Quantifying the relationship between skid resistance and crashes for Iowa roadways: A framework for a skid resistance policy

by

Wasama Abdel Razzaq Abdullah

A thesis submitted to the graduate faculty

in partial fulfillment of the requirements for the degree of

MASTER OF SCIENCE

Major: Civil Engineering (Transportation Engineering)

Program of Study Committee:
Omar Smadi, Major Professor
Roy Sturgill
Qing Li

Iowa State University

Ames, Iowa

2020

DEDICATION

To every working-student Mom… whenever you feel vulnerable … whenever you are about to give up … this success is dedicated to you… I did it, and you can do it!

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

ACKNOWLEDGMENTS

I would like to thank my Advisor, Dr. Omar Smadi who guided me throughout the course of this research with his knowledge and experience. I would also like to thank my committee members Dr. Qing Li, and Dr. Roy Sturgill for their willingness to participate on my committee and for their support. I also want to acknowledge Dr. Ahmad Alhassan's help and guidance through the early stages of scratching the hypothesis of this research. To my mentors and colleagues at the Institute for Transportation, the CCEE department faculty, and staff for making my time at Iowa State University a pleasant experience.

To my astoundingly supportive Mom, Jamilah Amer. Mom, thank you for sacrificing a lot, putting us first, putting in your heart and soul into seeing that we had everything we needed as we grew up. I can never repay your precious words of encouragement and prayers that you have sent my way, overseas, to make sure that I crossed the finish line. To my Dad, Abdelrazzaq Abdullah, my fighter, my backbone. The one who taught me how to fight for life; May your soul rest in peace. I hope that I have made you proud.

To everyone who has touched my life in any positive way, you all know who you are, and I am truly grateful for all of you. To my dearest friend and partner in crime, Ala Khaleel for her endless emotional support. Ala, I've been fortunate enough to have been blessed with having you in my life.

To my best friend and my darling husband, Ahmad Alzubaidi, my most stalwart supporter and knight, your faith in me is what propels me to pursue my education and to always be remarkable in anything that I do. This dream would not been possible without you believing in me, working with me, standing by my side, understanding me best as a Ph.D. yourself and helping us to have a great life together. Without you, I would not be where I am today. Finally,

to my eternal love, my son, Yazid Alzubaidi, you are truly a treasure, and a gem that I will always cherish and love. You have made me stronger, better and more fulfilled than I could have ever imagined. I love you to the moon and back. Your smile and giggles is what has gotten me through this stressful period in the most positive way, whenever I felt vulnerable, and whenever my heart ached for missing on so much time with you. Yazid, I pray that growing up watching your very busy and driven mother would only teach you the same lesson that I have learned....that we do not fall, and if we do, we always fall forward.

"There is no passion to be found, playing small and settling for the life that is less than the one you are capable of living".

Nelson Mandela

ABSTRACT

The lack of sufficient friction at the tire-pavement interface is a major contributing factor to traffic crashes. The relationship between surface friction and roadway safety has been recognized since the thirties. Minimum skid resistance guidelines have been the focus of intensive research efforts, but very little research in the U.S. has addressed the different friction demand categories that can be integrated into an effective skid resistance policy. It is crucial to quantify this relationship and determine the level of roadway surface friction needed (i.e., friction demand) to eliminate roadway surface friction-related crashes and reduce the severity of those that have more complex causation. This thesis quantifies the relationship between skid resistance and crashes for Iowa roadways. The correlation between skid resistance measured with a locked-wheel trailer and crash rates for wet, dry and roadway departure crashes is investigated through employing a two-parameter, two-level skid resistance model where factors like roadway geometry, roadway functional classification, traffic volume, speed, and pavement type are considered to determine friction demand investigatory and intervention levels. The research used crash data, skid measurements, traffic volumes and tangents information, from the Iowa Department of Transportation and the Institute for Transportation for the year 2018. Friction measurements are divided into intervals with increments of 2 friction units and the number of crashes for each friction interval were determined for the overall data and for the different analysis categories for which the crash rate models are generated and summarized. The specific crash rates were utilized in the friction demand regression models generation. Regression analyses indicated that there is statistically significant effect of skid resistance on wet, dry, and roadway departure crashes; as expected, skid resistance is a factor in explaining the variation in crash rates. For all sites evaluated, friction is found to be a significant factor affecting wet crash rates except for sites with low traffic where no tangible relationship is

detected between the two variables. Friction is found to be a significant factor affecting dry crash rates for most of the sites except for rural interstates. Friction was found to be a significant factor affecting roadway departure crashes at some sites where more roadway departure crashes were successfully matched with crash locations. However, no relationship was detected for urban freeways and expressways, urban principal arterials, rural minor arterials, sites with high speed limit, sites with low traffic as well as Portland cement concrete and asphalt concrete pavements. A larger study is needed to further investigate these findings with stratifying the roadway departure crashes by the surface contamination state. However, as expected, friction data tend to explain only a small portion of the variation in crash rates when considering individual crash sites and a statistically significant effect of skid resistance on the wet, dry and roadway departure crash rate is captured by grouping the crash sites by similar characteristics and better explained the variability in crash occurrence. Generally, based on the data studied, a target skid number (SN40) of 42 and 47 appears to have positive safety benefits with respect to wet and roadway departure crashes maintaining a crash risk of less than 500 crash per hundred million vehicle miles traveled on the network tangent segments.

CHAPTER 1. INTRODUCTION

General Background

For decades, the United States Department of Transportation (USDOT) has had a long-standing goal of reducing fatal crashes. The latest safety goal by the USDOT in 2010 was to decrease the number of fatalities from 33 thousand fatalities in 2010 by half over 20 years (USDOT). Nevertheless, today, in 2020, halfway through the goal designated period, the United States is witnessing an increase of 12% in the number of fatalities on its highway network (National Highway Transportation Safety Administration [NHTSA], 2020). There seems to be a pressing need for a more elaborate understanding of these vehicle crash events and their contributing factors.

Crashes are due to the interaction between five factors: driver, road, vehicle, traffic, and environment. Hence, a collision may be attributed to driving errors, vehicle malfunctions, poor geometric alignment of the roadway, weather, or lack of sufficient friction at the tire pavement interface. However, because the frictional properties of the roadway surface play a vital role in enhancing the driver's maneuvers on the road, the same driver behavior exhibited on a road with excellent pavement friction under wet conditions is probably less likely to result in a crash leading to a fatality or severe injury (Ivan et al., 2010).

The Federal Highway Administration (FHWA) implements updated policies to help satisfy the need for incorporating the safety management programs into the pavement management programs. Consequently, The FHWA recommended that each state Department of Transportation (DOT) develop a Pavement Friction Management Program (PFMP) within their guidelines to reduce the risk of crashes and take corrective action to address friction deficiencies (FHWA, 2010). However, although it is essential for all roadways to maintain some level of friction, different factors such as roadway geometry (i.e., curves and grades), roadway functional classification, traffic volume, speed, and the potential for conflicting movements (i.e., intersections) typically affect the level of friction required. Research in Europe has indicated that the relationship between surface friction properties and crash rates can be quantified and integrated into friction demand levels for certain pavement surface properties and specific sites of different friction demanding factors (Kuttesch, 2009).

PFM policies have been implemented for almost 2 decades in the United Kingdom (U.K.). Consequently, the U.K. is one of several European Union (E.U.) countries that were able to reduce fatalities by half over 20 years (Office for National Statistics, 2013). International practice experience suggests that it is critical to determine the level of roadway surface friction needed to eliminate friction-related crashes and casualties. However, the process of integrating the quantified friction-crash relationship into skid resistance policies and validating the European findings has not yet been implemented sufficiently nationwide in the United States. Obviously, "Two components, friction and safety management, are still disconnected and not synchronized efficiently" (Al Hassan et al., 2018). Therefore, there seems to be a need to expand the research efforts concerning PFM and, especially, level-based skid resistance policies to assist every U.S. DOT to use limited resources effectively while still providing safe, reliable, and economical roadways to the traveling public. This change would represent a robust movement towards U.S. DOTs long-standing goal of a 50% reduction in fatalities over the next 10 years as well as a step towards a future of "Zero Fatalities" on the U.S. roadway network.

Problem Statement

In the past 5 years, approximately 290,000 vehicle crashes have occurred on the state-maintained roadway network in Iowa, of which 1,700 were fatal and almost 32,000 involved injuries (Iowa DOT Crash Data, 2018). For the fatal and injury crashes, "Roadway departure crashes," "loss of control crashes," "speeding-related crashes ", and "impairment crashes" were the top four common contributing factors. These contributing factors point to some sort of erratic driver behavior; however, insufficient pavement friction can often be a determining factor for whether this behavior results in a crash and the severity of the crash. Since collisions are complex and random events that cannot be explained by the impact of driver behavior, roadway geometry, or skid resistance exclusively. In fact, "there is no existing method to determine the skid resistance threshold that will make a hazardous site 'safe'" (Long et al., 2014 p.7).

It is crucial to determine the level of roadway surface friction needed (i.e., friction demand) to eliminate roadway surface friction-related crashes and reduce the severity of those that have more complex causation. Federal Highway Administration (FHWA) recommends that state Departments of Transportation DOTs implement highway safety management programs related to pavement friction (FHWA, 2010). Moreover, much of the early research relating pavement friction to crash rates in the U.S. aimed to establish minimum thresholds for friction utilizing statistical analyses of historical skid resistance and crash data. These thresholds have been recommended as standards for the design and maintenance of the whole roadway network. However, single-point minimum friction thresholds have three significant limitations:

1. Implementing one rigid friction threshold value over the whole network will make friction "supply" meet or surpass friction "demand" over the entire network. Such practice would be restrictively expensive and superfluous, resulting in a cost/benefit impracticality (Hall et al. 2009).
2. Applying one-level statistical analyses will result in misleading crash-friction correlations that do not have the advantage of identifying the factors that have

important implications in the threshold setting process. Therefore, such an analysis is biased if it is not categorized by the factors that affect friction demand.

3. Providing a single-point friction level that defines the threshold between “safe” and “potentially unsafe” places additional legal responsibilities on the state highway agencies, as “case law indicates that states can be found liable for low friction conditions due to either their actions or lack of action” (Carlson, 1974). Therefore, this concept of single-point minimum threshold values as mandatory standards for pavement friction faced strong opposition from the transportation agencies (Kettusch, 2004).

The significant limitations of the concept of a single-point skid resistance threshold can be overcome by employing a two-parameter, two-level skid resistance model (Fwa, 2017). In such a model, factors like roadway geometry (i.e., curves and grades), roadway functional classification, traffic volume, speed, and potential for conflicting movement (i.e., intersections) are considered to determine what are called “friction demand levels.” These are guidelines rather than a “single point friction threshold” that sets a rigid requirement for the whole network.

Research Objectives

The objective of this thesis is to establish a framework that delivers a cost-effective, risk-based prioritized skid resistance program that can be tailored to the Iowa DOT budget and that will minimize the fatal and injury-causing friction-related crashes on the roadway network. Answering the following questions can meet this objective:

1. Can we detect a relationship between the skid resistance and the wet/dry crash rate on the roadway network in Iowa?

2. Is this association more significant when exploring variation in crash history among different traffic, speed, functional classification categories, and at locations with high expected braking frequency (i.e., intersections) on Iowa's roadway network?
3. What is the Investigatory Level (I.L.) of the skid resistance necessary for each of the categories across the network?
4. How can the Iowa DOT use the friction demand levels to prioritize the use of its funds in a PFM framework?

Significance of the Research

Providing guidelines on the desired frictional requirements of the site-specific pavement surfaces within the context of safety performance will help the Iowa DOT and probably other DOTs to develop a friction management program that can efficiently tolerate part of the road safety risk. A robust skid resistance program will help the Iowa DOT advance their network friction testing practice and use the data to develop proper maintenance decisions. The proposed friction demand levels will serve as skid resistance maintenance trigger levels that are readily implementable towards a consistent and pro-active PFMP to address Iowa's roadway network safety issues and ensure more efficient resource allocation. The benefits of the proposed skid resistance program are a reduction in crash rates and severity as well as savings in skid resistance related asset management activities (i.e., monitoring, maintenance, and resurfacing). Therefore, this research represents a significant step forward in the practices of friction management programs in Iowa.

Thesis Overview

This thesis is organized into six main chapters, which detail the background of the research problem of interest, provide context with respect to the research literature, outline the study methods, and demonstrate answers to the research questions of interest prior to presenting final conclusions. A brief description of these chapters follows:

Chapter 1: Introduction – This chapter provides background on the importance of friction in safety design of highways as well as developing pavement friction management programs. The chapter outlines the need for additional research in this area. The background section is followed by a presentation of the research statement and objectives that have been outlined to address the research questions as well as the research significance.

Chapter 2: Literature Review – This chapter is structured into five sections to extensively summarize the extant literature regarding pavement friction and pavement friction management programs. First, an overview of tire-pavement friction and roadway safety is provided through a review of research focused on the effects of skid resistance on crash occurrence. This is followed by demonstrating the basic concepts of tire-pavement friction then the factors affecting it. Next, a summary of the history of tire-pavement friction measurement methods and frequency of testing as well as national and international practices. Then, a review of the history of pavement friction management program and research focused on incorporating friction into safety analysis. Finally, the friction demand section outlines the national and international skid resistance polices and stresses on the friction demand models experience and lastly outlines the friction demand practice in Iowa and the need for additional research in this area.

Chapter 3: Methodology – This chapter discusses the proposed skid framework for quantifying the relationship between crash risks and skid resistance. As a part of the framework, the chapter first describes the data sources used in this thesis. It provides an overview of the datasets, highlights the interesting features of the data acquired for the purpose of this study, and describes the data integration methods and processes as well as assigns terminology for the research analysis categories. Next, the theoretical and statistical methods used for the purpose of this study are described thoroughly and a sample of the analysis is provided.

Chapter 4: Results and Discussion – This chapter presents the results of a series of statistical regression models developed over the course of this study. These results are accompanied by a discussion as to the practical implications of the findings, as well as a discussion of potential drawbacks and limitations.

Chapter 5: Conclusion – This chapter outlines a summary of the research findings along with a discussion on how these findings address the research questions. Findings are followed by general conclusions of this research study as well as its limitations. Finally, the chapter outlines potential directions for future research.

CHAPTER 2. LITERATURE REVIEW

In this chapter, a comprehensive literature review about tire pavement friction concepts, components, measuring techniques, pavement friction management as well as friction demand concepts and practices will be presented. Furthermore, the limitations of the current pavement friction management practices in the U.S. will be highlighted with an emphasis on the role of friction demand category-based skid resistance policies to overcome these limitations. Moreover, past efforts to research and implement friction demand levels in the U.S. and the international arena research will be reviewed.

Tire-Pavement Friction and Roadway Safety

Tire-Pavement friction is key to the safety of all traveling vehicles on the roadway network. Although the interaction of different factors causes most highway crashes, research has consistently demonstrated a link between collisions and pavement friction. Therefore, the friction between tire and pavement is a critical factor in reducing crashes (Hall et al., 2009; Henry 2000; Ivey et al., 1992).

Since the 1960s, considerable research has been conducted on pavement friction and its effect on traffic safety. Researchers in the U.S. and internationally have developed models to evaluate the association between friction and crash occurrence. Correlations between the pavement surface friction and the occurrence of crashes have been detected ever since (Al Hassan et al., 2018; Bray 2002; Davies et al., 2005; Flintsh et al., 2012; Giles et al., 1962; Kuttesh 2004; Murad et al., 2007; McCullough and Hankins, 1966; Najafi et al., 2015; Noyce et al., 2007; Rizenbergs et al., 1972; Schulze et al., 1976; Smith et al., 2012; Wallman and Astrom, 2001; Xiao et al., 2000). These correlations confirm that friction between tire and pavement is a critical factor in reducing crashes (Hall et al., 2009; Henry, 2000; Ivey 1992). Moreover, this profound relationship promotes friction as a critical performance measure of the roadway surface and the safety of the vehicle's operations over the network. Therefore, Pavement Friction Management (PFM) has received much attention from the decision-making authorities in the last decade.

There has been considerable research conducted on pavement friction and its effect on traffic safety. A detailed statistical study by (Gothie 1996) showed that not only that the crash rate increased when moving from a section with high friction supply to another with less friction supply, but also the risk and severity of crashes increased by approximately 50%.

An earlier comparison study by (Giles et al., 1967) revealed that the difference between the mean friction measurements for targeted sites with high crash rates and those of the randomly selected sites was considerably high and, thus, concluded that crashes are more likely to occur on pavements with a lower friction supply. Wallman et al. (2001) reported several earlier studies on this issue in a literature review where they all found correlations between the pavement surface friction and the occurrence of crashes (Giles et al., 1962; McCullough & Hankins, 1966; Rizenbergs et al., 1972; Schulze et al., 1976; Xiao et al., 2000). Many studies later developed different models to evaluate the association between friction and crash occurrence and demonstrated similar patterns (Al Hassan et al., 2018; Bray 2002; Davies et al., 2005; Flintsh et al., 2012; Kuttesh 2004; Murad et al., 2007; Najafi et al., 2015; Noyce et al., 2007; Smith et al. 2012; Wallman & Astrom 2001).

Basic Concepts of Tire-Pavement Friction and Pavement Texture

Tire-pavement friction is the force developed at the tire-pavement interface that resists the relative motion between a vehicle tire and a pavement surface (FHWA, 2010). This tire-pavement interaction prevents the tires from skidding (i.e., out of control sliding) on pavement surfaces under fast braking and cornering. Along with that, the tire-pavement friction is very crucial for highway safety as it keeps the vehicles on the road by allowing drivers to make safe maneuvers (Hall et al., 2009). The friction force between the tire and pavement is depicted by a dimensionless friction coefficient (μ), which is the ratio between the tangential force at the contact interface and the longitudinal load force on the wheel (Rizenbergs et al., 1986).

Figure 1 shows the tire while in contact with pavement and the forces generated when the tire is braking. The term "skid resistance (SN)" has been used in the literature to describe this tire-pavement interaction.

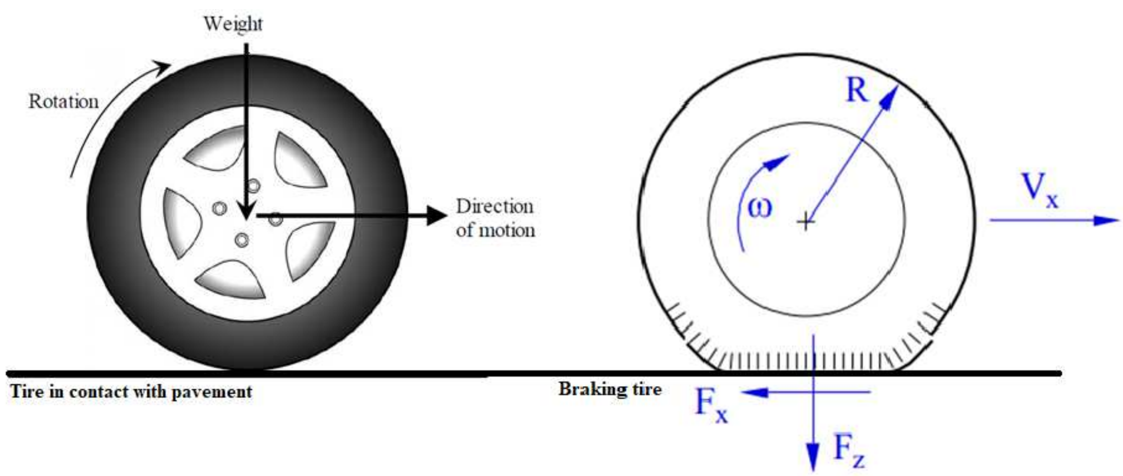


Figure 1: Tire Pavement Interface (Steve Karamihas, UMTRI; Hall et al., 2009)

The following equations show the fundamental physical relationship between the coefficient of friction (μ) and SN (Hall et al., 2009):

$$\mu = \frac{F_X}{F_z}$$

$$SN = \mu \times 100$$

Where:

F_X : The horizontal force applied to the test tire at the tire-pavement contact patch (Tractive Force).

F_z : The dynamic vertical load on the tire.

Pavement skid resistance is the retarding force generated when a tire that is prevented from rotating (i.e., locked-wheel) skids on a pavement surface (ASTM E 867, Highway Research Board, 1972). This definition refers to a situation in which the driver has attempted to decelerate the vehicle quickly and has locked the wheels in the process. A locked-wheel results from braking too hard at a high speed (Fu 2017). With a locked-wheel, the vehicle will skid no matter which way the steering wheel is turned. The skid resistance generated should prevent this skidding action.

Skid resistance on pavements depends primarily on the pavement texture. The AASHTO Guide defines pavement texture for pavement friction as "the deviations of the pavement surface from a true planar surface" (Hall et al., 2009, p.30). According to the Permanent International Association of Road Congress (PIARC) texture definitions, it is divided into four components: microtexture, macrotexture, mega texture, and roughness (Wambold et al., 1995). Based on the texture wavelength, the tire-pavement interaction is primarily related to micro-texture and macrotexture (Henry, 2000; see Figure 2). Macrotexture is seen with the bare eyes since it pertains to asperities greater than (0.02 inches) and up to about (0.20 inches) in size (Flintsch et al., 2003). It could be detected based on the openness of an A.C. surface, the jaggedness of a chip seal, tining, or grooving on a bridge deck (Schleppy, 2012). This texture depends on the size, shape, and spacing of the particles. Microtexture, on the other hand, is associated with a roughness on the surface of the aggregate less than (0.02 inches) in size (Flintsch et al., 2003). It is more easily felt than seen. It depends on the fine-scale texture of sand, aggregate particles, and cement paste as well as the degree of polish (Schleppi, 2012). Polishing is the wearing of the small surface particles of the aggregate under successive traffic loading (Ahammed, 2009).

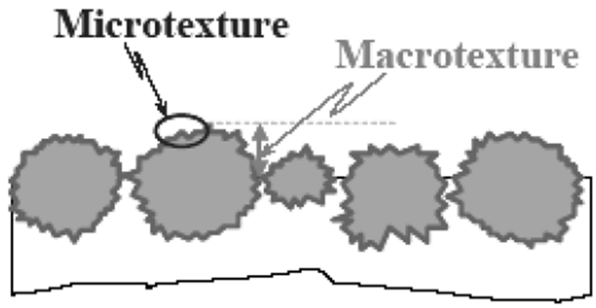


Figure 2: Macrotexture vs. Microtexture (Henry 2000)

At the micro-texture level; when the small-scale particles of the pavement meet the tire rubber, an adhesive friction component is generated. On the other hand, the larger particles of the pavement at the macrotexture level are what causes the deformation of the tire in the vicinity of the tire-pavement contact area. This deformation generates the hysteresis component of the friction force (Rizenbergs, 1968). The macrotexture also facilitates drainage of surface water and thus prevents hydroplaning (Rizenbergs, 1968). Hydroplaning occurs when the tires become separated from the surface and ride partially or entirely on a water layer, thus causing loss of traction (Hall et al., 2009; Rizenbergs 1968). The macrotexture defines how effective the micro-texture will be when the road is wet (Cook et al., 2010). Moreover, this interaction between the micro and macro textures is what is generating the two principle force components (adhesion and hysteresis) that interactively create tire-pavement friction, as illustrated in Figure 3. Adhesion, which is a molecular bonding, results from the shear frictional stresses at the tire-pavement interface. Hysteresis, on the other hand, is an energy loss. It results from the heat that is dissipated after the moving tire deforms against the mineral particles and relaxes (Rizenbergs, 1968).

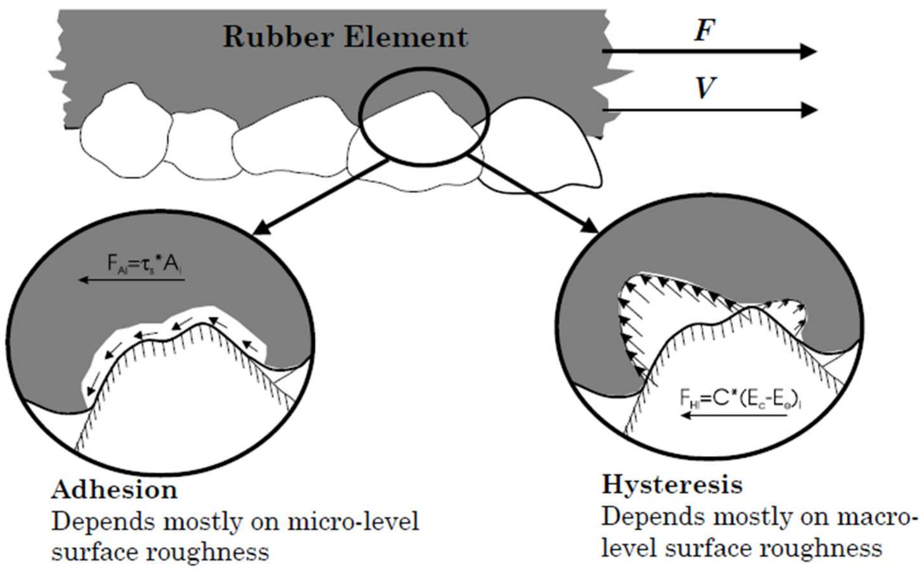


Figure 3: Tire-Pavement Friction (Byrd et al., 1981)

Factors Affecting the Tire-Pavement friction

In addition to the pavement surface characteristics, several factors affect pavement friction. According to Hall et al. (2009), these factors can be classified into four groups: pavement surface characteristics, driving maneuvers, tire properties, and environment (Hall et al., 2009).

Pavement Texture

The effect of pavement texture, which is a part of pavement skid resistance, has been investigated as early as in the seventies (e.g., Moore and Humphreys, 1972). Pavement texture was found to be a primary factor affecting skid resistance at speeds over 40 mph (Stroup-Gardiner et al., 2001). Furthermore, Pulugurtha (2012) assessed the effect of pavement macrotexture on Interstate I-40 Crashes in North Carolina. The results obtained from the research indicated that maintaining pavement macrotexture greater than or equal to 0.080 inches on tinned concrete pavement, and greater than or equal to 0.040 inches on asphalt pavement, would reduce crashes and enhance safety by improving braking performance (Pulugurtha, 2012).

Tire Properties

The earliest research report on the skid resistance of rubber tires on different road surfaces was published at the 1933 annual meeting of the Highway Research Board (Corsello, 1993; Moyer, 1933). The report elaborated on the many variables that influence tire-pavement friction. Some of the discussed properties are tire stiffness, inflation pressure, footprint, tread depth, load, and temperature. Although the features of the tire involved in the tire-pavement interaction are of great importance, this research will focus on the roadway side of the tire-pavement interaction.

Driving Maneuvers

Driving maneuvers are any changes in the vehicle's speed or direction. They include accelerating, decelerating, braking, and cornering. "The direction is forward in braking, backward in driving and accelerating, and sideways in cornering" (Rizenbergs, 1968, p.12). During free-rolling, the tire-pavement interface is instantaneously stationary, and the frictional supply is not fully utilized (Najafi et al., 2015). However, when a driver begins to maneuver, the previously discussed forces will develop at the interface in response to the specific maneuver activity. These forces are what enables the vehicle's driver to speed up, slow down, or track around a curve (Khasawneh et al., 2018).

Decelerating and Braking

In the process of braking and decelerating, Figure 4 demonstrates the tire-pavement friction versus slip (Henry, 2000). The tire slips while it transmits force to the pavement. Based on that, a slip results from the ratio of the slip velocity in a specific direction to the forward ground speed of the vehicle (Rizenbergs, 1968). At some point, while the tire is slipping on the road surface, the reacting force will keep increasing until it approaches a point at which the peak coefficient of friction available between the tire and the road is exceeded. If the friction between tire and road disappears (i.e., the friction demand exceeded the available friction), the motion of the wheel will not be a pure rolling motion, but it will be a rolling motion with slipping. This will eventually completely lock the wheel and turn into a skidding motion. The peak coefficient of friction is typically reached at between 18 to 30% slip (Rizenbergs et al., 1986). Most literature referred to a locked-wheel state as a 100% slip ratio and the free-rolling state as a zero-slip ratio (fully locked condition; Hall et al., 2009).

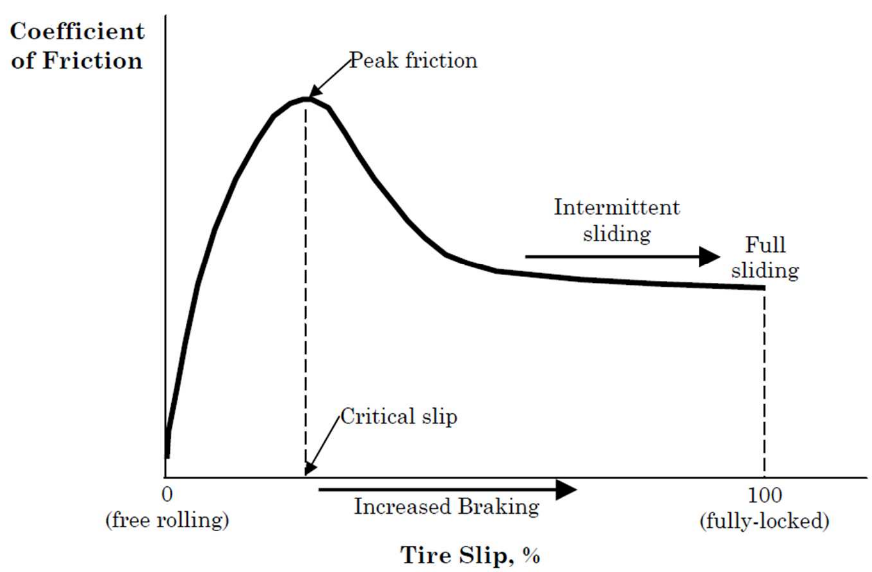


Figure 4: Tire-Pavement Friction Diagram (Henry, 2000)

It can be anticipated that exceeding the peak value can cause the vehicle to slide off the road. This range of friction (between demand and supply) is suggested as a design input parameter to improve road safety and to provide adequate driving conditions for vehicles

(Hayward et al., 1985; Lamm et al., 1991; Vaianna et al., 2017). Friction demand will be discussed in further detail throughout this document.

Accelerating

A high level of accelerating will be like braking. Exceeding the peak friction available will cause the wheel to start to slip, or in the extreme, to spin with little or no vehicle acceleration (Hall et al., 2009).

Cornering

Cornering generates side forces that allow the vehicle to follow a curved path. Curves usually demand higher friction that is due to the combination of speed and centrifugal force. Consequently, exceeding the friction supply at a curve might cause a rapid loss of control, causing the wheel to slip sideways (Najafi et al., 2015). In 1968, CALTRANS conducted Research to evaluate the effect of cornering maneuvers on friction-related crashes (Paige et al., 1968). The research concluded that curves have the highest crash rate, followed by weave sections and intersections, as one would expect, and that crash rates increase substantially for the same geometry type as the skid numbers decrease. More recently, Musey and Park (2016) performed a correlation analysis between pavement skid number, roadway curvature degree, crash rate, and crash severity. Their investigation revealed that wet crash rates were more profound at higher degrees of curvatures in conjunction with lower skid numbers.

Accelerating and Cornering

The worst case of a maneuver is a combination of Acceleration/Deceleration AND cornering. In this case, the available friction is shared by the two actions. Hence, exceeding the peak will cause the side-force to diminish and will lead to a complete loss of control on steering.

Speed

The correlation between wet pavement friction and vehicle speed has been recognized since the 1930s (Henry, 2000). Several models have described the relationship between friction and slip speed. Most of them conveyed that friction decreases as the slip speed of the tire increases (Antonio, 1976; Murad 2019; Van DE You 1995; Velds 1998; Yanase, 2014). A more recent study by Al Hassan et al. (2018) analyzed the effects of pavement conditions on traffic safety in a risk analysis scheme. In their research, Al Hassan et al. correlated pavement surface friction with roadway departure crash rates. The study concluded that higher friction values reduced the roadway departure crash rates. However, the relationship was more profound for segments with higher speed limits.

Environment

The characteristics of friction are susceptible to the environment. Seasonal variations, weather, and surface contamination have crucial impacts on the amount of friction available and the amount demanded.

Seasonal Variations and Weather

Available friction varies with the season. Bird and Scott (1931) first acknowledged a higher skidding resistance in winter and spring than in summer and fall. The available friction is relatively low in summer and fall, owing to contaminants (oil and dirt) deposition of surface particles. Rainfall cleans the contamination from the particles during the winter and exposes new particles to the surface (Jayawickrama and Thomas, 1998). Rain also washes any clogged debris in the pavement drainage channels. This flushing impact of rainfall is even thought to cause short-term changes in friction following significant rainfall events (Hill and Henry, 1982).

Researchers have made multiple attempts to quantify the impact of seasonal variation on the available friction supply (Bazlamit et al., 2005; Bianchini et al., 2011; FHWA, 2010; Flintsh, 2012; Flintsch et al., 2009; Hill and Henry 1981; Jayawickrama & Thomas, 1998). However, there is no practical model currently available for this purpose. Nevertheless, a lot of highway agencies seasonally correct the measured data for both within-year and between-year variations (Cook et al., 2011).

Surface Contamination (Wet, Dry, Dust)

Generally, there is often no difference between peak and sliding friction on a dry road surface in conjunction with a low-speed effect. In contrast, peak friction is usually lower on a wet road. Therefore, the main concern for road safety is the significantly reduced skid resistance during wet weather, when a water film is coating the pavement surface (Hall et al., 2009).

Moreover, wet pavement's skid resistance differs from the thickness of the water film on the surface. The strong dependency of friction supply on speed stems from the fact the efficiency with which the tire can expel water from that patch dictates the tire-pavement contact patch on wet pavements (Corsello, 1993). Such performance degrades at higher speeds owing to the water viscosity. The tire is, therefore, unable to maintain a dry contact patch. That is why most skidding problems occur when there are traction defects due to wetness on the road surface (Flintsch et al., 2012).

The literature confirms that the improved friction of the pavement will eliminate up to 70% of the wet pavement crashes (Henry, 2000). Several studies have quantified the impact of water film thickness on surface friction and proposed that reduced friction during wet weather would elevate the levels of vehicle crash (Kulakowski et al., 1990; Najafi 2015; Rose et al., 1997). Scientists have been able to prove that wet pavement crashes increase significantly as the friction of the pavement decreases (Al Hassan et al., 2018; Bray 2002; Giles et al., 1962; Griffin, 1984; McCullough and Hankins 1966; Murad et al., 2007; Najafi et al., 2015; Noyce et al., 2007; Rizenbergs et al., 1972; Schulze et al., 1976; Wallman and Astrom, 2001). For example, Griffin (1984) generated a multiple regression model of wet crashes in the U.S. and was able to detect a linear pattern in the correlation between crash occurrence and friction (i.e., reduction in friction was associated with a linear increase in crashes). Later, the Pennsylvania Transportation Institute (PTI) generated different models with different covariates to identify controlled relationships between wet crash rates and pavement friction. The models showed that the safety condition, measured by the percent reduction in wet pavement crashes, could be improved by nearly 60% if the skid number increased from 33.4 to 48 (Kuttesh 2004, Xiao et al., 2000). Moreover, Kuttesch (2004) developed a model to quantify the effect of friction on wet-weather crashes for the state of Virginia and concluded similar patterns. In addition to poor roadway conditions caused by wet pavements, there are also indications that dry pavements with inadequate friction can adversely affect the rate of roadway crashes (Hall et al., 2009; Najafi et al., 2015; Noyce et al., 2007; Smith et al., 2012).

Measuring Tire-Pavement Friction and Frequency of Testing

The pavement friction testing is considered a part of the asset management effort for each highway agency because of its importance in reducing crashes. Monitoring skid resistance has been in practice by many highway agencies both in the U.S. and in other countries (Long, 2013). Of transportation agencies, 55.4% collect friction measurements at the project level, and only 33.9% collect network-level friction data (Najafi, 2015). In addition, the roads with the highest traffic volumes and the highest likelihood of changes in friction over time require the most frequent monitoring of friction (Flintsch et al., 2009).

To collect network-level friction data, highway agencies have utilized friction testing equipment that offers high daily output. Some of the equipment measures the macrotexture of the pavement, and other equipment measures the microtexture. In the U.S., state DOTs have adopted different practices for skid resistance properties monitoring. The NCHRP 291 "Synthesis of Highway Practice" (Henry, 2000) and the NCHRP project 01-43 "guide for pavement friction" (Hall et al., 2009) summarized these practices. The classifications of the skid resistance measurement methods and equipment discussed in these reports are illustrated in Figure 5. Also, the statewide practices of friction testing in the U.S. is demonstrated spatially in Figure 6 with each testing method accompanied by the number of states adopting it. The map shows that three states, Kansas, Minnesota and Delaware, did not practice friction testing, instead, the DOTs at those states practiced common friction design restrictions. Additionally, reviewing the skid resistance measurement approaches statewide reveals that the locked-wheel trailers are the most predominant friction testing device used by state DOTs. Skid wheel test is not a direct measure of either micro texture or macrotexture but a response to both. The results of a locked-wheel test conducted under ASTM E-524 and E-501 specifications are reported as a skid number. Moreover, in this test and based on the ASTM standard, the vehicle should be brought to the desired testing speed of 40 mph to simulate the braking maneuvers in vehicles not equipped with anti-lock brakes (Najafi et al., 2015).

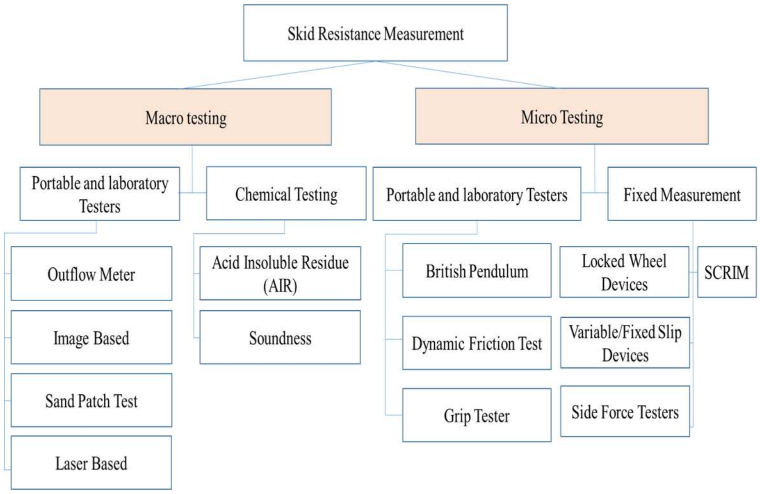


Figure 5: A Classification of Skid Resistance Measurement techniques (Hall et al., 2009; Henry 2000)

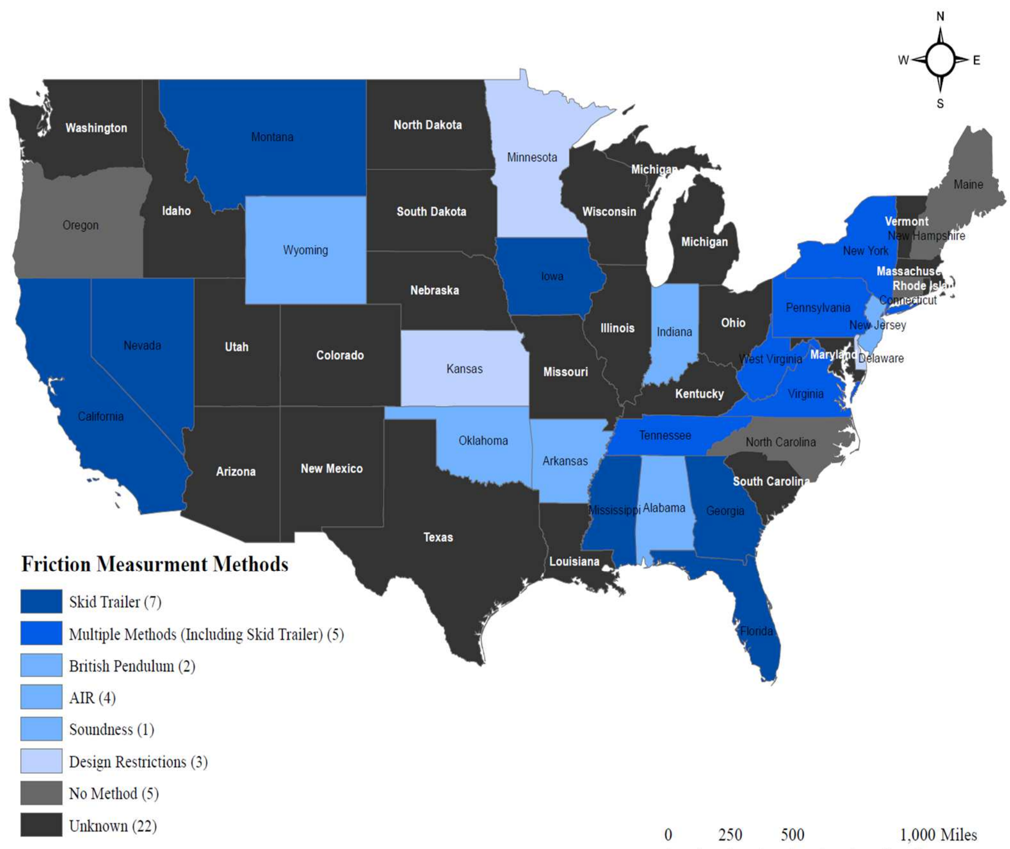
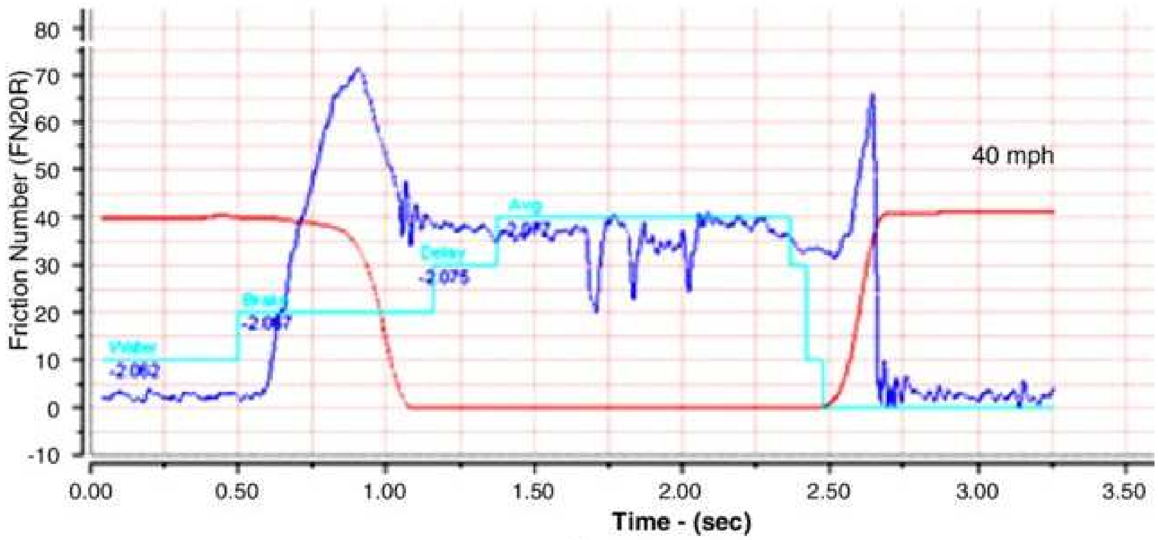


Figure 6: Statewide Pavement Skid Resistance Measurement Practices (Henry 2000, Hall et al., 2009)

Furthermore, two types of test tires can be used during the locked-wheel test: smooth (ASTM E-524) and ribbed (ASTM E-501). Accordingly, the terms SN40R and SN40S has been used in the literature to describe the measured friction at 40 mph using a ripped or a smooth tire, respectively. Regardless of the test tire used, every test pickup truck is supplied with a water tank, a water pump, and a computer system to control the testing and record the measurements. An advantage of the locked-wheel trailer is that the test variables are easy to understand and control (Henry, 2000). However, this test does not record continuous measurements along the test section.

The test cycle lasts approximately 2.5 seconds. Water is dispensed onto the pavement immediately ahead of the tire on the trailer to create an artificially wetted pavement (ASTM E274). Afterward, the trailer braking system is actuated to lock the test wheel, and the device starts recording measurements for tractive force. This horizontal force is applied to the test tire at the tire-pavement contact patch as well as at the vertical load of the vehicle for 1 second while the wheel is locked (Al Hassan et al., 2018). Figure 7 presents a sample of a locked-wheel test reading (per ASTM E-274 and AASHTO T-242). This sample test is performed at 40 mph. The blue line (i.e., SN) is almost zero when the tire is free rolling. Braking, the SN increases to reach the peak, right before regretting back to a fully locked SN, where the red line (i.e., rolling speed) reaches zero.



(c)

Figure 7: Locked-wheel test sample (Holzschuher et al., 2010)

Frequency of Friction Testing

Many agencies monitor the friction of their higher friction demand facilities on an annual basis, while a 2 to 3-year cycle may be appropriate for the parts of network with lower-risk (FHWA 2010). At the heart of this research is pavement friction management practices, especially for friction demand. Monitoring skid resistance on the network with the appropriate measuring equipment is an integral part of a typical PFM system. This document will describe pavement friction management practices in more detail in the following sections.

Pavement Friction Management Program

As early as 1976, "Guidelines for Skid-Resistant Pavement Design," published by AASHTO, recommended a list of specifications to produce the desired frictional properties in the newly designed pavement sections (AASHTO, 1976). The earlier National Highway Safety Act of 1966 required states to monitor the skid resistance of public highways and streets. Furthermore, FHWA Technical Advisory 5140.10, "Texturing and Skid Resistance of Concrete

Pavements and Bridge Decks," was published in 1979 right before the FHWA's Technical Advisory "Skid Accident Reduction Program" (T) 5040.17. Published in 1980, (T) 5040.17 states that "the State's program shall provide that there are standards for pavement design and construction with specific provision for high skid resistant qualities" (FHWA, 1980). However, almost a decade afterward, Jayawickrama et al. (1996) conducted a nationwide survey to record the state agencies' practices for controlling friction on hot mixed asphalt (HMA) pavements. At that time, almost 50 % of the state highway agencies did not have any design guidelines that specifically address friction. Further, in the NCHRP Synthesis 291, "Evaluation of Pavement Friction Characteristics," a questionnaire was sent out to all U.S. states and other countries to report the current practices of frictional characteristics evaluation used by highway agencies (Henry, 2000). At that time, the responses revealed that only 12 states have either suggested or established a requirement for minimum acceptable skid resistance level (Henry, 2000). These states are highlighted in Figure 8, which summarizes the statewide DOTs friction requirements at the time the questionnaire was administered (Henry, 2000).

While all the reported state agencies remained around a friction threshold of SN40R/SN40S of 30 to 35. Illinois and South Carolina each had very conservative values of SN40R of 45 and 41, respectively. On the other hand, Kentucky was practicing a minimum SN40R of 28, which is much lower than the practices of the other states. Furthermore, the remaining 75% of the states had no clear skid resistance control policy. However, "Based on the experience from many states, it is generally agreed that a skid number of *35* or greater gives adequate skid resistance under most conditions" (Wambolt et al., 1986). This examination of the statewide standards raised a lot of questions and expanded concerns concerning the safety of pavement surfaces.

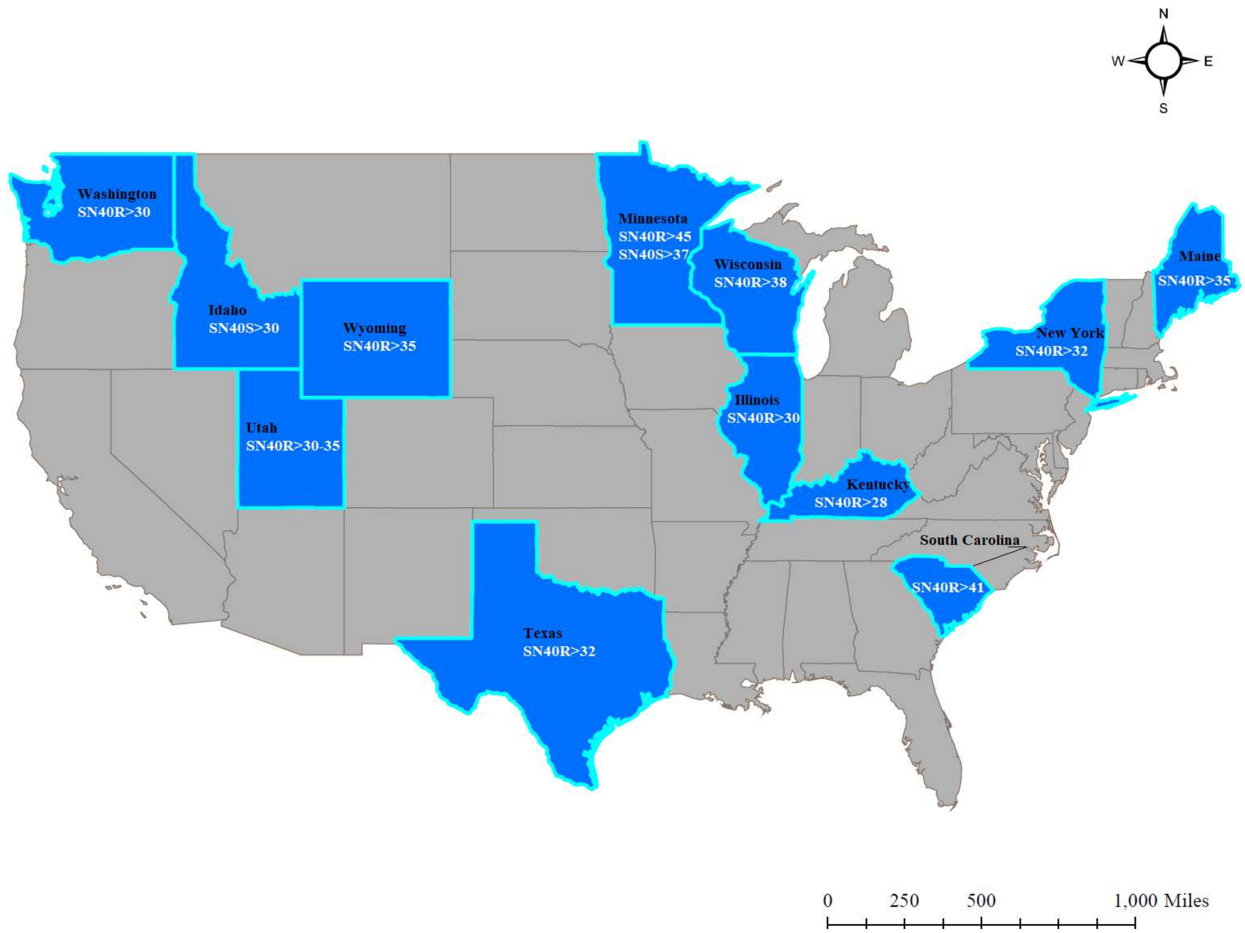


Figure 8: Statewide DOTs Friction Requirements (Hall et al., 2009, Henry, 2000)

Later, recognizing the urgent need to apply pavement friction management (PFM) concepts and technologies in the United States, FHWA issued several mandates superseding those published earlier on the surface and friction management of pavements. In addition to the “Surface Texture for Asphalt and Concrete” Technical Advisory T5040.36 -2005 that issued guidance on selecting techniques that provide adequate wet pavement friction and low tire/surface noise characteristics.

The FHWA Technical Advisory TA 5040.38 “Pavement Friction management” provided guidelines for providing adequate surface texture and friction (FHWA, 2010). It recommended that each state DOT develop a Pavement Friction Management Programs (PFMP) within the advisory guidelines to reduce the risk of fatal and injury-causing crashes and correct friction deficiencies (FHWA, 2010). The concept of PMFP was discussed earlier, in the 2009 AASHTO "Guide for Pavement Friction." That guide provided an approach for managing pavement friction and a process for implementing it and it serves as the model for future PFM programs (Hall et al., 2009).

The FHWA further stated that the primary purpose of a PFM program is to minimize friction-related vehicle crashes (FHWA, 2010). That can be achieved using a systematic approach to (a) measuring and monitoring the friction supply on the roadway network and (b) identifying the pavement surfaces that are or will soon require treatment. Finally, planning and budgeting for treatment and reconstruction activities to ensure adequate friction supply are imperative (AASHTO, 2008). The FHWA has sponsored a major, multi-year study to develop and demonstrate PFM programs at four state highway agencies using the best practices contained in their "Guide for Pavement Friction."

The role of a Pavement Friction Management Programs (PFMP) or policy is to provide a framework by which road engineers can follow appropriate pavement design, construction, monitoring, and evidence-based maintenance practices. Therefore, adopting a specific PFMP enables a highway agency to ensure its pavement surfaces are providing adequate skid resistance. Adequate skid resistance levels enable vehicles to reduce speeds more rapidly or allowing control to be retained for longer (Flintsh et al., 2018). PMFP requires balancing the risk of crashes with the cost and practicality of providing sufficient friction. Even though crashes will likely never be eliminated, a powerful PFMP can minimize the risk of friction-related crashes and reduce their severity when they do occur.

Single-Point Friction Thresholds

Minimum skid resistance guidelines have been the focus of intensive research efforts. Right after the 1980s, there was an upswing in acts and policies promoting the PMFP (Byrd et al., 1981; Corsello, 1993; FHWA 1980). Consequently, instead of using older specification methods based on either engineering judgment or experience, highway agencies started adopting a new practice of identifying rigid Single-Point skid resistance threshold as a maintenance trigger for the whole network (Hall et al., 2009). Any pavement section that has a skid resistance supply equal to or less than this threshold is marked as "unsafe" for wet weather driving. This movement towards data-driven threshold specifications was a significant step forward for PFMPs in the U.S.

McCullough and Hankins (1966) were among the pioneer researchers who handled the concept of the single-point friction threshold. They carried out a study to investigate the relationship between pavement friction and crashes from 571 sites in Texas. After performing a crash rate-friction history analysis and observing the point where the slope of crash rate versus friction decreased significantly. The researchers recommended a convenient minimum desirable skid number measured at 30 mph (SN30) of 40. One year later, Kummer and Meyer (1967) performed a silver stone study to determine frictional requirements for main rural highways. The study was sponsored by the Highway Research Board (HRB) and published as the National Cooperative Highway Research Program (NCHRP) Report 37. In this report, a minimum skid number (SN40R) of 37 was recommended for the national road network. However, the report elaborated on the importance of defining specific frictional requirements for different roadway conditions (NCHRP, 1967). Moreover, the HRB suggested continued research to develop a more refined skid resistance requirement (Corsello, 1993; Kummer and Meyer, 1967).

In Kentucky, Rizenbergs et al. (1972) analyzed crashes and friction on rural two-lane roadways using correlation analysis. Although they obtained low correlation coefficients (i.e., less than 0.430). Their results suggested that if the data are grouped by averaging the SN, a threshold approximately equal to 40 can be found. That threshold reflects the threshold value of 40 to manage skid resistance on Virginia Interstate highways suggested by Kuttesch (2004).

Moreover, Chelliah et al. (2003) performed another analysis in Maryland. A significant part of this research was the development of empirical models for various AADT ranges of and all wet crash data to predict wet pavement crashes from the friction number. The results ranged from 35 to 60 for each AADT range. More recently, Long et al. (2014) proposed thresholds for skid resistance to achieve a target crash reduction in Texas. They also tested their threshold values for benefit/cost convenience. Table 1 illustrates their benefit/cost analysis. The table shows that higher skid resistance thresholds will result in a significantly lower cost-benefit ratio. This result suggests that when an agency adopts a relatively conservative single-point minimum threshold level, it will start providing friction supply that is higher than the friction demand. At that point, the cost/benefit ratio starts decreasing, which means that the benefit of improving the pavement skid resistance is diminishing. It continues to decline until the ratio reaches a value of 1, where any improvement would yield no advantages in terms of crash reduction (Long et al., 2014).

Table 1: Benefit/Cost Ratios of SN Thresholds for Texas Crashes (Long et al., 2014)

Skid Number Thresholds	**Benefit/Cost Ratio**
14	39.64
28	20.04
73	1.00
74	0.99

This less conservative threshold was confirmed by Musey and Park (2016), who showed that at around 55, an increase in skid number no longer resulted in decreased crash rates. The cost/benefit impracticality resulting from a single-point skid resistance analysis is not the only concern associated with this analysis. The single-point skid resistance threshold analysis has two other main limitations. The first issue arises from a statistical analysis of historical records of skid resistance and crash data. The overall crash-friction correlations do not provide information necessary to identify the factors or causes that have important implications in the setting of the threshold. Therefore, such analysis is biased if not categorized by the factors that affect friction demand. Second, providing a single-point friction level that defines the threshold between “safe” and “potentially unsafe” places additional legal responsibilities on the state highway agencies. The case law indicates that states can be found liable for low friction conditions due to either their actions or lack of action” (Carlson, 1974).

Therefore, this concept of single-point minimum threshold values as mandatory standards for pavement friction faced strong opposition from the transportation agencies (Kettush, 2004). These two significant limitations of the concept of a single-point skid resistance threshold can be overcome by employing a two-parameter, two-level skid resistance model (Fwa, 2017). In such a model, factors such as roadway geometry (i.e., curves, grades), roadway functional classification, traffic volume, speed, and potential for conflicting movements (i.e., intersections) are considered. Applying this model would be useful for determining friction demand levels instead of setting a single point friction threshold for the whole network.

Friction Demand

In literature, there seems to be no general definition of friction demand, but it is understood to be the level of skid resistance that needs to be supplied at the tire-pavement interface to safely perform driving maneuvers (Najafi et al., 2015). Therefore, skid resistance can be thought of in terms of friction management as "the margin of safety" between the friction supply available and the friction demand generated at any particular time and for a specific driving maneuver (Corsello, 1993). The basic equation for the skid resistance is as follows:

$$Skid\ resistance = Friction\ supply - friction\ demand$$

Here, when the friction demand exceeds the available friction supply at the tire-pavement interface, skidding occurs (Kennedy et al., 1990). Although the perfect circumstance is to have friction "supply" meet or surpass friction "demand" over the whole network, such practice would be restrictively expensive and superfluous (Hall et al., 2009). Therefore, factors such as roadway geometry (i.e., curves, grades), roadway functional classification, traffic volume, speed, and potential for conflicting movements (i.e., intersections) should be considered for determining friction demand levels. Curves and intersections also tend to lose friction at a faster rate than other roadway locations, which explains their higher friction demand (Flintsh et al., 2018).

Factors that affect the friction Supply/Demand equation on the tire-pavement interface were summarized by (Byrd et al., 1984). These factors are outlined along with the friction demand/supply diagram of a moving vehicle shown in Figure 9. There, friction demand is increasing while the car is moving, and this increase is affected by different factors. On the other hand, the friction supply is diminishing at a rate that is affected by multiple factors. Some factors affect both demand and supply. In other words, "Friction demand is site specific. It's not one size fits all" (Schelppi, 2016).

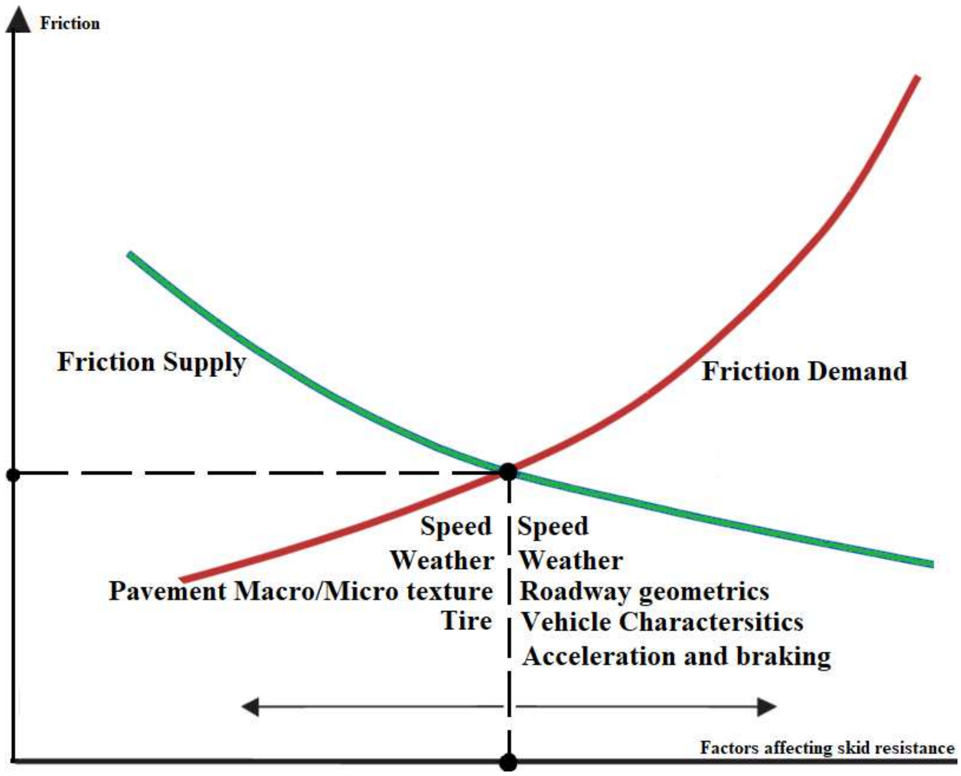


Figure 9: Factors Affecting Friction Demand and Supply (Byrd et al., 1984)

Accordingly, roadway networks should be stratified based on the factors that affect the friction demand and supply before an agency can follow on to the friction demand assignment. There are two distinctive types of friction demand levels. These levels, which serve as pavement friction management guidelines for highway agencies, are the investigatory level I.L. and the intervention level (Hall et al., 2009). The investigatory level is the desirable skid resistance level. Any increase in the friction supply above this level results in no crash reduction or cost-benefit. However, any friction supply below this level call the agency for monitoring the pavement skid resistance and crash level and start planning for maintenance or restoration actions. Whereas the intervention level is the minimum skid resistance level at which an agency must take immediate corrective action, such as a maintenance or surface treatment. The intervention level is “the skid resistance level below which the driving safety risk becomes unacceptable” (Fwa, 2017). Along these lines, a highway agency can assign different investigatory and intervention levels for each roadway category to normalize and minimize the risk of skid-related crashes through the network (Smith et al., 2012).

The AASHTO "Guide for Pavement Friction" has defined three methods to establish investigatory and intervention levels. The first method plots the friction loss versus pavement age. The friction value that encounters the first significant loss is selected as investigatory level, as shown in Figure 10. The intervention level can be defined at a fixed percentage below the investigatory level (AASHTO, 2008; Hall et al., 2009). The second method utilizes both friction deterioration curves and historical crash data. The investigatory level is set where there is a significant drop in the friction level. The intervention level is set where there is a substantial increase in crashes, as shown in Figure 11 (AASHTO, 2008; Hall et al., 2009).

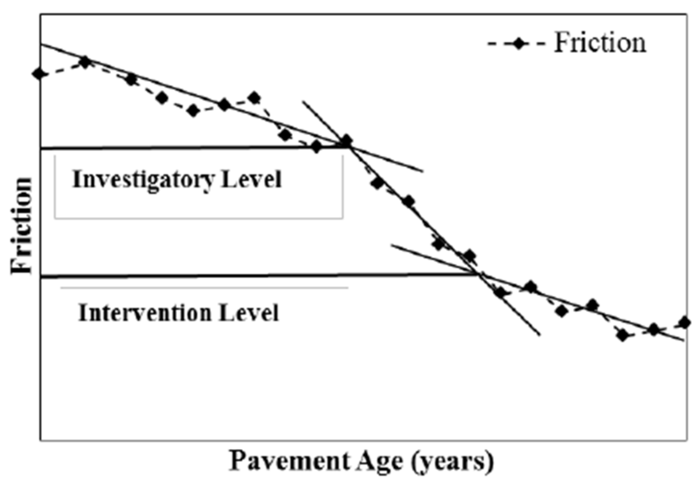


Figure 10: Method 1 – Friction Deterioration Curve (AASHTO, 2008; Hall et al., 2009)

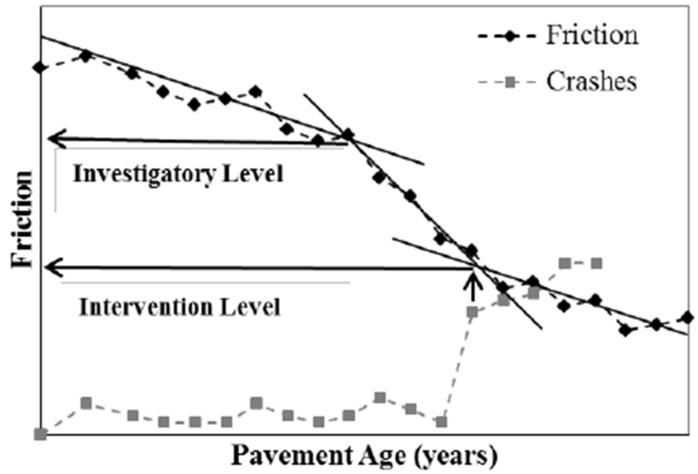


Figure 11: Method 2 – Friction Deterioration and Crash Rate (AASHTO, 2008; Hall et al., 2009)

Finally, the third method uses the friction distribution and crash rate to determine the investigatory level (I.L.) and the intervention level of friction. This method involves plotting a histogram of the pavement friction and wet-to-dry crash ratio, as shown in Figure 12, then calculating the mean and standard deviation of the friction distribution. Afterward, the investigatory level is set at the mean friction level minus a standard deviation and adjusted to the point where wet-to-dry crashes begin to increase rapidly. The intervention level, on the other hand, is set at the mean friction level minus a standard deviation and adjusted to minimum satisfactory wet-to-dry crash rate, or to address friction deficiencies where enough funding is available (AASHTO, 2008; Hall et al., 2009).

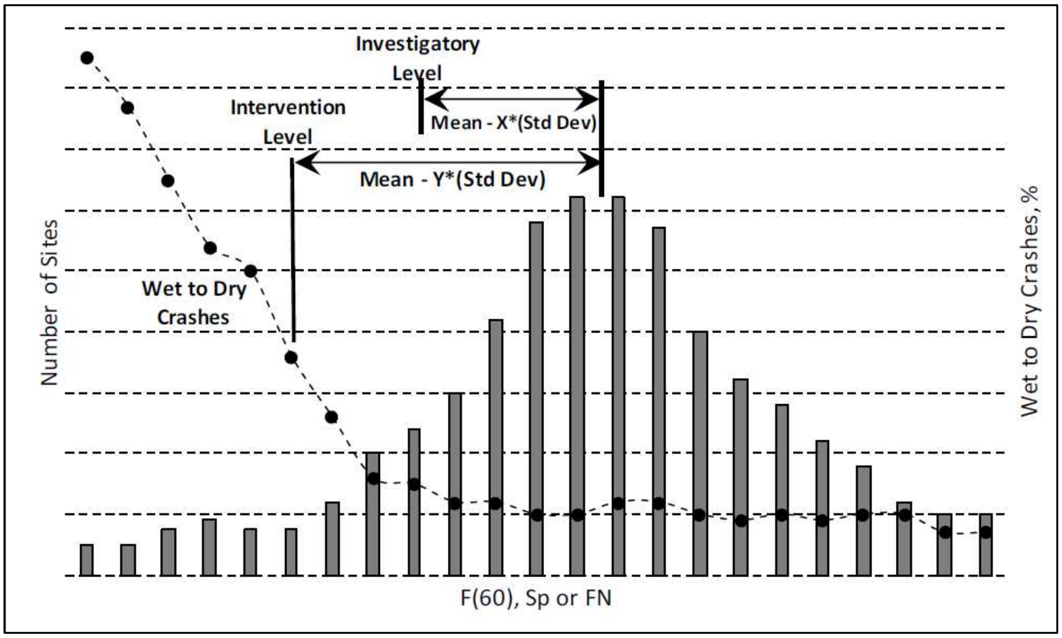


Figure 12: Method 3 – Friction Distribution and Wet-to-Dry Crash Ratio (AASHTO, 2008; Hall et al., 2009)

Many highway agencies outside of the United States have practiced managing skid resistance based on friction demand. Many of their studies have revealed that a skid resistance policy for appropriate skid resistance at various locations on the network results in crash reductions and is very effective when used reactively. The following section reviews some of these practices.

International Friction Demand Efforts

The international arena, particularly Great Britain, Italy, New Zealand, and Australia, use the Sideway-Force Coefficient Routine Investigation Machine (SCRIM) for friction testing, which is the most recognized friction test internationally. The SCRIM device provides continuous friction readings. It is supplied with a larger water tank compared to locked-wheel trailers to enable it to test a longer length of 125 miles of the roadway network (Fwa, 2005). The SCRIM records measurements of Mean Summer Scrim Coefficient (MSSC) AND Side Force

Coefficient (SFC). Hence, to be able to interpret a proper comparison between the United States and the international practice, it is crucial to elaborate on the conversion between a SCRIM measurement to a U.S. locked-wheel test measurement SN40R/SN40S. The SCRIM must be converted from SFC to a braking force coefficient (BFC) using the equation (Corsello, 1993):

$$BFC = SFC \times 0.08$$

The SCRIM measures SFC at 30 mph, which is 10 mph less than the locked-wheel trailer speed. The BFC should be converted to an SN40 using the equation (Corsello, 1993):

$$SN40 = (BFC \times 100) + 0.2\frac{SN}{mph}(30 - 40)$$

One of the first countries to establish a skid resistance policy with friction threshold levels was Australia. Australia has been following a skid resistance management policy since as early as in 1982 (Sinhal, 2005). In their policy, Austroads, the Australian/New Zealand transportation authority, provided investigatory and intervention levels for different roadway categories. However, because of the significant variations in traffic levels in Australia, Austroads divided the country into three generic zones based on the traffic-based friction demand and then incorporated that into a plan for zone testing frequency. Moreover, the most recent revised friction levels after recommending a minimum level of testing based on defined generic zones are presented in Figure 13. The revised standards were incorporated in Austroads policy as early as 2003 (Neaylon et al., 2011). The default I.L.s are the black areas. For example, the investigatory level for maneuver free divided roads (i.e., category 5) is 0.35 SFC at 30 mph, which is equal to SNR40 of 26 and lower than the minimum threshold practice recommended in the United States.

Moreover, in the United Kingdom, Highways England started establishing I.L. ranges for various types of roadway categories and geometric conditions in 1988 (Corsello, 1993). The agency measures friction using the parameter (CSC); Characteristic Skid Coefficient, a representative roadway surface friction value measured using the SCRIM and adjusted to consider fluctuations due to weather and seasonal effects throughout the year (Highways England, 2015). Their most recent skid resistance levels are summarized in Figure 14. The default I.L.s are the darker grey areas. For example, the investigatory level for the non-event carriageway with one –way traffic (i.e., category B) is 0.3 CSC at 30 mph (i.e., SNR40 of 26), which is the same as the practice in Australia.

Site Category	Site Description	Investigatory levels of SFC50 AT 50 KM/H or equivalent						
		0.30	0.35	0.40	0.45	0.50	0.55	0.60
		Corresponding risk ratings						
		1	2	3	4	5	6	7
1 (see notes)	Traffic light controlled intersections Pedestrian/school crossings Railway level crossings Roundabout approaches	Investigation Advised						
2	Curves with tight radius <= 250 m Gradients >= 5% and >= 50 m long Freeway/highway on/off ramps							
3 (see notes)	Intersections							
4	Manoeuvre-free areas of undivided roads							
5	Manoeuvre-free areas of divided roads							
Site Category	Site Description	Investigatory levels of SFC20 AT 50 KM/H or equivalent						
		0.30	0.35	0.40	0.45	0.50	0.55	0.60
		Corresponding risk ratings						
		1	2	3	4	5	6	7
6	Curves with radius <=100 m	Investigation Advised						
7	Roundabouts							

Key to thresholds at or below which investigation is advised	
	All primary roads, and for secondary roads with more than 2,500 vehicles per lane per day
	Roads with less than 2,500 vehicles per lane per day

Figure 13: Site-specific and Investigatory Levels in Australia (Sinhal, 2005)

Site Category and definition		IL for CSC data (Skid data speed corrected to 50km/h and seasonally corrected)							
		0.3	0.35	0.4	0.45	0.5	0.55	0.6	0.65
A	Motorway								
B	Non-event carriageway with one-way traffic								
C	Non-event carriageway with two-way traffic								
Q	Approaches to and across minor and major junctions, approaches to roundabouts and traffic signals (see note 5)								
K	Approaches to pedestrian crossings and other high risk situations (see note 5)								
R	Roundabout								
G1	Gradient 5-10% longer than 50m (see note 6)								
G2	Gradient > 10% longer than 50m (see note 6)								
S1	Bend radius <500m - carriageway with one-way traffic (see note 7)								
S2	Bend radius <500m - carriageway with two-way traffic (see note 7)								

Figure 14: Site categories and Investigatory Levels in the U.K. (Highways England, 2015)

New Zealand has implemented a skid resistance policy since 2004. All transportation agencies there are required to follow the policy to receive maintenance funding as part of the National Land Transport Program (NLTP), which is administered by the New Zealand Transportation Authority (NZTA; Cook et al., 2011). Their most recent skid resistance level specifications were published in the 2013 T10. Some of their results are summarized in Figure 15. The default I.L.s are the black areas. Although the site categorization is different from that of the U.K, the investigatory level for event-free divided carriageways (i.e., category 4) is 0.40 ESC at 30 mph (i.e., SNR40 of 30), which is more conservative than the U.K. and the Australian policies.

Site category	Skid site description	Investigatory level (IL), units ESC					
		0.35	0.40	0.45	0.50	0.55	0.60
1	Approaches to: a) Railway level crossings b) Traffic signals c) Pedestrian crossings d) Stop and Give Way controlled intersections (where state highway traffic is required to stop or give way) e) Roundabouts. One lane bridges: a) Approaches and bridge deck.						
2	a) Urban curves <250m radius						
	b) Rural curves <250m radius			L	M	H	
	c) Rural curves 250-400m radius		L	L	M	H	
	a) Down gradients >10%. b) On ramps with ramp metering.						
3	a) State highway approach to a local road junction. b) Down gradients 5-10% c) Motorway junction area including on/off Ramps d) Roundabouts, circular section only.						
4	Undivided carriageways (event-free).						
5	Divided carriageways (event-free).						

Figure 15: Site Categories and Investigatory Levels in New Zealand (T10, 2013)

The effectiveness of T10 specification was examined by (Davies, 2002) by performing a year-by-year comparisons of crash rate on New Zealand roadway network using data from 1998-2008. This period ranged from the evolution of skid resistance policies to the introduction of the robust T10 specifications for skid resistance, which allowed researchers to begin investigating both the initial and further effects of the policy on crash rates (Cook, 2011; Davies, 2002). The study revealed that the NZTA skid resistance policy has resulted in a significant reduction in the rural state highway network wet crash rate (Cook et al., 2011). After getting a cost-benefit ratio range of 13 to 35, the researcher elaborated: “While the policy has significant costs, we are confident that the policy is very effective and efficient” (p.15). These results confirm the benefit of targeted skid resistance investigations and interventions.

In Italy, Crisman and Roberti (2012) reviewed the levels of skid resistance for an existing road and were able to quantify the correlations between the friction number, road geometry, and speed. Consequently, the researchers we able to define the demand for traction at different sites. More recently, in Singapore, the Department of Civil and Environmental Engineering developed a mechanistically derived three-dimensional finite-element skid resistance simulation model to predict skid resistance. They presented an application of the proposed approach and the skid resistance prediction procedure in defining and predicting pavement friction demand (Fwa, 2017). The researchers promoted this analytical tool to enable pavement engineers to manage the skid resistance performance of pavements more effectively in a road network (Fwa, 2017).

Friction demand Efforts in the U.S.

The United States might not have as elaborate skid resistance policies as the ones that has already been implemented internationally. Very little research in the U.S. has addressed the different friction demand categories that can be integrated into an effective skid resistance policy (Long et al., 2014; Musick et al., 2019; Najafi et al., 2015; Speir et al., 2009). However, much research has been devoted to improving PFM policies nationwide. Furthermore, some state agencies started or have already implemented data-driven friction demand policies. Two decades after the national survey that was conducted in 1999 by the NCHRP (Henry, 2000), it is time to conduct a new survey that helps to formulate state-of-the-art pavement friction management practices. One of the first studies that moved from the single-point friction threshold analysis to a multi-level analysis was conducted by Cairney (1997), who stratified a roadway network into categories of functional class and traffic volumes and performed a statistical comparison between 120 selected sites with high skid-related crashes and another 100 random sites on the roadway network. Afterward, he computed the relative risk of a site being a skid-related crash site by dividing the number of skid-related crash sites by the number of control sites for different pavement friction categories. The risk of a skid-related crash was very low, at friction values of 60, suggesting SN40R of 60 as the I.L. Then started increasing profoundly for friction values below 50, indicating SN40R of 50 as the Intervention level for those sites (Cairney, 1997; Hall et al., 2009).

Later, researchers at the Pennsylvania Transportation Institute (PTI) developed two fuzzy logic models to identify the improvements in safety expected from improvements in different variables. Their analysis included multiple covariates suspected to have the most effect on the risk of skidding crashes at a site, such as posted speed, average daily traffic (ADT), pavement wet time, and driving difficulty. The models showed that safety conditions, measured by the percent reduction in wet pavement crashes, could be improved by nearly 60% if the skid number increased from 33.4 to 48 (Xiao et al., 2000). With the expansion of the research in this area, different states started implementing various prevention programs. Among these programs are the Wet Accident Reduction Programs (WARP) by Virginia and Texas DOTs (TxDOT, 2006; VDOT, 2006). The TxDOT WARP provided a framework for determining the pavement friction on existing roadways with a high number of wet crashes and used these values to determine the required resistance on surfaces of new pavement for various friction categories based on several aspects. Subsequently, the program did not propose I.L. s for the different friction categories. Instead, the TxDOT incorporated this friction information towards selecting the optimum roadway surface aggregate for new pavement design (TxDOT, 2006). However, an upswing in the PFM practices was inspired by the Maryland State Highway Administration in 2009. The agency sponsored a research report to help develop friction demand guidelines and policies along Maryland’s roadway network. The researchers defined friction demand categories along the network, with associated friction threshold levels for SN40 (see Table 2; Speir et al., 2009). The value of SN40R of 30 for the I.L. of divided highways with no constraints was similar to the I.L. implemented in New Zealand for the same roadway category.

Table 2: Site-Specific Friction Levels for Maryland Roadway Network (Speir et al., 2009)

Site Category	Site Description	Threshold SN	Investigatory SN	Intervention SN	Demand Category
1	Approach railroad crossings, traffic lights, pedestrian crossings, Stop and Give Way controlled intersections (SH only).	55	90	45	High
2	Curves with radius =<250m, downhill gradients > 10% and > 50 m long. Freeway/highway on/off ramp.	45	40	35	High
3	Approach to intersections, downhill gradients 5 to 10%.	45	40	35	High
4	Undivided Highways without other geometric constraints which influences frictional demand.	40	35	30	Low
5	Divided highways without any other geometrical constraints which influences frictional demand.	35	30	25	Low

Later on, Najafi et al. (2015) linked roadway wet and dry crashes with the tire-pavement friction by generating regression models for different types of urban roads on the New Jersey highway network. In their conclusions, the researchers focused on the fact that the correlations between friction and crash rates are not only limited to wet pavement crashes, as the correlations are also profound between friction and dry pavement crashes. Their developed regression models are recommended for defining the friction demand levels of different road categories.

Moreover, a very recent research in Virginia established an engineering-based procedure (Musick, 2019). In his Ph.D. dissertation, Musick proposed a framework for the West Virginia Division of Highways (WVDOH) to perform network roadway surface friction testing then use that information to define friction demand and determine the I.L.s of roadway surface friction for various route categories. The study found that for the United States Routes and the West Virginia routes, there is a relationship between crashes and friction, with lower crash rates as the roadway surface friction increases. Therefore, the friction I.L.s presented in Table 3 were determined for these two categories of routes. However, the researchers did not find a significant correlation for the interstates in Virginia and, thus, was unable to determine an I.L. for its interstates.

Table 3: Roadway category-specific investigatory levels in Virginia (Musick, 2019)

Route Category	Investigatory Level		Crash Risk
	GN	~ SN	crashes/100MVMT
Interstate	N/A	N/A	< 100
United States	0.30	46	270
West Virginia	0.24	41	220

Friction Demand Practices in Iowa

In Iowa, an early attempt by (Schram, 2011) was conducted to correlate between friction and wet/dry crash rates on Iowa interstates. Schram used Iowa DOT data to generate a model to define the I.L. of pavement friction on Interstates using the crash analysis and historical friction method proposed by The AASHTO "Guide for Pavement Friction" (Hall et al., 2009). Schram incorporated the results for friction requirements into aggregate frictional qualities specifications (Schram, 2011). Subsequently, the Iowa DOT updated its specifications for asphalt mixtures based on the findings of this research (Schram, 2011). Nevertheless, a site-specific correlation between crash data and skid resistance is not quantified yet in Iowa. These relationships are vital to determining the threshold levels for assisting Iowa DOT in the network monitoring process and prioritizing pavement maintenance activities. Therefore, this research aimed to determine quantitative relationships between skid resistance and crash occurrence on Iowa roadways while accounting for contributing factors such as roadway functional classification, geometry, traffic, speed, and some environmental conditions.

CHAPTER 3. METHODOLOGY

Overview

The main objective of this thesis is to establish a framework that quantifies the relationship between crash risks and pavement skid resistance to deliver a cost-effective, risk-based prioritized skid resistance program that can be tailored to the Iowa DOT budget and that will minimize the friction-related crashes on the roadway network. The framework will be aimed at employing a two-parameter, two-level skid resistance model, where factors like roadway geometry (i.e. tangents), roadway functional classification, traffic volume, speed and pavement type, are considered for determining the friction demand levels that will work as skid resistance maintenance trigger levels that are readily implement-able towards a consistent and pro-active PFMP to tackle Iowa's roadway network safety issues and ensuring efficient resource allocation.

In this chapter, the methodology used to achieve this objective is presented. The methodology proposes a multi-pronged network-level Pavement Friction Management Program by investigating the effect of friction on the rate of crashes on Iowa state-maintained roads. The effect of friction on the rate of wet condition and dry condition crashes as well as roadway departure (RwD) crashes is evaluated starting with the data acquisition and ending with determining the friction regression models and Investigatory Levels (I.L.s). After acquiring the data from multiple sources, the datasets are integrated into one dataset with records retaining category identifier, geometry, crash, friction number, and traffic data. Then, crash rate models are generated for each category and the relationship between crash rates and friction is identified. Next, the friction distribution along with the wet/dry crash and RwD/total ratios for each category are used to determine the I.L. of friction based on the AASHTO "Guide for Pavement Friction" methods to establish investigatory and intervention levels. Finally, an ANOVA regression analysis is performed to generate friction demand models for the different categories. The proposed methodology provides guidelines on the desired frictional requirements to develop a friction management program that can efficiently tolerate part of the road safety risk by enabling decision-makers to make proper maintenance decisions.

Framework

In the first part of this section, a conceptual framework has been developed to present the components of the entire process of quantifying the relationship between safety and roadway surface friction and defining the site-specific friction demand levels. Figure 16 summarizes the conceptual general steps of the proposed framework. In this conceptual framework, four steps were planned to achieve the research goals:

1. Collect and process data
2. Analyze and visualize data
3. Determine Friction I.L.s using the AASHTO method
4. Generate Friction Demand Models

The following sections describe the methodology in detail and demonstrate the general approach adopted to accomplish the research goals.

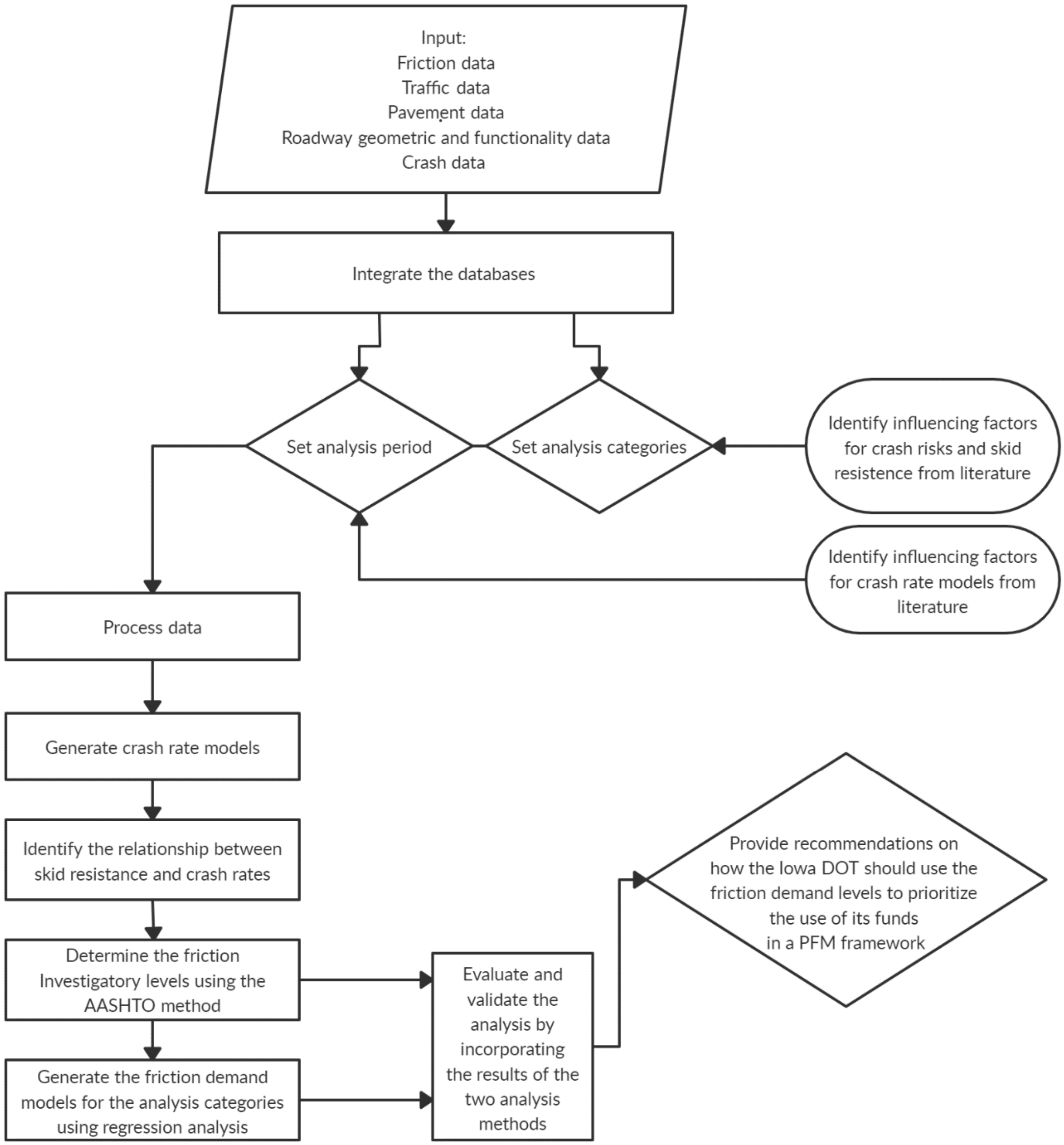


Figure 16: Conceptual Framework of the research

Data Acquisition and Processing

The data needed for this research effort had already been collected and stored by the Iowa DOT in different databases. Five major sources of data were used: Iowa Pavement Management Information System (PMIS) data, Iowa Roadway Asset Management System (RAMS) data, Iowa Intersections geodatabase, Iowa Curves data, and Iowa Crash data. These databases originated from different sources within Iowa DOT and were not stored using consistent referencing methods. Thus, the data needed significant processing prior to performing the analysis. This pre-processing involved filtering the different databases to include only valid attributes and aggregating each type of data into consistent sections on the basis of geospatial location, description, year, and mileposts. At that point, the four types of data could be linked, and the analysis performed. In addition, after integrating the data and prior to exploring the relationships between the friction, crash, and traffic data, it is necessary to examine the data independently to ensure that the data covers a wide range of scenarios and is not biased to certain types of sites. The following sub-sections will give generic details about the data sources, specific details about the available data as well as the processing performed.

PMIS Data

The PMIS database contains various levels of data on the pavement condition and history of Iowa Interstate and Primary routes collected between 2000 and 2018. PMIS data includes detailed information about the location, structure, functionality, and pavement condition information over time of the roadway sections. For the purpose of this research, the most important four elements in the PMIS data are the pavement friction values (i.e. Skid number measured at (40 mph) reported as (SN40), the location (route name and milepost) of the collected data and the year of the test, as well as the pavement type. These elements are conflated to the

RAMS database to get a more detailed roadway network information of the sections. The Iowa DOT carry out extensive efforts to process, report and integrate the friction measurements. Friction measurements are using a locked-wheel trailer and following the ASTM standard E274. The measurements are collected at the network level on a two-year cycle for interstate segments, a three to five year cycle for primary non-interstate segments depending on the AADT, and annual monitoring for segments with low SN's following the FHWA 2010 guidance.

These variations in the testing cycles for different sections is a major challenge for the data processing for this research. For example, a segment might be tested only once in a ten years span and another might be tested each year consecutively for the same ten years. After filtering the friction data to include only valid measurements, the database retained 29,466 records totaling 4,673 unique sites with at least one friction reading. Each record includes the skid number at a certain date for a certain milepost location along a route. Figure 17 presents the number of sections tested each year in the period (2009 – 2018). It can be seen that the highest number of tested sections was carried out in 2018.

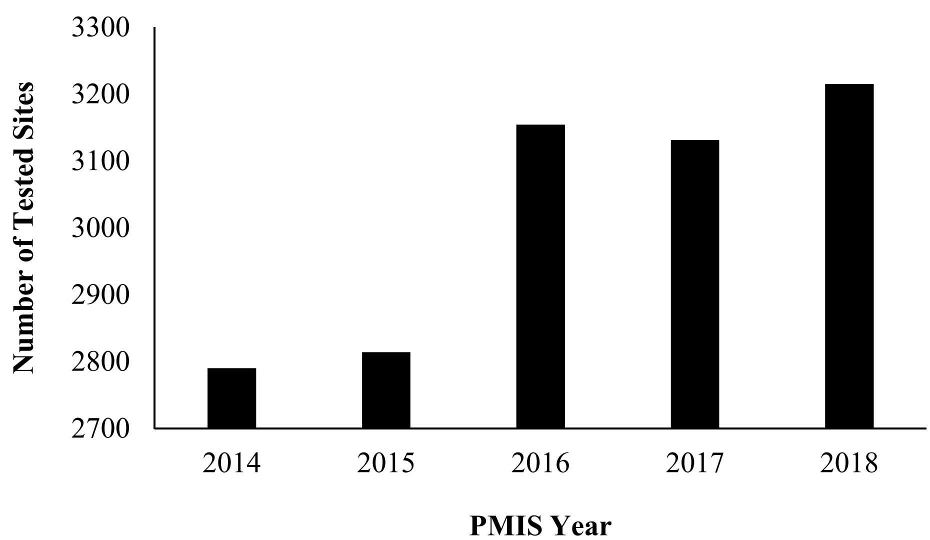


Figure 17: Pavement sections tested for friction by year

For practicality, only the data for one year (i.e. 2018) are analyzed. This is to avoid erroneous interpolations of traffic and friction measurements along a specific period where short-term random fluctuations in annual crash numbers might occur.

Section's Length

For 2018, the pavement section lengths range from 0.2 miles to 20 miles with an average length of 3.5 miles. Almost 34% of the sections are less than one mile long. Only few of the sections, around 4%, are longer than 10 miles. Figure 18 presents a frequency distribution of the pavement sections based on their lengths.

To reduce the effect of the ambiguity in the exact location of the friction measurement and to increase the confidence in the continuity of the friction reading along a segment, segments are filtered to those of lengths ranging from 0.2 mile to 10 miles.

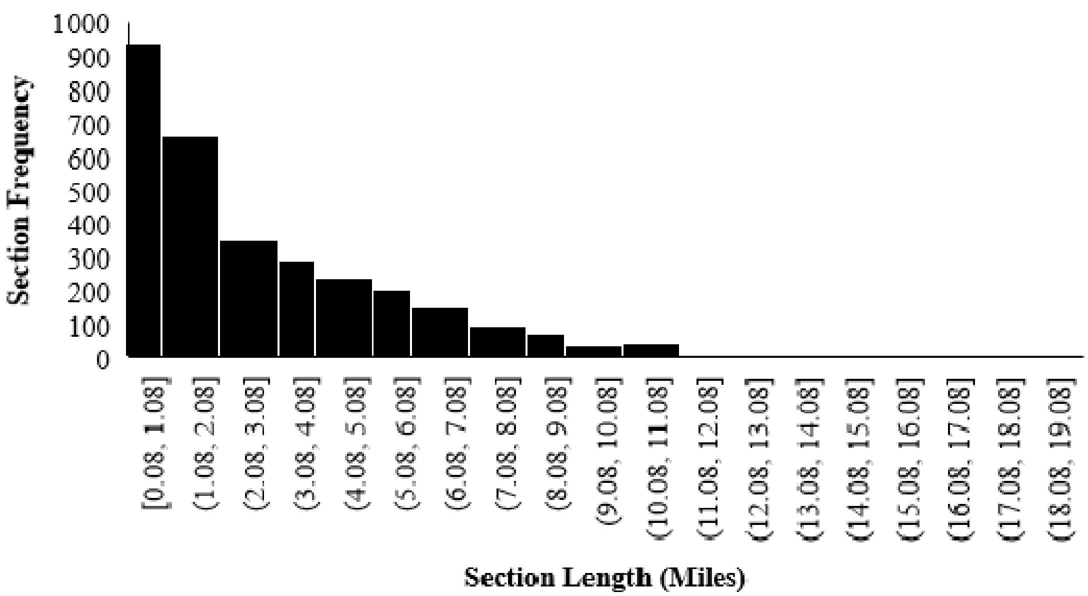


Figure 18: Pavement sections distribution by section length

Friction

To validate the data for the purpose of this research, friction data should include sites with poor friction as well as sites with good friction and should preferably exhibit a normal distribution. Figure 19 shows a histogram of the skid resistance data for the remaining 3295 sections. The mean of the skid numbers for these sections is 49.40, the standard deviation is 7.21, and the friction numbers range from 14 to 68.

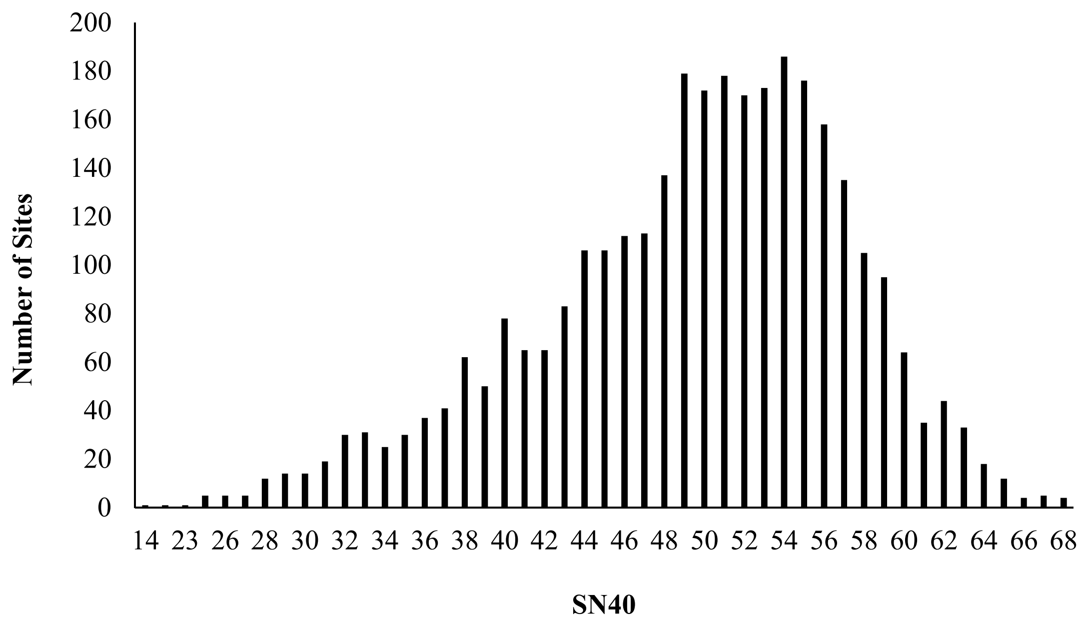


Figure 19: Friction distribution

There is a clear left skewed bell shape distribution to the friction data. However, there appears to be a wide range of skid numbers to be considered in the analysis. The scarcity of sites with very low skid resistance is most probably attributed to the fact that the Iowa DOT is doing a great job following the FHWA guidance on annually monitoring and treating segments that has been recorded with low SN's and identified as potential crash risk sites.

Pavement Type

The considered sections include a variety of pavement types with the composite pavement sections constituting 47% of the overall sections. In addition, 33 % and 12 % of the sections are Portland Cement (PC) and Asphalt Concrete (AC) pavement sections, respectively. Since these three types of pavements are the most common and are constituting almost 93% of the available data, only these three types are considered in the analysis of the friction demand levels to evaluate the relation between skid resistance and the macrotexture provided by each type. Figure 20 shows the distribution of pavement sections by pavement type. The terminology that will be used to describe the pavement type through this study is outlined in Table 4.

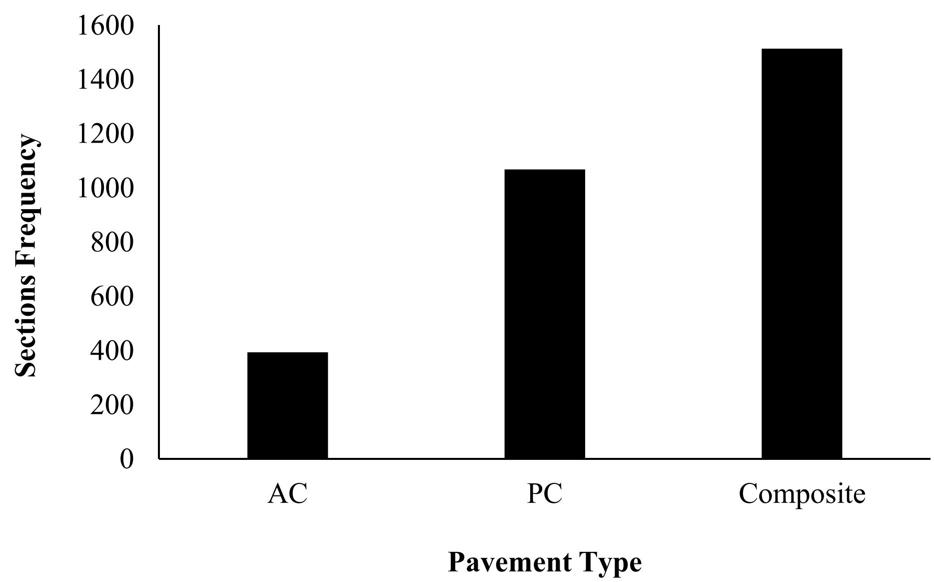


Figure 20: Pavement sections distribution by pavement type

Table 4: Pavement Types

Pavement Type	Description
PT1	PC
PT2	AC
PT3	Composite

Traffic

One more challenge with the PMIS data is the lack of actual count of the Annual Daily Average Traffic (AADT) records of the tested segments. AADT values are essential in calculating the exposure and developing the crash rate models. The data reports “generated” Average Daily Traffic (ADT) values only, which cannot be utilized by this research methodology that aims to generate crash rate models that are normalized by the annual traffic volumes to examine the influence of pavement friction on wet, dry and RwD crash rates. Consequently, the roadway traffic data is obtained from the RAMS database after conflating the PMIS database into the LRS based RAMS database using dynamic segmentation.

RAMS Data

The RAMS database stores the entire roadway network for the State of Iowa utilizing the Linear Referencing System (LRS). LRS reserves the advantage of providing the total miles of the roadway network in both directions instead of dealing with attributed segments at centerlines of the roads from the Geographic Information Management System (GIMS). The RAMS stores the roadway data such as functional classification, traffic, roadway geometrics, pavement condition data, as well as structure.

This research is aimed to determine the quantitative relationships between skid resistance and crash occurrence on Iowa roadways accounting for contributing factors such as roadway functional classification, Geometry, traffic, speed, and pavement type. Along this, the RAMS will be used to provide information about the road functional classification (Urban Interstate, Urban Other Principle Arterial, etc.), posted speed limits and AADT volumes.

Roadway Categories

In this research, the roadway segments will be categorized based on their federal functional classification. The federal functional classification is stratified based on a range of mobility and access functions that roadways serve. There are seven federal classifications included in the RAMS following the AASHTO functional classification definitions, a schematic diagram of the classification is demonstrated in Figure 21 based on AASHTO 2013.

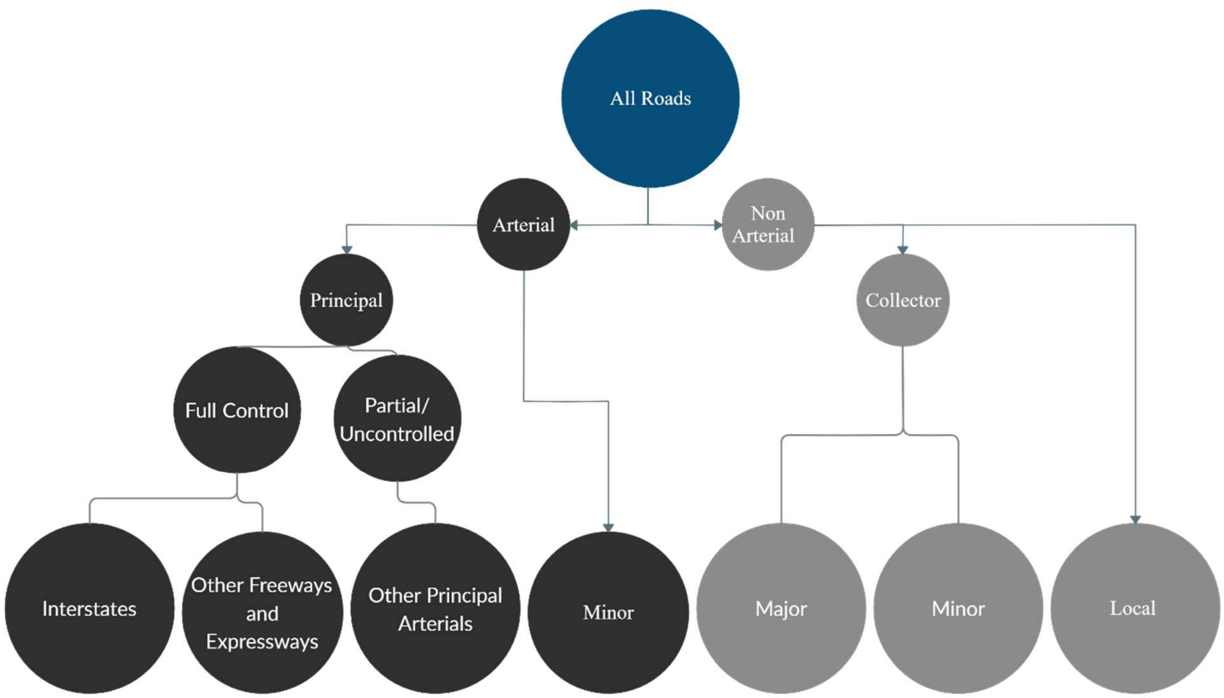


Figure 21: Federal Functional Classification of roadways in the RAMS (AASHTO 2013)

This research will not analyze the Minor collectors and local roads because of lack of friction measurements on them. The roadways are classified into ten categories based on their functional classification and urban/rural designation. These categories are outlined in Table 5.

Table 5: Roadway Categories

Classification	Description	Mileage
FC1	Urban Interstates	432.49
FC2	Rural Interstates	1009.13
FC3	Urban Principal Arterials – Freeways and Expressways	232.26
FC4	Rural Principal Arterials – Freeways and Expressways	315.55
FC5	Urban principal arterials - other	1280.38
FC6	Rural principal arterials - other	3420.89
FC7	Urban Minor Arterials	448.34
FC8	Rural Minor Arterials	1860.01
FC9	Urban Major Collectors	286
FC10	Rural Major Collectors	1395.04

Speed Limits

The distribution of pavement sections' speed in Figure 22 shows that most of the sections mandates a minimum speed limit of 50 mph. The sections are categorized into three speed limit levels to study the effect of each speed limit range on the friction demand. The low, medium, and high-speed roads were defined as those with speed limits of 35 mph or lower, between 40 and 55 mph, and 55 mph or higher, respectively. Similar speed limit levels revealed tangible relationships between the pavement surface condition and the crash rates in a study by Lee at al., (2015) to evaluate the effects of pavement surface conditions on traffic crash severity. Table 6 presents the three levels of speed limit along with their definition.

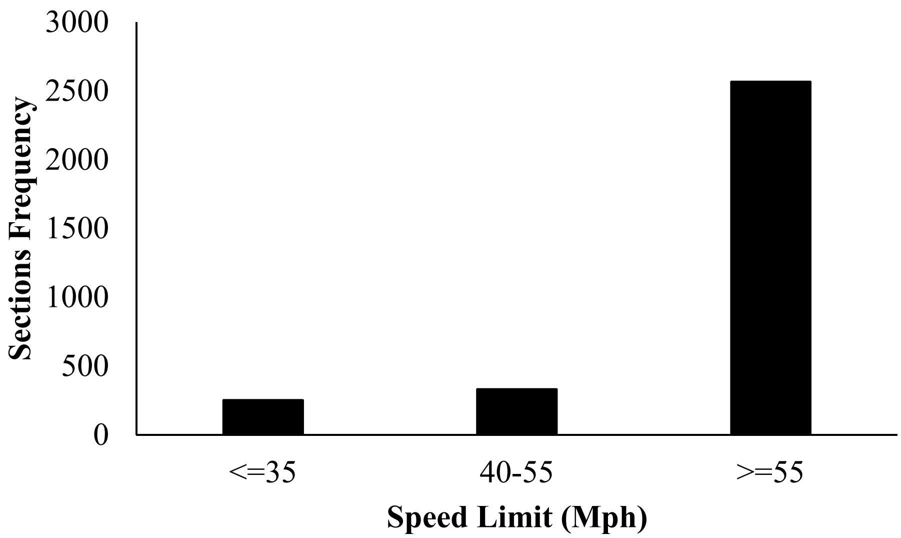


Figure 22: Distribution by speed limit

Table 6: Speed Limit Levels

Speed Level	Description
SL1	Low: <= 35 mph
SL2	Medium: 40-55
SL3	High: >=55

Traffic data

Traffic data is required for two purposes in this research. The first, is to compute crash rates and examine the influence of traffic volumes on wet, dry and RwD crash rates for all the proposed roadway categories. The second, is to categorize the roadway network into different levels based on the AADT exposure to provide a more detailed mean to correlate the pavement friction measurements with the AADT in a safety related scheme. The RAMS includes the AADT values between two mileposts along a given route as well as the count year for the given AADT. It also includes an Expanded AADT field for the current year (i.e. 2018) using jurisdiction specific growth factors.

Figure 23 shows the distribution of pavement sections based on AADT. The distribution of pavement sections' AADT is positively skewed. The reason behind this skew is the presence of almost 1% of sections with a very high AADT records (> 15,000). This indicates that the bulk of the sections lie between AADT values of 1,000 and 15,000. Consequently, each roadway segment was categorized into one of three volume ranges following guidance from the FHWA 2014 report: "Assessing roadway traffic count duration and frequency impacts on AADT estimation FHWA PL-015-008 2014" (FHWA, 2014). Table 7 presents the three levels of AADT designation.

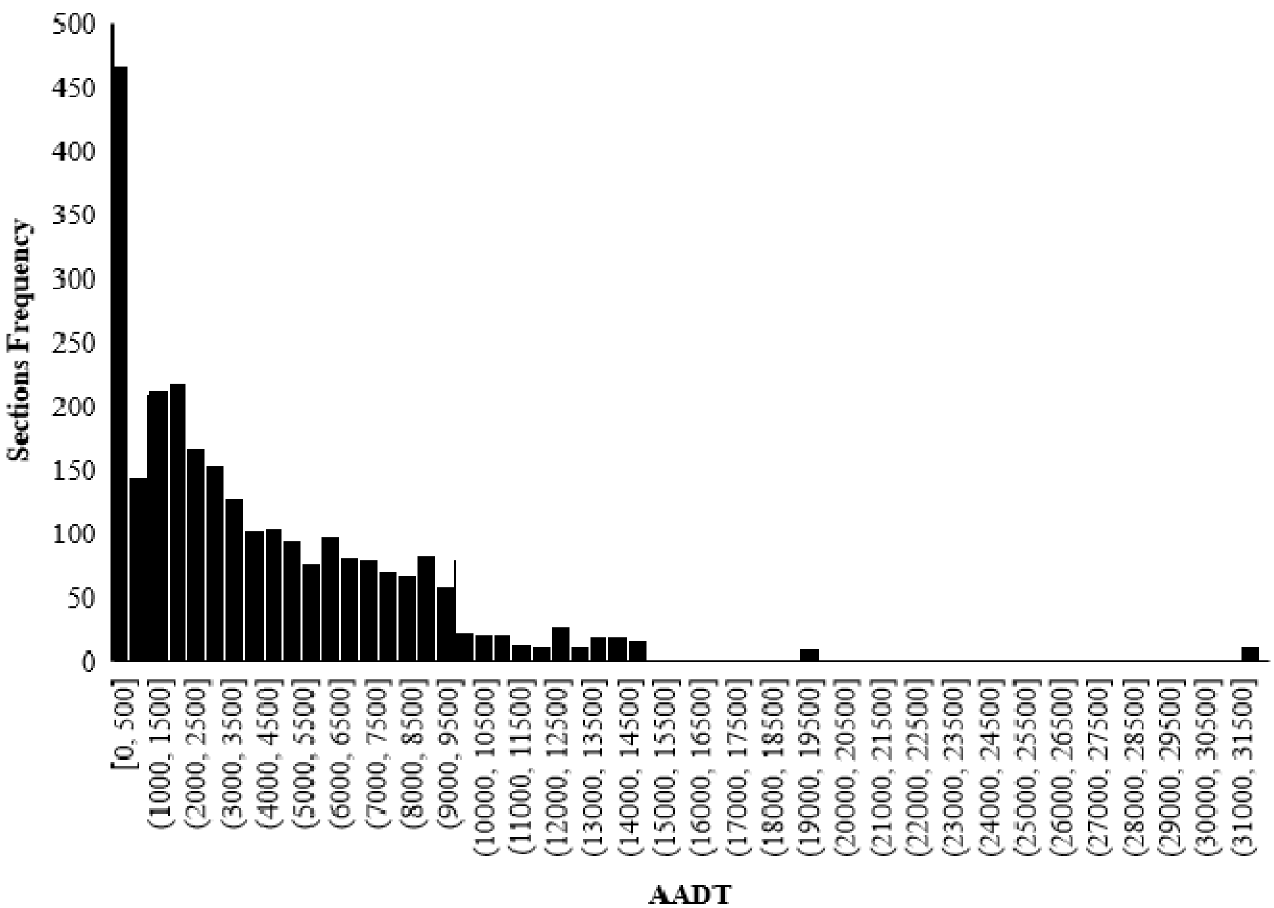


Figure.23: Pavement sections distribution by AADT

Table 7: Traffic Volume Ranges

Traffic Volume	Description
AADT1	Low: <1000
AADT2	Medium: 1000<=AADT<10000
AADT3	High: >=10000

Curves Data

The RAMS database does not provide a single attribute that identifies the locations of tangents on the network. Hence there is no way to automatically extract tangents only from RAMS. The curves database of Iowa is generated by the Center for Transportation Research and Education (CTRE). The data identifies the locations of the curves along with different attributes that were derived using post-processed GPS traces in conjunction with manual refinement for curve identification. The database contains only high-speed curves on secondary and primary roads and is divided to two subsets of data based on the number of lanes. (i.e. two lanes and multi-lane curves).

The RAMS database along with the curves database, does not yet provide sufficient information to distinguish the different types, locations, and geometries of tangents. The identification of such information is a time consuming and error-prone process and will not be included in this study. It is expected that tangents traversing into curves are exposed to increased friction demand where the roadway is at higher risk of polishing due to frequent braking. This means it often becomes prematurely polished, reducing the pavement friction and contributing to higher crash rates. However, using the available data, curves locations are identified, and tangents locations are identified as any segment that is not a curve. Those segments are conflated to the RAMS database to obtain traffic data on it as well as friction readings. This study will investigate the primary roads tangents that has available friction readings. Figure 24 presents a frequency of the matched tangent sections with friction readings. It can be noticed that the bulk of the tangent sections retain a skid value between 40 to 58 which indicates being in a very good condition. Generating the friction models for tangents will help facilitate generating friction models for curves in the future and gives an insight of the friction requirements for the network.

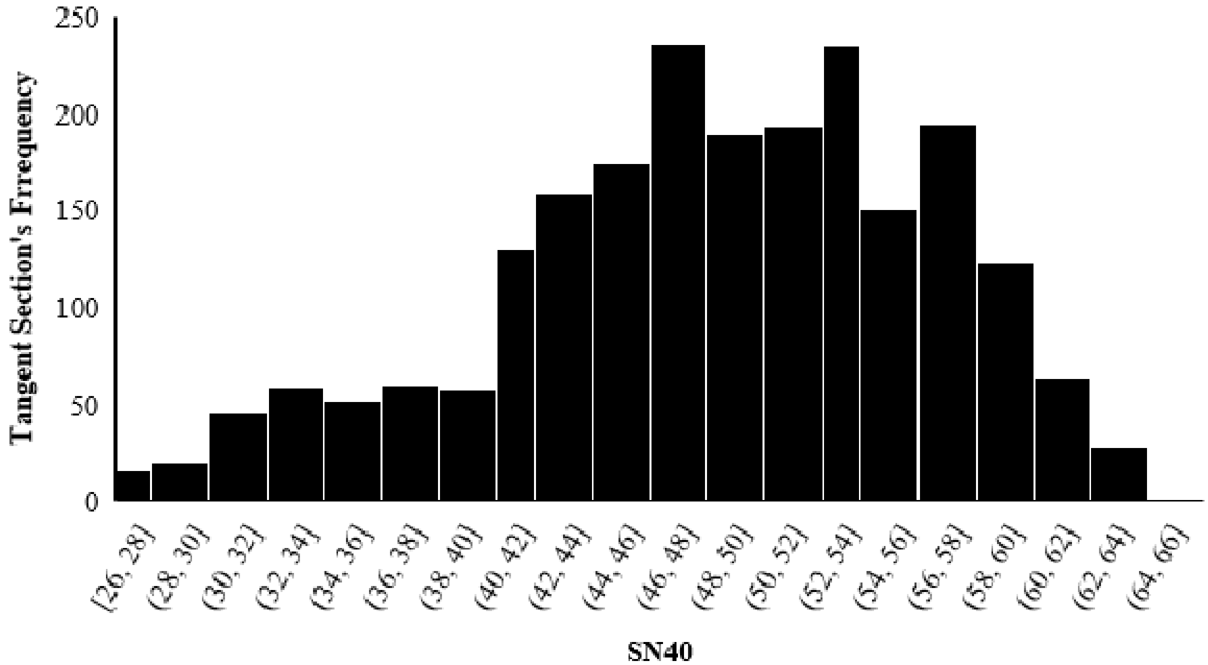


Figure 24: Tangents Segments Frequency by Friction

Crash Data

The Iowa DOT maintains public access to ten years of crash reports in a variety of tables. Each crash is geospatially located and there are many data tables available for each crash. There are three levels of data: crash-level (one record per crash), vehicle-level (one record per vehicle), and person level (one record per person involved in each crash). Each person (aside from non-motorists) is associated with a vehicle using a unique key and each vehicle is associated with a crash using a unique key. These keys can be used to associate the same vehicle or crash across tables as well. Each crash in the database is based on the responding officer's report. After that, the responding agency forwards the crash report to the DOT to process it. Most patrol cars in the state of Iowa are equipped with a GPS device to accurately locate the crashes. However, other crashes are manually located using a literal description of the crash location (Iowa DOT, 2014).

In this research, the crash locations are spatially assigned within 250 ft. of the PMIS sections in a relatively precise manner on the basis of geospatial location, description, as well as year. The segments are then matched with SN values from the PMIS database. For the purposes of this research, crash data for the year 2018 is extracted from the July 16, 2019 snapshot of the Iowa DOT crash database.

The crash data also includes information on the severity of the crash (fatal, injury-causing, etc.) and the road surface condition at the time of the crash (wet, dry, etc.). The analysis of this research considered all types of severity crashes. For 2018, the data contains records for almost 75 thousand crashes. For the purpose of this study, it is important to eliminate the effect of conflict points (i.e. intersections) on the occurrence of crashes as this study is aimed to correlating the friction-related crash occurrences with the variations of friction measurements. Hence, the intersection related crashes, constituting about 30% of the crashes, had to be excluded from the analysis. The intersection related crashes are identified using the Iowa DOT guidance as they fall within a vicinity of 250 ft. of an intersection as well as are reported as intersection/ramp related in the roadway type feature of the data. In addition, the crashes are then limited to only dry and wet crashes as reported in the environmental surface conditions feature of the data. After filtering the crashes, summaries of the approximately 22,000 crashes that are successfully matched with friction locations are generated.

In addition, this study investigates the friction-related crashes. In the literature, roadway departure crashes have been recognized for being related to friction most of the time. A roadway departure (RwD) crash is defined as a crash which occurs after a vehicle crosses an edge line or a center line, or otherwise leaves the traveled way (FHWA, 2019). In Iowa, approximately 33,000 Roadway departure crashes occurred in the period of 2014-2018, resulting in over 700 fatalities, which is about 45% percent of all the traffic fatalities in Iowa for the same period. This study further investigates the roadway departure crashes that constitutes about 15.5 % of the total crashes for the designated roadway categories to reveal more tangible relationships and provide a specific friction demand levels in a friction-related safety framework.

A primary issue in examining the crash data prior to any analysis is to ensure that the data represents a wide range of crash rates; specifically, it is important that sites with low crash rates, or sites with no crash incidents at all, are considered in the analysis. If only sites with a crash problem and high crash rates are considered, then any relationships found cannot be assumed to be valid for all types of sites. In this data set, 831, or almost 25% percent, of the 3295 sites experienced no crashes in 2018. Clearly, this is sufficient to ensure that the data set is not biased toward sites with crash problems. Further analysis is performed on the crash data to determine the total number of crashes across the different analysis categories. Table 8 shows a breakdown of the number of total crashes by surface condition as well as Rwd crashes for each category of the analysis.

The breakdown of the crash occurrences by functional classification reveals that the urban principal arterials (FC3), experienced the highest number of matched dry, wet and RwD crashes. The rural principal arterials – Freeways and Expressways (i.e. FC4) as well as Urban and Major Collectors (i.e. FC9 and FC10) had relatively low number of matched crashes. These categories are highlighted in grey in Table 8 and are not included in the analysis. In addition, the bulk of the network crashes happened at those sections with high speed limit (SL3). Furthermore, the crashes break down by the traffic volume explicitly show that sites with medium AADT (i.e.

AADT 2) witnessed the highest crash occurrence. Also, composite pavements had the highest crash exposure amongst the other pavement types. In addition, among all the analysis categories, the tangents segments experienced the highest RwD/total crash ratio. This can be attributed to the nature of the RwD crashes and that more skidding is required when a driver is traversing a tangent towards a curve.

Table 8: Crashes break down for the analysis categories

Category	Total crashes		RwD crashes
	Wet	Dry	
Functional Classification			
FC1	791	2017	273
FC2	828	2100	398
FC3	1499	2119	447
FC4	39	56	9
FC5	1335	3222	295
FC6	306	964	361
FC7	728	1241	200
FC8	494	598	164
FC9	39	123	16
FC10	92	105	35
Tangents			
Tangents	278	1029	453
Speed Limit			
SL1	1336	3696	334
SL2	1117	3249	277
SL3	4665	7124	1443
AADT Range			
AADT1	1778	3082	529
AADT2	3241	5682	918
AADT3	2707	5305	830
Pavement Type			
PT1	1976	2944	576
PT2	549	515	186
PT3	2071	2773	464

Data Analysis

This section details the analysis of the friction, crash, and traffic data that is acquired and pre-processed as described in the previous section. In addition to the previous abbreviations assigned to each analysis category in the previous section, the major variables that are discussed in this section are also abbreviated. These variables are the Wet Crash Rate per Hundred Million Vehicle Mile (HMVM), reported as (WCR), the dry crash rate per HMVM reported as (DCR) and the Roadway Departure crash rate per HMVM reported as (RwDCR).

Friction Analysis

In this study, Friction measurements are divided into intervals with increments of 2 friction units and the number of crashes for each friction interval were determined for the overall data and for the different analysis categories. The wet/dry crash distribution in relation to friction is illustrated in Figure 25. In addition, the roadway departure crash distribution in relation to friction is illustrated in Figure 26.

Figure 25 shows that the wet crashes distribution for the sites in this study constitutes at least 30% of the dry crashes' distribution for each bin. The trend also shows that the higher SN40, the higher that percentage is. Furthermore, Figure 26 shows that the RwD crashes distribution matches very closely the distribution of the total crashes. This indicates that the study sites represent a sample of the network with regards to skid-related crashes.

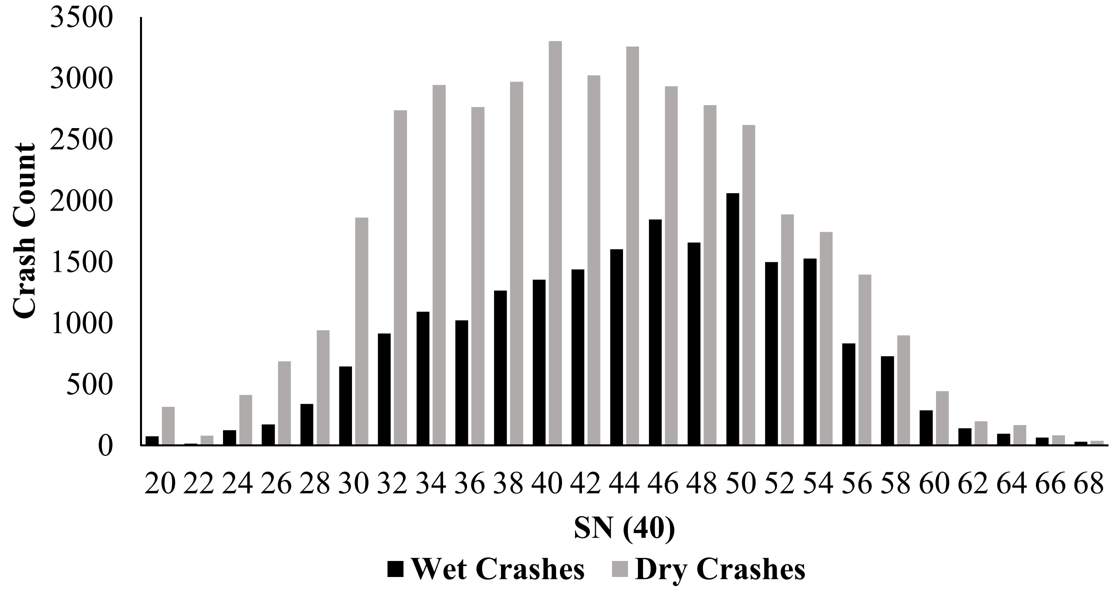


Figure 25: The wet/dry crash distribution in relation to friction

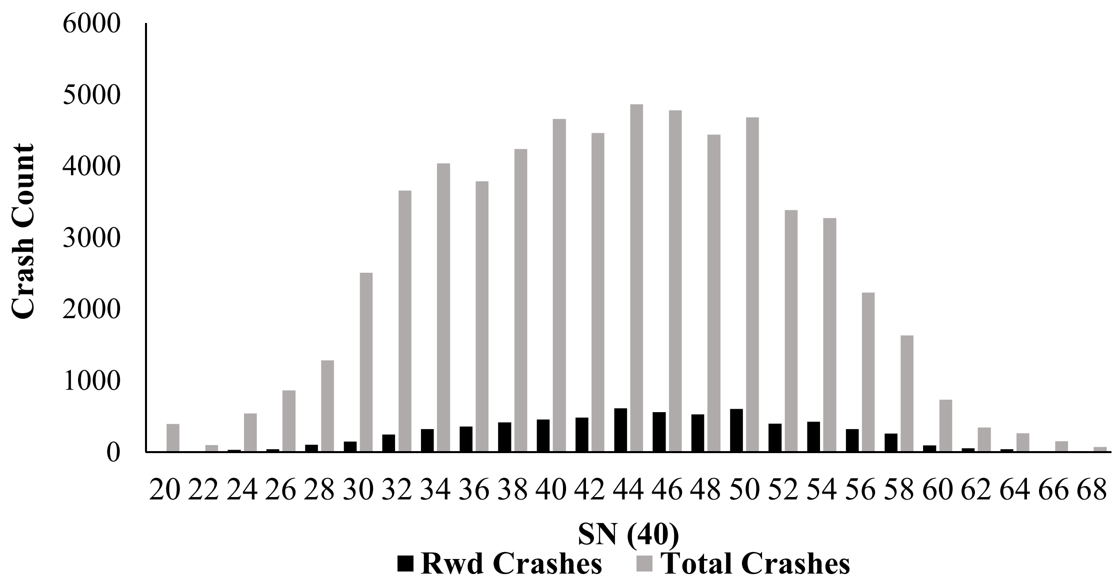


Figure 26: The roadway departure crash distribution in relation to friction

Crash Rates

The crash rate models for this study are generated as follows:

- The number of segments for each friction level is determined
- The total number of wet and dry crashes as well as Rwd crashes for all the segments at each friction level is determined.

- The AADT is identified for each segment in each level and a pro-rated AADT is calculated for each group of segments that shares the same friction level using the following equation (Porter, 2016):

$$AADT_{PR} = \frac{L_1 \times AADT_1 + L_2 \times AADT_2 + L_i \times AADT_i}{L_1 + L_2 + L_i}$$

Where:

L_i = Length of Segment #i (Miles)

$AADT_i$ = Average Annula Daily Traffic of Segment #i

- The WCR, DCR and RwDCR are calculated for each bin in HMVM using the following equation (Gan et al. 2012):

$$Crash.Rate = \frac{Number\ of\ Crashes\ \times 100{,}000{,}000}{AADT_{PR} \times 365 \times Y \times L}$$

Where:

$AADT_{PR}$ = Prorated Average Annual Average Daily Traffic

Y = study duration in years (1 year)

L = Total length of the roadway segments (Miles)

The crash rate models are generated and summarized for all the analysis categories. These summaries are utilized in the Friction demand models generation.

The overall WCR for the 3,295 sites is 208 wet crash/HMVM. Over 31 percent of the study sites have wet crash rates higher than the overall wet crash rate. For Comparison, the overall total crash rate for the study sites is 884 crash/HMVM. Nearly 45 percent of the study sites have a crash rate higher than the overall total crash rate.

Visual Diagnostics

Prior to attempting the regression analysis, the variables are plotted against each other for all 3295 sites to observe general trends and to assist in formulating the best models for the regression. The trends between WCR, DCR and RwDCR with SN40 are diagnosed in Figure 27. The three trends show a good amount of scatter and provides no tangible relationship between friction and crash rate. This elaborates on the need to quantify the relationships between skid resistance and crash occurrence on Iowa roadways accounting for the contributing factors that were previously discussed.

In addition, The WCR, DCR and RwDCR for each site is plotted versus the AADT as shown in Figure 28. This plot indicates a pronounced relationship between these variables, with the WCR, DCR and RwDCR decreasing with increasing traffic. One proposed explanation for obtaining high quality trends between the crash rates and AADT in contrary to the ones obtained against the friction is that the crashes are normalized by the AADT values to calculate the crash rates. This means that the two variables constitute an underlying mathematical relation.

Friction Investigatory Levels Determination – AASHTO Method

A highway agency can assign different investigatory and intervention levels for each roadway category to normalize and minimize the risk of skid-related crashes through the network (Larson et al. 2012). The AASHTO Guide for Pavement Friction recommends different methods to determine the friction Investigatory Level (I.L.) by evaluating the relationship between wet to dry crashes ratio and SN (AASHTO 2008). This study will utilize the third method which recommends using the friction distribution and wet to dry crash ratios to determine the I.L. In addition, to further examine the friction-related crashes, this study will examine the RwD to total crash ratio as well. To determine the I.L.s following the AASHTO method, the mean and standard deviation of the friction distribution for each category are calculated. The I.L. of friction is defined as the mean minus 'X or Y' standard deviation based on the wet/dry crash ratio pattern or the Rwd/total crash ratio pattern. The value of the fraction of the standard deviation to be subtracted from the mean is adjusted to match the point where a sudden drop in the wet to dry ratio was observed.

Regression Analysis – ANOVA

In this study, friction demand models are generated as regression models that can be used to define the minimum allowable friction level for various types of roads. Compared to traditional regression models that are developed to describe the relationship between pavement skid resistance and crash risks, the proposed two-parameter, two-level skid resistance models are specifically designed to support the management of skid resistance at the network level. The development of these models introduces an easy way to quantify and understand the quantitative relationship between skid resistance and crash risks.

Incorporating the generated friction demand models into a skid resistance policy would reduce crashes and enhance safety by improving the braking performance. In fact, the driver's choices, based on his/her perception of the road, of vehicle efficiency, of traffic and of environmental conditions, can become manoeuvres of the vehicle if there is friction, which in turn depends on the road, vehicle and environmental conditions. (Colona et al., 2016)

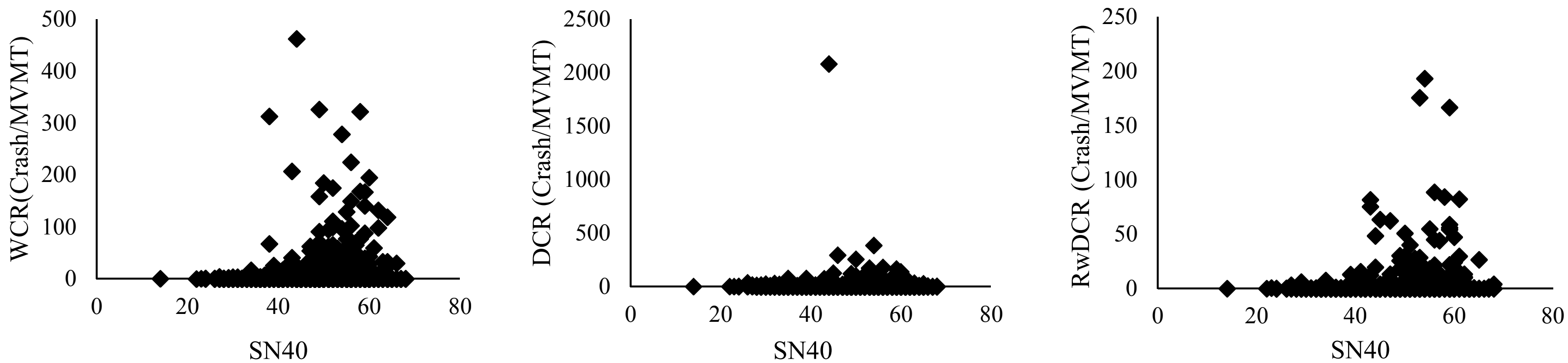


Figure 27: Crash Rate versus Friction

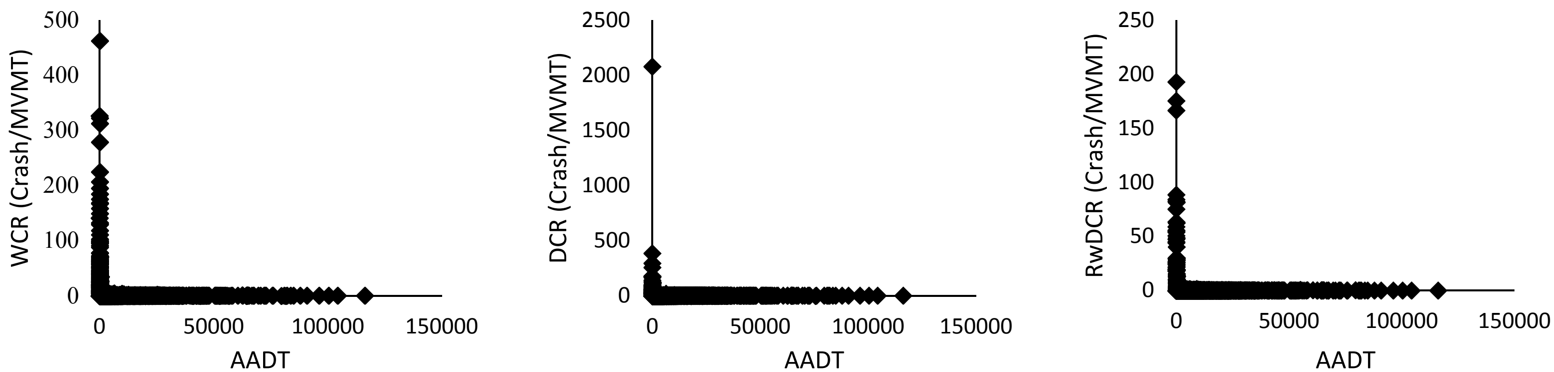


Figure 28: Crash Rate versus AADT

Reviewing the friction versus crash ratios distributions revealed high variability in the crash ratio's peaks amongst it. Consequently, to ensure that a drop is representative of the friction sites, the drop in the crash ratio must correspond to at least 5 of the cumulative site numbers to be considered significant. The AASHTO method also provide means for identifying Intervention Levels at minimum satisfactory wet/dry crash ratio, or to address friction deficiencies where enough funding is available. However, in this study, Intervention Levels are not investigated because Iowa DOT usually will not automatically trigger any kind of maintenance treatment to correct deficiency without a proper investigation. That being said, interventions are only triggered if the investigation concludes that it is necessary. Therefore, only I.L.s are investigated in this study.

Before generating the models, it is important to determine the significance of the effect of friction on the rate of wet, dry or RwD crashes. Therefore, an Analysis of Variance (ANOVA) is performed to check for significance. The ANOVA is to test the null hypothesis (H_0) that the population mean for each friction level are equal vs. the alternative hypothesis (H_a) that at least one mean among one friction level is different. Hence, the formal notation of the statistical hypotheses of this research is:

$$H_\circ : \mu_1 = \mu_2 = \cdots = \mu_k$$

$$H_a : not\ all\ means\ are\ equal$$

Where:

$$\mu_k : The\ mean\ of\ the\ crash\ rates\ for\ each\ friction\ level$$

This hypothesis is tested amongst the analysis categories using the JMP software. For this study, the basic regression models are conceptualized as if the total variation in the response data equals the variation in the mean response in addition to the residuals (Anscombe, 1948). This is illustrated in equation (Anscombe, 1948):

$$(y - \bar{y}) = (\hat{y} - \bar{y}) + (y - \hat{y})$$

Squaring each of these terms and adding over all the n observations gives the equation (Anscombe, 1948):

$$\sum(y - \bar{y})^2 = \sum(\hat{y} - \bar{y})^2 + \sum(y - \hat{y})^2$$

This equation may also be written as (Anscombe, 1948):

$$SST = SSM + SSE$$

Where:

SS is notation for sum of squares and T, M, and E are notation for total, model, and error, respectively.

The square of the sample correlation is equal to the ratio of the model sum of squares to the total sum of squares:

$$R^2 = \frac{SSM}{SST}$$

This formalizes the interpretation of R^2 as explaining the fraction of variability in the data explained by the regression model. In addition, the t-statistic is compared with the t distribution to determine the p-value which provides information on whether to reject the null or accept it. However, to verify the adequacy of the linear models, residual analysis is performed after analyzing the variances of data for each category. Residual analysis is a diagnostic method for examining the adequacy of the fit of a regression model. There are three basic assumptions that are made in any regression analysis. According to Montgomery et al. (2001), these assumptions are: First, the relationship between response and regressors is linear. Second, the error term has a zero mean and constant variance and, finally, the errors are uncorrelated and normally distributed.

The violation of these assumptions can create an unstable model in which different samples can result in totally different models with contradictory conclusions (Montgomery et al. 2001). Graphical analysis of the residuals is a common way of examining the adequacy of a regression model. This method is used to examine the adequacy of the models proposed for each roadway category. First, to check the normality assumption, a normal quantile plot of residuals is constructed using the JMP software. Ideally, the points should lie along a straight line in the normal quantile plot. Transformations are applied to the regressor at each model correspondingly. In this study, the logarithmic transformation is satisfactory to eliminate the nonlinear behavior of the residuals for the majority of the models.

A sample of the summary of ANOVA statistics and the model parameter estimates for wet crash rates at category FC1 before and after the logarithmic transformation are provided in Figures 29 and 30, respectively. In addition, a plot of the residuals before and after the transformation is provided in Figure 31 to demonstrate the graphical residual analysis utilized in this study.

Summary of Fit

RSquare	0.408624
RSquare Adj	0.373837
Root Mean Square Error	20.04439
Mean of Response	15.47368
Observations (or Sum Wgts)	19

Analysis of Variance

Source	DF	Sum of Squares	Mean Square	F Ratio
Model	1	4719.488	4719.49	11.7465
Error	17	6830.216	401.78	Prob > F
C. Total	18	11549.704		0.0032*

Parameter Estimates

Term	Estimate	Std Error	t Ratio	Prob>|t|
Intercept	73.794489	17.62684	4.19	0.0006*
SN	-1.395586	0.407194	-3.43	0.0032*

Prediction Expression

73.79448914 + -1.395586012 • SN

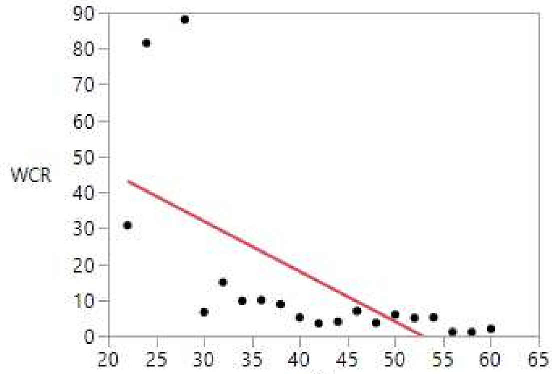


Figure 29: FC1 - ANOVA statistics and model parameter estimate before transformation

Not only the transformation has reformed the normal probability plot, but also has improved the model's coefficient of determination (R^2).

In this example, the data in the transformed ANOVA table provides enough information to reject the null and verify that not all means are equal and that the friction (SN) is a significant factor in the model, with a confidence level greater than 95% (P-value < 0.05).

Summary of Fit

RSquare	0.4655
RSquare Adj	0.434059
Root Mean Square Error	19.05613
Mean of Response	15.47368
Observations (or Sum Wgts)	19

Analysis of Variance

Source	DF	Sum of Squares	Mean Square	F Ratio
Model	1	5376.388	5376.39	14.8054
Error	17	6173.317	363.14	Prob > F
C. Total	18	11549.704		0.0013*

Parameter Estimates

Term	Estimate	Std Error	t Ratio	Prob>|t|
Intercept	229.08153	55.68641	4.11	0.0007*
Log[SN]	-57.84815	15.03415	-3.85	0.0013*

Prediction Expression

229.08153371 + -57.84814751 • Log[SN]

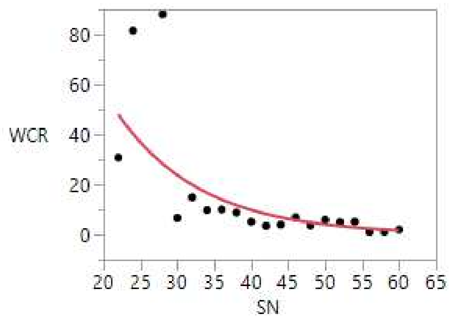


Figure 30: FC1 - ANOVA statistics and model parameter estimate after transformation

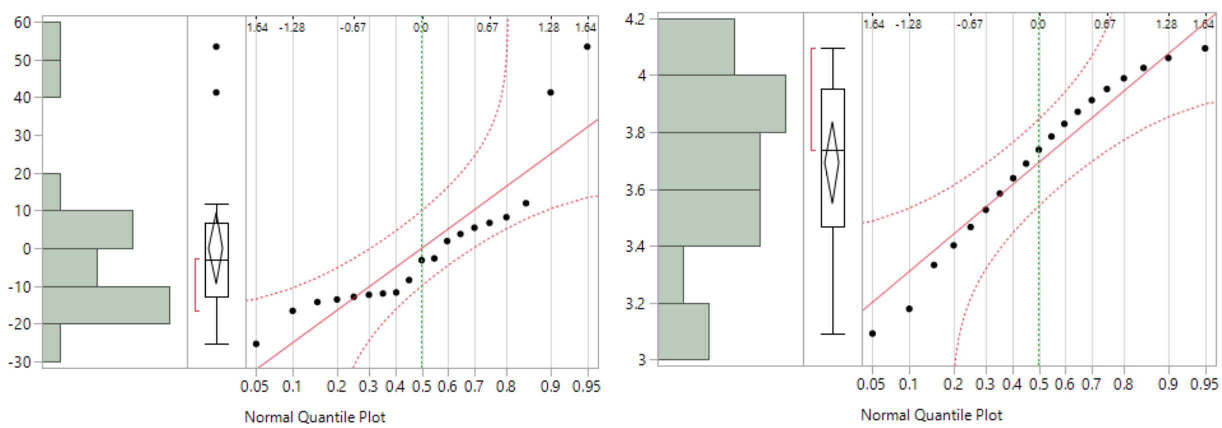


Figure 31: FC1 - Graphical residuals before and after transformation

However, in other analysis categories, due to the low number of RwD crashes that were matched with friction sites, the data did not provide enough information to reject the null and thus, a different probability test had to be performed whenever appropriate. For example, the analysis of variance of the friction and the RwDCR for the FC1 roadway classification, yielded a model with a p-value of 0.229 which means that there is no significant difference between any of the means of the RwDCR at any of the different friction levels. In addition, the model had an R^2

of 0.084 which means that only 8% of the variance in the RwDCR is accounted for. Usually, R^2 alone refers to the effect size. A small effect size does not indicate that the model is unworthy of being interpreted. However, a small R-square along with a high p-value indicate a small and insignificant effect. Hence, different generalized linear models had to be explored to best describe the data when the response variable retains a different distribution model other than a normal distribution. For example, for FC1, the RwD crash rates fit a negative binomial regression model.

The friction demand models for the wet and dry conditions as well as for the RwD crashes for the different analysis categories are demonstrated and discussed in the next chapter.

CHAPTER 4. RESULTS AND DISCUSSION

The goal of a PFM program or policy is to establish criteria to prioritize safety improvement projects based on the crash risk. The generated models in this research can be used to define the minimum allowable friction level for various types of roads. To emphasize on the benefits of the generated friction demand models, the discussion will demonstrate the usage of the regression models developed in this study to predict a crash risk by estimating the wet, dry and RwD crash risk associated with the suggested I.L.s by the AASHTO method.

This chapter introduces and discusses the results and findings of this research. The following sections will show, discuss, and provide explanation and guidance on the usage of the generated friction demand models for the different analysis categories. Each section presents the AASHTO method histograms accompanied with the suggested I.L.s. This is followed by the proposed friction demand models for the wet and dry conditions as well as for the RwD crashes for each category after proper logarithmic transformations has been applied to each crash condition/type. The models are demonstrated in Reports 1 through 7 for Functional Classification, Report 8 for Tangents, Reports 9 through 11 for Speed limit, Reports 12 through 14 for AADT, and Reports 15 through 17 for Pavement Type. Each report summarizes the model's information as well as the crash risk associated with each suggested I.L. The analysis categories sections are then followed by a general discussion of all the analysis categories as well as a summary of findings.

Friction demand models by functional classification

The AASHTO method histogram for each functional classification is presented in Figures 32 through 38. For each classification, the figures represent the wet/dry as well as RwD/total crash ratios shown by the black lines versus the friction distribution shown by the grey histogram blocks. In these histograms, the investigatory level is set at the value of friction where the mean friction is adjusted towards a specific fraction of the standard deviation. This adjustment is based on the crash ratio pattern. For example, for the urban interstates, the friction distribution of the sites has a mean friction of 52.08 and a standard deviation of 9.29. Based on the overall decreasing wet/dry crash ratios pattern with the increasing friction, one can observe a sharp drop at a friction value of 32. However, as discussed in the methodology chapter, a significant drop must correspond to at least 5% of the sites, hence, for this case, the next consecutive drop in the crashes around a friction value of 38 is considered. Consequently, the I.L. is set at a value of 37.22 which is the mean minus 1.6 of the standard deviation. The data for the same classification suggests an RwD I.L. of 45.58 which corresponds to 0.9 standard deviations shift from the mean. A summary of the suggested friction demand I.L.s for each classification based on its crash ratio and friction distribution histogram is provided in Table 9.

Table 9: Friction investigatory Levels by Functional Classification

Functional	Mean	Std	Wet/Dry	RwD/Total
FC1	52.08	9.29	37.22	45.58
FC2	43.71	6.93	34.01	35.95
FC3	49.03	7.27	40.31	40.93
FC5	44.10	6.66	30.11	34.11
FC6	44.77	6.76	33.95	38.00
FC7	51.84	9.33	35.07	42.51
FC8	43.43	6.58	30.26	32.23

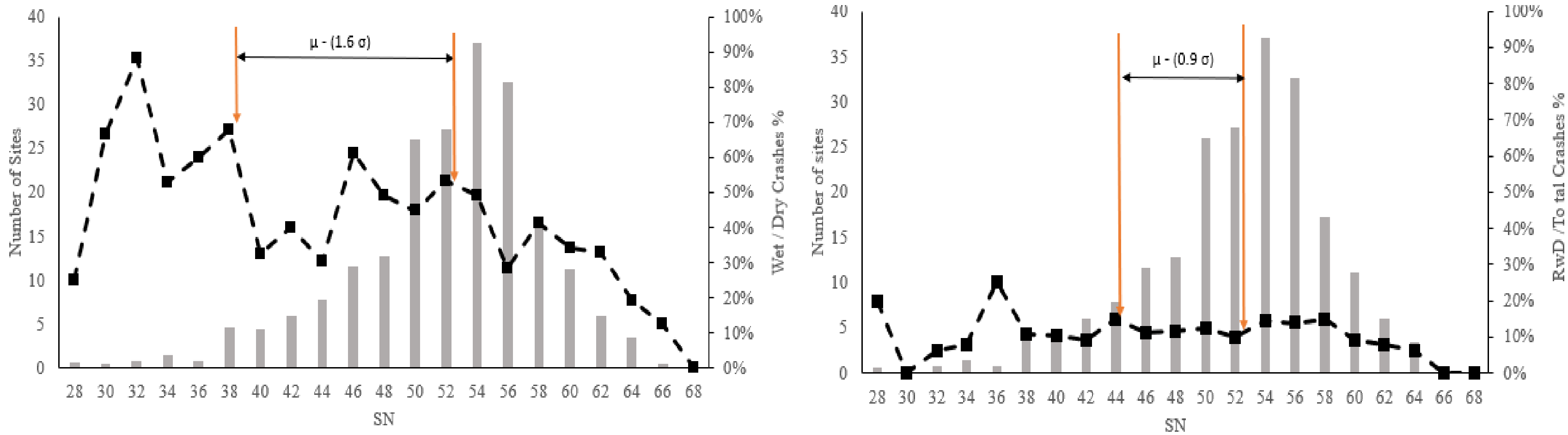


Figure 32: FC1 - Urban Interstates – Friction Distribution and Crash Ratio

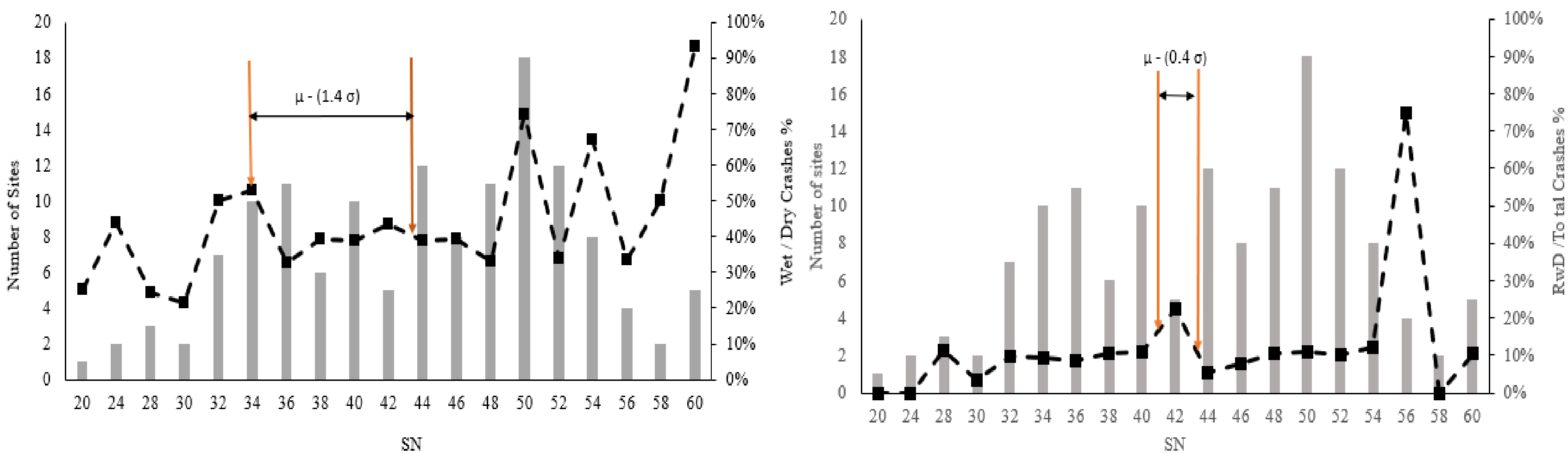


Figure 33: FC2 - Rural Interstates - Friction Distribution and Crash Ratio

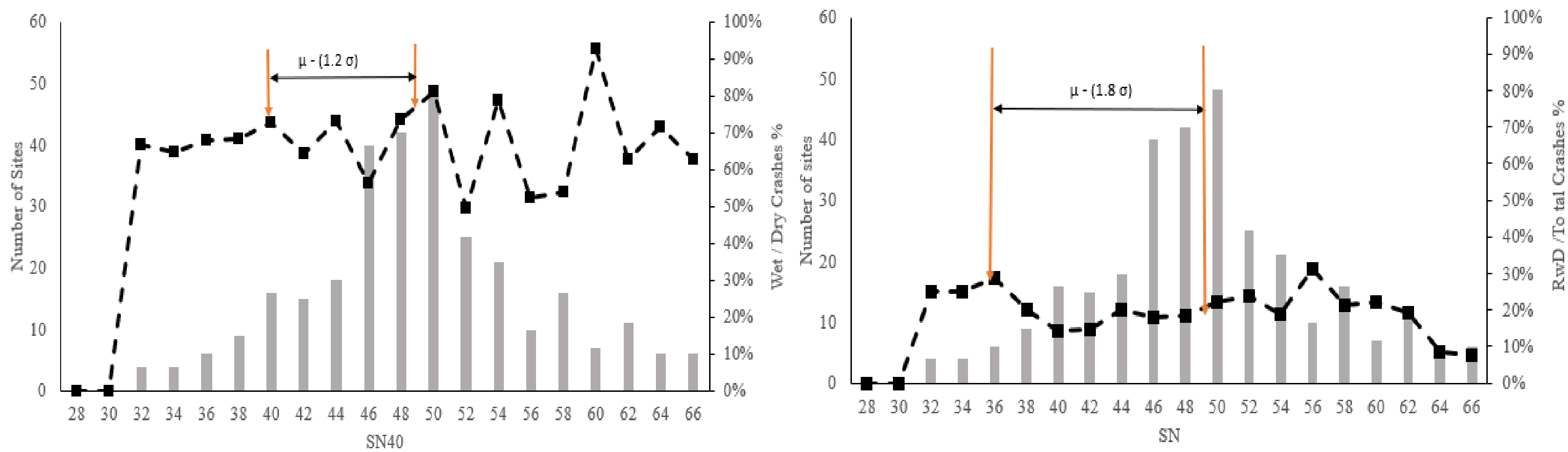


Figure 34: FC3 - Urban Freeways and Expressways - Friction Distribution and Crash Ratio

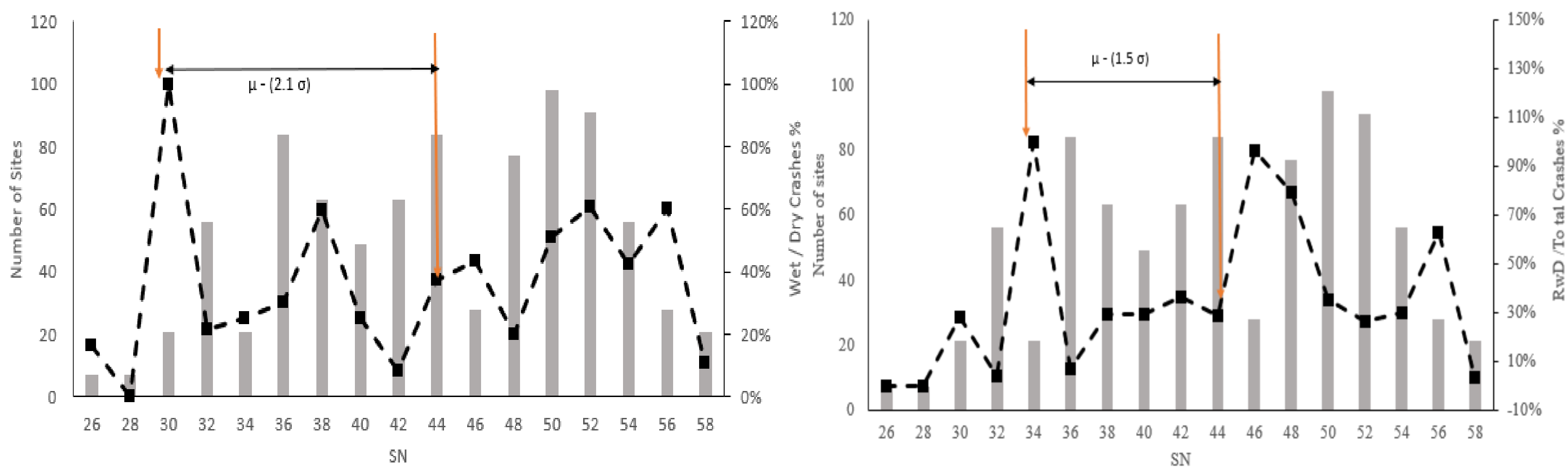


Figure 35: FC5 - Rural Freeways and Expressways - Friction Distribution and Crash Ratio

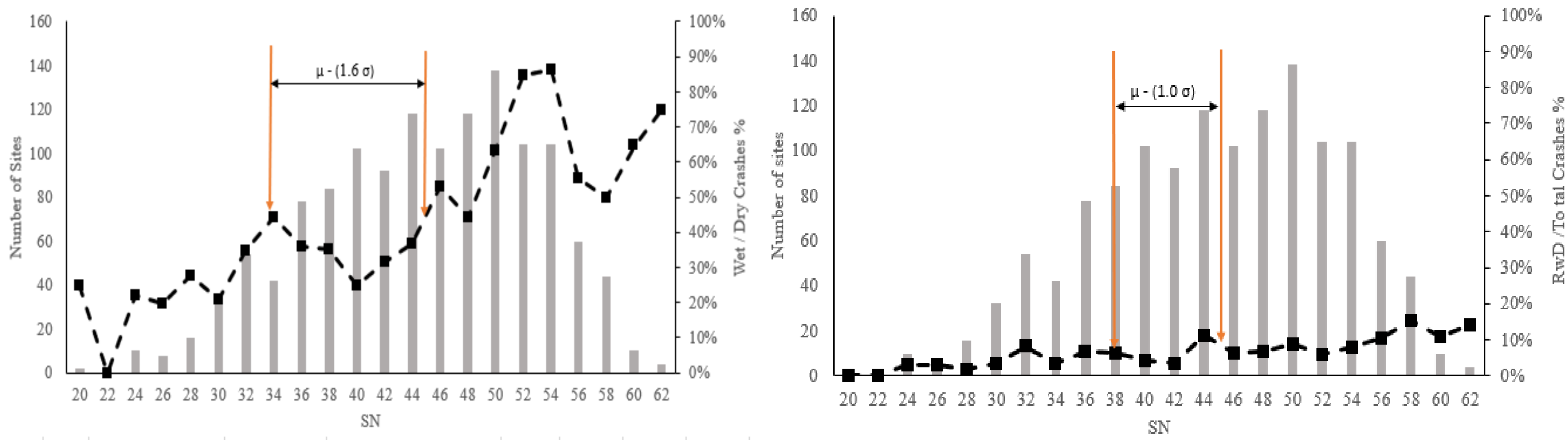


Figure 36: FC6 - Rural Principal Arterials - Friction Distribution and Crash Ratio

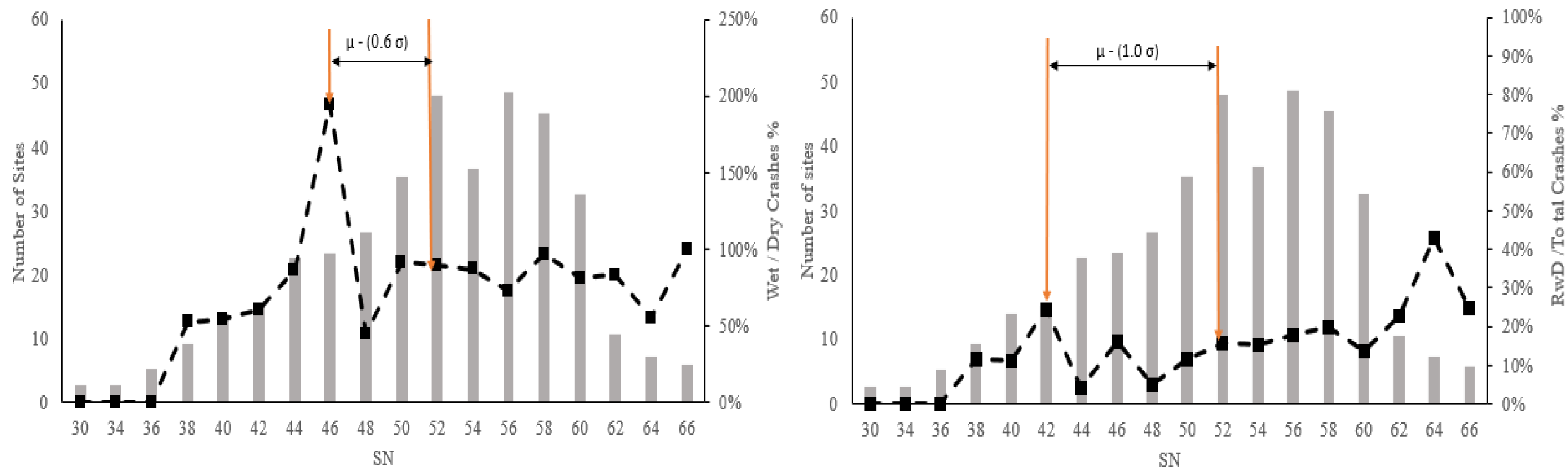


Figure 37: FC7 - Urban Minor Arterials - Friction Distribution and Crash Ratio

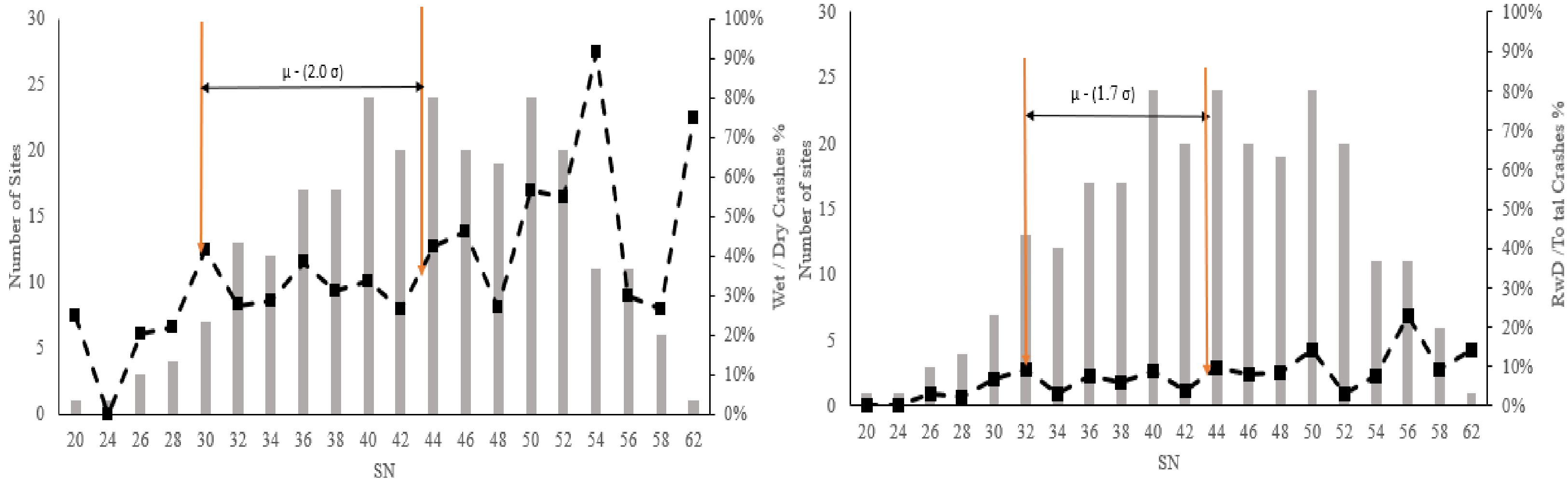


Figure 38: FC8 - Rural Minor Arterials - Friction Distribution and Crash Ratio

The friction distribution for the urban interstates and the urban minor arterials is negatively skewed indicating an overall good condition of the sections in these categories. The friction for the rest of the categories is normally distributed indicating a wide range of friction values, with the majority of the sections being maintained to intermediate friction values.

For the Urban Interstates, the crash rates appear to have a decreasing pattern with improved friction, with a high variability in the peaks amongst it. This reflects a strong association between the wet crashes or the RwD crashes with available friction supply. In contrary, all of the other categories like the rural principal arterials and the collectors, reflected inverted relationships. This could be because there is a strong association between the dry crashes and the available friction as well. This relationship between both, the wet crashes and the dry crashes with the friction can explain the lack of relationship between the wet/dry crash ratio and friction in most of the cases and would bias the use of the wet/dry crash ratio as a variable to define a friction demand level and promotes the usage of exclusive wet or dry crash rates.

However, the overall results show that Urban Interstates (FC1) and Urban Expressway and Freeways (FC3) has the highest suggested I.L.s of 37 and 40 SN40, respectively, when concerned with wet crashes. Not only that these two categories experience higher traffic volumes, but also Interstates and freeways have higher speed limits than those of the other roadway categories and the correlation between wet pavement friction and vehicle speed is recognized since the thirties (Henry 2000). In addition, the I.L. suggested for the interstates agree well with previous research on Iowa interstates by Schram (2001). The research suggested an I.L. of 36 for interstates using the same method (Schram 2011).

On the other hand, the Principal and Minor Arterials have typically lower speed limits than Interstates and Freeways. This results in lower friction requirements. However, the RwD

I.L. for the Urban Minor Arterials (FC7) is found to be higher than most of the other categories. This is probably related to the fact that this roadway category has a mean friction of 51.84 which is the highest among the other classifications indicating being maintained to better levels than the other routes or that the data may not provide enough sections in this category to determine a clear relationship between crashes and friction. The friction demand models for each functional classification are presented in Reports 1 through 7.

The slope of the regression line is negative for all the seven models presented in Reports 1 through 7, indicating an inverse relationship where crash rates are decreasing with increasing friction. However, according to P-values, the data provides enough information to reject the null and verify that not all means are equal and that the friction (SN) is a significant factor in the models that retained a (P-value < 0.05), with a confidence level greater than 95%.

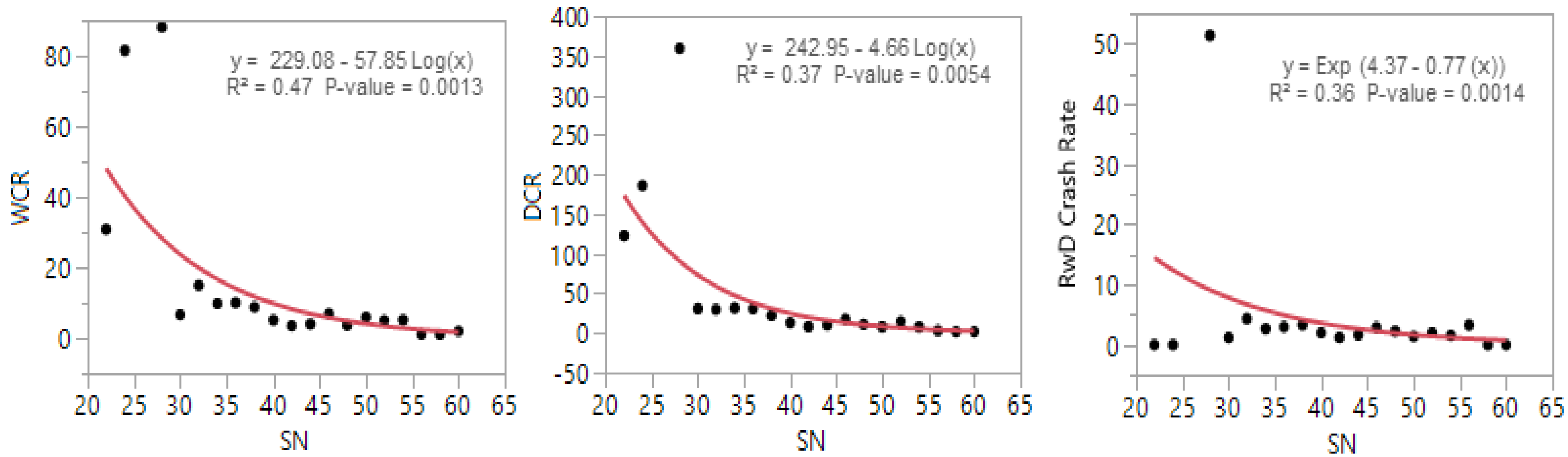
Additionally, to accommodate the low number of RwD crashes that were matched with friction sites in some of the categories where the data did not provide enough information to reject the null, a different probability test had to be performed whenever appropriate to fit a generalized regression model to the data. Some of the categories retained no tangible relationship no matter what probability test was used.

Accordingly, friction is a significant factor in all the models except for the following models (in which P-value > 0.05): Rural Interstates dry and roadway departure crashes, Freeways and Expressway dry-condition crashes, Rural Interstates roadway departure crashes, Urban Principal Arterials roadway departure crashes, and Rural Minor Arterials roadway departure crashes. Also, it is noted that R^2 for these types of roadway is very low and reviewing their plots shows no increase in crash rate with reduced friction.

Report 1: Urban Interstates

Category	Surface Condition/Crash Type	Regression Model	R^2	P-Value	AASHTO IL	Crash Risk at IL (HMVMT)
FC1	Wet	*Y= 229.08-57.85 Log(X)*	0.47	0.0013	37.22	138
	Dry	*Y= 242.95-4.66 Log(X)*	0.37	0.0054	37.22	236
	RwD	*Y= Exp (4.37-0.077 X)*	0.36	0.0014	43.72	3

Where: Y =Crash Rate in (HMVMT) and X = Skid Number (SN40)

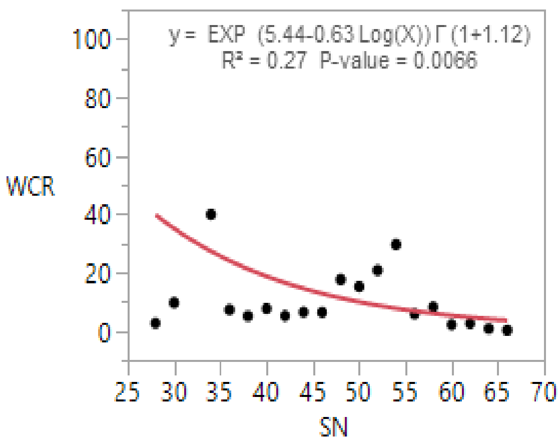


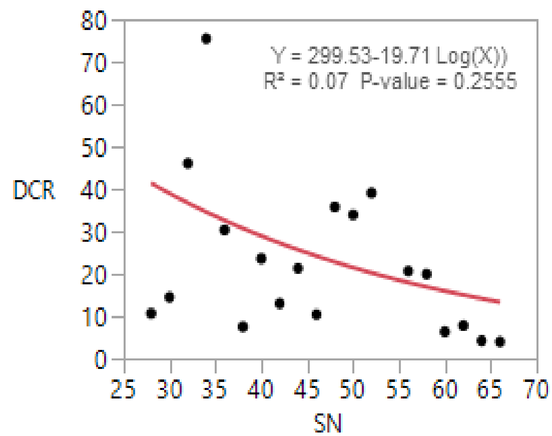
Notes: The slope of the regression fit is negative for the three cases. According to P-values, the three models are significant with a 95% confidence interval. R^2 for the three models is moderate and acceptable. Outliers exists in the three models. Both wet and dry crash rates are fitted to a logarithmic transformed linear model. The shape of the wet curve shows the crash rate appears to increase when SN values drop below 40 and then sharply increase when the friction drops below 38. For the dry curve, a gradual decrease in the crash rates is observed with increasing the friction with some variability where the rates remain somewhat constant for SN values between 40 and 53 before starting to decrease again. However, the RwD crash rates are fitted to a negative binomial model. The shape of the RwD curve shows the crash rate appears to increase when SN values drop below 40 and then sharply increase when the friction drops below 45.

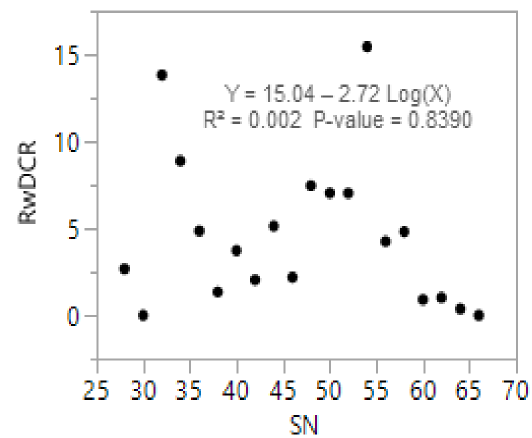
Report 2: Rural Interstates

Category	Surface Condition/ Crash Type	Regression Model	R^2	P-Value	AASHTO IL	Crash Risk at IL (HMVMT)
FC2	Wet	*EXP (5.44-0.63 Log(X)) Γ (1+1.12)*	0.27	0.0066	34.01	93
	Dry	*Y = 299.53-19.71 Log(X)*	0.07	0.2555	34.01	NA
	RwD	*Y = 15.04 – 2.72 Log(X)*	0.002	0.8390	35.95	NA

Where: Y =Crash Rate in (HMVMT) and X = Skid Number (SN40)





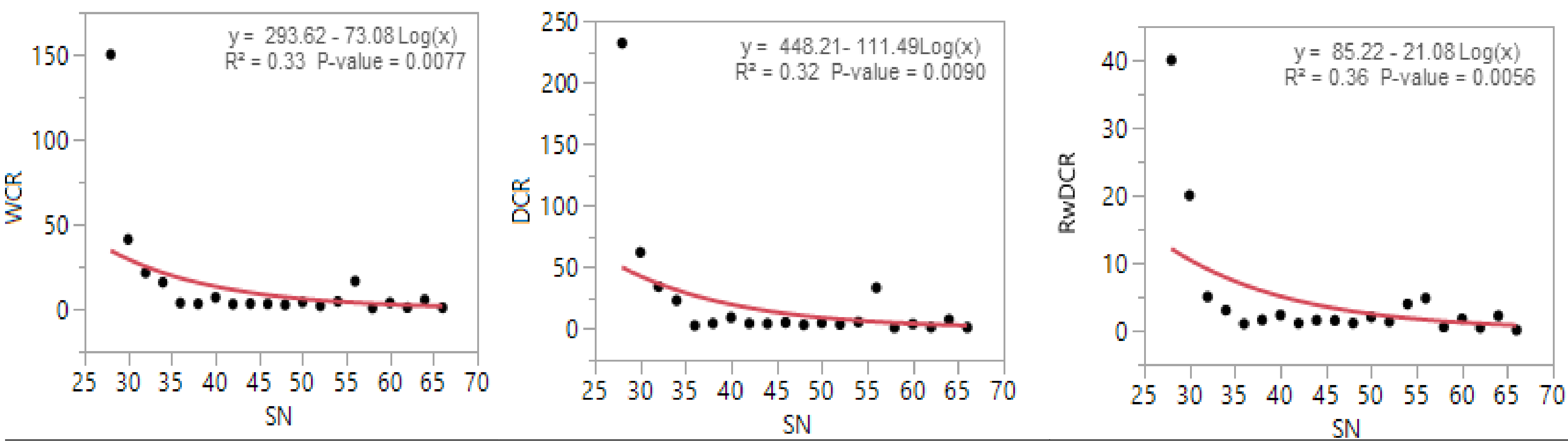


<u>Notes:</u> The slope of the regression fit is negative for the three cases. According to P-values, only the wet model is significant with a 95% confidence interval. Both dry and RwD models are insignificant and are highlighted in red. R^2 for these two models is very low. The wet crash rates model follows a Weibull distribution. The shape of the wet curve shows the crash rate appears to increase when SN values drop below 55 and then sharply increase when the friction drops below 35.

Report 3: Urban Principal Arterials – Freeways and Expressways

Category	Surface Condition/Crash Type	Regression Model	R^2	P-Value	AASHTO IL	Crash Risk at IL (HMVMT)
FC3	Wet	*Y= 293.62-73.08 Log(X)*	0.33	0.0077	40.31	176
	Dry	*Y= 448.21-111.49 Log(X)*	0.32	0.0090	40.31	270
	RwD	*Y= 85.22-21.08 Log(X)*	0.36	0.0056	40.93	51

Where: Y =Crash Rate in (HMVMT) and X = Skid Number (SN40)

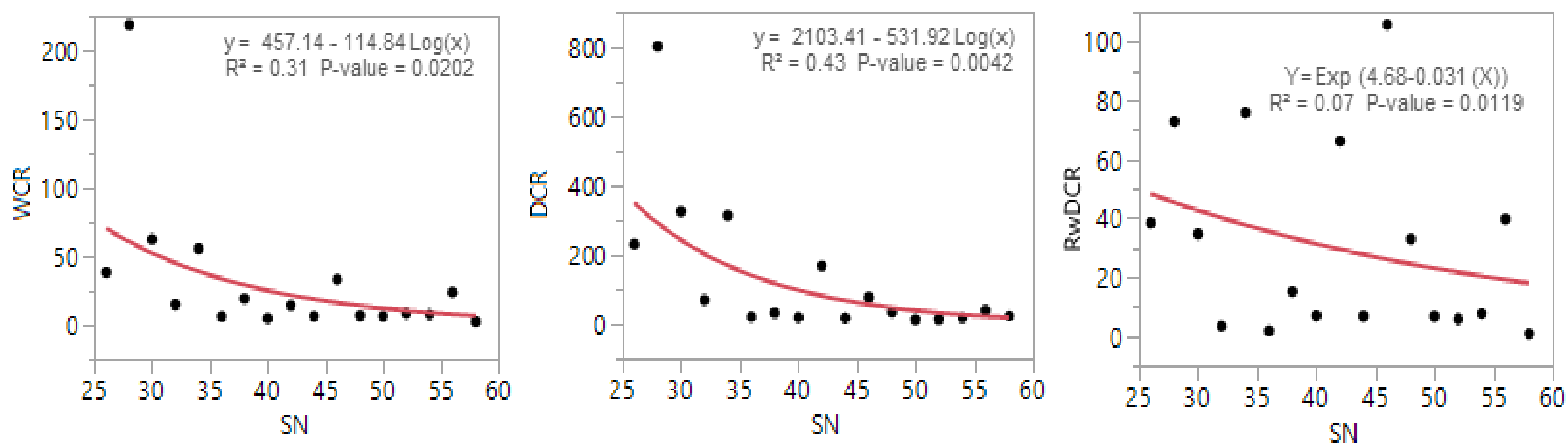


Notes: The slope of the regression fit is negative for the three cases. According to P-values, the three models are significant with a 95% confidence interval. R^2 for the three models is moderate and acceptable. Outliers exists in the three models. All three models are fitted to a logarithmic transformed linear model. The shape of the wet curve shows the crash rate appears to increase when SN values drop below 35. The dry and RwD curves follow similar pattern. The three curves show an increased crash rate at friction values between 50-55.

Report 4: Urban principal arterials – Other

Category	Surface Condition/Crash Type	Regression Model	R^2	P-Value	AASHTO IL	Crash Risk at IL (HMVMT)
FC5	Wet	*Y= 457.14-114.84 Log(X)*	0.31	0.0202	30.11	288
	Dry	*Y= 2103.41-531.92 Log(X)*	0.43	0.0042	30.11	1318
	RwD	*Y= Exp (4.68-0.031 X)*	0.07	0.0119	34.11	NA

Where: Y =Crash Rate in (HMVMT) and X = Skid Number (SN40)

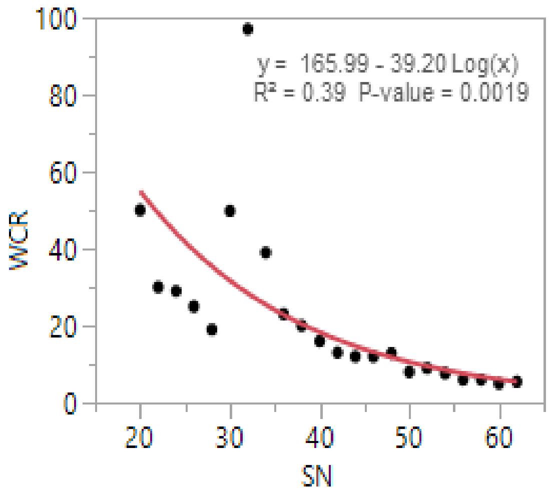


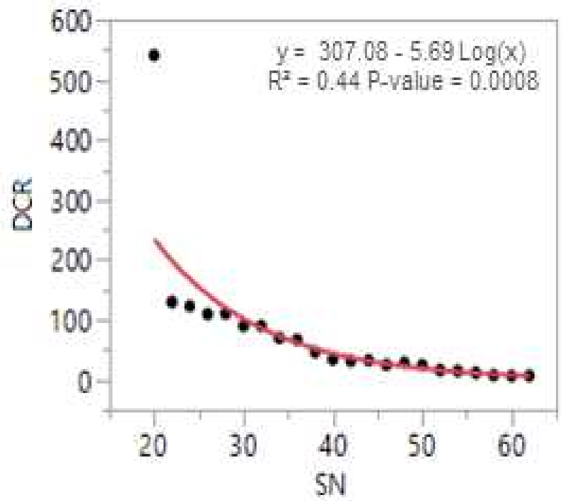
<u>Notes:</u> The slope of the regression fit is negative for the three cases. According to P-values, the wet and dry models are significant with a 95% confidence interval. The RwD model is insignificant and has a very small R^2, the model is highlighted in red. R^2 for the wet and dry models is moderate and acceptable. Outliers exists in the three models. Both wet and dry crash rates are fitted to a logarithmic transformed linear model. The shape of the two curves shows a gradual decrease in the crash rates with increasing the friction with some variability

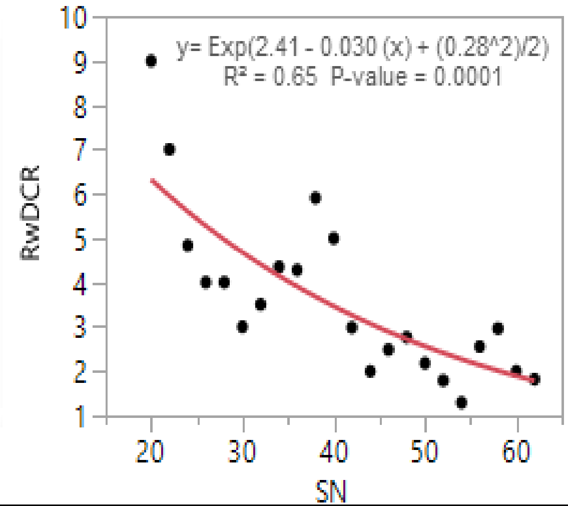
Report 5: Rural principal arterials – Other

Category	Surface Condition/Crash Type	Regression Model	R^2	P-Value	AASHTO IL	Crash Risk at IL (HMVMT)
FC6	Wet	*Y= 165.99-39.20 Log(X)*	0.39	0.0019	33.95	106
	Dry	*Y= 307.08-5.69 Log(X)*	0.44	0.0008	33.95	298
	RwD	*Y= Exp (2.41-0.030 X + (0.28^2)/2)*	0.65	0.0001	38	4

Where: Y =Crash Rate in (HMVMT) and X = Skid Number (SN40)





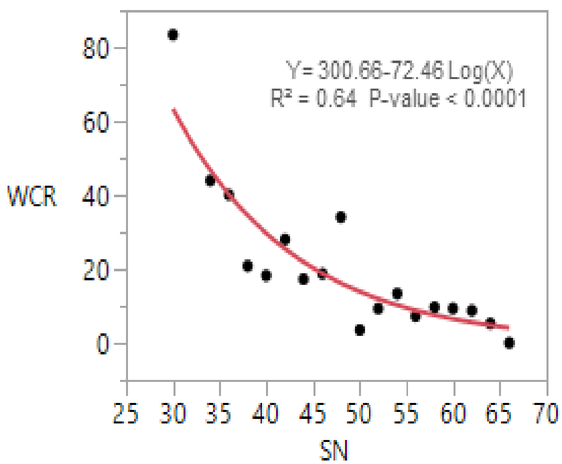


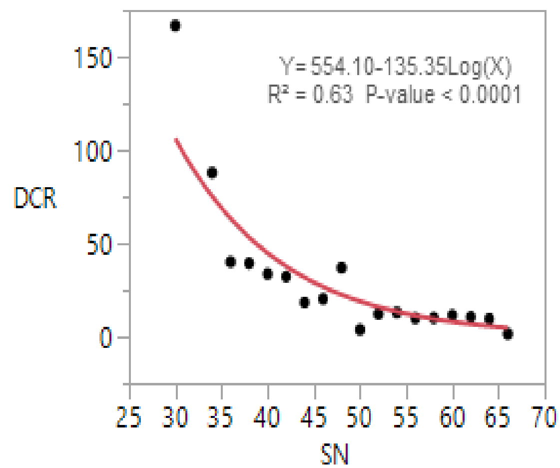
Notes: The slope of the regression fit is negative for the three cases. According to P-values, the three models are significant with a 95% confidence interval. R^2 for the three models is moderate and acceptable. Outliers exists in the three models. Both wet and dry crash rates are fitted to a logarithmic transformed linear model. The shape of the wet curve shows the crash rate appears to increase when SN values drop below 40 and then sharply increase when the friction drops below 38. For the dry curve, a gradual decrease in the crash rates is observed with increasing the friction with some variability where the rates remain somewhat constant for SN values between 40 and 53 before starting to decrease again. However, the RwD crash rates are fitted to a negative binomial model. The shape of the RwD curve shows the crash rate appears to increase when SN values drop below 40 and then sharply increase when the friction drops below 30.

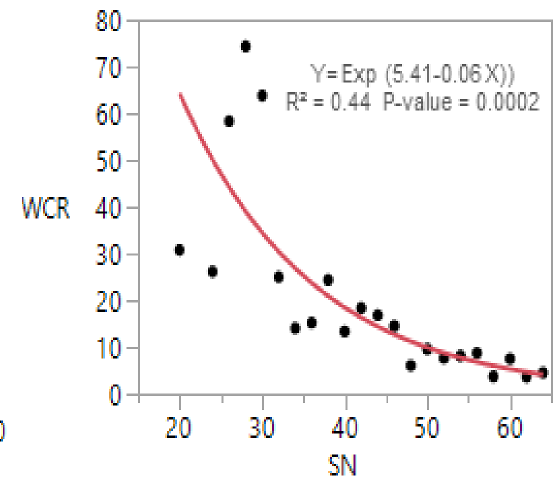
Report 6: Urban Minor Arterials

Category	Surface Condition/Crash Type	Regression Model	R^2	P-Value	AASHTO IL	Crash Risk at IL (HMVMT)
FC7	Wet	Y= 300.66-72.46 Log(X)	0.64	<0.0001	35.07	189
	Dry	Y= 554.10-135.35Log(X)	0.63	<0.0001	35.07	345
	RwD	Y= Exp (5.41-0.06 X)	0.44	0.0002	42.51	17

Where: Y =Crash Rate in (HMVMT) and X = Skid Number (SN40)





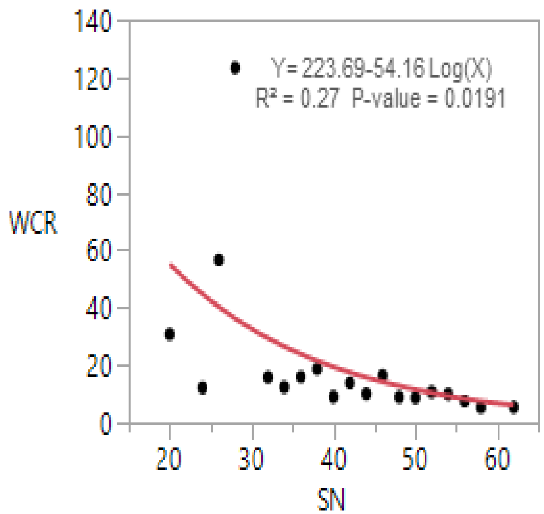


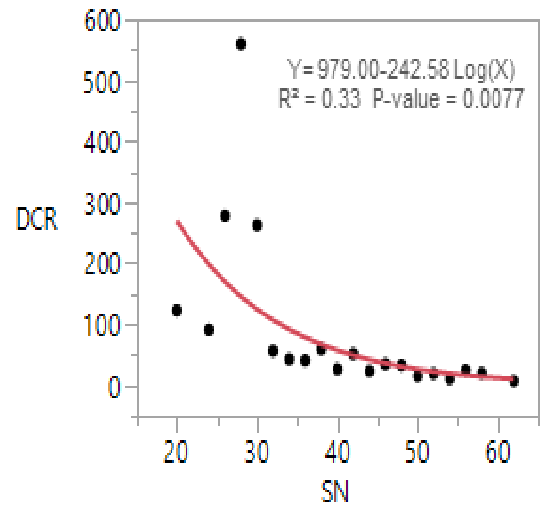
Notes: The slope of the regression fit is negative for the three cases. According to P-values, the three models are significant with a 95% confidence interval. R^2 for the three models is considerably high. Both wet and dry crash rates are fitted to a logarithmic transformed linear model. The shape of the wet curve shows an increase in the crash rates when SN values drop below 55 and then sharply increase when the friction drops below 35. For the dry curve, Similar pattern is observed. However, the RwD crash rates are fitted to a negative binomial model. The shape of the RwD curve shows the crash rate appears to increase when SN values drop below 50 and then sharply increase when the friction drops below 40.

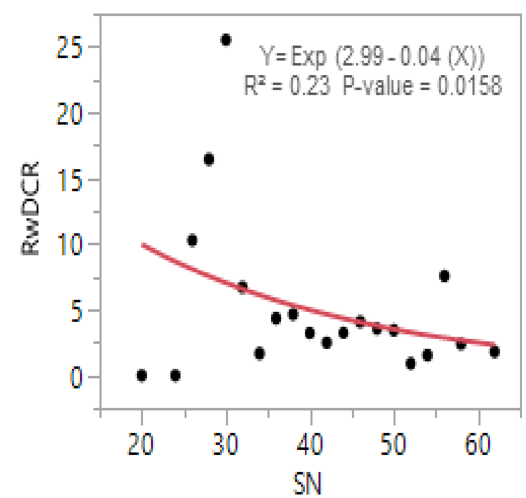
Report 7: Rural Minor Arterials

Category	Surface Condition/Crash Type	Regression Model	R^2	P-Value	AASHTO IL	Crash Risk at IL (HMVMT)
FC8	Wet	Y= 223.69-54.16 Log(X)	0.27	0.0191	30.26	143
	Dry	Y= 979.00-242.58 Log(X)	0.33	0.0077	30.26	620
	RwD	Y= Exp (2.99 - 0.04 X)	0.23	0.0158	32.23	5

Where: Y =Crash Rate in (HMVMT) and X = Skid Number (SN40)







Notes: The slope of the regression fit is negative for the three cases. According to P-values, the three models are significant with a 95% confidence interval. R^2 for the three models is moderate and acceptable. Outliers exists in the three models. Both wet and dry crash rates are fitted to a logarithmic transformed linear model. The shape of the wet curve shows the crash rate appears to increase when SN values drop below 40 and then sharply increase when the friction drops below 35. For the dry curve, a gradual decrease in the crash rates is observed with increasing the friction with some variability. However, the RwD crash rates are fitted to a negative binomial model. The shape of the RwD curve shows the crash rate appears to increase when SN values drop below 40 and then sharply increase when the friction drops below 32.

It was not possible to define an I.L. for these categories. This could be attributed to the fact that friction may not be as critical as in the other categories, or that the data does not provide enough sections in these categories to determine a clear relationship between crashes and friction. Following is an elaborate discussion of the significant reports:

Report 1:

For Urban interstate routes, there is a clear trend in the wet crash rate with respect to roadway surface friction on the investigated routes as indicated by the trend curve and equation illustrated in report 1. The shape of the curve shows that the wet crash rate appears to increase when SN values drop below 38 and then sharply increase when the friction drops below 35. Overall, a gradual decrease in the crash rates is observed with increasing the friction with some variability where the rates remains somewhat constant for SN values between 40 and 55 before starting to decrease again. These findings are consistent with those reported by Schram (2011) in Figure 39, which shows a polynomial fit that indicates a strong correlation between the two variables for Iowa Interstates for an analysis period of 8 years.

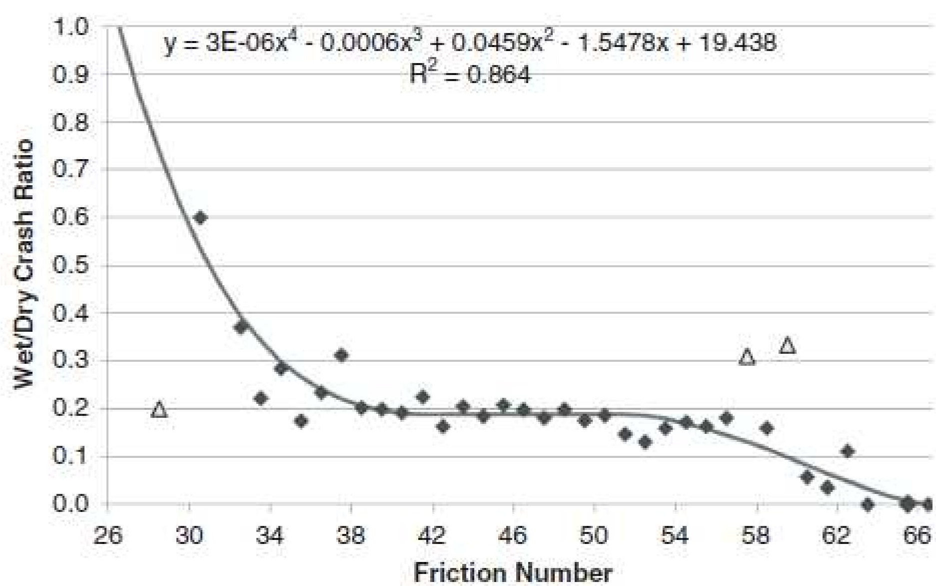


Figure 39: Iowa Interstate's wet to dry crash ratio versus friction number (Schram, 2011)

Furthermore, dry crash rates on urban interstates showed an increase with reduced friction at investigated urban interstate sites. This supports the hypothesis that increasing the friction level decreases the rate of both dry- and wet-condition car crashes and agrees with the findings of (Pardillo, 2009) and (Najafi, 2015).

Additionally, for the urban interstates, the models predict that the suggested I.L.s by the AASHTO method will keep the crash risk at lower than 300 HMVM. For example, the I.L. suggested by the AASHTO method for the WCR model is 37.22, this I.L. level corresponds to a crash risk of 138 HMVM.

Report 2:

For rural interstates, not only that no trend was detected for dry and RwD crashes, but also, the wet crashes model follows a Weibull distribution. This distribution best describes multiple-cause trends. This suggests that friction is not as critical in rural interstates as it is for urban interstates and suggests that urban areas demand higher friction values on its roadways. The WCR model predicts that the suggested I.L. by the AASHTO method of 34.01 corresponds to a crash risk of 91 HMVM.

Report 3:

For Urban Freeways and expressways, the relationship between crashes and friction for the investigated routes shows a clear trend as illustrated by the approximate trend line overlaid to the plot and the model equation in report 3. The shape of the curve shows the ratio of wet to dry crashes appears to increase when SN values drop below 40 and then sharply increase when the friction drops below 38. This agrees with the AASHTO method where the wet/dry crash ratio showed an increase in the slope at an SN level of approximately 40 (mean – 1.2 standard deviations). The models predict that the suggested I.L.s correspond to an approximate wet, dry and roadway departure crash rate of 176, 270 and 51 crash/HMVM, respectively. All below 300 HMVM.

Report 4:

For Urban Principal Arterials, the model presented in report 4 for wet crashes shows some association between crashes and friction, with an increase in crashes as friction decreases. However, no sharp increase in crashes can be observed, which makes it harder to define a threshold I.L. One possible reason is that urban arterials have typically lower posted speed limits than Interstates and freeways, which will result in lower friction requirements. This agrees with the results obtained from the AASHTO method were no patterns were detected for this category as well. These roads also include more intersections with even lower volume roads. However, the models predict that the suggested I.L.s by the AASHTO method will result in considerably higher crash risk than for the other categories. For example, the I.L. suggested by the AASHTO method for the DCR model is 30.11, this I.L. level corresponds to a crash risk of 1318 HMVM. This suggests that urban principal arterials experience more friction related crashes that other roadway categories.

Report 5:

For the rural principal arterials, the models predict that the suggested I.L.s by the AASHTO method will keep the crash risk at lower than 300 HMVM. For example, the I.L. suggested by the AASHTO method for the WCR model is 33.95, this I.L. level corresponds to a crash risk of 106 HMVM.

Report 6:

For urban minor arterials, the models predict that the suggested I.L.s by the AASHTO method will keep the crash risk at lower than 400 HMVM. For example, the I.L. suggested by the AASHTO method for the WCR model is 35.07, this I.L. level corresponds to a crash risk of 189 HMVM.

Report 7:

For rural minor arterials, the models predict that the suggested I.L.s by the AASHTO method will keep the crash risk at lower than 700 HMVM. For example, the I.L. suggested by the AASHTO method for the WCR model is 30.26, this I.L. level corresponds to a crash risk of 143 HMVM.

The overall results from the 7 reports show that Urban Interstates (FC1), Urban Expressway and Freeways (FC3) and Urban Arterials (FC4), demand higher friction supply to maintain the network at the same crash risk exposure. For example, a goal of a crash risk of less than a 100 HMVM can be achieved at friction levels as low as 30 for minor arterials but requires a friction supply of as high as 40 for urban freeways and expressways. This is justified by the higher traffic volumes and the higher speeds on these categories. This also reveals that, the friction requirements for urban areas are higher than those for rural areas. Possible reasons are that urban areas experience higher traffic volumes and more conflict points.

Friction demand models on Tangents

The friction distribution and the wet/dry as well as RwD/total crash ratios for tangents are presented in Figure 40. A summary of the suggested friction demand I.L.s for tangents is provided in Table 10. In these histograms, the investigatory level is set at the value of friction where the mean friction is adjusted towards a specific fraction of the standard deviation. This adjustment is based on the crash ratio pattern. To elaborate, the friction distribution of the sites has a mean friction of 49.33 and a standard deviation of 6.34. Based on the overall decreasing wet/dry crash ratios pattern with the increasing friction, one can observe a sharp drop at a friction value of 32. However, as discussed in the methodology chapter, a significant drop must correspond to at least 5% of the sites, hence, for this case, the next consecutive drop in the crashes around a friction value of 42 is considered. Consequently, the I.L. is set at a value of 41.72 which is the mean minus 1.2 of the standard deviation. The data for the same classification suggests an RwD I.L. of 46.16 which corresponds to 0.5 standard deviations shift from the mean.

The friction distribution for the tangents is also negatively skewed. The crash rates appear to have a decreasing pattern with a high variability in the peaks amongst it. Observing a sharp drop towards the highest friction values reveals that both wet and dry crash rates have dramatically decreased with increasing the friction. This relationship between both, the wet crashes and the dry crashes with the friction can explain the lack of relationship between the wet/dry crash ratio and friction in some cases.

Overall, the results show that the I.L.s required for tangents are the highest amongst all the other categories. In addition, if concerned in RwD crashes, higher friction is demanded. This is supporting the fact that the RwD crashes are more friction related. The friction demand models for tangents are presented in Report 8.

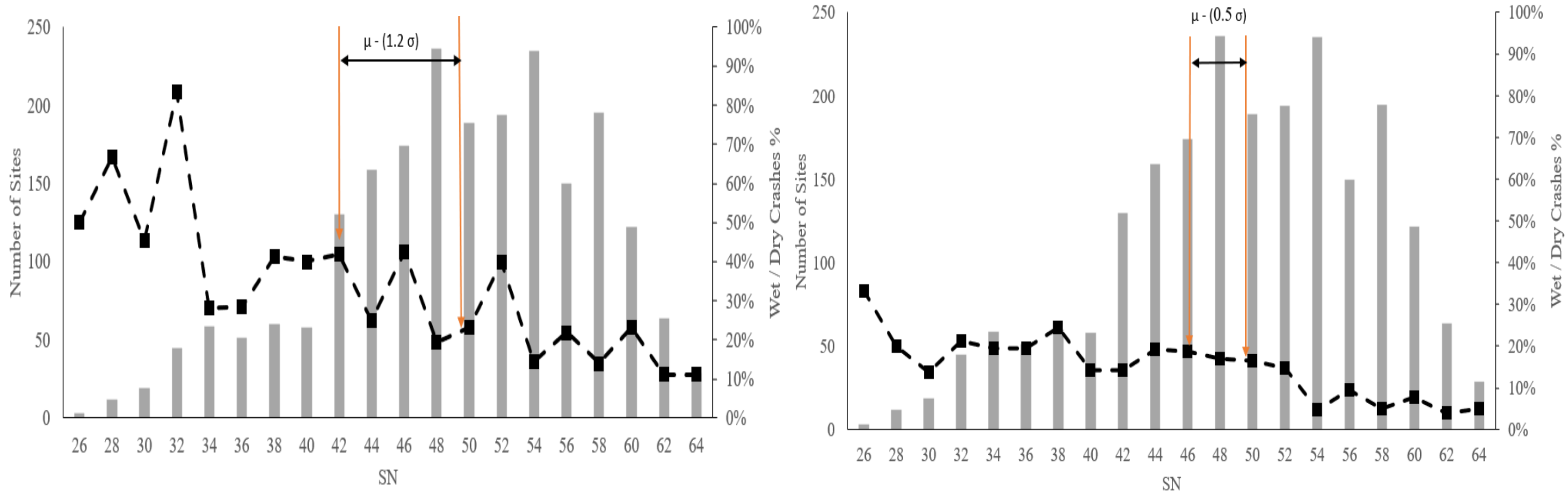


Figure 40: Tangents – Friction Distribution and Crash Ratio

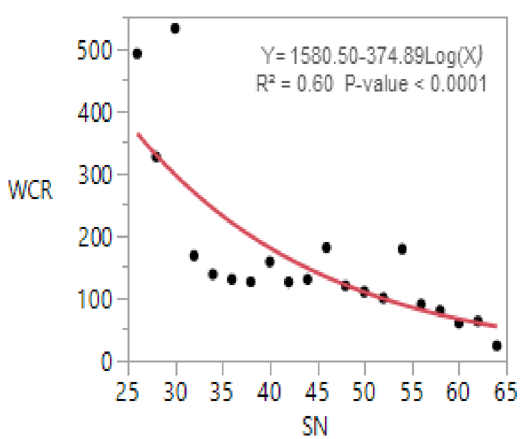
Table 10: Friction investigatory Levels for Tangents

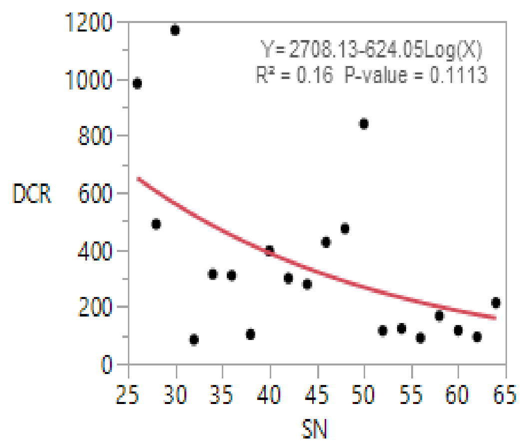
Roadway Geometry	Mean Friction (μ)	Std Dev. Friction (σ)	Wet/Dry IL	RwD/Total IL
Tangents	49.33	6.34	41.72	46.16

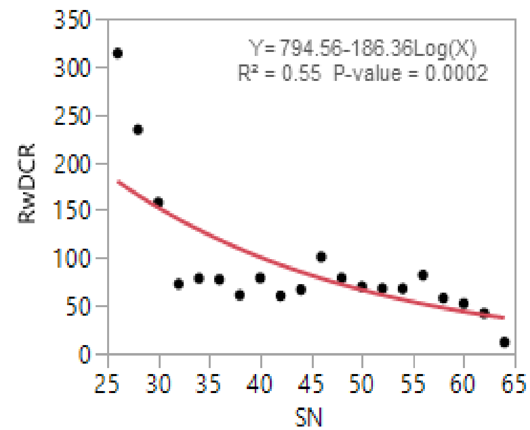
Report 8: Tangents

Category	Surface Condition/Crash Type	Regression Model	R^2	P-Value	AASHTO IL	Crash Risk at IL (HMVMT)
Tangents	Wet	Y= 1580.50-374.89Log(X)	0.60	<0.0001	41.72	973
	Dry	Y= 2708.13-624.05Log(X)	0.16	0.1113	41.72	NA
	RwD	Y= 794.56-186.36Log(X)	0.55	0.0002	46.76	483

Where: Y =Crash Rate in (HMVMT) and X = Skid Number (SN40)







Notes: The slope of the regression fit is negative for the three cases. According to P-values, the wet and RwD models are significant with a 95% confidence interval. R^2 for these models is considerably high. The wet model is insignificant and has a small R^2, the model is highlighted in red. Outliers exist in the three models. All crash rates are fitted to a logarithmic transformed linear model. The shape of the wet curve shows the crash rate appears to increase when SN values drop below 42 and then sharply increase when the friction drops below 40. For the RwD curve, a gradual decrease in the crash rates is observed with increasing the friction with some variability where the rates remain somewhat constant for SN values between 50 and 60 before starting to decrease again.

The slope of the regression line is negative, indicating an inverse relationship where crash rates are decreasing with increasing friction. However, according to P-values, the data provides enough information to reject the null and verify that not all means are equal and that the friction (SN) is a significant factor in the models that retained a (P-value < 0.05), with a confidence level greater than 95%. Accordingly, friction is a significant factor in the wet and the RwD models only. Also, it is noted that R^2 for the dry model is very low and reviewing the plot shows scattered points. Therefore, it was not possible to define an I.L. for the dry crash rates on tangents.

As one would expect, and based on the significance of the models, there is a strong association between the wet and the RwD crash rate with respect to roadway surface friction on the investigated tangents as indicated by the trend curve and equation illustrated in report 8. The shape of the curve shows the ratio of crash rate appears to increase when SN values drop below 42 and then sharply increase when the friction drops below 40. A contributing factor in the goodness of fit is that the investigated tangents are all on high speed routes.

These results are within the range of those reported by (Pardillo and Jurado 2009), who used crash data on two-way rural roads in the Spanish National Road System. The researchers suggested that an SFC50 of 55 and 60 (i.e. 46 to 50 SN40) should be set as the threshold of friction for tangents and curves, respectively. These thresholds are very conservative compared to the practiced policies in the U.S and even in the other European countries.

In addition, the I.L.s suggested by the AASHTO method of 41.72 and 46.76 for WCR and RwDCR, respectively, corresponds to risk levels of 973 and 483 HMVM respectively. This is higher than any crash risk predicted for all the categories at this high friction level.

Friction demand models by speed levels

The friction distribution and the wet/dry as well as RwD/total crash ratios for the speed levels are presented in Figures 41 through 43. In these histograms, the investigatory level is set at the value of friction where the mean friction is adjusted towards a specific fraction of the standard deviation. This adjustment is based on the crash ratio pattern. For example, for the high-speed limit SL1, the friction distribution of the sites has a mean friction of 47.22 and a standard deviation of 7.71. Based on the overall decreasing wet/dry crash ratios pattern with the increasing friction, one can observe a significant sharp drop at a friction value of 30. Consequently, the I.L. is set at a value of 30.22 which is the mean minus 2.2 of the standard deviation. The data for the same classification suggests an RwD I.L. of 33.89 which corresponds to 1.7 standard deviations shift from the mean. A summary of the suggested friction demand I.L.s for each classification based on its crash ratio and friction distribution histogram is provided in Table 11.

Table 11: Friction investigatory Levels by Speed Levels

Functional Classification	Mean Friction (μ)	Std Dev. Friction (σ)	Wet/Dry IL	RwD/Total IL
SL1	47.22	7.71	30.25	33.89
SL2	46.30	7.30	34.11	42.59
SL3	49.03	7.27	38.20	34.19

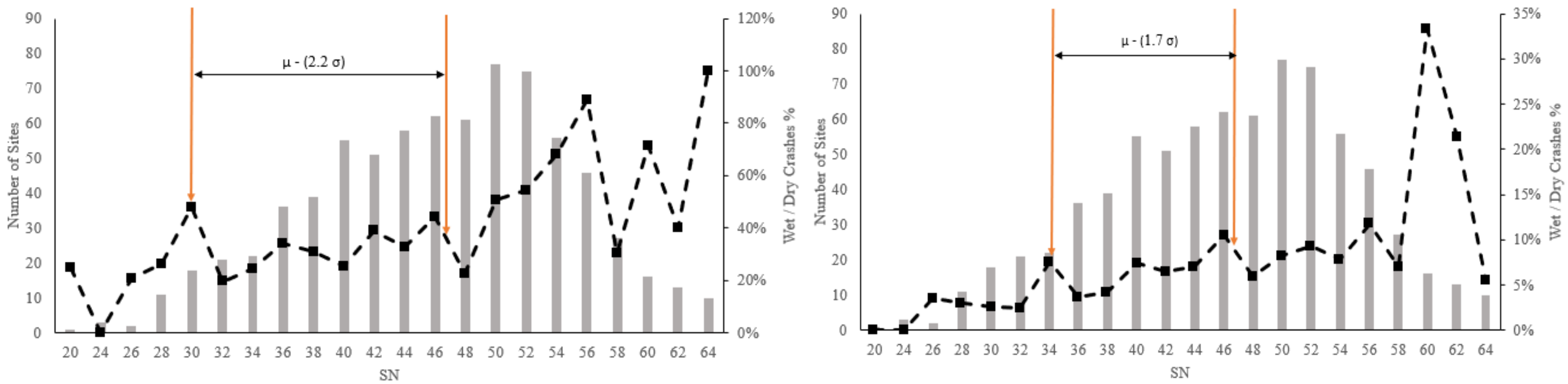


Figure 41: SL1 - Low Speed – Friction Distribution and Crash Ratio

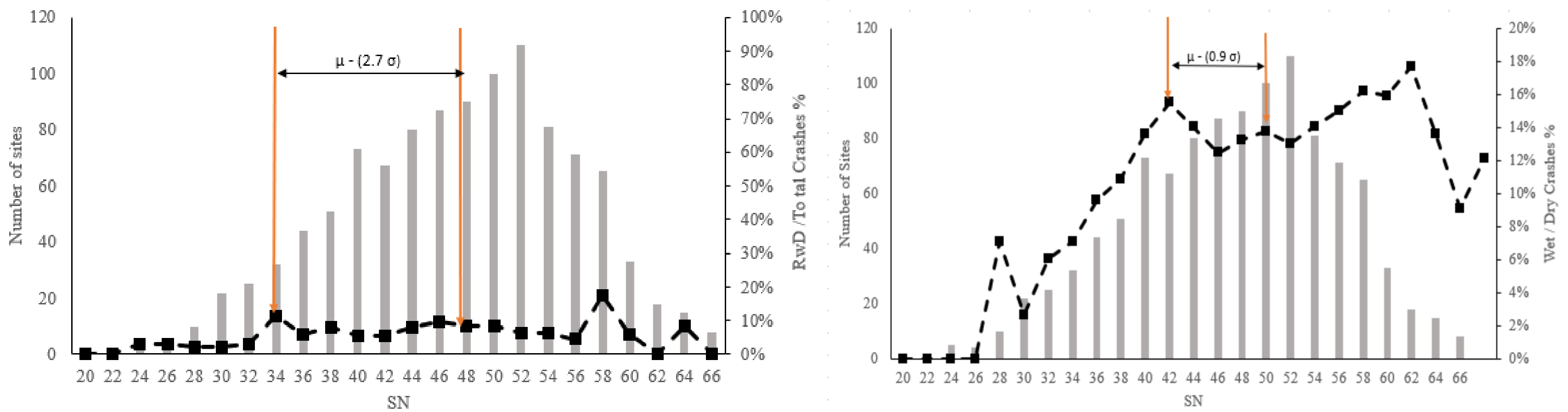


Figure 42: SL2 - Medium Speed – Friction Distribution and Crash Ratio

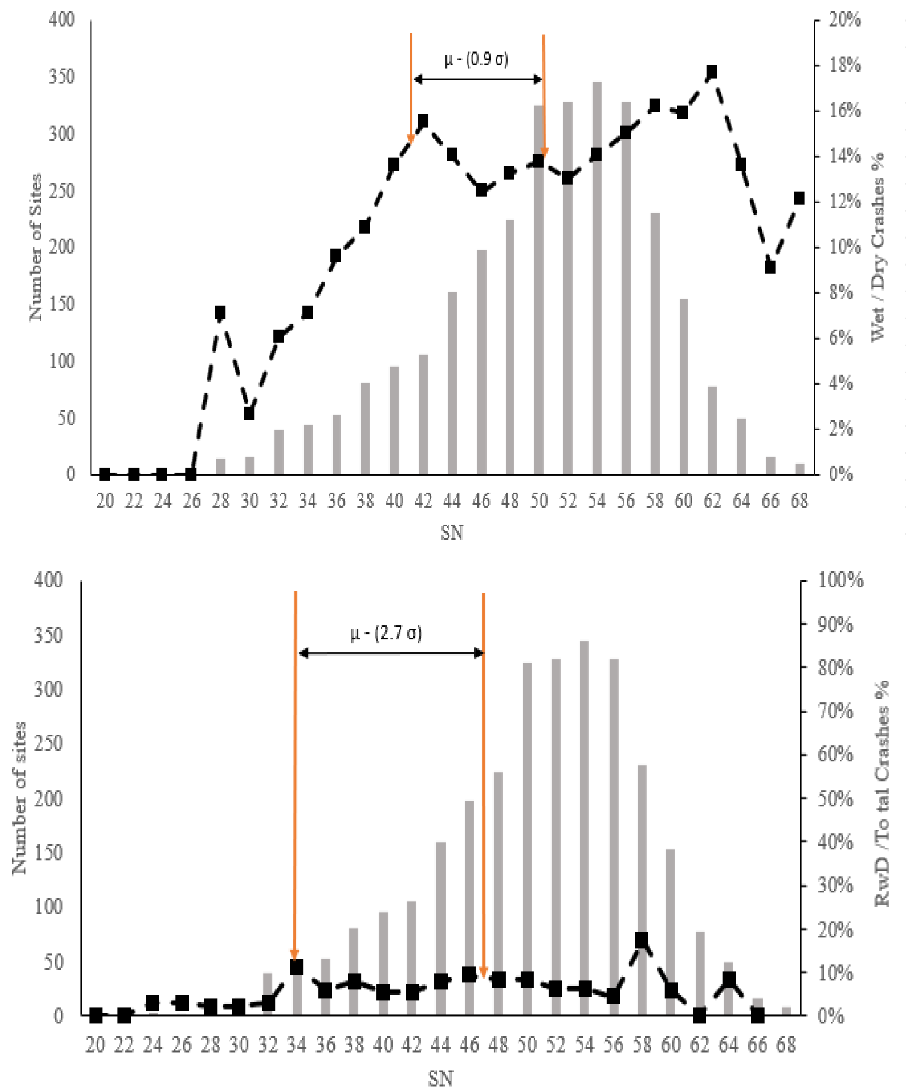


Figure 43: SL3 - High Speed – Friction Distribution and Crash Ratio

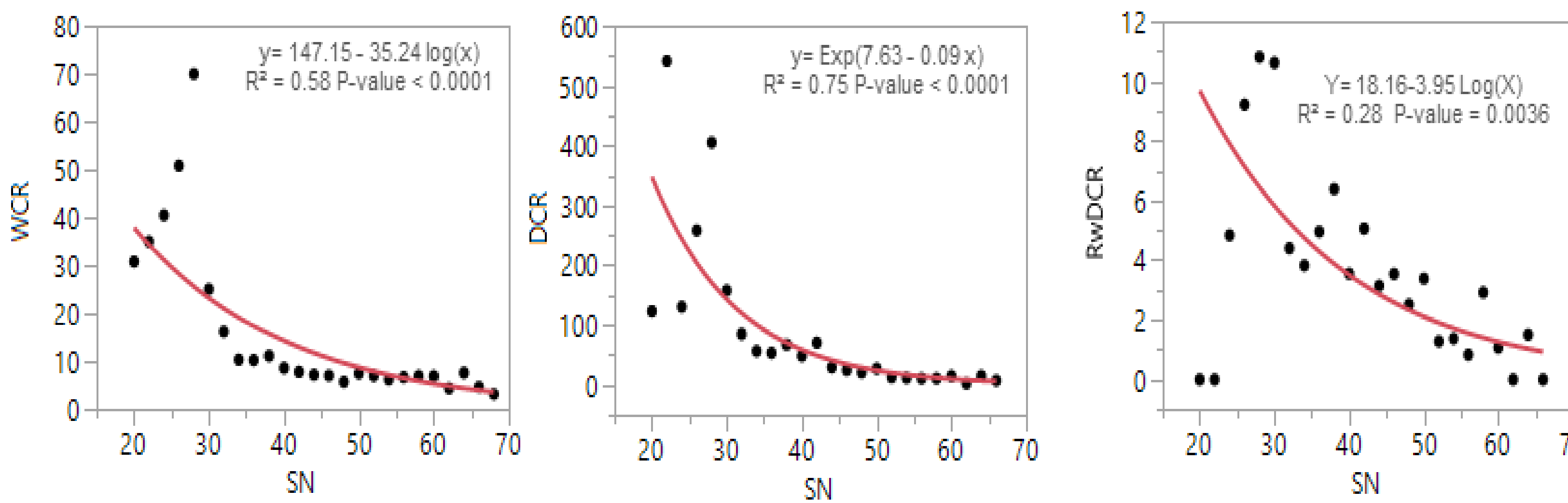
The friction values for the low and medium speeds seems to be normally distributed. However, the high-speed sections friction distribution is negatively skewed. The wet/Dry crash ratio doesn't appear to have a pattern with improved friction and here is a high variability in the peaks amongst it. This reflects a strong association between the wet crashes and the dry crashes with available friction supply. This relationship between both, the wet crashes and the dry crashes with the friction can explain the lack of relationship between the wet/dry crash ratio and friction in some cases.

Overall, the suggested I.L.s show that a higher friction level is required for higher speeds. One possible reason is that the interaction between the micro and macro textures is more pronounce at higher speeds (Roe and Sinhal 1998). This means that the friction decreases as the slip speed of the tire increases (Murad 2019; Yanase, 2014). These findings agree with a study by Al Hassan, et al. (2018). The study concluded that higher friction values reduced the roadway departure crash rates. However, the relationship was more profound on segments with higher speed limits. The friction demand models for each speed level are presented in Reports 9 through 11.

Report 9: Low Posted Speed Limit (<=35 mph)

Category	Surface Condition/Crash Type	Regression Model	R^2	P-Value	AASHTO IL	Crash Risk at IL (HMVMT)
SL1	Wet	Y= 147.15-35.24 Log(X)	0.53	<0.0001	30.25	95
	Dry	Y= Exp (7.63 - 0.09 X)	0.75	<0.0001	30.25	135
	RwD	Y= 18.16-3.95Log(X)	0.28	0.0036	33.89	12

Where: Y =Crash Rate in (HMVMT) and X = Skid Number (SN40)

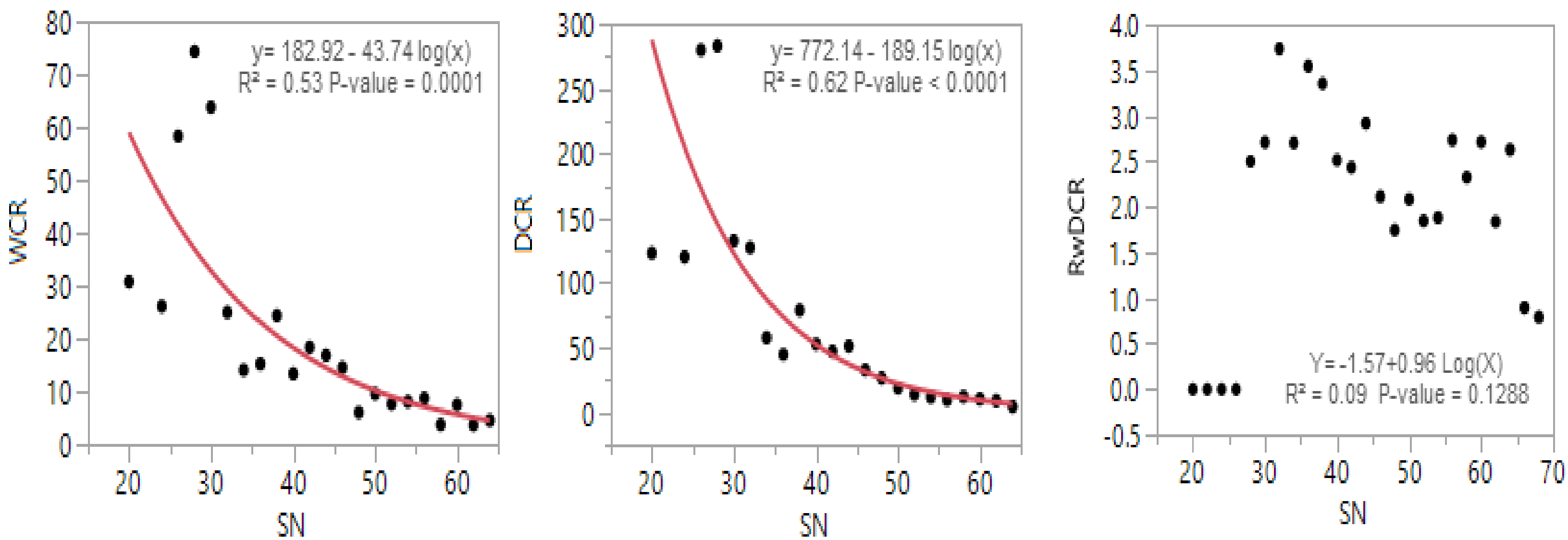


<u>Notes:</u> The slope of the regression fit is negative for the three cases. According to P-values, the three models are significant with a 95% confidence interval. R^2 for the wet and the RwD models is considerably high. R^2 For the dry model is moderate and acceptable. Outliers exist in the three models. Both wet and RwD crash rates are fitted to a logarithmic transformed linear model. The shape of the wet curve shows the crash rate appears to increase when SN values drop below 40 and then sharply increase when the friction drops below 35. For the dry curve, a gradual decrease in the crash rates is observed with increasing the friction with some variability where the rates remain somewhat constant for SN values between 45 and 60 before starting to decrease again. However, the RwD crash rates are fitted to a negative binomial model. The shape of the RwD curve shows the crash rate appears to retain a gradual increase in the crash rates with decreasing SN values.

Report 10: Medium Posted Speed Limit (40-55 mph)

Category	Surface Condition/Crash Type	Regression Model	R^2	P-Value	AASHTO IL	Crash Risk at IL (HMVMT)
SL2	Wet	Y= 182.92-43.74 Log(X)	0.53	0.0001	34.11	116
	Dry	Y= 772.14-189.15 Log(X)	0.62	<0.0001	34.11	482
	RwD	Y= -1.57+0.96 Log(X)	0.09	0.1288	42.59	NA

Where: Y =Crash Rate in (HMVMT) and X = Skid Number (SN40)

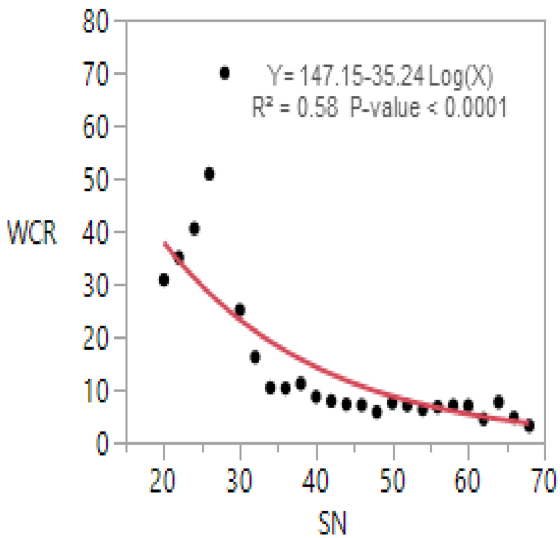


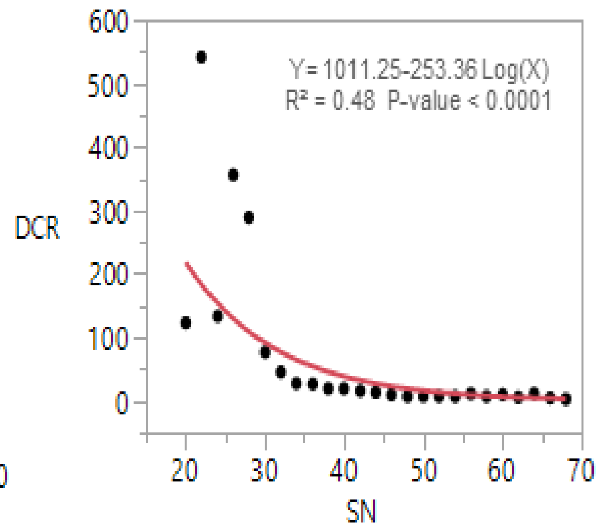
<u>Notes:</u> The slope of the regression fit is negative for the wet and dry models. According to P-values, the wet and dry models are significant with a 95% confidence interval. R^2 for the wet and dry models is considerably high. The RwD model is insignificant and has a very small R^2, the model is highlighted in red. Outliers exist in the three models. Both wet and dry crash rates are fitted to a logarithmic transformed linear model. The shape of the wet curve shows a gradual decrease in the crash rates with increasing the friction with some variability. For the dry curve, the crash rate appears to increase when SN values drop below 50 and then sharply increase when the friction drops below 35.

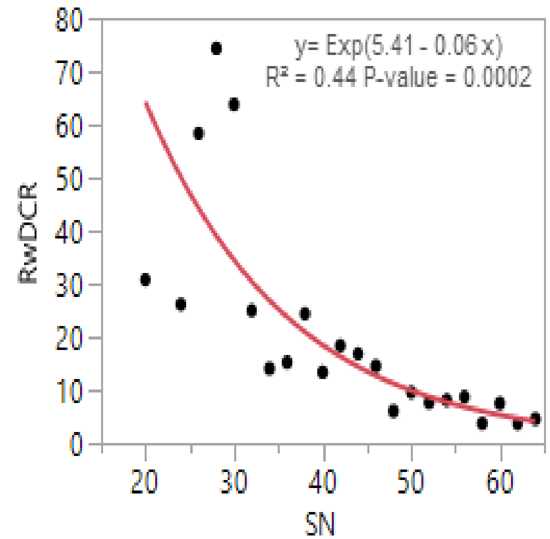
Report 11: High Posted Speed Limit (> 55 mph)

Category	Surface Condition/Crash Type	Regression Model	R^2	P-Value	AASHTO IL	Crash Risk at IL (HMVMT)
SL3	Wet	Y= 147.15-35.24 Log(X)	0.58	<0.0001	38.20	91
	Dry	Y= 1011.25-253.36 Log(X)	0.48	<0.0001	38.20	610
	RwD	Y= Exp (5.41 -0.06 X)	0.44	0.0002	38.2	29

Where: Y =Crash Rate in (HMVMT) and X = Skid Number (SN40)







Notes: The slope of the regression fit is negative for the three cases. According to P-values, the three models are significant with a 95% confidence interval. R^2 for the dry and RwD models is moderate and acceptable. R^2 For the wet model is considerably high. Outliers exist in the three models. Both wet and dry crash rates are fitted to a logarithmic transformed linear model. The shape of the wet curve shows the crash rate appears to sharply increase when SN values drop below 35. For the dry curve, a gradual decrease in the crash rates is observed with increasing the friction with some variability where the rates remain somewhat constant for SN values between 40 and 60 before starting to decrease again. However, the RwD crash rates are fitted to a negative binomial model. The shape of the RwD curve shows the crash rate appears to retain a gradual increase in the crash rates with decreasing SN values.

There appears to be a unique case of a positive regression line for RwD crashes at medium speeds as shown in Report 10. However, this model is insignificant and has a very low R^2, therefore, it was not possible to define an I.L. for this category. This could be attributed to the fact that friction may not be as critical as in the other categories, or that the data does not provide enough sections in this category to determine a clear relationship between crashes and friction.

However, the slope of the regression line is negative for the rest of the models presented in Reports 9 through 11. This indicate an inverse relationship where crash rates are decreasing with increasing friction. However, according to P-values, the data provides enough information to reject the null and verify that not all means are equal and that the friction (SN) is a significant factor in the models that retained a (P-value < 0.05), with a confidence level greater than 95%.

For all the speed categories, there is a clear trend in the Wet Crash Rate with respect to roadway surface friction on the investigated routes as indicated by the trend curves and equations illustrated in the reports. Following is an elaborate discussion of the significant reports:

Report 9:

For the low-speed range, the models predict that the suggested I.L.s by the AASHTO method will keep the crash risk at lower than 200 HMVM. For example, the I.L. suggested by the AASHTO method for the WCR model is 30.25, this I.L. level corresponds to a crash risk of 95 HMVM.

Report 10:

For the medium-speed limit category, the data failed to provide meaningful regression to describe the relationship between RwD crashes and friction. One possible reason is that the distribution of friction seems bimodal as it shows two different picks suggesting that this category includes different types of roads. Additionally, the reports show that the suggested I.L.s by the AASHTO method will keep the crash risk at lower than 500 HMVM. For example, the I.L. suggested by the AASHTO method for the WCR model is 34.11. This I.L. level corresponds to a crash risk of 482 HMVM.

Report 11:

For the high-speed range, the models predict that the suggested I.L.s by the AASHTO method will keep the crash risk at lower than 700 HMVM. For example, the I.L. suggested by the AASHTO method for the WCR model is 38.4, this I.L. level corresponds to a crash risk of 91 HMVM.

It can be seen that higher speeds demand higher friction supply to maintain the network at the same crash risk exposure. For example, A goal of a crash risk of less than a 100 HMVM can be obtained at friction levels as low as 31 for low speeds but requires a friction supply of as high as 38 for higher speed sections.

Friction demand models by AADT

The friction distribution and the wet/dry as well as RwD/total crash ratios for the AADT ranges are presented in Figures 44 through 46

In these histograms, the investigatory level is set at the value of friction where the mean friction is adjusted towards a specific fraction of the standard deviation. This adjustment is based on the crash ratio pattern. For example, for the high AADT range AADT3, the friction distribution of the sites has a mean friction of 46.57 and a standard deviation of 7.05. Based on the overall decreasing wet/dry crash ratios pattern with the increasing friction, one can observe a sharp drop at a friction value of 28. However, as discussed in the methodology chapter, a significant drop must correspond to at least 5% of the sites, hence, for this case, the next consecutive drop in the crashes around a friction value of 36 is considered. Consequently, the

I.L. is set at a value of 36.02 which is the mean minus 1.6 of the standard deviation. The data for the same classification suggests an RwD I.L. of 36.70 which corresponds to 1.4 standard deviations shift from the mean. A summary of the suggested friction demand I.L.s for each classification based on its crash ratio and friction distribution histogram is provided in Table 12.

Table 12: Friction investigatory Levels by AADT Range

Functional Classification	Mean Friction (μ)	Std Dev. Friction (σ)	Wet/Dry IL	RwD/Total IL
AADT1	50.71	9.11	36.13	36.13
AADT2	49.78	8.52	37.85	42.11
AADT3	46.57	7.05	36.02	36.70

The friction distribution for the three AADT ranges is negatively skewed. In addition, the crash rates appear to have a decreasing pattern with a high variability in the peaks amongst it. This means that the decrease in the dry rates is stronger than the decrease in the wet crash rates causing sharp drops towards the highest friction values. This reveals that both wet and dry crash rates have dramatically decreased with increasing the friction. Such sharp drops are observed in the wet/dry rate at low AADT, RwD rate at medium AADT and the RwD rate at high AADT.

Overall, the results show that a higher friction level is required for the medium AADT range. One possible reason is that this range underlies the highest percentage of the tested sections. The friction demand models for each AADT range are presented in Reports 12 through 14.

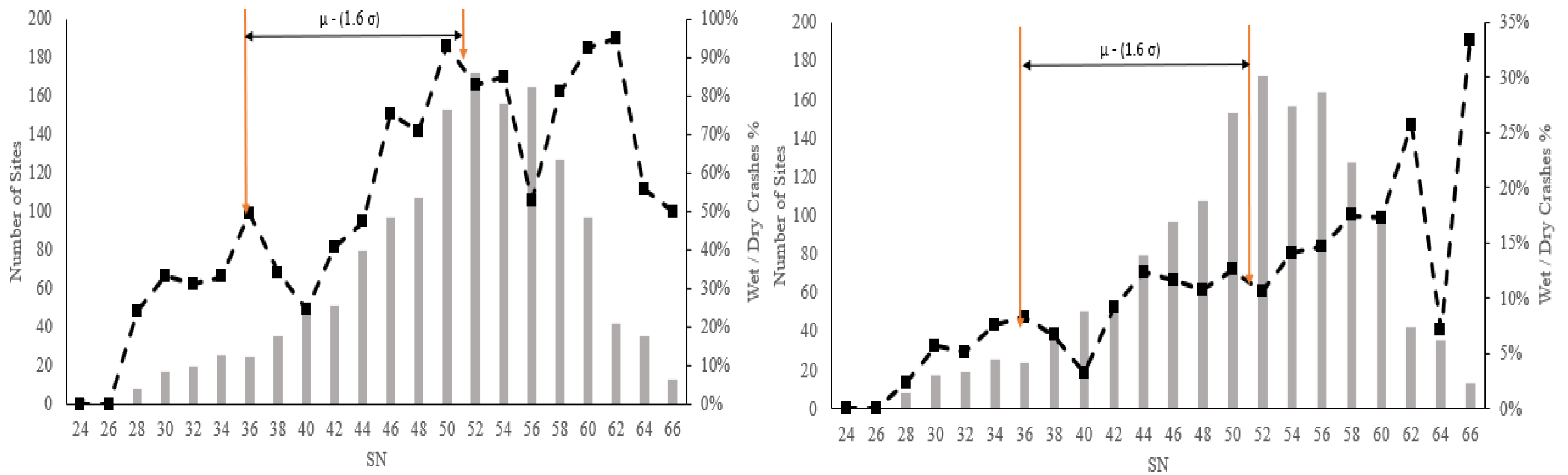


Figure 44: AADT1 - Low AADT – Friction Distribution and Crash Ratio

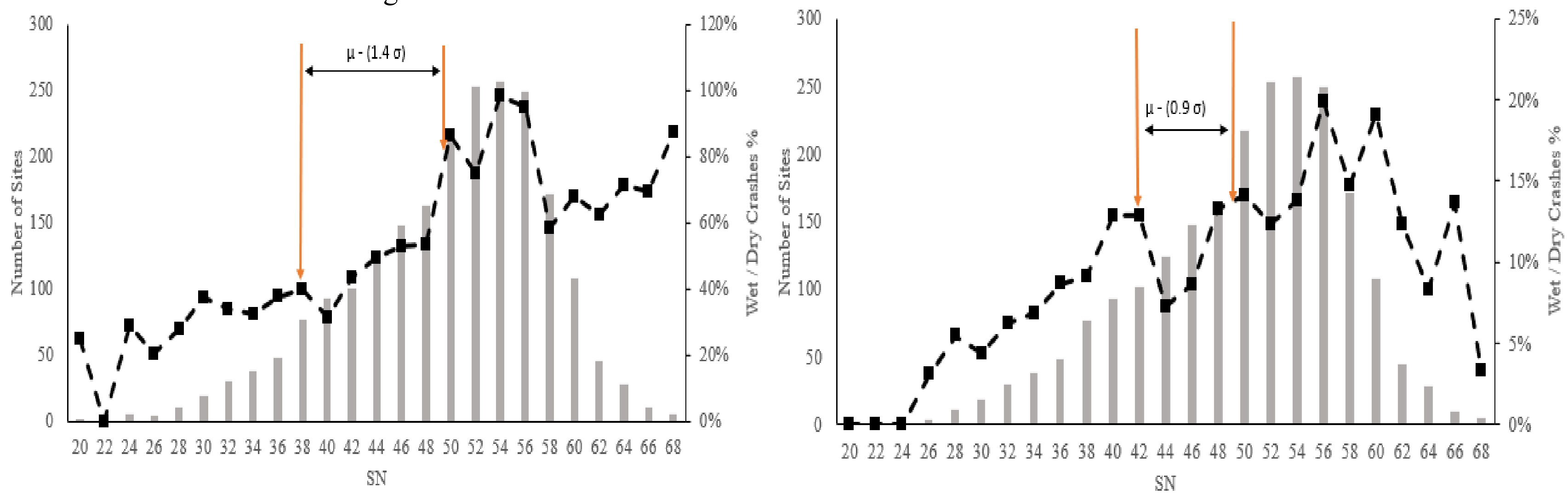


Figure 45: AADT2 - Medium AADT – Friction Distribution and Crash Ratio

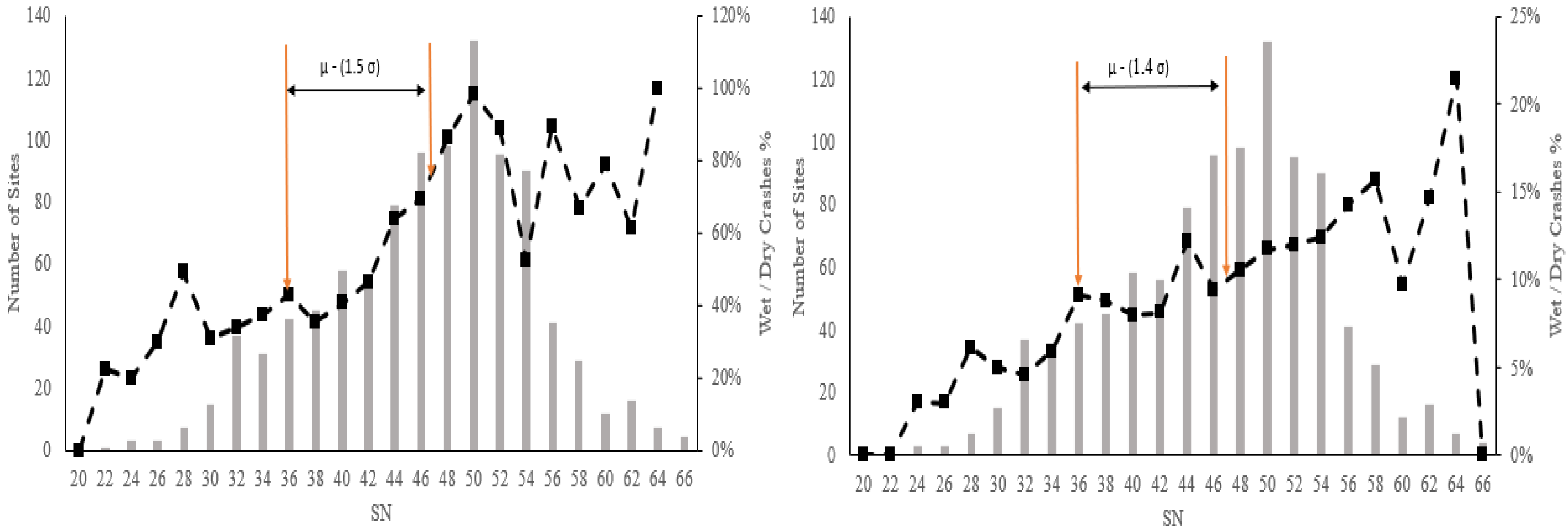
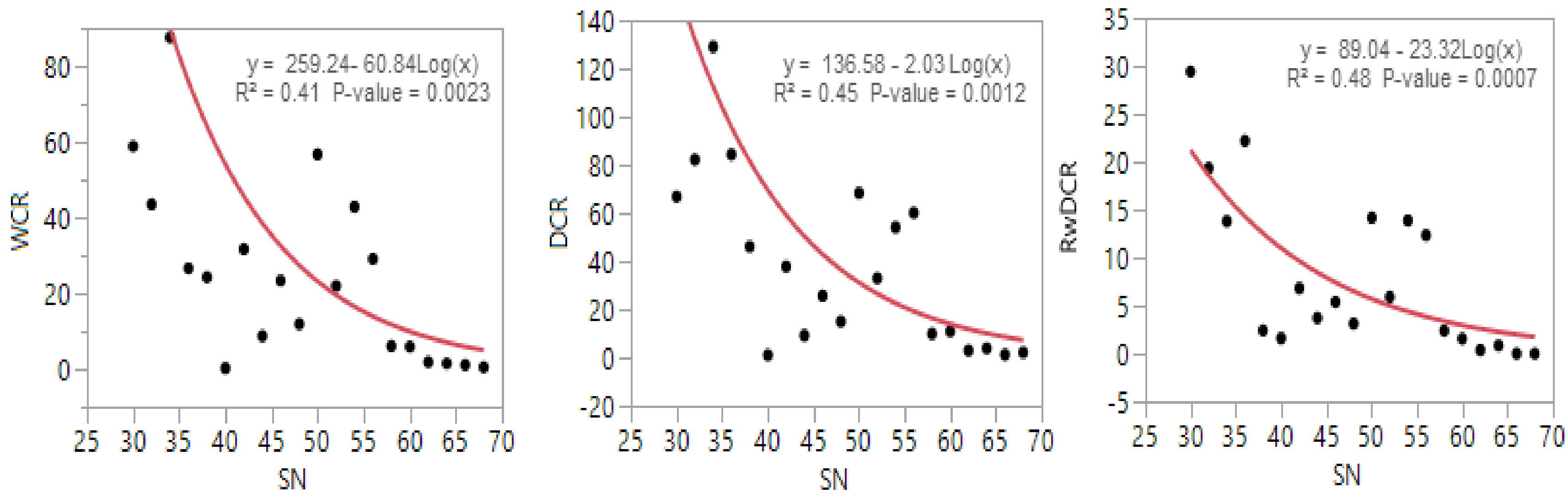


Figure 46: AADT3 - High AADT – Friction Distribution and Crash Ratio

Report 12: Low Traffic Volume (AADT < 1,000)

Category	Surface Condition/Crash Type	Regression Model	R^2	P-Value	AASHTO IL	Crash Risk at IL (HMVMT)
AADT1	Wet	*Y= 259.24 - 60.84 Log(X)*	0.14	0.2223	36.13	20
	Dry	y = 242.95 - 4.66 Log(x)	0.20	0.1112	36.13	611
	RwD	*Y= Exp (2.72 – 0.03X)*	0.27	0.1001	36.13	5

Where: Y =Crash Rate in (HMVMT) and X = Skid Number (SN40)

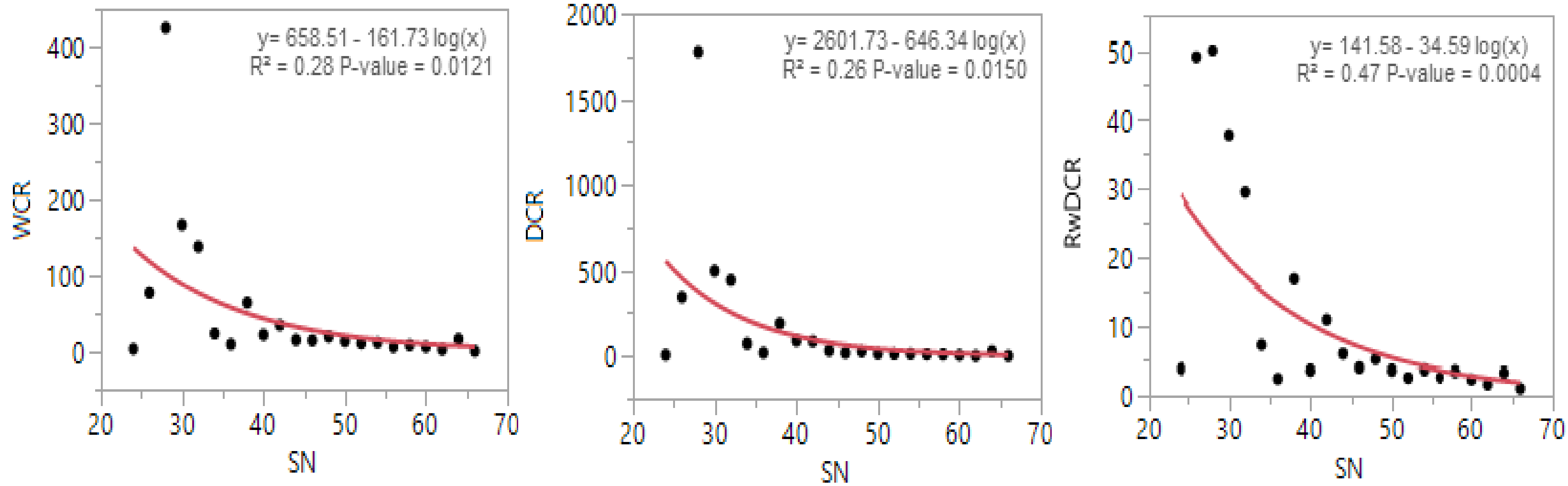


Notes: The slope of the regression fit is negative for the three cases. According to P-values, the three models are insignificant with a 95% confidence interval. The three models are highlighted in red. R^2 for the three models is low. The three models fail to provide information on the data trend.

Report 13: Medium Traffic Volume (1,000<=AADT < 10,000)

Category	Surface Condition/Crash Type	Regression Model	R^2	P-Value	AASHTO IL	Crash Risk at IL (HMVMT)
AADT2	Wet	*Y= 658.51-161.73 Log(X)*	0.28	0.0121	37.85	407
	Dry	*Y= 2601-646.34 Log(X)*	0.26	0.0150	37.85	1595
	RwD	*Y= 141.58-34.59 Log(X)*	0.47	0.0004	42.11	88

Where: Y =Crash Rate in (HMVMT) and X = Skid Number (SN40)

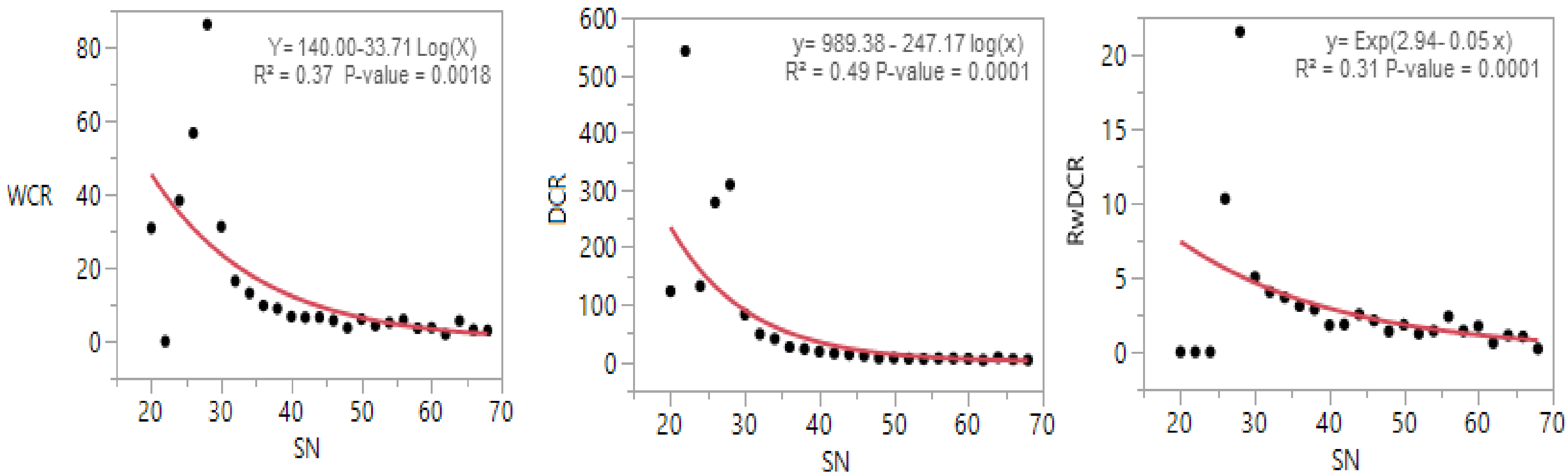


Notes: The slope of the regression fit is negative for the three cases. According to P-values, the three models are significant with a 95% confidence interval. R^2 for the three models is moderate and acceptable. Outliers exist in the three models. The three models are fitted to a logarithmic transformed linear model. The shape of the wet and dry curves show the crash rate appears to increase when SN values drop below 35. Both of the curves show a gradual decrease in the crash rates is with increasing the friction with some variability where the rates remain somewhat constant for SN values between 50 and 60. However, the RwD crash rates curve shows the crash rate appears to retain a gradual increase in the crash rates with decreasing SN values and then appears to experience a sharp increase in the crash rates when SN drops below 45.

Report 14: High Traffic Volume (AADT >= 10,000)

Category	Surface Condition/Crash Type	Regression Model	R^2	P-Value	AASHTO IL	Crash Risk at IL (HMVMT)
AADT3	Wet	Y= 140.00-33.71 Log(X)	0.37	0.0012	36.02	87
	Dry	Y= 989.38-247.17 Log(X)	0.49	0.0001	36.02	599
	RwD	Y= Exp(2.94-0.05X)	0.31	0.0001	36.70	3

Where: Y =Crash Rate in (HMVMT) and X = Skid Number (SN40)



Notes: The slope of the regression fit is negative for the three cases. According to P-values, the three models are significant with a 95% confidence interval. R^2 for the three models is moderate and acceptable. Outliers exist in the three models. Both wet and dry crash rates are fitted to a logarithmic transformed linear model. The shape of the wet curve shows the crash rate appears to sharply increase when SN values drop below 40. For the dry curve, a gradual decrease in the crash rates is observed with increasing the friction with some variability where the rates remain somewhat constant for SN values between 50 and 60. However, the RwD crash rates are fitted to a negative binomial model. The shape of the RwD curve shows the crash rate appears to retain a gradual increase in the crash rates with decreasing SN values.

The slope of the regression line is negative for all of the 3 AADT ranges presented in Reports 12 through 14, indicating an inverse relationship where crash rates are decreasing with increasing friction. However, according to P-values, the data provides enough information to reject the null and verify that not all means are equal and that the friction (SN) is a significant factor in the models that retained a (P-value < 0.05), with a confidence level greater than 95%.

Accordingly, friction is a significant factor in the medium and high AADT ranges (i.e. AADT2 & AADT3). However, the data failed to predict significant models for the low AADT range, where it is noted that R^2 for these types of roadway is very low and reviewing their plots shows no increase in crash rate with reduced friction. Therefore, it was not possible to define I.L.s for this category. This could be attributed to the fact that friction may not be as critical as in the other categories, or that the data does not provide enough sections in these categories to determine a clear relationship between crashes and friction. Following is an elaborate discussion of the significant models:

Report 12:

For the low AADT category, the data failed to provide meaningful regression to describe the relationship between all conditions/types of crashes and friction. One possible reason is that the sites with traffic levels above 40000 vehicles per day tend to have friction numbers below 40. This could indicate that the higher levels of traffic lead to more aggregate polishing and reduce the skidding resistance potential of those roads. This could also be another result of the procedure used to collect the friction data; roads with high traffic and good friction performance simply may not be tested and thus are underrepresented in the study. One other possible factor is that the distribution of friction seems bimodal as it shows two different picks suggesting that this category includes different types of roads.

To further investigate the test bias suggestion, SN40 is plotted against AADT values for all the study sites in Figure 47. The data shows that almost all the sites with very good friction (skid numbers greater than 50) have lower levels of traffic. Additionally, the sites with traffic levels above 40000 vehicles per day tend to have friction numbers below 40.

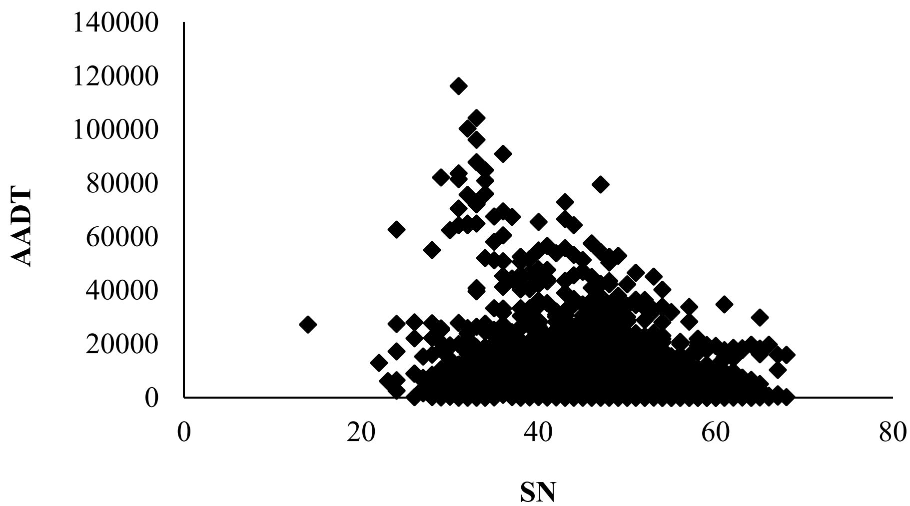


Figure 47: Friction versus AADT

Report 13:

For the medium AADT, there is a clear trend in the Wet Crash Rate with respect to roadway surface friction on the investigated routes as indicated by the trend curves and equations illustrated in the report 13. The shape of the curve shows that the crash rates appears to increase when SN values drop below 38 for the medium AADT range.

Additionally, the report shows that the suggested I.L.s by the AASHTO method for WCR will keep the crash risk at lower than 500 HMVM. For example, the I.L. suggested by the AASHTO method for the WCR model is 37.85. This I.L. level corresponds to a crash risk of 407 HMVM.

Report 14:

For the medium AADT, there is a clear trend in the Wet Crash Rate with respect to roadway surface friction on the investigated routes as indicated by the trend curves and equations illustrated in the report 14. The shape of the curve shows that the crash rates appears to increase when SN values drop below 36 for the high AADT range.

Additionally, the report shows that the suggested I.L.s by the AASHTO method for WCR will keep the crash risk at lower than 100 HMVM. For example, the I.L. suggested by the AASHTO method for the WCR model is 36.7. This I.L. level corresponds to a crash risk of 87 HMVM.

These findings agree with an analysis that was performed in Maryland by Chelliah et al. (2003). The research developed empirical models for various AADT ranges of all wet crash data. The results ranged from 35 to 60 for each AADT range. This supports this research findings that suggest I.L.'s that ranges from 36 to 42 for the different AADT ranges.

Moreover, it can be seen that the medium AADT range demands higher friction supply to maintain the network at the same crash risk exposure. For example, A goal of a crash risk of less than a 100 HMVM can be obtained at friction levels as low as 36 for high AADT, whereas the same crash risk requires a friction supply of higher than 40 for the medium AADT range. This is probably related to the fact that at some point, higher traffic volumes limit the vehicles ability to speed and thus reduces the friction demand as well as the crash rates. This also agrees with the findings of (Davies et al. 2005) that are demonstrated in Figure 48.

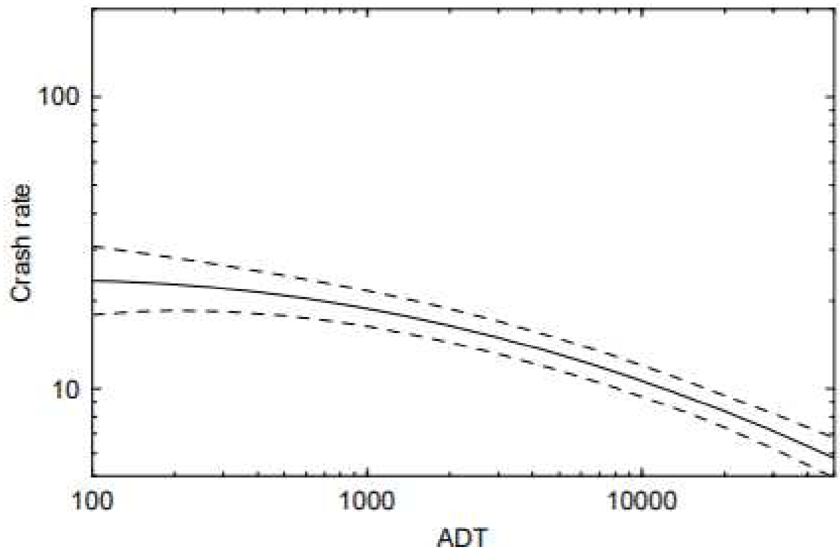


Figure 48: Crash rate versus ADT (Davies et al. 2005)

Friction demand models by Pavement Type

The friction distribution and the wet/dry as well as RwD/total crash ratios for the AADT ranges are presented in Figures 49 through 51. In these histograms, the investigatory level is set at the value of friction where the mean friction is adjusted towards a specific fraction of the standard deviation. This adjustment is based on the crash ratio pattern. For example, for the urban interstates, the friction distribution of the sites has a mean friction of 52.08 and a standard deviation of 9.29. Based on the overall decreasing wet/dry crash ratios pattern with the increasing friction, one can observe a sharp drop at a friction value of 32. However, as discussed in the methodology chapter, a significant drop must correspond to at least 5% of the sites, hence, for this case, the next consecutive drop in the crashes around a friction value of 38 is considered. Consequently, the I.L. is set at a value of 37.22 which is the mean minus 1.6 of the standard deviation. The data for the same classification suggests an RwD I.L. of 45.58 which corresponds to 0.9 standard deviations shift from the mean. A summary of the suggested friction demand

I.L.s for each classification based on its crash ratio and friction distribution histogram is provided in Table 13.

Table 13: Friction investigatory Levels by Pavement Type

Functional Classification	Mean Friction (μ)	Std Dev. Friction (σ)	Wet/Dry IL	RwD/Total IL
PT1	52.21	6.06	43.73	47.97
PT2	49.22	5.71	36.09	37.67
PT3	50.10	6.38	37.98	43.72

The friction distribution for the AC and Composite pavements is negatively skewed. The friction for the PC pavement is normally distributed. The crash rates do not appear to have a specific pattern and are showing a high variability in the peaks amongst it. This means that the decrease in the dry rates is stronger than the decrease in the wet crash rates. One can also observe sharp drops in the crash rates towards the highest friction values. This reveals that at that point, increasing the friction is resulting in no benefit in decreasing the dry rates, but is still decreasing the wet rates. The relationship between both, the wet crashes and the dry crashes with the friction can explain the lack of relationship between the wet/dry crash ratio and friction in some cases.

Overall, the suggested I.L.s show that the PC pavements require the highest level of friction for wet crashes. The friction demand models for each pavement type are presented in Reports 15 through 17.

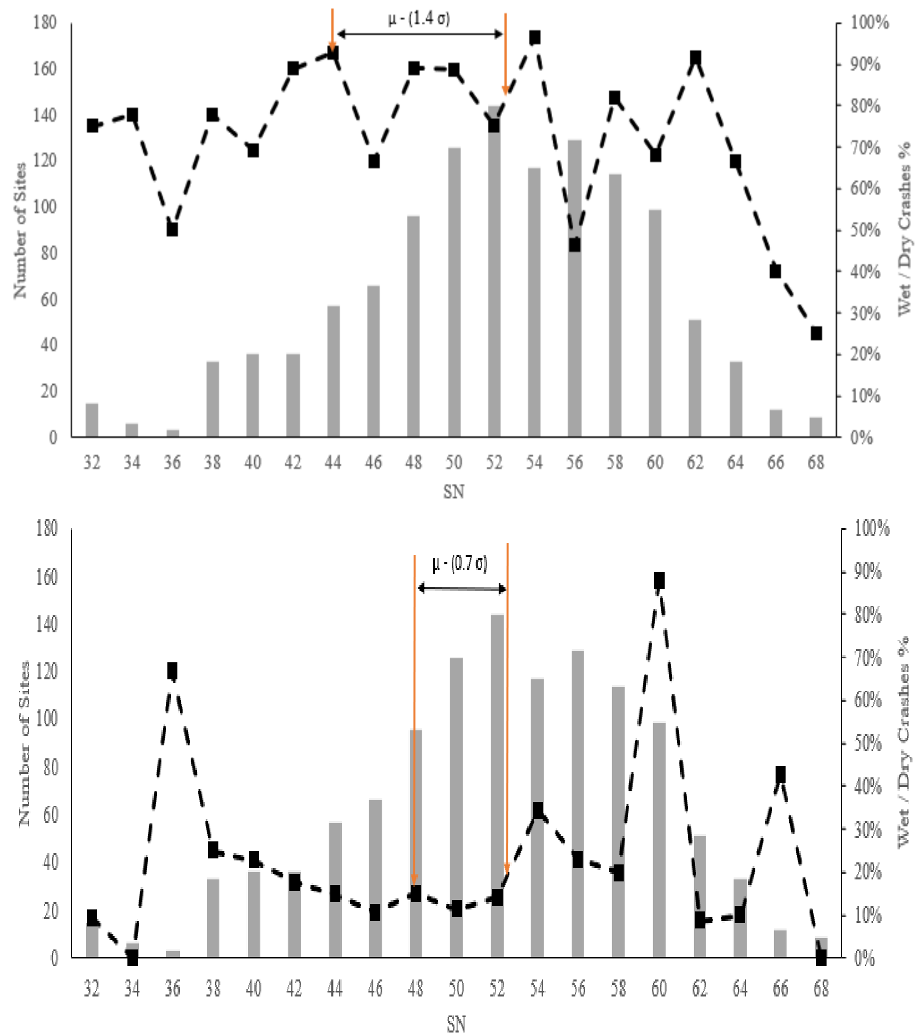


Figure 49: PT1 - PC Pavement – Friction Distribution and Crash Ratio

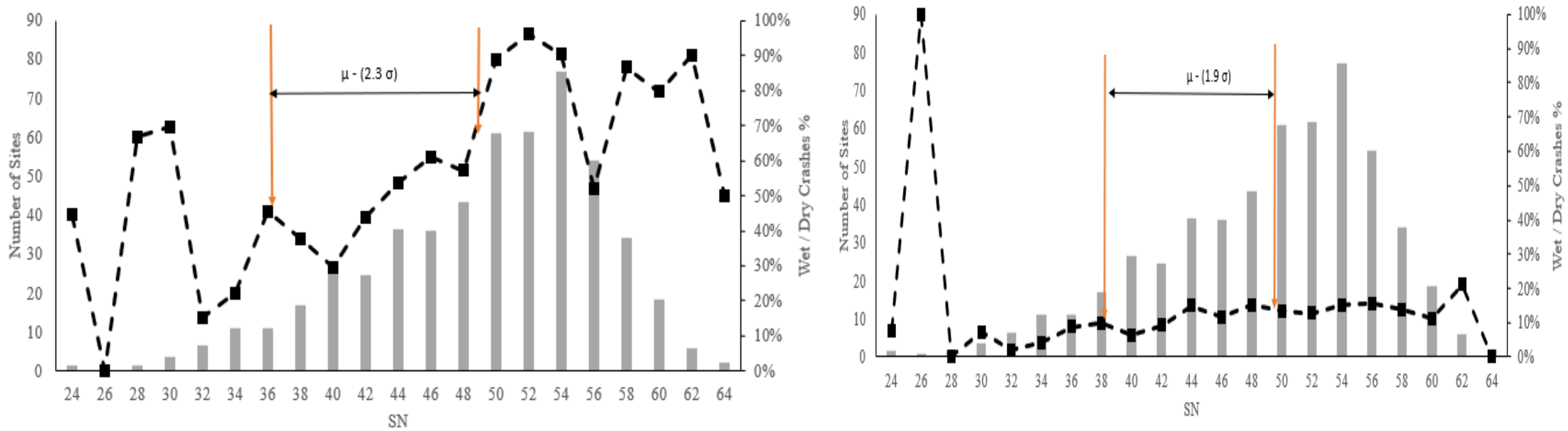


Figure 50: PT2 - AC Pavement – Friction Distribution and Crash Ratio

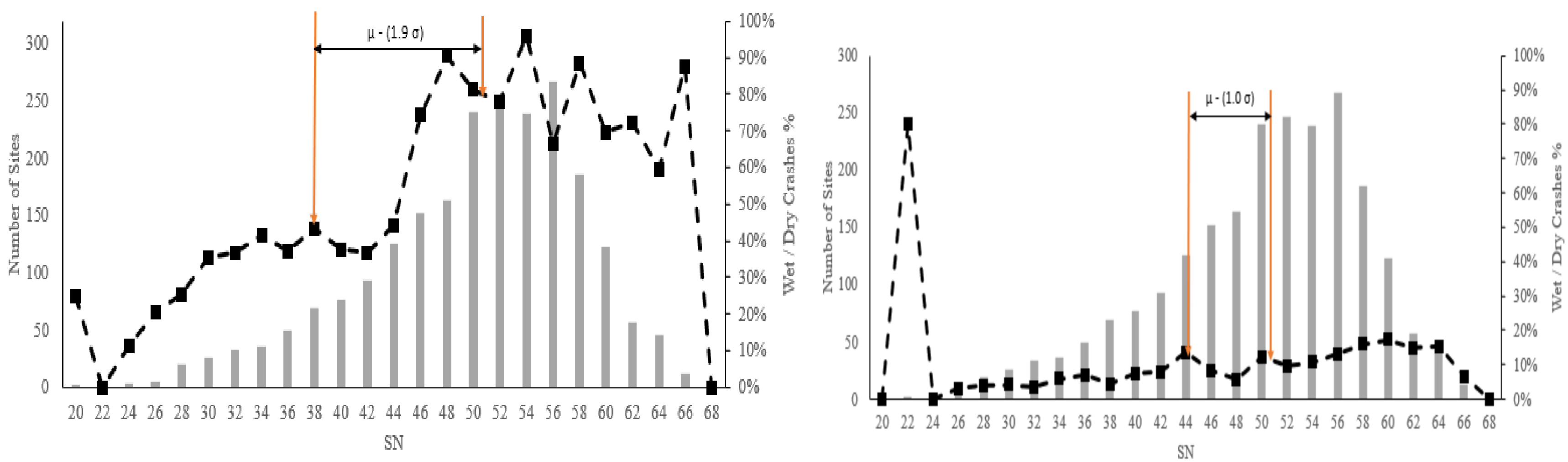
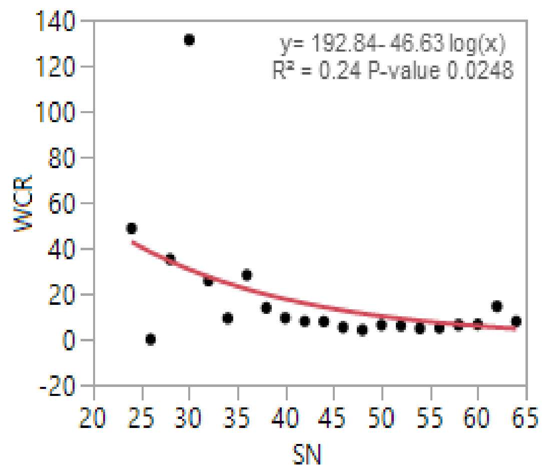


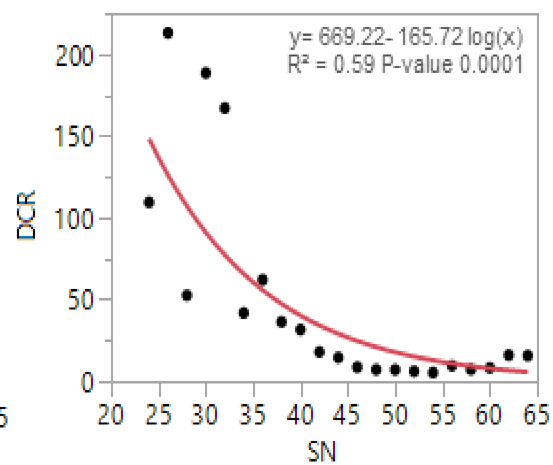
Figure 51: PT3 - Composite Pavement– Friction Distribution and Crash Ratio

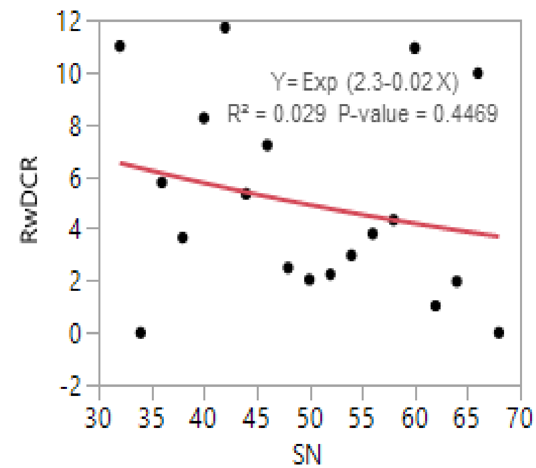
Report 15: Portland Cement Pavements (PC)

Category	Surface Condition/Crash Type	Regression Model	R²	P-Value	AASHTO IL	Crash Risk at IL (HMVMT)
PT1	Wet	*Y= 192.84-46.63 Log(X)*	0.24	0.0248	36.09	120
	Dry	*Y= 669.22-165.72 Log(X)*	0.59	0.0001	36.09	411
	RwD	*Y= Exp (2.3-0.02 X)*	0.029	0.4469	37.67	5

Where: Y =Crash Rate in (HMVMT) and X = Skid Number (SN40)





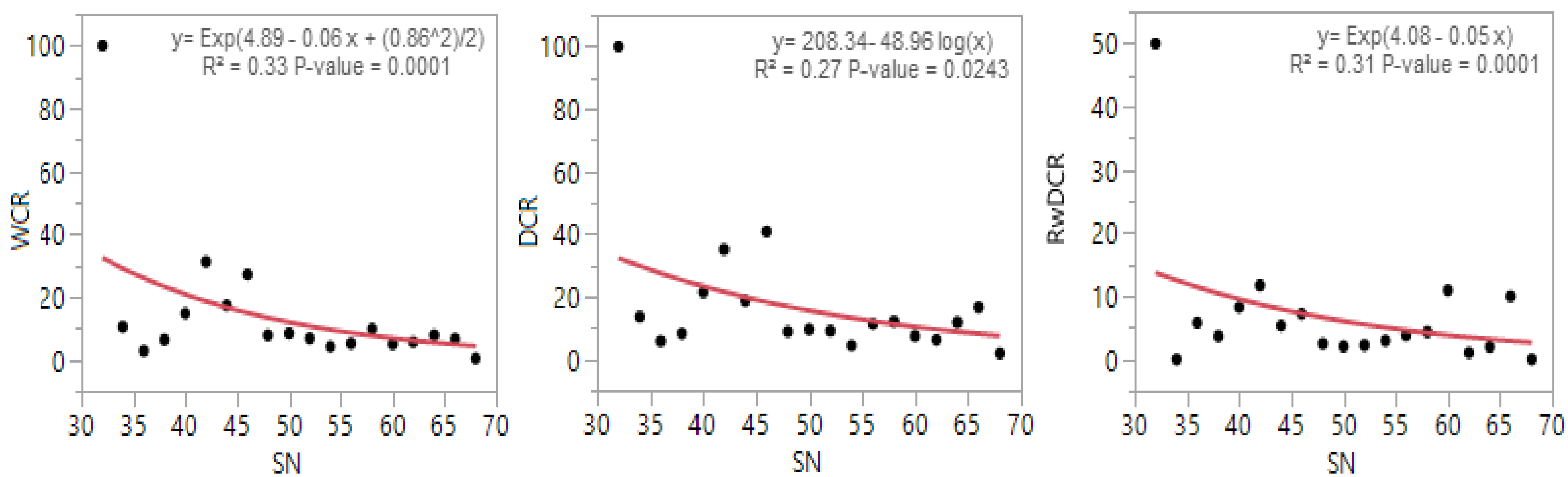


Notes: The slope of the regression fit is negative for the three cases. According to P-values, the wet and dry models are significant with a 95% confidence interval. R^2 for the wet model is moderate and acceptable. R^2 for the dry models is considerably high. The RwD model is insignificant and has a very small R^2, the model is highlighted in red. Outliers exists in the three models. Both wet and dry crash rates are fitted to a logarithmic transformed linear model. The shape of the wet curve shows a gradual decrease in the crash rates with increasing the friction with some variability. For the dry curve, the crash rate appears to increase when SN values drop below 45 and then sharply increase when the friction drops below 40.

Report 16: Asphalt Concrete Pavements (AC)

Category	Surface Condition/Crash Type	Regression Model	R^2	P-Value	AASHTO IL	Crash Risk at IL (HMVMT)
PT2	Wet	*Y= Exp (4.89-0.06 X + (0.86^2)/2)*	0.33	0.0001	43.73	14
	Dry	*Y= 208.34-48.96 Log(X)*	0.27	0.2043	43.73	128
	RwD	*Y= Exp (4.08-0.05 X)*	0.31	0.1111	47.97	5

Where: Y =Crash Rate in (HMVMT) and X = Skid Number (SN40)

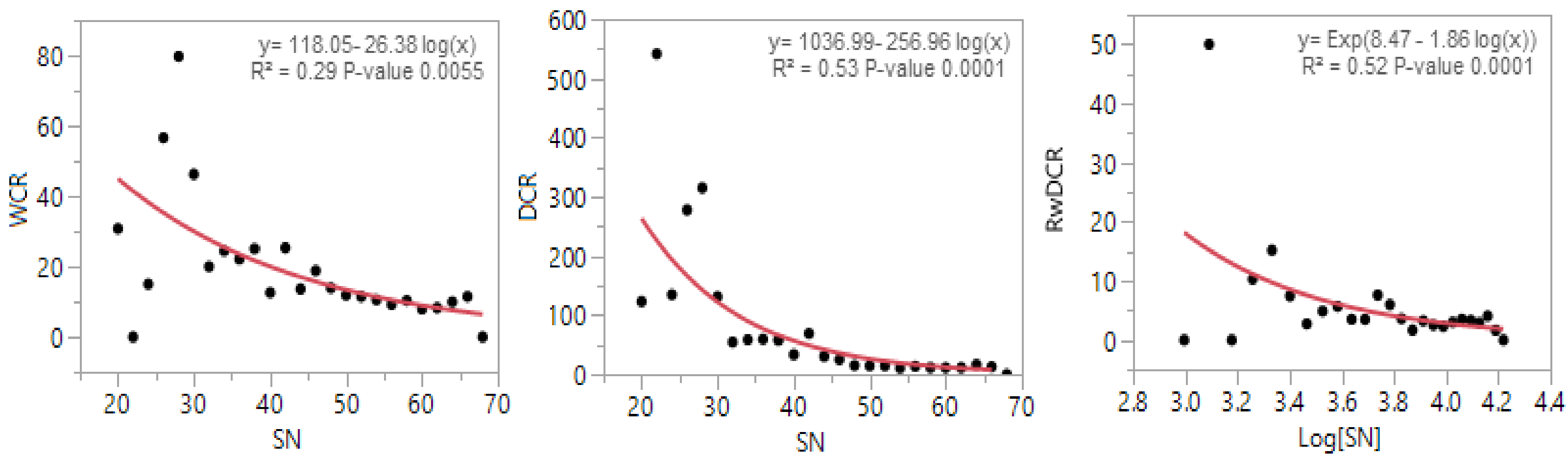


<u>Notes:</u> The slope of the regression fit is negative for the three cases. According to P-values, the wet and dry models are significant with a 95% confidence interval. R^2 for the wet and dry models is moderate and acceptable. The RwD model is insignificant and has a very small R^2, the model is highlighted in red. Outliers exists in the three models. The wet crash rates are fitted to a lognormal distribution. The dry crash rates are fitted to a transformed logarithmic model. The shape of the wet and dry curves shows a gradual decrease in the crash rates with increasing the friction with high variability in its peaks.

Report 17: Composite Pavements

Category	Surface Condition/Crash Type	Regression Model	R^2	P-Value	AASHTO IL	Crash Risk at IL (HMVMT)
PT3	Wet	Y= 118.05-26.38 Log(X)	0.29	0.0055	37.98	76
	Dry	Y= 1036.99-256.96 Log(X)	0.53	0.0001	37.98	631
	RwD	Y= Exp (8.47-1.86 Log(X))	0.52	0.0001	43.72	226

Where: Y =Crash Rate in (HMVMT) and X = Skid Number (SN40)



Notes: The slope of the regression fit is negative for the three cases. According to P-values, the three models are significant with a 95% confidence interval. R^2 for the dry and RwD models considerably high and is acceptable for the wet model. Outliers exist in the three models. Both wet and dry crash rates are fitted to a logarithmic transformed linear model. The RwD crash rates are fitted to an exponential model. The shape of the wet curve shows a gradual decrease in the crash rates with increasing the friction with some variability. For the dry curve, the crash rate appears to increase when SN values drop below 45 and then sharply increase when the friction drops below 30.

The slope of the regression line is negative for all the three pavement types presented in Reports 15 through 17, indicating an inverse relationship where crash rates are decreasing with increasing friction. However, according to P-values, the data provides enough information to reject the null and verify that not all means are equal and that the friction (SN) is a significant factor in the models that retained a (P-value < 0.05), with a confidence level greater than 95%.

Accordingly, friction is a significant for the wet and dry crash rates in the three pavement types. However, the data failed to predict significant models for the RwD crash rates among the AC and PC pavement sections, where it is noted that R^2 for these types of roadway is very low and reviewing their plots shows no increase in crash rate with reduced friction. Therefore, it was not possible to define I.L.s for these crash rates. This could be attributed to the fact the data does not provide enough sections in these categories to determine a clear relationship between crashes and friction. Following is an elaborate discussion of the significant models:

Report 15:

For the PC pavement, there is a clear trend in the Wet and Dry Crash Rate with respect to roadway surface friction on the investigated routes as indicated by the trend curves and equations illustrated in the report 15. The report shows that the suggested I.L.s by the AASHTO method will keep the crash risk at lower than 500 HMVM. For example, the I.L. suggested by the AASHTO method for the WCR model is 36.09. This I.L. level corresponds to a crash risk of 120 HMVM.

Report 16:

For the AC pavement, there is a clear trend in the Wet and Dry Crash Rate with respect to roadway surface friction on the investigated routes as indicated by the trend curves and equations illustrated in the report 16. The curves show that there is no sharp increase in crash rates, which makes it harder to define a precise threshold I.L.. One possible reason is that the 20 % of sections that the AC constitutes may not provide enough sections in this category to determine a clear relationship between crashes and friction. One other possible reason is that, contrary to PC pavements where supplemental treatments as tining or grooving are typically required to provide adequate macrotexture. AC pavements designed in conformance with Superpave mix design will generally provide adequate macrotexture and microtexture without supplemental treatments (“Surface Texture for Asphalt and Concrete” Technical Advisory T5040.36 -2005). That being said, friction may not be as critical for AC pavements as in the other categories since it might be maintained to better levels than the other pavement types. However, the report shows that the suggested I.L.s by the AASHTO method will keep the crash risk at lower than 150 HMVM. For example, the I.L. suggested by the AASHTO method for the WCR model is 43.73. This I.L. level corresponds to a crash risk of 14 HMVM.

Report 17:

For the composite pavement, there is a clear trend in the Wet and Dry Crash Rate with respect to roadway surface friction on the investigated routes as indicated by the trend curves and equations illustrated in the report 17. The report shows that the suggested I.L.s by the AASHTO method will keep the crash risk at lower than 700 HMVM. For example, the I.L. suggested by the AASHTO method for the WCR model is 37.98. This I.L. level corresponds to a crash risk of 76 HMVM. As expected, composite pavements and AC pavements retained somewhat similar relationships between wet crash rates and friction.

Overall, the results show that a higher friction level is required for Portland cement pavements followed by the composite pavements and Asphalt pavements which share the same surface macrotexture. These findings agree with the findings of Pulugurtha (2012). The researcher studied the effect of pavement macrotexture on Interstate I-40 Crashes in North Carolina. The results indicated that that PC pavements require maintaining greater macrotexture than asphalt pavements.

General Discussion

Reviewing the friction distributions revealed high variability in the crash ratio's peaks amongst it. For most of the categories, some of the distributions show that there are no apparent trends in the Wet/dry crash ratios with respect to roadway surface friction on the analyzed routes. This lack of relationship between the wet/dry crash ratio and friction might be attributed to a conclusion that has been recently introduced in the literature. In his research, Najafi (2015), concluded that there is a significant relationship between both wet and dry crash ratio and friction and suggested that models for the two environmental conditions should be investigated in future research (Musick, 2019; Najafi et al., 2014; Najafi, 2015). However, the distribution show that a significant drop in the Wet/Dry crash ratios is observed approximately around friction values of 30-42 for all categories. In addition, the poor trend between the RwD/Total crash ratios and friction could be attributed to the lack of sites with low friction values at which the RwD crashes are expected the most. Nevertheless, most of the categories show a significant drop in the RwD/Total crash ratios around friction values of 32 to 45. These findings agree with the FHWA instructional memorandum (FHWA, 1986) as well as the typical desirable SN values for most of the state's policies (i.e. 28-40) as demonstrated earlier in the literature review section of this research. In addition, these ranges include the threshold value of 40 suggested by (Rizenbergs et al., 1972) and (Kuttesch, 2004) in Kentuky and Virginia, respectively.

Overall, the results show that the I.L.s required if investigating RwD crashes are higher than those required if investigating Wet or Dry crashes. This is supporting the fact that the RwD crashes are more friction related. Hence, higher friction levels are likely to be needed depending on the agency's goal and targeted crashes.

All of these findings suggest more conservative I.L.s than those proposed and adopted internationally. For example, the I.L. for event free divided carriageways in New Zealand, Australia and the UK is set somewhere between 0.30 ESC to 0.40 ESC at 30 mph (i.e. SNR40 of 26 to 30). Additionally, the I.L. for event free un-divided carriageways is set somewhere between 0.35 ESC to 0.45 ESC at 30 mph (i.e. SNR40 of 28 to 35).

Summary of Findings

This study investigated the effect of friction on wet, dry and roadway departure crashes at different roadway categories and resulted in the friction demand models presented in Table 14 as well as the following findings:

- For all sites evaluated, friction is found to be a significant factor affecting wet crash rates except for sites with low AADT where no tangible relationship is detected between the two variables.
- Friction is found to be a significant factor affecting dry crash rates for most of the sites except for rural interstates. Hence, dry crashes should be considered along with wet crashes in future friction and safety studies.
- Friction was found to be a significant factor affecting roadway departure crashes at some sites where more roadway departure crashes were successfully matched with crash locations. However, no relationship was detected for urban freeways and expressways, urban principal arterials, rural minor arterials, sites with high speed limit, sites with low AADT as well as Portland cement and Asphalt

Concrete pavements. A larger study is needed to further investigate these findings with separating the wet crashes from the dry ones.

- It was possible to generate friction demand models and define friction I.L. for the categories that retained significant relationships.
- Based on the data studied,

  I. Tangent Segments have a relationship between crashes and friction, showing lower crash rates as the roadway surface friction increases. In addition, tangents appear to have the highest friction demand amongst other stratifying categories, i.e. a target SN40 of 42 and 47 appears to have positive safety benefits with respect to wet and roadway departure crashes maintaining a crash risk of less than 500 crash per HMVM.

  II. For urban interstates, consistent reductions in the mean crash rate are observed for increasing the target skid number beyond 37 which corresponds to a crash risk of less than 300 crashes per HMVM. However, freeways and expressways seem to demand higher friction amongst the other roadway classifications where a target of SN40 of 40 appears to have the same positive safety benefits to maintain a crash risk of less than 300 crashes per HMVM. Urban arterials on the other hand requires an SN40 of 30 to maintain the same crash risk as interstates and freeways, meaning that other principal arterials seem to require less friction supply than the other classifications.

III. Traffic volume was also found to be a significant factor in explaining the variation in crash rates. The crash rate decreases with increasing traffic volume.

IV. Higher speed limits require higher friction supply, i.e. when aiming for a wet crash risk of 100 crashes or less per HMVM, a target skid number (SN40) of 35, 38 and 39 appears to have positive safety benefits for low, medium and high speed limits, respectively.

V. For the Portland Cement and Composite pavements, there is a relationship between crashes and friction, showing lower crash rates as the roadway surface friction increases. The same relationship is not evident for the Asphalt Concrete Pavements. This could be because of the small sample of Asphalt Concrete Pavement segments available for the analysis. Portland Cement and Composite pavements which is basically AC pavement surfaces, appears to benefit from a skid resistance target of 40 and 38, respectively. Limiting the crash rates to less than 200 crashes per hundred million vehicle miles travelled.

Table 14: Friction Demand Models for Iowa Roadways

Category	Surface Condition/ Crash Type	Regression Model	R^2	P-Value	Category	Surface Condition/ Crash Type	Regression Model	R^2	P-Value
FC1	Wet	*Y= 229.08-57.85 Log(X)*	0.47	0.0013	**SL1**	Wet	*Y= 147.15-35.24 Log(X)*	0.53	<0.0001
	Dry	*Y= 242.95-4.66 Log(X)*	0.37	0.0054		Dry	*Y= Exp (7.63 - 0.09 X)*	0.75	<0.0001
	RwD	*Y= Exp (4.37-0.077 X)*	0.36	0.0014		RwD	*Y= 18.16-3.95Log(X)*	0.28	0.0036
FC2	Wet	*EXP (5.44-0.63 Log(X)) Γ (1+1.12)*	0.27	0.0066	**SL2**	Wet	*Y= 182.92-43.74 Log(X)*	0.53	0.0001
FC3	Wet	*Y= 293.62-73.08 Log(X)*	0.33	0.0077		Dry	*Y= 772.14-189.15 Log(X)*	0.62	<0.0001
	Dry	*Y= 448.21-111.49 Log(X)*	0.32	0.0090		RwD	*Y= Exp (5.41 -0.06 X)*	0.44	0.0002
	RwD	*Y= 85.22-21.08 Log(X)*	0.36	0.0056	**SL3**	Wet	*Y= 147.15-35.24 Log(X)*	0.58	<0.0001
FC5	Wet	*Y= 457.14-114.84 Log(X)*	0.31	0.0202		Dry	*Y= 1011.25-253.36 Log(X)*	0.48	<0.0001
	Dry	*Y= 2103.41-531.92 Log(X)*	0.43	0.0042	**AADT2**	Wet	*Y= 658.51-161.73 Log(X)*	0.28	0.0121
FC6	Wet	*Y= 165.99-39.20 Log(X)*	0.39	0.0019		Dry	*Y= 2601-646.34 Log(X)*	0.26	0.0150
	Dry	*Y= 307.08-5.69 Log(X)*	0.44	0.0008		RwD	*Y= 141.58-34.59 Log(X)*	0.47	0.0004
	RwD	*Y= Exp (2.41-0.030 X + (0.28^2)/2)*	0.65	0.0001	**AADT3**	Wet	*Y= 140.00-33.71 Log(X)*	0.37	0.0012
FC7	Wet	*Y= 300.66-72.46 Log(X)*	0.64	<0.0001		Dry	*Y= 989.38-247.17 Log(X)*	0.49	0.0001
	Dry	*Y= 554.10-135.35Log(X)*	0.63	<0.0001		RwD	*Y= Exp (2.94-0.05X)*	0.31	0.0001
	RwD	*Y= Exp (5.41-0.06 X)*	0.44	0.0002	**PT1**	Wet	*Y= 192.84-46.63 Log(X)*	0.24	0.0248
FC8	Wet	*Y= 223.69-54.16 Log(X)*	0.27	0.0191		Dry	*Y= 669.22-165.72 Log(X)*	0.59	0.0001
	Dry	*Y= 979.00-242.58 Log(X)*	0.33	0.0077	**PT2**	Wet	*Y= Exp (4.89-0.06 X + (0.86^2)/2)*	0.33	0.0001
	RwD	*Y= Exp (2.99 - 0.04 X)*	0.23	0.0158		Dry	*Y= 208.34-48.96 Log(X)*	0.27	0.2043
Tangents	Wet	*Y= 1580.50-374.89Log(X)*	0.60	<0.0001	**PT3**	Wet	*Y= 118.05-26.38 Log(X)*	0.29	0.0055
	RwD	*Y= 794.56-186.36Log(X)*	0.55	0.0002		Dry	*Y= 1036.99-256.96 Log(X)*	0.53	0.0001
						RwD	*Y= Exp (8.47-1.86 Log(X))*	0.52	0.0001

CHAPTER 5. CONCLUSION

Conclusions

Incorporating friction demand models into a skid resistance policy would reduce crashes and enhance safety by synchronizing the safety management and the pavement friction management into one framework. This is perceivable from the following conclusions:

- Skid resistance is a factor in explaining the variation in crash rates. However, as expected, friction data tend to explain only a small portion of the variation in crash rates when considering individual crash sites.
- Investigating the ratio of wet to dry crashes or the ratio of roadway departure crashes to total crashes assumes that there is no correlation between dry crashes or any other type of crashes and thus might not be the best approach to explain the variation in crashes with respect to skid resistance.
- A statistically significant effect of skid resistance on the wet, dry and roadway departure crash rate is captured by employing a two-parameter, two-level skid resistance model where grouping the crash sites by similar characteristics improved the ability to developing models that better explain the variability in crash occurrence.
- The development of friction demand models provides an easy way to quantify and understand the quantitative relationship between skid resistance and crash risks. Based on the developed models, skid resistance thresholds can be determined easily according to the target crash risk level or expected crash reduction. In addition, I.L.s can be used by the Iowa DOT to identify potentially hazardous locations that would need safety investigations.

Research Limitations

This study has certain limitations that future research may focus on, including the following:

- The studied SN numbers are discrete measurements on segments that are 0.2 to 10 miles long. This means that some of the longer segments might experience higher or lower friction at understudied parts of it and this will contribute to generating models that does not specifically serve the entire network. This effect needs to be further investigated and eliminated by moving towards using continuous friction measurement methods as the SCRIM and the Grip tester.
- The crash rates defined in this study were normalized using 365 days per year, without distinguishing between rainy and dry days. Factors such as seasonal variation and temperature changes can also affect the friction measurement. This effect needs to be further investigated.
- Discriminating on specifically roadway departure crashes for each weather condition provided limited data, especially for wet weather crashes. Therefore, a combination of wet and dry crash records was used for the development of the roadway departure models. In addition, wet and dry models were generated for all types of crashes including crashes that may not have a direct relationship with pavement conditions. Future research should be carried out to fine-tune the analysis by excluding crashes that are irrelevant to pavement conditions.
- Crash rate is influenced by a combination of factors rather than one single factor. Some of the categories adopted in this research implies underlying factors on the analysis. For Example, in addition to the limited access that urban interstates

provide, it also underlies higher speeds and higher AADT's. However, a heuristic approach is recommended for future research to develop a fine-tuned grouping structure.

Future work

Although this research has the potential to make an impact on the Iowa DOT skid resistance policy, additional improvement and research might be warranted in this area of study:

- Due to the lack of friction readings on curves, no investigation has been carried on providing a friction demand model on curves. However, Curves are very critical sites and usually demand higher friction that is due to the combination of speed and centrifugal force. Future efforts should be carried out to provide friction measurements on curves to be able to provide a robust friction management for curves.
- Although the data includes discrete friction measurements on tangent segments approaching curves, no investigation has been carried on providing a friction demand model on these segments. This is due to the lack of such information in the original road network database and the curves database. The identification of such information from road network and curves data is a time consuming and error-prone process. Future efforts should be carried out to integrate and automated GIS tool that enables such data extraction of road alignment to stratify the tangents and to carry on with identifying the friction requirements at those tangent segments where it is expected that these tangents are exposed to increased

friction demand where the roadway surface at often becomes prematurely polished, reducing the pavement friction and contributing to higher crash rates.

- Very limited friction readings were found in the vicinity of 250 ft. of intersections around the state. Thus, no investigation has been carried on correlating intersection related crashes with friction related crashes and developing a model that describes the two variables relationship. However, approaches to intersections and conflict sites usually require higher friction values and often becomes prematurely polished, reducing the pavement friction and contributing to higher crash rates. Future efforts should be carried out to provide a more site-specific friction measurements to be able to generate friction demand models for intersections.
- Future work should include a more in depth study of the PC pavements based on the surface treatment. It is expected that PC pavements with horizontal or vertical tinning behave different from regular PC pavements. Since concrete tining is a very common practice across Iowa roadways, such analysis is warranted to generate more precise models that tackles the need of the Iowa DOT pavement maintenance resource allocation.
- A robust movement towards the U.S. DOTs long-standing goal of a 50% reduction in fatalities over the next 10 years as well as a step towards a future of “Zero Fatalities” on the U.S. roadway network could be achieved by targeting high severity crashes and performing the same analysis conducted in this study on fatality and major injury causing crashes.

- A benefit/cost analysis could be carried on verifying the benefit of the proposed models and justify the recommended thresholds as well as prioritize pavement preservation techniques.
- There is a great potential for this study in the area of Safety Performance Functions. Safety Performance Functions should be developed for a multi-variable friction demand category.
- Friction and crash data are at the heart of PFM. However, as discussed in the data chapters, the data is diverse and has different data sources. Hence, it required a considerable effort of pre-processing. Moreover, when derived manually, such work cannot be reproduced in the future. In support of future analyses, an integrated database should be created where automated data processing can be implemented to extract the needed data effectively.
- Future work might also include incorporating the integrated data base along with the generated friction demand models into a web-based system that provides Iowa DOT with a tool that analyzes crash rates and SN values in a crash risk scheme at different site categories.
- Two decades after the national survey that was conducted in 1999 by the NCHRB (Henry, 2000), it is time to conduct a new survey that helps formulating the state-of-the-art pavement friction management practices. Future work must involve formulating a survey to collect and update information on issues pertaining to pavement friction characteristics, including methods of testing and monitoring as well as skid resistance policies and pavement friction management practices.

REFERENCES

Ahammed, M. Alauddin, and Susan L. Tighe. “Early-life, Long-Term, and Seasonal Variations in Skid Resistance in Flexible and Rigid Pavements.” *Transportation Research Record*, vol. 2094, no. 1, 2009, pp. 112-120.

Ahammed, M. Alauddin, and Susan L. Tighe. “Pavement surface friction and noise: integration into the pavement management system.” *Canadian Journal of Civil Engineering*, vol. 37, no. 10, 2010, pp. 1331-1340.

Alhasan, Ahmad, et al. “Impact of Pavement Surface Condition on Roadway Departure Crash Risk in Iowa.” *Infrastructures*, vol. 3, no. 2, 2018, pp. 14.

Banez, Bernadette. “Notes to Specification for Resistance Management.” nzta.govt.nz. https://nzta.govt.nz/assets/resources/skid-resistance-investigation-treatment-selection/docs/T10-Notes-to-specification-for-highway-skid-resistance-management-201306.pdf. Accessed 5 May 2020.

Byrd, Tallamy, and Lewis MacDonald. “Training Course: Skid Resistance Measurements and Design, Instructor Notebook.” U.S. Department of Transportation, Federal Highway Administration, and National Highway Institute Contract DTFH61-80-C-00023 FHWA, 1981.

Cairney, Peter. “Skid Resistance and Crashes: A Review of the Literature. Report No. ARR 311, Transport Research Ltd., Vermont, South Victoria, Australia. 1997.

Chelliah, T., et al. "Developing a design policy to improve pavement surface characteristics. CD-ROM." *Transportation Research Board, National Research Council, Washington, DC, USA* (2003).

Chu, L. J., and T. F. Fwa. “A Functional Approach for Determining Skid Resistance Threshold State of Porous Pavement.” *International Journal of Pavement Engineering*, vol. 20, no. 4, 2019, pp. 481-489.

Chu, L. J., and T. F. Fwa. “Pavement Skid Resistance Consideration in Rain-Related Wet-Weather Speed Limits Determination.” *Road Materials and Pavement Design*, vol 19, no. 2, 2018, pp. 334-352.

Chu, Longjia, and T. F. Fwa. “Evaluating Wet-Weather Driving Safety Risks of Pavement Ruts.” *Functional Pavement Design*, CRC Press, 2016, pp. 1523-1531.

Cook, D., D. Whitehead, and J. Towler. “An Evaluation of the Economics of New Zealand’s Skid Resistance Policy.” 3rd International Surface Friction Conference, Safer Road Surfaces–Saving Lives, Gold Coast, Australia, 2011.

Corsello, Patricia. Evaluation of surface friction guidelines for Washington state highways. Washington University Seattle Deptartment of Civil Engineering, 1993.

Corsello, Patricia. Evaluation of Surface Friction Guidelines for Washington state highways. Washington University Seattle Deptartment of Civil Engineering, 1993.

Crisman, Bruno, and Roberto Roberti. “Tire wet-Pavement Traction Management for Safer Roads.” *Procedia-Social and Behavioral Sciences*, vol. 53, 2012, pp. 1054-1067.

Davies, Robert B., Peter D. Cenek, and Robert J. Henderson. "*The effect of skid resistance and texture on crash risk.*" Proceedings Surface Friction Roads and Runways, Christchurch (2005): 1-4.

Davis, R. M., et al. “Effect of Wearing Surface Characteristics on Measured Pavement Skid Resistance and Texture.” 81st Transportation Research Board Annual Meeting, 2002.

Federal Highway Administration (FHWA). “Pavement Friction Management,” Technical Advisory T 5040.38, U.S. Department of Transportation, FHWA, Washington, D.C., 2010.

FHWA. “Surface Texture for Asphalt and Concrete Pavements.” Technical Advisory, Classification Code T 5040.36, U.S. Department of Transportation, FHWA, Washington, D.C., 2005.

Flintsch, Gerardo W., et al. "Pavement surface macrotexture measurement and applications." *Transportation Research Record* 1860.1 (2003): 168-177.

Flintsch, Gerardo W., et al. “Pavement Surface Macrotexture Measurement and Applications.” *Transportation Research Record*, vol. 1860, no. 1, 2003, pp. 168-177.

Flintsch, Gerardo W., et al. “The Little Book of Tire Pavement Friction.” *Pavement Surface Properties Consortium*, vol. 1, 2012.

Fwa, T. F. “Skid Resistance Determination for Pavement Management and Wet-Weather Road Safety.” *International Journal of Transportation Science and Technology*, vol. 6, no. 3, 2017, pp. 217-227.

Fwa, T. F., and L. Chu. “The Concept of Pavement Skid Resistance State.” *Road Materials and Pavement Design*, 2019, pp. 1-20.

Giles, Colum, B. Sabey, and K. H. Cardew. “Development and performance of the portable skid-resistance tester.” Symposium on Skid Resistance, ASTM International, 1962.

Gothie, M. “Relationship Between Surface Characteristics and Accidents.” International Symposium on Pavement Surface Characteristics, 1996, christchurch, new zealand. 1996.

Hall, J. W., et al. “Guide for Pavement Friction.” Final Report for NCHRP Project 1, 2009, pp. 43.

Healow, S. “Pavement Friction Management.” Oklahoma City, OK, 2011. Presentation.

Henry, John Jewett. *Evaluation of Pavement Friction Characteristics*, vol. 291. Transportation Research Board, 2000.

Hunt, Simon, and C. Pacey. "Skid Resistance Strategy for a Road Network." CD-ROM Proceedings of World Conference and Asset Management, Milan (Baveno), 2017.

Ivan, J., et al. "Incorporating Wet Pavement Friction into Traffic Safety Analysis." Connecticut Department of Transportation, Storrs, 2010).

Izeppi, Edgar de León, Gerardo Flintsch, and Ross McCarthy. "Evaluation of Methods for Pavement Surface Friction." Testing on Non-Tangent Roadways and Segments, no. FHWA/NC/2017-02, 2017.

Kuttesch, Jeffrey S. *Quantifying the Relationship between Skid Resistance and Wet Weather Accidents for Virginia Data*. 2004. Virginia Tech, PhD dissertation.

Lamm, Ruediger, E. M. Choueiri, and Theodore Mailaender. "Side Friction Demand Versus Side Friction Assumed for Curve Design on Two-Lane Rural Highways." *Transportation Research Record*, vol. 1303, 1991, pp. 11-21.

Lamm, Ruediger, et al. "Design Friction Factors of Different Countries Versus Actual Pavement Friction Inventories." *Transportation Research Record*, vol. 1260, 1990, pp. 135-146.

Larson, Roger M. "Using Friction and Texture Data to Reduce Traffic Fatalities, Serious Injuries, and Traffic Delays." International Conference for Surface Friction for Roads and Runways, Christchurch, New Zealand, 2005.

Long, Kan, et al. "Quantitative Relationship Between Crash Risks and Pavement Skid Resistance." No. FHWA/TX-13/0-6713-1, Texas. Dept. of Transportation, Research and Technology Implementation Office, 2014.

Mataei, Behrouz, et al. "Pavement Friction and Skid Resistance Measurement Methods: A Literature Review." *Open Journal of Civil Engineering*, vol. 60, no. 4, 2016, p. 537.

McCullough, B. Frank, and K. D. Hankins. "Skid Resistance Guidelines for Surface Improvements on Texas Highways." *Highway Research Record*, vol. 131, 1966.

Mohammad Ali Khasawneh, Nadim I. Shbeeb, and Aslam A. Al-Omari. "Analytical Tool to Shorten Polishing Time Based on Mean Texture Depth (MTD) of Flexible Pavements." *Road Materials and Pavement Design*, vol. 21, no. 3, 2020, 737-756,

Moyer, R. A. "Motor Vehicle Operating Costs, Road Roughness and Slipperiness of Various Bituminous and Portland Cement Concrete Surfaces." *Highway Research Board Proceedings*, vol. 22, 1943.

Musey, K., and S. Park. "Pavement Skid Number and Horizontal Curve Safety." *Procedia Engineering*, vol. 145, 2016, pp. 828-835.

Musick Jr, Ryland Wayne. *Network Roadway Surface Friction and Its Usage to Improve Safety and Project Performance along West Virginia Highways*. 2019. Virginia Tech, PhD dissertation.

Najafi, Shahriar, Gerardo W. Flintsch, and Alejandra Medina. "Linking Roadway Crashes and Tire–Pavement Friction: A Case Study." *International Journal of Pavement Engineering*, vol 18, no. 2, 2017, pp. 119-127.

Najafi, Shahriar. "Pavement Friction Management (PFM)-A Step Toward Zero Fatalities." 2016. Virginia Tech, PhD dissertation.

Neaylon, K. "Guidance for the Development of Policy to Manage Skid Resistance." No. AP-R374/11, 2011.

Noyce, David A., et al. "Incorporating Road Safety into Pavement Management: Maximizing Surface Friction for Road Safety Improvements." *Work*, vol. 5, 2007.

Noyce, David A., et al. "Incorporating Road Safety into Pavement Management: Maximizing Asphalt Pavement Surface Friction for Road Safety Improvements." Draft Literature Review and State Surveys, Midwest Regional University Transportation Center (UMTRI), Madison, Wisconsin, 2005.

Ong, Ghim Ping, and T. F. Fwa. "Wet-Pavement Hydroplaning Risk and Skid Resistance: Modeling." *Journal of Transportation Engineering*, vol. 133, no. 10, 2007, pp. 590-598.

Pardillo Mayora, José M, and Rafael Jurado Piña. "*An assessment of the skid resistance effect on traffic safety under wet-pavement conditions.*" Accident; analysis and prevention vol. 41,4 (2009): 881-6. doi:10.1016/j.aap.2009.05.004

Pulugurtha, Srinivas S., et al. "Preliminary Results Towards Developing Thresholds for Pavement Condition Maintenance: Safety Perspective." *Procedia-Social and Behavioral Sciences*, vol. 104, 2013, pp. 302-311.

Pulugurtha, Srinivas S., Prasanna R. Kusam, and Kuvleshay J. Patel. "Assessment of the effect of Pavement Macrotexture on Interstate Crashes." *Journal of Transportation Engineering*, vol. 138, no. 5, 2012, pp. 610-617.

R. Vaiana, T. Iuele, V. Gallelli, and D. Rogano. Demanded Versus Assumed Friction Along Horizontal Curves: An On-the-Road Experimental Investigation." *Journal of Transportation Safety & Security*, vol. 10, no. 4, 2018, pp. 318-344.

Rezaei, Arash, and Eyad Masad. "Experimental-based model for predicting the skid resistance of asphalt pavements." *International Journal of Pavement Engineering* 14.1 (2013): 24-35.

Rizenbergs, Rolands L. "Discussion on Skid Resistance of Pavement Surfaces." (1968).

Rizenbergs, Rolands L., James L. Burchett, and Cass T. Napier. "Skid Resistance of Pavements." Skid Resistance of Highway Pavements, ASTM International, 1973.

Rizenbergs, Rolands L., James L. Burchett, and Larry A. Warren. "Relation of Accidents and Pavement Friction on Rural, Two-Lane Roads." *Transportation Research Record*, vol. 633, 1977, pp. 21-27.

Schleppi, B. L., M. Y. Maikhail, and G. K. Chang. "International Experience and Perspective of Pavement Texture Measurements and Evaluation: Transportation Research Circular, E-C216, Washington, DC, Transportation Research Board, 2016.

Schram, Scott. "Specifications for Aggregate Frictional Qualities in Flexible Pavements: Case Study." *Transportation Research Record*, vol. 2209, no. 1, 2011, pp. 18-25.

Schulze, K. H., A. Gerbaldi, and J. Chavet. "Skidding Accidents, Friction Numbers, and the Legal Aspects Involved: Report of the PIARC Technical Committee on Slipperiness and Evenness." *Transportation Research Record*, vol. 623, 1977, pp. 1-10.

Sinhal, Ramesh. "The Implementation of a Skid Policy to Provide the Required Friction Demand on the Main road Network in the United Kingdom." International Conference of Surface Friction, 2005.

Smith, K. L., et al. "Improving Highway Safety Through Pavement Friction Management Programs." (2012).

Statistics How To [online], available at: https://www.statisticshowto.com/. accessed on December 06, 2019.

Subedi, Yogendra Prasad. "Evaluation of Louisiana Friction Rating Table by Field Measurements." (2015).

Thoma, T., and F. Vaca. "National Highway Traffic Safety Administration (NHTSA)." 2004, pp. 521-522.

Traffic and Safety Bureau -Iowa DOT [online], available at: https://iowadot.gov/traffic. accessed on September 10, 2019.

Ueckermann, Andreas, et al. "Calculation of Skid Resistance from Texture Measurements." *Journal of Traffic and Transportation Engineering* (English edition), vol. 2, no. 1, 2015, pp. 3-16.

Wallman, Carl-Gustaf, and Henrik Åström. "Friction Measurement Methods and the Correlation Between Road Friction and Traffic Safety: A Literature Review. Statens väg-och Transportforskningsinstitut, 2001.

Xiao, Jie, Bohdan T. Kulakowski, and Moustafa EI-Gindy. "Prediction of Risk of Wet-Pavement Accidents: Fuzzy Logic Model." *Transportation Research Record*, vol 17, no. 1, 2000, pp. 28-36.