Improvement to Highway Safety through Network Level Friction Testing and Cost Effective Pavement Maintenance

by

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AUTHOR'S DECLARATION

I hereby declare that I am the sole author of this thesis. This is a true copy of the thesis, including any required final revisions, as accepted by my examiners.

I understand that my thesis may be made electronically available to the public.

Abstract

Pavements encompass a significant component of the total civil infrastructure investment. In Ontario, the Ministry of Transportation (MTO) is responsible for the maintenance and construction of approximately 39,000 lane-kilometres of highway. In 2004, the province estimated the value of the total highway system at $39 billion dollars. Thus, managing this asset is an important factor to ensure a high level of service to the traveling public. One of the most important indicators of level of service for a road network is safety. Each year, thousands of motorists across North America are involved in motor vehicle collisions, which result in property damage, congestion, delays, injuries and fatalities. The MTO estimated that in 2002, vehicle collisions in Ontario cost nearly $11 billion.

Despite the importance of highway safety, it is usually not considered explicitly in the pavement management framework or maintenance analysis. A number of agencies across North America collect skid data to assess the level of safety at both the project and network level (Li et al, 2004). However, a number of transportation agencies still do not collect friction data as part of their regular pavement data collection programs. This is related to both liability concerns and lack of knowledge for how this data can be effectively used to improve safety. The transportation industry generally relies on information such as collision rates, black-spot locations and radius of curvature to evaluate the level of safety of an alignment (Lamm et al., 1999). These are important factors, but the use of complementary skid data in an organized proactive manner would also be beneficial.

In preparation for a considered Long Term Area Maintenance Contract, a project was initiated by the MTO to collect network level friction data across three regions in the Province of Ontario. This project represents the first time friction data was collected at the network level in Ontario. In 2006, approximately 1,800 km of the MTO highway network was surveyed as a part of this study. This research utilized the network level skid data along with collision data to examine the

relationships and model the impacts of skid resistance on the level of safety. Despite the value of collecting network level skid data, many Canadian transportation agencies still do not collect network level skid data due to the costs and potential liability associated with the collected data.

The safety of highway networks are usually assessed using various levels of service indicators such as Wet-to-Dry accident ratio (W/D), surface friction (SN), or the collision rate (CR). This research focused on developing a framework for assessing the level of safety of a highway network in terms of the risk of collision based on pavement surface friction. The developed safety framework can be used by transportation agencies (federal, state, provincial, municipal, etc.) or the private sector to evaluate the safety of their highway networks and to determine the risk or probability of a collision occurring given the level of friction along the pavement section of interest. As a part of the analysis, a number of factors such as Region, Season of the Year, Environmental Conditions, Road Surface Condition, Collision Severity, Visibility and Roadway Location were all investigated. Statistical analysis and modeling were performed to developed relationships which could relate the total number of collisions or the collision rate (CR) to the level of available pavement friction on a highway section. These models were developed using over 1,200 collisions and skid test results from two Regions in the Province of Ontario. Another component of this study examined the Wet-to-Dry accident ratio and compared it to the Skid Number. A number of Transportation Agencies rely on the Wet-to-Dry accident ratio to identify potential locations with poor skid resistance. The results of the comparison further demonstrated the need and importance of collecting network level skid data.

Another component of this study was to evaluate the effectiveness of various preservation treatments used within the Long Term Pavement Performance (LTPP) study. In addition, modeling was performed which examined the historical friction trends over time within various environment zones across North America to investigate skid resistance deterioration trends. The results of the analysis demonstrated that commonly used preservation treatments can increase skid resistance and improve safety.

The cost effectiveness of implementing preservation and maintenance to increase the level of safety of a highway using Life Cycle Cost Analysis (LCCA) was evaluated. A Decision Making Framework was developed which included the formulation of a Decision Matrix that can be used to assist in selecting a preservation treatment for a given condition. The results of this analysis demonstrate the savings generated by reducing the number of collisions as a result of increasing skid resistance.

The results of this research study have demonstrated the importance of network level friction testing and the impact of skid resistance on the level of safety of a highway. A review of the literature did not reveal any protocol or procedures for sampling or minimum test interval requirements for network level skid testing using a locked-wheel tester. Network level friction testing can be characterized as expensive and time-consuming due to the complexity of the test. As a result, any reduction in the required number of test points is a benefit to the transportation agency, private sector (consultants and contractors) and most importantly, the public. An analysis approach was developed and tested that can be used to minimize the number of required test locations along a highway segment using common statistical techniques.

Acknowledgements

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- To my family: My father, Halim; My Mother, Beverley; My Sister, Vanessa; my Brothers Karim and Tarek; and My Seto; Sooad…your unconditional love and support throughout my life has been so valuable. To all of you, I thank you from the bottom of my heart.
- Most importantly….to my best friend, wife and soul mate Ilona Baranyai and my beautiful daughter Mya Ilona Abd El Halim, you both have always been there to support me, encourage me, love me and pick me up when I was down and out. You both know first hand what *we* went through to get this done. Thanks for your sacrifices and support…without you; this would not have been possible.

Sincerely with thanks…Amir

Dedication

In the name of God, the Beneficent, the Merciful…

…This thesis and Ph.D. is dedicated to my two *precious* daughters Mya Ilona Abd El Halim and Lily Amira Abd El Halim. You have both given me the inspiration and final push to finish this thesis and I love you always and forever and beyond my last breath…

Invictus (Unconquered)

Out of the night that covers me,
Black as the pit from pole to pole,
I thank whatever Gods may be
For my unconquerable soul.

In the fell clutch of circumstance
I have not winced nor cried aloud.
Under the bludgeonings of chance
My head is bloody, but unbowed.

Beyond this place of wrath and tears
Looms but the Horror of the shade,
And yet the menace of the years
Finds and shall find me unafraid.

It matters not how strait the gate,
How charged with punishments the scroll,
I am the master of my fate:
I am the captain of my soul.

William Ernest Henley

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Chapter 1
Introduction

This chapter provides background information related to pavement engineering and safety management. The problem statement is presented and deficiencies in the current practice are outlined. The research work plan is defined and a summary of each chapter is also provided.

1.1 Background

Civil infrastructure is a vital component of our society's health, safety, and economy. Each year, billions of taxpayer's dollars are invested in constructing, maintaining, and rehabilitating all forms of civil infrastructure. In Canada, civil infrastructure represents a $1.6 trillion dollar asset (CSCE, 2003). Despite the significant investment in civil infrastructure, a $60 billion dollar backlog in municipal infrastructure exists and 79% of the infrastructure has reached the end of its service life. In the United States, this problem is even more profound. According to the American Society of Civil Engineers (ASCE), a total budget of $1.6 trillion dollars is required over the next 5 years. Furthermore, deteriorated roads cost American motorists $54 billion dollars in repair and operating costs each year (ASCE, 2005).

Roads and pavements are an integral component of civil infrastructure and our nation's highway system. Everyday, billions of dollars in goods and services are transported along our highway network. The Canadian transportation network is comprised of 1,042,300 kilometres of roadway, of which 415,600 km are paved (including 17,000 km of highway) and 626,700 km of unpaved roadway (CIA, 2007). The Ontario Ministry of Transportation (MTO) is responsible for the maintenance and construction of approximately 39,000 lane-kilometres of highway. In 2004, the Province estimated the value of the total highway system at $39 billion dollars (OAGO, 2005). Due to the size and significance of this considerable infrastructure asset, cost-effective maintenance and management of this asset is essential.

One of the most important indicators of level of service for a road network is safety. Each year, thousands of motorists across North America are involved in motor vehicle collisions which result in property damage, congestion, delays, injuries and fatalities (Lamm et. al, 1999). The MTO estimated that in 2002, vehicle collisions in Ontario cost nearly $11 billion. It also estimated that for every dollar spent on traffic management, 10 times that amount could be saved on collision-related expenditures, including health care and insurance claims (OAGO, 2005). Collision data for the province of Ontario from 1996 to 2004 are presented in Figures 1.1 to 1.4.

In 2000, all of the provincial and territorial agencies in Canada endorsed the Road Safety Vision 2010. The aim of this national initiative is to make Canadian roads among the safest in the world and to reduce the average number of deaths and serious injuries resulting from motor vehicle collisions by 30% (OAGA, 2005). Despite these ambitions, a safety problem still exists on our nation's roadways.

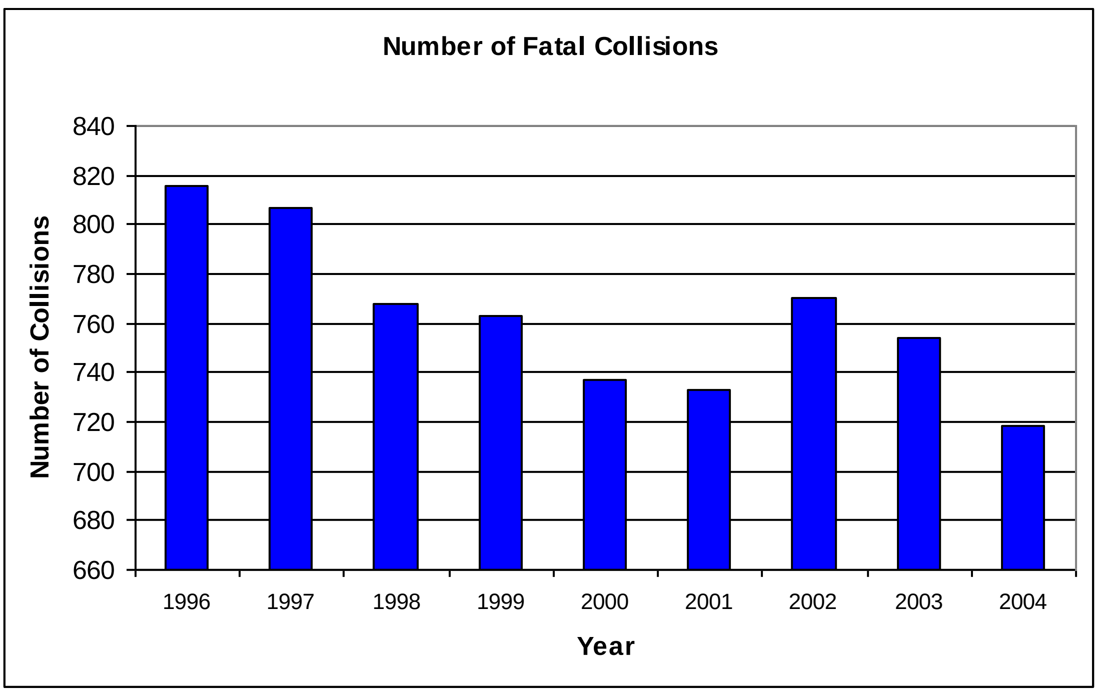

Figure 1.1: Number of Fatal Collisions (OAGO, 2005)

Figure 1.2: Number of Collisions Resulting in Personal Injury (OAGO, 2005)

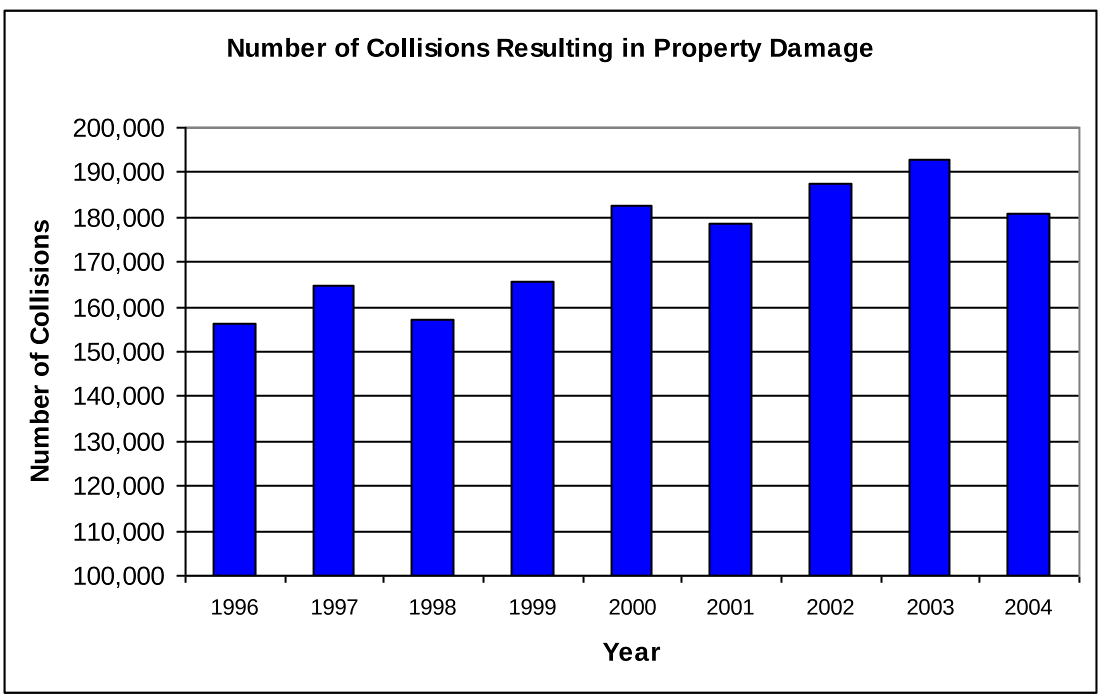

Figure 1.3: Number of Collisions Resulting in Property Damage (OAGO, 2005)

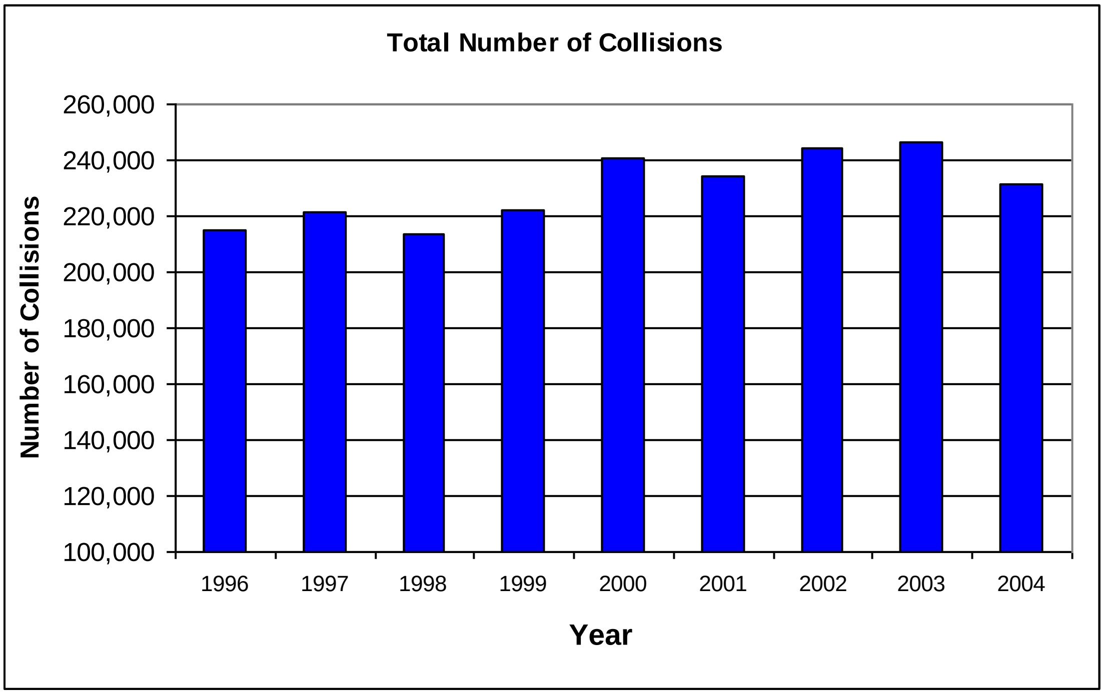

Figure 1.4: Total Number of Collisions (OAGO, 2005)

In 1996, there were 215,024 motor vehicle collisions reported in Ontario consisting of 816 fatal collisions, 57,791 collisions resulting in personal injury, and 156,417 collisions resulting in property damage only. In 2004, there were 231,548 motor vehicle collisions consisting of 718 fatal collisions, 49,948 collisions resulting in personal injury, and 180,882 collisions resulting in property damage. From 1996 to 2004, an increasing trend in the total number of collisions is observed in the Province of Ontario (Figure 1.4). Although the major automobile manufacturers are designing and building safer vehicles with safety systems such as airbag systems, antilock braking systems (ABS) and traction control, the total number of collisions and fatalities is alarming.

Despite the fact that the number of fatalities on Ontario highways decreased slightly from 1999 to 2004, the total number of motor vehicle collisions increased by almost 10%. A strong relationship exists between safety, highway design and pavement performance. Outdated and poor geometric design practices along with deteriorated pavement conditions influence the safety of a highway alignment. In the United States, over 43,000 fatalities occur on the nation’s roadways and

30% of all fatal highway collisions can be attributed to these factors (ASCE, 2005). As a result of this complex relationship, an effective methodology and approach are required to evaluate, analyze, research and manage our highway networks. The fundamental purpose is to provide the public with a safe, efficient, and cost effective, transportation system.

1.2 Problem Statement

As the volume of traffic and movement of goods on our highway network continues to increase, the need for an effective management framework related to pavements and safety is even more crucial. Each year, there are approximately 60,000 collisions and over 700 fatalities on Ontario's highway network (ORSAR, 2005). These statistics do not include the number of collisions or fatalities at the municipal or county level or for other provinces or transportation agencies across Canada. The cost of these collisions in terms of property damage, injury, insurance costs, traffic delays, emissions, and many other items is substantial. Therefore, finding ways to reduce or mitigate the number of collisions and fatalities should be a priority and is generally a part of every Canadian transportation agency's mission statement.

Several limitations and drawbacks to the current-state-of-the-practice of pavement management exist. As mentioned previously in the Section 1.1, pavements encompass a significant component of our nation's total civil infrastructure investment. As our nation's population and economy continues to grow and develop, our dependence on the highway network as a major mode of transportation will continue to increase. Along with increased traffic volumes comes a significant increase in traffic loadings which has a profound effect on the overall condition of a pavement structure. Both the functional and structural performance of a pavement is significantly affected by traffic loadings. Furthermore, with most of our highway networks at or near their intended design capacity, highway safety is becoming a greater concern. A good example of this is Highway 401 through the City of Toronto. During peak hours, this vital section of highway is at or beyond a Level of Service E as defined by the Highway Capacity Manual. A single minor collision on this section of highway can create significant user delays, increased emissions, and economic loss. Therefore, effective pavement and safety management practices are required to ensure cost-effective decision-making and a high level of service are offered to the end user.

Traditionally, a reactive approach rather than a proactive approach towards safety has been accepted and practiced across the pavement and highway industry. Furthermore, safety is not always considered explicitly in the pavement management process. If a location along a highway section is considered a “black-spot” due to a high number of collisions over a given period of time, then an agency may consider corrective action. Improving sight distance, increasing the radius of a horizontal curve and improving surface friction (skid resistance) are all examples of corrective improvements that can be performed as a part of a proactive approach towards improving the level of safety.

A reactive approach is not always cost-effective or a good example of sound pavement or safety management. A proactive approach considers all factors such as environmental conditions, pavement performance factors, driver behaviour, vehicle dynamics and geometric design considerations and offers the most cost effective solution to the pavement and safety problem. In short, it is a comprehensive approach that includes several factors. The questions that need to be answered include: “How can safety be integrated within a Pavement Management System (PMS) effectively?”, “Why do some highway sections experience higher collision rates?”, “What factors influence safety”, and “What can be done to limit or minimize the number of collisions through pavement engineering practices?”

1.2.1 Deficiencies in Existing Practice

Current pavement management practices offer a significant improvement over their predecessors. In the past, agencies typically based their selection of candidate projects using a worst first approach with little or no use of prioritization or optimization in the selection process. This approach does not provide the most cost-effective results. Today, many agencies use an index such as the International Roughness Index (IRI) or Pavement Condition Index (PCI) to rate their pavement sections and trigger Maintenance, Rehabilitation and Reconstruction (M, R&R) activities. The MTO relies on the Distress Manifestation Index (DMI) and the Ride Comfort Index (RCI) to trigger pavement sections for M, R&R activities. These pavement performance indices do not consider the impact of collisions or the economic benefits of reducing collisions.

Safety is usually not considered explicitly in the pavement management framework or maintenance analysis. A number of agencies across North America collect skid data to assess the level of safety at both the project and network level (Li et al, 2004). However, a number of transportation agencies still do not collect friction data as part of their regular pavement data collection programs. This is related to both liability concerns and lack of knowledge for how this data can be effectively used to improve safety. The transportation industry generally relies on information such as collision rates, black-spot locations and radius of curvature to evaluate the level of safety of an alignment (Lamm et al., 1999). These are important factors, but the usage of complementary skid data in an organized proactive manner would also be beneficial.

Another issue lies directly within the various transportation agencies. Many State and Provincial Departments of Transportation (DOTs) have two separate departments within the agency that are responsible for handling pavement and safety management. These two groups typically work independently, sometimes at different geographic locations making communication and coordinated efforts more challenging. Furthermore, their various management systems (i.e. pavement, bridge, traffic, safety, etc.) and databases are not typically integrated and generally function independently with limited communication, integration, and functionality existing between the systems. A problem with liability also arises when dealing with any safety concerns or issues due to the risks of lawsuits and litigation. An agency may collect or record safety data such as collision rates, collision locations, and surface friction but may not publicly release the information or make it available.

Furthermore, many agencies across North America and around the world continue to rely on a single measure to quantify safety or do not utilize a proactive approach towards managing pavements. A more comprehensive investigation that considers the key influencing factors along with their interactions and resulting impacts, must be studied, modeled, and quantified.

1.2.2 Research Objectives and Scope

The overall objectives of this research study are to examine the value of network level friction testing, develop a proactive approach towards pavement and safety management and examine the cost-effectiveness of safety related pavement preservation and maintenance through the usage of life cycle cost analysis (LCCA).

As previously mentioned, a number of transportation agencies across Canada and the United States do not collect network level friction data as a part of their regular pavement management or data collection programs. This is despite the fact that surface friction has a significant impact on the level of safety of a roadway. In addition, pavements are typically managed without significant consideration of safety or collisions on a highway network. There is a need for an integrated approach towards pavement and safety management that considers the key influencing factors along with their interactions. One of the main objectives of this research is to investigate the relationships between the various factors influencing the level of available pavement friction and the collision rate.

The scope of this research is limited to factors that can be measured or predicted in the field. Factors such as driver behaviour are difficult to quantify and model due to significant differences in the driver population. The skill, age, experience, health, concentration, and many other factors vary from driver to driver making it difficult to quantify or predict behaviour. As a result, this research is limited to measurable factors such as environmental and climatic data, skid resistance, geometrics and pavement conditions.

Several issues can arise when trying to develop relationships between highway safety and any factor such as pavement performance (skid resistance). One of the reasons for this is due to how the various factors are measured and referenced. Pavement performance data are typically collected by state-of-the-art data collection technologies that are linked to an accurate distance measuring instrument and Global Positioning Satellite (GPS) systems. As a result, data is collected and referenced very accurately along the length of a pavement section, roadway or highway. Depending on the data attribute, data can be summarized in 0.1 m increments along the length of a section or by the kilometre. This is usually not the same approach for safety data. Safety data is generally collected through police records or accident reports. Generally, the police officer at the scene of the collision fills out an accident report and estimates the location using the nearest kilometre post or intersection.

Another major factor that affects the development of safety-based relationships and produces several data outliers is the variability in driver behaviour. A pavement section might be in excellent functional and structural condition, the weather conditions are clear and sunny, and the geometric components are designed sufficiently but the driver loses control because he or she is intoxicated or not paying attention. Many collisions occur due to driver error or by uncontrollable factors such as animal collisions, talking on mobile phones, and tire blowouts. From a management perspective, these types of collisions tend to produce many outliers and reduce the statistical significance of any relationship.

Other statistical methods such as Cluster and Bayesian analyses have proven to be powerful and useful when investigating collision data. It is important to note that this research is not solely focused on developing relationships between the various factors and safety. Rather, the research is focused on demonstrating the importance and value of collecting network level skid data, the development of an integrated approach to pavement and safety management and to demonstrate how project-level life cycle cost analysis (LCCA) can also provide benefits in terms of safety. Finally, an approach is developed which can be used to determine minimum test interval requirements for network level skid testing using a locked-wheel skid tester. This latter objective is important for providing guidance to the industry and other researchers.

1.3 Research Approach

To examine the relationship between safety and skid resistance and evaluate the cost-effectiveness of safety related preservation and maintenance, a workplan has been developed which consists of five major components or modules. The first module, Module 1, consists of assembling the friction, collision, PMS and traffic data sources into a structured database using Microsoft Access and Excel. This initial module is a required component of all other modules. Once the data sources have been linked and integrated into a common database, statistical analyses utilizing common statistical techniques such as simple linear regression and Analysis of Variance (ANOVA) will be employed to develop models relating the level of friction to the collision rate (Module 2). Module 3 involves an examination of friction data from the SPS-5 Experiment sections in the Long Term Pavement Performance (LTPP) database. As a part of this module, friction data is modeled over time to identify trends and to investigate how the skid number deteriorates with time. The effectiveness of various pavement preservation techniques such as asphalt concrete overlays, mill and overlays, and surface treatments will be examined in terms of skid resistance. Module 4 utilizes the results from Modules 2 and 3 to perform project level life cycle cost analyses (LCCA) to examine the cost-effectiveness of performing functional pavement improvements to increase the skid resistance and improve highway safety. Module 5 involves the development of a framework for determining the minimum test interval requirements for network level skid testing using a locked-wheel tester. The sections herein provide specific details related to each research module.

1.3.1 Module 1: Data Integration

The development of any models or statistical analysis requires both high quality and structured data. The data for this research study consists primarily of traffic data, pavement management data (i.e. pavement types, pavement surface friction, highway referencing, etc.), and highway safety data.

The pavement management data were obtained from the MTO's Highway Pavement Management Application (MTO-HPMA). This data consists of pavement type, pavement surface friction in terms of skid number and highway referencing data. The friction data was collected as a part of a potential Area Maintenance Contract from approximately 1,800 km of highways located in the province of Ontario. This data was obtained with permission from the Pavement Management Division within the MTO. Pavement data within the MTO-HPMA has passed rigorous Quality Control/Quality Assurance protocols prior to being loaded into the system. The data exists in a well defined, structured, and documented database. All data is referenced using a highway definition and linearly using the kilometre-post. The referencing system is referred to as the Linear Highway Referencing System (LHRS). The data can be exported to Microsoft ACCESS or Microsoft EXCEL for further processing and manipulation.

The highway safety data was obtained from the MTO's Highway Safety Division. The data consists of collision occurrences at various locations. All data is referenced by highway number, LHRS number and kilometre post and is in Microsoft EXCEL format. This data can be imported to Microsoft ACCESS.

Once all data was formatted and checked for quality, it was integrated using Microsoft ACCESS and EXCEL into a structured data set for further processing and analyses. In this format, the data can be filtered, queried, manipulated, and transformed.

1.3.2 Module 2: Statistical Analysis

To examine the relationships and interactions between the various components of a collision and the level of pavement surface friction, several statistical methods are employed. Statistical software packages such as EXCEL, SPSS, and SAS are utilized in this module to examine relationships between variables.

Initially, a simple linear regression between the various factors and highway safety are performed. All significant factors along with their interactions are identified. The influence of the various parameters on highway safety are investigated and modeled.

1.3.3 Module 3: Examination of SPS-5 LTPP Friction Data

The Long Term Pavement Performance (LTPP) Project is the largest field pavement research study in North America. Pavement performance data has been collected from over 2,400 pavement sections located across Canada and the United States. These pavement sections consist of a variety of pavement structures in various environmental zones, built on different subgrades and exposed to various levels of traffic.

To investigate and quantify the effects of pavement friction over time, friction data extracted from the LTPP database for the SPS-5 test sites are examined. The LTPP data used in this study were extracted from the DataPave Online website (LTPP IMS Data Release 23.0 VR 2009.01). The SPS-5 experiment examines the effects of various pavement rehabilitation activities on flexible (asphalt) pavement sections. Each SPS-5 test site has 8 flexible pavement sections with different rehabilitation activities, in addition to a control section for benchmarking purposes. The objective of this module is to examine how friction varies with time and to evaluate the effectiveness in terms of skid resistance for various pavement rehabilitation and maintenance strategies.

1.3.4 Module 4: Cost Effectiveness Analysis

To demonstrate the importance of a proactive approach towards safety and pavement management, project level life cycle cost analysis (LCCA) is performed. The models developed in Modules 2 and 3 in addition to unit costs for various pavement maintenance treatments are used to perform the LCCA. The goal of the LCCA is to assess the benefits of investing in preservation and maintenance to improve skid resistance and the associated level of safety of a highway. The benefits include the reduction in collisions as a result of increasing the level of pavement surface friction using common preservation and maintenance treatments.

1.3.5 Module 5: Determination of Minimum Skid Testing Requirements

Network level friction testing can be characterized as expensive and time consuming due to the complexity of the test and the size of most highway networks. Currently, no guidelines or skid testing interval requirements are provided for network level skid testing. For this module, network level skid data from 1,800 km of provincial highway are used to develop an approach for determining the minimum skid test interval requirements.

The friction data used for this study were collected to determine a baseline of the current network friction levels in terms of a skid number. Testing was carried out at an interval of 1.0 km across the length of each highway segment. As a result, any reduction in the required number of test points is a benefit to the transportation agency, private sector (consultants and contractors) and most importantly, the public. This module demonstrates an approach that can be used to minimize the number of required test locations along a highway segment using common statistical techniques.

1.4 Thesis Organization

An organized research outline representing the various stages and tasks of the thesis is presented in Figure 1.5. The chapters of this thesis are organized as follows:

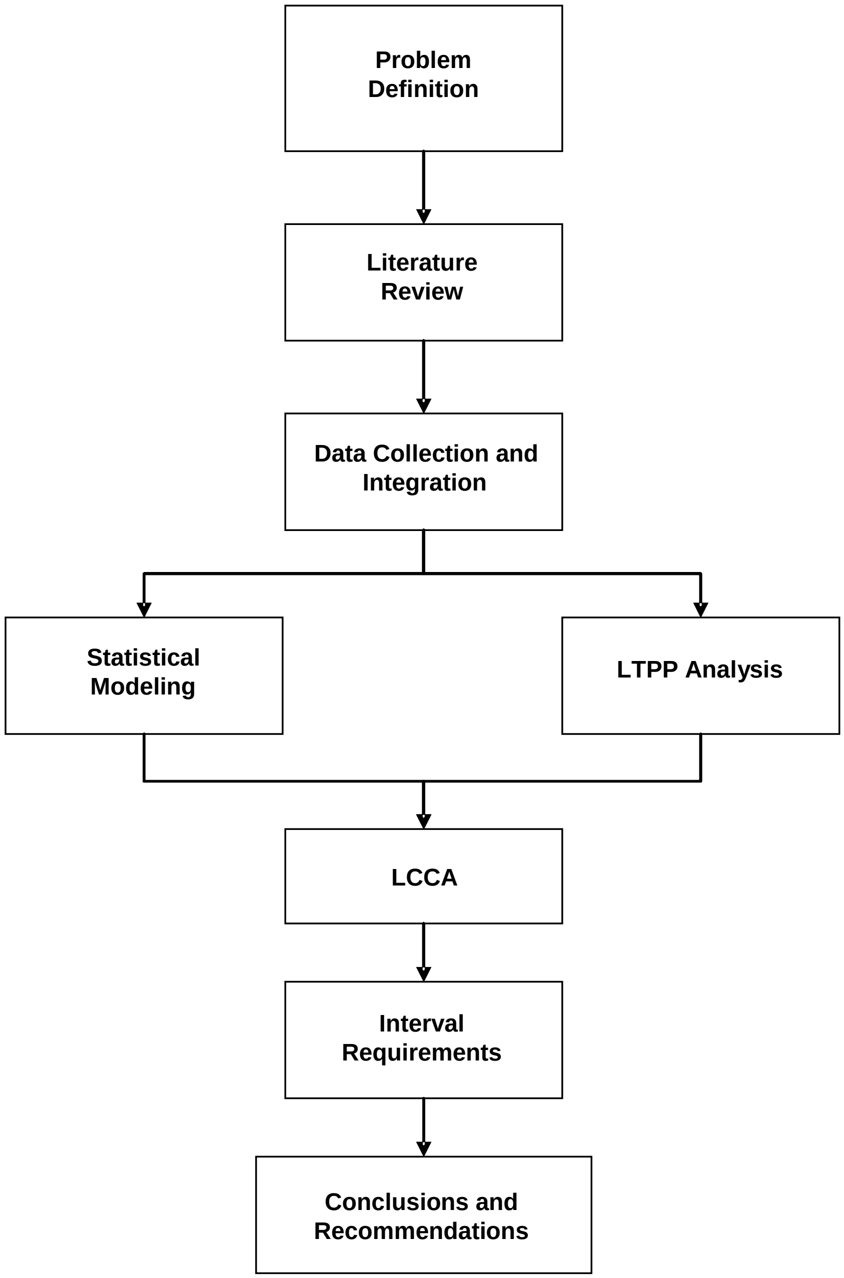

Figure 1.5: Research Methodology

- Chapter 2 contains a detailed literature review on some of the major components of pavement management, highway safety, pavement surface friction, preventative maintenance and life cycle cost analysis.

- Chapter 3 outlines the various data sets and parameters used in the analysis process and for the research. The data sets discussed are the LHRS data, skid data, collision data and LTPP data.
- Chapter 4 describes the linkage and integration of the data sets and variables outlined in Module 1. The development of any models or statistical analysis requires both high quality data and a structured database. As previously mentioned, the data for this research study consists primarily of pavement performance data and highway safety data.
- Chapter 5 presents the results of the statistical analysis and model development. The relationships and interactions between skid resistance and highway safety will be examined and several statistical methods will be employed. The relationship between the Wet-to-Dry accident ratio and surface friction will also be examined.
- Chapter 6 provides details on the examination of the SPS-5 LTPP friction data. The friction level over time is examined across various environmental zones for the SPS-5 pavement test sections. The performance of various pavement preservation treatments in terms of skid resistance is also evaluated.
- Chapter 7 investigates the economic effectiveness of implementing preservation and maintenance to improve the level of safety on a pavement section. The cost savings and benefits of increasing the level of friction are examined using Life Cycle Cost Analysis (LCAA) techniques.
- Chapter 8 provides details on an approach to determine the minimum test interval requirements for network level skid testing using a locked-wheel skid tester.
- Chapter 9 highlights the main findings and results of this research. Conclusions and recommendations are provided. Also outlined are areas of further research and study.

Chapter 2
Literature Review

This chapter provides a brief overview of pertinent research to the thesis and identifies existing gaps in the literature.

2.1 Pavement Management Systems

A Pavement Management System (PMS) is an essential component of most federal, state, or provincial transportation agencies. Since the late 1970s, the capability and functionality of a PMS have improved significantly with advancements in technology, computing power, and improvements to optimization/prioritization techniques. A management system is a tool for providing co-ordination of design, maintenance and rehabilitation activities, improved awareness, better communication within an agency and between departments, and improved use of funds. These systems also provide engineers and decision makers with a reliable source of data and integrated tools based on sound engineering and economic principles.

Today, most provinces and states own and operate their own PMS. These systems are used for inventory of data, performance modeling, budget analyses, and many other important business fuctions. More recently, the focus of the management systems has shifted towards an integrated approach whereby pavement management systems are integrated with other systems such as bridge management, traffic management into an overall asset management system.

2.1.1 The Concept of Integration

Over the past decade, a major move within the asset management community has been towards the full integration of civil infrastructure assets within a management system or framework. The goal or objective of implementing an integrated IMS within an agency is to provide decision-makers with processed quantitative data that can be used to examine the impacts of various alternative scenarios.

Integration represents an organized approach to help manage the infrastructure more effectively and efficiently (Hudson et al., 1997).

The integration of assets within a management system should produce several major savings and benefits to the agency and public at large. An integrated approach minimizes life cycle costs, impacts to the environment, and disruptions to local traffic and residents. It also ensures that the management of civil infrastructure is proactive, and that a high level of service is provided to the public. Furthermore, it ensures that full cost accounting is improved and that long range planning is enhanced in terms of technical, financial and risk management. Despite the benefits of an integrated approach, some potential risks to the agency do exist. Some of these risks include additional required resources, high renewal costs in the short term, and a potential lack of support from key stakeholders (operators, politicians, and the public) (Infraguide, 2003). Despite these potential problems, the savings and benefits incurred by an integrated approach clearly outweigh the potential risks.

2.2 Relationship Between Pavements and Highway Safety

Pavements encompass a significant proportion of most agencies' total civil infrastructure value. The City of Edmonton, in Alberta Canada estimates that the value of their civil infrastructure is approximately $200 billion, with pavements encompassing approximately 20% of this value. In 2004, the Ontario Ministry of Transportation (MTO) estimated that the current value of the provincial highway system is approximately $39 billion (ORSAR, 2005). Therefore, an effective and reliable management system based on sound engineering principles and theories is essential to ensure that pavements are managed in a systematic and efficient manner.

Highway and traffic safety data such as roadway geometry, traffic volumes, collision locations, pavement and material types, as well as pavement history and performance data are stored in a Pavement and Safety Management System (SMS). These systems are also used for Life Cycle Cost Analysis (LCCA), priority programming, budget optimization, and project-level decision making. An integrated approach is key to maximizing the savings and benefits generated by these two management systems. Many transportation agency's PMS and SMS are handled by two independent groups within the agency.

Despite the strong correlation between pavement performance and highway safety, both software applications/programs generally function independently with very little communication, integration, and functionality existing between the two systems. Several pavement performance factors such as pavement roughness, rutting, and surface friction, have a significant impact on highway safety. Currently, the pavement industry uses surface friction as the primary measure of safety for a highway alignment (NCHRP, 2009). The transportation industry tends to rely on collision location, geometric design components such as radius of curvature, sight distance, etc (Lamm et al., 1999).

One very important factor that is not clearly addressed, understood, or easily quantified is the interaction of pavement performance and geometric design and the resulting impact on safety. An example of this phenomenon is a highway alignment with poor ride quality and sharp horizontal curvature (small R_{min}). Another example is an alignment with poor sight distance and low surface friction. Furthermore, relying on a single measure of safety such as the collision rate or the Wet-to-Dry accident ratio can be problematic.

2.3 Road Safety on Ontario Highways

The Province of Ontario recognizes itself as a leader in road safety and has made road safety a top priority. In 2006, Ontario roads were found to be the safest in North America, based on the fatality rates for all jurisdictions across the continent (ORSAR, 2006). Ontario's fatality rate was the lowest ever recorded at 0.87 per 10,000 licensed drivers. Fatalities among drivers aged 65 and over fell by almost 11 % between 2005 and 2006, and motorcycle fatalities dropped 28.4% compared to 2005 levels (ORSAR, 2006).

Each year, the Ontario Ministry of Transportation issues their Ontario Road Safety Annual Report (ORSAR) to review current practice and identify and implement innovative ways to save lives and reduce injuries on Ontario roads and highways. ORSAR is a comprehensive annual review of road safety figures and statistics for the Province of Ontario. For more than 50 years, Ontario has collected major road safety statistics and tracked and recorded long-term trends in road safety (ORSAR, 2006). The report comments and takes pride in the fact that: "Over the past 20 years, the number of licensed drivers has increased as the Province's population has grown at a rapid rate. Despite this fact, there has been a drop in the number of fatalities over this same time period (Figure 2.1)".

Examining the past 10 years, there has been a less rapid reduction in the overall number of fatalities. A similar trend is observed with the total number of major and minor injuries. Overall, there appears to be a drop in the total number of injuries over the past 25 years. However, looking at the last 10 years, there has been a less rapid reduction in the overall number of fatalities (Figure 2.2).

For the total number of fatalities as a result of large truck collisions, there was a 16% reduction in fatalities from 1990 to 2006. Similar to the other two cases, when examining the past 10 years, there has been a less rapid reduction in the overall number of fatalities as a result of large truck collisions (Figure 2.3). These figures illustrate the caution and care that must be taken when interpolating collision or safety statistics.

Over the past 20 years, there have been major improvements to our civil infrastructure, vehicle enhancement and development, driver education, and enforcement and legislation. However, as these figures illustrate, the safety problem still exists and must be addressed. To improve the level of safety on Ontario's roads and highways, the Province invests significant resources into education, improvement and enforcement. The One-Person, One-Seatbelt" Legalisation was identified as the single most effective action that people can take to protect themselves and passengers in motor vehicle collisions (ORSAR, 2006).

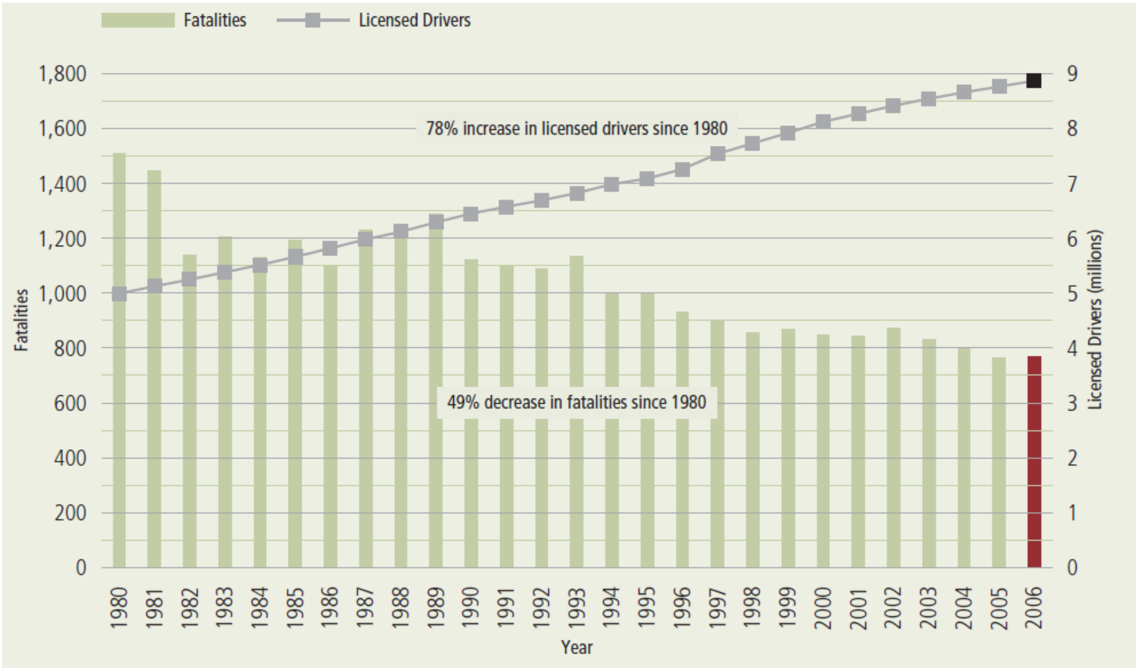

Figure 2.1: Total Number of Fatalities and Licensed Drivers (ORSAR, 2006)

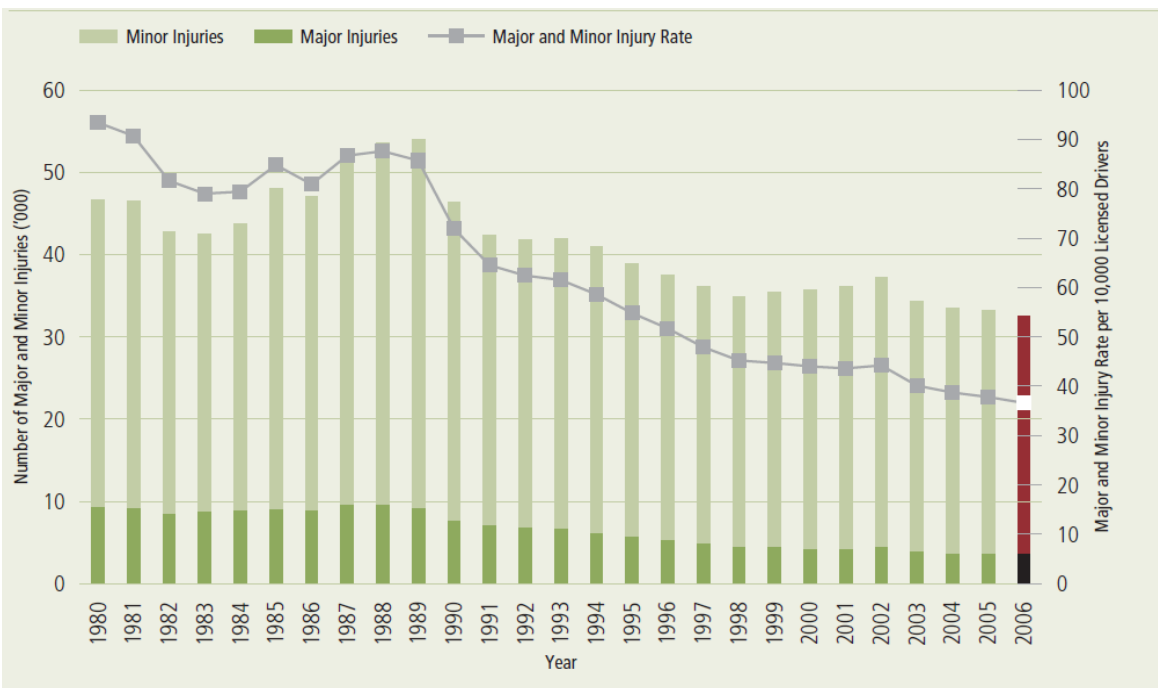

Figure 2.2: Total Number of Major and Minor Injuries (ORSAR, 2006)

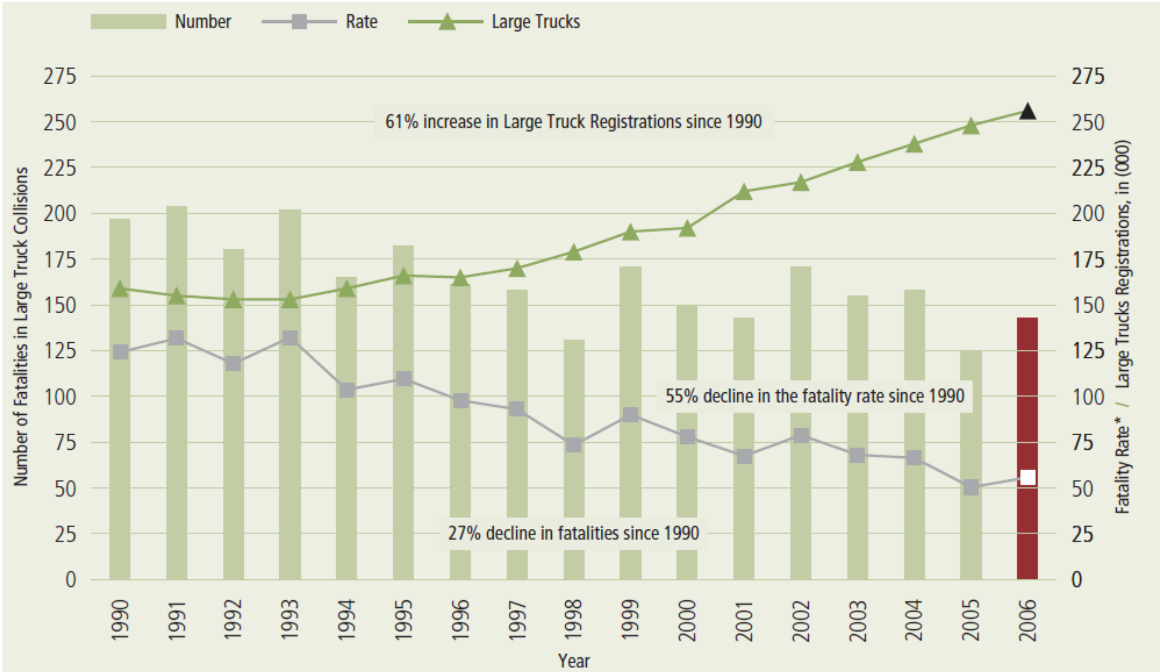

Figure 2.3: Total Number of Fatalities in Large Truck Collisions (ORSAR, 2006)

In 2006, one in every four drivers or passengers killed on Ontario roads was not wearing a seatbelt (ORSAR, 2006). Another example was the "Smart Love" advertising campaign to remind parents and caregivers that properly used child car safety seats save lives and are now the law in Ontario. The campaign included television, radio, and newspaper ads across the province and was also displayed on COMPASS traffic signs on 400-series highways (ORSAR, 2006).

The Province also has invested significant resources into improving public transit. In 2006, the MTO invested $1.3 billion into public transit improvements. Another example of improvements was the High Occupancy Vehicle (HOV) lanes that were implemented on a number of highways in Ontario. The first HOV lanes were implemented on Highways 403 and 404 and commuters generally saved 14 and 17 minutes respectively, on each highway during peak-hour commutes compared to non-HOV lanes (ORSAR, 2006).

Ontario is the third-largest financial area in North America and every year, $1.2 trillion worth of good are transported on the Provincial highway network (ORSAR, 2006). Every day, $650 million worth of products cross the Ontario/US border by road. In 2006, the Province invested over $1.4 billion for highway improvements and rehabilitation. The investment included $3.4 billion over five years to construct 130 km of new highways and 64 bridges and to repair 1,600 km of highways and 200 bridges in Southern Ontario (ORSAR, 2006). An additional $1.8 billion was invested over five years with the Ministry of Northern Development and Mines to improve Northern Ontario's highways by expanding 62 km of highway, adding or replacing 54 bridges and repairing 2,000 km of highway and 200 bridges (ORSAR, 2006). These facts demonstrate the provinces efforts to provide a safe and efficient road and highway network and highlight the importance of highway safety. With the billions of dollars in infrastructure improvements being invested on our road and highway system, it is obvious that pavements are an integral component of the highway safety problem.

Despite these major improvements to our civil infrastructure and safety initiatives implemented by the Province, collisions continue to occur and cost the Province billions of dollars each year. Motor vehicle collisions in Ontario in 2004 had a social cost of $17.9 billion (Vodden et al., 2007). Although fatal collisions accounted for less than 1% of the 231,548 reported collisions, they represented $11.5 billion or 64% of the total social costs estimated in 2004. Injury collisions made up 27% of all collisions and 27% or $5.0 billion of all costs. Property Damage Only (PDO) collisions, while the largest collision group at 73%, resulted in $1.3 billion or 8% of social costs. Based on these statistics, the average social cost of a collision by collision severity was (Vodden et al., 2007):

- Fatal: $15.7 million
- Injury: $82,000
- PDO: $8,000

Thus, the average collision had a social cost of $77,000 in 2004 (Vodden et al., 2007). These figures illustrate the significant costs vehicle collision cause to families, society, and the Province.

2.4 Pavement Performance Factors

Pavement structures are a significant component of our investment in civil infrastructure. Generally, the condition of a pavement deteriorates over time with repeated traffic loadings and environmental cycles. Several pavement performance factors influence highway safety. The next sections provide an overview of a number of pavement performance factors and examine their effects on safety. Since the primary focus of this research study is on pavement surface friction and how it impacts safety, the section covering surface friction has been presented as a standalone section in this chapter.

2.4.1 Ride Quality

Ride quality or pavement roughness is an important measure of the serviceability of a pavement. The serviceability of a pavement is defined as the ability to accommodate the road users at a reasonable level of comfort (TAC, 1997). Roughness is defined as a distortion of the pavement surface that contributes to an undesirable or uncomfortable ride (Hudson et al., 1997). Rough pavement results in driver discomfort, decreased speeds, potential vehicle damage, increased operating costs, and increased emissions. Roughness is also an important indicator of safety since it directly affects the driver and vehicle. The magnitude or severity of roughness is related to the amplitude and frequency of the pavement distortions, vehicle suspension characteristics, and the speed of the vehicle. Profiles are detailed recordings of the surface characteristics of pavement and are used to characterize roughness. Short-wavelength roughness is generally attributed to localized pavement distresses such as depressions and cracking. Environmental processes in combination with pavement layer properties are the typical cause of long-wavelength roughness.

The main causes of pavement roughness are traffic loading, environmental effects, construction materials, and construction quality (Shaheen, 1994). Pavement roughness increases with exposure to traffic and the environment. In Canada, pavements built on silty materials are highly susceptible to frost heave as a result of freeze-thaw cycles. This may result in increased pavement roughness during the early spring.

The International Roughness Index (IRI) was developed as a roughness measurement index in an attempt to standardize roughness data collection and analysis techniques for pavements. IRI is a roughness statistic that is valid for any road surface type and covers all levels of roughness. It is based on a quarter-car simulation. An IRI value of 0 m/km indicates absolute smoothness, while a value of 10 m/km represents a rough unpaved roadway (TAC, 1997).

There are four classes (Class I to 4) of roughness measurement methods defined as (TAC, 1997):

- Class 1 – Precision Profilers (Dipstick, Rod and Level, Profilometer, etc.)
- Class 2 – Other Profilomteric Methods (RT 3000, ARAN, Dynatest Model 5051, etc.)
- Class 3 – Response Type Devices (Mays Ride Meter, K.J. Law Model 8300, etc.)
- Class 4 – Subjective Ratings (Riding Comfort Index, PSR, etc.)

The Class 1 method is the most accurate, while the Class 4 is the least accurate. There is a definite trade off between the level of accuracy and speed of the test. For higher accuracy (Class 1), the test time is much longer then the high speed/automated methods (Class 2 or Class 3).

The relationship between ride quality and safety is obvious since pavement roughness is felt directly by the driver through the suspension of the car. On rough roads, some drivers tend to reduce their speed to maintain a reasonable level of comfort. This can result in a safety hazard as the majority of drivers tend to travel well beyond the 85^{th} percentile operating speed [Lamm et al, 1999]. A number of studies conducted in 1972 by Quinn and Hildebrand (1974), Brickman et al. (1972), and by Wambold et al. (1973) demonstrated the impacts of pavement roughness on the availability of tire-pavement friction and impacted the steering and traction performance of vehicles. Magnusson and Arnberg (1977) found that roughness affects a driver's ability to collect information and perform various manoeuvres. It was reported that a driver's ability to perform a motor task is reduced by vibrations caused by pavement roughness.

Short-wavelength roughness caused by potholes and localized areas of distresses can be a safety hazard as they may cause a driver to wander within the traveled lane or into oncoming traffic. Many of these distresses are unexpected and can result in dangerous avoidance manoeuvres, loss of control, mechanical breakdown thus increasing the risk of collisions. This may result in a potential head on collision or loss of control. The safety impact of pavement roughness was found to vary by accident type (Al Masaeid, 1997).

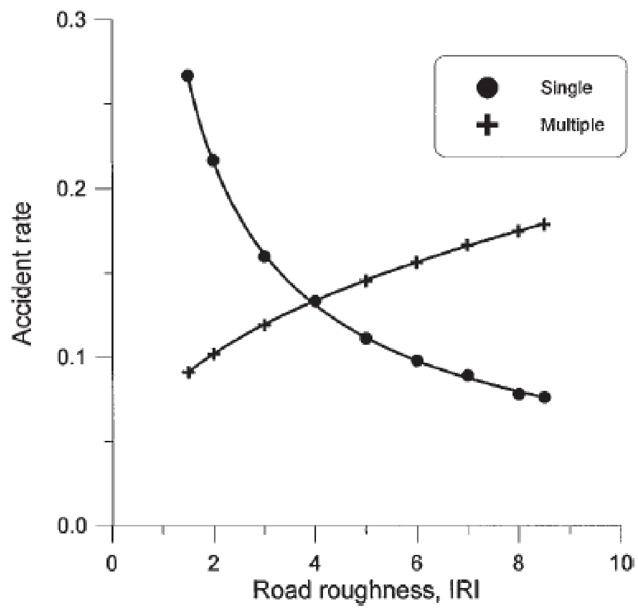

Figure 2.4: Estimated Single and Multi-Vehicle Accident Rates versus IRI (Al Masaeid, 1997)

The single vehicle accident rate was found to decrease as pavement roughness increases due to drivers reducing their speed for comfort. However, for the multi-vehicle accident rate, an increase was found due to lateral shifts and speed differentials between road users (Figure 2.4). Cenek & Davies (2002) found a positive relationship between the crash rate and $\log_{10}$IRI. Wambold et al., (2009) present a thorough summary of the affects and influences of pavement roughness on highway safety and the collision rate.

2.4.2 Pavement Rutting

Pavement surface ruts have a major impact on highway safety. Ruts are categorized as being traffic-load associated deformation, wear related, or a combination of the two (TAC, 1997). Typical causes of surface ruts include abrasion and/or studded tire related wear in the wheel paths and deformation of either the entire pavement structure in wheel paths (structural ruts), or instability in the form of compaction of one or more asphalt layers in the wheel paths (instability ruts) (TAC, 1997).

Rutting affects the handling characteristics of a vehicle (TAC, 1999). Ruts also allow water to pond, which can result in hydroplaning, skidding, or a loss of vehicle control during or after inclement weather. Hydroplaning is defined as the condition when a vehicle's tire is separated from the road by a fluid. The pressure of the fluid beneath the tire is able to lift the tire from the surface of the road (Start, 1997). Studies have shown that hydroplaning can occur with water depths as little as 7.6 mm. This phenomena startles or surprises many drivers and can result in loss of control or rear end collisions. Along with hydroplaning, asymmetric water drag on a vehicle can cause instability. This can occur when the rut depths in one wheel path are significantly greater than the ruts in the other. Asymmetrical water drag also influences steering control and can lead to drivers over correcting which can result in a loss of vehicle control. Studies have also shown that rut depths have an impact on the collision rate along a highway alignment. A study investigating the relationship between safety and rut depths in the State of Wisconsin found that collision rates increase dramatically for rut depth measurements exceeding 7.6 mm (Start, 1997).

It was also reported that rutting affects vehicle handling. Handling for smaller vehicles is impaired when they drive in a rut pattern that has been established by trucks. There is also concern that the wider 2590 mm trucks are having steering consistency problems because of rut patterns formed by the more common 2438 mm wide trucks (Start, 1997). The shape characteristics of ruts have also been shown to influence driver behaviour, vehicle dynamics and safety (Kazuya et al., 2007). To measure rut depths, a visual inspection consisting of a straightedge and a measuring stick or automated techniques that employ lasers or ultrasound to measure the transverse profiles at highway speeds are currently used by most transportation agencies.

2.4.3 Pavement Distresses

Evaluation of the surface condition of a pavement is an important component of pavement management and highway safety. This provides the ability to maintain the required level of service and to program maintenance work. Pavement distresses are a result of traffic loading, environmental loading, material and construction quality, and many other factors. Some examples of distresses related to asphalt concrete pavements are longitudinal cracking, transverse cracking, alligator cracking, ravelling, polishing, bleeding and potholes. Examples of Portland Cement Concrete distresses are block cracking, edge cracking, spalling, blowouts, scaling and map cracking. Potholes are a common occurrence on Canadian highways as a result of freeze thaw cycles and cause a safety hazard to motorists. Wambold et al., (2009) present a thorough summary of the effects and influences of potholes on highway safety. A number of studies demonstrate that potholes impact vehicle dynamics, can lead to vehicle damage such as tire blow outs and impact highway safety (Baker 1977, Klein et al., 1976, and Zimmer and Ivey, 1983). Distresses such as bleeding and ravelling affect the surface friction of a pavement and thus also directly impact safety. Other distresses such as thermal cracking and longitudinal cracking contribute to short-wave length roughness, which impacts the ride quality of a pavement.

To measure or evaluate the surface condition of pavement, the type, severity, and extent of the distress must be quantified. This can be done using manual evaluations, semi-automated and/or automated measures. Manual distress ratings are considered the most accurate measurement of surface condition as each distress type is measured and mapped directly in the field by an experienced rater. Automated and semi-automated methods consist of a vehicle traveling at highway speeds with a rater using a keyboard to record distresses. Other methods include the use of high quality digital images to record a continuous image of the pavement surface using a downward “line-scan” camera. The image is then digitized/processed by trained technicians using data reduction software.

The surface condition of a pavement can be reported using an index such as the Surface Distress Index (SDI) or the Pavement Condition Index. The PCI (Shaheen, 1994) is measured on a scale of 100 with a deduct occurring for each occurrence of a distress type. In Ontario, the MTO uses the Distress Manifestation Index (DMI) as a measure of surface condition.

2.5 Influence of Geometric Design on Highway Safety

A strong link exists between highway safety and geometric design. Geometric design of highways refers to the design of the visible dimensions of such features as horizontal and vertical alignments, cross sections, intersections, and bicycle and pedestrian facilities (Easa, 2003). The main objective of geometric design is to produce a highway with safe, efficient, and economic traffic operations, while still maintaining aesthetic and environmental quality. Generally, it costs significantly more money to adjust or improve the geometrics of a highway alignment compared to rehabilitating a pavement structure. The next few sections provide an overview of various highway geometric design features that influence safety.

2.5.1 Sight Distance

One of the most important aspects in highway safety and more specifically in geometric design is sight distance. Sight distance is defined as the length of roadway ahead that is visible to a driver. A driver’s ability to see ahead is of the utmost importance in the safe an efficient operation of a vehicle on a highway (AASHTO, 2001). Sight distance is highly variable and is a challenging parameter to measure or quantify since the ability of a driver to operate, perceive, and react is different from person to person. The skill of a driver is dependent on several factors such as age, health, and driver education. Another factor is the high variability in vehicle characteristics such as power, dimensions, weights, and dynamics. It is very important that highway designers provide sight distance of adequate length that would allow drivers to control their vehicles and avoid crashing into an unexpected object on the roadway (Lamm et al., 1999). Several types of sight distance such as stopping, passing, decision, and intersection sight distance are identified and addressed in the major geometric design guides (TAC and AASHTO). Sight distance can be a safety issues at vertical (crest and sag) and horizontal curves. Locations with insufficient sight distance are potential locations for high accident or collision occurrences and should be considered for improvement or corrective action during maintenance or rehabilitation activities.

2.5.2 Horizontal Curves

The main objective of horizontal curve design is to provide a radius and superelevation rate that combine to yield a safe and comfortable lateral acceleration for a reasonably large percentage of drivers (Bonneson, 2001). The minimum radius of a horizontal curve is typically the limiting value of curvature for a given design speed and is determined from the maximum superelevation and the maximum side friction factor for that given design. The use of sharp curves for a particular design speed would require superelevation rates that are beyond the limit considered practical or for operation with tire friction and lateral acceleration beyond what is considered comfortable by many drivers (AASHTO, 2001). The minimum radius of the curve is based largely on the level of driver comfort rather than safety.

As a vehicle moves in a circular path, it undergoes a radial acceleration that acts towards the centre of curvature. This acceleration is sustained by a component of the vehicle's weight related to the roadway superelevation, by the side friction developed between the vehicle's weight related to the roadway superelevation, by the side friction developed between the vehicle's tires and the pavement surface, or by a combination of the two (AASTHO, 2001). The design of horizontal curves should be based on an appropriate relationship between the design speed and curvature, and on their joint relationships with superelevation and side friction. The relationship between the radius, speed, superelevation, and the lateral friction factor as defined by (TAC, 1999) is given by,

$$e + f = \frac{V^2}{127R} \quad [1]$$

Where,

e	=	Pavement superelevation
f	=	Lateral friction force factor between the vehicle tire and roadway pavement.
V	=	Speed of Vehicle (km/h)
R	=	Radius of curve (m)

Horizontal curves with sharp radii are potential locations for high collision occurrences. It is usually difficult to improve a horizontal curve (increase radius) due to several factors such as high construction costs and right of way issues (Abd El Halim, 2004). Identifying locations of sharp curvature along a highway alignment is important to transportation agencies and the public safety since traffic signs such as a hazard/warning or reduced speed signs can be placed to provide warning, alert drivers, and improve safety. Several studies have shown the relationship between curve radius and the collision rate. In general, for sharp radius horizontal curves, the rollover risk and collision rates are higher, especially for trucks that travel at or near the posted speed limit (Abd El Halim, 2004).

2.5.3 Maximum Rate of Superelevation

The transition from a tangent or normal crown section to a curved superelevation section must be accomplished without any appreciable reduction in speed and in such a manner to ensure the safety and comfort of all occupants in the vehicle. The maximum superelevation that can be applied in the design of highways is affected by climatic conditions, terrain, type of environment, frequency of slow moving vehicles, and maintenance (TAC, 1999). In areas where snow and ice are a factor, the rate of superelevation should not exceed the rate on which a stationary or slow moving vehicle would slide toward the centre of the curve under icy conditions. For vehicles traveling at high rates of speeds on curves with poor drainage, hydroplaning can occur. Some vehicles with a high centre of gravity such as Sport Utility Vehicles (SUVs) and trucks have a high percentage of their weight carried by the inner tires while traversing a circular curve. This can cause a rollover if the vehicle is travelling at slow speeds.

In Canada, the maximum values for superelevation used are 0.04 m/m, 0.06 m/m, and 0.08 m/m (4%, 6%, and 8%) depending on environment and the degree of surface icing that is likely to occur (TAC, 1999). In the United States, the highest superelevation rate for highways in common use is 0.1 m/m (10%) and in some instances, values as high as 0.12 m/m (12%) have been used. In areas where snow and icy conditions are likely to occur, superelevation rates greater than 0.08 m/m should not be used (AASHTO, 2001).

Generally, a curve of minimum radius requires superelevation and side friction factors at their maximum, while a curve with a very large radius (very flat radii), requires minimal superelevation (i.e., normally sloped cross section). Nicholson (1998) found that having the superelevation and friction varying linearly with the degree of curvature leads to greater alignment consistency and improved safety. A method for selecting the optimum superelevation rates for a system of horizontal curves has been developed (Easa, 2003).

2.5.4 Tangent Length

Tangents are straight sections of the horizontal alignment, and can be beneficial to a highway design. Tangents can be used to achieve passing sight distance on two-lane highways, and for adapting the alignment to railroad sections, canals, and other man-made constraints (Lamm et al., 1999). Long tangents having a constant grade have several drawbacks. Some of the disadvantages are they lead to excessive speeding, increase the effects of glare from oncoming vehicles at night, and can be monotonous causing driver fatigue and boredom. Tangents are typically separated into two separate groups depending on the length of the tangent. An independent tangent is defined as a tangent having a length long enough to permit a driver to exceed the 85th percentile speed differences. Non-independent tangents are tangents that are too short to exceed the possible 85th percentile speed difference. The maximum speed achieved on a tangent section is largely dependent on the tangent length, the sharpness of the curves on either end of the tangent, and driver's desires based on the general character of the roadway (Ottesen, et al., 2000).

When designing a tangent, one critical factor that must be addressed is the length of the tangent. Due to glare caused by headlights at night, and the danger of drowsiness, tangent lengths should not exceed 20 times the design speed V_D (Lamm et al., 1999). Some highway agencies suggest that desirable tangent lengths should be at least six times the design speed.

2.5.5 Vehicle Stability

Vehicle dynamics focuses on the movements of different vehicles (automobiles, trucks, buses, and special purpose vehicles) on a road surface. The primary forces by which a high-speed motor vehicle is controlled are developed in four locations – the contact area between the tires and the road. To understand vehicle dynamics, the forces and moments generated by the rubber tires at the ground must be clearly defined. Vehicle performance and dynamics consists of several different motions – acceleration, braking, and handling. The forces imposed on the vehicle from the tires, gravity, and aerodynamics determines the dynamic behaviour.

To evaluate vehicle stability and its effects on safety, it is important to consider the rollover thresholds of various vehicles (Table 2.1). Rollover thresholds are critical since if a vehicle such as a tractor-trailer is operating on a sharp horizontal curve with very low pavement friction and is traveling slightly over the posted speed limit, a rollover or loss of control can occur. Therefore, knowing the lateral acceleration levels a vehicle experiences while operating on a highway alignment is helpful in identifying locations that may be hazardous or unsafe.

Table 2.1: Rollover Threshold Values for Different Vehicles (Gillespie, 1992)

Vehicle	CG Height (inches)	Tread (inches)	Rollover Threshold (g)
Sports Car	18-20	50-60	1.2-1.7
Compact Car	50-60	50-60	1.1-1.5
Luxury Car	20-24	60-65	1.2-1.6
Pickup Truck	30-35	65-70	0.9-1.1
Passenger Van	30-40	65-70	0.8-1.1
Medium Truck	45-55	65-75	0.6-0.8
Heavy Truck	60-85	70-72	0.4-0.6

2.6 Pavement Surface Friction

Pavement surface friction is defined as the force that resists the relative motion between a vehicle tire and the surface of a pavement. As a tire rolls or slides over a pavement surface, a resistive force is generated. The forces acting on a rotating tire are presented below in Figure 2.5 (Hall et al., 2009).

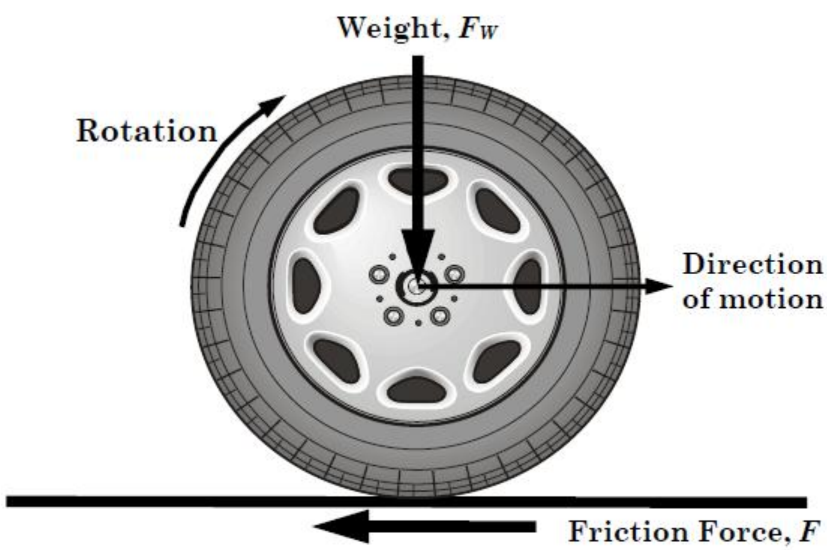

Figure 2.5: Forces acting on a Rotating Tire (Hall et al., 2009)

The resistive force, μ, is the ratio of the tangential force (F) between the tire tread and the horizontal surface to the vertical load or weight, F_w, and is calculated from the following equation:

$$\mu = \frac{F}{F_W} \qquad [2]$$

Surface friction between a vehicles tires and the pavement surface has a significant effect on highway safety. A driver must be able to adapt their behaviour to changing friction conditions in order to maintain an acceptable level of safety (Wallman and Astrom, 2001). When road surfaces are dry, the friction generated between the tires and pavement is generally sufficiently high to provide adequate levels of safety. During wet or winter weather conditions, water can create a critical situation by increasing the potential for hydroplaning or skidding, especially when surface friction of a pavement is low (Shaheen, 1994). When surface friction is low, the driver may not be able to stop the vehicle or retain stability on wet pavements.

2.6.1 Types of Pavement Frictional Forces

For roadways and highways, there are generally two major types of frictional forces that are considered in the design and management of pavements – longitudinal and lateral frictional forces. The next two sections describe how they are defined and calculated.

2.6.1.1 Longitudinal Frictional Forces

The longitudinal frictional forces are defined as forces that occur between a rolling tire and the road surface when operating in the free rolling or constant-braked mode in the longitudinal direction. During free-rolling mode, the relative speed between the tire circumference and the pavement (slip speed) is equal to zero. In the constant-brake mode, the slip speed increased from zero to a maximum of the traveling speed of the vehicle. Slip speed is calculated from the following equation (Hall et al., 2009 and Meyers, 1982):

$$S = V - V_P = V - (0.68 \times \omega \times r) \qquad [3]$$

Where,

S = Slip speed, km/hr
V = Vehicle speed, km/hr
V_P = Average peripheral speed of tire, km/hr
ω = Average velocity of tire, km/hr
r = Average radius of tire, m

When the vehicle is in the free-rolling state, V_P is equal to the speed of the vehicle resulting in a slip speed, S = 0. The free-rolling state is generally referred to as the zero percent slip ratio. For the locked-wheel state while the locked wheel state is referred to as the 100 percent slip ratio (Hall et al., 2009). The slip ratio is calculated from the following equation:

$$SR = \frac{V - V_P}{V} \times 100 = \frac{S}{V} \times 100 \qquad [4]$$

Where,

SR = Slip ratio
V = Vehicle speed, km/hr
V_P = Average peripheral speed of tire, km/hr
S = Slip speed, km/hr

When the vehicle is in the free-rolling state, V_P is equal to the vehicle speed and SR = 0%. For the locked wheel state, V_P = 0, and S equals the speed of the vehicle, which results in SR = 100% (Hall et al., 2009).

The forces and moments generated from the interaction between the tire and the pavement surface are presented in Figure 2.6 for a free-rolling tire at a constant speed and a constant-braked wheel on a dry pavement surface (Andresen and Wambold, 1999). For the free rolling tire, the ground force, F_G, is located at the centre of pressure of the tire contact area and located a distance of, a, from the centre of the tire. This distance is a function of the speed and increases with speed. In the constant-braked mode, the braking slip force (F_B) is required to counter the added moment (M_B) caused by the braking (Hall et al., 2009).

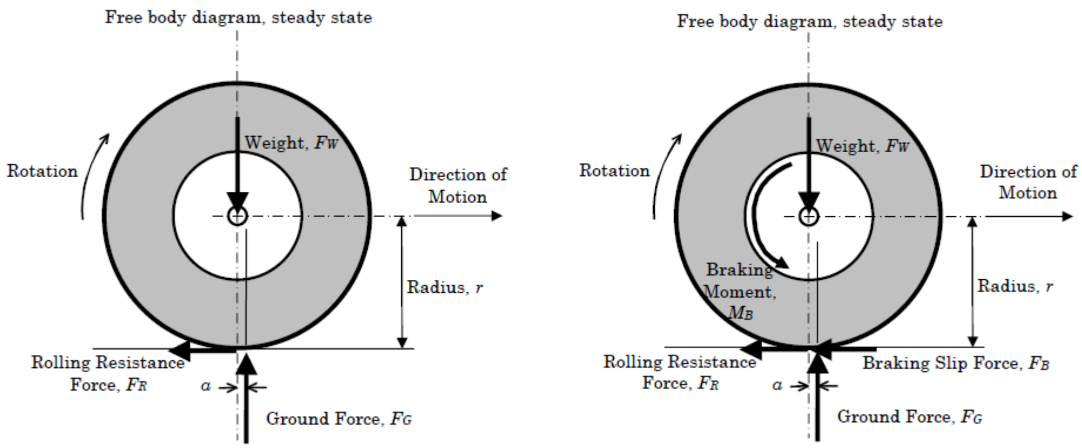

Figure 2.6: Rolling resistance force with a free-rolling tire at a constant speed on a bare dry pavement surface (left) and forces and moments of a constant-braked wheel on a bare-dry pavement surface (right) (Andresen and Wambold, 1999).

This force is proportional to the level of braking and the resulting slip ratio. The total frictional force is equal to the sum of F_R and F_B (Hall et al., 2009). The level of available friction between a tire and the pavement is a function of the slip. The coefficient of friction increases with increasing slip to a peak value which is typically between 10 and 20% slip (critical slip). The friction then decreases to a level referred to as the coefficient of sliding friction which occurs at 100% slip (Hall et al., 2009).

2.6.1.2 Lateral Frictional Forces

The lateral frictional forces or side-force friction occurs as a vehicle changes direction or compensates for pavement superelevation. The side friction factor (*f*) is the ratio of the lateral friction force and the component of the weight of the vehicle perpendicular to the pavement (Figure 3). This force is applied to the vehicle at the tires and is toward the centre of the curve producing a radial acceleration (TAC, 1999). Since the speed that drivers operate their vehicles varies greatly on curves, there is generally an unbalanced force that is produced. This force results in side thrust that is counterbalanced by the friction between the tires and the pavement surface (AASHTO, 1993). The coefficient of friction, *f*, is the frictional force divided by the component of the weight perpendicular to the pavement surface. The coefficient f is also referred to as the lateral ratio, cornering ratio, unbalanced centrifugal ratio, friction factor, and side friction factor. The upper limit of the side friction factor is the point at which the tire begins to loose traction or skid. This is referred to as the point of impending skid. Since horizontal curves on highways are designed with a high degree of margin of safety to avoid any chances of skidding, the *f* values used in the design process are substantially lower than the value of friction at the point of impending skid. The side friction factor at the point of impending skid is dependent on factors such as vehicle speed, roadway surface conditions, types of vehicles, and tire type and pressure. Morrall and Talarico (1994) conducted a survey that measured the side friction demanded and lateral acceleration of several different vehicles at the moment of impending skid conditions on horizontal curves.

The skid resistance condition of the pavement surface is critical to highway safety. Emergency or evasive manoeuvres such as braking, sudden lane changes and directional adjustments within a single lane can significantly add to the frictional demands on the roadway geometry (AASTHO, 2001). When these manoeuvres are performed, it is usually only for a short period of time and although high friction demands may exist, the amount of time required may not be enough to facilitate a corrective response from the driver resulting in unsafe operating conditions.

One major issue with the side friction factors used by the major international highway agencies is that the factors were developed almost 50 years ago. The vehicle and tire technology, as well as pavement surfaces, have all evolved with the time and thus the relevance of these factors is now questioned. Today's drivers have become more tolerant of lateral acceleration through changes in attitudes and vehicle cornering capabilities (Bonneson, 2001).

The relationship between the forces acting on the tire of the vehicle and the pavement surface as a vehicle operates on a curve, changes lanes or compensates for lateral forces is calculated from:

$$F_S = \frac{V^2}{15R} - e \qquad [5]$$

Where,

F_s = lateral friction or side friction

V = Vehicle Speed, km/hr

R = Radius of curvature in a curve (m)

E = Pavement superelevation (m/m)

2.6.2 Mechanisms of Pavement Surface Friction

The pavement surface friction mechanism is created by a complex relationship between two major frictional force components – adhesion and hysteresis (Hall et al., 2009). Adhesion is the friction that results from the small-scale bonding/interlocking of the tire of the vehicle and the pavement surface. It is a function of the interface shear strength and the contact area. The hysteresis component of frictional forces results from the energy loss due to bulk deformation of the vehicle tire. The deformation is referred to as enveloping of the tire around the texture (Hall et al., 2009). When a tire compresses against the pavement surface, the stress distribution causes the deformation energy to be stored within the rubber. As the tire relaxes, part of the stored energy is recovered, while the other part is lost in the form of heat (hysteresis). That loss leaves a net frictional force to help stop the forward motion. As a result, friction can be viewed as the sum of the adhesion and hysteresis frictional forces (Hall et al., 2009):

$$F = F_A + F_H \quad [6]$$

Both forces are directly related to the pavement surface characteristics, the contact between the tire and pavement and the properties of the tire. Since rubber is a visco-elastic material, temperature and sliding speed also affect both forces.

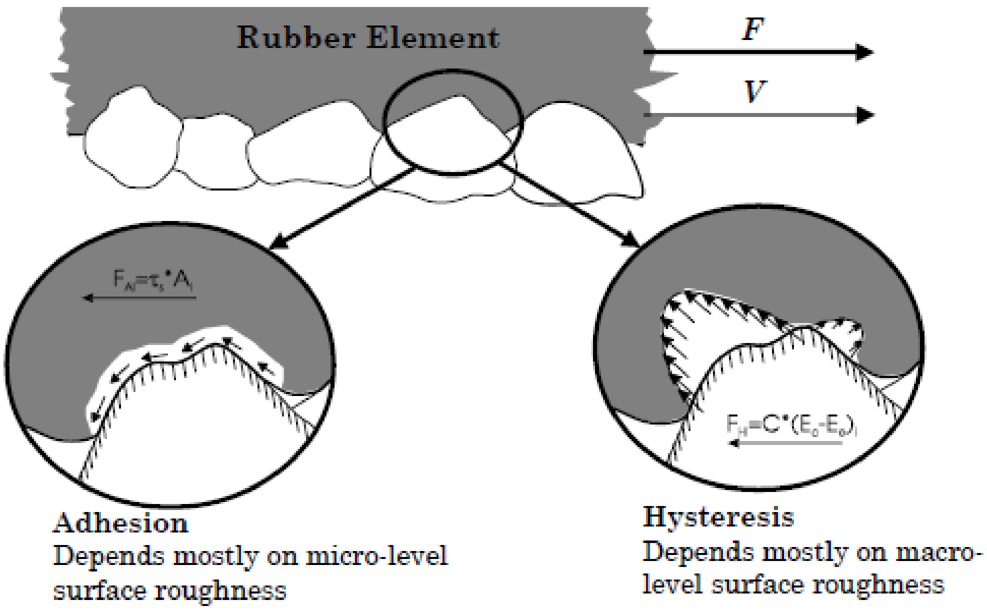

Figure 2.7 Adhesion and Hysteresis Forces (Hall et al., 2009)

Since the adhesion force is developed at the pavement-tire interface, it is most responsive to the micro-texture of the aggregate particles contained in the pavement surface. Hysteresis forces developed within the tire are most responsive to the macro-texture formed in the surface by mix design or construction techniques. As a result of this phenomenon, adhesion governs the overall friction on smooth-textured and dry pavements, while hysteresis is the over-riding component on wet and rough textured pavements (Hall et al., 2009).

2.6.3 Factors Influencing Available Pavement Surface Friction

As discussed in the previous section, pavement surface friction is a complex force which affects the braking dynamics of a vehicle. A number of factors influence pavement friction forces. The major factors influencing pavement surface friction can be grouped into four major categories;

- Pavement Surface Characteristics
- Vehicle Operational Parameters
- Tire Properties
- Environmental Factors

The factors influencing pavement friction along with the various factors comprising each major category are presented below in Table 2.2. The next subsections provide details on the major factors influencing surface friction.

Table 2.2: Factors influencing Pavement Surface Friction (modified from Hall et al., 2009)

Pavement Surface Characteristics	Vehicle Dynamics	Tire Specification and Properties	Environment and Climatic Conditions
▪ Microtexture ▪ Macrotexture ▪ Megatexture/unevenness ▪ Temperature	▪ Slip speed o Vehicle speed o Braking action ▪ Driving manoeuvre o Turning o Passing	▪ Foot print ▪ Tire tread design and condition ▪ Rubber composition and hardness ▪ Inflation pressure ▪ Load ▪ Temperature	▪ Climate o Wind o Temperature o Water (rainfall, condensation o Snow and Ice ▪ Contaminants o Anti-skid material (salt, sand, etc.) o Dirt, mud, debris, etc.

2.6.3.1 Pavement Surface Characteristics

The microtexture and macrotexture of the pavement surface greatly influences the level of available friction. Microtexture is related to the degree of roughness of individual aggregate particles within the asphalt mixture; while macrotexture is related to the degree of roughness caused by deviations in the aggregate particles. At low speeds, microtexture is responsible for pavement friction. At higher speeds, macrotexture produces most of the available pavement friction.

The pavement surface characteristics can vary greatly depending on the mix design, aggregate type and source, binder type, traffic, etc. In the province of Ontario, there has been a lot of discussion related to the skid resistance of Stone Mastic Asphalt (SMA) pavements once open to traffic (Lane et al., 2008).

2.6.3.2 Vehicle Dynamics

Vehicle characteristics such as size, weight, and number of tires influence the level of available surface friction. As vehicles have changed over the past 50 years, the required surface friction requirements have also changed.

2.6.3.3 Tire Specifications and Properties

Tire properties such as material type, tread type, tread depth, tire quality, and condition influence the level of available surface friction. Tire inflation pressure is another important factor as under-inflation can significantly reduce friction at high speeds. Over-inflation produces a smaller loss in pavement friction compared with under-inflation.

2.6.3.4 Environmental and Climatic Conditions

Environmental conditions such as thermal conditions, water, snow, ice and contaminants all influence the level of available surface friction. During and after a rain event, water on the pavement surface can reduce the level of surface friction and cause hydroplaning under braking. In Canada, winter driving conditions can be dangerous. Debris such as dirt, sand and oil on a pavement surface can also significantly reduce the available pavement friction.

2.6.4 Pavement Friction Testing

The need and requirement for measuring the level of friction or skid resistance of a roadway has been an important requirement for many transportation agencies around the world. This data is collected for pavement management purposes, research needs, safety assessments and project level decision making. A variety of equipment devices and procedures are available to determine these properties. However, the differences in the methods and results of the measurements can be significant (Hall et al., 2009). A number of equipment manufacturers have developed and built friction testing devices that can be used to test roadways. Some of these devices can operate at the posted speed limit or higher (100 km/h or more) with fixed or variable slip. In addition, variable test tire conditions such as load, size, tread design and construction, and inflation pressure can be evaluated. To measure the surface texture, a number of devices are available such as rubber sliding contact devices, volumetric techniques and water drainage techniques (Wallman and Astrom, 2001 and Hall et al., 2009).

The cost and complexity of a friction testing device increases for devices which operate at highway speeds with no traffic control. These devices are typically more complex and expensive to maintain and operate. Devices requiring lane closures such as the British Pendulum are generally less complex and relatively inexpensive to own and operate. The ASTM has developed a set of surface characteristics standards and measurement practices to ensure comparable, repeatable and accurate measurements for surface texture and friction data collection. The next two sections provide an overview of Friction and Surface Texture measuring techniques.

2.6.4.1 Low Complexity Testing

The British Pendulum Tester (BPT) and the Dynamic Friction Tester (DFT) are two common devices that can be used to measure pavement friction in the lab or in the field. Details on the test methods and procedures for the BTM test can be found in AASTHO T278 or ASTM E 3030. Details on the test methods and procedures for the DFT can be found in ASTM E 1911. These devices measure frictional properties by determining the loss in kinetic energy of a sliding pendulum or rotating disk when in contact with the pavement surface (Hall et al., 2009). The loss in kinetic energy is converted to a frictional force which is pavement friction. The advantages of these test methods are the low complexity, low equipment costs, and ease of portability. The disadvantage of these test methods are that they are not suitable for network level testing since they generally require a lane closure and do not simulate the stopping forces of an actual vehicle tire.

2.6.4.2 High Complexity Testing

For high-speed friction data collection, pavement friction properties are typically determined in four modes:

- Locked Wheel
- Side Force
- Fixed-Slip
- Variable Slip

The most common device used for measuring pavement friction in the United States is the locked wheel method (ASTM E 274). The Ontario Ministry of Transportation (MTO) owns and operates a locked wheel skid tester. The locked wheel skid tester simulates frictional properties under emergency braking conditions without anti-lock brakes. The locked wheel tester tests at a slip speed equal to the vehicle speed, which means the wheel is locked and not able to rotate (Henry, 2000). The results of the locked wheel test are reported as a friction number, FN, or skid number, SN, which is calculated from the following equation:

$$FN(V) = 100\mu \times (F / W) \quad [6]$$

Where,

V = Velocity of the test tire, mi/hr

F = Tractive horizontal force applied to the tire, lb

W = Vertical load applied o the tire, lb

Locked wheel skid testers typically operate at a speed between 60 km/h to 100 km/h. Testing can be performed with a single tire or dual tire and using a smooth tire or ribbed tire. Details on the testing procedure and methods are provided in ASTM E 524 and ASTM E 501 for smooth and ribbed tires, respectively.

Another method to measure surface friction is using the side-force method which is detailed in ASTM E 670 which measures a vehicles ability to maintain control in a horizontal curve. The side-force friction is measured using the following equation:

$$SFC(V, \alpha) = 100 \times (\frac{Fs}{W}) \quad [7]$$

Where,

V = Velocity of the test tire, mph

α = Yaw angle

F_s = Force perpendicular to plane of rotation

The Mu-Meter and the Side-Force Coefficient Road Inventory Machine (SCRIM) are the two most common side-friction testing devices. The side-force measuring devices provide the ability for continuous friction measurements through the test section, while the locked wheel tester provides friction measurements at a discrete point (Henry, 2000).

Another device that can be used to measure pavement friction is the fixed-slip test device. This device measures the pavement friction experienced by vehicles equipped with anti-lock brakes. The frictional forces between the pavement surface and the tire are measured and the percent slip is calculated from the following equation:

$$Percent\ Slip = \frac{V - r \times \omega}{V}\ x\ 100 \qquad [8]$$

Where,

$Percent\ Slip$ = Ratio of slip speed to test speed
V = Test Speed
r = Effective tire rolling radius
ω = Angular velocity of test tire

Variable-slip devices (ASTM E 1859) measure the frictional force, as the tire is taken through a predetermined set of slip ratios. A summary of a variety of high speed and low speed friction test devices is presented below in Table 2.3.

Table 2.3: Overview of Skid Testing Devices (modified from Hall et al., 2009)

Pavement Friction Test Method	Test Method	Associated Test Standard	Description	Equipment	Applications	Advantages	Disadvantages
Highway Speed	Locked Wheel	ASTM E 274	This device is installed on a trailer which is towed behind the measuring vehicle at a typical speed of 40 mi/hr (64 km/hr). Water (0.02 in [0.5 mm] thick) is applied in front of the test tire, the test tire is lowered as necessary, and a braking system is forced to lock the tire. Then the resistive drag force is measured and averaged for 1 to 3 seconds after the test wheel is fully locked. Measurements can be repeated after the wheel reaches a free rolling state again.		Field Testing straight sections Network Level-friction testing	Well Developed Widely used (over 40 States) User friendly, simple, safe and efficient	Only used on straight sections (no curves, T-sections, or roundabouts) Can miss slippery spots since measurements are intermittent (point-by-point)
Highway Speed	Side-Force	ASTM E 670	Measures the pavement side friction or cornering force perpendicular to the direction of travel of one or two skewed tires. Water is placed on the pavement surface (4 gal/min [1.2 L/min]) and one or two skewed, free rotating wheels are pulled over the surface (typically at 40 mi/hr [64 km/hr]). Side force, tire load, distance, and vehicle speed are recorded. Data is typically collected every 1 to 5 in (25to 125 mm) and averaged over 3-ft (1-m) intervals.	 	Field Testing straight sections, curves, steep grades Data in different applications collected separately	Well controlled skid condition Measurements are continuous throughout test Method is commonly used in Europe	Very sensitive to road irregularities (potholes, crack, etc.) which destroy tire quickly Mu-Meter is primarily only used on airports in North America
Highway Speed	Fixed-Slip	Various	Fixed-slip devices measure the rotational resistance of smooth tires slipping at a constant slip speed (12 to 20 percent). Water (0.02 in [0.5 mm] thick) is applied in front of a retracting tire mounted on a trailer or vehicle typically traveling 40 mi/hr [64 km/hr]. Test tire rotation is inhibited to a percentage of the vehicle speed by a chain or belt mechanism or a hydraulic braking system. Wheel loads and frictional forces are measured by force transducers or tension and torque measuring devices. Data are typically collected every 1 to 5 in (25 to 125 mm) and averaged over 3-ft (1-m) intervals.	 	Field Testing straight sections Network-level friction monitoring Project-level friction monitoring	Continuous, high resolution friction data collected	Fixed-slip devices take readings at specialized slip speed. May not coincide over ice or snow covered surfaces Uses large amounts of water in continuous mode Requires skilful data reduction
Highway Speed	Variable-Slip	ASTM E 1859	Variable-slip devices measure friction as a function of slip (0 to 100 percent) between the wheel and the highway surface. Water (0.02 in [0.5 mm] thick) is applied to the pavement surface and the wheel is allowed to rotate freely. Gradually the test wheel speed is reduced and the vehicle speed, travel distance, tire rotational speed, wheel load, and frictional force are collected at 0.1-in (2.5-mm) intervals or less. Raw data are recorded for later filtering, smoothing, and reporting.		Field Testing straight or curved sections Network-level friction monitoring Project-level friction monitoring	Can provide continuously any desired fixed or variable slip friction results Can provide the Rado shape factor for detailed evaluation	Large complex equipment with high maintenance costs Complex data processing and analysis Uses large amounts of water
Requires Traffic Control	Stopping Distance Measurement	ASTM E 445	The pavement surface is sprayed with water until saturated. A vehicle is driven at a constant speed (40 mi/hr [64 km/hr] specified) over the surface. The wheels are locked, and the distance the vehicle travels while reaching a full stop is measured. Alternatively, different speeds and a fully engaged antilock braking system (ABS) have been used.		Field Testing straight sections Crash investigations	Simplest method for determining pavement surface friction	Test values obtained are not very repeatable Traffic control is required

Pavement Friction Test Method	Test Method	Associated Test Standard	Description	Equipment	Applications	Advantages	Disadvantages
Requires Traffic Control	Deceleration Rate Measurement	ASTM E 2101	Testing is typically done in winter contaminated conditions. While traveling at standard speed (20 to 30 mi/hr [32 to 48 km/hr]), the brakes are applied to lock the wheels, until deceleration rates can be measured. The deceleration rate is recorded for friction computation.		Field testing (straight segments) Crash investigations	System is easy to use, small, portable, lightweight, and easy to install and remove	Requires a sudden braking manoeuvre to be made and may not be operationally desirable Cannot be used for network level evaluation Requires traffic control
Requires Traffic Control	Portable Testers	ASTM E 303 ASTM E 1911	Portable testers can be used to measure the frictional properties of pavement surfaces. These testers use pendulum or slider theory to measure friction in a laboratory or in the field. The British Pendulum Tester (BPT) produces a low-speed sliding contact between a standard rubber slider and the pavement surface. The elevation to which the arm swings after contact provides an indicator of the frictional properties. Data from five readings are typically collected and recorded by hand. The Dynamic Friction Tester measures the torque necessary to rotate three small, spring-loaded, rubber pads in a circular path over the pavement surface at speeds from 3 to 55 mi/hr (5 to 89 km/hr). Water is applied at 0.95 gal/min (3.6 L/min) during testing. Rotational speed, rotational torque, and downward load are measured and recorded electronically.	 	BPT provides friction and micro texture indicators for any pavement in the field or lab It can also evaluate the effect of wear on friction and texture The DFT can be used for field and lab testing for QC/QA	BPT is used worldwide. It is suitable for both lab and field. BPT can be used to measure the longitudinal and lateral pavement-tire friction DFT provide good repeatability and reproducibility and unaffected by operator or wind. It provides friction coefficients of high speed values. It can produce IFI statistic and correlated to BPN	BPN variability is large and can be affected by operator or procedures and wind effects Traffic control is required for both tests Do not always simulate pavement-tire characteristics Both devices only collect spot measurements Cannot be used for network level testing
Highway Speed	Electro-optic (laser) method	ASTM E 1845 ISO 13473-1 ISO 13473-2 ISO 13473-3	Non-contact very high-speed lasers are used to collect pavement surface elevations at intervals of 0.01 in (0.25 mm) or less. This type of system, therefore, is capable of measuring pavement surface macro-texture (0.5 to 50 mm) 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.	 \ 	Network level testing	Collects continuous data at high speeds Correlates well with MTD Can be used to provide a speed constant to accompany friction data	Equipment is very expensive Requires skilled operators for collection and data processing

2.6.5 Surface Friction and Safety

The impact of surface friction on highway safety is a complex problem. It consists of a relationship that involves the driver and vehicle, environmental conditions, and the pavement surface. The ability of a driver to accurately assess or estimate the friction conditions is poor (Wallman and Astrom, 2001). This perspective is supported by several research studies such as speed measurements during different roadway conditions, driver interviews during slippery conditions, and vehicle simulator experiments. The main premise for these studies is that if the stopping distance for a dry pavement condition is considered an indicator of safe speed, then a reduction in speed as a result of poor surface friction (wet or icy conditions) should result in an equivalent stopping distance. A study was carried out where vehicle speeds were recorded under different road conditions. For the studied highway (7-m wide, posted speed of 90 km/h), the average speeds were found to be 85 km/h to 95 km/h for dry pavement conditions. During winter conditions, a 6 to 10 km/h decrease in the posted speed limit was recorded despite icy and snow packed pavement conditions. To maintain equivalent "dry" pavement surface stopping distances, the speed of the vehicle should be reduced to 56 km/h (Wallman and Astrom, 2001). Several other studies have shown similar findings.

Many research studies investigating collision data and surface friction in European countries such as the Netherlands, Germany, and France have shown that the number of collisions and the relative proportion of collisions at skid-prone sites increase sharply when the friction coefficient decreases. For example, when the friction interval is between 0.35 to 0.44, the collision rate is 0.20 (personal injuries/million veh-km). When the friction interval is less than <0.15, the collision rate increases by 300% (Wallman and Astrom, 2001). Recent research has shown the benefits of mix design and hot mix asphalt technologies on the surface friction of newly constructed pavements (Neves and Fernandez, 2006). Presented in Table 2.4 is a set of criteria for identifying low friction pavement surfaces from the Transportation Association of Canada (TAC, 1997).

Table 2.4: Criteria For Identifying Low Friction Pavement Surfaces (TAC, 1997)

Category	Skid Number	Collisoin Problem	Comments
A	< 31	Yes	Improvements considered for programming on the Betterment of General Maintenance Programs in a prudent manner consistent with District priorities.
B	31-34	Yes	Maintain surveillance and take corrective action as required
C	34 or less	No	
D	35-40	-	
E	$\geq$ 40		No further action required

Pavement surface friction plays a significant role in highway safety. It is a complex problem that is sometimes difficult to define, quantify or model. A summary of a number of relevant research findings related to pavement surface friction and safety is presented below:

- José M. Pardillo Mayora and Rafael Jurado Piña, 2009 – The study examined pavement surface friction measured with a SCRIM and collision data from over 1,750 km of two-lane rural roads in Spain. Both wet- and dry-pavement crash rates presented a decreasing trend as skid resistance values increased. Thresholds in SCRIM coefficient values associated with significant decreases in wet-pavement crash rates were determined. Pavement friction improvement schemes were found to yield significant reductions in wet-pavement crash rates averaging 68%. The results confirm the importance of maintaining adequate levels of pavement friction to safeguard traffic safety as well as the potential of pavement friction improvement schemes to achieve significant crash reductions.
- Burns et al., 2009 – A research study was completed which provided an overview of the influences of roadway surface discontinuities on road and highway safety. The study found that variations in friction coefficients with and between wheel paths may produce difficulties in controlling a vehicle when brakes are applied. It also concluded that a difference in friction coefficients between the wheel paths could be potentially hazardous, even though the average surface friction is relatively high. Severe vehicle response can occur when a driver releases their brakes after the vehicle begins to spin.
- Zhonghyin et al., 2009 – This study focused on examining the level of available surface friction in road tunnels in the provinces of Guizhou and Yunnan, in China. It was found that skid resistance decreased rapidly inside tunnels and was significantly different than the skid resistance from the outside of the tunnel. The researchers identified this as a significant safety hazard since a collision within a tunnel can result in a major catastrophe such as fires and explosions. The research focused on investigating various pavement surfaces to provide long-lasting skid resistance, including porous concrete and open graded friction course (OGFC) surface.
- Reddy et al., 2008 – As a part of this study conducted in the State of Florida, six innovative safety treatments were evaluated to examine their impacts on improving the level of safety. One of these treatments was the Tyregrip High Friction Surface System. This treatment was found to be effective in increasing the friction between the roadway and vehicle tries. The treatment was also found to be effective in helping motorists maintain their lane position

under wet pavement conditions. The study found that increasing the level of available pavement surface friction increased the level of safety.

- Noyce et al., 2007 – This study focused on the relationship between asphalt mix design, pavement surface friction and highway safety. Friction and crash data collected over 10 years at six study sites in Wisconsin were analyzed. The results of the analysis did not indicate a relationship between crash frequency and pavement skid friction. Although some evidence suggests that the number of wet pavement crashes increased as the pavement life increased (and skid friction values decreased), the frequency of crashes was not sufficient to statistically support this conclusion. The study concluded that more crashes occurred at low friction numbers (FNs), which is an important indication that skid resistance may indeed be a factor affecting wet weather crashes. The researchers also indicate that a friction value less than 35 (SN) is problematic from a safety standpoint. FN values less than 35 (SN) should trigger a safety monitoring program and those pavements should be scheduled for future rehabilitation or reconstruction.
- Murad, 2007 – As a part of this study, pavement surface friction data (SN) and collision history were examined for 500 km of two-lane highway sections selected randomly across the United States. Approximately 20% of the collisions occurred on wet-pavements. Low skid resistance was found to have been a contributing factor to a large portion of the wet weather collisions. A result of the statistical modeling indicated a negative correlation existed between SN (skid number corrected for temperature) and the variable WMY (wet-pavement collisions per mile per year) indicating that higher levels of skid resistance lead to lower potential for wet-pavement collisions.
- Zimmerman et al., 2005 – This study provided a review of the benefits associated with the use of pavement preservation program to improve safety characteristics. The study reviewed a number of agencies practices such as the Texas Department of Transportation's Wet Weather Accident Analysis Program. Internationally, work being conducted in the United Kingdom and New Zealand on continuous friction measurements and the use of the data to identify pavement sections where poor texture/friction may be contributing to higher than average crash rates was reviewed. Other examples, such as Australia's recently established goal of achieving 19% of their 40% per capita collision reduction by providing safer roads was also highlighted.
- Kuttesch, 2004 – As a part of this study, analysis showed that vehicle crashes are more likely to occur on wet pavements with lower friction levels. Also, as level of available pavement

surface friction decreases, the crash rate increases. The findings also concluded that when pavement friction falls below a site-specific threshold value, the risk of wet crashes increases significantly.

- Bray, 2002 – This study examined locations with very high amounts of wet weather crashes. In total, 40 pavement sections were examined before and after a new asphalt concrete overlay was performed to increase the level of pavement surface friction. The results of this study showed a significant reduction in the number of wet weather crashes as a result of the new overlay.
- Xiao et al, 2000 – This research study focused on the development of fuzzy logic models to predict wet-pavement crashes. The model was developed using the following variables: skid number, posted speed, average daily traffic, driving difficulty and pavement wet time. The listed variables were found to have the greatest effect on the risk of skidding collisions at a crash location. The models were used to estimate the improvement in safety expected from improvements in each of the input variables. The study concluded that the level of safety could be improved by nearly 60% if the skid number increased from 33.4 (SN) to 48 (SN).
- Cairney, 1997 – This study examined pavement surface friction at 120 collision locations where a collision involving skidding occurred in addition to 100 randomly selected control sites on highways of similar functional class and traffic levels. The relative risk of a site being a skid-related crash site was calculated. It was found that the risk of a skid-related crash was small for friction values above 60 (SN), but increased rapidly for friction values below 50 (SN).
- Kamel and Gartshore, 1982 – This study examined a number of pavement section that had low surface friction and high rates of wet pavement crashes on highways in Ontario. These sections were resurfaced to increase pavement friction. At intersections, the resurfacing resulted in a 46% reduction in the total number of collisions (21% for dry conditions and 71% for wet conditions). For freeways, the resurfacing resulted in a 29% reduction in the total number of collisions (16% for dry conditions and 54% for wet conditions).

A review of the literature shows a long list of excellent research studies dating back as early as the 1960s which examine the effects and influences of pavement surface friction on highway safety.

2.6.6 Friction Testing in North America

The collection of pavement friction data has many important applications for Highway Departments or Transportation Agencies. For a state or provincial level highway agency, friction testing can be collected for both network level or project level management. Network level testing is typically conducted to evaluate pavement safety by identifying potential slippery locations, to provide data for planning resurfacing activities and to establish network friction databases for pavement management purposes (Li et al., 2004). The project-level testing is usually conducted to identify friction conditions on a specific road for pavement design and rehabilitation decision making or to implement quality control for new construction. In addition, friction testing is also often collected for collision investigations, forensics applications, or for research studies on pavement materials (Li et al, 2004).

In a survey completed in 1990, forty-four agencies responded that they conducted friction testing for the purpose of wet-pavement collision investigations, forty-two agencies responded that they conducted friction testing for the purpose of pavement inventory, forty-six agencies responded that they conducted friction testing for the purpose of research, and eleven agencies responded that they conducted friction testing for the purpose of new construction acceptance. In another survey completed in 2000, three agencies started inventory friction testing but thirteen agencies ceased inventory friction testing. Two agencies started to conduct friction testing for collision investigations but nine agencies discontinued this type of friction testing. More agencies started to implement friction requirements for new construction. A study was completed by the Ohio Department of Transportation (ODOT) in 2002 which surveyed a number of DOTs use of pavement friction data across the United States and the results are summarized below in Figure 2.8. Although the importance of skid resistance and friction testing is well known, there must be a mechanism in place to collect and use the data to improve safety.

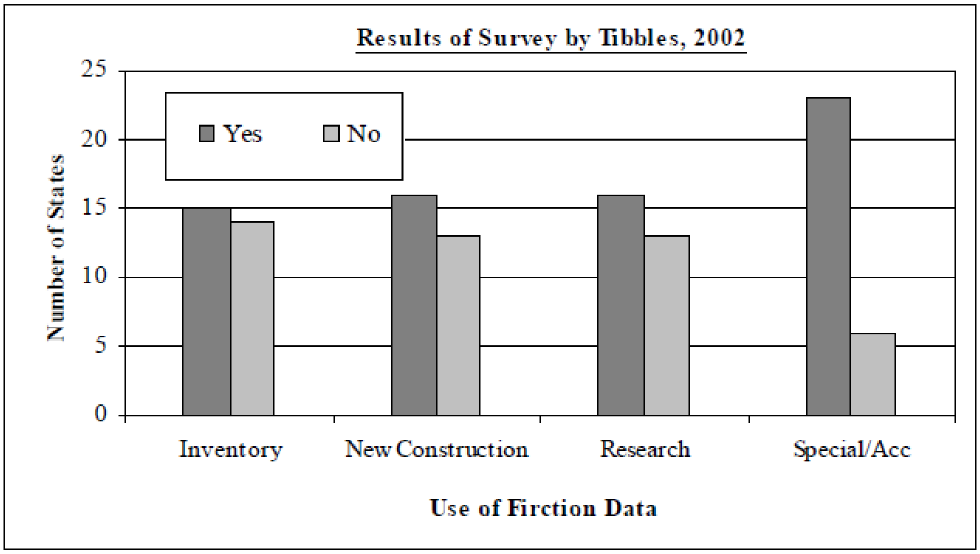

Figure 2.8: Results of ODOT Survey (Li et al, 2004).

2.6.7 Friction Data and Liability

A major issue related to the collection of skid data is the risk of liability and litigation to Highway Departments and Transportation Agencies. Since pavement friction contributes significantly to the level of safety of a highway or road, Agencies may find themselves exposed to liability for collisions that result from what used to be considered purely weather-related, drive-related, or vehicle-related causes (Carlson, 1974). Network level skid testing is an important component of a pavement and safety management framework and is imperative for early detection of low skid resistance areas. The use of mandatory minimum skid numbers is warned against as it may expose the agency to adverse legal implications (Carlson, 1974).

In a survey conducted of sixty-six North American roadway agencies in 1990, thirty had indicated that they had litigation as a result of wet weather collisions. Only 16 agencies believe that litigation was a significant problem. Some of the comments were (Glennon and Hill, 2004):

- 100 claims have been related to friction number and/or wet weather
- 20 to 25 claims had been related to snow, ice, pooled water, frost
- Three cases in 1986
- Two cases settled in 1981 at $750,000 with other pending

- Several favourable defence verdicts and 2 or 3 claims settled favourably
- Any litigation is a problem.
- No settlement yet

Slippery pavement is not a common aspect of tort claims and proving that a pavement with low skid resistance caused a collision is usually difficult (Glennon and Hill, 2004). United States federal rules of evidence (23 USC 409), however, which have been adopted in many States, preclude the discovery of collected data whose ultimate purpose is to make roadways safer. The results of this survey clearly illustrate the risk and liability that agencies face when dealing with friction data and collisions that occur at locations with low skid resistance. However, it is important to note that implementing network-level friction data collection shows the agency is being proactive in dealing with the safety problem related to poor skid resistance any may help reduce the risk of tort litigation (Hall et al., 2009).

2.7 Preservation and Maintenance

Conventional or routine pavement maintenance practices can be characterized as reactive in nature (unplanned), performed on failing pavements, does not contribute to long-term performance, not cost effective and often performed under harsh or severe conditions (AASHTO, 1999). They are also focused primarily on activities of a structural or corrective nature (Labi and Sinha, 2003). Preservation and maintenance is an essential component of an effective pavement management framework or strategy. Preservation and maintenance can be defined as a strategy intended to arrest light deterioration, retard progressive failures, and reduce the need for routine maintenance and service activities (Louis O'Brien, NCHRP 153). Dollars invested in implementing preservation strategies are significantly less than allowing a pavement structure to deteriorate until major pavement rehabilitation or reconstruction is required. Preservation not only increases performance and service life, but also shows much promise in reducing long-term costs of highway facilities (Labi and Sinha, 2003).

The benefits of practicing preservation and maintenance are higher customer satisfaction, better informed decisions, improved strategies and techniques, improved pavement condition, reduced life cycle costs and increased safety. Studies have shown that highway improvements such as increasing the radius of a horizontal curve or increasing the skid resistance of a pavement can result in a reduction in the number of collisions and/or improved levels of service. Evaluating the effectiveness or performance of a Maintenance, Rehabilitation or Reconstruction (M, R, & R) activity is beneficial to agencies and contractors so they can determine what treatments or strategies offer the best “bang for the buck”. The practice of optimum preservation requires an adequate balance between sustained performance and increased maintenance costs. If pavement preservation is applied too often, it will be uneconomical while if it’s too infrequent, user costs increase and overall repair costs can increase dramatically (Labi and Sinha, 2003).

2.7.1 Pavement Preservation in North America

Many transportation agencies are currently practicing some form of preservation to effectively manage their pavement networks. In 1999, a survey by the American Association of State Highway and Transportation Officials (AASHTO) Lead States Team on Pavement Preservation surveyed transportation agencies in 50 States (including the District of Columbia and Puerto Rico) and six Canadian Provinces on the nature of their pavement preventive maintenance (PPM) programs and practices. The survey asked the various agencies where it had a PPM program, how long has the program been around and whether it was integrated with a pavement management system, what level of annual funding is provided and if the program’s administration is centralized, decentralized or some combination of the two. The survey also asked what PPM treatments were used, had any PPM guidelines been developed, where test sections implements and what condition levels pavement are being treated with preservation techniques.

Based on the 41 agencies that responded to the survey, thirty-six (85%) had established PPM programs, two others were in the process of developing a program and forty-one were all using a variety of preservation treatments. Three years prior (1996), a similar survey was completed and only twenty-six of forty-three reporting agencies said they had extensive preservation programs, 56% had moderate programs and 19% had very little PPM. Nearly half of the survey respondents had PPM programs that had existed for more than 10 years. In thirty-one of the forty-one agencies, the pavement preservation program had been integrated with pavement management systems. Half the reporting agencies characterized their PPM administration as a mixture of centralized and decentralized, while only six programs were characterized as being completely centralized. For funding, eighteen out of thirty-six agencies had funding less than $25 million.

In 2006, a similar survey was conducted by Cuelho et al., 2006 and distributed to members of the AASHTO Research Advisory Committee (RAC). The objectives of the survey were to solicit States and Provinces to determine the types of pavement preventive maintenance systems they currently use, their use of materials and techniques, and how preventive maintenance systems are evaluated in their respective programs. The survey was completed in March, 2006 with forty-seven individual responses to the survey from thirty-four states and five provinces. The majority of the respondents (91.3 %) had indicated that their agency had a preventive maintenance program for their pavements, with 8.7% indicating no program existed. This represents a slight increase from the 1999 survey. For program funding, 67.4% of the respondents had dedicated budgets, 28.3% did not and 4.4% did not know. The range in funding varied from $2 million to $150 million with an average of $40 million for all respondents.

2.7.2 Pavement Preservation and Safety

From a safety standpoint, the level of safety of a pavement has typically been measured as a function of the skid resistance. A strong relationship exists between safety, highway design and pavement performance.

Based on a review of the literature and current practice of North America’s major transportation agencies, very little focus or attention was provided on safety related preservation, maintenance and pavement management at large. Both survey results of the AASHTO 1999 and Cuelho et al., 2006 studies did not show any agency practicing safety related preservation.

2.8 Life Cycle Cost Analysis in Pavement Design

Life Cycle Cost Analysis (LCCA) is a useful tool and analysis technique that is based on economic principles and theories to evaluate the over-all-long-term economic efficiency between competing alternative investment options (FWHA, 1998). It incorporates initial and discounted future agency, user and other relevant costs over the life cycle of alternative investments. The goal or objective of the LCCA is to identify the most cost-effective (lowest long term cost that satisfied the performance objective) for investment expenditures.

The LCCA should be conducted as early in the project development cycle as possible. For pavement design and rehabilitation projects, the appropriate time for conducting the LCCA is during the project design stage. Typical LCCA models based on primary pavement management strategies can be used to minimize unnecessary repetitive analysis (FHWA, 2001). Inclusion of all potential LCCA factors in every analysis is not necessary and cumbersome. Instead, all LCCA factors and assumptions should be addressed, even if only limited to an explanation of the rationale for not including eliminated factors in details (FHWA, 1999).

2.8.1 LCCA Economic Principles

The following are a brief description and overview of some of the key economic principles of LCCA for selecting alternatives.

2.8.1.1 Analysis Period

The LCCA analysis period or life cycle over which the alternatives are evaluated should be sufficient to reflect long-term cost differences associated with reasonable design strategies (FWHA, 1999). While FHWA's LCCA Policy Statement recommends an analysis period of at least 35 years for all pavement projects, including new or total reconstruction projects as well as rehabilitation, restoration, and resurfacing projects, an analysis period range of 30 to 40 years is not unreasonable (FHWA, 1998). To evaluate or compare alternatives (Figure 2.9), shorter life cycle periods can be used since the effectiveness of the treatment is generally in the three to ten year range.

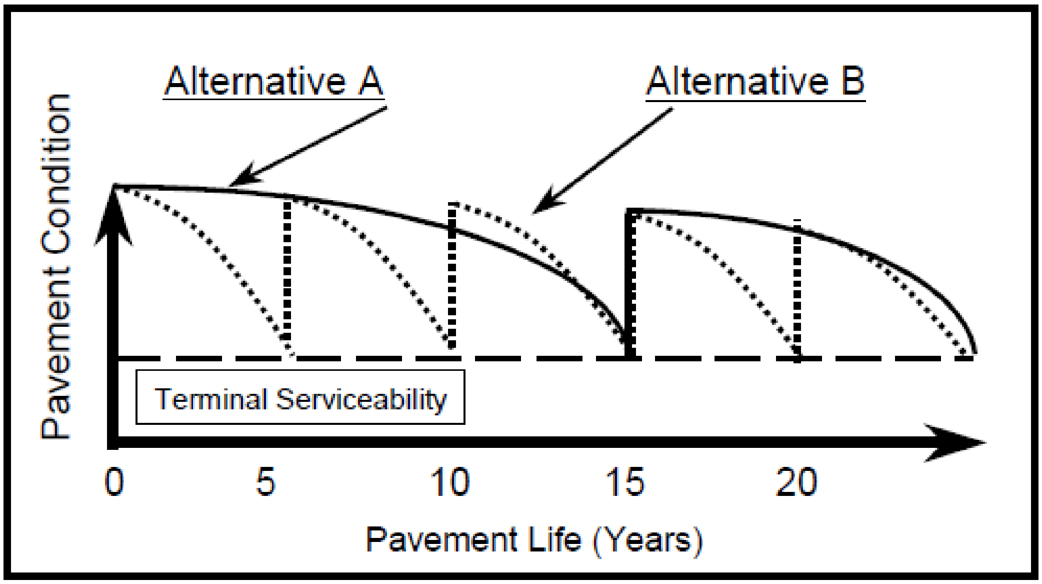

Figure 2.9: Performance Condition Comparison of Two Alternatives

2.8.1.2 Benefit/Cost Analysis or Ratio

The Benefit/Cost Ratio represents the net discounted benefits of an alternative divided by net discounted costs. B/C ratios greater than 1.0 indicate that benefits exceed cost. The B/C ratio approach is generally not recommended for pavement analysis because of the difficulty in sorting out benefits and costs for use in developing B/C ratios.

2.8.1.3 Internal Rate of Return

The internal rate of return is primarily used in private industry, represents the discount rate necessary to make discounted cost and benefits equal. While the IRR does not generally provide an acceptable decision criterion, it does provide useful information, particularly when budgets are constrained or there is uncertainty about the appropriate discount rate.

2.8.1.4 Net Present Worth

Net Present Worth (NPW), is the discounted monetary value of expected net benefits (i.e., benefits minus costs). NPV is computed by assigning monetary values to benefits and costs, discounting future benefits ($PV_{benefits}$) and costs (PV_{costs}) using an appropriate discount rate, and subtracting the sum total of discounted costs from the sum total of discounted benefits. The Net Present Worth converts all the treatment costs to the present using the following formula:

$$PW_n = \frac{F}{(1+i)^n} \qquad [9]$$

Where,

PW_n = Total present worth of the strategy in year n

n = Age of the pavement within the life cycle analysis period

F = Cost of the treatment

i = Discount rate

The net present worth (NPW) of the strategy combines all the treatments costs over the analysis period:

$$NPW = \sum_{n=1}^{N} \frac{F}{(1+i)^n} \qquad [10]$$

Where,

NPW = Total present worth of the strategy

N = Life cycle analysis period

n = Age of the pavement within the life cycle analysis period

F = Cost of the treatment

i = Discount rate

The incremental Present Worth is calculated as follows:

$$IncPW_n = PWn \text{ for n=0} \qquad [11]$$

$$IncPW_n = IncPW_{n-1} + PWn \text{ for n>0} \qquad [12]$$

Where,

$IncPW_n$ = Incremental present worth for year n

$IncPW_{n-1}$ = Incremental present worth for year n-1

PW_n = Present worth value in year n

2.8.1.5 Equivalent Annual Uniform Cost

The equivalent annual uniform cost (EAUC) is an annuity that is mathematically equivalent to a generally more complicated cash flow. The EAUC is used to calculate the regular annuity, given the present worth and is calculated as follows:

$$EAUC = NPW \frac{i(1+i)^N}{(1+i)^N - 1} \qquad [13]$$

Where,

EAUC = Equivalent annual uniform cost

NPW = Total net present worth of the strategy over the analysis period

I = Discount rate

N = Analysis period

The incremental costs of EAUC are calculated similar to the incremental costs for the present worth. First all the equivalent annual costs are converted to an annual Present Worth cost, then each annual present worth cost is added to the previous annual present worth cost.

2.8.1.6 Discount Rate

Similar to costs, LCCA can use either real or nominal discount rates. Real discount rates reflect the true time value of money with no inflation premium and should be used in conjunction with non inflated dollar cost estimates of future investments. Nominal discount rates include an inflation component and should only be used in conjunction with inflated future dollar cost estimates of future investments.

2.9 Summary

The relationship between skid resistance and safety has been established in the literature. However, a review of the literature revealed that many agencies are not collecting network level skid data and do not practice effective safety management. Furthermore, safety related pavement preservation is not commonly practiced by Highway Departments or Transportation Agencies across North America.

Chapter 3
Data Sources

This chapter outlines the data sources and parameters used in the analytical work of this research study. The data used for this research study was obtained from the Ontario Ministry of Transportation (MTO) and the Long Term Pavement Performance (LTPP) Project's DataPave Online database. The analytical work consisted of four major tasks:

- Examining the influences of surface friction on various parameters and attributes related to collisions,
- Evaluating the effectiveness of various preservation and maintenance treatments in terms of skid resistance using LTPP data,
- Performing Life Cycle Cost Analysis (LCCA) to demonstrate the benefits of a safety-related preservation and maintenance approach to managing pavements, and
- Developing a methodology to determine the minimum skid test interval requirements for network level skid testing using a locked-wheel skid tester.

3.1 Data Parameters and Attributes

All friction, collision, traffic, and pavement referencing data for this study were obtained from the Ontario Ministry of Transportation (MTO). Each data element was checked for completeness; Quality Assurance/Quality Control, and formatted prior to analysis. The next sections provide an overview of the various data elements of this study.

3.1.1 Highway Referencing

The MTO uses the Linear Highway Referencing System (LHRS) to section their highway segments into manageable pavement sections. Each highway segment is referenced with a unique highway identification number. As an example, a highway might have a length of 100 km and have 12 unique individual LHRS sections each of varying length. Each unique LHRS section "resets" the linear offset distance at the start of a new section. All collision data and locations are referenced to the LHRS and the correct linear offset. Presented below in Table 3.1 is a summary of the total number of LHRS sections per Highway in each Region. As can be seen in Table 3.1, the shortest LHRS section is 0.51 km which is located in Region C on Highway 14. The longest LHRS section is 31.0 km which is located in Region C on Highway 15.

Table 3.1: Summary of LHRS Sections Per Highway

Region	Highway Number	Number of LHRS Sections	Minimum Length of LHRS Section (km)	Maximum Length of LHRS Section (km)	Average Length of LHRS Section (km)
A & B	1	9	3.90	10.60	7.98
	2	18	1.65	26.93	12.18
	3	6	1.20	12.00	6.65
	4	7	5.90	17.50	10.27
	5	7	8.00	14.00	10.41
	6	12	2.10	14.70	7.86
	7	15	2.10	16.10	10.81
	8	10	1.60	16.10	9.64
	9	5	4.50	16.00	10.54
	10	10	1.30	13.40	7.83
C	11	18	1.70	19.00	6.81
	12	9	4.50	15.80	9.80
	13	7	2.52	14.00	9.72
	14	11	0.51	18.50	5.57
	15	5	6.59	31.00	18.52
	16	5	1.91	20.11	15.06
	17	6	0.60	17.28	8.31
	18	13	1.24	10.27	5.17
	19	1	2.80	2.80	2.80
	20	5	8.90	20.10	14.18
	21	5	2.30	25.30	10.62
	22	2	2.30	17.30	9.80
	23	2	3.90	12.59	8.25
	24	1	4.43	4.43	4.43
	25	1	10.90	10.90	10.90
	26	1	18.80	18.80	18.80

3.1.2 Network Level Friction Data

As a part of a potential long term area maintenance contract, network level skid data was collected on a large portion of the pavement network from the Ontario Provincial Highway System. In 2006, approximately 1,800 km of the provincial highway network was surveyed for friction data. This data was collected across 33 individual highway segments consisting of an assortment of functional classes (2 lane undivided, 4 lane divided, etc.). This friction data was collected to determine a baseline of the current network friction levels in terms of a skid number. Testing was carried out at an interval of 1.0 km along the length of each highway segment.

As described in Chapter 2, a locked-wheel skid tester is a common device used to assess the friction level of pavements in terms of a Skid Number (SN). The MTO retained Applied Research Associates, Inc. to collect the network level skid data. A locked-wheel device, conforming to ASTM E274 (Skid Resistance of Paved Surfaces Using a Full-Scale Tire) and ASTM E501 (Standard Rib Tire for Pavement Skid Resistance Tests) were utilized. A Dynatest 1295 Pavement Friction Tester (PFT) conforming to these requirements was used to complete the testing. The 1295 PFT can travel the highway at normal traffic speeds and produce an accurate measurement and record of highway friction values using full size ASTM E501 test tires. The 1295 PFT fully complies with the requirements set out in ASTM E-274 "Standard Test Method for Skid Resistance of Paved Surfaces Using a Full-Scale Tire". In accordance with ASTM E-274, the testing was completed at a test speed of 65 km/hr where possible. As a part of the data collection program, the following was considered:

- The start and end distances from the centreline of permanent landmarks at start and end of each highway surveyed were recorded.
- Kilometre distances of each Skid Number (SN) test result and speed encountered at each test cycle, using the left (driver side) tire mounted on an ASTM E274 trailer, each referenced from the starting point of each roadway were recorded.

- Minimum of one centreline distance of a permanent landmark (i.e. roadway overpass/underpass, river/creek bridges, railways) within each ten kilometres (10 km) were recorded.

To perform the test, water was dispensed onto the pavement immediately ahead of the tire on the trailer and the trailer braking system was actuated to lock the test wheel (typically, only the wheel on the driver's side of the trailer is used to test). The system detects and records the horizontal tractive force, which is the force necessary to slide the locked test tire along the pavement at the test speed, the vertical load on the test wheel, and the vehicle speed (PENNDOT, 2008).

A test cycle takes approximately 2.5 seconds. Water dispersion begins 0.1 seconds prior to wheel lock (and continues during the entire test cycle), it takes approximately 1 second to lock the wheel (the higher the speed, the longer it takes to lock the wheel), and measurements are made for one second while the wheel is locked (200 measurements are recorded during that 1 second interval). Water is dispensed at the rate of approximately 28 gallons per minute. The average skid number (SN) for each test cycle equals the Horizontal Tractive Force divided by the Vertical Load, multiplied by 100 (PENNDOT, 2008).

Due to the sensitivity of the friction data and the potential legal risks to the MTO, the regions will be referenced within this research study as Regions A, B, and C. In addition, Highways will also be referenced using an arbitrary number. For the testing, a 1.0 km sampling interval was selected as a reasonable measuring interval to perform the skid testing. This was not based on a specified skid testing protocol or test methodology. At each test point, the skid number (SN), average test vehicle speed, and kilometre post were recorded. Since the skid resistance and aggregate properties of Region A and B are similar, the two regions have been grouped together. Region C has significantly different skid resistance and aggregate properties compared to Regions A and B. As a result, summary statistics are grouped for Regions A and B, while Region C is treated independently. The average network level skid number across the 1,800 km of pavements is 37.5. The highway with the lowest average Skid Number was Highway 6 within Region A and B with an average SN of 33.2, while the highway with the highest average Skid Number was Highway 15 within Region C with an average SN of 56.7. The average skid number for Regions A, B, and C are 36.3, 37.7, and 51.8 respectively. Summary Statistics of the Skid Number results for each highway is presented below in Table 3.2.

Table 3.2: Summary Statistics of Skid Number Results per Highway

Region	Highway Number	Minimum SN	Maximum SN	Average SN	Standard Deviation	Coefficient of Variance
A & B	1	31.8	56.2	41.7	4.6	11%
	2	24.9	57.7	36.7	7.1	19%
	3	27.7	50.9	35.1	5.0	14%
	4	29	45.9	36.2	5.1	14%
	5	25.7	51.5	38.6	6.5	17%
	6	19.6	45.4	33.2	4.9	15%
	7	24.1	48	37.9	4.1	11%
	8	25.7	64.9	40.6	7.2	18%
	9	10.3	56.5	35.4	8.9	25%
	10	30	57.7	40.8	6.0	15%
C	11	45.3	64.8	53.7	2.8	5%
	12	35.1	60.4	49.2	5.8	12%
	13	22.3	67.6	55.0	7.0	13%
	14	41.1	55.5	46.6	3.8	8%
	15	39	68.6	55.0	4.2	8%
	16	35.9	55.6	48.4	4.8	10%
	17	34.4	52.5	46.8	4.0	9%
	18	46.4	60.4	53.6	3.0	6%
	19	49.2	58	54.1	4.5	8%
	20	43.1	66.1	54.7	4.7	9%
	21	37.3	58.2	50.6	4.4	9%
	22	44.2	57.8	50.3	3.3	7%
	23	32.8	63.3	54.3	8.4	15%
	24	26.1	62.6	43.0	14.6	34%
	25	46.7	53.3	49.9	2.1	4%
	26	46.1	62.8	56.7	3.7	7%

Histograms were developed to examine the distributions of the Skid Number for all highways evaluated within the highway network. A histogram representing the distribution of all SN values for all highways under study is presented below in Figure 3.1. It shows approximately 80% of the SN values fall between an SN of 30 and 55, approximately 6% of the SN values are below 30, and approximately 14% of the SN values are greater than 55. As previously mentioned, Regions A and B have similar skid resistance and aggregate properties compared to Region C. As a result, a histogram was developed for Regions A and B (Figure 3.2) and Region C (Figure 3.3).

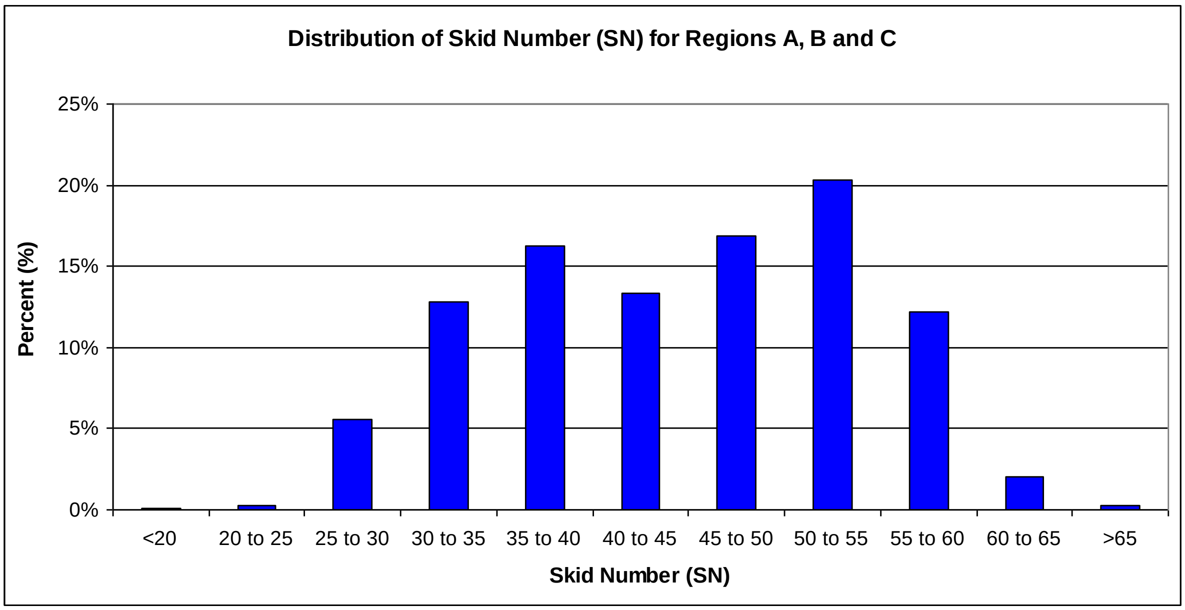

Figure 3.1: Distribution of Skid Number Values for all Regions

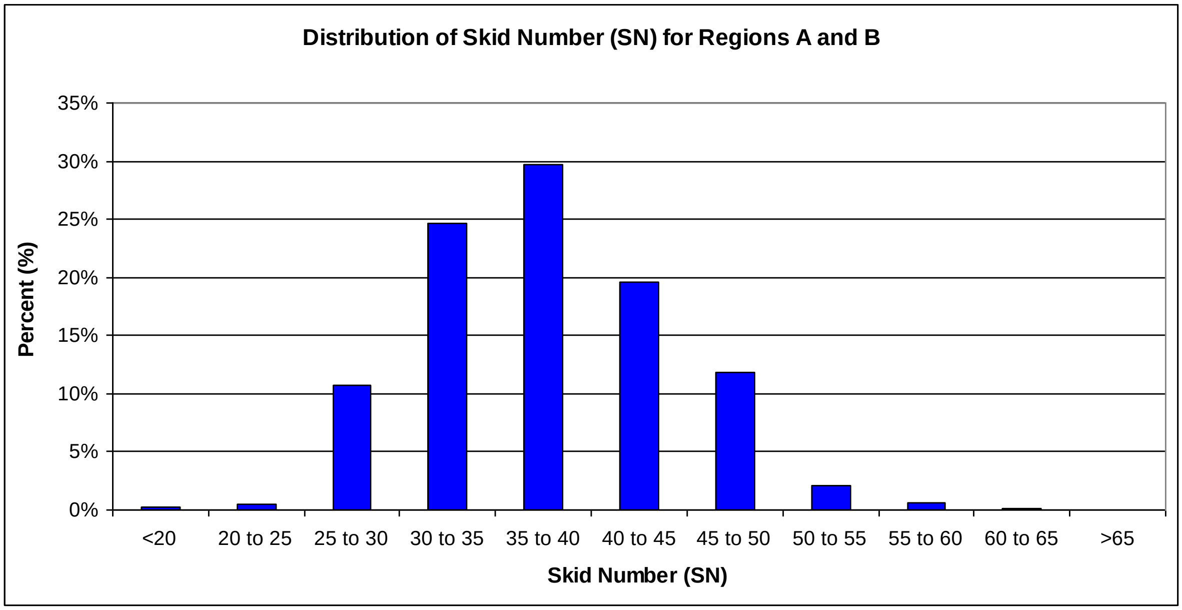

Figure 3.2: Distribution of Skid Number Values for Regions A and B

For Regions A and B, approximately 86% of the SN values fall between an SN of 30 and 50, approximately 11% of the SN values are below 30, and approximately 3% of the SN values are greater than 55. For Region C, approximately 93% of the SN values fall between an SN of 40 and 60, approximately 3% of the SN values are below 40, and approximately 4% of the SN values are greater than 60.

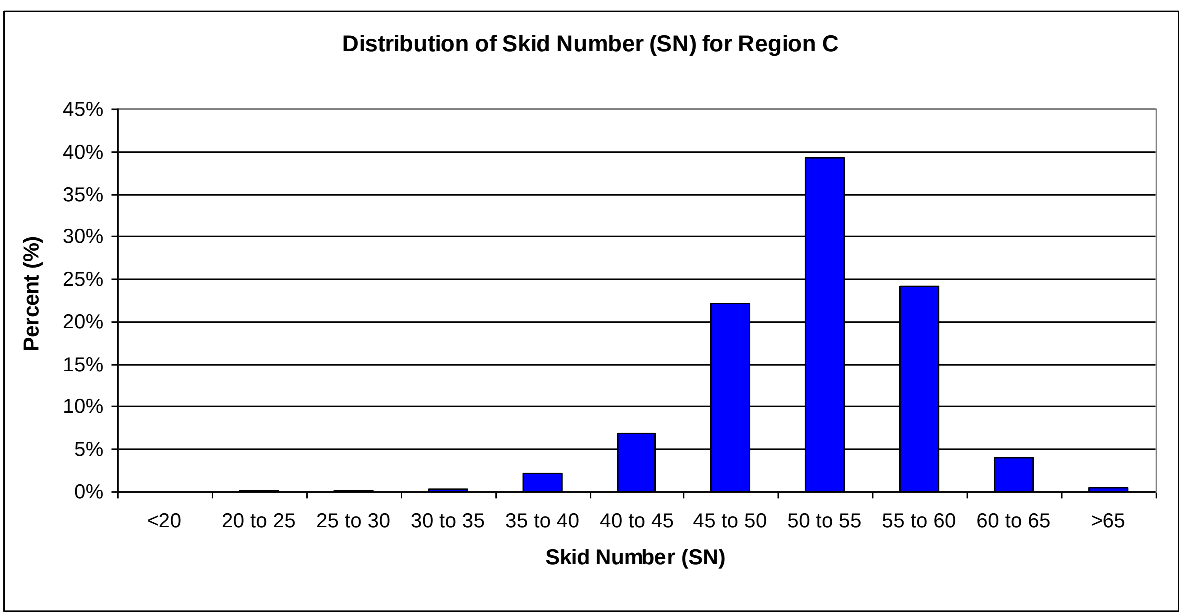

Figure 3.3: Distribution of Skid Number Values for Region C

3.1.3 Collision Data

The Traffic Division at the MTO is responsible for collecting and maintaining a comprehensive vehicle collision database. When a collision occurs on a highway segment, provincial police officers produce a detailed record of the collision including such factors as collision type, weather conditions, surface conditions, location, object of impact, etc. This data is then entered into a Traffic Management System that can be queried and manipulated to extract data and key fields of interest. Due to the sensitivity and confidentiality of the collision data, only information related to the driver's age, gender and condition is provided. No personal information such as name or address is available to the public or researchers.

The collision data set has several attributes associated with each collision record. Attribute data such as Surface Condition, Driver Condition, Sex of Driver, Environment Condition, Collision Severity and many others are included in the data set. Summaries of the collision data by Season, Severity, Surface Condition and Driver Sex are presented below in Figure 3.4. A summary of the Collision Data is presented in Table 3.3.

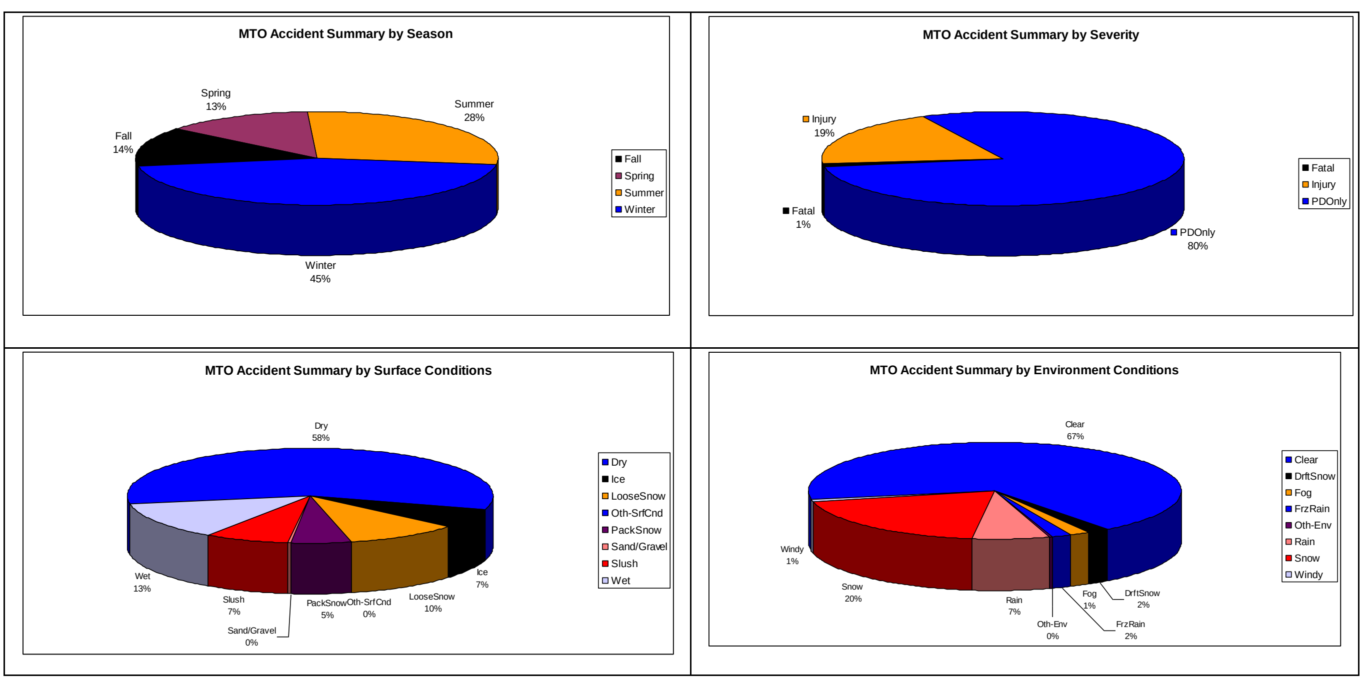

Figure 3.4: Distributions of Collisions by Season, Severity, Surface Condition, and Environment Condition

Table 3.3: Summary of Collisions per Highway

Region	Highway Number	Number of LHRS Sections	Total Number of Collisions	Minimum Number of Collisions	Maximum Number of Collisions
A and B	1	9	399	1	13
	2	18	118	0	34
	3	6	110	0	26
	4	7	96	5	26
	5	7	103	6	22
	6	12	67	0	38
	7	15	32	0	27
	8	10	2	0	17
	9	5	25	7	37
	10	10	12	0	25
C	11	18	6	0	64
	12	9	5	3	35
	13	7	1	6	40
	14	11	3	0	41
	15	5	153	9	32
	16	5	56	4	26
	17	6	273	0	10
	18	13	96	0	23
	19	1	60	2	2
	20	5	83	2	7
	21	5	80	0	8
	22	2	223	1	5
	23	2	187	1	4
	24	1	74	1	1
	25	1	112	3	3
	26	1	N/A	N/A	N/A

3.1.4 Traffic Data

A critical component of any pavement or safety-related study is traffic data. Factors such as the annual average daily traffic (AADT), annual average daily truck traffic (AADTT), and % commercial truck traffic all influence pavement performance and the level of safety of a highway alignment.

Table 3.4: Summary of AADT per Highway

Region	Highway Number	Number of LHRS Sections	Minimum AADT	Maximum AADT	Average AADT
A and B	1	9	5,050	12,900	7,375
	2	18	2,200	18,900	6,757
	3	6	5,900	9,850	7,467
	4	7	3,850	7,800	5,586
	5	7	1,950	5,850	3,571
	6	12	5,300	22,100	9,496
	7	15	1,950	9,450	5,533
	8	10	3,400	9,000	5,713
	9	5	5,350	7,850	6,030
	10	10	2,850	11,400	6,525
C	11	18	7,350	21,500	14,719
	12	9	1,150	8,100	3,017
	13	7	1,500	9,250	4,314
	14	11	6,100	11,200	8,055
	15	5	1,450	5,000	3,020
	16	5	770	5,100	1,934
	17	6	1,350	2,700	1,779
	18	13	8,650	12,200	9,454
	19	1	150	150	150
	20	5	260	1,250	824
	21	5	710	3,350	1,682
	22	2	2,100	2,250	2,175
	23	2	120	1,300	710
	24	1	570	570	570
	25	1	1,050	1,050	1,050
	26	1	220	220	220

Traffic data was obtained from the MTO between the years 2003 and 2005. This data was collected from various fixed traffic data collection sensors (WIM and weigh scales) located across the three regions. A summary of the traffic data for each Highway under study is presented in Table 3.4.

3.1.5 Long Term Pavement Performance Data

The Long Term Pavement Performance (LTPP) Project is the largest pavement research study in North America. Pavement performance data has been collected from over 2,400 pavement sections located across Canada and the United States. These pavement sections consist of a variety of pavement structures in various environmental zones, built on different subgrades and exposed to various levels of traffic.

The LTPP program was initiated in 1987 as a part of the Strategic Highway Research Program (SHRP). The main objective for the LTPP program is to establish a national long-term pavement database to support SHRP objectives and future needs. Currently, the project is managed by the Federal Highway Administration (FHWA) and consists of over 2,400 sections at 932 locations on in-service highways located across North America. The LTPP test sections are classified into a number of studies: General Pavement Studies (GPS) and Specific Pavement Studies (SPS) sections. A GPS test site typically would have one test section, while an SPS test site would have multiple test sections incorporating a controlled set of experiment design and construction features.

LTPP data is collected in a consistent manner at a specific level of accuracy and checked through a series of Quality Assurance (QA) checks. Also, maintenance activities are monitored and recorded, thus addressing some of the possible sources of inconsistencies in historic performance data. The construction activities in the LTPP are defined at a very detailed level to allow for further research into specific treatments. This includes details in the maintenance activities, such as overlays or surface treatments that were implemented on the sections after the original rehabilitation activity. These maintenance activities will have an impact on the pavement performance. Therefore, the performance data considered in the analysis were those collected before and after the M, R & R activities were initiated. When an LTPP site is first entered into the study, it is identified as Construction Number 1. When the test site undergoes an M, R &R treatment, it changes Construction Numbers from 1 to 2. The reason for the change is also documented by a code which represents the various M, R, & R treatments. The data sources used from DataPave Online are presented below in Table 3.5.

Table 3.5: Data Types and Sources from LTPP Database

Data Type	LTPP DataPave Module	LTPP Table Name
Construction Date and M&R activities type	Administration	EXPERIMENT_SECTION
Pavement type, lane width, and other general information	Inventory	INV_GENERAL
Section location, route number, mileposts	Inventory	INV_ID
Historical precipitation data	Climate	CLM_VWS_PRECIP_ANNUAL
Historical temperature data	Climate	CLM_VWS_TEMP_ANNUAL
Friction (SN) measurements	Pavement monitoring	MON_FRICTION

Chapter 4
Data Integration and Aggregation

An important component of any research study involving large databases and statistical analyses is a structured and complete data set. This section outlines the linkage and aggregation of the various data sets and attributes.

4.1 Data Linkage

In Chapter 3, an overview of the data sets used as a part of this study was provided. As previously mentioned, friction data in terms of a skid number (SN) was collected along a single direction of each highway segment in the network. The friction data was collected continuously with increasing chainage (kilometres) for the positive directions (north and east) and decreasing chainage for the negative directions (south and west). It is very important to note that the referencing for the friction data is significantly different than the referencing for the LHRS, collision locations, and traffic data by referencing cross roads, bridges, and jurisdictional boundaries.

The collision data is referenced by the kilometre-post at the location of the collision and is referenced to the LHRS and correct offset. Each highway section is segmented into manageable LHRS sections that "reset" the linear offset distance along the length of each new section (LHRS). Skid data was collected at an interval of 1.0 km along the length of each highway section (and LHRS) and a collision record can occur anywhere along the length of each LHRS.

Traffic data were referenced similar to the collision data using the LHRS system which is referenced to the kilometre-post at the location of the traffic measurement. Each highway section is segmented into manageable LHRS sections that "reset" the linear offset distance along the length of each new section (LHRS). This allowed for straight-forward and efficient linkage of the LHRS, collision and traffic data sets.

To correctly assign a friction value to a collision location, the nearest friction value (SN) was assigned to the corresponding collision location. Since the skid data and collision data are referenced using different linear offsets, a considerable effort was undertaken to link or integrate the two independent datasets. In addition, the data file structure of friction data was not in a consistent format. The simplest and most effective approach to link large datasets would have been to use Database software such as Microsoft ACCESS or to develop a macro or computer program to link the four independent data sets. However, due to the inconsistencies in the friction data file formats, a cumbersome (manual) approach had to be undertaken. This was a critical step for any model development and statistical analysis and stresses the importance of an integrated test-setup, protocols, and procedures when performing any data collection activities. A simplified example of the data integration for the friction data, collision locations, traffic data and LHRS is illustrated in Figure 4.1 and summarized in Table 4.1. This is very similar to "Dynamic Sectioning" which creates homogeneous sections and is a critical component of most Pavement Management Systems (PMS).

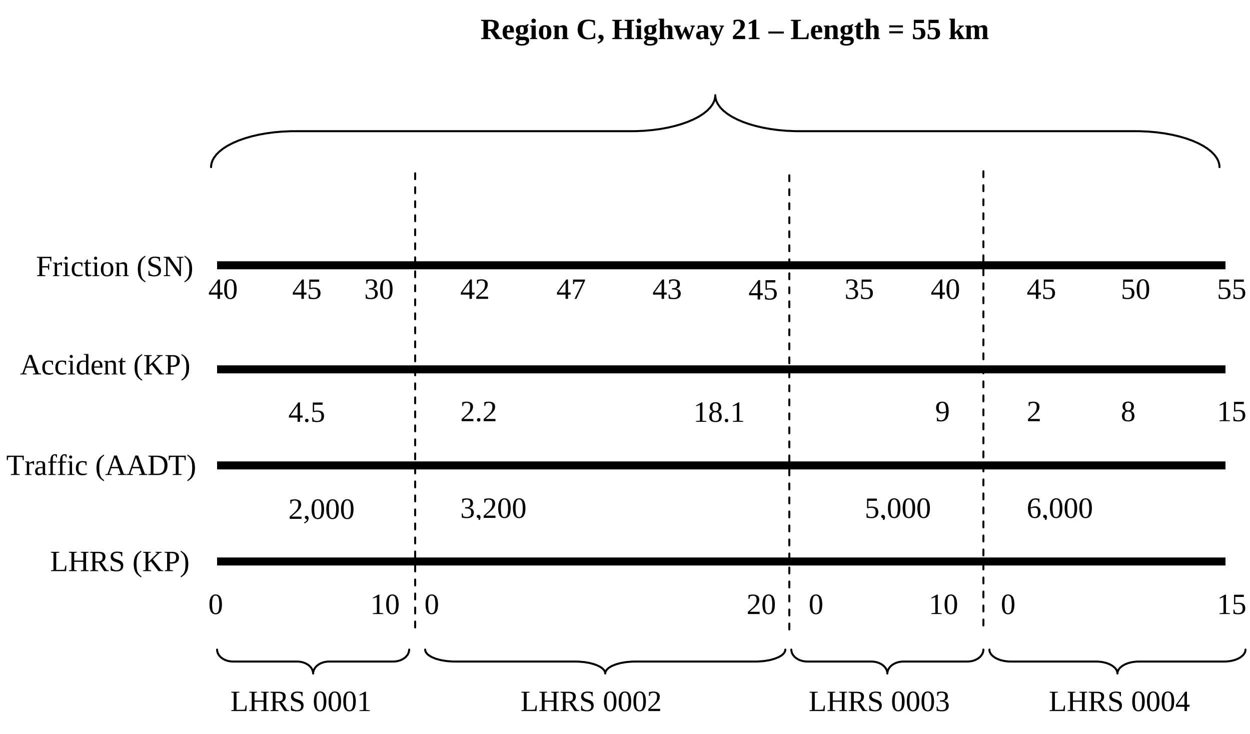

Figure 4.1: Simplified Example of Data Integration

Table 4.1: Results of Data Integration for Highway 21

Highway Number	LHRS	Location (KP)	Traffic (AADT)	Skid Number
21	0001	4.5	2,000	45
21	0002	2.2	3,200	42
21	0002	18.1	3,200	44 (avg of 43 and 45)
21	0003	9	5,000	40
21	0004	2	6,000	45
21	0004	8	6,000	50
21	0004	15	6,000	55

Upon completion of the integration, the dataset was checked for quality and completeness by filtering and applying queries to the data set. Invalid or erroneous skid test results were filtered from the data set. At a few locations, there were issues with the skid testing results due to equipment issues or debris on the surface. These results were filtered from the dataset. Length and completeness checks were also performed to ensure that all data was linked to the proper LHRS.

4.2 Data Aggregation

The vehicle collision database is a comprehensive data set with several fields related to various attributes and characteristics of the collision. Each data field is typically a categorical or descriptive variable. Since many of the categorical variables are similar in nature, they were “grouped” or aggregated into similar variable classes. This reduces the complexity and size of the dataset and redundancy within the variable classes.

As an example, for the data field "*surface condtion*", the following variables are attributed to surface condition: Dry, Ice, Loose Snow, Other, Packed Snow, Sand/Gravel, Slush and Wet. The surface condition field was reduced from 8 descriptive variables into 5 as a result of the aggregation. The variables that were investigated as a part of this study are presented below in Table 4.2.

Table 4.2: Collision Variables Included in Study

Collision Database Parameter	Number of Variable Classes	Variables	Number of Aggregated Classes	Variables
Region	3	▪ A ▪ B ▪ C	2	▪ A ▪ B
Collision Severity	3	▪ Property Damage ▪ Injury ▪ Fatality	N/A	N/A
Season	4	▪ Spring ▪ Winter ▪ Summer ▪ Fall	N/A	N/A
Environmental Condition	8	▪ Clear ▪ Drift Snow ▪ Fog ▪ Freeze Rain ▪ Other ▪ Rain ▪ Snow ▪ Windy	N/A	N/A
Visibility	5	▪ Clear ▪ Fog ▪ Other ▪ Rain ▪ Snow	N/A	N/A

Collision Database Parameter	**Number of Variable Classes**	**Variables**	**Number of Aggregated Classes**	**Variables**
Surface Condition	8	▪ Dry ▪ Ice ▪ Loose Snow ▪ Other ▪ Packed Snow ▪ Sand/Gravel ▪ Slush ▪ Wet	5	▪ Dry ▪ Ice ▪ Snow ▪ Wet ▪ Other
Roadway Location	13	▪ 2 Way LT Lane ▪ Left Shoulder ▪ Left Turn Lane ▪ Off Highway ▪ Off Road Left ▪ Off Road Right ▪ Other ▪ Passing Lane ▪ Right Shoulder ▪ Right Turn Lane ▪ Right Turn Chan ▪ Thru Lane ▪ Within Intersection	4	▪ Intersection ▪ Mainline ▪ Off Road ▪ Shoulder

Chapter 5
Assessing the Level of Safety of a Highway Network Using Network Level Friction Data

This Chapter presents the development of a framework for assessing the level of safety of a highway network in terms of the risk of collision based on pavement surface friction. The developed safety framework can be used by transportation agencies (federal, state, provincial, municipal, etc.) or the private sector (consultants, contractors, concessionaires, etc.) to evaluate the safety of their highway networks and to determine the risk or probability of a collision occurring given the level of friction along the pavement section of interest.

5.1 Study Objectives

The purpose of this study is to develop an approach for assessing the level of safety of a highway network using network level friction and collision data. No previous studies of this kind have been performed in the Province of Ontario at the network level. As a part of this research study, a significant data collection program consisting of network level friction data, traffic data, collision data, and highway attribute data was undertaken. The development of any models or statistical analysis requires both high quality data and a structured data set. To assess the safety and risk of collision on the highway network, a multi-step procedure was developed (Figure 5.1). This section provides an outline of the various data elements, provides an overview of the data integration and linkage, and presents the results of the statistical analysis and model development.

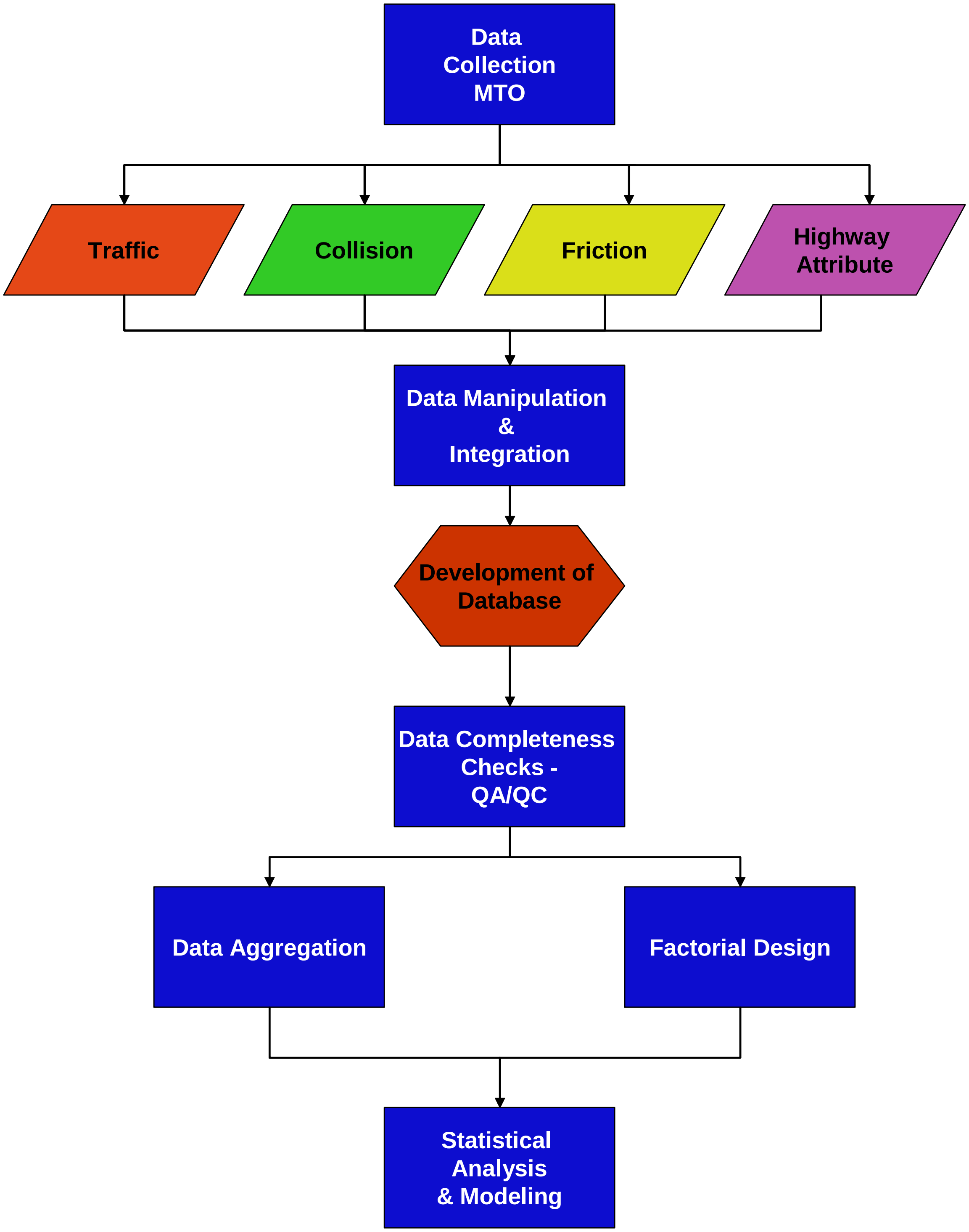

Figure 5.1: Analysis Approach

5.2 Data Attributes

As described in detail in Chapter 3, all data for this study was obtained with the permission of the Ontario Ministry of Transportation (MTO). Each data element was checked for completeness; QA/QC'd, and formatted prior to analysis. The next subsections provide a brief overview of the various data elements of this study.

5.2.1 Surface Friction

In preparation for a considered Long Term Area Maintenance Contract, a project was initiated by the Ontario Ministry of Transportation (MTO) to collect network level friction data across three regions in the Province of Ontario. In 2006, approximately 1,800 km of the Provincial highway network was surveyed as a part of this study. Due to the sensitivity of the data and the potential risk to the agency, the regions will be referenced within this thesis as Regions A, B, and C. Friction data was collected to determine a baseline of the current network friction levels in terms of a skid number. In addition to friction data, traffic and collision data were also obtained from the Traffic Department within MTO. A trailer mounted locked-wheel skid tester was used to collect the friction data. A 1.0 km sampling interval was selected as a reasonable measuring interval to perform the skid testing. At each test point, the skid number (SN), average test vehicle speed, and kilometre post were recorded.

For skid testing, generally only one direction was tested. The direction with the more pronounced downhill gradient was surveyed, where it was assumed that a higher portion of the collisions had taken place. The average network level skid number for the three regions is 37.5. The average skid number for Regions A, B, and C are 36.3, 37.7, and 51.8 respectively.

5.2.2 Traffic Data

A critical component of any pavement or safety related study is traffic data. Factors such as the annual average daily traffic (AADT), annual average daily truck traffic (AADTT), and % commercial truck traffic all influence pavement performance and the level of safety of a highway alignment. Traffic data was obtained from the MTO between the years 2003 and 2005. This data was collected from various fixed traffic data collection sensors (WIM and weigh scales) located across the three regions. The traffic data was used to compute the Collision Rate (CR).

5.2.3 Collision Data

The Traffic Department at the MTO is responsible for collecting and maintaining a comprehensive vehicle collision database. When a collision occurs on a highway segment, provincial police officers produce a detailed record of the collision including such factors as collision type, weather conditions, surface conditions, location, object of impact, etc. This data is then entered into a Traffic Management System that can be queried to extract data and key fields of interest. Due to the sensitivity and confidentiality of the collision data, only information related to the driver's age, gender and mental condition is provided. No personal information such as name or address is available to the public or researchers.

5.2.4 Highway Attribute Data

The MTO uses the Linear Highway Referencing System (LHRS) to section their highway segments into manageable pavement sections. Each highway segment is referenced with a unique highway identification number. As an example, a highway might have a length of 100 km and have 12 unique individual LHRS sections each of varying length. Each unique LHRS section "resets" the linear offset distance at the start of each new section. All collision data and locations are referenced to the LHRS and the correct linear offset.

5.2.5 The Highway Network

The highway network in question is comprised of approximately 1,800 km of highway located across three regions in Southern Ontario. This data was collected across 33 individual highway segments consisting of an assortment of functional classes (2 lane undivided, 4 lane divided, 6 lane divided, etc.).

5.3 Data Linkage and Integration

An important component of any research study involves usage and linking of a large data set and completion of statistical analyses in a structured manner. This section outlines the data linkage and aggregation requirements for the various data and describes the attributes that are examined in this study. Details on the data linkage and integration were provided in Chapter 4.

5.3.1 Data Linkage

There were several challenges involved with developing the linkages between the various types of data. The friction data in terms of a skid number (SN) was collected along a single direction of each highway segment in the network. The skid data was collected continuously with increasing chainage (kilometres) for the positive directions (north and east) and decreasing chainage for the negative directions (south and west). It is important to note that the referencing is different than the LHRS and collision locations, by referencing cross roads, bridges, and jurisdictional boundaries. The collision data is referenced by the kilometre-post at the location of the collision and is referenced to the LHRS and correct offset. Each highway section is segmented into manageable LHRS sections that "reset" the linear offset distance along the length of each new section (LHRS).

As previously mentioned, skid data was collected at an interval of 1.0 km along the length of each highway section (and LHRS) and a collision record can occur anywhere along the length of each LHRS. To correctly assign a friction value to a collision location, the closest friction value (SN) was assigned to the corresponding collision location. Since the skid data and collision data are referenced using different linear offsets, a considerable effort was undertaken to link or integrate the two independent datasets. As previously mentioned in Chapter 4.0, the inconsistencies in the file formats of the collected skid data created a significant challenge and resulted in considerable time and efforts. This is a critical step for any model development and statistical analysis and stresses the importance of an integrated test-setup, protocols, and procedures when performing any data collection activities.

5.3.2 Data Aggregation

The vehicle collision database is a comprehensive data set with several fields related to various attributes and characteristics of the collision. Each data field is typically a categorical or descriptive variable. Since many of the categorical variables are similar in nature, they were “grouped” or aggregated into similar variable classes. This reduces the complexity and size of the dataset and redundancy within the variable classes. Details on the aggregation of the collision data were provided in Chapter 4.0.

5.4 Analysis and Modeling

The following sections discuss the various statistical analyses and model development components that were completed as a part of this study. A multi-step approach was used to develop the safety framework to asses the level of risk or probability of collision occurrence as a function of the friction level along a pavement section or highway network.

5.4.1 Categorization of Data and Descriptive Statistics

As a first step, the friction and collision data was examined on a point-by-point basis. At this level, a relationship between the collision rate (or number of collisions) and the level of friction (skid number) were developed. As can be seen in Figure 5.2, there is no correlation in the data (R^2 = 0.0024). In addition, when the friction levels were examined in detail within each region, the pavements in Region C showed significantly higher friction levels compared to the other two regions.

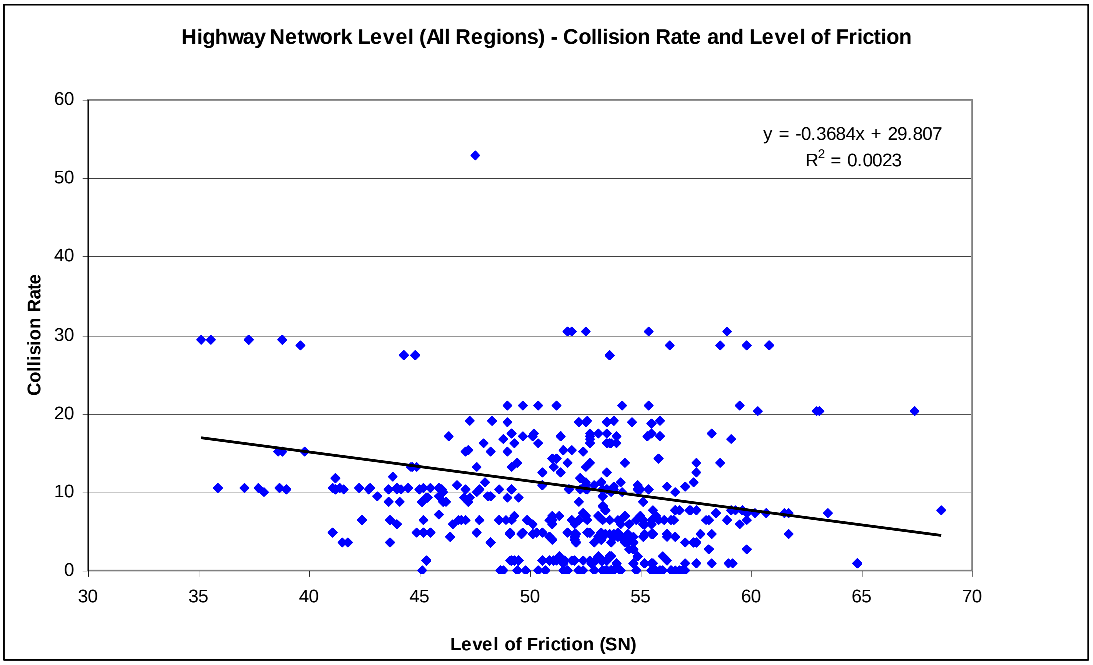

Figure 5.2: Relationship Between Friction and Collision Rate on a Point-by-Point Basis

The minimum, maximum, and average skid number values in Region C are 35.1, 68.6 and 51.8 respectively. Since Region C is located within the Canadian Shield, which is well known for its very hard rock formations, the aggregate sources in this region have excellent skid resistance properties. As a result, other factors such as geometric design and site distance may have a more profound affect on highway safety within this region. Due to this fact, Region C was removed from the analysis since it did not represent typical pavement surface friction levels in Southern Ontario, where the greatest part of the province's highway network is located.

Since the first method did not provide meaningful relationships or strong correlations, the friction and collision data were examined in grouped ranges or bins. At this level, a relationship between the total number of collisions within a given friction level were examined at the network level (with Region C now removed from the study). The friction data was categorized or grouped into the following bins presented in Table 5.1.

Table 5.1: Categorized Friction Data

Bin Number	Skid Number Range	Mid Point of Range	Comments
1	SN <= 32	30	TAC identifies potential collision problem
2	32 <= SN < 34	33	
3	34 <= SN < 36	35	No collision problem. TAC recommends surveillance and take corrective action as required
4	36 <= SN < 38	37	
5	38 <= SN < 40	39	
6	40 <= SN < 42	41	No further action required (TAC)
7	42 <= SN < 44	43	
8	44 <= SN < 46	45	
9	46 <= SN < 48	47	
10	48 <= SN < 50	49	
11	50 <= SN < 52	51	
12	SN >= 52	55	

The bin ranges were established by examining the distribution of the skid number within each region (Region A and B), a review of the literature (TAC, 1997) & (VTRI, 2001), and engineering judgement. Descriptive statistics summarizing the average, minimum, maximum, range and standard deviation for Regions A and Region B are presented below in Table 5.2.

It is worth noting that this is a common modeling technique used in the pavement management and engineering field. An excellent example of this technique being implemented and used on a large scale is in a number of models found within the new Mechanistic-Empirical Pavement Design Guide (M-E PDG).

Table 5.2: Descriptive Statistics for Regions A and B

MTO Region	Bin No.	Bin Range	Total Number of Collisions	SN_{avg}	SN_{min}	SN_{max}	SN_{range}	SN_{stdev}
Region A	1	<=32	241	29.0	10.3	31.9	21.6	2.6
	2	32 to 34	77	33.0	32.1	34.0	1.9	0.6
	3	34 to 36	70	35.1	34.1	35.8	1.7	0.5
	4	36 to 38	91	37.2	36.1	38.0	1.9	0.5
	5	38 to 40	63	39.1	38.1	40.0	1.9	0.7
	6	40 to 42	50	40.7	40.1	41.7	1.6	0.5
	7	42 to 44	24	43.1	42.2	44.0	1.8	0.5
	8	44 to 46	38	44.9	44.2	45.8	1.6	0.4
	9	46 to 48	41	47.1	46.3	47.9	1.6	0.5
	10	48 to 50	9	48.8	48.2	49.8	1.6	0.7
	11	50 to 52	4	51.1	50.4	51.5	1.1	0.5
	12	>=52	5	55.4	52.5	57.7	5.2	2.7
Region B	1	<=32	107	30.2	25.0	31.9	6.9	1.5
	2	32 to 34	63	33.1	32.2	34.0	1.8	0.6
	3	34 to 36	86	35.1	34.2	36.0	1.8	0.5
	4	36 to 38	61	37.2	36.1	37.9	1.8	0.5
	5	38 to 40	61	38.9	38.1	40.0	1.9	0.6
	6	40 to 42	44	40.8	40.2	42.0	1.8	0.5
	7	42 to 44	37	43.1	42.1	44.0	1.9	0.7
	8	44 to 46	28	45.1	44.1	45.9	1.8	0.6
	9	46 to 48	20	46.9	46.1	48.0	1.9	0.5
	10	48 to 50	8	48.7	48.1	49.3	1.2	0.5
	11	50 to 52	11	50.5	50.1	50.9	0.8	0.4
	12	>=52	3	61.6	55.1	64.9	9.8	5.7

5.4.2 Model Development

The friction and collision data were examined in grouped ranges or bins. At this level, the total number of collisions and the collision rate within a given friction level (bin) were analyzed at the network level. A non-parametric linear regression was performed to develop a number of Model Classes with the total number of collisions as the dependent variable and the level of friction (SN) as the independent variable. In addition, models were also developed with the collision rate (CR) as the dependent variable and the level of friction (SN) as the independent variable. The collision rate (CR) is calculated from the following equation:

$$CR = \frac{\text{Total Number of Collisions} * 1{,}000{,}000 \text{ vehicles}}{\text{AADT} * \text{length of highways} * 365 \text{ days}} \qquad [3]$$

A CR was calculated for each level of friction (bins). The total number of collisions, the AADT and length of highways was calculated for all highways within the given bin. In total, 7 Model Classes were developed and a summary of the developed models for each class is presented in Table 5.3. In addition, statistical testing was performed on the seven Model Classes using an Analysis of Variance (ANOVA) or a Student’s T-Test to determine if the various means within each of the model classes are significantly different at the 95% confidence interval.

5.4.2.1 Region

Two regions were examined as a part of this study; Regions A and B. In total, three models were developed within the Region Model Class. A model was developed for each region as well as a combined model representing the conditions within both regions. Region A had 712 collisions, while Region B had 529 collisions for a total of 1,242 collisions. Models were developed with the total number of collisions as the dependent variable and the level of pavement surface friction (SN) as the independent variable. An exponential model was used to best fit the data with R^2 values ranging from 0.85 to 0.91. Similar to the model for total number of collisions, models were also developed based on the collision rate (CR) as the dependent variable and the level of pavement surface friction (SN) as the independent variable. An exponential model was used to best fit the data with R^2 values ranging from 0.86 to 0.92. The models for the total number of accidents and the collision rate are presented in Figures 5.3 and 5.4 for Both Regions, respectively. The models for Region A and B are presented in Appendix A.

A summary of the Region Model Classes is presented in Table 5.3. The results of the statistical analysis clearly illustrate the impact of the level of available pavement surface friction on the total number of collisions and the collision rate for a highway network. As the level of available friction increases, the collision rate was observed to decrease. In addition, an average increase in the collision rate of approximately 100% was observed when the skid number drops from a level of 32 to 35 SN to a skid number value less than 32. This may indicate that an SN between 32 and 35 may be a critical level of friction which has been reported by TAC and other agencies.

An Analysis of Variance (ANOVA) test was completed to determine if there were significant differences within the Regions. The results of the ANOVA indicate that the region Model Classes are significantly different at the 95% confidence interval as presented in Table 5.4. This indicates that there are different levels of performance within each Region.

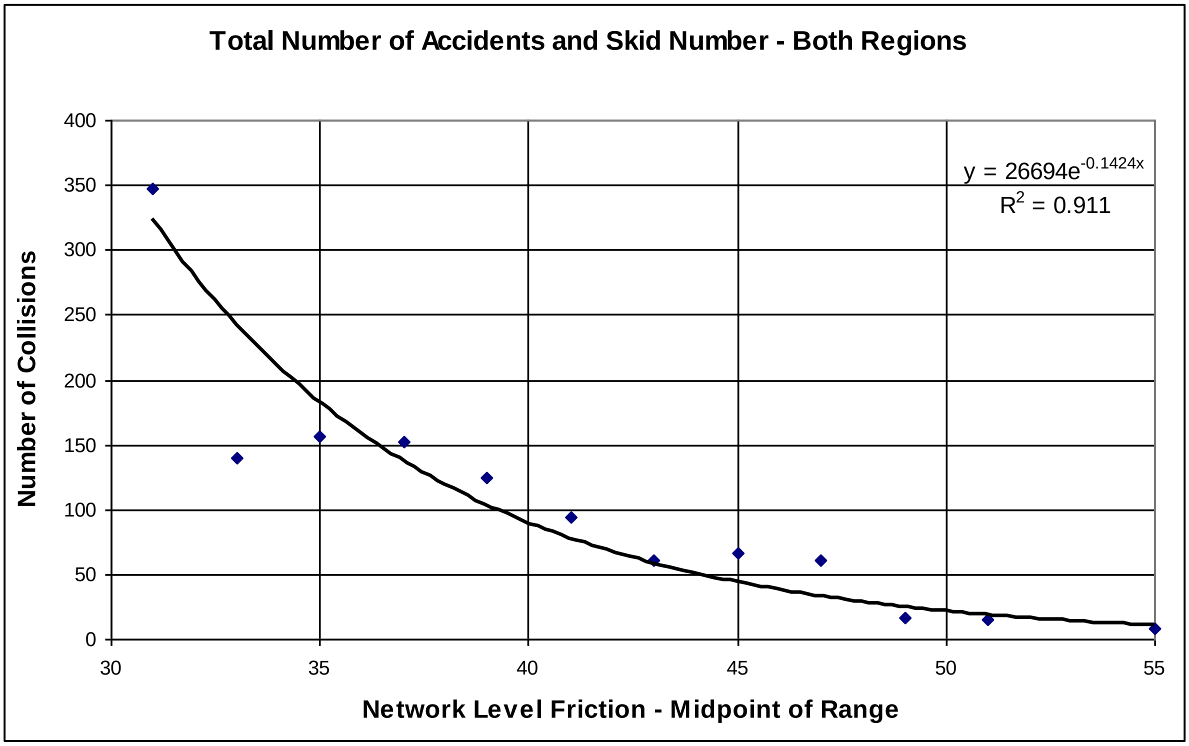

Figure 5.3: Number of Accidents and Skid Number – Both Regions

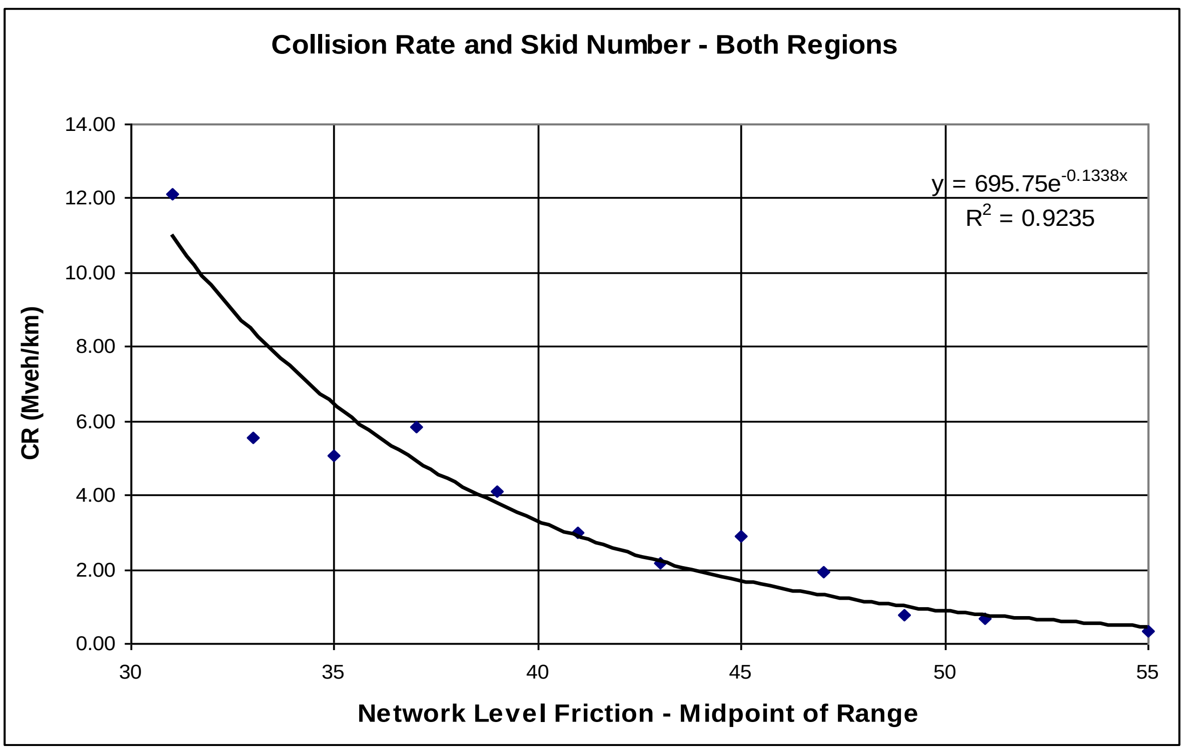

Figure 5.4: Collision Rate and Skid Number – Both Regions

5.4.2.2 Collision Severity

The MTO's vehicle collision database identified three collision severity types: fatal, personal injury, and property damage. In total, 3 models were developed within the Collision Severity Model Class. A model was developed representing each collision severity type. In the two combined regions, there were 12 fatal collisions, 253 personal injury collisions, and 977 collisions involving property damage.

For the personal injury and property damage collision severity types, an exponential based model was used to best fit the data with R^2 values of 0.77 and 0.92 respectively. For the fatal collisions, a power based model was used to best fit the data with an R^2 value of 0.35. Similar to the model for total number of collisions, models were also developed based on the collision rate (CR) as the dependent variable and the level of pavement surface friction (SN) as the independent variable. For the fatal and property damage collision severity types, an exponential model was used to best fit the data with R^2 values ranging from 0.67 to 0.94. For the injury collisions, a power based model was used to best fit the data with an R^2 value of 0.73. The model for the fatal severity class had the lowest correlation (R^2 value) compared to the injury and property damage classes possibly due to the low number of observations within the class. As was previously mentioned, there were only 12 fatal collisions compared to 253 personal injury and 977 collisions involving property damage. The models for the total number of accidents and the collision rate are presented in Figures 5.5 and 5.6 for fatal collisions, respectively. The models for injury and collisions involving property damage are presented in Appendix A.

A summary of the Collision Severity Model Classes is presented in Table 5.3. The results of the ANOVA indicate that collision severity are significantly different at the 95% confidence interval as presented in Table 5.4.

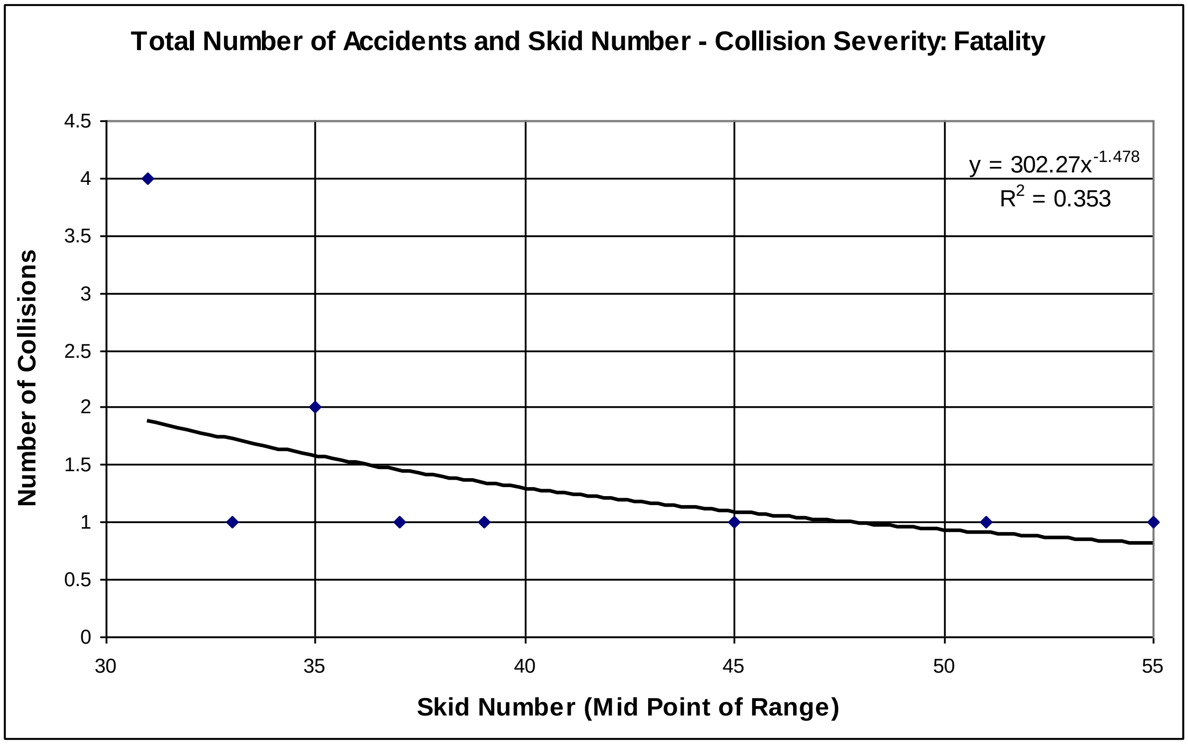

Figure 5.5: Number of Accidents and Skid Number – Fatal

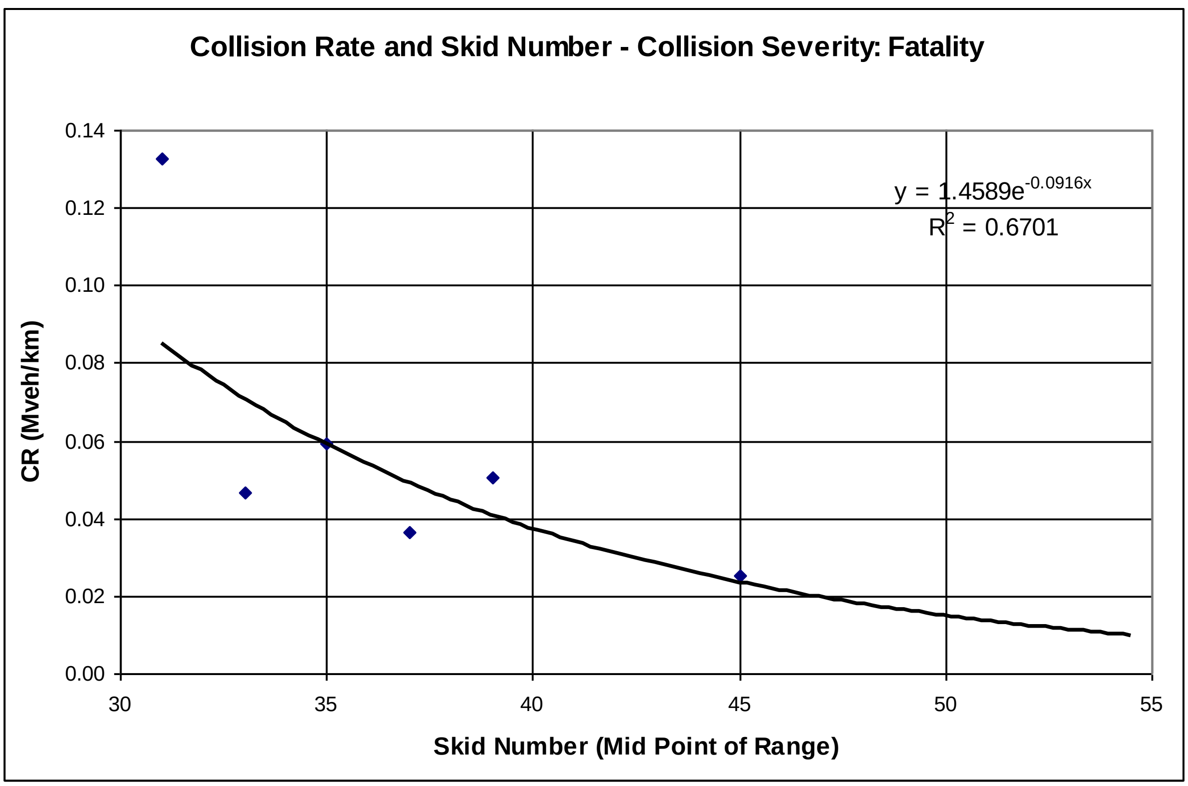

Figure 5.6: Collision Rate and Skid Number – Fatal

5.4.2.3 Environmental Condition

Seven environmental condition classes were examined as a part of this study; clear, drifting snow, fog, freezing rain, rain, snow, and wind. For the environmental condition classes, clear had the highest number of collisions with 800, followed by snow with 271, rain with 69, drifting snow with 45, fog with 20, freezing rain with 18, and wind with 18.

The higher number of collision for the clear conditions could be attributed to human factors. When environmental conditions are clear, drivers tend to exceed the posted speed limit and the 85th percentile operating speed. For clear, rain, snow and windy conditions, an exponential-based model was used to best fit the data with R^2 values of 0.88, 0.75, 0.88, and 0.185 respectively. For drifting snow, fog, and freezing rain conditions, power based models were used to best fit the data with R^2 values of 0.93, 0.79 and 0.77 respectively. Similar to the models for total number of collisions, models were also developed based on the collision rate (CR) as the dependent variable and the level of pavement surface friction (SN) as the independent variable. For clear, drifting snow, fog, snow and wind conditions, an exponential based model was used to best fit the data with R^2 values of 0.88, 0.75, 0.01, 0.88 and 0.02 respectively. For freezing rain and rain conditions, power based models were used to best fit the data with R^2 values of 0.58 and 0.88, respectively. The models for the total number of accidents and the collision rate are presented in Figures 5.7 and 5.8 for clear collisions, respectively. The models for the other environment conditions are presented in Appendix A.

The low correlations for the freezing rain, fog and wind conditions may be attributed to the fact that friction was probably not a significant factor since visibility (fog) and vehicle dynamics/driver error (wind) were the probable cause of those collisions. Freezing rain can significantly reduce the surface friction of a pavement even if the pavement has high insitu skid resistance properties. A summary of the Environmental Condition Model Classes is presented in Table 5.3. The results of the ANOVA indicate that the environmental condition Model Classes are not significantly different at the 95% confidence interval as presented in Table 5.4.

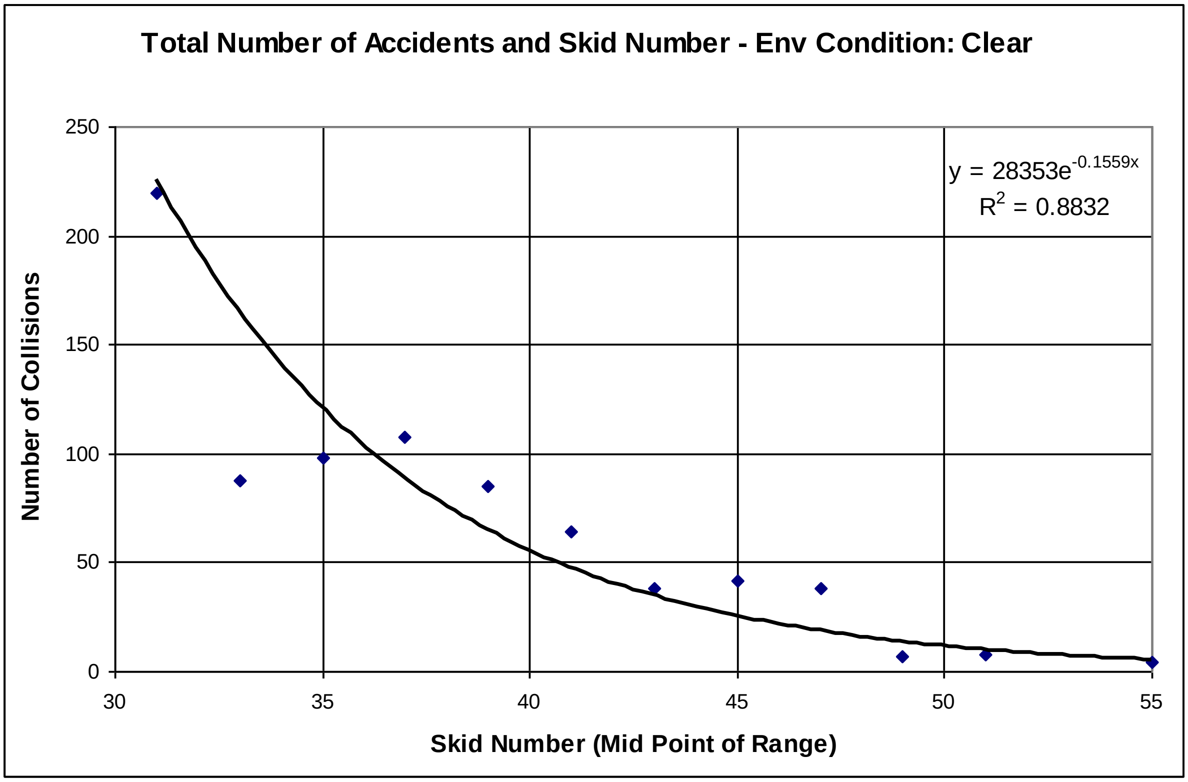

Figure 5.7: Number of Accidents Collisions and Skid Number – Clear

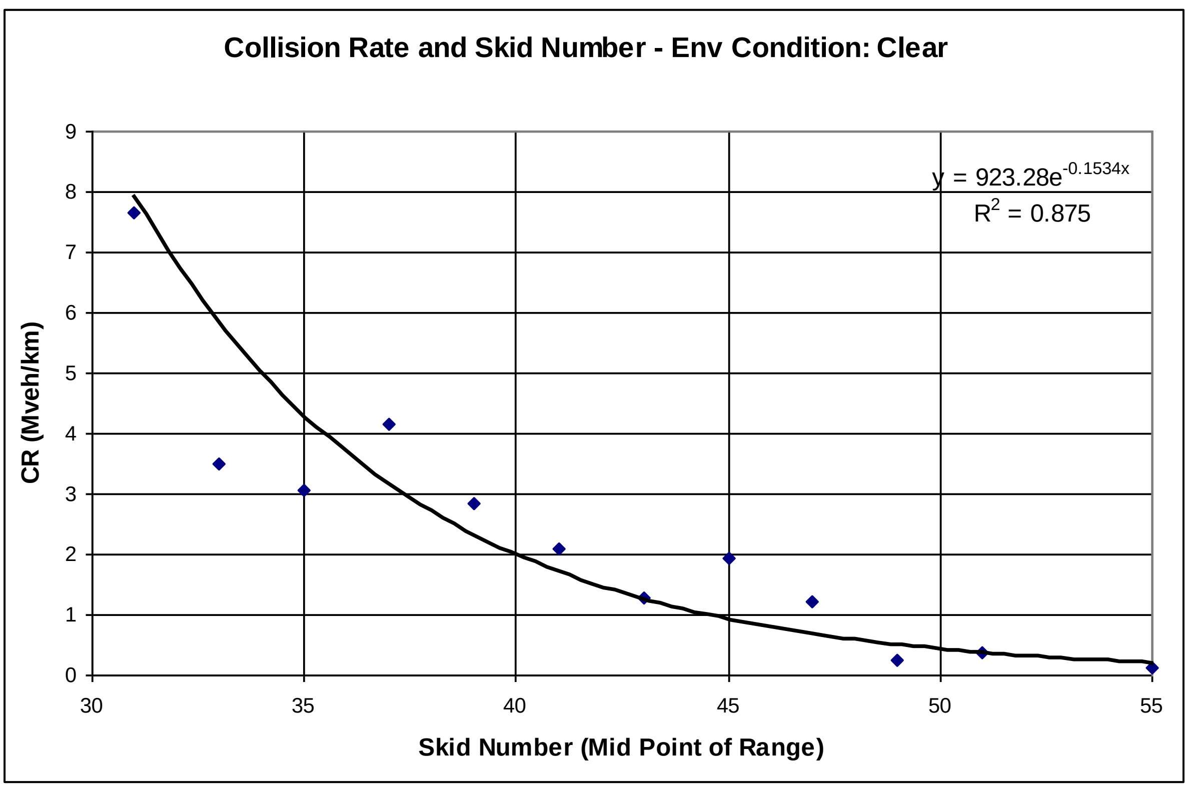

Figure 5.8: Collision Rate and Skid Number – Clear

5.4.2.4 Season

Four models were developed to represent each season of the year – winter, spring, summer and fall. The winter had the highest number of collisions with 581, followed by the summer with 329, the spring with 173, and the fall with 159. For the winter, spring, summer and fall seasons, an exponential based model was used to best fit the data with R^2 values of 0.91, 0.93, 0.86 and 0.70, respectively.

Similar to the model for total number of collisions, models were also developed based on the collision rate (CR) as the dependent variable and the level of pavement surface friction (SN) as the independent variable. For the winter, spring, summer and fall seasons, exponential based models were used to best fit the data with R^2 values of 0.94, 0.92, 0.85 and 0.65, respectively. The models for the total number of accidents and the collision rate are presented in Figures 5.9 and 5.10 for collisions occurring in the spring, respectively. The models for the other seasons are presented in Appendix A.

A summary of the Season Model Classes is presented in Table 5.3. The results of the ANOVA indicate that the season Model Classes are not significantly different at the 95% confidence interval as presented in Table 5.4.

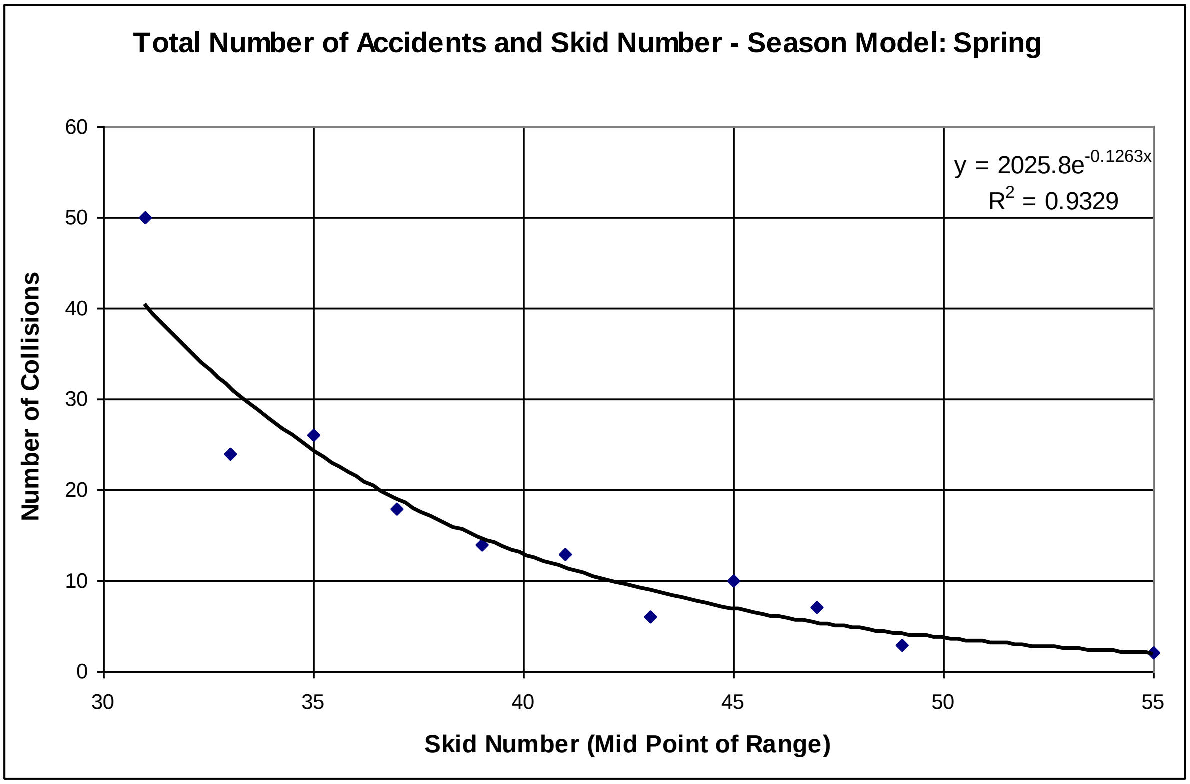

Figure 5.9: Number of Accidents and Skid Number – Spring

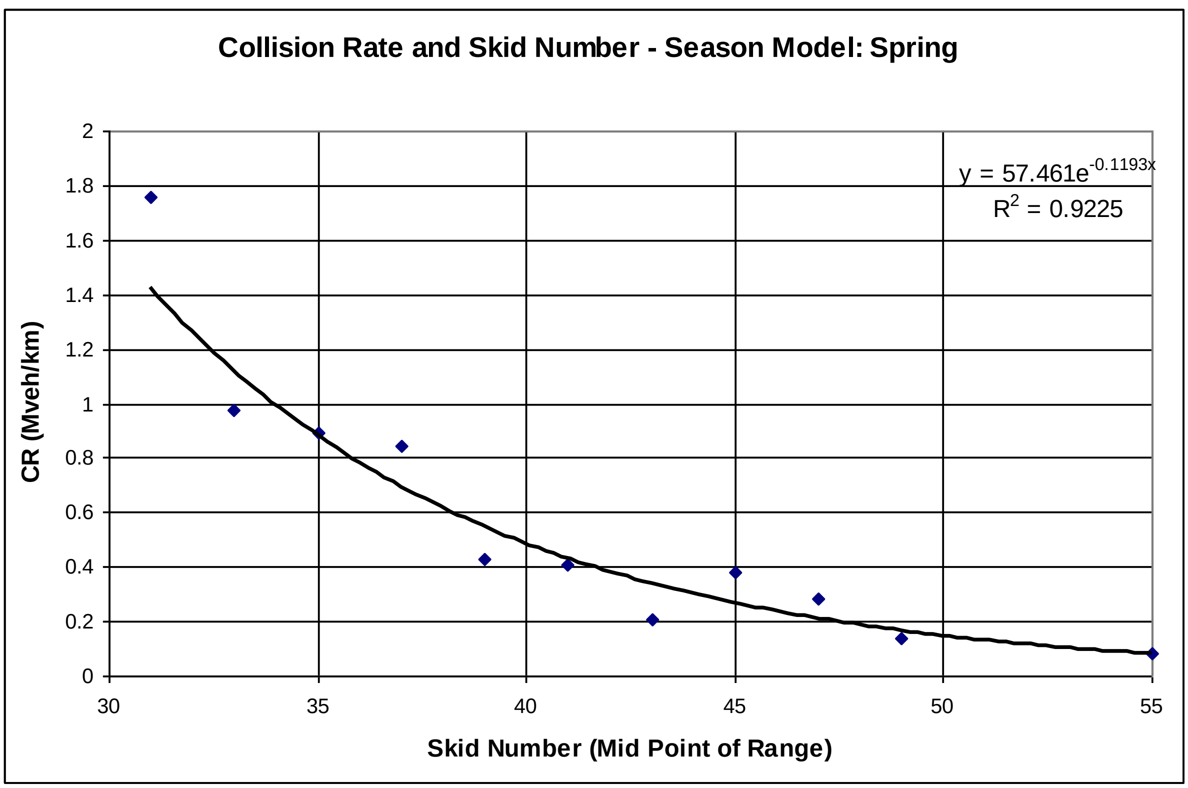

Figure 5.10: Collision Rate and Skid Number – Spring

5.4.2.5 Road Surface Condition

Four models were developed within the Road Surface Condition Model Class. A model was developed for dry, ice, snow and wet pavement surfaces. Dry surface condition had the highest number of collisions with 676, followed by snow surface condition with 308, wet surface condition with 164, and ice surface condition with 90. The higher number of collision for the dry surface could be attributed to human factors. When road surfaces are covered in snow or ice, drivers tend to reduce their speeds and drive with more caution. When pavement surfaces are ideal, drivers tend to exceed the posted speed limit and 85th percentile operating speed (Lamm et al., 1999). In addition, there could have been more "dry surface" days over the year compared to the snow or ice covered surfaces due to winter maintenance activities. For the dry, snow covered and wet surfaces; an exponential based model was used to best fit the data with R^2 values of 0.86, 0.90 and 0.83 respectively. For iced surfaces, a power based model was used to best fit the data with an R^2 value of 0.89.

Similar to the model for total number of collisions, models were also developed based on the collision rate (CR) as the dependent variable and the level of pavement surface friction (SN) as the independent variable. For the dry and snow covered surfaces, an exponential based model was used to best fit the data with R^2 values of 0.86 and 0.87, respectively. For iced and wet surfaces, a power based model was used to best fit the data with R^2 values of 0.90 and 0.72, respectively. The models for the total number of accidents and the collision rate are presented in Figures 5.11 and 5.12 for dry collisions and 5.13 and 5.14 for wet collisions, respectively. The models for the other road surface conditions are presented in Appendix A.

A summary of the Road Surface Condition Model Classes is presented in Table 5.3. The results of the ANOVA indicate that the surface type Model Class is not significantly different at the 95% confidence interval. As a result, the data was re-grouped into dry and wet collisions and a student's t-test was performed which showed that there was a significant difference between the two surface conditions as presented in Table 5.4.

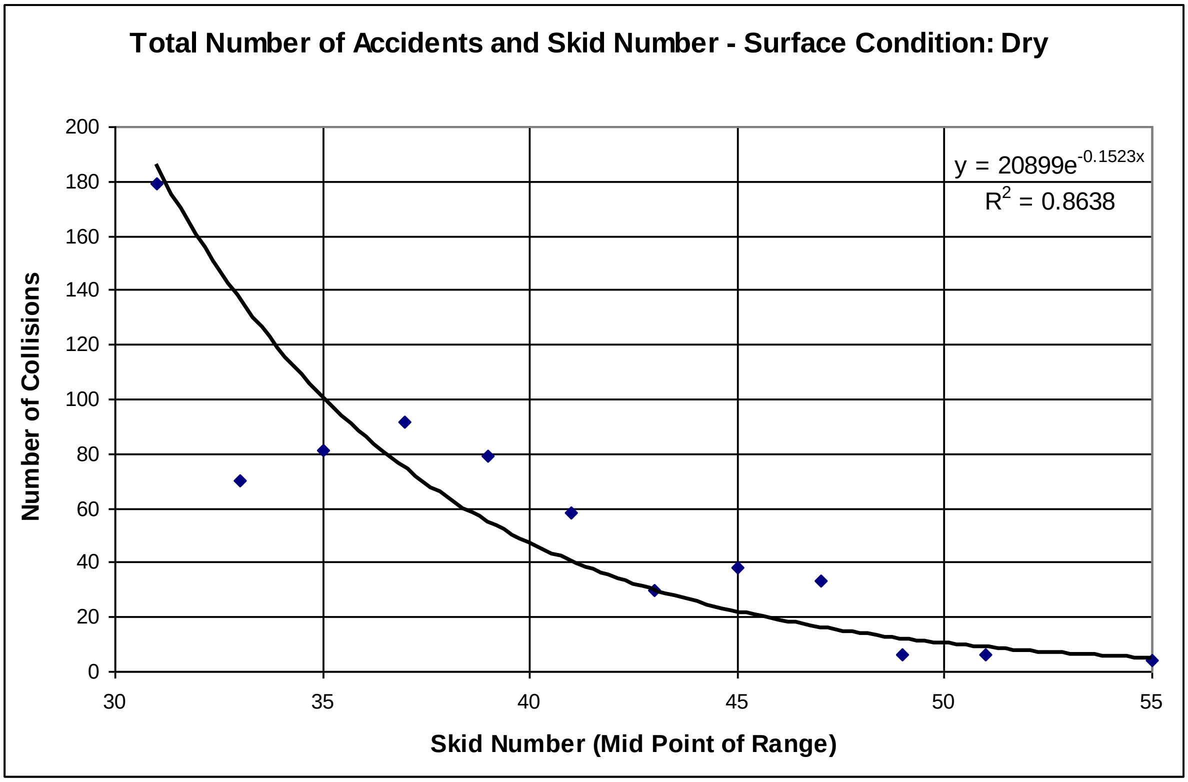

Figure 5.11: Number of Accidents and Skid Number – Dry

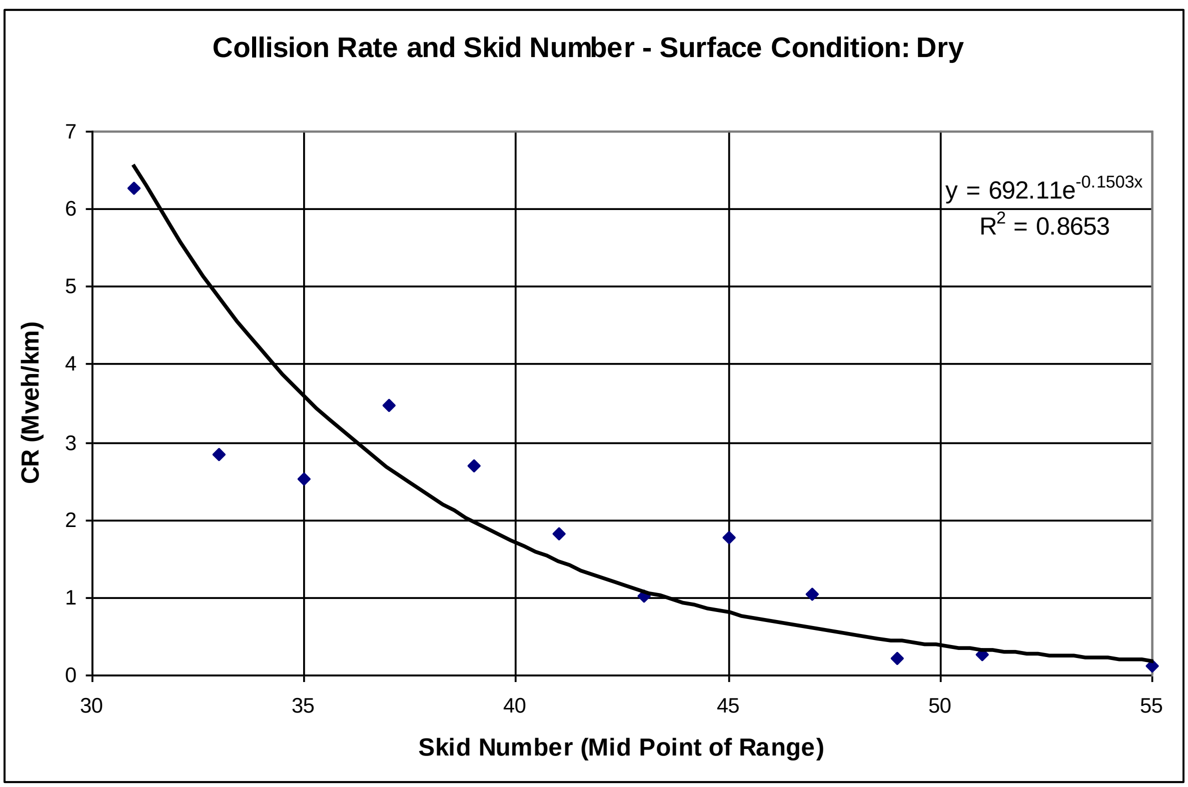

Figure 5.12: Collision Rate and Skid Number – Dry

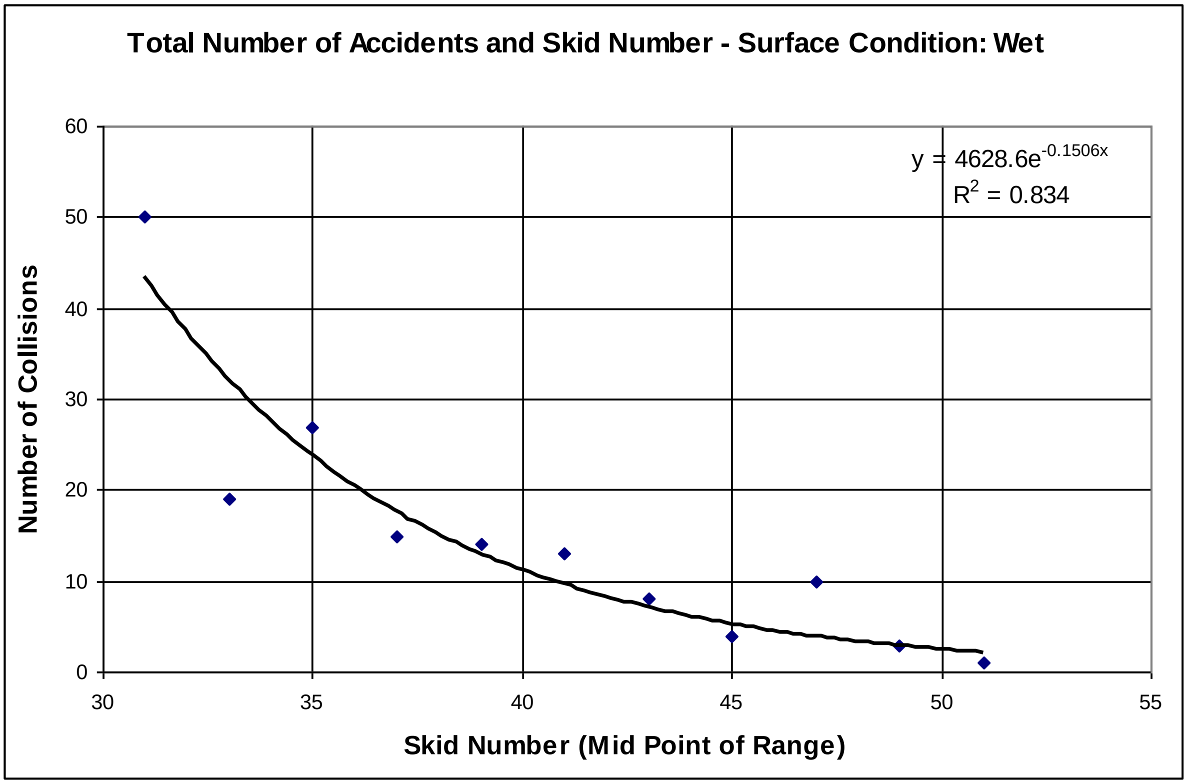

Figure 5.13: Number of Accidents and Skid Number – Wet

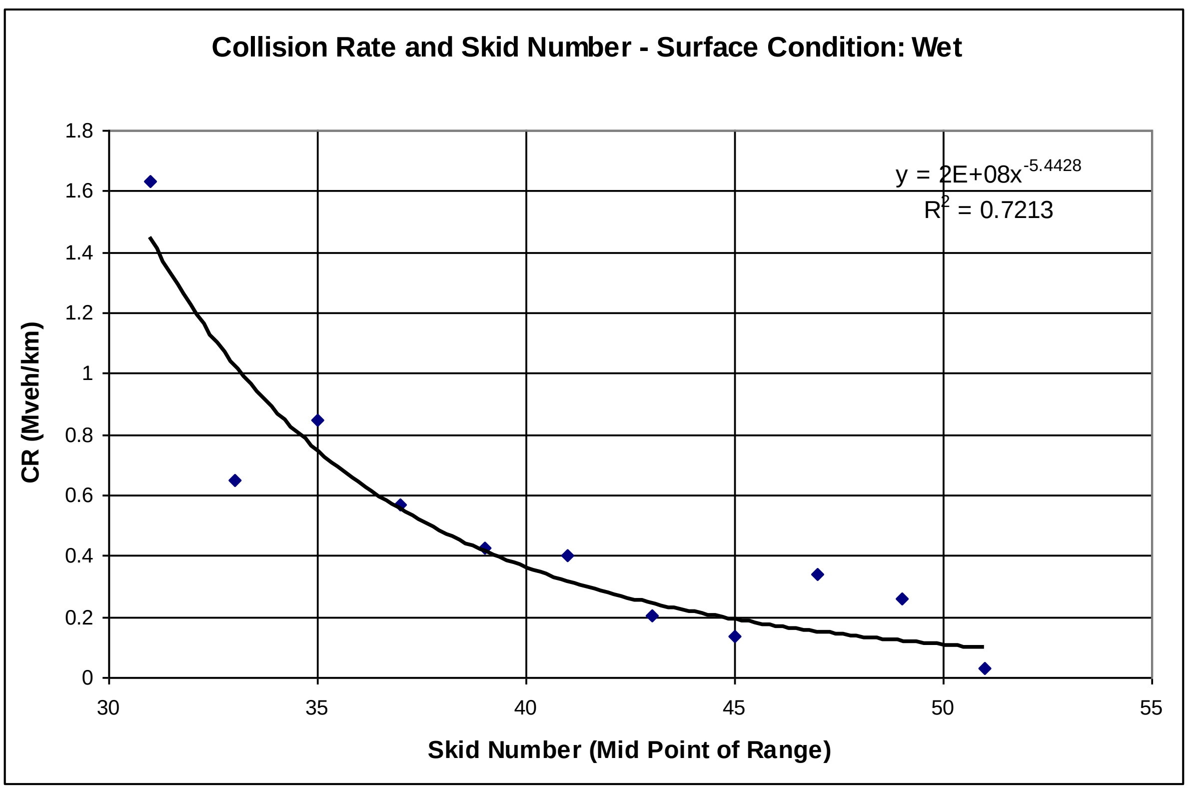

Figure 5.14: Collision Rate and Skid Number – Wet

5.4.2.6 Visibility

Visibility is directly related to weather and climatic conditions. In total, 4 models were developed within the Visibility Model Class. A model was developed to represent the following visibility conditions – clear, fog, rain and snow. Clear visibility had the highest number of collisions with 811, followed by snow with 323, rain with 87, and fog with 20. For clear, fog and snow visibility conditions, an exponential based model was used to best fit the data with R^2 values of 0.88, 0.04 and 0.93 respectively. For the rain visibility conditions, a power based model was used to best fit the data with an R^2 value of 0.86.

Similar to the model for total number of collisions, models were also developed based on the collision rate (CR) as the dependent variable and the level of pavement surface friction (SN) as the independent variable. For clear, fog and snow visibility conditions, an exponential based model was used to best fit the data with R^2 values of 0.87, 0.01 and 0.92 respectively. For the rain visibility conditions, a power based model was used to best fit the data with an R^2 value of 0.67. The models for the total number of accidents and the collision rate are presented in Figures 5.15 and 5.16 for clear visibility and 5.17 and 5.18 for foggy conditions, respectively. The models for the other visibility classes are presented in Appendix A. As was previously discussed, the low correlations for the foggy visibility could be attributed to the fact that friction was probably not a significant factor since visibility was the probable cause of those collisions. In addition, there were not many foggy collisions observed within the visibility class (20 collisions).

A summary of the Visibility Model Classes is presented in Table 5.3. The results of the ANOVA indicate that the visibility Model Classes are not significantly different at the 95% confidence interval as presented in Table 5.4.

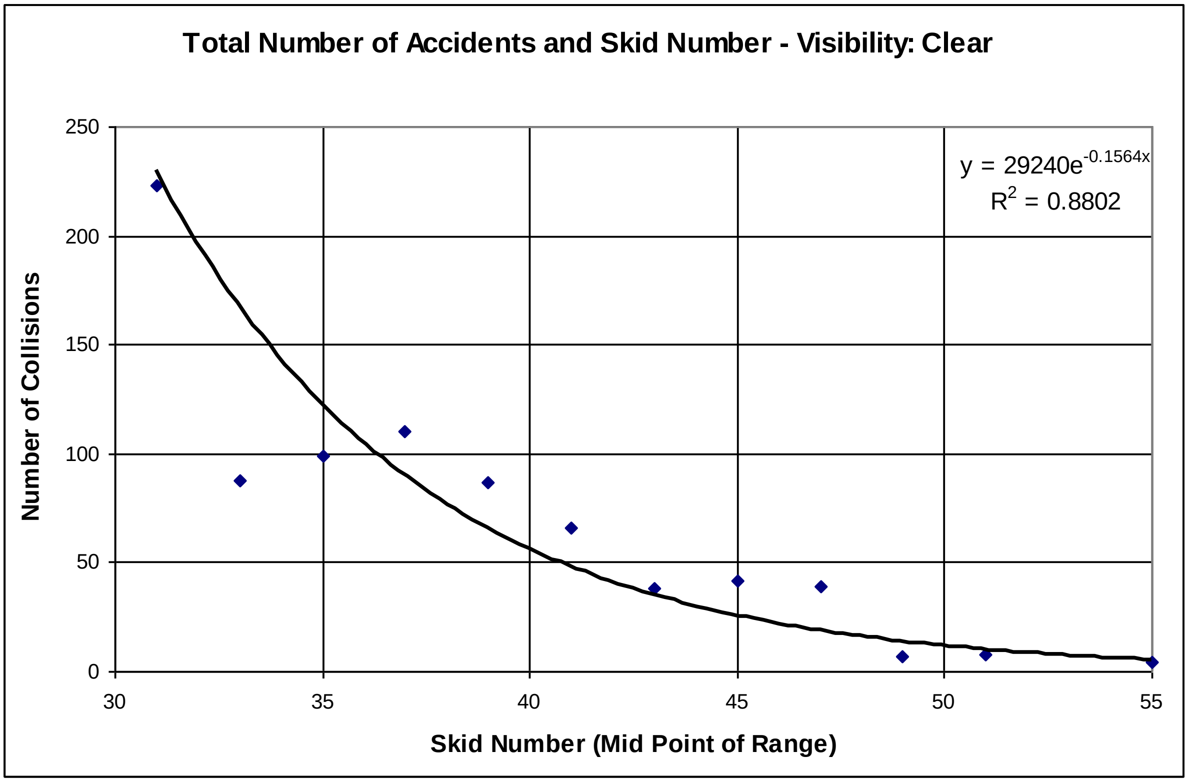

Figure 5.15: Number of Accidents and Skid Number – Clear

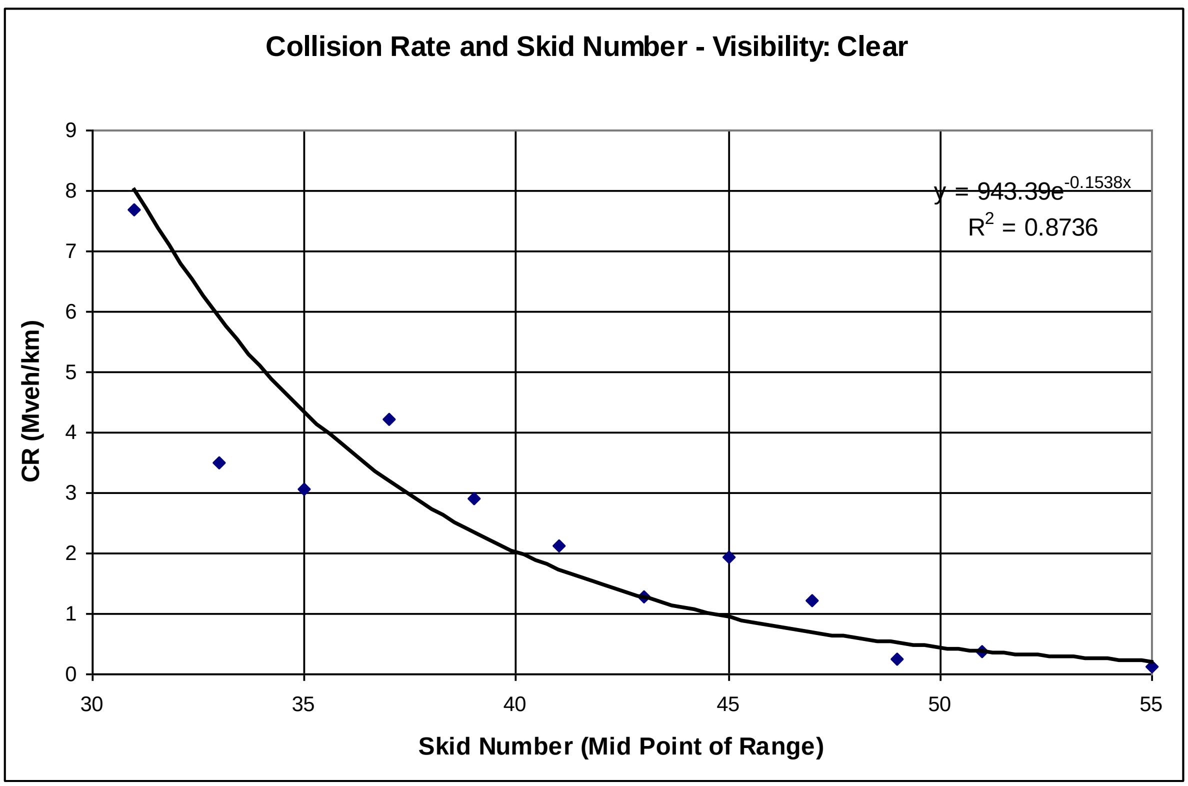

Figure 5.16: Collision Rate and Skid Number – Clear

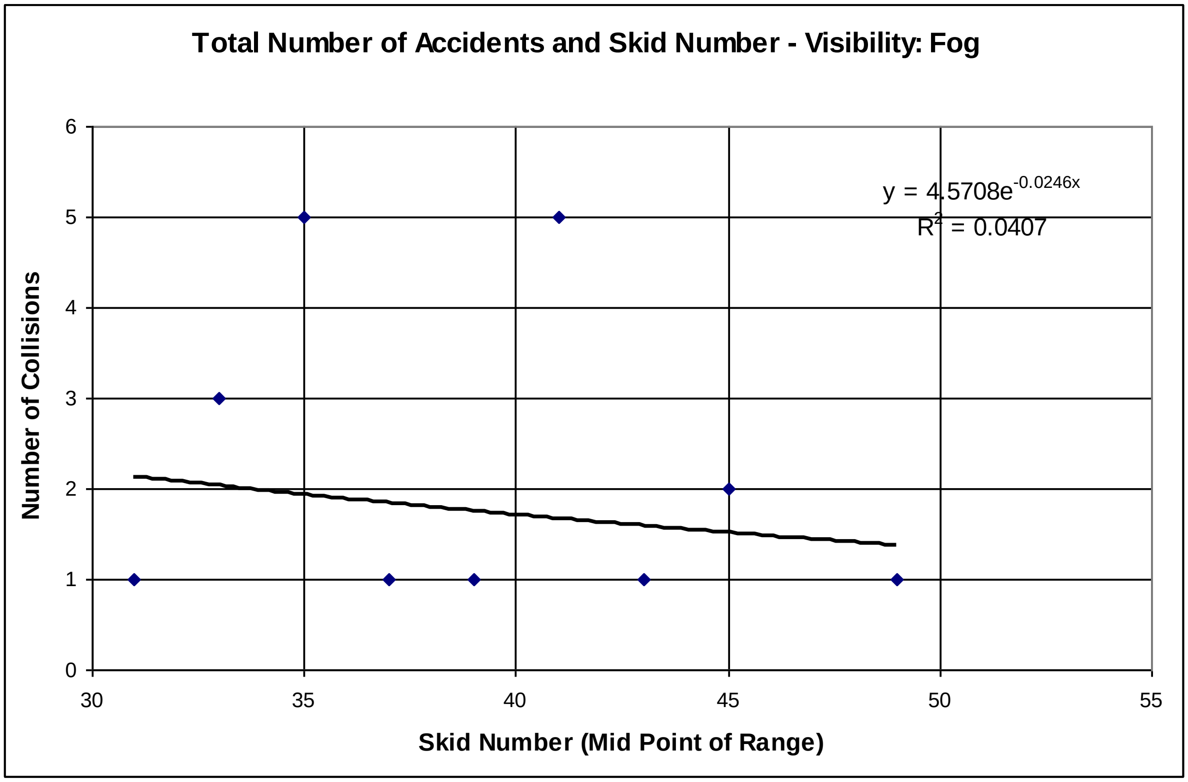

Figure 5.17: Number of Accidents and Skid Number – Fog

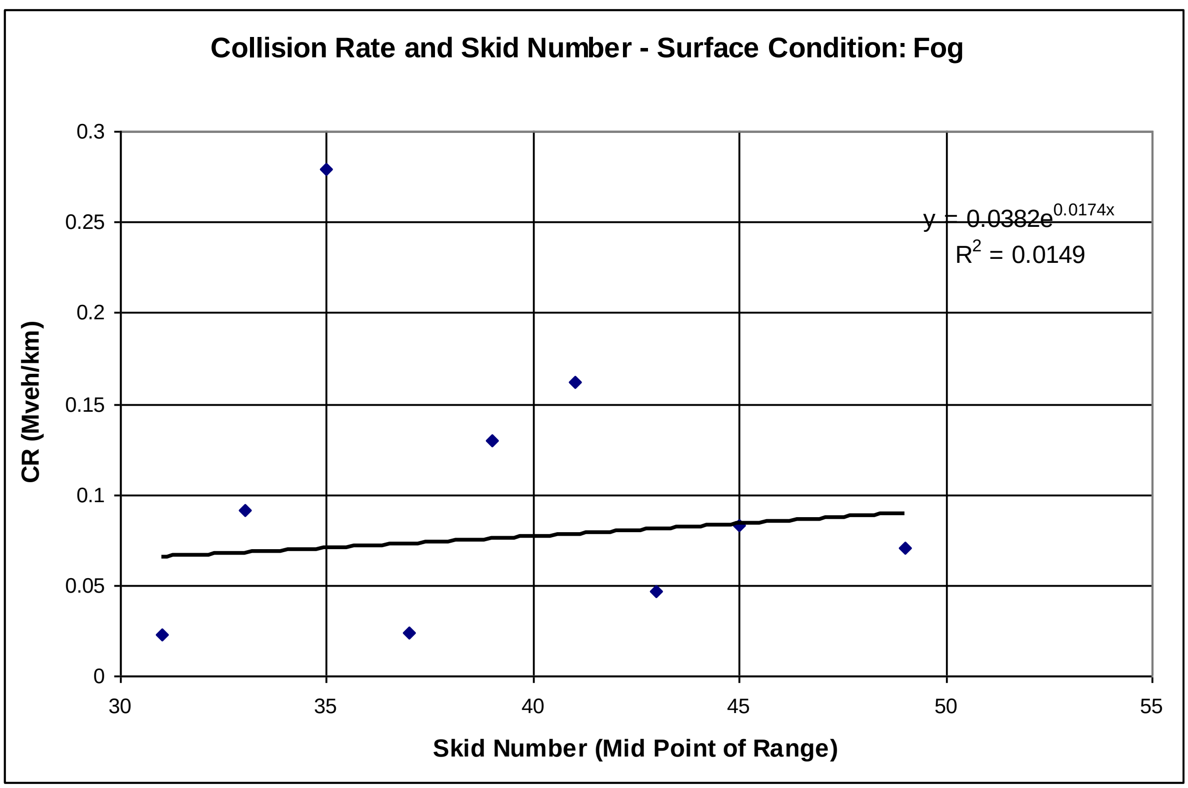

Figure 5.18: Collision Rate and Skid Number – Fog

5.4.2.7 Roadway Location

Four models were developed within the Roadway Location Model Class. A model was developed to represent the following conditions – intersection, mainline, off-road and shoulder. The mainline location had the highest number of collisions with 719, followed by the off-road location with 260, intersection with 132, and shoulder with 109. For intersection, mainline, and off-road collision locations, an exponential based model was used to best fit the data with R^2 values of 0.80, 0.89 and 0.82, respectively. For the shoulder, a power based model was used to best fit the data with an R^2 value of 0.72.

Similar to the model for total number of collisions, models were also developed based on the collision rate (CR) as the dependent variable and the level of pavement surface friction (SN) as the independent variable. For intersection and mainline collision locations, an exponential based model was used to best fit the data with R^2 values of 0.88 and 0.90, respectively. For the off road and shoulder collision locations, a power based model was used to best fit the data with R^2 values of 0.81 and 0.55, respectively. The models for the total number of accidents and the collision rate are presented in Figures 5.19 and 5.20 for mainline collision locations, respectively. The models for the other roadway locations are presented in Appendix A.

A summary of the Roadway Location Model Classes is presented in Table 5.3. The results of the ANOVA indicate that the roadway location Model Classes are significantly different at the 95% confidence interval as presented in Table 5.4.

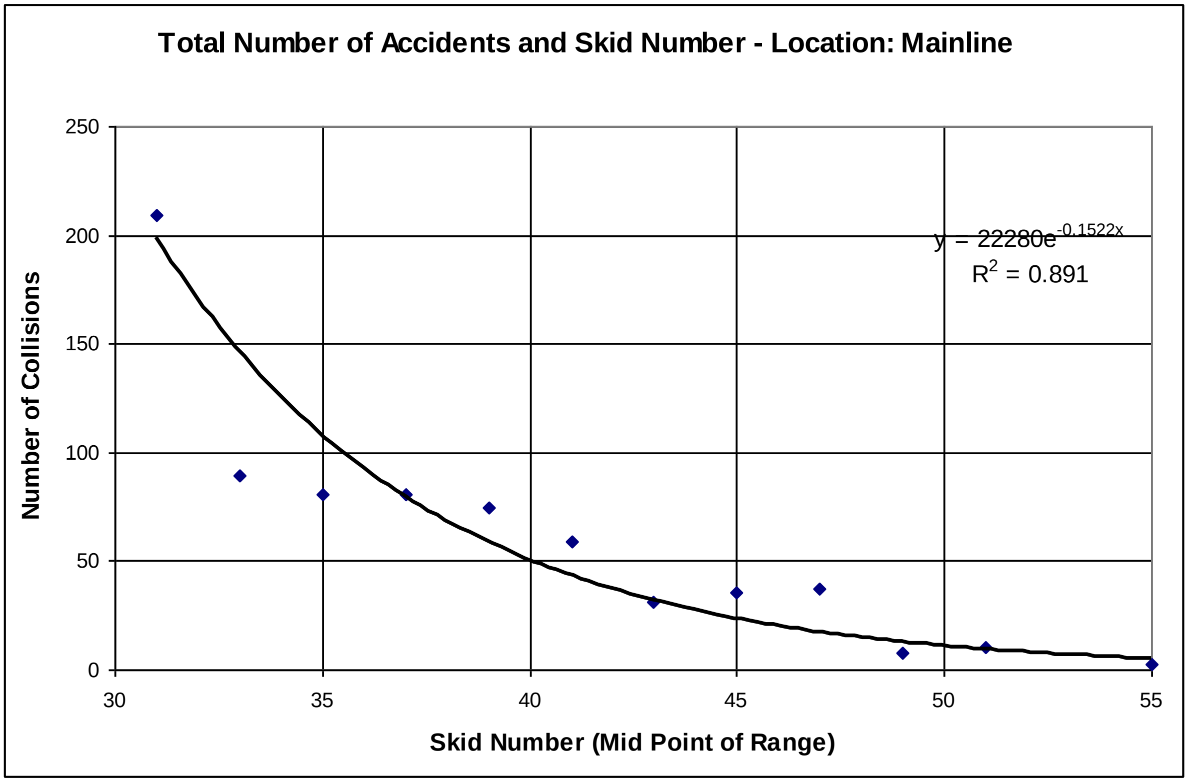

Figure 5.19 Number of Accidents and Skid Number – Mainline

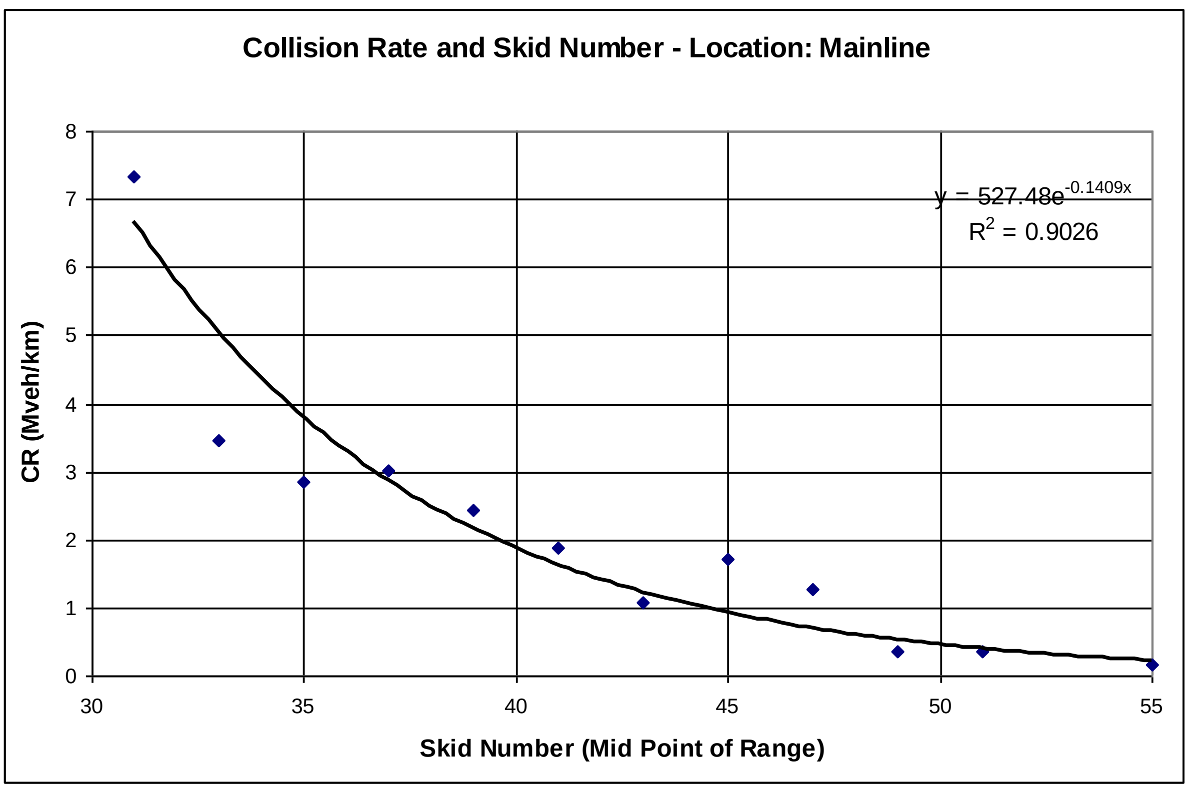

Figure 5.20: Collision Rate and Skid Number – Mainline

Table 5.3: Summary of Model Classes and Best Fit Curve

MODEL CLASS	CLASS ID	MODEL	Total Number of Collisions and SN			Collision Rate (CR) and SN		
			m	b/exp	R^2	m	b/exp	R^2
Region	R1	Both Regions	26,694	-0.1424	0.911	179.14	-0.1164	0.8575
	R2	Region A	2,0039	-0.1512	0.8475	476.33	-0.1415	0.8821
	R3	Region B	9,301	-0.1316	0.902	4,870	-0.120	0.934
Collision Severity	CT1	Fatal	302.27	-1.478	0.353 (power)	1.4589	-0.0916	0.6701
	CT2	Injury	2020.6	-0.1164	0.7697	9E+06	-4.471	0.7276 (power)
	CT3	Property	22140	-0.1439	0.9177	572.52	-0.1345	0.9414
Season	S1	Winter	13480	-0.1443	0.9146	255.09	-0.1272	0.9396
	S2	Spring	2025.8	-0.1263	0.9329	47.461	-0.1193	0.9225
	S3	Summer	12094	-0.1567	0.8574	434.01	-0.1566	0.8521
	S4	Fall	2490.3	-0.1356	0.7028	43.989	-0.1152	0.6508
Surface Condition	ST1	Dry	20899	-0.1523	0.8638	692.11	-0.1503	0.8653
	ST2	Ice	3E+08	-4.8225	0.888 (power)	1E+06	-4.1692	0.8982 (power)
	ST3	Snow	2909.8	-0.121	0.9002	64.602	-0.1063	0.8724
	ST4	Wet	4628.6	-0.15606	0.834	2E+08	-5.4428	0.7213 (power)
Environment Condition	EC1	Clear	28353	-0.1559	0.8832	923.8	-0.1534	0.875
	EC2	Drifting Snow	3E+07	-4.2786	0.9288 (power)	6.629	-0.0908	0.7479
	EC3	Fog	321210	-3.3154	0.7923 (power)	0.0395	0.0165	0.0147
	EC4	Freezing Rain	45193	-2.7627	0.7741 (power)	124.59	-1.9512	0.2356 (power)
	EC5	Rain	368.88	-0.1074	0.7496	441955	-4.0067	0.5829 (power)
	EC6	Snow	5527.6	-0.1409	0.8755	90.664	-0.1196	0.8831
	EC7	Wind	6.22	-0.0348	0.185	0.0998	-0.0108	0.0191
Visibility	V1	Clear	29249	-0.1564	0.8802	943.39	-0.1538	0.8736
	V2	Fog	4.5708	-0.0246	0.0407	0.0382	-0.0174	0.0149
	V3	Rain	723.34	-0.1189	0.8577	977883	-4.1492	0.6738 (power)
	V4	Snow	3451.1	-0.124	0.9299	77.405	-0.1103	0.9239
Roadway Location	RL1	Intersection	1800.8	-0.1297	0.7967	157.33	-0.1557	0.876
	RL2	Mainline	22280	-0.1522	0.891	527.48	-0.1409	0.9026
	RL3	Off Road	1421.9	-0.1067	0.8149	3E+06	-4.0884	0.8071 (power)
	RL4	Shoulder	8E+07	-4.3931	0.715 (power)	1E+06	-4.1155	0.5526 (power)

Table 5.4: Results of ANOVA and Student's T-Test at 95% Confidence Level

Model Class	Number of Variables	P value	F	F-critical	T	T-Critical	Significant
Region	3	0.000117	9.089	2.999	-	-	YES
Collision Type	2	0.0414	-	-	-1.742	1.651	YES
Season	4	0.545	0.712	2.612	-	-	NO
Surface Condition	2	0.050	-	-	-1.650	1.650	YES
Environmental Condition	7	0.261	1.284	2.106	-	-	NO
Visibility	4	0.372	1.043	2.612	-	-	NO
Roadway Location	4	0.00879	3.412	2.380	-	-	YES

5.5 Analysis of Wet-to-Dry Accident Ratio

It has been well established in a review of the literature (Hall et al., 2009) and (Wallman and Astrom, 2001) that the Wet-to-Dry or Wet-to-Total accident ratio was related to highways with low pavement surface friction. A number of U.S. transportation agencies rely on the use of the ratio of wet-to-total collisions ratio for network level friction analyses.

The wet-to-dry accident ratio was calculated for each highway and LHRS section within Regions A and B. Similar to the analysis methodology presented in Section 5.4.1, the friction values were aggregated based on the bin classes presented in Table 5.1. Based on the friction ranges presented in Table 5.1, the average Wet-to-Dry ratio was calculated for each bin class. The results of this analysis are presented below in Figure 5.21. A linear regression was performed to develop a best fit model which resulted in an R^2 value of 0.22.

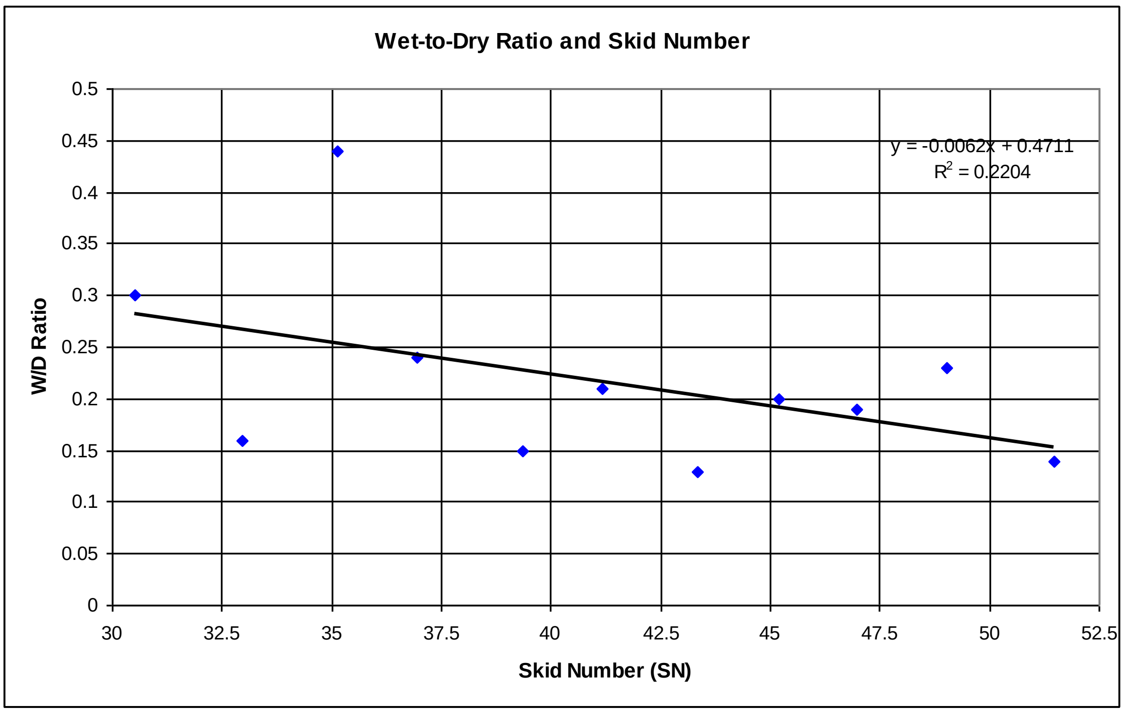

Figure 5.21: Wet-to-Dry Ratio and Skid Number (SN)

The model illustrates that there is a reduction in the Wet-to-Dry Ratio for an increase in the level of available pavement surface friction. It is worth noting that the relationship is not very strong (compared to the *CR*) and shows the limitation in relying only on the Wet-to-Dry Ratio to evaluate the level of safety of a highway or network. It also demonstrates the importance and value of collecting network level friction data.

Similarly, the collision rate (CR) was calculated using the model developed in Section 5.4.2.1 in (Region Class – Both Regions) and compared to the Wet-to-Dry Ratio. As a part of this study, 91 highway segments from 10 highways within Regions A and B were evaluated and compared. Based on the results presented in the Ontario Road Safety Annual Reports (2001 to 2003), the provincial average of Wet-to-Dry accidents is approximately 0.3. The Wet-to-Dry accident ratio was categorized into the following three categories:

- Wet-to-Dry Ratio < 0.3 (Low, shaded green)
- 0.3 >= Wet-to-Dry Ratio < 0.45 (Medium, shaded orange)
- Wet-to-Dry Ratio >=0.45 (High, shaded red)

A review of the literature indicates that highway sections with a high Wet-to-Dry Ratio may be indicative of locations with low pavement surface friction. Highway segments with a high collision rate are also considered not safe and potentially hazardous. The highway sections were ranked based on the Wet-to-Dry accident ratio and the collision rate based on a worst-to-first approach. For this methodology, the Wet-to-Dry Ratio was sorted from the highest value (1) to the lowest value (91). A similar procedure was used to rank the Collision Rate with the highest CR value (1) to the lowest CR value (*91*). The difference between the two ratings (or ranks) was calculated and is presented in Table 5.5 as Delta Rank. As an example, for Highway 2, LHRS No. 6 (Region A), the Wet-to-Dry accident ratio was ranked as 8 out of 91, while the collision rate was ranked as 2 out 91. The delta rank value would be (Delta Rank: 8-2 = 6).

Table 5.5: Rank of Highways in terms of W/D Ratio and Collision Rate for Region A and B

Region	Highway Number	LHRS Number	SN	Number of Dry Collisions	Number of Wet Collisions	W/D Ratio	CR (f^n of SN)	Rank Based on W/D	Rank Based on CR	Delta Rank (Absolute Value)
A	2	6	8.2	11	6	0.55	16.02	8	2	6
A	2	9	29.3	6	2	0.33	13.90	22	4	18
A	2	8	29.6	16	4	0.25	13.33	34	5	29
A	2	3	30.4	6		0.00	11.98	71	8	63
A	2	11	31.2	1		0.00	10.70	74	11	63
A	2	4	31.4	4		0.00	10.45	72	12	60
A	2	2	31.8	8	1	0.13	9.82	58	15	43
A	2	1	32.0	14	6	0.43	9.65	17	18	1
A	2	10	35.4	13	3	0.23	6.08	43	39	4
A	2	7	37.6	8	3	0.38	4.53	20	54	34
A	2	5	38.0	4		0.00	4.32	73	57	16
A	2	14	43.8	8		0.00	1.98	75	76	1
A	2	13	43.9	1	2	2.00	1.96	2	77	75
A	2	12	47.0	11	5	0.45	1.29	16	89	73
A	5	15	33.7	9	1	0.11	7.69	61	27	34
A	5	16	41.3	5		0.00	2.76	76	68	8
A	5	17	46.3	4	2	0.50	1.42	10	87	77
A	5	18	46.6	3	1	0.33	1.37	23	88	65
A	6	26	28.1	2		0.00	16.30	77	1	76
A	6	25	29.0	7	3	0.43	14.39	18	3	15
A	6	28	31.9	13	3	0.23	9.76	45	16	29
A	6	29	32.6	7	1	0.14	8.87	54	21	33
A	6	23	32.7	13	3	0.23	8.79	44	23	21
A	6	27	33.8	4	1	0.25	7.53	37	28	9
A	6	24	34.2	4	1	0.25	7.19	36	31	5
A	6	22	35.4	16	2	0.13	6.12	59	37	22
A	6	21	36.7	20	5	0.25	5.13	35	47	12
A	6	19	36.8	26	8	0.31	5.09	29	48	19
A	6	20	39.2	13	4	0.31	3.65	30	59	29
A	7	32	34.1	3		0.00	7.26	78	29	49
A	7	33	35.4	4		0.00	6.14	79	36	43
A	7	30	35.6	7	1	0.14	5.93	55	40	15
A	7	31	35.8	3	1	0.33	5.77	24	41	17
A	7	35	36.3	5	1	0.20	5.42	47	46	1
A	7	36	36.9	11	1	0.09	5.00	70	52	18
A	7	37	37.7	14	2	0.14	4.48	56	55	1
A	7	34	41.0	6	3	0.50	2.90	11	65	54
A	9	40	30.5	8	5	0.63	11.83	7	9	2
A	9	38	31.2	13	7	0.54	10.75	9	10	1
A	9	39	32.6	17	4	0.24	8.87	41	22	19
A	9	41	41.8	14	3	0.21	2.59	46	69	23
A	9	42	44.9	3		0.00	1.70	80	82	2
A	10	45	37.3	7	2	0.29	4.71	32	53	21
A	10	43	44.7	2	1	0.50	1.77	12	80	68
A	10	44	45.3	17	4	0.24	1.62	42	85	43

Region	Highway Number	LHRS Number	SN	Number of Dry Collisions	Number of Wet Collisions	W/D Ratio	CR (f^n of SN)	Rank Based on W/D	Rank Based on CR	Delta Rank (Absolute Value)
B	1	54	35.4	1	3	3.00	6.10	1	38	37
B	1	53	40.0	4		0.00	3.32	83	61	22
B	1	52	40.6	2	1	0.50	3.06	14	62	48
B	1	46	43.0	3	1	0.33	2.21	25	72	47
B	1	47	43.5	5	1	0.20	2.05	48	74	26
B	1	48	43.7	6	1	0.17	2.02	51	75	24
B	1	51	44.1	1		0.00	1.90	82	79	3
B	1	50	44.9	2		0.00	1.71	81	81	0
B	1	49	45.1	2	1	0.50	1.67	13	84	71
B	2	56	30.0	5	4	0.80	12.65	5	6	1
B	2	55	31.9	17	6	0.35	9.74	21	17	4
B	2	57	32.2	10	3	0.30	9.31	31	20	11
B	3	59	31.8	7	1	0.14	9.90	57	14	43
B	3	58	33.7	16	4	0.25	7.70	38	26	12
B	3	61	36.1	10	1	0.10	5.55	67	44	23
B	3	62	36.8	1		0.00	5.06	84	49	35
B	3	60	41.1	3	1	0.33	2.83	26	66	40
B	4	68	32.0	5		0.00	9.58	88	19	69
B	4	64	32.9	3		0.00	8.52	85	24	61
B	4	69	33.5	9	1	0.11	7.90	62	25	37
B	4	65	35.1	4	1	0.25	6.35	39	33	6
B	4	67	41.3	7		0.00	2.76	87	67	20
B	4	63	42.0	8	1	0.13	2.53	60	70	10
B	4	66	43.2	6		0.00	2.15	86	73	13
B	5	70	30.2	11	3	0.27	12.17	33	7	26
B	5	71	34.1	9	1	0.11	7.25	63	30	33
B	5	72	44.1	4	3	0.75	1.90	6	78	72
B	7	74	35.2	9	1	0.11	6.24	65	34	31
B	7	75	35.9	12	2	0.17	5.69	52	42	10
B	7	73	38.2	9	1	0.11	4.22	64	58	6
B	7	77	39.9	5	1	0.20	3.35	49	60	11
B	7	76	40.6	20	2	0.10	3.03	68	63	5
B	7	78	40.8	9	1	0.11	2.96	66	64	2
B	8	86	31.5	4	1	0.25	10.35	40	13	27
B	8	81	35.0	5	2	0.40	6.48	19	32	13
B	8	79	35.3	2	2	1.00	6.17	3	35	32
B	8	80	36.0	1	1	1.00	5.62	4	43	39
B	8	85	43.0	10	1	0.10	2.22	69	71	2
B	8	84	45.0	3	1	0.33	1.68	28	83	55
B	8	83	45.9	3		0.00	1.49	89	86	3
B	8	82	51.7	3	1	0.33	0.69	27	91	64
B	10	91	36.3	6	1	0.17	5.44	53	45	8
B	10	87	36.8	6		0.00	5.05	90	50	40
B	10	90	36.9	10	5	0.50	5.01	15	51	36
B	10	88	37.9	5	1	0.20	4.38	50	56	6
B	10	89	51.5	2		0.00	0.71	91	90	1

The Delta Rank values were then categorized into three levels:

- Delta Rank < 10
- 10 >= Delta Rank < 20
- Delta Rank >=20

The purpose of this evaluation is to determine how close or different the two methods (W/D and CR) were at ranking or evaluating the level of safety of the various highway segments. The results of this analysis are presented below in Figure 5.22; 27% of the highway segments were classified within 10 Delta Rank values, 18% between 10 and 20 Delta Rank values and 55% were classified with Delta Rank greater than 20. The results indicate that the Wet-to-dry accident ratio can correctly classify the level of safety for the various highway segments based solely on accident information (no surface friction) approximately 27% of the time, while for the remaining 63%, there is a relatively high margin in error in classifying the various highway segments.

The results of this analysis demonstrate the importance of network level friction testing for evaluating the level of safety of a highway network. Relying solely on collision or traffic data can result in errors when assessing the level of safety of a highway or highway network.

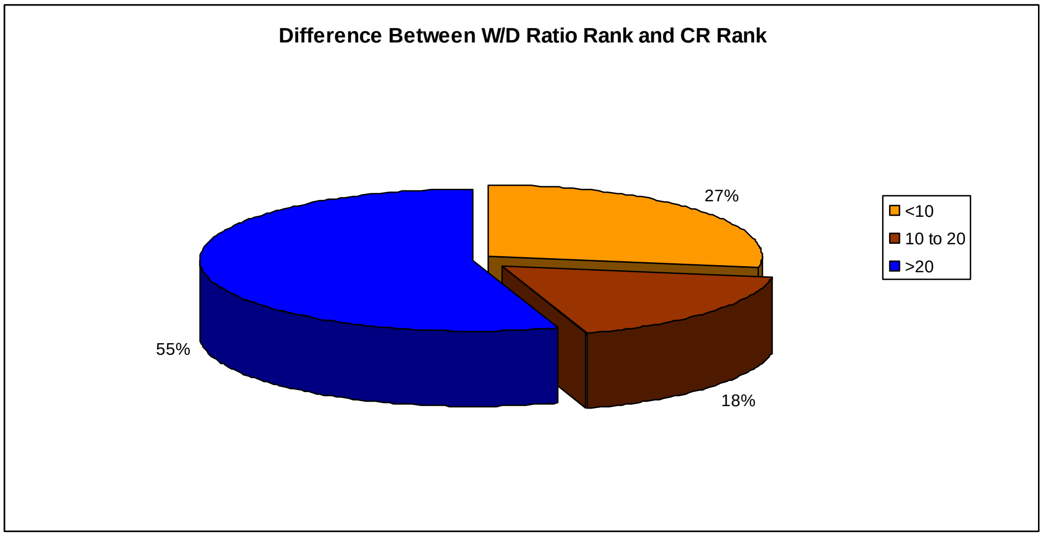

Figure 5.22: Differences Between Wet-to-Dry Accident Ratio and Collision Rate (Delta Rank)

5.6 Estimating Level of Risk

To estimate the level of risk, or probability of a collision occurring given a known level of friction along a pavement section, the normal distrubtion of the skid number (SN) for Regions A and B were examined. With the normal distribution known, the risk of a collision occurring given a known level of friction can be estimated by calculating the area under the normal curve.

A random variable *X* whose distribution has the shape of a normal curve is called a normal random variable. The random variable for this case is the skid number, or level of friction. This random variable *X* is said to be normally distributed with mean μ and standard deviation σ if its probability distribution given by;

$$f(x) = \frac{1}{\sigma\sqrt{2\pi}} e^{-(x-u)^2/2\sigma^2} \qquad [14]$$

The standard normal distribution is a special case of the normal distribution. It is the distribution that occurs when a normal random variable has a mean of zero and a standard deviation of one. The normal random variable of a standard normal distribution is called a standard score or a z-score. Every normal random variable *X* can be transformed into a z-score using the following equation:

$$Z = \frac{X - \mu}{\sigma} \qquad [15]$$

For this study, each normal random variable (or skid number) was transformed into its corresponding z-score using the above equation. The standard normal probability distribution function of the skid numbers (SN) for Regions A and B was then generated using the calculated z-scores and the probability distribution function. The standard normal probability distribution function for Regions A and B is presented below in Figure 5.23. The probability of a continuous normal variable *X* found in a particular interval [*a*, *b*] is the area under the curve bounded by *x* = *a* and *x* = *b* and is given by;

$$P(a < X < b) = \int_a^b f(x)dx \qquad [16]$$

This area is dependent upon the values of μ and σ. The areas under the curve are bounded by the ordinates $z = 0$ and any positive value of z are found in a ***z***-Table. From this table the area under the standard normal curve between any two ordinates can be found by using the symmetry of the curve about $z = 0$.

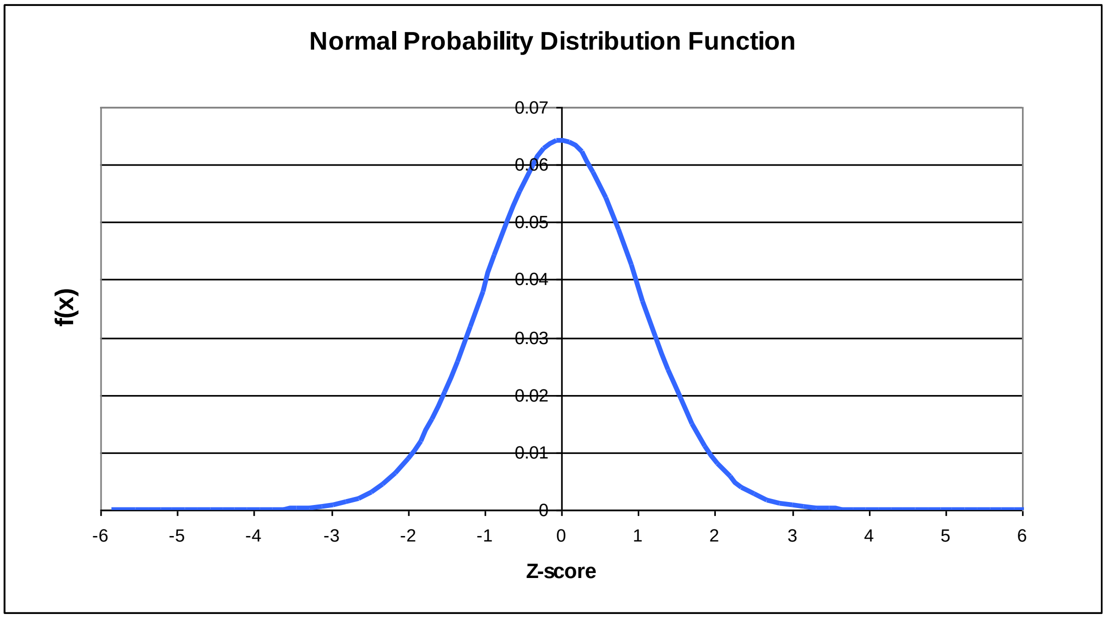

Figure 5.23: Distribution Function for Skid Numbers in Regions A and B

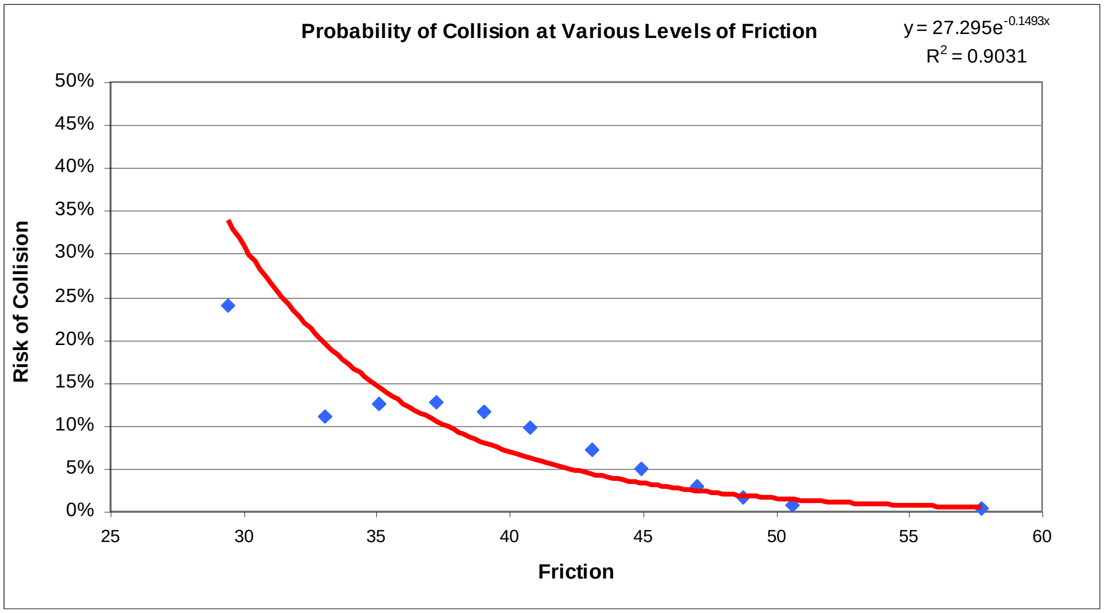

Figure 5.24: Network Level Model – Collision Risk and Level of Friction

The level of risk or probability of collision occurrence was determined for various levels of friction. A model was then generated to represent the risk of collision as a function of the level of friction as presented in Figure 5.24. This model can be used to estimate the risk of collision for a given pavement section when the Skid Number is known. It can also be used as a tool to estimate the benefits in terms of “a reduction in risk” as a result of increasing the level of friction on a pavement section due to a maintenance, rehabilitation, or reconstruction (M, R, and R) activity such as an asphalt overlay, slurry seal, or chip seal.

5.7 Summary

Several advanced methods such as Bayesian Statistical Techniques, Cluster Analysis, and Artificial Neural Networks are used by transportation and safety experts to assess the level of safety of a highway network. Some disadvantages to those analysis techniques are they are very complex and require someone with an advanced statistical background/education. Many agencies do not have the resources or in-house expertise to carry out such types of analyses. This research study presents a framework or approach that a transportation agency, contractor, or consultant can use to assess the level of safety of a highway network. As a part of this framework, data collection requirements, data integration and linkage methods, statistical analysis and estimating the probability of collision based on the level of friction were presented and outlined.

Chapter 6

Effectiveness of Preservation and Maintenance Treatments within the LTPP Study

As a part of the study presented in this Chapter, friction data from all SPS-5 sites in the Long Term Pavement Performance (LTPP) project were used to examine how skid resistance deteriorates with time for various environment zones. The SPS-5 experiment examines the effects of a number of rehabilitation activities on flexible pavement sections. Another component of the study was to evaluate the performance of a selected group of commonly implemented preservation and maintenance strategies.

6.1 Background

Conventional or routine pavement maintenance practices can be characterized as reactive in nature (unplanned), performed on deteriorated pavements, that do not contribute to long-term performance, are not generally cost effective and are often performed under harsh or severe conditions (i.e. pot hole patching in spring). Maintenance is an essential component of an effective pavement management framework or strategy. Preservation can be defined as a strategy intended to arrest light deterioration, retard progressive failures, and reduce the need for routine maintenance and service activities (Louis O'Brien, NCHRP 153). Dollars invested in implementing preservation and maintenance strategies are significantly less than allowing a pavement structure to deteriorate until major pavement rehabilitation or reconstruction is required.

The benefits of practicing preservation are higher customer satisfaction, better informed decisions, improved strategies and techniques, improved pavement condition, reduced life cycle costs and increased safety. Studies have shown that highway improvements such as increasing the radius of a horizontal curve or increasing the skid resistance of a pavement can result in a reduction in the number of collisions and improved levels of service. Evaluating the effectiveness or performance of a Maintenance, Rehabilitation or Reconstruction (M, R, & R) activity is beneficial to agencies and contractors so they can determine what treatments or strategies offer the best “bang for the buck”.

6.2 The Long Term Pavement Performance (LTPP) Project

The Long Term Pavement Performance (LTPP) Project is the largest pavement research study performed in North America. Pavement performance data has been collected from over 2,400 pavement sections located across Canada and the United States. These pavement sections consist of a variety of pavement structures in four environment zones, built on a number of different subgrades and exposed to various levels of traffic.

The LTPP program was initiated in 1987 as a part of the Strategic Highway Research Program (SHRP). The main objective for the LTPP program was to establish a national long-term pavement database to support SHRP objectives and future needs. Currently, the project is managed by the Federal Highway Administration (FHWA) and consists of over 2,400 sections at 932 locations on in-service highways located across North America. The LTPP test sections are classified into a number of studies: General Pavement Studies (GPS) and Specific Pavement Studies (SPS) sections. A GPS test site typically would have one test section, while an SPS test site would have multiple test sections incorporating a controlled set of experiment design and construction features. LTPP data is collected in a consistent manner at a specific level of accuracy and checked through a series of Quality Assurance (QA) checks. Also, maintenance activities are monitored and recorded, thus addressing some of the possible sources of inconsistencies in historic performance data.

6.3 Study Approach

To quantify the effectiveness of a preservation or maintenance strategy, historical pavement performance data is required. Pavement performance data such as deflection measurements collected from a Falling Weight Deflectometer (FWD), roughness in terms of the International Roughness

Index (IRI) and skid resistance can be used to evaluate the effectiveness of an M, R, & R treatment. Most of this data has been collected over the past 20 years as a part of the LTPP Project and is stored in the LTPP DataPave database.

When determining the performance or improvement provided by an M, R & R treatment, two important factors must be known. First, the condition of the pavement (or level of pavement surface friction) just prior to implementing the M, R, & R treatment must be known. In an ideal situation, this data is collected or surveyed just prior to construction. Secondly, the condition of the pavement (level of pavement surface friction) just after the implementation of the M, R & R treatment must be identified. Ideally, this data should be collected after construction, sometime after the pavement has been re-opened to traffic. With the before-and-after conditions of the pavement known, the improvement, or increase in structural, functional or safety performance can be quantified. The LTPP database is an excellent source of before-and-after pavement performance data.

6.3.1 Data Manipulation

As stated earlier, the LTPP database includes an extensive amount of data, designed to address the requirements of a large variety of pavement research objectives. Subsequently, only the data required for this study was extracted from the LTPP database for analysis purposes. Furthermore, some of the data had to be filtered and/or reformatted for analysis purposes. The data used in the analysis and their sources in the LTPP database are shown in Table 6.1. For the collection of friction data from the LTPP sites, a skid number is recorded at the start and end of the 500 foot section. For analysis purposes, the skid number for the start and end were averaged.

The construction activities in the LTPP are defined at a very detailed level to allow for further research into specific treatments. This includes details in the maintenance activities, such as overlays or surface treatments that were implemented on the sections after the original rehabilitation activity. These maintenance activities will have an impact on the pavement performance and level of pavement surface friction. Therefore, the performance data considered in the analysis were those collected before and after the M, R & R activities were initiated. When an LTPP site is first entered into the study, it is identified as Construction Number 1. When the test site undergoes an M, R &R treatment, it changes from Construction Number from 1 to 2. The reason for the change is also documented by a code which represents the various M, R, & R treatments.

Table 6.1: Data Types and Sources from LTPP Database

Data Type	LTPP DataPave Module	LTPP Table Name
Construction Date and M&R activities type	Administration	EXPERIMENT_SECTION
Pavement type, lane width, and other general information	Inventory	INV_GENERAL
Section location, route number, mileposts	Inventory	INV_ID
Historical precipitation data	Climate	CLM_VWS_PRECIP_ANNUAL
Historical temperature data	Climate	CLM_VWS_TEMP_ANNUAL
Friction (SN) measurements	Pavement monitoring	MON_FRICTION

One of the major parameters that influence pavement performance are the environmental and climatic factors. The LTPP was primarily designed considering four environmental zones, as a combination of wet versus dry, and freeze versus no freeze. These classifications are based on the amount of annual precipitation and freezing index. Climatic data in terms of annual precipitation and historical temperature data were extracted from the LTPP database to evaluate how skid resistance deteriorates with time in each environment zone. Depending on the agency requirements, the limits for defining the environment zone can be changed. However, for the scope of this case study, the environmental zones were defined based on the limits set by the FHWA, which are a freezing index of 83 degree C-days as a boundary between No-Freeze and Freeze Zones, and a precipitation of 50 mm/year as a boundary between Wet and Dry Zones.

6.3.2 Rate of Deterioration of Skid Resistance over Time

As previously mentioned, data from all SPS-5 sites within the LTPP experiment were used for this analysis. The SPS-5 experiment examines the effects of various rehabilitation activities on flexible pavement sections. Each SPS-5 test site will have eight flexible pavement sections with different rehabilitation activities, in addition to a control section. Friction data from all SPS-5 sites in the LTPP experiment was used in the analysis. The rehabilitation activities implemented in each SPS-5 are:

- Thin Asphalt Concrete (AC) overlay
- Medium AC overlay
- Cold Mill + Thin AC Overlay
- Cold Mill + Medium AC Overlay
- Thin Recycled AC Overlay
- Medium Recycled AC Overlay
- Cold Mill + Thin Recycled AC Overlay
- Cold Mill + Medium Recycled AC Overlay

In total, data from 14 SPS-5 sites with a total of 165 test sections was evaluated. Table 6.2 shows the location, year of construction, and environment zone, as described in the LTPP database, for these test sites. The friction data for the SPS-5 test sites along with the corresponding environment zone were extracted from the LTPP database. Data for a single Construction Number was examined to eliminate the effects of an increase in SN due to an M, R & R treatment.

A model that best fits the data for each environment zone was developed (Figures 6.1 to 6.4). For three out of the four environment zones (Dry No Freeze, Wet Freeze, and Wet No Freeze) the level of friction is observed to increase with time. The Dry Freeze environment zone shows a slight decrease in skid resistance over time. It is important to note the magnitude of the slope for each model, which generally indicates no relationship. This result is similar to a study conducted by Indiana Department of Transportation (Li et al., 2004). This trend was also observed when examining the friction data across all environment zones (Figure 6.5).

Table 6.2: LTPP SPS 5 Test Sites

SPS-5 Site	State/Province	Year of Construction	Environmental Zone
010500	Alabama	1991	Wet - No Freeze
040500	Arizona	1990	Dry - No Freeze
060500	California	1992	Dry - No Freeze
080500	Colorado	1991	Dry - Freeze
120500	Florida	1995	Wet - No Freeze
130500	Georgia	1993	Wet - No Freeze
230500	Maine	1995	Wet - Freeze
240500	Maryland	1992	Wet - Freeze
300500	Montana	1991	Dry - Freeze
340500	New Jersey	1992	Wet - Freeze
350500	New Mexico	1996	Dry - No Freeze
400500	Oklahoma	1997	Wet - No Freeze
810500	Alberta	1990	Wet - Freeze
830500	Manitoba	1989	Wet - Freeze

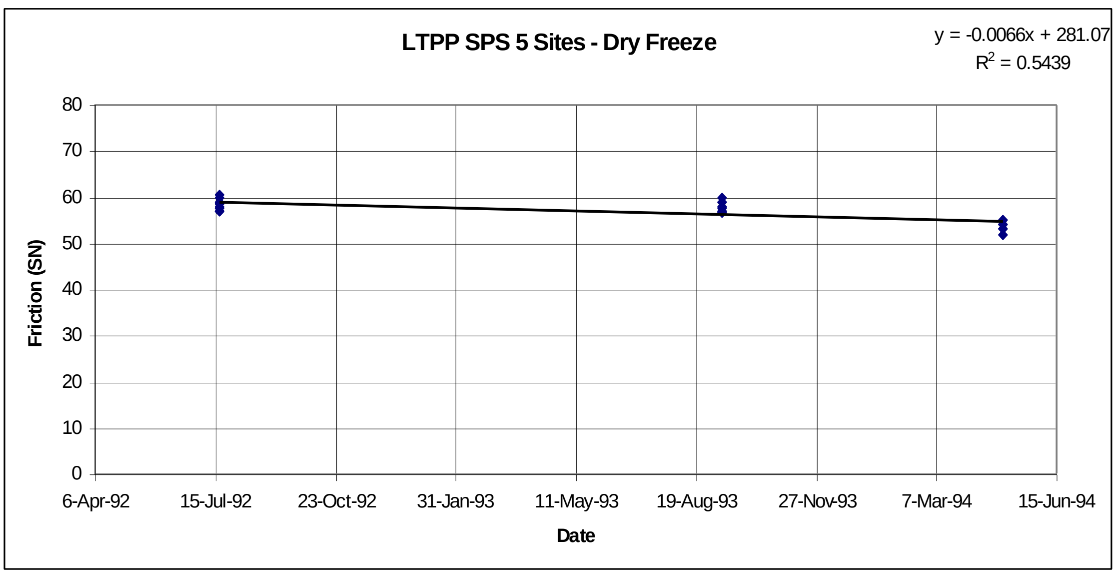

Figure 6.1: Skid Resistance over time for Dry Freeze Environmental Zone

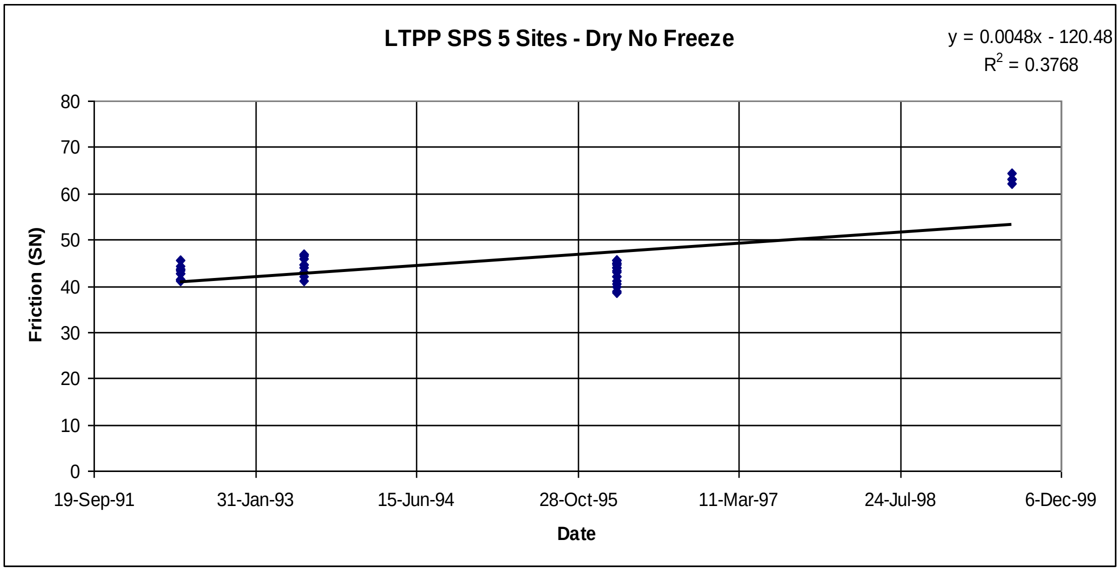

Figure 6.2: Skid Resistance over time for Dry No Freeze Environment Zone

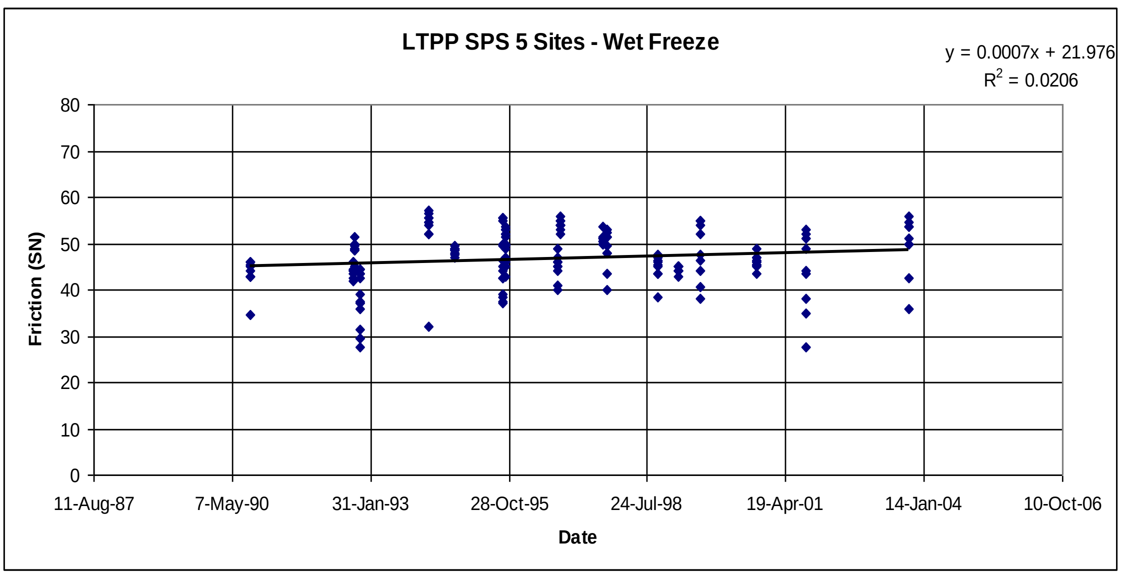

Figure 6.3: Skid Resistance over time for Wet Freeze Environment Zone

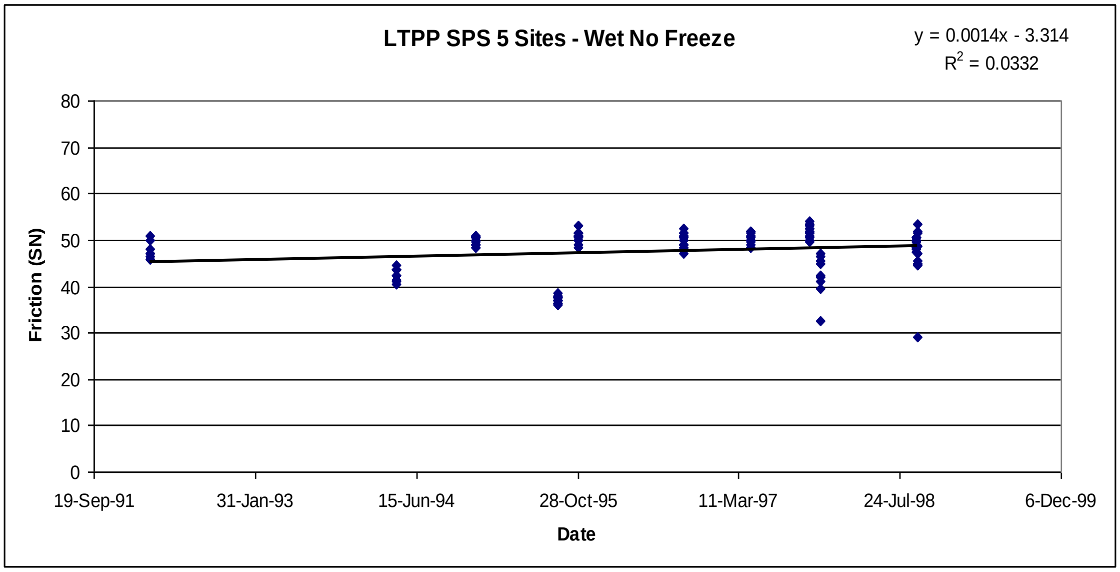

Figure 6.4: Skid Resistance over time for Wet No Freeze Environment Zone

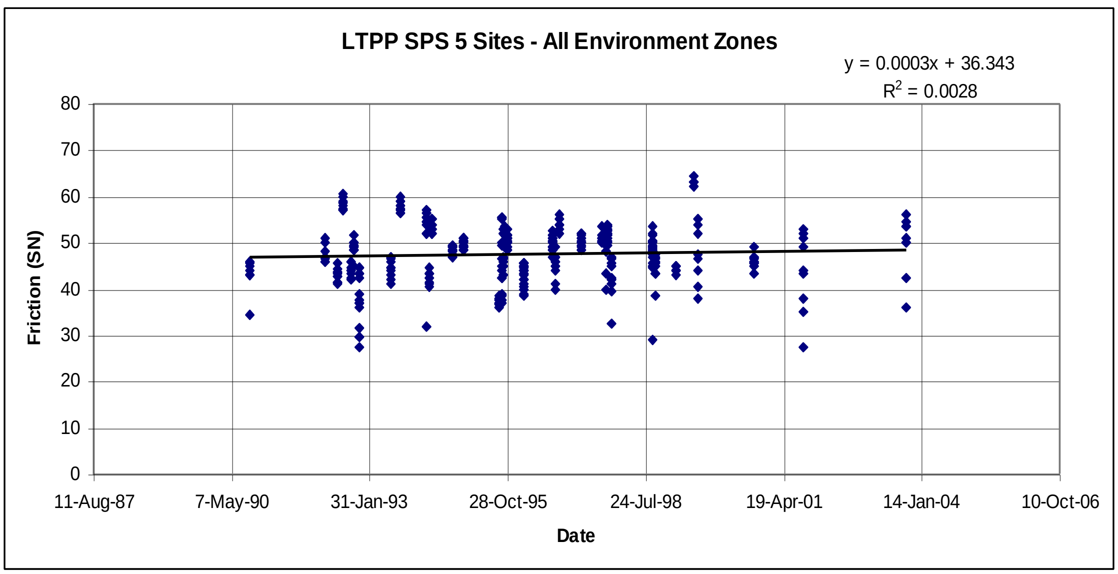

Figure 6.5: Skid Resistance over time for All Environment Zones

It is generally expected that a pavement will deteriorate with time as a result of traffic loadings and environmental factors. Structural performance quantified in terms of a Structural Adequacy Index (SAI) obtained from deflection measurements generally decreases with time. Pavement distresses such as alligator cracking, longitudinal and transverse cracking and rutting also initiate and propagate with time. Functional performance such as ride quality described in terms of roughness (IRI) obtained from a high speed profiler will generally increase (worsen) over time.

It would be expected that skid resistance would also decrease with time due to traffic and climatic factors similar to structural performance or pavement distress. However, this was not observed with friction measurements. This may be explained due to the fact that as a pavement ages and the surface starts to exhibit signs of distress such as raveling, the surface texture of the pavement may actually become rougher. This is an important factor to consider when examining historical friction trends, conducting life cycle cost analysis, and developing pavement performance models.

6.3.3 Evaluating Safety Performance of Preservation Treatments

To quantify the skid resistance performance of various preservation and maintenance treatments, friction data was extracted from the DataPave database. LTPP data from the MON_FRICTION table was used for the analysis. Data from all flexible pavement sections in the LTPP experiment which had recorded friction data was used in the analysis. Data from the EXPERIMENT_SECTION table provided information related to the M, R & R treatment including the year it was implemented for each of the LTPP sections. In total data from 347 LTPP sites were examined as a part of this analysis.

The various LTPP sections were then categorized by treatment. Since combinations of maintenance activities are typically performed at a single time (i.e., crack sealing, shoulder repair and asphalt concrete overlay), the treatments were aggregated into major groups. Using information obtained from the EXPERIMENT_SECTION Table, the following major treatment groups were developed:

- AC Overlay
- Recycled AC Overlay
- Mill and AC Overlay
- Slurry Seal Coat
- Aggregate Seal Coat
- Sand Seal Coat
- Fog Seal

The friction data was then filtered and split into two unique data sets. The first data set included all friction data obtained from Construction Number 1 (or prior to the M, R & R Treatment). The last recorded friction measurement was selected and used to represent the level of friction prior to construction. The second data set included all friction data obtained from Construction Number 2 (or after M, R&R Treatment). The first recorded friction measurement was selected and used to represent the level of friction after construction. The difference between these two friction measurements represents the impact of the treatment on the level of friction. The percent change in friction level was then calculated for each LTPP site. This was calculated from the following equation:

$$\text{Percent Change in SN} = (SN_{after} - SN_{prior})/SN_{prior} \qquad [17]$$

The average percent change in SN was calculated for each treatment group. This value represents the overall impact of applying each treatment on the level of skid resistance. The analysis was performed at three different levels. For the first level, the pre- and post-construction skid numbers were examined for all flexible LTPP sections and the average group treatment level was calculated. The duration between skid testing cycles was observed to vary from 0 years to 13 years after construction. As a result, for the second level of analysis, only the LTPP sections with less than 5 years between the pre- and post-construction skid numbers were included in the analysis. Upon further examination of the data, it was observed that a number of LTPP sections showed a decrease in the level of friction as a result of the treatment. This could be attributed to a number of factors such as an invalid SN, skid testing performed during different times of the year (seasonal impacts such as wet weather), operator error, etc. As a result, the third level of analysis examined LTPP sections with less than 5 years between the pre- and post-construction skid numbers and sections that only showed an improvement in SN as a result of the various treatments. Summary statistics were determined for each treatment group and the results are presented below in Tables 6.3 and 6.4 for the pre- and post-construction conditions. The percent change in SN condition for each treatment group for the three levels of analysis is presented in Table 6.5.

Table 6.3: Summary Statistics for Pre-Construction SN Condition

Treatment	Minimum SN	Maximum SN	Average SN	Standard Deviation
AC Overlay	19.0	90.0	43.2	12.3
Recycled AC Overlay	32.0	69.0	45.1	9.6
Mill and Overlay	15.0	67.0	42.8	15.0
Slurry Seal Coat	23.0	63.0	43.2	11.8
Aggregate Seal Coat	42.0	44.0	43.0	1.4
Sand Seal Coat	36.0	60.0	45.0	11.3
Fog Seal	30.0	84.2	44.8	9.7

Table 6.4: Summary Statistics for Post-Construction SN Condition

Treatment	Minimum SN	Maximum SN	Average SN	Standard Deviation
AC Overlay	27.0	99.0	45.4	10.1
Recycled AC Overlay	34.0	*65.0*	46.9	7.4
Mill and Overlay	31.3	85.0	46.6	8.0
Slurry Seal Coat	42.0	66.0	54.6	6.4
Aggregate Seal Coat	25.5	61.8	44.7	10.3
Sand Seal Coat	54.0	56.0	55.0	1.4
Fog Seal	37.0	61.0	45.6	11.1

Table 6.5: Percent Change in Average SN

Treatment	Level 1 % Change	Level 2 % Change	Level 3 % Change
AC Overlay	8	10	27
Recycled AC Overlay	5	6	14
Mill and Overlay	6	8	17
Slurry Seal Coat	20	22	33
Aggregate Seal Coat	0	-1	30
Sand Seal Coat	25	25	25
Fog Seal	1	1	4

6.4 Discussion of Results

As can be observed from Table 6.5 (Level 3), the largest increase in SN was for the Slurry Seal Coat and Aggregate Seal Coat treatment Groups. Both of these treatments can be characterized as a surface treatment. The lowest increase in SN was for the Fog Seal Group. This is logical since a Fog Seal generally results in a smoother surface with lower surface friction. The Asphalt Concrete Overlay Group has an increase of 27% and was found to be greater than asphalt overlays which contained recycled asphalt concrete (14%).

An important factor to highlight is that even though Surface Treatments (Slurry and Aggregate Seal Coats) offer similar or greater performance to the AC Overlay or Mill and Overlay treatments at lower costs, the service lives for the surface treatments are typically much lower. For example, on a roadway with high traffic volumes, the service life for a surface treatment might be one to two years, where an Asphalt Overlay may last 8 to 12 years. A Life Cycle Cost Analysis (LCCA) that identifies the most cost effective M, R & R strategies for the given pavement structure which considers all design parameters and site conditions should be performed prior to selecting and implementing the treatment.

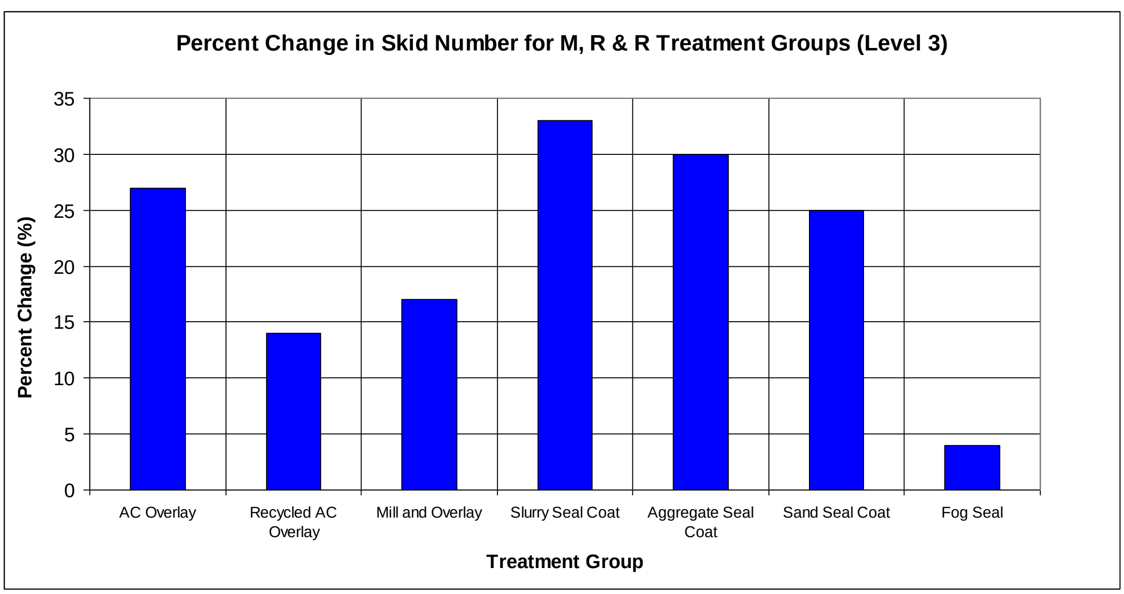

Figure 6.6: Percent Increase in SN for Various M, R and R Treatments

6.5 Summary

Despite that fact that the preservation and maintenance concept has been around for several decades, a number of transportation agencies still practice the old reactive approach despite the benefits to life cycle costs. As safety is becoming more of a concern on our nation's highways and the number of collisions continues to increase, agencies will be examining methods and techniques to increase the level of safety of their pavements and highway alignments. The level of safety of a pavement has typically been measured as a function of its skid resistance.

Studies have shown that highway improvements such as increasing the radius of a horizontal curve or increasing the skid resistance of a pavement can result in a reduction in the number of collisions and improved levels of service. The costs of increasing the radius of a horizontal curve or widening a highway alignment tend to be significantly higher than a preservation or maintenance activity such as an Asphalt Concrete Overlay or a Surface Treatment. Evaluating the effectiveness or performance of a treatment is useful to an agency so they can determine what treatments or strategies offer the best "bang for the buck".

Skid data is not readily available to researchers or the public due to the sensitivity of the data and the potential risk to the agency (lawsuits and litigation). The LTPP project provides engineers and researchers with a valuable source of high quality pavement performance data that can be used to evaluate pavement performance. The LTPP database has an extensive skid data set from sites located across Canada and the United States.

Chapter 7
Cost Effectiveness of Safety Initiated Preservation and Maintenance

This Chapter demonstrates the benefits of using preservation and maintenance to increase skid resistance and improve the level of safety of a highway alignment. Life Cycle Cost Analysis (LCCA) is performed to quantify the savings and benefits as a result of preservation and maintenance. A Decision Matrix is also developed to assist with the selection of treatments based on the level of traffic, total number of collisions, and level of pavement surface friction.

7.1 Introduction

Conventional or routine pavement maintenance practices can be often characterized as reactive in nature, performed on failing pavements and not always contribute to long-term performance. They are often not cost effective and sometimes performed under harsh or severe conditions. Preservation or maintenance is an essential component of an effective pavement management framework or strategy and can be used to improve the level of safety of a roadway or highway network.

As a part of this study, the benefits of using preservation strategies to improve the level of safety of a highway and the associated Life Cycle Cost Analysis (LCCA) was performed. In addition, a Decision Making Framework was developed which includes a Decision Matrix which can be used to assist in selecting a preventive maintenance treatment based on a number of factors.

7.2 Development of Decision Making Framework

The purpose of this evaluation is to develop a decision making framework to assist in determining what preventive maintenance treatment should be selected for improving the level of safety of a highway. A number of factors such as traffic levels, existing pavement conditions, level of pavement surface friction and the number of collisions influence the selection of an appropriate treatment. As a result, a framework for developing a Decision Making Matrix to aid in the selection criteria for improving the level of safety of a highway was developed. The following parameters were considered in the framework:

- Level of Pavement Surface Friction (SN)
- Number of Collisions
- Traffic volumes (AADT)
- Pavement Type
- Rehabilitation Strategies
- Existing Pavement Condition (PCI)

7.2.1 Pavement Surface Friction

The level of available pavement friction influences the level of safety and preservation and maintenance strategy. Based on the model developed in Chapter 5.0, a significant reduction in the collision rate (CR) occurs when the skid number (SN) is increased. The was also demonstrated by Kamel and Gartshore (1982) who observed significant decreases in the number of collisions as a result of asphalt concrete overlays. The collision rate for a pavement section with a known SN can be calculated from the following equation, which was developed in Chapter 5.0:

$$CR = 695.75e^{(-0.1338*SN)} \quad [18]$$

Where,

CR = collision rate (Mveh/km)
SN = Skid Number

The reduction in the *CR*, or improvement in level of safety, can be calculated from the following equation, when the existing SN (pre-construction) and SN after rehabilitation (post-construction) are known:

$$CR_R = 695.75e^{(-0.1338*SNe)} - 695.75e^{(-0.1338*SNr)} \quad [19]$$

Where,

CR_R = *Reduction in Collision Rate*
SNe = *Existing Skid Number*
SNr = *Skid Number after rehabilitation*

7.2.2 Number of Collisions

Motor vehicle accidents have a significant cost to the province and society at large. The cost of a motor vehicle accident in the Province of Ontario in 2004 was $77,000 (Vodden et al., 2007), based on the total proportion of all fatal, injury and accidents involving only property damage. The current cost of a motor vehicle accident in the province of Ontario in 2009 is approximately $84,100. This number was determined using the Bank of Canada inflation rates from 2004 to 2009. Any reduction in the total number of accidents along a highway results in direct savings which can be equated to a benefit. When a preservation treatment such as an asphalt concrete overlay or microsurfacing is used to increase the skid resistance, a reduction in the collision rate occurs and savings are generated.

7.2.3 Traffic Volumes

The number of vehicles on a highway will influence both the level of safety and pavement performance. In addition, traffic volumes will also impact the selection of a preventive maintenance treatment. For highways with heavy traffic volumes, surface treatments such as micro surfacing and chip seals are not cost effective due to the wear and resulting shorter service lives. Alternatively, when traffic volumes are lower, a surface treatment can be a cost effective treatment to improve skid resistance and increase the level of safety.

7.2.4 Pavement Type

The type of pavement influences the selection of a preventive maintenance treatment. Since all of the highways evaluated as a part of this study have an asphalt concrete surface layer, the focus of the LCCA will be on strategies to improve pavements with an asphalt concrete surface layer.

7.2.5 Existing Pavement Condition

The existing pavement condition will influence the selection of a preventive maintenance treatment. If the surface has pavement distress such as transverse and fatigue cracking, an asphalt concrete overlay may not be a suitable option due the risk of reflective cracking. A mill and overlay may be a more suitable and cost effective treatment as it removes any surface distress and irregularities. To evaluate the existing pavement condition, the MTO relies on the Distress Manifestation Index (DMI) to evaluate the surface condtion of a pavement.

7.2.6 Rehabilitation Strategies

A number of rehabilitation strategies that are widely used in Ontario were reviewed and selected for use within the analysis. The preservation scenarios are presented in Appendix B. The following treatments were considered in the LCCA:

- Asphalt Concrete (AC) Overlay – Premium Mix
- AC Overlay – Standard Mix
- Mill and AC Overlay
- Micro Surfacing
- Crack Sealing
- Fog Seal
- Do Nothing

7.3 Decision Matrix

Based on the framework parameters outlined in Section 7.2, a Decision Matrix was developed. The Matrix is a function of the level of traffic, the total number of collisions and the level of existing pavement surface friction. Each Decision Matrix consists of 50 cells or possible combinations representing the number of collisions and level of available pavement surface friction. Within each cell are various treatments scenarios that can be considered for each combination. The objective of developing the Decision Matrix is to provide a tool that can be used to select the most cost-effective treatment for the combination of parameters. LCCA was performed for all treatment groups within each cell and the most cost effective treatment was selected to represent the preventive maintenance treatment for that cell. It is worth noting that the Do Nothing option is also considered within each cell. In total, LCCA was performed for 50 combinations representing two levels of traffic (Low AADT < 3,000 and High AADT>= 3,000), five levels of collisions (Collision Count = Very Low=5, Low=10, Medium=15, High = 20, Very High = 30), and five levels of pavement surface friction (SN = Very Poor =15, Poor = 25, Fair = 35, Good = 45, Very Good =55).

Table 7.1: Decision Matrix for Selecting Preventive Maintenance Treatment

Traffic Level	Number of Collisions	Level of Pavement Surface Friction				
		Very Poor	**Poor**	**Fair**	**Good**	**Very Good**
Low Traffic Volumes	Very Low	AC Ovly Mill & AC Ovly Micro Surface Recy AC Ovly Do Nothing	AC Ovly Mill & AC Ovly Micro Surface Recy AC Ovly Do Nothing	AC Ovly Mill & AC Ovly Micro Surface Recy AC Ovly Do Nothing	AC Ovly Mill & AC Ovly Micro Surface Recy AC Ovly Do Nothing	AC Ovly Mill & AC Ovly Micro Surface Recy AC Ovly Do Nothing
	Low	AC Ovly Mill & AC Ovly Micro Surface Recy AC Ovly Do Nothing	AC Ovly Mill & AC Ovly Micro Surface Recy AC Ovly Do Nothing	AC Ovly Mill & AC Ovly Micro Surface Recy AC Ovly Do Nothing	AC Ovly Mill & AC Ovly Micro Surface Recy AC Ovly Do Nothing	AC Ovly Mill & AC Ovly Micro Surface Recy AC Ovly Do Nothing
	Medium	AC Ovly Mill & AC Ovly Micro Surface Recy AC Ovly Do Nothing	AC Ovly Mill & AC Ovly Micro Surface Recy AC Ovly Do Nothing	AC Ovly Mill & AC Ovly Micro Surface Recy AC Ovly Do Nothing	AC Ovly Mill & AC Ovly Micro Surface Recy AC Ovly Do Nothing	AC Ovly Mill & AC Ovly Micro Surface Recy AC Ovly Do Nothing
	High	AC Ovly Mill & AC Ovly Micro Surface Recy AC Ovly Do Nothing	AC Ovly Mill & AC Ovly Micro Surface Recy AC Ovly Do Nothing	AC Ovly Mill & AC Ovly Micro Surface Recy AC Ovly Do Nothing	AC Ovly Mill & AC Ovly Micro Surface Recy AC Ovly Do Nothing	AC Ovly Mill & AC Ovly Micro Surface Recy AC Ovly Do Nothing
	Very High	AC Ovly Mill & AC Ovly Micro Surface Recy AC Ovly Do Nothing	AC Ovly Mill & AC Ovly Micro Surface Recy AC Ovly Do Nothing	AC Ovly Mill & AC Ovly Micro Surface Recy AC Ovly Do Nothing	AC Ovly Mill & AC Ovly Micro Surface Recy AC Ovly Do Nothing	AC Ovly Mill & AC Ovly Micro Surface Recy AC Ovly Do Nothing
High Traffic Volumes	Very Low	AC Ovly Mill & AC Ovly Recy AC Ovly Do Nothing	AC Ovly Mill & AC Ovly Recy AC Ovly Do Nothing	AC Ovly Mill & AC Ovly Recy AC Ovly Do Nothing	AC Ovly Mill & AC Ovly Recy AC Ovly Do Nothing	AC Ovly Mill & AC Ovly Recy AC Ovly Do Nothing
	Low	AC Ovly Mill & AC Ovly Recy AC Ovly Do Nothing	AC Ovly Mill & AC Ovly Recy AC Ovly Do Nothing	AC Ovly Mill & AC Ovly Recy AC Ovly Do Nothing	AC Ovly Mill & AC Ovly Recy AC Ovly Do Nothing	AC Ovly Mill & AC Ovly Recy AC Ovly Do Nothing
	Medium	AC Ovly Mill & AC Ovly Recy AC Ovly Do Nothing	AC Ovly Mill & AC Ovly Recy AC Ovly Do Nothing	AC Ovly Mill & AC Ovly Recy AC Ovly Do Nothing	AC Ovly Mill & AC Ovly Recy AC Ovly Do Nothing	AC Ovly Mill & AC Ovly Recy AC Ovly Do Nothing
	High	AC Ovly Mill & AC Ovly Recy AC Ovly Do Nothing	AC Ovly Mill & AC Ovly Recy AC Ovly Do Nothing	AC Ovly Mill & AC Ovly Recy AC Ovly Do Nothing	AC Ovly Mill & AC Ovly Recy AC Ovly Do Nothing	AC Ovly Mill & AC Ovly Recy AC Ovly Do Nothing
	Very High	AC Ovly Mill & AC Ovly Recy AC Ovly Do Nothing	AC Ovly Mill & AC Ovly Recy AC Ovly Do Nothing	AC Ovly Mill & AC Ovly Recy AC Ovly Do Nothing	AC Ovly Mill & AC Ovly Recy AC Ovly Do Nothing	AC Ovly Mill & AC Ovly Recy AC Ovly Do Nothing

Notes: AC = Asphalt Concrete
Ovly = Overlay

7.4 Life Cycle Cost Analysis (LCCA)

To determine what treatment is the most cost effective option within each cell of the Decision Matrix, LCCA was performed. LCCA is a process for evaluating the total economic worth of a project by analyzing initial costs and discounted future costs, including maintenance, rehabilitation, and reconstruction, over the life of a project. Since the objective of this study is focussed primarily on preventive maintenance, rehabilitation and reconstruction costs are not considered as a part of the LCCA. The following sections describe the maintenance and preventive maintenance treatments considered, the strategies for each scenario, and the basic assumptions of the LCCA.

The following assumptions were used to determine the unit rates for the various maintenance and rehabilitation treatments. These rates represent recent material prices for the province of Ontario:

- Asphalt cost = \$95/tonne; Asphalt density = 2.46 tonne/m^3
- Granular A cost = \$22/tonne; Granular A density = 2.40 tonne/m^3
- Granular B cost = \$19/tonne; Granular B density = 2.0 tonne/m^3

The maintenance and rehabilitation treatment costs used in the LCCA are presented below in Table 7.2. These treatments were used as part of the preventive maintenance strategies considered in the analysis. It is worth noting that the prices for each treatment were obtained from recent rates used in the Province of Ontario.

Each treatment strategy found within the various cells of the Decision Matrix consists of a number of maintenance activities throughout the life cycle that was evaluated. In total, three life cycle periods were evaluated (7 year, 10 year and 15 year periods). These life cycle periods were selected since preventive maintenance is being evaluated. As an example, the Asphalt Concrete Overlay strategy has Crack Seal (10% cracking) at year 3, while the Recycle Asphalt Concrete Overlay strategy has Crack Seal (20% cracking) at year 4 and a Fog Seal at year 7.

Table 7.2: Maintenance and Rehabilitation Treatment Costs and Service Lives

Model Identifier	**Treatment**	**Unit Cost ($/m^2)**
Fog Seal	Sealant application to prevent weathering and raveling	$2.50
Crack Seal	Routing and Sealing of crack	$5.00
Micro Surface	Micro Surface existing pavement	$5.00
Mill 50 mm + 80 mm AC O/L	Mill 50 mm AC and Overlay of 80 mm AC	$28.10
50 mm AC Overlay	Overlay of 50 mm AC	$12.55
50 mm Recycled AC Overlay	Overlay of 50 mm Recycled AC	$11.55

7.4.1 LCCA Assumptions

To carry out LCCA, a number of assumptions were made regarding the key inputs and parameters. These were based on a review of the literature and engineering judgement. The following basic assumptions were used in the LCCA:

- Discount rate is 4%
- Area of pavement section = 100,000 m^2
- Costs shown in 2009 dollars (no inflation)
- Analysis periods (7, 10 and 15 years)
- Present Worth (PW) and Equivalent Annual Uniform Cost (EAUC) were used as the basis for comparison between strategies
- The costs associated with noise, emissions, user delay, etc. were not considered in the LCCA as they are difficult to model and quantify.

7.4.1.1 Present Worth

The Present Worth converts all the treatment costs to the present using the following formula:

$$PW_n = \frac{F}{(1+i)^n} \qquad [20]$$

Where,

PW_n = Total present worth of the strategy in year n

n = Age of the pavement within the life cycle analysis period

F = Cost of the treatment

i = Discount rate (4%)

The net present worth (NPW) of the strategy combines all the treatments costs over the analysis period:

$$NPW = \sum_{n=1}^{N} \frac{F}{(1+i)^n} \quad [21]$$

Where,

NPW = Total present worth of the strategy

N = Life cycle analysis period

n = Age of the pavement within the life cycle analysis period

F = Cost of the treatment

i = Discount rate (4%)

The incremental Present Worth is calculated as follows:

$$IncPW_n = PWn \text{ for n=0} \quad [22]$$

$$IncPW_n = IncPW_{n-1} + PWn \text{ for n>0} \quad [23]$$

Where,

IncPWn = *Incremental present worth for year n*

IncPWn-1 = *Incremental present worth for year n-1*

PWn = *Present worth value in year n*

7.4.1.2 Equivalent Annual Uniform Cost

The equivalent annual uniform cost (EAUC) is an annuity that is mathematically equivalent to a generally more complicated cash flow. The EAUC is used to calculate the regular annuity, given the present worth and is calculated as follows:

$$EAUC = NPW \frac{i(1+i)^N}{(1+i)^N - 1} \quad [24]$$

Where,

EAUC = Equivalent annual uniform cost

NPW = Total net present worth of the strategy over the analysis period

I = Discount rate (4%)

N = Analysis period

The incremental costs of EAUC are calculated similar to the incremental costs for the present worth. First all the equivalent annual costs are converted to an annual Present Worth cost and then each annual present worth cost is added to the previous annual present worth cost.

7.4.2 LCCA Results

The results of the net present worth and equivalent annual uniform total incremental costs for each strategy are shown in Tables 7.3 and 7.4, Tables 7.5 and 7.6 and Tables 7.7 and 7.8 for seven, ten and fifteen year life cycle periods, respectively. It is important to note that cells with N/A indicate that the rehabilitation strategy is not applicable for the case scenario. The most cost-effective strategy for each case scenario is bolded red in Tables 7.3 to 7.8, and summarized in Tables 7.9 to 7.11, respectively. For the seven year life cycle period, the AC Overlay strategy was found to be the most cost effective strategy for more than half (27 out of 50) of the case scenarios. Whereas, the Micro Surface strategy represents the most economical strategy for 36 percent (18 out of 50) of the case scenarios. The Recycled AC Overlay strategy was found to be the most economical strategy for 20 percent (10 out of 50) of the case scenarios. For the fifteen year life cycle period, the Recycled AC Overlay strategy was found to be the most cost effective strategy for more than half (27 out of 50) of the case scenarios. Whereas, the Micro Surface strategy represent the most economical strategy for 24 percent (12 out of 50) of the case scenarios. The AC Overlay strategy was found to be the most economical strategy for 26 percent (13 out of 50) of the case scenarios. The LCCA was performed to identify the most cost-effective strategies based on the assumed unit rates and case scenarios. It is important to note that an agency may wish to consider using an alternative strategy when the costs between the strategies are very close (i.e. AC Overlay vs. Micro Surfacing for many cases). Results, in terms the incremental equivalent annual uniform cost representing four cases is presented in Figures 7.1 to 7.4.

Table 7.3: Net Present Worth (NPW) Results (7 years)

Case Scenario			Preservation Strategy				
Traffic	**Number of Collisions**	**Level of Friction**	**Mill and AC Overlay ($)**	**AC Overlay ($)**	**Recycled AC Overlay ($)**	**Micro Surface ($)**	**Do Nothing ($)**
Low	Very Low	Very Poor	295,543	**669,770**	641,988	647,087	-2,944,364
Low	Very Low	Poor	279,660	**653,887**	608,106	639,478	-1,192,260
Low	Very Low	Fair	219,120	593,347	504,704	**610,475**	-1,192,260
Low	Very Low	Good	-11,620	362,607	317,518	**499,933**	-1,192,260
Low	Very Low	Very Good	-891,067	-516,840	-562,621	**78,709**	-1,192,260
Low	Low	Very Poor	1,482,153	**1,856,380**	1,810,599	1,836,640	-2,384,520
Low	Low	Poor	1,450,053	**1,824,267**	1,778,832	1,821,422	-2,384,520
Low	Low	Fair	1,337,628	1,703,188	1,657,753	**1,763,416**	-2,384,520
Low	Low	Good	867,827	1,250,707	1,196,965	**1,542,332**	-2,384,520
Low	Low	Very Good	-891,067	-516,840	-544,622	**699,692**	-2,384,520
Low	Medium	Very Poor	2,668,764	**3,042,991**	2,997,902	3,026,194	-3,576,780
Low	Medium	Poor	2,621,113	2,995,339	2,950,251	**3,003,365**	-3,576,780
Low	Medium	Fair	2,439,495	2,813,722	2,768,633	**2,916,357**	-3,576,780
Low	Medium	Good	1,747,273	2,121,500	1,503,745	**2,454,749**	-3,576,780
Low	Medium	Very Good	-891,067	-516,840	-561,929	1,320,771	-3,576,780
Low	High	Very Poor	3,855,374	**4,229,601**	4,184,512	4,215,747	-4,769,040
Low	High	Poor	3,791,839	4,166,066	4,120,977	**4,185,309**	-4,769,040
Low	High	Fair	3,549,682	3,923,909	3,878,820	**4,069,298**	-4,769,040
Low	High	Good	2,626,720	3,001,280	2,955,859	**3,627,131**	-4,769,040
Low	High	Very Good	-891,067	-516,840	-561,929	1,941,850	-4,769,040
Low	Very High	Very Poor	6,228,594	6,602,821	6,557,732	6,594,853	-7,153,559
Low	Very High	Poor	6,133,292	6,507,519	6,462,430	6,549,197	-7,153,559
Low	Very High	Fair	5,770,056	6,144,283	6,099,194	6,375,180	-7,153,559
Low	Very High	Good	4,385,614	4,759,841	4,714,752	5,711,929	-7,153,559
Low	Very High	Very Good	-891,067	-516,840	-561,929	-99,366	-7,153,559
High	Very Low	Very Poor	729,863	1,442,692	**1,572,146**	N/A	-1,192,260
High	Very Low	Poor	279,660	568,305	**569,601**	N/A	-1,192,260
High	Very Low	Fair	219,120	**507,766**	452,795	N/A	-1,192,260
High	Very Low	Good	-11,620	**277,025**	2,324	N/A	-1,192,260
High	Very Low	Very Good	-891,067	**-602,422**	-1,573,122	N/A	-1,192,260
High	Low	Very Poor	1,482,153	1,770,799	**1,781,840**	N/A	-2,384,520
High	Low	Poor	1,450,053	**1,738,339**	1,719,821	N/A	-2,384,520
High	Low	Fair	1,329,307	**1,617,953**	1,484,826	N/A	-2,384,520
High	Low	Good	867,827	**1,173,779**	583,884	N/A	-2,384,520
High	Low	Very Good	-891,067	**-602,422**	-2,815,362	N/A	-2,384,520
High	Medium	Very Poor	2,668,764	2,957,409	**2,964,455**	N/A	-3,576,780
High	Medium	Poor	2,621,113	**2,909,758**	2,871,426	N/A	-3,576,780
High	Medium	Fair	2,439,495	**2,728,140**	2,516,856	N/A	-3,576,780
High	Medium	Good	1,747,273	**2,035,919**	786,752	N/A	-3,576,780
High	Medium	Very Good	-891,067	**-602,422**	-637,077	N/A	-3,576,780
High	High	Very Poor	3,855,374	4,144,019	**4,145,685**	N/A	-4,769,040
High	High	Poor	3,791,839	**4,080,484**	4,021,647	N/A	-4,769,040
High	High	Fair	3,549,682	**3,838,327**	3,548,887	N/A	-4,769,040
High	High	Good	2,626,720	**2,916,031**	1,764,311	N/A	-4,769,040
High	High	Very Good	-891,067	**-602,422**	-656,357	N/A	-4,769,040
High	Very High	Very Poor	6,228,594	**6,517,239**	6,508,145	N/A	-7,153,559
High	Very High	Poor	6,133,292	**6,421,937**	6,322,088	N/A	-7,153,559
High	Very High	Fair	5,770,056	**6,058,701**	5,612,948	N/A	-7,153,559
High	Very High	Good	4,385,614	**4,674,259**	2,910,124	N/A	-7,153,559
High	Very High	Very Good	-891,067	**-602,422**	-694,918	N/A	-7,153,559

Table 7.4: Equivalent Annual Uniform Total Incremental Cost (EAUC) Results (7 years)

Case Scenario			Preservation Strategy				
Traffic	**Number of Collisions**	**Level of Friction**	**Mill and AC Overlay ($)**	**AC Overlay ($)**	**Recycled AC Overlay ($)**	**Micro Surface ($)**	**Do Nothing ($)**
Low	Very Low	Very Poor	729,863	**1,654,042**	1,585,432	1,598,024	-1,192,260
Low	Very Low	Poor	690,638	**1,614,816**	1,501,757	1,579,232	-2,944,364
Low	Very Low	Fair	541,132	1,465,310	1,246,400	**1,507,608**	-1,192,260
Low	Very Low	Good	-28,697	895,481	784,132	**1,234,618**	-2,944,364
Low	Very Low	Very Good	-2,200,548	-1,276,370	-1,389,429	**194,377**	-1,192,260
Low	Low	Very Poor	3,660,275	**4,584,453**	4,471,394	4,535,704	-5,888,728
Low	Low	Poor	3,581,002	**4,505,147**	4,392,942	4,498,120	-5,888,728
Low	Low	Fair	3,303,360	4,206,135	4,093,931	**4,354,871**	-5,888,728
Low	Low	Good	2,143,155	3,088,703	2,955,983	**3,808,891**	-5,888,728
Low	Low	Very Good	-2,200,548	-1,276,370	-1,344,979	**1,727,935**	-5,888,728
Low	Medium	Very Poor	6,590,687	**7,514,865**	7,403,515	7,473,384	-8,833,092
Low	Medium	Poor	6,473,009	7,397,187	7,285,838	**7,417,008**	-8,833,092
Low	Medium	Fair	6,024,492	6,948,670	6,837,320	**7,202,135**	-8,833,092
Low	Medium	Good	4,315,006	5,239,185	3,713,597	**6,062,163**	-8,833,092
Low	Medium	Very Good	-2,200,548	-1,276,370	-1,387,720	3,261,731	-8,833,092
Low	High	Very Poor	9,521,098	**10,445,276**	10,333,927	10,411,064	-11,777,456
Low	High	Poor	9,364,195	10,288,373	10,177,024	**10,335,895**	-11,777,456
Low	High	Fair	8,766,172	9,690,350	9,579,000	**10,049,398**	-11,777,456
Low	High	Good	6,486,858	7,411,858	7,299,687	**8,957,438**	-11,777,456
Low	High	Very Good	-2,200,548	-1,276,370	-1,387,720	4,795,526	-11,777,456
Low	Very High	Very Poor	15,381,922	16,306,100	16,194,750	**16,286,423**	-17,666,184
Low	Very High	Poor	15,146,567	16,070,745	15,959,395	**16,173,671**	-17,666,184
Low	Very High	Fair	14,249,532	15,173,710	15,062,360	**15,743,925**	-17,666,184
Low	Very High	Good	10,830,561	11,754,739	11,643,390	**14,105,984**	-17,666,184
Low	Very High	Very Good	-2,200,548	-1,276,370	-1,387,720	**-245,390**	-17,666,184
High	Very Low	Very Poor	295,543	584,189	**636,608**	N/A	-2,944,364
High	Very Low	Poor	690,638	1,403,466	**1,406,666**	N/A	-2,944,364
High	Very Low	Fair	541,132	**1,253,960**	1,118,207	N/A	-2,944,364
High	Very Low	Good	-28,697	**684,132**	5,740	N/A	-2,944,364
High	Very Low	Very Good	-2,200,548	**-1,487,720**	-3,884,929	N/A	-2,944,364
High	Low	Very Poor	3,660,275	4,373,104	**4,400,371**	N/A	-5,888,728
High	Low	Poor	3,581,002	**4,292,942**	4,247,211	N/A	-5,888,728
High	Low	Fair	3,282,812	**3,995,640**	3,666,875	N/A	-5,888,728
High	Low	Good	2,143,155	**2,898,724**	1,441,941	N/A	-5,888,728
High	Low	Very Good	-2,200,548	**-1,487,720**	-6,952,721	N/A	-5,888,728
High	Medium	Very Poor	6,590,687	7,303,515	**7,320,915**	N/A	-8,833,092
High	Medium	Poor	6,473,009	**7,185,838**	7,091,175	N/A	-8,833,092
High	Medium	Fair	6,024,492	**6,737,320**	6,215,542	N/A	-8,833,092
High	Medium	Good	4,315,006	**5,027,835**	1,942,935	N/A	-8,833,092
High	Medium	Very Good	-2,200,548	**-1,487,720**	-1,573,302	N/A	-8,833,092
High	High	Very Poor	9,521,098	10,233,927	**10,238,040**	N/A	-11,777,456
High	High	Poor	9,364,195	**10,077,024**	9,931,720	N/A	-11,777,456
High	High	Fair	8,766,172	**9,479,000**	8,764,209	N/A	-11,777,456
High	High	Good	6,486,858	**7,201,330**	4,357,081	N/A	-11,777,456
High	High	Very Good	-2,200,548	**-1,487,720**	-1,620,917	N/A	-11,777,456
High	Very High	Very Poor	15,381,922	**16,094,750**	16,072,290	N/A	-17,666,184
High	Very High	Poor	15,146,567	**15,859,395**	15,612,811	N/A	-17,666,184
High	Very High	Fair	14,249,532	**14,962,360**	13,861,544	N/A	-17,666,184
High	Very High	Good	10,830,561	**11,543,390**	7,186,742	N/A	-17,666,184
High	Very High	Very Good	-2,200,548	**-1,487,720**	-1,716,145	N/A	-17,666,184

Table 7.5: Net Present Worth (NPW) Results (10 years)

Case Scenario			Preservation Strategy				
Traffic	**Number of Collisions**	**Level of Friction**	**Mill and AC Overlay ($)**	**AC Overlay ($)**	**Recycled AC Overlay ($)**	**Micro Surface ($)**	**Do Nothing ($)**
Low	Very Low	Very Poor	840,664	**1,326,873**	1,260,032	1,130,165	-2,018,562
Low	Very Low	Poor	813,773	**1,299,981**	1,233,140	1,117,281	-2,018,562
Low	Very Low	Fair	711,276	**1,197,485**	1,130,644	1,068,178	-3,831,132
Low	Very Low	Good	320,620	806,828	739,988	**881,025**	-2,018,562
Low	Very Low	Very Good	-1,168,332	-682,124	-748,964	**167,869**	-3,831,132
Low	Low	Very Poor	2,849,661	**3,335,870**	3,269,029	3,144,144	-4,037,124
Low	Low	Poor	2,849,661	**3,282,086**	3,215,245	3,118,378	-4,037,124
Low	Low	Fair	2,590,884	**3,077,093**	3,010,252	3,020,171	-4,037,124
Low	Low	Good	1,809,572	2,295,781	2,228,940	**2,645,864**	-4,037,124
Low	Low	Very Good	-1,168,332	-682,124	-748,964	**1,219,227**	-4,037,124
Low	Medium	Very Poor	4,858,658	**5,344,866**	5,278,026	5,158,124	-6,055,686
Low	Medium	Poor	4,785,260	**5,264,191**	5,197,350	5,119,474	-6,055,686
Low	Medium	Fair	4,470,493	4,956,701	4,889,860	**4,972,164**	-6,055,686
Low	Medium	Good	3,298,524	3,784,733	3,717,892	**4,410,704**	-6,055,686
Low	Medium	Very Good	-1,157,515	-675,808	-660,492	**2,250,739**	-5,944,071
Low	High	Very Poor	6,867,655	**7,353,863**	7,287,022	7,172,103	-8,074,248
Low	High	Poor	6,760,087	**7,246,295**	7,179,454	7,120,570	-8,074,248
Low	High	Fair	6,350,101	6,836,310	6,769,469	**6,924,157**	-8,074,248
Low	High	Good	4,787,476	5,273,685	5,206,844	**6,175,544**	-8,074,248
Low	High	Very Good	-1,168,332	-682,124	-748,964	**3,322,270**	-8,074,248
Low	Very High	Very Poor	10,885,648	11,371,857	11,305,016	11,200,062	-12,111,372
Low	Very High	Poor	10,724,296	11,210,505	11,143,664	11,122,763	-12,111,372
Low	Very High	Fair	10,109,318	10,595,526	10,528,685	**10,828,142**	-12,111,372
Low	Very High	Good	7,765,381	8,251,589	8,184,748	**9,705,223**	-12,111,372
Low	Very High	Very Good	-1,168,332	-682,124	-748,964	**-133,621**	-12,111,372
Hi	Very Low	Very Poor	830,222	1,216,242	**1,237,513**	N/A	-2,018,562
Hi	Very Low	Poor	803,330	1,189,350	**1,210,621**	N/A	-2,018,562
Hi	Very Low	Fair	700,833	1,086,854	**1,108,125**	N/A	-3,831,132
Hi	Very Low	Good	310,177	696,198	**717,469**	N/A	-2,018,562
Hi	Very Low	Very Good	-1,178,775	-792,754	**-771,484**	N/A	-3,831,132
Hi	Low	Very Poor	2,839,218	3,225,239	**3,246,510**	N/A	-4,037,124
Hi	Low	Poor	2,785,434	3,171,455	**3,192,726**	N/A	-4,037,124
Hi	Low	Fair	2,580,442	2,966,462	**2,987,733**	N/A	-4,037,124
Hi	Low	Good	1,799,129	2,185,150	**2,206,421**	N/A	-4,037,124
Hi	Low	Very Good	-1,178,775	-792,754	**-771,484**	N/A	-4,037,124
Hi	Medium	Very Poor	4,848,215	5,234,236	**5,255,507**	N/A	-6,055,686
Hi	Medium	Poor	4,767,539	5,153,560	**5,174,831**	N/A	-6,055,686
Hi	Medium	Fair	4,460,050	4,846,071	**4,867,341**	N/A	-6,055,686
Hi	Medium	Good	3,288,081	3,674,102	**3,695,373**	N/A	-6,055,686
Hi	Medium	Very Good	-1,167,861	-695,060	**-682,803**	N/A	-5,944,071
Hi	High	Very Poor	6,857,212	7,243,233	**7,264,503**	N/A	-8,074,248
Hi	High	Poor	6,749,644	7,135,665	**7,156,935**	N/A	-8,074,248
Hi	High	Fair	6,339,658	6,725,679	**6,746,950**	N/A	-8,074,248
Hi	High	Good	4,777,033	5,163,054	**5,184,325**	N/A	-8,074,248
Hi	High	Very Good	-1,178,775	-792,754	**-771,484**	N/A	-8,074,248
Hi	Very High	Very Poor	10,875,205	11,261,226	**11,282,497**	N/A	-12,111,372
Hi	Very High	Poor	10,713,853	11,099,874	**11,121,145**	N/A	-12,111,372
Hi	Very High	Fair	10,098,875	10,484,896	**10,506,166**	N/A	-12,111,372
Hi	Very High	Good	7,754,938	8,140,958	**8,162,229**	N/A	-12,111,372
Hi	Very High	Very Good	-1,178,775	-810,551	**-771,484**	N/A	-12,111,372

Table 7.6: Net Present Worth (EAUC) Results (10 years)

Case Scenario			Preservation Strategy				
Traffic	Number of Collisions	Level of Friction	Mill and AC Overlay ($)	AC Overlay ($)	Recycled AC Overlay ($)	Micro Surface ($)	Do Nothing ($)
Low	Very Low	Very Poor	1,595,540	**2,518,340**	2,391,479	2,144,997	-3,831,132
Low	Very Low	Poor	1,544,500	**2,467,300**	2,340,440	2,120,545	-3,831,132
Low	Very Low	Fair	1,349,967	**2,272,767**	2,145,907	2,027,349	-3,831,132
Low	Very Low	Good	608,521	1,531,321	1,404,460	**1,672,142**	-3,831,132
Low	Very Low	Very Good	-2,217,437	-1,294,637	-1,421,498	**318,607**	-3,831,132
Low	Low	Very Poor	5,408,517	**6,331,317**	6,204,457	5,967,431	-7,662,263
Low	Low	Poor	5,408,517	**6,229,238**	6,102,377	5,918,528	-7,662,263
Low	Low	Fair	4,917,372	**5,840,171**	5,713,311	5,732,136	-7,662,263
Low	Low	Good	3,434,479	4,357,279	4,230,418	**5,021,721**	-7,662,263
Low	Low	Very Good	-2,217,437	-1,294,637	-1,421,498	**2,314,034**	-7,662,263
Low	Medium	Very Poor	9,221,494	**10,144,294**	10,017,434	9,789,866	-11,493,395
Low	Medium	Poor	9,082,189	**9,991,175**	9,864,315	9,716,510	-11,493,395
Low	Medium	Fair	8,484,776	9,407,576	9,280,715	**9,436,923**	-11,493,395
Low	Medium	Good	6,260,437	7,183,237	7,056,376	**8,371,300**	-11,493,395
Low	Medium	Very Good	-2,217,437	-1,294,637	-1,265,297	4,311,712	-11,386,983
Low	High	Very Poor	13,034,472	**13,957,272**	13,830,411	13,612,300	-15,324,527
Low	High	Poor	12,830,313	**13,753,113**	13,626,252	13,514,493	-15,324,527
Low	High	Fair	12,052,180	12,974,980	12,848,120	**13,141,710**	-15,324,527
Low	High	Good	9,086,395	10,009,195	9,882,335	**11,720,879**	-15,324,527
Low	High	Very Good	-2,217,437	-1,294,637	-1,421,498	**6,305,505**	-15,324,527
Low	Very High	Very Poor	20,660,426	**21,583,226**	21,456,366	21,257,169	-22,986,790
Low	Very High	Poor	20,354,188	**21,276,988**	21,150,128	21,110,458	-22,986,790
Low	Very High	Fair	19,186,989	20,109,789	19,982,929	20,551,283	-22,986,790
Low	Very High	Good	14,738,311	15,661,111	15,534,251	18,420,037	-22,986,790
Low	Very High	Very Good	-2,217,437	-1,294,637	-1,421,498	-253,607	-22,986,790
Hi	Very Low	Very Poor	1,575,720	2,308,368	**2,348,739**	N/A	-3,831,132
Hi	Very Low	Poor	1,524,680	2,257,329	**2,297,700**	N/A	-3,831,132
Hi	Very Low	Fair	1,330,147	2,062,796	**2,103,166**	N/A	-3,831,132
Hi	Very Low	Good	588,701	1,321,349	**1,361,720**	N/A	-3,831,132
Hi	Very Low	Very Good	-2,237,257	-1,504,609	**-1,464,238**	N/A	-3,831,132
Hi	Low	Very Poor	5,388,697	6,121,346	**6,161,716**	N/A	-7,662,263
Hi	Low	Poor	5,286,618	6,019,266	**6,059,637**	N/A	-7,662,263
Hi	Low	Fair	4,897,552	5,630,200	**5,670,571**	N/A	-7,662,263
Hi	Low	Good	3,414,659	4,147,307	**4,187,678**	N/A	-7,662,263
Hi	Low	Very Good	-2,237,257	-1,504,609	**-1,464,238**	N/A	-7,662,263
Hi	Medium	Very Poor	9,201,675	9,934,323	**9,974,694**	N/A	-11,493,395
Hi	Medium	Poor	9,048,556	9,781,204	**9,821,575**	N/A	-11,493,395
Hi	Medium	Fair	8,464,956	9,197,604	**9,237,975**	N/A	-11,493,395
Hi	Medium	Good	6,240,617	6,973,266	**7,013,636**	N/A	-11,493,395
Hi	Medium	Very Good	-2,237,257	-1,331,518	**-1,308,037**	N/A	-11,386,983
Hi	High	Very Poor	13,014,652	13,747,300	**13,787,671**	N/A	-15,324,527
Hi	High	Poor	12,810,493	13,543,142	**13,583,512**	N/A	-15,324,527
Hi	High	Fair	12,032,360	12,765,009	**12,805,380**	N/A	-15,324,527
Hi	High	Good	9,066,575	9,799,224	**9,839,594**	N/A	-15,324,527
Hi	High	Very Good	-2,237,257	-1,504,609	**-1,464,238**	N/A	-15,324,527
Hi	Very High	Very Poor	20,640,606	21,373,255	**21,413,625**	N/A	-22,986,790
Hi	Very High	Poor	20,334,368	21,067,017	**21,107,387**	N/A	-22,986,790
Hi	Very High	Fair	19,167,169	19,899,818	**19,940,188**	N/A	-22,986,790
Hi	Very High	Good	14,718,491	15,451,140	**15,491,511**	N/A	-22,986,790
Hi	Very High	Very Good	-2,237,257	-1,538,387	**-1,464,238**	N/A	-22,986,790

Table 7.7: Net Present Worth (NPW) Results (15 years)

Case Scenario			Preservation Strategy				
Traffic	**Number of Collisions**	**Level of Friction**	**Mill and AC Overlay ($)**	**AC Overlay ($)**	**Recycled AC Overlay ($)**	**Micro Surface ($)**	**Do Nothing ($)**
Low	Very Low	Very Poor	1,743,545	**2,349,759**	2,266,133	1,929,243	-3,251,399
Low	Very Low	Poor	1,700,229	**2,306,442**	2,222,817	1,418,001	-3,251,399
Low	Very Low	Fair	1,535,132	**2,141,346**	2,057,721	1,829,398	-4,862,293
Low	Very Low	Good	406,916	982,962	939,158	**1,118,155**	-2,561,865
Low	Very Low	Very Good	-1,492,447	-886,233	-969,858	**379,225**	-4,862,293
Low	Low	Very Poor	4,979,536	**5,585,750**	5,502,124	5,173,260	-6,502,797
Low	Low	Poor	4,979,536	**5,499,118**	5,415,492	5,131,757	-6,502,797
Low	Low	Fair	4,562,712	**5,168,925**	5,085,300	4,973,570	-6,502,797
Low	Low	Good	3,304,213	3,910,426	3,826,801	**4,370,656**	-6,502,797
Low	Low	Very Good	-1,492,447	-886,233	-969,858	**2,072,701**	-6,815,064
Low	Medium	Very Poor	8,215,528	**8,821,741**	8,738,116	8,417,278	-9,754,196
Low	Medium	Poor	8,094,816	**8,691,793**	8,608,167	8,355,023	-9,754,196
Low	Medium	Fair	7,590,291	**8,196,504**	8,112,879	8,117,743	-9,754,196
Low	Medium	Good	5,702,542	6,308,756	6,225,130	**7,213,372**	-9,754,196
Low	Medium	Very Good	-1,472,252	-873,052	-853,698	**3,685,500**	-9,492,016
Low	High	Very Poor	11,451,519	**12,057,733**	11,974,107	11,661,295	-13,005,595
Low	High	Poor	11,278,254	**11,884,468**	11,800,842	11,578,288	-13,005,595
Low	High	Fair	10,617,870	11,224,083	11,140,458	**11,261,915**	-13,005,595
Low	High	Good	8,100,872	8,707,086	8,623,460	**10,056,087**	-13,005,595
Low	High	Very Good	-1,492,447	-886,233	-969,858	**5,460,177**	-10,247,458
Low	Very High	Very Poor	17,923,502	**18,529,716**	18,446,090	18,149,330	-19,508,392
Low	Very High	Poor	17,663,604	**18,269,818**	18,186,192	18,024,820	-19,508,392
Low	Very High	Fair	16,673,028	17,279,241	17,195,616	**17,550,260**	-19,508,392
Low	Very High	Good	12,897,531	13,503,745	13,420,119	**15,741,518**	-19,508,392
Low	Very High	Very Good	-1,492,447	-886,233	-969,858	**-106,400**	-19,508,392
Hi	Very Low	Very Poor	1,720,637	2,200,115	**2,216,669**	N/A	-3,251,399
Hi	Very Low	Poor	1,677,321	2,156,799	**2,173,353**	N/A	-3,251,399
Hi	Very Low	Fair	1,512,225	1,991,703	**2,008,257**	N/A	-4,862,293
Hi	Very Low	Good	900,353	1,362,453	**1,379,008**	N/A	-3,251,399
Hi	Very Low	Very Good	-1,515,354	-1,035,876	**-1,038,630**	N/A	-4,862,293
Hi	Low	Very Poor	4,956,629	5,436,107	**5,452,661**	N/A	-6,502,797
Hi	Low	Poor	4,956,629	5,349,474	**5,366,028**	N/A	-6,502,797
Hi	Low	Fair	4,539,804	5,019,282	**5,035,836**	N/A	-6,502,797
Hi	Low	Good	3,281,305	3,760,783	**3,777,337**	N/A	-6,502,797
Hi	Low	Very Good	-1,515,354	-1,035,876	**-1,019,322**	N/A	-6,502,797
Hi	Medium	Very Poor	8,192,620	**8,672,098**	**8,688,652**	N/A	-9,754,196
Hi	Medium	Poor	8,062,671	8,542,149	**8,558,703**	N/A	-9,754,196
Hi	Medium	Fair	7,567,383	8,046,861	**8,063,415**	N/A	-9,754,196
Hi	Medium	Good	5,679,635	6,159,113	**6,175,667**	N/A	-9,754,196
Hi	Medium	Very Good	-1,505,150	-907,681	**-902,492**	N/A	-9,492,016
Hi	High	Very Poor	11,428,612	11,908,089	**11,924,643**	N/A	-13,005,595
Hi	High	Poor	11,255,347	11,734,824	**11,751,378**	N/A	-13,005,595
Hi	High	Fair	10,594,962	11,074,440	**11,101,436**	N/A	-13,005,595
Hi	High	Good	8,077,964	8,557,442	**8,573,996**	N/A	-13,005,595
Hi	High	Very Good	-1,515,354	-1,035,876	**-1,019,322**	N/A	-13,005,595
Hi	Very High	Very Poor	17,900,594	18,380,072	**18,396,626**	N/A	-19,508,392
Hi	Very High	Poor	17,640,697	18,120,175	**18,136,729**	N/A	-19,508,392
Hi	Very High	Fair	16,650,120	17,129,598	**17,146,152**	N/A	-19,508,392
Hi	Very High	Good	12,874,624	13,354,101	**13,370,655**	N/A	-19,508,392
Hi	Very High	Very Good	-1,515,354	-1,035,876	**-1,019,322**	N/A	-19,508,392

Table 7.8: Net Present Worth (EAUC) Results (15 years)

Case Scenario			Preservation Strategy				
Traffic	**Number of Collisions**	**Level of Friction**	**Mill and AC Overlay ($)**	**AC Overlay ($)**	**Recycled AC Overlay ($)**	**Micro Surface ($)**	**Do Nothing ($)**
Low	Very Low	Very Poor	2,607,378	**3,513,939**	3,388,881	2,885,080	-4,862,293
Low	Very Low	Poor	2,542,601	**3,449,162**	3,324,104	2,120,545	-4,862,293
Low	Very Low	Fair	2,295,709	**3,202,269**	3,077,211	2,735,767	-4,862,293
Low	Very Low	Good	608,522	1,469,968	1,404,460	1,672,142	-3,831,132
Low	Very Low	Very Good	-2,231,874	-1,325,314	-1,450,372	567,111	-4,862,293
Low	Low	Very Poor	7,446,631	**8,353,191**	8,228,133	7,736,335	-9,724,586
Low	Low	Poor	7,446,631	**8,223,637**	8,098,579	7,674,269	-9,724,586
Low	Low	Fair	6,823,292	**7,729,852**	7,604,794	7,437,709	-9,724,586
Low	Low	Good	4,941,274	5,847,834	5,722,776	**6,536,083**	-9,724,586
Low	Low	Very Good	-2,231,874	-1,325,314	-1,450,372	**3,099,614**	-9,724,586
Low	Medium	Very Poor	12,285,883	**13,192,443**	13,067,386	12,587,590	-14,586,880
Low	Medium	Poor	12,105,365	**12,998,112**	12,873,054	12,494,491	-14,586,880
Low	Medium	Fair	11,350,874	**12,257,435**	12,132,377	12,139,651	-14,586,880
Low	Medium	Good	8,527,848	9,434,408	9,309,350	**10,787,212**	-14,586,880
Low	Medium	Very Good	-2,231,874	-1,323,511	-1,294,170	**5,587,066**	-14,389,506
Low	High	Very Poor	17,125,136	**18,031,696**	17,906,638	17,438,845	-19,449,173
Low	High	Poor	16,866,027	**17,772,587**	17,647,530	17,314,713	-19,449,173
Low	High	Fair	15,878,457	16,785,018	16,659,960	**16,841,594**	-19,449,173
Low	High	Good	12,114,421	13,020,982	12,895,924	**15,038,341**	-19,449,173
Low	High	Very Good	-2,231,874	-1,325,314	-1,450,372	**8,165,403**	-15,324,527
Low	Very High	Very Poor	26,803,640	**27,710,201**	27,585,143	27,141,355	-29,173,759
Low	Very High	Poor	26,414,978	**27,321,538**	27,196,480	26,955,156	-29,173,759
Low	Very High	Fair	24,933,623	25,840,183	25,715,126	**26,245,478**	-29,173,759
Low	Very High	Good	19,287,569	20,194,130	20,069,072	**23,540,600**	-29,173,759
Low	Very High	Very Good	-2,231,874	-1,325,314	-1,450,372	**-159,116**	-29,173,759
Hi	Very Low	Very Poor	2,573,122	3,290,155	**3,314,911**	N/A	-4,862,293
Hi	Very Low	Poor	2,508,344	3,225,378	**3,250,134**	N/A	-4,862,293
Hi	Very Low	Fair	2,261,452	2,978,485	**3,003,241**	N/A	-4,862,293
Hi	Very Low	Good	1,346,429	2,037,476	**2,062,232**	N/A	-4,862,293
Hi	Very Low	Very Good	-2,266,131	-1,549,097	**-1,553,215**	N/A	-4,862,293
Hi	Low	Very Poor	7,412,374	8,129,407	**8,154,163**	N/A	-9,724,586
Hi	Low	Poor	7,412,374	7,999,853	**8,024,609**	N/A	-9,724,586
Hi	Low	Fair	6,789,035	7,506,068	**7,530,824**	N/A	-9,724,586
Hi	Low	Good	4,907,017	5,624,050	**5,648,806**	N/A	-9,724,586
Hi	Low	Very Good	-2,266,131	-1,549,097	**-1,524,342**	N/A	-9,724,586
Hi	Medium	Very Poor	12,251,626	12,968,660	**12,993,416**	N/A	-14,586,880
Hi	Medium	Poor	12,057,295	12,774,328	**12,799,084**	N/A	-14,586,880
Hi	Medium	Fair	11,316,618	12,033,651	**12,058,407**	N/A	-14,586,880
Hi	Medium	Good	8,493,591	9,210,624	**9,235,380**	N/A	-14,586,880
Hi	Medium	Very Good	-2,281,746	-1,376,007	**-1,368,140**	N/A	-14,389,506
Hi	High	Very Poor	17,090,879	17,807,912	**17,832,668**	N/A	-19,449,173
Hi	High	Poor	16,831,770	17,548,804	**17,573,559**	N/A	-19,449,173
Hi	High	Fair	15,844,201	16,561,234	**16,601,605**	N/A	-19,449,173
Hi	High	Good	12,080,165	12,797,198	**12,821,954**	N/A	-19,449,173
Hi	High	Very Good	-2,266,131	-1,549,097	**-1,524,342**	N/A	-19,449,173
Hi	Very High	Very Poor	26,769,384	27,486,417	**27,511,173**	N/A	-29,173,759
Hi	Very High	Poor	26,380,721	27,097,754	**27,122,510**	N/A	-29,173,759
Hi	Very High	Fair	24,899,366	25,616,400	**25,641,156**	N/A	-29,173,759
Hi	Very High	Good	19,253,312	19,970,346	**19,995,102**	N/A	-29,173,759
Hi	Very High	Very Good	-2,266,131	-1,549,097	**-1,524,342**	N/A	-29,173,759

Table 7.9: Most Cost Effective Strategy based on the LCCA (7 years)

Case Scenario			Most Cost Effective Strategy	
Traffic	**Number of Collisions**	**Level of Friction**	**Strategy**	**Cost ($)**
Low	Very Low	Very Poor	AC Overlay	669,770
Low	Very Low	Poor	AC Overlay	653,887
Low	Very Low	Fair	Micro Surface	610,475
Low	Very Low	Good	Micro Surface	499,933
Low	Very Low	Very Good	Micro Surface	78,709
Low	Low	Very Poor	AC Overlay	1,856,380
Low	Low	Poor	AC Overlay	1,824,267
Low	Low	Fair	Micro Surface	1,763,416
Low	Low	Good	Micro Surface	1,542,332
Low	Low	Very Good	Micro Surface	699,692
Low	Medium	Very Poor	AC Overlay	3,042,991
Low	Medium	Poor	Micro Surface	3,003,365
Low	Medium	Fair	Micro Surface	2,916,357
Low	Medium	Good	Micro Surface	2,454,749
Low	Medium	Very Good	Micro Surface	1,320,771
Low	High	Very Poor	AC Overlay	4,229,601
Low	High	Poor	Micro Surface	4,185,309
Low	High	Fair	Micro Surface	4,069,298
Low	High	Good	Micro Surface	3,627,131
Low	High	Very Good	Micro Surface	1,941,850
Low	Very High	Very Poor	AC Overlay	6,602,821
Low	Very High	Poor	Micro Surface	6,549,197
Low	Very High	Fair	Micro Surface	6,375,180
Low	Very High	Good	Micro Surface	5,711,929
Low	Very High	Very Good	Micro Surface	-99,366
High	Very Low	Very Poor	Recycled AC Overlay	1,572,146
High	Very Low	Poor	Recycled AC Overlay	569,601
High	Very Low	Fair	AC Overlay	507,766
High	Very Low	Good	AC Overlay	277,025
High	Very Low	Very Good	AC Overlay	-602,422
High	Low	Very Poor	Recycled AC Overlay	1,781,840
High	Low	Poor	AC Overlay	1,738,339
High	Low	Fair	AC Overlay	1,617,953
High	Low	Good	AC Overlay	1,173,779
High	Low	Very Good	AC Overlay	-602,422
High	Medium	Very Poor	Recycled AC Overlay	2,964,455
High	Medium	Poor	AC Overlay	2,909,758
High	Medium	Fair	AC Overlay	2,728,140
High	Medium	Good	AC Overlay	2,035,919
High	Medium	Very Good	AC Overlay	-602,422
High	High	Very Poor	Recycled AC Overlay	4,145,685
High	High	Poor	AC Overlay	4,080,484
High	High	Fair	AC Overlay	3,838,327
High	High	Good	AC Overlay	2,916,031
High	High	Very Good	AC Overlay	-602,422
High	Very High	Very Poor	AC Overlay	6,517,239
High	Very High	Poor	AC Overlay	6,421,937
High	Very High	Fair	AC Overlay	6,058,701
High	Very High	Good	AC Overlay	4,674,259
High	Very High	Very Good	AC Overlay	-602,422

Table 7.10: Most Cost Effective Strategy based on the LCCA (10 years)

Case Scenario			Most Cost Effective Strategy	
Traffic	Number of Collisions	Level of Friction	Strategy	Cost ($)
Low	Very Low	Very Poor	AC Overlay	1,326,873
Low	Very Low	Poor	AC Overlay	1,299,981
Low	Very Low	Fair	AC Overlay	1,197,485
Low	Very Low	Good	Micro Surface	881,025
Low	Very Low	Very Good	Micro Surface	167,869
Low	Low	Very Poor	AC Overlay	3,335,870
Low	Low	Poor	AC Overlay	3,282,086
Low	Low	Fair	AC Overlay	3,077,093
Low	Low	Good	Micro Surface	2,645,864
Low	Low	Very Good	Micro Surface	1,219,227
Low	Medium	Very Poor	AC Overlay	5,344,866
Low	Medium	Poor	AC Overlay	5,264,191
Low	Medium	Fair	Micro Surface	4,972,164
Low	Medium	Good	Micro Surface	4,410,704
Low	Medium	Very Good	Micro Surface	2,250,739
Low	High	Very Poor	AC Overlay	7,353,863
Low	High	Poor	AC Overlay	7,246,295
Low	High	Fair	Micro Surface	6,924,157
Low	High	Good	Micro Surface	6,175,544
Low	High	Very Good	Micro Surface	3,322,270
Low	Very High	Very Poor	AC Overlay	11,371,857
Low	Very High	Poor	AC Overlay	11,210,505
Low	Very High	Fair	Micro Surface	10,828,142
Low	Very High	Good	Micro Surface	9,705,223
Low	Very High	Very Good	Micro Surface	-133,621
High	Very Low	Very Poor	Recy	1,237,513
High	Very Low	Poor	Recy	1,210,621
High	Very Low	Fair	Recy	1,108,125
High	Very Low	Good	Recy	717,469
High	Very Low	Very Good	Recy	-771,484
High	Low	Very Poor	Recy	3,246,510
High	Low	Poor	Recy	3,192,726
High	Low	Fair	Recy	2,987,733
High	Low	Good	Recy	2,206,421
High	Low	Very Good	Recy	-771,484
High	Medium	Very Poor	Recy	5,255,507
High	Medium	Poor	Recy	5,174,831
High	Medium	Fair	Recy	4,867,341
High	Medium	Good	Recy	3,695,373
High	Medium	Very Good	Recy	-682,803
High	High	Very Poor	Recy	7,264,503
High	High	Poor	Recy	7,156,935
High	High	Fair	Recy	6,746,950
High	High	Good	Recy	5,184,325
High	High	Very Good	Recy	-771,484
High	Very High	Very Poor	Recy	11,282,497
High	Very High	Poor	Recy	11,121,145
High	Very High	Fair	Recy	10,506,166
High	Very High	Good	Recy	8,162,229
High	Very High	Very Good	Recy	-771,484

Table 7.11: Most Cost Effective Strategy based on the LCCA (15 years)

Case Scenario			Most Cost Effective Strategy	
Traffic	**Number of Collisions**	**Level of Friction**	**Strategy**	**Cost ($)**
Low	Very Low	Very Poor	AC Overlay	2,349,759
Low	Very Low	Poor	AC Overlay	2,306,442
Low	Very Low	Fair	AC Overlay	2,141,346
Low	Very Low	Good	Micro Surface	1,118,155
Low	Very Low	Very Good	Micro Surface	379,225
Low	Low	Very Poor	AC Overlay	5,585,750
Low	Low	Poor	AC Overlay	5,499,118
Low	Low	Fair	AC Overlay	5,168,925
Low	Low	Good	Micro Surface	4,370,656
Low	Low	Very Good	Micro Surface	2,072,701
Low	Medium	Very Poor	AC Overlay	8,821,741
Low	Medium	Poor	AC Overlay	8,691,793
Low	Medium	Fair	AC Overlay	8,196,504
Low	Medium	Good	Micro Surface	7,213,372
Low	Medium	Very Good	Micro Surface	3,685,500
Low	High	Very Poor	AC Overlay	12,057,733
Low	High	Poor	AC Overlay	11,884,468
Low	High	Fair	Micro Surface	11,261,915
Low	High	Good	Micro Surface	10,056,087
Low	High	Very Good	Micro Surface	5,460,177
Low	Very High	Very Poor	AC Overlay	18,529,716
Low	Very High	Poor	AC Overlay	18,269,818
Low	Very High	Fair	Micro Surface	17,550,260
Low	Very High	Good	Micro Surface	15,741,518
Low	Very High	Very Good	Micro Surface	-106,400
High	Very Low	Very Poor	Recy	3,314,911
High	Very Low	Poor	Recy	3,250,134
High	Very Low	Fair	Recy	3,003,241
High	Very Low	Good	Recy	2,062,232
High	Very Low	Very Good	Recy	-1,549,097
High	Low	Very Poor	Recy	8,154,163
High	Low	Poor	Recy	8,024,609
High	Low	Fair	Recy	7,530,824
High	Low	Good	Recy	5,648,806
High	Low	Very Good	Recy	-1,524,342
High	Medium	Very Poor	Recy	12,993,416
High	Medium	Poor	Recy	12,799,084
High	Medium	Fair	Recy	12,058,407
High	Medium	Good	Recy	9,235,380
High	Medium	Very Good	Recy	-1,368,140
High	High	Very Poor	Recy	17,832,668
High	High	Poor	Recy	17,573,559
High	High	Fair	Recy	16,601,605
High	High	Good	Recy	12,821,954
High	High	Very Good	Recy	-1,524,342
High	Very High	Very Poor	Recy	27,511,173
High	Very High	Poor	Recy	27,122,510
High	Very High	Fair	Recy	25,641,156
High	Very High	Good	Recy	19,995,102
High	Very High	Very Good	Recy	-1,524,342

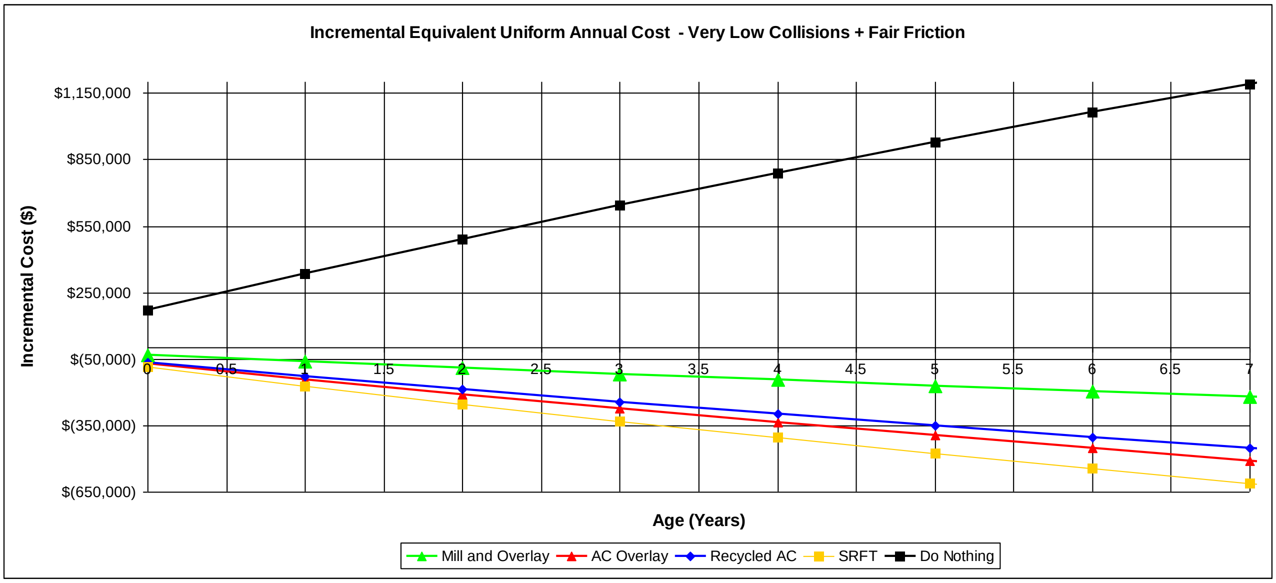

Figure 7.1: Incremental EAUC – Low Traffic/Very Low Collisions/Fair Friction

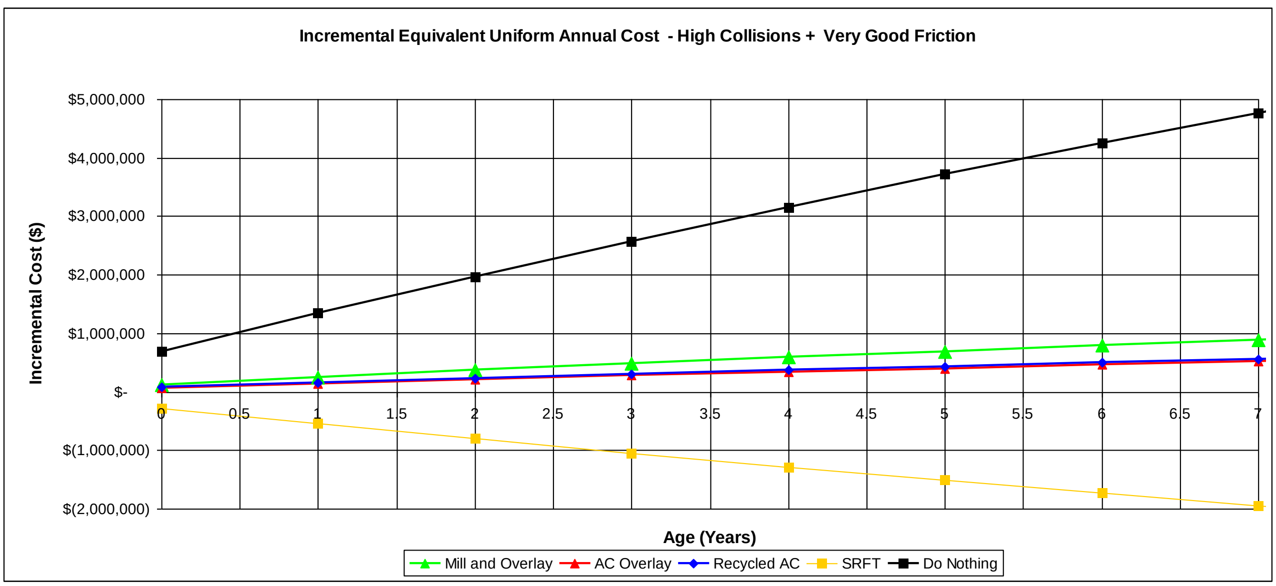

Figure 7.2: Incremental EAUC – Low Traffic/High Collisions/Very Good Friction

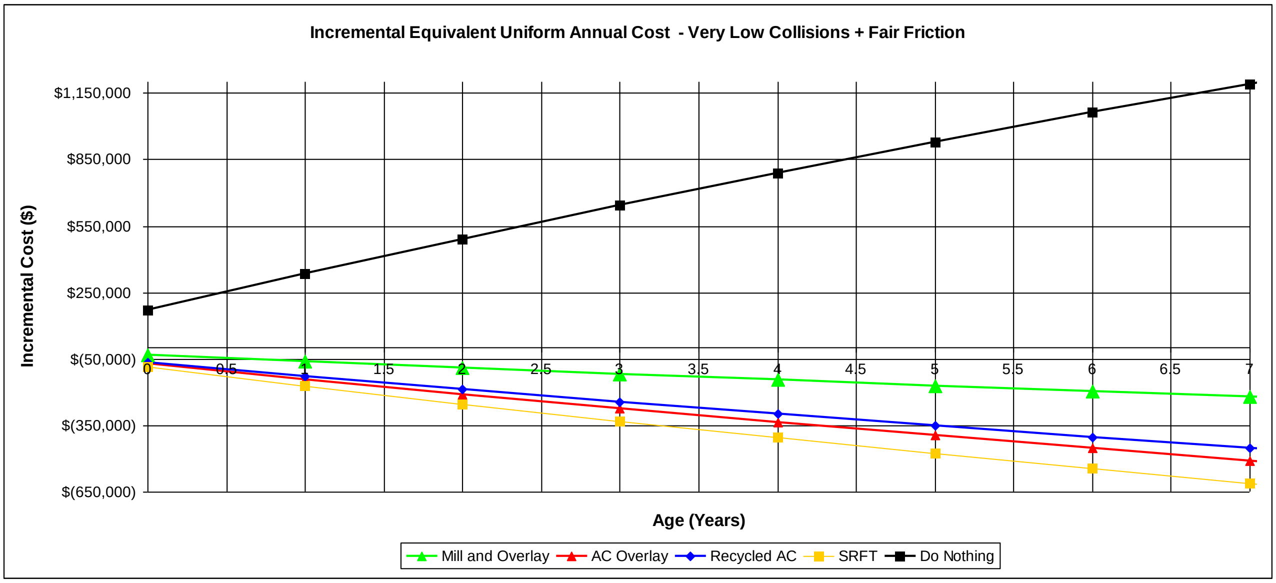

Figure 7.3: Incremental EAUC – High Traffic/Very Low Collisions/Fair Friction

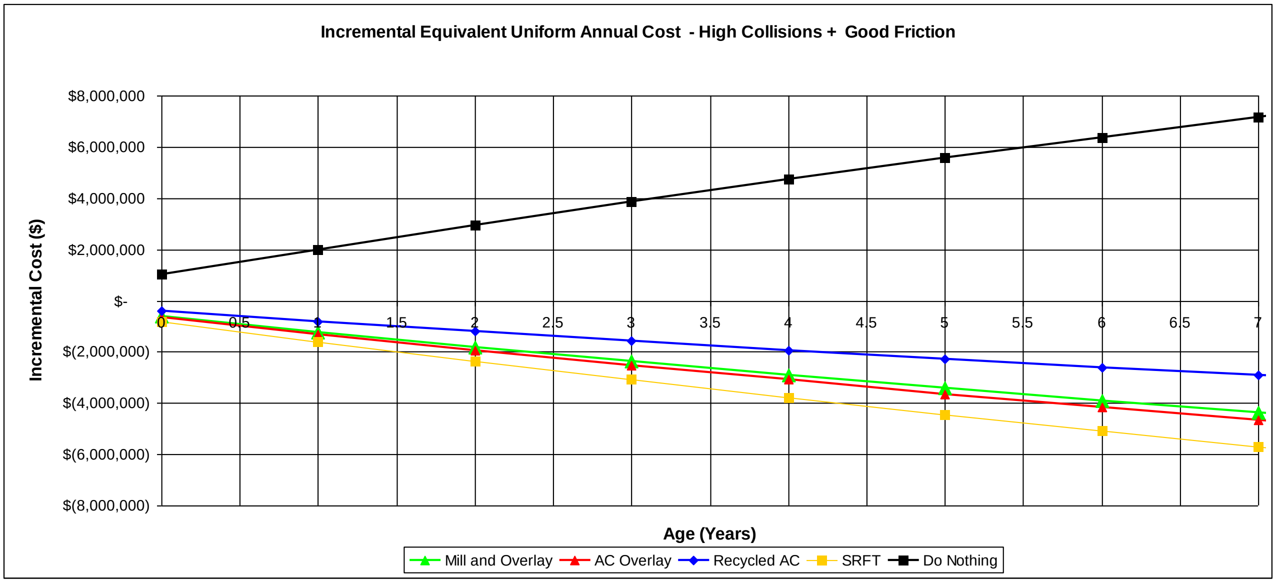

Figure 7.4: Incremental EAUC – High Traffic/High Collisions/Good Friction

Table 7.12: Final Decision Matrix Based on LCCA (7 Years)

Traffic Level	Number of Collisions	Level of Pavement Surface Friction				
		Very Poor	Poor	Fair	Good	Very Good
Low Traffic Volumes	Very Low	AC Ovly	AC Ovly	Micro Surface	Micro Surface	Micro Surface
	Low	AC Ovly	AC Ovly	Micro Surface	Micro Surface	Micro Surface
	Medium	AC Ovly	Micro Surface	Micro Surface	Micro Surface	Micro Surface
	High	AC Ovly	Micro Surface	Micro Surface	Micro Surface	Micro Surface
	Very High	AC Ovly	Micro Surface	Micro Surface	Micro Surface	Micro Surface
High Traffic Volumes	Very Low	Recycled AC Ovly	Recycled AC Ovly	AC Ovly	AC Ovly	AC Ovly
	Low	Recycled AC Ovly	AC Ovly	AC Ovly	AC Ovly	AC Ovly
	Medium	Recycled AC Ovly	AC Ovly	AC Ovly	AC Ovly	AC Ovly
	High	Recycled AC Ovly	AC Ovly	AC Ovly	AC Ovly	AC Ovly
	Very High	AC Ovly	AC Ovly	AC Ovly	AC Ovly	AC Ovly

Table 7.13: Final Decision Matrix Based on LCCA (10 Years)

Traffic Level	Number of Collisions	Level of Pavement Surface Friction				
		Very Poor	Poor	Fair	Good	Very Good
Low Traffic Volumes	Very Low	AC Ovly	AC Ovly	AC Ovly	Micro Surface	Micro Surface
	Low	AC Ovly	AC Ovly	AC Ovly	Micro Surface	Micro Surface
	Medium	AC Ovly	AC Ovly	Micro Surface	Micro Surface	Micro Surface
	High	AC Ovly	AC Ovly	Micro Surface	Micro Surface	Micro Surface
	Very High	AC Ovly	AC Ovly	Micro Surface	Micro Surface	Micro Surface
High Traffic Volumes	Very Low	Recycled AC Ovly	Recycled AC Ovly	Recycled AC Ovly	Recycled AC Ovly	Recycled AC Ovly
	Low	Recycled AC Ovly	Recycled AC Ovly	Recycled AC Ovly	Recycled AC Ovly	Recycled AC Ovly
	Medium	Recycled AC Ovly	Recycled AC Ovly	Recycled AC Ovly	Recycled AC Ovly	Recycled AC Ovly
	High	Recycled AC Ovly	Recycled AC Ovly	Recycled AC Ovly	Recycled AC Ovly	Recycled AC Ovly
	Very High	Recycled AC Ovly	Recycled AC Ovly	Recycled AC Ovly	Recycled AC Ovly	Recycled AC Ovly

Table 7.14: Final Decision Matrix Based on LCCA (15 Years)

Traffic Level	Number of Collisions	Level of Pavement Surface Friction				
		Very Poor	**Poor**	**Fair**	**Good**	**Very Good**
Low Traffic Volumes	Very Low	AC Ovly	AC Ovly	AC Ovly	Micro Surface	Micro Surface
	Low	AC Ovly	AC Ovly	AC Ovly	Micro Surface	Micro Surface
	Medium	AC Ovly	AC Ovly	AC Ovly	Micro Surface	Micro Surface
	High	AC Ovly	AC Ovly	Micro Surface	Micro Surface	Micro Surface
	Very High	AC Ovly	AC Ovly	Micro Surface	Micro Surface	Micro Surface
High Traffic Volumes	Very Low	Recycled AC Ovly	Recycled AC Ovly	Recycled AC Ovly	Recycled AC Ovly	Recycled AC Ovly
	Low	Recycled AC Ovly	Recycled AC Ovly	Recycled AC Ovly	Recycled AC Ovly	Recycled AC Ovly
	Medium	AC Ovly	Recycled AC Ovly	Recycled AC Ovly	Recycled AC Ovly	Recycled AC Ovly
	High	Recycled AC Ovly	Recycled AC Ovly	Recycled AC Ovly	Recycled AC Ovly	Recycled AC Ovly
	Very High	Recycled AC Ovly	Recycled AC Ovly	Recycled AC Ovly	Recycled AC Ovly	Recycled AC Ovly

7.5 Sensitivity Analysis – Discount Rate

A discount rate of 4% was utilized for the Life Cycle Cost Analysis computations. To examine the impact of a change in the discount rate on the various outcomes of the preservation strategies, sensitivity analysis was completed. The LCCA analysis was repeated for a discount rate of 6% and 8% to compare to the base case of 4%. The results illustrated that a change in the discount rate had no significant impact on the selection of the preservation treatment.

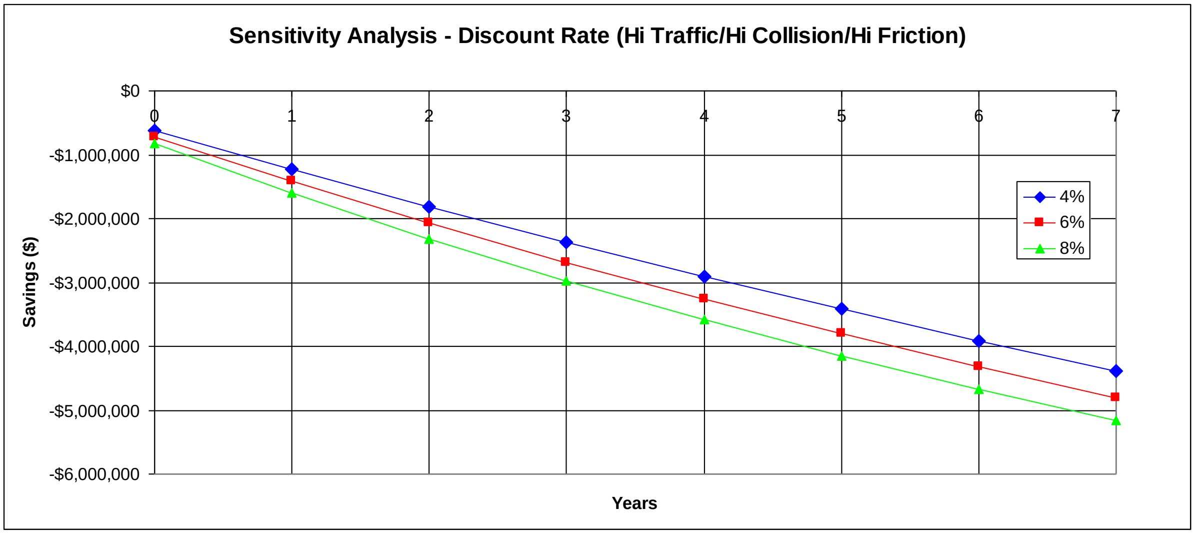

Figure 7.5: Sensitivity Analysis – Mill and Overlay Treatment

7.6 Summary

The results of the LCCA demonstrate that preservation and maintenance can be used to increase the level of safety of a highway and generate savings in terms of a reduction in the number of collisions. The final Decision Matrix based on the results of the LCCA is presented in Tables 7.12 to 7.14. As was described in detail, the matrix is comprised of 50 cells or combinations of the following parameters; traffic levels, total number of collisions, surface friction. The treatments presented within each cell were the most cost-effective over the 7, 10 and 15 year life cycles based on the LCCA. Sample LCCA calculations are presented in Appendix C.

It is worth noting that the existing pavement condition in terms of roughness and distress should also be considered when selecting treatments. As an example, a highway section with poor ride quality and cracking should be milled prior to the placement of the new asphalt concrete overlay. Even though an Asphalt Concrete Overlay might be the most cost-effective treatment, the Mill and AC Overlay should be selected.

Chapter 8
Optimization of Required Tests for Network Level Skid Testing

The study presented within this Chapter demonstrates a method that can be used to minimize the number of required skid test locations along a highway segment using common statistical techniques. It is also very timely in light of Public Private Partnerships (PPP) where friction testing at the network level will become more commonplace.

8.1 Framework Approach

The purpose of this study is to develop an approach or framework to determine the minimum test interval requirements for network level skid testing using a locked-wheel skid tester. As a part of this study, a significant data collection program consisting of network level skid data and highway attribute data was undertaken. The framework approach is based on common statistical techniques which were evaluated at the 95% confidence interval.

The following sections provide background information on the data and analysis methodology. Three levels of analysis were examined;

- Network Level,
- Highway Level, and;
- Group Level Analysis

The three levels of analysis were performed to illustrate the effects of increasing the test interval requirements; the next sections outline the data attributes, statistical approach and results of analysis.

8.2 Data Attributes

As was described in detail in Chapter 3, the data from this study was obtained from a highway network located in Ontario. Each data element was checked for completeness,; Quality Control and Assurance and formatted prior to analysis.

8.2.1 The Highway Network

In 2006, approximately 1,800 km of highway network was surveyed across three regions in Ontario. This data was collected across 33 individual highway segments consisting of an assortment of functional classes (2 lane undivided, 4 lane divided, etc.).

8.2.2 Surface Friction

As was previously mentioned, friction data was collected to determine a baseline of the current network friction levels in terms of a skid number. A trailer mounted locked-wheel skid tester was used to collect the skid data in terms of a skid number (SN). A 1.0 km sampling interval was selected as a reasonable measuring interval to perform the skid testing. At each test point, the skid number (SN), average test vehicle speed, and kilometre post were recorded.

8.3 Statistical Approach

To determine the minimum skid testing interval requirements, an analysis approach consisting of common statistical techniques was developed. The first step was to examine the descriptive or summary statistics of the skid data collected within each region. Next, tests for goodness of fit for normal distribution analysis were performed. Finally, a comparison of means was conducted using a Student's T-test and an Analysis of Variance (ANOVA) to evaluate the differences between groups at the 95% confidence interval. These tests are used to determine if the testing interval could be increased. A similar analysis approach was carried out to examine the testing interval requirements for network level Falling Weight Deflectometer (FWD) testing along a number of interstates in the

State of Virginia (Alam et al., 2007). The Virginia research study demonstrated that the FWD testing interval could be increased (number of tests reduced) using statistical techniques.

8.3.1 Descriptive Statistics and Normality Tests

A detailed analysis examining the descriptive statistics is performed to determine the variability and distribution of the collected skid data. The mean, standard deviation and coefficient of variation as well as maximum, minimum and range were calculated for the skid data collected within each Region. In addition, skewness and kurtosis tests of the skid data for each region were also performed. The skewness and kurtosis tests are used to determine if the collected skid data followed a normal distribution.

As was previously mentioned, Regions A and B have similar friction properties and similar distributions and were combined into a single region (Region A&B). The third region is located within the Canadian Shield, which is well known for its very hard rock formations, the aggregate sources in this region have excellent skid resistance properties. This region was treated as a single region (Region C). Presented in Table 8.1 are descriptive statistics for Regions A&B and C.

Table 8.1: Summary SN Statistics for Regions A&B and C

Summary Statistic	Region A&B	Region C
Mean	37.6	52
Standard Deviation	6.5	5.5
Coefficient of Variation	0.17	0.11
Kurtosis	0.1	1.74
Skewness	0.3	0.657
Range	54.6	46.3
Minimum	10.3	22.3
Maximum	64.9	68.6
N	905	869

8.3.2 Comparison of Means

Statistically, any parameter can be estimated from a data set either by a point estimate or a confidence interval. Frequently, however, the objective of an investigation is not to estimate a parameter but to decide which two contradictory statements about a parameter is correct. The outcome of these tests is the acceptance or rejection of the null hypothesis (H_0). For the skid data, the objective was to determine if the number of test locations across a highway network or segment could be reduced. For this case, the Null Hypothesis is that the means of the two data sets are equal; while the alternate hypothesis is that the means of the two data sets are statistically different.

This test was performed to determine if a statistical difference was present between the various alternatives. If the data sets are found to be equal at the 95% confidence interval, then we can conclude that both groups are the same and the number of skid tests can be reduced. For two groups, a Student's T-test was used to test for differences. For three or more groups, an Analysis of Variance (ANOVA) was performed.

8.4 Results and Analysis

As a part of this study, three levels of analysis were performed. The first level of analysis was conducted at the Network Level, which included all highway sections under study. The second level of analysis consisted of a Highway Level Analysis which was performed on three individual highway segments. The third level of analysis performed is a Group Level Analysis on 33 individual highway segments within the network.

8.4.1 Network Level Analysis

For the Network Level Analysis, skid data was examined across three regions in Ontario. The skid data was grouped into a number of subsets to see if the testing interval could be extended from the current 1-km interval up to an interval of 5 km. The grouping of skid data into subsets is presented in Figure 8.1 and Table 8.2.

Table 8.2: Skid Data Grouped Into Subsets Based on Testing Interval

Region	Highway	Distance (km)	2 km Interval	3 km Interval	4 km Interval	5 km Interval	Random 5 km Interval	Skid Number (SN)
Region A	1	1.324	1	1	1	1	2	44.3
Region A	1	2.324	2	2	2	2	1	45.8
Region A	1	3.324	1	3	3	3	4	45.2
Region A	1	4.324	2	1	4	4	5	44.8
Region A	1	5.324	1	2	1	5	3	43.3
Region A	1	6.324	2	3	2	1	3	44.5
Region A	1	7.324	1	1	3	2	5	49.8
Region A	1	8.324	2	2	4	3	2	10.3
Region A	1	9.324	1	3	1	4	1	50.8
Region A	1	10.324	2	1	2	5	4	47.0

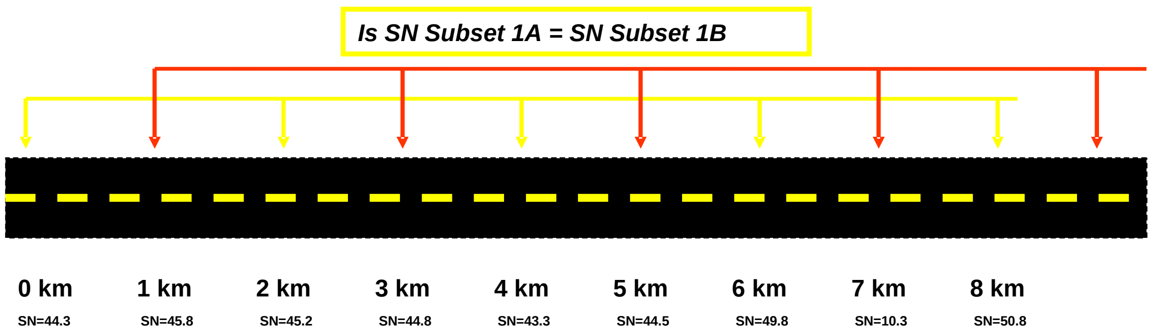

Figure 8.1: Comparison of Subsets

8.4.1.1 Case 1: Extend Test Interval from 1 km to 2 km

To investigate whether the testing interval could be extended from 1 km to 2 km, two subsets of data were compared. The first subset included data from every even kilometre post (Subset 1A → 0, 2, 4, 6, etc.) for each highway segment. The second subset included data from every odd kilometre post (Subset 1B → 1, 3, 5, 7, etc.) for each highway segment. This comparison was performed for all highway segments within the three Regions. To determine if there were any statistical differences between the two subsets, a test of "comparison of means by a two sample t-test with 95% level of confidence" was performed. The two subsets were compared to the population (all regions) as well as to each other. For all scenarios, Subsets 1A and 1B were not found to be statistically different at the 95% confidence interval indicating that the two subsets are similar and that the testing interval can be extended to 2 km. Results of the Student's T-test for Case 1 are presented in Table 8.3. A paired Student's T-test was also performed to determine if there were any statistical differences between Subset 1A and Subset 1B. The results of paired T-test were not found to be statistically different at the 95% confidence level indicating that the two subsets are similar.

Table 8.3: Results of T-Test (Two Sample Assuming Unequal Variances) for 2 km Test Interval

Results of T-Test	Subset 1A	Subset 1B
Mean	44.6	44.7
Variance	86.4	89.6
Observations	897	877
Hypothesized Mean Difference	0	
df	1769	
t Stat	-0.0180	
P(T<=t) one-tail	0.49	
t Critical one-tail	1.65	
P(T<=t) two-tail	0.99	
t Critical two-tail	1.96	

8.4.1.2 Case 2: Extend Test Interval from 2 km to 3 km

To investigate whether the testing interval could be extended from 2 km to 3 km, three subsets of data were compared. The first subset includes data starting at the first test location within the highway segment and every 3 km thereafter (Subset 2A → 0, 3, 6, etc.). The second subset includes data starting at the second test point within the highway segment and every 3 km thereafter (Subset 2B → 1, 4, 7, etc.). The third subset includes data starting from the third test point within the highway segment and every 3 km thereafter (Subset 2C → 2, 5, 8, etc.).

This comparison was performed for all highway segments within the three Regions. To determine if there were any statistical differences between the three subsets, an Analysis of Variance (ANOVA) was performed. Results of the ANOVA indicate that the three subsets are not statistically different at the 95% confidence interval indicating that the three subsets are similar and that the testing interval can be extended to 3 km. Results of the ANOVA for Case 2 are presented in Table 8.4.

8.4.1.3 Case 3: Extend Test Interval from 3 km to 5 km

To investigate whether the testing interval could be extended from 3 km to 5 km, five subsets of data were compared. The first subset includes data starting at the first test location within the highway segment and every 5 km thereafter (Subset 3A → 0, 5, 10, etc.). The second subset includes data starting at the second test point within the highway segment and every 5 km thereafter (Subset 3B → 1, 6, 11, etc.). The third subset includes data starting from the third test point within the highway segment and every 5 km thereafter (Subset 3C → 2, 7, 12, etc.). The fourth subset includes data starting from the fourth test point within the highway segment and every 5 km thereafter (Subset 3D → 3, 8, 13, etc.). The fifth subset includes data starting from the fifth test point within the highway segment and every 5 km thereafter (Subset 3E → 4, 9, 14, etc.).

This comparison was performed for all highway segments within the three Regions. To determine if there were any statistical differences between the five subsets, an Analysis of Variance (ANOVA) was performed. Results of the ANOVA indicate that the three subsets are not statistically different at the 95% confidence interval indicating that the five subsets are similar and that the testing interval can be extended to 5 km. Results of the ANOVA for Case 3 are presented in Table 8.4.

8.4.1.4 Case 4 – Extend Skid Test Interval Based on Random Testing

The results from the three previous cases are based on grouped subsets with a known or regular interval (i.e. even and odd test points). This may result in each subset having distributions of data which are similar to the original data set since it contains all subsets within its population. Therefore, to ensure a sound approach in conducting the Analysis of Variance (ANOVA) between the various subsets, a final analysis was performed where the grouped subsets were developed using a random number generator. This ensures that a friction test could be performed anywhere within each 5 km interval along the highway segment and still produce statistically significant results.

The average level of SN for each of the 5 randomly generated subsets (1A through 1E) is 36.6, 36.4, 36.54, 37.2, and 36.9. To determine if there were any statistical differences between the five subsets, an Analysis of Variance (ANOVA) was performed. Results of the ANOVA indicate that the five subsets are not statistically different at the 95% confidence interval indicating that the 5 subsets are similar and that the testing interval can be extended to 5 km. Results of the ANOVA for Case 4 are presented in Table 8.4.

Table 8.4: Results of ANOVA for Network Level Analysis

RESULTS OF ANALYSIS OF VARIANCE FOR CASE 2 – 3 KM INTERVAL						
Source of Variation	**SS**	**df**	**MS**	**F**	**P-value**	**F crit**
Between Groups	42.342	2	21.171	0.241	0.786	3.001
Within Groups	155862.033	1771	88.008			
RESULTS OF ANALYSIS OF VARIANCE FOR CASE 3 – 5 KM INTERVAL						
Source of Variation	**SS**	**df**	**MS**	**F**	**P-value**	**F crit**
Between Groups	62.986	4	15.746	0.178741	0.949	2.377
Within Groups	155841.4	1769	88.096			
RESULTS OF ANALYSIS OF VARIANCE FOR CASE 4 – 5 KM RANDOM INTERVAL						
Source of Variation	**SS**	**df**	**MS**	**F**	**P-value**	**F crit**
Between Groups	172.409	4	43.102	0.490	0.743	2.377
Within Groups	155732	1769	88.034			

8.4.1.5 Summary of Network Level Analysis

Based on the results of the Student's t-test and Analysis of Variance (ANOVAs), the skid testing interval can be extended from one km up to five km since there is no significant difference (at the 95% confidence interval) within each of the subsets for all cases. The testing interval was not increased beyond five km since testing beyond this distance may not provide a meaningful representation of the friction properties along a highway section or network. Since the level of friction is a function of the material properties and characteristics of the pavement surface layer, increasing the distance beyond five km may increase the variability in the skid data across a highway segment due to the variations in asphalt mixes or aggregates being used along a section of highway or project.

8.4.2 Highway Level Analysis

The results of the Network Level Analysis demonstrated that the skid testing interval could be increased from one km to five km and statistically provide the same level of friction at the 95% confidence interval. A Highway Level Analysis was performed to test if extending the testing interval affected the significance of the results along three individual high segments. Due to the sensitivity of the skid data, the three highways are referred to as Highway I, Highway II, and Highway III within this study.

Similar to the Network Level Analysis, skid data was grouped into five subsets. The first subset includes data starting at the first test location along each highway and every five km thereafter (Subset 1A → 0, 5, 10, etc.). The second subset includes data starting at the second test point along each highway and every 5 km thereafter (Subset 1B → 1, 6, 11, etc.). The third subset includes data starting from the third test point along each highway and every five km thereafter (Subset 1C → 2, 7, 12, etc.). The fourth subset includes data starting from the fourth test point along each highway and every five km thereafter (Subset 1D → 3, 8, 13, etc.). The fifth subset includes data starting from the fifth test point along each highway and every five km thereafter (Subset 1E → 4, 9, 14, etc.).

8.4.2.1 Case 1 - Extend Skid Testing Interval from 1 km to 5 km along Highway I

Highway I is located within Region A in the province of Ontario. It is generally a two lane undivided highway. In total, approximately 200 km of pavement were tested along Highway I. Since testing was conducted every one km, 203 individual skid tests were performed along the length of this highway segment. Similar to the approach for the Network Level Analysis, skid data was grouped into five subsets. The average level of SN for each of the five subsets (1A through 1E) is 36.6, 36.4, 36.5, 37.2, and 36.9. To determine if there were any statistical differences between the five subsets, an Analysis of Variance (ANOVA) was performed. Results of the ANOVA indicate that the five subsets are not statistically different at the 95% confidence interval indicating that the three subsets are similar and that the testing interval can be extended to 5 km along Highway I. Results of the ANOVA for Case 1 are presented in Table 8.5.

8.4.2.2 Case 2 - Extend Skid Testing Interval from 1 km to 5 km along Highway II

Highway II is located within Region A in the province of Ontario. It is generally a two lane undivided highway. In total, approximately 160 km were tested along Highway II. Since testing was conducted every one km, 158 individual skid tests were performed along the length of this highway segment. Similar to the approach for the Network Level Analysis, skid data was grouped into five subsets. A comparison was then performed for the five subsets along Highway II. To determine if there were any statistical differences between the five subsets, an Analysis of Variance (ANOVA) was performed. Results of the ANOVA indicate that the five subsets are not statistically different at the 95% confidence interval indicating that the three subsets are similar and that the testing interval can be extended to five km along Highway II. Results of the ANOVA for Case 2 are presented in Table 8.5.

8.4.2.3 Case 3 - Extend Skid Testing Interval from 1 km to 5 km along Highway III

Highway III is located within Region B in the province of Ontario. It is generally a two lane undivided highway. In total, approximately 90 km were tested along Highway III. Since testing was conducted every one km, 87 individual skid tests were performed along the length of this highway segment. Similar to the approach for the Network Level Analysis, skid data was grouped into five subsets. A comparison was then performed for the five subsets along Highway III. To determine if there were any statistical differences between the five subsets, an Analysis of Variance (ANOVA) was performed. Results of the ANOVA indicate that the five subsets are not statistically different at the 95% confidence interval indicating that the three subsets are similar and that the testing interval can be extended to five km along Highway III. Results of the ANOVA for Case 3 are presented in Table 8.5.

8.4.2.4 Summary of Highway Level Analysis

Based on the results of the Analysis of Variance (ANOVAs), the skid testing interval can be extended from one km up to five km since there is no significant difference (at the 95% confidence interval) within each of the subsets for the three highway segments. This analysis demonstrates that reducing the skid testing interval along three independent highway sections results in similar friction levels to testing at an interval of one km. These results support the findings of the Network Level Analysis.

Table 8.5: Results of ANOVA for Highway Level Analysis

RESULTS OF ANALYSIS OF VARIANCE FOR HIGHWAY I						
Source of Variation	**SS**	**df**	**MS**	**F**	**P-value**	**F crit**
Between Groups	38.021	4	9.505	0.547	0.701	2.431
Within Groups	2656.837	153	17.365			
RESULTS OF ANALYSIS OF VARIANCE FOR HIGHWAY II						
Source of Variation	**SS**	**df**	**MS**	**F**	**P-value**	**F crit**
Between Groups	38.021	4	9.505	0.547	0.701	2.431
Within Groups	2656.837	153	17.365			
RESULTS OF ANALYSIS OF VARIANCE FOR HIGHWAY III						
Source of Variation	**SS**	**df**	**MS**	**F**	**P-value**	**F crit**
Between Groups	116.616	4	29.154	0.828	0.511	2.483
Within Groups	2885.78	82	35.192			

8.4.3 Group Level Analysis

For the Group Level Analysis, skid number values were categorized into three classes based on the following criteria;

- Poor: SN <31.5
- Fair: 31.5<=SN<=35
- Good: SN >=35.0

These levels were established based on a review of the TAC pavement design and management guide and using engineering judgement. A similar analysis approach to the previous two levels was performed to examine if the skid testing interval could be extended to five km for all highway segments. Skid data was grouped into the five subsets. A rating of Poor (1), Fair (2) or Good (3) was assigned to each skid number value. The group average was then calculated for each of the five subsets for all highways. Since the results of the network and highway level analysis showed that all subsets were not significantly different at the 95% confidence interval, it was expected that the group level analysis should produce similar results.

For this level of analysis, the rating of each subset was expected to be equal to the overall "subjective" rating of the highway segment. Presented in Table 8.6 are a summary of each highway and the subjective rating of each subset and the overall highway. As can be seen in Table 8.6, for the 33 highway segments, 7 are categorized as "Fair" and 26 are categorized as "Good". In addition, only two of the highway segments (Highway 10 and 27) have the overall condition differing from one or more of the five subsets. The results of this analysis show that reducing the number of required skid tests does not result in an over-or under estimation of the friction levels along a highway segment.

Table 8.6 : Group Level Analysis – Subjective Rating

Region	Highway	Subjective Rating					
		Subset 1	Subset 2	Subset 3	Subset 4	Subset 5	Overall
A	1	Fair	Fair	Fair	Fair	Fair	Fair
	2	Good	Good	Good	Good	Good	Good
	3	Good	Good	Good	Good	Good	Good
	4	Fair	Fair	Fair	Fair	Fair	Fair
	5	Good	Good	Good	Good	Good	Good
	6	Fair	Fair	Fair	Fair	Fair	Fair
	7	Fair	Fair	Fair	Fair	Fair	Fair
	8	Fair	Fair	Fair	Fair	Fair	Fair
	9	Good	Good	Good	Good	Good	Good
	10	Fair	Fair	Fair	Good	Good	Fair
B	11	Good	Good	Good	Good	Good	Good
	12	Good	Good	Good	Good	Good	Good
	13	Good	Good	Good	Good	Good	Good
	14	Good	Good	Good	Good	Good	Good
	15	Good	Good	Good	Good	Good	Good
	16	Good	Good	Good	Good	Good	Good
	17	Good	Good	Good	Good	Good	Good
	18	Good	Good	Good	Good	Good	Good
	19	Good	Good	Good	Good	Good	Good
	20	Good	Good	Good	Good	Good	Good
	21	Good	Good	Good	N/A	N/A	Good
	22	Good	Good	Good	Good	Good	Good
	23	Good	Good	Good	Good	Good	Good
	24	Good	Good	Good	Good	Good	Good
	25	Good	Good	Good	Good	Good	Good
	26	Good	Good	Good	Good	Good	Good
	27	Good	Poor	Fair	Good	Good	Fair
	28	Good	Good	Good	Good	Good	Good
	29	Good	Good	Good	Good	Good	Good
	30	Good	Good	Good	Good	Good	Good
	31	Good	Good	Good	Good	Good	Good
	32	Good	Good	Good	Good	Good	Good
	33	Good	Good	Good	Good	Good	Good

8.5 Benefits of Reducing Number of Skid Tests

The results of the analysis demonstrate that the skid testing interval for a locked wheel tester can be extended from one km up to five km in length without jeopardizing the accuracy of the results for network level skid testing. It is important to note that the results are based on the 1,800 km network that was surveyed and should not be applied to the rest of the Ontario Highway network or any other agency's network. All results were found to be statistically significant at the 95% confidence interval. By extending the skid testing interval, this results in a reduction in the number of required skid tests along a highway segment or network. This is a benefit to the transportation agencies, data collection providers, contractors, and most importantly the public.

8.5.1 Transportation Agency

Network level skid testing is an important tool that can be used to assess the level of safety of a highway network. Consultants and data collection providers typically collect data such as deflection or skid data by the test-point and charge agencies based on the number of tests performed. With a reduction in the total number of required test points, agencies are able to survey higher percentages of their networks more frequently. This savings will allow for resources to be allocated to project or detailed level skid testing at collision prone or "black spot" locations.

8.5.2 Data Collection Providers

The cost of skid testing is considerable in terms of equipment costs, operating and maintenance costs, staffing, mobilization, and training. If the number of required skid test points can be reduced, data collection providers can complete projects earlier and move on to their next assignments. This also reduces the wear-and-tear and depreciation of the skid testing equipment.

8.5.3 Contractors

As transportation agencies begin to shift the ownership and ultimate responsibility of their civil infrastructure assets such as highway networks to the private sector, contractors are going to be required to survey their networks on a regular basis. A reduction in the total number of tests is an obvious savings in costs and allows budgets to be spent on maintenance or other improvements.

8.6 Summary

The importance of network level testing to assess pavement performance is an essential component of any pavement management system. Skid testing is an important tool that can be used to assess the level of friction and safety of a highway network. It is important to note that the results of this research study should not be applied to other agencies highway networks. This is due to the obvious facts that the materials, traffic loadings, subgrade conditions, aggregate types, environmental conditions, etc. are all specific to this region of this study. Extrapolating the results from this study onto another agency's highway network may result in an over or under estimation of the network level friction levels. However, it is recommended that the analysis methodology or framework developed as a part of this research study be used to determine if the total number of skid test points can be reduced for another agency's highway network. This analysis should be carried out on a year-to-year basis prior to the testing cycle.

Chapter 9
Summary and Conclusions

9.1 Summary

Pavements encompass a significant component of the total civil infrastructure investment. In Ontario, the Ministry of Transportation (MTO) is responsible for the maintenance and construction of approximately 39,000 lane-kilometres of highway. In 2004, the province estimated the value of the total highway system at $39 billion dollars. Thus, managing this asset is an important factor to ensure a high level of service to the traveling public.

Highway safety is a major concern for Transportation Agencies across North America and around the world. The MTO estimated that in 2002, vehicle collisions in Ontario cost nearly $11 billion. It also estimated that for every dollar spent on traffic management, 10 times that amount could be saved on collision-related expenditures, including health care and insurance claims. Thus an effective management strategy or framework that considers both pavement performance and safety is critical.

In preparation for a considered Long Term Area Maintenance Contract, a project was initiated by the Ontario Ministry of Transportation (MTO) to collect network level friction data across three regions in the Province of Ontario. This project represents the first time friction data was collected at the network level in Ontario. In 2006, approximately 1,800 km of the MTO highway network was surveyed as a part of this study. This research utilized the network level skid data along with collision data to examine the relationships and model the impacts of skid resistance on the level of safety. Despite the value of collecting network level skid data, many Canadian transportation agencies still do not collect network level skid data due to the costs and potential liability associated with the collected data.

The safety of highway networks are usually assessed using various levels of service indicators such as Wet-to-Dry accident ratio (W/D), surface friction (SN), or the collision rate (CR). This research focused on developing a framework for assessing the level of safety of a highway network in terms of the risk of collision based on pavement surface friction. The developed safety framework can be used by transportation agencies (federal, state, provincial, municipal, etc.) or the private sector to evaluate the safety of their highway networks and to determine the risk or probability of a collision occurring given the level of friction along the pavement section of interest. As a part of the analysis, a number of factors such as Region, Season of the Year, Environmental Conditions, Road Surface Condition, Collision Severity, Visibility and Roadway Location were all investigated. Statistical analysis and modeling were performed to developed relationships which could relate the total number of collisions or the collision rate (CR) to the level of available pavement friction on a highway section. These models were developed using over 1,200 collisions and skid test results from two Regions in the province of Ontario. Another part of this study examined the Wet-to-Dry accident ratio and compared it to the Skid Number. A number of Transportation Agencies rely on the Wet-to-Dry accident ratio to identify potential locations with poor skid resistance. The results of the comparison further demonstrated the need and importance of collecting network level skid data.

Another component of this study was to evaluate the effectiveness of various preservation treatments used within the Long Term Pavement Performance study. In addition, modeling was performed which examined the historical friction trends over time within various environment zones across North America to investigate skid resistance deterioration trends. The results of the analysis demonstrated that commonly used treatments can increase skid resistance and improve safety.

The cost effectiveness of implementing preservation maintenance to increase the level of safety of a highway using Life Cycle Cost Analysis (LCCA) was evaluated. A Decision Making Framework was developed which included the formulation of a Decision Matrix that can be used to assist in selecting a preservation treatment for a given condition. The results of this analysis demonstrate the savings generated by reducing the number of collisions as a result of increasing skid resistance.

The results of this research study have demonstrated the importance of network level friction testing and the impact of skid resistance on the level of safety of a highway. A review of the literature did not reveal any protocol or procedures for sampling or minimum test interval requirements for network level skid testing using a locked wheel tester. Network level friction testing can be characterized as expensive and time-consuming due to the complexity of the test and the traffic control requirements. As a result, any reduction in the required number of test points is a benefit to the transportation agency, private sector (consultants and contractors) and most importantly, the public. An analysis approach was developed and tested that can be used to minimize the number of required test locations along a highway segment using common statistical techniques.

9.2 Conclusions

The main objectives of this research were to demonstrate the importance of network level skid testing for pavement and safety management and to develop a framework for assessing the level of safety of a highway network. The results of the statistical analyses showed a strong correlation between the level of available pavement surface friction in terms of the skid number (SN) and the total number of collisions or collision rate (CR) for a highway network. This indicates that increasing the skid resistance of a pavement will result in a reduction in the collision rate and an improvement in safety. Overall, an average 100% increase in the *CR* was observed when the skid number dropped from an

SN of 32 to 35 to a level below 32. This indicates that a pavement with a skid number ranging from 32 to 35 is at a critical state and corrective maintenance should be performed to improve skid resistance and the level of safety. This critical SN level was also reported by TAC (1997) and other transportation agencies. Furthermore, relying solely on the Wet-to-Dry (W/D) accident ratio may result in underestimating the number of highway sections with poor skid resistance. A comparison of the W/D accident ratio and the skid number was performed and the W/D was not able to accurately identify highway segments with low levels of friction.

Another major objective of this research was to demonstrate the benefits of a proactive approach to managing pavements and safety using preservation. The effectiveness of a number of commonly used preservation treatments were evaluated using LTPP data. The increase in skid resistance as a result of various treatments was evaluated. The increase in SN was observed to range from 14% to 27% for various types of overlays and from 25% to 33% for various types of surface treatments. Models were also developed for each environment zone within the LTPP study to model how skid resistance deteriorates with time. Generally, very little deterioration or reduction in skid resistance was observed over time within the four environment zones. This result is similar to a study completed by Indiana Department of Transportation (Li et al., 2004). This is an important factor to consider when examining historical friction trends, conducting life cycle cost analysis, and developing pavement performance models.

Life Cycle Cost Analysis (LCCA) was performed to demonstrate the benefits of a proactive approach towards pavement and safety management. The results of the LCCA demonstrated that by increasing the level of skid resistance for a highway using a treatment such as micro surfacing or an asphalt concrete overlay, a reduction in the Collision Rate occurs which generates savings and benefits. The results also demonstrate that not only is preservation cost-effective from a pavement management perspective, it also addresses and complements the safety component.

The importance of network level testing to assess pavement performance is an essential component of any pavement management system. Skid testing using a locked wheel tester is an important tool that can be used to assess the level of friction and safety of a highway network. This study provided a methodology or framework to examine if the skid testing interval can be reduced for a highway network. The results from this study indicate that the skid testing interval can be increased from 1.0 km to 5.0 km and provide statistically the same results at the 95% confidence interval. Reducing the number of skid test points results in an immediate savings to the transportation agency, the data collection providers, contractors, and the public.

9.3 Recommendations

Current pavement management practices do not consider safety directly in the management process. Furthermore, most pavement and safety management systems or frameworks are not fully integrated. The results of this research study have demonstrated the influences of skid resistance on highway safety and the importance of collecting network level skid data for assessing the level of safety of a highway network.

Based on the results of this study, the following recommendations are presented and areas for future research are offered:

- Pavement surface friction has an effect on highway safety and on the probability of collision occurrence. It is not always easy to model or predict.
- New pavement design procedures should consider skid resistance within the analysis framework
- A proactive approach in dealing with the friction-collision problem would be advantageous. Network level friction testing should be carried out on an annual or bi-annual basis to screen the network and identify potential collision prone locations.

- Other factors such as highway geometrics (curve radius, tangent length, superelevation, sight distance, etc.) have an effect on the driver, vehicle and highway safety. Unfortunately, at the time of this research study, no comprehensive geometric data set was available from MTO. It is recommended that once this data is available, it should be closely examined to identify any relationships and correlations.

- One of the most time consuming components of this study was the integration and linkage of the data sets. A major reason for this issue is a problem many DOTs currently face - each data attribute was obtained from different departments within the agency (i.e., Pavement and Materials, Transportation and Safety, etc.). This demonstrates the benefits of integrating management systems such as a Traffic Safety Management System and a Pavement Management System.

- Network-level friction testing is an important component of any pavement management system and for determining the level of safety of a highway network. Therefore, it is recommended that skid testing be considered in any transportation agency's data collection cycle. Network level friction testing should be carried out on an annual or bi-annual basis to screen the network and identify potential collision prone locations.

- This study provides a methodology or framework to examine if the skid testing interval can be reduced for a highway network. The results from this study indicate that the skid testing interval can be increased from 1.0 km to 5.0 km and provide statistically the same results at the 95% confidence interval.

- Reducing the number of skid test points results in an immediate savings to the transportation agency, the data collection providers, contractors, and the public.

- It is important to note that the results from this study should not be extrapolated onto another agency’s highway network. However, the approach or methodology developed as a part of this study can be used to determine if the skid testing interval can be reduced for another agency’s highway network.

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Appendix A

Statistical Models

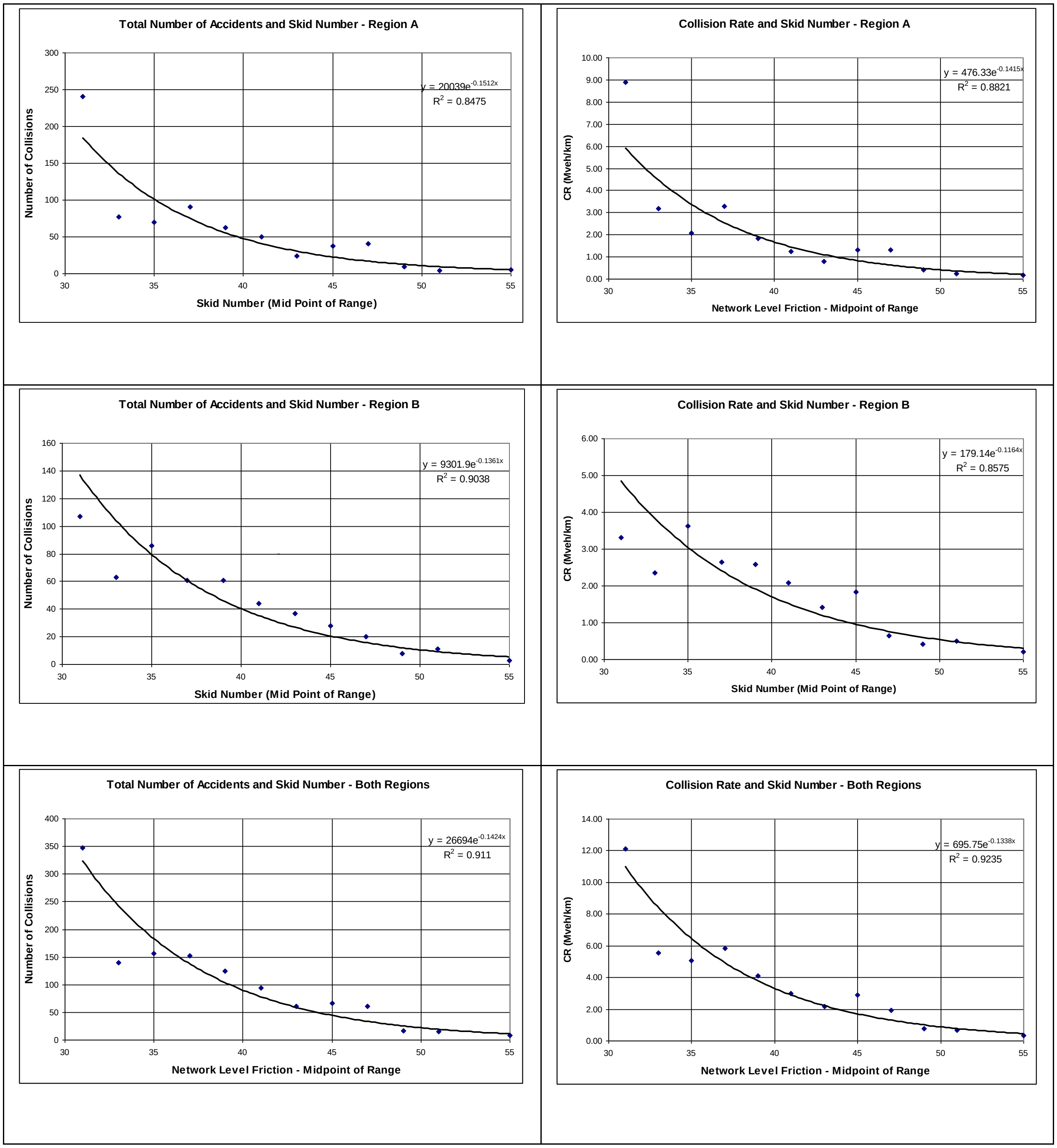


Figure A1: Region

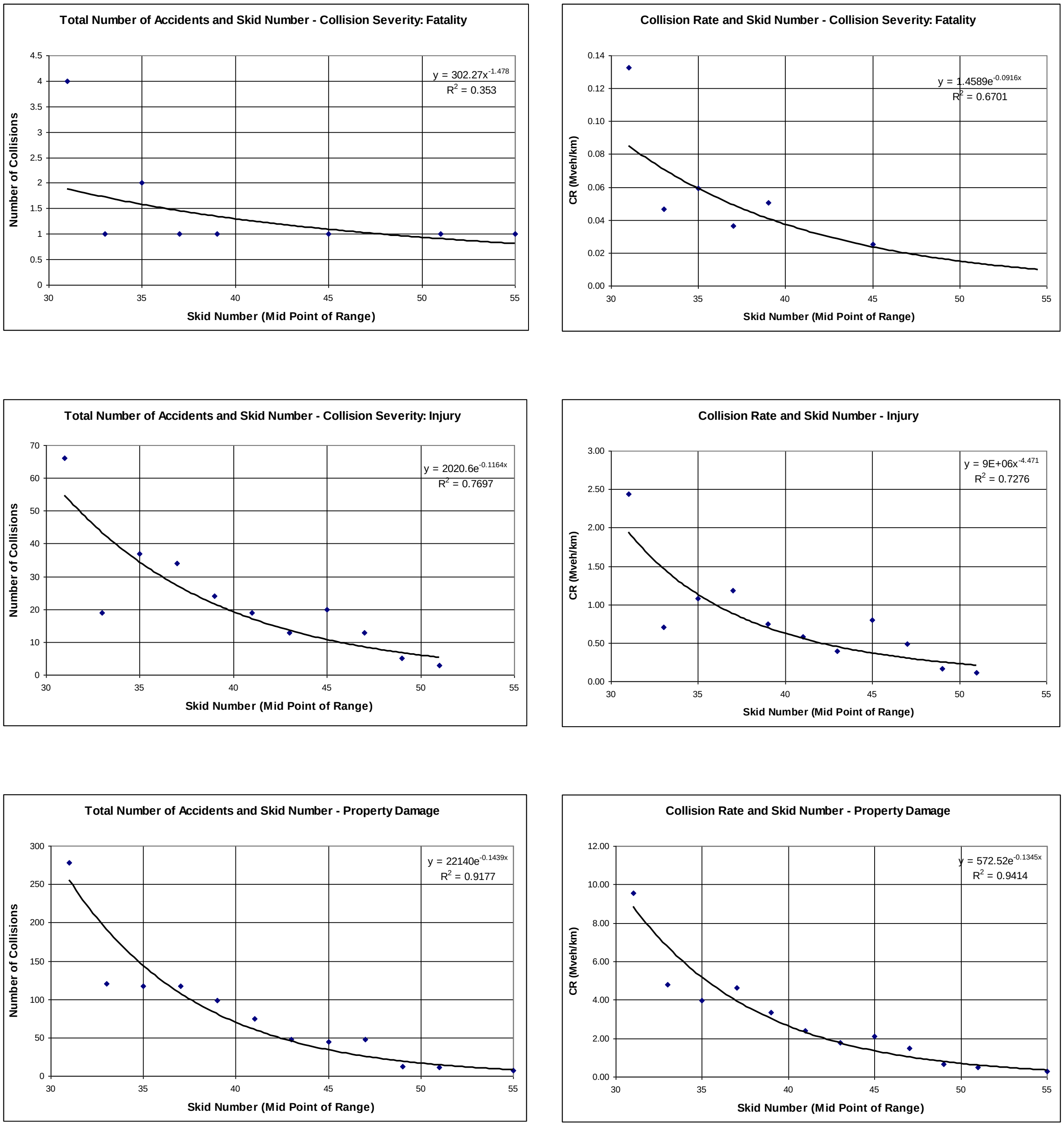


Figure A2: Collision Severity Models

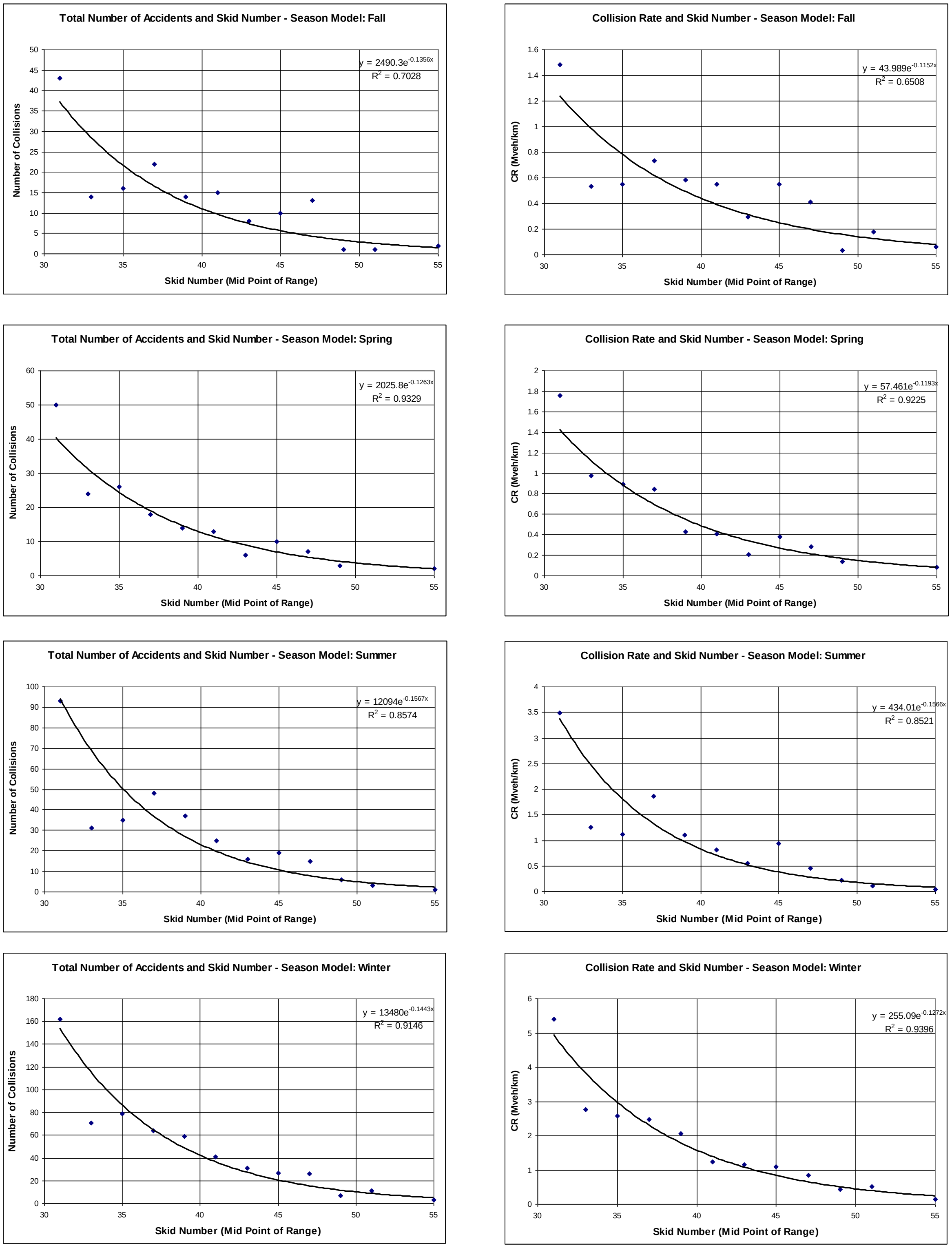


Figure A3: Season Model

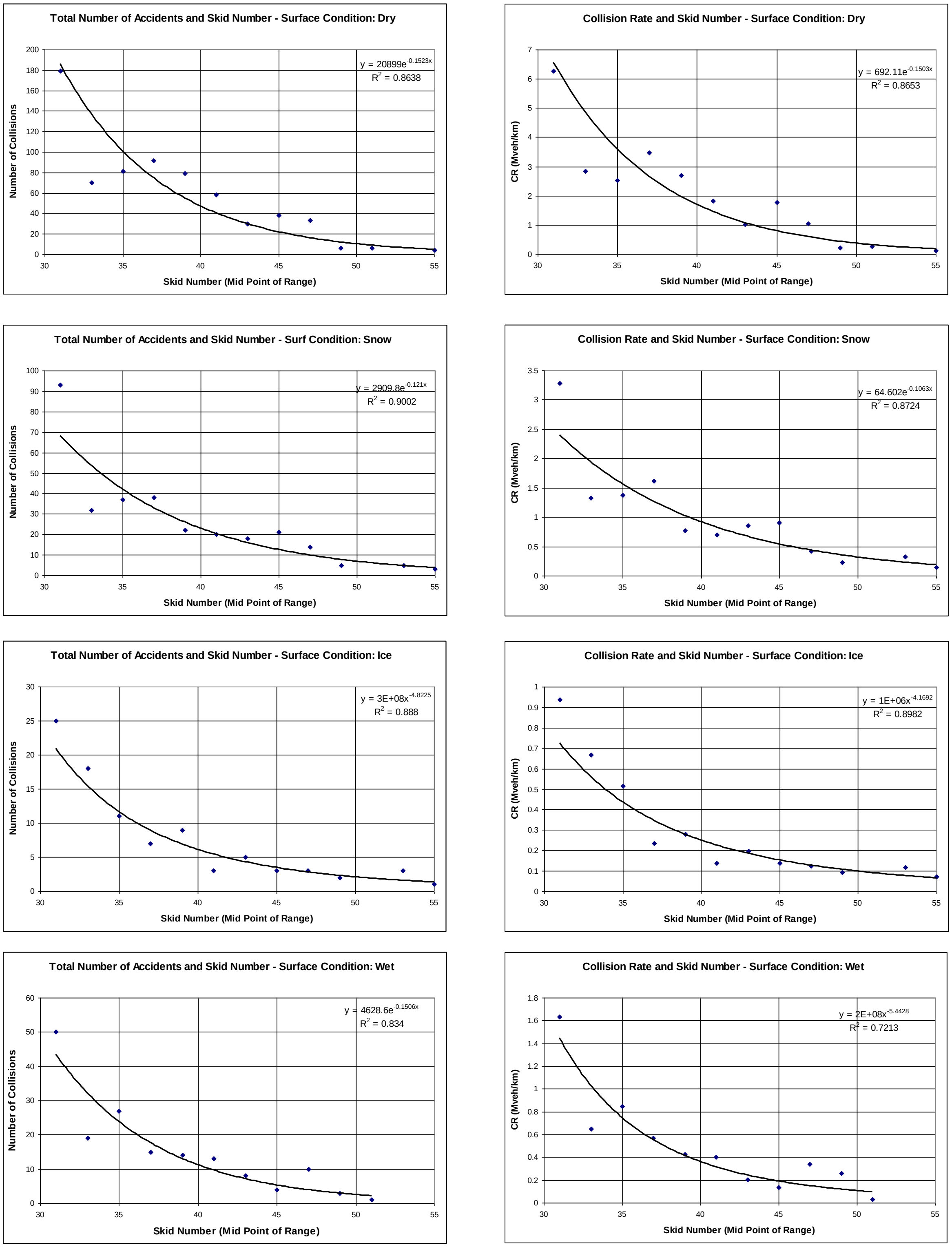


Figure A4: Road Surface Condition

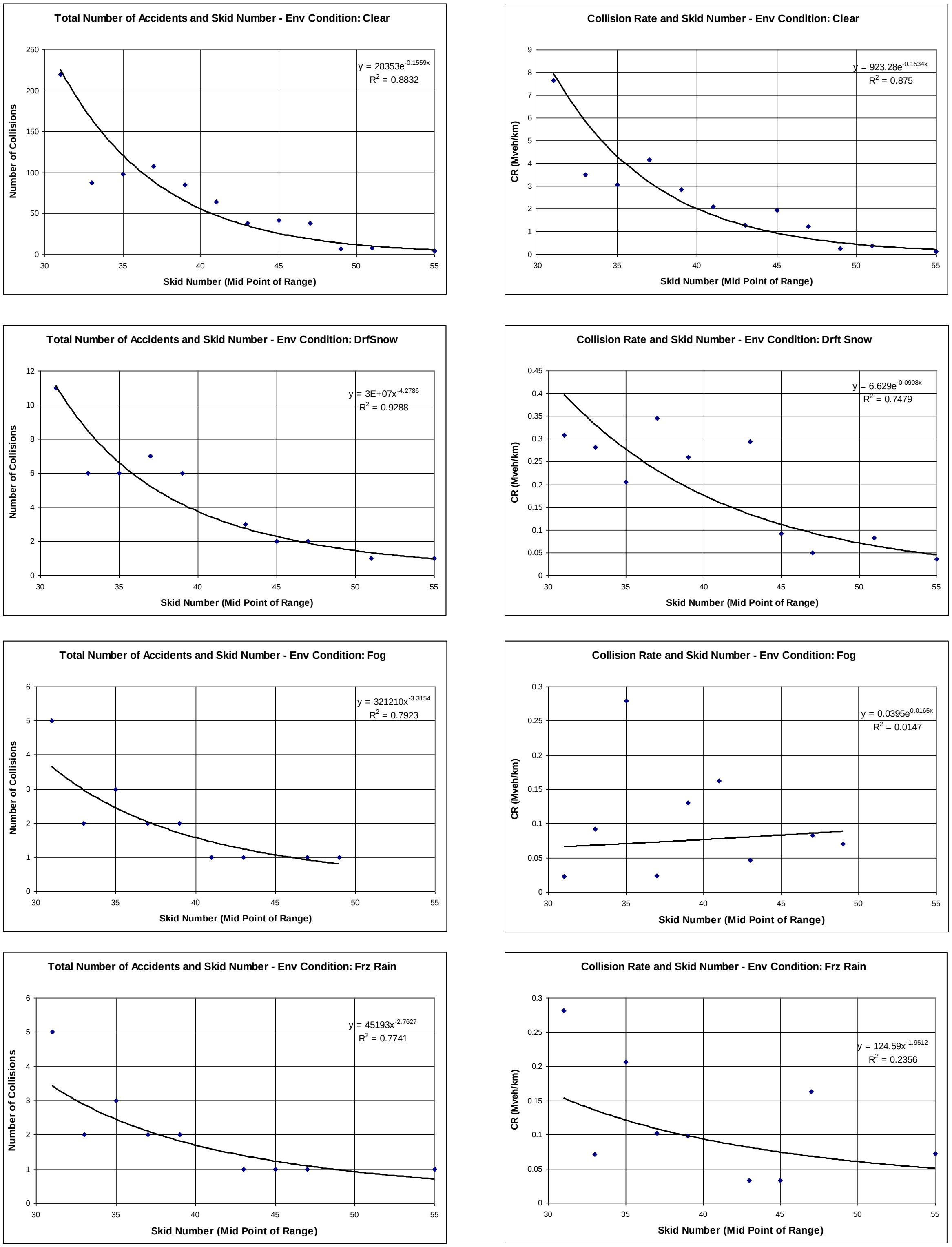


Figure A5: Environmental Condition

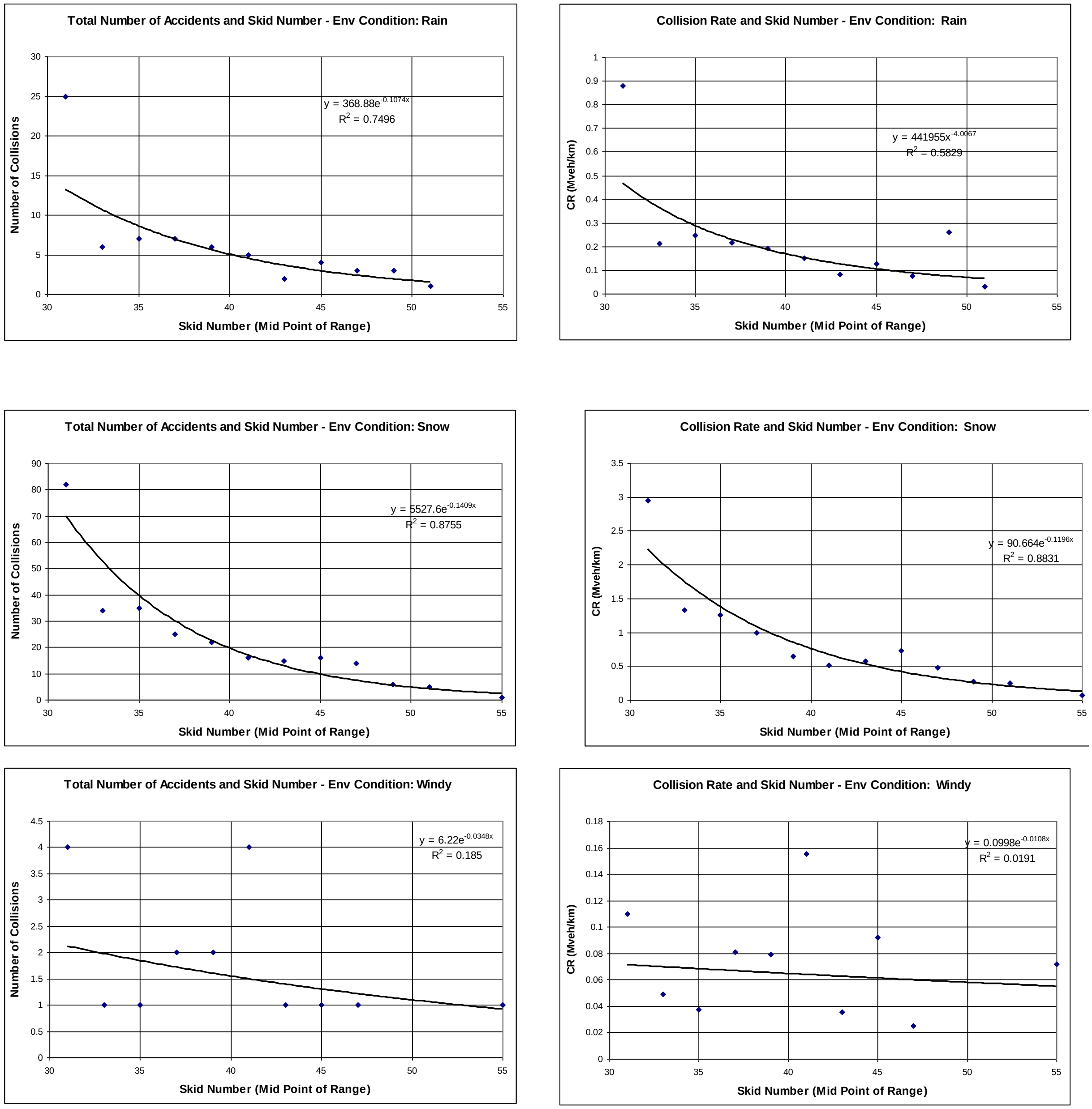
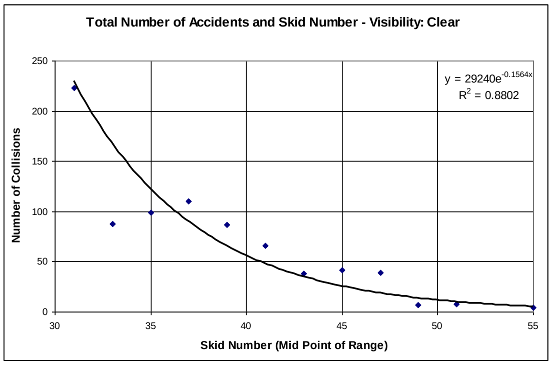
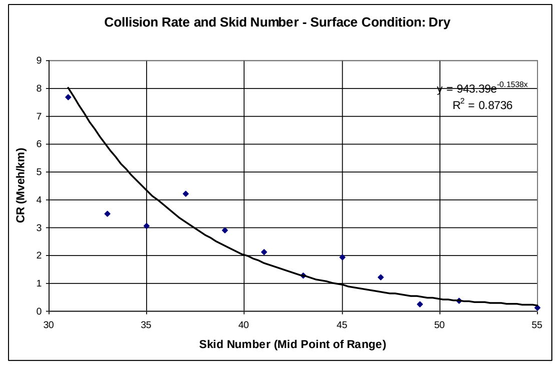
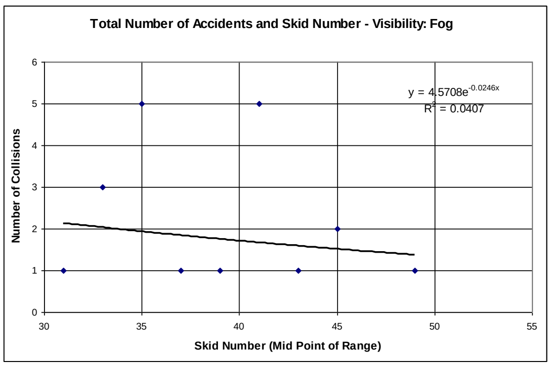
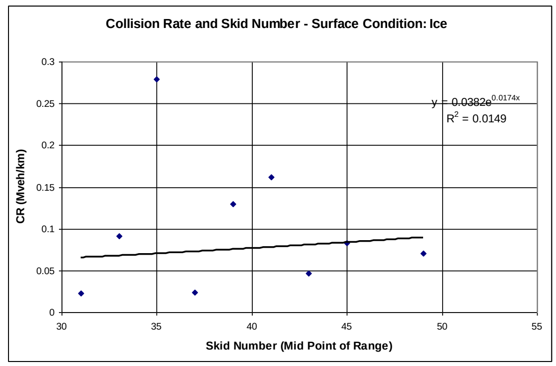


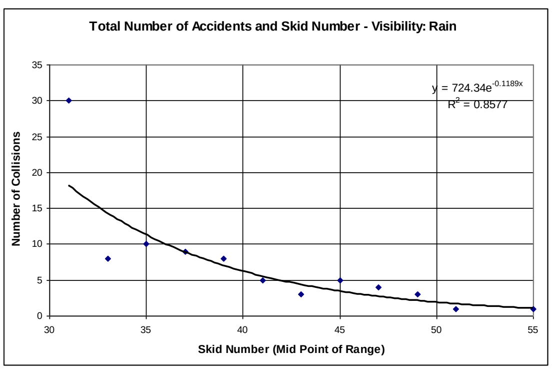
Figure A6: Environmental Condition (cont')

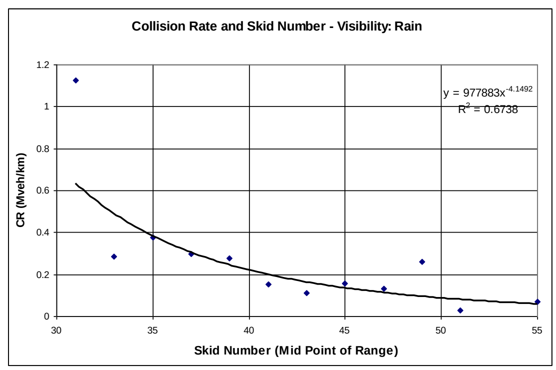


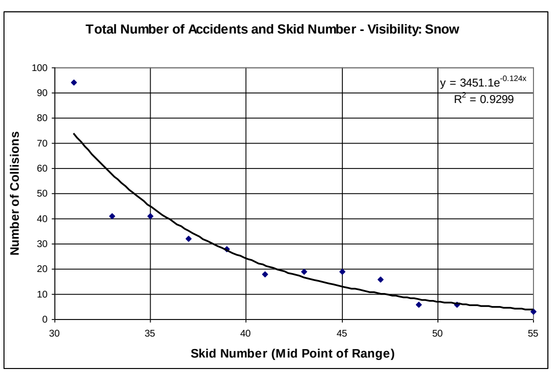












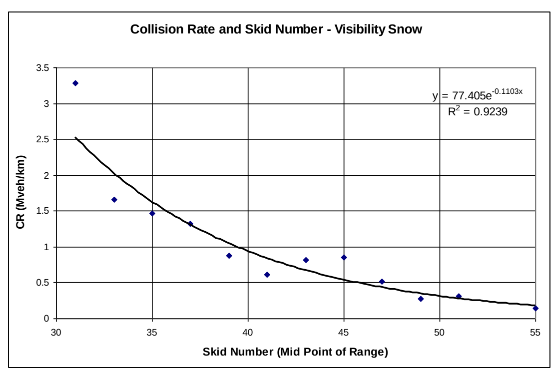


Figure A7: Visibility

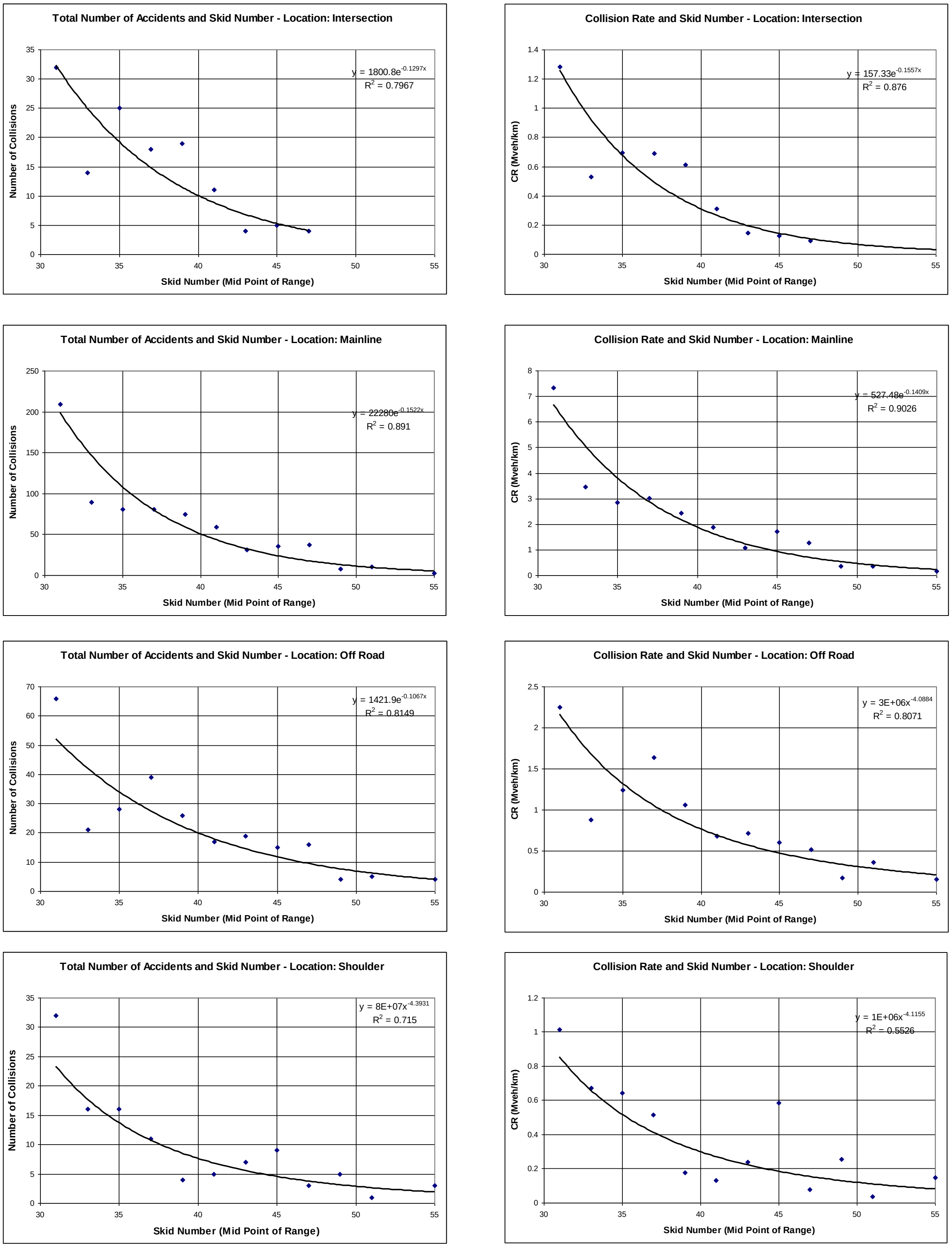


Figure A8: Collision Location

Appendix B
Preservation Scenarios

Table B1: Preservation Scenario for 7 Year LCCA

Traffic	Age	Preservation Scenario				
		Mill and Overlay	**AC Overlay**	**Recycled AC Overlay**	**Micro Surface**	**Do Nothing**
Low Traffic	0	Mill 50 mm + 80 mm AC O/L	50 mm AC O/L	50 mm RAC O/L	Micro Surface	
	1					
	2					
	3					
	4		Crack Seal (5%)	Crack Seal (10%)	Micro Surface	
	5	Crack Seal (5%)				
	6					
	7		Fog Seal	Fog Seal	Micro Surface	
High Traffic	0	Mill 50 mm + 80 mm AC O/L	50 mm AC O/L	50 mm RAC O/L		
	1					
	2					
	3					
	4		Crack Seal (10%)	Crack Seal (20%)		
	5	Crack Seal (5%)				
	6					
	7		Fog Seal	Fog Seal		

Table B2: Preservation Scenario for 10 Year LCCA

Traffic	Age	Preservation Scenario				
		Mill and Overlay	AC Overlay	Recycled AC Overlay	Micro Surface	Do Nothing
Low Traffic	0	Mill 50 mm + 80 mm AC O/L	50 mm AC O/L	50 mm RAC O/L	Micro Surface	
	1					
	2					
	3					
	4		Crack Seal (5%)	Crack Seal (10%)	Micro Surface	
	5	Crack Seal (5%)				
	6					
	7		Crack Seal (5%)	Crack Seal (10%)	Micro Surface	
	8	Crack Seal (5%)				
	9					
	10		Crack Seal (5%)	Crack Seal (10%)		
High Traffic	0	Mill 50 mm + 80 mm AC O/L	50 mm AC O/L	50 mm RAC O/L		
	1					
	2					
	3					
	4	Crack Seal (5%)	Crack Seal (10%)	Crack Seal (20%)		
	5					
	6					
	7	Crack Seal (5%)	Fog Seal	Fog Seal		
	8					
	9					
	10	Crack Seal (5%)	Crack Seal (10%)	Crack Seal (20%)		

Table B3: Preservation Scenario for 15 year Life Cycle

Traffic	Age	Preservation Scenario				
		Mill and Overlay	**AC Overlay**	**Recycled AC Overlay**	**Micro Surface**	**Do Nothing**
Low Traffic	0	Mill 50 mm + 80 mm AC O/L	50 mm AC O/L	Recycled AC	Microsurfacing	
	1					
	2					
	3				Microsurfacing	
	4		Crack Seal (5%)	Crack Seal (10%)		
	5	Crack Seal (5%)				
	6				Microsurfacing	
	7			Fog Seal		
	8		Crack Seal (5%)			
	9					
	10	Crack Seal (5%)		Crack Seal (10%)	Microsurfacing	
	11		Crack Seal (5%)			
	12					
	13					
	14	Crack Seal (5%)	Crack Seal (5%)	Crack Seal (10%)	Microsurfacing	
	15					
Low Traffic	0	Mill 50 mm + 80 mm AC O/L	50 mm AC O/L	Recycled AC		
	1					
	2					
	3					
	4	Crack Seal (5%)	Crack Seal (10%)	Crack Seal (20%)		
	5					
	6					
	7	Crack Seal (5%)	Fog Seal	Fog Seal		
	8					
	9					
	10	Crack Seal (5%)	Crack Seal (5%)	Crack Seal (10%)		
	11					
	12		Crack Seal (5%)	Crack Seal (10%)		
	13					
	14	Crack Seal (10%)	Crack Seal (10%)	Crack Seal (10%)		
	15					

Appendix C

Sample Calculations

Figure C1 : High Traffic/Very Low Collision/Very Poor SN

AGE	Mill and Overlay						
	TREATMENT	COST ($)	PW	Inc_PW	EAUC_yr	EAUC_PW	In_EAUC_PW
0	Mill 50 mm + 80 mm AC O/L	$ 1,761,493	$ 1,761,493	$ 1,761,493	$ (148,804)	$ (148,804)	$ (148,804)
1		$ (418,507)	$ (402,411)	$ 1,359,082	$ (148,804)	$ (143,081)	$ (291,885)
2		$ (418,507)	$ (386,934)	$ 972,148	$ (148,804)	$ (137,578)	$ (429,462)
3		$ (418,507)	$ (372,052)	$ 600,096	$ (148,804)	$ (132,286)	$ (561,748)
4	Crk Seal (10%cracking)	$ (393,507)	$ (336,372)	$ 263,725	$ (148,804)	$ (127,198)	$ (688,946)
5		$ (418,507)	$ (343,983)	$ (80,258)	$ (148,804)	$ (122,306)	$ (811,252)
6		$ (418,507)	$ (330,752)	$ (411,010)	$ (148,804)	$ (117,602)	$ (928,854)
7	Crk Seal (10%cracking)	$ (393,507)	$ (299,033)	$ (710,044)	$ (148,804)	$ (113,079)	$ (1,041,933)
8		$ (418,507)	$ (305,799)	$ (1,015,843)	$ (148,804)	$ (108,730)	$ (1,150,662)
9		$ (418,507)	$ (294,038)	$ (1,309,881)	$ (148,804)	$ (104,548)	$ (1,255,210)
10	Crk Seal (10%cracking)	$ (393,507)	$ (265,839)	$ (1,575,720)	$ (148,804)	$ (100,527)	$ (1,355,737)
11		$ (418,507)	$ (271,854)	$ (1,847,574)	$ (148,804)	$ (96,660)	$ (1,452,397)
12		$ (418,507)	$ (261,398)	$ (2,108,973)	$ (148,804)	$ (92,942)	$ (1,545,339)
13		$ (418,507)	$ (251,345)	$ (2,360,318)	$ (148,804)	$ (89,368)	$ (1,634,707)
14	Crk Seal (10%cracking)	$ (368,507)	$ (212,804)	$ (2,573,122)	$ (148,804)	$ (85,931)	$ (1,720,637)

AGE	AC Overlay						
	TREATMENT	COST($)	PW	Inc_PW	EAUC_yr	EAUC_PW	In_EAUC_PW
0	50 mm AC O/L	$ 836,493	$ 836,493	$ 836,493	$ (190,270)	$ (190,270)	$ (190,270)
1		$ (418,507)	$ (402,411)	$ 434,082	$ (190,270)	$ (182,952)	$ (373,222)
2		$ (418,507)	$ (386,934)	$ 47,148	$ (190,270)	$ (175,915)	$ (549,137)
3		$ (418,507)	$ (372,052)	$ (324,904)	$ (190,270)	$ (169,149)	$ (718,287)
4	Crk Seal (10%cracking)	$ (368,507)	$ (315,002)	$ (639,905)	$ (190,270)	$ (162,644)	$ (880,930)
5		$ (418,507)	$ (343,983)	$ (983,888)	$ (190,270)	$ (156,388)	$ (1,037,318)
6		$ (418,507)	$ (330,752)	$ (1,314,640)	$ (190,270)	$ (150,373)	$ (1,187,691)
7	Fog Seal	$ (168,507)	$ (128,052)	$ (1,442,692)	$ (190,270)	$ (144,590)	$ (1,332,281)
8		$ (418,507)	$ (305,799)	$ (1,748,491)	$ (190,270)	$ (139,028)	$ (1,471,309)
9		$ (418,507)	$ (294,038)	$ (2,042,529)	$ (190,270)	$ (133,681)	$ (1,604,990)
10	Crk Seal (10%cracking)	$ (393,507)	$ (265,839)	$ (2,308,368)	$ (190,270)	$ (128,540)	$ (1,733,530)
11		$ (418,507)	$ (271,854)	$ (2,580,223)	$ (190,270)	$ (123,596)	$ (1,857,126)
12	Crk Seal (10%cracking)	$ (393,507)	$ (245,784)	$ (2,826,006)	$ (190,270)	$ (118,842)	$ (1,975,968)
13		$ (418,507)	$ (251,345)	$ (3,077,351)	$ (190,270)	$ (114,271)	$ (2,090,239)
14	Crk Seal (10%cracking)	$ (368,507)	$ (212,804)	$ (3,290,155)	$ (190,270)	$ (109,876)	$ (2,200,115)

AGE	Do Nothing						
	TREATMENT	COST($)	PW	Inc_PW	EAUC_yr	EAUC_PW	In_EAUC_PW
0	Collision	$ 420,500	$ 420,500	$ 420,500	$ 281,187	$ 281,187	$ 281,187
1	Collision	$ 420,500	$ 404,327	$ 824,827	$ 281,187	$ 270,372	$ 551,559
2	Collision	$ 420,500	$ 388,776	$ 1,213,603	$ 281,187	$ 259,973	$ 811,532
3	Collision	$ 420,500	$ 373,823	$ 1,587,426	$ 281,187	$ 249,974	$ 1,061,506
4	Collision	$ 420,500	$ 359,445	$ 1,946,871	$ 281,187	$ 240,360	$ 1,301,866
5	Collision	$ 420,500	$ 345,620	$ 2,292,491	$ 281,187	$ 231,115	$ 1,532,981
6	Collision	$ 420,500	$ 332,327	$ 2,624,819	$ 281,187	$ 222,226	$ 1,755,207
7	Collision	$ 420,500	$ 319,545	$ 2,944,364	$ 281,187	$ 213,679	$ 1,968,886
8	Collision	$ 420,500	$ 307,255	$ 3,251,619	$ 281,187	$ 205,461	$ 2,174,347
9	Collision	$ 420,500	$ 295,438	$ 3,547,057	$ 281,187	$ 197,558	$ 2,371,905
10	Collision	$ 420,500	$ 284,075	$ 3,831,132	$ 281,187	$ 189,960	$ 2,561,865
11	Collision	$ 420,500	$ 273,149	$ 4,104,280	$ 281,187	$ 182,654	$ 2,744,518
12	Collision	$ 420,500	$ 262,643	$ 4,366,924	$ 281,187	$ 175,629	$ 2,920,147
13	Collision	$ 420,500	$ 252,541	$ 4,619,465	$ 281,187	$ 168,874	$ 3,089,020
14	Collision	$ 420,500	$ 242,828	$ 4,862,293	$ 281,187	$ 162,378	$ 3,251,399

AGE	Recyled AC					
	COST ($)	PW	Inc_PW	EAUC_yr	EAUC_PW	In_EAUC_PW
0	$ 736,493	$ 736,493	$ 736,493	$ (191,702)	$ (191,702)	$ (191,702)
1	$ (418,507)	$ (402,411)	$ 334,082	$ (191,702)	$ (184,328)	$ (376,030)
2	$ (418,507)	$ (386,934)	$ (52,852)	$ (191,702)	$ (177,239)	$ (553,269)
3	$ (418,507)	$ (372,052)	$ (424,904)	$ (191,702)	$ (170,422)	$ (723,691)
4	$ (318,507)	$ (272,261)	$ (697,165)	$ (191,702)	$ (163,867)	$ (887,558)
5	$ (418,507)	$ (343,983)	$ (1,041,148)	$ (191,702)	$ (157,565)	$ (1,045,123)
6	$ (418,507)	$ (330,752)	$ (1,371,900)	$ (191,702)	$ (151,505)	$ (1,196,628)
7	$ (168,507)	$ (128,052)	$ (1,499,952)	$ (191,702)	$ (145,677)	$ (1,342,305)
8	$ (418,507)	$ (305,799)	$ (1,805,751)	$ (191,702)	$ (140,074)	$ (1,482,380)
9	$ (418,507)	$ (294,038)	$ (2,099,789)	$ (191,702)	$ (134,687)	$ (1,617,067)
10	$ (368,507)	$ (248,950)	$ (2,348,739)	$ (191,702)	$ (129,507)	$ (1,746,573)
11	$ (418,507)	$ (271,854)	$ (2,620,594)	$ (191,702)	$ (124,526)	$ (1,871,099)
12	$ (368,507)	$ (230,169)	$ (2,850,762)	$ (191,702)	$ (119,736)	$ (1,990,835)
13	$ (418,507)	$ (251,345)	$ (3,102,107)	$ (191,702)	$ (115,131)	$ (2,105,966)
14	$ (368,507)	$ (212,804)	$ (3,314,911)	$ (191,702)	$ (110,703)	$ (2,216,669)

AGE	SRFT						
	TREATMENT	COST ($)	PW	Inc_PW	EAUC_yr	EAUC_PW	In_EAUC_PW
0	Microsurfacing	$ 80,455	$ 80,455	$ 80,455	$ (166,844)	$ (166,844)	$ (166,844)
1		$ (419,545)	$ (403,409)	$ (322,954)	$ (166,844)	$ (160,427)	$ (327,272)
2		$ (419,545)	$ (387,893)	$ (710,848)	$ (166,844)	$ (154,257)	$ (481,529)
3	Microsurfacing	$ 80,455	$ 71,524	$ (639,324)	$ (166,844)	$ (148,324)	$ (629,853)
4		$ (419,545)	$ (358,629)	$ (997,953)	$ (166,844)	$ (142,619)	$ (772,472)
5		$ (419,545)	$ (344,836)	$ (1,342,789)	$ (166,844)	$ (137,134)	$ (909,606)
6	Microsurfacing	$ 80,455	$ 63,584	$ (1,279,204)	$ (166,844)	$ (131,860)	$ (1,041,466)
7		$ (419,545)	$ (318,820)	$ (1,598,024)	$ (166,844)	$ (126,788)	$ (1,168,254)
8		$ (419,545)	$ (306,558)	$ (1,904,582)	$ (166,844)	$ (121,912)	$ (1,290,166)
9		$ (419,545)	$ (294,767)	$ (2,199,349)	$ (166,844)	$ (117,223)	$ (1,407,388)
10	Microsurfacing	$ 80,455	$ 54,352	$ (2,144,997)	$ (166,844)	$ (112,714)	$ (1,520,102)
11		$ (419,545)	$ (272,529)	$ (2,417,526)	$ (166,844)	$ (108,379)	$ (1,628,481)
12		$ (419,545)	$ (262,047)	$ (2,679,572)	$ (166,844)	$ (104,211)	$ (1,732,692)
13		$ (419,545)	$ (251,968)	$ (2,931,540)	$ (166,844)	$ (100,202)	$ (1,832,894)
14	Microsurfacing	$ 80,455	$ 46,461	$ (2,885,080)	$ (166,844)	$ (96,349)	$ (1,929,243)