State of Florida

Develop Statistical Models Quantifying the Relationship Between Pavement Surface Friction Characteristics and Traffic Accident Rates

FINAL REPORT

FDOT Contract Number: BE385

November 2019

Submitted By:
Hyung S. Lee, Ph.D., P.E.
Jim Hall, Ph.D., P.E.
Mark Stanley

APPLIED RESEARCH ASSOCIATES, INC.
100 Trade Centre Dr., Suite 200
Champaign, Illinois 61820

DISCLAIMER

The opinions, findings, and conclusions expressed in this publication are those of the authors and not necessarily those of the State of Florida Department of Transportation.

Technical Report Documentation Page

1. Report No.	2. Government Accession No.	3. Recipient's Catalog No.
4. Title and Subtitle Develop Statistical Models Quantifying the Relationship Between Pavement Surface Friction Characteristics and Traffic Accident Rates		5. Report Date November 27, 2019
		6. Performing Organization Code
7. Author(s) Hyung S. Lee, Jim Hall, and Mark Stanley		8. Performing Organization Report No.
9. Performing Organization Name and Address Applied Research Associates, Inc. 100 Trade Centre Dr., Suite 200 Champaign, IL 61820		10. Work Unit No. (TRAIS)
		11. Contract or Grant No. BE385
12. Sponsoring Agency Name and Address Florida Department of Transportation State Materials Office 5007 N.E. 39th Avenue Gainesville, FL 32609		13. Type of Report and Period Covered Final Report June 2017 to December 2019
		14. Sponsoring Agency Code
15. Supplementary Notes		

16. Abstract

The objectives of this research project are to (1) quantify the relationship between pavement surface friction characteristics, traffic accident rates, and any other factor(s) deemed important and (2) determine and recommend critical threshold levels below which crash rates would significantly increase.

Following a literature review and gap analysis on FDOT’s existing practice, a statistical relationship was developed between Florida’s crash rates, pavement friction and texture characteristics, as well as other pavement-related data. A number of preliminary analyses have been conducted in regards to crash, friction, and traffic distributions for determining the friction demand categories. The new statistical model was accompanied by a reliability-risk analysis for determining the recommended levels of friction. Furthermore, recommendations on FDOT’s Friction Guidelines, Friction Course Policy, and Safety Analysis Practice were developed and documented.

17. Key Word	18. Distribution Statement
Safety, Pavement Friction, Pavement Texture, Crash, Safety Performance Function, Empirical Bayes, Pavement Performance	No restrictions

19. Security Classif. (of this report)	20. Security Classif. (of this page)	21. No. of Pages	22. Price
Unclassified	Unclassified	133	

Form DOT F 1700.7 (8-72) Reproduction of completed page authorized

EXECUTIVE SUMMARY

Ideally, the friction demand for roadways should be determined by examining the trend between pavement friction and accident rates. However, early attempts to relate crash statistics to pavement friction were unsuccessful. The issue was that, although pavement friction is a crucial factor for improving highway safety and for reducing traffic accidents, it is not the only factor influencing the cause of crashes. In fact, traffic accidents are complicated events resulting from a combination of pavement friction and various other factors that are driver-related (e.g., distraction), vehicle-related (e.g., tires, brake system), pavement-related (e.g., structural and functional distresses, pavement marking issues), roadway-related (e.g., geometry, visibility), and weather-related (e.g., rainfall intensity).

Due to the difficulties associated with assessing the effect of pavement friction on crash rates and assessing the friction demand, most State Highway Agencies (SHA) have established the required and desired levels of friction based on empirical friction demand. As a result, many SHAs have specified universal values for the required and desired levels of friction for the entire highway network, while a few SHAs have defined the thresholds for a limited group of roadways categorized based on the posted speed limit, facility type, and roadway classes. Although it may be practical to maintain a certain level of pavement friction for all pavement sections within the highway network (or one of its subcategories), a single level of friction cannot be used to define a threshold that distinguishes between “safe” and “unsafe” roadways because different highways are subjected to different conditions and circumstances. In addition, the frictional characteristics of a given pavement surface change over time. Therefore, such a practice is prohibitively expensive and would not generate the cost-benefits associated with a better-targeted strategy.

The objectives of this research project are to (1) quantify the relationship between pavement surface friction characteristics, traffic accident rates, and any other factor(s) deemed important and (2) determine and recommend critical threshold levels below which crash rates would significantly increase. It is believed that, if these objectives are met, the study would allow for a more objective friction target setting process and a more cost-effective friction design procedure. To meet the above objectives, this study was carried out in four major tasks: (1) Literature review, (2) Gap analysis, (3) Statistical model development, and (4) Recommendations.

The summary of literature review in this report provides a summary of relationships between crash rates and pavement surface friction, as well as other factors that affect friction and crash rates. In addition, available literature describing other SHAs’ friction measurement and management practices, friction demand setting, and friction number requirements were reviewed.

Following the review of literature, FDOT’s current practice and other SHA practices as well as those recommended by Federal Highway Administration (FHWA), American Association of State Highway and Transportation Officials (AASHTO), and National Cooperative Highway Research Program (NCHRP) have been reviewed further. Based on the review of the various practices, a gap analysis was conducted to determine any shortcomings of FDOT’s current practice. The gap analysis results are presented in great detail in the second chapter of this report. As a quick summary, although no significant shortcomings were found when FDOT’s practices were compared to those of other SHAs in the U.S, several gaps were identified when FDOT’s practices were compared to the recommended practice or to some international agencies (e.g., United Kingdom and New Zealand). Furthermore, lack of a procedure for determining the desired level of friction (or friction demand) in an objective manner based on the crash counts and other factors that affect crashes was found to be one of the most significant shortcomings.

A statistical relationship was developed between Florida's crash rates, pavement friction and texture characteristics, as well as other pavement-related data. A number of preliminary analyses have been conducted in regards to crash, friction, and traffic distributions for determining the friction demand categories. The new statistical model was accompanied by a reliability-risk analysis for determining the recommended levels of friction. Furthermore, recommendations on FDOT's Friction Guidelines, Friction Course Policy, and Safety Analysis Practice were developed and documented. The following provides a quick summary.

1. Based on the Safety Performance Functions developed in this study, the Desired as well as the Questionable and Review thresholds for pavement friction were established. However, it should be noted that the questionable and review thresholds were established more or less in an empirical manner by examining FDOT's existing friction distribution, crash and crash rate distributions, and wet to dry crash ratios. Therefore, it is recommended that these thresholds be evaluated further for their effectiveness.

2. According to the available data, FDOT's roadways with a speed limit of 45 mph exhibit the largest number of crashes (Dry and Wet weather). The design speed of 45 mph also corresponds to the minimum speed considered in FDOT's Hydroplaning Design Guidance. As such, it is recommended that FDOT consider using OGFCs on multi-lane facilities with design speed of 45 mph.

3. FDOT's current methodology for identifying the wet weather crash locations are found to be effective. However, it is recommended that another criterion [more specifically, sections with minimum of six wet weather crashes and 10 percent or more wet weather crashes] be added to the existing criteria to capture more sections with higher crash potential.

It is emphasized again that although pavement friction is an important factor, it is not the only factor affecting roadway crashes. Nevertheless, it is believed that FDOT will see further improvement in safety (i.e., further reduction in crashes) by implementing the recommendations provided herein.

TABLE OF CONTENTS

Disclaimer ii
Executive Summary iv
Table of Contents vi
List of Figures ix
LIST OF TABLES xii
Introduction 1
Research Objective 3
Literature Review 4
  Review of Pavement Friction and Texture 4
    Overview of Pavement Friction 4
      Definition and Mechanism of Pavement Friction 4
      Factors Affecting Available Pavement Friction 5
      Friction Equipment 6
    Overview of Pavement Texture 10
      Definition and Significance of Texture 10
      Factors Affecting Texture 11
      Texture Equipment 13
  Agency Practices for Measuring Friction and Texture 19
  Review of Pavement Friction Management 20
    Recommended Practice by AASHTO 20
    Agency Practices 24
    International Practices 26
  Review of Highway Safety and Crash 28
    Factors Affecting Highway Safety 28
    Relationship between Crash, Friction, and Other Factors 30
      Equation Forms for Predicting Crash 30
      Statistical Approach for Predicting Crash 33
  Summary 38
Gap Analysis 39
  Review of FDOT Practice 39
    FDOT's Friction & Texture Data Collection and Management 39
    FDOT Friction Guidelines and Friction Course Policy 40
    FDOT Friction Restoration Practice 41

Other State Agency Practices ... 41
Friction & Texture Data Collection and Management ... 41
Friction Guidelines and Friction Course Policy ... 42
Friction Restoration Practice ... 44
Recommended Practice for Pavement Friction and Texture ... 45
Shortcomings of FDOT Practice (Gap Analysis) ... 57
Summary ... 59
Statistical Model Development ... 60
Construction of Integrated Database ... 60
Sources of Data ... 60
Skid Hazard Reporting (SHR) Database ... 60
Texture Database ... 61
Pavement Condition Survey (PCS) Database ... 61
Pavement Marking Management (PMM) Database ... 62
Traffic Database ... 63
Crash Analysis Reporting System (CARS) Database ... 63
Integrated Database ... 68
Preliminary Analysis ... 71
Friction Number ... 71
Nomenclatures for Surface Type and Mix Type ... 71
Friction Number Distributions ... 71
Friction Number Degradation ... 73
Traffic Distributions ... 78
Crash Count and Rate Distributions (Initial Assessment) ... 79
Friction Demand Categories ... 84
Speed Limit Category ... 84
Traffic Level Category ... 87
Recommended Friction Demand Categories ... 89
Safety Performance Function Development ... 92
Safety Performance Function: Flexible Pavements ... 92
SPF Including All Variables ... 93
SPF Including Significant Variables Only ... 96
SPF Including Significant Variables and Mix Family Information Only ... 98
SPF without Geographic Inputs ... 100

Safety Performance Function: Rigid Pavements 101
Empirical Bayes Method 102
Network Level Crash Analysis 106
Reliability vs Risk Analysis for Optimum Levels of Friction 109
Motivation 109
Concept of Reliability and Risk 111
Reliability and Risk Curves 114
Summary 120
Recommendations 121
Recommendations for FDOT's Friction Guidelines 121
Recommendations for FDOT's Friction Course Policy 123
Recommendations for FDOT's Safety Analysis Practice 125
Summary 129
References 130
Appendix A A-1

LIST OF FIGURES

Figure 1. Simplified diagram of forces acting on a rotating wheel (Hall et. al., 2006). 4
Figure 2. Key mechanisms of pavement–tire friction (Hall et. al., 2006). 5
Figure 3. Pavement friction testing equipment. 6
Figure 4. Simplified illustration of the various texture ranges that exist for a given pavement surface (Sandburg, 1998). 11
Figure 5. Texture wavelength influence on pavement–tire interactions (adapted from Henry, 2000). 11
Figure 6. Pavement texture testing equipment. 14
Figure 7. Screenshot of GripVal. 20
Figure 8. Recommended practice for friction management program (Hall et. al., 2006)............ 21
Figure 9. Current friction categories and investigatory levels in U.K. (Viner et. al., 2004) 27
Figure 10. Current Austroads friction categories and investigatory levels (Neaylon, 2011) 28
Figure 11. U.K. crash model for friction and texture (Viner et. al., 2004) 33
Figure 12. Description of Regression to Mean bias (Herbal et. al., 2010) 36
Figure 13. Illustration of Empirical Bayes method (Herbal et. al., 2010) 36
Figure 14. Observed crash count from I-81 in Virginia along with SPF and EB predictions (de Leon Izeppi et. al., 2016) 37
Figure 15. Survey Results on Porous Friction Course Use (Stanard et. al., 2007) 43
Figure 16. States with NGCS Construction (shown in Blue) (After Scofield, 2017).................. 51
Figure 17. Recommended practice for selecting adequate friction course or texturing technique (after Hall et. al., 2006; Hall and Smith, 2009).. 53
Figure 18. (a) New and (b) old longitudinally-ground surfaces (after Holzschuher, 2017a) 55
Figure 19. Texture measurements from high-speed (2D) vs. site specific devices (Holzschuher, 2017a) 56
Figure 20. PCS Rating vs amount of cracks, ride number, and rut depth (Choubane et. al., 2017) 62
Figure 21. Mapping of dry and wet weather crashes (2011 – 2017) 64
Figure 22. FDOT Districts 65
Figure 23. Florida’s (a) dry and (b) wet weather crashes by County (2011 – 2017).................... 66
Figure 24. Florida’s (a) total fatalities and (b) total crash cost by County (2011 – 2017) 67
Figure 25. Construction of the Integrated Database 68
Figure 26. (a) Dry and (b) wet weather crashes in the Integrated Database by County (2011 – 2017) 70

Figure 27. Florida’s statewide distribution of friction (2009 – 2017) 72
Figure 28. Florida’s distribution of friction by District (2009 – 2017) 73
Figure 29. FN40R vs. Pavement age 74
Figure 30. FN40R vs. Pavement age for FC125 mix family 75
Figure 31. FN40R vs. Pavement age for FC95 mix family 76
Figure 32. FN40R vs. Pavement age for FC5 mix family 77
Figure 33. AADT distribution by District (2009 – 2017))... 78
Figure 34. Statewide Distribution of Crash vs. FN40R (2011 – 2017) 79
Figure 35. Statewide Distribution of Crash vs. AADT (2011 – 2017) 80
Figure 36. Statewide Distribution of Crash Rates (2011 – 2017)... 81
Figure 37. Dry Weather Crash Rate Distribution by Speed Limit and District (2011 – 2017) 82
Figure 38. Wet Weather Crash Rate Distribution by Speed Limit and District (2011 – 2017).... 83
Figure 39. Dry Weather Crash Distribution by Speed Limit and District (2011 – 2017)............ 85
Figure 40. Wet Weather Crash Distribution by Speed Limit and District (2011 – 2017) 86
Figure 41. AADT Distribution for Different Speed Categories 87
Figure 42. Dry Weather Crash Count vs AADT Distribution for Different Speed Categories 88
Figure 43. Wet Weather Crash Count vs AADT Distribution for Different Speed Categories.... 88
Figure 44. Distribution of Crash vs. FN40R for Different Friction Demand Categories 90
Figure 45. Dry Weather Crash Rates for Different Friction Demand Categories 91
Figure 46. Wet Weather Crash Rates for Different Friction Demand Categories 91
Figure 47. Observed Crash Count, SPF Prediction, and EB Crash Counts 103
Figure 48. Crash Counts, AADT, and FN40R.. 104
Figure 49. Estimated Crash Counts Before (Top) and After (Bottom) Friction Restoration 105
Figure 50. Estimated Crash Counts with Increasing FN40R Threshold.................................... 107
Figure 51. Estimated Crash Rates with Increasing FN40R Threshold 108
Figure 52. Estimated Crash Modification Factors with Increasing FN40R Threshold 108
Figure 53. Dry Weather Crash Modification Factors with Confidence Intervals....................... 110
Figure 54. Wet Weather Crash Modification Factors with Confidence Intervals 110
Figure 55. Illustration No. 1. Reliability and Risk based on CMF Curve 111
Figure 56. Illustration No. 2. Reliability and Risk based on CMF Curve and FN40R Distribution 112
Figure 57. Illustration No. 3. Example of Risk... 113
Figure 58. Illustration No. 3. Example of Risk and Reliability 114

Figure 59. FN40R Distribution for the Recommended Friction Demand Categories 115
Figure 60. Dry Weather Benefit (Reliability) vs Risk Curves 116
Figure 61. Wet Weather Benefit (Reliability) vs Risk Curves 116
Figure 62. Dry Weather Simulation Results for Different Surface Types 124
Figure 63. Wet Weather Simulation Results for Different Surface Types 124
Figure 64. Excess Crash Counts 126
Figure 65. Percent Wet Weather vs. Total Wet Weather Crashes with FDOT's Existing Criteria (Excess Crash greater than 4 shown in Red Triangles) 126
Figure 66. Percent Wet Weather vs. Total Wet Weather Crashes with Recommended Criteria (Excess Crash greater than 4 shown in Red Triangles) 127
Figure 67. Percent Wet Weather vs. Total Wet Weather Crashes with Recommended Criteria (FN40R less than 35 shown in Red Triangles) 128

LIST OF TABLES

Table 1. Factors affecting available pavement friction (modified from Wallman and Astrom, 2001)........ 6

Table 2. Overview of pavement friction equipment........ 7

Table 3. Factors affecting pavement micro-texture and macro-texture (Hall et. al., 2006)........ 13

Table 4. Overview of pavement texture equipment........ 15

Table 5. Guide for Pavement Friction recommended methods for determining investigatory and intervention levels of friction (after Hall et. al., 2006)........ 22

Table 6. FDOT's Friction Guideline........ 24

Table 7. Intervention levels of friction (after Henry, 2000)........ 24

Table 8. Texas DOT friction demand classification (TxDOT, 2006)........ 25

Table 9. Maryland DOT straight segment friction demand classification (after Speir et. al., 2009)........ 26

Table 10. Equation forms relating crash, friction, and other variables........ 31

Table 11. FDOT's Friction Course Policy........ 40

Table 12. SHA purposes of friction measurement (after Henry, 2000)........ 42

Table 13. Other States concrete texturing practices (after Hall and Smith, 2009)........ 44

Table 14. Summary of other SHAs friction restoration practice (after McGovern et. al., 2009) 45

Table 15. Strengths and weaknesses of HMA mix types (NAPA, 2001)........ 46

Table 16. Strengths and weaknesses of PCC texturing methods (Hall and Smith, 2009)........ 47

Table 17. Rankings for PCC texturing methods (Hall and Smith, 2009)........ 50

Table 18. Nomenclatures for Surface and Mix Types in FDOT's SHR Database........ 71

Table 19. Mean and percentiles of Florida's statewide friction data (2009 – 2017)........ 72

Table 20. Mean and percentiles of Florida's friction data by District (2009 – 2017)........ 73

Table 21. Mean and percentiles of AADT (×1,000) by District (2009 – 2017)........ 79

Table 22. AADT (×1,000) Values for 33 and 66 Percentiles of Crash........ 89

Table 23. Recommended Friction Demand Categories........ 89

Table 24. Significant Variables from NB Regression [Eq. (14) and (15)]........ 94

Table 25. NB Coefficients for Numerical Variables [Eq. (14) and (15)]........ 94

Table 26. NB Coefficients for FDOT's District [Eq. (14) and (15)]........ 95

Table 27. NB Coefficients for Categorical Variables: Aggregate [Eq. (14) and (15)]........ 95

Table 28. NB Coefficients for Categorical Variables: System [Eq. (14) and (15)]........ 95

Table 29. NB Coefficients for Categorical Variables: Mix Type [Eq. (14) and (15)]........ 95

Table 30. NB Coefficients for Numerical Variables [Eq. (16) and (17)] 97
Table 31. NB Coefficients for FDOT's District [Eq. (16) and (17)] 97
Table 32. NB Coefficients for Categorical Variables: Aggregate [Eq. (16) and (17)]................. 97
Table 33. NB Coefficients for Categorical Variables: System [Eq. (16) and (17)]...................... 97
Table 34. NB Coefficients for Categorical Variables: Mix Type [Eq. (16) and (17)]................. 98
Table 35. NB Coefficients for Numerical Variables [Eq. (18) and (19)] 99
Table 36. NB Coefficients for FDOT's District [Eq. (18) and (19)] 99
Table 37. NB Coefficients for Categorical Variables: Aggregate [Eq. (18) and (19)]................. 99
Table 38. NB Coefficients for Categorical Variables: System [Eq. (18) and (19)]...................... 99
Table 39. NB Coefficients for Categorical Variables: Mix Family [Eq. (18) and (19)].............. 99
Table 40. NB Coefficients for Numerical Variables [Eq. (20) and (21)] 100
Table 41. NB Coefficients for Categorical Variables: Aggregate [Eq. (20) and (21)]............... 100
Table 42. NB Coefficients for Categorical Variables: System [Eq. (20) and (21)].................... 100
Table 43. NB Coefficients for Categorical Variables: Mix Family [Eq. (20) and (21)]............. 101
Table 44. Significant Variables from NB Regression [Eq. (22) and (23)] 102
Table 45. NB Coefficients for Numerical Variables [Eq. (22) and (23)] 102
Table 46. NB Coefficients for Categorical Variables: System [Eq. (22) and (23)].................... 102
Table 47. Recommended Levels for Probability of Benefit (Percent)....................................... 117
Table 48. FN40R Thresholds from Benefit Curves... 117
Table 49. Recommended Minimum Values of FN40R 117
Table 50. Estimated Crash Reduction Per Year... 118
Table 51. Recommended Friction Demand Categories and Levels of Friction.......................... 119
Table 52. FDOT's Existing Friction Guidelines (Same as Table 6)... 119
Table 53. Recommended Friction Guidelines 122
Table 54. Recommended Friction Course Policy. 125

INTRODUCTION

Due to their direct impact on roadway safety, pavement surface friction and texture have been recognized as important since the early days of motor vehicle transportation. Within the U.S., initial efforts for assessing the frictional characteristics of a pavement surface using full-scale tires date back to the 1950s, where the stopping distance was directly measured using a standard passenger vehicle. With the development of a first-generation locked wheel friction tester in the late 1960s and the evolution of friction measurement technology over time, assessment of pavement friction has become a common practice by all state highway agencies (SHA).

The evolution of friction equipment was accompanied by the development of new or updated policies, standards, and guidelines, from National Cooperative Highway Research Program (NCHRP) Report 37 published in 1967, through the various ASTM and American Association of State Highway and Transportation Officials (AASHTO) standards and Federal Highway Administration (FHWA) Code of Federal Regulations (CFR) and technical advisories, to the NCHRP 1-43 study that produced the current AASHTO Guide for Pavement Friction. All these efforts led the way to the current state-of-practice in which most SHAs evaluate pavement friction at a network level for pavement management or highway safety improvement programs.

FHWA's most recent Technical Advisory T 5040.38 indicates that SHAs should establish investigatory and intervention levels of pavement friction and texture as part of the friction management program (FHWA, 2010). The investigatory level, known as the "desired" level of friction, is the level of friction below which the specific pavement section needs to be evaluated for friction-related crash potential. The intervention level is defined as the level of friction below which the owner/agency is required to take corrective action. It is also frequently referred to as the "minimum required" level of pavement friction. The Technical Advisory also recommends that the investigatory and intervention friction levels be determined based on the specific needs of a facility—that is, the level of friction needed for drivers to safely operate a vehicle while performing necessary maneuvers such as accelerating, braking, and steering (FHWA, 2010; Flintsch et al., 2012; Hall et al., 2006). This level is called the "friction demand."

Ideally, the friction demand should be determined by examining the trend between pavement friction and accident rates. However, early attempts to relate crash statistics to pavement friction were unsuccessful (Henry, 2000). In addition, few recent studies have concluded that pavement friction is statistically not correlated to crash rates or crash counts (Noyce et. al., 2007; Buddhavarapu et. al., 2015). These studies further suggest that pavement friction alone is not sufficient to explain the safety characteristics of a roadway. In other words, although pavement friction is a crucial factor for improving highway safety and for reducing traffic accidents, it is not the only factor influencing the cause of crashes. In fact, traffic accidents are complicated events resulting from a combination of pavement friction and various other factors that are driver-related (e.g., distraction), vehicle-related (e.g., tires, brake system), pavement-related (e.g., structural and functional distresses, pavement marking issues), roadway-related (e.g., geometry, visibility), and weather-related (e.g., rainfall intensity, fog, ice).

Due to difficulties associated with assessing the effect of pavement friction on crash rates and assessing the friction demand, most SHAs, including the Florida Department of Transportation (FDOT), have established “required” and “desired” levels of friction based on empirical friction demand (additional discussion on FDOT’s empirical friction thresholds are discussed under “Friction Management – Agency Practices” section). As a result, many SHAs have specified universal values for the required and desired levels of friction for the entire highway network, while a few SHAs have defined the thresholds for a limited group of roadways categorized based on the posted speed limit, facility type, and roadway classes (Hall et al., 2006; Henry, 2000). Although it may be practical to maintain a certain level of pavement friction for all pavement sections within the highway network (or one of its subcategories), a single level of friction cannot be used to define a threshold that distinguishes between “safe” and “unsafe” roadways because different highways are subjected to different conditions and circumstances. In addition, the frictional characteristics of a given pavement surface change over time. Therefore, such a practice is prohibitively expensive and would not generate the cost-benefits associated with a better-targeted strategy.

A better approach may be to determine the friction demand for each roadway section in an objective manner and then design and maintain the pavement surface such that the available friction meets or exceeds the friction demand. This approach ensures adequate friction levels for a variety of roadway (intersections, approaches to traffic signals, tight curves, and ramps) and traffic conditions. Furthermore, the process of determining the friction demand or setting a friction target objectively may allow for the selection of appropriate friction course type (for flexible pavements), texturing methods (for rigid pavements and bridge decks), and materials that can meet the pavement and safety performance goals while maximizing the benefit/cost ratio of each treatment option.

As mentioned previously, the friction demand or the target friction should be determined from the relationship between crash statistics, roadway characteristics, and pavement friction. However, past research studies have indicated that friction values alone do not correlate well with crash statistics. Therefore, there is a need to re-examine the relationship between pavement friction and traffic accidents while also considering factors such as texture, mixture type, material type, friction degradation over time, and pavement condition.

RESEARCH OBJECTIVE

To meet the research needs discussed in the previous section, the objectives of this study are to:

- Quantify the relationship between pavement surface friction characteristics, traffic accident rates, and any other factor(s) deemed important.
- Determine and recommend critical threshold levels below which crash rates would significantly increase.

LITERATURE REVIEW

As a first task of the research study, a comprehensive literature review has been conducted with a focus on the relationship between crash rates and pavement surface friction, as well as other factors affecting friction and crash rates. In addition, available literature has been reviewed for other SHAs' friction measurement and management practices, friction demand setting, and friction number requirements.

REVIEW OF PAVEMENT FRICTION AND TEXTURE

Overview of Pavement Friction

Definition and Mechanism of Pavement Friction

Pavement friction is the force that resists the relative motion between a vehicle tire and a pavement surface. This resistive force, illustrated in Figure 1, is generated as the tire rolls or slides over the pavement surface.

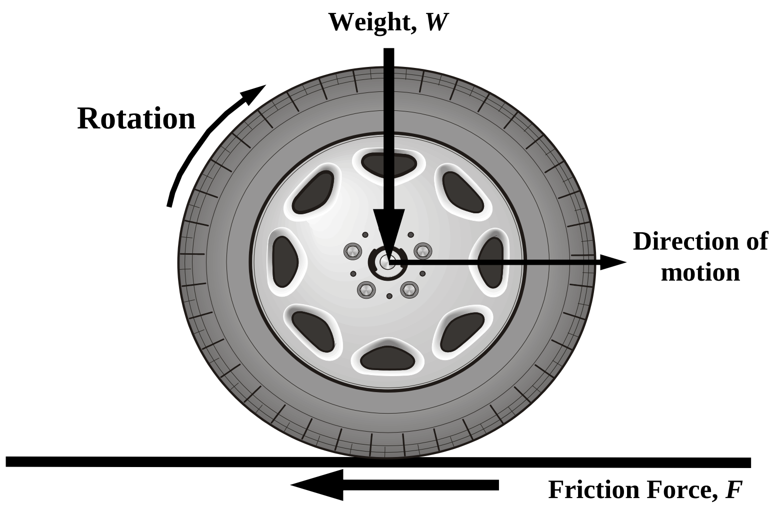


Figure 1. Simplified diagram of forces acting on a rotating wheel (Hall et. al., 2006).

The resistive force, characterized using the non-dimensional friction coefficient, μ, is the ratio of the tangential friction force (F) between the tire tread rubber and the horizontal traveled surface to the perpendicular force or vertical load (W) and is computed using the following equation.

$$\mu = \frac{F}{W} \qquad (1)$$

Pavement friction is achieved by the interaction of two principal frictional components — adhesion and hysteresis. The mechanisms of these components are illustrated in Figure 2. Adhesion corresponds to the bonding or interlocking of the vehicle tire and the pavement surface upon contact. The hysteresis component results from the energy loss due to bulk deformation or the "enveloping effect" of the tire around the surface texture. When a tire is compressed against texture, the energy due to the deformation is stored within the tire. When the tire is relaxed, part of the stored energy is recovered, while the other part is lost in the form of heat (hysteresis). This loss in energy is irreversible and leaves a net frictional force to help stop the forward motion. Although there are other components of pavement friction (e.g., tire rubber shear), they are relatively insignificant when compared to the primary components mentioned above. Thus, friction can be viewed as the sum of the adhesion and hysteresis frictional forces.

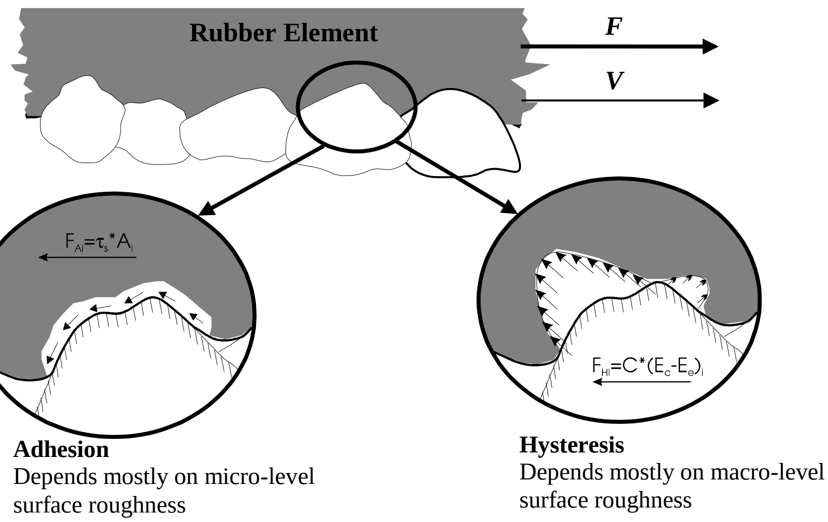


Figure 2. Key mechanisms of pavement–tire friction (Hall et. al., 2006).

Because adhesion force is developed at the pavement–tire interface, it is mostly dependent on the micro-texture of the aggregate particles. On the other hand, the hysteresis force is mostly dependent on the macro-texture formed in the surface via mix design and/or construction techniques. As a result, adhesion governs the overall friction on smooth-textured and dry pavements, while hysteresis is the dominant component on wet and rough-textured pavements. (Micro-texture and macro-texture will be defined in following sections of this report).

Factors Affecting Available Pavement Friction

The factors that influence pavement friction forces can be grouped into four categories: pavement surface characteristics, vehicle operational parameters, tire properties, and environmental factors. Table 1 lists the various factors in each category, with the more critical factors shown in bold. Among these factors, those that are considered to be within an SHA's control and should be considered in the friction targeting and design, are micro-texture and macro-texture, pavement material properties, and vehicle speed.

Table 1. Factors affecting available pavement friction (modified from Wallman and Astrom, 2001).

Pavement Surface Characteristics	Vehicle Operating Parameters	Tire Properties	Environment
• **Micro-texture** • **Macro-texture** • Mega-texture/ unevenness • **Material properties** • Temperature	• **Slip speed** ➢ Vehicle speed ➢ Braking action • Driving maneuver ➢ Turning ➢ Overtaking • Tire wear	• Footprint • **Tread design and condition** • Rubber composition and hardness • **Inflation pressure** • Load • Temperature	• Climate ➢ Wind ➢ **Temperature** ➢ **Water (rainfall, condensation)** ➢ **Snow and ice** • **Contaminants** ➢ Anti-skid material (salt, sand) ➢ Dirt, mud, debris

Note: Critical factors are shown in bold.

Friction Equipment

A list of field testing equipment for pavement friction is shown in Figure 3. As shown, the equipment can be separated into two categories: (1) full-scale, vehicle mounted equipment that can be operated at highway speed without traffic control and (2) portable devices that are used for site-specific testing under traffic control. The vehicle mounted equipment can be further categorized into those that provide continuous friction measurements along the roadway and those that are designed to test for friction intermittently (or in a discrete manner). Additional information on the friction testing equipment is provided in Table 2.

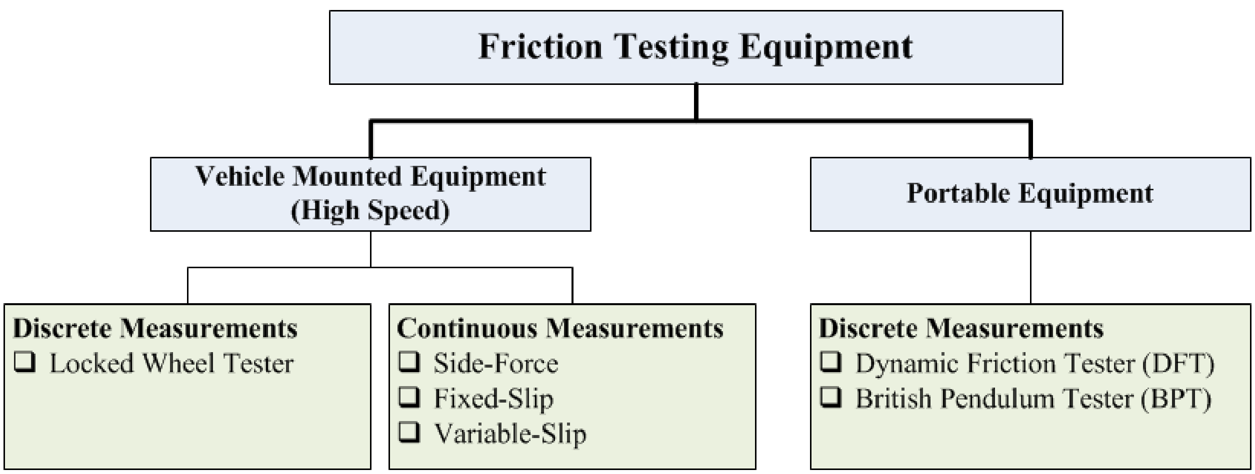


Figure 3. Pavement friction testing equipment.

Table 2. Overview of pavement friction equipment.

Test Method	Description	Equipment	Advantages	Disadvantages
Locked-Wheel	The locked wheel tester is a full-sized friction testing device standardized in ASTM E 274. During testing, the test tire is completely locked up (typically for 1 to 3 seconds) and dragged over the pavement. The system records both the horizontal friction force and the dynamic vertical load of the friction trailer during testing. Based on these two measurements, the FN can be calculated as follows: $FN = 100 \cdot \mu = 100 \cdot (F/W)$ where F is the sum of all horizontal forces acting on the test tire at the pavement–tire contact area and W is the dynamic vertical load applied to the test wheel. The friction number is simply the coefficient of friction expressed in percentage. The FN measured at 40 mph using an ASTM E 501 ribbed tire is designated as FN40R. Similarly, if a smooth tire as specified in ASTM E 524 is used for the locked wheel test at 40 mph, then the friction number is designated as FN40S.	 	• Well developed and very widely used in the U.S. • User friendly, relatively simple, and not time consuming.	• Can only be used on straight segments (no curves, T-sections, or roundabouts). • Measurements are intermittent.
Side-Force	The side-force method is standardized in ASTM E 670 and measures the ability of vehicles to maintain control in curves. The test tire is fixed at a constant angle (known as the yaw angle, typically between 7.5 and 20°) from the direction of travel. The side-force coefficient (*SFC*) is calculated as follows: $SFC\ (V, \alpha) = 100 \times (F_S/W)$ where V is the velocity of the test vehicle, α is the yaw angle, F_S is the force perpendicular to plane of rotation, and W is the vertical load applied to the tire. The two most common side-force measuring devices are the Mu-Meter and the Side-Force Coefficient Road Inventory Machine (SCRIM), both of which were originally developed in the U.K.	 Mu-Meter  SCRIM	• Relatively well controlled skid condition similar to fixed-slip device results. • Measurements are continuous throughout a test pavement section. • Commonly used in Europe.	• Very sensitive to road irregularities (potholes, cracks, etc.) which can destroy tires quickly. • Mu-Meter is primarily only used for airports in the U.S.

Table 2. Overview of pavement friction equipment (continued).

Test Method	Description	Equipment	Advantages	Disadvantages
Fixed-Slip	Fixed-slip devices measure the rotational resistance of smooth tires slipping at a constant slip speed (typically 10 to 20 percent). The test tire maintains a constant, predefined slip as the vertical load is applied to the test tire. The frictional force in the direction of motion between the tire and pavement is measured, and the percent slip (% *Slip*) is computed as follows: $\% \text{ Slip} = 100 \cdot (V - r\omega)/V$ where V is the test speed, r is the effective tire rolling radius, and ω is the angular velocity of test tire.	 Runway friction testers  U.K. Grip-Tester	• Continuous, high resolution friction data collected.	• The slip speeds do not always coincide with the critical slip speed value. • Uses large amounts of water in continuous mode. • Requires skillful data reduction.
Variable-Slip	Variable-slip devices (ASTM E 1859) measure the frictional force, as the tire is taken through a predetermined set of slip ratios (0 to 100 percent). At the start of testing, water is applied to the pavement surface and the wheel is allowed to rotate freely. Then, the test wheel speed is reduced gradually and the vehicle speed, travel distance, tire rotational speed, wheel load, and frictional force are collected continuously. Raw data are recorded for later filtering, smoothing, and reporting.		• Continuous measurement of any desired fixed or variable slip force..	• Large, complex equipment with high maintenance costs. • Complex data processing and analysis needs. • Uses large amounts of water in continuous mode.
DFT	The DFT is a portable device for obtaining friction measurements of flat surfaces as standardized in ASTM E 1911. It consists of a horizontal 13.75-inch-diameter spinning disk with three spring loaded rubber sliders. The disk is driven by a motor and suspended over a pavement surface until the tangential speed reaches 55 mph. Water is then applied to the pavement surface and the disk is lowered to the surface. The friction force is measured by a transducer as the disk spin slows.	 	• Continuous measurement of friction with respect to the tangential speed of the spinning disk. • Allows for determining the friction and velocity relationship.	• Site specific. • Requires traffic control.

Table 2. Overview of pavement friction equipment (continued).

Test Method	Description	Equipment	Advantages	Disadvantages
BPM	The British Pendulum Tester (BPT) produces a low-speed sliding contact between a standard rubber slider and the pavement surface (AASHTO T 278 or ASTM E 303). The elevation to which the arm swings after contact provides an indicator of the frictional properties. Data from five readings are typically collected and manually recorded.		• Easy to operate.	• Site specific. • Requires traffic control.

Overview of Pavement Texture

Definition and Significance of Texture

Pavement surface texture is defined as the deviations of the pavement surface from a true planar surface. These deviations occur at three distinct levels of scale, each defined by the wavelength (λ) and peak-to-peak amplitude (A) of its components. The three levels of texture, as established in 1987 by the World Road Association (WRA), formerly known as the Permanent International Association of Road Congresses (PIARC), are as follows:

- Micro-texture (λ < 0.02 in., A = 0.04 to 20 mils) — Surface roughness quality at the sub-visible or microscopic level. It is a function of the surface properties of the aggregate particles contained in the asphalt or concrete paving material.
- Macro-texture (λ = 0.02 to 2 in., A = 0.005 to 0.8 in.) — Surface roughness quality defined by the mixture properties (shape, size, and gradation of aggregate) of asphalt paving mixtures and the method of finishing/texturing (dragging, tining, grooving; depth, width, spacing and orientation of channels/grooves) used on a concrete paved surfaces.
- Mega-texture (λ = 2 to 20 in., A = 0.005 to 2 in.) — Texture with wavelengths in the same order of size as the pavement–tire interface. It is largely defined by the distress, defects, or "waviness" on the pavement surface.

Wavelengths longer than the upper limit of mega-texture are defined as roughness or unevenness (Henry, 2000). Figure 4 illustrates the three texture ranges, as well as a fourth level—roughness/unevenness—representing wavelengths longer than the upper limit (20 in.) of mega-texture.

It is widely recognized that pavement surface texture influences many different pavement–tire interactions including friction and noise. Figure 5 shows the ranges of texture wavelengths affecting various vehicle–road interactions, including friction, interior and exterior noise, splash and spray, rolling resistance, and tire wear. As can be seen, friction is primarily affected by micro-texture and macro-texture, which correspond to the adhesion and hysteresis friction components, respectively.

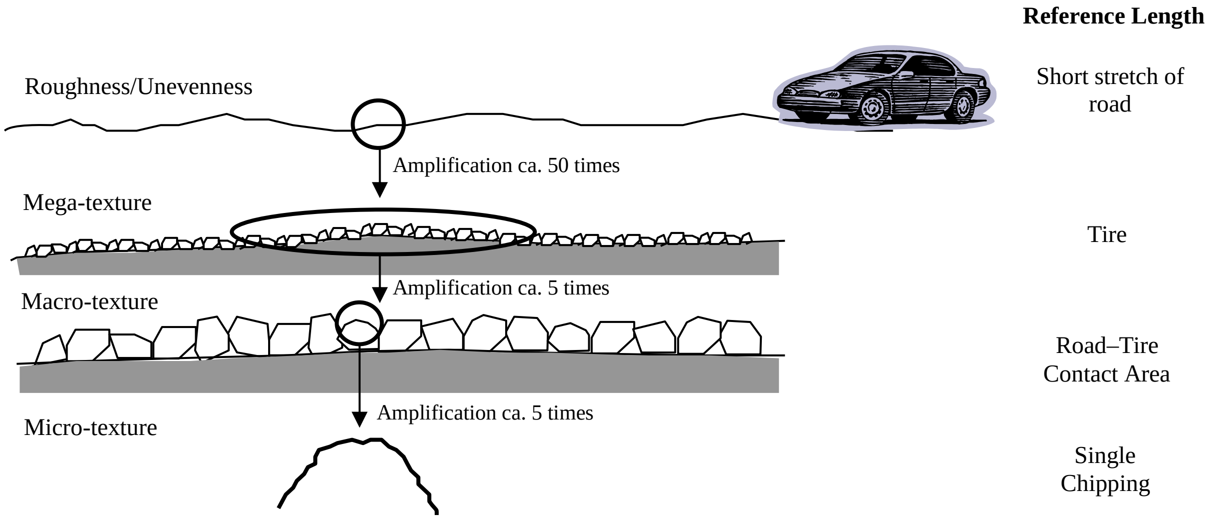


Figure 4. Simplified illustration of the various texture ranges that exist for a given pavement surface (Sandburg, 1998).

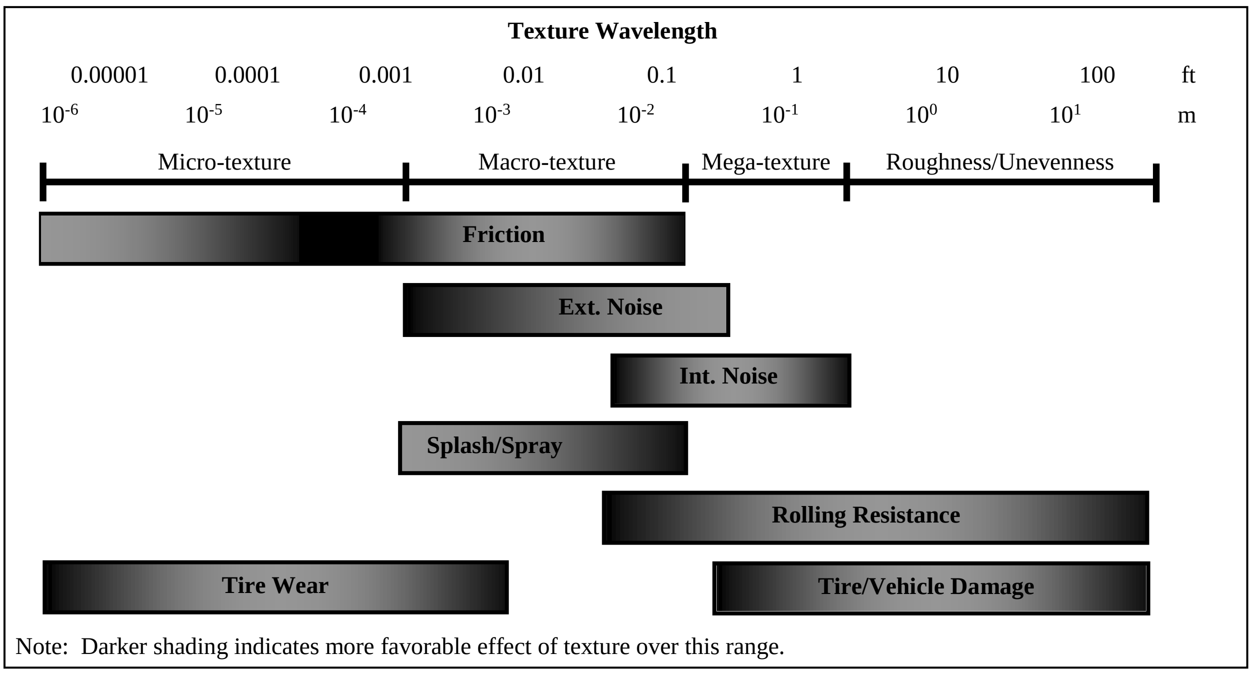


Figure 5. Texture wavelength influence on pavement–tire interactions (adapted from Henry, 2000).

Factors Affecting Texture

The texture of an asphalt surface is mostly achieved from the coarse aggregate characteristics and mixture properties, and by the placement of the surface layer, without supplemental treatments. Therefore, the macrotexture of asphalt surface is greatly dependent on the type of asphalt surface course placed (e.g., dense-graded, open-graded, and stone-matrix surface course). Additional factors that affect asphalt pavement surface texture, which relate to the aggregate properties, asphalt binder, and mix properties are as follows:

- Maximum Aggregate Dimensions—The size of the largest aggregates in an asphalt concrete (AC) provides the dominant macro-texture wavelength, if closely and evenly spaced.
- Coarse Aggregate Type—The coarse aggregate type will control the particle angularity, shape factor, and durability.
- Fine Aggregate Type—The angularity and durability of the fine aggregate is controlled by the aggregate composition, and mineralogy and whether or not it is crushed.
- Binder Viscosity and Content—Asphalt binders with low viscosities tend to cause bleeding more readily than higher viscosity grades. Also, excessive amounts of binder (all types) can result in bleeding. Bleeding results in a reduction or total loss of pavement surface micro-texture and macro-texture. Because binder also holds the aggregate particles in position, a binder with good resistance to weathering is very important.
- Mix Gradation—Gradation of the mix, particularly for porous pavements, will affect the stability and air voids of the pavement.
- Air Voids—Increased air void content, such as open graded mixes with at least 15 percent voids designed to provide increased water drainage and reduced potential for hydroplaning, may also affect pavement friction and safety of roadways.
- Layer Thickness—Increased layer thickness for porous pavements provides a larger volume for water dispersal. On the other hand, increased thickness reduces the frequency of the peak sound absorption.

The macrotexture of a Portland Cement Concrete (PCC) surface is more or less “manufactured” in the sense that it is mostly achieved by a supplemental treatment after material placement (e.g., burlap- or turf-drag, tining, grinding, grooving, exposed aggregate, etc). Hence, the macrotexture of a concrete surface is not only affected by the mixture characteristics but also any texturing done to the material after placement. Factors that affect texture of concrete pavement include:

- Coarse Aggregate Type—The selection of coarse aggregate type will control the stone material, its angularity, its shape factor, and its durability. This is particularly critical for exposed aggregate PCC pavements.
- Texture Dimensions—The dimensions of tining, grooving, grinding, and turf dragging of PCC surfaces affect the macro-texture, and therefore friction and noise.
- Texture Spacing—Spacing of transverse tining and grooving not only increases the amplitude of certain macro-texture wavelengths, but can affect the noise frequency spectrum.
- Texture Orientation—PCC surface texturing can be oriented transverse, longitudinal, and diagonally to the direction of traffic. The orientation affects tire vibrations and, hence, noise.
- Isotropic or Anisotropic—Consistency in the surface texture in all directions (isotropic) will minimize longer wavelengths, thereby reducing noise.
- Texture Skew—Positive skew results from the majority of peaks in the macro-texture profile, while negative skew results from a majority of valleys in the profile.

Table 3 provides a summary of how these factors influence micro-texture and macro-texture. These factors can be optimized to obtain pavement surface characteristics required for a given design situation.

Table 3. Factors affecting pavement micro-texture and macro-texture (Hall et. al., 2006).

Pavement Surface Type	Factor	Macro-Texture	Micro-Texture
Asphalt	Maximum Aggregate Dimensions	X	
	Coarse Aggregate Types	X	X
	Fine Aggregate Types	X	
	Mix Gradation	X	
	Mix Air Content	X	
	Mix Binder	X	
Concrete	Coarse Aggregate Type	X (For Exposed Agg. PCC)	X (For Exposed Agg. PCC)
	Fine Aggregate Type		X
	Mix Gradation	X (For Exposed Agg. PCC)	
	Texture Dimensions And Spacing	X	
	Texture Orientation	X	
	Texture Skew	X	

Texture Equipment

Nearly all of the current standards established by ASTM, ISO, and PIARC use the concept of texture depth to quantify or characterize pavement surface texture. Texture depth is defined as the average distance within a given surface area (in the same order of size as the tire/road interface) between the surface and a plane through the top of the three highest particles "well spaced" within the surface area.

Figure 6 shows a list of equipment for pavement texture measurement. Information on each texture measuring device, and relevant texture indices or parameters, is provided in Table 4.

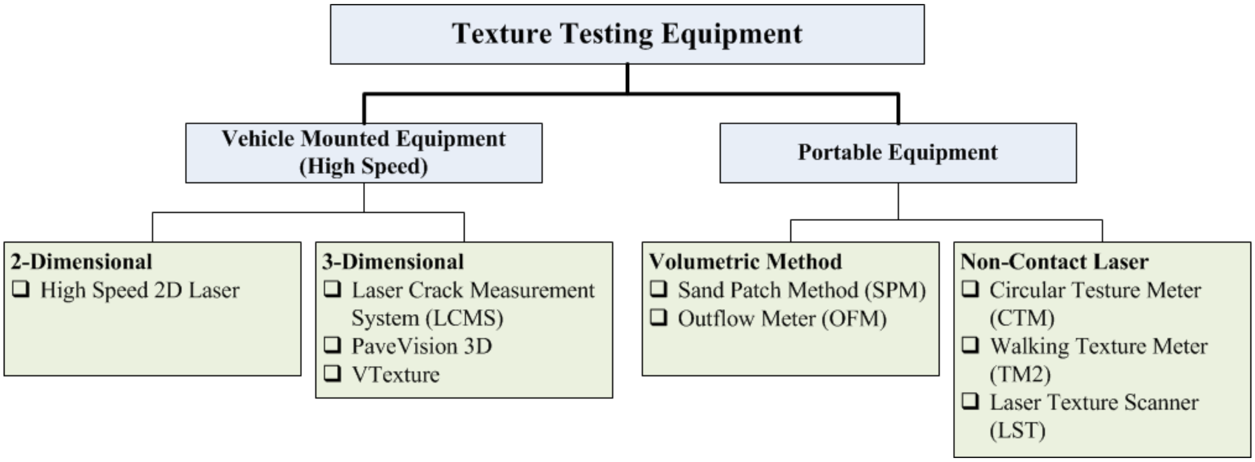


Figure 6. Pavement texture testing equipment.

Table 4. Overview of pavement texture equipment

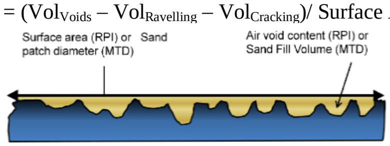
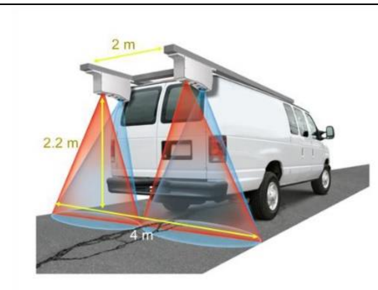
Test Method/ Equipment	Description	Equipment	Advantages	Disadvantages
High Speed, 2-D Laser	Non-contact very high-speed lasers are used to collect pavement surface elevations at intervals of 0.01 inch or less. High-speed laser texture measuring equipment uses a combination of a horizontal distance measuring device and a very high speed (64 kHz or higher) laser triangulation sensor. Vertical resolution is usually 0.002 inch or better. The laser equipment is mounted on a high-speed vehicle, and data is collected and stored in a portable computer. This type of system, therefore, is capable of measuring pavement surface macro-texture profiles and indices. Global Positioning Systems (GPS) are often added to this system to assist in locating the test site. Data collecting and processing software filters and computes the texture profiles and other texture indices. These devices can be operated at speeds of up to 70 mph and allow for the calculation of a Mean Profile Depth (MPD) which is which is a two-dimensional estimate of the three-dimensional texture. Calculation of MPD is based on ASTM E 1845 or ISO 13473-1,2,3.	 	• Collects continuous data at high speeds. • Correlates well with MTD. • Can be used to provide a speed constant to accompany friction data.	• Equipment is expensive. • Only measures pavement profile and hence texture characteristics over a single longitudinal path.
LCMS (3-D Laser System)	The laser crack measurement system (LCMS) uses laser line projectors, high speed cameras and advanced optics to acquire high resolution 3D profiles of the road. This unique 3D vision technology allows for automatic pavement condition assessment of asphalt, porous asphalt, chipseal and concrete surfaces. The LCMS acquires both 3D and 2D image data of the road surface with 1 mm resolution over a 4 m lane width at survey speeds up to 60 mph. Using the measured texture profiles, the system computes the MPD, RMS, and Road Porosity Index (RPI) based on a Digital Sand Patch model. RPI is defined as the volume of the voids in the road surface that would be occupied by the sand divided by a user defined surface area. Since the standard sand patch method specifies that texture should not be measured at locations where cracking or other surface defects are present, the volume of 'holes' present because of cracking, ravelling or other defects are subtracted from the RPI calculation. In other words, $RPI = (Vol_{Voids} - Vol_{Ravelling} - Vol_{Cracking}) / \text{Surface Area}$  	 	• Collects continuous data at high speeds. • Correlates well with MPD. • Covers entire pavement width	• Equipment is very expensive. • Skilled operators are required for collection and data processing. • Extremely large amount of data storage needed.

Table 4. Overview of pavement texture equipment (continued)

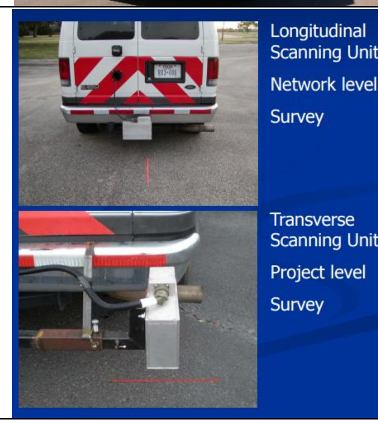
Test Method/ Equipment	Description	Equipment	Advantages	Disadvantages
PaveVision 3D (3-D Laser System)	Integrated vehicular surveying platform that allows for measuring cracking, rutting, texture/hydroplaning, and longitudinal profiling. Uses 30KHz 3D profile scanning rate, to allow for 1mm resolution in three dimensions at 60 mph. Using the measured texture profiles, the system computes the MPD for each transverse profile collected.		• Collects continuous data at high speeds. • Correlates well with MTD. • Covers entire pavement width	• Equipment is very expensive. • Skilled operators are required for collection and data processing. • Extremely large amount of storage needed.
VTexture (3-D Laser System)	Based on high speed 3D laser technology. The system samples 2048 points for every 2 inches of travel, covering a 12 inches wide surface with 10 µm vertical resolution and 0.007 inch spatial (x, y) resolution. The device can be mounted to measure the profile in the longitudinal or transverse direction. Using the measured texture profiles, the system computes the MPD for each transverse profile collected.		• Collects continuous data at high speeds. • Correlates well with MTD. • Equipment can be mounted to collect MPD longitudinally or transversely.	• Does not cover full pavement width (laser footprint is 12 inches wide).
Sand Patch Method (SPM)	This volumetric-based spot test method provides the mean depth of pavement surface macro-texture in accordance with ASTM E 965 or ISO 10844. The operator spreads a known volume of glass beads in a circle onto a cleaned surface and determines the diameter and subsequently mean texture depth (*MTD*). The MTD is computed as: $$MTD = \frac{4V}{\Pi \times D^2}$$ where V is the sample volume and D is the average diameter of sand patched area.		• Simple and inexpensive methods and equipment. • When combined with other data, can provide friction information.	• Method is slow and requires lane closure. • Only represents a small area. • Only macro-texture is evaluated. • Sensitive to operator variability. • Labor intensive.

Table 4. Overview of pavement texture equipment (continued)

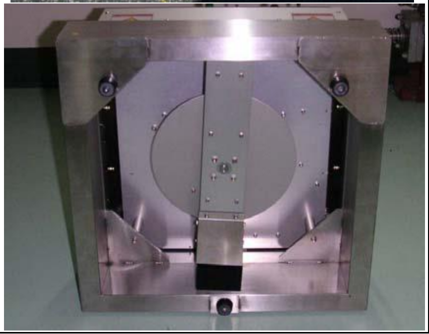
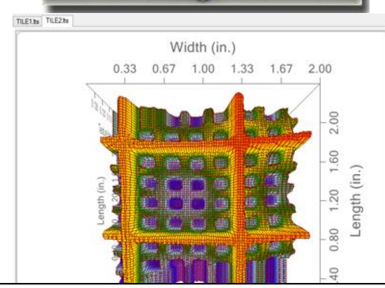
Test Method/ Equipment	Description	Equipment	Advantages	Disadvantages
Outflow Meter (OFM)	This volumetric test method measures the water drainage rate through surface texture and interior voids (ASTM E 2380). It indicates the hydroplaning potential of a surface by relating to the escape time of water beneath a moving tire. Correlations with other texture methods have also been developed. Equipment is a cylinder with a rubber ring on the bottom and an open top. Sensors measure the time required for a known volume of water to pass under the seal or into the pavement. The texture index is given as the Outflow Time (*OFT*) which is the time in milliseconds for outflow of specified volume of water. Shorter outflow times indicate rougher surface texture.		• Simple methods and relatively inexpensive equipment. • Provides an indication of hydroplaning potential in wet weather.	• Method is slow and requires lane closure. • Only represents a small area of the pavement surface. • Output does not have a good correlation with MPD or MTD
Circular Texture Meter (CTM)	The CTM uses a charge couple device (CCD) laser displacement sensor to measure surface texture and is designated in ASTM E 2157. The laser sensor is mounted on an arm that rotates around a central point at a fixed distance above the pavement and measures the change in surface elevation. The CTM is portable, collects data in an 11.2-inch diameter, and is often used along with the DFT to measure texture in the same path friction was measured. Indices provided by the CTM include the mean profile depth (*MPD*) and the root mean square (*RMS*) macro-texture.	 	• Measures same diameter as DFT, allowing texture–friction comparisons. • Repeatable, reproducible, and independent of operators • Correlates well with *MTD*. • Measures positive and negative texture.	• Method is slow (about 45 seconds to complete) and requires lane closure. • Represents a small surface area.

Table 4. Overview of pavement texture equipment (continued)

Test Method/ Equipment	Description	Equipment	Advantages	Disadvantages
Walking Texturemeter TM2	The texture is measured over a 4.0 inches laser footprint projected transversely to the direction of travel at approximately every 0.08 inch. The texture values (MPD and RMS) calculated over these transverse profiles are averaged and can be reported at any desired interval between 33 ft. and 164 ft. Equipment includes a portable computer with touch screen, the texture meter device, and a battery that allows for continuous operation for up to 10 hours. Indices provided by the CTM include the mean profile depth (*MPD*) and the root mean square (*RMS*) macro-texture.		• Repeatable, reproducible, and independent of operators • Measures positive and negative texture. • Is small and portable. • Setup time is short (less than 1 minute)	• Method is slow and requires lane closure. • Represents a small surface area.
Laser Texture Scanner	This non-contact laser device measures the surface profile over a 2.84 inches by 4.25 inches area, creating a 3-D profile of the pavement surface. The texture is reported in terms of MPD, Texture Profile Index (TPI), and elevation variance/slope variance. The device can also provide the texture in terms of Estimated Texture Depth (ETD), which is the MTD estimated from the MPD using regression models Equipment includes a laser sensor with a footprint of 0.002 inch, having a vertical resolution of 0.0006 inch. It also includes a battery and a GPS receiver. Indices provided by the Laser Texture Scanner include the mean profile depth (*MPD*), the root mean square (*RMS*), the Texture Profile Index (TPI), and slope/elevation variance of macro-texture.		• Repeatable, reproducible, and independent of operators • Measures positive and negative texture. • Small and portable.	• Method is slow and requires lane closure. • Represents a small surface area.

AGENCY PRACTICES FOR MEASURING FRICTION AND TEXTURE

An agency survey conducted by Henry (2000) indicated that the most common method for measuring pavement friction in the U.S. is the locked-wheel method (ASTM E 274) with ribbed test tires (ASTM E 501). This finding was confirmed by more recent surveys conducted by Hall et. al. (2006) and Noyce et. al. (2007).

It is also worthwhile to note that most of the European countries are using the Continuous Friction Measurement Equipment (CFME) such as SCRIM, Mu-Meter, and Grip-Testers (See Table 2) for network level friction testing. Unlike the CFMEs, the locked-wheel method is conducted in a "discrete" or "intermittent" manner, as the testing involves a complete lock up of the test tire for a duration of 3 seconds or less. Henry (2000) also reported that in the U.S., the locked-wheel testing is mostly conducted at a frequency of 1 to 5 lock ups per mile, with only a few agencies (e.g., Illinois, New York, Pennsylvania, and Wisconsin) conducting 10 lock-ups per mile.

Similar to other State agencies in the U.S., FDOT's current practice for network level friction testing involves the use of a locked-wheel tester. The testing is conducted on the driving lanes of multi-lane roads or on a single lane for two-lane, two-way roadways. The normal frequency of locked-wheel testing is 3 lock ups per mile.

A joint effort by FHWA and six State DOTs acquired CFMEs (Grip-Testers to be more specific) and initiated a CFME loan program starting in 2007 (de Leon Izzepi et. al., 2011). As part of the CFME loan program, a software called GripVal was developed for analysing the continuous friction measurement (Figure 7). Although CFMEs are frequently used in the U.S. for friction testing of airport runways, implementation of CFMEs for roadway friction testing is still in the pilot stage. Virginia recently acquired a Grip-Tester and is currently evaluating its use for pavement friction management (de Leon Izzepi et. al., 2016). Additional information regarding this pilot effort will be provided subsequently in this report.

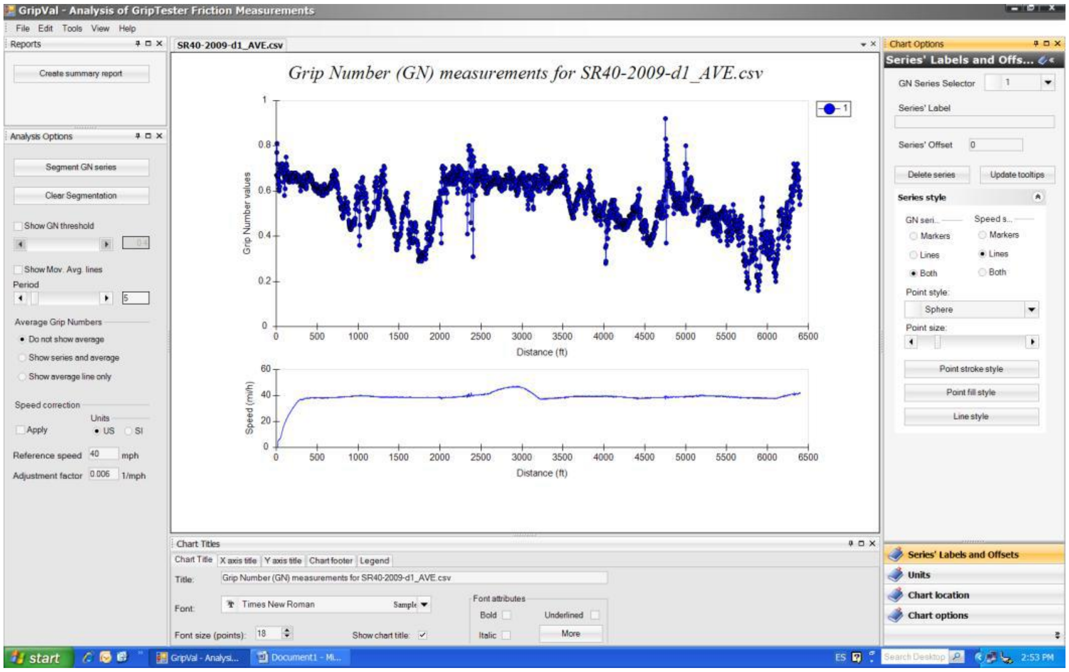


Figure 7. Screenshot of GripVal.

A survey of the SHA practices showed that texture measurements were not conducted as frequently as friction measurements, especially for routine, inventory testing (Henry, 2000). This is due to the traditional, site-specific testing methods that are labor intensive and require traffic control. Literature reviews indicated that the SHAs are still using these site-specific technologies for texture measurements (Speir et. al., 2009; Hall and Smith, 2009; Lu and Steven, 2008).

With the introduction of high-speed, vehicle-mounted 2D laser systems in the late 1990’s for measuring texture (see Table 4), collection of texture data at a network level became potentially possible. However, the 2D laser system has its inherent limitations as the texture data is collected along a single longitudinal path. This limitation poses challenges in texture measurements especially on rigid pavements with directional characteristics such as longitudinal grinding or grooving (Hall and Smith, 2009; Holzschuher, 2017a).

Recent developments of the high-speed 3D laser systems for texture measurement showed potential for overcoming the limitations of the 2D systems. However, the technology is still evolving and there is no standard regarding the texture indices, data collection protocols, analysis procedures, and use of the massive amount of texture data from these systems. An on-going NCHRP study (NCHRP 10-98) is expected to provide more guidelines on these topics.

REVIEW OF PAVEMENT FRICTION MANAGEMENT

This section of the report provides a summary of the recommended AASHTO friction program along with some SHA practices on friction management.

<u>Recommended Practice by AASHTO</u>

The AASHTO Guide for friction management program includes five steps shown in Figure 8. The fourth step involves studying the friction and crash trends for determining the investigatory and intervention levels of friction. As indicated the AASHTO Guide provides three methods for determining these friction levels based on availability of data. Table 5 summarizes the three methods.

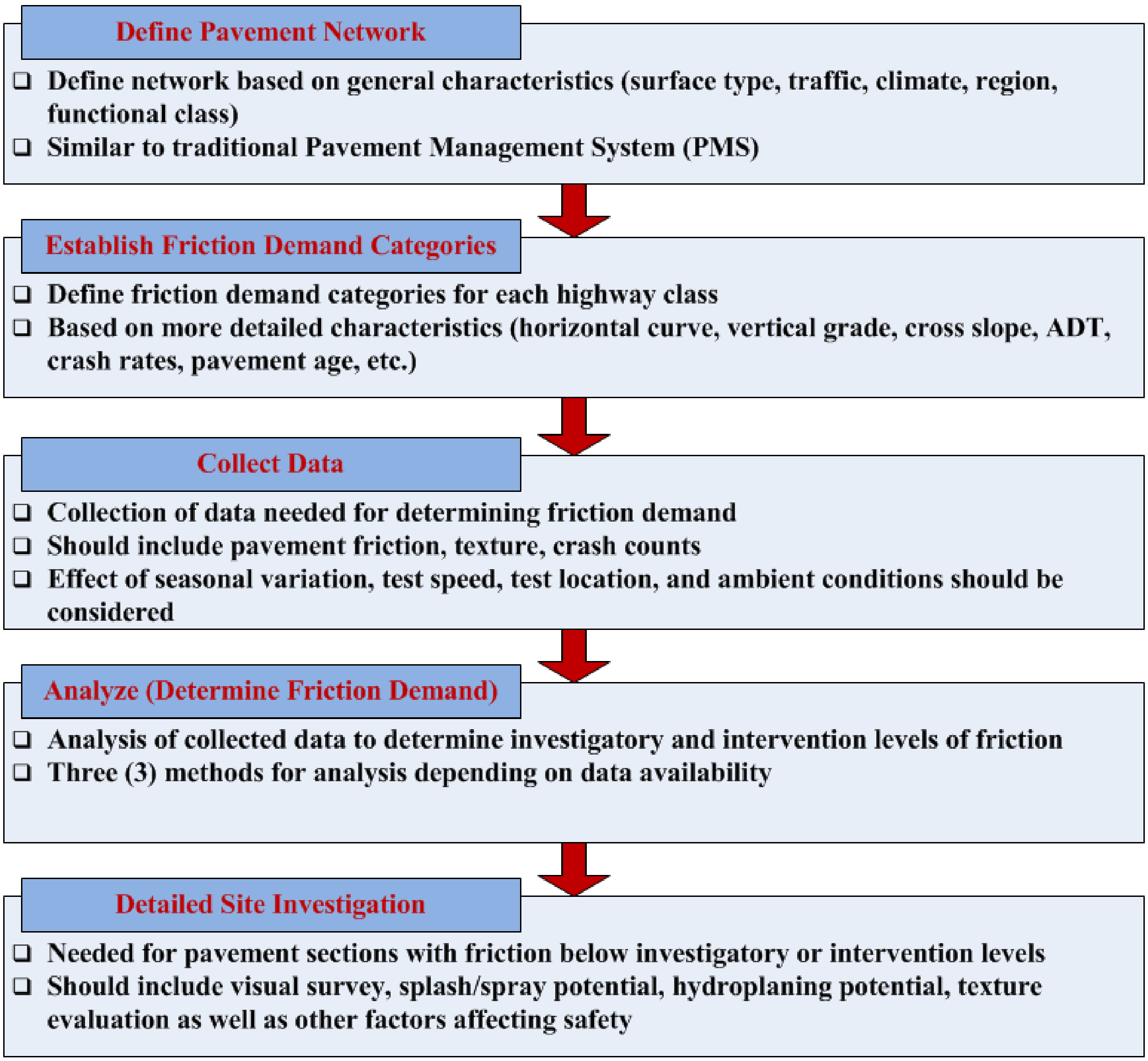


Figure 8. Recommended practice for friction management program (Hall et. al., 2006)

Table 5. Guide for Pavement Friction recommended methods for determining investigatory and intervention levels of friction (after Hall et. al., 2006)

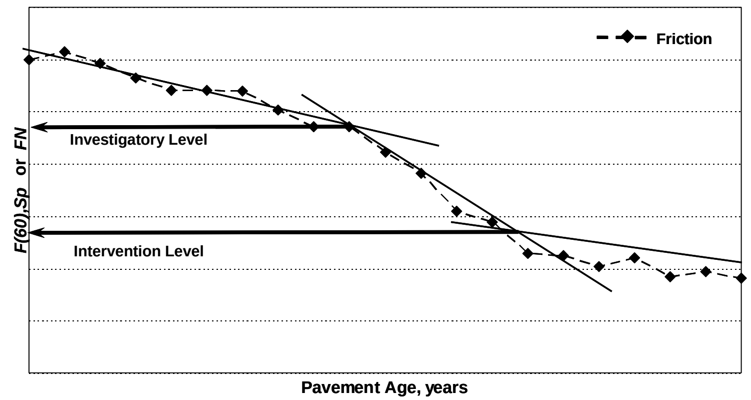
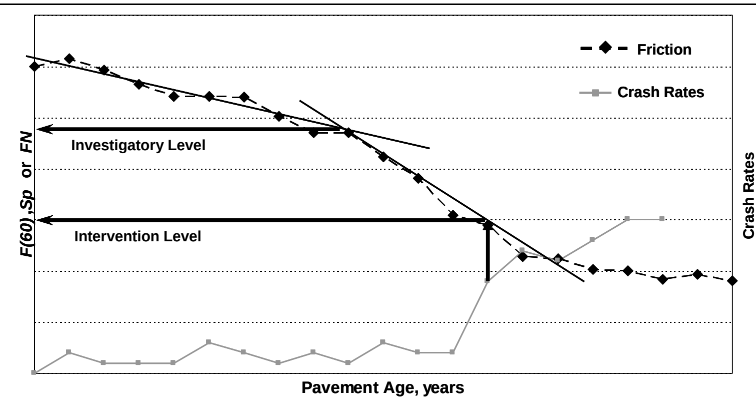
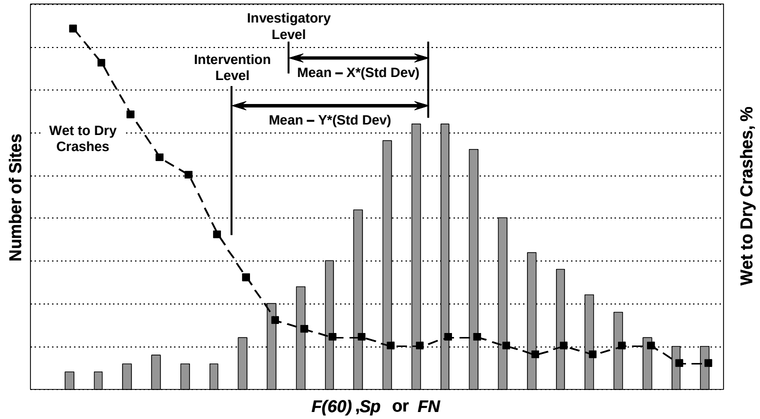
Method	Illustration	Description
1		Method 1 is based on the historical trends of friction loss over pavement age or time for a specific friction demand category. • The investigatory level is set at the value where friction loss begins to increase at a faster rate. • The intervention level is set at a certain amount of friction or percentage below the investigatory level.
2		Method 2 uses the historical pavement friction loss (similar to Method 1) as well as the crash data for the given friction demand category. • The investigatory level is set at the value where friction loss begins to increase at a faster rate (same as Method 1). • The intervention level is set to the friction value below which there is a significant increase in the number of crashes.

Table 5. Guide for Pavement Friction recommended methods for determining investigatory and intervention levels of friction (after Hall et. al., 2006), continued

Method	Illustration	Description
3		Method 3 uses the distribution of friction data (instead of friction loss over time) and the number of crashes for the friction demand category. • The investigatory level is set to a certain standard deviations below the mean friction value. The wet-to-dry crashes should be studied for determining the number of standard deviations. • The intervention level is also set to a certain standard deviations below the mean friction value. The minimum satisfactory wet-to-dry crash rate should be used for determining the multiplier for the standard deviation.

Agency Practices

FDOT's current friction guideline implemented as part of the highway safety improvement program is summarized in Table 6. As shown in the table, FDOT specifies a desired FN40R of 35 for roadways with a posted speed limit greater than 45 mph and a desired FN40R of 30 for all other roadways. New or existing pavements with FN40R below these thresholds are subjected to a safety review, investigation, and/or crash monitoring. The fact that a higher level of friction is specified for roadways with speed limit greater than 45 mph, clearly reflects that the friction demand is higher for high-speed facilities. However, it is also noted that these thresholds were established without a rigorous relationship to crash statistics. In other words, these thresholds were determined in an empirical manner.

Table 6. FDOT's Friction Guideline

Speed Limit or Design Speed (mph)	All Highway Surfaces		
	Questionable[1]	Review[2]	Desired[3]
	FN40R	FN40R	FN40R
Less than or equal to 45	25	26 - 28	30
Greater than 45	27	28 - 30	35

Note 1: FN below these thresholds warrants investigation (existing pavements) or crash monitoring (new pavements).
Note 2: FN below these thresholds warrants review (existing pavements) or crash monitoring (new pavements).
Note 3: Desired FN values for new pavements.

Similarly, most SHAs have established the investigatory and intervention levels of friction in an empirical manner, but not following the AASHTO Friction Guide recommendations. Table 7 summarizes some of the empirical intervention levels used by the SHAs in the U.S. The reason behind the empirical establishment of these friction levels is believed to be due to (1) the challenges and lack of understanding related to the relationship between pavement friction and crash, and (2) the empirical thresholds have been established a few decades ago when the SHAs began the friction testing and the practice has not changed since then.

Table 7. Intervention levels of friction (after Henry, 2000)

Agency	Interstate	Primary	Secondary	Local
Arizona	34 (Mu-Meter)			N/A
Idaho	FN40S > 30			N/A
Illinois	FN40R > 30			N/A
Kentucky	FN40R > 28	FN40R > 25		
New York	FN40R > 32			
South Carolina	FN40R > 41	FN40R > 37		N/A
Texas	FN40R > 30	FN40R > 26	FN40R > 22	N/A
Utah	FN40R > 30-35	FN40R > 35		N/A
Washington	FN40R > 30			
Wyoming	FN40R > 35			N/A

McGovern et. al. (2011) conducted a detailed study of wet weather crash reduction programs in California, Florida, Michigan, New York, and Virginia. They found that all of these five states actively maintain pavement friction databases. The study also indicated that these states use empirical friction thresholds and that none have implemented a systematic approach focused specifically on friction-related wet weather crashes. Instead, the focus of the current practice is on spot-improvements at locations of high wet weather crashes. The friction measurements are used either to identify potential problematic locations (Michigan) or to evaluate areas with high crash rates (all five states).

As mentioned previously, it would be ideal if the friction demand for each roadway section is determined in an objective manner and to design and maintain the pavement surface such that the available friction meets or exceeds the friction demand. Selecting pavement types and materials to meet the friction demand of a pavement may allow for a better allocation of the resources. In the U.S., however, very few agencies have developed a methodology for determining the friction demand of a given roadway. Texas and Maryland are among the first to establish a procedure for estimating the friction demand.

According to Texas DOT's Wet Weather Accident Reduction Program (WWARP), the first step is toward determination of the overall frictional demand of a road surface is consideration of factors shown in Table 8 (TxDOT, 2006). After determining the overall friction demand (low, moderate, or high), the designer is advised to alter the factors that influence the available friction (Table 8) to meet or exceed the friction demand determined previously. Nevertheless, it should be noted that a given roadway may not fall under a single friction demand category for every factor shown in the table. As an example, a roadway may fall under the low category based on rainfall, but under high category based on traffic and moderate category based on speed, etc. After considering all these factors, the designer is advised to determine the overall friction demand based on engineering judgment. As such, the TxDOT procedure for determining the friction demand remains subjective (Hall et. al., 2006).

Table 8. Texas DOT friction demand classification (TxDOT, 2006)

Demand for Friction	**Low**	**Moderate**	**High**
Rainfall, in/yr	$\leq$ 20	>20 $\leq$ 40	> 40
Traffic, ADT	$\leq$ 5000	>5000 $\leq$ 15,000	> 15,000
Speed, mi/hr	$\leq$ 35	>35 $\leq$ 60	> 60
Percent Trucks	$\leq$ 8	>8 $\leq$ 15	> 15
Vertical grade, %	$\leq$ 2	>2 $\leq$ 5	> 5
Horizontal curve, deg.	$\leq$ 3	>3 $\leq$ 7	> 7
Driveways per mi	$\leq$ 5	>5 $\leq$ 10	> 10
Intersecting Roadway ADT	$\leq$ 500	>500 $\leq$ 750	> 750
Available Friction	**Low**	**Moderate**	**High**
Cross slope, in/ft	$\leq$ 0.25	0.25 – 0.375	0.375 – 0.5
Design life, yr	> 7	>3 $\leq$ 7	$\leq$ 3
Proposed macro-texture	Fine	Medium	Coarse

The Maryland DOT differentiates friction demand on straight segments and curves. For straight segments, five demand categories are defined as shown in Table 9 (Speir et. al., 2009). The table also shows the priorities (high or low) for each demand category along with the desired level of friction as well as the investigatory and intervention levels of friction. It was also pointed out that the demand categories as well as the friction levels should be updated regularly due to the evolving nature of the roadway conditions (Speir et. al., 2009).

Table 9. Maryland DOT straight segment friction demand classification (after Speir et. al., 2009).

Site/Demand Category	Site Description	Threshold FN	Investigatory FN	Intervention FN
1 (High)	Approach railroad crossing, traffic lights, pedestrian crossing, stop and give way controlled intersections	55	50	45
2 (High)	Curves with radius < 820 ft (250 m), downhill gradient > 10 percent, and > 164 ft (50 m) highway on/off ramp	50	45	40
3 (High)	Approach to intersections, downhill gradient 5 to 10 percent	45	40	35
4 (Low)	Undivided highways without any other geometrical constraints which influences friction demand	40	35	30
5 (Low)	Divided highways without any other geometrical constraints which influences friction demand	35	30	25

International Practices

The U.K has implemented a friction management program since 1988. According to Hall et. al., (2006) and Larson et. al. (2008), the U.K program represents one of the most comprehensive friction management practices and is the closest agency implementation to what was recommended by the AASHTO Guide for Pavement Friction. At the establishment of their friction program in 1988, 13 friction demand categories were defined along with the corresponding investigatory levels. However, these categories were reduced to 11 categories in 2004 based on an updated analysis of crash risk and pavement friction. The updated categories and their investigatory levels are shown in Figure 9 in which the darker shading indicates the investigatory levels for normal conditions and the lighter shading indicates those levels for low risk areas (i.e., very low traffic) (Viner et. al., 2004).

Site category and definition		Investigatory level at 50km/h							
		0.30	0.35	0.40	0.45	0.50	0.55	0.60	0.65
A	Motorway class								
B	Dual carriageway non-event								
C	Single carriageway non-event								
Q	Approaches to and across minor and major junctions, approaches to roundabouts								
K	Approaches to pedestrian crossings and other high risk situations								
R	Roundabout								
G1	Gradient 5 to 10% longer than 50m								
G2	Gradient >10% longer than 50m								
S1	Bend radius <500m – dual carriageway								
S2	Bend radius <500m – single carriageway								

Figure 9. Current friction categories and investigatory levels in U.K. (Viner et. al., 2004)

It is also noted that the U.K. friction practice does not specify an intervention level of friction below which an immediate action is needed. This is because (1) the relationship between crash risk and pavement parameters (friction, texture, etc.) are highly variable and as such, (2) a low value of friction alone does not indicate that the roadway is dangerous (Viner et. al., 2005). Therefore, the U.K. has been setting their investigatory levels at a higher level, rather than defining an intervention level.

The Austroads friction management program implemented in Australia and New Zealand is fundamentally based on the U.K. friction model, but tailored to the Australian environment. The original set of investigatory levels were established in 1982 and revised in 1996. The current set of friction demand categories and the investigatory levels specified by Austroads are shown in Figure 10.

Site category	Site description	Investigatory levels of SFC_{50} 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 $\leq$ 250 m Gradients $\geq$ 5% and $\geq$ 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 SFC_{20} 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 $\leq$ 100 m	INVESTIGATION ADVISED						
7	Roundabouts							

Key to thresholds at or below which investigation is advised	
(dark shading)	All primary roads, and for secondary roads with more than 2500 vehicles per lane per day
(light shading)	Roads with less than 2500 vehicles per lane per day

Notes:
- The difference in sideways force coefficient values between wheelpaths (differential friction levels) should be less than 0.10 where the speed limit is over 60 km/h; or less than 0.20 where the speed limit is 60 km/h or less.
- Investigatory levels are based on the minimum of the four-point rolling average skid resistance for each 100 m section length.
- Investigatory levels for site categories 1 and 3 are based on the minimum of the four-point rolling average skid resistance for the section from 50 m before to 20 m past the feature, or for 50 m approaching a roundabout.

Figure 10. Current Austroads friction categories and investigatory levels (Neaylon, 2011)

REVIEW OF HIGHWAY SAFETY AND CRASH

Factors Affecting Highway Safety

As mentioned previously, adequate pavement friction and surface texture are key components of a safe roadway. However, friction and texture are not the only factors that affect the roadway safety. In addition to friction and texture, the cause of crashes can be related to other factors that are driver-related (e.g., distraction), vehicle-related (e.g., tires, brake system), pavement-related (e.g., structural and functional distresses, pavement marking issues), roadway-related (e.g., geometry, visibility), and weather-related (e.g., rainfall intensity, fog). Among these, the factors related to pavements and roadways are of interest in this report. There is a vast amount of literature that reports the statistical correlation between highway safety and the pavement/roadway features. In the following, a brief summary of the recent literature relevant to this topic is provided.

Li and Huang (2014) studied the effect of pavement condition on crash counts using the data from Texas. They concluded that in general, the crash counts reduced significantly on pavements that are in good condition. More specifically, the study indicated that the mean crash rate on roadways with severe surface distresses and/or rough riding characteristics (i.e., higher International Roughness Index, IRI) was more than twice the crash rate found on roadways that are smooth and free of surface distresses. The researchers also speculated that the poor pavement condition could affect the driver behavior and result in unexpected maneuvers that may cause crashes.

Tehrani et. al. (2017) also studied the effect of pavement condition on roadway crashes using data from Alberta, Canada. The factors studied include IRI, rut depth, traffic, horizontal & vertical alignment, and weather condition. Although the study concluded that rut depth is not correlated to number of crashes, it was concluded that the number of crashes is affected by the other factors studied. Furthermore, this study found that IRI showed the best correlation with crash rates.

A significant correlation between IRI and crash rate was also found by an Iowa study conducted by Bektas et. al. (2016). These researchers indicated that while pavement friction was correlated to crash counts, other distresses including rut depth and faulting did not have a significant effect on roadway safety. Another interesting finding from this study is that the pavement marking retroreflectivity is highly correlated to roadway crashes, especially on multilane roadways. More specifically, it was reported that the roadways with higher retroreflectivity of white and yellow edge lines showed a significant reduction in the number of crashes.

In addition to the number of crashes or crash rates, it was reported that pavement condition is correlated to the severity of the crashes (Lee et. al., 2015). This study by the University of Central Florida (UCF) researchers indicated that poor pavement condition increased the severity of single vehicle crashes on high-speed facilities and multiple vehicle crashes on all facilities. However, the study did not differentiate the significance of each type of pavement distress (crack, ride, and rut) monitored by FDOT. Instead, the study combined the three distress modes and used an overall pavement condition index for studying the correlation with crash severity.

Musey et. al. (2016) also used data from FDOT to study the effect of friction and horizontal curves on crash rates and severity. They concluded that due to the reduction in available friction with increasing degree of horizontal curvature, both the crash rate and severity increased with increasing curvature. The study also indicated that regardless of the horizontal curvature, the crash count reduced significantly when the friction number approached 60 or greater.

As seen from the above, the recent literature generally indicate that besides pavement friction and texture, there are other pavement or roadway related factors that affect roadway safety (or crash rates and severity). These factors include the traffic, horizontal & vertical curves, pavement surface distress, ride quality, etc. However, it is important to note that these factors do not always show significant correlation with the number of crashes or crash rates. As an example, the horizontal curvature may be a significant factor for high-speed facilities but its effect may not be as pronounced on low-speed facilities. As such, previous research studies have defined the friction demand categories based on the detailed site factors (e.g., Figure 9, Viner et. al., 2004, 2005) or simply based on their functional class – e.g., Interstate, primary, and secondary roads (de Leon Izzepi et al., 2016a, 2016b). Then, the relationship between crash statistics and the influencing factors have been determined for each friction demand category.

Relevant literature on the statistical relationship between crash and the influencing factors is provided subsequently.

Relationship between Crash, Friction, and Other Factors

Equation Forms for Predicting Crash

The equation forms found in the literature for relating the crash count (or crash rate) to the pavement and roadway related site factors are summarized in Table 10. It is seen that with the exception of the equation form proposed for intersection crashes by Larson et. al. (2008), most of the equations involve nonlinear functions such as the exponential or the logarithmic functions.

The exponential function is the form used to define the Safety Performance Function (SPF) recommended by FHWA and AASHTO (Srinivasan and Bauer, 2013). The purpose of the SPF is to identify roadway sites that may benefit from a safety treatment by estimating the number of crashes for a given roadway with a specified length. More specifically, the SPF in its most basic form is given as the following equation.

$$\begin{aligned} \mu &= L \cdot e^{\beta_0} \cdot AADT^{\beta_1} \\ &= L \cdot e^{\beta_0 + \beta_1 \cdot \ln(AADT)} \end{aligned} \tag{2}$$

where μ is the expected number of crashes, L is the segment length, $AADT$ is the annual average daily traffic, and β_0 and β_1 are regression coefficients.

Table 10. Equation forms relating crash, friction, and other variables.

Reference	Equation Form	Comments
Kuttesch (2004)	$CR = e^{2.54-0.01492 \cdot \text{FN40S} - 0.000026 \cdot AADT}$	CR = Crash Rate
Long et. al. (2014)	$CRR = 3.894 \cdot e^{-0.04605 \cdot \text{FN50S}} + 0.9205$ (for total crashes) $CRR = 5.023 \cdot e^{-0.05292 \cdot \text{FN50S}} + 0.9264$ (for wet crashes, FN50S < 39) $CRR = 3.894 \cdot e^{-0.04605 \cdot \text{FN50S}} + 0.9205$ (for wet crashes, FN50S ≥ 39)	CRR is the Crash Rate Ratio defined as: $CRR = \frac{P_{CR}}{P_{LM}}$ where P_{CR} and P_{LM} are cumulative percentage of total crashes and lane miles below a specific friction number, respectively.
De Leon Izzepi et. al. (2016a, 2016b) McCarthy et. al. (2016)	$\mu = e^{-0.35+1.25\ln(AADT)-1.19GN}$ (for Interstate Routes) $\mu = e^{-0.25+0.37\ln(AADT)-1.00GN+0.04/CV}$ (for Primary Routes) $\mu = e^{-0.55+0.75\ln(AADT)-0.56GN}$ (for Secondary Routes)	μ = mean crash count per 0.1 mile segment GN = Grip Number from Grip-Tester CV = Roadway Horizontal Curvature
Ivan et. al., (2010, 2014)	$\mu = e^{\beta_0 + \beta_1 \cdot \text{FN40R} + \beta_3 \log(AADT)}$	μ = mean crash count per 0.5 mile segment $\beta_1 - \beta_3$ = regression coefficients
Viner et. al. (2004)	$\mu = k \cdot Q^{\alpha} L^{\beta} e^{a_1 x_1 + a_2 x_2 + \cdots + a_i x_i}$	Q = Traffic L = Segment length α, β, a_i = regression parameters x_i = independent variables (friction, texture, etc.)
Musey et. al. (2016)	$\mu = -24.91\ln(\text{FN40R}) + 109.59$	μ = Mean crash count

Larson et. al. (2008)	$\mu = a \cdot \mathrm{FN20R} + b \cdot MTD$ for intersection pavement sections	μ = Total crash count

As seen from equation (2), the only mandated variable in the SPF is the traffic (*AADT*). However, FHWA Office of Safety further recommends that the above equation can be generalized to include additional site factors such as the lane width, shoulder width, horizontal curvature, and the presence of turn lanes, intersections, and traffic control (Srinivasan and Bauer, 2013). The generalized form of the equation, with these variables included, can be written as:

$$\mu = L \cdot e^{\beta_0 + \beta_1 \cdot \ln(AADT) + \sum \beta_i \cdot X_i} \tag{3}$$

where X_i is the additional site factors to be included and β_i is the corresponding regression coefficient. It is also noted that while FHWA's SPF document does not mention pavement friction and texture as potential site factors, these terms can easily be included in the generalized SPF. In fact, several equations shown in Table 10 are in the form of the generalized SPF shown in Equation (3) (e.g., Kuttesch, de Leon Izzepi, Ivan, and Viner equations). As an example, Figure 11 shows the U.K. crash model as a function of pavement friction and texture (Viner et. al., 2004).

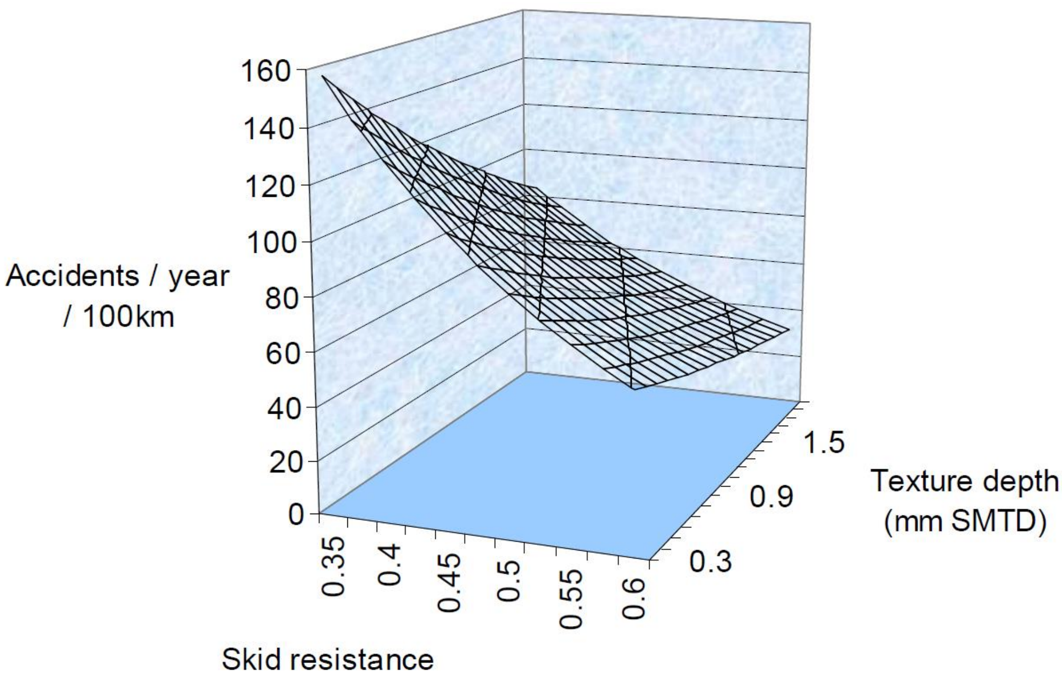


Figure 11. U.K. crash model for friction and texture (Viner et. al., 2004)

Statistical Approach for Predicting Crash

The SPF described above or the closed-form regression models shown in Table 10 do not allow for the parameters to vary across different observations. In other words, the effect of the explanatory variable (e.g., friction) on the frequency of crashes is constrained to be the same for all segments within the predefined friction demand category. However, because of the factors that influence crash but cannot be measured or are not measurable, the crash statistics typically show large variations from one roadway segment to another.

In order to address such variability in the crash counts, it is necessary to model the statistical distribution of the crash counts and the associated probabilities.

It was also pointed out that because crash counts are discrete, non-negative integers, application of ordinary least-squares or ordinary normal distribution should not be used for modelling the distribution of crash-frequency data (Lord and Mannering, 2010). Instead, FHWA's recommendation is to use the Poisson or Negative Binomial (NB) distribution for modelling the crash statistics. The Poisson model is given by the following equation.

$$P(y_i) = \frac{e^{-\mu_i} \mu_i^{y_i}}{y_i!} \tag{4}$$

where $P(y_i)$ is the probability of section i having y_i crashes per year and μ_i is the mean or expected number of crashes determined from the SPF shown in Equation (3). However, the drawback of the Poisson model is that the variance of the distribution is equal to its mean, and does not allow for modelling the over-dispersion of the data (variance being greater than the mean) which is frequently encountered in crash data (Lord and Mannering, 2010; Srinivasan and Bauer, 2013; Herbal et. al., 2010; Cameron and Trivedi, 2005).

Due to the above limitation of the Poisson model, FHWA recommends the use of the NB model for Highway Safety Improvement Program (HSIP) purposes (Herbel et. al., 2010). The NB model is derived by rewriting the SPF in Equation (3) as:

$$\lambda = \mu\upsilon = L \cdot e^{\beta_0 + \beta_1 \cdot \ln(AADT) + \sum \beta_i \cdot X_i} \cdot e^{\varepsilon} \tag{5}$$

where υ is the gamma-distributed, random error term with a mean of 1.0 and a variance of α. Given λ and υ, the NB model is written as the following (Cameron and Trivedi, 2005).

$$P(y_i) = \frac{\Gamma\left(\alpha^{-1} + y_i\right)}{\Gamma\left(\alpha^{-1}\right)\Gamma(1+y_i)} \left\{\frac{\alpha^{-1}}{\alpha^{-1}+\lambda}\right\}^{\alpha^{-1}} \left\{\frac{\lambda}{\alpha^{-1}+\lambda}\right\}^{y_i} \tag{6}$$

The mean of the above distribution is equal to λ as given by Equation (5) and the variance is equal to $\lambda(1+\alpha\lambda)$.

In addition to the NB model shown above, FHWA's HSIP manual recommends the use of the Empirical Bayes (EB) method for combining the observed crash counts with the predicted counts from the SPF to calculate the statistically expected crash count for a given section (Herbal et. al., 2010). The EB method is based on the assumption that crash counts from a given pavement section are not the only evidence of the safety of that pavement. Another evidence or clue that should be considered is the information given for other pavements with similar characteristics. Hauer et. al. (2002) provides simple examples behind the concept of EB method as the following:

“For example, consider Mr. Smith, a novice driver in Ontario who had no accidents during his first year of driving. Let it also be known that an average novice driver in Ontario has 0.08 accident/year. It would be absurd to claim that Smith is expected to have zero accidents/year (based on his record only). It would also be peculiar to estimate his safety to be 0.08 accident/year (by disregarding his accident record). A sensible estimate must be a mixture of the two clues. Similarly, to estimate the safety of a specific segment of, say, a rural two-lane road, one should use not only the accident counts for this segment, but also the knowledge of the typical accident frequency of such roads in the same jurisdiction.”

Mathematically, the EB method is written as the following:

$$EB_i = W_i \lambda_i + (1 - W_i) y_i \tag{7}$$

where EB_i is the crash count for section *i* estimated from the EB method and W_i is the weight factor given as:

$$W_i = \frac{1}{1 + \lambda_i \alpha} \tag{8}$$

The primary purpose of the EB method is to eliminate the Regression to Mean (RTM) bias and to improve the precision of the estimated crash counts. As an example to explain the RTM phenomenon which often causes erroneous conclusions in highway safety analysis, consider the crash counts shown in Figure 12. Given the random fluctuations in crash counts shown in this figure, FHWA’s HSIP manual illustrates the RTM bias as the following:

“(the figure) shows an example to demonstrate this concept. It shows the history of crashes at an intersection, which might have been identified as a high-hazard location in 2003 based upon the rise in crashes in 2002. Even though a treatment may have been introduced early in 2003, any difference between the frequencies of crashes in 2002 and those in 2003 and 2004 would, to some unknown degree, not be attributed to the treatment, but to the RTM phenomenon. The RTM phenomenon may cause the perceived effectiveness of a treatment to be overestimated. Thus, there would be a “threat to validity” of any conclusions drawn from a simple comparison of conditions before and after a change at a site.”

Essentially, the RTM bias is caused by not incorporating for the random fluctuation of the crash counts in the analysis. In order to eliminate the RTM bias, the EB method pulls the observed crash count from a given pavement towards the mean by combining the observed crash count with the predicted SPF predicted crash count, as shown in Figure 13. Therefore, the expected or corrected crash count based on the EB method is always between the observed value and the predicted value from the SPF.

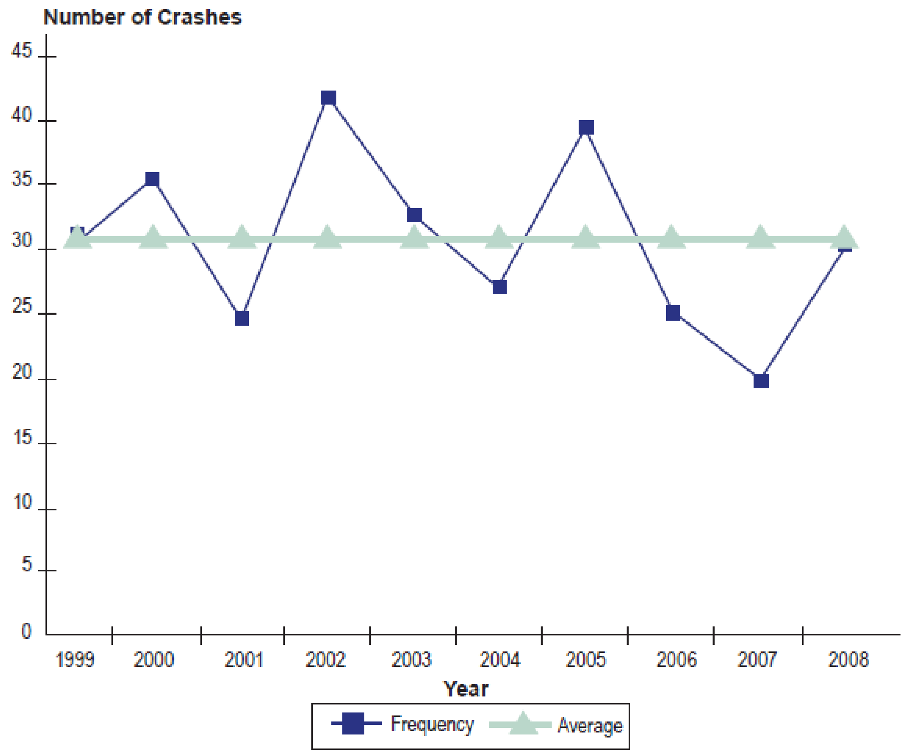


Figure 12. Description of Regression to Mean bias (Herbal et. al., 2010)

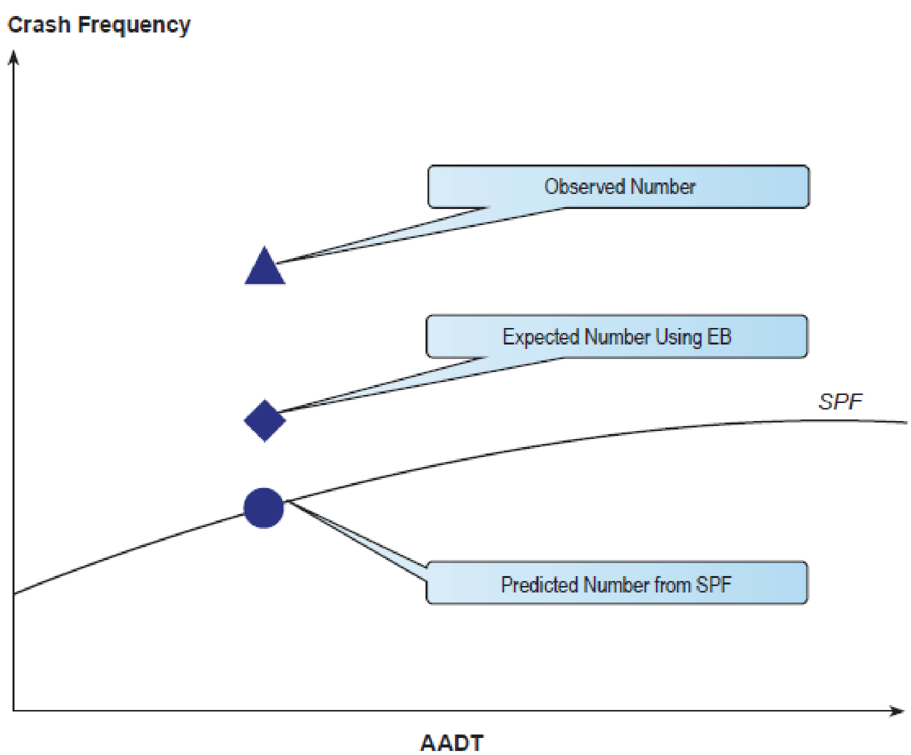


Figure 13. Illustration of Empirical Bayes method (Herbal et. al., 2010)

The application of the NB and EB methods have been illustrated in great detail by de Leon Izeppi et. al. (2016a, 2016b) and McCarthy et. al. (2016) as part of a pilot effort for incorporating the CFME measurements into roadway safety decision process. The researchers used the negative binomial SPF for modelling the crash versus friction relation relationship, and the EB method for predicting the crashes that occurred on a segment of I-81 in Virginia. Their results are as shown in Figure 14.

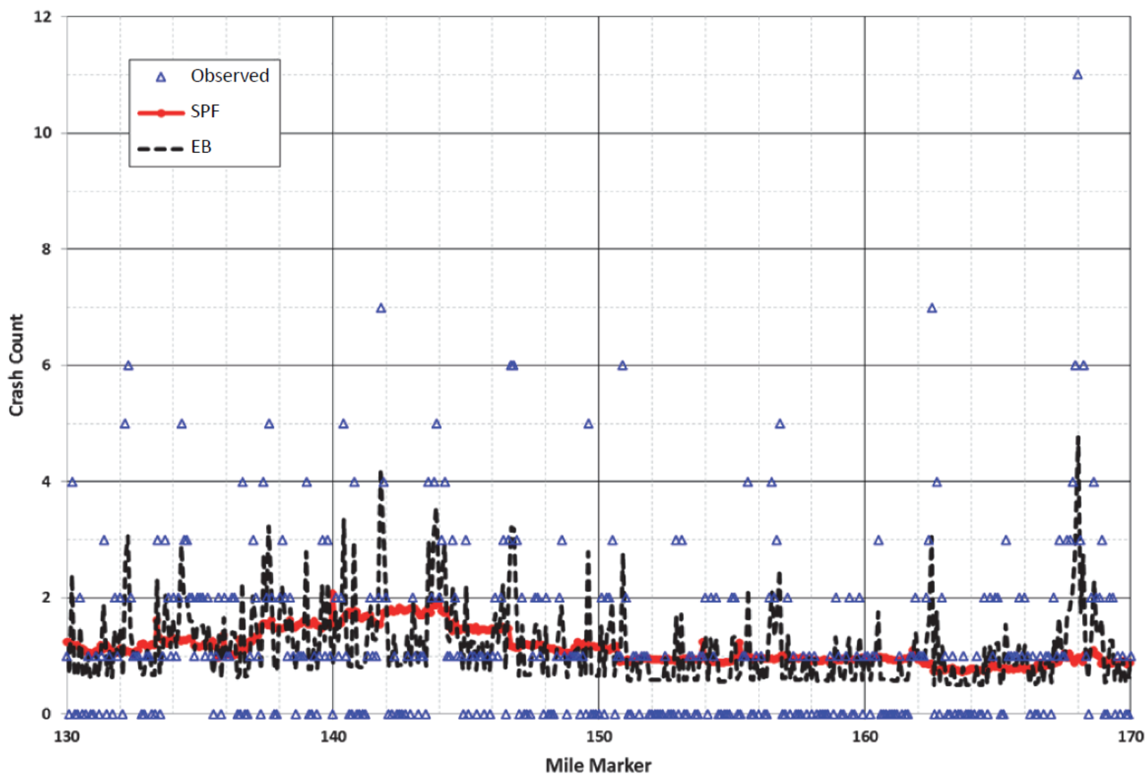


FIGURE 3 Observed crashes, rate (SPF), and predicted (EB) crashes on I-81, MM 130 to MM 170, Salem District.

Figure 14. Observed crash count from I-81 in Virginia along with SPF and EB predictions (de Leon Izeppi et. al., 2016)

SUMMARY

This chapter provided a summary of available literature on relationships between crash rates and pavement surface friction, as well as other factors that affect friction and crash rates. In addition, available literature describing other SHAs' friction measurement and management practices, friction demand setting, and friction number requirements was reviewed.

Although recommendations were made through the AASHTO Friction Guide for determining the friction thresholds, most SHAs have defined friction thresholds in an empirical manner. This is partly because the thresholds were set a few decades ago, but mostly because of the challenges associated with linking the pavement friction/texture information to the observed number of crashes.

Recent recommendations by FHWA and AASHTO provided additional guidance on the statistical modeling of the crash frequency data. As was shown in this literature review report, the guidance was not specifically targeted at linking the crash data to pavement friction or texture. Nonetheless, the guidance was made generic enough that additional factors can be added by SHAs.

GAP ANALYSIS

The previous chapter (i.e., literature review) of this report was focused on the relationship between crash rates and pavement surface friction, as well as other factors affecting friction and crash rates. In addition, other state highway agencies (SHAs) friction measurement, management, and friction demand setting practices, as well as the friction number requirements were reviewed and summarized in the literature review chapter.

The objectives of the current chapter are to (1) further review FDOT's current friction and safety related practice and (2) conduct an in-depth gap analysis to compare FDOT's practice to other SHAs' as well as to the recommendations provided in national and/or international guidelines such as the AASHTO Guide for Pavement Friction (Hall et al., 2006). More specifically, the gap analysis was conducted in the following areas of interest:

- Friction and texture data collection practice to include equipment, test protocols, and analysis.
- Friction management practice including friction demand setting, friction design and restoration, and benefit/cost analysis.

This chapter summarizes FDOT's current practice and documents the results of the gap analysis.

REVIEW OF FDOT PRACTICE

FDOT's Friction & Texture Data Collection and Management

FDOT's standard method of friction testing uses full-scale, fully automated locked wheel testers in accordance with ASTM E 274. The locked wheel tester consists of a full-sized pick-up truck and an instrumented two-wheel trailer with a wheel locking system. The tow vehicle supplies all the mechanical and electrical power required to perform testing as well as all support systems, including a control panel and a data acquisition system to collect and store information from the travelled surface. A distance-measuring instrument (DMI) and a global positioning system (GPS) antenna determine the position along the road. The locked wheel testers are equipped with a controlled water distribution system for wet pavement testing. The Friction Number (FN) is typically measured at 40 mph using an ASTM E 501 ribbed tire, and the resulting FN is designated as FN40R.

Currently, FDOT conducts network-level testing on a 2-year cycle for interstate highways, turnpike, and toll roads, and on a 3-year cycle for all other state highways. Normally, the locked wheel measurements are conducted in the left wheel path of the lane tested with a frequency of three lockups per mile. Friction measurements are classified according to the purpose of the testing:

- **Inventory** – Data are collected to monitor current friction characteristics of the state highway system as outlined in the Skid Accident Reduction Program (FHWA Technical Advisory T 5040.17).

- **Spot Hazard** – Locations where pavements may have an unusual number of accidents (wet) or potential problems.
- **Special Request** – Road sections requested for research, off-system, accidents, litigation, or safety related.
- **Overlay** – Roadways that have been resurfaced.
- **New Construction** – Roadways that have new, added, or reconstructed lanes with new friction surface.
- **Re-test** – Roadway sections that originally did not meet the initial overlay/new construction desired friction characteristics or values are retested within 1 year of the original test date.

All friction measurements are entered into the Skid Hazard Reporting (SHR) system. The system offers FDOT's District personnel with options for retrieving and monitoring friction results. Friction data obtained from new construction and/or overlay projects are also recorded in FDOT's internal texture database, which is the home for the texture data in terms of Mean Profile Depth (MPD) collected for newly constructed pavements using the high-speed 2D laser system mounted underneath the towing truck of the locked wheel tester. The texture database also stores the friction number as well as the mix design information (mix type, aggregate type and source, binder grade, etc.).

FDOT Friction Guidelines and Friction Course Policy

FDOT's current friction guidelines implemented as part of the highway safety improvement program as well as other SHA's friction thresholds, were summarized in the previous chapter (see Table 6). As shown in Table 6, FDOT specifies a desired FN40R of 35 for roadways with a posted speed limit greater than 45 mph and a desired FN40R of 30 for all other roadways. New or existing pavements with FN40R below these thresholds are subjected to a safety review, investigation, and/or crash monitoring.

Table 11 summarizes FDOT's current friction course policies for both asphalt and concrete surfaced pavements. For asphalt surfaced pavements, FDOT requires dense graded friction courses (DGFC) on all two-lane roadways, while open graded friction courses (OGFC) are required on multi-lane roadways with a design speed greater than 45 mph. For concrete surfaced roadways, FDOT specifies longitudinal grinding (LGD) for pavements and a combination of longitudinal grinding and transverse grooving (TGV) for bridge decks.

Table 11. FDOT's Friction Course Policy.

<table>
<tr><th rowspan="2">Design Speed (mph)</th><th colspan="2">Asphalt Surface</th><th colspan="2">Concrete Surface</th></tr>
<tr><th>Two-Lane</th><th>Multi-Lane</th><th>Pavement[1]</th><th>Bridge Deck[1]</th></tr>
<tr><td>Less than or Equal to 45</td><td rowspan="2">FC125 or FC95</td><td>FC125 or FC95</td><td rowspan="2">LGD</td><td rowspan="2">LGD + TGV</td></tr>
<tr><td>Greater than 45</td><td>FC5</td></tr>
</table>

Note 1: LGD = longitudinal grinding, TGV = transverse grooving.

FDOT Friction Restoration Practice

FDOT's crash data is housed and managed by the State Safety Office in Tallahassee. FDOT's internal database known as the Crash Analysis Reporting System (CARS) is used to access the data in the crash database and summarize the necessary statistics such as cost per crash, total number of crashes (wet and dry), crashes per million vehicle miles, etc. Using CARS, the District Safety Office identifies the locations with high wet weather crash rates within the roadway network. Although the analysis is typically conducted on an annual basis, the District Safety Engineers (DSEs) may conduct the wet weather crash analysis at any given time.

The first step in the safety analysis involves identification of high wet weather crash locations based on the last five years of crash data in CARS (McGovern et. al., 2011). The sections are identified as high crash locations when they have (1) a minimum of four wet weather crashes with 25 percent or more wet weather crashes or (2) 50 percent or more wet weather crashes during the five-year analysis period. The analysis is conducted on 0.3-mile segments (including intersections) moving at an increment of 0.1-mile. Based on the results of this analysis, the DSEs may submit a request to the State Materials Office (SMO) for additional friction tests (i.e., Spot Hazard friction testing) if deemed necessary. However, one drawback of the CARS system is that it takes FDOT approximately 2 years to have the CARS data available for a given year (i.e., the database is 2 years behind).

If the friction number is determined to be below the desired level (see Table 6) for locations identified from the crash analysis, the State's work program is reviewed by the DSEs to determine if the roadway is programmed for resurfacing. If the roadway is not programmed for resurfacing, the DSEs conduct a more detailed investigation to identify the factors contributing to the high wet weather crash rates, such as the crash reports and other roadway conditions (e.g., geometrics, surface condition, drainage, etc.). If pavement friction is identified as a contributing factor, the DSEs identify and recommend appropriate mitigation strategies which may include high-friction surface treatments (HFST), friction overlay with granite aggregates, and/or installation of warning signs, enhancing the roadway visibility and lane delineation as temporary solutions until a more permanent solution can be put in place.

OTHER STATE AGENCY PRACTICES

Friction & Texture Data Collection and Management

As mentioned in the previous chapter, the locked-wheel tester is the predominant equipment used by U.S. agencies for testing pavement friction. The testing is mostly conducted at a frequency of 1 to 5 lock-ups per mile, with only a few agencies (e.g., Illinois, New York, Pennsylvania, and Wisconsin) conducting 10 lock-ups per mile (Henry, 2000).

In addition to the above, Henry (2000) also reported that out of the 42 states responding to the survey, Florida was one of the nine states conducting friction testing for inventory, spot-hazard, new construction (including overlay), and accident investigation purposes (along with Arkansas, California, Louisiana, Maryland, Mississippi, New Jersey, North Carolina, and Oregon). Table 12 summarizes the number of states conducting friction tests for each purpose.

Table 12. SHA purposes of friction measurement (after Henry, 2000)

Purpose of Friction Measurement	Inventory	Spot-Hazard	New Construction	Accidents (Special Request)
Number of States (Out of 42)	27	19	20	25

Note: A given state may be collecting friction data for multiple purposes (e.g., Inventory and Accidents).

It was also reported that routine testing for pavement texture is rarely conducted by U.S. agencies. According to Henry (2000), only Louisiana, Mississippi, and Virginia were collecting pavement texture for inventory purposes, while Minnesota was the only state collecting the texture data on new construction projects. However, with the recent developments on the high-speed 3D laser systems for texture measurement and the on-going NCHRP 10-98 study on network level texture collection, it is anticipated that more agencies will be collecting the texture data for inventory and pavement management purposes.

Friction Guidelines and Friction Course Policy

A summary of other SHAs' intervention levels of friction were summarized in the previous chapter (see Table 7) which showed that the intervention levels from other SHAs are mostly between 30 and 35 (in terms of FN40). However, it is noted again that these friction thresholds were mostly determined empirically without following the AASHTO Friction Guide recommendations.

As previously shown in Table 11, FDOT specifies the use of OGFC or DGFC on asphalt surfaced pavements depending on the number of lanes and the design speed. According to a survey conducted by Stanard et. al. (2007), 17 states (or 37 percent of the 46 states that responded to the survey) are also using Porous Friction Courses (PFC) on a regular basis, mostly on interstate highways. Georgia places OGFCs on all roadways with speed limit greater than 55 mph.

On the other hand, it was reported that 21 states (46 percent) are not using PFC or OGFC. It is also noted that 8 out of the 21 states, namely Colorado, Connecticut, Maryland, Michigan, Minnesota, Ohio, Pennsylvania, and Virginia, have used PFC in the past, but have discontinued its use due to poor performance (likely due to snow and cold winter in these states). A quick summary of the survey results by Stanard et. al. (2007) is shown in Figure 15.

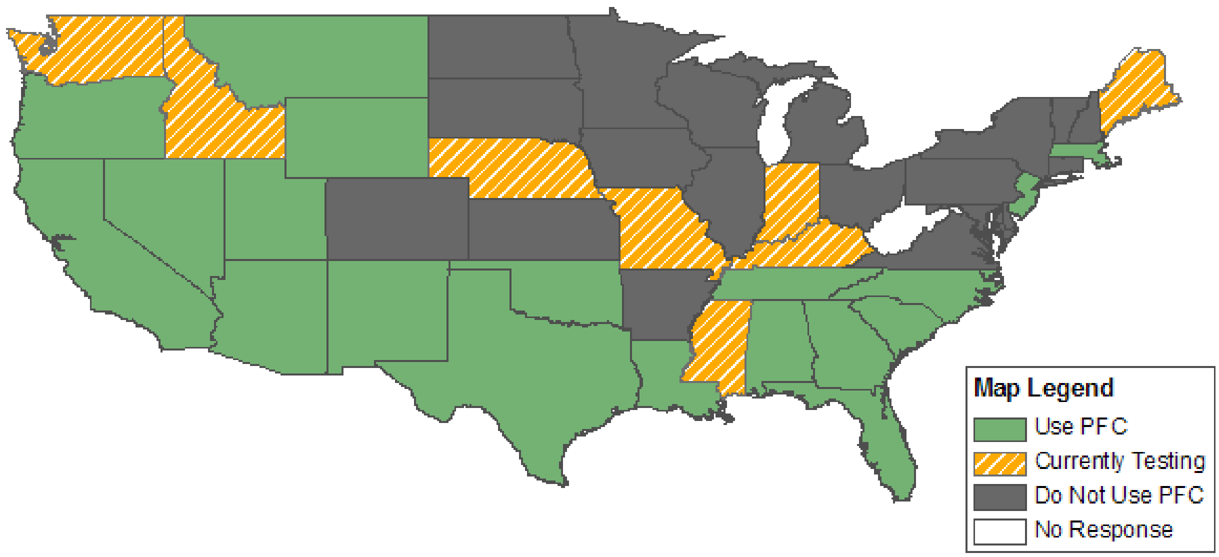


Figure 15. Survey Results on Porous Friction Course Use (Stanard et. al., 2007)

Table 13 summarizes the primary (required for high-speed facilities) and optional (allowed for low-speed facilities or experimental sections being evaluated for future use) texturing methods used by other SHAs for newly constructed rigid pavements (Hall and Smith, 2009). The table indicates that most states are primarily using turf or burlap drag followed by longitudinal or transverse tines with various patterns and dimensions. In addition, Hall and Smith (2009) also indicated that the Exposed Aggregate Concrete (EAC) is the predominant method for achieving surface friction in many European countries (e.g. Austria, Belgium, Netherlands, Sweden, and United Kingdom).

Table 13. Other States concrete texturing practices (after Hall and Smith, 2009)

State	Texturing Method	
	Primary	Optional
Alabama	Tran Tine (13 to 25 mm variable) w/ Burlap Drag	
California	Long Tine (19 mm) w/ Burlap Drag	Burlap Drag, Long Groove
Colorado	Long Tine (19 mm)	
Florida	Long Grind	Long Grind & Trans Groove (Bridge Decks)
Illinois	Tran Tine (19 mm) w/ Long Turf Drag	Tran Tine (17 to 54 mm variable) w/ Long Turf Drag
Indiana	Tran Tine (variable) w/ Long Turf Drag or Burlap Drag	
Iowa	Tran Tine (19 mm) w/ Long Turf Drag or Burlap Drag	Long Tine (19 mm), Tran Tine (9.5 to 41 mm)
Kansas	Long Tine (19 mm) w/ Long Turf Drag or Burlap Drag	
Michigan	Tran Tine (13 mm)	
Minnesota	Long Turf Drag	
Missouri	Any method (≥0.7 mm MTD)	Tran Tine (13 mm), Long Tine (13 mm), Long Grind
North Carolina	Tran Tine (13 to 19 mm variable) w/ Burlap Drag	
North Dakota	Tran Tine (13 to 71 mm variable) w/ Long Turf Drag	
Ohio	Long Tine (19 mm) w/ Burlap Drag	Tran Tine (10 to 45 mm variable)
Pennsylvania	Tran Tine (15 to 54 mm variable)	
Texas	Tran Tine (25 mm) w/ Long Turf Drag	
Wisconsin	Tran Tine (13 to 54 mm variable) w/ Long Turf Drag	

Friction Restoration Practice

Table 14 summarizes the current practice of four states for identifying locations prone to wet weather crashes and for restoring the friction of pavement surfaces (McGovern et. al., 2009). Similar to FDOT's practice, the sections identified as high wet weather crash zones typically undergo a more detailed site investigation which include additional friction testing and review of other contributing factors (e.g., roadway geometry, splash/spray, etc) prior to taking any action.

Table 14. Summary of other SHAs friction restoration practice (after McGovern et. al., 2009)

State	Identification of Wet Weather Crash Locations	Practice for Friction Restoration	Analysis Frequency
California	• Crash count significantly higher than Statewide average (95% confidence) • Minimum of 9, 6, or 3 wet weather crashes within 36-, 24-, or 12-month period, respectively	• Superelevation correction • OGFC overlay • Grooving • HFST	Annual
Florida	• Minimum of 4 wet weather crashes with 25 percent or more wet weather crashes • 50 percent or more wet weather crashes during the five-year analysis period	• Friction overlay • Warning signs • Florida is also using HFST in areas were friction-based crashes are a concern • Visibility improvement	Annual
Michigan	• FN40R less than 30 and crash count significantly higher than Statewide average	• Overlay (including ultra-thin overlay) • Mill and resurface • Microsurfacing • Surface seal • Chip seal • Diamond grinding	Annual
New York	• Minimum of 6 (rural) and 10 (urban) wet weather crashes during a 2-year period • Areas where wet weather crash count exceeds 35% of total crash count	• Resurfacing • Microsurfacing	Annual
Virginia	• Three or more crashes in the previous year • Area exposed to wet weather condition at least 20% of the time	• Microsurfacing • Chip seal • Overlay • Diamond grinding • Grooving	Annual

RECOMMENDED PRACTICE FOR PAVEMENT FRICTION AND TEXTURE

The friction program, including the practice for friction target setting, recommended by the AASHTO Friction Guide was reviewed under the previous chapter (see Figure 8). The information summarized herein builds on the AASHTO recommended friction program and is more focused on the recommended practice for selecting the adequate friction courses (for AC pavements) or texturing techniques (for PCC pavements).

FHWA Technical Advisory (T 5040.36) on Surface Texture for Asphalt and Concrete Pavements recommends several HMA mix types and texturing techniques that can provide adequate pavement friction and texture for newly constructed pavements (and overlays) as well as for friction restoration of existing pavements (FHWA, 2005). These mix types or techniques are:

- For Asphalt Pavements:
  - Dense-Graded (DG) Hot Mix Asphalt (HMA) including DGFC
  - Open-Graded Friction Course (OGFC)
  - Stone Matrix Asphalt (SMA)
- For Rigid Pavements:
  - Tining (Longitudinal or Transverse)
  - Diamond Grinding or Grooving
  - Burlap or Turf Drag
  - Exposed Aggregate Concrete (EAC)
  - Ultra-Thin Epoxy Laminates

Table 15 and Table 16 summarize the strengths and weaknesses of the techniques shown above for asphalt and concrete pavements, respectively (NAPA, 2001; Hall and Smith, 2009).

Table 15. Strengths and weaknesses of HMA mix types (NAPA, 2001)

Mix Type	Strengths	Weaknesses
Dense-Graded (DG) HMA	• Satisfactory for all HMA layers (Structural, Friction, Leveling, & Patching) • Adequate friction and noise	• Splash/Spary • Higher hydroplaning potential than OGFC
Open-Graded Friction Course (OGFC)	• Reduced splash/spray • Reduced hydroplaning potential • Adequate friction and noise	• More expensive than DGFC • Few agencies report reduced life as compared to DGFC • Special winter maintenance required (for freezing climates)
Stone-Matrix Asphalt (SMA)	• Improved rutting resistance	• Very expensive

Table 16. Strengths and weaknesses of PCC texturing methods (Hall and Smith, 2009)

Texture Direction	Method	Strengths	Weaknesses
Transverse	Transverse Tine (0.75 in. [19 mm] or 0.5 in. [12.5 mm] uniform tine spacing)	• Durable high friction (with good aggregates) • Water drains in channels (less splash/spray) • Automated or manual construction	• Very high noise and tonal whine • Variable depending on weather and operator • Possible less friction on horizontal curves than longitudinal textures
	Transverse Tine (Variably spaced)	• Durable high friction (with good aggregates) • Water drains in channels (less splash/spray) • Automated or manual construction • No tonal whine if properly designed & constructed	• High noise • Variable depending on weather and operator • Possible less friction on horizontal curves than longitudinal textures
	Transverse Tine (Skewed and variably spaced)	• Durable high friction (with good aggregates) • Water drains in channels (less splash/spray) • No tonal whine if properly designed & constructed	• High noise • Additional effort required to construct
	Transverse Groove	• Provides retrofitted macro-texture to old roads • Water drains in channels • Minimal traffic interruption or worker exposure	• Slow and expensive operation • Requires equipment entry into adjacent lanes • Possible less friction on horizontal curves than longitudinal textures
	Transverse Drag	• Small positive subsurface water drainage flow	• Slow and expensive operation
Longitudinal	Longitudinal Tine	• Higher friction, lower noise, and no tonal whine • Possible greater stability on curves • Automated construction	• No positive surface drainage channels (more splash/spray)
	Longitudinal Plastic Brush	• Automated or manual application • Attractive, consistent appearance	• Generally low macro-texture • Surface wears quickly under heavy traffic

Texture Direction	Method	Strengths	Weaknesses
		• Good noise properties	
	Longitudinal Burlap Drag	• Automated, simple application • Attractive, consistent appearance • Good noise properties	• Only applies to moderate macro-texture • Moderate initial friction • Surface wears quickly under heavy traffic
	Longitudinal Turf Drag	• Lower noise, higher friction • Simple application • Early cure application for greater strength • Attractive, consistent appearance	• Long-term friction not well defined • Aggregate and mortar strength are critical • Difficult to achieve under high wind and extreme temperatures
	Longitudinal Groove	• Provide retrofitted macro-texture to old roads • Minimal traffic interruption or worker exposure	• No positive surface drainage channels (more splash/spray) • Does not increase micro-texture
	Longitudinal Grind	• Provide retrofitted macro-texture to old roads • Improves friction and noise • Low worker exposure • Increased smoothness	• Friction decreases rapidly on polish susceptible coarse aggregate with heavy traffic • No positive surface drainage channels (more splash/spray)
Other	Exposed Aggregate	• Some with good noise and friction properties • Long-term noise relatively stable • Allows use of recycled aggregates and two-layer systems	• Special equipment and methods are required • High variability in noise properties • Contractor experience critical • Additional time required for setting and brushing • Air void loss could lead to durability problems
	Shotblasting	• Provide retrofitted macro-texture to old roads • Can increase macro-texture • Minimal traffic interruption or worker exposure	• Limited improvement in noise properties • Long-term performance depends on aggregate properties • Noise level increase if aggregate is large

Texture Direction	Method	Strengths	Weaknesses
			• Does not remove whine from transverse tines
	Porous PCC	• Very good noise properties • High friction • Low splash/spray	• Mostly experimental designs • Noise reduction reduces with void clogging • Vacuuming debris needed
	Ultra-thin epoxied laminates	• Little noise improvement over ground PCC • Good friction	• Extremely expensive
	Ultra-thin bonded wearing course	• Good noise, high friction, low splash/spray • Fast application • Improved smoothness	• Clearance slightly decreased

After a thorough review of the literature and evaluation of such variables as performance characteristics, range of initial texture, friction, noise, cost and constructability, Hall and Smith (2009) developed a benefit rankings table for rigid pavement texturing techniques as shown in Table 17.

Table 17. Rankings for PCC texturing methods (Hall and Smith, 2009)

Method	Friction	Noise	Cost	Constructability
Transverse Tine (0.75 in.)	1	8	1	2
Transverse Tine (0.5 in.)	1	6	1	2
Transverse Tine (Variable Spacing)	1	7	1	2
Transverse Groove	1	7	4	3
Transverse Drag	2	6	-	2
Longitudinal Tine	1	4	1	1
Longitudinal Groove	1	5	3	3
Longitudinal Grind	1	3	3	3
Longitudinal Burlap Drag	4	3	1	1
Longitudinal Turf Drag	2	3	1	1
Longitudinal Plastic Brush	3	3	1	1
Exposed Aggregate	2	3	3	4
Shotblasting	1	7	2	3
Porous PCC	1	1	5	4
Untra-Thin Epoxied Laminate	1	2	6	3
Ultra-Thin Bonded Wearing Course	2	2	3	3

Note: Lower number indicates better or higher ranking.

It should be noted that the rankings shown in Table 17 were determined from subjective assessment of the available information, and it is unlikely that any one of these texturing techniques will be the best choice for all conditions. In other words, the specific demands for levels of friction, texture, noise, cost, and constructability should be considered in determining the adequate texturing technique (or friction courses for asphalt pavements) for a particular project. Low-speed rural or industrial projects in a dry climate with no curves and intersections will demand less noise reduction and less friction than an urban, high-speed throughway that includes several curves and intersections and bisects a residential community.

In addition to the traditional friction courses or texturing methods mentioned above, it is recommended that newer technologies be considered for improving pavement surface friction. These technologies include the HFST and the Next Generation Concrete Surface (NGCS). The HFST, specified in FDOT’s Specification No. 333, is a treatment intended to restore and maintain pavement friction to reduce crashes. It consists of a thin layer of high-quality polish-resistant aggregate (typically calcined bauxite) bonded to the pavement surface using polymer resin binder (typically epoxy-resin, polyester-resin, or polyurethane-resin). FDOT published a HFST guideline based on their experience (Holzschuher, 2017b). It was reported that while the installation cost of the HFST is higher than the traditional friction courses, the average 5-year benefit to cost ratio of the HFST was found to be 24.5 on tight curves.

The NGCS is a new texturing technique that is currently being evaluated by several agencies in the U.S. As shown in Figure 16, fifteen states are currently evaluating the NGCS for future use (Scofield, 2017). The precursor of the NGCS was developed using the Tire-Pavement Test Apparatus (TPTA) at Purdue University (Dare et. al., 2009). The hypothesis of the Purdue study was that the positive, highly nonhomogeneous macrotexture provided by the conventional diamond ground surface and aggrevated by traffic wear/tear, is responsible for the increased level of tire-pavement noise. To evaluate this hyphothesis, the study conceived a surface with no positive texture; a surface that is first ground smooth followed by an additional texture imparted by grooving. Such a “manufactured” surface providing downward or negative texture later became what is currently know as the NGCS. Evaluation results from the NGCS test sections throughout the nation indicated that the new texture showed a reduction in tire-pavement noise (approximately 3 dB) and a stable pavement friction when compared to the traditional diamond ground concrete surface (Scofield, 2017). California Department of Transportation carried out a pilot study in which the performance of NGCS was compared to the conventional rigid pavement texture in terms of pavement noise, friction, and smoothness (Guada et. al., 2012). Although this pilot study concluded that the NGCS is effective in improving the pavement noise and smoothness, no conclusion was drawn for pavement friction due to the lack of friction measurements. FDOT is also considering to construct a NGCS test section in their full-scale rigid pavement test sections to be constructed in the near future.

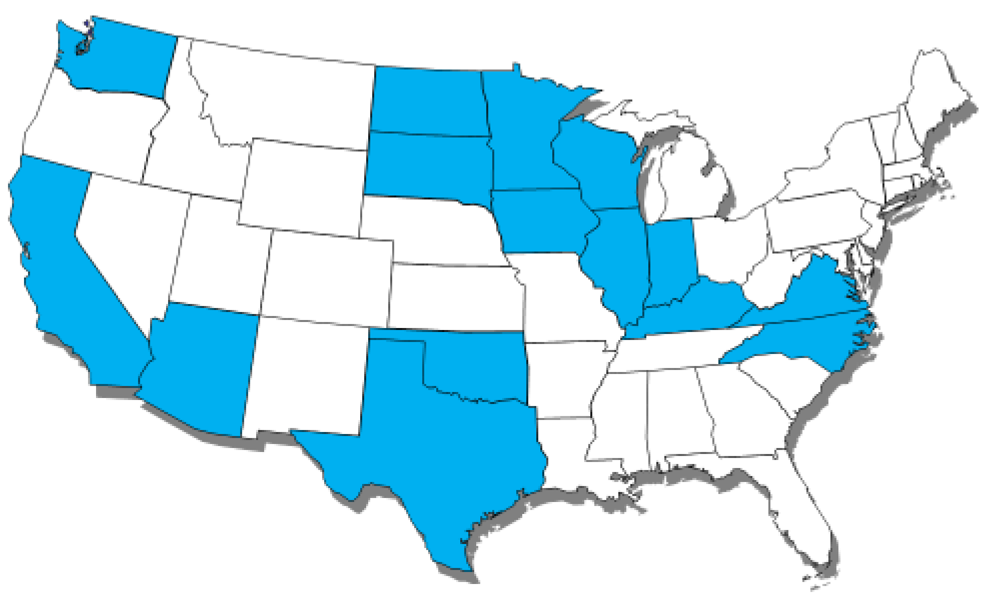


Figure 16. States with NGCS Construction (shown in Blue) (After Scofield, 2017)

The recommended practice for selecting the friction course type (for flexible pavements) or the texturing technique (for rigid pavements) for a particular highway project is to employ a logical, rational process which involves: (1) gathering and reviewing available critical information about the project, (2) identifying potential constraints/limitations (both internally and externally) in terms of available resources/technologies and performance/ cost expectations, (3) considering alternative feasible solutions, and (4) determining the most economical and practical alternative (Hall and Smith, 2009).

In accordance with the above, Figure 17 shows the recommended process for determining the adequate friction courses or surface texturing options at the project level (Hall et. al., 2006; Hall and Smith, 2009). This process first involves gathering of the necessary information about the project to establish target levels for friction, noise, and other surface characteristics (Step 1). The target or the desired level of friction is then determined based on the procedures recommended by the AASHTO Friction Guide (Step 2). Based on the identified target friction level, the need for texture and noise preferences (or regulations) are reviewed to identify the feasible friction course or texturing options along with the available information regarding the aggregate types, contractor & agency experience as well as agency policies (Steps 3 and 4). Then, the final friction course should be selected by considering other surface characteristics such as splash/spray potential and the cost of each friction course (Step 5). This five-step process can be applied to both new construction/reconstruction projects and rehabilitation (friction restoration) projects. Additional information regarding each step of the process is provided below.

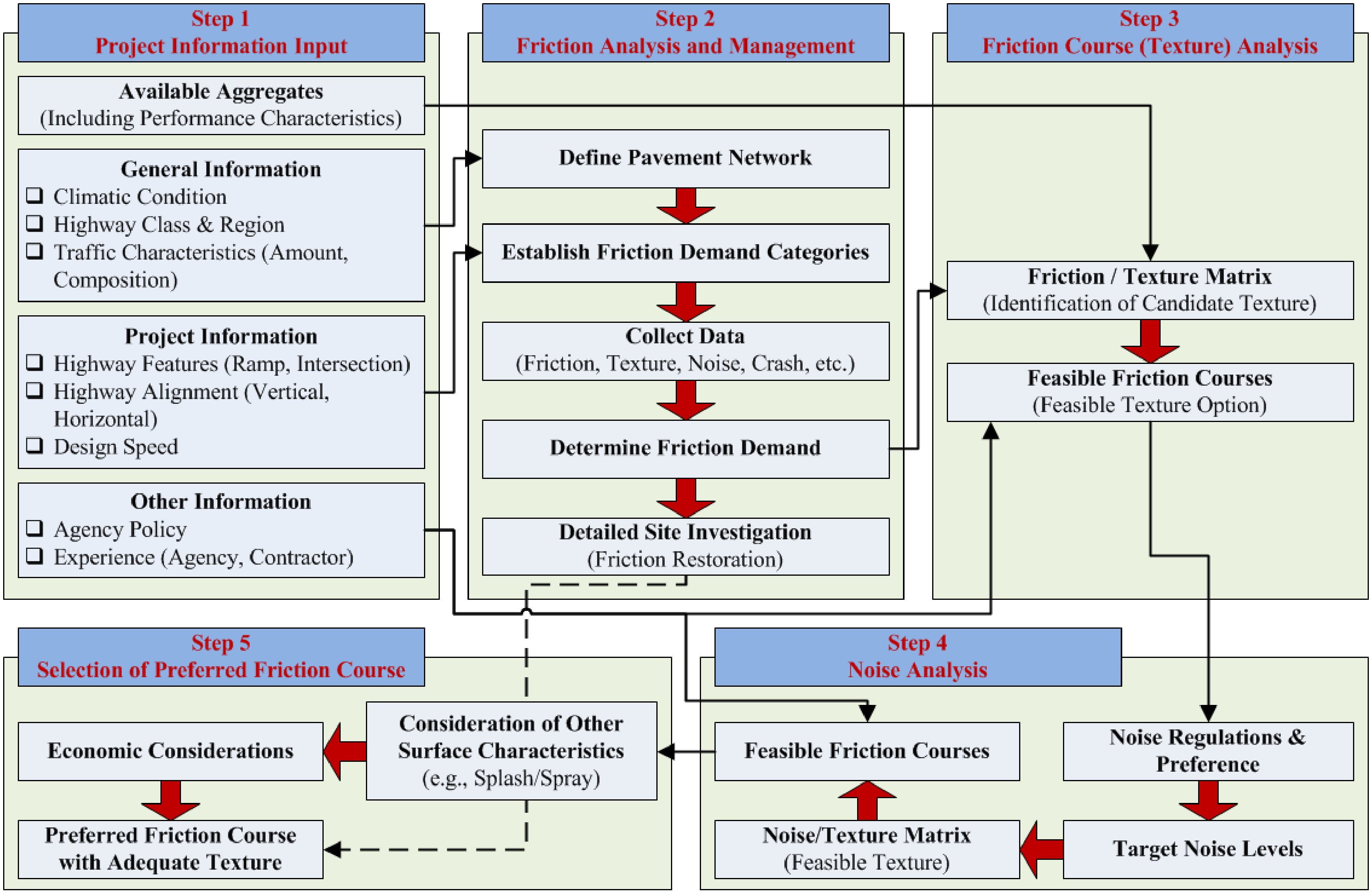


Figure 17. Recommended practice for selecting adequate friction course or texturing technique (after Hall et. al., 2006; Hall and Smith, 2009)

Step 1—Project Information Gathering

For each roadway project, information pertaining to the needs and expectations of friction, noise, and other related surface characteristics must first be gathered. Such information includes the following (Hall and Smith, 2009; FHWA, 2005).

- Climatic Conditions—A higher threshold level of friction (and thus requiring greater amounts of texture) may be necessary for locations with increased probability of wet-weather conditions (Hall and Smith, 2009; FHWA, 2005), especially if only polish-susceptible aggregates are available.

- Highway Alignment—Increased friction demand associated with horizontal and vertical curves is often addressed through increases in the horizontal radius of curvature, increases in super-elevation, and/or reductions in longitudinal grades. However, the alignments for some projects (particularly, those in which the existing alignment will be kept) may preclude taking these measures. In lieu of posting reduced speed limit signs, specifying a pavement surface with increased texture depth may be a viable solution.

- Highway Features/Environment—Highway geometric features and environment influence traffic flow and thus friction. Traffic flow is defined largely by the level of interacting traffic situations (e.g., entrance/exit ramps, access drives, unsigned/unsignalized intersections), the presence of controlled (signed/signalized) intersections, the presence of specially designated lanes (e.g., separate turn lanes at intersections, center left-turn lanes, through versus traffic lanes), the presence and type of median barriers, and the setting (urban versus rural) of the roadway facility (Hall et. al., 2006).

- Design Speed—The design traffic speed influences both friction and noise. As speed increases, the level of friction decreases, reaching a minimum at approximately 60 mph (FHWA, 2005).

- Design Traffic Characteristics—Both traffic volume and composition affect friction as follows: The higher the traffic volume, the greater the number of driving maneuvers (per segment of highway), which increases the risk of accidents, especially in high-speed areas (Hall and Smith, 2009). Pavements with higher traffic volumes may require greater amounts of texture to provide a higher level of friction (FHWA, 2005).

Step 2—Friction Management and Analysis

The target friction level should be determined in accordance with the procedure outlined in the AASHTO Guide for Friction Management program which includes five steps shown in Figure 17. Additional details regarding this process were provided in Figure 8 and Table 5.

Step 3—Feasible Textures Based on Friction Requirements

After determining the desired (initial) level of friction and reviewing all relevant project information, an assessment should be made to determine the type of friction courses (or textures) that can provide adequate friction over the life of the pavement. The factors that need to be considered include not only the initial values of friction and texture (micro- and macro-) but also their degradation over time due to environmental and traffic wear (Figure 18).

(a)

(b)

Figure 18. (a) New and (b) old longitudinally-ground surfaces (after Holzschuher, 2017a)

Pavement texture is not only important for adequate pavement friction but also for the vehicle hydroplaning potential during wet weather conditions. FDOT currently utilizes a tool for predicting the hydroplaning speed of different textures for design purposes.

It is also noted that measuring the rigid pavement texture using the high-speed 2D laser system (see Table 4) is a known challenge, due to the directional characteristics of rigid pavement textures. In general, no specific relationship was found between the texture values measured by the high-speed 2D laser and the site specific equipment such as the Circular Track Meter (Hall and Smith, 2009; Holzschuher, 2017a). An example comparison of the MPD measured by these systems are shown in Figure 19 (Holzschuher, 2017a). While both devices are in good agreement for the asphalt surfaces whose texture does not show any directional dependencies, the correlation is poor for rigid pavement with longitudinal texture.

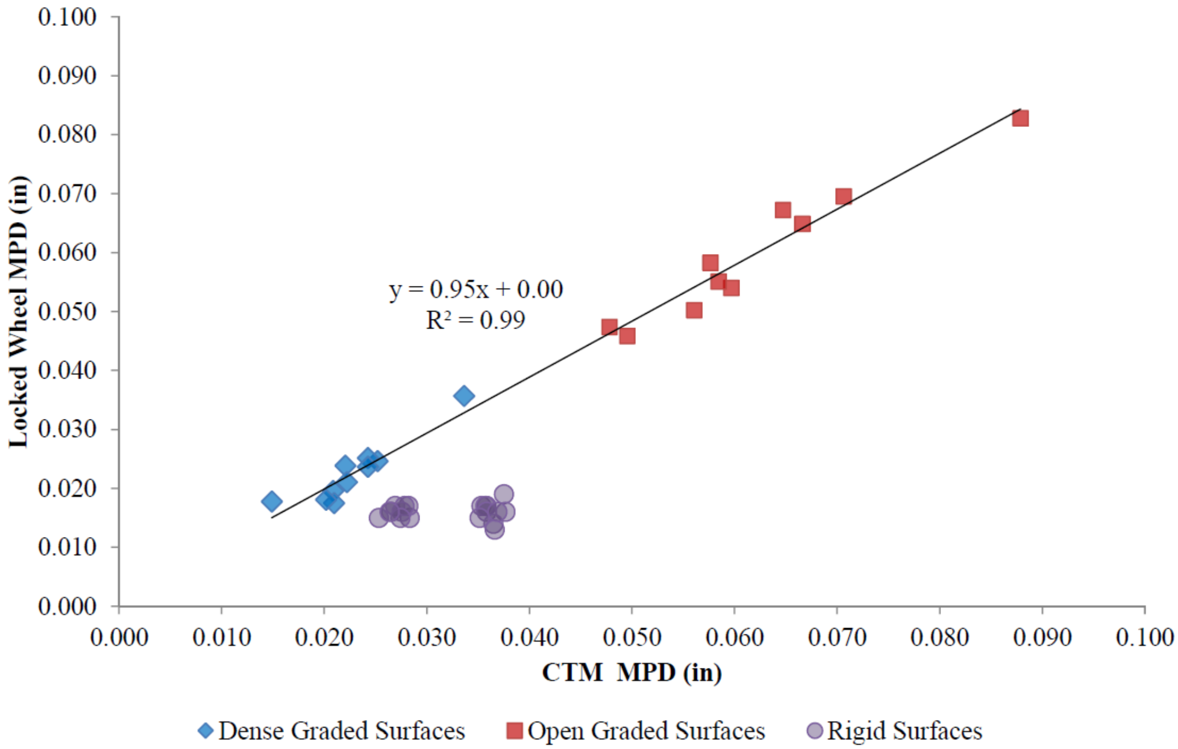


Figure 19. Texture measurements from high-speed (2D) vs. site specific devices (Holzschuher, 2017a)

Step 4—Feasible Textures Based on Noise Requirements and Preferences

There is no nationally recognized requirement for the maximum level of noise (either at the source or at a point on the wayside) that can be generated by a highway pavement. However, if an agency is capable of collecting the noise data at a network level (FDOT has the capability of collecting the noise data), the same process used in the previous step (for pavement texture) can be used to incorporate the short-term and long-term pavement noise characteristics in determining the adequate friction course and texture. There is no trade-off between friction and noise – adequate friction must be achieved but some surfaces and texturing techniques offer lower noise options.

Step 5—Selection of the Preferred Texturing Alternative

The last step in the texture selection process involves evaluating the adequacy of feasible textures with consideration of other important surface characteristics, such as splash/spray, fuel consumption and rolling resistance, and cost-effectiveness (Hall and Smith, 2009; FHWA, 2005).

The final step in selecting feasible friction course or texture involves costs— both the initial cost of constructing the texture and its long-term or life-cycle cost. The Life-Cycle cost analysis (LCCA) or the Benefit-Cost analysis (BCA) which include the estimated reduction in crashes may allow an agency to determine the most beneficial friction course or texture for a given pavement. As mentioned, FDOT reports that the average 5-year benefit to cost ratio of the HFST was approximately 24.5 on tight curves. It is recommended that such analysis (LCCA or BCA) be conducted on other surfaces as well.

SHORTCOMINGS OF FDOT PRACTICE (GAP ANALYSIS)

In the last two chapters of this report, FDOT's practices on friction (and texture) data collection, management, and friction course polices were reviewed. In order to determine any shortcomings of FDOT's current practice, other SHA practices as well as those recommended by FHWA, AASHTO, and NCHRP have been reviewed. The results of the above review are summarized in the following paragraphs.

No significant gap was identified when FDOT's practices were compared to those of other SHAs in the U.S. FDOT's practices regarding friction/texture data collection and management, friction course policy, and friction restoration are similar to, if not more advanced than, most other U.S. agencies. Nonetheless, when FDOT's practices were compared to the recommended AASHTO practice or some international agencies (e.g., United Kingdom and New Zealand), the following gaps were identified:

- Locked-wheel testing for friction measurement only provide the results in a "discrete" or "intermittent" manner. While the locked-wheel friction testing is the predominant method used in the U.S., most of the European countries are using the Continuous Friction Measurement Equipment (CFME) for network level friction testing. As mentioned previously, a joint effort by FHWA and six State DOTs have completed a pilot program for implementing the CFMEs for roadway friction testing in the U.S. It was reported that the granularity provided with a CFME accommodates coupling of crash data with pavement friction data, permitting for improved crash rate estimates and an ability to detect and mitigate negative conditions that might contribute to higher crash risks. The intermittent nature of the locked-wheel system does not allow for effective testing in tight curves and may fail to identify highly localized friction issues (de Leon Izzepi et. al., 2016)

- FDOT is collecting the pavement texture data on new construction / overlay projects using the high-speed 2D laser system. However, the limitations of the 2D laser system may not allow for collecting reliable texture data on rigid pavements, especially those having longitudinal textures (e.g., Longitudinal grinding). Furthermore, long-term texture data (i.e., texture degradation) has not been available in the past as the texture data was not collected for inventory purposes. FDOT is currently looking into collecting network-level texture data along with the friction data using line laser and 3D laser systems.

- Similar to the other SHAs in the U.S., FDOT's friction requirements (Table 6) were determined in an empirical manner. More specifically:

    - The design speed is the only criterion for categorizing the friction requirement. The friction demand of roadways with a speed limit of 25 mph or less may not be as high as the friction demand of roadways having a speed limit of 45 mph.

- The friction guideline does not consider other factors that are important in establishing the friction demand, such as the amount of traffic, crash counts, degradation of friction & texture over time, roadway grade & cross-slope, and pavement surface condition.

- Recently, FDOT studied the effect of testing speed on the locked wheel test results and established relationships between FN values measured at 30 mph, 40 mph, and 50 mph (Choubane et al., 2012). However, these relationships only address the speed dependency of the locked wheel testing and not the friction guidelines.

- FDOT's friction guideline does not account for the precision of locked wheel testers. The repeatability and reproducibility of FN40R values obtained using the locked wheel testers, as determined by FDOT, is approximately 4.0 (Choubane et al., 2006). Thus, it is possible that FDOT may obtain slightly different FN40R values from consecutive locked wheel testing.

Implementation of the objective friction target setting process, as recommended by the AASHTO friction guide is recommended to overcome the weaknesses shown above.

- The FDOT friction course policy does not provide a variety of options for the asphalt friction courses and rigid pavement texturing.

  - For example, only DGFCs are permitted on all two-lane roads with a design speed of up to 60 mph. In addition, transverse grooving is only applied on bridge decks.

  - The friction course policy does not provide clear guidance on some specific areas, such as ramps and other areas that are curved with considerable cross-slope. Literature indicates that these areas tend to lose friction at a faster rate (Hall et al., 2006).

With an objective friction / texture target setting process, a variety of friction courses (or textures) with different aggregates and/or recycled materials may be allowed for different roadways depending on the region, traffic, speed limit, and other confounding factors (e.g., splash/spray) along with the long-term values of friction and texture (based on their degradation characteristics).

- FDOT's practice on identification and friction restoration of high wet weather crash locations are focused on spot improvements. In other words, areas with high crashes are evaluated for friction and other contributing factors on a spot by spot basis.

  - A better approach would be to compare the crash characteristics of a given location to other locations with similar friction, geometry, traffic, etc. by means of the Safety Performance Function (SPF) as recommended by FHWA (Herbel et. al., 2010). This approach allows for a systematic analysis of the crash and safety characteristics of the entire roadway network. FDOT currently has the capability of surveying the roadway grade and cross-slope at a network level with a Multi-Purpose Survey Vehicle (MPSV). It is recommended that such valuable information be used for developing the SPF.

- The above systematic approach requires that a sound relationship be established between crash counts and the factors influencing crashes (friction, texture, traffic, roadway geometry, etc.). However, such a relationship has not been developed using FDOT's data.

SUMMARY

Following the literature review conducted in the previous chapter, FDOT's current practice and other SHA practices as well as those recommended by FHWA, AASHTO, and NCHRP have been reviewed further. Based on the review of the various practices, a gap analysis was conducted to determine any shortcomings of FDOT's current practice.

Although no significant shortcomings were found when FDOT's practices were compared to those of other SHAs in the U.S, several gaps were identified when FDOT's practices were compared to the recommended practice or to some international agencies (e.g., United Kingdom and New Zealand).

The most significant gap identified is the lack of a procedure for determining the desired level of friction (or friction demand) in an objective manner based on the crash counts and other factors that affect crashes. Once such a procedure is established based on a sound statistical approach, FDOT's practice on friction management, friction courses, and safety analysis may be further enhanced in accordance with the recommended AASHTO practice.

STATISTICAL MODEL DEVELOPMENT

In this chapter, a statistical relationship has been developed for Florida's crash rates which incorporated the pavement friction and texture characteristics as primary inputs. In addition, other pavement-related data that were available in FDOT's various databases were included in the statistical model development. Using the statistical model, further analysis has been conducted for determining the recommended levels of friction.

This chapter details the development of the above mentioned statistical model and presents the model results for FDOT's future use.

CONSTRUCTION OF INTEGRATED DATABASE

It was already mentioned that although adequate pavement friction and surface texture are key components of a safe roadway, they are not the only factors affecting the roadway safety. In addition to friction and texture, crashes can be caused by other factors that are related to the driver, vehicle, pavement, roadway, weather, etc. Among the many factors contributing to traffic accidents, this study is focused on those related to pavements and roadways. As such, a number of FDOT's available databases containing the relevant pavement or roadway data attributes have been provided to the research team for consideration. These databases and their data attributes are described herein followed by the development of an integrated database.

Sources of Data

As mentioned, one of the primary objectives of this study was to develop a statistical relationship between traffic accident counts, pavement surface characteristics (i.e., friction and texture), and other pavement (or roadway) related factors. However, the necessary data attributes (friction, texture, crash counts, and others) were not readily available in a single database. Instead, the information was spread out in a number of databases that needed to be integrated into a single database. The databases that were used for this study are described in the following paragraphs.

Skid Hazard Reporting (SHR) Database

The SHR database is the primary database for FDOT's pavement friction data. It is hosted in FDOT's Bluezone Mainframe server for storing all of FDOT's friction test results. The database stores friction data that date back to 1977. The historical friction data from the SHR were exported into a CSV file format for this study. The SHR data was the primary source for studying the trend, distribution, and degradation of FDOT's friction numbers.

FDOT's standard method of friction testing involves full-scale, fully automated locked wheel testers in accordance with ASTM E 274 and ribbed test tires standardized in ASTM E 501. Currently, FDOT conducts network-level friction testing on a 2-year cycle for interstate highways and on a 3-year cycle for all other state highways. Normally, the locked wheel measurements are conducted in the left wheel path of the lane tested with a frequency of three lockups per mile.

The corresponding Friction Number (FN) measured at a standard speed of 40 mph is designated as FN40R. However, due to the safety issues associated with friction testing at different facilities, FDOT allows for the locked wheel testing to be conducted at non-standard speeds, i.e., 30 mph, 50 mph, and 60 mph (i.e., FN30R, FN50R, and FN60R, respectively). These FN values obtained at non-standard speeds are directly entered into the SHR database without any correction for test speed. As such, the FN values in SHR were corrected for speed using FDOT's available conversion equations (Choubane, et. al., 2012) prior to database integration. The FN-speed conversion equations for FDOT's Dense-Graded Friction Courses (DGFC) are given as the following.

$$FN40R = \begin{cases} 0.87 \times FN30R + 3.45 \\ 0.99 \times FN50R + 3.47 \end{cases} \tag{9}$$

For the Open-Graded Friction Courses (OGFC), the following equations are given for speed conversion.

$$FN40R = \begin{cases} 1.12 \times FN50R - 3.09 \\ 1.19 \times FN60R - 5.59 \end{cases} \tag{10}$$

Similarly, the following equations were developed for rigid pavement surfaces.

$$FN40R = \begin{cases} 0.98 \times FN30R - 2.90 \\ 0.96 \times FN50R + 5.73 \\ 0.95 \times FN60R + 9.87 \end{cases} \tag{11}$$

Texture Database

FDOT's friction data obtained from new construction and/or overlay projects are also recorded in FDOT's texture database. The texture database is internal to FDOT's State Materials Office (SMO) and is hosted in Microsoft Excel. This database stores the friction/texture test results from newly placed pavement surface as well as the mixture related information such as sources of aggregate, voids in mineral aggregate, and contractor. This database was implemented after FDOT's implementation of the 64 kHz laser for high speed texture measurement and hence the data only goes back to 2006.

Pavement Condition Survey (PCS) Database

FDOT's PCS database stores all of the historical pavement condition related information. The PCS survey is conducted annually on all FDOT's roadways. However, it is noted that not all lanes are surveyed annually. FDOT's flexible and rigid pavement condition survey handbooks indicate that the surveyed (or rated) lane should be the one having the worst pavement condition (SMO, 2017a & 2017b). As such, the rated lanes frequently correspond to the outer lanes (both directions) of a divided highway and one of the outside lanes for a composite (undivided) highway.

The PCS data include ratings of FDOT's pavements in terms of crack, ride, and rut. All three ratings are evaluated on a scale of 0.0 to 10.0, where a rating of 10.0 is equivalent to a pavement with no distress. Figure 20 shows the relationship between the PCS rating scale and the amount of cracks, ride number, and rut depth (Choubane et. al., 2017). Also shown in the figure is the deficiency threshold of 6.5 below which a pavement is considered to have failed. A pavement requires resurfacing when any of the three ratings falls below this threshold. Although FDOT has historically used the ride rating obtained from the Ride Number (RN) for both ride acceptance and pavement management, International Roughness Index (IRI) has recently been implemented both for acceptance and pavement management. As such, the IRI has been selected as the measure of pavement smoothness for this study.

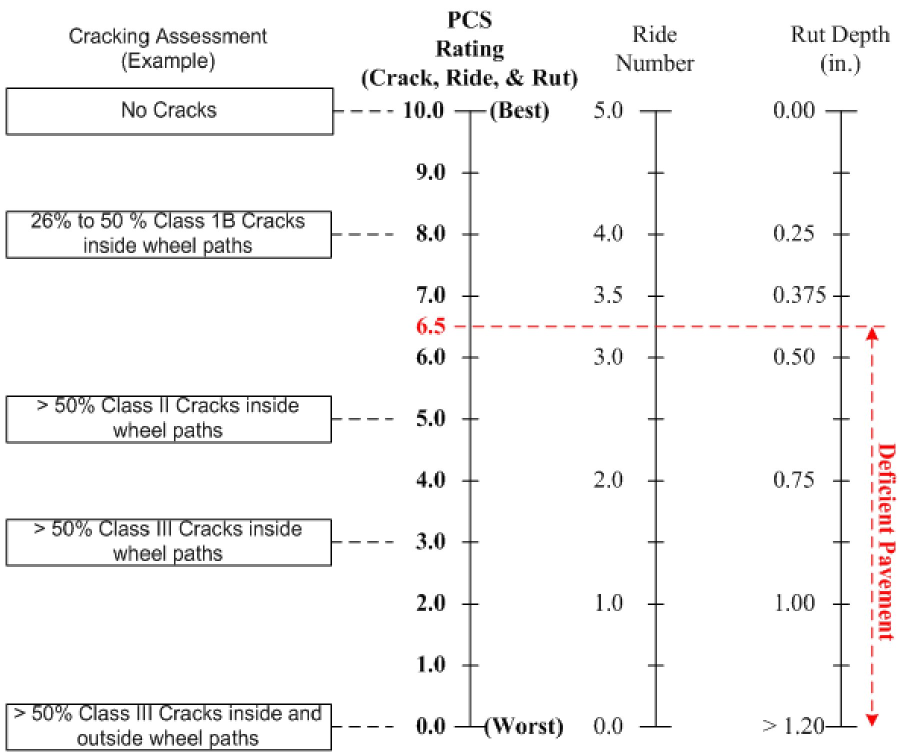


Figure 20. PCS Rating vs amount of cracks, ride number, and rut depth (Choubane et. al., 2017)

Since 2009, FDOT had also put together a more detailed PCS database which stores the RN, IRI, and rut depth (for flexible pavements only) of FDOT's roadways at every 0.1 mile (hence, this database is also known as 0.1-mile PCS database). However, it is noted that the 0.1-mile PCS database does not include crack rating.

Pavement Marking Management (PMM) Database

FDOT's PMM database is a relatively new database that stores FDOT's network-level pavement marking retroreflectivity data. The data is collected using a Mobile Retroreflectivity Unit (MRU). The data is gathered on an annual basis for all of the yellow center-line markings for all state roads in one direction (approximately 12,000 line-miles). In addition, approximately 8,000 line-miles of other line markings such as skip and edge lines are tested each year.

Traffic Database

FDOT's historical traffic data dating back to 1999 were provided to the research team in GIS database format. The database included the Annual Average Daily Traffic (AADT) for all of FDOT's roadways.

Crash Analysis Reporting System (CARS) Database

FDOT's crash database is housed and managed by the Safety Office in Tallahassee. FDOT's internal program called Crash Analysis Reporting System (CARS) can be used to access the data in the crash database and summarize the necessary statistics such as crash per cost, total number of crashes (wet and dry), crashes per million vehicle miles, etc. The CARS can also be used to generate a list of individual crashes along with the relevant information (e.g., weather condition, roadway condition, cause of crash, etc.).

Overview of FDOT's Crash Data

For this study, crash data was extracted from FDOT's CARS system for a period of 8 years (from 2011 and 2017). The extracted data included a large number of attributes such as the following.

1. Location of crash in GPS coordinates as well as in FDOT's linear referencing system.
2. Severity of crash including minor injuries and fatalities.
3. Weather condition such as rain, fog, etc.
4. Lighting condition such as daylight, dawn, dusk, dark, etc.
5. Roadway surface condition such as dry, wet, standing water, oil, mud, etc.
6. Contributing circumstances such as work zone, severe rut, debris, etc.
7. Other potential causes of crash such as drinking & driving, cell phone use, vehicle defects, etc.
8. Cost of crash including vehicle and property damage costs.

In summary, approximately a total of 1.13 million crashes that occurred on FDOT's roadway systems (i.e., not including crashes that occurred on County roads and City streets, etc.) were extracted from the database for the 8 year period. Figure 21 shows all of the individual crashes mapped onto Florida's map. These crashes correspond to 930,512 dry weather crashes and 200,005 wet weather crashes which indicate that on a statewide basis, approximately 360 percent more crashes occurred during dry weather when compared to wet weather.

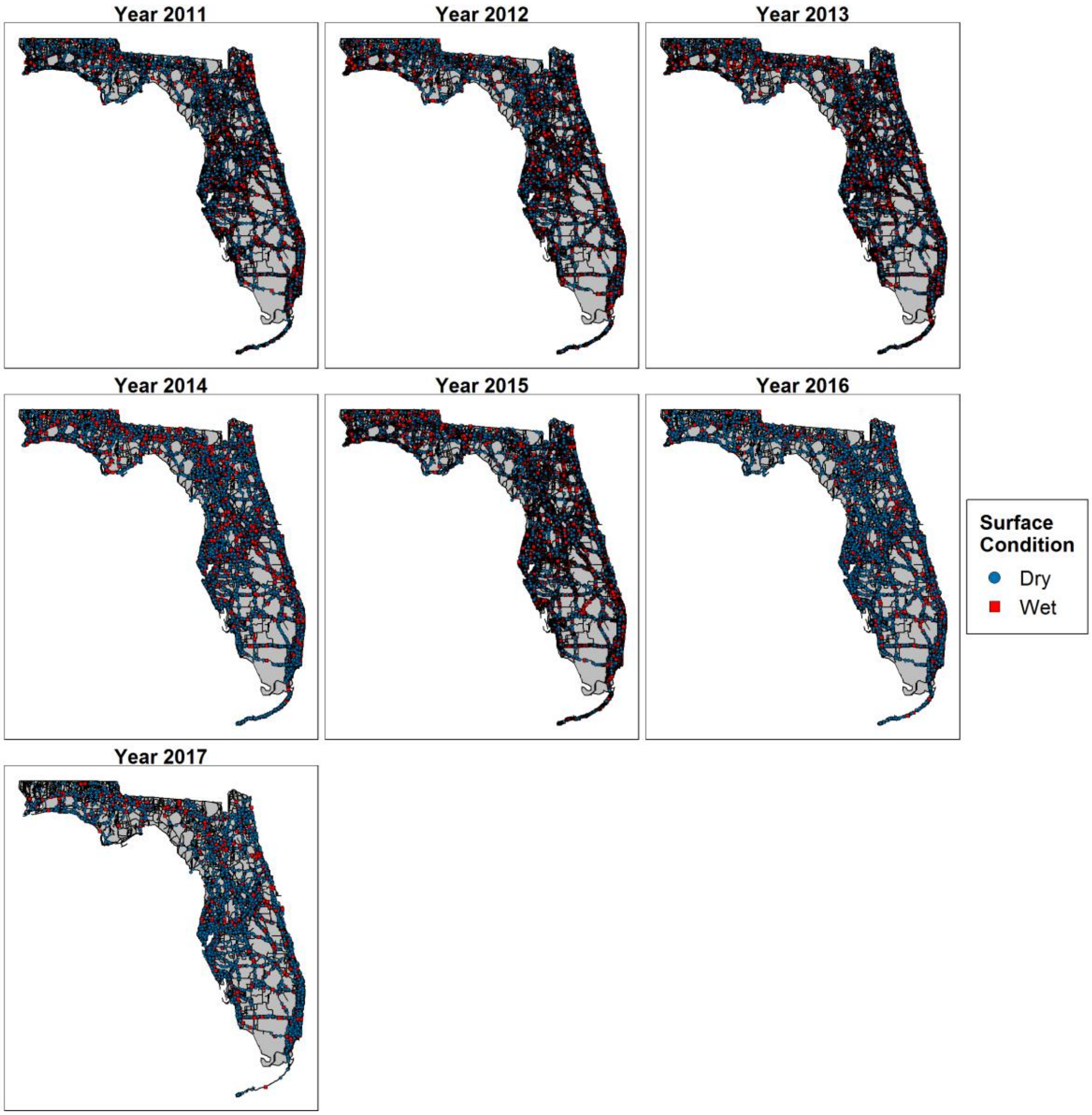


Figure 21. Mapping of dry and wet weather crashes (2011 – 2017)

For reference purposes (i.e., for those that are not familiar with FDOT's Districts), Figure 22 shows a map of FDOT's Districts 1 through 7 (labeled as "D1" through "D7"). Figure 23 shows the total number of dry and wet weather crashes broken down for Florida's Counties. Similarly, Figure 24 shows the total number of fatalities and the total cost of crash on a County basis for the same 8 year period. Regardless of the roadway condition (i.e., dry vs. wet), these figures generally show that fewer number of crashes (and hence fewer fatalities and lower cost) were observed in FDOT's panhandle area (D3) whereas significantly higher number of crashes occurred in southern Florida Districts (D4 and D6).

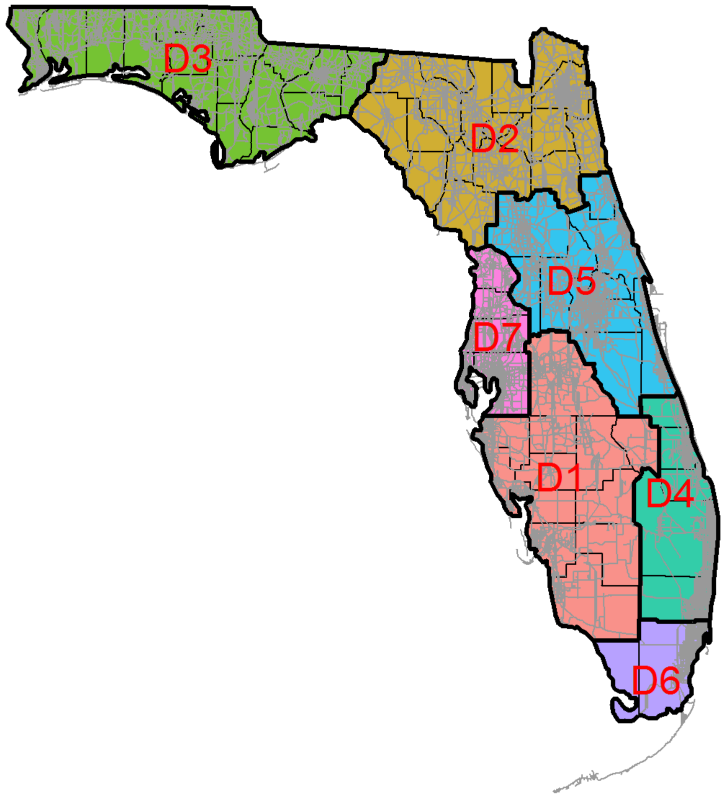
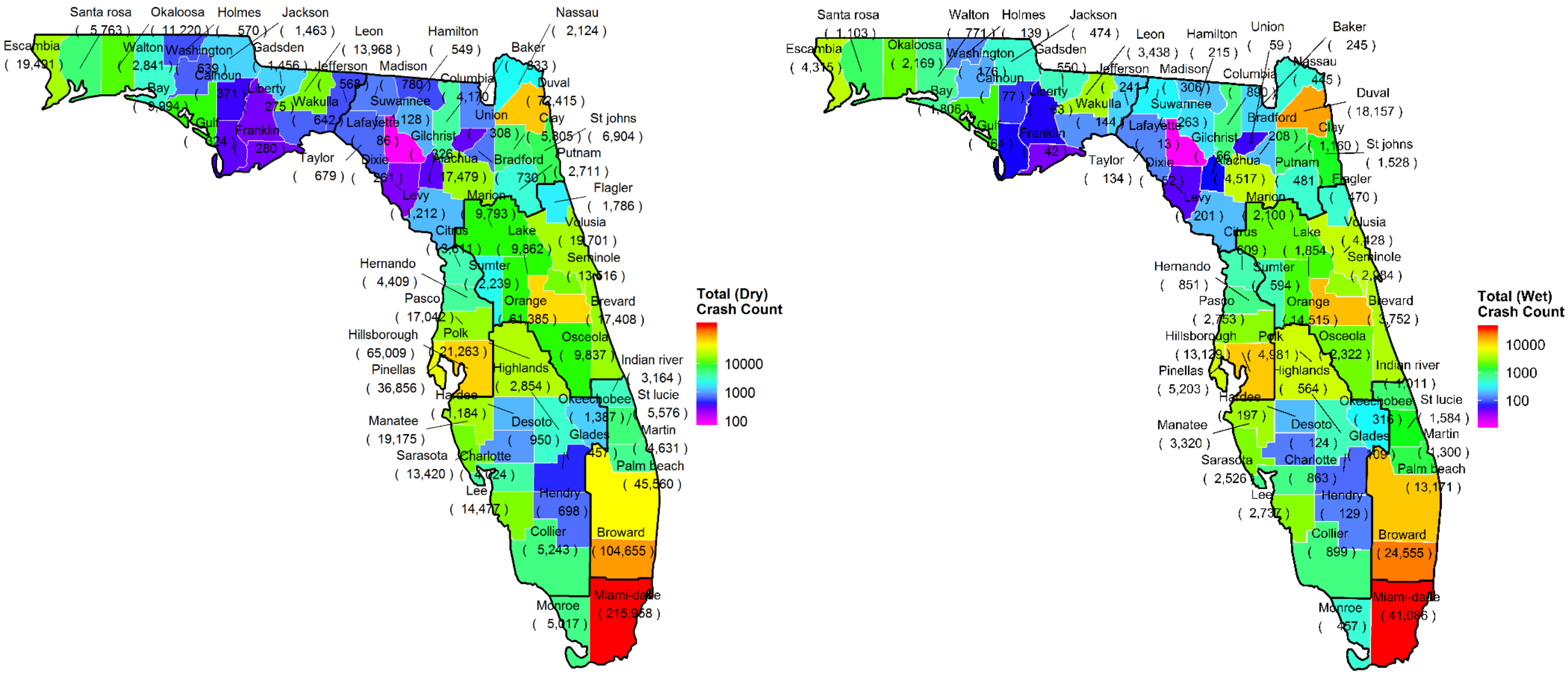


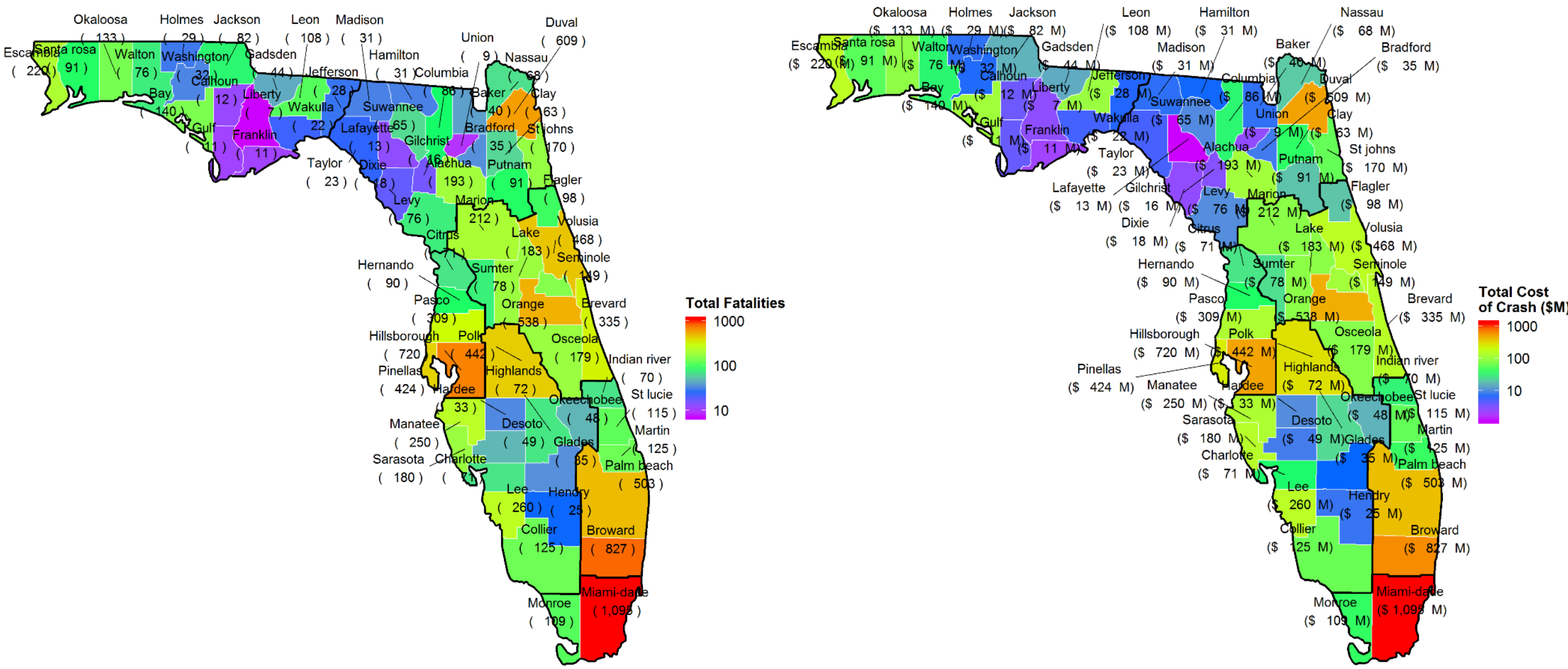
Figure 22. FDOT Districts



(a)

(b)

Figure 23. Florida's (a) dry and (b) wet weather crashes by County (2011 – 2017)



(a) (b)

Figure 24. Florida's (a) total fatalities and (b) total crash cost by County (2011 – 2017)

Integrated Database

The schematics of the integrated database is shown in Figure 25. Essentially, the new database was built around the 0.1-mile PCS database which included the IRI, RN, and rut depth values at every 0.1 mile increment for the entire network.

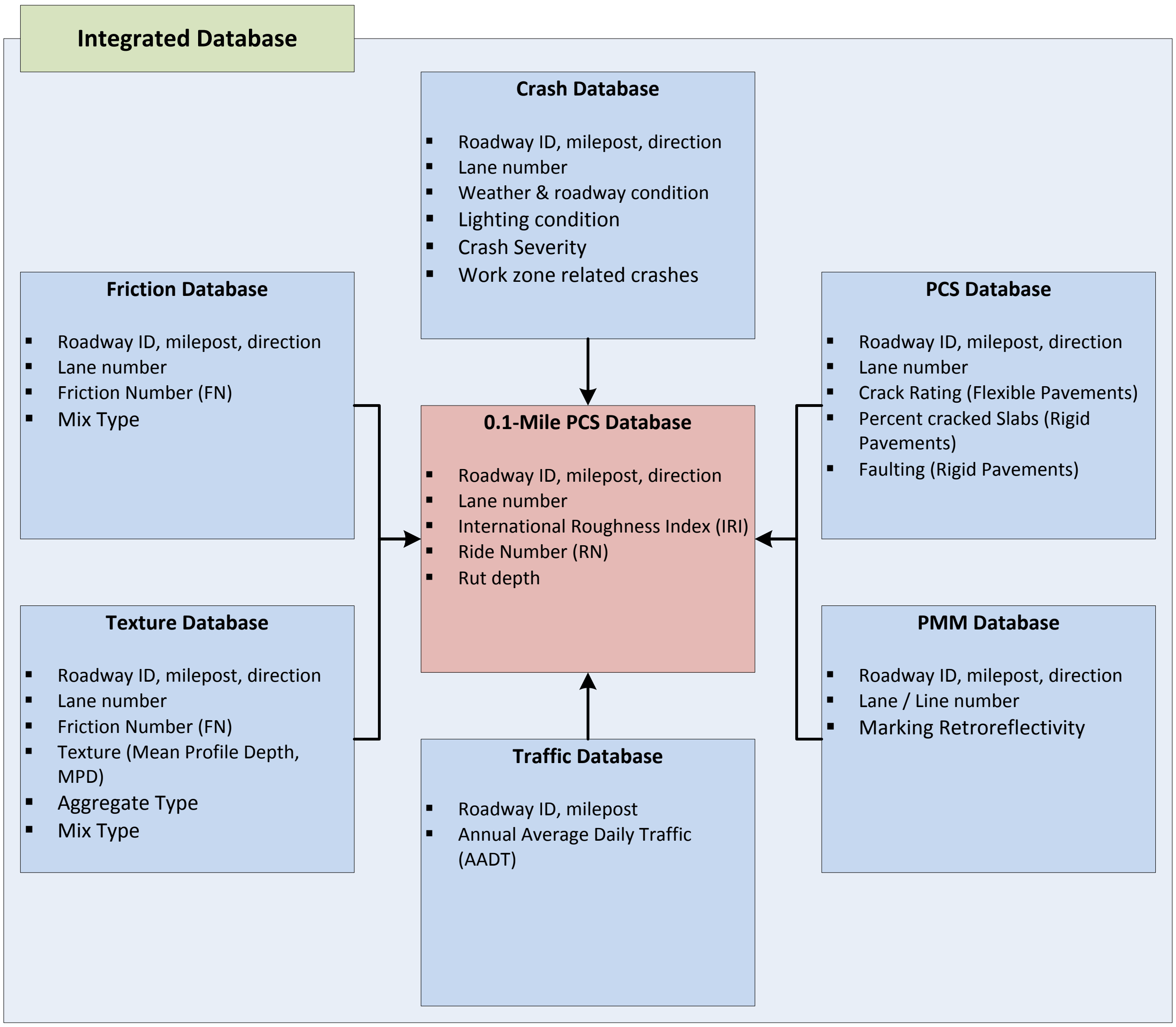


Figure 25. Construction of the Integrated Database

The remaining data from the above-mentioned databases were integrated into this 0.1-mile database. The data integration was carried out by identifying and matching the roadway location based on FDOT's Straight Line Diagram (SLD) milepost that were commonly available in all of the above databases. However, due to the differences in survey practice (frequency, surveyed lanes, etc) of different datasets (friction, IRI, crack rating, etc), several challenges were encountered during data integration. These challenges and the assumptions for data integration are summarized in the following.

- The friction data was not collected on an annual basis (i.e., 2-year cycle for Interstates and 3-year cycle on other roads). If the friction data was not available for a particular year then the FN from the most recent friction testing was used to populate the friction data for the missing years.
- Although all lanes are generally tested for newly placed surfaces (friction and PCS), this was not true for network-level inventory testing. I.e., there were discrepancies in the surveyed lane (e.g., PCS survey was done in the southbound lane of a 2-lane road whereas the friction testing was conducted in the northbound lane). In these cases, the friction number from the closest lane was selected and populated for the integrated database.
- Pavement age was not included in any of the available databases. Therefore, each of the 0.1-mile segment was mapped to the historical PCS and SHR database and traced back in time until the segment was marked to be a new surface (i.e., construction year). The construction year was then used to calculate the pavement age.
- The texture database was limited to the data collected for new construction and overlay projects. In other words, the change in texture over the pavement life could not be established. As such, it was assumed that the texture data remains unchanged for the entire pavement life.
- As mentioned, the 0.1-mile PCS database only includes the data that are collected in an automated fashion, namely IRI, RN, and rut depth (for flexible pavements). As such, other data attributes (e.g., crack rating from PCS database and FN from SHR, etc.) were assumed to be constant within the limits identified in the respective database.
- Although the PMM data was stored in 0.1-mile basis, the pavement line markings surveyed for PMM did not necessarily match the lanes surveyed for PCS or friction. As such, the available PMM data was averaged for all available line markings for a given 0.1-mile segment prior to being integrated.
- The crash database included a significant number of records that could not be mapped back to the integrated database. These records include crashes that occurred on ramps, turn lanes, roadway median or shoulder, and other lanes that were not surveyed for PCS or friction at a network level.
- Again, the focus of this study is to study the roadway features that contributes to crashes. As such, crashes that were primarily attributed to other reasons (e.g., vehicle defects, driving under influence, and cell phone use while driving) were removed prior to the data integration.

Figure 26 shows a quick summary of the crashes that were mapped to the integrated database. In total, 125,091 crashes (approximately 11 percent of all crashes in the crash database) were made available in the final database. More specifically, 105,093 dry weather (approximately 11 percent) and 19,998 wet weather (approximately 16 percent) crashes were mapped and stored in the integrated database. These numbers correspond to 825,419 dry weather crashes and 180,007 wet weather crashes that were not mapped to the integrated database.

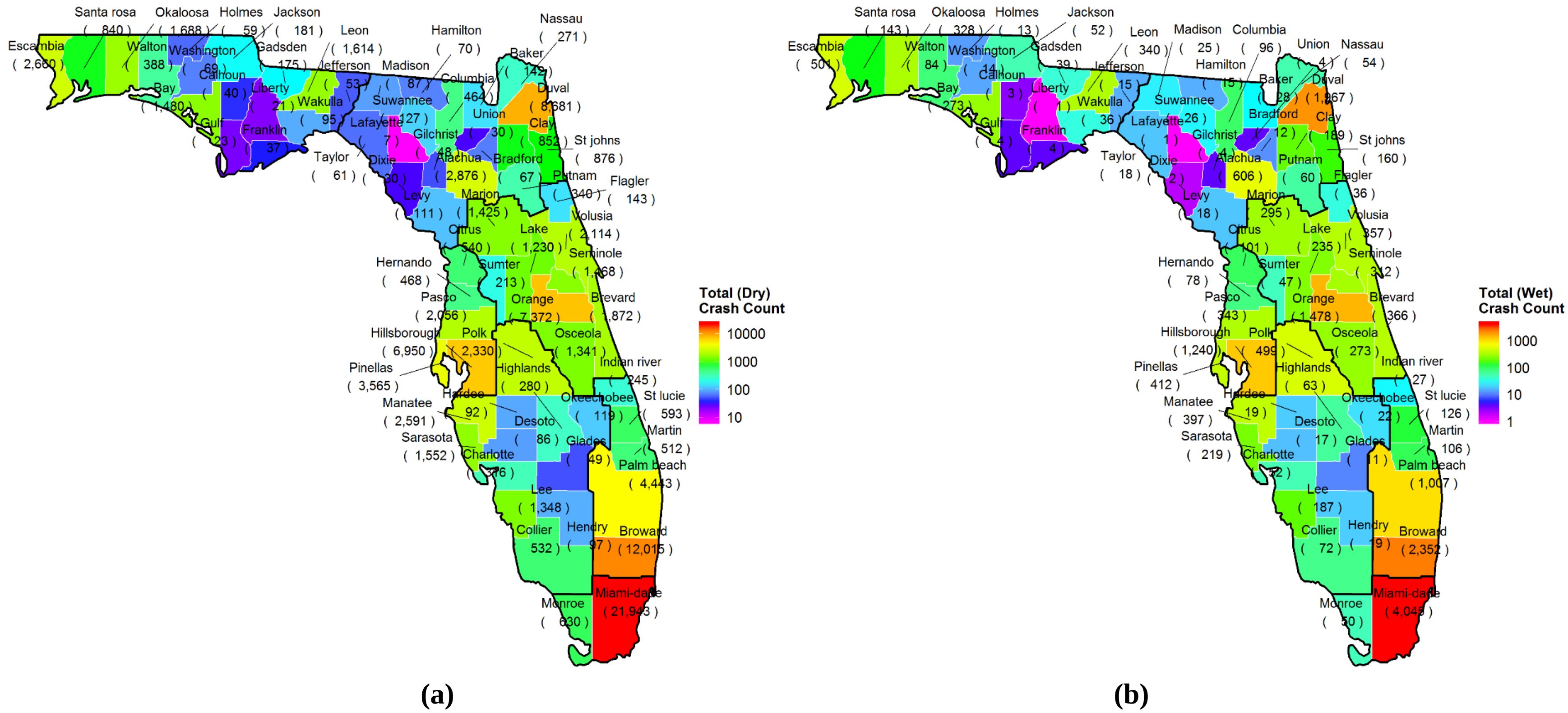


Figure 26. (a) Dry and (b) wet weather crashes in the Integrated Database by County (2011 – 2017)

PRELIMINARY ANALYSIS

Prior to developing the statistical models between crash counts and roadway related features, a preliminary analysis was conducted to understand the trends in friction, traffic, and crash. The results and findings of the preliminary analyses are summarized in this section of the report.

Friction Number

Nomenclatures for Surface Type and Mix Type

Prior to discussing the preliminary analysis of friction, it is deemed necessary that the nomenclatures for FDOT's surface type and mix type be defined. Table 18 summarizes the surface and mix type nomenclatures used in FDOT's SHR and Texture databases. Currently, Longitudinal Grinding (LGD) is the only allowed finishing texture for FDOT's rigid pavements. The asphalt surfaces can be categorized first into open and dense graded surfaces, with different mix families in each surface.

Table 18. Nomenclatures for Surface and Mix Types in FDOT's SHR Database

Pavement Type	Surface Type	Mix Type / Finishing Texture
Rigid	Rigid	Longitudinal Grinding (LGD)
Flexible	Open-Graded Friction Course (OGFC)	FC5 Mix Family (FC5, FC5M, etc.)
	Dense-Graded Friction Course (DGFC)	FC95 Mix Family (FC95, FC95MW, etc.) FC125 Mix Family (FC125, FC125MW,etc)

As an example, FC125M is a dense graded friction course with Nominal Maximum Aggregate Size (NMAS) of 12.5 mm. The letter "M" at the end indicates that a polymer modified asphalt binder was used for this particular mixture. Similarly, additional letters can be added to the Mix Type nomenclature. These letters are:

- **M**: Polymer modified binder
- **R**: Recycled mix
- **A**: Asphalt rubber binder
- **W**: Warm mix

Friction Number Distributions

Figure 27 shows the distribution of all friction numbers (FN40R, after speed conversion) in the integrated database. The mean and various percentiles corresponding to this distribution is summarized in Table 19.

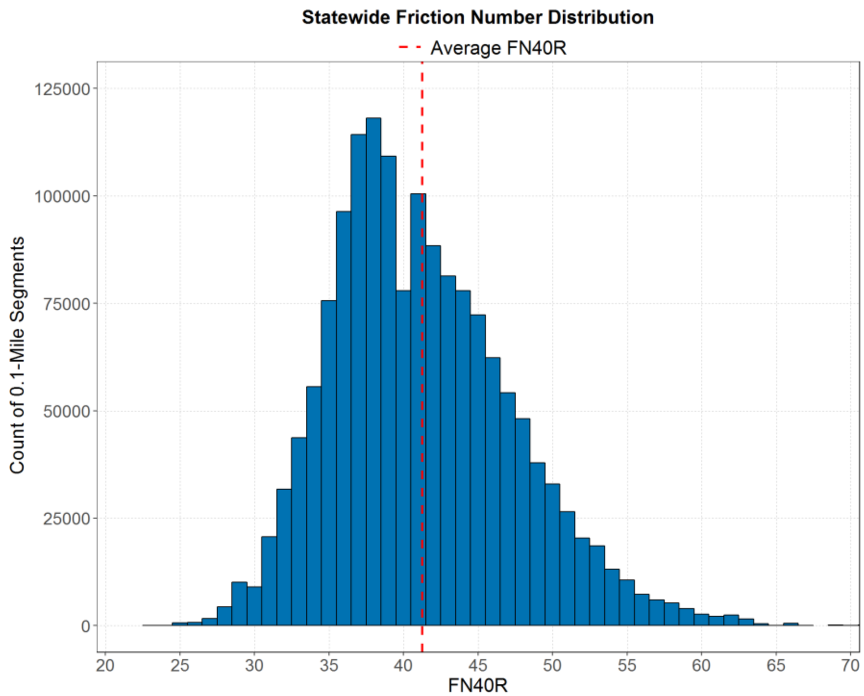


Figure 27. Florida's statewide distribution of friction (2009 – 2017)

Table 19. Mean and percentiles of Florida's statewide friction data (2009 – 2017)

Mean	Percentiles						
	5 %-tile	15 %-tile	25 %-tile	Median	75 %-tile	85 %-tile	95 %-tile
41	32	35	37	41	45	48	52

The above figure and table show that the FN40R values are within a relatively narrow range (i.e., 50 percent of FN40R between 37 and 45, and 70 percent between FN40R of 35 and 48). In addition, it is seen that only a small fraction of roadways exhibited FN40R values less than 35 or 30. This is likely due to FDOT's friction restoration program taking place when FN40R drops below the desired value and FDOT's aggregate approval program, which has strict approval requirements for aggregates used in friction courses.

Similar to the above, Figure 28 shows the FN40R distribution on a District basis with the mean and percentile values summarized in Table 20. These results show that District 3 (Florida's panhandle area) generally shows higher friction whereas Districts 4 and 6 (southern Florida) show relatively lower friction characteristics. It is believed that this is primarily due to the different aggregates used in these regions. More specifically, District 3 mostly uses granite aggregates in their friction courses whereas limestone aggregates are primarily used in Districts 4 and 6.

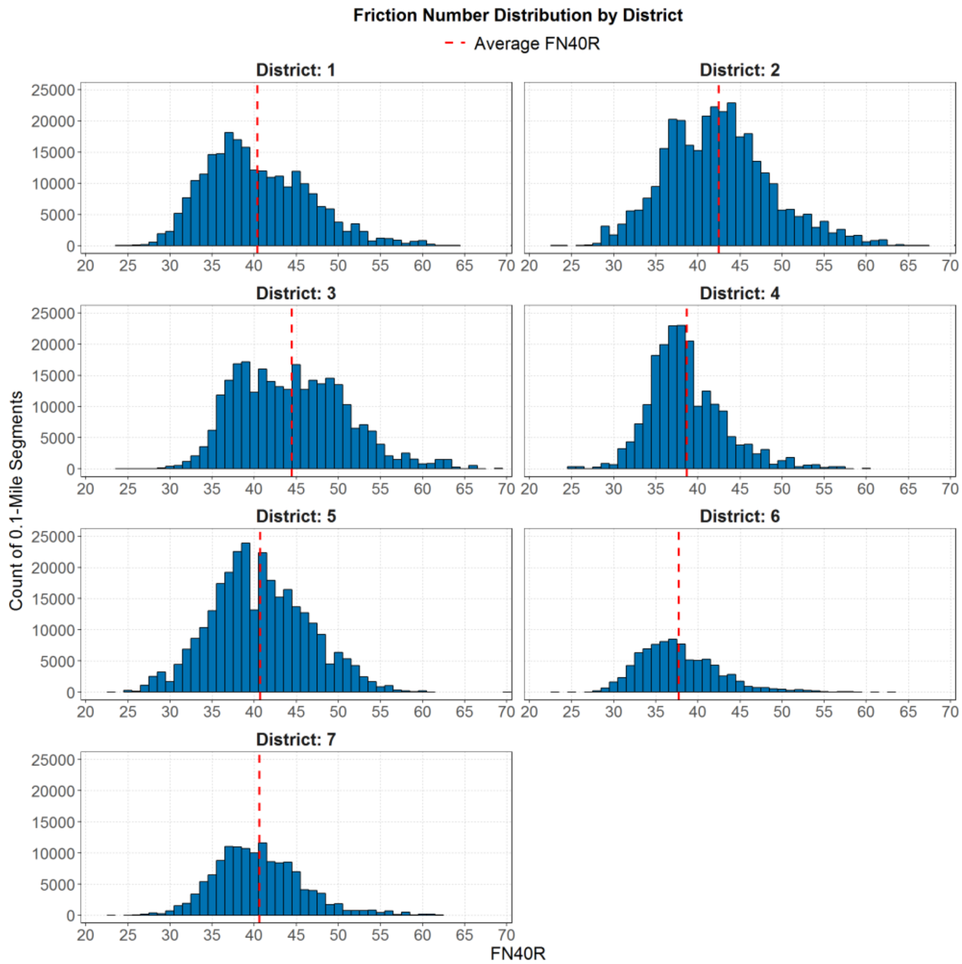


Figure 28. Florida's distribution of friction by District (2009 – 2017)

Table 20. Mean and percentiles of Florida's friction data by District (2009 – 2017)

District	Mean	Percentiles						
		5 %-tile	15 %-tile	25 %-tile	Median	75 %-tile	85 %-tile	95 %-tile
1	40	32	34	36	39	45	47	52
2	42	33	36	38	42	46	49	54
3	44	35	38	39	44	49	51	55
4	38	32	35	36	38	41	43	47
5	41	32	35	37	40	45	47	51
6	38	31	33	34	37	41	42	46
7	41	33	36	37	40	44	46	50

Friction Number Degradation

Revisiting Figure 27 and Table 19, it is seen that approximately 15 percent of FN40R values were less than 35 between years 2009 and 2017. Due to FDOT's existing Friction Guidelines, it is likely that these roadways did exhibit higher FN40R (e.g., greater than 35) when they were resurfaced, but the friction number reduced over time (i.e., friction degradation). As such, it is also of interest to assess how fast a newly placed surface deteriorates in terms of friction.

Figure 29 shows the overall plot of FN40R versus pavement age, for the three major surface types (rigid, dense, and open). Although the coefficient of determination (R^2) values are low, the trend lines embedded within each plot may provide a rough idea of how these surfaces are generally performing over time. As an example, rigid surfaces (i.e., LGD) will likely have an initial FN40R of 41 and its value will drop at a rate of 0.39 per year.

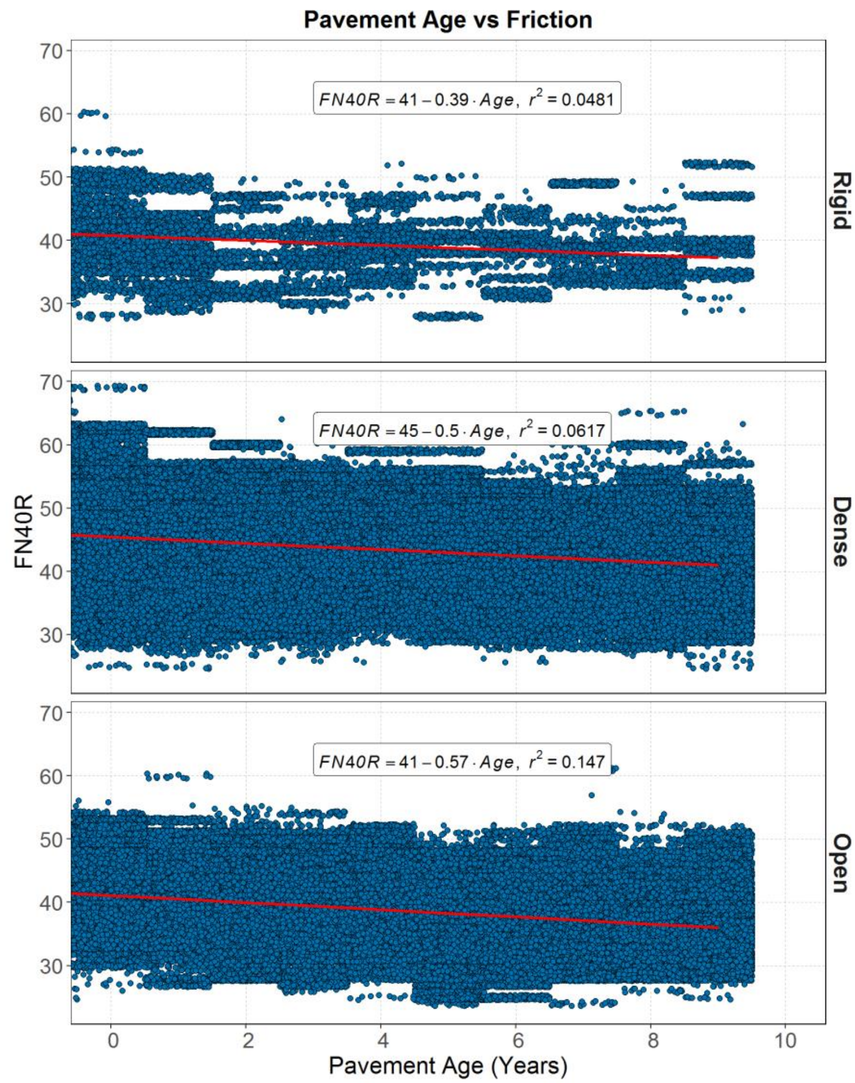


Figure 29. FN40R vs. Pavement age

Similarly, the FN40R versus pavement age plots were generated for each individual mixtures defined in Table 18 and for the primary aggregate types (i.e., granite and limestone) used in FDOT's mixtures. These results are shown in Figures 30, 31, and 32 for the FC125, FC95, and FC5 mix families, respectively.

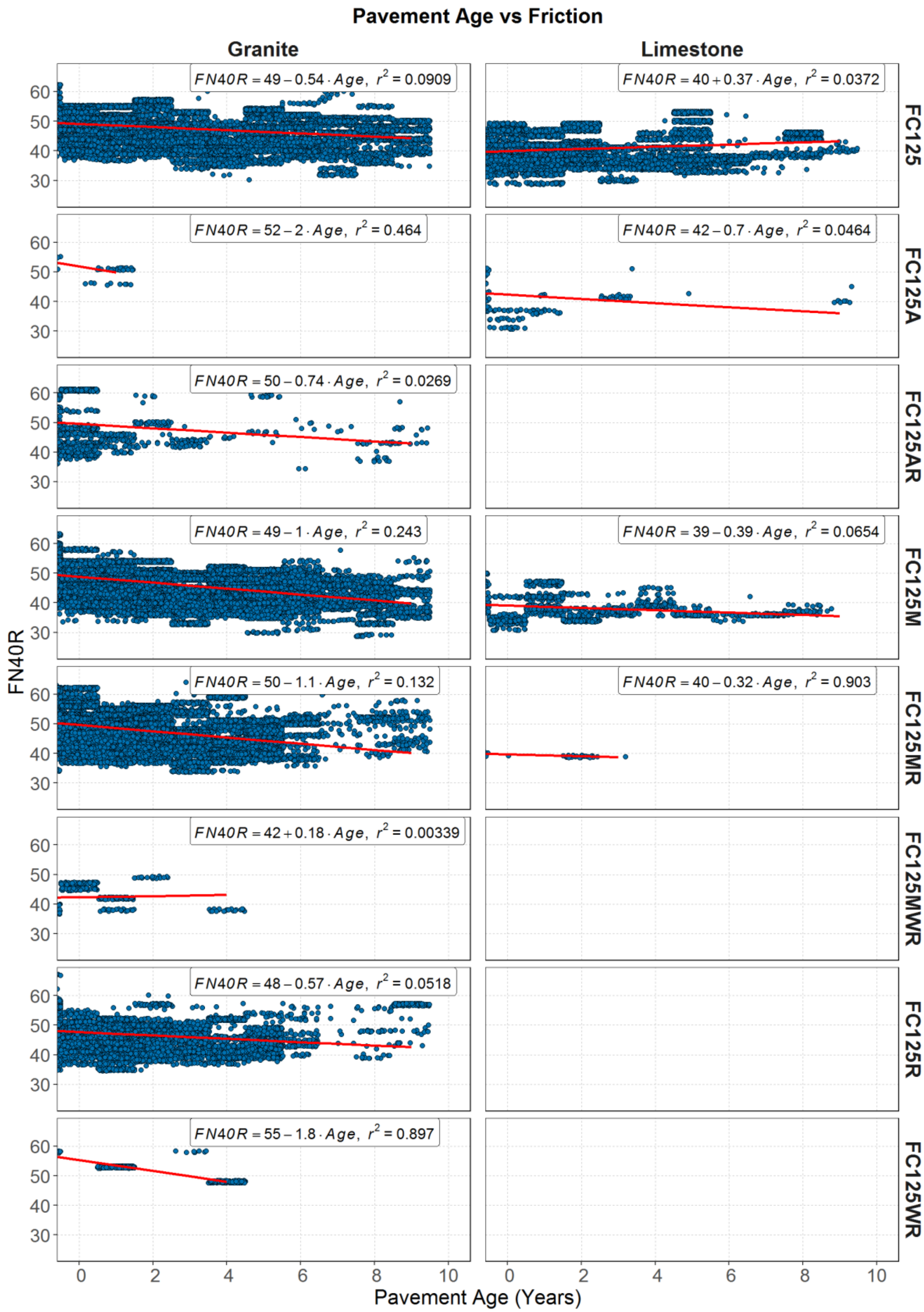


Figure 30. FN40R vs. Pavement age for FC125 mix family

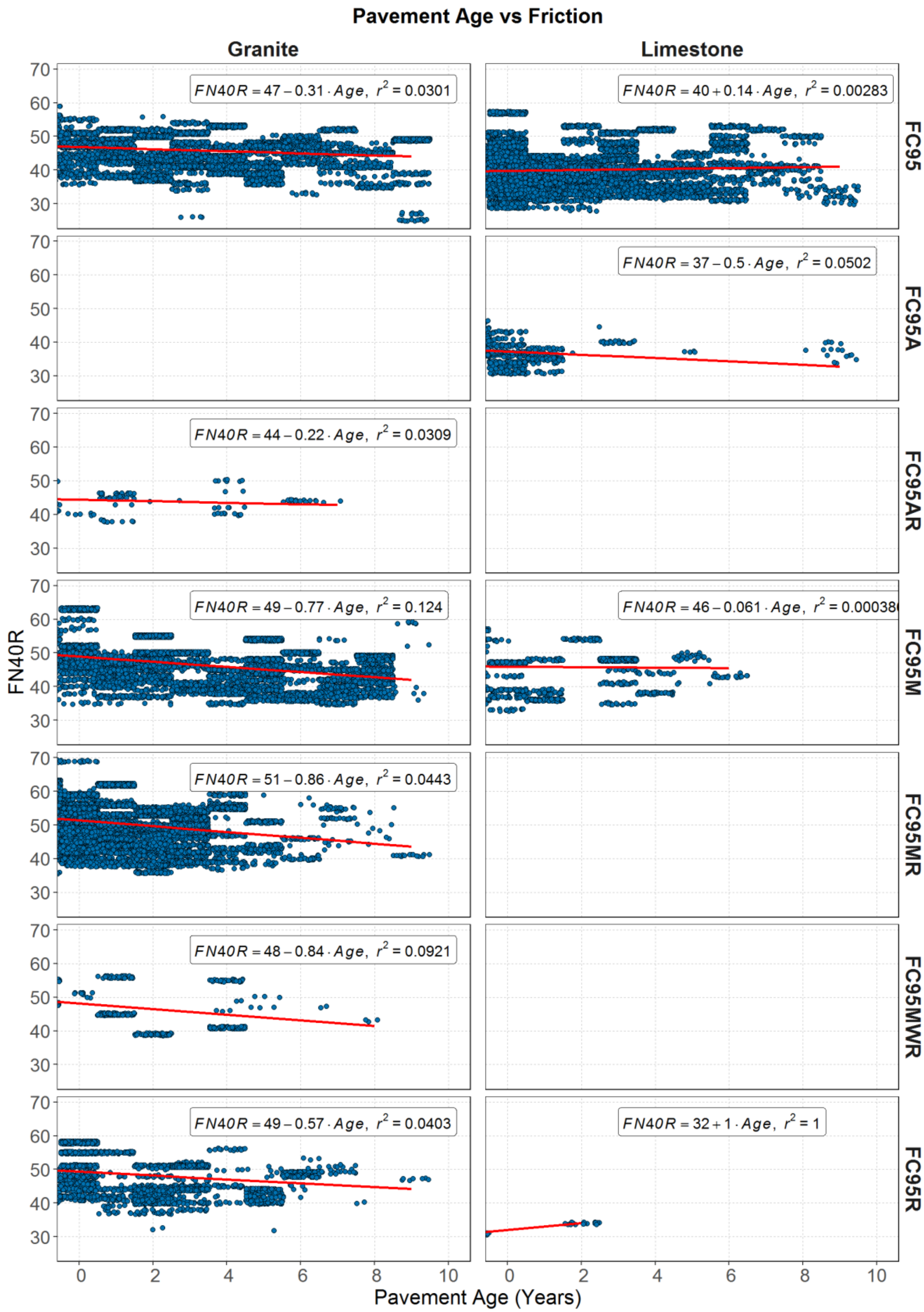


Figure 31. FN40R vs. Pavement age for FC95 mix family

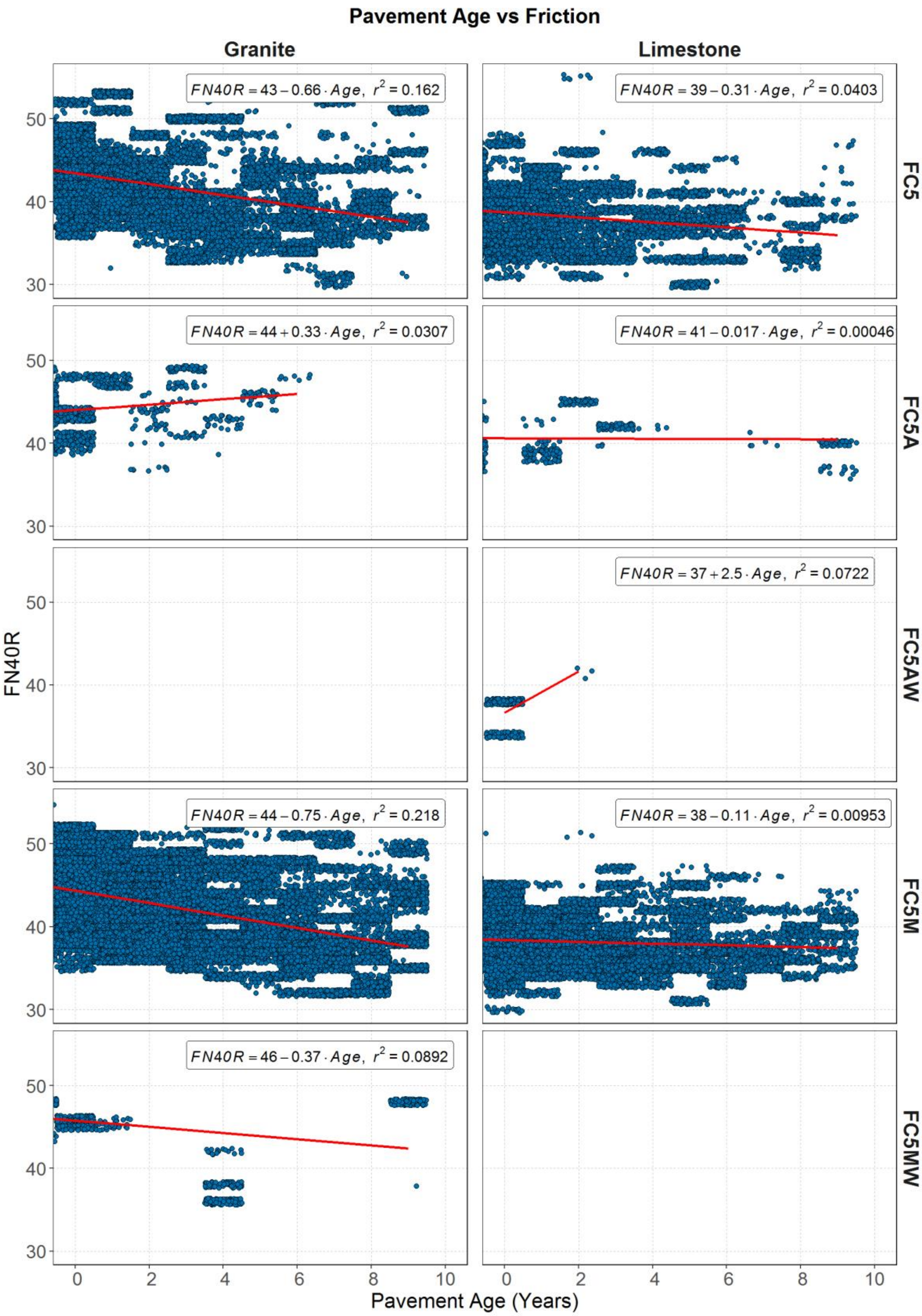


Figure 32. FN40R vs. Pavement age for FC5 mix family

Traffic Distributions

As indicated by Srinivasan and Bauer (2013), the only variables included in the most basic form of the Safety Performance Function (SPF) are the segment length and traffic (in terms of AADT). Therefore, the AADT distribution is briefly studied herein.

Figure 33 and Table 21 summarize the AADT distribution on a District basis, along with the average AADT observed within each District. Despite the skewed distributions, these results clearly show that there is a significantly higher amount of traffic in southern Florida (D4 and D6) with the least amount of traffic in the panhandle area (D3).

As discussed above, the lower number of crashes observed in D3 may be attributed to higher FN40R and lower AADT. Conversely, the increased number of crashes in D4 and D6 are likely due to the lower FN40R and higher amount of traffic. Although this provides fairly reasonable inferences on crash, friction, and traffic, it is challenging to separate the effects of friction and traffic if these distributions are studied without connection.

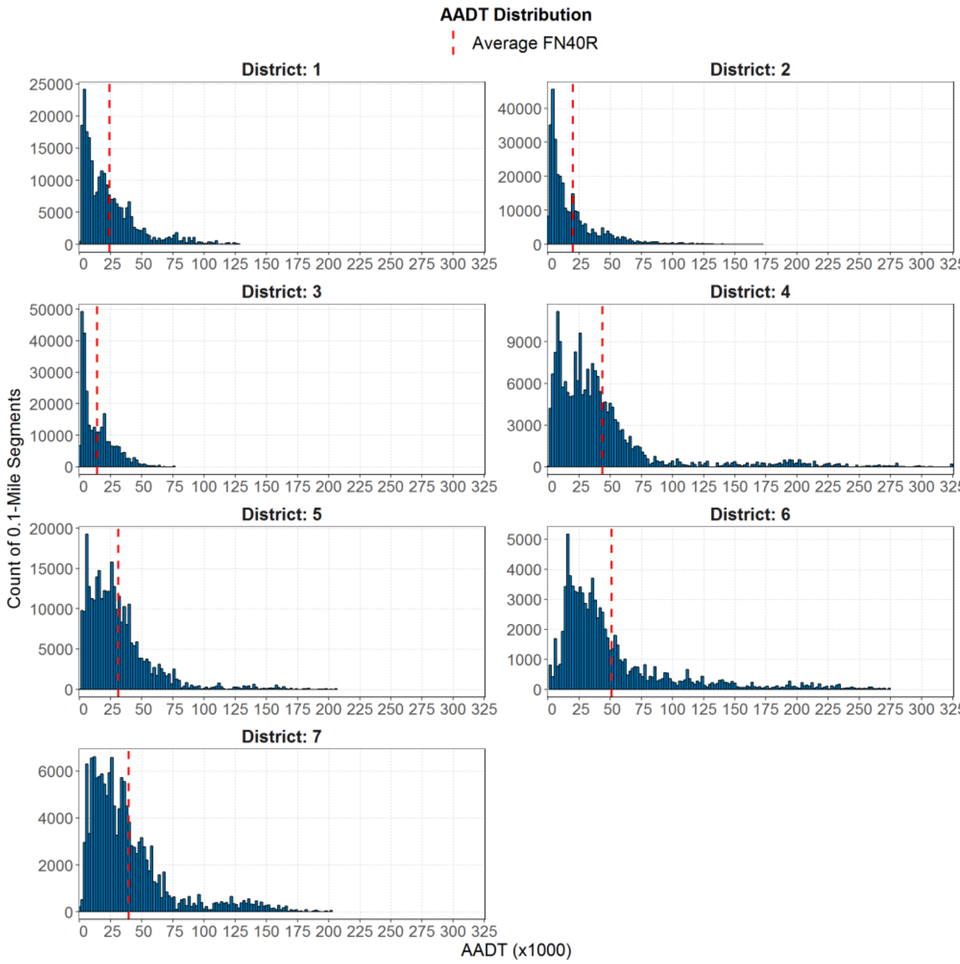


Figure 33. AADT distribution by District (2009 – 2017))

Table 21. Mean and percentiles of AADT (×1,000) by District (2009 – 2017)

District	Mean	Percentiles						
		5 %-tile	15 %-tile	25 %-tile	Median	75 %-tile	85 %-tile	95 %-tile
1	24.0	2.6	4.5	7.0	17.7	33.5	42.0	75.5
2	19.7	1.7	3.4	4.8	11.3	25.5	39.5	65.0
3	14.0	1.4	2.4	3.5	9.9	20.9	28.4	39.5
4	43.5	4.9	9.2	15.3	32.0	50.5	64.0	163.5
5	30.9	4.2	8.1	13.7	25.5	40.0	51.0	75.5
6	51.1	11.2	16.6	21.5	36.0	58.0	87.0	154.0
7	39.2	6.2	11.5	16.8	30.5	49.5	63.0	122.0

Crash Count and Rate Distributions (Initial Assessment)

As a preliminary to studying the combined effects of friction and traffic on crash counts, Figures 34 and 35 show the statewide crash count distributions plotted against FN40R and AADT, respectively. The figures also show the Wet-to-Dry (W/D) crash ratios calculated from the same dataset.

Figure 34 shows that both the dry and wet weather crash distributions are fairly normal with respect to FN40R. In addition, the W/D ratio shows a steady, gradual reduction with increasing friction for FN40R between 30 and 45. For FN40R below 30, the W/D ratio increases very rapidly with reduction in FN40R. However, it is not clear if such a rapid increase is caused by reaching a critical level of friction or simply due to lack of data. In other words, such rapid increase in W/D ratio may be an artifact of not having sufficient roadway sections and corresponding crash data, similar to the spikes observed at FN40R of 63 and 65.

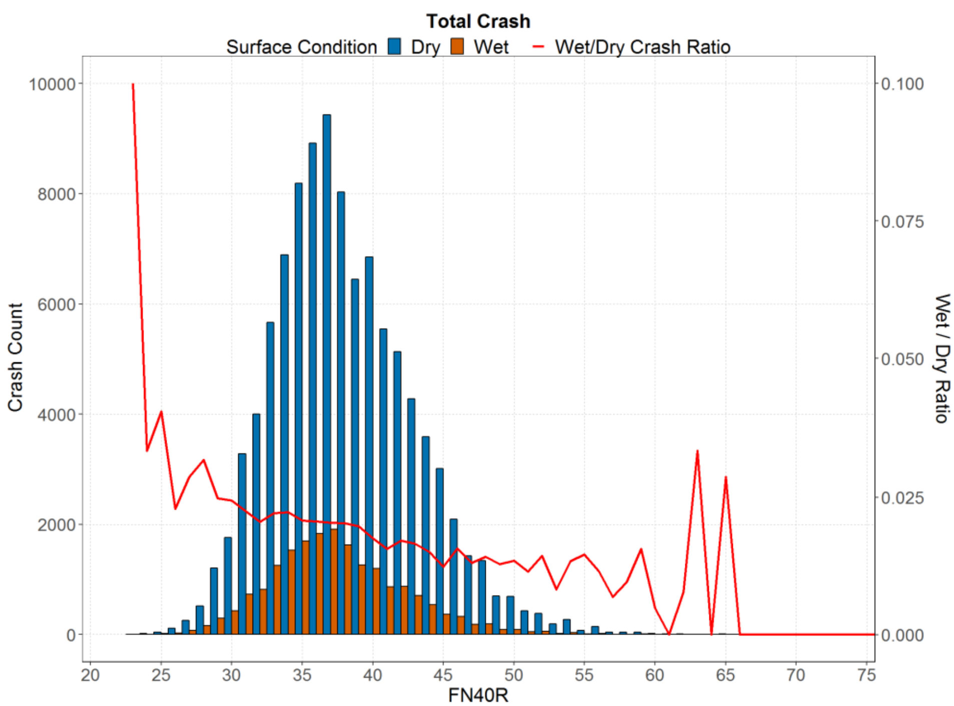


Figure 34. Statewide Distribution of Crash vs. FN40R (2011 – 2017)

Figure 35 shows the distribution of wet and dry weather crashes as well as the W/D ratio with respect to AADT. It is seen that distributions are significantly skewed due to the limited number of segments with AADT in excess of 75K. In addition, the W/D ratio initially shows a steady increase with increasing AADT, but shows a rapid increase when it reached AADT of 75K. Similar to the discussion made previously, it is not clear if this sudden jump in W/D ratio is indicative of a critical level of AADT or if it is simply due to lack of data. However, the noise seen in the W/D ratio for AADT greater than 100K suggests that lack of sufficient data may have caused these issues.

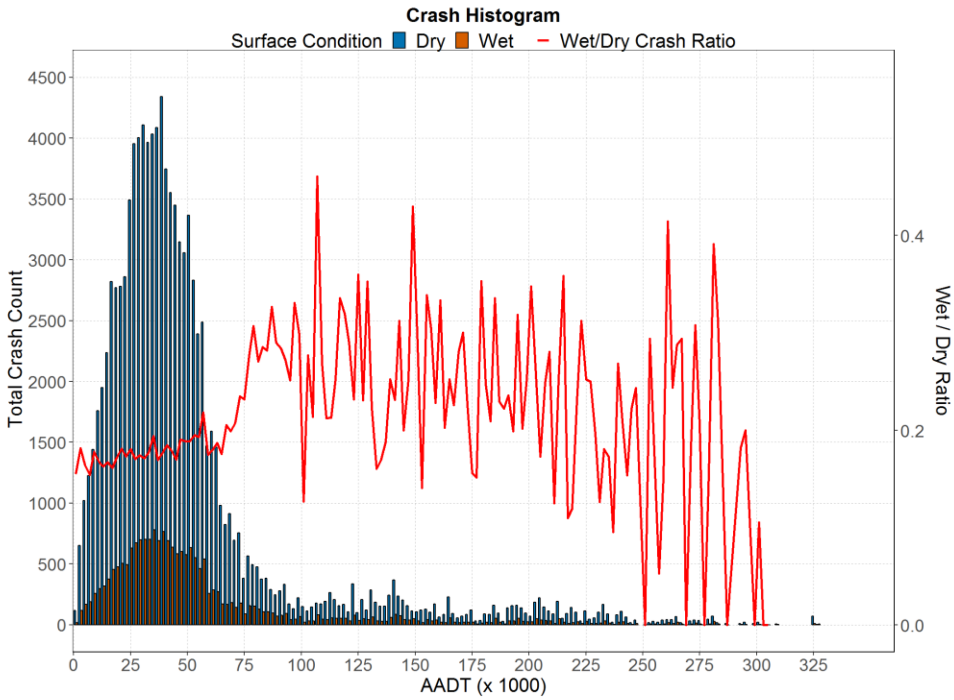


Figure 35. Statewide Distribution of Crash vs. AADT (2011 – 2017)

To study the combined effects of friction and traffic on the number of crashes, it was decided to normalize the crash counts by the amount of traffic through the concept of crash rates. The crash rate, *CR*, is defined as the following

$$CR_i = \frac{C_i \times 10^8}{365 \times Y \times AADT_i \times L} \tag{12}$$

where CR_i is the crash count per 10^8 vehicle-miles travelled (VMT) for the i^{th} segment (or category), Y is the number of years, $AADT_i$ is the traffic for the i^{th} segment, and L is the segment length in miles.

Figure 36 shows the dry and wet weather crash rates calculated on a statewide basis, and plotted against FN40R. The figure shows that the overall trends of dry and wet crash rates are similar to each other. The figure also shows intuitive trends for FN40R greater than 30 (i.e., crash rates reduce with increasing FN40R). Nevertheless, the trends shown for FN40R below 30 do not show a very clear trend.

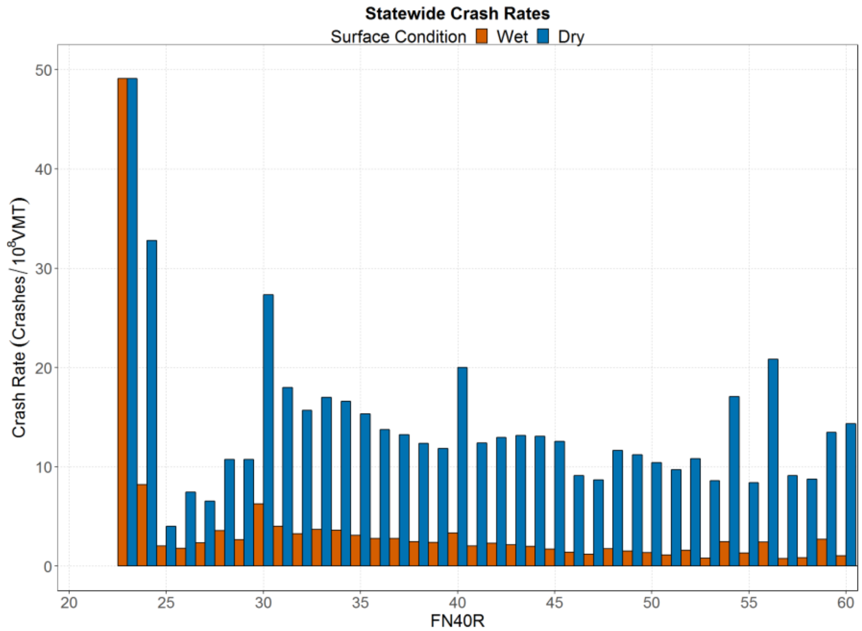


Figure 36. Statewide Distribution of Crash Rates (2011 – 2017)

It is believed that the unexpected trend shown in the above figure (for the lower FN40R range) is due to the insufficient number of roadways below FN40R value of 30. To assess if there were any particular speed zones and/or geographic locations that are responsible for this trend, the crash rates were calculated for different speed limits (from 25 to 70 mph) and District. These results are shown in Figures 37 and 38 for dry and wet weather crashes, respectively.

Both Figures 37 and 38 reveal that there is no clear trend in crash rates for low speed facilities with speed limits of 25 mph and 30 mph. On the other extreme, the high speed facilities (speed limits 50 mph through 70 mph) do not seem to have sufficient data to yield any notable trends. Although the trends shown for speed limits 35 mph through 45 mph were slightly more reasonable (e.g., see Figure 37 for D6 at 45 mph), these graphs are in general, very noisy. Furthermore, these figures fail to show the difference between low and high traffic facilities.

The above observations suggest that the available data may need to be combined over a range of speed limits and the crash distributions as well as the crash rates be re-evaluated. In other words, the friction demand categories may need to be established prior to studying the crash statistics.

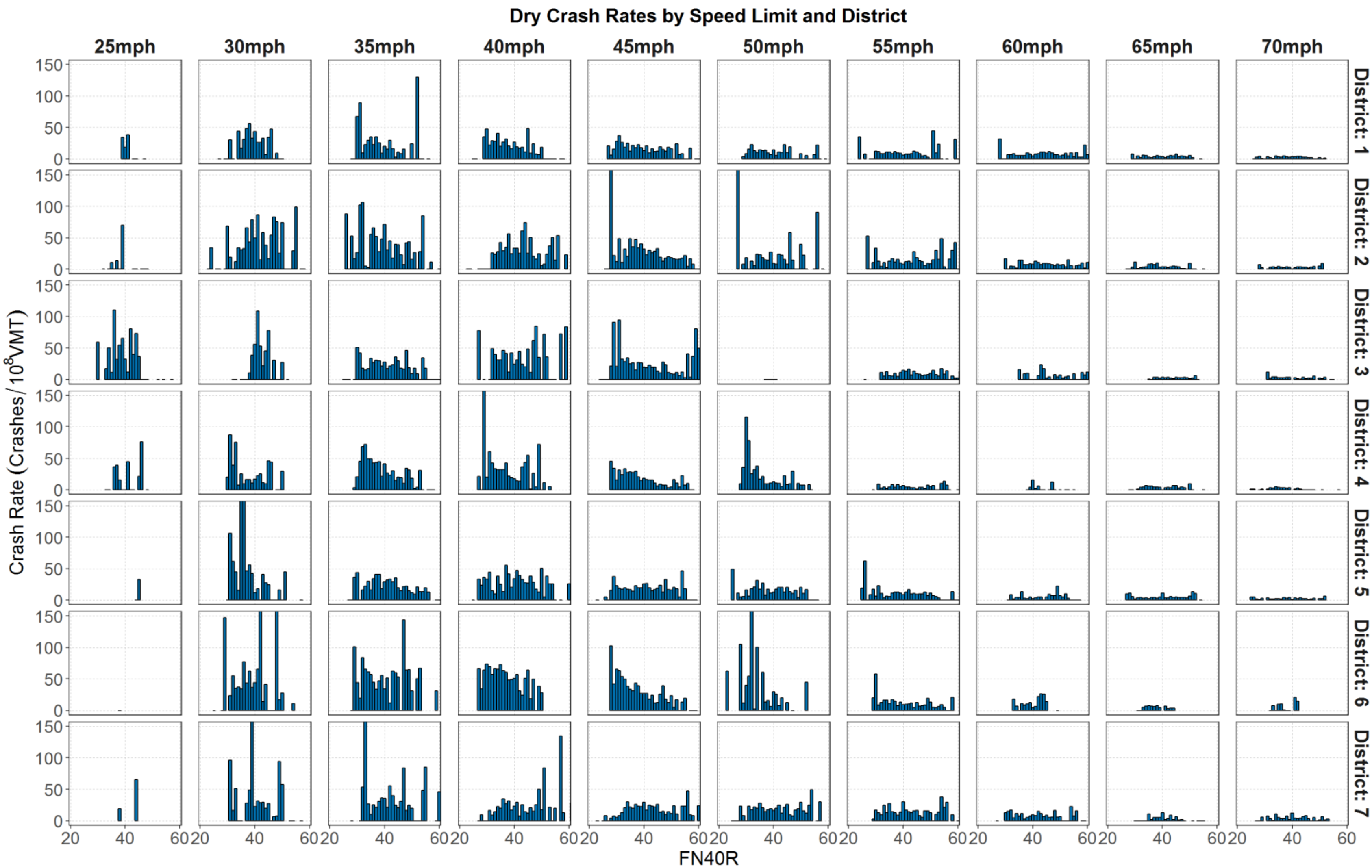


Figure 37. Dry Weather Crash Rate Distribution by Speed Limit and District (2011 – 2017)

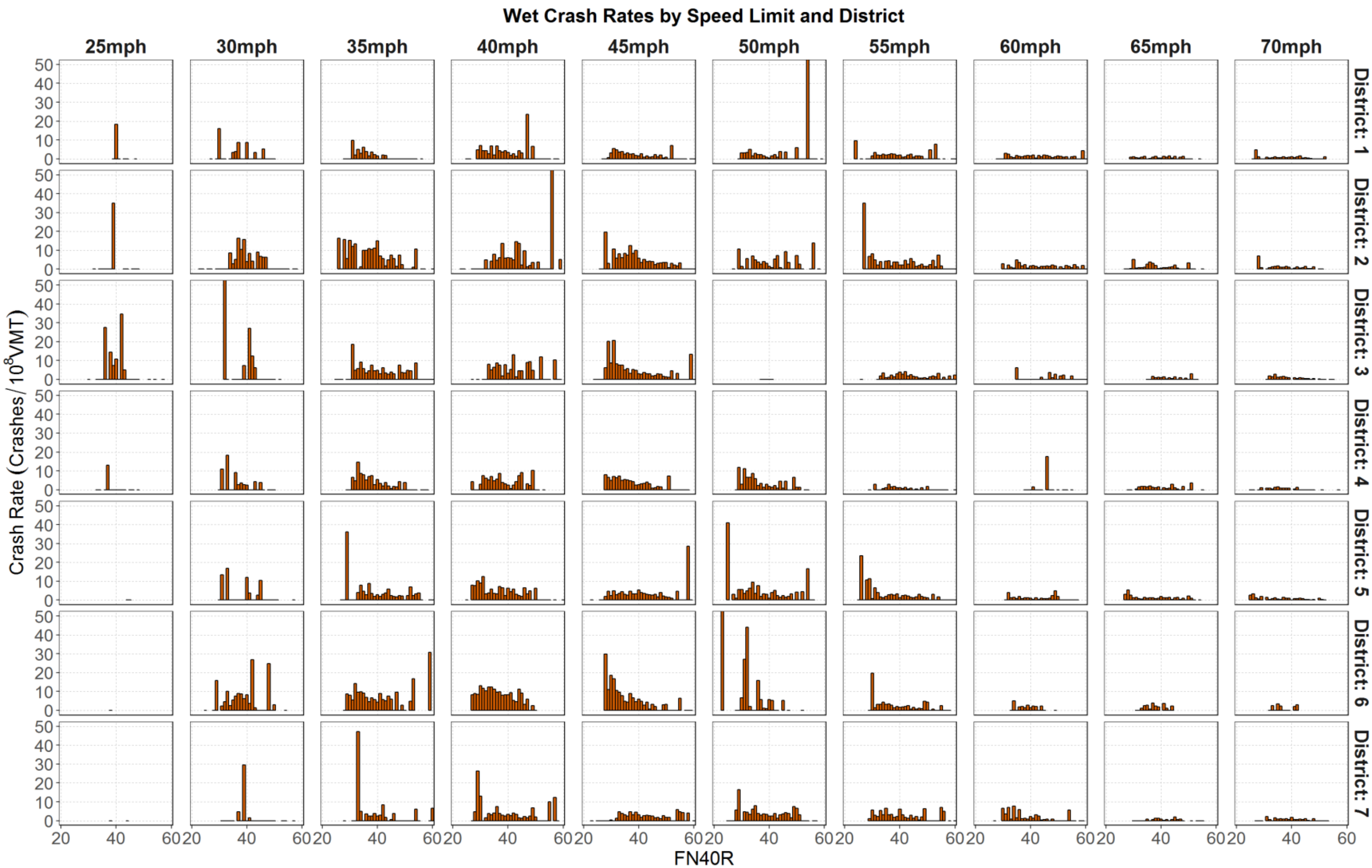


Figure 38. Wet Weather Crash Rate Distribution by Speed Limit and District (2011 – 2017)

FRICTION DEMAND CATEGORIES

As discussed previously, it was deemed necessary that the friction demand categories be established prior to assessing the necessary levels of friction. This is to ensure that there is sufficient amount of friction data as well as crash data for establishing the critical levels of friction.

Ideally, the friction demand categories should be defined while considering a number of factors such as speed limit, traffic levels, geographic region, highway functional class, crash severity, etc. However, having a large number of friction demand categories is not practical nor effective for friction and pavement management purposes. Furthermore, many of these factors are related to each other to a certain degree (e.g., highway functional class is strongly related to speed limit) or are beyond FDOT's control (e.g., geographic region or District). As such, it was decided to consider two of the most basic but important factors, namely speed limit and traffic levels.

Speed Limit Category

As discussed previously, FDOT's current friction guideline is only based on speed limit (> 45 mph or ≤ 45 mph). However, Figures 37 and 38 showed that the crash rates at low speed facilities (25 mph and 30 mph) did not exhibit a clear trend. This may indicate that the crashes in these low speed facilities may be attributed to other unknown factors and that the necessary level of friction for these roadways may not be as high as those of having speed limits 45 mph or greater. Extrapolating this logic to high speed facilities, it can also be argued that the high speed facilities may require higher levels of friction than 45 mph roadways.

Based on the above discussion, it was deemed appropriate to divide FDOT's roadways into low, medium, and high speed facilities for fiction management. In order to determine the boundaries for the speed categories, the crash count distributions were regenerated with respect to speed limit and District for both dry and wet weather as shown in Figures 39 and 40, respectively. These figures also highlight the speed limit zones with the highest and second-highest number of crashes within each District. Regardless of the weather condition (and District to a certain degree), these figures clearly show that the majority of Florida's crashes occurred in 40 mph, 45 mph, and 55 mph zones.

In light of the above, the following speed categories are recommended for FDOT's friction management.

- Low Speed: Speed limit less than 40 mph.
- Medium Speed: Speed limits 40 mph through 50 mph.
- High Speed: Speed limit greater than 50 mph.

The recommended speed definition shown above will be used for the remainder of this report.

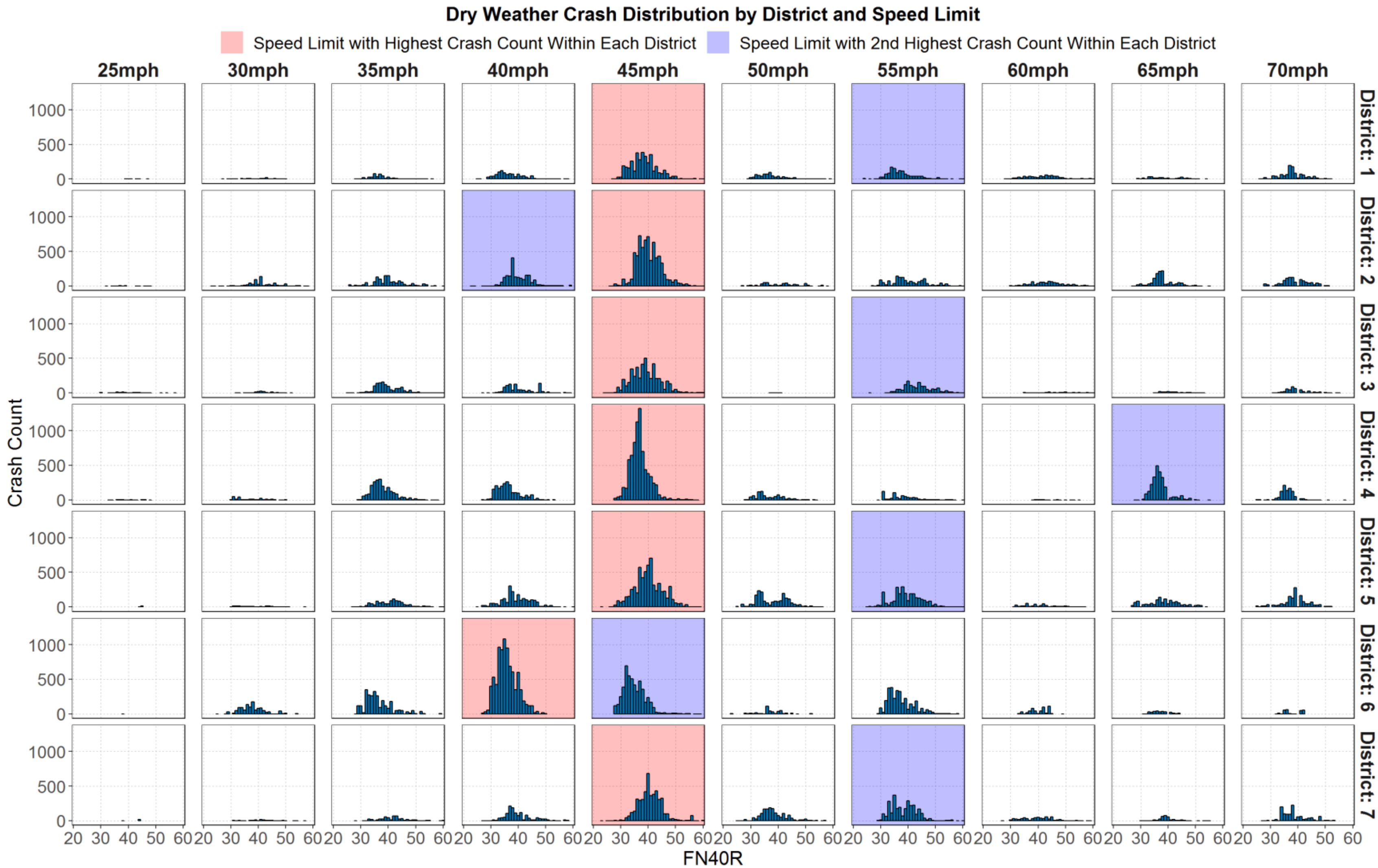


Figure 39. Dry Weather Crash Distribution by Speed Limit and District (2011 – 2017)

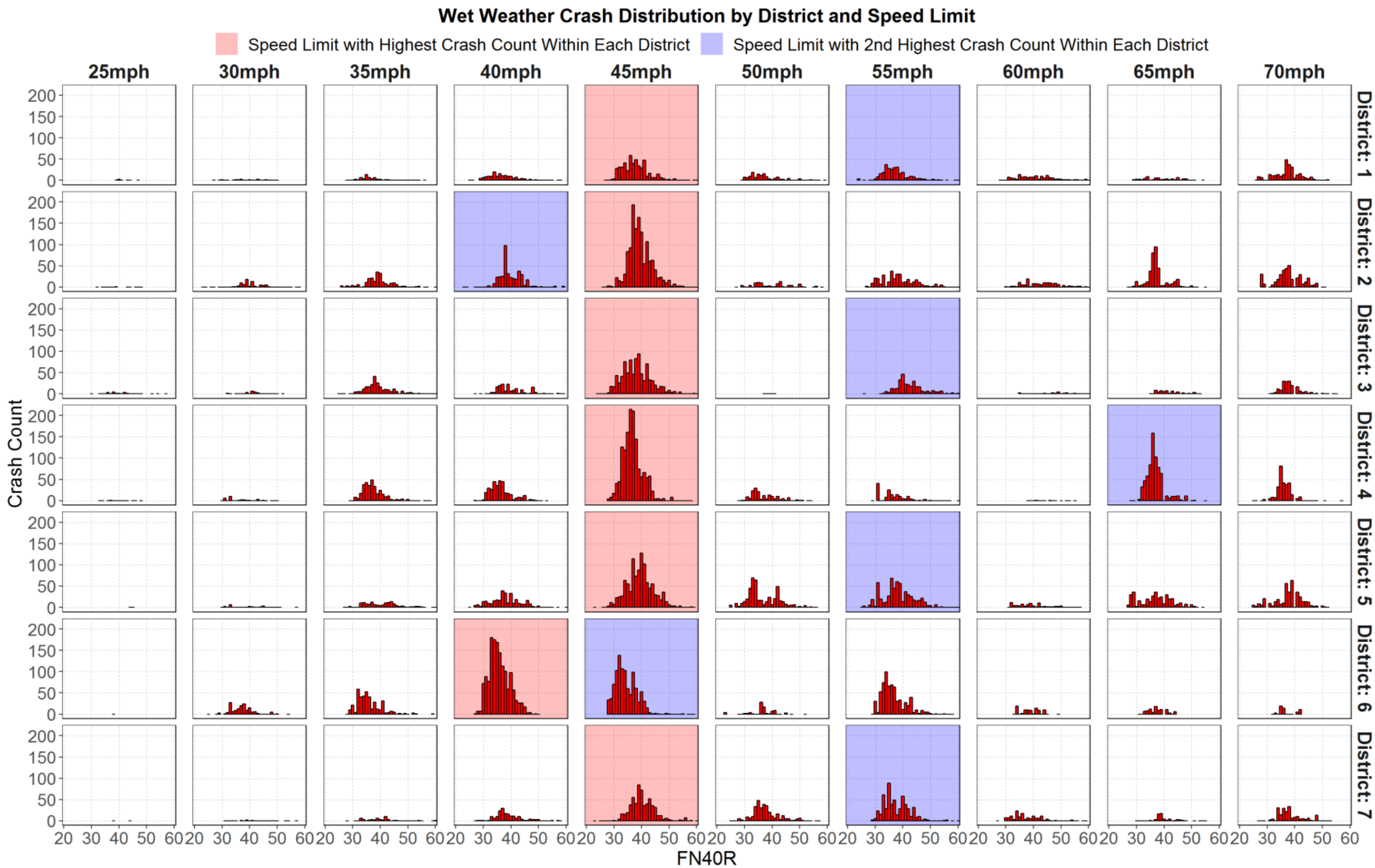


Figure 40. Wet Weather Crash Distribution by Speed Limit and District (2011 – 2017)

Traffic Level Category

Similar to the speed category, it is recommended that the level of traffic be categorized into low, medium, and high AADT for friction management. As such, Figure 41 shows the distribution and cumulative distribution of AADT for the three speed categories. This figure shows that in general, low speed facilities have lower traffic and the highly trafficked roadways are high speed facilities.

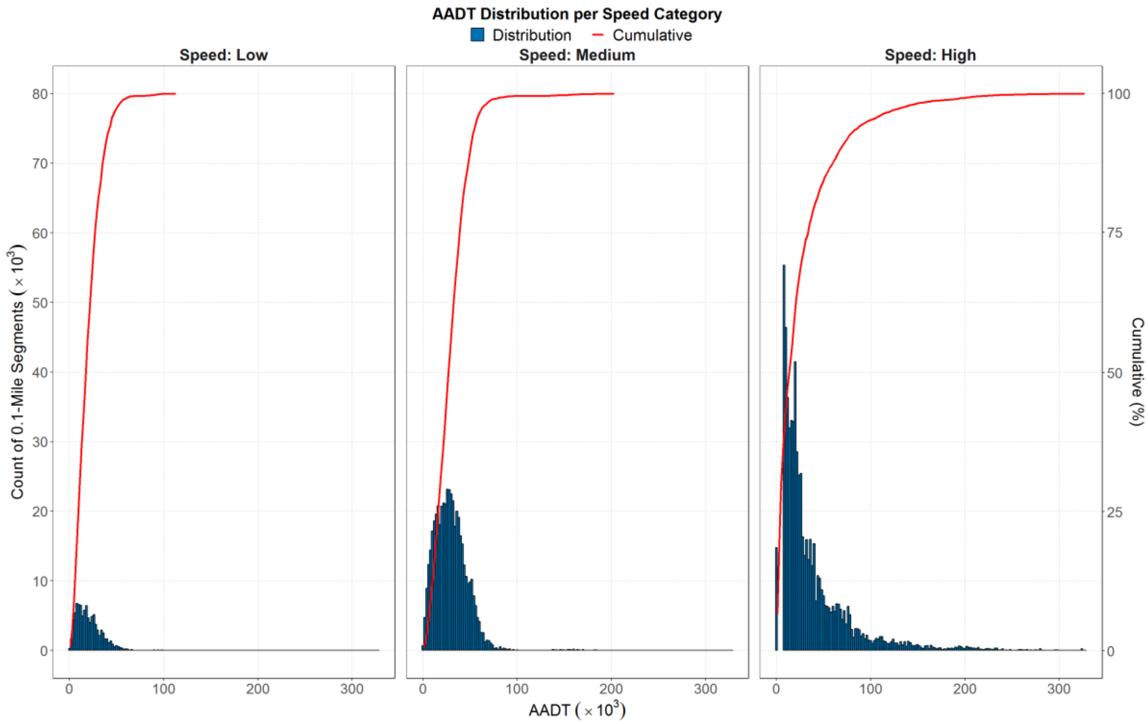


Figure 41. AADT Distribution for Different Speed Categories

Although the above AADT distributions can be used to determine the traffic levels for each speed category, an immediate limitation was encountered. The above distributions are not related to crash counts and hence, do not ensure that sufficient number of crashes are included for further analysis for each traffic level to be defined.

Therefore, rather than defining the traffic levels simply based on AADT alone, the crash count vs. AADT distribution was investigated for this purpose. Figures 42 and 43 show these distributions corresponding to low, medium, and high speed categories for dry and wet weather incidents, respectively. Also shown in the figures are the AADT corresponding to 33 and 66 percentiles of crash counts. For the low speed category as an example, 33 percent of dry weather crashes occurred at roadways with AADT less than 23K, another 33 percent between AADT of 23K and 35K, and the remaining 34 percent crashes occurred at roadways with AADT greater than 35K.

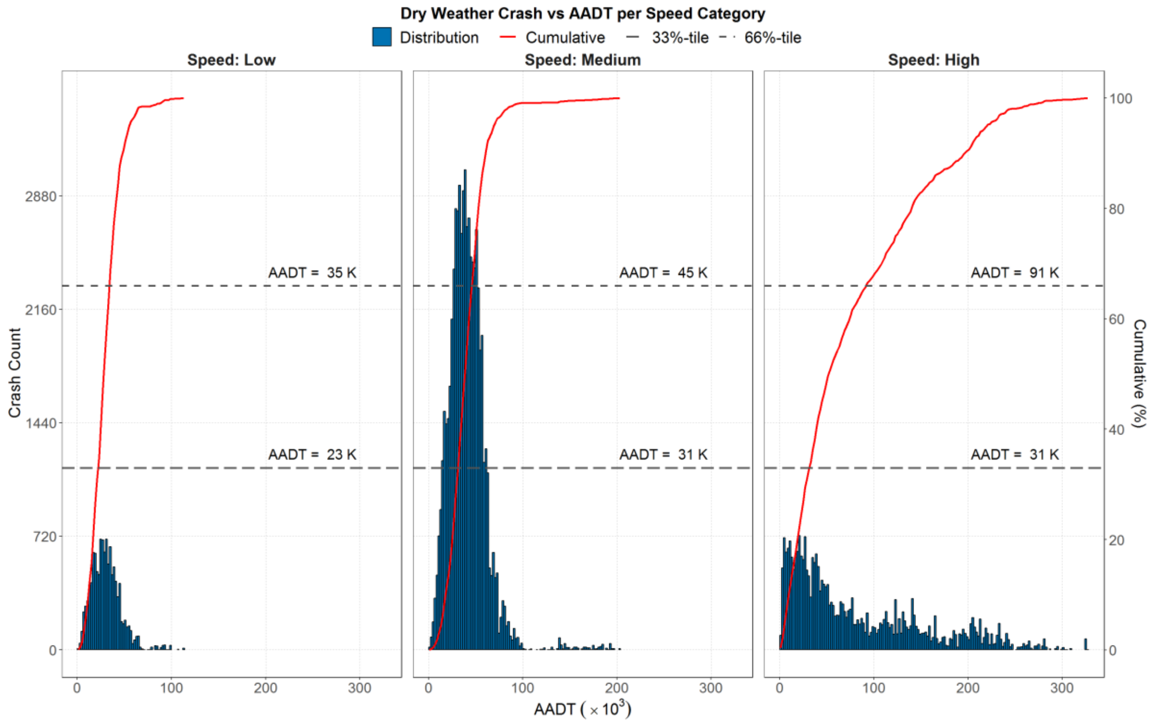


Figure 42. Dry Weather Crash Count vs AADT Distribution for Different Speed Categories

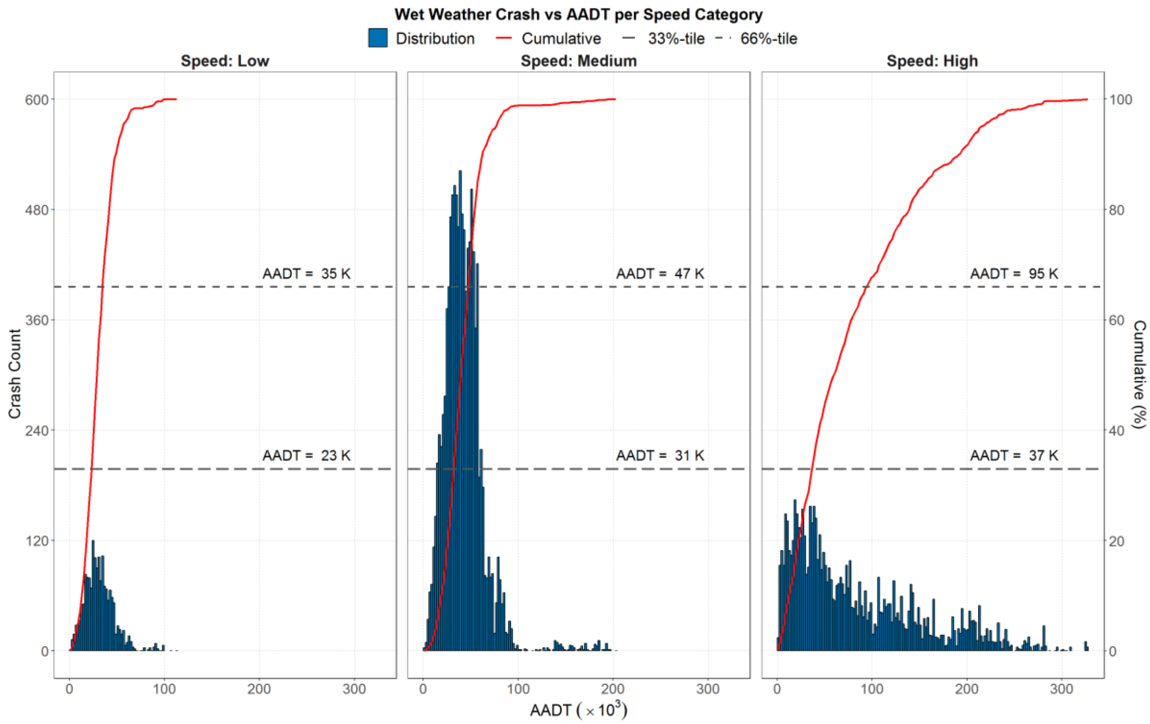


Figure 43. Wet Weather Crash Count vs AADT Distribution for Different Speed Categories

The AADT values corresponding to 33 and 66 percentiles of crash counts are summarized in Table 22 for both dry and wet weather crashes. The table indicates that these AADT values are not significantly different between dry and wet weather crashes. As such, the AADT category thresholds were determined as the lower value of the respective percentile (dry vs. wet) and rounding it down to the nearest 5K. These values are also shown in Table 22.

Table 22. AADT (×1,000) Values for 33 and 66 Percentiles of Crash

Speed Category	AADT (× 1000)					
	33%-tile		66%-tile		AADT Category Thresholds	
	Dry	Wet	Dry	Wet	Low/Medium	Medium/High
Low	23	23	35	35	20	35
Medium	31	31	45	47	30	45
High	31	37	91	95	30	90

Recommended Friction Demand Categories

By combining the speed and traffic categories determined above, Table 23 shows the final recommended friction demand categories for FDOT's friction management.

Table 23. Recommended Friction Demand Categories

Speed Category	AADT	
	Category	Value
Low (Speed Limit < 40 mph)	Low	AADT < 20 K
	Medium	20 K ≤ AADT < 35 K
	High	AADT ≥ 35 K
Medium (40 mph ≤ Speed Limit ≤ 50 mph)	Low	AADT < 30 K
	Medium	30 K ≤ AADT < 45 K
	High	AADT ≥ 45 K
High (Speed Limit > 50 mph)	Low	AADT < 30 K
	Medium	30 K ≤ AADT < 90
	High	AADT ≥ 90 K

Figure 44 shows the crash distributions and the W/D ratios corresponding to each friction demand category defined above. The figure shows that although the crash data is still limited for the low speed category (compared to medium and high speed categories), all W/D ratios are showing a relatively steady trend for FN40R between 30 and 45 where most of the data is made available. Nonetheless, the W/D ratios outside this FN40R range are still subject to noise (again due to lack of data). As such, it is necessary that a statistical model be developed to minimize the effect of such noise in determining the required levels of friction.

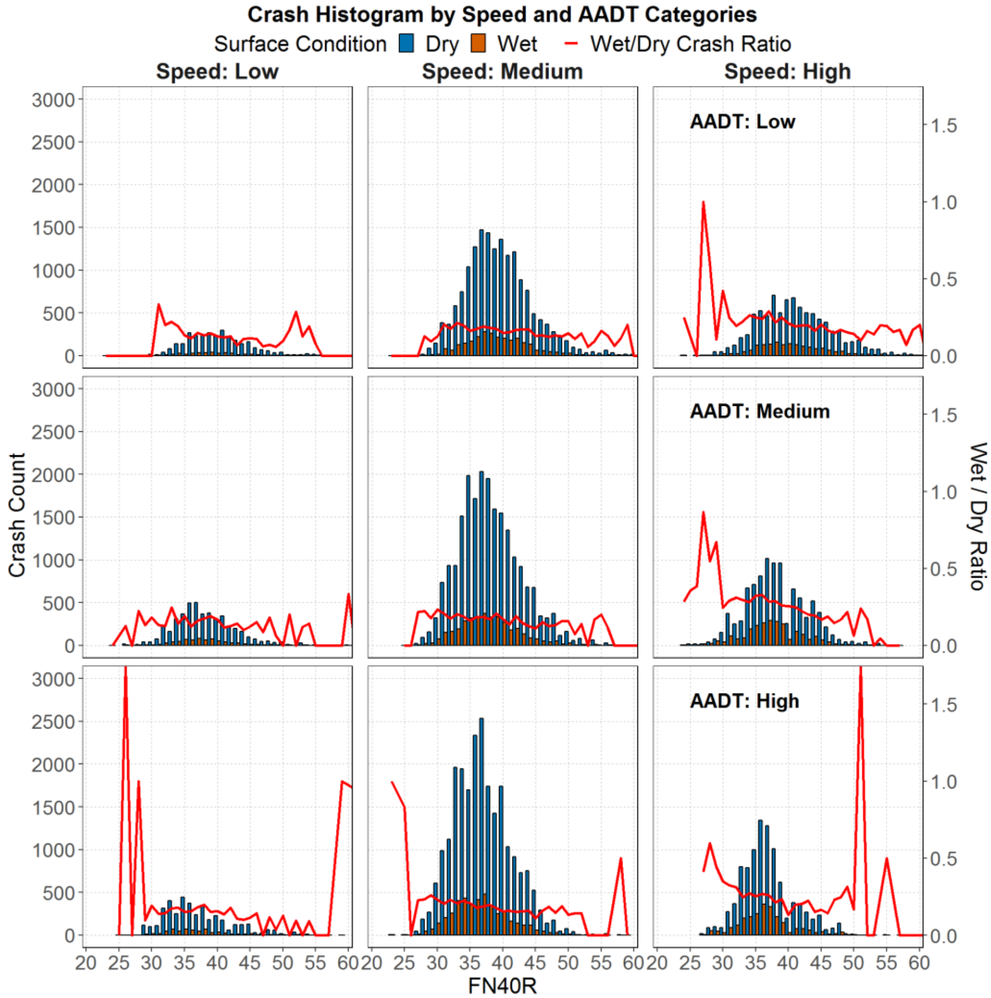


Figure 44. Distribution of Crash vs. FN40R for Different Friction Demand Categories

Similarly, Figures 45 and 46 show the dry and wet weather crash rates calculated for all friction demand categories, respectively. Both these figures clearly show more reasonable and intuitive trends than those previously shown in Figures 37 and 38, especially for low and medium speed categories. In addition, these figures indicate that although the overall crash counts were lower (Figure 44), the low speed facilities may also benefit from friction restoration and management. On the other hand, the trends shown for the high speed facilities are not as clear suggesting that a statistical model may be needed to better understand the friction benefits.

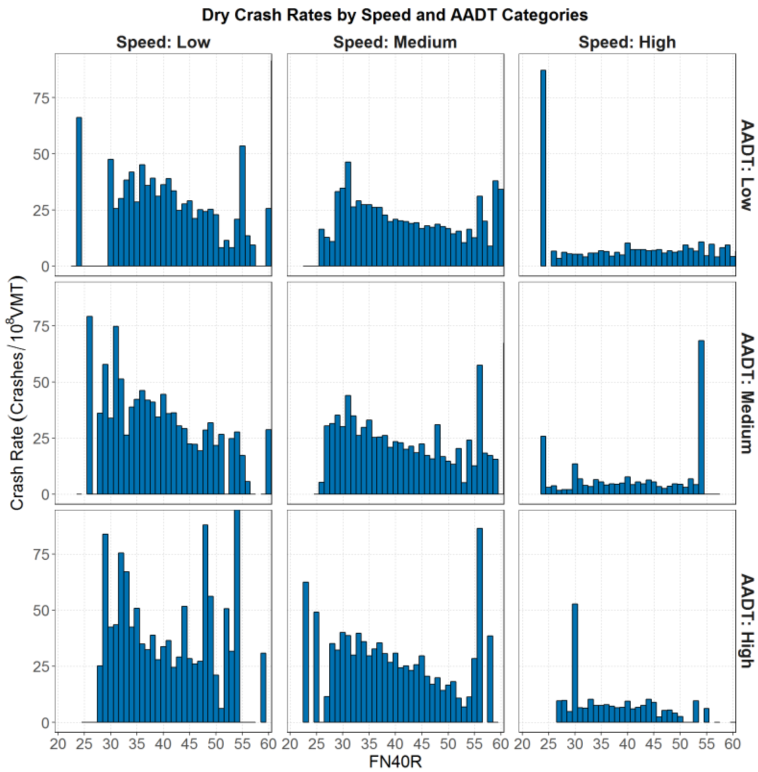


Figure 45. Dry Weather Crash Rates for Different Friction Demand Categories

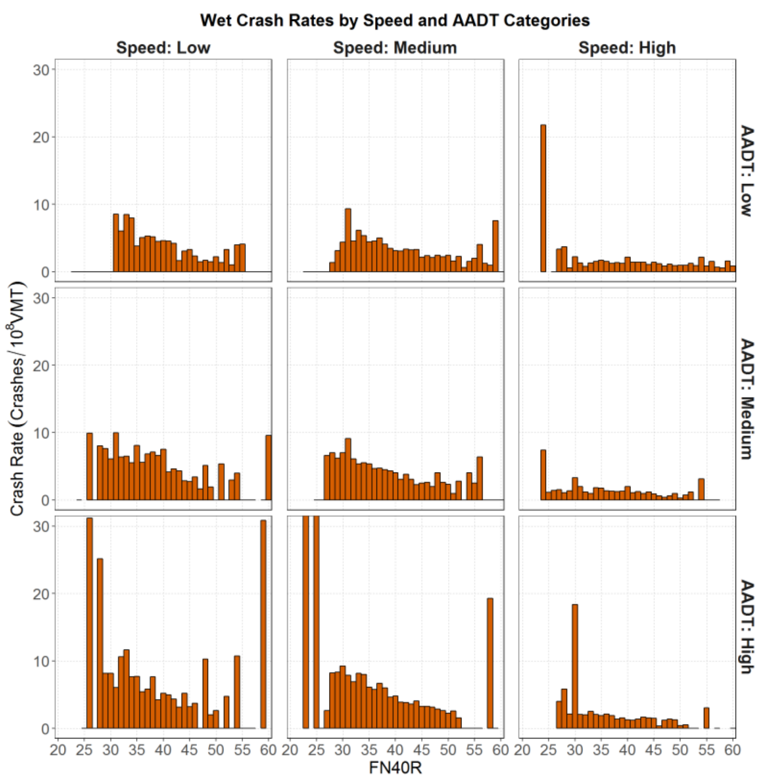


Figure 46. Wet Weather Crash Rates for Different Friction Demand Categories

SAFETY PERFORMANCE FUNCTION DEVELOPMENT

As recommended by FHWA and AASHTO, the statistical model for FDOT's crash data has been developed through the concept of Safety Performance Function (SPF) as well as the use of Negative Binomial (NB) and Empirical Bayes (EB) methods (Hauer et. al., 2002; Srinivasan and Bauer, 2013). Recall that the SPF in its most basic form is given as the following equation (Srinivasan and Bauer, 2013).

$$\mu = L \cdot e^{\beta_0 + \beta_1 \cdot \ln(AADT) + \sum \beta_i \cdot X_i} \tag{13}$$

where μ is the expected number of crashes, L is the segment length, $AADT$ is the annual average daily traffic, β_0, β_1, and β_i are regression coefficients, and X_i's are additional variables that may be used for developing the model.

<u>Safety Performance Function: Flexible Pavements</u>

As discussed, a primary objective of this study was to develop a statistical model for FDOT's crashes that incorporates pavement surface characteristics (i.e., friction and texture) as well as other pavement (or roadway) related factors. Therefore, the following variables from the integrated database were used for both the dry and wet weather SPFs:

- Pavement Variables
  - Friction (FN40R)
  - Pavement Texture (Mean Profile Depth [MPD], in inches)
  - Crack Rating
  - Rut Depth (in inches)
  - International Roughness Index (IRI, in in/mi)
  - Pavement Age (in years)
- Roadway Variables
  - AADT (2-Way)
  - Segment Length (in miles)
  - Pavement Marking Retroreflectivity (from MRU, in units of mcd/m2/lux)
  - Speed Limit (in mph)
  - Number of Lanes (per Direction)
- Material Variables
  - Aggregate Type (Granite or Limestone)
  - Mix Type (as defined in Table 18)
- Geographic & Functional Variables
  - District (D1 through D7, see Figure 22)
  - System Type (Primary, Toll, and Interstate)

SPF Including All Variables

Mathematically, the SPF including all of the above independent variables is written in terms of an exponential function given as the following.

$$\mu = L^{\beta_l} \cdot e^{\beta_0 + \beta_a \cdot \ln(AADT) + \beta_f \cdot FN40R + \beta_t \cdot Texture + \beta_s \cdot Speed + f(\beta)} \tag{14}$$

in which the function *f*(*β*) is written as:

$$\begin{aligned} f(\beta) = {} & \beta_m \cdot Marking + \beta_i \cdot IRI + \beta_r \cdot Rut + \beta_c \cdot Crack \\ & + \beta_L \cdot Lanes + \beta_A * Age + \beta_{District} + \beta_{Agg} + \beta_{System} + \beta_{MixType} \end{aligned} \tag{15}$$

Using the R statistical package, NB regression was carried out to fit the above function to the data in the integrated database (see Appendix A for the R summary reports).

In order to determine the significance of each independent variable or the variables having a significant effect on crashes, the NB regression was repeated by eliminating one of the input variables at a time. Then, the two NB models (with and without the variable in question) were compared by means of hypothesis testing. The null hypothesis is that the two NB models are statistically equivalent, which means that the particular variable does not have a significant effect on the original NB model. The alternate hypothesis is that the two NB models are statistically different, which means that the input variable does have a significant effect in the NB model and should not be eliminated.

Table 24 highlights the significant variables from the regression along with the p-values associated with each of the variables, while Tables 25 through 29 show the coefficients determined for Equations (14) and (15). Table 24 also highlights the variables that were determined to have a significant effect on crash counts, based on a significance level of 0.05. If the p-value shown in the table is less than 0.05, it indicates that there is less than 5 percent chance that the two NB models (with and without that particular variable) are equivalent and the null hypothesis should be rejected.

As an example, the p-value corresponding to FN40R was found to be less than 2×10^{-16}, for both dry and wet weather crashes. This means there is almost no chance that the NB model with all variables included is equivalent to the NB model without FN40R, i.e., FN40R has a very significant effect on crashes.

On the other hand, the p-value for Crack Rating was found to be 0.387 for wet weather NB model. This indicates that there is more than 5 percent chance that the two NB models (with and without Crack Rating) are equivalent, i.e., eliminating Crack Rating did not have a significant effect on the NB model and hence, this is not a significant variable.

Table 24. Significant Variables from NB Regression [Eq. (14) and (15)]

Variable Category	Variable	Significant Variables* (p-value)	
		Dry Weather	Wet Weather
N/A	Intercept	S (< 2e-16)	S (< 2e-16)
Pavement Variables	Friction (FN40R)	S (< 2e-16)	S (< 2e-16)
	Pavement Texture	S (4.3e-4)	NS (0.147)
	Crack Rating	NS (0.238)	NS (0.387)
	Rut Depth	S (0.005)	NS (0.842)
	IRI	S (< 2e-16)	S (< 2e-16)
	Pavement Age	S (4.4e-13)	NS (0.338)
Roadway Variables	AADT	S (< 2e-16)	S (< 2e-16)
	Segment Length	S (< 2e-16)	S (< 2e-16)
	Retroreflectivity	NS (0.272)	NS (0.575)
	Speed Limit	S (< 2e-16)	S (7.0e-4)
	Number of Lanes	S (< 2e-16)	S (3.4e-9)
Material Variables	Aggregate Type	S (6.1e-5)	NS (0.075)
	Mix Type	S (< 2e-16)	S (2.3e-6)
Geographic & Functional Variables	District	S (< 2e-16)	S (1.8e-6)
	System Type	S (< 2e-16)	S (3.8e-4)

Note*: S = Significant Factor, NS = Not Significant Factor, based on significance level of 0.05.

Table 24 shows that for dry weather, all variables except for Crack Rating and Pavement Marking Retroreflectivity were found to have significant effects on crashes. For wet weather crashes, a less number of variables were found to be significant. More specifically, Pavement Texture and Aggregate Type were found to be insignificant based on a significance level of 0.05. It is believed that this may be an artifact of having less number of wet weather crashes (approximately 5 times less than dry weather crashes). As such, for practical reasons, these variables were not eliminated in any of the subsequent analyses.

Table 25. NB Coefficients for Numerical Variables [Eq. (14) and (15)]

Coefficient	Related Variable	Dry	Wet
β_0	Intercept	-11.407	-12.745
β_l	Length	1.164	0.906
β_a	AADT	1.456	1.417
β_f	FN40R	-0.039	-0.056
β_t	Texture	-9.342	-7.521
β_s	Speed	-0.025	-0.016
β_m	Marking	-1.0×10^{-4}	-1.1×10^{-4}
β_i	IRI	0.008	0.007
β_r	Rut Depth	-0.523	0.073
β_c	Crack	0.014	-0.020
β_L	No. of Lanes	-0.348	-0.300
β_A	Age	0.017	0.005

Table 26. NB Coefficients for FDOT's District [Eq. (14) and (15)]

District	Dry	Wet
1	0.000	0.000
2	-0.132	0.189
3	-0.260	-0.113
4	0.107	0.176
5	0.132	0.323
6	0.522	0.426
7	0.369	0.403

Table 27. NB Coefficients for Categorical Variables: Aggregate [Eq. (14) and (15)]

Aggregate Type	Dry	Wet
Granite	0.000	0.000
Limestone	-0.233	-0.203

Table 28. NB Coefficients for Categorical Variables: System [Eq. (14) and (15)]

System	Dry	Wet
Primary	0.000	0.000
Toll	-0.662	-0.330
Interstate	-0.523	-0.470

Table 29. NB Coefficients for Categorical Variables: Mix Type [Eq. (14) and (15)]

Mix Family	Mix Type	Dry	Wet
FC5 (Open)	FC5	-0.026	0.094
	FC5A	-0.834	-0.258
	FC5AW	-0.727	-31.495
	FC5M	-0.071	-0.155
	FC5MW	-0.282	-0.390
FC95 (Dense)	FC95	0.164	-0.033
	FC95A	-0.055	-0.121
	FC95AR	-0.875	-33.621
	FC95M	0.056	-0.354
	FC95MR	0.546	0.219
	FC95MWR	1.016	1.308
	FC95R	0.456	0.120
FC125 (Dense)	FC125	0.000	0.000
	FC125A	-0.369	-0.939
	FC125AR	-0.284	-1.160
	FC125M	0.156	0.012
	FC125MR	0.413	0.083
	FC125MWR	0.534	-0.668
	FC125R	-0.030	-0.309
	FC125WR	0.518	1.660

Although the SPF including all available variables may be useful, it is recognized that Equations (14) and (15) may become cumbersome due to the large number of variables and coefficients associated with them. Furthermore, it is possible that FDOT may not have all necessary data to use these equations for future studies (e.g., safety analysis before/after friction restoration). Therefore, it was deemed useful to simplify the SPF further. The following sections document the SPFs simplified in different aspects.

SPF Including Significant Variables Only

The first attempt to simplify the SPF was to eliminate the variables that were found to be insignificant. As seen from Table 24, Crack Rating and Marking Retroreflectivity were found to have insignificant effects on both dry and wet weather crashes, and hence eliminated from regression. It is not clear as to why Crack Rating was not found to be a significant factor but it is possible that the influence of cracks (if any) may be reflected through other pavement condition indices such as IRI. On the other hand, it is believed that Reflectivity was found to be insignificant because the crashes were not distinguished for daytime vs nighttime. I.e., the effect of retroreflectivity may have a more significant effect on nighttime crashes rather than daytime – this effect was not captured in this study.

In addition to the above, Pavement Age and Rut Depth variables were eliminated for wet weather crash regression due to the large p-values associated with them. Similar to the Crack Rating, the influence of Pavement Age may have been reflected through other variables. As for the Rut Depth, it is possible that the insignificant amount of rut (mostly less than 0.15 in. in the integrated database) may have caused challenges in regression.

After eliminating the above variables, the SPF is re-written as:

$$\mu = L^{\beta_l} \cdot e^{\beta_0 + \beta_a \cdot \ln(AADT) + \beta_f \cdot FN40R + \beta_t \cdot Texture + \beta_s \cdot Speed + f(\beta)} \tag{16}$$

where

$$f(\beta) = \beta_i \cdot IRI + \beta_r \cdot Rut + \beta_L \cdot Lanes + \beta_A * Age + \beta_{District} + \beta_{Agg} + \beta_{System} + \beta_{MixType} \tag{17}$$

Tables 30 through 34 show the NB coefficients corresponding to Equations (16) and (17).

Table 30. NB Coefficients for Numerical Variables [Eq. (16) and (17)]

Coefficient	Related Variable	Dry	Wet
β_0	Intercept	-11.290	-12.914
β_l	Length	1.159	0.890
β_a	AADT	1.457	1.415
β_f	FN40R	-0.038	-0.057
β_t	Texture	-9.200	-8.192
β_s	Speed	-0.025	-0.016
β_i	IRI	0.008	0.007
β_r	Rut Depth	-0.541	0.000*
β_L	No. of Lanes	-0.346	-0.299
β_A	Age	0.015	0.000*

Note*: These coefficients are "Zero" because the associated variables were not included in wet weather regression.

Table 31. NB Coefficients for FDOT's District [Eq. (16) and (17)]

District	Dry	Wet
1	0.000	0.000
2	-0.138	0.202
3	-0.271	-0.094
4	0.108	0.164
5	0.127	0.330
6	0.518	0.430
7	0.368	0.406

Table 32. NB Coefficients for Categorical Variables: Aggregate [Eq. (16) and (17)]

Aggregate Type	Dry	Wet
Granite	0.000	0.000
Limestone	-0.236	-0.199

Table 33. NB Coefficients for Categorical Variables: System [Eq. (16) and (17)]

System	Dry	Wet
Primary	0.000	0.000
Toll	-0.665	-0.321
Interstate	-0.523	-0.466

Table 34. NB Coefficients for Categorical Variables: Mix Type [Eq. (16) and (17)]

Mix Family	Mix Type	Dry	Wet
FC5 (Open)	FC5	-0.024	0.102
	FC5A	-0.851	-0.203
	FC5AW	-0.708	-31.476
	FC5M	-0.071	-0.144
	FC5MW	-0.310	-0.362
FC95 (Dense)	FC95	0.170	-0.052
	FC95A	-0.061	-0.066
	FC95AR	-0.885	-33.591
	FC95M	0.060	-0.345
	FC95MR	0.552	0.202
	FC95MWR	1.015	1.279
	FC95R	0.450	0.115
FC125 (Dense)	FC125	0.000	0.000
	FC125A	-0.387	-0.865
	FC125AR	-0.296	-1.134
	FC125M	0.154	0.013
	FC125MR	0.414	0.070
	FC125MWR	0.535	-0.707
	FC125R	-0.027	-0.333
	FC125WR	0.517	1.641

SPF Including Significant Variables and Mix Family Information Only

As seen in Tables 29 and 34, there is a lot of mix types that have been included in the integrated database. However, as evidenced by Figures 30 through 32, not all mix types have sufficient data. Therefore, it was determined that the SPF based on mix families (rather than mix types) may be beneficial. All other variables are kept the same as the previous SPF. The SPF is written as:

$$\mu = L^{\beta_l} \cdot e^{\beta_0 + \beta_a \cdot \ln(AADT) + \beta_f \cdot FN40R + \beta_t \cdot Texture + \beta_s \cdot Speed + f(\beta)} \tag{18}$$

where

$$f(\beta) = \beta_i \cdot IRI + \beta_r \cdot Rut + \beta_L \cdot Lanes + \beta_A * Age + \beta_{District} + \beta_{Agg} + \beta_{System} + \beta_{MixFamily} \tag{19}$$

The resulting NB coefficients are summarized in Tables 35 through 39.

Table 35. NB Coefficients for Numerical Variables [Eq. (18) and (19)]

Coefficient	Related Variable	Dry	Wet
β_0	Intercept	-11.197	-12.777
β_l	Length	1.130	0.941
β_a	AADT	1.471	1.429
β_f	FN40R	-0.038	-0.061
β_t	Texture	-12.058	-11.288
β_s	Speed	-0.025	-0.015
β_i	IRI	0.008	0.007
β_r	Rut Depth	-0.897	0.000*
β_L	No. of Lanes	-0.362	-0.330
β_A	Age	0.014	0.000*

Note*: These coefficients are "Zero" because the associated variables were not included in wet weather regression.

Table 36. NB Coefficients for FDOT's District [Eq. (18) and (19)]

District	Dry	Wet
1	0.000	0.000
2	-0.089	0.223
3	-0.184	0.015
4	0.033	0.197
5	0.069	0.318
6	0.391	0.444
7	0.300	0.392

Table 37. NB Coefficients for Categorical Variables: Aggregate [Eq. (18) and (19)]

Aggregate Type	Dry	Wet
Granite	0.000	0.000
Limestone	-0.303	-0.210

Table 38. NB Coefficients for Categorical Variables: System [Eq. (18) and (19)]

System	Dry	Wet
Primary	0.000	0.000
Toll	-0.579	-0.421
Interstate	-0.526	-0.592

Table 39. NB Coefficients for Categorical Variables: Mix Family [Eq. (18) and (19)]

Mix Family	Dry	Wet
FC5	-0.097	0.158
FC95	0.150	0.014
FC125	0.000	0.000

SPF without Geographic Input

All of the SPFs developed above included the geographical input in terms of FDOT's District. In the last SPF to be presented below, this geographic input has been eliminated. This SPF may be useful when FDOT intends to study the crash predictions on a statewide basis, without any discrepancies in District. All other variables from the previous SPF were included. The SPF is now written as:

$$\mu = L^{\beta_l} \cdot e^{\beta_0 + \beta_a \cdot \ln(AADT) + \beta_f \cdot FN40R + \beta_t \cdot Texture + \beta_s \cdot Speed + f(\beta)} \tag{20}$$

where

$$f(\beta) = \beta_i \cdot IRI + \beta_r \cdot Rut + \beta_L \cdot Lanes + \beta_A * Age + \beta_{Agg} + \beta_{System} + \beta_{MixFamily} \tag{21}$$

Tables 40 through 43 summarize the NB coefficients for the SPF shown in Equations (20) and (21).

Table 40. NB Coefficients for Numerical Variables [Eq. (20) and (21)]

Coefficient	Related Variable	Dry	Wet
β_0	Intercept	-11.778	-12.917
β_l	Length	1.158	0.972
β_a	AADT	1.544	1.477
β_f	FN40R	-0.036	-0.058
β_t	Texture	-16.330	-14.393
β_s	Speed	-0.026	-0.017
β_i	IRI	0.009	0.007
β_r	Rut Depth	-0.914	0.000*
β_L	No. of Lanes	-0.390	-0.342
β_A	Age	0.014	0.000*

Note*: These coefficients are "Zero" because the associated variables were not included in wet weather regression.

Table 41. NB Coefficients for Categorical Variables: Aggregate [Eq. (20) and (21)]

Aggregate Type	Dry	Wet
Granite	0.000	0.000
Limestone	-0.182	-0.173

Table 42. NB Coefficients for Categorical Variables: System [Eq. (20) and (21)]

System	Dry	Wet
Primary	0.000	0.000
Toll	-0.637	-0.401
Interstate	-0.611	-0.628

Table 43. NB Coefficients for Categorical Variables: Mix Family [Eq. (20) and (21)]

Mix Family	Dry	Wet
FC5	0.144	0.320
FC95	0.132	0.002
FC125	0.000	0.000

<u>Safety Performance Function: Rigid Pavements</u>

Due to the inherent differences in the type of distresses measured for rigid pavements, the SPF for concrete surfaces has been developed separately. It is also noted that due to the limited number of rigid pavements in Florida, there were not sufficient number of crashes to develop an SPF with all possible input variables. More specifically, the number of 0.1-mile segments included in the integrated database was 9,830, with a total of 2,723 crashes (2,346 dry weather and 377 wet weather crashes). Therefore, the independent variables that could be used for SPF development were limited as shown below.

- Pavement Variables
  - Friction (FN40R)
  - Percent Cracked Slabs
  - Faulting (in inches)
  - International Roughness Index (IRI, in in/mi)
- Roadway Variables
  - AADT (2-Way)
  - Segment Length (in miles)
  - Speed Limit (in mph)
  - Number of Lanes (per Direction)
- Functional Variables
  - System Type (Primary, Toll, and Interstate)

The SPF including the above input variables is given as:

$$\mu = L^{\beta_l} \cdot e^{\beta_0 + \beta_a \cdot \ln(AADT) + \beta_f \cdot FN40R + \beta_S \cdot Speed + f(\beta)} \tag{22}$$

where

$$f(\beta) = \beta_i \cdot IRI + \beta_F \cdot Fault + \beta_L \cdot Lanes + \beta_S * CrackedSlabs + \beta_{System} \tag{23}$$

Table 44 summarize the p-values obtained from NB regression for all variables while Tables 45 and 46 show the regression coefficients. Table 44 show that Faulting and the Number of Lanes were not significant variables affecting both dry and wet weather crashes. In addition, Percent Cracked Slabs, IRI, and System were also determined to be insignificant for wet weather crashes. It should be noted that these outcomes should not be taken for granted. All these regression results for rigid pavements are subject to lack of data. Therefore, no further effort has been conducted to simplify the SPF for rigid pavements.

Table 44. Significant Variables from NB Regression [Eq. (22) and (23)]

Variable Category	Variable	Significant Variables* (p-value)	
		Dry Weather	Wet Weather
N/A	Intercept	S (< 2e-16)	S (1.7e-4)
Pavement Variables	Friction (FN40R)	S (< 2e-16)	S (1.6e-8)
	% Cracked Slabs	S (3.1e-11)	NS (0.003)
	Faulting	NS (0.804)	NS (0.641)
	IRI	S (4.0e-09)	NS (0.069)
Roadway Variables	AADT	S (< 2e-16)	S (9.4e-11)
	Segment Length	S (2.4e-05)	S (0.038)
	Speed Limit	S (3.7e-08)	S (1.5e-4)
	Number of Lanes	NS (0.96)	NS (0.434)
Functional Variable	System Type	S (8.8e-06)	NS (0.453)

Note*: S = Significant, NS = Not Significant, based on significance level of 0.05.

Table 45. NB Coefficients for Numerical Variables [Eq. (22) and (23)]

Coefficient	Related Variable	Dry	Wet
β_0	Intercept	-9.907	-9.570
β_l	Length	0.791	1.105
β_a	AADT	1.331	1.317
β_f	FN40R	-0.074	-0.095
β_s	Speed	-0.028	-0.038
β_i	IRI	0.005	0.003
β_F	Faulting	-0.653	-2.551
β_S	Percent Cracked Slabs	-0.399	-0.390
β_L	No. of Lanes	0.002	-0.058

Table 46. NB Coefficients for Categorical Variables: System [Eq. (22) and (23)]

System	Dry	Wet
Primary	0.000	0.000
Toll	-0.254	-0.705
Interstate	-0.836	-0.429

Empirical Bayes Method

As described in the first chapter of this report, FHWA recommends that the NB regression results be integrated with the observed number of crashes through the use Empirical Bayes (EB) method for estimating the statistically expected crash counts (Herbal et. al., 2010). An extensive review of the EB method shall not be repeated herein.

Mathematically, the EB method is written as the following:

$$EB_i = W_i \lambda_i + (1 - W_i) y_i \tag{24}$$

where EB_i is the expected crash count for section *i* estimated from the EB method, λ_i is the dispersion parameter obtained from NB regression, and W_i is the weight factor given as:

$$W_i = \frac{1}{1+\lambda_i \alpha} \tag{25}$$

The estimated crash counts from the EB method [Equation (24)] has a standard deviation calculated as the following.

$$\sigma_{EB,i} = \sqrt{(1-W_i)\cdot EB_i} \tag{26}$$

To demonstrate an example of the above statistic models, Figure 47 shows the wet weather crash counts (Observed, SPF prediction from Equation (14), and EB estimate from Equation (24)) for both travel directions from a roadway in south Florida. The figure also shows four different mix types along the roadway (FC5, FC95A, FC95, and FC125 from left to right). The areas with no mix type information indicate that one or more of the SPF inputs (e.g., texture) was missing and hence the crash count predictions could not be made.

The observed crash counts in Figure 47 generally show that (1) there were more crashes in the eastbound lane than in the westbound lane and (2) more crashes were observed in the FC95 and FC125 surfaces between mileposts 15 and 20. The figure also shows that the SPF predictions closely follow this trend which demonstrates the effectiveness of the SPF. The EB prediction is simply a weighted average of the observed and SPF predictions and hence it is closer to the observed crash counts.

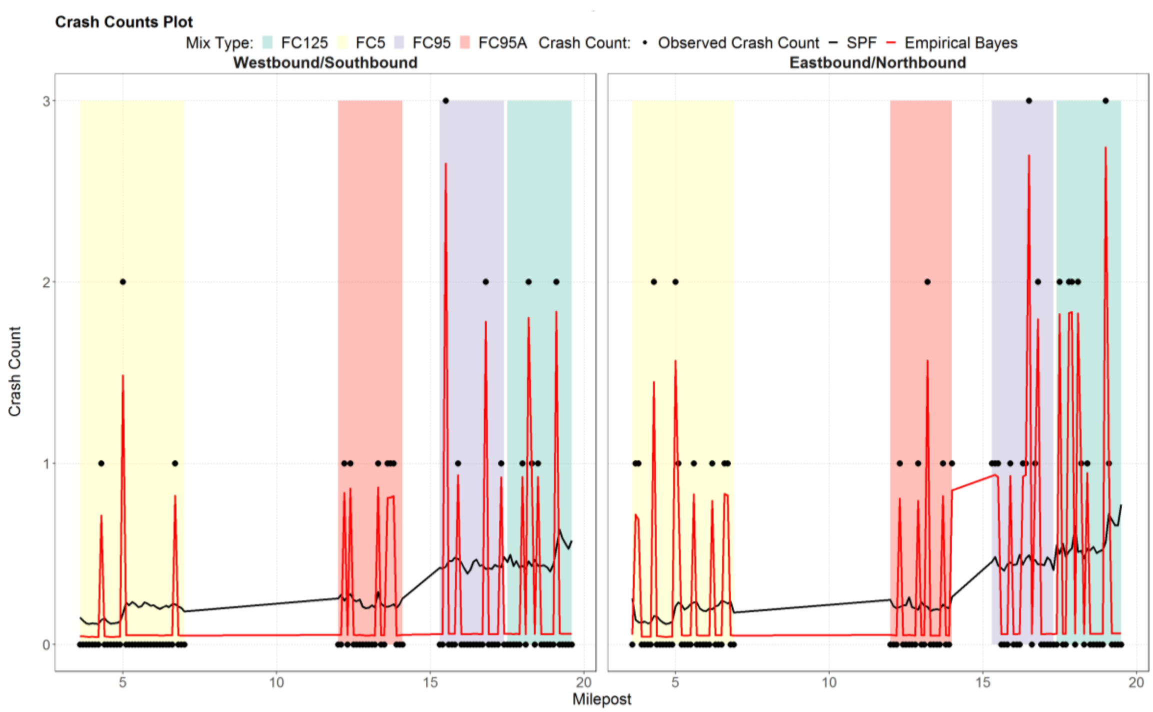


Figure 47. Observed Crash Count, SPF Prediction, and EB Crash Counts

To understand the cause of the increased SPF predictions between mileposts 15 and 20, Figure 48 shows the crash count plot as well as the plots of AADT and FN40R from the same pavement section. Although there are additional effects from other variables not shown in this figure (e.g., texture, IRI, etc.), the figure clearly shows that the AADT in the areas with higher crash counts (FC95 and FC125) is significantly higher or almost double the AADT of FC5 section with lower number of crashes. Furthermore, the FN40R values in the eastbound FC125 section is significantly lower than that of the westbound section, which may explain the increased number of crashes in the eastbound lanes.

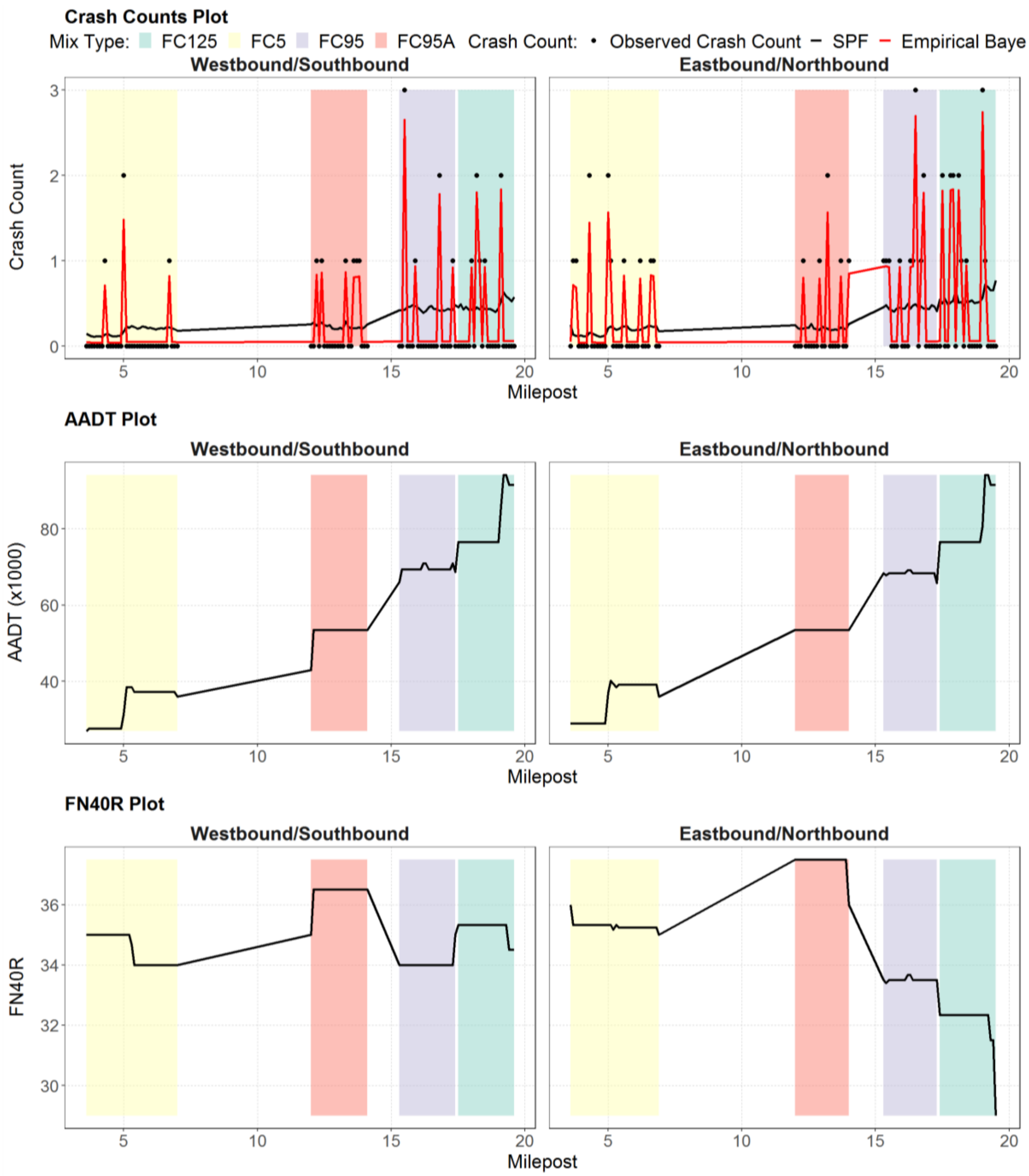


Figure 48. Crash Counts, AADT, and FN40R

The above example clearly demonstrates the effect of AADT and friction on wet weather crashes. Obviously, the crashes in the FC95 and FC125 surfaces may be reduced if AADT is reduced or if FN40R values were increased in these segments (note that the eastbound FN40R values for these segments are below 35). However, reducing the AADT is likely beyond FDOT's control for existing roadways. Adding additional lanes may alleviate the issue of higher traffic in these areas, but this option is usually very expensive and/or very difficult if this portion of the roadway is located in urban areas. Therefore, increasing the FN40R on these roadways may be a feasible option to reduce the crashes.

Based on the above, a hypothetical scenario was created in which the FN40R of this entire roadway was increased from its current value to a high value of 75 (say using a High Friction Surface Treatment). Figure 49 shows the comparison of the expected crash counts before and after this treatment. The figure clearly shows the benefit of this (hypothetical) treatment as seen by the reductions in the expected number of crashes.

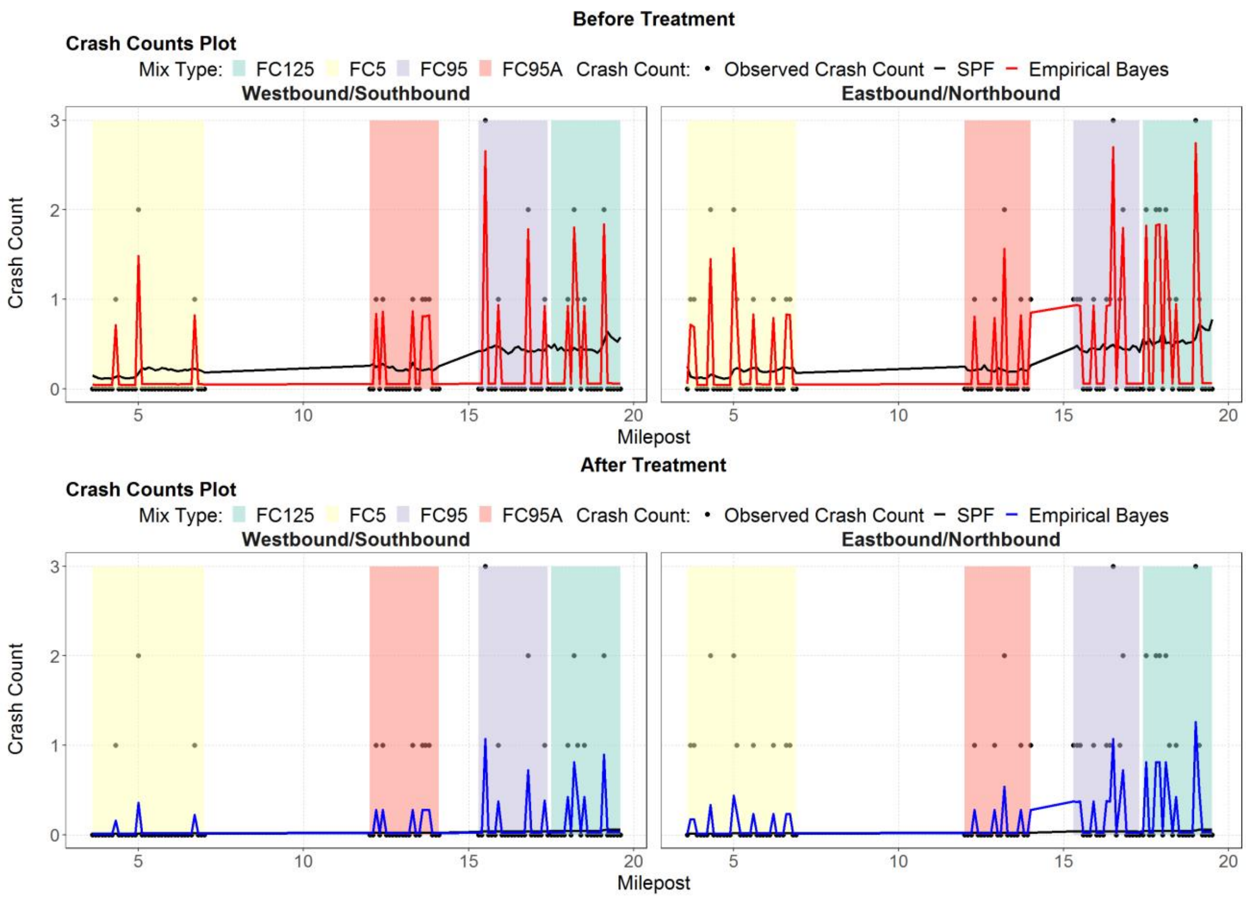


Figure 49. Estimated Crash Counts Before (Top) and After (Bottom) Friction Restoration

Network Level Crash Analysis

The previous example shown in Figures 47 through 49 clearly demonstrated that higher friction numbers may reduce crashes at a project (or roadway) level. However, it is also of interest to understand how FDOT's roadways may benefit from increased friction numbers at a network level. It is envisioned that such network level analysis would provide more insight towards the target friction numbers for the friction categories identified previously. As such, the hypothetical example was extended to the network level analysis. For this analysis, several what-if scenarios were simulated in which a minimum friction number was enforced and maintained for the roadway segments in the entire integrated database. More specifically, the following describes the what-if analysis procedure:

1. The minimum FN40R value to be enforced (this value will be referred to as "FN40R threshold" for this analysis) was gradually increased from 23 to 70. Note the minimum value of 23 was chosen as it was the lowest FN40R value available in the integrated database.
2. For a given FN40R threshold, identify all 0.1-mile segments with friction numbers below this threshold and replace the existing FN40R values with the threshold value (again, the assumption here is that the FN40R threshold is "enforced and maintained"). The segments with friction higher than the threshold remain unchanged.
3. Run the SPF [Equations (14) and (15) for flexible pavements and Equations (22) and (23) for rigid pavements] and EB [Equation (24)] models to get an updated number of expected crashes.
4. Repeat steps 2 and 3 for all FN40R thresholds.

Figure 50 shows how the crash counts are expected to reduce (per year) with increasing FN40R threshold for all 3 speed categories and all 3 traffic categories, while Figure 51 shows the expected crash reduction in terms of crash rates. Note that an FN40R threshold of 23 is equivalent to taking "no action at all" and retaining the current condition of FDOT's roadways as all of the friction numbers in the integrated database were greater than or equal to this value.

Figures 50 and 51 indicate that there is no noticeable reduction in the expected crash counts for FN40R threshold values below 35 (approximately). This is because FDOT's current friction guidelines require this level of friction for high speed facilities. Nevertheless, the figure clearly shows that FDOT may expect additional crash reduction as the minimum FN40R value is increased beyond the current required level of friction.

It is also emphasized that the number of crashes (or its reduction) shown in Figure 50 is only based on the crashes that were mapped back to the integrated database. Given that only 11 percent of all crashes in the crash database were mapped back to the integrated database for the analysis, the actual benefit (crash reduction) gained by maintaining and enforcing a higher FN40R threshold may be significantly higher than what is determined from Figure 50. For example, consider the dry weather crash curve in the medium speed, low AADT category. If the minimum required FN40R is increased from 35 to 45, then this curve shows a crash reduction of approximately 2,500 (from 17,500 to 15,000) for this category. This 2,500 crash reduction is based only on the 11 percent of the crash data included in the integrated database, and the actual crash reduction is anticipated to be much larger if the remaining 89 percent of crash data (that were not mapped back to the integrated database) are to be considered.

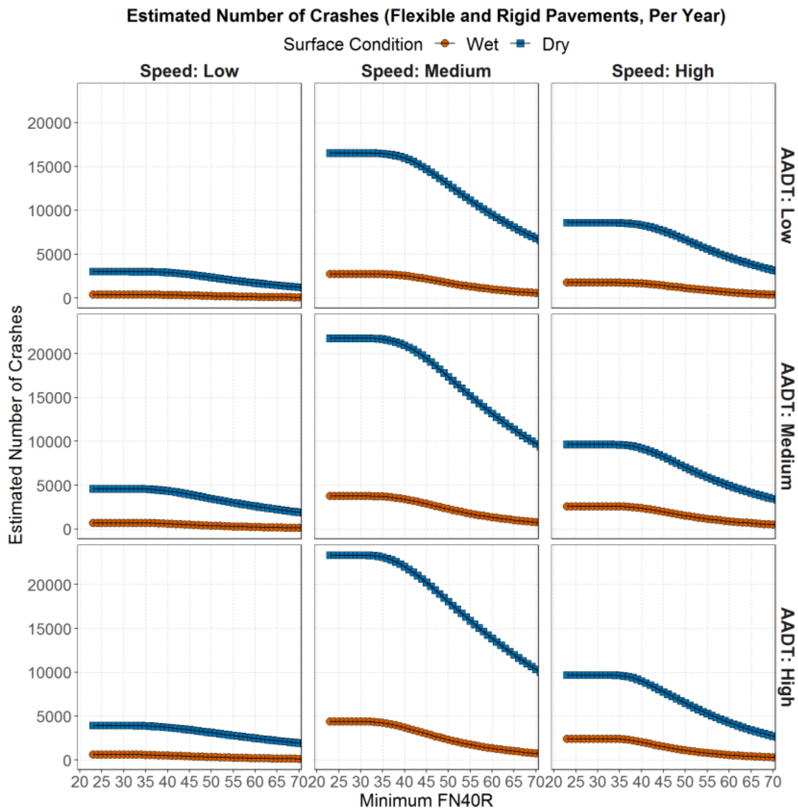


Figure 50. Estimated Crash Counts with Increasing FN40R Threshold

Figure 50 also shows that in general, more significant crash reduction is expected for dry weather crashes than wet weather, and the most significant reduction is seen in the medium speed category. This is simply because there are more dry weather crashes than wet weather and the majority of the crashes are located in this particular speed category (see Figure 44).

To show the relative effectiveness of the FN40R threshold, Figure 52 shows the Crash Modification Factors (CMF) obtained as the ratio of the expected crash count corresponding to a given FN40R threshold and the current crash counts. This figure clearly shows that wet weather crashes will benefit more from friction restoration than dry weather crashes, regardless of speed and traffic categories.

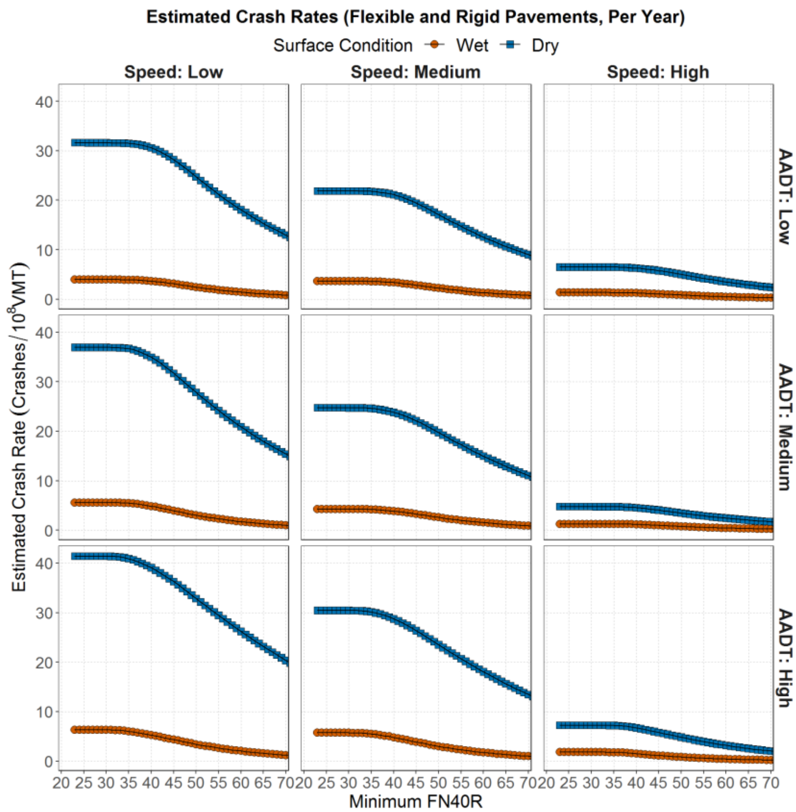


Figure 51. Estimated Crash Rates with Increasing FN40R Threshold

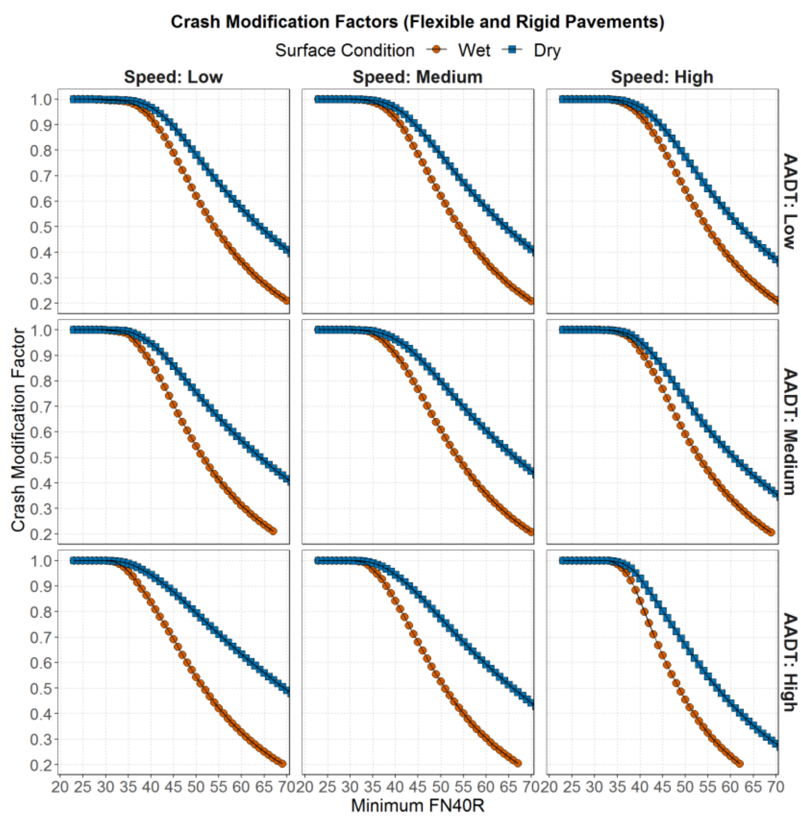


Figure 52. Estimated Crash Modification Factors with Increasing FN40R Threshold

RELIABILITY VS RISK ANALYSIS FOR OPTIMUM LEVELS OF FRICTION

Motivation

In the previous section of the report, the benefit of increasing the levels of friction on FDOT's roadways has been demonstrated in terms of CMF or estimated reduction in the number of crashes. The original research plan included a Benefit/Cost Analysis (BCA) for determining the optimum level of friction such that the benefit/cost ratio is maximized. Given the estimated crash reduction curves shown in Figure 50, the benefit could be quantified relatively easily by multiplying the average cost of crash by the estimated crash reduction per year.

However, the cost of achieving higher friction levels which was the other necessary piece of information for BCA could not be established, primarily because friction is not included in FDOT's pay item. Stated differently, although the initial level of friction shows a relatively wide range (see Figures 29 through 32), FDOT's payment to the contractor is not affected by the level of friction achieved. I.e., whether the achieved FN40R is 40, 50, or 60, the contractor gets paid based on tonnage and other incentive/disincentive specifications that do not include friction. Therefore, it was impossible to differentiate FDOT's cost associated with achieving an initial FN40R of 40 vs. 50 vs. 60, etc.

Due to the challenges associated with assessing the cost for different levels of friction, the BCA was eliminated from the research plan and replaced with a reliability-risk analysis (which will be referred to as "Risk Analysis" in the remainder of the report) for assessing the optimum levels of friction. The following paragraphs explain the concept and procedure for the risk analysis.

The risk analysis was inspired by the fact that the EB prediction (i.e., the weighted average of SPF predictions and the observed number of crashes) also has uncertainties as indicated by the standard deviation shown in Equation (26). This means that the crash reduction curves and the CMF curves shown in Figures 31 and 33 are also subject to uncertainties. More specifically, Figures 53 and 54 show the 95 percent confidence intervals of the CMF curves for dry and wet weather crashes, respectively. These figures show that the 95 percent confidence intervals are relatively narrow for the medium and high speed categories, due to the large amount of observed crash counts. Conversely, the relatively wider range of confidence intervals seen for the low speed category is due to the less number of observed crashes found in the integrated database (see Figure 44). Nevertheless, these intervals are still considered to be narrow, considering the overall number of crashes in these categories (Figure 44).

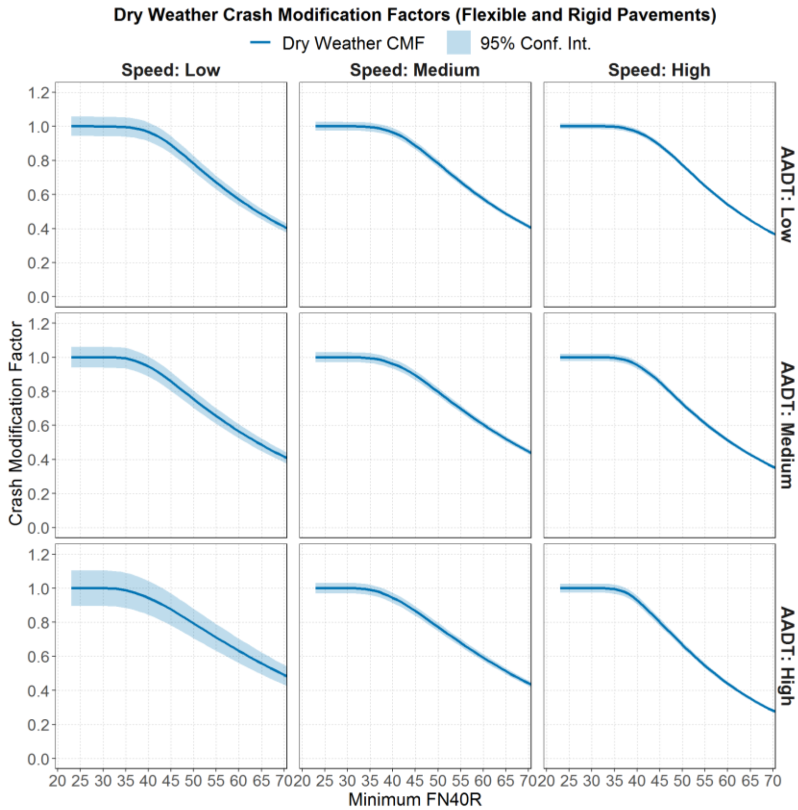


Figure 53. Dry Weather Crash Modification Factors with Confidence Intervals

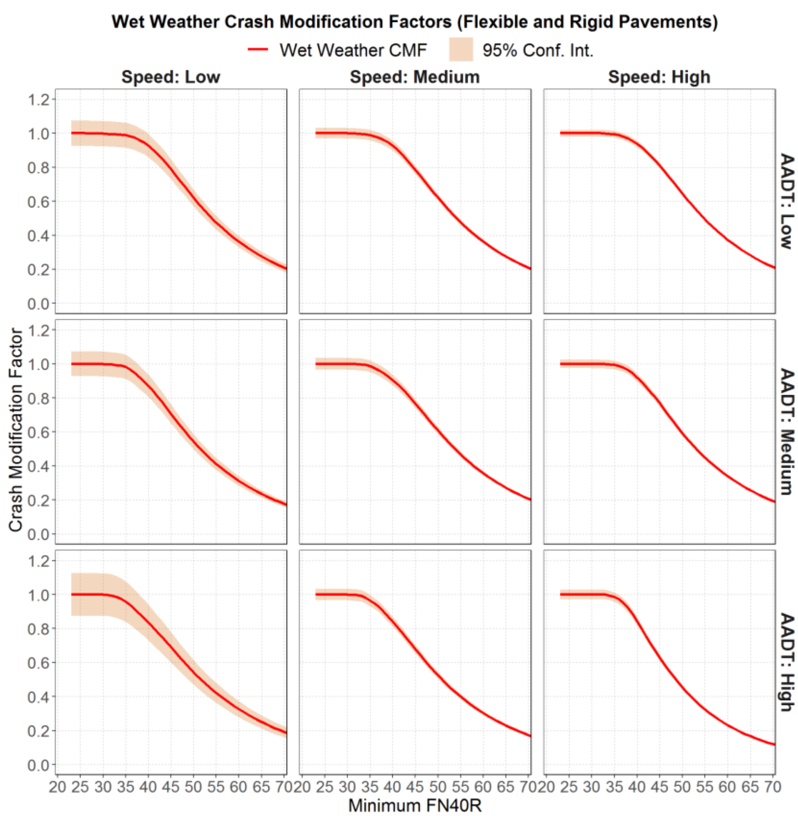


Figure 54. Wet Weather Crash Modification Factors with Confidence Intervals

Concept of Reliability and Risk

The concept of reliability and risk for the context of this report is described herein using a series of figures created for illustration. Since the standard deviation of the EB prediction is easily obtained using Equation (26), the distribution of estimated crash counts for any FN40R threshold can be calculated; note that the 95 percent confidence intervals shown in Figures 53 and 54 are only specific examples corresponding to ± 1.96 standard deviations from the average expected values.

The above concept is illustrated further in Figure 55. As a very simple scenario, consider the pavements with FN40R of 23. If these pavements get resurfaced and end up with a new FN40R value of 35, it is quite obvious that there will be a good chance of reducing the crash counts. However, due to the uncertainties associated with the estimated crash counts, there is still a relatively small chance (i.e., Risk) that the crash counts will increase after resurfacing. The probability of risk is obtained as the area under the bell curve highlighted in red. On the other hand, the Reliability can be obtained as the probability corresponding to improved safety which is the green area in Figure 55.

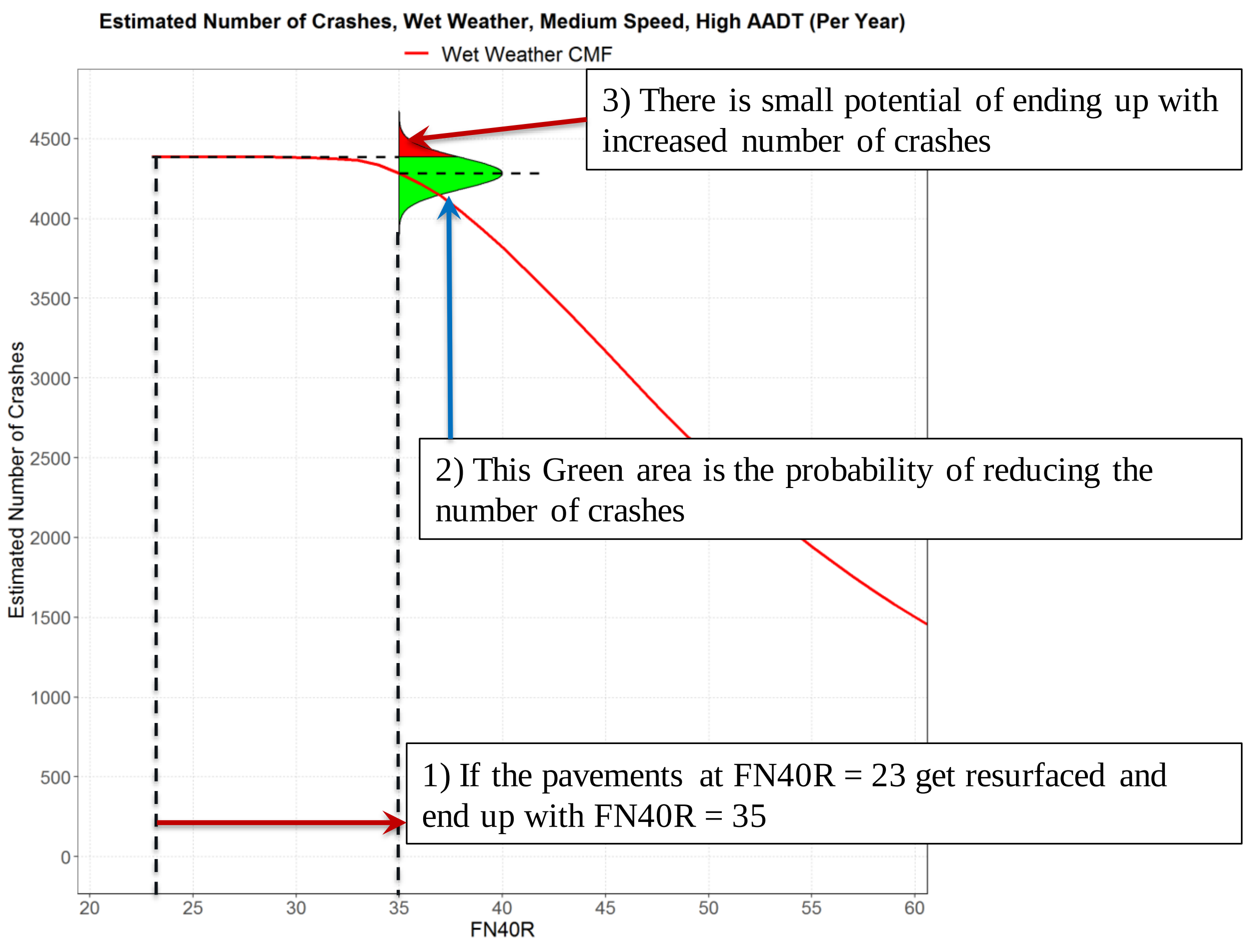


Figure 55. Illustration No. 1. Reliability and Risk based on CMF Curve

The concept of reliability and risk shown in Figure 55 was only based on the uncertainties associated with the CMF. However, the normal distribution that was drawn in Figure 55 and used for assessing the reliability/risk, is valid if and only if an FN40R value of 35 was obtained as a result of resurfacing. Such an FN40R value is not guaranteed in reality and as evidenced in Figures 29 through 32, it is possible that the achieved FN40R may be higher or lower than this artificial value. In other words, the uncertainties associated with the achieved FN40R after resurfacing (or any other friction restoration) need to be considered for the analysis (Figure 56).

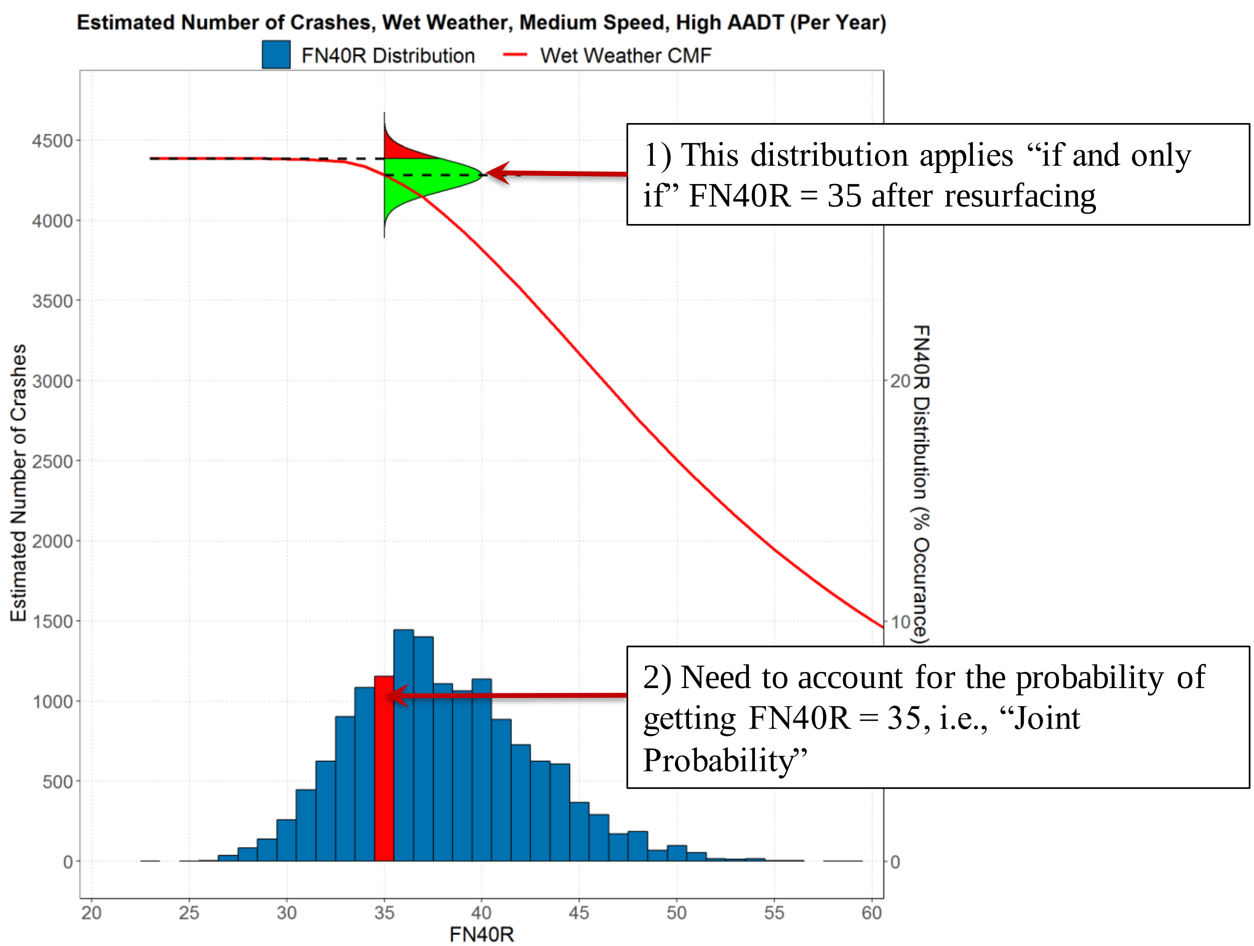


Figure 56. Illustration No. 2. Reliability and Risk based on CMF Curve and FN40R Distribution

As another example incorporating the FN40R distribution, consider a pavement with FN40R = 45 as shown in Figure 57. Although the level of friction for this roadway may still be in good shape, the road may need to be resurfaced due to other reasons (e.g., crack, roughness, or rut depth). If an FN40R value of 35 is achieved after resurfacing, it is likely that the number of crashes will increase as a result of reduction in FN40R. As shown in Figure 57, the probability of risk in this case is close to 100 percent with almost no chance of improving safety.

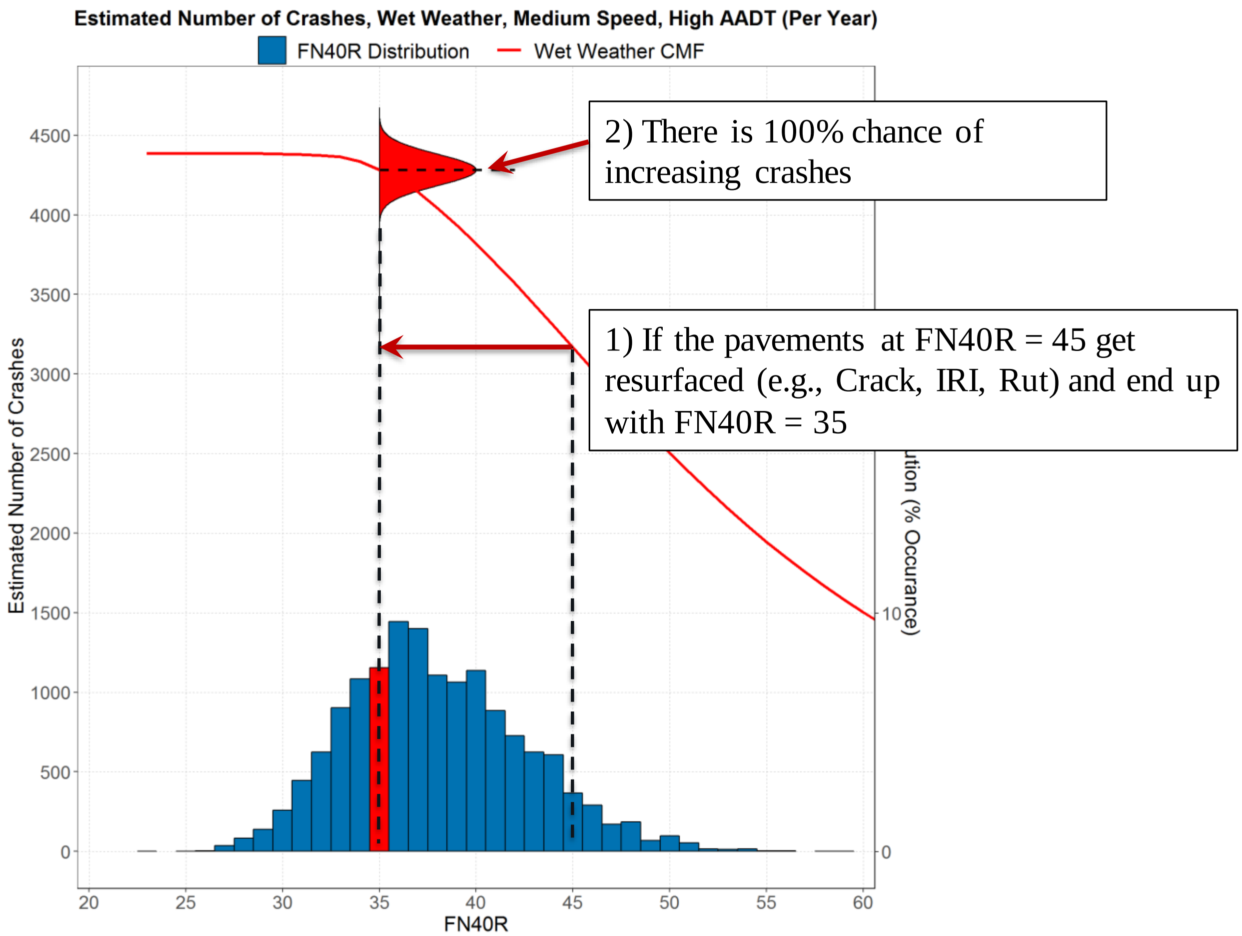


Figure 57. Illustration No. 3. Example of Risk

The scenario shown in Figure 57 was under the assumption that an existing roadway with FN40R of 45 gets resurfaced and the FN40R is reduced to 35 after resurfacing. However, it is possible that a higher FN40R may be achieved due to resurfacing. Figure 58 shows the scenario where the FN40R is increased to 50. Obviously in this case, the probability of risk is close to 0 percent with almost 100 percent chance of safety improvement due to the increase in FN40R. Nevertheless, it should be noted that the probability of achieving an FN40R of 50 is relatively small compared to the probability of achieving an FN40R value of 35.

Based on the above illustrations, the reliability (i.e., the probability of safety improvement or benefit) and risk (i.e., the probability of increased crashes) has been calculated for every combination of existing FN40R and achieved FN40R values after resurfacing, as the joint probability between friction distribution and CMF distribution. These results will be presented in the following section of the report.

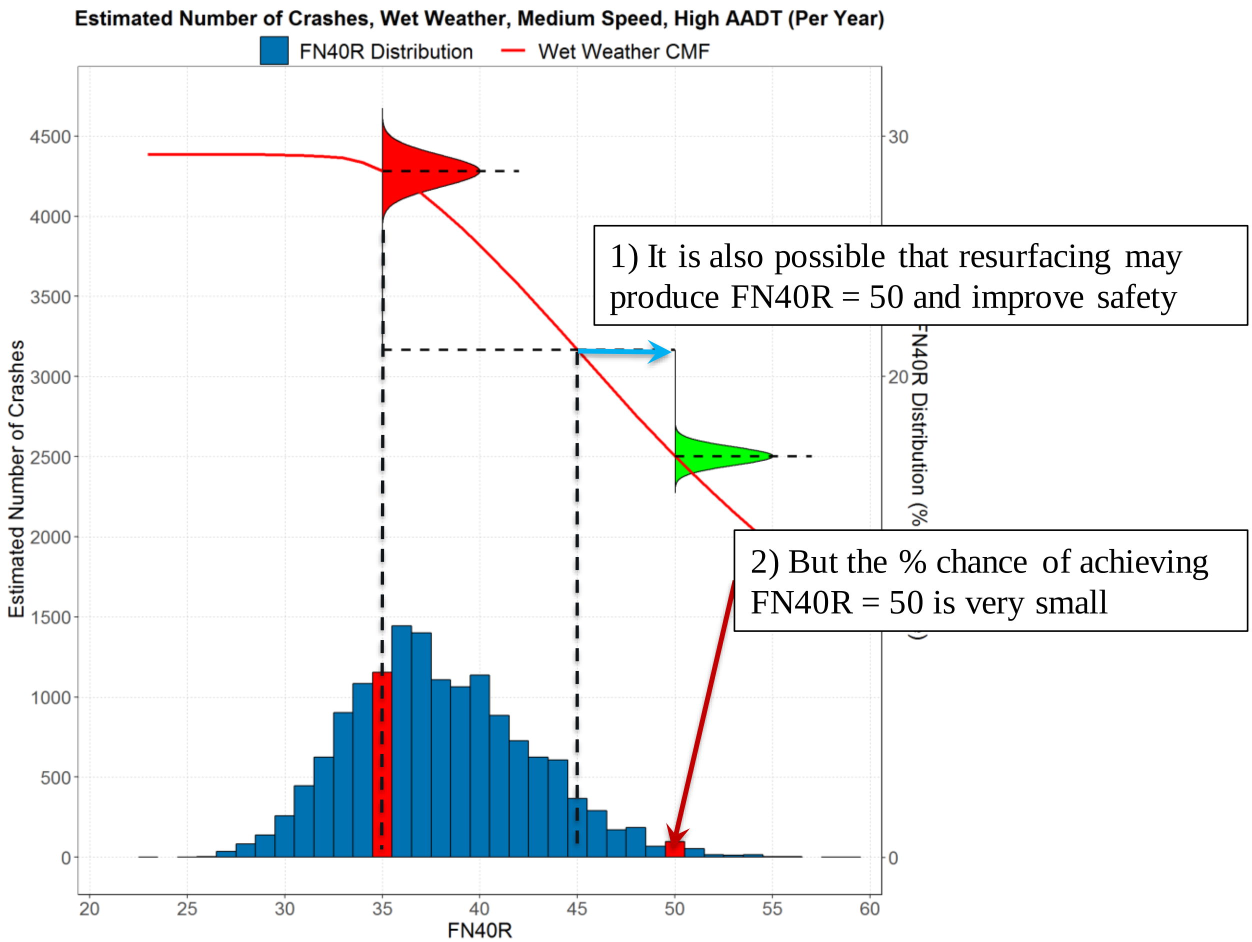


Figure 58. Illustration No. 3. Example of Risk and Reliability

<u>Reliability and Risk Curves</u>

As discussed above, the probabilities of benefit (or reliability) and risk has been calculated for all combinations of before/after FN40R while incorporating the FN40R distributions. Figure 59 shows the FN40R distributions obtained from the integrated database and used for the risk analysis. Also indicated in the figure is the average FN40R (dashed vertical line in red) for the respective categories. The following is observed from this figure:

- Regardless of the speed category, the low AADT category shows the highest FN40R values. This is due to the fact that the majority of D3 pavements (including the high speed, interstate highways) carried lower traffic (see Figure 33) but showed higher friction values (see Figure 28) compared to the other Districts.
- On the other hand, the high AADT category showed the lowest FN40R values, regardless of the speed category. Similar to the above, this is because the majority of high traffic facilities in D4 and D6 (see Figure 33) showing relatively lower FN40R values (see Figure 28) due to the use of limestone aggregates.

The above observations will be revisited for determining the target (recommended) friction levels.

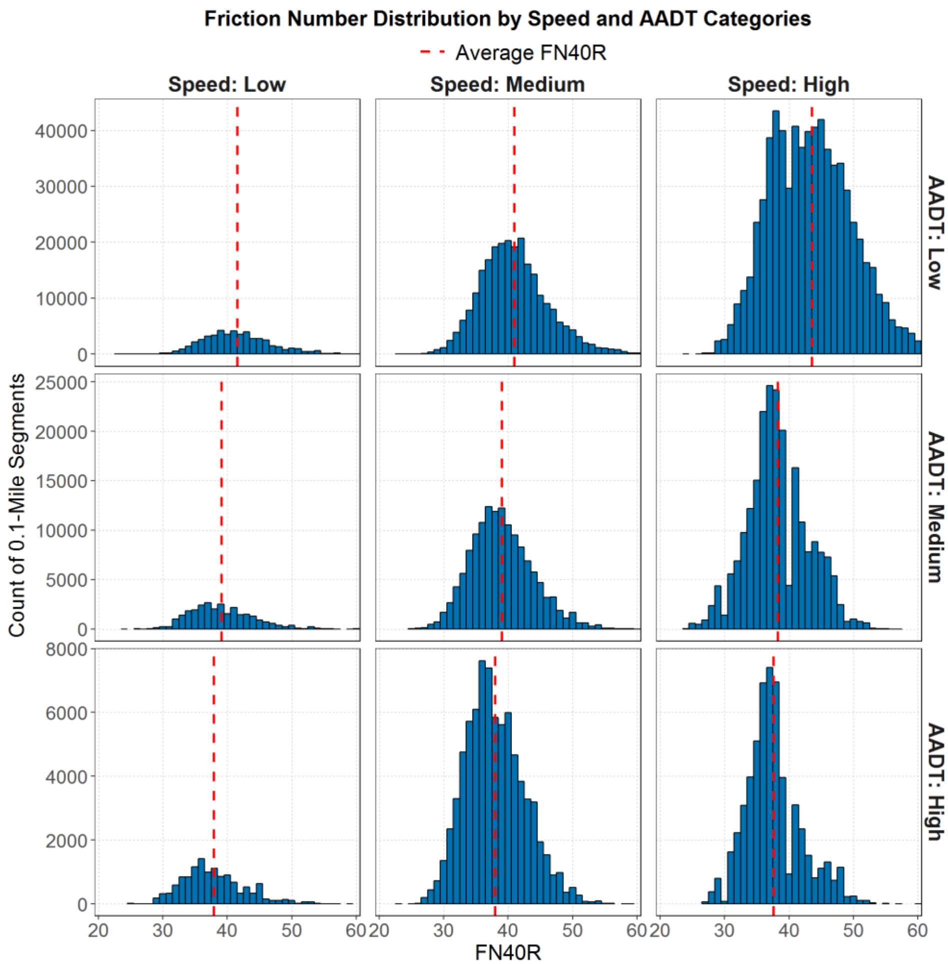


Figure 59. FN40R Distribution for the Recommended Friction Demand Categories

Figures 60 and 61 show the benefit (or reliability) vs risk curves obtained from dry weather and wet weather crashes respectively. As expected, both figures clearly show that the chances of reducing crashes increase as higher FN40R value is enforced and maintained. However, it is emphasized that these figures are not the traditional BCA curves.

To explain the interpretation of these figures, consider the benefit curve corresponding to high speed, low AADT, dry weather crashes (Figure 60) as an example. This benefit curve is relatively flat for FN40R values below 35 (approximately). This is because there is only a small fraction of roadways with FN40R less than 35 in this category (see Figure 59). In order to gain potential for additional benefit (i.e., the region where the benefit curve shows a steep increase), the FN40R threshold has to be increased further such that a larger portion of the lower tail of the existing FN40R distribution is eliminated.

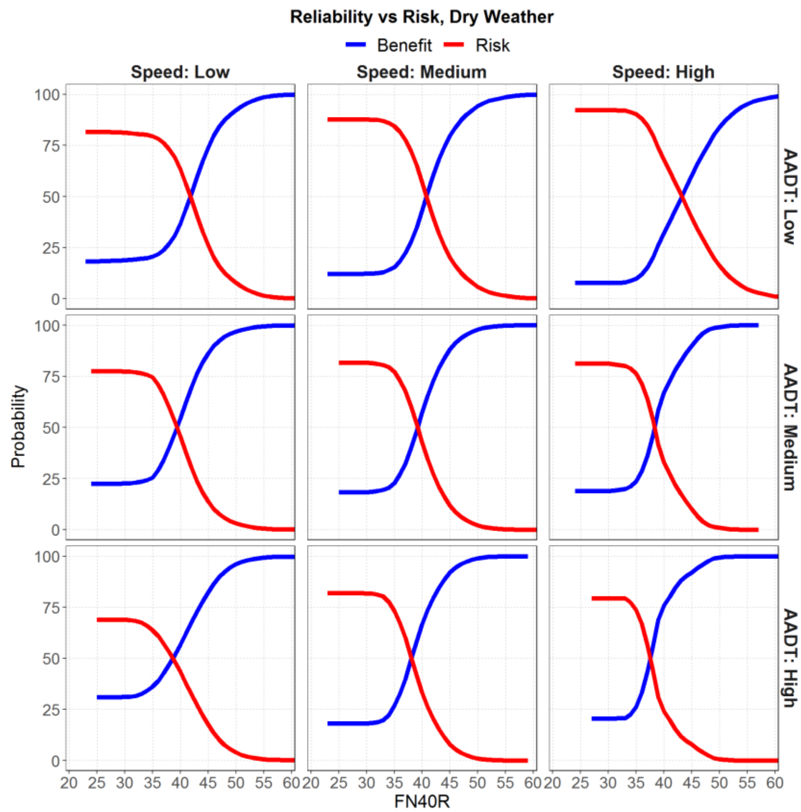


Figure 60. Dry Weather Benefit (Reliability) vs Risk Curves

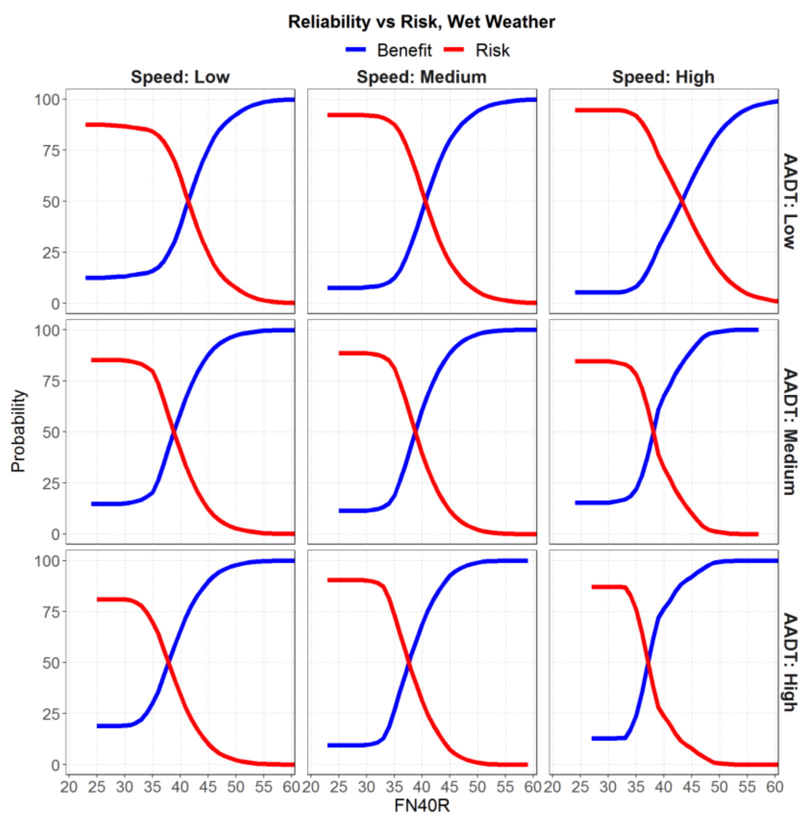


Figure 61. Wet Weather Benefit (Reliability) vs Risk Curves

The above discussion indicates that the desired level of friction should be determined while considering the level of benefit as well as the current distributions of friction and crash counts. It is also noted that the friction target should be set such that it is not unrealistic but achievable by the FDOT and the contractors. Furthermore, it is desirable that the probability levels of benefit (reliability) be higher for higher speed and higher traffic categories where the crash severities are expected to increase. Based on these discussions, the recommended probability levels of benefit have been selected and are summarized in Table 47 for both the dry weather and wet weather crashes (note that the benefit curves are very similar for dry and wet weather crashes).

Table 47. Recommended Levels for Probability of Benefit (Percent)

AADT Category	Speed Category		
	Low	Medium	High
Low	20	20	30
Medium	30	55	60
High	40	70	80

Table 48 summarizes the FN40R thresholds corresponding to the benefit probabilities shown above for both dry and wet weather crashes.

Table 48. FN40R Thresholds from Benefit Curves

AADT Category	Dry Weather			Wet Weather		
	Speed Category			Speed Category		
	Low	Medium	High	Low	Medium	High
Low	34	36	39	36	36	39
Medium	36	39	39	36	39	38
High	36	40	40	36	40	41

The above table shows that the FN40R thresholds obtained from dry and wet weather crashes are fairly similar to each other. As such, the FN40R thresholds have been simplified further by combining the two weather types and are shown in Table 49. These FN40R thresholds are the recommended minimum friction values for FDOT's friction management program.

Table 49. Recommended Minimum Values of FN40R

AADT Category	Speed Category		
	Low (Speed Limit < 40 mph)	Medium (40 mph ≤ Speed Limit ≤ 50 mph)	High (Speed Limit > 50 mph)
Low	34	36	39
Medium	36	39	39
High	36	39	40

The desired FN40R values in the above table may seem to span only a narrow range from 34 to 40 and hence, the effectiveness of the above levels of friction may also seem questionable. Therefore, the estimated number of crash reduction has been calculated assuming that the FN40R values in Table 49 were enforced and maintained. Table 50 shows the estimated crash reduction and percent reduction based on the data in the integrated database.

Table 50. Estimated Crash Reduction Per Year

AADT Category	Estimated Reduction of Number of Crashes (% Reduction)*					
	Dry Weather			Wet Weather		
	Speed Category			Speed Category		
	Low	Medium	High	Low	Medium	High
Low	9 (0.3)	121 (0.7)	194 (2.3)	3 (0.9)	48 (1.7)	81 (4.6)
Medium	54 (1.2)	616 (2.8)	308 (3.2)	23 (3.4)	276 (7.4)	145 (5.6)
High	71 (1.8)	1,018 (4.4)	681 (7.0)	37 (6.2)	567 (12.9)	382 (15.8)

Note*: These numbers are only based on the crash data that was mapped to the Integrated Database.

Based on the numbers shown in the above table, the total reduction is estimated to be 4,634 crashes per year. Although this may not seem to be a large number of crash reduction, it is emphasized that this number of derived based on 125K crashes that were mapped back to the integrated database which corresponds to approximately 3.7 percent crash reduction per year. If this rate of crash reduction (3.7 percent) is applied to 1.13 million statewide crashes included in the crash database provided to the research team for the 8 year period, this results in an estimated crash reduction of 41K over the analysis period.

Furthermore, the average cost of a crash from FDOT's CARS system was estimated to be approximately $150K based on the crash data between 2011 and 2015. This translates the estimated crash reduction of 41K to a saving in excess of $6.2B dollars for the 8 year period.

It is noted again that the above estimates on savings were based on the data that were available to the research team. In other words, the crashes and savings that may occur on County roads and City streets were not accounted for. Furthermore, the above benefits were estimated based only on improvement in pavement friction. The benefits gained from resurfacing or other friction restoration activities (e.g., added lanes, new texture, smoother pavement, etc.) were not incorporated. Based on these discussions, it is concluded that the actual benefits to be gained are much greater than what was estimated herein.

Finally, by combining the preliminary friction demand categories in Table 23 and the recommended levels of friction shown in Table 49, and further simplifying the table by combining the AADT categories with similar desired FN40R value, the recommended friction guideline is summarized as shown in Table 51.

To compare the recommended FN40R values with FDOT's existing FN40R requirements, FDOT's existing friction guideline is shown again in Table 52 (same as Table 6). It is seen that newly recommended friction thresholds are generally higher than FDOT's existing thresholds. This does not mean that FDOT's existing friction guideline is not effective. In fact, FDOT's existing friction guideline is simple, effective, and is working well for its purpose. Nonetheless, the statistical model developed in this study clearly indicated that the crash counts will reduce further with increasing friction on FDOT's roadways. As such, it is emphasized that the friction numbers in Table 51 are only provided as recommendations for further reducing the crashes on FDOT's roadways, with the estimated crash reduction provided above (Table 50).

Table 51. Recommended Friction Demand Categories and Levels of Friction

Speed Category	AADT		Desired FN40R
	Category	Value	
Low (Speed Limit < 40 mph)	Low	AADT < 20 K	34
	Medium - High	AADT ≥ 20 K	36
Medium (40 mph ≤ Speed Limit ≤ 50 mph)	Low	AADT < 30 K	
	Medium - High	AADT ≥ 30 K	39
High (Speed Limit > 50 mph)	Low - Medium	AADT < 90 K	
	High	AADT ≥ 90 K	40

Table 52. FDOT's Existing Friction Guidelines (Same as Table 6)

Speed Limit or Design Speed (mph)	All Highway Surfaces		
	Questionable[1]	Review[2]	Desired[3]
	FN40R	FN40R	FN40R
Less than or equal to 45	25	26 - 28	30
Greater than 45	27	28 - 30	35

Note 1: FN below these thresholds warrants investigation (existing pavements) or crash monitoring (new pavements).
Note 2: FN below these thresholds warrants review (existing pavements) or crash monitoring (new pavements).
Note 3: Desired FN values for new pavements.

SUMMARY

The primary objective of this chapter was to develop a statistical relationship between Florida's crash rates, pavement friction and texture characteristics, as well as other pavement-related data. A number of preliminary analyses have been conducted in regards to crash, friction, and traffic distributions for determining the friction demand categories. The new statistical model was accompanied by a reliability-risk analysis for determining the recommended levels of friction for FDOT's friction guidelines.

Furthermore, discussions were provided on the estimated crash reduction as well as the savings from reduced number of crashes. If the friction values recommended in this study are implemented, it was estimated that FDOT may anticipate a crash reduction of 41K which also translates to a crash savings in excess of $6.2B dollars over a period of 8 years. These benefits, however, do not account for those from County roads and City streets as well as the effects of other variables that may be altered due to friction restoration or resurfacing (e.g., texture and ride quality improvement). As such, it is believed that the actual benefits to be gained from the recommended friction levels are much greater.

RECOMMENDATIONS

In the previous chapter, a statistical relationship was developed for Florida's crash counts, pavement friction and texture characteristics, as well as other pavement-related data. In addition, recommendations were provided on the desired levels of friction for the nine friction demand categories (three speed categories and three traffic categories).

In this chapter, the previously developed statistical model was utilized further to develop recommendations regarding FDOT's Friction Guidelines, Friction Course Policy, and Safety Analysis Practice.

RECOMMENDATIONS FOR FDOT'S FRICTION GUIDELINES

Tables 52 and 51 showed FDOT's existing Friction Guidelines and the desired levels of friction recommended from this study, respectively. Although the desired levels of friction were recommended in Table 51, it is still lacking the Questionable and Review thresholds when compared to the existing Friction Guidelines. As such, recommendations are made herein for these missing thresholds such that a complete recommendation can be made on FDOT's Friction Guidelines.

A quick comparison of the desired level of friction in Tables 52 and 51 immediately reveals that the newly recommended levels of desired FN40R values are higher than the existing values. It is emphasized that this does not mean FDOT's existing Friction Guidelines (Table 52) are not effective. As seen in Figure 27 and Table 19, less than 5.0 percent of FDOT's roadways are exhibiting FN40R values below 32. In addition, Figure 34 showed a significantly lower number of crashes on roadways with FN40R below 30. These are clear evidences that indicate FDOT's current Friction Guidelines are effective and working.

The recommendations for higher levels of desired FN40R in Table 51 were made based on the statistical model developed in this study that indicated the crash counts will reduce further and the safety performance on FDOT's roadways will improve with increasing level of friction. In order for the higher levels of desired FN40R values to be effective upon implementation, it is deemed appropriate that the Questionable and Review thresholds be established at higher levels than the existing thresholds.

As suggested by Hall et. al. (2006), the crash distributions and the Wet-to-Dry (W/D) crash ratio are revisited for establishing the Questionable and Review thresholds. Figure 34 showed the statewide W/D ratio along with the dry and wet weather crash distributions. Although it is based on limited amount of data as evidenced by the crash distributions, the statewide W/D ratio clearly showed a rapid increase for FN40R below 30.

In addition, the following observations are made from Figure 44, which shows the crash distributions and the W/D ratio for the 9 friction demand categories previously defined:

1. Similar to the statewide trend, the W/D ratios for high speed categories are subjected to lack of crash data, especially for FN40R below 30. Nonetheless, higher W/D ratios are generally observed for this particular FN40R range.

2. The W/D ratios for medium speed categories do not show a clear trend. However, it is clear from the crash distributions that the majority of crashes are occurring in the medium speed category. As such, the medium speed category may gain significant benefit by performing safety reviews or crash monitoring, for those roadways with FN40R less than 30.

3. The low speed categories only show a minimal amount of crashes (compared to other speed categories) and their W/D ratios do not show a clear trend (see Figure 44). However, the crash rates shown in Figures 45 and 46 indicate that both dry and wet weather crashes may still benefit from increased level of friction.

Based on the above observations, the recommended FN40R thresholds for Questionable and Review categories are shown in Table 53. Essentially, an FN40R value of 30, below which the statewide W/D ratio showed a significant increase, was assigned as the Questionable threshold to 8 of the 9 friction demand categories. For these categories, the Review thresholds have been established approximately midway between Questionable and Desired levels of FN40R. As for the low speed, low AADT category, the FN40R threshold of 30 was assigned to the Review category. Note that this is 4 points below the Desired level, and hence the Questionable threshold has been established at another 4 points below the Review threshold (i.e., FN40R = 26).

Table 53. Recommended Friction Guidelines

<table>
<tr><th rowspan="2">Speed Category</th><th colspan="2">AADT</th><th colspan="3">FN40R</th></tr>
<tr><th>Category</th><th>Value</th><th>Questionable</th><th>Review</th><th>Desired</th></tr>
<tr><td rowspan="2">Low
(Speed Limit < 40 mph)</td><td>Low</td><td>AADT < 20 K</td><td>26</td><td>30</td><td>34</td></tr>
<tr><td>Medium - High</td><td>AADT ≥ 20 K</td><td rowspan="5">30</td><td rowspan="2">33</td><td rowspan="2">36</td></tr>
<tr><td rowspan="2">Medium
(40 mph ≤ Speed Limit ≤ 50 mph)</td><td>Low</td><td>AADT < 30 K</td></tr>
<tr><td>Medium - High</td><td>AADT ≥ 30 K</td><td rowspan="2">34</td><td rowspan="2">39</td></tr>
<tr><td rowspan="2">High
(Speed Limit > 50 mph)</td><td>Low - Medium</td><td>AADT < 90 K</td></tr>
<tr><td>High</td><td>AADT ≥ 90 K</td><td>35</td><td>40</td></tr>
</table>

In order to see if the Questionable and Review thresholds are practical for FDOT's typical friction courses, the trends of FN40R degradation shown in Figure 29 is revisited. Taking the high speed, high AADT category as an example, the FN40R thresholds for Desired and Review levels are 40 and 35, respectively (i.e., only 5 point difference). Then, a practical question one would have to ask is: "How long does it take for a typical friction course to reach the review threshold?". According to Figure 29, the OGFCs show the lowest initial FN40R (equal to 41 on average) and the highest degradation rate (equal to 0.57 average reduction in FN40R per year).

Said differently, this means that a typical OGFC will reach the Review threshold at 11 years of age and the Questionable threshold at 22 years of age. Given that the service life of OGFCs are estimated to be 12 to 15 years, it is concluded that these thresholds have passed the sanity check.

Lastly, it is emphasized that the recommended Questionable and Review thresholds in Table 53 have been established in an empirical manner (i.e., through examining the FN40R distribution, crash and crash rate distributions, and W/D crash ratio). In other words, these thresholds may need to be evaluated further (e.g., through pilot projects) to ensure that these values support the higher levels of Desired FN40R values.

RECOMMENDATIONS FOR FDOT'S FRICTION COURSE POLICY

As shown in FDOT's current Friction Course Policy (Table 11), FDOT is specifying OGFC (i.e., FC-5) on multi-lane facilities with design speed in excess of 45 mph. To see if the Safety Performance Functions (SPF) developed in this study support this policy, a couple of simulations have been conducted and described below.

The simulation was conducted based on all data that was mapped to the Integrated Database, and was carried out by changing the surface type and the texture of all available roadways in the Database to one of the following:

- FC125 (Dense Graded) with MPD = 0.018 in.
- FC95 (Dense Graded) with MPD = 0.018 in.
- FC5 (Open Graded) with MPD = 0.061 in.

Note that the MPD values for the above surface types correspond to the median values obtained from FDOT's roadways constructed between 2014 and 2018 (FDOT, 2019). Using the above inputs, the expected crash counts (both Dry and Wet Weather) were calculated for the entire network.

Figures 62 and 63 show the total expected crash counts obtained from the SPF models for Dry and Wet weather, respectively. Both figures indicate that FC5 may reduce the number of crashes, regardless of the type of crash (Dry vs. Wet) and Speed Limit. However, it should also be noted that based on FDOT's current policy, FC5 is not placed on roadways with a design speed of 45 mph or less. In other words, the FC5 results shown in Figures 62 and 63 for these low speed facilities are "extrapolated" without any field data that would be required for a stronger conclusion. Similarly, the FC95 and FC125 results for high speed facilities (e.g., 70 mph speed limit) are from extrapolation of the SPF models and are not supported by field data.

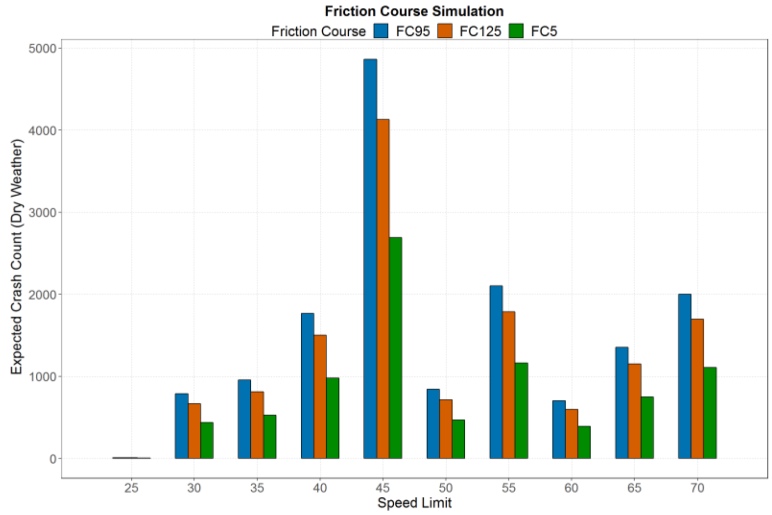


Figure 62. Dry Weather Simulation Results for Different Surface Types

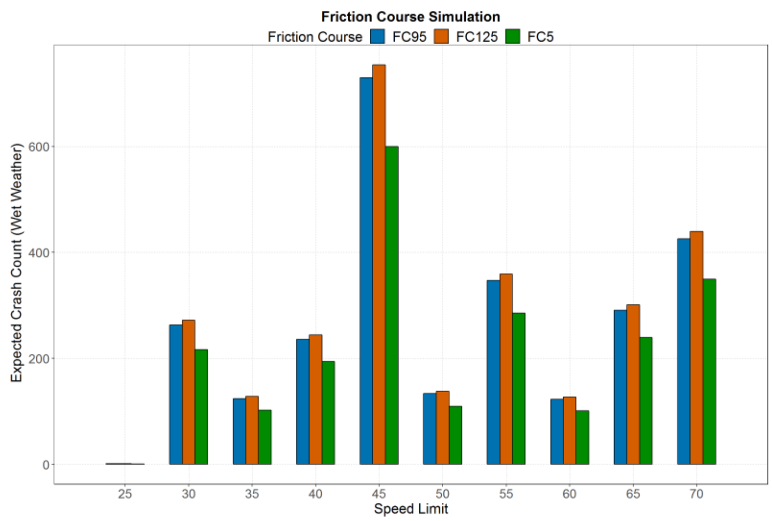


Figure 63. Wet Weather Simulation Results for Different Surface Types

The primary role of an OGFC is to improve surface drainage and to reduce the potential for Splash/Spray and hydroplaning potential. As such, it is believed that the OGFC should continue to be used on high speed (i.e., Speed Limit greater than or equal to 50 mph), multi-lane facilities. On the other hand, it is recommended that FDOT continue to use Dense Graded surfaces (FC125 or FC95) on low speed facilities (i.e., Speed Limit less than or equal to 45? mph) and two-lane roads, as hydroplaning risk and Splash/Spray potential are not as critical as on the high speed facilities.

FDOT's current policy (Table 11) specifies the use of DGFC on multi-lane facilities with a design speed of 45 mph. However, as shown in Figures 62 and 63, facilities with a speed limit of 45 mph is where the majority of crashes are expected (and observed, based on Figures 39 and 40). In addition, the design speed of 45 mph is the minimum speed considered in FDOT's Hydroplaning Design Guidance. As such, it is recommended that FDOT consider using FC5 in addition to FC125 and FC95, on multi-lane facilities with design speed of 45 mph. It is also recommended that the decision (FC5 vs FC125 or FC95) be made on a project-by-project basis (i.e., by conducting hydroplaning risk analysis or through evaluation of past crash histories on the given section). Table 54 summarizes the recommended Friction Course Policy.

Table 54. Recommended Friction Course Policy.

<table>
<tr><th rowspan="2">Design Speed (mph)</th><th colspan="2">Asphalt Surface</th><th colspan="2">Concrete Surface</th></tr>
<tr><th>Two-Lane</th><th>Multi-Lane</th><th>Pavements</th><th>Bridge Decks</th></tr>
<tr><td>40 mph or Less</td><td rowspan="3">FC125 or FC95</td><td>FC125 or FC95</td><td rowspan="3">LGD</td><td rowspan="3">LGD + TGV</td></tr>
<tr><td>45 mph</td><td>FC125, FC95, or FC5*</td></tr>
<tr><td>50 mph or Greater</td><td>FC5</td></tr>
</table>

Note*: FC5 should be considered on roadways requiring improved surface drainage.

RECOMMENDATIONS FOR FDOT'S SAFETY ANALYSIS PRACTICE

As discussed previously, the first step in FDOT's safety analysis involves identification of high wet weather crash locations based on the last five years of crash data. The high crash locations are identified based on the following criteria.

(1) A minimum of four wet weather crashes with 25 percent or more wet weather crashes or
(2) Fifty (50) percent or more wet weather crashes during the five-year analysis period.

The above analysis is conducted on 0.3-mile segments (including intersections) moving at an increment of 0.1-mile.

To see if the above criteria are effective, analysis was conducted to compare the wet weather crash locations identified from the above criteria to those from a different method recommended by FHWA's Highway Safety Improvement Manual (Herbel et. al., 2010). The FHWA method involves calculating the "Excess Number of Crashes" as the observed number of crashes minus the expected crash count from the statistical model (i.e., SPF). The "problematic" crash locations are identified as those where the Excess Crash is greater than zero (i.e., more crashes were observed than expected).

Figure 64 shows the Excess Crash calculated from FDOT's data made available in the Integrated Database. As the sections with negative Excess Crash (i.e., less number of crashes than expected) are not of concern, these sections were removed from further analysis. For the remaining sections where the Excess Crash was greater than zero, the total number of wet weather crashes and percent wet weather crashes were calculated based on the observed crash data between 2013 and 2017, and plotted in Figure 65.

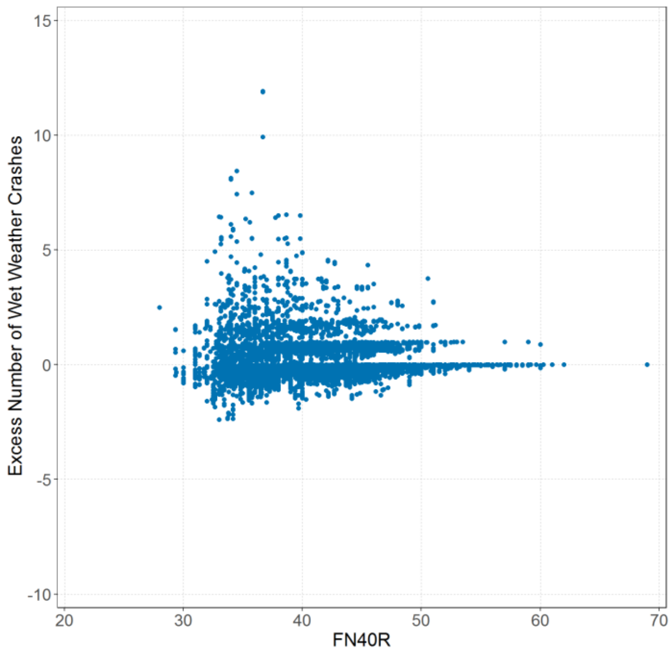


Figure 64. Excess Crash Counts

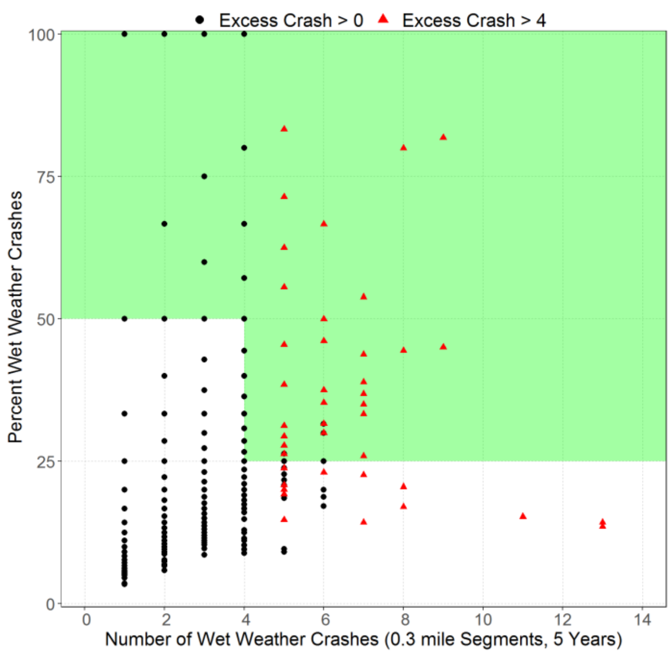


Figure 65. Percent Wet Weather vs. Total Wet Weather Crashes with FDOT's Existing Criteria (Excess Crash greater than 4 shown in Red Triangles)

In Figure 65, the sections with Excess Crash greater than 4 are shown separately in red triangles. The figure also highlights (in light green) the area where the sections meet FDOT's current criteria for wet weather crash locations. The figure clearly shows that FDOT's existing method is picking up most of the sections with high Excess Crash. However, the figure also shows that the existing method missed some of the sections with high wet weather crash counts but low percent wet weather crashes.

Higher wet weather crash counts with lower percentage of wet weather crashes is a consequence of even higher dry weather crash counts. For example, if a section had 10 wet weather crashes with 10 percent wet weather, this means that the section showed 90 dry weather crashes (i.e., total of 100 crashes). Although the significant increase in dry weather crashes may be due to a combination of various factors (friction, roadway curvature, speed, etc.), it is generally seen that wet and dry weather crash distributions follow similar trends (see Figures 34 and 44). In other words, if the Department was successful in reducing the number of wet weather crashes on a given section, it is likely that the section will also show a reduction in dry weather crashes. As such, it is recommended that another criterion be added to FDOT's existing criteria of identifying wet weather crash locations for safety review. The following shows the recommended criteria including FDOT's existing ones.

(1) A minimum of four wet weather crashes with 25 percent or more wet weather crashes, or
(2) Fifty (50) percent or more wet weather crashes during the five-year analysis period, or
(3) A minimum of six wet weather crashes with 10 percent or more wet weather crashes.

Figure 66 highlights (in light green) the area where the sections would have been identified as wet weather crash locations based on all three of the above recommended criteria. The addition of the new criterion allowed for capturing more sections with higher Excess Crash counts.

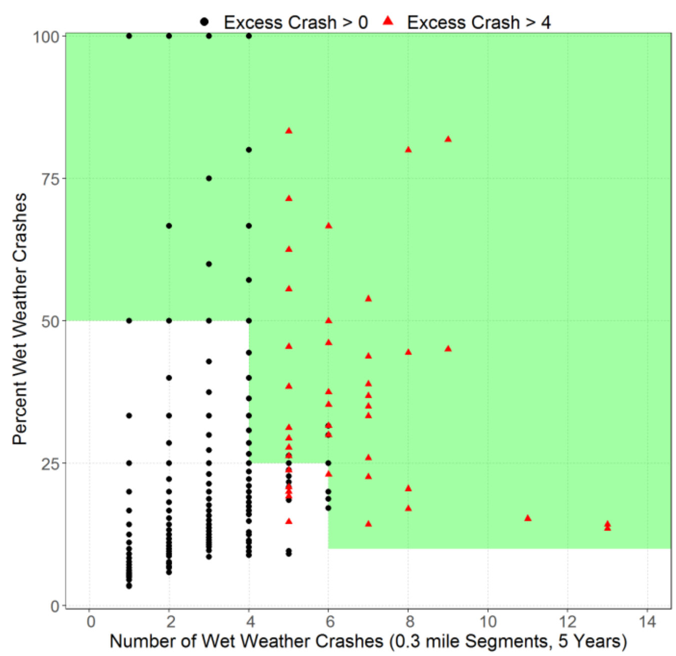


Figure 66. Percent Wet Weather vs. Total Wet Weather Crashes with Recommended Criteria (Excess Crash greater than 4 shown in Red Triangles)

As shown in Figure 66, the recommended criteria allows for identifying the sections with excess number of crashes. The advantage of the new criteria is that the wet weather crash locations are identified with minimal change to FDOT's existing practice. I.e., the SPF is not required for identification of problematic sections. The sections identified in the above manner are effectively capturing most of the sections with higher Excess Crash counts.

It should also be noted that the sections not identified as wet weather crash locations (i.e., sections below the green area in Figure 66) are not being neglected either. To prove this point, Figure 67 plots the same data as Figure 66. In Figure 67, however, the sections with FN40R less than 35 are highlighted in red triangles (regardless of Excess Crash). Note that these sections will be reviewed for safety under the recommended Friction Guidelines (Table 53), if implemented.

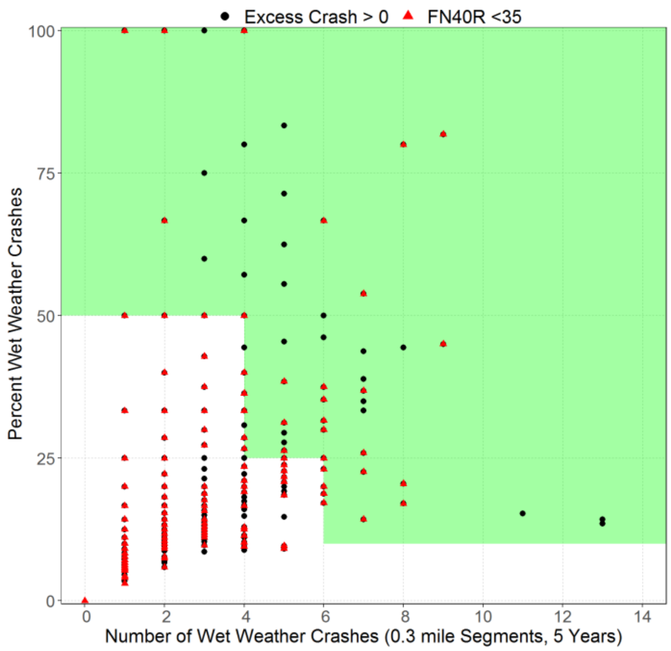


Figure 67. Percent Wet Weather vs. Total Wet Weather Crashes with Recommended Criteria (FN40R less than 35 shown in Red Triangles)

Figures 66 and 67 clearly demonstrates that the newly recommended Friction Guidelines combined with the recommended method for identifying wet weather crash locations, may allow FDOT for identifying locations with safety concerns effectively.

SUMMARY

In this chapter, recommendations on FDOT's Friction Guidelines, Friction Course Policy, and Safety Analysis Practice were developed and documented. The following provides a quick summary.

1. Based on the Desired FN40R levels determined from the previous chapter, the recommended levels for Questionable and Review thresholds were established herein (Table 53). These thresholds were established in an empirical manner by examining the FN40R distribution, crash and crash rate distributions, and W/D crash ratio. Therefore, it is recommended that these thresholds be evaluated further (e.g., through pilot projects) to ensure that these values support the higher levels of Desired FN40R values.

2. According to the available data, FDOT's roadways with a speed limit of 45 mph exhibit the largest number of crashes (Dry and Wet weather). The design speed of 45 mph also corresponds to the minimum speed considered in FDOT's Hydroplaning Design Guidance. As such, it is recommended that FDOT consider using OGFCs on multi-lane facilities with design speed of 45 mph.

3. FDOT's current methodology for identifying the wet weather crash locations are found to be effective. However, it is recommended that another criterion [more specifically, sections with minimum of six wet weather crashes and 10 percent or more wet weather crashes] be added to the existing criteria to capture more sections with higher crash potential.

It is emphasized again that although pavement friction is an important factor, it is not the only factor affecting roadway crashes. As demonstrated in this report, FDOT's current approach is effectively in line with this statement because the sections with either (1) low FN40R [based on the Friction Guidelines] or (2) high number of crashes [based on Safety Analysis criteria] undergo a safety review. Nevertheless, it is believed that FDOT will see further improvement in safety (i.e., further reduction in crashes) by implementing the recommendations provided herein.

REFERENCES

Bektas, B.A., Gkritza, K., and Smadi, O. 2016. Pavement Marking Retroreflectivity and Crash Frequency: Segmentation, Line Type and Imputation Effects. Journal of Transportation Engineering, Vol. 142, No. 8, DOI: 10.1061/(ASCE)TE.1943-5436.0000863.

Buddhavarapua, P., Smit, A.F., Prozzi, J.A. 2015. A Fully Bayesian Before– After Analysis of Permeable Friction Course (PFC) Pavement Wet Weather Safety. Accident Analysis and Prevention. Vol. 80. pp. 89-96.

Cameron, A. C., and P. K. Trivedi. Chapter 20: Models of Count Data. In Microeconometrics: Methods and Applications. Cambridge University Press, New York, 2005, pp. 665–691.

Choubane, B., Chun, S., Lee, H.S., Upshaw, P., Greene, J., and Abdenour, N. 2017. Evaluation of Warm-Mix Asphalt (WMA) Performance in Florida. Transportation Research Board 93rd Annual Meeting Compendium of Papers, Washington, D.C.

Choubane, B. Holzschuher, C., Gokhale, S. and Fletcher, J. Precision of Smooth and Ribbed Tire Locked Wheel Testers for Measurement of Roadway Surface Friction Characteristics. Transportation Research Record, 1949, TRB, Washington, D. C., 2006, pp. 137-147.

Choubane, B., Lee, H.S., Holzschuher, C., Upshaw, P., and Jackson, N.M. 2012. Harmonization of Texture and Friction Measurements on Florida's Open-Graded and Dense-Graded Pavements. Transportation Research Record No. 2306. Transportation Research Board (TRB), Washington, D.C. pp. 122-130.

Dare, T., Thornton, W., Wulf, T., Bernhard, R., 2009. Acoustical Effects of Grinding and Grooving on Portland Cement Concrete Pavements. Final Report HL 2009-1, Purdue University's Institute of Safe, Quiet, and Durable Highways.

De Leon Izeppi, E., Flintsch, G.W., Archilla, A.R., and Sequeira, W. 2011. Data Processing and Analysis Software for Continuous Friction Measurement Equipment. Transportation Research Record: Journal of the Transportation Research Board, No. 2227, Transportation Research Board, Washington, D.C., pp. 163–170.

De Leon Izeppi, E., Katicha, S.W., Flintsch, G.W., and McGhee, K.K. 2016a. Pioneering Use of Continuous Pavement Friction Measurements to Develop Safety Performance Functions, Improve Crash Count Prediction, and Evaluate Treatments for Virginia Roads. Transportation Research Record: Journal of the Transportation Research Board, No. 2583, Transportation Research Board, Washington, D.C., pp. 81–90.

De Leon Izeppi, E., Katicha, S.W., Flintsch, G.W., McCarthy, R., and McGhee, K.K. 2016b. Continuous Friction Measurement Equipment as a Tool for Improving Crash Rate Prediction – A Pilot Study. Report No. VTRC 16-R8. Virginia Department of Transportation, Richmond, VA.

FDOT. 2019. “State Materials Research Park 2014-2018 NC/OVR Friction and Texture Summary”. State Materials Office. Gainesville, FL.

Federal Highway Administration (FHWA). 2005. Surface Texture for Asphalt and Concrete Pavements. Technical Advisory T 5040.36. FHWA. Washington, D.C.

Flintsch, G.W., McGhee, K.K., Izeppi, E., and Najafi, S. 2012. The Little Book of Tire Pavement Friction. Version 1.0. Pavement Surface Consortium.

Guada, I.M., Rezaei, A., Harvey, J.T., and Spinner, D. 2012. Evaluation of Grind and Groove (Next Generation Concrete Surface) Pilot Projects in California. Report No. UCPRC-RR-2013-01. California Department of Transportation, CA.

Hall, J.W., Smith, K.L, Titus-Glover, L., Wambold, J.C., Yager, T.J., and Rado, Z. 2006. Guide for Pavement Friction. NCHRP Project 1-43, National Cooperative Highway Research Program (NCHRP), Washington, D.C.

Hall, J.W. and Smith, K.L. 2009. Texturing of Concrete Pavements. NCHRP Report 634. Transportation Research Board, Washington, D.C.

Hauer, E., Harwood, D.W., Council, F.M., and Griffith, M.S. 2002. Estimating Safety by the Empirical Bayes Method - A Tutorial. Transportation Research Record: Journal of the Transportation Research Board, No. 1784, Transportation Research Board, Washington, D.C., pp. 126–131.

Henry, J.J. 2000. Evaluation of Pavement Friction Characteristics – A synthesis of Highway Practice. NCHRP Synthesis 291. Transportation Research Board, Washington, D.C.

Herbel, S., Liang, L., and McGovern, C. 2010. Highway Safety Improvement Manual. Report No. FHWA-SA-09-029. Federal Highway Administration. Washington, D.C.

Holzschuher, C. 2017a. Florida Texture Characteristics Overview. Transportation Research Board Webinar Presentation. Transportation Research Board, Washington, D.C.

Holzschuher, C. 2017b. High Friction Surface Treatment Guidelines; Project Selection, Materials, and Construction. State Materials Office, Florida Department of Transportation, Gainesville, FL.

Holzschuher, C. 2018. FDOT Pavement Marking Management System and Quality Assurance Practices. Transportation Research Board Webinar Presentation. Transportation Research Board, Washington, D.C.

Ivan, J.N., Ravishanker, N., Jackson, E., Aronov, B., and Guo, S. 2010. Incorporating Wet Pavement Friction into Traffic Safety Analysis. Report No. JHR 10-324. Connecticut Department of Transportation. Rocky Hill, CT.

Ivan, J.N., Ravishanker, N., Jackson, E., Aronov, B., and Guo, S. 2012. A Statistical Analysis of the Effect of Wet-Pavement Friction on Highway Traffic Safety. Journal of Transportation Safety & Security, Vol. 4, pp. 116-136.

Kuttesch, J.S. 2004. Quantifying the Relationship between Skid Resistance and Wet Weather Accidents for Virginia Data. Master's Thesis. Virginia Polytechnic Institute and State University. Blacksburg, VA.

Larson, R., Hoerner, T.E., Smith, K.D., and Wolters, A.S. 2008. Relationship Between Skid Resistance Numbers Measured With Ribbed and Smooth Tire and Wet Accident Locations. Report No. FHWA/OH-2008/11. Ohio Department of Transportation. Columbus, OH.

Lee, J., Nam, B., and Abdel-Aty, M. 2015. Effects of Pavement Surface Conditions on Traffic Crash Severity. Journal of Transportation Engineering, American Society of Civil Engineers. DOI: 10.1061/(ASCE)TE.1943-5436.0000785.

Li., Y. and Huang, J. 2014. Safety Impact of Pavement Conditions. Transportation Research Record: Journal of the Transportation Research Board, No. 2455, Transportation Research Board, Washington, D.C., pp. 77–88.

Long, K., Wu, H., Zhang, Z., and Murphy, M. 2014. Quantitative Relationship between Crash Risks and Pavement Skid Resistance. Report No. FHWA/TX-13/0-6713-1. Texas Department of Transportation. Austin, TX.

Lord, D. and Mannering, F. 2010. The Statistical Analysis of Crash-Frequency Data: A Review and Assessment of Methodological Alternatives. Transportation Research Part A. Vol. 44. pp. 291-305.

Lu, Q. and Steven, B. 2008. Friction Testing of Pavement Preservation Treatments: Literature Review. Report No. UCPRC-TM-2006-10. California Department of Transportation. CA.

McCarthy, R., Flintsch, G.W., Katicha, S.W., McGhee, K.K., and Medina-Flintsch, A. 2016. New Approach for Managing Pavement Friction and Reducing Road Crashes. Transportation Research Record: Journal of the Transportation Research Board, No. 2591, Transportation Research Board, Washington, D.C., pp. 23–32.

McGovern, C., Rusch, P., and Noyce, D. 2011. State Practices to Reduce Wet Weather Skidding Crashes. Report No. FHWA-SA-11-21. Federal Highway Administration. Washington, D.C.

Musey, K., Park, S., and McFadden, J. 2016. Exploring Friction Modification to Improve the Safety of Horizontal Curve Roadways. International Conference on Transportation and Development. American Society of Civil Engineers.

Najafi, S., Flintsch, G.W., and Khaleghian, S. 2016. Pavement friction management – artificial neural network approach, International Journal of Pavement Engineering, DOI: 10.1080/10298436.2016.1264221

Najafi, S., Flintsch, G.W., and Medina, A. 2017. Linking Roadway Crashes and Tire–Pavement Friction: A Case Study. International Journal of Pavement Engineering, 18:2, 119-127, DOI: 10.1080/10298436.2015.1039005

National Asphalt Pavement Association (NAPA). 2001. HMA Pavement Mix Type Selection Guide. Information Series 128, NAPA. Lanham, MD.

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

Noyce, D.A., Bahia, H.U., Yambo, J., Chapman, J., and Bill, A. 2007. Incorporating Road Safety into Pavement Management: Maximizing Surface Friction for Road Safety Improvements. Report No. MRUTC 04-04. Wisconsin Department of Transportation. Madison, WI.

Sandburg, U. 1998. Influence of Road Surface Texture on Traffic Characteristics Related to Environment, Economy, and Safety: A State-of-the-Art Study Regarding Measures and Measuring Methods. VTI Report 53A-1997, Swedish National Road Administration, Borlange, Sweden.

Sandburg, U. and J.A. Ejsmont, 2002. Tyre/Road Noise Reference Book, ISBN 91-631-2610-9, Informex, Kisa, Sweden.

Scofield, L. 2017. Development and Implementation of the Next Generation Concrete Surface – 2017 Report Living Document, American Concrete Pavement Association.

Sinhal, R., and Viner, H.E. 2008. Evaluating the Success of the UK Skid Policy. Second Safer Roads Conference.

Smit, A.F. and Prozzi, J.A. 2013. Quantification of the Reduction of Wet Weather Accidents Using Porous Friction Courses (PFC). Periodica – Social and Behavioral Sciences. Vol. 96. pp. 2745-2755.

Speir, R., Puzin, T., Barcena, R., and Desaraju, P. 2009. Development of Friction Improvement Policies and Guidelines for the Maryland State Highway Administration. Report No. MD-07-SP708B4F. Maryland State Highway Administration. Baltimore, MD.

Srinivasan, R. and Bauer, K. 2013. Safety Performance Function Development Guide: Developing Jurisdiction-Specific SPFs. Report No. FHWA-SA-14-005. Federal Highway Administration. Washington, D.C.

State Materials Office (SMO). 2017a. Flexible Pavement Condition Survey Handbook. FDOT. Gainesville, FL.

State Materials Office (SMO). 2017b. Rigid Pavement Condition Survey Handbook. FDOT. Gainesville, FL.

Stanard, C., Candaele, R., Charbeneau, R., and Barrett, M. 2007. State of the Practice: Permeable Friction Courses. FHWA/TX-08/0-5220-1. Center for Transportation Research, University of Texas at Austin.

Tehrani, S.S., Falls, L.C., and Mesher, D. 2017. Effects of Pavement Condition on Roadway Safety in the Province of Alberta. Journal of Transportation Safety & Security, Vol. 9, No. 3, pp. 259-272.

Texas Department of Transportation (TxDOT). 2006. Wet Weather Accident Reduction Program (WWARP).

Viner, H., Sinhal, R., and Parry, A. 2004. Review of UK Skid Resistance Policy. Fifth Symposium on Pavement Surface Characteristics for Roads and Airports. Toronto, Ontario, Canada.

Viner, H., Sinhal, R., and Parry, A. 2005. Linking Road Traffic Accidents with Skid Resistance: Recent UK Developments. International Surface Friction Conference: roads and runways: improving safety through assessment and design, Christchurch, New Zealand.

Wallman, C.G., and Astrom, H. 2001. Friction Measurement Methods and the Correlation Between Road Friction and Traffic Safety. Swedish National Road and Transport Research Institute, VTI Meddelande 911A, Linkoping, Sweden.

APPENDIX A

Negative Binomial Regression Results from the Statistical Package R

```
Call:
glm.nb(formula = RD_SRFC_COND_1 ~ log.AADT + log.netlength +
    MRU + IRIa + Profiler_RutAvg + Crack_Rating + Texture_Data +
    FN40R + Speed_Limit + Pavement_Age + District + Aggregate_Type +
    Mix_Type + System_Number + Num_Lanes, data = reduced.table.complete.cases,
    init.theta = 0.3377107046, link = log)

Deviance Residuals:
    Min       1Q   Median       3Q      Max
-2.4606  -0.3733  -0.2045  -0.1164   6.4200

Coefficients:
                          Estimate Std. Error z value Pr(>|z|)
(Intercept)             -1.141e+01  4.271e-01 -26.710  < 2e-16 ***
log.AADT                 1.456e+00  2.869e-02  50.747  < 2e-16 ***
log.netlength            1.164e+00  1.021e-01  11.409  < 2e-16 ***
MRU                      1.028e-04  9.352e-05   1.099 0.271803
IRIa                     8.180e-03  3.482e-04  23.494  < 2e-16 ***
Profiler_RutAvg         -5.228e-01  1.879e-01  -2.783 0.005391 **
Crack_Rating             1.367e-02  1.160e-02   1.179 0.238395
Texture_Data            -9.342e+00  2.655e+00  -3.519 0.000433 ***
FN40R                   -3.852e-02  3.168e-03 -12.160  < 2e-16 ***
Speed_Limit             -2.470e-02  2.264e-03 -10.908  < 2e-16 ***
Pavement_Age             1.713e-02  2.366e-03   7.242 4.42e-13 ***
District2               -1.323e-01  5.078e-02  -2.605 0.009183 **
District3               -2.600e-01  5.727e-02  -4.540 5.62e-06 ***
District4                1.066e-01  6.200e-02   1.720 0.085463 .
District5                1.321e-01  4.999e-02   2.642 0.008237 **
District6                5.223e-01  6.758e-02   7.729 1.08e-14 ***
District7                3.688e-01  5.137e-02   7.179 7.03e-13 ***
Aggregate_TypeLimestone -2.332e-01  5.755e-02  -4.052 5.08e-05 ***
Mix_TypeFC125A          -3.695e-01  2.138e-01  -1.729 0.083890 .
Mix_TypeFC125AR         -2.840e-01  1.211e-01  -2.345 0.019032 *
Mix_TypeFC125M           1.565e-01  6.360e-02   2.460 0.013878 *
Mix_TypeFC125MR          4.127e-01  5.958e-02   6.927 4.31e-12 ***
Mix_TypeFC125MWR         5.337e-01  2.190e-01   2.437 0.014796 *
Mix_TypeFC125R          -3.047e-02  6.716e-02  -0.454 0.650051
Mix_TypeFC125WR          5.176e-01  7.209e-01   0.718 0.472788
Mix_TypeFC5             -2.590e-02  1.172e-01  -0.221 0.825081
Mix_TypeFC5A            -8.338e-01  1.980e-01  -4.211 2.54e-05 ***
Mix_TypeFC5AW           -7.266e-01  7.294e-01  -0.996 0.319227
Mix_TypeFC5M            -7.079e-02  1.256e-01  -0.564 0.572984
Mix_TypeFC5MW           -2.820e-01  2.240e-01  -1.259 0.208033
Mix_TypeFC95             1.641e-01  5.239e-02   3.132 0.001737 **
Mix_TypeFC95A           -5.546e-02  1.002e-01  -0.554 0.579774
Mix_TypeFC95AR          -8.747e-01  3.469e-01  -2.521 0.011689 *
Mix_TypeFC95M            5.605e-02  9.332e-02   0.601 0.548080
Mix_TypeFC95MR           5.464e-01  7.192e-02   7.598 3.00e-14 ***
Mix_TypeFC95MWR          1.016e+00  2.165e-01   4.693 2.70e-06 ***
Mix_TypeFC95R            4.560e-01  9.718e-02   4.692 2.70e-06 ***
System_Number3          -6.618e-01  7.756e-02  -8.533  < 2e-16 ***
System_Number4          -5.235e-01  6.002e-02  -8.721  < 2e-16 ***
Num_Lanes               -3.483e-01  2.475e-02 -14.076  < 2e-16 ***
---
Signif. codes:  0 '***' 0.001 '**' 0.01 '*' 0.05 '.' 0.1 ' ' 1

(Dispersion parameter for Negative Binomial(0.3377) family taken to be 1)

    Null deviance: 46813  on 151134  degrees of freedom
Residual deviance: 32215  on 151095  degrees of freedom
  (37 observations deleted due to missingness)
AIC: 66434

Number of Fisher Scoring iterations: 1


              Theta:  0.33771
          Std. Err.:  0.00972

 2 x log-likelihood:  -66352.00300
```

Figure A.1. Dry Weather Negative Binomial Regression Results [Eqs. (14) and (15)]

```
Call:
glm.nb(formula = RD_SRFC_COND_1 ~ log.AADT + log.netlength +
    IRIa + Profiler_RutAvg + Texture_Data + FN40R + Speed_Limit +
    Pavement_Age + District + Aggregate_Type + Mix_Type + System_Number +
    Num_Lanes, data = reduced.table.complete.cases, init.theta = 0.3374629638,
    link = log)

Deviance Residuals:
    Min       1Q   Median       3Q      Max
-2.4527  -0.3734  -0.2048  -0.1165   6.4236

Coefficients:
                          Estimate Std. Error z value Pr(>|z|)
(Intercept)             -1.129e+01  4.151e-01 -27.195  < 2e-16 ***
log.AADT                 1.457e+00  2.868e-02  50.803  < 2e-16 ***
log.netlength            1.159e+00  1.018e-01  11.383  < 2e-16 ***
IRIa                     8.141e-03  3.472e-04  23.445  < 2e-16 ***
Profiler_RutAvg         -5.405e-01  1.869e-01  -2.892 0.003833 **
Texture_Data            -9.200e+00  2.646e+00  -3.478 0.000506 ***
FN40R                   -3.810e-02  3.143e-03 -12.119  < 2e-16 ***
Speed_Limit             -2.467e-02  2.263e-03 -10.902  < 2e-16 ***
Pavement_Age             1.547e-02  2.022e-03   7.648 2.03e-14 ***
District2               -1.376e-01  5.064e-02  -2.717 0.006594 **
District3               -2.712e-01  5.681e-02  -4.773 1.81e-06 ***
District4                1.078e-01  6.187e-02   1.742 0.081436 .
District5                1.272e-01  4.986e-02   2.552 0.010711 *
District6                5.175e-01  6.744e-02   7.673 1.67e-14 ***
District7                3.683e-01  5.135e-02   7.171 7.43e-13 ***
Aggregate_TypeLimestone -2.361e-01  5.750e-02  -4.106 4.02e-05 ***
Mix_TypeFC125A          -3.870e-01  2.140e-01  -1.809 0.070486 .
Mix_TypeFC125AR         -2.962e-01  1.209e-01  -2.451 0.014262 *
Mix_TypeFC125M           1.540e-01  6.356e-02   2.422 0.015419 *
Mix_TypeFC125MR          4.141e-01  5.944e-02   6.968 3.22e-12 ***
Mix_TypeFC125MWR         5.349e-01  2.189e-01   2.443 0.014566 *
Mix_TypeFC125R          -2.740e-02  6.702e-02  -0.409 0.682650
Mix_TypeFC125WR          5.166e-01  7.210e-01   0.716 0.473697
Mix_TypeFC5             -2.357e-02  1.170e-01  -0.202 0.840289
Mix_TypeFC5A            -8.506e-01  1.978e-01  -4.301 1.70e-05 ***
Mix_TypeFC5AW           -7.079e-01  7.293e-01  -0.971 0.331695
Mix_TypeFC5M            -7.053e-02  1.254e-01  -0.562 0.573822
Mix_TypeFC5MW           -3.103e-01  2.226e-01  -1.394 0.163362
Mix_TypeFC95             1.703e-01  5.227e-02   3.257 0.001126 **
Mix_TypeFC95A           -6.100e-02  9.992e-02  -0.611 0.541527
Mix_TypeFC95AR          -8.847e-01  3.478e-01  -2.543 0.010979 *
Mix_TypeFC95M            6.024e-02  9.327e-02   0.646 0.518398
Mix_TypeFC95MR           5.516e-01  7.172e-02   7.692 1.45e-14 ***
Mix_TypeFC95MWR          1.015e+00  2.165e-01   4.688 2.76e-06 ***
Mix_TypeFC95R            4.505e-01  9.709e-02   4.640 3.49e-06 ***
System_Number3          -6.652e-01  7.745e-02  -8.590  < 2e-16 ***
System_Number4          -5.226e-01  5.998e-02  -8.714  < 2e-16 ***
Num_Lanes               -3.461e-01  2.471e-02 -14.008  < 2e-16 ***
---
Signif. codes:  0 '***' 0.001 '**' 0.01 '*' 0.05 '.' 0.1 ' ' 1

(Dispersion parameter for Negative Binomial(0.3375) family taken to be 1)

    Null deviance: 46804  on 151134  degrees of freedom
Residual deviance: 32211  on 151097  degrees of freedom
  (37 observations deleted due to missingness)
AIC: 66433

Number of Fisher Scoring iterations: 1


              Theta:  0.33746
          Std. Err.:  0.00971

 2 x log-likelihood:  -66354.57400
```

Figure A.2. Dry Weather Negative Binomial Regression Results [Eqs. (16) and (17)]

```
Call:
glm.nb(formula = RD_SRFC_COND_1 ~ log.AADT + log.netlength +
    IRIa + Profiler_RutAvg + Texture_Data + FN40R + Speed_Limit +
    Pavement_Age + District + Aggregate_Type + Mix.Family + System_Number +
    Num_Lanes, data = reduced.table.complete.cases, init.theta = 0.3302996941,
    link = log)

Deviance Residuals:
    Min       1Q   Median       3Q      Max
-2.3873  -0.3747  -0.2046  -0.1174   6.4958

Coefficients:
                          Estimate Std. Error z value Pr(>|z|)
(Intercept)             -1.120e+01  4.122e-01 -27.166  < 2e-16 ***
log.AADT                 1.471e+00  2.859e-02  51.452  < 2e-16 ***
log.netlength            1.130e+00  1.025e-01  11.028  < 2e-16 ***
IRIa                     7.992e-03  3.526e-04  22.666  < 2e-16 ***
Profiler_RutAvg         -8.974e-01  1.863e-01  -4.818 1.45e-06 ***
Texture_Data            -1.206e+01  2.609e+00  -4.621 3.81e-06 ***
FN40R                   -3.777e-02  3.072e-03 -12.296  < 2e-16 ***
Speed_Limit             -2.459e-02  2.265e-03 -10.861  < 2e-16 ***
Pavement_Age             1.394e-02  2.007e-03   6.945 3.78e-12 ***
District2               -8.910e-02  5.007e-02  -1.779 0.075167 .
District3               -1.843e-01  5.299e-02  -3.479 0.000504 ***
District4                3.284e-02  6.063e-02   0.542 0.588116
District5                6.928e-02  4.854e-02   1.427 0.153508
District6                3.911e-01  6.536e-02   5.984 2.18e-09 ***
District7                3.003e-01  4.866e-02   6.172 6.75e-10 ***
Aggregate_TypeLimestone -3.032e-01  5.703e-02  -5.316 1.06e-07 ***
Mix.FamilyFC5           -9.679e-02  1.114e-01  -0.869 0.384858
Mix.FamilyFC95           1.499e-01  3.439e-02   4.358 1.31e-05 ***
System_Number3          -5.789e-01  7.381e-02  -7.844 4.37e-15 ***
System_Number4          -5.264e-01  5.636e-02  -9.340  < 2e-16 ***
Num_Lanes               -3.623e-01  2.480e-02 -14.609  < 2e-16 ***
---
Signif. codes:  0 '***' 0.001 '**' 0.01 '*' 0.05 '.' 0.1 ' ' 1

(Dispersion parameter for Negative Binomial(0.3303) family taken to be 1)

    Null deviance: 45727  on 148653  degrees of freedom
Residual deviance: 31568  on 148633  degrees of freedom
  (2518 observations deleted due to missingness)
AIC: 65311

Number of Fisher Scoring iterations: 1


              Theta:  0.33030
          Std. Err.:  0.00953

 2 x log-likelihood:  -65266.95500
```

Figure A.3. Dry Weather Negative Binomial Regression Results [Eqs. (18) and (19)]

```
Call:
glm.nb(formula = RD_SRFC_COND_1 ~ log.AADT + log.netlength +
    IRIa + Profiler_RutAvg + Texture_Data + FN40R + Speed_Limit +
    Pavement_Age + Aggregate_Type + Mix.Family + System_Number +
    Num_Lanes, data = reduced.table.complete.cases, init.theta = 0.3236176294,
    link = log)

Deviance Residuals:
    Min       1Q   Median       3Q      Max
-2.4587  -0.3765  -0.2051  -0.1186   6.4109

Coefficients:
                          Estimate Std. Error z value Pr(>|z|)
(Intercept)             -1.178e+01  4.079e-01 -28.879  < 2e-16 ***
log.AADT                 1.544e+00  2.816e-02  54.824  < 2e-16 ***
log.netlength            1.158e+00  1.023e-01  11.322  < 2e-16 ***
IRIa                     8.749e-03  3.475e-04  25.176  < 2e-16 ***
Profiler_RutAvg         -9.135e-01  1.836e-01  -4.977 6.46e-07 ***
Texture_Data            -1.633e+01  2.559e+00  -6.380 1.77e-10 ***
FN40R                   -3.625e-02  3.053e-03 -11.872  < 2e-16 ***
Speed_Limit             -2.580e-02  2.200e-03 -11.731  < 2e-16 ***
Pavement_Age             1.444e-02  2.010e-03   7.182 6.86e-13 ***
Aggregate_TypeLimestone -1.822e-01  3.264e-02  -5.582 2.38e-08 ***
Mix.FamilyFC5            1.435e-01  1.090e-01   1.316    0.188
Mix.FamilyFC95           1.318e-01  3.183e-02   4.142 3.44e-05 ***
System_Number3          -6.373e-01  7.148e-02  -8.915  < 2e-16 ***
System_Number4          -6.108e-01  5.534e-02 -11.039  < 2e-16 ***
Num_Lanes               -3.901e-01  2.430e-02 -16.051  < 2e-16 ***
---
Signif. codes:  0 '***' 0.001 '**' 0.01 '*' 0.05 '.' 0.1 ' ' 1

(Dispersion parameter for Negative Binomial(0.3236) family taken to be 1)

    Null deviance: 45477  on 148653  degrees of freedom
Residual deviance: 31578  on 148639  degrees of freedom
  (2518 observations deleted due to missingness)
AIC: 65496

Number of Fisher Scoring iterations: 1


              Theta:  0.32362
          Std. Err.:  0.00929

 2 x log-likelihood:  -65464.09100
```

Figure A.4. Dry Weather Negative Binomial Regression Results [Eqs. (20) and (21)]

```
Call:
glm.nb(formula = RD_SRFC_COND_2 ~ log.AADT + log.netlength +
    MRU + IRIa + Profiler_RutAvg + Crack_Rating + Texture_Data +
    FN40R + Speed_Limit + Pavement_Age + District + Aggregate_Type +
    Mix_Type + System_Number + Num_Lanes, data = reduced.table.complete.cases,
    init.theta = 0.2688058165, link = log)

Deviance Residuals:
    Min       1Q   Median       3Q      Max
-1.8559  -0.1831  -0.1054  -0.0573   4.8699

Coefficients:
                          Estimate Std. Error z value Pr(>|z|)
(Intercept)             -1.274e+01  8.608e-01 -14.805  < 2e-16 ***
log.AADT                 1.417e+00  5.927e-02  23.910  < 2e-16 ***
log.netlength            9.057e-01  1.971e-01   4.594 4.34e-06 ***
MRU                     -1.060e-04  1.892e-04  -0.560 0.575283
IRIa                     6.933e-03  6.003e-04  11.550  < 2e-16 ***
Profiler_RutAvg          7.348e-02  3.690e-01   0.199 0.842175
Crack_Rating            -2.026e-02  2.344e-02  -0.864 0.387359
Texture_Data            -7.521e+00  5.190e+00  -1.449 0.147309
FN40R                   -5.633e-02  6.666e-03  -8.450  < 2e-16 ***
Speed_Limit             -1.597e-02  4.716e-03  -3.387 0.000708 ***
Pavement_Age             4.855e-03  5.069e-03   0.958 0.338117
District2                1.890e-01  1.045e-01   1.809 0.070519 .
District3               -1.126e-01  1.234e-01  -0.913 0.361344
District4                1.762e-01  1.236e-01   1.425 0.154047
District5                3.234e-01  1.035e-01   3.125 0.001778 **
District6                4.265e-01  1.334e-01   3.198 0.001386 **
District7                4.030e-01  1.091e-01   3.693 0.000221 ***
Aggregate_TypeLimestone -2.033e-01  1.140e-01  -1.783 0.074553 .
Mix_TypeFC125A          -9.387e-01  6.082e-01  -1.543 0.122757
Mix_TypeFC125AR         -1.160e+00  3.581e-01  -3.240 0.001196 **
Mix_TypeFC125M           1.232e-02  1.252e-01   0.098 0.921587
Mix_TypeFC125MR          8.329e-02  1.231e-01   0.677 0.498595
Mix_TypeFC125MWR        -6.676e-01  5.480e-01  -1.218 0.223133
Mix_TypeFC125R          -3.091e-01  1.443e-01  -2.142 0.032185 *
Mix_TypeFC125WR          1.660e+00  1.017e+00   1.631 0.102821
Mix_TypeFC5              9.405e-02  2.267e-01   0.415 0.678225
Mix_TypeFC5A            -2.577e-01  3.400e-01  -0.758 0.448497
Mix_TypeFC5AW           -3.149e+01  4.107e+06   0.000 0.999994
Mix_TypeFC5M            -1.547e-01  2.449e-01  -0.632 0.527641
Mix_TypeFC5MW           -3.897e-01  4.311e-01  -0.904 0.366050
Mix_TypeFC95            -3.321e-02  1.027e-01  -0.323 0.746423
Mix_TypeFC95A           -1.210e-01  1.925e-01  -0.629 0.529561
Mix_TypeFC95AR          -3.362e+01  6.229e+06   0.000 0.999996
Mix_TypeFC95M           -3.543e-01  2.084e-01  -1.700 0.089126 .
Mix_TypeFC95MR           2.195e-01  1.550e-01   1.416 0.156720
Mix_TypeFC95MWR          1.308e+00  3.760e-01   3.478 0.000506 ***
Mix_TypeFC95R            1.202e-01  2.121e-01   0.567 0.570833
System_Number3          -3.305e-01  1.429e-01  -2.313 0.020705 *
System_Number4          -4.696e-01  1.165e-01  -4.032 5.53e-05 ***
Num_Lanes               -2.995e-01  5.067e-02  -5.911 3.41e-09 ***
---
Signif. codes:  0 '***' 0.001 '**' 0.01 '*' 0.05 '.' 0.1 ' ' 1

(Dispersion parameter for Negative Binomial(0.2688) family taken to be 1)

    Null deviance: 13894  on 151134  degrees of freedom
Residual deviance: 11201  on 151095  degrees of freedom
  (37 observations deleted due to missingness)
AIC: 18897

Number of Fisher Scoring iterations: 1


              Theta:  0.2688
          Std. Err.:  0.0279

 2 x log-likelihood:  -18815.1560
```

Figure A.5. Wet Weather Negative Binomial Regression Results [Eqs. (14) and (15)]

```
Call:
glm.nb(formula = RD_SRFC_COND_2 ~ log.AADT + log.netlength +
    IRIa + Texture_Data + FN40R + Speed_Limit + District + Aggregate_Type +
    Mix_Type + System_Number + Num_Lanes, data = reduced.table.complete.cases,
    init.theta = 0.2680176125, link = log)

Deviance Residuals:
    Min       1Q   Median       3Q      Max
-1.8731  -0.1833  -0.1051  -0.0574   4.8444

Coefficients:
                          Estimate Std. Error z value Pr(>|z|)
(Intercept)             -1.291e+01  8.275e-01 -15.606  < 2e-16 ***
log.AADT                 1.415e+00  5.910e-02  23.949  < 2e-16 ***
log.netlength            8.899e-01  1.958e-01   4.546 5.47e-06 ***
IRIa                     7.009e-03  5.948e-04  11.784  < 2e-16 ***
Texture_Data            -8.192e+00  5.168e+00  -1.585 0.112916
FN40R                   -5.708e-02  6.530e-03  -8.741  < 2e-16 ***
Speed_Limit             -1.605e-02  4.720e-03  -3.400 0.000675 ***
District2                2.021e-01  1.036e-01   1.951 0.051056 .
District3               -9.377e-02  1.228e-01  -0.764 0.445006
District4                1.641e-01  1.223e-01   1.342 0.179543
District5                3.299e-01  1.033e-01   3.195 0.001398 **
District6                4.297e-01  1.323e-01   3.249 0.001160 **
District7                4.060e-01  1.091e-01   3.721 0.000199 ***
Aggregate_TypeLimestone -1.995e-01  1.131e-01  -1.763 0.077817 .
Mix_TypeFC125A          -8.647e-01  6.057e-01  -1.428 0.153393
Mix_TypeFC125AR         -1.134e+00  3.569e-01  -3.176 0.001493 **
Mix_TypeFC125M           1.260e-02  1.249e-01   0.101 0.919651
Mix_TypeFC125MR          6.967e-02  1.219e-01   0.571 0.567756
Mix_TypeFC125MWR        -7.068e-01  5.478e-01  -1.290 0.196961
Mix_TypeFC125R          -3.330e-01  1.431e-01  -2.326 0.020008 *
Mix_TypeFC125WR          1.641e+00  1.017e+00   1.614 0.106562
Mix_TypeFC5              1.020e-01  2.263e-01   0.451 0.652287
Mix_TypeFC5A            -2.032e-01  3.383e-01  -0.601 0.548148
Mix_TypeFC5AW           -3.148e+01  4.107e+06   0.000 0.999994
Mix_TypeFC5M            -1.437e-01  2.443e-01  -0.588 0.556352
Mix_TypeFC5MW           -3.620e-01  4.268e-01  -0.848 0.396275
Mix_TypeFC95            -5.155e-02  1.022e-01  -0.505 0.613821
Mix_TypeFC95A           -6.619e-02  1.901e-01  -0.348 0.727699
Mix_TypeFC95AR          -3.359e+01  6.231e+06   0.000 0.999996
Mix_TypeFC95M           -3.446e-01  2.082e-01  -1.655 0.097996 .
Mix_TypeFC95MR           2.020e-01  1.544e-01   1.308 0.190793
Mix_TypeFC95MWR          1.279e+00  3.753e-01   3.409 0.000651 ***
Mix_TypeFC95R            1.147e-01  2.112e-01   0.543 0.587192
System_Number3          -3.210e-01  1.423e-01  -2.257 0.024024 *
System_Number4          -4.663e-01  1.163e-01  -4.008 6.11e-05 ***
Num_Lanes               -2.995e-01  5.051e-02  -5.929 3.04e-09 ***
---
Signif. codes:  0 '***' 0.001 '**' 0.01 '*' 0.05 '.' 0.1 ' ' 1

(Dispersion parameter for Negative Binomial(0.268) family taken to be 1)

    Null deviance: 13887  on 151134  degrees of freedom
Residual deviance: 11199  on 151099  degrees of freedom
  (37 observations deleted due to missingness)
AIC: 18893

Number of Fisher Scoring iterations: 1


              Theta:  0.2680
          Std. Err.:  0.0278

 2 x log-likelihood:  -18819.1290
```

Figure A.6. Wet Weather Negative Binomial Regression Results [Eqs. (16) and (17)]

```
Call:
glm.nb(formula = RD_SRFC_COND_2 ~ log.AADT + log.netlength +
    IRIa + Texture_Data + FN40R + Speed_Limit + District + Aggregate_Type +
    Mix_Type + System_Number + Num_Lanes, data = reduced.table.complete.cases,
    init.theta = 0.2680176125, link = log)

Deviance Residuals:
    Min       1Q   Median       3Q      Max
-1.8731  -0.1833  -0.1051  -0.0574   4.8444

Coefficients:
                          Estimate Std. Error z value Pr(>|z|)
(Intercept)             -1.291e+01  8.275e-01 -15.606  < 2e-16 ***
log.AADT                 1.415e+00  5.910e-02  23.949  < 2e-16 ***
log.netlength            8.899e-01  1.958e-01   4.546 5.47e-06 ***
IRIa                     7.009e-03  5.948e-04  11.784  < 2e-16 ***
Texture_Data            -8.192e+00  5.168e+00  -1.585 0.112916
FN40R                   -5.708e-02  6.530e-03  -8.741  < 2e-16 ***
Speed_Limit             -1.605e-02  4.720e-03  -3.400 0.000675 ***
District2                2.021e-01  1.036e-01   1.951 0.051056 .
District3               -9.377e-02  1.228e-01  -0.764 0.445006
District4                1.641e-01  1.223e-01   1.342 0.179543
District5                3.299e-01  1.033e-01   3.195 0.001398 **
District6                4.297e-01  1.323e-01   3.249 0.001160 **
District7                4.060e-01  1.091e-01   3.721 0.000199 ***
Aggregate_TypeLimestone -1.995e-01  1.131e-01  -1.763 0.077817 .
Mix_TypeFC125A          -8.647e-01  6.057e-01  -1.428 0.153393
Mix_TypeFC125AR         -1.134e+00  3.569e-01  -3.176 0.001493 **
Mix_TypeFC125M           1.260e-02  1.249e-01   0.101 0.919651
Mix_TypeFC125MR          6.967e-02  1.219e-01   0.571 0.567756
Mix_TypeFC125MWR        -7.068e-01  5.478e-01  -1.290 0.196961
Mix_TypeFC125R          -3.330e-01  1.431e-01  -2.326 0.020008 *
Mix_TypeFC125WR          1.641e+00  1.017e+00   1.614 0.106562
Mix_TypeFC5              1.020e-01  2.263e-01   0.451 0.652287
Mix_TypeFC5A            -2.032e-01  3.383e-01  -0.601 0.548148
Mix_TypeFC5AW           -3.148e+01  4.107e+06   0.000 0.999994
Mix_TypeFC5M            -1.437e-01  2.443e-01  -0.588 0.556352
Mix_TypeFC5MW           -3.620e-01  4.268e-01  -0.848 0.396275
Mix_TypeFC95            -5.155e-02  1.022e-01  -0.505 0.613821
Mix_TypeFC95A           -6.619e-02  1.901e-01  -0.348 0.727699
Mix_TypeFC95AR          -3.359e+01  6.231e+06   0.000 0.999996
Mix_TypeFC95M           -3.446e-01  2.082e-01  -1.655 0.097996 .
Mix_TypeFC95MR           2.020e-01  1.544e-01   1.308 0.190793
Mix_TypeFC95MWR          1.279e+00  3.753e-01   3.409 0.000651 ***
Mix_TypeFC95R            1.147e-01  2.112e-01   0.543 0.587192
System_Number3          -3.210e-01  1.423e-01  -2.257 0.024024 *
System_Number4          -4.663e-01  1.163e-01  -4.008 6.11e-05 ***
Num_Lanes               -2.995e-01  5.051e-02  -5.929 3.04e-09 ***
---
Signif. codes:  0 '***' 0.001 '**' 0.01 '*' 0.05 '.' 0.1 ' ' 1

(Dispersion parameter for Negative Binomial(0.268) family taken to be 1)

    Null deviance: 13887  on 151134  degrees of freedom
Residual deviance: 11199  on 151099  degrees of freedom
  (37 observations deleted due to missingness)
AIC: 18893

Number of Fisher Scoring iterations: 1


              Theta:  0.2680
          Std. Err.:  0.0278

 2 x log-likelihood:  -18819.1290
```

Figure A.7. Wet Weather Negative Binomial Regression Results [Eqs. (18) and (19)]

```
Call:
glm.nb(formula = RD_SRFC_COND_2 ~ log.AADT + log.netlength +
    IRIa + Texture_Data + FN40R + Speed_Limit + Aggregate_Type +
    Mix.Family + System_Number + Num_Lanes, data = reduced.table.complete.cases,
    init.theta = 0.256689294, link = log)

Deviance Residuals:
    Min       1Q   Median       3Q      Max
-1.8462  -0.1864  -0.1052  -0.0582   4.8515

Coefficients:
                          Estimate Std. Error z value Pr(>|z|)
(Intercept)             -1.292e+01  8.258e-01 -15.642  < 2e-16 ***
log.AADT                 1.477e+00  5.785e-02  25.525  < 2e-16 ***
log.netlength            9.719e-01  2.077e-01   4.678 2.89e-06 ***
IRIa                     7.430e-03  5.934e-04  12.521  < 2e-16 ***
Texture_Data            -1.439e+01  5.014e+00  -2.870  0.00410 **
FN40R                   -5.843e-02  6.217e-03  -9.400  < 2e-16 ***
Speed_Limit             -1.714e-02  4.541e-03  -3.775  0.00016 ***
Aggregate_TypeLimestone -1.727e-01  6.262e-02  -2.758  0.00582 **
Mix.FamilyFC5            3.198e-01  2.142e-01   1.493  0.13534
Mix.FamilyFC95           1.960e-03  6.765e-02   0.029  0.97688
System_Number3          -4.015e-01  1.284e-01  -3.126  0.00177 **
System_Number4          -6.282e-01  1.075e-01  -5.843 5.12e-09 ***
Num_Lanes               -3.420e-01  4.910e-02  -6.966 3.26e-12 ***
---
Signif. codes:  0 '***' 0.001 '**' 0.01 '*' 0.05 '.' 0.1 ' ' 1

(Dispersion parameter for Negative Binomial(0.2567) family taken to be 1)

    Null deviance: 13565  on 148653  degrees of freedom
Residual deviance: 11007  on 148641  degrees of freedom
  (2518 observations deleted due to missingness)
AIC: 18633

Number of Fisher Scoring iterations: 1


              Theta:  0.2567
          Std. Err.:  0.0265

 2 x log-likelihood:  -18604.7880
```

Figure A.8. Wet Weather Negative Binomial Regression Results [Eqs. (20) and (21)]

```
Call:
glm.nb(formula = RD_SRFC_COND_1 ~ log.AADT + log.netlength +
    IRIa + Faulting + Prcnt_Crackd_Slabs + FN40R + Speed_Limit +
    System_Number + Num_Lanes, data = reduced.table.complete.cases.rigid,
    init.theta = 0.3563100637, link = log)

Deviance Residuals:
    Min       1Q   Median       3Q      Max
-1.2467  -0.6157  -0.4229  -0.1750   6.8343

Coefficients:
                     Estimate Std. Error z value Pr(>|z|)
(Intercept)        -9.9071504  1.1809492  -8.389  < 2e-16 ***
log.AADT            1.3314623  0.0975924  13.643  < 2e-16 ***
log.netlength       0.7914663  0.1873943   4.224 2.41e-05 ***
IRIa                0.0051050  0.0008678   5.883 4.03e-09 ***
Faulting           -0.6531013  2.6261982  -0.249    0.804
Prcnt_Crackd_Slabs -0.3990416  0.0600808  -6.642 3.10e-11 ***
FN40R              -0.0739621  0.0079665  -9.284  < 2e-16 ***
Speed_Limit        -0.0280648  0.0050982  -5.505 3.70e-08 ***
System_Number3     -0.2543482  0.3637134  -0.699    0.484
System_Number4     -0.8356088  0.1781545  -4.690 2.73e-06 ***
Num_Lanes           0.0021184  0.0410006   0.052    0.959
---
Signif. codes:  0 '***' 0.001 '**' 0.01 '*' 0.05 '.' 0.1 ' ' 1

(Dispersion parameter for Negative Binomial(0.3563) family taken to be 1)

    Null deviance: 5435.8  on 9829  degrees of freedom
Residual deviance: 4083.8  on 9819  degrees of freedom
AIC: 9923.1

Number of Fisher Scoring iterations: 1


              Theta:  0.3563
          Std. Err.:  0.0210

 2 x log-likelihood:  -9899.1450
```

Figure A.9. Rigid Dry Weather Negative Binomial Regression Results [Eqs. (22) and (23)]

```
Call:
glm.nb(formula = RD_SRFC_COND_2 ~ log.AADT + log.netlength +
    IRIa + Faulting + Prcnt_Crackd_Slabs + FN40R + Speed_Limit +
    System_Number + Num_Lanes, data = reduced.table.complete.cases,
    init.theta = 0.4575614122, link = log)

Deviance Residuals:
    Min       1Q   Median       3Q      Max
-0.7424  -0.3024  -0.1946  -0.1198   3.4871

Coefficients:
                    Estimate Std. Error z value Pr(>|z|)
(Intercept)        -9.569519   2.543135  -3.763 0.000168 ***
log.AADT            1.316742   0.203304   6.477 9.37e-11 ***
log.netlength       1.014714   0.489213   2.074 0.038063 *
IRIa                0.002973   0.001637   1.816 0.069308 .
Faulting           -2.550851   5.470227  -0.466 0.640990
Prcnt_Crackd_Slabs -0.389765   0.132078  -2.951 0.003167 **
FN40R              -0.094516   0.016606  -5.692 1.26e-08 ***
Speed_Limit        -0.038131   0.010044  -3.796 0.000147 ***
System_Number3     -0.705445   0.828373  -0.852 0.394434
System_Number4     -0.428582   0.361692  -1.185 0.236043
Num_Lanes          -0.058441   0.074635  -0.783 0.433615
---
Signif. codes:  0 '***' 0.001 '**' 0.01 '*' 0.05 '.' 0.1 ' ' 1

(Dispersion parameter for Negative Binomial(0.4576) family taken to be 1)

    Null deviance: 2028.2  on 9829  degrees of freedom
Residual deviance: 1647.1  on 9819  degrees of freedom
AIC: 2843.4

Number of Fisher Scoring iterations: 1


              Theta:  0.458
          Std. Err.:  0.109

 2 x log-likelihood:  -2819.424
```

Figure A.10. Rigid Wet Weather Negative Binomial Regression Results [Eqs. (22) and (23)]