TRANSFERRING ALABAMA'S SMOOTHNESS SPECIFICATIONS

FROM PI-BASED TO IRI-BASED

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TRANSFERRING ALABAMA'S SMOOTHNESS SPECIFICATIONS FROM PI-BASED TO IRI-BASED

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Date of Graduation

THESIS ABSTRACT

TRANSFERRING ALABAMA'S SMOOTHNESS SPECIFICATIONS

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Wen Huang

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Currently, Profilograph Index (PI) is deployed as the pavement smoothness evaluation index in the Alabama DOT's smoothness specifications. The specifications set the incentive, full and disincentive payment levels to encourage the construction of smoother pavement. The problems of this index are the poor correlation between PI and the driving comfort, and its walking-speed operation, which makes it infeasible to keep track of pavement smoothness condition over time and traffic. With the development of inertial profilers, smoothness specifications based on International Roughness Index (IRI), which can accurately evaluate the driving quality right after construction up to rehabilitation needs, are expected to address these problems.

An analysis was conducted on the profile database pooled from a range of Alabama asphalt concrete pavements and Quebec Portland cement concrete pavements. Correlations between PI and IRI were developed by several statistic methods. According

to these relationships, the PI-based smoothness specifications were transferred to IRI-based smoothness specifications.

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CHAPTER ONE

INTRODUCTION

Pavement smoothness, defined as the lack of roughness, is considered as one of the most important indicators of overall construction quality and subsequent riding comfort (Smith et al. 1997). Initially smooth pavement, which is the result of a good construction quality, provides a longer service life than initially rough pavement (Smith et al. 1997). For the driving public, smoothness is the primary means of assessing pavement quality. A rough-riding pavement increases fuel costs, vehicle maintenance and repair costs, slows traffic flow which can increase congestion, and in extreme cases, creates safety issues. Due to the importance of pavement smoothness, smoothness specifications are applied to encourage the construction of good ride quality of the final surface. Good-riding smooth pavements can earn the incentives, while contractors building rough-riding pavement product are only paid a reduced portion of the contract price (i.e., disincentives).

1.1 Background

The nationwide application of smoothness specifications has led to the development of a variety of devices to measure pavement profiles, which generate various ride quality statistics as the outputs. The most commonly employed device is the California-type profilograph, used to calculate the profile index (PI) as the index to assess pavement smoothness. The PI represents the total accumulated deviations of the longitudinal profilograph beyond a tolerance zone, which is also referred as a blanking band.

Until recently, the Alabama Department of Transportation (ALDOT) deployed the McCracken California-style profilograph as the standard measuring device, and Profile Index with 0.2 inch blanking band as the smoothness index. Contractors received a 5% bonus by providing pavements with a PI of less than 2 inch/mile (ALDOT, 2002). However, an analysis study conducted by ALDOT in 1999 indicated that 0.2 inches blanking band specification raised some concerns (Bowman et al., 2003). The most important one was that the wide blanking band (0.2") ignores defects (localized bumps) in the surface that are felt by the driving public but not necessarily identified as a penalty to the contractor. In this analysis, more than three-quarters of all 0.1 mile segments were found falling within the bonus range for the contractor without improving the public's ride comfort. Therefore, Profile Index calculated with a 0.2 blanking band has a limited ability to reflect riding quality of the newly constructed pavement, which results in the failure of the PI to motivate good construction.

After 2003, ALDOT decreased the 0.2" blanking band to 0.0" blanking band, which helps to count irregularities hidden by the blanking band. However, PI still represents the physically accumulated pavement deviations, which do not directly connect to the ride quality of the pavement. And besides, since California-type profilograph is hand-propelled and operated in walking speed, it is extremely time-consuming and infeasible for PI to keep track of the pavement smoothness condition during the whole service life because of the required traffic control.

With the development of inertial profilers, especially light-weight inertial profilers, the longitudinal profiles of pavement can be collected at highway speeds, even right after paving is finished. These technologies make International Roughness Index (IRI) a universally accepted ride quality statistic. IRI accumulates the response of vehicle to the roughness of the road surface. It can precisely evaluate the riding comfort by simulating the way a reference vehicle would response to the pavement roughness and accumulating the vehicle suspension travel. And also, the inertial profilers are operated at highway speeds, which provides an efficiently fashion to investigate the smoothness of the new pavement and monitor the subsequent pavement condition over traffic and time. All these evident advantages encourage the development of IRI as a portable and repeatable smoothness scale to evaluate both short and long-term pavement ride quality.

Although PI is used in the present ALDOT pavement smoothness specification, an urge to employ IRI in specification is claimed by ALDOT because of the advantages of IRI. In order to transfer the current PI based specifications to the corresponding specifications with IRI, the relationships between the PI and IRI indices are needed to connect different smoothness indices.

Currently, most agencies including ALDOT measure the pavement smoothness over a 0.1 mile segment during the quality assessment. But as observed in the quality assessment and construction, localized irregularities at the construction joint or caused by discontinuous paving practices can be averaged in the whole 0.1 mile interval without being noticed. In order to mark these bumps and accurately evaluate pavement smoothness, a smaller interval, such as a 0.01 mile segment, has the potential for identifying and quantifying these localized irregularities.

1.2 Objective

The main objective of this study was to move the current PI-based smoothness specifications to the corresponding IRI specifications. To address this transfer from PI to IRI in the specifications, the correlation between these two indices needed to be established. Based on these connections, the IRI limits, corresponding to PI limits for bonus, full pay, and penalty pay range, can be calculated and determined.

1.3 Scope

The 57 sets of longitudinal profiles from a range of Alabama asphalt concrete pavements and Quebec concrete pavements were collected for this study. All asphalt pavement sections are HMA overlay sections located in the same climatic zone (a wet, no-freeze region), while concrete pavement sections come from wet and freeze climate zone. Due to the different climate zone and other different conditions, PCC data from Quebec has the limitation to be applied in Alabama specification, PCC data was only used to primarily compare with AC data, and to present the way for different smoothness indexes to evaluate the pavement roughness.

Both IRI and mathematical-simulated PI value were calculated for each profile using the ProVAL Version 2.5 software, for 0.1 mile and 0.01 mile interval. The transferred specifications were only based on 0.1 mile interval. Since 0.01 mile interval is just used in localizing the bumps (WFLHD, 2003), the 0.01 mile interval specification for bump detection needs future development.

CHAPTER TWO

LITERATURE REVIEW

According to the definition of roughness (i.e., lack of smoothness) from ASTM E 867 (1998), traveled surface roughness is the deviations of a pavement surface from a true planar surface with characteristic dimensions that affect vehicle dynamics, ride quality, dynamic loads, and drainage, for example, longitudinal profiles, transverse profile and cross slope. Therefore, pavement roughness can be described by the magnitudes of the profile irregularities and their distribution on the measured surface.

2.1 Roughness Index

The primary objective for any ride quality index is to indicate information about a pavement surface that is sufficient to estimate the satisfaction of riding comfort. Mathematically, a pavement profile can be described as a combination of varied sine waves, which includes the long wavelengths like slope of pavement and the short wavelengths like the teeth-jarring waves (Sayers and Karamihas, 1998). Not all waves contribute to the driver's perception of pavement roughness. Good design of the vehicle suspension system and tire system are used to filter out the effect of some pavement wavelengths. The wavelengths that can not be filtered out with vehicle design and cause the unwanted vehicle vibration are felt as the pavement roughness. Consequently, the roughness index is required to attenuate the unnecessary road features and highlight the driving-discomfort wavelengths.

As a matter of fact, different indices use different mechanical filters or mathematical algorithms to collect pavement roughness information. Profile index is the typical representative for mechanical filter based indices; International Roughness Index is for profile based indices. Due to the different filter methods, some wavelength bands may be noticed by one roughness index and ignored by another index.

2.1.1 Profilograph Index

Profilograph Index (PI), also called as profile index, is derived from low-speed rolling system, which uses its own geometry to filter the profile. PI is derived from rolling straightedge systems such as California profilograph, which is a 25 ft long truss with a set of wheels at either end that travels over the pavement surface, presented in Figure 2.1 and Figure 2.2 (FAA, 2005). The wheel in the center of the truss is attached to a recording device (e.g. chart recorder), which documents the deviations. This rolling system functions as the mechanical filter. The wheels of truss except the middle wheel establish the average surface, and then the middle wheel records the deviation from this surface. According to this filter method, the long surface wavelength is removed by the average; the high-frequency wavelength is emphasized.

Figure 2. 1 McCracken California Profilograph

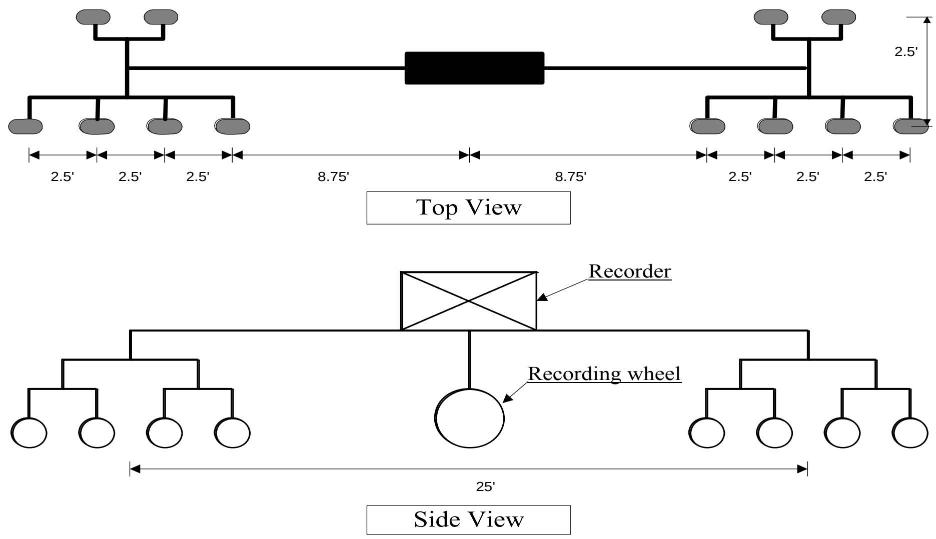


Figure 2. 2 A Typical California Profilograph with 12 Support Wheels (FAA, 2005).

The calculation procedure to produce the profilograph recording from the rolling systems can be expressed as in Equation 2.1(FAA, 2005). This equation is also the algorithm for profile software to simulate PI value from pavement profiles collected by the inertial profiler.

$$R(x) = (\sum_{i=1}^{N} C_i * P_i(x - d_i)) - P_r(x - d_r) \quad \text{(Equation 2.1)}$$

Where,

R(x) = the computed profilograph recording at the position x, mm

N = the total number of the wheels in the left and right side of the support system

P_i = the profile on which the ith wheel is traveling, mm

Ci = the influence coefficient corresponding to the ith wheel. It is equal to the vertical displacement at the recorder position caused by a unit vertical movement at the ith wheel. From the structure geometry and the definition of the influence coefficients, Ci = 1/16 for the 8 right side wheels and Ci = 1/8 for the 4 left side wheels is used here.

D_i = the offset distance from the location x for each wheel, mile

Items with subscript r refer to those of the recording wheel.

After recording the profilograph in the field, the operator needs to return to the office to have the chart paper profiles processed. The analysis starts with the location of a floating blanking band, which is determined by tracing these curve outlines. Figure 2.3 presents one sample of this process. The blanking band is located for allowing as many of irregularities as possible to be covered and blanked out. Since defects within blanking band are considered having no effect on riding quality, these defects are excluded from calculating PI values. In Figure 2.3, the two dash lines indicate the location of the blanking band. Each deviation exceeding the blanking band is called a scallop, with the number of scallops being accumulated to compute PI. PI value has the unit of slope, in/mile or m/km. A length of 0.1 miles (528 ft) is used as the interval over which the number of scallops is considered.

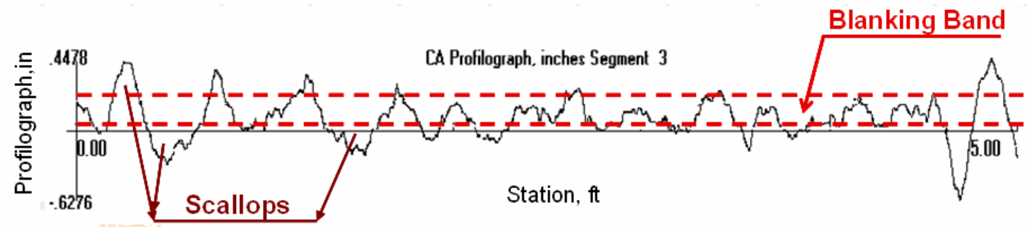


Figure 2. 3 Profilograph Trace (FAA, 2005).

0.0, 0.1 and 0.2 inches, are the commonly used blanking band to calculate PI value. From Federal Highway Administration survey results (Smith et al. 1997), 19 states used 0.2 inches blanking band for AC pavement quality assurance; one state used 0.1 inches blanking band and two states used 0.0 inches blanking band. 2000 America Concrete Pavement Association database shows that the different usages of blanking band for PCC pavement are distributed: 0.2" blanking band 77.2%, 0.1" blanking band 13.9%, 0.0" blanking band 11.1%. These different blanking bands can generate the different effect on evaluating the pavement ride quality. The vertical deviations smaller than 0.2 inches are not counted when 0.2" blanking band is applied to compute the PI value. This has raised some concerns because in some cases newly constructed pavements received the riding quality complaints even though they met the smoothness criterion (Bowman et al. 2003). 0.0 inches blanking band can count every irregularities to better assess the pavement roughness. There is a trend to move toward 0.0" blanking band to compute PI value.

Two methods are widely used to conduct PI calculation, manual method or automated method (ProScan™, shown in Figure 2.4). Manual tracing includes the personal judgment about the location of blanking band, which leads to variations in the final result. Alabama uses automatic tracing program, ProScan™ system to process the trace (ALDOT, 2003). In general, the profiler curve is scanned to digitize its tracing. An image enhancement program is then used to prepare the image for analysis. After the enhancement, mathematical filtering is applied to the digitized traces to reduce the noise of the traces and to mimic the process of an operator drawing the outline on the trace. A linear regression analysis is then performed to establish the location of a floating centerline and blanking band, along the outline of the trace (Pellinen et al. 2003).

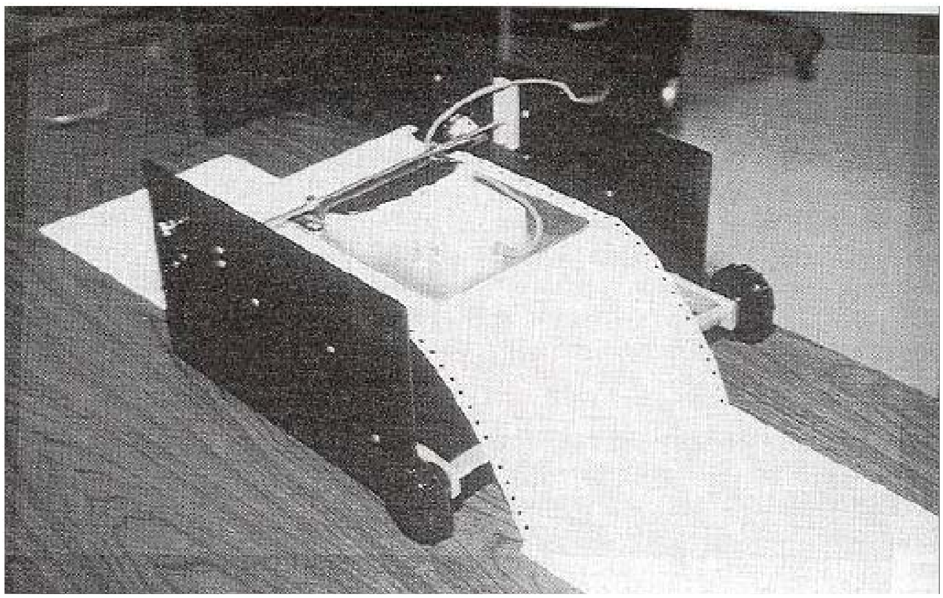


Figure 2. 4 ProScan (Smith et al., 1997).

Figure 2.5 shows the sensitivity of PI to the wavelengths, where a gain equals 1 for the true profile (Smith et al. 2002). If the gain value corresponding to one wavelength is larger than 1, this wavelength is considered as having an important effect on the discomfort riding and would be amplified in PI calculation. On the other hand, if the gain value is less than 1, the wavelength is recognized to have an insensitive effect on riding quality and would be attenuated in PI calculation. According to Figure 2.5, it is indicated that PI addresses the wavelengths from 0.3 to 23 m (1 to 75 ft), especially wavelengths from 0.3 to 1 m. The filtering function of the rolling system is limited by its own geometry, which minimizes the impact of wavelengths shorter than 0.3 m or longer than

23 m.

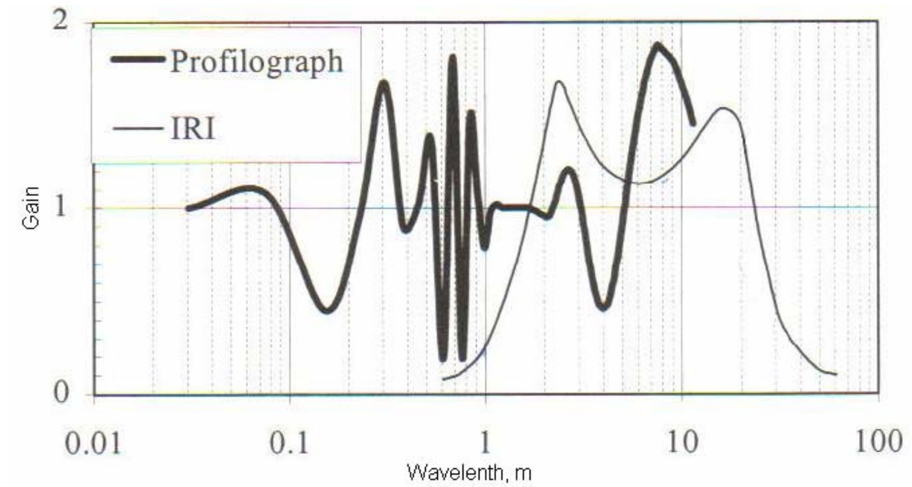


Figure 2. 5 Sensitivity of PI and IRI to Wavelength (Evans et al. 2003).

2.1.2 International Roughness Index

International Roughness Index (IRI) is the ride quality statistic deriving from the response-type road roughness measuring systems (RTRRMS). In RTRRMS, the devices (also called as roadmeters) accumulate the suspension motion of a passenger car running over a pavement surface at a given speed. IRI mathematically standardizes the PTRRMS and duplicates the vibrations level of the vehicles.

IRI is based on the response of a generic passenger car (known as the quarter-car model) to the roughness of a pavement surface. This simple dynamic model is a sprung mass resting on a suspension system with stiffness and damping (Figure 2.6). The wheel contacts the road through a tire-like spring. Road inputs to the car flex the tire, stroke the suspension and cause the sprung and unsprung masses to vibrate in the vertical direction (Shahin, 1994). The vertical velocity difference between sprung mass body and unsprung mass body produces the stroke of the suspension system, which is perceived by human body as the roughness of pavement. Equation 2.2 illustrates the algorithm used by IRI to record these acceleration differences (Sayers, 1995). About 70% of vertical vibration of a passenger experiences can be described by the response of quarter-car to pavement. The IRI is the accumulated vehicle vibration divided by the distance traveled to give a ride quality statistic with units of slope (in/mile, m/km).

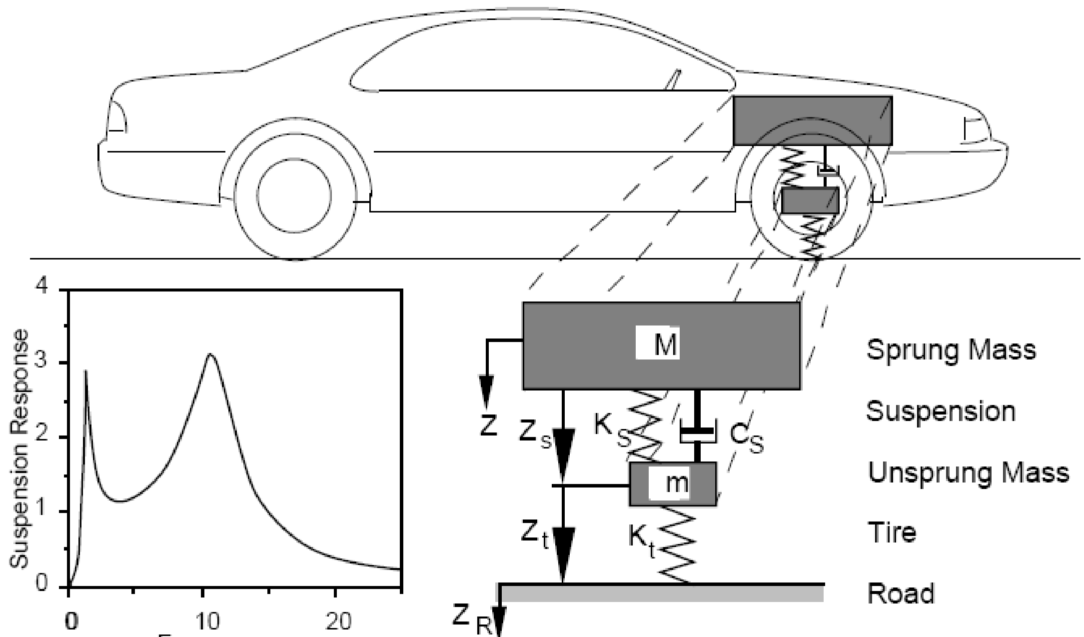


Figure 2. 6 Quarter Car Model. (Gillespie, T.D., 1992)

$$IRI = \frac{1}{L}\int_0^{\frac{L}{V}} \left| \dot{z}_s - \dot{z}_t \right| dt \qquad \text{(Equation 2.2)}$$

Where,

IRI = International Roughness Index, in/ft;

L = the distance quarter-car travels over, ft;

V = the velocity of quarter-car, ft/s,

$\dot{z}_s$ = the vertical velocity of sprung mass, ft/s^2

$\dot{z}_t$ = the vertical velocity of unsprung mass, ft/s^2

As viewed in Figure 2.5, IRI has sensitive gain value larger than 1 for wavelengths from 2.2 to 16.1 m (7.1 to 52.5 ft), which means this wavelength range are sensitive to the pavement ride quality based on IRI algorithm. This wavelength range is within the sensitivity band range for the PI statistic (i.e., between 0.3 to 23 m). However, it is also evident that PI focuses more on the smaller wavelengths around 1 meter, whereas IRI amplifies the larger band wavelengths from 3 to 11 m.

2.1.3 Comparison of PI with IRI

Automotive engineers measure accelerations on the seat of the car to evaluate the suspension performance and the riding comfort of passengers. From numerous studies of the human body sensitivity to vibration in a sitting position, a vertical frequency of around 5 Hz is critical to the riding comfort. It is generally recognized that the human body has minimum tolerance to vertical vibration when the vibration frequency is about 5 Hz due to resonance of the abdominal cavity (Sayers and Karamihas, 1998). For example, Figure 2.7 shows that in the SAE J6A research, human body only can endure about 0.13 g acceleration when the vibration frequency equals to 5 Hz, while when the vibration is decreased to 1 Hz or increased to 20 Hz, the tolerable vertical acceleration for human body can be about 0.8g. Therefore, the pavement wavelengths raising the critical frequency should be emphasized by the pavement roughness index. In other words, the roughness index needs to have gain value larger than 1 to this wave band and be sensitive to these wavelengths.

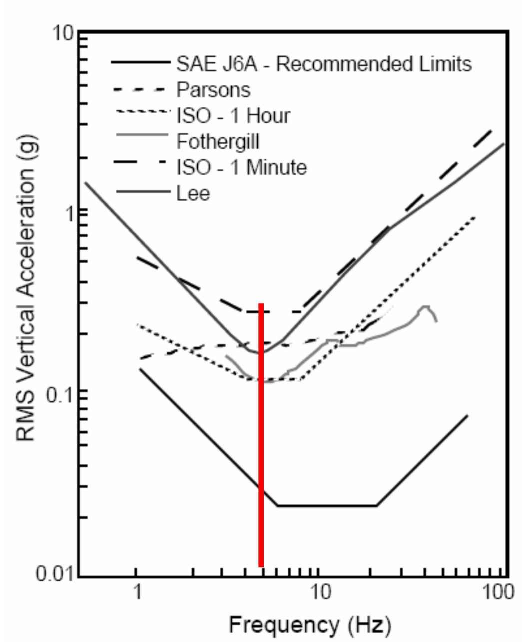


Figure 2. 7 Human Body Sensitivity of the Vertical Vibration (Sayers and Karamihas, 1998).

Assume the average speed of the vehicle ranges from 25 mi/hr to 80 mi/hr, the pavement surface wavelengths from 2.2 m to 9.4 meters can cause the vertical vibration of 5 Hz, which is mostly uncomfortable to passengers. A good ride quality index that accurately reflects user discomfort is required to make these wavelengths pronounced in evaluating the pavement roughness. Based on the former discussion about the sensitivity range of IRI and PI, it can be concluded that IRI well covers this critical wavelengths from 2.2 m to 9.4 m responsible for creating vertical vibration. As for PI, it not only covers this critical wavelength but also emphasizes other wavelengths unnecessary to producing vibration. This means that IRI can more accurately assess the ride quality through focusing on these uncomfortable pavement features.

The quarter-car model uses the suspending system and pneumatic tire damping to isolate the effect of some speed-related vertical frequencies, and records the accelerations of the passenger seat. Its rational algorithm makes this model more related to the vertical acceleration of vehicle than the hand-operated California profilograph rolling systems. Therefore, IRI can better represent the driving comfort than PI.

2.1.4 Correlation between PI and IRI

Since PI and IRI statistic amplifies or attenuates profile features occurring at different wavelengths range, it is difficult to find an exact correlation between these two indices. Nevertheless, Figure 2.5 also presents that both of indices amplify the wavelengths from 2 to 10, even though at different degree. It makes the possibility to develop the connection between these two indices. Some previous research has presented that there is a relatively good statistic relationship between PI and IRI.

In 1989, Pennsylvania Transportation Institute (PTI) conducted a full-scale field-testing program on behalf of Federal Highway Administration (FHWA) to develop calibration procedures for profilograph and evaluate equipments for measuring the smoothness of new pavement surfaces (Kulakowski and Wambold, 1989). In this project, 26 individual 0.1 mile long sections were selected from five different locations around Pennsylvania, including new or newly surfaced concrete pavements and asphalt pavements. Pavement roughness was recorded by profilograph and laser-type inertial profiler. Table 2.1 shows the relationship developed in this correlation. Solely based on the data from this research, the regression was not considerably different between concrete sections and asphalt sections. The manually calculated $PI_{0.2}$ had a correlation equation with IRI different from the correlation equation between computer-generated $PI_{0.2}$ and IRI. Slope from the regression equation from computer-generated $PI_{0.2}$ was considerably flatter.

1992 saw Arizona Department of Transportation (AZDOT) initiated a study to determine the feasibility of their inertial profiler (K.J.Law 690 DNC profilometer) on measuring the initial PCC pavement smoothness (Kombe and Kalevela, 1993). To examine the correlation between the profiler (IRI) and profilograph (PI) output, twelve typical newly-constructed 0.1-mile PCC pavement sections were selected to measure the smoothness by both devices. Simple linear regression (presented in Table 2.1) were performed between IRI and $PI_{0.2}$ values and indicated that generally good correlation existed between these two indexes with high R^2 of 0.93.

During developing the new smoothness specifications for rigid and flexible pavements in Texas, University of Texas operated an investigation between McCracken California-type profilograph and the Face Dipstick, a manual profile measurement device in 1993 (Scofield, 1993). After collecting smoothness of 18 pavement sections including both asphalt and concrete pavements using these two devices, linear regression analysis (presented in Table 2.1) showed a strong collection (R^2=0.92) between IRI and $PI_{0.2}$.

In order to compare its current rolling straightedge with other available measurement devices such as inertial profilers, Florida DOT conducted a study in 1997 (FLDOT, 1997). Twelve 0.5-mile sections from newly-constructed or resurfaced asphalt pavement were chosen for testing. Two type sensors were equipped in the inertial profilers, laser sensor and ultrasonic sensors. The linear relationships between IRI and $PI_{0.0}$ were developed respectively for each kind of inertial profiles. Both correlations (presented in Table 2.1) were fairly strong, with R^2 value of 0.88 and 0.67. Since the ultrasonic-based measurement adds the smoothness sensitivity to surface texture, cracking and temperature, the measurements deriving from ultrasonic profiler were higher than laser-based and resulted in higher intercept in the regression equation.

In 1996, as part of research on transfer a profilograph-based smoothness specification to a profile-based specification, Texas Transportation Institute (TTI) was involved to evaluate the relationship between PI and IRI (Fernando, 2000). Longitudinal surface profiles from 48 newly resurfaced AC pavement sections throughout Texas were measured to produce PI and IRI values. PI values were simulated by using ProScan software, IRI was automatically created from the inertial profiling system. In the relationship evaluation, a much stronger trend (presented in Table 2.1) was found between IRI and $PI_{0.0}$ than between IRI and $PI_{0.2.}$ Since the application of blanking band mask the effect of certain component of the roughness, $PI_{0.2}$ was found to have a poorer relationship with IRI than $PI_{0.0}$.

In developing a series of relationships between IRI and PI that can assist states in transitioning to IRI or $PI_{0.0}$ smoothness specifications for AC and PCC pavement, research project using the Long Term Pavement Performance (LTPP) DataPave database to establish the relationships was sponsored and conducted by FHWA in 2002, hereafter referred as 2002 LTPP. A total of 1,793 LTPP test sections located in 47 states and 8 Canadian Provinces, which span all four climatic zone (dry freeze, dry nonfreeze, wet freeze and wet nonfreeze), formed the database for this evaluation (Smith.K.L et al. 2002). All these archived profile were measured with K.J. Law T-6600 inertial profiler from 1996 to 2001.PI and IRI values were generated from “Indexer”, a profiler software developed by K.J. Law in 1995. Finally, the linear regression models were developed between IRI and $PI_{0.0}$, $PI_{0.1}$, $PI_{0.2}$. Different pavement type and climate zone were found to have a significant effect on the regression model. The models in wet nonfreeze climate zone, where Alabama belongs, are presented in Table 2.1.

The regression equations from all these research are summarized in Table 2.1. Since the blanking band covers some components of pavement roughness, the correlation between IRI and $PI_{0.0}$ was found generally stronger than correlation between IRI and $PI_{0.2}$. Table 2.1 shows that both the slope and intercept of the regression equations are dependent on the blanking band selected for calculating the PI ride quality statistic.

When a 0.2 blanking band is used, the average slope is 3.7, and the average intercept is 64.6 in/mi. The values are various among different studies. When a 0.0 blanking band is used, both the slope and intercept decrease. The average of slope is 2.2, and the intercept is 18.2 in/mi. The values are more consistent between different studies than values in 0.2 blanking band.

Data from PTI, ADOT, University of Texas and FLDOT research were developed by calculating one statistic for each of two independently obtained profiles. It is extremely difficult to track the identical profile with two different devices which can have a large influence in the quality of the correlations obtained. The data of 2002 LTPP and TTI were developed using a single source of raw profile data, then calculating both the IRI and PI from the same profile. One single source raw profile data eliminates the variation between two profilers used to respectively calculate IRI and PI value. The correlations would be sensitive only to the choice of blanking band and not of changes in profile characteristics.

Table 2. 1 Summary of Documented PI-IRI Relationships.

Study (Year)	Pavement Types	No. of Test Sections	Remarks	Linear Regression Equation, m/km	Linear Regression Equation, in/mi	R^2
PTI (1989)	AC and PCC	26	Manual profilograph PI Laser-type inertial profiler	$IRI = 4.02 * PI_{0.2} + 1.11$	$IRI = 4.02 * PI_{0.2} + 70.13$	0.57
PTI (1989)	AC and PCC	26	Computerized profilograph PI Laser-type inertial profiler	$IRI = 2.46 * PI_{0.2} + 1.04$	$IRI = 2.46 * PI_{0.2} + 66.22$	0.58
Arizona DOT (1992)	PCC	12	Computerized profilograph PI Laser-type inertial profiler	$IRI = 6.10 * PI_{0.2} + 0.83$	$IRI = 6.10 * PI_{0.2} + 52.90$	0.93
University of Texas (1992)	AC and PCC	18	Computerized profilograph PI Manually computed IRI (Dipstick)	$IRI = 2.83 * PI_{0.2} + 1.16$	$IRI = 2.83 * PI_{0.2} + 73.70$	0.92
Texas Transportation Institute(1996)	AC overlays	48	Computer-simulated PI Laser-type inertial profiler	$IRI = 4.08 * PI_{0.2} + 0.84$	$IRI = 4.08 * PI_{0.2} + 52.74$	0.56
LTPP (2002)	AC Overlay (wet nonfreeze)	5126	LTPP Measurement data	$IRI = 3.43 * PI_{0.2} + 0.88$	$IRI = 3.43 * PI_{0.2} + 55.54$	0.63
LTPP (2002)	PCC (wet nonfreeze)	2888	LTPP Measurement data	$IRI = 2.87 * PI_{0.2} + 1.23$	$IRI = 2.87 * PI_{0.2} + 77.89$	0.74
Florida DOT (1996)	AC	12	Computerized profilograph PI Laser-type inertial profiler	$IRI = 2.19 * PI_{0.0} + 0.22$	$IRI = 2.19 * PI_{0.0} + 13.75$	0.90
Florida DOT (1996)	AC	12	Computerized profilograph PI Ultrasonic-type inertial profiler	$IRI = 2.20 * PI_{0.0} + 0.31$	$IRI = 2.20 * PI_{0.0} + 19.36$	0.88
Texas Transportation Institute(1996)	AC overlays	48	Computer-simulated PI Laser-type inertial profiler	$IRI = 2.14 * PI_{0.0} + 0.31$	$IRI = 2.14 * PI_{0.0} + 19.33$	0.85
LTPP (2002)	AC Overlay (wet nonfreeze)	5126	LTPP Measurement data	$IRI = 2.42 * PI_{0.0} + 0.30$	$IRI = 2.42 * PI_{0.0} + 19.12$	0.84
LTPP (2002)	PCC (wet nonfreeze)	2888	LTPP Measurement data	$IRI = 2.36 * PI_{0.0} + 0.32$	$IRI = 2.36 * PI_{0.0} + 20.09$	0.84

2.2 Smoothness Specifications Conversion Methods

With the update of the pavement roughness measurement devices or evaluation method, some states already had the experience on moving their former smoothness specifications to the new specifications. There are several methods widely used for making this conversion.

The first method is based on engineering judgment without performing any comparative measurements. Indiana DOT and Missouri DOT selected their new reasonable IRI specifications from the practical knowledge and field experience of old specifications (Pellinen et al., 2003).

The second method is to build the regressed correlation equations between old smoothness index and the new IRI index. Through the regress equations, the old smoothness index based specifications are transferred to specifications based on the new smoothness index. Illinois DOT established the regressed relationship between IRI and PI from an available database, such as LTPP (Rufino et al., 2001). The bonus and penalty range for the new index IRI, corresponding to the old PI index range, were determined by the correlations, shown in figure 2.8.

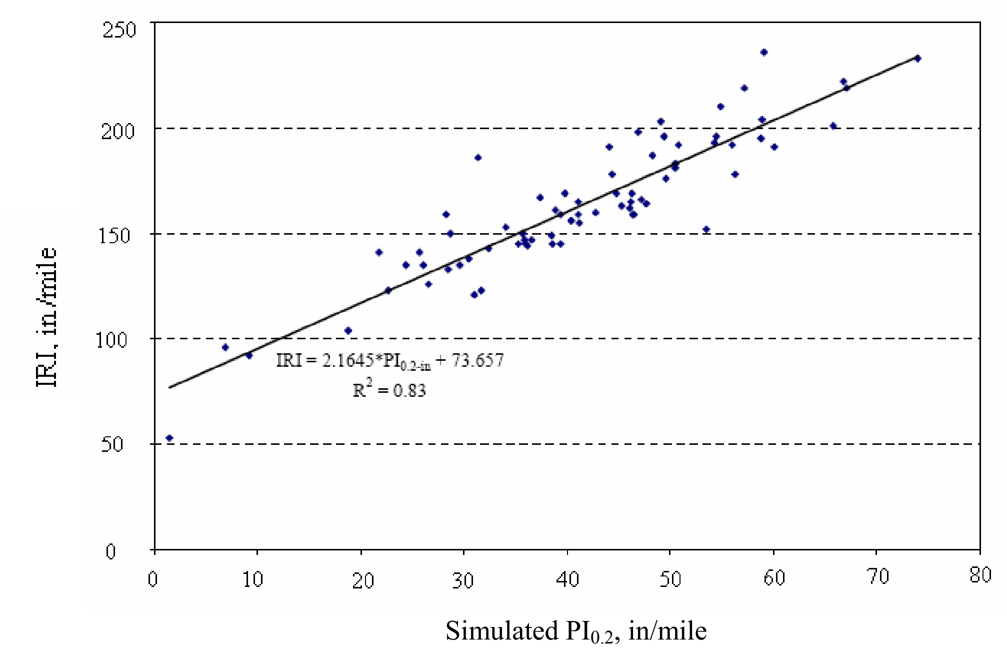


Figure 2. 8 Relationship between Simulated $PI_{0.2}$ and IRI in ILDOT Bridge Smoothness Study (Rufino et al., 2001).

The third method is to statistically examine the surface smoothness data, and plot the probability or distribution curve for both old and new index. The new index limits for incentive/disincentive pay ranges correspond to the limits of old index by having the same amount of sections in each smooth level. Kansas DOT, Minnesota DOT and Wisconsin DOT applied this histogram method to establish new index specifications (Pellinen et al., 2003). Figure 2.9 shows an example how this approach is used.

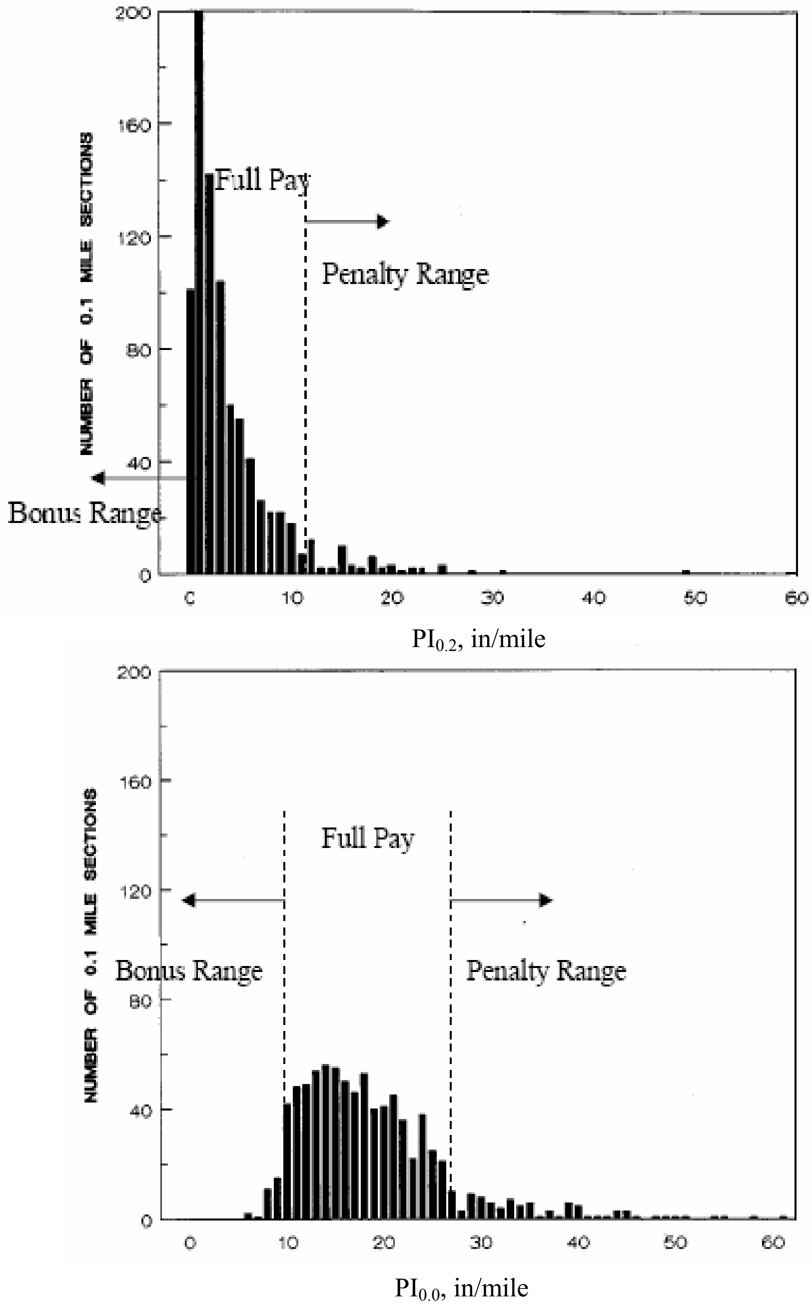


Figure 2. 9 Conversion from Old Smoothness Specification to New One by Distribution Method (Hossain et al., 1995).

From the distribution plot of $PI_{0.2}$, it can be calculated that based on $PI_{0.2}$ specification from Kansas, 10% segments having $PI_{0.2}$ value less than 2 in/mile were qualified to the incentive, 80% segment would be full paid, 10% segments with $PI_{0.2}$

value larger than 10 in/mile located in the penalty range. Therefore, based on distribution method, in order to allow 10% segments still could achieve bonus, the lower limit for $PI_{0.0}$ full pay range needed to be set at 10 in/mile. For having 80% segments in full pay range, the upper limit for $PI_{0.0}$ full pay range would be 26 in/mile. Consequently, the specifications based on the new roughness index were determined after setting those limits.

2.3 Effect of Short Interval on Estimating Pavement Smoothness

Some bumps or localized irregularities are not detected by the average IRI values over long distance. Figure 2.10 shows a continuous plot of average IRI values over 0.1 mile interval and 0.01 mile interval of one pavement section. Assume the upper limit of average roughness considered barely acceptable without correction is 95 in/mi (WFLHD, 2003), 17% segments at 0.1 mile interval are recognized as bumps with needed correction, while 23% segments at 0.01 mile interval are detected as irregularities. By examining the IRI values at short interval, some of the segments requiring correction can be readily identified as the result of poorly constructed joints.

Compared to long interval spacing, short interval spacing more accurately locates and quantifies localized ride quality problems.

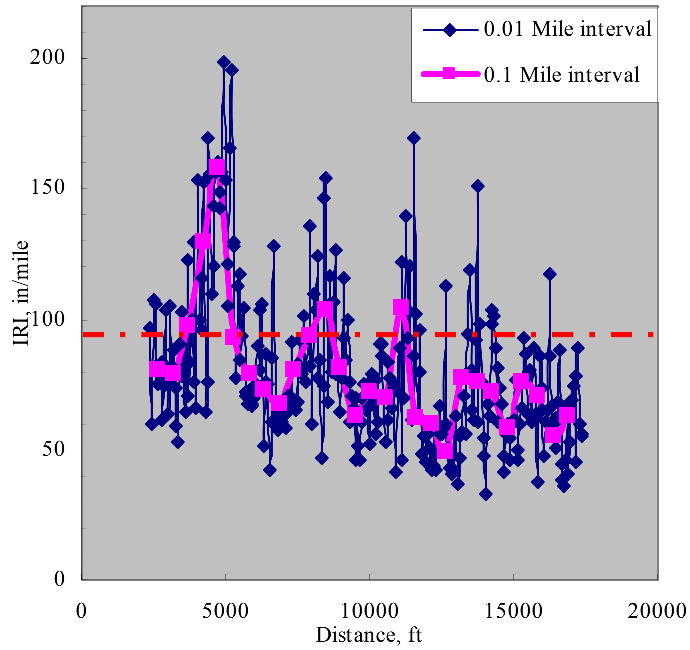


Figure 2. 10 Comparison of IRI Value at 0.1 mile interval with 0.01 mile interval.

2.4 Smoothness Specification

2.4.1 ALDOT Smoothness Specifications

As of 2002, ALDOT had different pavement smoothness specifications for asphalt and concrete pavements (Table 2.2 and Figure 2.11). Both of the specifications were PI-based using a 0.2 inches blanking band. The smoothness values were required to be measured as soon as practical after paving and compaction. The measurement interval in quality assessment was 0.1 mile. The specifications for asphalt pavement combined continuous and step function pay factors for different smoothness levels. Concrete pavement had the step function pay factors for each smoothness level. Pay factors for concrete pavement were higher than asphalt, either in bonus range or penalty range.

Table 2. 2 Alabama Pavement Smoothness Specifications for $PI_{0.2}$ (ALDOT, 2002).

Pavement Type	Equipment	Section Length	Blanking Band	Price Adjustments	
				Profile Index, in/mile	Contract Price Adjustment of pavement unit bid price, %
Asphalt Pavement	California profilograph	0.1 mile	0.2 inches	Under 2	105 - (profile index/4)
				2 to 4	100
				4 to 10	100 - (profile index-4)/0.3
				Over 10	Unacceptable
Concrete Pavement	California profilograph	0.1 mile	0.2 inches	Under 3	105
				3 to 6	100
				6 to 8	95
				8 to 10	90
				Over 10	Unacceptable

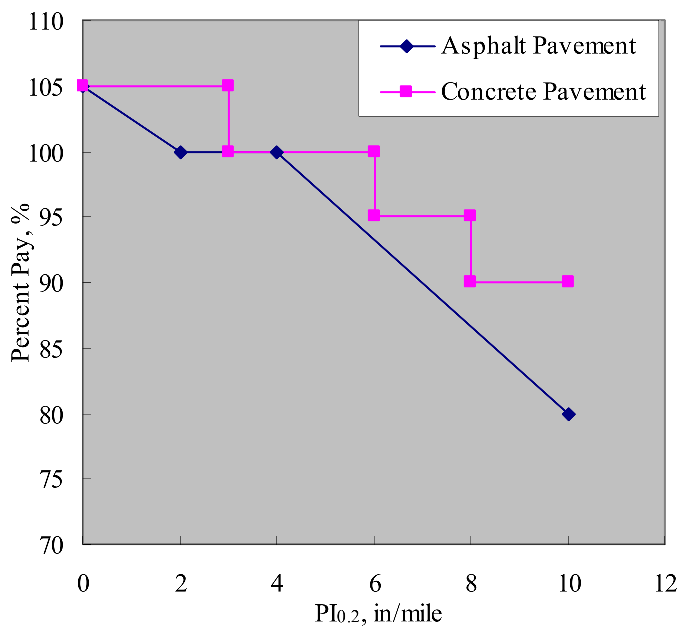


Figure 2. 11 2002 ALDOT Specification for Pavement Roughness.

The ALDOT smoothness specifications were changed in 2003 so that ride quality would be evaluated using a 0.0 blanking band ($PI_{0.0}$). At the same time, the separate specifications for asphalt concrete and Portland cement concrete pavements were eliminated. There is only one specification for ride quality, regardless of the type of pavement. Pavement products are paid only by the ride service they can provide, concrete pavements are required to reach the same comfort level as asphalt pavement to earn the same pay. The pay functions of concrete were also changed from step functions to the combination of step and continuous functions. Table 2.3 states the current $PI_{0.0}$ ALDOT smoothness specifications.

Table 2. 3 Alabama Pavement Smoothness Specifications for $PI_{0.0}$ (ALDOT, 2003).

Profile Index In/mi/Section	Contract Price Adjustment Percent of Pavement Unit Price
Under 10.0	105 – (PI/2)
10.0 to less than 20.0	100
20.0 thru 50.0	100 - (PI -20)/1.5
Over 50.0	Unacceptable

While the current ALDOT specification is based on PI using the 0.0 blanking band, the analyses in the following chapters will include the evaluation of PI calculated with both blanking bands and the IRI. The $PI_{0.2}$ is included because a number of states still use this value; there is also a substantial amount of previous research based on this value.

2.4.2 Smoothness Specifications of Other DOTs

After changing its smoothness specification toward 0.0 inches blanking band, it is still necessary for ALDOT to track the implement of this new specification. In this study, smoothness specifications based on $PI_{0.0}$ from other states were collected to compare and evaluate the current ALDOT smoothness specification.

The specifications from five states (plotted in Figure 2.12 and Figure 2.13) state that these states employ different smoothness specifications for AC pavements and PCC pavements. Figure 2.12 records the PI $_{0.0}$ specifications for AC pavement. In this figure, these five states all deploy the step functions to pay the pavements at each smoothness level. And also, the incentive and disincentive ranges are divided into several steps to have an accurate pay for each riding quality level. The lower limits for $PI_{0.0}$ full pay range are averaged around 17 in/mile; the average of upper limits is 27 in/mile. Compared to current ALDOT $PI_{0.0}$ specifications, ALDOT uses continuous function for paying different smoothens level. And also, ALDOT conducts considerable stricter specifications, where $PI_{0.0}$ full pay range from 10 in/mile to 20 in/mile, than these states.

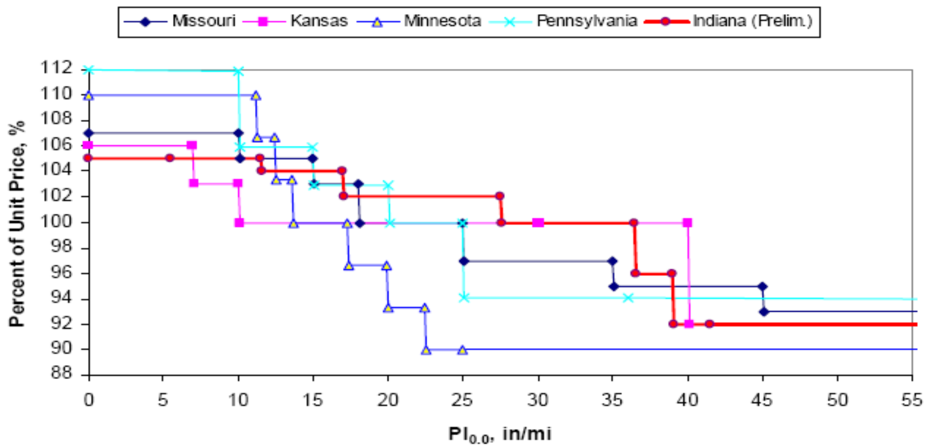


Figure 2. 12 $PI_{0.0}$ Specifications for AC Pavement from other DOTs (Pellinen et al., 2003).

Figure 2.13 plots the $PI_{0.0}$ specifications for PCC pavement from other DOTs. All these state have the more lenient specifications for PCC pavement than AC pavement. For example, Kansas DOT pays more incentive for smooth PCC pavement than smooth AC pavement, and has the same penalty for pavement generating $PI_{0.0}$ values larger than 40 in/mile for both pavement types.

The step functions are still used for paying concrete pavement. The incentive and disincentive ranges are also separated into several steps to have an accurate pay for each riding quality level. The lower limits of full pay range are averaged around 26 in/mile. States like Indiana and Pennsylvania have no penalty to the PCC pavement. The upper limits of full pay range are around 42 in/mile averaged from Kansas and Wisconsin specifications.

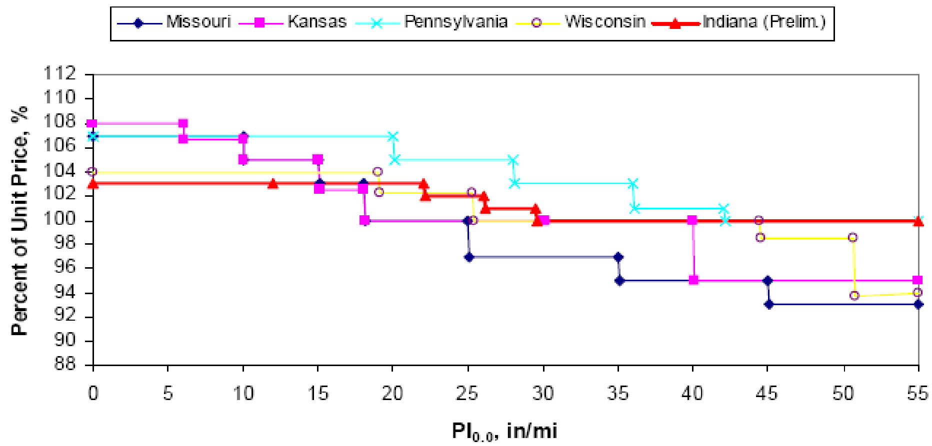
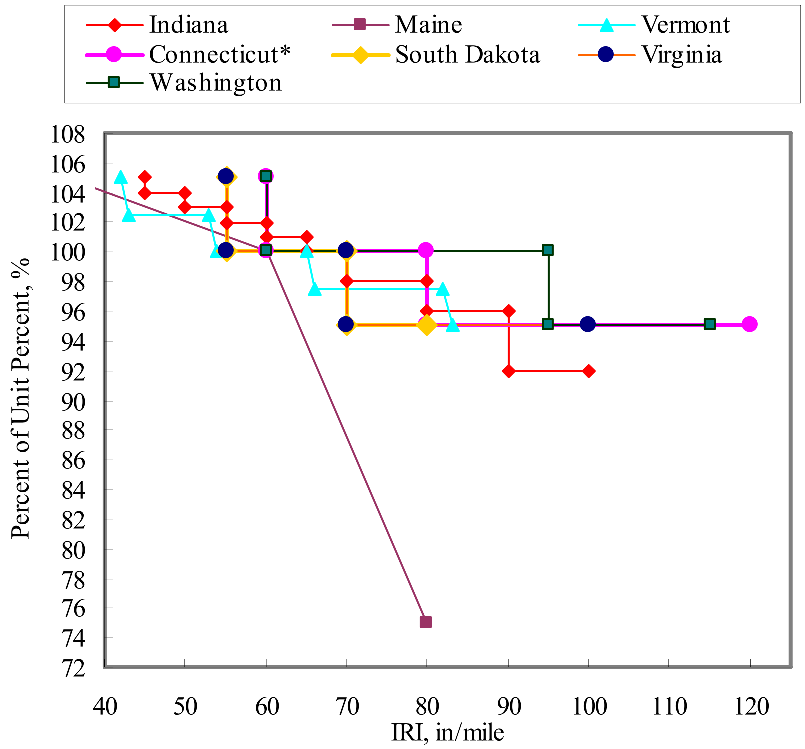


Figure 2. 13 $PI_{0.0}$ Specifications for PCC Pavement from other DOTs (Pellinen et al., 2003).

Currently, IRI is already applied in quality assessment of some states. Since the main objective of this study is to transfer PI based specification to IRI based, the IRI specifications from other states are plotted in Figure 2.14 to provide a reference for establishing ALDOT IRI specifications.

Within the seven states in Figure 2.14, Maine and Virginia have the same specifications for AC pavement and PCC pavement. In other states, like Connecticut, South Dakota, Vermont and Washington, IRI-based specifications are just for evaluating flexible pavement; PI-based specifications are still used to investigate rigid pavement. Except Maine, other states use the step function to pay the pavement at different smoothness levels. The lower limits of full pay range from these seven states have the average of 58.5 in/mile; the average of the upper limits is around 73 in/mile.



*: For Connecticut, South Dakota, Virginia and Washington, there is no detailed pay factor value available. 105% and 95% was assumed as the bonus and penalty pay factor.

Figure 2. 14 IRI Specifications from other DOTs.

2.5 Summary

From the literature reviews discussed in this chapter, several conclusions can be drawn as follows:

- PI is the physical accumulation of pavement deviations. The geometry of the rolling system limits PI sensitive to pavement wavelength from 0.3 to 23 m, especially from 0.3 to 1 m. However, IRI is the accumulated vertical vibration simulated by Quarter-car model. This index is sensitive to the wavelengths spanning from 2 to 16 m. IRI wavelength range well covers the waves responsible for 5Hz critical frequency vibration, which ranges from 2 m to 10 m and human body has the minimum tolerance to. Therefore, IRI better represents the pavement riding quality.

- The short interval spacing makes the localized bumps pronounced. Some of the bumps averaged in the 0.1 mile interval can be detected in the 0.01 mile interval. Short interval localizes and quantifies the local riding problems.
- ALDOT moved its smoothness specifications from $PI_{0.2}$ to $PI_{0.0}$ in 2003. In $PI_{0.0}$ specifications, AC pavement and PCC pavement have the same pay standard. The specifications provide full pay for pavement smoothness ranged from 10 in/mile to 20 in/mile. Compared to ALDOT, most of other states have different specifications for each pavement type. Either for AC pavement or PCC pavement, the specifications from several other states are more lenient than ALDOT specifications. Base on the specifications from five states plotted in Figure 2.12 and Figure 2.13, the average lower limit of full pay for AC pavement is 17 in/mile, upper limit is 27 in/mile. The average lower limit of full pay for PCC pavement is 26 in/mile, upper limit is 46 in/mile.
- As the specifications of seven states using IRI plotted in Figure 2.14, step functions are used to pay the different smooth level pavement. The lower limits of full pay range from these seven states have the average of 58.5 in/mile; the average of the upper limits is around 73 in/mile.

CHAPTER THREE

DATA COLLECTION AND DATABASE DEVELOPMENT

3.1 Data Collection

The Roadware ARAN (automated pavement analyzer) vehicle was used to collect pavement longitudinal profiles in this study. This vehicle has several subsystems, which can collect the raw profile data in each wheel path for calculating ride quality statistics, such as IRI and PI. Other pavement condition information collected includes rut depth estimates (both wheel paths) and pavement macro texture in the right wheel path only. Auburn University has an ARAN van of model 4300, which uses the South Dakota Profiler (SDP) inertial profiling system sensor set-up. This is a laser-accelerometer combination system to measure the longitudinal profile. This system measures the pavement profile at intervals as short as 100 mm (4 in) at variable speeds up to 100 km/h (60 mph) (Roadware, 2005). An automated standard moving-average filter from ARAN translates the digital sensor data into a representation of the relative surface profiles. Therefore, the output profiles from ARAN system are considered pre-filtered before any further analysis is conducted.

Pavement longitudinal profile measured by laser inertial profiler, like ARAN van, covers a slice of pavement. With the variation between different driver and the variation of start point, it is hard to repeat the exact same profile measure. But the former research has indicated that inertial profiler has high repeatability. In 2000 and 2001, Highway

Research Center in Auburn University operated the repeatability estimates for inertial profiler in National Center for Asphalt Technology (NCAT) test track. The research showed that IRI had the coefficient of variance (COV) around 9% between different repeat measures. For rough and high ESALs pavement, COV value increased to 15% (Stroup Gardiner, 2004). It was suggested that the one-time measurement of profile from ARAN was sufficient.

In this study, the ARAN Van was driven over a range of asphalt pavement and concrete pavement projects to collect longitudinal profiles (total 20 sections) in both right and left wheelpath. The longitudinal profiles of all sections were measured at least twice (i.e., 2 replicates). When possible, the profiles were measured three times for one section, ending up with a total of 57 pavement profiles.

Figure 3. 1 ARAN Van Model 4300 (Roadware, 2005).

3.1.1 Asphalt Pavement Profiles

Longitudinal profiles of asphalt pavement were collected from four Alabama paving projects using ARAN inertial profiler. These projects are briefly described in Table 3.1. Projects were HMA overlays after an initial mill only, or a mill and chip seal preparation.

Longitudinal profiles were collected as soon as practical after the paving and rolling was completed.

Table 3. 1 Project Descriptions (Alabama Mill and Fill Projects).

Project	Location	Layer	Mix Design	Max. Agg. Size, in	Traffic Level	Preparation
1	US 280	Binder	Superpave 424	1	ESAL E[1]	Milling and chip seal
		Wearing Surface	Superpave 424	0.75	ESAL C/D[1]	Patching and chip seal
2	Selma	Binder	Superpave 424	1	ESAL E	Milling
		Wearing Surface	Superpave 424	0.75	ESAL C/D	None
3	US 82	Binder	Superpave 424	1	ESAL E	Milling
		Wearing Surface	Superpave 424	0.75	ESAL E	None
4	Opelika	Binder	SMA 423	1	ESAL E	Milling

[1] ESAL C/D range: $1.0X10^6 \leq$ ESALs < $1.0X10^7$
E range: $1.0X10^7 \leq$ ESALs < $3.0X10^7$ (ALDOT, 2002)

Project 1, 2 and 3 had Superpave bituminous concrete binder and wearing surface layers constructed according to Section 424 of *ALDOT 2002 Specification*. Project 4 had SMA 423 as binder concrete according to Section 395 of *ALDOT 2002 Specification*. These detailed gradation information about these mixtures were presented in somewhere else (Williams, 2003).

3.1.2 Concrete Pavement Profiles

Concrete pavement data were surveyed in Montréal, Quebec. The description of the four concrete sections is stated in Table 3.2 (Carter, 2005). All of these sections are new concrete pavement, except project 1 with short slabs. While project 1 was not new, the concrete pavement was still in its good shape and condition.

Table 3. 2 Descriptions of Concrete Pavements.

Project	Slabs	Texture	Year of Construction	Length of Section, Km
1	Short Slabs	Skid abrader	1997	1.5
2	Continuous Slabs	Transverse Tinning	2004	1.5
3	Continuous Slabs	Transverse Tinning	2004	0.5
4	Continuous Slabs	Longitudinal Tinning	2004	0.5

Since the different climate conditions between Alabama and Quebec, this concrete profile database has the limitation to be used in Alabama smoothness specification development. However, these four concrete pavement projects located in the same urban highway system, had the structure of the typical 9" thick slab, and were built by the same contractor in the recent years. The data based on them can be considered as a homogenous database deriving from newly-constructed concrete pavements. Moreover, Alabama has very few new concrete pavement constructions, which make it difficult to build a new concrete pavement database.

Therefore, concrete pavement database from Québec were just used to compare the way different roughness indices evaluate pavement riding quality. Only asphalt pavement data were used to transfer ALDOT specifications.

3.2 ProVAL

ProVAL was performed as the analysis tool in this study. ProVAL (Profile Viewing and Analysis), published by Federal Highway Administration in 2005, is an engineering software application that allows users to view and analyze pavement profiles in many different ways (ProVAL, 2005). This software can perform various filters to pavement profiles, provide power spectral density information of profiles, simulate profilograph trace and operate the smoothness statistic analysis. Also, ProVAL can complete these analyses with two unit systems: Metric and USCS. Finally, an analysis report can be created automatically. Figure 3.2 shows the major function tab of this software.

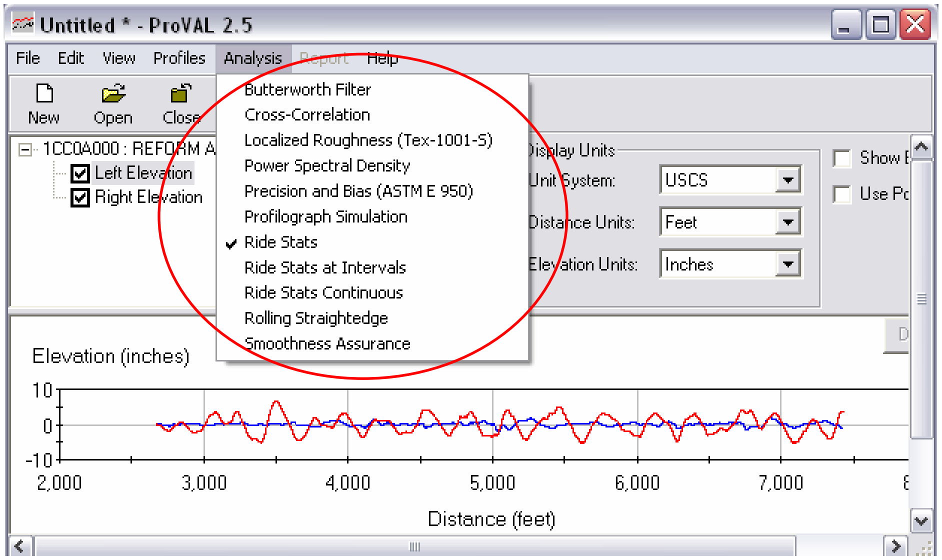


Figure 3. 2 Main Function of ProVAL 2.5.

Profilograph simulation is designed to emulate profilograph traces, like California Profilograph, for the profiles collected using inertial profilers. The default wheel offsets is the geometry of the California rolling system. The algorithm here to calculate the deviations of pavement similarly follows Equation 2.1. The elevation of the referred surface can be computed by averaging the elevations of wheel groups. The deviation of the recording wheel can be calculated from the disparity of its elevation from the surface. The location of the blanking band is determined by the least squares linear fit, which makes the centerline of the blanking band pass through the middle of the profile. Therefore, the blanking band can cover as many of irregularities as possible.

In this software, after setting the input value of blanking band, minimum scallop width, minimum scallop height and scallop rounding increment, the button of **Run Filter** is pressed to perform the Profilograph simulation filter. As a result, the California

Profilograph trace appears on the screen with the default interval set as 0.1 mile (528 feet). If smaller, larger segment or part of the profiles is interested to be analyzed, the **Segments** button allows adding and deleting segments, even changing the desired analysis interval. After the input of all parameters, the **Analyze** button is pressed to run and compute the California Profilograph Index. Consequently, the simulated PI values are calculated for each segment of profiles. In this study, ProVAL2.5 was used to model the California profilograph trace and calculate the PI values in different blanking band (0.0and 0.2 inches) and different segment intervals (0.01 and 0.1 miles).

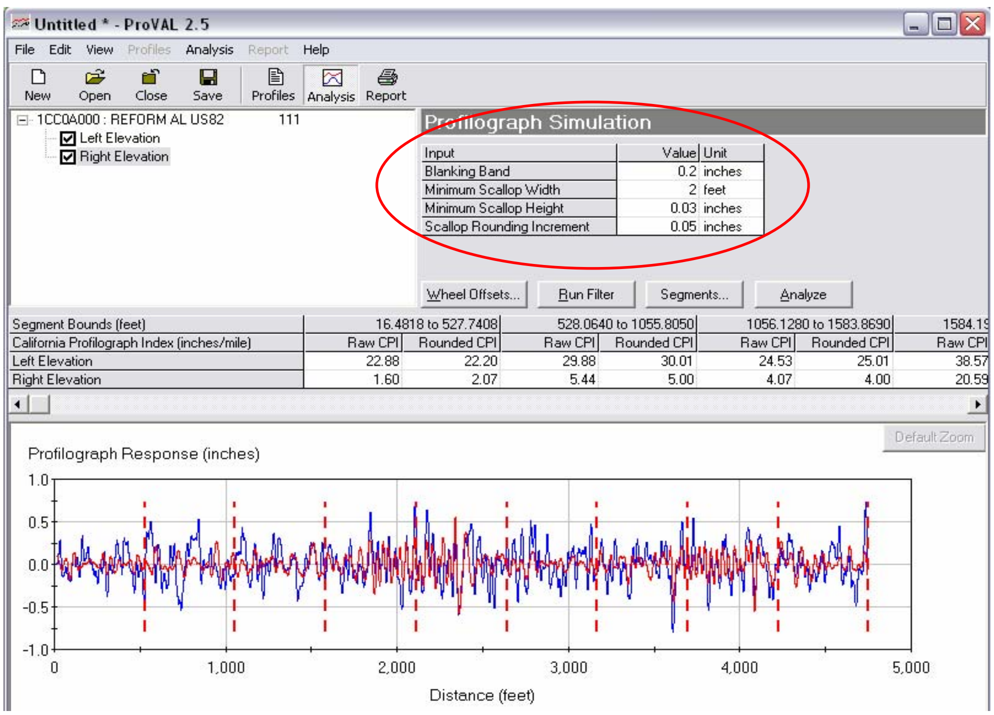


Figure 3. 3 Profilograph Simulation Function Tab.

The second main function of ProVAL is to compute ride statistics, such as International Roughness Index and Half-car Roughness Index, which is the IRI algorithm applied to average of two wheelpath profiles. In ProVAL, the algorithm of quarter-car model is used for calculating IRI value. The raw profile provides the height information of the unsprung and sprung mass body. With parameters of the suspension system and the tire system in quarter-car, the vertical acceleration difference between these two body parts can be computed with integration method.

In ProVAL, the default values of vehicle velocity and segment length are 80 km/h and 528 feet, respectively. If the input profiles are not pre-filtered, the required 250 mm moving average filter or other desired filters can be performed on the raw profiles before further analysis. After that, the **Analyze** button starts to run the analysis. As a result, IRI value of each segment of each wheelpath appears on the screen. Figure 3.4 presents one ride statistics analysis example. This study applied ProVAL to calculate IRI for each segment of each section tested, with 0.01 mile and 0.1 mile segment interval.

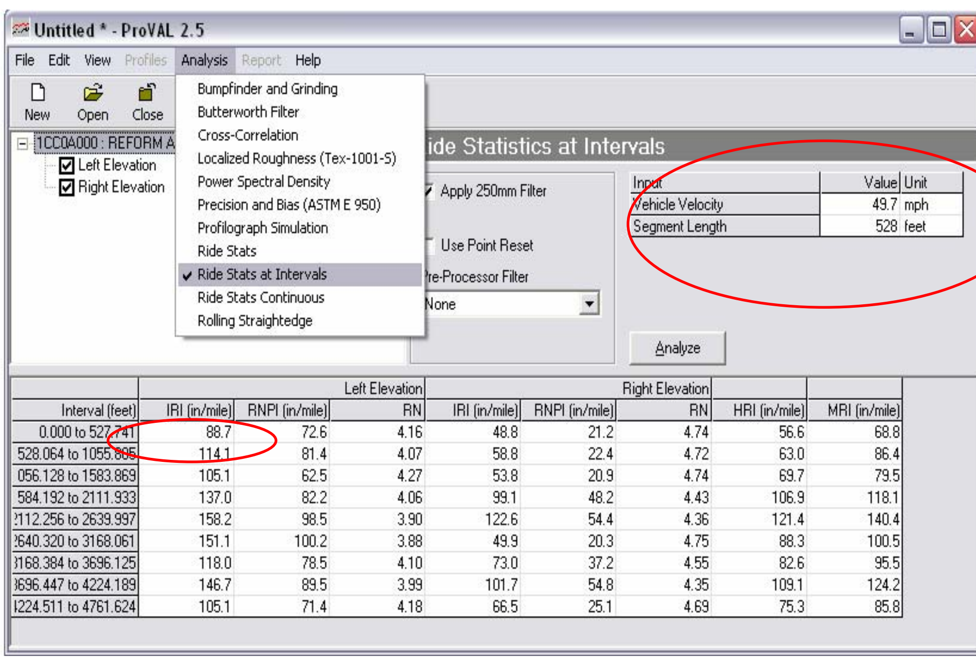


Figure 3. 4 Ride Statistics Function Tab.

CHAPTER FOUR

DATA ANALYSIS

4.1 Data Quality

After profiles were processed using the ProVAL2.5 software, database consisting of IRI and simulated PI values were developed for the further analysis. During the data collection using ARAN Van, sometimes optical triggers were placed on the pavement before the segment collections to indicate the start of another segment. The triggers produced evident peaks on the profiles. To eliminate the effect of those peaks, the remainder of the database were evaluated and deleted as outliers, which were defined as values beyond plus and minus three standard deviations of the average. After removing these abnormal values, the data were plotted in Figure 4.1 to 4.13 to evaluate the quality.

4.1.1 Smoothness Data of Asphalt Concrete (AC) Pavement

Figures 4.1 to 4.7 present the range and distribution of IRI and PI values at each of two intervals (0.1 m, 0.01 mi) for asphalt concrete pavement. These figures demonstrate that PI and IRI values fully cover the range of typically reported smoothness values of new construction and AC overlays (i.e., IRI between 50 to 125 in/mi, $PI_{0.2}$ between 0 and 15 in/mi) (Smith et al. 2002). Therefore, the assembled AC overlay database can be considered as a representative of asphalt overlay pavement projects in Alabama.

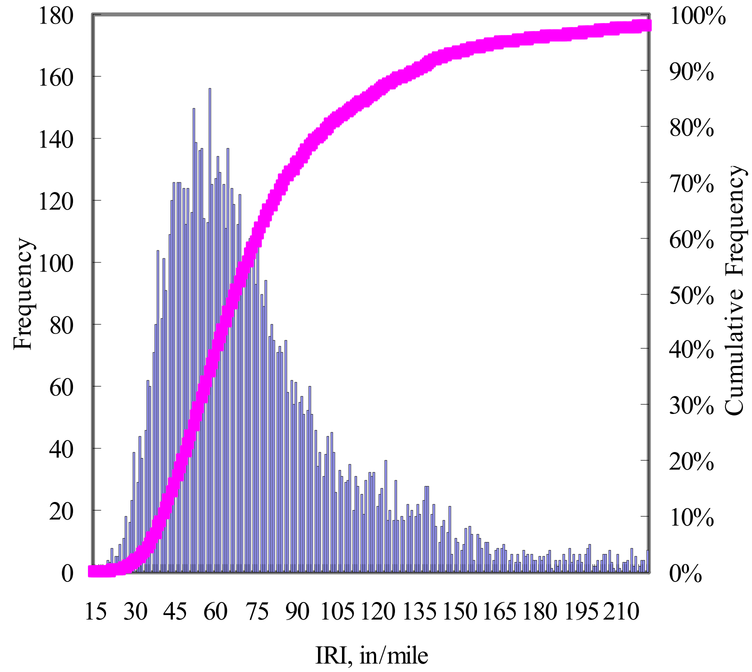


Figure 4. 1 Histogram of AC IRI Value Distribution at 0.01 Mile Interval.

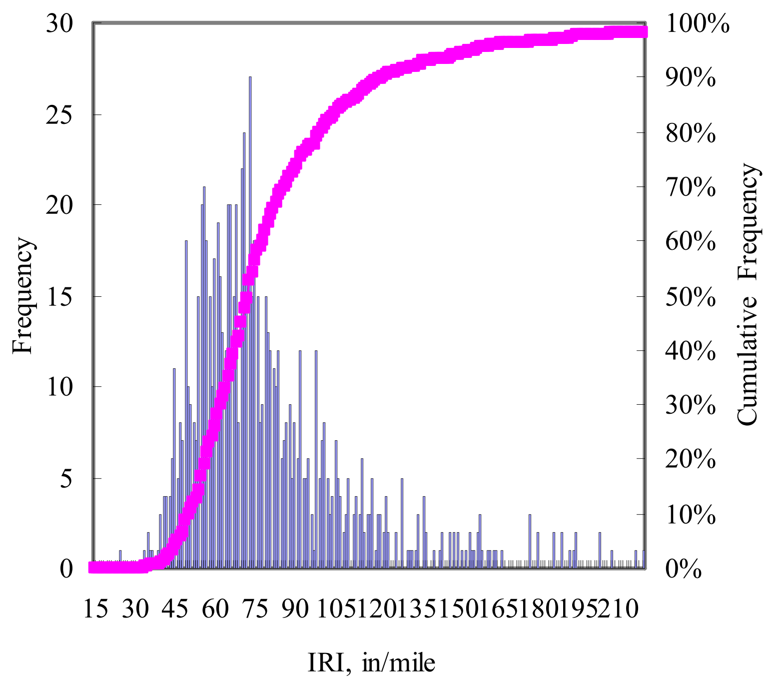


Figure 4. 2 Histogram of AC IRI Value Distribution at 0.1 Mile Interval.

Figures 4.1 and 4.2 indicate that distributions of IRI values at 0.01 mile interval and 0.1 mile interval are similar. The 50th percentile of 0.1 mile interval values is 72 in/mile, 0.01 mile interval values has close 50th percentile of 68 in/mile. As it was expected, the distribution of the IRI values calculated at 0.01 mile interval has a flatter distribution with more data spreading into both tails than 0.1 mile interval. The 0.1 mile interval averages the bumps, and therefore smoothes out the tail in the longer distance to gain standard deviation of 38 in/mile. However, the smaller interval, accounting for the shorter areas with localized irregularities, spreads data to wider tails and has larger standard deviation of 49 in/mile.

As seen in Figure 4.1 and Figure 4.2, a transformation of the database may be helpful in order to obtain a more normally distributed distribution of IRI data. However, as already presented in smoothness specifications, pavements are sorted into four population by its smoothness according to the practical experiences and engineer judgments: very smooth pavement which is the product of excellent construction and is qualified for the incentive, smooth pavement which is the result of qualified construction and would earn the full pay, the rough pavement which is created by the unqualified construction and only achieves parts of the bid price, the very rough pavement which is produced by the poor construction and could not be accepted without correction. Therefore, there would actually be several populations represented by the data, but the separation of different population is not readily evident. There is no sufficient data in these particular projects to provide project-specific information, which is needed to sort each data base into independent databases of low, med, and high roughness.

Compared to the IRI values distribution, $PI_{0.2}$ values have completely different trends, either for 0.01 mile interval or for 0.1 mile interval. For the 0.01 mile interval, Figure 4.3 shows that 64% segments have $PI_{0.2}$ value of 0 in/mi. These data initially appear to have a very limited distribution. However, this appearance is a function of the high frequency of values at 0 in/mi. If 0 in/mile values were taken out, the remaining data present other populations (Figure 4.4).

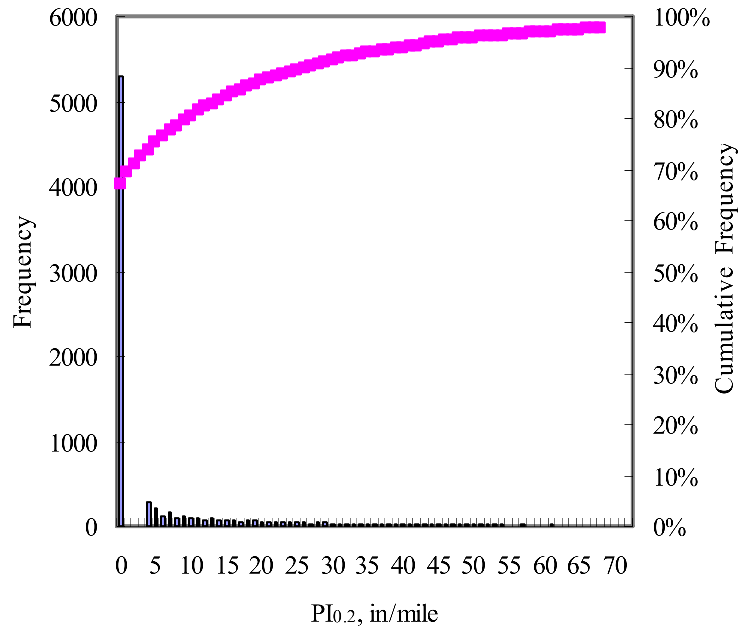


Figure 4. 3 Histogram of AC $PI_{0.2}$ Value Distribution at 0.01 Mile Interval.

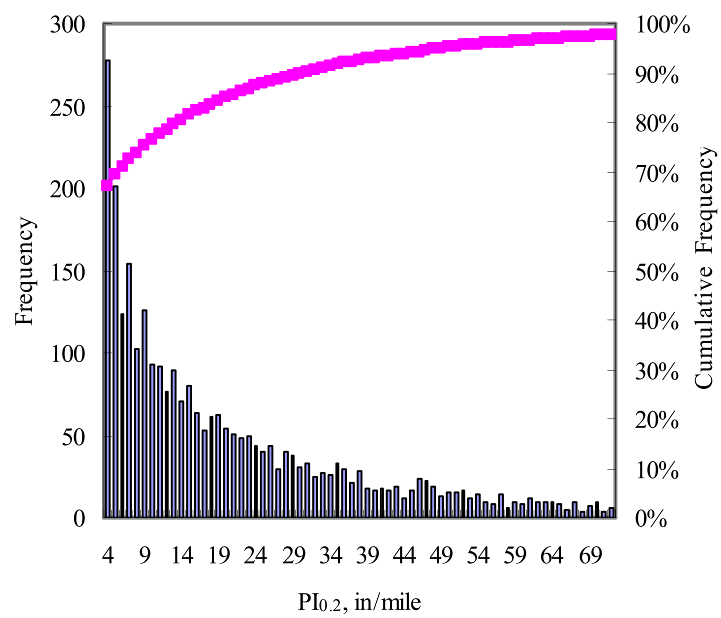


Figure 4. 4 Histogram of AC $PI_{0.2}$ Value at 0.01 Mile Interval after Taking out $PI_{0.2}$ Values of 0 in/mile.

The data distribution for 0.1 mile interval shows that $PI_{0.2}$ values calculated using at 0.1 mile interval comprise 18.5% of the segments having a value less than 2 in/mi. These segments would qualify for a 5% bonus by the pre-2003 specifications. Based on pre-2003 specifications, 13% segments associated with $PI_{0.2}$ from 2 to 4 in/mile can receive full pay; 32% segments would have deducted pay; 36.5% segments are unacceptable without correction.

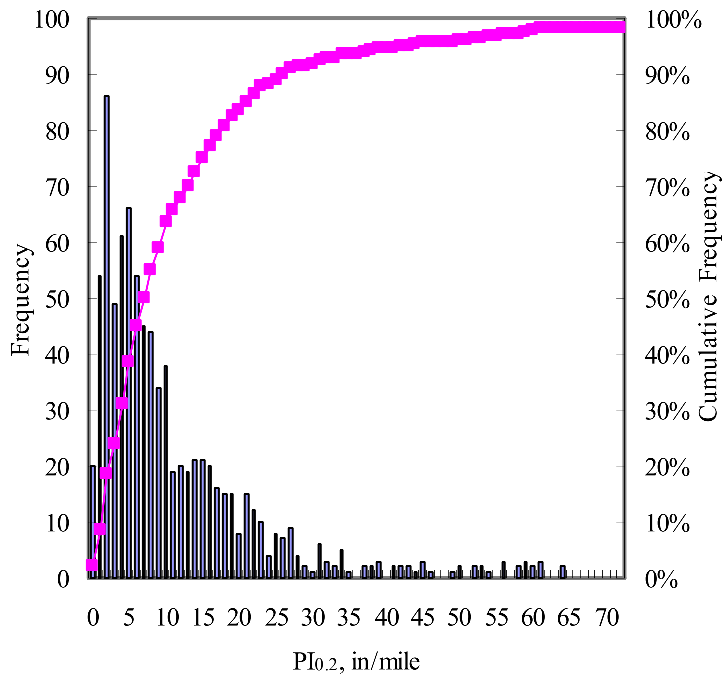


Figure 4. 5 Histogram of AC $PI_{0.2}$ Value Distribution at 0.1 Mile Interval.

Figure 4.6 and figure 4.7 present distributions of $PI_{0.0}$ values, like IRI distributions are skewed to the left. The 50th percentile is associated with a $PI_{0.0}$ of 32 in/mi when using an interval of 0.01 miles. The current specified interval of 0.1 mile shows 50th percentile of $PI_{0.0}$ value is 27 in/mile. 0.1 mile interval also has smaller standard deviation of 18 in/mile than 0.01 mile standard deviation of 28 in/mile. Smaller interval moves more data to the tails of the distribution and creates higher standard deviation.

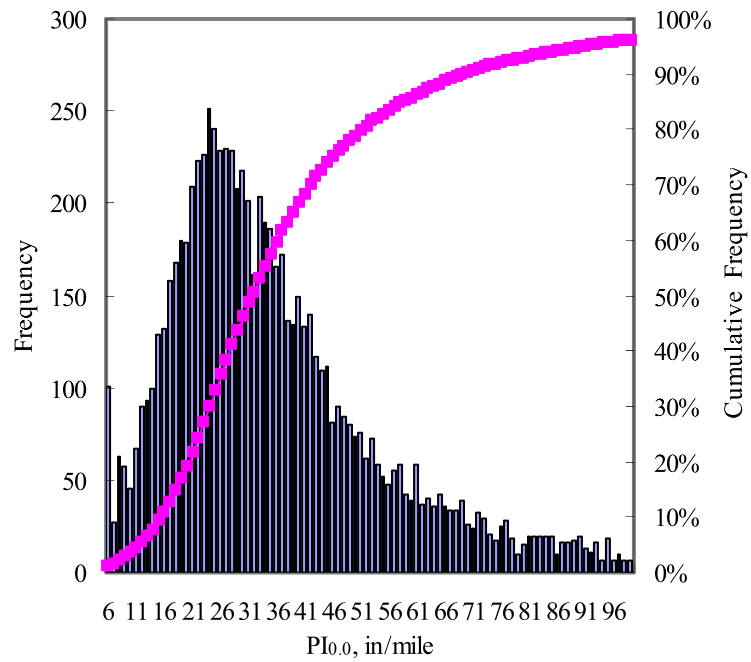


Figure 4. 6 Histogram of AC $PI_{0.0}$ Value Distribution at 0.01 mile interval.

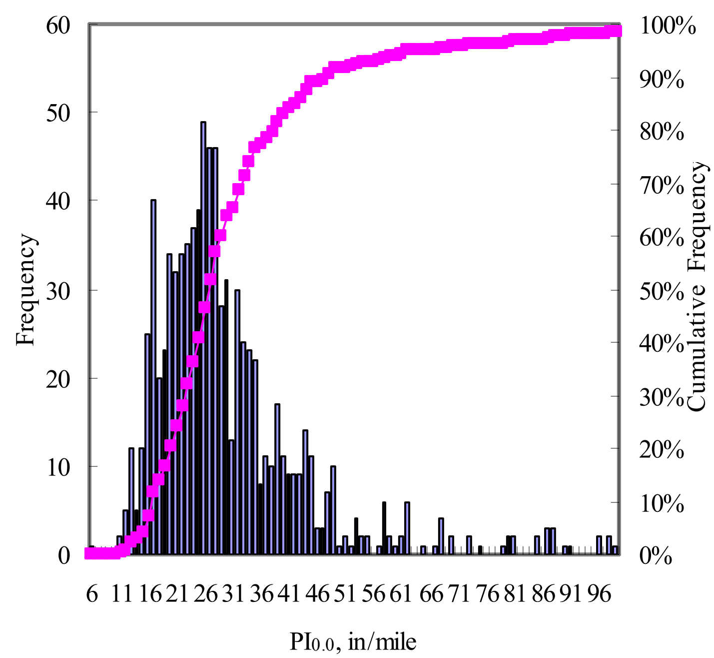


Figure 4. 7 Histogram of AC $PI_{0.0}$ Value Distribution at 0.1 Mile Interval.

4.1.2 Smoothness Data of Portland Cement Concrete Pavement

Figure 4.8 to Figure 4.13 demonstrate the range of PI and IRI values for a range of differently textured PCC pavements at 0.1 mile and 0.01 mile intervals.

According to these figures, it can be seen that PI and IRI values fully cover the range of typical smoothness values of newly constructed PCC pavement (i.e., IRI between 50 to 150 in/mi, $PI_{0.2}$ between 0 and 25 in/mi) (Smith et al. 2002). Therefore, this new PCC pavement database can be considered as one representative of new PCC pavement.

As seen from figure 4.8 and figure 4.9, IRI values of concrete pavement at both 0.1 mile interval and 0.01 mile interval have slightly skewed distributions, with 50th percentile around 95 in/mi. Like IRI value distributions of AC pavement, the 0.01 mile interval has a larger standard deviation than the 0.1 mile interval, which flattens the distribution curve and brings more segments into the right side tails. IRI values using 0.01 mile interval have a standard deviation of 40 in/mile; the 0.1 mile interval has a standard deviation of 20 in/mile.

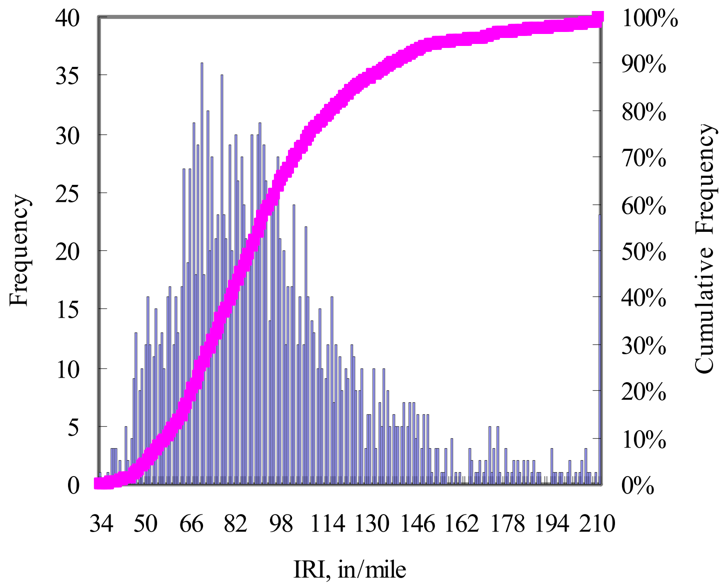


Figure 4. 8 Histogram of IRI Value Distribution of PCC at 0.01 Mile Interval.

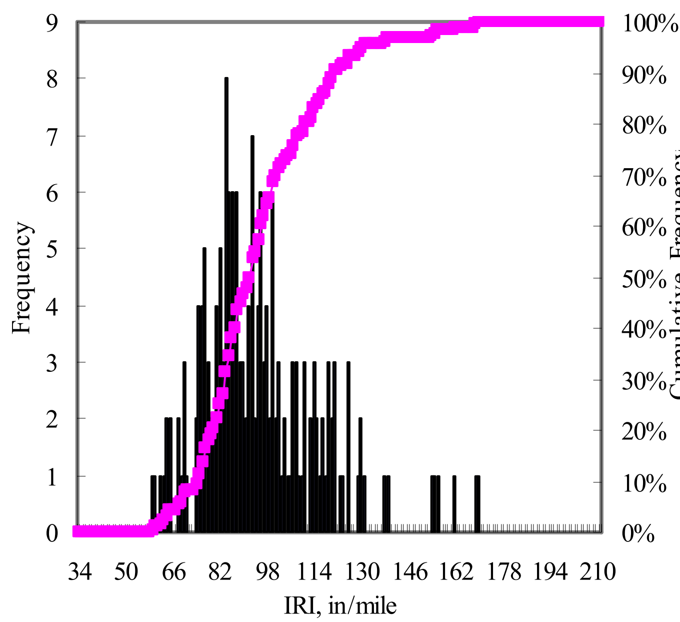


Figure 4. 9 Histogram of IRI Value Distribution of PCC at 0.1 Mile Interval.

The $PI_{0.2}$ data distribution of concrete pavement is also similar to asphalt pavement. When the interval changes from 0.1 mile to 0.01 mile, 50% segments focus on the $PI_{0.2}$ of zero. This emphasizes 0.2 inches blanking band is unable to record small roughness and produces a large percent of segments reaching the bonus.

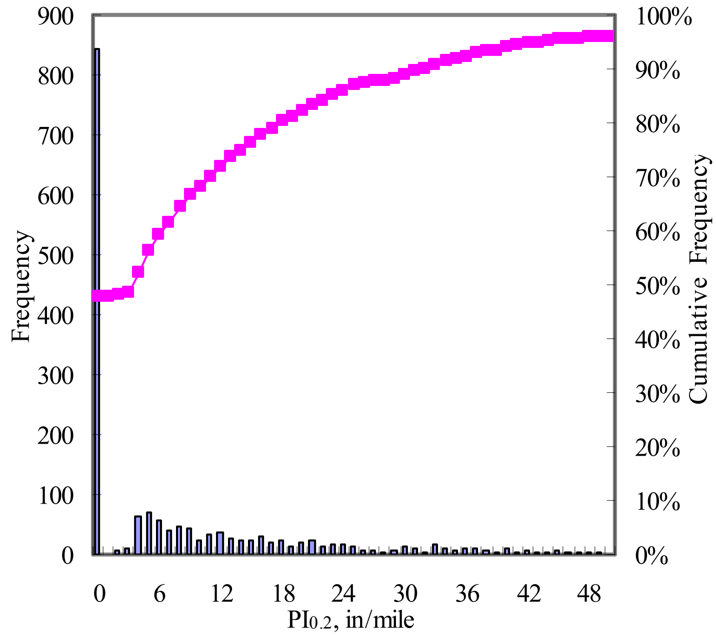


Figure 4. 10 Histogram of $PI_{0.2}$ Value Distribution of PCC at 0.01 Mile Interval.

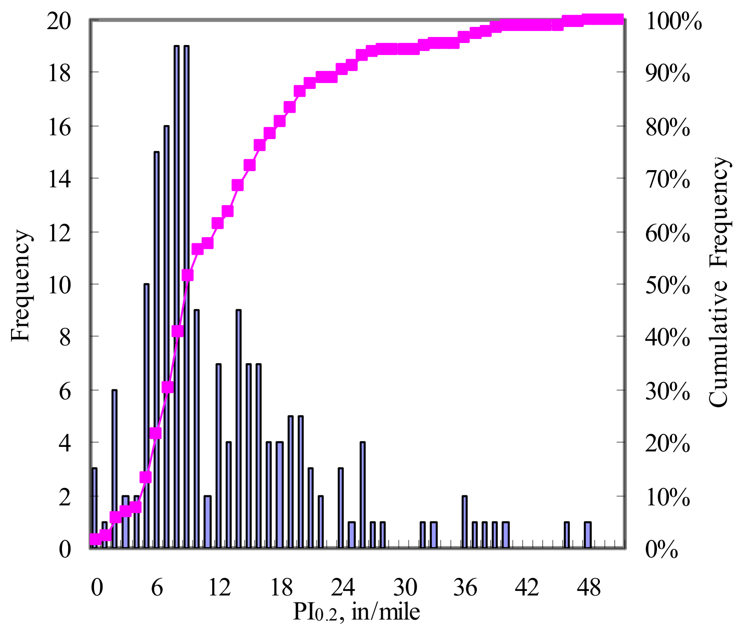


Figure 4. 11 Histogram of $PI_{0.2}$ Value Distribution of PCC at 0.1 Mile Interval.

As for $PI_{0.0}$ data of 0.1 mile or 0.01 mile interval, concrete pavement also has slightly skewed distributions. $PI_{0.0}$ values have almost same shape of distribution curves with IRI.

0.1 mile interval generates the $PI_{0.0}$ value of concrete pavement with 50th percentile of 41 in/mile, with a standard deviation of 13 in/mile. The 0.01 mile interval creates larger average of 44 in/mile and larger standard deviation of 23 in/mile. Unlike the $PI_{0.2}$, the different intervals present dissimilar distribution patterns; IRI and $PI_{0.0}$ have a similar pattern either for 0.1 mile interval or 0.01 mile interval.

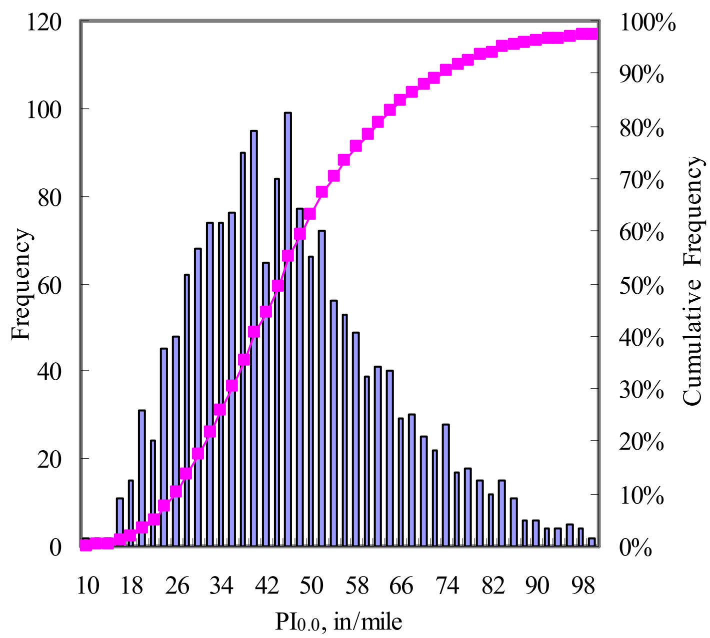


Figure 4. 12 Histogram of $PI_{0.0}$ Value Distribution of PCC at 0.01 Mile Interval.

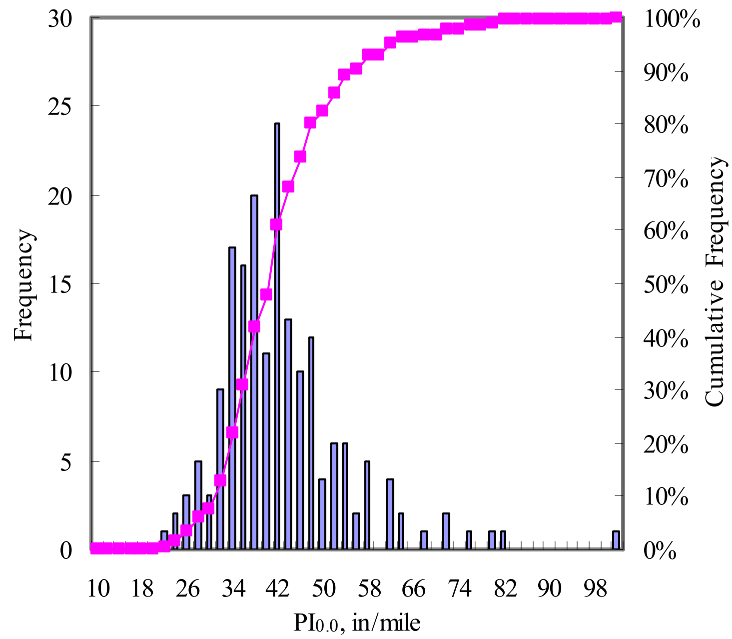


Figure 4. 13 Histogram of $PI_{0.0}$ Value Distribution of PCC at 0.1 Mile Interval.

4.2 Effect of blanking band on Evaluating Pavement Smoothness

Numbers of states still use $PI_{0.2}$ in the quality assessment, especially for concrete pavement. As known in literature review, 0.2" blanking band covers some components of pavement roughness. And also, the same specifications were recommended for both AC and PCC pavements (Smith et al. 1997), so it is meaningful to see whether this blanking band has the same influence on the AC pavement and PCC pavement.

Since the database in this study came from limited projects, there are limitations for these data to represent the roughness feature of the whole new pavements. Therefore, the emphasis here focuses on the comparison of the effects of different roughness indexes, not the comparison of the roughness of different pavement type.

From $PI_{0.2}$ distributions of AC and PCC pavements in Figure 4.14, it can be seen that these two groups of AC and PCC pavements have close roughness condition based on

$PI_{0.2}$. If paid by Alabama Pre-2003 specification, contractors from both industries can achieve similar degree of pay for providing the $PI_{0.2}$- based ride quality. There would be approximately 31% asphalt segment ($PI_{0.2}$ between 0 and 4 in/mi) and 22% concrete segment ($PI_{0.2}$ between 0 and 6 in/mi) receiving full pay or bonus. 32% asphalt segments and 30% concrete segments would get penalty price. 34% asphalt segments and 44% concrete segments ($PI_{0.2}$ larger than 10 in/mile) need to be corrected.

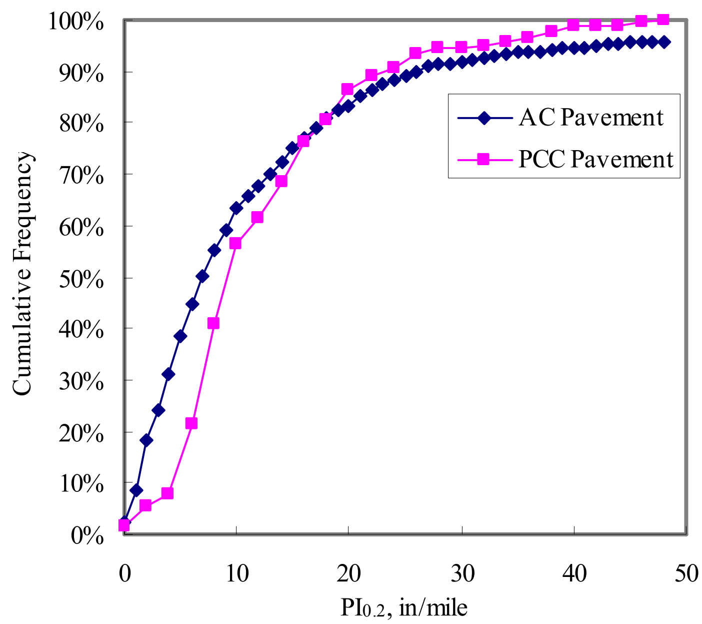


Figure 4. 14 Comparison of $PI_{0.2}$ of AC and PCC Pavement at 0.1 Mile Interval.

However, the $PI_{0.0}$ cumulative frequency curves display a large disparity between these two groups of asphalt pavements and concrete pavements (Figure 4.15). When using the old $PI_{0.2}$ specification, similar pay for asphalt and concrete pavements could be obtained. But for the same pavement profile database, the current $PI_{0.0}$ specification highlights the rougher service provided by these concrete pavements compared to asphalt pavements in this study. Following the current Alabama $PI_{0.0}$ specification, 20% of the AC projects would receive full pay while 0% of the PCC projects would receive full pay.

Concrete pavements have 85% segments get disincentive pay and 15% segments need extra correction.

Based on $PI_{0.0}$ values, contractors of those concrete pavements would need a large improvement in construction procedures to achieve the same ride quality and earn the same pay as those AC pavement contractors. Although smoothness specifications need to provide fair competition between asphalt pavement and concrete pavement industry, there is no reason to accept worse ride quality with the same pay.

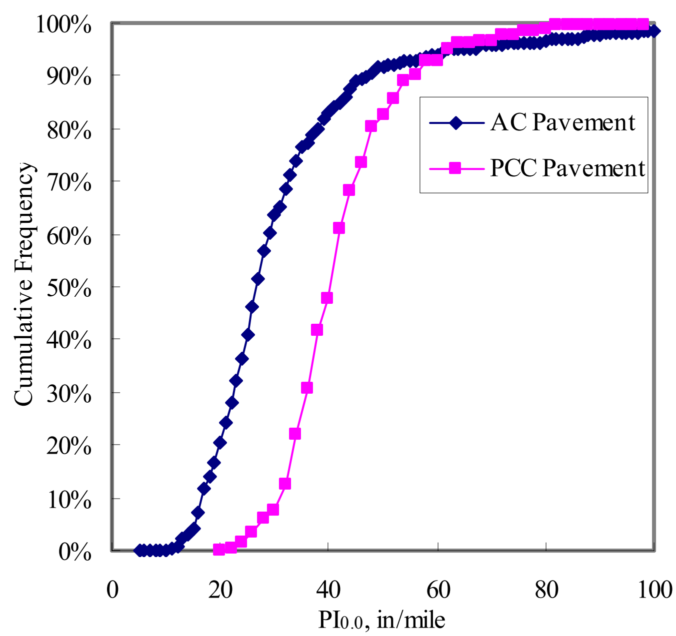


Figure 4. 15 Comparison of $PI_{0.0}$ of AC and PCC Pavement at 0.1 Mile Interval.

Figure 4.16 also shows an evident difference between the IRI value distributions of asphalt pavement and concrete pavements. Assumed that full-pay upper limits of IRI is set on 75 in/mile, only 12% of the concrete segments could achieve the full pay, while 62% asphalt segments would be qualified for 100% pay.

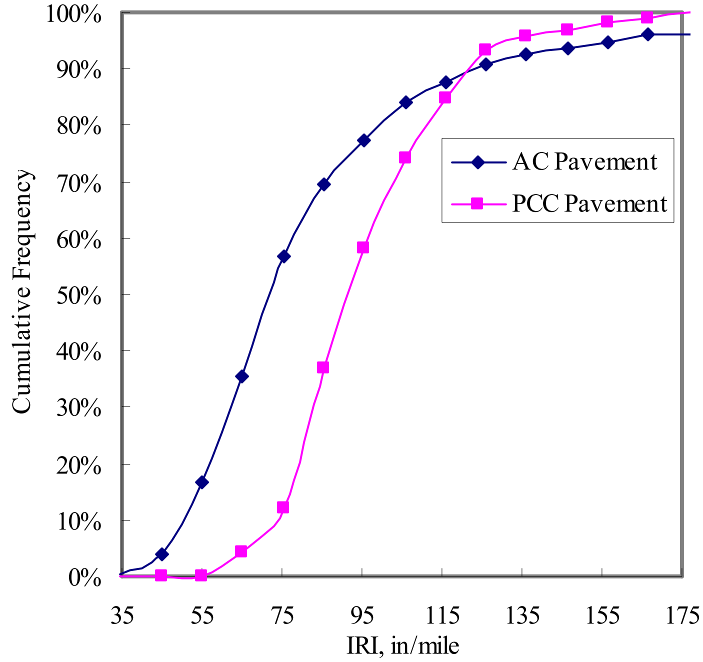


Figure 4. 16 Comparison of IRI of AC and PCC Pavement at 0.1 Mile Interval.

These two comparisons further prove that PI with 0.2 inches blanking band makes the small roughness unnoticeable and moves segments to "smooth" level. As $PI_{0.0}$ and IRI are more sensitive to smaller vertical displacements, those segments defined as "smooth" by $PI_{0.2}$ would be considered as rough segments by both the $PI_{0.0}$ and IRI.

Although 0.2" blanking band covers the small defects for both pavements, 0.2" blanking band has different effect on evaluating the smoothness of asphalt and concrete pavements in this study. Based on the database developed in this study, it shows that more amounts of irregularities from those concrete pavements are concealed by 0.2" blanking band than these AC pavements. 0.2" blanking band allows the worse-quality PCC pavement to earn the same pay as AC pavement. Accordingly, those concrete pavements contractors need more rapid improvement on the roughness measurement and smoothness specification. Deleting the blanking band would promote smoother concrete pavements.

4.3 Conversion of PI Specifications to IRI Specifications

4.3.1 Specification Conversion Using Regression Equations

As indicated by the scatter plot in Appendix A and the previous research about the correlations between PI_X and IRI, the simple linear relation model was chosen to describe the relationship between PI_X and IRI. The model is shown in equation 4.1.

$$IRI = \beta_0 + \beta_1 * PI_X \qquad \text{(Equation 4.1)}$$

Where,

IRI = International Roughness Index, in/mile

PI_X = Simulated Profile Index for blanking band x (x= 0.0, 0.1 or 0.2 inches), in/mile

β_0, β_1 = Regression parameter

In the 2002 LTPP study, regression equations from different climate zones have significant differences between each other. Asphalt pavement data used in this study was collected in Alabama. This corresponds with the LTPP population of asphalt pavement in the wet no-freeze (WNF) climate zone. The equations based on this database should be applicable to the profiles obtained for this study.

The concrete database used in this study was gathered at Quebec, Canada, which is located in wet-freeze (WF) climate zone. Due to the climate limitation and other construction or material difference between Quebec PCC pavement and Alabama PCC pavement, the regression equations developed on this database could not adapt to Alabama. So the correlation model of WNF zone PCC pavement in 2002 LTPP was applied here to transfer Alabama specifications.

By following models of equation 4.1, the regression equations for asphalt pavement were developed and shown in Table 4.1. Compared with the regression equations from 2002 LTPP study (asphalt pavement at 0.1 mile interval), the equations developed in this study and those for the LTPP study have similar intercepts: 55 in/mile for the 0.2" blanking band and 18 in/mi for 0.0" blanking bands. The slopes between IRI and PI_x from the 2002 LTPP equations are slightly higher than those found in this study.

Equations for 0.01 mile interval are distinct from 0.1 mile interval equations, with a slightly higher slope and a noticeably higher intercept. The short interval creates the database with a higher variation than 0.1 mile interval, contributing to the smaller R^2.

Table 4. 1 Correlation Equations between IRI and PI in this Study and LTPP (Asphalt Overlay Pavement).

	Climate Zone	Number of segments	Interval (mile)	Correlation Equation (IRI,PI=in/mile)	
				$PI_{0.2}$	$PI_{0.0}$
This Study	WNF[a]	8332	0.01	$IRI=1.9295*PI_{0.2}+62.82$, $R^2=0.70$	$IRI=1.5699*PI_{0.0}+19.91$, $R^2=0.79$
		869	0.1	$IRI=2.3688*PI_{0.2}+54.10$, $R^2=0.91$	$IRI=2.0708*PI_{0.0}+17.84$, $R^2=0.92$
LTPP(2002)	WNF	5126	0.1	$IRI=3.4267*PI_{0.2}+55.54$, $R^2=0.63$	$IRI=2.4230*PI_{0.0}+19.12$, $R^2=0.84$

[a] WNF: Wet-Nonfreeze climate zone

Table 4.1 shows that regression equations for asphalt pavement in this study have high significance of regression with R^2 values consistently above 0.9. Even for 0.01 mile interval, regression models still have a good R^2 (around 0.75). In other words, 75% change of IRI can be explained by the linear change of PI. Using these developed equations, the current Alabama asphalt pavement PI based specification could be reasonably transferred to IRI based specification. Table 4.2 presents the converted IRI-based specification results. The continuous pay factor functions were retained through these regression models.

Table 4. 2 Converted IRI Specifications for Asphalt Pavement at 0.1 Mile Interval by Regression Equations.

Price Adjustment of Pavement Unit Bid Price by $PI_{0.2}$	$PI_{0.0}$, in/mi	IRI, in/mi	Price Adjustment of Pavement Unit Bid Price by IRI
105-(PI/20)	Under 10	Under 38	109.3 - 0.24*IRI
100	10 to 20	38 to 60	100
100-(PI-20)/1.5	20 to 50	60 to 121	119.1 - 0.322*IRI
Unacceptable	Over 50	Over 121	Unacceptable

Owning to the absence of Alabama rigid pavement data, the linear regression model of WNF zone PCC pavement from 2002 LTPP study (Table 2.1) were used for concrete pavement smoothness specification transfer. According to regression equations established for WNF climate zone from 2002 LTPP study, the current ALDOT concrete pavement $PI_{0.0}$ specifications were changed to IRI base specification, shown in table 4.3.

Table 4. 3 IRI Specifications for PCC Pavement.

Price Adjustment of pavement Unit Bid Price by $PI_{0.2}$	$PI_{0.0}$, in/mi	IRI, in/mi (LTPP)	Price Adjustment of pavement Unit Bid Price by IRI
105-(PI/20)	Under 10	Under 44	112 - 0.24* IRI
100	10 to 20	44 to 67	100
100-(PI-20)/1.5	20 to 50	67 to 138	119 – 0.282* IRI
Unacceptable	Over 50	Over 138	Unacceptable

4.3.2 Specification Conversion Using Distribution Method

Pavements with different smoothness levels can be paid for different percentages of the initial bid: bonus pay, full pay or penalty pay. For contractor, if an existing smoothness specification is converted to new specification based on another index, the same pavement product is expected to receive the same pay either based on former smoothness index or new one. But for the agency and the public, the transfer of smoothness index is for more accurately evaluating the pavement roughness and promoting the good construction. If the pavement product does not improve the driving comfort but is paid the incentive by the former index, its payment needs to be adjusted in the new index.

The distribution method is to transfer specification limits between different indices by using the concept that each index will have the same number of segments in the same payment level. The percentages of bonus, full or penalty pay pavement determined by the former specifications are used as the reference to start a new specification. This conversion makes the change of evaluation system comfortable for contractors, but it also makes the public having the risk to receive the worse paving product with paying the same amount.

Therefore, the result from the distribution method is just a first step to establish the new specifications. With the application of this primary result, the further adjustments are needed to decide the reasonable percentage of pavement having incentive/disincentive. Herein, the distribution method provided a primary result for moving PI-based specifications to IRI-based; the further adjustment is out of the range of this study.

The distribution curves of $PI_{0.0}$ were employed to determine the number of segments at different pay levels: the incentive range, full pay range, disincentive range and the unacceptable range, respectively. Consequently, the limits of IRI specifications can be determined by having the same number of segments in each roughness level as $PI_{0.0}$ specifications.

Due to the current ALDOT $PI_{0.0}$ specification, 0% segment of asphalt pavement projects used in this study would reach the bonus pay; $PI_{0.0}$ range for full payment is from 0^{th} to 20^{th} percentile. It should be pointed out that the projects in this study were all mill and overlay over existing distressed HMA pavements, which is a contributing factor to the contractors' ability to restore a new pavement ride. Figure 4.17 also indicates that

71% of asphalt segments would have a penalty pay, while 9% would be unacceptable without correction.

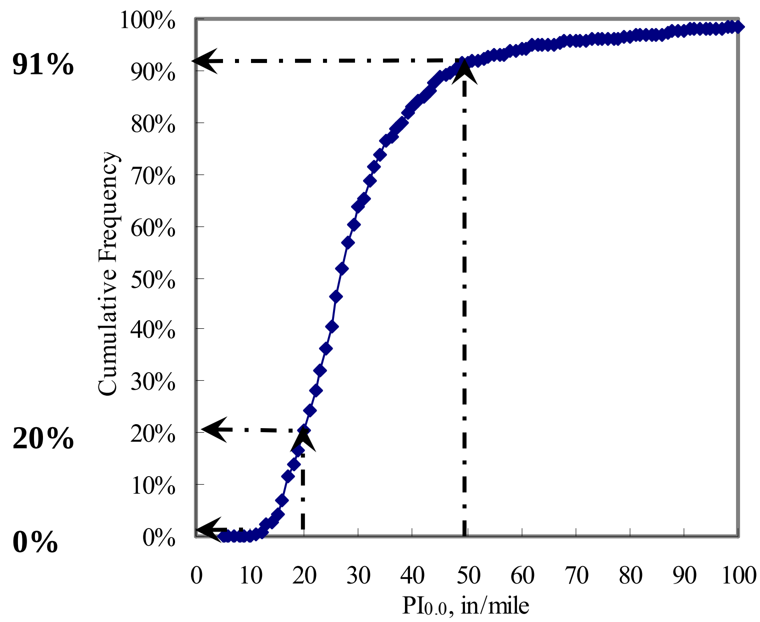


Figure 4. 17 Pavement Percentages of AC Pavements in Each Pay Level according to $PI_{0.0}$ Specifications.

Since these data profiles of asphalt pavement come from overlay projects, overlay pavements are possibly rougher than totally new-constructed pavements. That is one of the reasons that just small amount of the asphalt pavement segments in this database are reached bonus or full pay limits.

Another reason is the strict requirement of $PI_{0.0}$ full pay limits in current ALDOT specification ($PI_{0.0}$ value from 10 in/mile to 20 in/mile), which results in small number of full-pay segments and bonus-pay segments. Figure 2.12 shows that the lower limit of full pay from other DOTs is 17 in/mile; the upper limit of full pay from other DOTs is 27 in/mile. This means that currently ALDOT specifications are stricter than most other DOT ride quality specifications, which suggests that a little lenient range in limits could be more reasonable.

If the ALDOT specification is adjusted to the average limits of full pay range from other DOT ($PI_{0.0}$ value from 17 in/mile to 27 in/mile), 11% asphalt segments would achieve a bonus, 40% segments would earn full pay, 40% segments would receive penalty payment and 9% segments would need to be corrected.

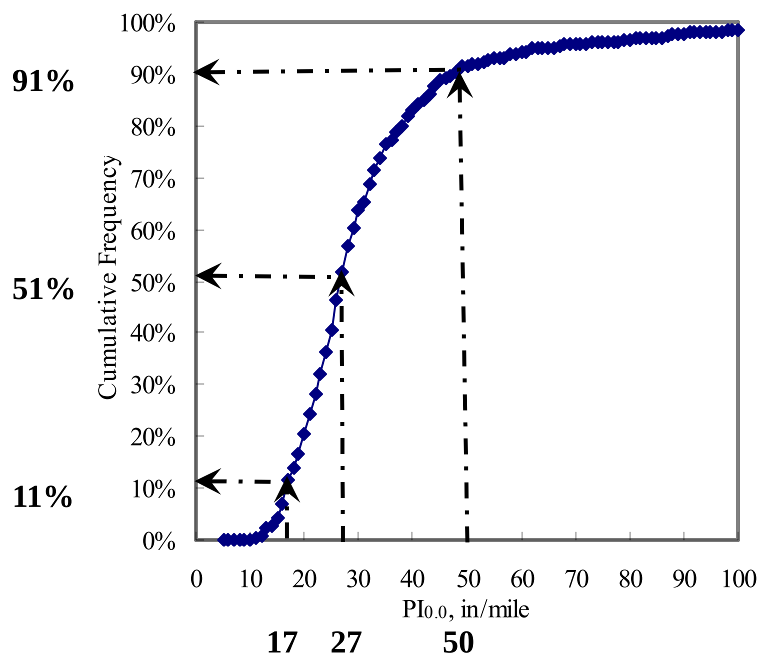


Figure 4. 18 Pavement Percentages of AC Pavements in Each Pay Level according to Adjusted $PI_{0.0}$ Specifications.

According to the percentage ranges calculated after adjusting the $PI_{0.0}$ limits for different pay levels, the limits of IRI-based specifications were determined for having the same number of segments for each pay level. In order to have 11% segments receiving the incentive, the lower limit of full pay range for IRI equals to 52 in/mile based on the cumulative frequency curve. The upper limit of full pay range for IRI is 72 in/mile for having 40% full-pay segments. The upper limit of penalty range is 128 in/mile to make 9% segments unacceptable. Figure 4.19 and Table 4.4 presents the result of the limits of each pay range for IRI at 0.1 mile interval.

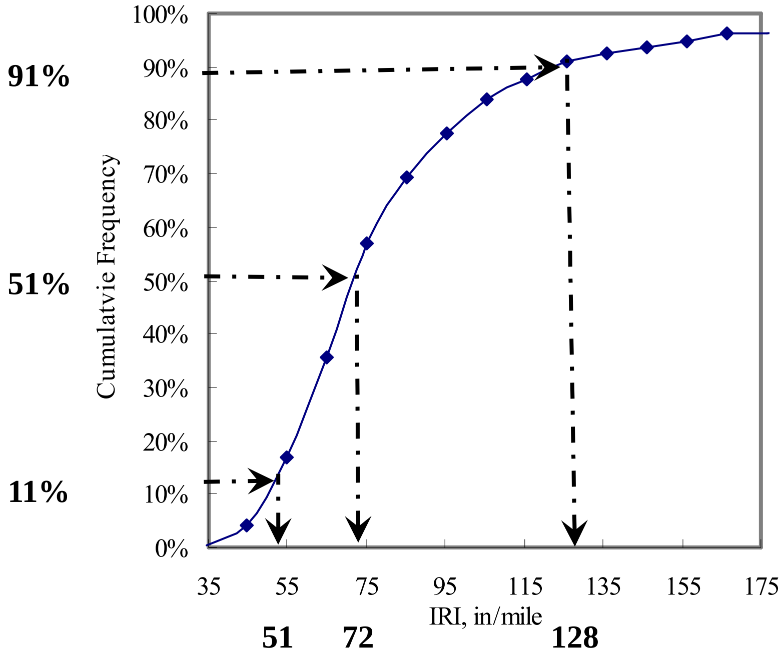


Figure 4. 19 Limits of Each Pay Range for IRI

Table 4. 4 Converted IRI Specifications for AC Using Distribution Method.

Price Adjustment of pavement Unit Bid Price	Current $PI_{0.0}$ at 0.1 mile, in/mile	Adjusted $PI_{0.0}$ at 0.1 mile, in/mile	Percent of segments in different pay level based on adjusted $PI_{0.0}$	$PI_{0.0}$ at 0.01 mile, in/mile	IRI at 0.1 mile, in/mile	IRI at 0.01 mile, in/mile
105 – (PI/20)	Under 10	Under 17	11.5%	Under 17	Under 51	Under 42
100	10 to 20	17 to 27	40%	17 to 32	51 to 72	42 to 68
100-(PI-20)/1.5	20 to 50	27 to 50	40%	32 to 74	72 to 128	68 to 140
Unacceptable	Over 50	Over 50	8.5%	Over 74	Over 128	Over 140

Moreover, the statistical relationships between 0.1 mile and 0.01 mile smoothness indices were developed during the analysis for possible use of the smaller interval for localized bump detection in further studies.

4.3.3 Effect of Material Transfer Devices (MTD) on Asphalt Pavement Smoothness

One of the important purposes of smoothness specification is to encourage contractors provide better products and pursue higher payment by employing new technologies. Hence the payment level should be set to motivate contractors to use these technologies. During the asphalt pavement paving projects, material transfer devices, also called remixers, are proven to play an important role on decreasing the material segregation and yielding smooth pavement (Roberts et al., 1996). MTD is used between the paver and the loading truck in the construction. Because of it, the paver can process the paving at a more uniform speed with less stop. MTD also remixes the material before supplying them to the paver and decreases the segregation of the materials.

In this study, the pavement smoothness data were collected from paving projects using MTD and projects without MTD. Figure 4.20 plots the distributions of pavement smoothness data at 0.1 mile interval with and without using MTD. The figure shows that MTD has a strong affect on the distribution of segments having IRI value less than 70 in/mile.

Paving projects with using MTD provide 26% segments having IRI value less than 55 in/mile, but only 5% segments in paving projects without MTD have IRI value less than 55 in/mile. IRI value of 55 in/mile reveals the biggest disparity between projects with MTD and without MTD. Consequently, IRI value of 55 in/mile is a good value as incentive limit to encourage contractors to pursuit the incentive with using MTD.

Projects constructed without MTD or with MTD but not using best paving practices would both have penalties assessed when the IRI is greater than 70 in/mi. Given that the cost of purchasing, using, and maintaining a MTD is high; it is to the contractors' advantage to make sure that the equipment is used properly. Alternatively, lower traffic volume roadways can have a higher initial IRI and still be considered acceptable. It is also difficult to use some of the MTD equipment in single lane paving operations, as is common on two-lane roadway paving. In this case, projects that would be acceptable with an IRI of 70 in/mi would not use an MTD, which would result in a lower bid for the agency and both less capital cost and maintenance for the contractor. IRI value of 70 in/mile is a good value as the upper limits of full pay. With the proper paving practices, contractor can provide the IRI less than 70 in/mile and achieve the 100% pay, either using MTD or not.

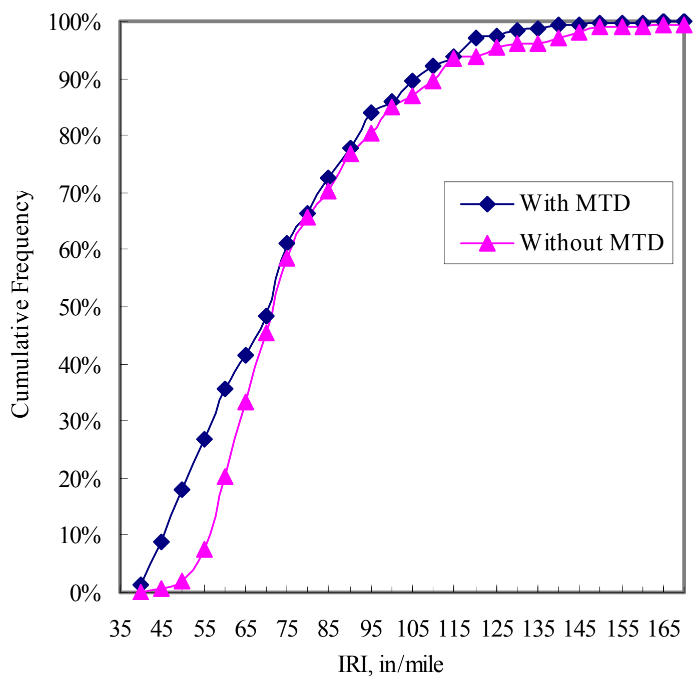


Figure 4. 20 Effect of MTD on Pavement Smoothness at 0.1 mile interval.

4.4 IRI-based Specification

The converted IRI specifications based on the above methods and analysis provides the reasonable references to determine the final IRI specifications recommendation. Currently, most of states still use 0.1 mile as the test interval, and 0.01 mile interval is just employed to further detect localized bump for some states (WFLHD, 2003). Therefore, this study recommends the IRI based specification at 0.1 mile interval; leaving the 0.01 mile interval specification for bump detection for future development.

From the preceding analysis, Table 4.5 provides the combined analysis results from regression method, distribution method, literature review, and effectiveness of a material transfer device.

Table 4. 5 Combination of Converted IRI Specifications.

Price Adjustment of Pavement Unit Bid Price by $PI_{0.2}$ at 0.1 mile interval	Current $PI_{0.0}$ at 0.1 mile, in/mile	IRI at 0.1mile from AC Regression, in/mi	IRI at 0.1mile from PCC Regression, in/mi	IRI at 0.1 mile from AC Distribution Method, in/mile	IRI at 0.1 mile suggested by MTD application, in/mile
105 – (PI/20)	Under 10	Under 38	Under 44	Under 51	Under 55
100	10 to 20	38 to 60	44 to 67	51 to 72	55 to 70
100-(PI-20)/1.5	20 to 50	60 to 121	67 to 138	72 to 128	------
Unacceptable	Over 50	Over 121	Over 138	Over 128	------

Table 4.5 indicates that transferred IRI specifications developed from both methods reach similar conclusions. Asphalt pavement and concrete pavement also have close smoothness limits after conversion. In addition, the analysis result of MTD effects, for asphalt pavement at 0.1 mile interval, that IRI of 55in/mile is suitable for incentive limit and IRI of 70 in/mile is for 100% pay upper limit, also closely follow the converted IRI specifications by other methods. Since the continuous specification is more accurate to evaluate the pavement smoothness than stepped pay specifications, continuous functions were also considered in the recommendations for an IRI-based specification. Balancing the final recommendation to account for these limitations, the final IRI specifications for asphalt pavement at 0.1 mile interval were determined in Table 4.6.

Table 4. 6 IRI Specification at 0.1 mile interval.

Price Adjustment of Pavement Unit Bid Price by $PI_{0.2}$ at 0.1 mile interval	Current ALDOT $PI_{0.0}$ at 0.1 mile, in/mile	Suggested IRI at 0.1 mile interval, in/mile	Price Adjustment of Pavement Unit Bid Price by IRI
105 – (PI/20)	Under 10	Under 55	112 -0.22*IRI
100	10 to 20	55 to 70	100
100-(PI-20)/1.5	20 to 50	70 to 110	121-0.37*IRI
Unacceptable	Over 50	Over 110	Unacceptable

4.5 Comparison of Converted IRI Specification with Other DOT's Specifications

Since some other DOTs have applied IRI in evaluating pavement roughness, current specifications from other DOT were plotted together to verify the feasibility of transferred IRI smoothness specification for Alabama. All of the DOT specifications included for comparisons in Figures 4.21 use a 0.1 mile segment interval to test pavement smoothness.

Figure 4.21 shows that most of IRI full pay ranges are from 55 to 85 in/mile. The full pay range of transferred Alabama IRI-based specification is from 55 to 70 in/mile, which belongs the typical full pay range. It is also seen in Figure 4.21 that compared with other states expect Maine, the transferred Alabama IRI specifications pay less bonus for the smooth pavement with IRI value less than 55 in/mile, and make a higher penalty for the pavement roughness higher than IRI of 85 in/mile.

Therefore, the transferred Alabama IRI-based specifications are within the typical pay factor function trend as other DOT's specifications, and slightly stricter in the incentive and disincentive range.

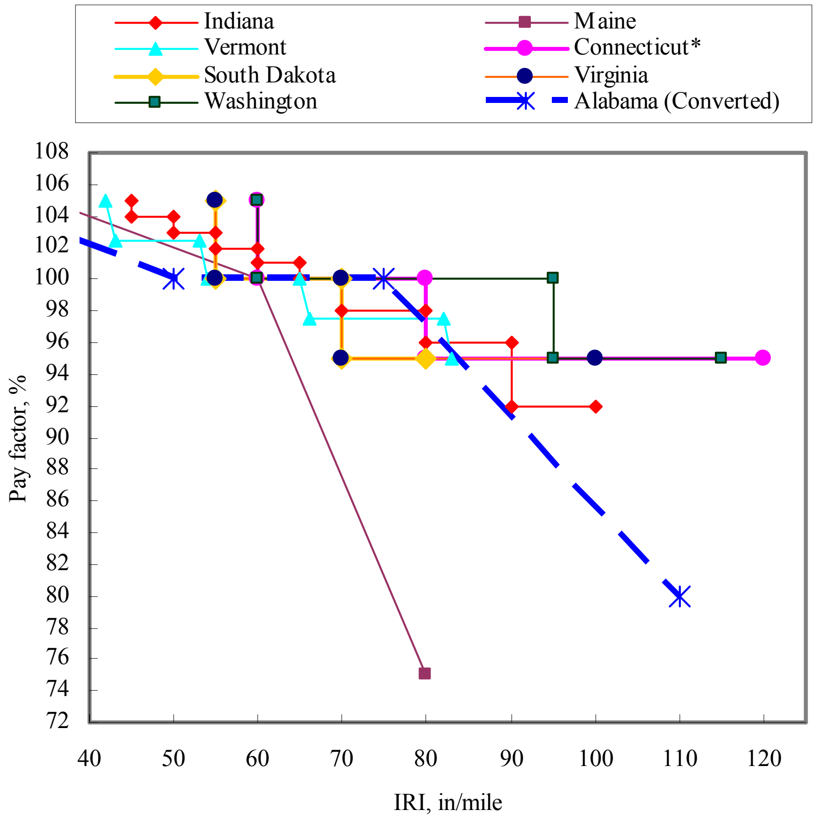


Figure 4. 21 Comparison of Transferred AL IRI specifications with Specifications from other DOT's.

CHAPTER FIVE

CONCLUSIONS

Over recent years, both the inertial profiler electronic technology and mathematical algorithms for evaluating the user's perception of ride quality have developed rapidly. Inertial profilers can record pavement profiles at highway speed, encouraging IRI to become widely used as both an initial smoothness acceptance assessment and an evaluation of ride quality changes with time and traffic. The IRI ride quality statistic accumulates the vertical movement response of a vehicle running over a pavement surface at a given speed. This method of profiling better highlights the wavelengths that reflect the riding comfort than other smoothness indices calculating the physical deviation of pavement surface beyond certain tolerance band, such as the PI obtained from the California-style profilograph. And also, IRI is an index estimating the pavement condition from immediately after construction up to rehabilitation needs, which makes pavement management more efficient and economical. For these reasons, the Alabama Department of Transportation is considering moving from a $PI_{0.0}$ based specification to an IRI based specification. The reasonable and practical relationships between PI and IRI needed for a specification conversion were developed in this study. The current specifications use 0.1 mile segment as the test interval, which averages the roughness and makes the localized irregularities unnoticed. To address this problem, the shorter interval, 0.01 mile was utilized to analyze the pavement roughness.

5.1 Conclusions

This study is based on 57 pavements longitudinal profiles from four Alabama asphalt pavement projects and four Quebec concrete pavement projects, measured with the ARAN inertial profiler. The raw profiles were analyzed using the ProVAL 2.5 software, which conducted the calculation of both the PI and IRI values for the each obtained raw profile.

The asphalt pavement database fully covered the typical smoothness value range of newly surfaced AC pavement, and was considered as the representative of Alabama (wet and no-freeze climate zone) asphalt pavement smoothness database. Since concrete pavements examined in this study were located in Quebec (wet and freeze climate zone), this database was just used to compare the blanking band effect on AC pavement and PCC pavement, and not used in Alabama specification development. Through the analysis of the database, the following conclusions can be drawn from this research:

- 0.2” blanking band hides the irregularities of pavements and has the limitation to evaluate pavement roughness, especially in short interval like 0.01 mile interval. It also shows that 0.2” blanking band has much more influence on evaluating PCC pavements than AC pavements in this study. According to the database herein, 0.2” blanking band covers much more amount of defects of rigid pavements than flexible pavements in this study, which allows rougher-driving concrete pavement to earn the same payment as asphalt pavement.
- Good linear regressions ($R^2 > 0.7$) between PI and IRI were developed. According to the correlation analysis, the current Alabama pavement smoothness specifications were moved to the single IRI-based smoothness specifications. The IRI based

specifications were decided at 0.1 mile interval, presented in table 5.1.

Table 5. 1 Transferred IRI based Smoothness Specifications for Asphalt and Concrete Pavement in Alabama.

Pavement Type	Equipment	Section Length	Price Adjustments	
			IRI, in/mile	Contract Price Adjustment of pavement unit bid price, %
AC and PCC Pavement	Inertial Profiler	0.1 mile	Under 55	112 -0.22*IRI
			55 to 70	100
			70 to 110	121-0.37*IRI
			Over 110	Unacceptable

- In addition, the statistical relationships between 0.1 mile and 0.01 mile smoothness indices were established in Table 4.4 for possible use of the smaller interval for investigating localized bumps in further studies.

5.2 Limitations

- Smoothness data from Alabama concrete pavement need to be verified if or when new concrete pavements are constructed in Alabama. Due to dataset of concrete pavement used herein collected from Montréal, Québec, even the database falling in the typical concrete pavement smoothness value range, the different climate zone still has an obvious impact on the relationship linking IRI and PI. In order to accurately move the PI based specifications to IRI based in Alabama, correlations developed by the smoothness data from Alabama are required.
- Since there is limited database in this study, in order to examine the effect of blanking band on evaluating all type of pavement smoothness, the database qualified for representing all new pavements need to be collected.
- Transferring the specifications from 0.1 mile interval to 0.01 mile interval is based on distribution method, which ensures contractor earning the same pay level for

either index limits. The primary reason for introducing 0.01 mile interval is to motivate fewer localized bumps in an otherwise good quality pavement. Specific recommendations for use of this approach for minimizing localized bumps are beyond the scope of this current research project.

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APPENDICES

Appendix A

Regression Relationship between IRI and PIx at 0.1 and 0.01 Mile Interval

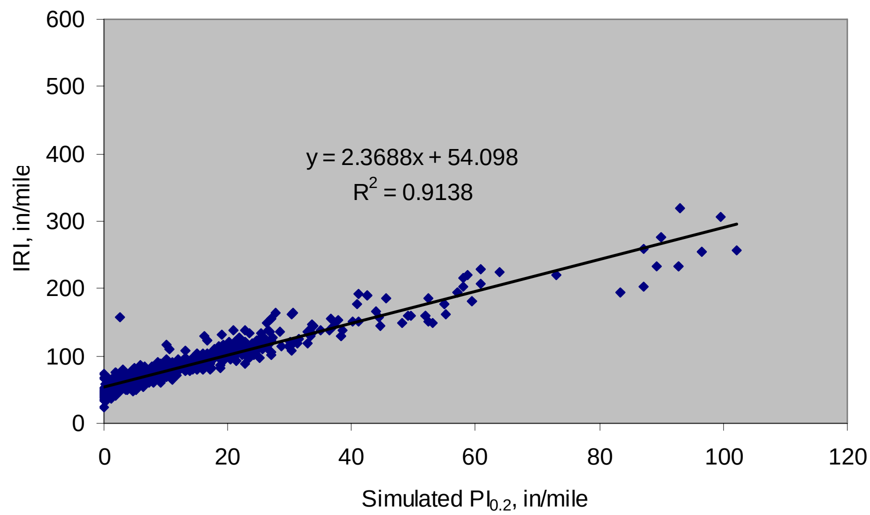

Figure A. 1 $PI_{0.2}$ vs. IRI for AC at 0.1 mile interval.

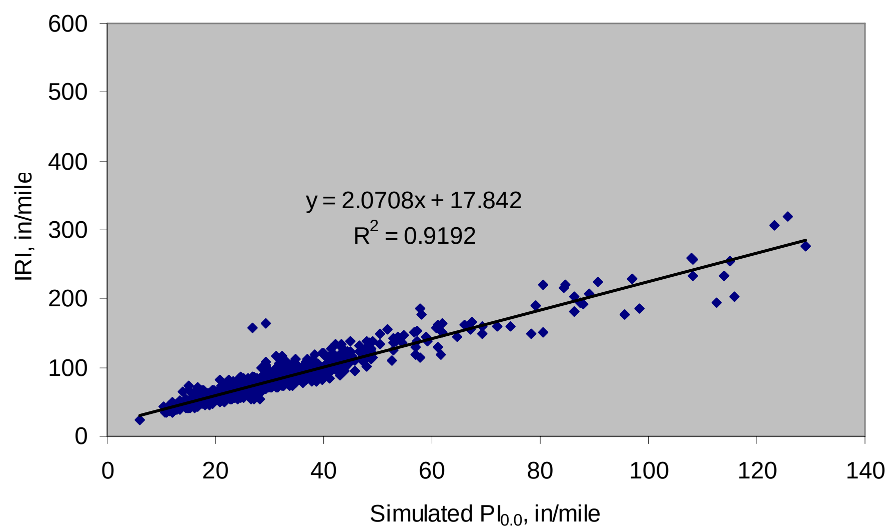

Figure A. 2 $PI_{0.0}$ vs. IRI for AC at 0.1 mile interval.

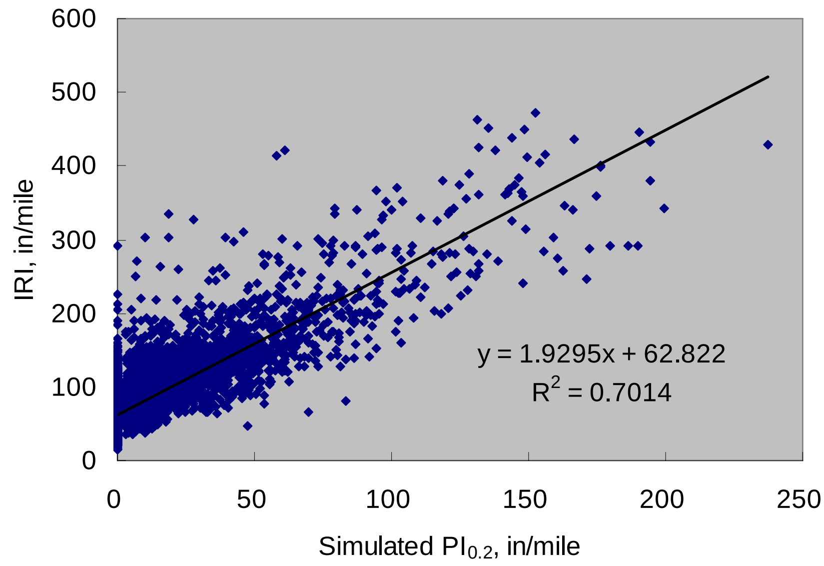

Figure A. 3 PI0.2 vs. IRI for AC at 0.01 mile interval.

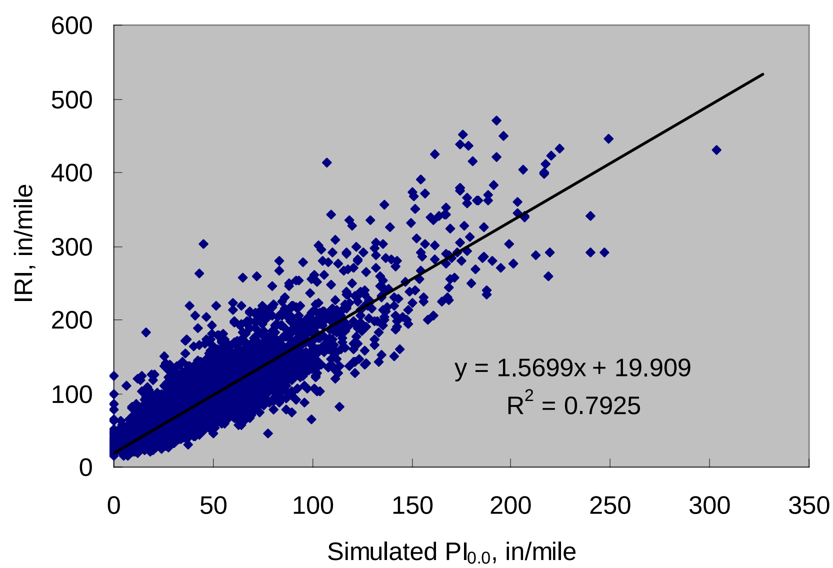

Figure A. 4 PI0.0 vs. IRI for AC at 0.01 mile interval.

Appendix B

Histogram Distribution of PI and IRI values of AC and PCC Pavement

Table B. 1 Histogram Distribution of $PI_{0.2}$ Value of AC Pavement at 0.1 mile interval.

Bin	Frequency	Cumulative	Bin	Frequency	Cumulative
0	20	2.30%	26	15	89.87%
2	140	18.41%	28	13	91.37%
4	110	31.07%	30	3	91.71%
6	120	44.88%	32	9	92.75%
8	89	55.12%	34	7	93.56%
10	72	63.41%	36	1	93.67%
12	39	67.89%	38	4	94.13%
14	40	72.50%	40	3	94.48%
16	41	77.22%	42	4	94.94%
18	31	80.78%	44	3	95.28%
20	23	83.43%	46	4	95.74%
22	27	86.54%	48	0	95.74%
24	14	88.15%	And more	37	100.00%

Table B. 2 Histogram Distribution of $PI_{0.0}$ Value of AC Pavement at 0.1 mile interval.

Bin	Frequency	Cumulative	Bin	Frequency	Cumulative
6	1	0.11%	69	7	95.88%
10	0	0.11%	73	2	96.11%
14	23	2.75%	77	1	96.22%
18	92	13.27%	81	5	96.80%
22	126	27.69%	85	2	97.03%
26	152	45.08%	89	7	97.83%
30	158	63.16%	93	1	97.94%
34	92	73.68%	97	4	98.40%
38	49	79.29%	101	1	98.51%
42	48	84.78%	105	0	98.51%
46	37	89.02%	109	3	98.86%
50	23	91.65%	113	1	98.97%
54	8	92.56%	117	5	99.54%
58	8	93.48%	121	0	99.54%
62	10	94.62%	125	1	99.66%
65	4	95.08%	And more	3	100.00%

Table B. 3 Histogram Distribution of IRI Value of AC Pavement at 0.1 mile interval.

Bin	Frequency	Cumulative	Bin	Frequency	Cumulative
25	1	0.11%	177	1	96.22%
35	2	0.34%	187	7	97.02%
45	32	4.01%	197	6	97.71%
55	111	16.74%	207	2	97.94%
65	163	35.44%	217	2	98.17%
75	187	56.88%	227	3	98.51%
85	109	69.38%	238	5	99.08%

96	70	77.41%	248	0	99.08%
106	58	84.06%	258	3	99.43%
116	32	87.73%	268	1	99.54%
126	27	90.83%	278	2	99.77%
136	14	92.43%	288	0	99.77%
146	10	93.58%	299	0	99.77%
156	11	94.84%	309	1	99.89%
167	11	96.10%	And more	1	100.00%

Table B. 4 Histogram Distribution of $PI_{0.2}$ Value of PCC Pavement at 0.1 mile interval.

Bin	Frequency	Cumulative	Bin	Frequency	Cumulative
0	3	1.66%	26	5	93.37%
2	7	5.52%	28	2	94.48%
4	4	7.73%	30	0	94.48%
6	25	21.55%	32	1	95.03%
8	35	40.88%	34	1	95.58%
10	28	56.35%	36	2	96.69%
12	9	61.33%	38	2	97.79%
14	13	68.51%	40	2	98.90%
16	14	76.24%	42	0	98.90%
18	8	80.66%	44	0	98.90%
20	10	86.19%	46	1	99.45%
22	5	88.95%	48	1	100.00%
24	3	90.61%	And more	0	100.00%

Table B. 5 Histogram Distribution of $PI_{0.0}$ Value of PCC Pavement at 0.1 mile interval.

Bin	Frequency	Cumulative	Bin	Frequency	Cumulative
20	0	0.00%	72	2	97.80%
22	1	0.55%	74	0	97.80%
24	2	1.65%	76	1	98.35%
26	3	3.30%	78	0	98.35%
28	5	6.04%	80	1	98.90%
30	3	7.69%	82	1	99.45%
32	9	12.64%	84	0	99.45%
34	17	21.98%	86	0	99.45%
36	16	30.77%	88	0	99.45%
38	20	41.76%	90	0	99.45%
40	11	47.80%	92	0	99.45%
42	24	60.99%	94	0	99.45%
44	13	68.13%	96	0	99.45%
46	10	73.63%	98	0	99.45%
48	12	80.22%	100	0	99.45%
50	4	82.42%	102	0	99.45%
52	6	85.71%	104	0	99.45%
54	6	89.01%	106	0	99.45%

56	2	90.11%	108	0	99.45%
58	5	92.86%	110	0	99.45%
60	0	92.86%	112	0	99.45%
62	4	95.05%	114	0	99.45%
64	2	96.15%	116	0	99.45%
66	0	96.15%	118	0	99.45%
68	1	96.70%	120	0	99.45%
70	0	96.70%	And more	1	100.00%

Table B. 6 Histogram Distribution of IRI Value of PCC Pavement at 0.1 mile interval.

Bin	Frequency	Cumulative
25	0	0.00%
35	0	0.00%
45	0	0.00%
55	0	0.00%
65	8	4.40%
75	14	12.09%
85	45	36.81%
96	39	58.24%
106	29	74.18%
116	19	84.62%
126	16	93.41%
136	4	95.60%
146	2	96.70%
156	3	98.35%
167	1	98.90%
177	2	100.00%
And more	0	100.00%

Table B. 7 Histogram Distribution of $PI_{0.2}$Value of AC Pavement at 0.01 mile interval.

Bin	Frequency	Cumulative	Bin	Frequency	Cumulative
0	5300	63.61%	51	15	95.31%
1	0	63.61%	52	16	95.50%
2	0	63.61%	53	12	95.64%
3	0	63.61%	54	14	95.81%
4	278	66.95%	55	9	95.92%
5	201	69.36%	56	8	96.02%
6	124	70.85%	57	14	96.18%
7	154	72.70%	58	6	96.26%
8	102	73.92%	59	9	96.36%
9	126	75.43%	60	8	96.46%
10	93	76.55%	61	12	96.60%
11	92	77.65%	62	10	96.72%
12	77	78.58%	63	10	96.84%

13	89	79.64%	64	9	96.95%
14	71	80.50%	65	8	97.05%
15	80	81.46%	66	5	97.11%
16	63	82.21%	67	9	97.22%
17	53	82.85%	68	4	97.26%
18	61	83.58%	69	7	97.35%
19	62	84.33%	70	10	97.47%
20	54	84.97%	71	3	97.50%
21	51	85.59%	72	6	97.58%
22	48	86.16%	73	6	97.65%
23	50	86.76%	74	5	97.71%
24	44	87.29%	75	3	97.74%
25	40	87.77%	76	4	97.79%
26	43	88.29%	77	3	97.83%
27	30	88.65%	78	5	97.89%
28	40	89.13%	79	6	97.96%
29	38	89.58%	80	4	98.01%
30	31	89.95%	81	7	98.09%
31	33	90.35%	82	4	98.14%
32	25	90.65%	83	5	98.20%
33	27	90.97%	84	3	98.24%
34	26	91.29%	85	3	98.27%
35	33	91.68%	86	2	98.30%
36	29	92.03%	87	9	98.40%
37	21	92.28%	88	3	98.44%
38	28	92.62%	89	2	98.46%
39	18	92.83%	90	3	98.50%
40	16	93.03%	91	3	98.54%
41	18	93.24%	92	4	98.58%
42	17	93.45%	93	3	98.62%
43	19	93.67%	94	1	98.63%
44	12	93.82%	95	7	98.72%
45	16	94.01%	96	6	98.79%
46	24	94.30%	97	4	98.84%
47	22	94.56%	98	0	98.84%
48	19	94.79%	99	1	98.85%
49	13	94.95%	100	1	98.86%
50	15	95.13%	And more	95	100.00%

Table B. 8 Histogram Distribution of $PI_{0.0}$ Value of AC Pavement at 0.01 mile interval.

Bin	Frequency	Cumulative	Bin	Frequency	Cumulative
0	48	0.58%	130	15	98.28%
4	18	0.79%	133	11	98.42%
7	63	1.55%	137	14	98.58%
11	201	3.96%	140	6	98.66%

14	316	7.75%	144	11	98.79%
18	521	14.01%	147	2	98.81%
21	650	21.81%	151	8	98.91%
25	820	31.65%	154	4	98.96%
28	808	41.35%	158	8	99.05%
32	705	49.81%	161	5	99.11%
35	665	57.79%	165	3	99.15%
39	538	64.25%	169	9	99.26%
42	484	70.06%	172	5	99.32%
46	377	74.58%	176	7	99.40%
49	293	78.10%	179	6	99.47%
53	251	81.11%	183	5	99.53%
56	192	83.41%	186	4	99.58%
60	175	85.51%	190	5	99.64%
63	155	87.37%	193	4	99.69%
67	135	88.99%	197	2	99.71%
70	110	90.31%	200	1	99.72%
74	95	91.45%	204	3	99.76%
77	81	92.43%	207	3	99.80%
81	51	93.04%	211	0	99.80%
84	67	93.84%	214	1	99.81%
88	56	94.52%	218	3	99.84%
91	50	95.12%	221	3	99.88%
95	47	95.68%	225	1	99.89%
98	30	96.04%	228	0	99.89%
102	29	96.39%	232	0	99.89%
105	33	96.78%	235	0	99.89%
109	17	96.99%	239	0	99.89%
112	29	97.34%	242	2	99.92%
116	17	97.54%	246	0	99.92%
119	18	97.76%	249	2	99.94%
123	16	97.95%	253	1	99.95%
126	13	98.10%	And more	4	100.00%

Table B. 9 Histogram Distribution of IRI Value of AC Pavement at 0.01 mile interval.

Bin	Frequency	Cumulative	Bin	Frequency	Cumulative
25	38	0.46%	175	58	95.40%
35	325	4.36%	185	48	95.98%
45	944	15.68%	195	47	96.54%
55	1290	31.17%	205	50	97.14%
65	1271	46.42%	215	38	97.60%
75	1074	59.31%	225	34	98.01%
85	827	69.23%	235	18	98.22%
95	585	76.25%	245	13	98.38%
105	393	80.97%	255	13	98.54%
115	281	84.34%	265	10	98.66%

125	271	87.59%	275	9	98.76%
135	199	89.98%	285	18	98.98%
145	196	92.33%	295	16	99.17%
155	118	93.75%	305	11	99.30%
165	80	94.71%	And more	58	100.00%

Table B. 10 Histogram Distribution of $PI_{0.2}$ Value of PCC Pavement at 0.01 mile interval.

Bin	Frequency	Cumulative	Bin	Frequency	Cumulative
0	842	47.84%	26	7	87.27%
1	0	47.84%	27	8	87.73%
2	6	48.18%	28	2	87.84%
3	9	48.69%	29	6	88.18%
4	63	52.27%	30	14	88.98%
5	69	56.19%	31	10	89.55%
6	56	59.38%	32	5	89.83%
7	40	61.65%	33	16	90.74%
8	46	64.26%	34	11	91.36%
9	42	66.65%	35	8	91.82%
10	25	68.07%	36	9	92.33%
11	32	69.89%	37	9	92.84%
12	37	71.99%	38	7	93.24%
13	27	73.52%	39	4	93.47%
14	23	74.83%	40	10	94.03%
15	24	76.19%	41	5	94.32%
16	29	77.84%	42	7	94.72%
17	19	78.92%	43	4	94.94%
18	24	80.28%	44	2	95.06%
19	13	81.02%	45	6	95.40%
20	19	82.10%	46	2	95.51%
21	22	83.35%	47	3	95.68%
22	15	84.20%	48	3	95.85%
23	16	85.11%	49	3	96.02%
24	16	86.02%	50	1	96.08%
25	15	86.88%	And more	69	100.00%

Table B. 11 Histogram Distribution of $PI_{0.0}$ Value of PCC Pavement at 0.01 mile interval.

Bin	Frequency	Cumulative	Bin	Frequency	Cumulative
10	2	0.11%	58	49	76.14%
12	2	0.23%	60	39	78.35%
14	2	0.34%	62	41	80.68%
16	11	0.97%	64	40	82.95%

18	15	1.82%	66	29	84.60%
20	31	3.58%	68	30	86.31%
22	24	4.94%	70	25	87.73%
24	45	7.50%	72	22	88.98%
26	48	10.23%	74	28	90.57%
28	62	13.75%	76	17	91.53%
30	68	17.61%	78	18	92.56%
32	74	21.82%	80	15	93.41%
34	74	26.02%	82	12	94.09%
36	76	30.34%	84	15	94.94%
38	90	35.45%	86	11	95.57%
40	95	40.85%	88	6	95.91%
42	65	44.55%	90	6	96.25%
44	84	49.32%	92	4	96.48%
46	99	54.94%	94	4	96.70%
48	77	59.32%	96	5	96.99%
50	66	63.07%	98	4	97.22%
52	72	67.16%	100	2	97.33%
54	56	70.34%	And more	47	100.00%
56	53	73.35%			

Table B. 12 Histogram Distribution of IRI Value of PCC Pavement at 0.01 mile interval.

Bin	Frequency	Cumulative	Bin	Frequency	Cumulative
34	1	0.06%	137	78	89.77%
45	24	1.42%	149	63	93.35%
57	134	9.03%	160	27	94.89%
68	224	21.76%	172	18	95.91%
80	298	38.69%	183	20	97.05%
91	298	55.63%	195	13	97.78%
103	244	69.49%	207	13	98.52%
114	161	78.64%	And more	26	100.00%
126	118	85.34%			