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FINAL REPORT ~ FHWA-OK-20-02

CONTINUOUS FRICTION MEASUREMENT EQUIPMENT (CFME) FOR HIGHWAY SAFETY MANAGEMENT IN OKLAHOMA

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CONTINUOUS FRICTION MEASUREMENT EQUIPMENT (CFME) FOR HIGHWAY SAFETY MANAGEMENT IN OKLAHOMA

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An important part of the pavement friction management (PFM) process is the selection of the most appropriate friction measuring equipment. In this project, the capabilities of the Grip Tester, a type of continuous friction measurement (CFME) device, and its ability to provide information to support PFM programs were evaluated based on comprehensive field data collection. Various statistical and comparisons analyses were performed, suggesting that CFME can acquire repeatable and reproducible friction profiles. The friction measurements from the Grip Tester and the locked-wheel skid trailer were tested to be statistically correlated. In addition, several potential implementations of CFME data were discussed for PFM and highway safety applications.

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SI* (MODERN METRIC) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

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

APPROXIMATE CONVERSIONS FROM SI UNITS

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

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

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CHAPTER 1 INTRODUCTION

1.1 Background

Pavement skid resistance plays a significant role in road safety as the friction between tire and pavement surface is a critical contributing factor in reducing potential crashes and improve roadway safety. Therefore, it is important that Departments of Transportation (DOTs) monitor the friction performance of their pavement networks in a systematic manner and establish Pavement Friction Management (PFM) programs. The Federal Highway Administration (FHWA) Technical Advisory T 5040.38, 2010 recommends that “to provide roadway data that will establish the relative severity of locations identified for highway safety improvement projects, a state highway agency should implement a program to manage pavement friction on its public roads.” The aim of this program is to minimize friction-related vehicle crashes by ensuring that pavements provide adequate friction properties throughout their lives. A proactive friction management program can help identify areas that have elevated friction-related crash rates, investigate road segments with friction deficiencies, and prioritize use of resources to reduce friction-related vehicle crashes in a cost-effective manner (AASHTO 2008, Flintsch 2009).

An important part of the friction management process is the selection of the most appropriate friction measuring equipment. There are many devices used around the world to measure friction, which can be classified into four groups:

locked-wheel, fixed slip, side force and variable slip. Locked-wheel (LW) skid trailers simulate the typical vehicular response to sudden braking before the widespread implementation of anti-lock braking systems (ABSs). The side force friction measurement devices simulate the ability of a vehicle to maintain control during vehicular maneuver. Fixed slip and variable slip devices are designed to measure the friction around the critical slip of an ABS. The locked-wheel skid tester (AASHTO T 242) is the predominant high-speed device used on U.S. roads. This device requires a tow vehicle and a locked wheel skid trailer, equipped with either standard smooth tires (ASTM E-524) and ribbed tires (ASTM E-521). Friction measurements are obtained by locking the testing tire on a wetted pavement surface while traveling at a specified speed (40mph in standard). The smooth tire is more sensitive to pavement macro-texture, while the ribbed tire is more sensitive to micro-texture change in the pavement.

Although this technology has been useful for several decades, the locked-wheel device based on the 1950s technology have major limitations: (1) it completely locks the wheel during testing, which does not accurately represent the braking conditions experienced in modern vehicles equipped with ABS; (2) it only provides a single average reading of friction over long distances (typically 1056 ft), which could result in inaccurate or loss of testing at intersections, ramps, sharp curves etc. where the friction demands are high; (3) it consumes approximately 2 gallons of water per mile and requires expensive testing tires, which hinders the wide application of friction measurements at the network level. As a result, agencies around the world have started using Continuous Friction Measurement Equipment

(CFME). CFMEs have the advantage of operating under conditions similar to modern vehicles equipped with ABS. These devices can measure friction continuously around the critical slip ratio of a vehicle braking with ABS at highway speed across the entire stretch of a road. CFMEs also use less water, possibly providing a more practical alternative for both project and network level friction management.

The Grip Tester (GT), one type of CFME, has been used extensively in the UK and Germany. It continuously measures the longitudinal friction along the wheel path and operates at a fixed slip ratio at highway speeds using users defined water film thickness. The friction measurements are recorded at an interval of 3-ft (0.9 m) by default or another value set by the users (Findlay Irvine Ltd 2017). Because CFME devices are developed based on the modern ABS technology, the friction measurements from CFME may not correspond precisely with historical data collected with locked-wheel trailers. In addition, although CFME provides information with greater details about spatial variability of pavement friction, processing large amount of data may be cumbersome and time consuming at this time due to the lack of available practical software for roadway applications. Therefore, research is needed to understand the various factors that affect the CFME measurements, to evaluate the capabilities of CFME devices and its ability to support PFM programs in Oklahoma.

1.2 Project Objective

The main objective of this study is to evaluate the capabilities of the Grip Tester, one type of CFME device, and its ability to provide information to support PFM programs. Specifically, the research aims to address the following sub-objectives:

- Design an experiment using various types of pavement surfaces in Oklahoma to investigate the effect of various operational factors on CFME friction measurements,
- Compare CFME measurements from the Grip Tester with data from the ODOT locked-wheel trailer,
- Investigate the repeatability of CFME measurements,
- Investigate the potential implementation of CFME Grip Tester data sets for pavement friction management and highway safety applications in Oklahoma.

1.3 Project Tasks

The aforementioned project objectives were achieved through the following project tasks, each outlined in one chapter in this report:

- Literature Review. This task included a comprehensive literature review in order to develop an in-depth understanding of various aspects related to this project.

- Experimental Design. Working closely with ODOT, asphalt and concrete testing sections with various surface characteristics (texture and materials) were selected in Oklahoma as the testing bed for the implementation of Grip Tester. Various operation characteristics of a Grip Tester were integrated into the field data collection to evaluate the CFME friction measurements.
- Field Data Collection. Several state-of-the-art instruments were used (1) to collect pavement surface and macro-texture data at highway speed, and (2) to measure pavement surface skid resistance using the Grip Tester under various operational conditions.
- Evaluation of Grip Tester. Statistical testing was conducted to determine whether the Grip Tester and locked wheel friction measurements were significantly different. The repeatability of multiple friction measurements from the Grip Tester was evaluated. Also the effect of various operational factors on CFME friction measurements was investigated.
- Implementation of CFME Data. There were several potential implementations of CFME data sets for pavement friction management and highway safety applications. Combining the CFME and pavement condition data sets collected from this project with the Oklahoma safety database, updated Safety Performance Function (SPF) was developed to improve the prediction accuracy of expected average crash frequency. With detailed CFME data, pavement network could be better segmented into homogenous sections for road maintenance scheduling and

management. Several algorithms were tested for this purpose. In addition, in order to ease the use of CFME data, a CFME data analysis software interface was developed, which is able to upload and visualize CFME measurements, and perform statistical and comparison analyses for data reporting. Finally, how CFME data and the findings from this study can be used to assist the pavement friction management was discussed.

- Final Reporting. This task was the submission of the final report documenting each of the tasks conducted as part of this study and the lessons learned.

1.4 Report Outline

Accordingly, the annual report is organized as below:

- Chapter 1 provides the background and objectives of the project.
- Chapter 2 documents the comprehensive literature review of related aspects for the project.
- Chapter 3 introduces the field experimental design, the list of field testing sites, field data collection devices and their operation conditions.
- Chapter 4 provides the data analysis for the evaluation of Grip Tester, and the influences of operation testing factors on friction CFME measurements.
- Chapter 5 and Chapter 6 presents the potential implementation of CFME for pavement friction management and highway safety analysis. The

features and functions of the developed data analysis software interface are also introduced.

- Chapter 7 summarizes the key findings and future work of this project.

CHAPTER 2 LITERATURE REVIEW

An extensive literature review was completed to develop an in-depth understanding of various aspects related to this project. Several important related topics were summarized in this Chapter, including the pavement friction measurement and surface characteristics, testing equipment, existing safety prediction models and the relationships between friction and highway safety, and pavement friction management.

2.1 Pavement Friction Characteristics

Pavement friction is the force resisting the relative motion between the vehicle tire and pavement surface, and it is a critical factor influencing the crash ratios on both wet and dry conditions for roads (Hall et al., 2009; Najafi et al., 2015). The resistive force, illustrated in Figure 2-1, is the result of the interaction between the tire and the pavement (Flintsch et. al, 2012). It is dominated by the texture of the pavement surface, with different texture components making different contributions.

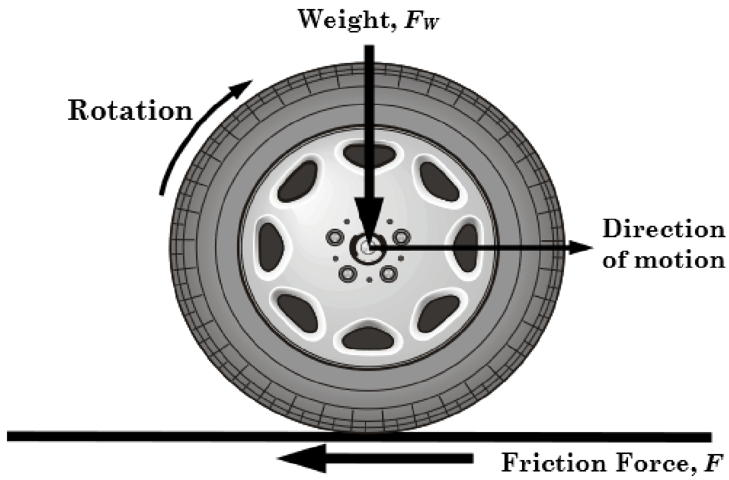


Figure 2-1 Forces on a Rotating Wheel (Hall et al., 2009)

Pavement micro- and macro-texture are the main pavement surface characteristics that affect tire-pavement friction. Micro-texture is generally provided by the relative roughness of the aggregate particles in asphalt pavements and by the fine aggregate in concrete surfaces. Macro-texture is generally provided by proper aggregate gradation in asphalt pavement and by a supplemental treatment such as tinning, broom, diamond grinding or grooving in concrete surfaces. The reduction in friction with increasing testing speed would be lower for the pavements with higher macro-texture since they have more channels for the water to escape while pressed between the tire and the pavement (Henry, 2000). Kanafi et al. (2015) monitored variations of pavement texture and observed that macro-texture decreased and micro-texture increased during summer time. Early rapid reduction followed by an increase and subsequent gradual decline of macro-texture change of asphalt concrete samples was observed in lab using close range photogrammetry (Millar et al. 2009). Wavelet analysis was applied to interpret macro-texture collected by Circular Texture Meter (CTM) to determine the wavelength ranges and energy content that affect the macro-texture properties of asphalt pavements (Zelelew et al. 2013; Zelelew et al. 2014).

To better visualize the role of texture with the contact region of a tire on a wet pavement, the Three Zone Concept, first suggested by Gough and later extended by Moore, is shown in Figure 2-2 (Moore, 1966). In zone 1, water is squeezed out by the macro-texture of the pavement surface, whereas in zone 2, the micro-texture dominates. In zone 3, the tire gains dry contact with the pavement's surface. It is in this last zone, that the forces of adhesion and hysteresis come into play. Adhesion and hysteresis are the two main components of tire pavement friction. Adhesion is due to the molecular bonding between the tire and the pavement surface while hysteresis is the result of energy loss due to tire deformation. Both hysteresis and adhesion are related to surface characteristics and tire properties (Hall et al. 2009).

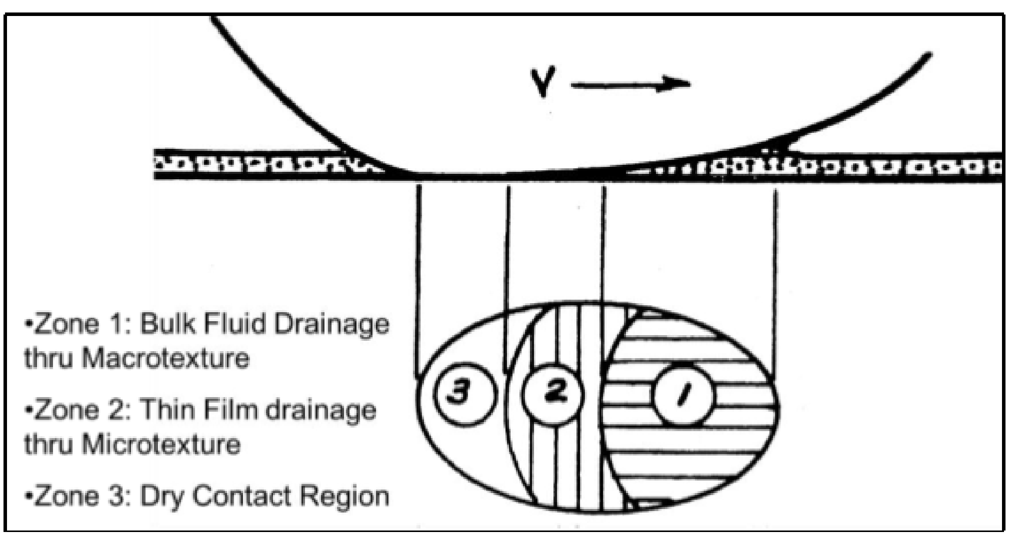


Figure 2-2 Texture Three Zone Concept of a Wet Surface (Moore, 1966)

As drivers maneuver their vehicles (i.e. braking, accelerating, or changing their vehicle's direction of travel), tire-pavement friction is produced at the tire-pavement contact patch. In the situation where a driver applies the brakes, the relative difference between the peripheral speed of the tire and the velocity of the vehicle result in tire slipping over the pavement surface. Literature commonly refers to this slippage as (longitudinal) slip speed, S, which is the relative difference between the directional velocity of a vehicle, V, and the average peripheral velocity of the tire, Vp, during constant braking or free rolling. Figure 2-3 illustrates the process of slip speed as a result of applied braking which is also expressed as the percentage of slip calculated by taking the ratio of S over V, multiplied by 100 (Hall, 2009).

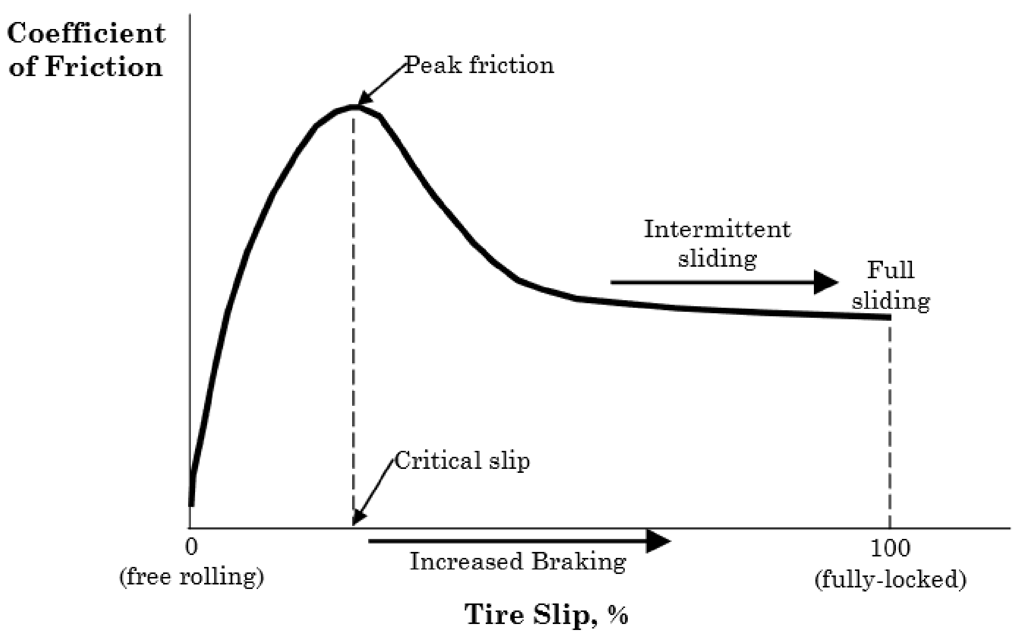


Figure 2-3 Friction vs. Tire Slip (Hall et al, 2009)

Flintsch et al. (2012) illustrated in “The Little Book of Tire Pavement Friction” that on a dry road surface, there is often little difference between peak and sliding friction and relatively little effect of speed. However, on a wet road, peak friction is often lower than that in dry conditions, the sliding friction is typically lower than peak friction, and both usually (but not always) decrease with increasing speed. The differences between wet and dry, peak and sliding friction, depend not only on vehicle speed and tire properties (including tread depth and pattern), but also to a large extent on the characteristics of the road surface, particularly its state of micro-texture, the form and magnitude of the macro-texture, and the amount of water and other contaminants on the pavement. It is important to point out that when friction measurements occur on the left side of the peak, these will be mostly influenced by the characteristics of the tire, whereas those measurements made on the right side of the peak, will be influenced by those properties of the surface (macro-texture).

2.2 Pavement Friction Measurement and Influencing Factors

This section discusses the four common pavement friction measurement devices and the influencing factors of pavement friction measurements.

2.2.1 Testing Equipment

In the context of roadway safety management, there are several methods for pavement friction measurements, the majority of which obtain measurements by moving a tire or slider over a wetted pavement surface (Do and Roe, 2008). The methods can be grouped into two categories: high-speed equipment, and low-speed or stationary equipment (Hall et al., 2009). The slow-moving and static test methods, also referred to as laboratory methods, can be used in the field or in a lab. Two devices that are typical for industrial and research use are the Dynamic Friction Tester (DFT) and the British Pendulum Tester (BPT). For network-level management, an optimal method for measuring skid resistance could be the use of high-speed equipment. The high-speed equipment is often subcategorized into four groups: locked-wheel (longitudinal friction force), sideway-force (sideway “lateral” friction factor), and variable-slip (Hall et al., 2009), fixed-slip (longitudinal friction force). Table 2-1 summaries the characteristics of these four groups of friction test equipment. The last three devices can be characterized as continuous friction measurement equipment (CFME) because they collect friction measurements continuously. More agencies around the world have started using CFME instruments for highway friction management. CFMEs have the advantage that they continuously measure the friction across the entire stretch of a road, providing greater detail about spatial variability of the tire pavement frictional properties (Flintsch et al. 2012), using either sideway-force or fixed-slip device.

Table 2-1 Pavement Friction Test Equipment (Henry, 2000)

Test Method	Associated Standard	Description	Equipment	Measurement Index	Application	Cost
Locked Wheel	ASTM E 274	This device is installed on a trailer which is towed behind the measuring vehicle at a speed of 40 mi/hr (64 km/hr). Water may be applied in front of the test tire, a braking system is forced to lock the tire, and the resistive drag force is measured and averaged for 1 sec after the test wheel is fully locked.	Measuring vehicle and locked-wheel skid trailer, equipped with either a ribbed tire (ASTM E 501) or a smooth tire (ASTM E 524). ASTM E 274 recommends the ribbed tire.	The measured resistive drag force and the wheel load applied to the road are used to compute the coefficient of friction, μ. Friction is reported as FN.	Field testing (straight segments) and curves up to a side acceleration of 0.3 Gs.	Equipment: \$100,000 to \$200,000 Test Rate: Highway speeds Other: Not continuous collection.
Side Force	ASTM E 670	Side-force friction measuring devices estimate the road surface friction at an angle to the direction of motion (usually perpendicular).	British Mu-Meter (measures the side force developed by two yawed wheels). British Sideway Force Coefficient Routine Investigation Machine (SCRIM) (has a wheel yaw angle of 20°).	The side force perpendicular to the plane of rotation is measured and used to compute the sideways force coefficient, SFC.	Field testing (straight and curved sections).	Equipment: \$50,000 and up Test Rate: Highway speeds.
Fixed Slip	Under ASTM ballot	Fixed-slip devices perform tests typically between 10 and 20 percent slip speed.	Roadway and runway friction testers (RFTs) Airport Surface friction Tester (ASFT) Saab friction Tester (SFT) Grip Tester.	The measured resistive drag force and the wheel load applied to the road are used to compute the coefficient of friction, μ. Friction is reported as FN.	Field testing (straight segments).	Equipment: \$35,000 to \$150,000 Test Rate: Highway speeds.

Test Method	Associated Standard	Description	Equipment	Measurement Index	Application	Cost
Variable Slip	ASTM E 1859	Variable-slip devices measure friction as a function of slip between the wheel and the highway surface. They provide information about the frictional characteristics of the tire and highway surfaces, such as the initial increasing portion of the friction slip curve is dependent upon the tire properties, whereas the portion after the peak is dependent upon the road surface characteristics.	French IMAG Norwegian Norsemeter RUNAR, ROAR, and SALTAR systems. ASTM E 1551 specifies the test tire suitable for use in variable-slip devices (ASTM 1998f).	The measured resistive drag force and the wheel load applied to the road are used to compute the coefficient of friction, µ. Friction is reported as FN.	Field testing (straight segments).	Equipment: $40,000 to $500,000 Test Rate: Highway speeds

2.2.2 Influencing Factors

According to NCHRP (2009), the factors that influence pavement friction forces can be grouped into four categories: pavement surface characteristics, vehicle operational parameters, tire properties, and environmental factors. Table 2-2 lists the various factors comprising each category, in which the more critical factors are shown in bold. Particularly, friction is affected primarily by micro-texture and macro-texture. At low speeds, micro-texture dominates the wet and dry friction level. At higher speeds, the presence of high macro-texture facilitates the drainage of water so that the adhesive component of friction afforded by micro-texture is re-established by being above the water. Pavement material properties influence both micro- and macro-texture, and also affect the long-term durability of texture under accumulated traffic and environmental loadings.

Table 2-2 Factors Affecting Available Pavement Friction (AASHTO 2008)

Pavement Surface Characteristics	Vehicle Operating Parameters	Tire Properties	Environment
Micro-texture **Macro-texture** Mega-texture **Material properties** Temperature	**Slip speed** • Vehicle speed • Braking action Driving maneuver • Turning • Overtaking	Foot Print **Tread design and condition** Rubber composition and hardness **Inflation pressure** Load Temperature	Climate • Wind • **Temperature** • **Water** • **Snow and ice** Contaminants • Anti-skid material • Dirt, mud, debris

Previous research has also indicated that pavement surfaces of a given type, vary tremendously in skid resistant properties. Several different surface mix types and finishing/texturing techniques are available to use in constructing new pavements and overlays, or for restoring friction on existing pavements, as shown in Table 2-3 with typical macro-texture levels.

Table 2-3 Common Mix Types and Texturing Techniques

Application	Mix / Texture Type	Typical Macro-Texture Depth
New AC or AC Overlay	Dense Fine-Graded HMA	0.015 to 0.025 in
New AC or AC Overlay	Dense Coarse-Graded HMA	0.025 to 0.05 in
New AC or AC Overlay	Gap-Graded HMA or Stone Matrix Asphalt	exceeds 0.04 in
New AC or AC Overlay	Open-Graded Friction Course (OGFC)	0.06 to 0.14 in
Friction Restoration(AC)	Chip Seal	exceeds 0.04 in
Friction Restoration(AC)	Slurry Seal	0.01 to 0.025 in
Friction Restoration(AC)	Micro-Surfacing	0.02 to 0.04 in
Friction Restoration(AC)	HMA Overlay	
Friction Restoration(AC)	Ultra-Thin Polymer-Modified Asphalt (e.g., NovaChip)	exceeds 0.04 in
Friction Restoration(AC)	Epoxied Synthetic Treatment (e.g., HFST)	exceeds 0.06 in
Retexturing (AC)	Micro-Milling	exceeds 0.04 in
New PCC Or Overlay	Broom Drag	0.008 to 0.016 in
New PCC Or Overlay	Artificial Turf Drag (longitudinal)	0.008 to 0.016 in
New PCC Or Overlay	Burlap Drag (longitudinal)	0.008 to 0.016 in
New PCC Or Overlay	Longitudinal Tine	0.015 to 0.04 in
New PCC Or Overlay	Transverse Tine	0.015 to 0.04 in
New PCC Or Overlay	Diamond Grinding (longitudinal)	0.03 to 0.05 in
New PCC Or Overlay	Porous PCC	exceeds 0.04 in
New PCC Or Overlay	Exposed Aggregate PCC	exceeds 0.035 in
Friction Restoration	HMA Overlay	
Retexturing (PCC)	Diamond Grinding (longitudinal)	0.03 to 0.05 in
Retexturing (PCC)	Longitudinal Diamond Grooving	0.035 to 0.055 in
Retexturing (PCC)	Transverse Diamond Grooving	0.035 to 0.055 in
Retexturing (PCC)	Shot Abrading	0.025 to 0.05 in

Specifically, several important factors that can influence the available pavement friction are discussed as below, including pavement texture, types of surfacing, water film thickness, slip speed, temperature, and road geometry (AASHTO, 2008 and Austroads, 2009).

Pavement Texture

Pavement micro- and macro-texture are the main pavement surface characteristics that affect tire pavement friction. Micro-texture (fine scale texture of less than 0.5 mm depth) is the dominant factor in determining wet skid resistance at low to moderate speeds. At high speeds, micro-texture is still important but the macro-texture (coarse texture in the range of 0.5 mm to 15.0 mm) becomes dominant, as it provides rapid drainage routes between the tire and road surface (Vicroads, 2015).

Najafi (2010) also pointed out that the reduction in friction with increasing testing speed would be lower for the pavements with higher macro-texture since they have more channels for the water to escape while pressed between the tire and the pavement. Macro-texture data can be used to predict the changes of friction with speed (Najafi et al., 2011).

Currently there is no widely accepted specification on pavement texture within the U.S., while some countries have such a requirement to maintain proper performance. UK required a mean texture depth (MTD) of 1.5 mm (0.06'') for new AC pavements (Henry, 2000), while a minimum 0.65 mm (0.026'') sand patch MTD or 1.0 mm (0.04'') laser-based MTD for transversely textured new PCC surfaces was required to meet the skid resistance requirement (Ahammed and Tighe, 2010). France recommended the desired glass beads MTD ⩾ 0.40 mm (0.016'') to ⩾ 0.70 mm (0.028'') for urban and suburban roads, ⩾ 0.60 mm (0.024'') to ⩾ 0.80 mm (0.031'') MTD for rural (interurban) roads, depending on speed, longitudinal slope, curve radius, and number of lanes per direction (Dupont and Bauduin, 2005). China specified texture depth to be greater than 0.55 mm (0.022'') for AC interstate pavements, and from 0.77 mm (0.030'') to 1.1 mm (0.043'') for interstate PCC pavements. Larson et al. (2004) recommended a minimum macrotexture for Ohio, whose thresholds are the same as the French specification for intervention at network level, but a 1.0 mm (0.04'') as an investigatory (desirable) value for network and project levels. Minnesota required an MTD greater than 0.8 mm (0.031'') on new PCC surfaces (Henry, 2000). Roe et al. (1991, and 1998) applied high-speed texture meter to assess texture depth and found that accident risk started to increase when texture depth was less than 0.7 mm (0.028'').

<u>Types of Surfacing</u>

Several studies have been conducted to investigate pavement friction with different types of pavement surfacing. Gardiner (2001) measured friction on sites with Superpave and Marshall Mix designs, and found that friction related more to the nominal maximum size of aggregate rather than mix design practices. Li et al. (2007) evaluated the influence of the aggregates characteristics on pavement friction performance considering different mixture designs, and concluded pavements with coarse aggregate generated more consistent friction performance than regular mixes. Asi (2007) tested friction for different pavement mixes and observed that harder aggregate induced higher friction value while vice versa for asphalt content. Kumar and Wilson (2010) demonstrated more than 24% improvement in skid resistance performance when Grade 6 was used compared to Grade 4 for two geologically similar sourced aggregate chips. Prapaitrakul et al. (2005) investigated the skid resistance effectiveness of fog seal, while Li et al. (2012) evaluated the long-term friction performance of chip seal, fog-chip, rejuvenating seal, micro-surfacing, ultrathin bonded wearing course (UBWC), and thin overlay. Wang (2013) applied boxplot and Fisher's Least Significance Difference test to rank the effectiveness of preservation treatments on friction. Li et al. (2016) studied the effectiveness of high friction surface treatment (HFST) in improving pavement friction.

Water Film Thickness

NCHRP (2009) points out that water, can act as a lubricant, significantly reducing the friction between tire and pavement. The effect of water film thickness on friction is minimal at low speeds (<20 mph or 32 km/h) and quite pronounced at higher speeds (>40mph or 64 km/h). In fact, a water film thickness of 0.002 inches reduces the tire pavement friction by 20 to 30 percent of the dry surface friction (Merritt et al., 2015).

Figure 2-4 presents an idealized representation of the loss of friction with time during a rain event from a dry surface to wet and then dry again. In dry conditions, clean surfaced roads have high friction because the vehicle tires can keep in good contact with the road surface. When a road surface transitions from dry to being slightly wet, there is a sharp reduction in the coefficient of friction due to the presence of the water film, which acts as a lubricant between the tire and road surface (Wilson, 2006). Harwood et al. (1987) reported that even 0.025 mm depth of water on the pavement can reduce the tire–pavement friction by as much as 75% on surfaces having poor skid resistance characteristics. Thin films of water have also been shown to be sufficient to produce hydroplaning. The micro drainage routes provided by the surface texture roughness (macro-texture) together with the tire tread help to eliminate the bulk of the water. However, the penetration of the remaining water film can only be achieved if there are sufficient fine-scale sharp edges (micro-texture) on which high pressures can build up as the tire passes. These high pressures are needed to break through the water film to establish dry contact between road and tire (Rogers & Gargett, 1991).

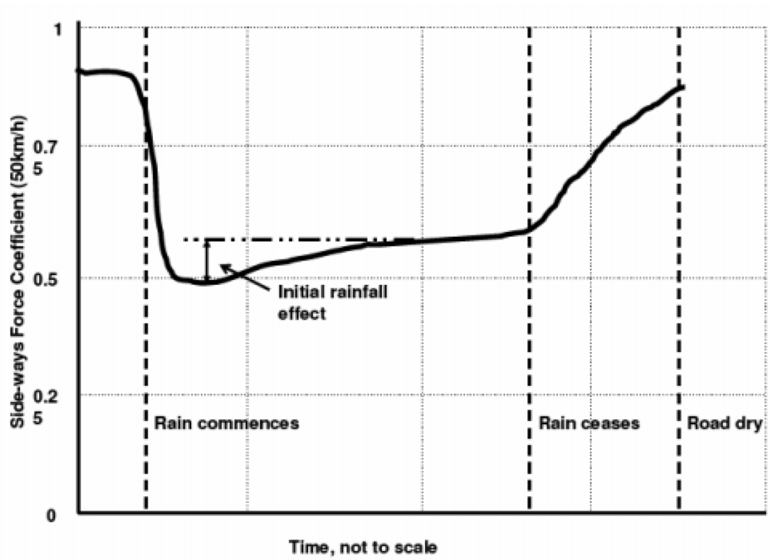


Figure 2-4 Friction Variation during a Rainfall Event (Wilson, 2006)

<u>Slip Speed</u>

Slip speed contains vehicle speed and braking action (NCHRP, 2009). In dry conditions the level of surface friction is considered to be constant with increasing vehicle speed. However, in wet conditions, the level of surface friction reduces rapidly with increasing vehicle speed (Vicroads, 2015). The coefficient of friction between a tire and the pavement is increasing rapidly to a peak value usually at 10 to 20 percent slip. The friction then decreases to a value known as the coefficient of sliding friction that occurs at full sliding.

Standards for locked-wheel friction measurements (SN, skid number) are set at 40 mph (Flintsch et al., 2012). For Grip Tester, a speed modification factor (0.007/ mph) found in a previous study was used to convert the results to the 40 mph standard (Flintsch et al., 2010). Because on open roadways, it is very difficult to maintain a constant speed as required by friction specifications, normally 40 mph.

Temperature

As both tire rubber and bituminous materials are viscoelastic materials, these materials are sensitive to change in temperature and will subsequently affect pavement friction. Many studies have been performed in the past several decades with key findings summarized below (Jayawickrama 1981, Oliver 1989, Henry 2000, Grosch, 2005):

- pavement surface temperature plays a significant role in influencing the tire temperature and hence the skid resistance,
- temperature change has more effect on the frictional properties of the tire, leading to an indirect effect on friction as measured by testers,
- measured coefficient of friction tends to decrease with increasing air temperature,

- water temperature has negligible effect on measured coefficients of friction.
- tire slip condition has a major influence on temperature development in a tire and a resulting effect on friction.

Road Geometry

VicRoads (2015) pointed out that the highest rates of loss to surface friction are found at sites where the highest vehicle stresses are imparted onto the surface aggregates, such as at tight curves and the approaches to intersections. At these sites, polishing of the surface aggregate occurs. It is also recognized that crossfall and superelevation have an effect on the propensity of water ponding on a road surface.

2.3 Pavement Friction and Highway Safety

Highway safety is a critical transportation issue in the United States. The DOT has had a long-standing goal of reducing the fatality rate by a certain amount over a certain time period; the most recent goal being a reduction from 1.5 fatalities per 100 million vehicle miles travelled (VMT) in 2003 to 1.0 fatality per 100 million VMT in 2008 (Ostensen 2005). There are a number of factors that contribute to the high number of traffic crashes and the resulting fatalities and injuries. The factors fall under three broad categories: Human (driver and/or passenger behavior); Vehicle (design and condition); Roadway environment (design and condition). Merritt et al. (2015) pointed out that one factor that is fairly well understood is the link between pavement friction and safety, or more specifically, the probability of wet-weather skidding crashes. The probability of wet-skidding crashes is reduced when friction between a vehicle tire and pavement is high. The FHWA and National Transportation Safety Board (NTSB) (2016) indicate that up to 70 percent of wet-pavement crashes can be prevented or minimized (in terms of damage) by improved pavement friction (FHWA, 2016).

As documented in the AASHTO *Guide for Pavement Friction* (2008), although a basic relationship exists between pavement friction and wet-crash rates, no specific threshold values have been established for pavement friction that make a pavement more or less safe. Pavement friction demand, which is specific to the characteristics of a particular roadway, must be considered when establishing any sort of threshold. Pavement friction demand is dictated by site conditions (such as longitudinal grade, superelevation, radius of curvature, terrain, climatic conditions), traffic characteristics (volume and mix of vehicle types), and driver behavior (prevailing speed, response to conditions, etc.). These conditions are continually changing over time and are different for every roadway, making it difficult to establish a “one size fits all” friction threshold (Merritt et al., 2015).

2.3.1 Safety Performance Function

Safety performance functions (SPFs) are essentially mathematical equations used to predict the average number of crashes per year at a location as a function of traffic volume (AADT), and in some cases site characteristics, such as lane width and shoulder width (Srinivasan, Carter, and Karin Bauer, 2013). The HSM identifies three types of SPF applications:

- Network screening to identify locations with promise, which are locations that may benefit the most from a safety treatment.
- Determination of safety impacts of design changes: When SPFs are used in project-level decision making, they are used for estimating the average expected crash frequency for existing conditions, alternatives to existing conditions, or proposed new roadways.
- Determination of safety effects of engineering treatments (before/after). These are usually implemented in combination with Empirical Bayes (EB) methods to address potential bias due to regression to the mean.

In mathematical form, an SPF can be represented in the following manner:

$$Np = f\,(AADT, X1, X2, X3, X4, \ldots) \qquad \text{(Eq. 2.1)}$$

Where Np is the predicted number of crashes during a particular time as a function of traffic volume (AADT), and other site characteristics, X1, X2, X3, X4, and so on (f is a mathematical function that relates the predicted number of crashes with AADT and other site characteristics). The relevant site characteristics may also be different depending on the type of road and whether it is a roadway segment, intersection, or ramp (Srinivasan, Carter, and Karin Bauer, 2013).

CHAPTER 3 FIELD TESTING SITES AND DATA COLLECTION

This Chapter discusses the experimental design of field testing sites and the corresponding data collection devices for this project. Specifically, built on the extensive literature review and technical guidance from ODOT, the field testing sites was finalized as the test beds to evaluate the capabilities of the Grip Tester. Subsequently, pavement surface characteristics, including surface friction and texture properties were acquired form the filed sites using several state-of-the-art instruments.

3.1 Field Testing Sites

Based on the comprehensive literature review presented in Chapter 2, the most influential factors to pavement friction were considered in the experimental design of the field testing sites. In particular, several influencing factors, including pavement surface conditions, friction testing speeds, roadway geometry, traffic volume, preventive treatment type, and treatment age, were included in the selection of the testing sites. In addition, the research team worked closely with ODOT engineers to finalize these sites.

Currently, various pavement preventive treatments have been applied in Oklahoma to retrieve surface characteristics including pavement skid resistance. Pavement micro- and macro-texture are the main pavement surface characteristics that affect tire pavement friction. Therefore, different pavement preventive treatments with various texture properties should be considered in the experimental design. Nine commonly used treatment types in Oklahoma were investigated in this project, including chip seal, ultra-thin bonded wearing course (UTBWC), resurfacing (asphalt), micro-surfacing, warm mix asphalt (WMA) thin overlay, resurfacing (concrete), next generation concrete surface (NGCS), longitudinal grooving, and high friction surfacing treatment (HFST). In addition, for the ODOT SP&R 2275 project, forty-five field sites with various surface treatments and aggregate sources were monitored, and the data and findings from this project were utilized in this study to leverage existing/current resources and data sets.

Besides, ODOT maintains an annual skid resistance testing program to gather locked-wheel friction data on Oklahoma interstates and US-69. Since one of the goals of this project is to explore the relationship between Grip Tester and locked-wheel tester measurements. It is important to work closely with ODOT to include the annual testing of those highways and take full advantage of the abundant data resources from the statewide data collected program.

In addition, pavement friction is a major factor for highway safety. Sites with high crash rates should maintain high skid demand. In order to manage and improve safety performance of the pavement network in Oklahoma, it is necessary to understand how various testing factors could impact pavement surface friction on those high friction demand sites. Crash data from the past ten years was obtained from the ODOT Safe-T database, and subsequently the roadway segments with high observed crashes, particular those within the ODOT skid testing program (interstates and US-69), were identified as the potential field testing locations for this project.

For practical purposes, the accessibility of the testing sites should also be considered. For each site, multiple friction measurements were acquired at various testing conditions such as different water film thicknesses and testing speeds. Whether the testing vehicle can be easily turned around is important for the need of multiple data collection.

Considering the aforementioned factors, the field testing sites was selected as shown in Table 3-1, which had been approved by ODOT and were severed as the testing bed for this project. It should be noted that the LTPP SPS-10 WMA site on State Highway 66 selected in this project contains six sections with different WMA mixtures.

In addition to the sites listed in Table 3-1, ODOT Bridge Division showed interests in the skid resistance performance of bridge deck surfaces. ODOT Bridge engineer, Mr. Peters Walt suggested including bridge decks on north bound of Interstate 35 into the monitoring process. After exploring the 2017 Oklahoma National Bridge Inventory (NBI) data, ten bridge decks with six different surface treatment types were identified for testing as shown in Table 3-2. The selected pavement and bridge testing sites are displayed in GoogleMap® in Figure 3-1.

Table 3-1 Testing Sites for ODOT Project SP&R 2306

Treatment Type	Highway	Completion Date	# of Lanes	Age③ (yrs)	AADT	Start Latitude	Start Longitude	End Latitude	End Longitude
Chip Seal	SH-11	Aug 2017	2	2.1	1,572	36.811312	-98.074823	36.811395	-98.019699
Chip Seal	US-81	Late 2016	4	2.8	7,025	35.883830	-97.932807	35.89863	-97.932984
UTBWC	I-35 ①	10/31/2015	4	4.0	117,263	35.320032	-97.489843	35.333824	-97.489964
UTBWC	US-69 ①	4/10/2015	4	4.3	15,938	34.630777	-95.956830	34.618544	-95.966249
UTBWC	US-177	5/6/2016	4	3.4	6,400	36.601094	-97.075970	36.615428	-97.075769
Resurface (HMA)	I-40 ①	8/27/2013	4	6.1	39,652	35.384105	-97.142144	35.384043	-97.124881
Resurface (HMA)	US-69 ①	10/23/2016	4	3.0	19,600	34.878994	-95.795848	34.874646	-95.800424
Resurface (HMA)	SH-51	9/5/2014	2	5.1	3,967	36.116240	-96.872502	36.116223	-96.855061
Resurface (HMA)	US-64 ②	9/1/2017	4	2.1	6,400	36.289580	-97.326090	36.289663	-97.308270
Micro-surfacing	Lakeview Rd ②	3/11/2015	4	4.5	5,443	36.145126	-97.070381	36.144978	-97.087410
WMA Overlay	SH-66(LTPP)	6/11/2015	4	4.3	5,767	35.507971	-97.792731	35.507987	-97.824039
Resurface(Concrete)	I-35 (Exit 170-174)	NA	4	NA	24, 000	36.055158	-97.345019	36.111081	-97.345006
Resurface(Concrete)	SH-33	6/1/2015	4	4.3	10,592	35.880167	-97.412350	35.878079	-97.395522
NGCS	US-77	?/?/2009	4	9.4	22,523	35.265704	-97.477576	35.274498	-97.481400
Long Grooving	I-40 ①	NA	6	NA	46,130	35.490895	-97.768640	35.482942	-97.753825
HFST	I-40 ① -1	10/3/2015	4	4.1	55,200	35.435075	-97.392165	35.434341	-97.398857
HFST	I-40 ①② -2	10/3/2015	6	3.1	62,248	35.434340	-97.405709	35.436512	-97.411423
HFST	SH-20 ②-1	8/29/2015	2	4.1	3,200	36.310404	-95.132119	36.311426	-95.128759
HFST	SH-20 ② -2	12/22/2013	2	5.8	3,200	36.323257	-95.121682	36.325060	-95.120359

Note: ① ODOT skid resistance program sites; ②ODOT SP&R 2275 sites; ③ age in years till Oct 2019.

Table 3-2 Bridge Decks on I-35-NB

#	Latitude	Longitude	Length (ft)	Year Reconstructed	Surface Type
1	35°52'40.56"N	97°23'39.59"W	49.4	1989	5 - Epoxy overlay
2	35°59'04.92"N	97°21'10.91"W	244.8	1980	4 - Low slump concrete
3	35°59'26.89"N	97°21'07.00"W	415.8	/	1 - Monolithic concrete
4	36°00'13.47"N	97°20'58.69"W	64	1988	1 - Monolithic concrete
5	36°04'19.67"N	97°20'41.96"W	31.1	2006	4 - Low slump concrete
6	36°13'36.43"N	97°19'40.80"W	74.4	1974	4 - Low slump concrete
7	36°17'06.60"N	97°19'39.18"W	40.5	2013	0 - None
8	36°20'56.88"N	97°19'36.33"W	103.9	1975	6 - Bituminous
9	36°28'31.91"N	97°19'36.65"W	100.6	1975	3 - Latex concrete or similar additive
10	36°40'30.84"N	97°20'44.35"W	178	1984	4 - Low slump concrete

Note: All deck structure is concrete cast-in-place.

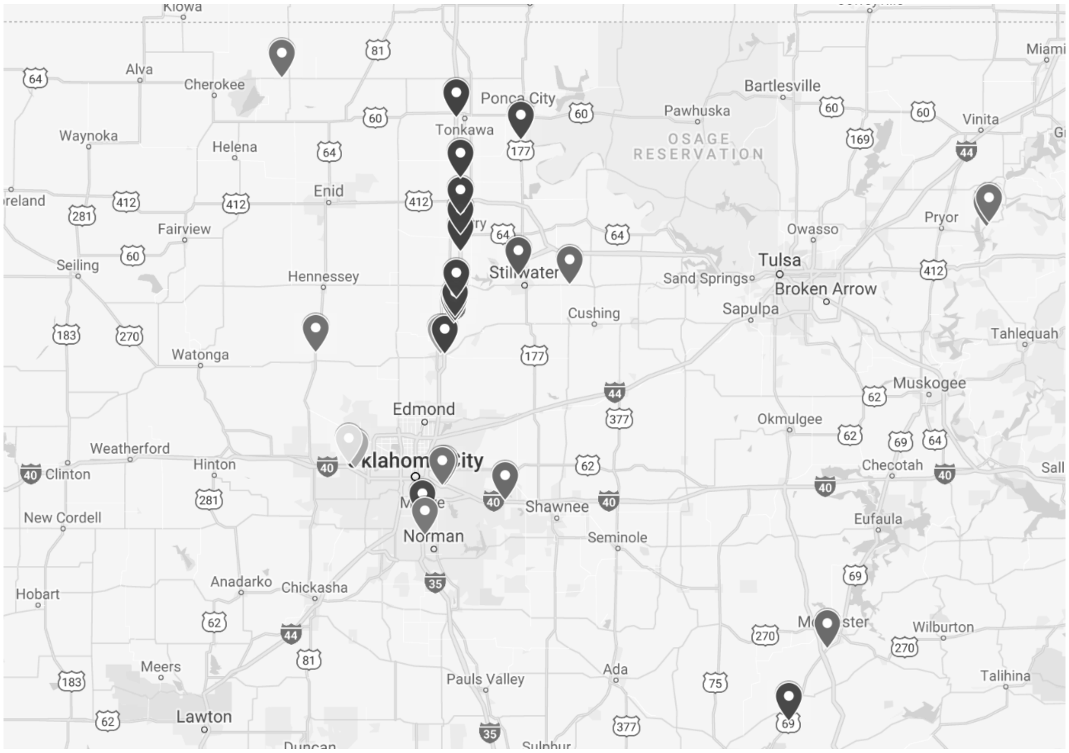


Figure 3-1 Locations of Field Testing Sites on GoogleMap

3.2 Field Data Collection

For each testing site, multiple runs of friction data collection were conducted using the Grip Tester at various testing speeds, water film thicknesses, and ambient temperatures. The data collected at these sites were further used to assess the repeatability of CFME friction measurements, and to compare the measurements from locked-wheel friction trailer and Grip Tester. The collected field data were also used for the development of crash rate prediction models.

According to the ASTM "*Standard Test Method for Skid Resistance of Paved Surfaces Using a Full-Scale Tire*" (ASTM E274 / E274M - 15), the standard operational conditions of locked-wheel friction measurement are 40 mph in terms of testing speed and 0.25 mm of water film thickness. For safety considerations, ODOT collects locked-wheel friction data at 50 mph for its annual skid resistance testing program on the Oklahoma interstates and US-69 truck corridor. Therefore, Grip Tester friction measurements were made at both 40 mph and 50 mph of testing speeds, so that the comparisons with ODOT locked-wheel testing data are possible. Two additional testing: 30 mph on minor arterials and 60mph on major arterials, were also performed to understand the influence of testing speed on friction measurements. For water film thickness, three scenarios including 0.25 mm (standard), 0.5 mm and 1.00 mm, were applied during the testing of each location to simulate various levels of rain intensity.

Besides friction measurements, pavement surface texture data were acquired using the PaveVision3D data vehicle equipped with the AMES® high speed profiler at highway speed with full-lane coverage. An overview of each instrument and its role in this project is given below.

<u>Grip Tester</u>

Grip Tester has been used in recent years by FHWA on many demonstration projects in the United States. It is designed to continuously measure the longitudinal friction along the wheel path operating around the critical slip of an anti-braking system (ABS). The device has the capability to test at highway speeds (50 mph) as well as low speeds (20 mph) using a constant water film thickness. The collected data are recorded in 3-ft (0.9 m) interval by default and can be adjusted by the user.

This method follows the ASTM E274 - 11 "Standard Test Method for Skid Resistance of Paved Surfaces Using a Full-Scale Tire".

(a) Grip Tester

(b) ODOT Locked Wheel Tester

(c)DHDV with PaveVision3D Ultra

(d) AMES 8300 Survey Pro Profiler

Figure 3-2 Data Collection Devices

<u>Locked-Wheel Friction Tester</u>

Locked-wheel friction tester (AASHTO T 242) is the predominant friction measurement device used in the United States. This device requires a tow vehicle and a locked-wheel skid trailer, equipped with either a standard ribbed tire (AASHTO M 261) or a standard smooth tire (AASHTO M 286). Friction measurements are obtained by locking the test tire (ribbed or smooth) on a wetted pavement surface while traveling at a specified standard speed of 40 mph (AASHTO T 242). The smooth tire is more sensitive to pavement macro-texture, while the ribbed tire is more sensitive to micro-texture of pavement surfaces.

PaveVision3D Data Vehicle

The PaveVision3D data vehicle is used to collect 1mm 3D surface data with full-lane coverage for pavement distresses, transverse and longitudinal profiles, and texture data at highway speed of up to 60 mph. In particular, pavement geometric data (including horizontal curve, longitudinal grade, and cross slope) are continuously acquired with an Inertial Measurement Unit (IMU) equipped in PaveVision3D. The collected 1 mm data sets can be utilized to analyze the impacts of pavement conditions and surface characteristics on CFME friction measurements, while the geometric data form IMU can be used to develop crash rate prediction models along with the CFME data for pavement friction management in Oklahoma.

AMES® High Speed Profiler

The Model 8300 Survey Pro High Speed Profiler is designed to collect macro surface texture data along with standard profile data at highway speeds. Multiple texture indices such as Mean Profile Depth (MPD) can be calculated from the testing data. This High Speed Profiler meets or exceeds the ASTM E950 Class 1 profiler specifications, AASHTO PP 51-02 and Texas test method TEX 1001-S. Texture data were used to investigate the impacts of texture on pavement friction and to develop the relationship between friction and texture.

CHAPTER 4 EVALUATION OF GRIP TESTER MEASUREMENTS

Pavement friction and surface characteristics data used in this Chapter were collected in August and November 2018 on 22 selected field testing sites. For each site, pavement friction was measured using the Grip Tester under nine operating conditions at three different vehicle speeds (40, 50, 60 mph for major arterials; 30, 40, 50 mph for minor arterials) and three different water film thicknesses (0.25, 0.50, 1.00 mm). The 1 mm 3D pavement surface and macro-texture data were collected using the PaveVision3D data vehicle at posted highway speed under dry surface condition since these characteristics are measured via non-contact methodology and do not vary with changing testing conditions. In this Chapter, the CFME testing results under various conditions were summarized with descriptive statistics. Further, the performance of CFME was evaluated from three aspects: (1) the correlation with Locked-wheel measurements, (2) the repeatability of CFME data, and (3) the influence of operational characteristics on CFME measurements.

4.1 Descriptive Statistics of CFME Measurements

The descriptive testing results summarized in this section include (1) Friction by treatment type; (2) Friction by testing speed; (3) Friction by water film thickness, and (4) Friction of the 10 bridge desks on I-35.

4.1.1 Friction by Treatment Type

Figure 4-1 presents the average friction number by treatment type measured at the standard testing conditions (40 mph of testing speed and 0.25 mm of water film thickness). The treatment types herein include Chip Seal (ChipS), Ultra-Thin Bounded Wearing Course (UTBWC), Asphalt Resurface (HMA), Micro-surfacing (MicroS), Concrete Resurface (PC), Longitudinal Groove (LGroove), Transverse Groove (TGroove), Next Generation Diamond Grinding (NGDG), High Friction Surface Treatment (HFST), and Warm Mix Asphalt (WMA). As shown in Figure 4-1, the two HFST sites on I-40 show the best friction performance, followed by the transverse grooving site on US-77. The other testing sites, with various surface types, are observed to have comparable friction performance ranging from 0.35 to 0.62. Meanwhile, since friction number could be highly related to treatment age and traffic exposure, Figure 4-2 and Figure 4-3 also provide the treatment age for each site and its traffic information in terms of Annual Average Daily Traffic (AADT). Missing data are shown as blank columns in the figure.

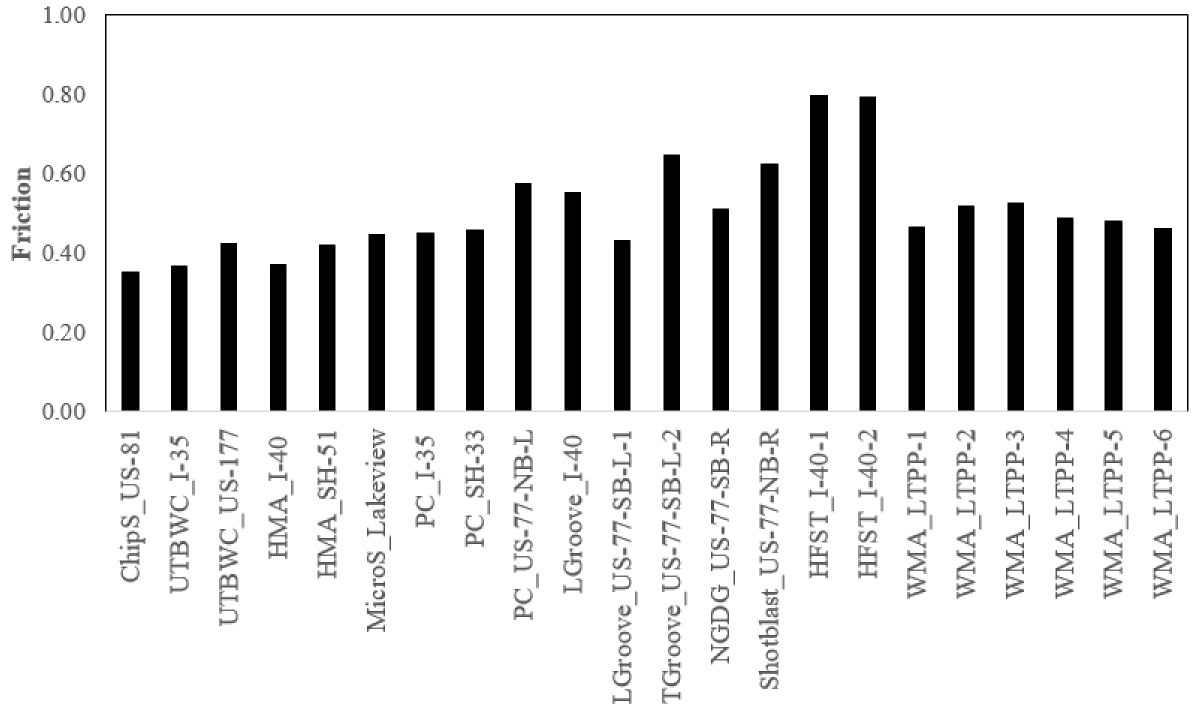


Figure 4-1 Friction vs. Treatments

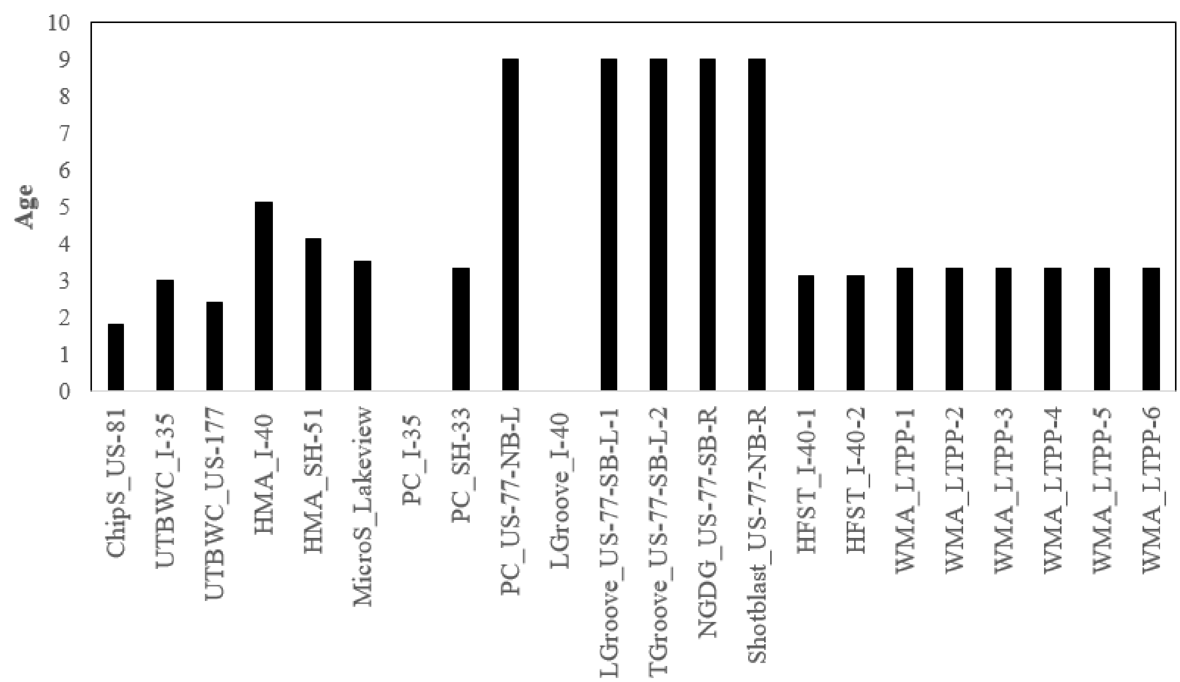


Figure 4-2 Treatment Ages of Testing Sites

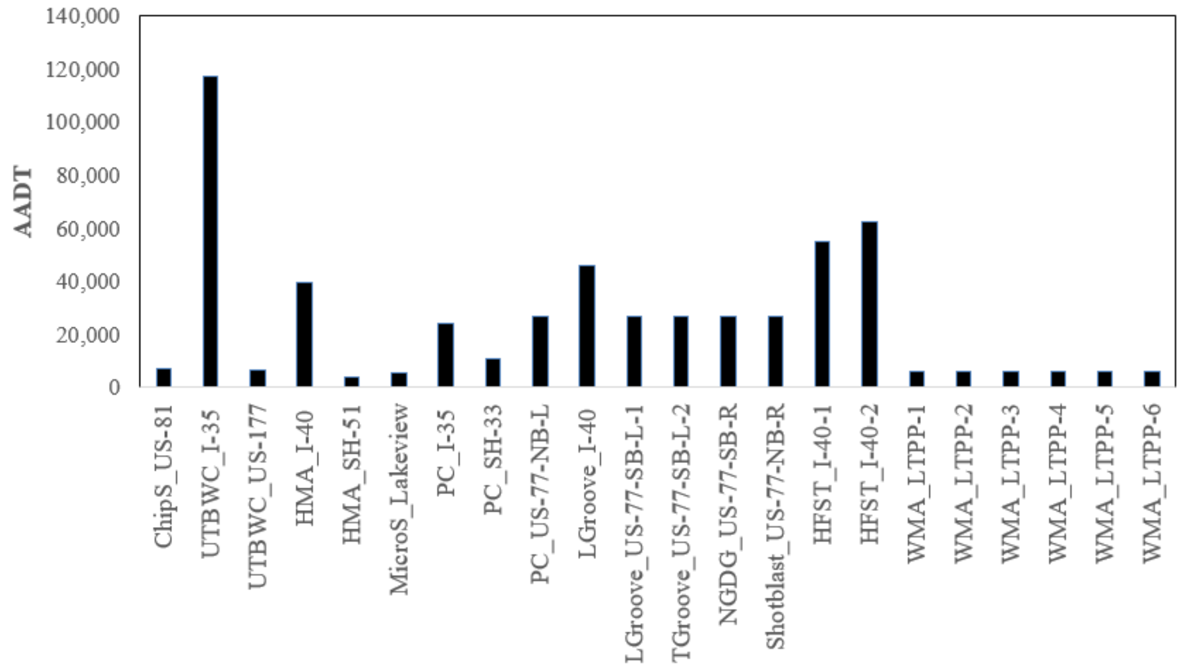


Figure 4.3 AADT of Testing Sites

4.1.2 Friction by Testing Speed

To investigate the effects of testing speeds on pavement friction, the comparison of friction numbers at different testing speeds under the “base” water film thickness (0.25 mm) is shown in Figure 4-4. For most sites, with increasing testing speeds, the friction numbers show decreasing trend, which is consistent with previous research work. However, the effects of varying testing speed differ for different treatment types. Some sites demonstrate pronounced reduction in friction performance with increasing speeds, such as chip seal, UTBWC, HMA and some WMA sites, while others are not as significant. Particularly, HFST as the example, it seems that their friction performance does not depend on testing speeds.

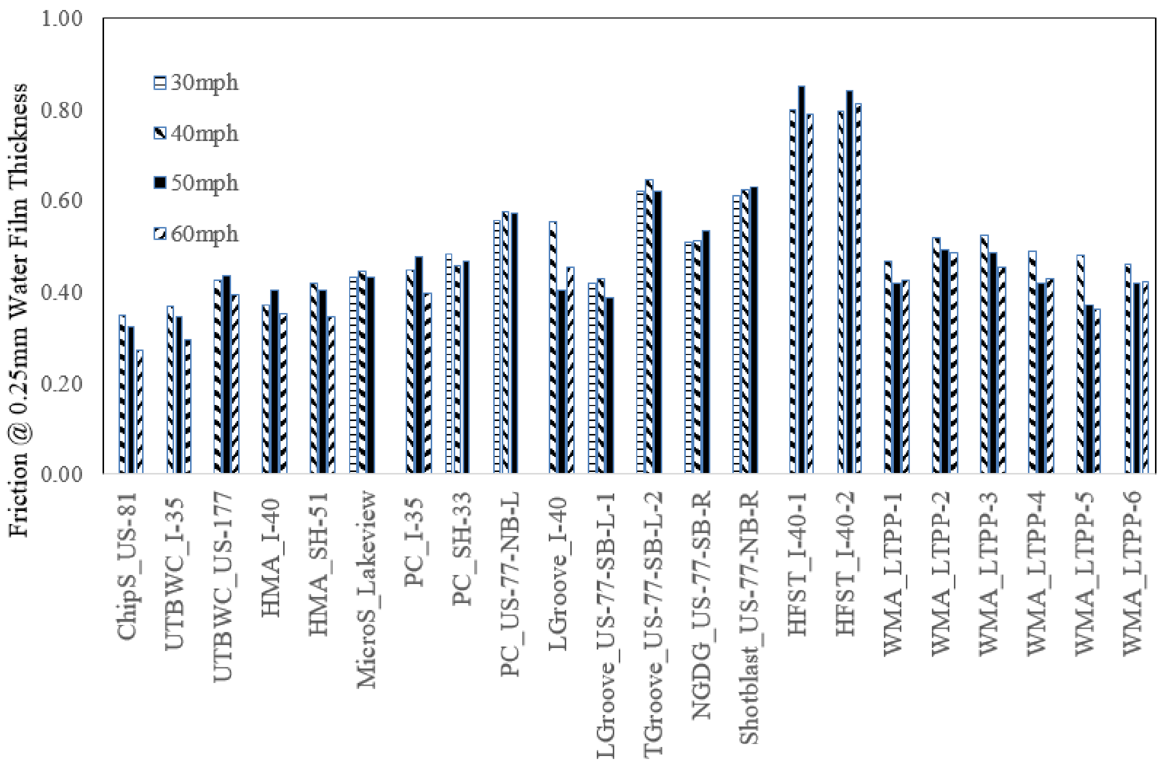


Figure 4-4 Friction vs. Vehicle Speeds

4.1.3 Friction by Water Film Thickness

Similarly, friction numbers were measured at three different water film thicknesses but compared at the standard testing speed (40 mph), as shown in Figure 4-5. It is shown that friction numbers decrease when larger water film thickness is applied. For some sites, the friction values are significantly reduced when the water film thickness is increased from 0.25 mm to 0.5 mm. On the other hand, HFST sites for example, the impact of water film thickness for some sites does not demonstrate such significant influences.

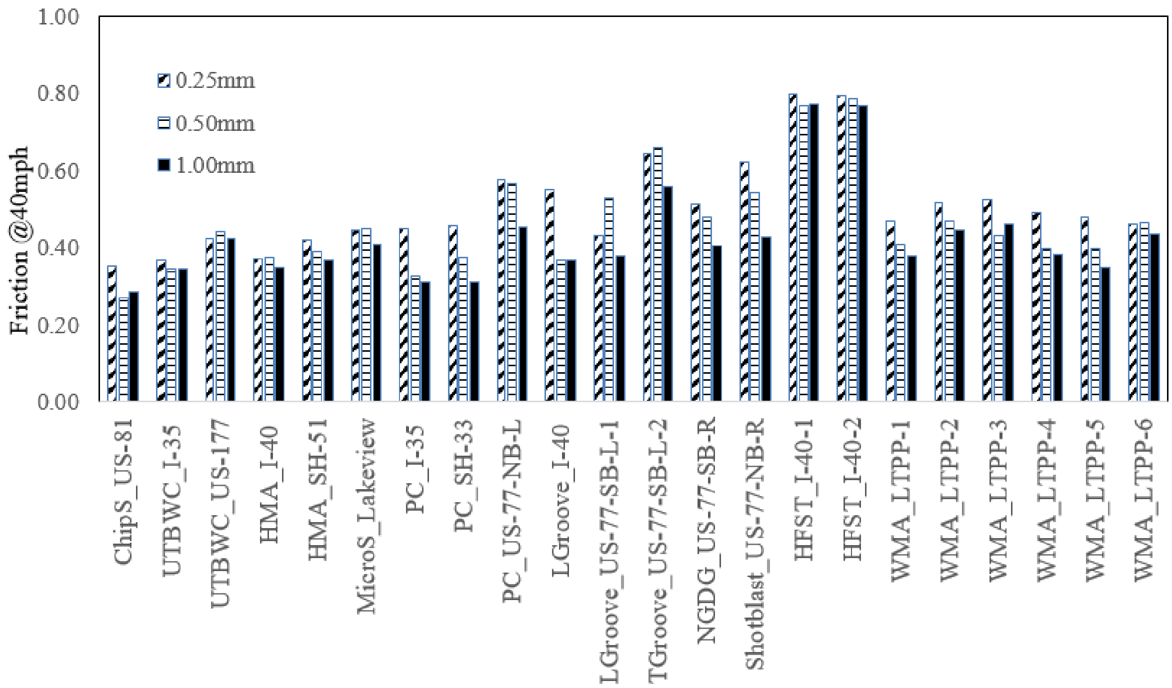


Figure 4-5 Friction vs. Water Film Thickness

4.1.4 Friction on Bridge Desks

The friction values of the 10 bridges on I-35, both the South and North bound, are presented in Figure 4-6 and Figure 4-7 from 2011 to 2018. It should be noted that the friction data, with the range from 0 to 100, were measured from the ODOT locked-wheel testing trailer.

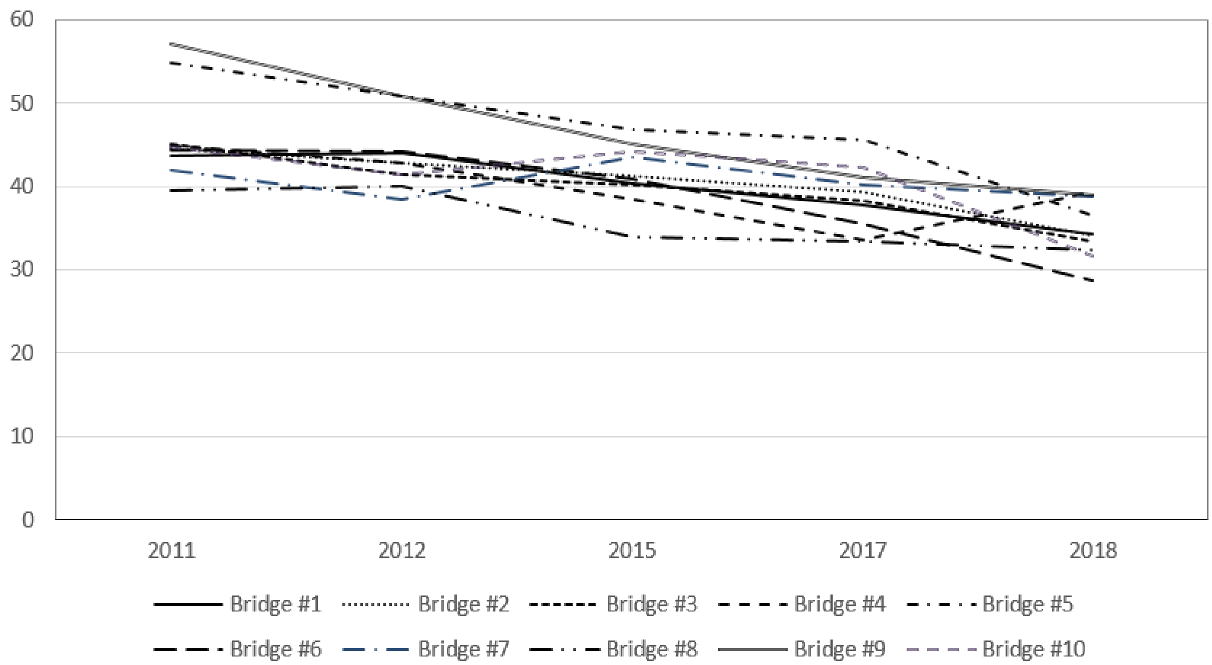


Figure 4-6 Friction on Bridge Decks (Northbound I-35)

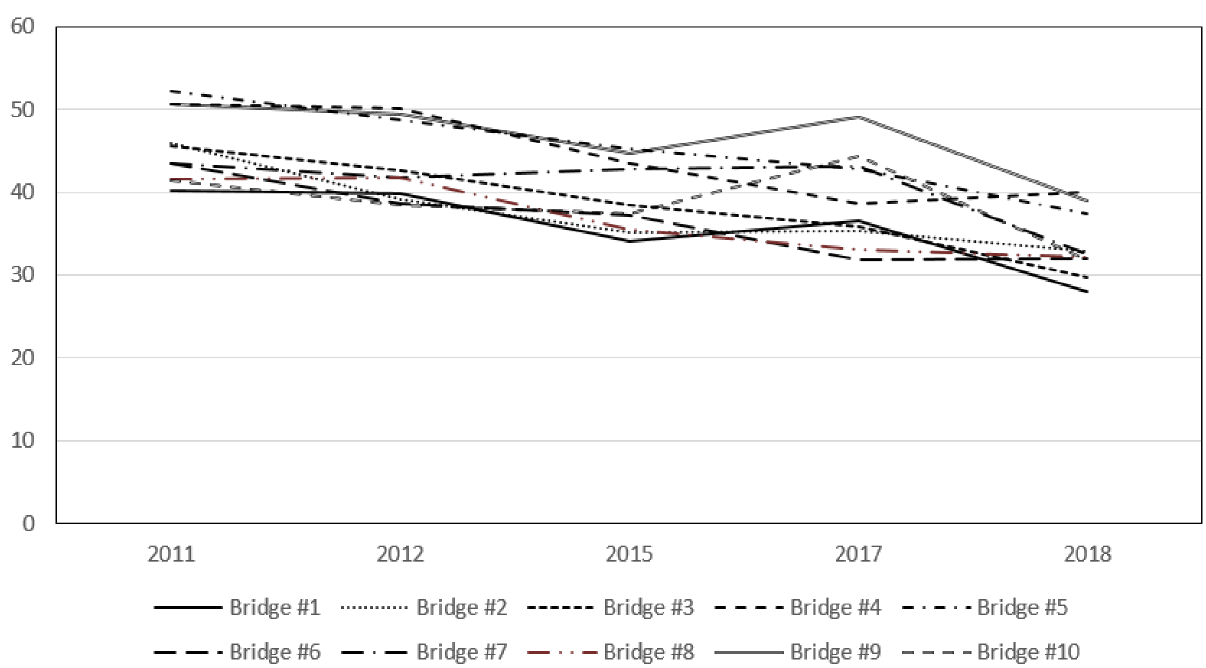


Figure 4-7 Friction on Bridge Decks (Southbound I-35)

4.2 Comparison Analysis with Locked-Wheel Measurements

The Oklahoma Department of Transportation (ODOT), like most of the states in the United States, uses locked-wheel skid tester (LWST) fitted with a ribbed tire (ASTM E274) as the predominant method for pavement skid resistance measurements. This method is believed to behave well in detecting micro-texture properties of pavement surfaces (McCarthy et al. 2018). Besides, the measurement of an LWST is periodic and discrete, which is the average fiction value measured in a fully locked state over a specific period (ASTM E501). In recent years, continuous friction measurement equipment (CFME) has been widely used for the measurement of pavement friction. Grip Tester is one of the CFME device that measure pavement friction at an interval up to 1 meter or 3 ft. In this section, the correlation of friction measurements from LWST and Grip Tester are evaluated.

4.2.1 Data Collection

Grip Numbers (GN) were collected using the OSU Grip Tester on 4 ODOT control sections (35-42-30, 35-60-29, 35-52-33 and 35-36-25 respectively), with a total of 85 miles on Interstate-35 (I-35). The testing speed was 40mph, the measurement water film depth was 0.25 mm, and the GN was reported at 1-m intervals. A Global Positioning System (GPS) was integrated in the Grip Tester to record the latitude and longitude at 10Hz frequency. The collected GNs were mapped as slim column on Google Maps®, as shown in Figure 4-8. The column in blue, green, yellow and red represents GN numbers collected from section. During the measurement, the water tank of the Grip Tester was filled twice, and thus the friction measurements were disrupted, as showed by the two blanks in the figure.

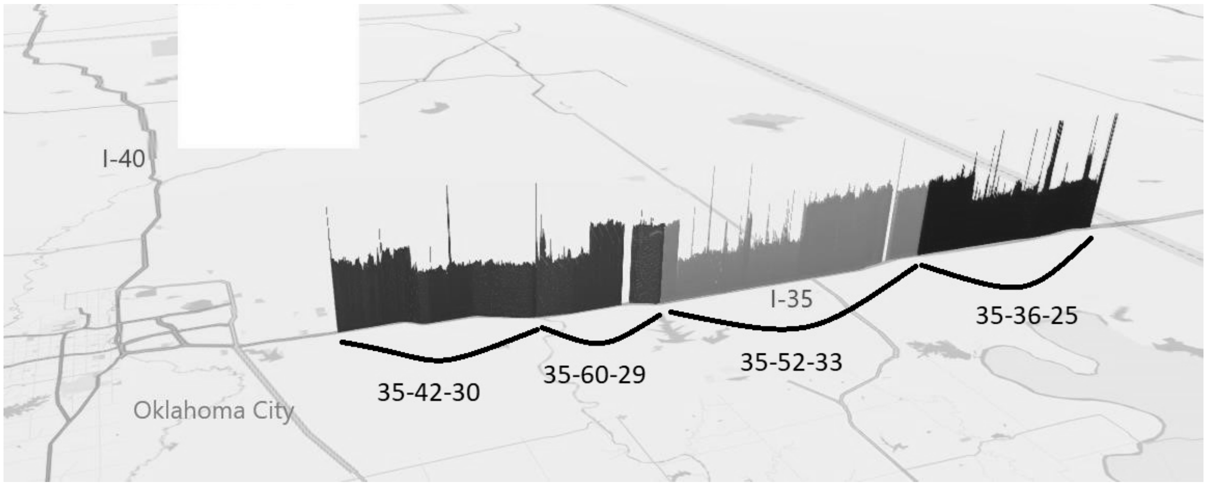


Figure 4-8 Grip Numbers on Google Maps

Meanwhile, the Skid Number (SN) on the same sections were also measured by the ODOT LWST. The testing interval was approximately 0.5 mile. The LWST data collection always started at the beginning of each control section, and tested every 0.5 miles thereafter. Each data point corresponded to a particular mile post of the control section.

In addition, the ODOT Pavement Management System (PMS) data for those sections were also acquired, which were saved at an interval of 0.01 miles. The PMS data included various data sets, such as traffic flow, roadway geometry, pavement roughness, surface macrotexture, and pavement distresses. The PMS data were referenced by the mile post of the control section and also the GPS coordinates.

4.2.2 Data Compilation

The data collection interval and their geographical reference information were summarized in Table 4-1. The GN are collected at an interval of 1 meter (3 feet) with GPS coordinates recorded, while the SN collected by LWST was at the interval of 0.5 mile with recorded mile post for each control section. The intervals of the PMS data were 0.01 mile with both GPS coordinate and mile post. Therefore, the three data sets were linked per the mile post, or GPS coordinates. In total 188 data pairs with SN, GN and corresponding PMS data were obtained.

Table 4-1 Location Reference and Reporting Interval

Data Item	Data Interval	Mile post	GPS coordinates
SN	0.5 mile	×	
PMS	0.01 mile	×	×
GN	3 feet		×

The friction data were considered as random variables whose dispersion is attributed to random errors or noise of measurement and heterogeneities on the surface. The random errors, statically named as outliers, were identified and removed using the Hampel based filtering. The Hampel filter is a sliding window implementation of the Hampel identifier to calculate the outlier sensitive z-score. To provide robust estimation of μ and σ in the contaminated data, the Hampel process uses the median and Median Absolute Deviation (MAD) as the outlier resistant parameters. Assuming a sliding window containing the prior w values of grip numbers $GN_i^w = \{GN_{i-w}, ..., GN_i\}$, the median and the scale (MAD) of GN_i^w is computed as follows.

$$\varphi_w = \mathit{median}(GN_i^w) \qquad \text{(Eq. 4-1)}$$

$$S_w = 1.4826 \times \mathit{median}\{|GN_i^w - \varphi_w|\} \qquad \text{(Eq. 4-2)}$$

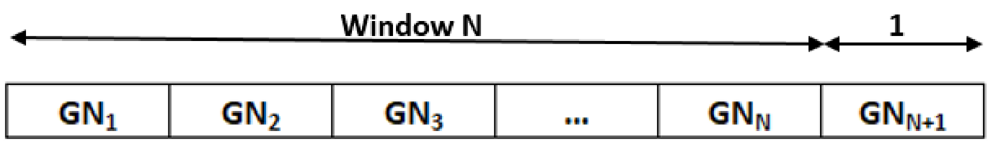


Figure 4-9 Sliding Window of the Hampel Filter

After replacing the mean μ by the median φ_w, and the standard deviation σ by Sw, the z-score is used to test if the new value Ei+1 is abnormal, where k is a threshold value (k = 1.96 in this study).

$$|E_{i+1} - \varphi_w| \geq k \times S_w \qquad \text{(Eq. 4-3)}$$

As illustrated in Figure 4-9, the first step of Hamper filter is to get the median, φ_w of a sliding window (Eq. 4-1) that is composed of a point and its N surrounding samples, N/2 per side. Then the MAD of the window can be estimated by Eq. 4-2. If a sample differs from the median by a threshold value, it is identified as an outlier, as shown in Figure 4-10(b). The detected outliers are replaced by φ_w, the median of the sliding window. A denoised pavement friction profiler is finally obtained and ready for further analysis, as shown in Figure 4-10(c). This process is completed in R programming.

It is generally agreed that any denoising process could sacrifice some details of the data sets. The data sets shown in Figure 4-10 were several miles long collected on I-35 with several bridge decks. The data sets were carefully examined and it was found that most majority if not all of the spikes in the data occurred near joints (at transitions from regular roadway to bridge or vice versa, and joints on the decks). This is logic because the friction trailer maybe bounced up to some degree and the contact between the testing tire and roadway surface was partially lost. In addition, the data interval of the Grip Tester is a meter. A significant difference of friction between consecutive data points is highly problematic. Therefore, it is concluded that this denoising method is valid and necessary.

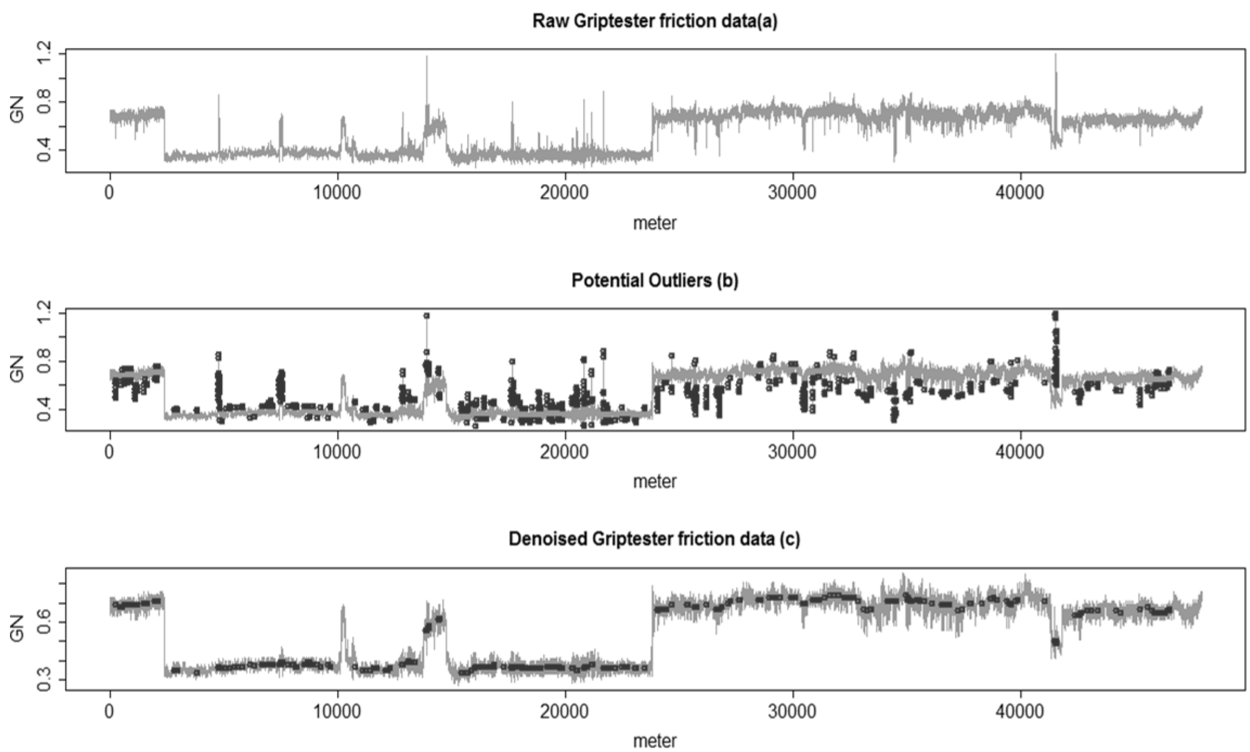


Figure 4-10 Grip Number (GN) Before and After Outlier Removal

4.2.3 Data Comparisons

The SNs from LWST and GNs from Grip Tester were compared in Figure 3. The GN, which was saved at the range of 0 to 1, were multiplied by 100 so that they fell into the same data range of SN. In general, the GNs are larger than the SNs for the same roadway sections. This is justifiable since the Grip Tester operates at the critical slip while the locked wheel trailer operates at 100% full sliding conditions. Per the friction-slip curve as shown in Figure 4-10, the friction number measured at the critical slip is greater than that at the fully-locked.

The pavement type of each section was also labeled in the figure. There are three types of pavements: asphalt concrete (AC), jointed concrete pavement (JCP) and continuously reinforced concrete pavements (CRCP). As shown in Figure 4-11, the SN, obtained with LWST, is not as sensitive as the GN to the change of pavement types. Since the LWST used ribbed tires during testing, its measurements are more sensitive to the surface micro-texture performance (McCarthy et al. 2018), while the friction on JCP and CRCP are primarily provided by their macro-texture components resulting from surface grooving. As a result, the Grip Tester recorded much higher friction numbers on concrete pavements.

Per Figure 4-11, pavement surface type seems to be a significant factor to correlate SN and GN. Figure 4-12 plots the simple linear regression results of SN and GN: (a) for all the data samples, (b) and (c) for asphalt and Portland pavements separately. The coefficient of determination, R2 indicates the strength of the model, ranging from 0 to 1 where 1 supports the absolute exact linear regression. Overall all the three linear models are not robust with the highest R^2 value being 0.3185. The trend for asphalt pavements seem to be more consistent than that for the rigid surfaces.

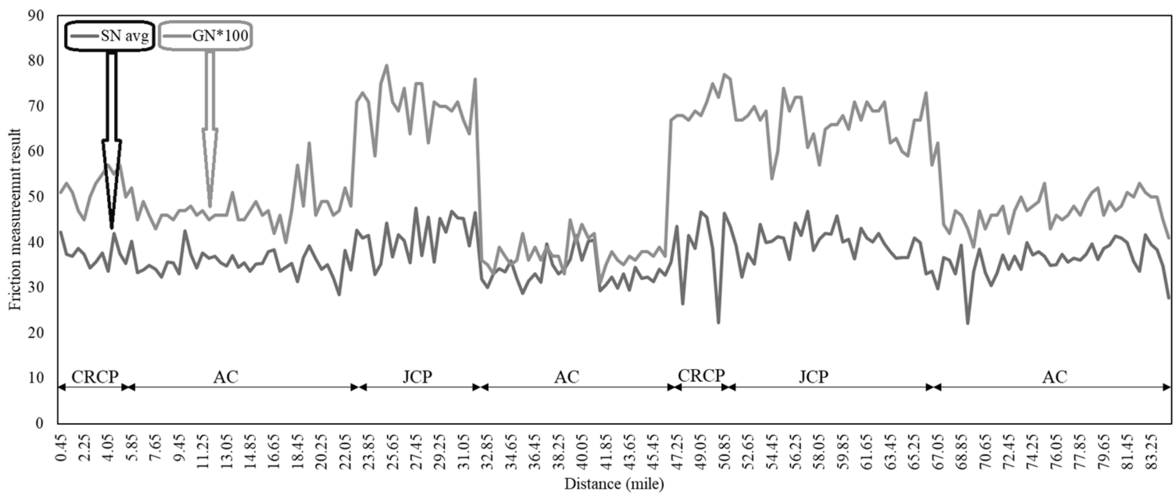


Figure 4-11 Comparison of SN and GN (I-35 NB)

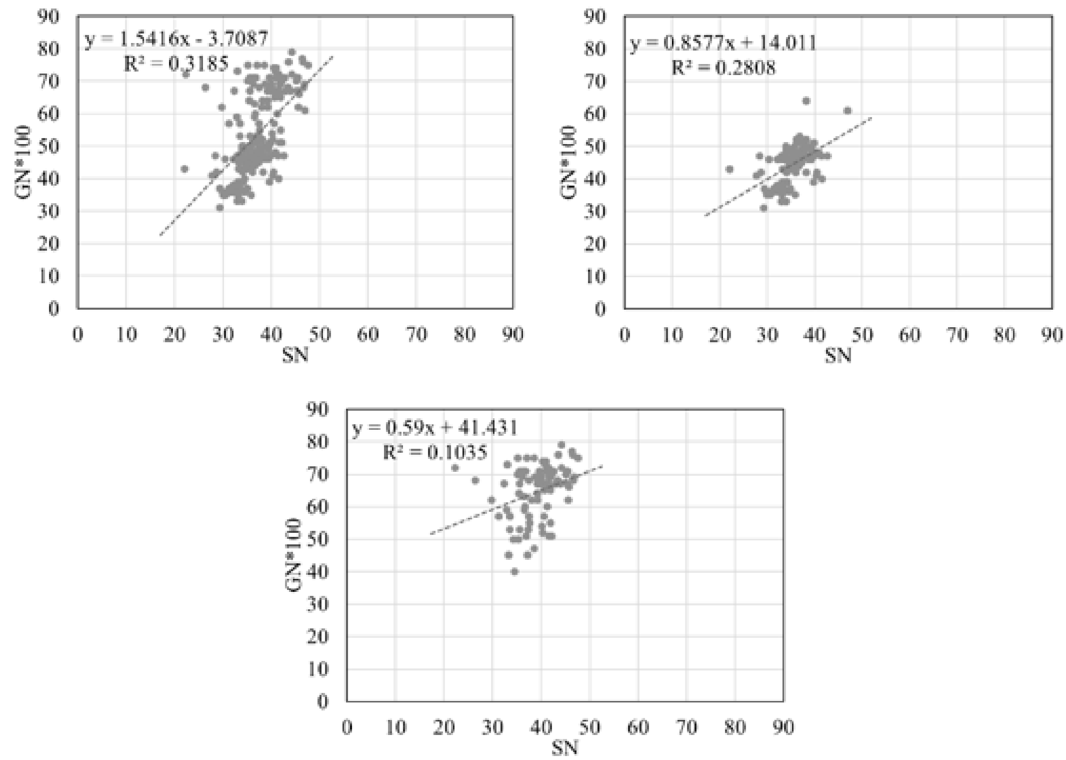


Figure 4-12 Linear Correlation Results (a) All Samples, (b) AC, (c) PC Samples

4.2.4 Model Development

Multi-variant regression was applied to develop the correlation between SN and GN. The dependent variable was the grip number (GN) from the Grip Tester measurements. The potential influencing factors included the skid number (SN) from the LWST, roadway surface characteristics (surface macrotexture in terms of mean profile depth – MPD; pavement roughness index – IRI; and the ODOT specific performance indices), roadway geometry (degree of curvature, longitudinal grade, cross fall), type of terrain (rural and urban), and pavement type (asphalt and concrete). The degree of curvature data in the PMS database for the parallel testing section on I-35 were mostly zeros, and thus it was removed.

ODOT's primary performance measure for pavement condition is Pavement Quality Index (PQI). Each pavement type has several summary condition indices as well as an overall PQI that can be calculated based on aggregated subsection pavement distress data. The PQI score, on a scale of 0 to 100, is the weighted average of various PQI component that reflects the primary distresses found in each pavement type in Oklahoma.

- For asphalt concrete pavements, such components in PQI include ride, rut, functional, and structural indices.
- For JPCP pavements, the components in PQI include ride index, faulting/joint measures, and slab index.
- For CRCP pavements, it only includes ride and structural indices.

Correlation analysis was conducted among the various pavement surface characteristics to remove the parameters who exhibited strong correlations and remove their potential multicollinearity for regressional model development. Correlation coefficient of 0 means that there is no correlation, -1 denotes a perfect negative correlation, while +1 suggests a perfect positive correlation between the two variables. A correlation greater than 0.8 is generally described as strong, whereas a correlation less than 0.5 is generally described as weak (MathBits, 2019).

Three sets of correlation analyses and models were developed herein: one for all the data samples (both asphalt and concrete pavements), another for asphalt pavement data only, and the last for JPCP pavement data only. The correlation analyses revealed that PQI and ride index had strong correlations with IRI for both asphalt and concrete pavements. Therefore, IRI was kept in the model development while PQI and ride index removed. As a result, the following pavement condition indices were maintained as the influencing variables in the three sets of statistical models.

- For all data samples: IRI;
- For asphalt samples: IRI, rut index, and structural index;
- For concrete samples: IRI, faulting index, and slab index.

In addition, although the automated watering system for the grip tester was set to generate 0.25 mm water film thickness and the testing vehicle was under cruise control at the speed of 40 mph, the water flow recorded in the testing data sets showed noticeable variations. Therefore, the water flow rate (in liters per minute) was also included as one of the independent variables.

Variable selection is a process to determine a set of independent variables for the final regression model from a pool of candidate variables. On one hand, the subset of the independent variables needs to be as complete and realistic as possible. On the other hand, the independent variables included should be as few as possible to eliminate irrelevant variables, which will decrease the precision of the model as well as increase the complexity of data collection. To balance the goodness-of-fit and model simplicity, the backward stepwise method was implemented for the model development. It is a stepwise regression approach which begins with a full (saturated) model and at each step gradually eliminates the least significant variables from the regression model for a reduced model that best explains the data. The multivariate method with backward regression was conducted in R programming. The Akaike information criterion (AIC), an estimator of out-of-sample prediction error and thereby relative quality of statistical models for a given set of data, was used as the parameter for model evaluation.

Accordingly, three sets of regressional models were developed: one based on data from all pavement types, one for asphalt sections and the third for concrete pavement sections. The statistical results are shown in Table 4-2 to Table 4-4. The p-value for each factor tests the null hypothesis that the coefficient is equal to zero (no effect). A low p-value (< 0.05) indicates that the null hypothesis can be rejected. In other words, a predictor that has a low p-value is likely to be a meaningful addition to the model because changes in the predictor's value are related to changes in the response variable. The skid number from LWST is a significant factor to correlate with grip number for all the models. Meanwhile, there are several other factors that have showed statistical significance to the grip tester measurements. The R squared value of the first model reaches 0.77, and the p-value is significantly smaller than 0.05, indicating the correlation between the model and dependent variable is statistically significant. However, the R squared values for the separated models for asphalt and concrete pavements are much lower (0.35 and 0.39 respectively), while the p-values remain very small, indicating the models are statistically significant but the scatter around the regression line increases. In case of a low R-squared value but the independent variables are statistically significant, important conclusions can still be drawn about the relationships between the variables. Statistically significant coefficients continue to represent the mean change in the dependent variable given a one-unit shift in the independent variable.

Table 4-2 Backward Stepwise Final Model Results: All Sections

Category	Variable	Estimate	Std. Error	t value	P-value	Sig. Level
Constant	Intercept	26.52	4.81	5.52	0.00	***
GT	Water Flow	-0.54	0.15	-3.53	0.00	***
LWST	Skid Number	0.53	0.11	4.68	0.00	***
Surface Condition	IRI	-0.03	0.01	-2.05	0.04	*
Roadway Geometry	Cross Fall	-1.42	0.69	-2.07	0.04	*
Roadway Geometry	Longitudinal Grade	-1.13	0.64	-1.77	0.08	.
Others	Surface Type	6.80	0.39	17.63	< 2e-16	***

Notes: (1) Significance codes: 0 - '***', 0.001 - '**', 0.01 - '*', 0.05 - '.', 0.1 - ' ';

(2) Residuals: Min (-18.70), 1Q (-3.68), Median (0.33), 3Q (3.44), Max (18.13);

(3) Multiple R-squared: 0.7816, Adjusted R-squared: 0.7742;

(4) F-statistic: 105.6 on 6 and 177 DF, p-value: <2.2e-16

Table 4-3 Backward Stepwise Final Model Results: Asphalt Sections

Variable	Symbol	Estimate	Std. Error	t value	P-value	Sig. Level
Constant	(Intercept)	6.50	4.77	1.36	0.18	
LWST	Skid Number	0.78	0.13	6.03	0.00	***
Surface Condition	Functional Index	0.06	0.02	2.52	0.01	*
Others	Terrain Type	0.58	0.28	2.06	0.04	*

Notes: (1) Significance codes: 0 - '***', 0.001 - '**', 0.01 - '*', 0.05 - '.', 0.1 - ' ';

(2) Residuals: Min (-10.36), 1Q (-2.76), Median (0.27), 3Q (2.65), Max (13.13);

(3) Multiple R-squared: 0.3486, Adjusted R-squared: 0.3292;

(4) F-statistic: 18.02 on 3 and 101 DF, p-value: <1.932e-09

Table 4-4 Backward Stepwise Final Model Results: Concrete Sections

Category	Variable	Estimate	Std. Error	t value	P-value	Sig. Level
Constant	Intercept	60.71	7.82	7.77	0.00	***
GT	Water Flow	-1.02	0.26	-4.01	0.00	***
LWST	Skid Number	0.40	0.17	2.33	0.02	*
Surface Condition	IRI	-0.11	0.04	-2.92	0.00	**
Surface Condition	Macrotexture	6.22	3.31	1.88	0.06	.
Surface Condition	Average Faulting	152.35	57.67	2.64	0.01	*
Others	Terrain Type	-1.36	0.86	-1.59	0.12	

Notes: (1) Significance codes: 0 - '***', 0.001 - '**', 0.01 - '*', 0.05 - '.', 0.1 - ' ';

(2) Residuals: Min (-17.67), 1Q (-3.17), Median (0.60), 3Q (4.90), Max (11.92);

(3) Multiple R-squared: 0.3855, Adjusted R-squared: 0.3343;

(4) F-statistic: 7.529 on 6 and 72 DF, p-value: <2.768e-06.

One important item in the model output is the residuals. Residuals are essentially the difference between the actual observed response values and the response values that the model predicted. A residual plot is a graph that shows the residuals on the vertical axis and the independent variable on the horizontal axis. If the points in a residual plot are randomly dispersed around the horizontal axis, a linear regression model is appropriate for the data; otherwise, a non-linear model is more appropriate. In addition, the norm Q-Q plot, or quantile-quantile plot, is a graphical tool to help assess if a set of data plausibly came from normal distribution. The residual and Q-Q plots for the first model (with all data samples) are shown in Figure 4-13. It can be seen that the model residual presents a symmetrical distribution across these points on the mean value zero, and the norm Q-Q plot suggests it follows the normal distribution and thus linear regression model is statistically valid. On the other hand, on the residual plot, it is observed that the residuals scatter with a wide range. This is also echoed in the model summary statistics as shown in Table 4-2. The Residuals section of the model output breaks it down into 5 summary quantile points. The median residual is 0.33, the first and third quartiles are -3.68 and 3.44. In other words, 50% of the grip number predictions have less than 4.0 of difference (out of 100 scale), implying the model is a good fit. However, the maximum and minimum of residuals is -18.70 and 18.31, indicating that some of the predictions will have significant variations and thus the variability present in the field friction measurement data. Further research is therefore highly needed to reduce such variability for friction measurements and enhance the reliability of friction testing instruments.

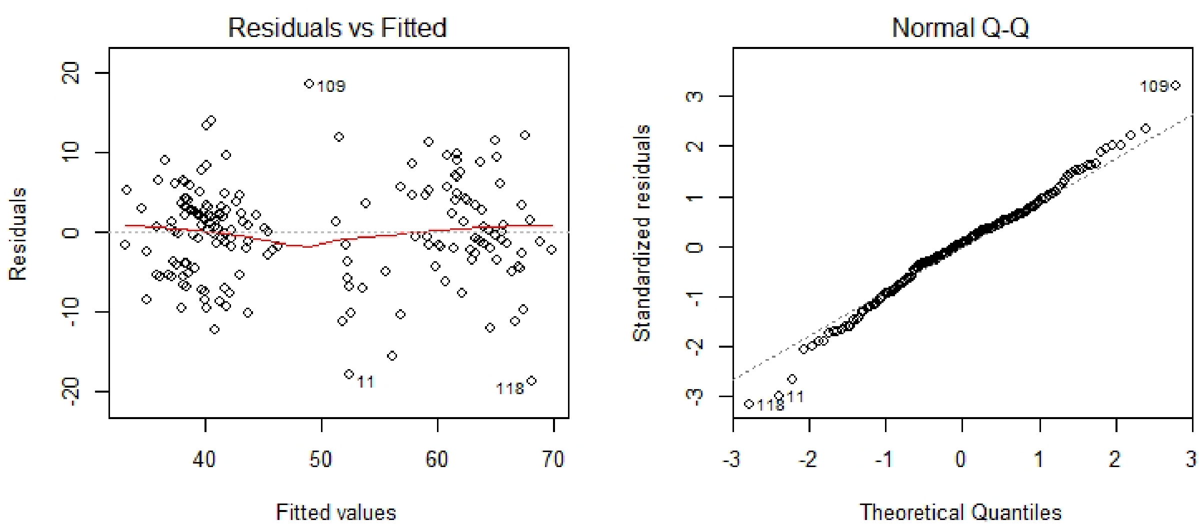


Figure 4-13 Residual and Q-Q Plots

4.3 Repeatability Analysis

The repeatability, also known as precision, of any measuring device is a primary concern of equipment users. It is desired that continuous friction measurements are not altered from repeating runs taken with the same equipment under unchanged conditions. If the system is repeatable, the measurement error can be mapped and compensated for. If the measurements differ greatly, one can argue that the findings derived from the measurements are inaccurate. However, continuous friction measurements may differ to a certain degree from multiple runs due to the potential vehicle wandering during driving and the measurement variations from various sensor components of the Grip Tester.

Friction measurements can be affected by several factors, such as pavement surface type, testing speed, water film thickness, tire characteristics, and ambient temperature et al (AASHTO, 2008). To investigate the repeatability of Grip Tester measurements, both concrete and asphalt pavement testing sites with various pavement preventive treatments were included, as described in Chapter 3. For each site, multiple repeating runs were performed by the OSU Grip Tester under the same operational conditions (0.25 mm of water film thickness and 40 mph of testing speed). Five testing sites were tested over the course of two field trips, three of which were tested in November 2018: Lakeview Road in Stillwater (flexible pavement with Micro-surfacing), SH-33 in Perkins (concrete pavement with longitudinal grooving), and US-77 North Bound in Norman (concrete pavement with five types of preventive treatments including shotblasting). The other two testing sites were tested in March 2019: I-40 in Oklahoma City (high friction surface with lead in and lead out asphalt pavements) and US-177 in Ponca City (ultra-thin bounded wearing course (UTBWC) asphalt pavements). The impact of temperature is neglected since the testing was generally completed within an hour. Example friction measurement of each site is displayed in Figure 4-14.

Various methodologies are available for repeatability testing. Traditionally, repeatability is often reported in terms of standard deviation, which measures the variation of measurements taken by a single device under the same conditions. Table 4-5 summarizes the average friction numbers and their standard deviations on each site. The average differences (the maximum minus minimum friction) and standard deviation of the five measurements are 0.039 and 0.051 respectively, indicating the Grip Tester can measure surface friction in a consistent manner.

Table 4-5 Evaluation of Repeatability: Mean and Standard Deviations (STD)

Site	Type	Run 1	Run 2	Run 3	Run 4	Run 5	Diff.	STD
Lakeview Rd	Micro-surface	0.468	0.470	0.479	0.500	0.474	0.032	0.0107
SH-33	Long. Groove	0.604	0.612	0.625	0.615	0.630	0.026	0.0004
US-77 NB	Shotblasting	0.628	0.586	0.602	0.565	/	0.063	0.0012
I-40	HFST	0.591	0.619	0.652	0.645	0.653	0.062	0.0125
US-177	UTBWC	0.595	0.596	0.592	0.604	0.602	0.012	0.0006
Average							0.039	0.0051

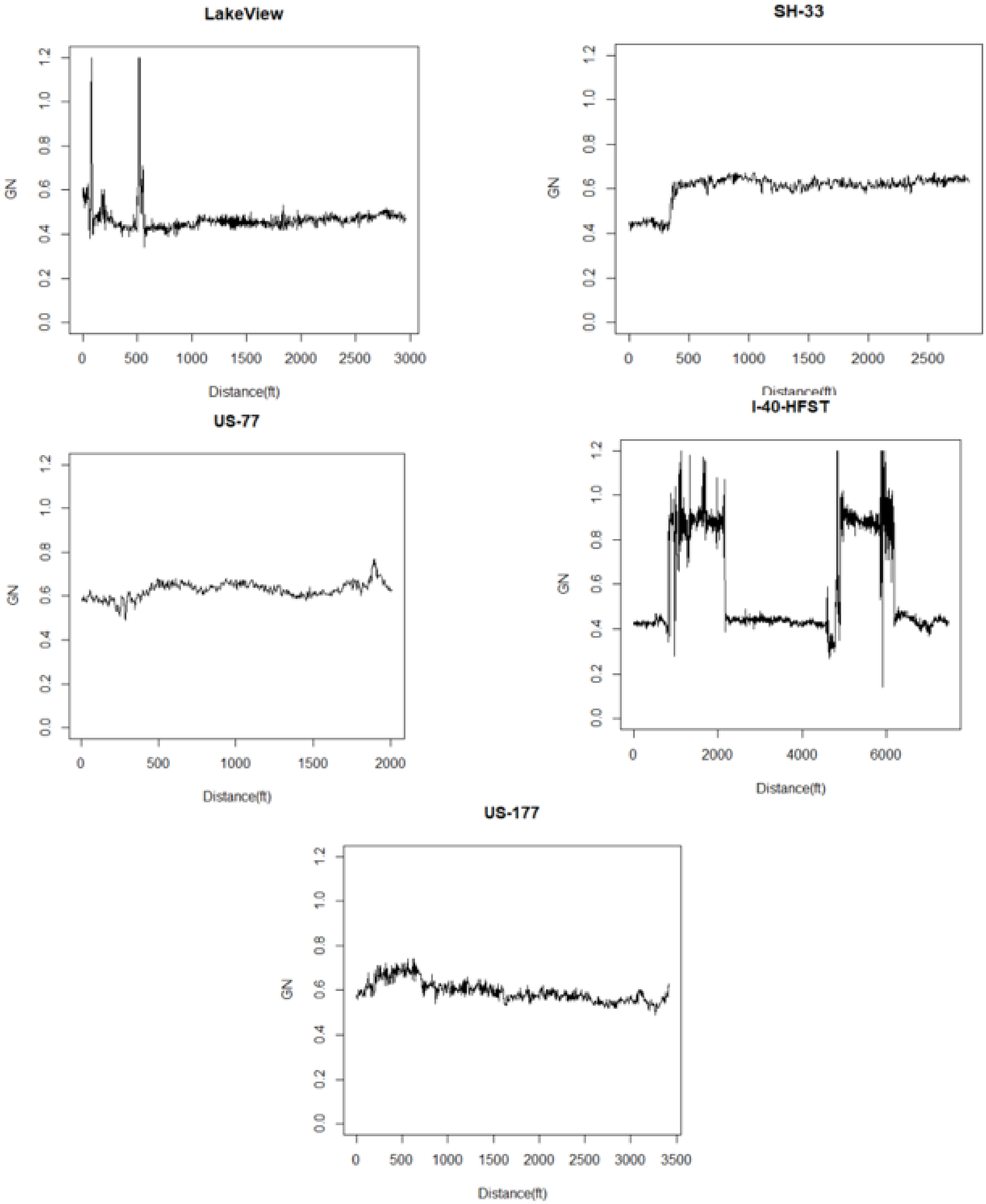
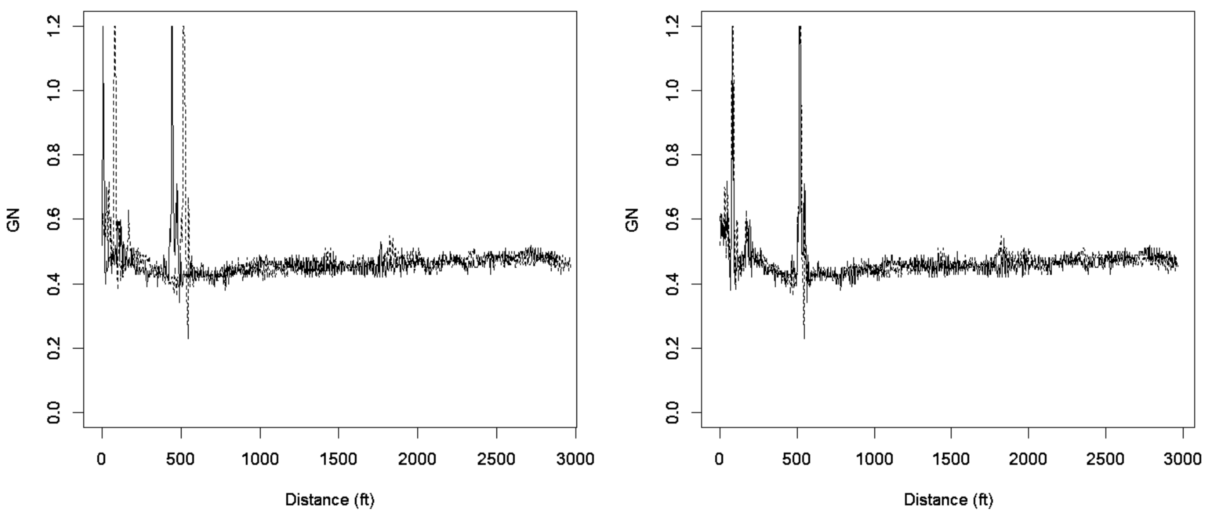


Figure 4-14 Grip Tester Friction Measurements

In addition, cross-correlation is a more rigorous statistical approach to identifying the similarity among repeating measurements, which has been applied successfully in many disciplines, including pavement engineering in the areas of pavement longitudinal profiles (Karamihas, 2004) and friction profiles (Najafi et al., 2017), for repeatability and accuracy testing. The output of this methodology yields a single value, the cross-correlation coefficient, whose range is -1.0 to 1.0, whereby 1.0 indicates a perfect positive correlation, and -1.0 indicates a perfect negative correlation.



(a) 0 Lag　　(b) 22 Lags

Figure 4-15 Cross-correlation Methodology

Cross-correlation may also be employed to determine the lag one waveform should be shifted to obtain the best match with another waveform for optimal synchronization. The two friction profiles measured on Lakeview Road in Stillwater are shown in Figure 4-15(a): the first run (black line) and the second run (dotted red line). The two profiles were shifted in waveform by various number of lags using the cross-correlation method so that the highest cross-correlation coefficient is achieved. The calculation process was performed by the R programming language. For this case study, the maximum cross-correlation coefficient is 0.833 after the second run (dotted red line) is shifted to the left by 22 lags, as exhibited in Figure 4-15(b).

Table 4-6 Maximum Cross-correlation Value for Evaluation Repeatability

Site	# of Run	Run 1	Run 2	Run 3	Run 4	Run 5
Lakeview Rd.	Run 1	1.000	0.833	0.592	0.573	0.714
Lakeview Rd.	Run 2	/	1.000	0.676	0.569	0.725
Lakeview Rd.	Run 3	/	/	1.000	0.582	0.794
Lakeview Rd.	Run 4	/	/	/	1.000	0.644
Lakeview Rd.	Run 5	/	/	/	/	1.000
SH-33	Run 1	1.000	0.916	0.912	0.939	0.925
SH-33	Run 2	/	1.000	0.863	0.878	0.881
SH-33	Run 3	/	/	1.000	0.902	0.878
SH-33	Run 4	/	/	/	1.000	0.887
SH-33	Run 5	/	/	/	/	1.000
US-77	Run 1	1.000	0.534	0.633	0.484	/
US-77	Run 2	/	1.000	0.782	0.735	/
US-77	Run 3	/	/	1.000	0.621	/
US-77	Run 4	/	/	/	1.000	/
I-40	Run 1	1.000	0.955	0.926	0.943	0.943
I-40	Run 2	/	1.000	0.954	0.959	0.950
I-40	Run 3	/	/	1.000	0.941	0.950
I-40	Run 4	/	/	/	1.000	0.950
I-40	Run 5	/	/	/	/	1.000
US-177	Run 1	1.000	0.817	0.828	0.662	0.838
US-177	Run 2	/	1.000	0.873	0.637	0.855

Site	# of Run	Run 1	Run 2	Run 3	Run 4	Run 5
US-177	Run 3	/	/	1.000	0.688	0.871
US-177	Run 4	/	/	/	1.000	0.647
US-177	Run 5	/	/	/	/	1.000

Cross-correlation analysis was subsequently applied to the possible combinations of the multiple runs, whose results for the five testing sites are shown in Table 4-6. The highest repeatability is obtained on the HFST site, while there is relatively weaker repeatability on the shotblasting site. The range of the friction on the shotblasting site is small and thus the friction profile is uniform in shape without distinct changes and profile features (Figure 4.13). As a result, the synchronization process based on cross-correlation is not as robust. For the HFST site, since there is a significant increase of friction from the non-HFST to HFST surface, the synchronization of the friction measurements is more accurate and easier to implement. Additionally, the wandering of the testing vehicle during measurements could reduce the correlation among different runs. Overall, the repeatability analysis results suggest that the Grip Tester based CFME measurements have sufficient repeatability.

4.4 Operational Characteristics of Grip Tester

Understanding the influences of various factors on CFME measurements could have many benefits for state agencies to effectively implement CFME for pavement friction management. The objective of this section is to assess the impacts of various operational factors on CFME friction measurements so that friction measured at different testing conditions can be adjustable to achieve universal comparability of tire/pavement friction measurements.

The experimental design in this study includes twenty-two pavement sections with ten different treatment types and nine testing conditions. The ambient temperature during the testing ranges from 56°F to 110°F.

- Ten types of preventive maintenance treatments: chip seal, ultra-thin bounded wearing course (UTBWC), asphalt resurface, warm mix asphalt, micro-surface, high friction surface treatment (HFST), concrete surface without grooving, concrete surface with longitudinal grooving, next generation concrete surface (NGCS), and shotblasting.
- Nine combinations of friction testing conditions: three water film thicknesses (WFT) (0.25, 0.50, 1.00mm) combined with three different testing speeds (40, 50, 60 mph for major arterials, while 30, 40, 50 mph for minor arterials).

Pavement friction data was collected by the Grip Tester, while the corresponding pavement macro-texture (in terms of MPD) and roughness (in terms of IRI) data were acquired by the AMES 8300 Survey Pro High Speed Profiler.

To access the impact of friction measurements due to the change of operational characteristics (testing speed, WFT, ambient temperature), the base scenario was selected as the standard testing conditions of Grip Tester, i.e. 50 mph, 0.25 mm (WFT) and 700F. Multivariate regression analysis was performed to quantify the change of friction measurements under various operational testing characteristics. Pavement surface texture (in terms of MPD) and roughness (in terms of IRI) levels were also included in the model development as the independent variables to characterize the testing site conditions. The regression model was developed following the backward stepwise analysis. This process starts with the inclusion of all independent variables in the model, and then gradually removes the least significant variable step by step until the highest model of fit is achieved. IRI was found insignificant to the friction measurement and thus was removed in the model. The statistical result is summarized in Table 4-7.

The adjusted R-squared of the model is 0.68, indicating that the model has satisfactory statistical performance. Ambient temperature, testing speed, and water film thickness show negative effects on the friction measurements. In other words, friction measurements tend to decrease with the increasing of these operational characteristics. The average MPD has a positive effect on friction measurements. Such findings are consistent with those in several previous studies (Hall et al., 2009; Vicroads, 2018). Utilizing the multivariate regression model in Table 4-7, state agencies can adjust/correct the friction collected at different testing conditions to the baseline condition and be comparable for further decision making.

Table 4-7 Multiple Regression Model

Coefficients	**Estimate**	**Std. Error**	**t value**	**Pr(>|t|)**	**Significance Level**
(Intercept)	-0.09151	1.28E-02	-7.16E+00	2.30E-11	***
Average MPD	2.67801	2.52E-01	1.06E+01	< 2e-16	***
ΔTemperature	-0.00089	2.36E-04	-3.79E+00	2.06E-04	***
ΔSpeed	-0.00381	4.08E-04	-9.35E+00	< 2e-16	***

Coefficients	Estimate	Std. Error	t value	Pr(>|t|)	Significance Level
ΔWFT	-0.12472	1.08E-02	-1.15E+01	< 2e-16	***

Note: (1) Signif. codes: 0 '***' 0.001 '**' 0.01 '*' 0.05 '.' 0.1 ' ' 1; (2) Multiple R-squared: 0.6861, Adjusted R-squared: 0.6788; (3) F-statistic: 93.45 on 4 and 171 DF, p-value: < 2.2e-16

CHAPTER 5 CRASH RATE PREDICTION MODELS USING CFME DATA

One of the goals in the FHWA Strategic Plan (2018) is improving roadway safety through a data-driven approach to reducing transportation-related fatalities and serious injuries across the transportation system. In this chapter, a framework is proposed to integrate the roadway characteristics data collected by the Grip Tester (for pavement friction) and 1 mm 3D laser imaging technology (for roadway surface conditions) into the development of crash rate prediction model. A case study is subsequently provided to implement the framework utilizing various sources of data sets.

5.1 Proposed Framework

The AASHTO Highway Safety Manual (HSM, 2010) provides an approach that utilizes regression equations, the Safety Performance Functions (SPFs), to predict the crash frequency for a specific site type. Factors contributing to roadway crashes are generally classified into three categories (HSM, 2010):

- Human behavior,
- Vehicle performance,
- Roadway condition and characteristics.

At the era of autonomous driving and connected vehicles, the role of roadway on crashes is becoming even more important. Factors in the roadway category include surface conditions (skid resistance, IRI, texture, etc.) and roadway geometry (curves, grade, shoulders, etc.). Miller and Zoloshnja (2009) found that inadequate roadway condition was a contributing factor, 52.7 percent of the nearly 42,000 American deaths from motor vehicle crashes each year and 38 percent of the non-fatal injuries, resulting in more than $217 billion of economy loss each year. Many studies have consistently shown a link between crashes and roadway characteristics. The National Transportation Safety Board and FHWA concluded that about 70 percent of wet pavement crashes could be prevented or minimized by improved pavement surface friction (FHWA, 2016). Wallman and Astrom (2001) developed relationship between pavement friction and roadway crash rate, revealing that higher friction can significantly reduce the crash rate (Table 5-1).

Table 5-1 Friction Coefficient and Crash Rate (Wallman and Astrom, 2001)

Frictional Coefficient	Crash Rate (injuries per million vehicle km)
<0.15	0.80
0.15-0.24	0.55
0.25-0.34	0.25
0.35-0.44	0.20

Although roadway condition is widely recognized as one important category of contributing factors to roadway crash, it is not yet fully considered in the current SPFs. The SPFs for highway segments in the AASHTO HSM are functions of the annual average daily traffic (AADT) and the segment length, which are both crash exposure indicators (HSM, 2010). Highway agencies are encouraged to develop state-specific SPFs for different roadway facilities and crash types (Merritt, et al., 2015). For example, in Virginia SPF was developed to include skid resistance and the radius of curvature for interstate and primary highway systems (de León Izeppi, et al., 2016).

Jurisdiction-specific SPFs are likely to enhance the reliability of safety predictive method (HSM, 2010; Lu et al., 2012). Built on extensive literature review results (HSM, 2010; Srinivasan, Carter, and Karin Bauer, 2013; de León Izeppi, et al., 2016; Merritt, et al., 2015), the following step-by-step framework is proposed for the development of enhanced SPFs with roadway characteristics integrated.

Step 1 - Identify facility type

Depending on whether the SPF is being estimated for project-level analysis or network screening, state agencies can decide which facility types they are most interested in. The most frequent analysis types include highway segments and intersections.

Step 2 – Determine the data needs and compile necessary data

The selection of explanatory (independent) variables is an important step in the development of SPFs. The list of explanatory variables may depend on the proposed application of the SPF. Such data sets may include but not limited to: section length, traffic volume, crash history, surface characteristics (pavement friction, surface texture, IRI, rutting, cracking), roadway geometry (roadway curvature, longitudinal grade, number of lanes, presence of shoulder and/or median).

Step 3 - Determine functional form

Given the discrete non-negative data nature of traffic crashes, count data modeling techniques are typically used to predict crash frequencies. Common techniques include Poisson Generalized Linear Models (GLMs) and Negative Binomial GLMs. The SPFs in Part C of the AASHTO HSM are negative binomial regression models with a log-linear relationship between crash frequency and site characteristics. Agencies can determine their state-specific functional forms based on the characteristics of available data.

Step 4 - Develop the SPF

A common approach for identifying significant variables is a stepwise regression approach, which could be based on either a forward selection or a backward elimination procedure. Examples of model comparison criteria include t-statistic, chi-square statistic, Akaike's information criterion (AIC), and Bayesian Information Criterion (BIC). The model development can be readily implemented in many statistical software tools as long as the model is a generalized linear model (GLM), such as commercially available software packages SAS, STATA, and GENSTAT. Other software such as R, an open source programming language, and the widely available Microsoft Excel spreadsheet can be used as well.

Step 5 - Conduct model diagnostics

Many statistics can be used for model diagnostics, such as checking the sign of the parameters' coefficients, examining residuals via residual plots and cumulative residual plots (i.e., CURE plots), and identifying potential outliers using Cook's D or other methods, and examining the goodness-of-fit measures.

Step 6 - Re-estimate the SPF model

Based on the results from Step 5, the SPF may have to be re-estimated using a different statistical model or functional form. The SPF may also need to be re-estimated after removing outliers that were identified in the diagnostics step.

Step 7 – Crash estimation

The statistically reliability of average crash estimation can be improved by combining observed crash frequency and estimates of the average crash frequency, using the Empirical Bayes predictive method (EB Method) to compensate for the potential bias resulting from regression-to-the-mean (RTM). The RTM is the tendency of crash fluctuations where a comparatively high crash frequency is followed by a low crash frequency (Hauer, 1996). Failure to account for the RTM bias may result in an over- or under- estimation of long-term crash frequency. The Empirical Bayes (EB) method is commonly known to address two problems of safety estimation: it increases the precision of estimates beyond what is possible when one is limited to the use of two-three years of history accidents, and it corrects for the

RTM bias. The EB method uses a weighted adjustment factor, w, which is a function of the SPF's over-dispersion parameter, k, in the negative binomial distribution:

$$w = 1/(1 + k\times\sum_{\text{all study years}} N_{\text{predicted}}) \qquad \text{(Eq. 5-1)}$$

Therefore, the expected average crash frequency for the analyzed period is:

$$N_{\text{expected}}=w\times N_{\text{predicted}}+(1-w)\times N_{\text{observed}} \qquad \text{(Eq. 5-2)}$$

5.2 Case Study

A case study is provided in this section to demonstrate the development of enhanced SPF using CFME friction and 1 mm pavement condition data sets collected from this project and two additional projects conducted by the research team: the ODOT project titled "*Development of Aggregate Characteristics-Based Preventive Maintenance Treatments for Optimized Skid Resistance of Pavements*" (Li et al., 2018), and the FHWA project titled "*3D Laser Imaging based Real Time Pavement Surface Evaluation for High Friction Surfacing Treatments (HFST)*". The team has used the CFME Grip Tester as the data collection instrument, and conducted multiple years of field monitoring for dozens of testing sites which have different preventive treatment types and installation ages, various climate and traffic conditions, and roadway geometry characteristics. It should be noted that the data herein for this case study is only used for demonstration purpose due to the limited number of testing sites and the duration of the monitoring cycle (less than 2 years).

The crash data in Oklahoma for this case study was obtained from the SAFE-T database managed by ODOT, while additional crash data were acquired directly from state agencies through email communications. The traffic volume in terms of AADT was obtained manually from the AADT Maps managed by each DOT. The friction data was collected using the Grip Tester owned by the OSU research team (Figure 3-2 (a)). The pavement surface texture and roadway geometry data were obtained at highway speed with full-lane coverage using the 1 mm laser imaging data vehicle equipped with the AMES high speed profiler (Figure 3-2 (c) and (d)). Mean profile depth (MPD), International roughness index (IRI), number of lanes, and presence or absence of shoulder and median.

After data acquisition, the enhanced SPF model can be developed following the framework proposed previously in this chapter.

Step 1 - Identify Facility Type.

The data herein was collected on highway segments. Each testing site included in this project defined one highway segment.

Step 2 - Determine the Data Needs and Compile Necessary Data.

The acquired data was attached to each highway segment, including the AADT data, surface characteristics (surface friction, MPD, IRI), and roadway geometry (# of lanes, presence or absence of shoulder and/or median). The crash data in five years, obtained from the SAFE-T database, divided by the length of the segment length, in terms of crash rate in five years per mile, was used as the dependent variable for the SPF model.

Step 3 - Determine Functional Form.

A negative binomial regression model with a log-linear relationship between crash rate and traffic volume was adopted in this study, the same functional form as that for the HSM SPF model shown in Equation 5.3.

Step 4 - Develop the SPF.

The statistical analysis and model development were performed in R programming. Backward regression analysis was used to select the most significant variables for crash rate prediction model. This process started with the full model including all the identified variables (in Step 2) and removed the least significant variable at each step until the remaining variables were statistically significant. The final model results are shown in Table 5-2. It can be seen that the predicted number of crashes per mile in 5 years is most significant to AADT, followed by pavement friction at 99.999% significance level.

$$N_{SPF\ per\ mile\ per\ 5\ yrs} = a \times AADT \times \exp(\alpha + \sum \beta_i \times x_i) \qquad \text{(Eq. 5.3)}$$

Table 5-2 Enhanced SPF Model Results for the Case Study

Coefficients	Estimate	Std. Error	z value	Pr(>|z|)	Significance Level
(Intercept)	0.5095	1.3305	0.383	0.7018	
AADT	0.4953	0.1175	4.214	2.51E-05	***
Friction	-2.3916	0.883	-2.708	0.00676	**

Note: (1) Signif. codes: 0 '***' 0.001 '**' 0.01 '*' 0.05 '.' 0.1 ' ' 1; (2) Dispersion parameter for Negative Binomial: 0.485; (3) AIC: 660.98

Step 5 - Conduct Model Diagnostics.

As shown in Table 5-2, the crash per mile in five years is expected to increase with the increase of traffic volume or the decrease of surface friction coefficient, which is consistent with engineering judgement and the findings in previous studies. The model has the dispersion parameter of 0.485, indicating that the crash data is over-dispersed and thus the negative binomial model is suitable for this case study. For a real-world engineering practice, additional diagnostics can be conducted based on the various needs from the agency.

Step 6 - Re-estimate the SPF.

The SPF may be re-estimate based on the feedback in Step 5, however, this step is eliminated in this case study. Furthermore, with the enhanced SPF, the statistical reliability of the crash estimation can be further improved by the Empirical Bayes method to compensate for the potential bias resulting from RTM. Per Eq. 5.1, the weighted adjustment factor, w, can be integrated to estimate the expected average crash rate in Eq. 5.2.

5.3 Summary

Besides traffic volume and segment length that are considered in the HSM SPF model, the statistical results of the case study have shown that pavement friction is also critical to crash rate prediction.

The case study provided in this Chapter is only for demonstrate purpose, which has two main limitations. First, in order to increase the size of the samples for model development, the testing sites were selected from different States that may not be appropriate for developing state-specific SPF. Second, the data size for the model development used in this Chapter is limited in terms of the number of sites and the number of influencing variables. In total seventy-six samples were included in the data set and seven independent variables for the SPF model development. Besides, all the segments considered in this study were chosen on tangent sections on level terrain. Longitudinal grade and curvature of highway segments, which could have significant influence on highway safety, were not included in the model development.

CHAPTER 6 PRILIMINARY APPLICATIONS OF CFME DATA

In this Chapter, several potential implementations of CFME data sets were illustrated for pavement friction management and highway safety applications. With detailed CFME data, pavement network could be better segmented into homogenous sections for road maintenance scheduling and management. Several algorithms were tested for this purpose. In addition, in order to ease the using of CFME data, a CFME data analysis software interface was developed, which is able to upload and visualize CFME measurements, and perform statistical and comparison analyses for data reporting. Finally, how CFME data and the findings from this study can be used to assist the pavement friction management was discussed.

6.1 Dynamic Segmentation of CFME Data

6.1.1 Introduction

Pavement network should be divided into roadway segments at different levels for the convenience of management and maintenance. Traditional Pavement Management System (PMS) define sections with a fixed length (such as 0.1 mile) or variable length. Fixed-length segments must be short enough to accurately represent uniform pavement performance, which could result in data redundancy if the condition does not change much among multiple segments. Variable length segments address this issue but can be cumbersome and requires the breakage of an integrated road section. One typical variable method is the Cumulative Difference Approach (CDA) recommended by AASHTO (1993). Though powerful and simple for continuous and constant response values, CDA was pointed out to be highly sensitive to what the exact part of the data series is used for the analysis (Thomas, 2003). The Oklahoma Department of Transportation (ODOT) defines pavement subsections based on more than a dozen of criteria primarily related to traffic characteristics, roadway geometry, construction activities, and political jurisdiction.

Recent advancement of testing technologies has made the collection of detailed pavement data at a much smaller interval or resolution possible. With large amount of data sets, dynamic segmentation (DS) was believed to be a better solution to this problem. Researchers have applied different segmentation methods, such as the binary segmentation (Scott and Knott, 1974), the neighborhood segmentation algorithm (Auger and Lawrence, 1989), and Pruned Exact Linear Time (PELT) method (Li et al., 2015) for dynamic segmentation under different circumstances. Kenneth [6] developed a framework for the evaluation of dynamic segmentation based on a mile-point linear data model.

The objective of this implementation is to investigate and evaluate different dynamic segmentation methods based on pavement performance data (surface friction data herein). Three dynamic segmentation methods: binary, neighborhood and PELT methods were applied. The results were compared with the ODOT subsections and their efficiency was evaluated according to the computation time and the robustness per the residual check for individual segment and the overall roadway.

The CFME data used in this case study was collected on northbound of I-35. The details on CFME data collection and the data pre-processing can be referred to Section 4.2.

6.1.2 Change Point Detection Methods

The problem of segmentation can be led back to the detection of change points that divide measures into homogenous segments along the measured data series (Cafiso and Graziano, 2012). A change point is the point where the statistical properties of a sequence change. The property can be mean, variance, slope, or their combination. For example, suppose a series of data $y_{1:n}=(y_1,\ldots,y_n)$ has change points in order at positions $\tau_{1:m}=(\tau_1,\ldots,\tau_m)$, where $\tau_0=0$ and $\tau_{m+1}=n$. As a result, the series of data were split into *m*+1 segment, and each segment $y_{\tau k:\tau k+1}$, $0 \le k \le m$, has different statistical properties as compared with their adjacent segments. Arguably, the most common approach to identify the locations of change points is to minimize:

$$\sum_{\boldsymbol{i=1}}^{\boldsymbol{m+1}}\left[\boldsymbol{C}\left(\boldsymbol{y}_{(\boldsymbol{\tau_{i-1}}+\mathbf{1}):\boldsymbol{\tau_{i-1}}}\right)\right]+\boldsymbol{\beta f(m)} \qquad \text{(Eq. 6-1)}$$

Where C is a cost function for a segment and βf(m) is a penalty to guard against over-fitting.

In this study, three multiple change point algorithms were applied to minimize such objective: binary segmentation (Edwards and Cavalli-Sforza, 1965), segment neighborhood algorithm (Auger and Lawrence, 1989), and the more recent PELT method (Killick et al., 2012). Binary segmentation repeats the procedure of applying a single change point segmentation to the existed data segments in previous round and splitting in current round if a change point is identified. The data series are split further if new change points are identified in either of the new data sets. As any change point locations are conditional on change points identified previously, binary segmentation is considered an approximate algorithm but computationally fast as it only considers a subset of the 2^{n-1} possible solutions (Killick and Eckley, 2014). The computational complexity of the algorithm is O(nlogn) but this speed can come at the expense of accuracy of the resulting change points (Killick et al., 2012).

The segment neighborhood algorithm was proposed by Auger and Lawrence (1989) and further explored in Bai and Perron (1998). The algorithm minimizes the expression given by Eq. 6-1 exactly using a dynamic programming technique to obtain the optimal segmentation for m+1 change points reusing the information that was calculated for m change points. This reduces the computational complexity from O(2n) for a naive search to $O(Qn^2)$ where Q is the maximum number of change points to identify. Whilst this algorithm is exact, the computational complexity is considerably higher than that of binary segmentation.

The binary segmentation and segment neighborhood algorithms would appear to indicate a trade-off between speed and accuracy. The PELT algorithm proposed by Killick is similar to the segment neighborhood algorithm in which it provides an exact segmentation (Killick et al., 2012). However, due to the construction of the PELT algorithm, it can be shown to be more computationally efficient, due to its use of dynamic programming and pruning, which can result in an O(n) search algorithm. Indeed, the main assumption that controls the computational time is that the number of change points increases linearly as the size of data set.

6.1.3 Segmentation Results

<u>Influence of Segmentation Criteria</u>

Segmentation results could be influenced by different criteria considered in Equation 6-1. As shown in Fig. 4-8, the 29.6-mile section consists of three pavement surface types: Joint Concrete pavement (JCP), Asphalt Concrete (AC) and Continuous Reinforced Concrete Pavement (CRCP), and ten bridge decks (BR) (shown in Figure 6-1 with short vertical lines). Currently, the entire testing section was divided into 28 ODOT subsections. The breaking rules for the roadway sub-control sections include any traffic, functional, geometry, geographical or jurisdiction changes of the roadway, such as changing of highway function class, junction of highways, terrain area type, surface width or type, shoulder width or type, political regions (municipal limits, urban area, county or state line), or maintenance responsibility (maintenance division) etc. For example, as shown in Figure 6-1, the ODOT subsections always break at junctions where pavement surface type changes. However, the difference between the ODOT subsections and the dynamic segmentation results, whether based on mean, variance or both, suggests that the specific subsection sectioning practice used in this case study does not take pavement friction herein into consideration. It is acknowledged that ODOT makes dynamic segmentation based on pavement performance (such as roughness, rutting, and cracking etc.) in-house if demanded by individual projects.

The dynamic segmentation results based on mean, variance or both, could also detect the change of different surface types. The segmentation results based on the change of variance seems to be insensitive to pavement surface change within a short distance, since variance reflects the level of pavement performance uniformity. In this case study, the segmentation based on variance only generate the least number (16 in total) of homogeneous sections, followed by the ODOT subsections (28), the mean and variance method (46), and lastly the mean only method (48 sections).

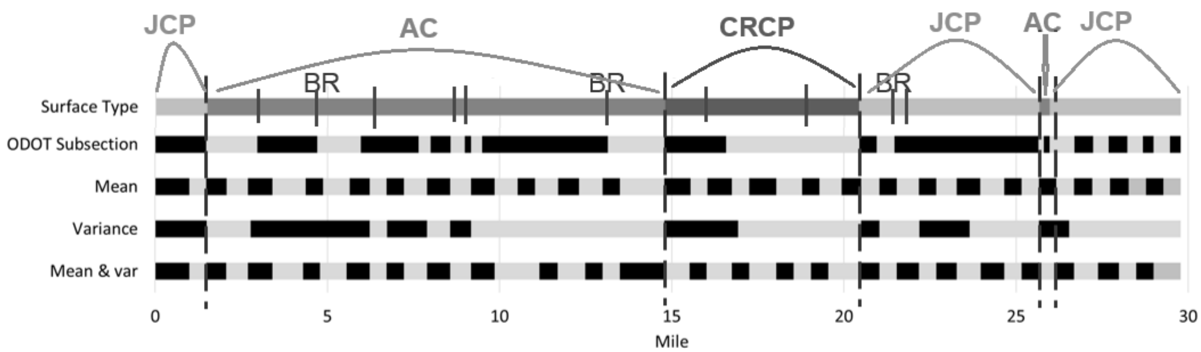


Figure 6-1 Comparisons of Results based on Different Segmentation Criterion

<u>Influence of Segmentation Methods</u>

The challenge of dynamic segmentation is the tradeoff between the convenience of engineering practice (which prefer a reasonable number of segments) and the deviations of characteristics within each segment (which could lead to a large number of segments). In other words, a proper segmentation method should balance the number of segments and the uniformity in each segment. For the binary segmentation method, the number of segments can be directly determined since its procedure splits new segments based on the results from previous iteration. When the total number of segments or change points are reached, the procedure stops, and the segmentation result is obtained.

As for the PELT method, the minimum length of the segment controls the number of change points and the penalty value defined in Eq. 6-1. The minimum length is set to 0.01-mile, the smallest decision-making unit used in ODOT. The relationship between the penalty value and the number of change points are presented in Figure 6-2. As the penalty value increases, the number of change points decreases rapidly in the beginning and then stabilizes. The point of maximum curvature change is used as the optimal balanced number of segments. In this case, the balanced number of change points is 10 and the penalty value of approximately 60.

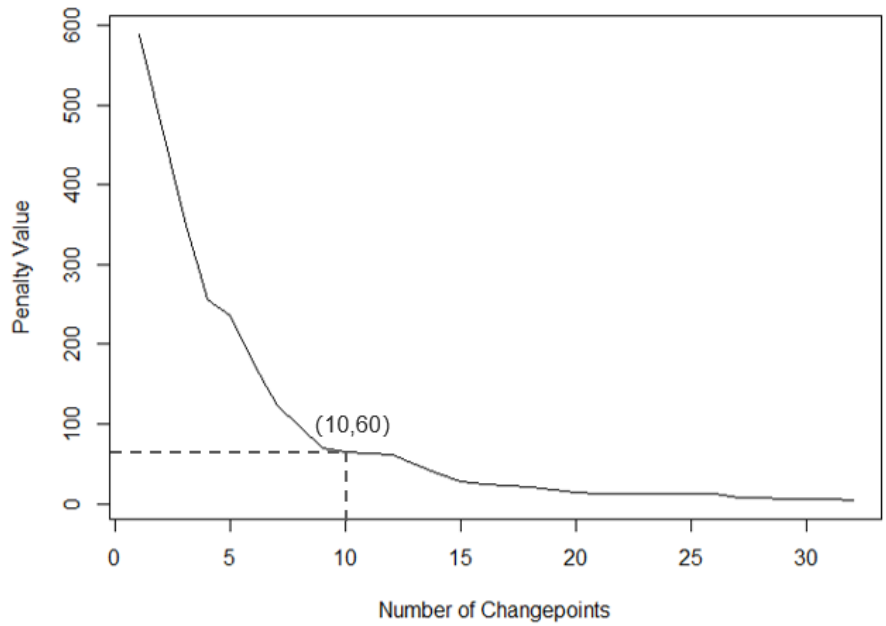


Figure 6-2 Number of Change Points vs Penalty Value in the PELT Method

With the optimal number of segments, the PELT and binary segmentation results were presented in Figure 6-3 with different outcomes. The junctions between two different pavement surfaces were all detected by the PELT method, which could assist in the treatment selection for maintenance and management. On the contrary, the binary method failed to detect the pavement surface change in some cases, such as the first junction transiting from JCP to AC. This comparison confirmed the limitation of the binary method as a proximate method.

In order to compare the dynamic segmentation with the ODOT subsection results, the number of homogenous segments is set to 28 (which equals to the number of ODOT subsections), and the PELT and binary methods results were illustrated in Figure 6-3. Within each homogenous section of the 10-segment results, multiple sections were further divided in the 28-segment results. The beginning and ending locations of these segments match well between the two methods. However, even with the same total number of segments, the PELT and binary methods gain different segmentation results, as compared to the ODOT subsection method. Therefore, the uniformity of each segment needs to be evaluated for comparisons of these methods.

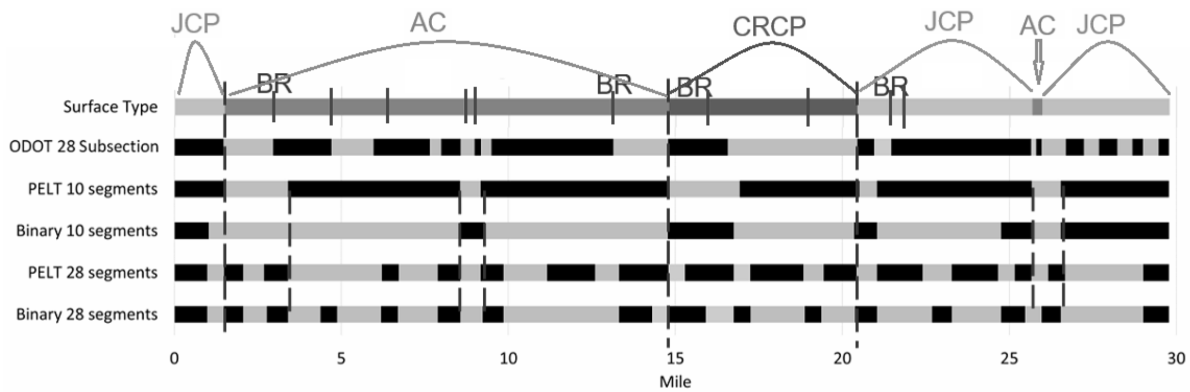


Figure 6-3 PELT and Binary Segmentation Results: 10 vs. 28 Segments

6.1.4 Results Evaluation

An ideal segmentation should consist of the minimum number of segments, and the data within each segment should be normally distributed. To evaluate the uniformity of the segmentation results, residual plots were provided before and after the segmentation. The frequency distribution of the raw friction data was presented in Figure 6-4. The raw GN data presents an obvious bi-modal distribution. The Shapiro-Wilk normality test was carried out with the W value of 0.91086, p-value of 1.31e-09. A higher W value closer to one supports the sample being from a normally distributed population only if the p-value is higher than 0.05. However, in this case, the p-value is less than 0.05, indicating there is evidence that the raw data was not normally distributed and further segmentation is needed (Liang et al., 2009).

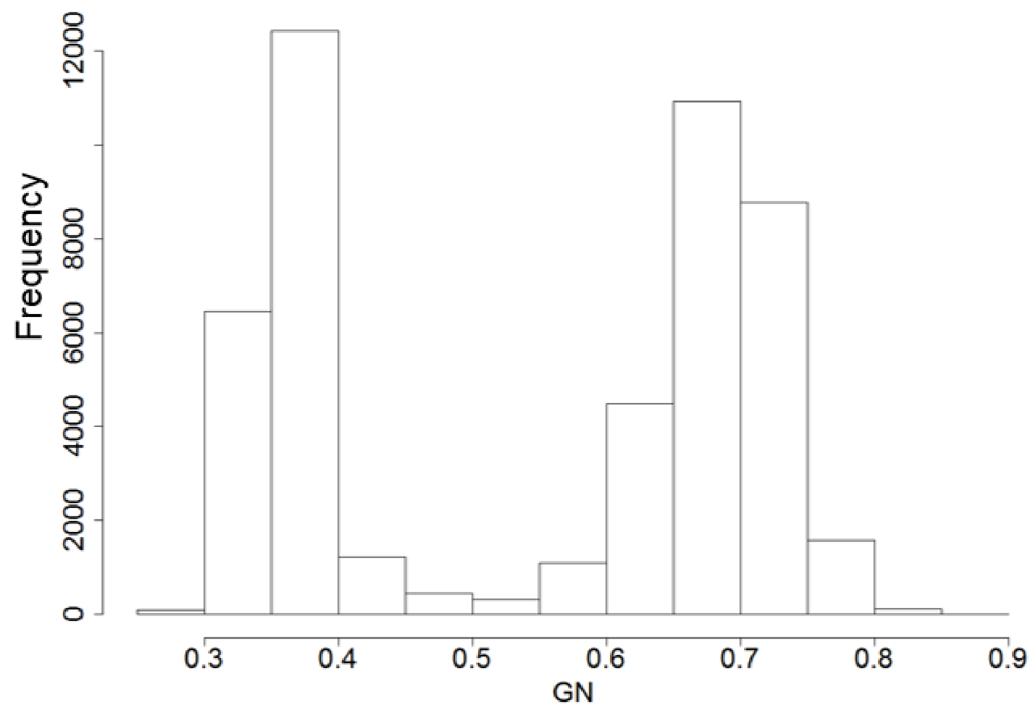


Figure 6-4 Frequency Distribution of Raw Friction Data

After the segmentation process, each segmented section was assessed using the Q-Q plot to evaluate the normality. A Q-Q plot is a scatterplot created by plotting two sets of quantiles against one another. If both sets of quantiles came from the same distribution, the points should form a straight line. A good segmentation result should have a large percent of segments with “good” normal Q-Q plots as possible. The composition of “good” normal Q-Q plot segment for the ODOT subsections, the PELT and binary segmentation results were presented in Table 6-1. Compared to the ODOT subsection results, the Binary and PELT method with 28 sections have higher percentage of “good” segments, 57.14% and 67.90% respectively as compared to the ODOT method 28.60%. In addition, as the number of segments increases from 10 to 28, the percentage of “good” segments increased for both the binary and PELT methods, and the PELT being the best performer.

Table 6-1 Comparisons of Percentage of “Good” Segments

Segmentation	**ODOT 28 Subsection**	**Binary 10 segments**	**Binary 28 segments**	**PELT 10 segments**	**PELT 28 segments**
Percentage	28.60%	30.00%	57.14%	40.00%	67.90%

An alternative comparison of the three segmentation results is to check the total residuals of the model fit, illustrated by the normal Q-Q plots shown in Figure 6-5. The closer the points are to the approximation line; the smaller the total residuals were gained. The PELT method has a smaller residual than the Binary method and the ODOT subsection method has the largest residual, indicating that the PELT segmentation method generates the most uniform segments, which is consistent with its higher percentage of “good” segments shown in Table 6-1.

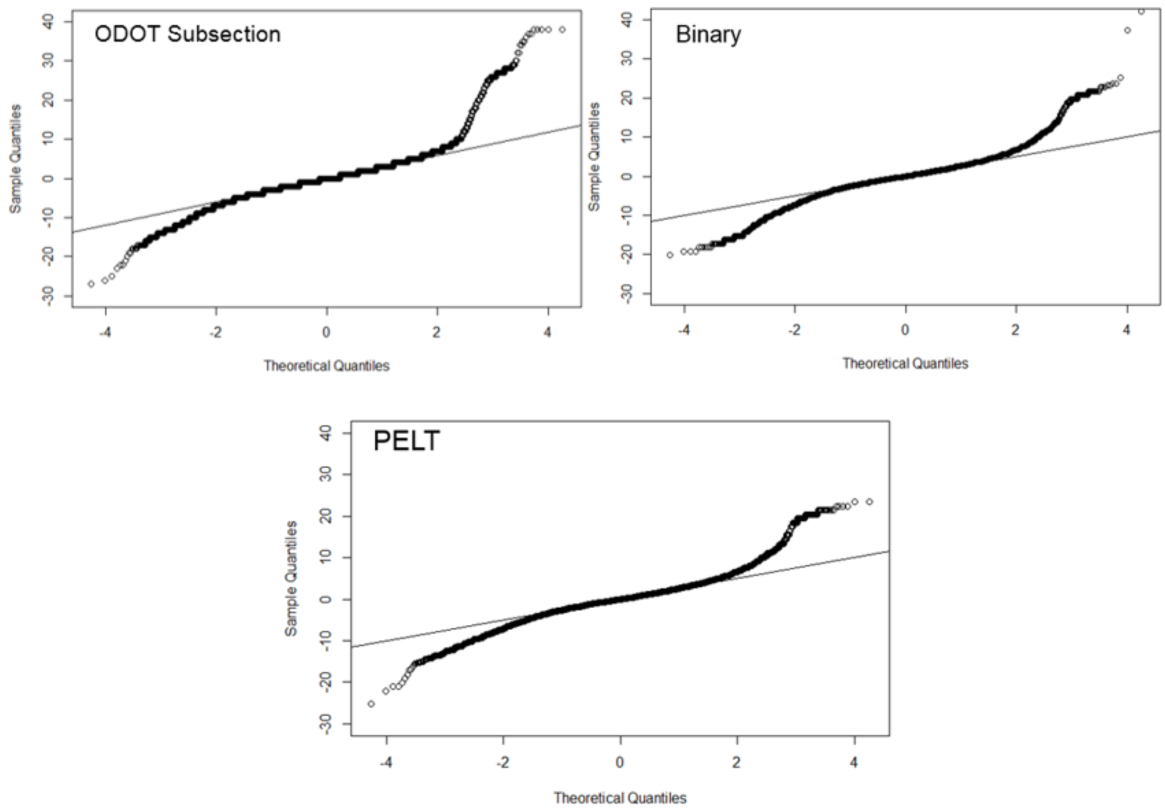


Figure 6-5 Q-Q Plot of Overall Residual Check: 28 Segments.

6.2 CFME Software Interface

It became evident that, although CFME friction profiles provide more information than what presently is being obtained by most locked wheel testers, its processing becomes sophisticated. It is thus important to develop a software interface to allow the users to view and perform basic analysis on CFME pavement friction profiles. The developed software interface has three major capabilities.

6.2.1 *Data Import and Visualization*

Firstly, the software interface is able to import CFME measurements and allows users to conduct direct comparisons of CFME friction measurements from multiple testing runs under different operating conditions (at various testing speeds and water film depths) in the same chart.

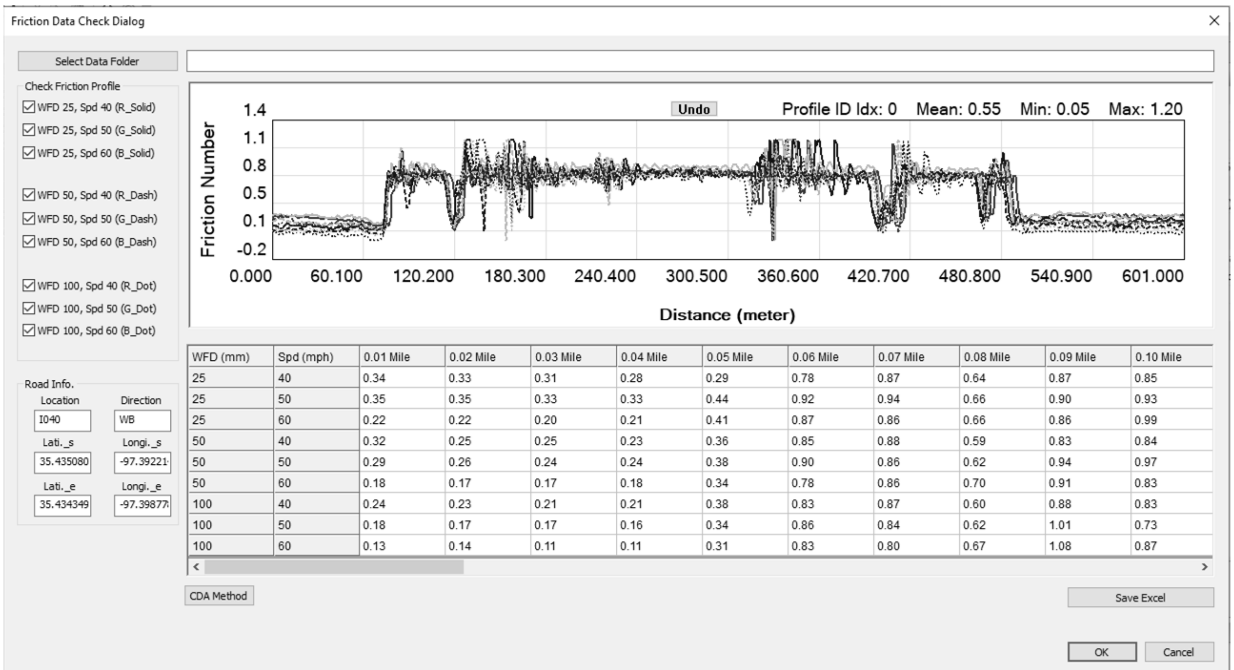


Figure 6-6 CFME Data Import and Visualization

6.2.2 *Profile Synchronization and Repeatability Analysis*

In order to make direct comparisons between different runs of the same road segment, the software allows friction profiles to be shifted distance wise for synchronization and making friction profile comparisons for repeatability. Basic statistics, mean and standard deviation, are provided. In addition, cross-correlation is employed to determine the lag one waveform should be shifted to obtain the best match with another waveform for optimal synchronization among multiple testing runs. It is also applied for repeatability and accuracy testing. The cross-correlation coefficient ranges from -1.0 to 1.0, whereby 1.0 indicates a perfect positive correlation, and -1.0 indicates a perfect negative correlation, as shown in Figure 6-7. For the example in the figure, the results suggest that the Grip Tester based CFME measurements have sufficient repeatability.

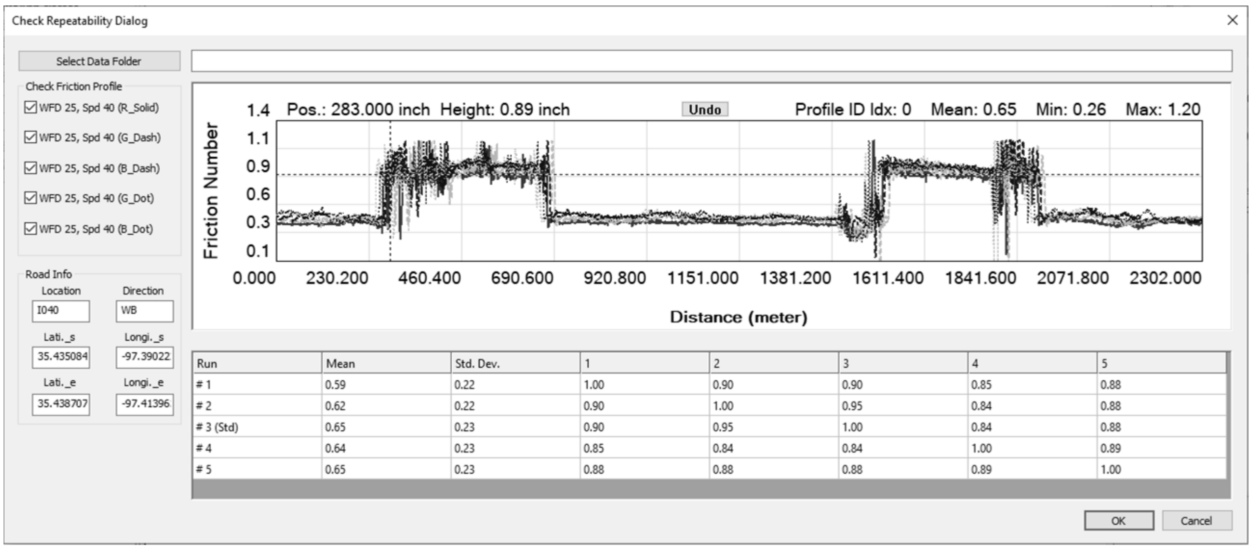


Figure 6-7 CFME Profile Repeatability Analysis

6.2.3 Homogeneous Segments

Lastly, the software is able to create uniform segmentations with statistically similar CFME friction properties. Segmenting pavement network into homogenous sections is important for decision makers to properly evaluate road maintenance scheduling and management work required as evidenced by the individual condition of each segment. In the software, two types of segmentation approaches are included: fixed-length segments and dynamic segmentation. Although the Pruned

Exact Linear Time (PELT) method has been demonstrated with slightly better performance in identifying homogenous segments, this method is excluded from the software for practical applications due to its complexity of computation requirements.

The fixed-length static method breaks highway routes into every 0.01 miles, which is the default reporting interval for the ODOT pavement management systems (PMS). The results can be directly imported into Excel spreadsheet for further analysis (as shown in Figure 6-6).

In addition, the cumulative difference approach (CDA), advocated by the AASHTO Guide (AASHTO, 1993) which compares the sequence of actual cumulative sums in a measurement series with the sums resulted from the adding averages, is also implemented in the software for identification of homogenous segments. In the CDA, the term Z_x is defined as the difference in the cumulative area values at a given distance (x) between the actual response and the average response (AASHTO, 1993). Mathematically,

$$Z_x = \sum_{i=1}^{n} a_i - \left(\frac{\sum_{i=1}^{n} x_i}{L_p} \right) \sum_{i=1}^{n_t} a_i \quad \text{(Eq. 6-2)}$$

$$a_i = \left(\frac{r_{i-1} + r_i}{2} \right) x_i \quad \text{(Eq. 6-3)}$$

where, x_i is the distance along the road section up to i^{th} response measurement; n is the n^{th} pavement response measurement; n_t is the total number of pavement response measurements taken in the project; r_i is the pavement response value of the i^{th} measurement; L_p is the total project length.

The Z_x values are plotted versus distance along the road length and the points where the slope of the Z_x curve changes its algebraic sign are taken as the border line between the consecutive homogeneous sections (AASHTO, 1993). These points have been described as change-points. A sample plot of the pavement response values as read from in the AASHTO's Guide (1993) and the resulting Z_x values, are plotted in Figure 6-8. The change points, thus identified, are shown by vertical lines intersecting the Z_x curve.

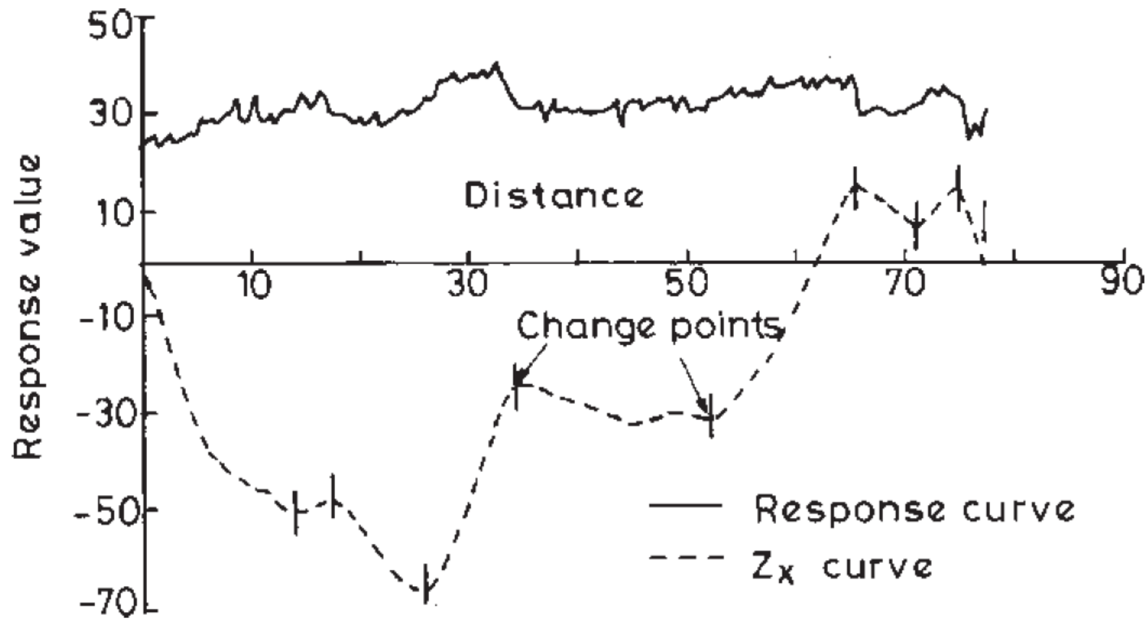


Figure 6-8 Homogeneous Section Using CDA (AASHTO, 1993)

An example screenshot of the CDA approach is provided in Figure 6-9. It successfully identifies the sections with different friction levels. CDA, though a quick and straight-forward method for identification of homogeneous sections, has certain limitations. CDA does not have any control over the number of homogeneous sections a designer intends to choose. There may not be any restriction on maximum length of a homogenous section, but from practical considerations, there must be a provision for specifying minimum length of a homogeneous section. Because, frequent changes in specification along the road stretch may cause inconvenience from the construction point of view. Choice of minimum section length cannot be incorporated in CDA.

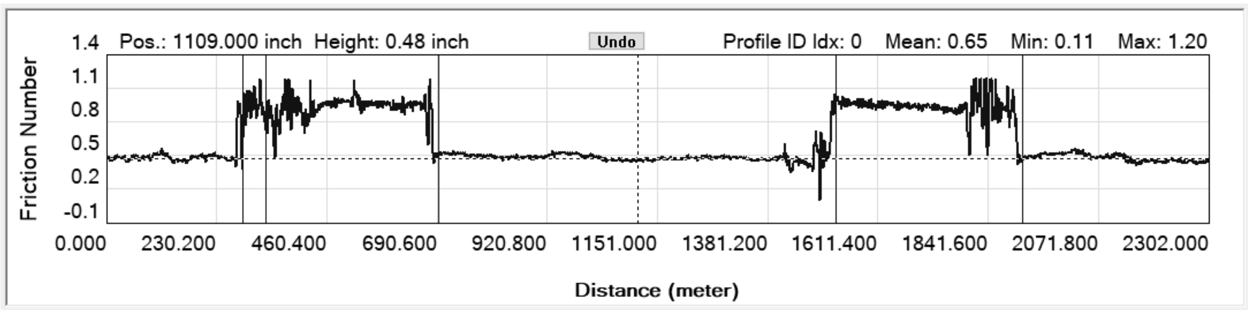


Figure 6-9 CDA based CFME Homogeneous Sections

6.3 CFME for Pavement Friction Management (PFM)

6.3.1 The AASHTO Guideline

The Technical Advisory 5040.38 Pavement Friction Management (FHWA, 2010) provides guidance to highway agencies towards developing or improving pavement friction management programs (PFMPs) to ensure pavement surfaces are designed, constructed, and maintained to provide adequate and durable friction properties that reduce friction-related crashes in a cost-effective manner (FHWA, 2017). The 2008 AASHTO Guide for Pavement Friction defines PFM as a systematic approach to measuring and monitoring the friction qualities and wet crash rates of roadways, identifying those pavement surfaces and roadway situations that are or will immediately be in need of remedial treatment, and planning and budgeting for treatments and reconstruction work that will ensure appropriate friction characteristics (AASHTO 2008). The AASHTO Guide provides an overall approach for managing pavement friction and a rigorous process for implementation (AASHTO 2008). This approach calls for routine friction testing of the roadway network and the subsequent analysis of friction and crash data to identify locations with potentially inadequate levels of friction/texture, comprising of the following components.

- **Step 1 Network Definition** - subdivide the highway network into distinct pavement sections and group the sections according to levels of friction need
    - Define pavement sections;
    - Establish friction demand categories.
- **Step 2 Network-Level Data Collection** - Gather all the necessary information
    - Establish field testing protocols (methods, equipment, frequency, conditions, etc.) for measuring pavement friction and texture;
    - Collect friction and texture data and determine overall friction of each section;
    - Collect crash data.
- **Step 3 Network-Level Data Analysis** - Analyze friction and/or crash data to assess overall network condition and identify friction deficiencies

- Establish investigatory and intervention levels for friction, which are defined as the levels that prompt the need for a detailed site investigation or the application of a friction restoration treatment;
- Identify pavement sections requiring detailed site investigation or intervention.

- **Step 4 Detailed Site Investigation** - Evaluate and test deficient pavement sections to determine causes and remedies
  - Evaluate non-friction-related items, such as alignment, the layout of lanes, intersections, and traffic control devices, the presence, amount, and severity of pavement distresses, and longitudinal and transverse pavement profiles;
  - Assess current pavement friction characteristics, both in terms of micro-texture and macro-texture;
  - Identify deficiencies that must be addressed by restoration;
  - Identify uniform sections for restoration design over the project length.
- **Step 5 Selection and Prioritization of Short- and Long-Term Restoration Treatments** - Plan and schedule friction restoration activities as an integral part of the overall pavement management process
  - Identify candidate restoration techniques best suitable for correcting existing pavement deficiencies;
  - Compare costs and benefits of the different restoration alternatives over a defined analysis period;

- Consider monetary and non-monetary factors and select one pavement rehabilitation strategy.

6.3.2 Use of Friction Data for PFM

Following the AASHTO Guideline (AASHTO, 2008), friction data from CFME or other skid trailers (such as locked-wheel tester at ODOT) can be used in several components of a pavement friction management program.

<u>Step 1 Network Pavement Sections Definition</u>

According to the NCHRP Project 01-43 (Hall et al. 2009), the friction demand levels should be estimated by evaluating three broad categories of influencing factors: highway alignment, highway features/environment, and highway traffic characteristics. At ODOT, control section has been widely implemented as the primary key for GIS and roadway asset management. Control section numbers contain the county number, a continuous route number, and the control number. A control section generally starts or ends at state/county line, a major intersection/highway junction, etc. by different roadway functional classes (interstate, freeway, principal arterial, minor arterial, major collector). In addition, one control section is further divided into multiple sections based on data owners' needs.

Therefore, the currently used ODOT subsections of a control section meet the criteria as recommended in NCHRP Project 01-43, and can be used as the basic network pavement sections. The location of a pavement section can be identified from various information such as route number, direction, county name, control section number, milepost, and GPS coordinates. Since control sections and subsections are used in the ODOT Pavement Management System (PMS), the recommended definition of pavement sections for PFM programs can be easily integrated into existing ODOT programs. However, the current field friction database at ODOT are slightly different in terms of data items from year to year. All data includes county name, route number, control section number, but some data are saved per reference milepost while others also have GPS coordinates. Subsections of a control section are not provided for all the data sets from 2010.

In addition, the control sections at ODOT are defined mainly for roadway segments. However, roadway segments and intersections, two major facility types in the AASHTO Highway Safety Manual (AASHTO, 2010), could require different friction demand levels. For this reason, the Illinois DOT (IDOT) defines roadway segments or intersection into peer groups based on fundamental roadway data elements in their roadway safety analysis. Therefore, the IDOT approaches should be referenced for the defining of roadway sections but also intersections.

Table 6-2 Illinois SPF Peer Groups

Segments	Intersections
1 – Rural two-lane highways	1 – Rural minor-road STOP control
2 – Rural multilane undivided highways	2 – Rural all-way STOP control
3 – Rural multilane divided highways	3 – Rural signalized
4 - Rural freeways – four lanes	4 – Rural undetermined
5 - Rural freeways – six plus lanes	5 – Urban minor-road STOP control
6 – Urban two-lane arterials	6 – Urban all-way STOP control
7 - Urban multilane undivided arterials	7 – Urban signalized

Segments	Intersections
8 - Urban multilane divided arterials	8 – Urban undetermined
9 – Urban one-way arterials	
10 – Urban freeways – four lanes	
11 - Urban freeways – six lanes	
12 - Urban freeways – eight plus lanes	

Step 2 Network-Level Data Collection

The key data inputs for a PFM program are pavement friction, pavement texture, and crash rates. For friction measurements at the network level, both the ODOT used locked-wheel tester and the CFME based Grip Tester are recommended testing instruments, since the measurements from these two devices show statistical correlations at various confidence levels (in Chapter 4). However, if possible, continuous friction measurement devices, such as grip tester and SCRIM (Sideway-force Coefficient Routine Investigation Machine), are preferred because they can provide much more detailed information of pavement friction properties at a much shorter testing interval. Friction measurements on sharp curves, bridge decks, intersections are possible with CFME method, while such scenarios are generally challenging for locked-wheel testers.

The standard testing conditions of friction measurements are 50mph with 0.25mm water film depth for highways with speed limits over 65mph, while 40 mph with 0.25mm water film for other lower speed highways. Under different testing conditions, measured friction number can be adjusted using the relationship developed in Chapter 4.

For pavement texture data collection at ODOT, they are collected simultaneously using a laser-based high-speed profiler with the statewide automated collection of pavement condition data. The data are saved at every 0.01 miles, same as other PMS data sets, in terms of mean profile depth (MPD). The current data collection practices on texture are considered as adequate for PFM purposes. It is widely recognized that pavement surface texture is important for pavement friction and roadway safety. MPD has been the primary index used to characterize macro-texture. However, MPD is a two-dimensional (2D) height indicator calculated from a single profile, which is insufficient to represent surface texture especially for friction analysis. Many studies have found that MPD failed to capture the differences and variations in friction performance. As a result, new texture parameters have been tested and developed to relate pavement texture with friction performance at both the macro- and micro-level. In order to be able to better characterize pavement texture performance, it is therefore recommended that ODOT should store the raw texture profiles so that such analysis is feasible.

Regarding the data collection frequency for friction and texture, it is desirable to test all network pavement sections annually because of the year-to-year variation in pavement friction. However, due to the limitation of available resources, a cost-effective approach should be conducted to identify the pavement sections with priority in terms of friction and texture. The first level of priority should be tested annually, of which the roadway segments have high friction demand and high risk of crashes. The second level of priority can be tested once every few years, such as a 2-year to 3-year based program, half or one-third of the network is tested each year.

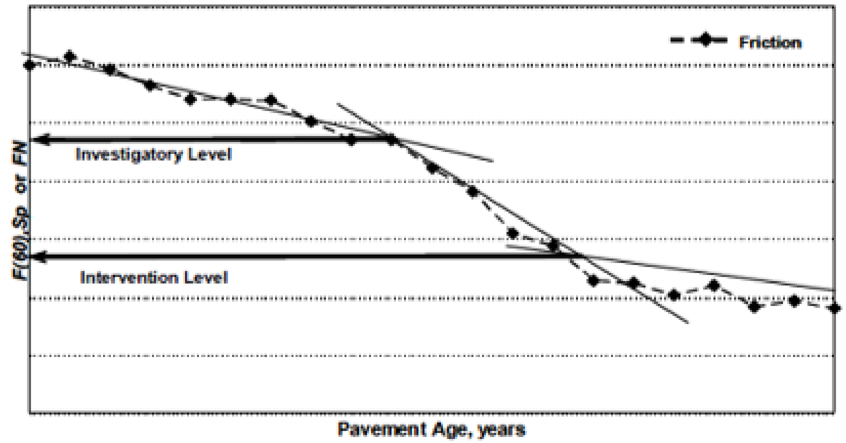
While the third level of priority can be selected based on statistical sampling, of which the pavement sections have low friction demand and low risk of crashes.

Crash data is an important data component for a successful PFM program. ODOT currently manages a computer-based crash database called Statewide Analysis for Engineering & Technology (SAFE-T), to save all highway crashes in the state since 1998. The crash data and related information can be gathered from SAFE-T database, including the severity, type, location of each crash, and corresponding pavement condition (wet or dry) etc. In addition, a comprehensive data check process is also conducted before the crash data are imported into the Safe-T database. It is thus believed the quality of crash data is satisfactory. From those aspects, the rich crash data are sufficient for the need of developing PFM.

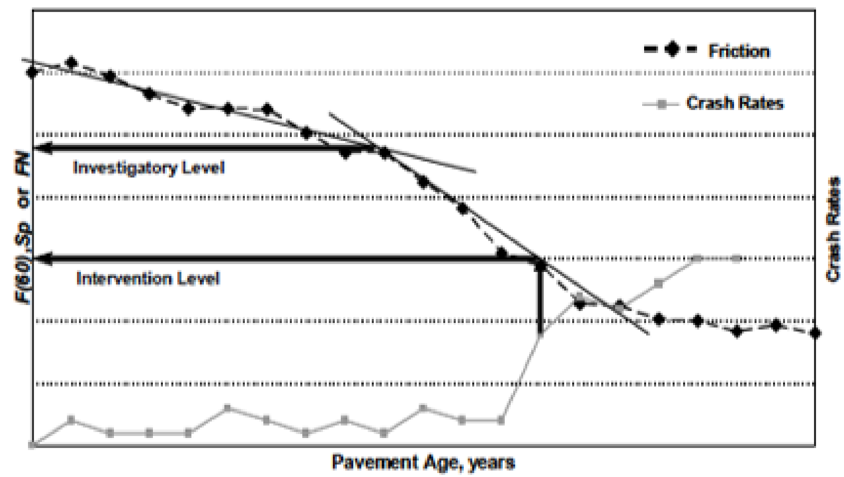
Step 3 Network-Level Data Analysis

The collected network-level data should be assessed to identify roadway segments with potential safety risk. Firstly, pavement network should be segmented into homogeneous sections, using the change point methodologies discussed in Chapter 5, or approaches that are used or desired at ODOT. Subsequently, the adequacy of friction of these segments can be assessed and compared with the investigatory and intervention friction demand levels. Pavement sections with measured friction below investigatory level are subjected to detailed site investigation to determine the need for warning or remedial actions, such as performing more frequency testing and analysis or applying short-term restoration treatment. Once the site friction reaches the intervention level, remedial actions are needed.

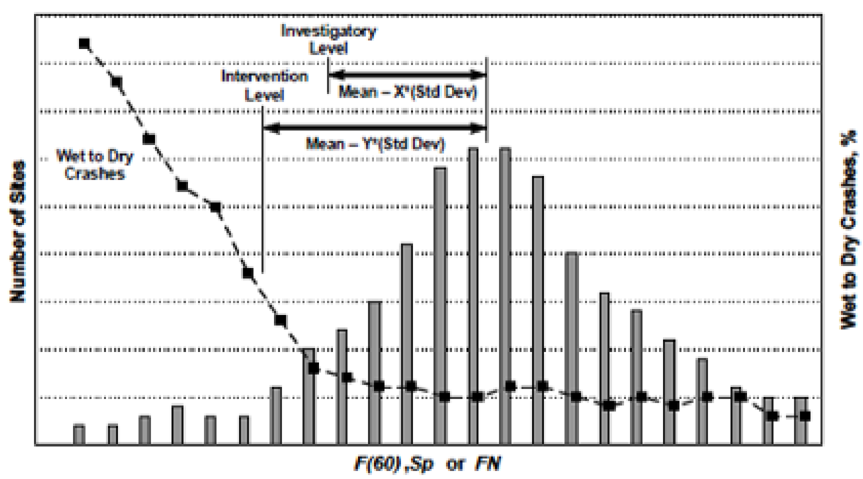
It should be noted that the establishment of investigatory and intervention friction levels is out of the scope of this project and should be carried out for each friction demand category. The friction thresholds can be determined using one of three methods presented in the NCHRP Project 01-43 project (Hall et al. 2009) based on the data availability. The first method (Figure 6-10a) sets investigatory level as the friction value where there is a significant increase in friction loss. The intervention level is set as a certain percentage below the investigatory level. The second method (Figure 6-10b) examines both historical friction and wet-to-dry crash data trends. The investigatory level is set corresponding to a large change in friction loss rate while the intervention level is set where there is a significant increase in crashes. The third method (Figure 6-10c) is the most detailed approach, which uses the distribution of friction data versus the crash rates that correspond with the friction for the category of roadway for which the levels are being set.



(a) Method 1 – Using Time History of Pavement Friction



(b) Method 1 – Using Time History of Friction and Crash Rate



(c) Method 3 – Using Pavement Friction Distribution and Crash Rate-Friction Trend

Figure 6-10 Setting of Investigatory and Intervention Levels (Hall et al. 2009)

Step 4 Detailed Site Investigation

The goal of detailed site investigation for highway sections is to determine the specific causes and evaluate non-friction-related items. The detailed site investigation could involve visual, video, or laser based roadway condition data survey, micro- and macro-texture data evaluation.

The first step is to check the roadway features that may be compounding the friction problems, such as the layout of the lanes, traffic control devices, roadway geometry characteristics, and the extent and severity of pavement distresses (e.g. cracking, rutting, patches, potholes, etc.). The second step involves the testing of pavement surface texture. The collection of macro-texture data is becoming mature with the use of high speed laser profilers. While the micro-texture can be evaluated using Dynamic Friction Tester (DFT) on representative sections. In addition, other relevant information, including traffic volume and the truck percentages, pavement surface type and materials, signs of polishing aggregates should be collected if possible.

Step 5 Selection and Prioritization of Friction Restoration Treatments

The final step is to plan and schedule friction restoration activities best suited to correct existing pavement deficiencies and compare costs and benefits of the different restoration alternatives over a defined analysis period. The prioritization of the highway sections can be identified based on pavement friction, other condition data, and site crash features. In analyzing pavement friction data, the minimum desirable outcome is to ensure that the most “deficient” sites are detected and given reasonable priority. Two possible actions could be considered:

- Continue to monitor the site: Such a decision typically would be reached where (a) current crash rates are sufficiently low and an increase is not expected to significantly impact safety and (b) the pavement surface does not require maintenance because of other factors.
- Select for remedial action to improve pavement friction (e.g., resurface, retexture): This usually would apply where an increase in crash rate might occur if friction remains the same or continues to decrease, and such an increase would significantly impact safety. Preventive maintenance is a cost-effectiveness option to restore pavement performance and extend service life. It is critical to determine the optimal timing of the appropriate treatment.

CHAPTER 7 CONCLUSIONS AND RECOMMENDATIONS

7.1 Conclusions

Pavement skid resistance plays a significant role in road safety. The application of new technologies can be used to determine the low friction areas or vulnerable accident sites at both the project and network levels. An important part of the friction management process is the selection of the most appropriate friction measuring equipment. Continuous friction devices are one of the newest systems currently used in European countries that have recently been introduced to the United States. In this project, the capabilities of the Grip Tester, a type of continuous friction measurement (CFME) device, and its ability to provide information to support PFM programs were evaluated based on a comprehensive field data collection and statistical analysis.

An extensive literature review was conducted to develop an in-depth understanding of various aspects related to friction measurements. The most influential factors to pavement friction were summarized and considered in the field data collection experimental design. In addition, crash data during the past ten years was obtained, and the roadway segments with high observed crashes, particular those within the ODOT skid testing program (interstates and US-69), were also identified as the potential field testing locations. After close consultation with ODOT, twenty-two testing sites were included in this project with nine commonly used treatment types in Oklahoma: chip seal, ultra-thin bonded wearing course (UTBWC), resurfacing (asphalt), micro-surfacing, warm mix asphalt (WMA) thin overlay, resurfacing (concrete), next generation concrete surface (NGCS), longitudinal grooving, and high friction surfacing treatment (HFST).

For each testing site, nine runs of friction data collection were conducted using the Grip Tester at three testing speeds (40mph, 50mph, 60mph for interstates and major arterials; 30mph, 40mph, 50mph for minor arterials), three water film thicknesses (0.25mm, 0.50mm, 1.0mm). Pavement surface condition and texture data were also acquired for each site. In addition, approximate 90-mile of friction data collection were conducted in parallel on northbound of mainline I-35 using the Grip Tester and the ODOT locked-wheel friction trailer. The data collected at these sites along with Oklahoma crash data were further used to assess the operation characteristics and repeatability of CFME friction measurements, to compare the measurements from locked-wheel friction trailer and Grip Tester, and to develop enhanced crash rate prediction models.

Various statistical and comparisons analyses were performed. First, the CFME testing results under various conditions were summarized with descriptive statistics. Further, the performance of CFME was evaluated from three aspects: (1) the correlation with Locked-wheel measurements, (2) the repeatability of CFME data, and (3) the influence of operational characteristics on CFME measurements. The key findings are summarized below:

- The HFST sites show the best friction performance, followed by the transverse grooving site on US-77. The other testing sites, with various surface types, are observed to have comparable friction performance

- For most sites, with increasing testing speeds, the friction numbers show decreasing trend. However, the effects of varying testing speed differ for different treatment types. Some sites demonstrate pronounced reduction in friction performance with increasing speeds, such as chip seal, UTBWC, HMA and some WMA sites, while others are not as significant. HFST as the example, it seems that their friction performance does not depend on testing speeds.
- Friction numbers decrease when larger water film thickness is applied. For some sites, the friction values are significantly reduced when the water film thickness is increased from 0.25mm to 0.5mm. On the other hand, HFST sites for example, the impact of water film thickness for some sites does not demonstrate such significant influences.
- Comparing to the friction measurements from LWST, Grip Numbers from the Grip Tester measured on the same sites are consistently larger, and are more sensitive to pavement surface types. Multi-variant regression analysis was applied to develop the correlation between SN and GN. The skid number from LWST is statistically significant when correlating to the grip number, while there are several other factors that have showed statistical significance to the Grip Tester measurements. Even though the correlation models between the two measurements are statistically significant, the R-squared value is not strong due to the high variabilities presented in the field friction measurements using contact-based method for both LWST and grip tester.

- Various methodologies were implemented for repeatability testing. The average differences (the maximum minus minimum friction) and standard deviation of the repeating measurements are 0.039 and 0.051 respectively, indicating the Grip Tester can measure surface friction in a consistent manner. The correlation based repeatability analysis results suggest that the Grip Tester based CFME measurements have sufficient repeatability. On average, the cross-correlation coefficient is 94.71% for the HFST testing sites, and 63.15% for the shotblasting site. The cross-correlation analysis method is more robust for friction profiles with more distinct changes and pronounced features.
- To access the impact of friction measurements due to the change of operational characteristics (testing speed, WFT, ambient temperature), regression model was developed following the backward stepwise analysis. The adjusted R-squared of the model is 0.68, indicating that the model has satisfactory statistical performance. Ambient temperature, testing speed, and water film thickness show negative effects on the friction measurements, while the average MPD has a positive effect on friction measurements. State agencies can utilize the multivariate regression model to adjust/correct the friction collected at different testing conditions to the baseline condition and be comparable for further decision making.

The continuous friction data provides much more detailed skid resistance information, and can have several potential implementations for pavement friction management and highway safety applications.

- Combining the CFME friction and pavement condition data sets collected from this project with the Oklahoma safety database, a framework was proposed to integrate the roadway characteristics into the development of crash rate prediction model. A case study was subsequently provided to implement the framework for the development of enhanced safety performance function (SPF) using CFME friction data. The statistical significant factors for crash includes AADT, segment length, and pavement surface friction. The number of crashes per mile in five years was expected to increase with the increase of traffic volume or the decrease of surface friction coefficient. The model indicated that the crash data was over-dispersed and thus the negative binomial model was suitable for this case study.
- With detailed CFME data, pavement network could be better segmented into homogenous sections for road maintenance scheduling and management. Surface friction data were collected in Oklahoma using a Grip Tester and used for the case study. Three dynamic segmentation methods: binary, neighborhood and PELT methods were implemented. The segmentation results were compared with the subsection-based method currently used in the pavement management system (PMS) at

ODOT, and their efficiency and robustness were evaluated for each individual segment and the overall roadway.

- In order to ease the using of CFME data, a CFME data analysis software interface was developed, which is able to upload and visualize CFME measurements, and perform basic statistical and comparison analyses for data reporting.

In addition, the AASHTO Guide for Pavement Friction provides an overall approach for managing pavement friction, which calls for routine friction testing of the roadway network and the subsequent analysis of friction and crash data to identify potentially inadequate locations. How CFME data and the findings from this study can be used to assist the pavement friction management was discussed.

7.2 Recommendations

The use of slip-wheel CFMEs devices has gained popularity in Europe and recently started using in U.S. for roadway applications. These devices can measure friction continuously and provide more spatial details than locked-wheel devices. This project presents the research effort to explore the use of CFME, Grip Tester herein, as a tool for pavement friction management and roadway safety practices. The direct comparison results suggest that CFME can provide repeatable and reproducible friction profiles, and the measurements from Grip Tester and locked-wheel skid trailer are statistically correlated. However, such relationships are not very strong in some cases and impacted by several other factors. It should be noted that the data used in this report as case studies is only for demonstration purpose due to the limited number of testing sites and the duration of the monitoring cycle (less than 2 years). Therefore, it is recommended that ODOT should consider adopting CFME for their skid resistance program. Through a wide range of measurements and analysis, an in-depth understanding of the testing methodology could be achieved to (1) quantify the effect of operational factors, and (2) establish standard testing condition and approaches.

Besides, several other challenges and opportunities remain for the wide implementation of CFME for friction management and roadway safety practices. The Grip Tester has been proved by several studies to be an accurate and economical way to measure skid resistance on roads. Meanwhile, it is also found that the variations of measurements could be high on segments with bad surface conditions (such as high IRI, faulted roads) while traveling at highway speed. This is probably due to the low weight of the trailer. During the field testing, the team depended on vehicular cruise control capability and was able to maintain a stable testing speed (generally within 1 mph range). However, the water flow data recorded by the Grip Tester software could vary significantly in some occasions, which could result in variations of friction measurements.

Grip Tester is one of the CFME devices on the market. Others in use include Side-Force Coefficient Routine Investigation Machine (SCRIM), Mu-Meters, Airport Surface Friction Testers (ASFT), Dynatest Pavement Friction Tester (DPFT), etc. In particular, SCRIM has been implemented in the very recent years with support from Federal Highway Administration (FHWA) and many state agencies through a transportation pooled fund study. Although both the Grip Tester and the SCRIM continuously measure wet pavement friction, they use different test tires, different tire orientation, and different slip ratios (and slip speeds). In addition to measuring friction, the SCRIM generally also measures macrotexture (MPD, in mm), longitudinal grade (%), cross-slope (%), and horizontal curvature (1/m). As a result, different CFME devices should be investigated and evaluated for ODOT's consideration regarding its capabilities, advantages, disadvantages, and costs etc. for potential future adoptions.

Furthermore, no matter which friction testing instruments are used, the existing methodologies are all contact based, which require dragging a tire/rubber pad across a road/specimen to measure roadway skid resistance. Such technology could result in inconsistent friction measurements depending on many operational factors, such as the viscoelastic properties of testing tires under various temperatures, testing speed, and surface conditions with various water film depth, and the wandering of testing vehicle. In addition, it requires in-vehicle water tank in order to wet testing surface and consumes testing rubber pads or tires which could wear quickly due to the physical contact with pavement surfaces. Therefore, it is long desired to develop non-contact methods, such as surface texture collection, to supplement and/or replace the existing contacting pavement friction measurements with less inconsistency and variations. With the fast development of non-contact 3-Dimensioal (3D) measurement technologies and the improvement in the computing and processing power of computers, it is becoming feasible and desirable to describe road surface texture in both macro- and micro-scale under 3D at high resolution for pavement skid resistance characterization.

It is also found that the existing contact-based friction measurement methods (such as the LWST with a ribbed tire) can be insensitive to macrotexture under specific circumstances. It thus should be recommended that CFME friction testing be complemented by macro-texture measurement, in order to which could facilitate the definitions of investigatory friction and macro-texture levels to support the state's pavement friction management program. This will allow areas with potentially deficient macro-texture to be identified and investigated at the project level and corrected, if necessary, before the occurrence of wet-weather crashes.

In recent years, the scope of the ODOT Skid Studies Program has been downsized to annual testing of US-69, all the Interstate Highways, as well as the Strategic Highway Research Program (SHRP) sites. In order to develop a comprehensive pavement friction program, a wider range of friction measurements is desired for common pavement types, overlays, and surface treatments on various highways with different traffic levels and environmental zones in the state. Subsequently, more advanced analysis could be conducted for enhanced roadway safety management, such as:

- The incorporation of CFME friction measurements into safety performance functions (SPF) to improve the assessment of overall network condition and identify friction deficiencies that might cause crashes,

- Developing preservation policies that improve the safety of the roadway network and decrease the number of skidding-related crashes through engineered safety improvements.

Finally, there is also a serious need to predict friction and macro-texture properties during mix design, when the mix can be adjusted to meet frictional demand at the project location. The detailed information used to develop mix-specific performance equations can then be used to predict (and accurately monitor) the substantial benefits that can be expected from skid-resistant surface mixes and surface treatments, which will be further related to highway safety in terms of the reduction of traffic crashes.

REFERENCES

AASHTO (1993), Guide for design of pavement structures, American Association of State Highway and Transportation Officials, Washington, DC.

AASTHO. (2013a). Quantifying cracks in asphalt pavement surfaces from collected images utilizing automated methods. AASHTO Designation PP67-10. Washington, D.C.

AASTHO. (2013b). Standard practice for determining pavement deformation parameters and cross slope from collected transverse profiles. AASHTO Designation: PP69-10. American Association of State Highway and Transportation Officials. Washington, D.C.

Adelle W. (2006). Quantitative Characterisation of Surface Finishes on Stainless Steel Sheet using 3D Surface Topography Analysis. Doctoral thesis, University of Huddersfield.

Arhin, S.A., Noel, E.C., and Ribbiso, A. (2015). Acceptable International Roughness Index Thresholds based on Present Serviceability Rating. Journal of Civil Engineering Research, 5(4), pp. 90-96, doi:10.5923/j.jce.20150504.03.

ASTM Designation D6433-18. (2018). Standard Practice for Roads and Parking Lots Pavement Condition Index Surveys. ASTM International. West Conshohocken, PA, DOI: 10.1520/ D6433-18.

ASTM Standard E1845-15. (2015). Standard Practice for Calculating Pavement Macrotexture Mean Profile Depth. ASTM International. West Conshohocken, PA, DOI: 10.1520/E1845-15.

ASTM Standard E1911-09a. (2009). Standard Test Method for Measuring Paved Surface Frictional Properties Using the Dynamic Friction Tester. ASTM International, West Conshohocken, PA, DOI: 10.1520/E1911-09AE01.

ASTM Standard E2157-15. (2015). Standard Test Method for Measuring Pavement Macrotexture Properties Using the Circular Track Meter. ASTM International, West Conshohocken, PA, DOI: 10.1520/E2157-15.

ASTM Standard E274/E274M-115. (2015). Standard Test Method for Skid Resistance of Paved Surfaces Using a Full-Scale Tire. ASTM International, West Conshohocken, PA, DOI: 10.1520/E0274_E0274M-15.

ASTM Standard E303-93. (2013). Standard Test Method for Measuring Surface Friction Properties Using the British Pendulum Tester. ASTM International, West Conshohocken, PA, DOI: 10.1520/E0303-93R13.

ASTM Standard E950-98. (1998). Measuring Test Method for Measuring the Longitudinal Profile of Traveled Surfaces with an Accelerometer Established Inertial Profiling Reference. ASTM International, West Conshohocken, PA.

ASTM Standard E965-15. (2015). Standard Test Method for Measuring Pavement Macrotexture Using a Volumetric Technique. ASTM International, West Conshohocken, PA, DOI: 10.1520/E0965-15.

Auger I.E., C.E. Lawrence (1989). Algorithms for the optimal identification of segment neighborhoods. Bulletin of mathematical biology 51(1), pp 39-54.

Bai J.S., P. Perron (1998). Estimating and testing linear models with multiple structural changes. Econometrica 66(1), pp 47-78.

Bitelli G. et al. (2012). Laser Scanning on Road Pavements: A New Approach for Characterizing Surface Texture. Sensors, Vol. 12, pp. 9110-9128, DOI: 10.3390/s120709110.

Bonaquist, R. (2011). Mix Design Practices for Warm Mix Asphalt. NCHRP Report 691, Transportation Research Board, National Research Council, Washington, D.C.

Bower N. et al. (2012). Evaluation of the Performance of Warm Mix Asphalt in Washington State. Washington DOT. Olympia, WA.

Button J.W., Estakhri C. and Wimsatt A., (2007). A Synthesis of Warm-Mix Asphalt. Publication FHWA/TX-07/0-5597-1. FHWA and Texas Department of Transportation.

Cafiso S., A. Di Graziano (2012). Definition of Homogenous Sections in Road Pavement Measurements, Procedia - Social and Behavioral Sciences 53, pp 1069-1079.

Cerezo, V., Do, M., Prevost, D., and Bouteldja, M. (2014). Friction/Water Depth Relationship-In Situ Observations and Its Integration in Tire/Road Friction Models. Journal of Engineering Tribology, 228(11), pp. 1285-1297.

Copeland, A. et al. (2010). Field Evaluation of High Reclaimed Asphalt Pavement/Warm Mix Asphalt Project in Florida: A Case Study. Transportation Research Board Annual Meeting CD-ROM, Transportation Research Board of the National Academies, Washington, D.C.

Dave, E.V., Hanson, C.E., Helmer, B., Dailey, J., and Hoplin, C.M. (2015). Laboratory Performance Test for Asphalt Concrete. Publication MN/RC 2015-24, Minnesota Department of Transportation, St. Paul, Minnesota.

D'Angelo J. et al. (2008). Warm-Mix Asphalt: European Practice. Publication FHWA-PL-08-007. Washington D.C., Federal Highway Administration Office of International Programs.

Deltombe, R., Kubiak, K.J., Bigerelle, M. (2011). How to Select the Most Relevant 3D Roughness Parameters of a Surface Scanning, Vol. 36, No. 1, pp. 150-160, DOI: 10.1002/sca.21113.

Diefenderfer, S. and A. Hearon. (2008). Laboratory Evaluation of a Warm Asphalt Technology for Use in Virginia. Publication VTRC 09-R11, Virginia Transportation Research Council, Charlottesville, VA.

Do, M. et al. (2007). Pavement Polishing-Development of a Dedicated Laboratory Test and its Correlation with Road Results. Wear, Vol. 263, pp. 36-42.

Do, M., Cerezo, V., Beautru, Y., and Kane, M. (2014) Influence of Thin Water Film on Skid Resistance. Journal of Traffic and Transportation Engineering, 2(1), pp. 36-44.

Dueker K.J., R. Vrana (1992). Dynamic segmentation revisited: a milepoint linear data model. Journal of the Urban and Regional Information Systems Association 4(2), pp 94-105.

Dunford A.M. et al. (2012). Three-Dimensional Characterization of Surface Texture for Road Stones Undergoing Simulated Traffic Wear. Wear, Vol. 292-293, pp. 188-196.

Edwards A.W., L.L. Cavalli-Sforza (1965). A method for cluster analysis, Biometrics, pp 362-375.

Ergun, M., Iyinam, S., and Iyinam, A. (2005). Prediction of Road Surface Friction Coefficient Using Only Macro- and Microtexture Measurements. Journal of Transportation Engineering, Vol. 131, No. 4, pp. 311-319.

Federal Highway Administration (FHWA). (2016). “Pavement Friction.” U.S. Department of Transportation, FHWA, Washington, D.C. <https://safety.fhwa.dot.gov/roadway_dept/pavement_friction/> (Accessed on April 1, 2019)

Flintsch G.W. et al. (2012). The Little Book of Tire Pavement Friction. Pavement Surface Properties Consortium.

Hall J.W. et al. (2009). NCHRP Web Document 108: Guide for Pavement Friction. Transportation Research Board, National Research Council, Washington, D.C.

Henry, J.J. (2000). NCHRP Synthesis 291: Evaluation of Pavement Friction Characteristics. TRB, National Research Council, Washington, D.C.

Hartikainen L., Petry F. and Westermann S. (2014). Frequency-wise Correlation of the Power Spectral Density of Asphalt Surface Roughness and Tire Wet Friction. Wear, Vol. 317, pp. 111-119.

Hurley, G. C. et al. (2006) "Evaluation of potential processes for use in Warm Mix Asphalt." Journal of the Association of Asphalt Paving Technologists, Savannah, GA, United States, 41-90.

Izeppi E., Flintsch G. and McGhee K. (2010). Field Performance of High Friction Surfaces. Publication FHWA/VTRC 10-CR6. FHWA, U.S. Department of Transportation.

Kanafi M.M. et al. (2015). Macro- and Micro-Texture Evolution of Road Pavements and Correlation with Friction. International Journal of Pavement Engineering, Vol. 16, No. 2, pp. 168-179, DOI: 10.1080/10298436.2014.937715.

Kane M., Rado Z. and Timmons A. (2015). Exploring the Texture-Friction Relationship: from Texture Empirical Decomposition to Pavement Friction. International Journal of Pavement Engineering, Vol. 16, No. 10, pp.919-918, DOI:10.1080/10298436.2014.972956.

Killick R., P. Fearnhead, I.A. Eckley (2012). Optimal detection of changepoints with a linear computational cost. Journal of the American Statistical Association 107(500), pp 1590-1598.

Killick R., I. Eckley, changepoint (2014). An R package for changepoint analysis. Journal of statistical software 58(3), pp 1-19.

Kvasnak, A., J. Moore, A. Taylor, and B. Prowell. (2010). Evaluation of Warm Mix Asphalt Field Demonstration: Nashville, Tennessee. NCAT Report 10-01, National Center for Asphalt Technology. Auburn, AL.

Leach R. (2012). Characterisation of Areal Surface Texture. Springer-Verlag Berlin Heidelberg, DOI: 10.1007/978-3-642-36458-7.

Li L., Wang K.C.P., and Li Q.J. (2016). Geometric Texture Indicators for Safety on AC Pavements with 1 mm 3D Laser Texture Data. International Journal of Pavement Research and Technology, Vol. 9, pp. 49-62.

Li Q., K. Wang, G. Yang (2015). PELT based dynamic segmentation for network level pavement evaluation with 1 mm 3D data. 6th International Conference on Bituminous Mixtures and Pavements, Thessaloniki, Greece.

Li S. et al. (2016). Pavement Surface Microtexture: Testing, Characterization and Frictional Interpretation.

Liang J., M.-L. Tang, P.S. Chan (2009). A generalized Shapiro–Wilk W statistic for testing high-dimensional normality. Computational statistics & data analysis 53(11), pp 3883-3891.

Liu Q. and Shalaby A. (2015). Relating Concrete Pavement Noise and Friction to Three-Dimensional Texture Parameters. International Journal of Pavement Engineering, DOI: 10.1080/10298436.2015.1095897.

Michigan Metrology. (2014). Michigan Metrology Surface Texture Parameters Glossary. Michigan Metrology, LLC, MI.

Mohammad, L., Hassan, M., Vallabhu, B., and Kabir, M. (2015). Louisiana's Experience with WMA Technologies: Mechanistic, Environmental, and Economic Analysis. Journal of Materials in Civil Engineering, 27(6): 04014185.

Nataadmadja A.D. et al. (2012). Quantifying Aggregate Microtexture with Respect to Wear-Case of New Zealand Aggregates. Wear, Vol. 332-333, pp. 907-917.

National Asphalt Pavement Association (NAPA). (2002). Designing and Constructing SMA Mixtures—State-of-the-Practice. Quality Improvement Series 122, NAPA, Lanham, Maryland.

Prowell, B. D., G. C. Hurley, and E. Crews. (2007). Field Performance of Warm Mix Asphalt at the NCAT Test Track. Transportation Research Board Annual Meeting CD-ROM, Transportation Research Board of the National Academies, Washington, D.C.

West, R., et al. (2014). Field Performance of Warm Mix Asphalt Technologies. NCHRP REPORT 779. Transportation Research Board of the National Academies, Washington, D.C.

Salem O., Y. Liu, A. Mehaoua (2013). A lightweight anomaly detection framework for medical wireless sensor networks. IEEE Wireless Communications and Networking Conference (WCNC), IEEE, pp. 4358-4363.

Sargand, S., Nazzal, M.D., Al-Rawashdeh, A., and Powers, D. (2012). Field Evaluation of Warm-Mix Asphalt Technologies. Journal of Materials in Civil Engineering, 24(11), pp. 1343-1349. DOI: 10.1061/(ASCE)MT.1943-5533.0000434.

Scott A.J., M. Knott (1974). A cluster analysis method for grouping means in the analysis of variance. Biometrics, pp 507-512.

Serigos P.A. (2013). The Contribution of Micro- and Macro-Texture to the Skid Resistance of Pavements. http://www.ectri.org/YRS13/Documents/Presentations/Session5b-6a/YRS13_Session5b-6a_Serigos_TRB.pdf. Accessed 12 May, 2016.

Thomas F. (2003). Statistical approach to road segmentation, J Transp Eng 129(3), pp 300-308.

Ueckermann et al. (2015). A Contribution to Non-contact Skid Resistance Measurement. International Journal of Pavement Engineering, Vol. 16, No. 7, pp. 646-659, DOI:10.1080/10298436.2014.943216.

Villani M.M. et al. (2014). Application of Fractal Analysis for Measuring the Effects of Rubber Polishing on the Friction of Asphalt Concrete Mixtures. Wear, Vol. 320, pp. 179-188.

Wang, K.C.P. et al. (2015). Network Level Pavement Evaluation with 1 mm 3D Survey System. Journal of Transportation Engineering (English Edition), 2(6), pp. 391–398.

West, R. et al. (2011). Field Performance of Warm Mix Asphalt Technologies. NCHRP Report 779, Transportation Research Board, National Research Council, Washington, D.C.

Wielinski, J., A. Hand, and D. M. Rausch. (2009). Laboratory and Field Evaluations of Foamed Warm Mix Asphalt Projects. Transportation Research Record, No. 2126, Transportation Research Board, National Research Council, Washington, D.C., pp. 125-131.

Williams, Brett A., Audrey Copeland, and T. Carter Ross (2018). Asphalt Pavement Industry Survey on Recycled Materials and Warm-Mix Asphalt Usage: 2017. Information Series 138 (8th edition). National Asphalt Pavement Association. Lanham MD.

Yang, G. et al. (2018). Wavelet based macrotexture analysis for pavement friction prediction. KSCE Journal of Civil Engineering, 22(1), 117–124. https://doi.org/10.1007/s12205-017-1165-x.

Zelelew H.M. et al. (2014). Wavelet-based Characterisation of Asphalt Pavement Surface Macro-texture. Road Materials and Pavement Design, Vol. 15, No. 3, pp. 622-641, DOI:10.1080/14680629.2014.908137.

Zhang, A. et al. (2017). Automated Pixel-Level Pavement Crack Detection on 3D Asphalt Surfaces Using a Deep-Learning Network. Computer-Aided Civil and Infrastructure Engineering, 32, pp. 805–819.

Zhang, A. et al. (2018). Pavement Lane Marking Detection Using Matched Filter. Measurement, 130, pp. 105-117.