IDAHO TRANSPORTATION DEPARTMENT

RESEARCH REPORT

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RP 244

Safety Impacts of Using Wider Pavement Markings on Two-Lane Rural Highways in Idaho

By

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16. Abstract

This project evaluated the safety effects of wider pavement markings on rural two-lane highways in Idaho using two analyses: 1) Observational Before-and-After Studies using a before-and-after comparison group analysis and an Empirical Bayes before-and-after analysis, and 2) Driver Simulation-Based Studies. The study also examined and modeled the deterioration characteristics of pavement marking used in Idaho highways. The findings from the Empirical Bayes analysis were consistent with those obtained from the comparison group. Both showed that wider pavement markings reduce the number of crashes by 17 percent and fatal and severe injury crashes by 14 percent, reducing crash rates at 5.53 percent and 12.59 percent respectively. The reduction in crash rates for total crashes is statistically significant at the 90 percent confidence level while the reduction in crash rates for fatal and severe injury crashes is statistically significant at the 95 percent confidence level. The results of the driver simulation study have showed subtle differences in driver behavior that occur at different deterioration levels and different widths. With such crash reduction potential, the cost-to-benefit ratio of implementing wide pavement marking throughout the state's two-lane rural highway segments is approximately 1:25.

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METRIC (SI*) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
in	inches	25.4	millimeters	mm
ft	feet	0.3048	meters	m
yd	yards	0.914	meters	m
mi	Miles (statute)	1.61	kilometers	km
		AREA		
in^2	square inches	645.2	millimeters squared	cm^2
ft^2	square feet	0.0929	meters squared	m^2
yd^2	square yards	0.836	meters squared	m^2
mi^2	square miles	2.59	kilometers squared	km^2
ac	acres	0.4046	hectares	ha
		MASS (weight)		
oz	Ounces (avdp)	28.35	grams	g
lb	Pounds (avdp)	0.454	kilograms	kg
T	Short tons (2000 lb)	0.907	megagrams	mg
		VOLUME		
fl oz	fluid ounces (US)	29.57	milliliters	mL
gal	Gallons (liq)	3.785	liters	liters
ft^3	cubic feet	0.0283	meters cubed	m^3
yd^3	cubic yards	0.765	meters cubed	m^3

Note: Volumes greater than 1000 L shall be shown in m^3

		TEMPERATURE (exact)		
°F	Fahrenheit temperature	5/9 (°F-32)	Celsius temperature	°C
		ILLUMINATION		
fc	Foot-candles	10.76	lux	lx
fl	foot-lamberts	3.426	candela/m^2	cd/cm^2
		FORCE and PRESSURE or STRESS		
lbf	pound-force	4.45	newtons	N
psi	pound-force per square inch	6.89	kilopascals	kPa

APPROXIMATE CONVERSIONS FROM SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	Miles (statute)	mi
		AREA		
mm^2	millimeters squared	0.0016	square inches	in^2
m^2	meters squared	10.764	square feet	ft^2
km^2	kilometers squared	0.39	square miles	mi^2
ha	hectares (10,000 m^2)	2.471	acres	ac
		MASS (weight)		
g	grams	0.0353	Ounces (avdp)	oz
kg	kilograms	2.205	Pounds (avdp)	lb
mg	megagrams (1000 kg)	1.103	short tons	T
		VOLUME		
mL	milliliters	0.034	fluid ounces (US)	fl oz
liters	liters	0.264	Gallons (liq)	gal
m^3	meters cubed	35.315	cubic feet	ft^3
m^3	meters cubed	1.308	cubic yards	yd^3

		TEMPERATURE (exact)		
°C	Celsius temperature	9/5 °C+32	Fahrenheit temperature	°F
		ILLUMINATION		
lx	lux	0.0929	foot-candles	fc
cd/cm^2	candela/m^2	0.2919	foot-lamberts	fl
		FORCE and PRESSURE or STRESS		
N	newtons	0.225	pound-force	lbf
kPa	kilopascals	0.145	pound-force per square inch	psi

Technical Advisory Committee

Each research project is overseen by a technical advisory committee (TAC), which is led by an ITD project sponsor and project manager. The Technical Advisory Committee (TAC) is responsible for monitoring project progress, reviewing deliverables, ensuring that study objective are met, and facilitating implementation of research recommendations, as appropriate. ITD's Research Program Manager appreciates the work of the following TAC members in guiding this research study.

Project Sponsor/Project Manager – Jared Hopkins

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Executive Summary

The primary objective of this project is to evaluate the safety effects of wider pavement markings on rural two-lane highways in Idaho using two analyses: 1) Driver Simulation-Based Studies, and 2) Observational Before-and-After Studies. The study also examined and modeled the deterioration characteristics of pavement marking used in Idaho highways.

The driver simulator study examined the relationship between driver lane deviation and varying combinations of edge line pavement marking widths and deterioration levels. The simulated environment encountered by all participants represented daytime and nighttime driving conditions. During real-world conditions with visible light, external factors such as signage, trees, and the presence of guardrails may impact driver behavior, though these elements were not simulated as part of this research. However, for these reasons, testing during nighttime conditions when such cues are not visible and when pavement markings are more heavily relied upon was included.

Edge line deterioration is an expected event that occurs due to weathering, plowing, and wearing from recurrent vehicle tire loading. The results of the driver simulation study have shown subtle differences in driver behavior that occur at different deterioration levels and different widths. For public agencies who are responsible for the operations of these facilities, proper maintenance and upkeep of edge line markings, regardless of width, ensures that vehicle operators, under normal driving conditions, will be most likely to maintain lane position when visibility of these markings is highest.

Waterborne marking retroreflectivity deterioration was modeled using field and laboratory data. Field retroreflectivity data for each targeted rural road in Idaho was collected at three different times over twelve months to establish deterioration curves. The endurance of waterborne markings under different conditions were tested and correlated with field data.

- The results show that the best fit curve to predict waterborne pavement markings retroreflectivity deterioration in Idaho was a logarithmic decay function.
- There was a strong relationship between the deterioration rate of pavement marking retroreflectivity and the weighted average Normalized Ground Snow Load (NGSL) for all districts in Idaho. The higher the NGSL, the faster deterioration (loss) in retroreflectivity. This finding could be attributed to the increase in winter maintenance activities (e.g. snowplowing).
- An increase in pavement markings lightness, total color change, and percent loss was directly correlated to a loss in retroreflectivity.

The 6-inch edge line markings degrade at a relatively slower rate than the 4-inch-wide markings did. However, the difference in retroreflectivity degradation rate between the 6-inch and the 4-inch edge line markings decreases with time.

A before-and-after study was done to examine the safety effect of wider pavement markings. Two different methods were employed: a before-and-after comparison group analysis and Empirical Bayes before-and-after analysis.

The results of the comparison group analysis show that the average crash rate for total crashes, night crashes, fatal and severe injury crashes, and fatal and severe injury night crashes decreased after implementating wide pavement markings. The reduction in the crash rate for total crashes, night crashes, and fatal and severe injury crashes were 12.22 percent, 8.81 percent, and 18.68 percent, respectively. These crash rate reductions, however, are not statistically significant at the 95 percent confidence level (the fatal and severe injury crash rate reduction of 18.68 percent is statistically significant at the 90 percent confidence level). Again, fatal and severe injury night crashes showed a statistically significant crash rate reduction, at the 95 percent confidence interval, of 39.39 percent as a result of wider pavement marking implementation.

The results of Empirical Bayes analysis were consistent with the results obtained from the comparison group analysis. For crash frequency, the Empirical Bayes unbiased estimates for the reduction of crashes as a result of the implementation of wider pavement markings are 17 percent and 14 percent for total crashes and fatal and severe injury crashes, respectively. For crash rates, these reductions are 5.53 percent and 12.59 percent, respectively. The reduction in crash rates for total crashes in statistically significant at the 90 percent confidence level. The reduction in crash rates for fatal and severe injury crashes is statistically significant at the 95 percent confidence level.

Based on the 2012 to 2016 Crash data the average cost of fatal and serious injury ROR crashes in two-lane rural highways in Idaho is approximately $382.05 million a year. Wide pavement marking implementation has the potential to reduce fatal and serious ROR crashes by 12.59 percent with an expected cost savings of $48.1 million.The cost-to-benefit ratio of implementing wide pavement marking throughout the state's two-lane rural highway segments is approximately 1:25.

Chapter 1
Introduction

The purpose of this research is to determine if the use of wider longitudinal pavement markings for center lines, lane lines and edge lines can provide critical information to drivers that will help them to better identify the roadway alignment and maintain appropriate lane position. In the State of Idaho, based on the 2015 crash data, the largest contributors to single vehicle crashes have been the inability of the driver to maintain his or her lane position (22 percent) and the driver's travel speed (22 percent). Driver inattention and distraction (11 percent), driver overcorrection (5 percent), and driving to the left of center (2 percent) also represent contributing circumstances. This research project supports the Idaho Transportation Department's Strategic Highway Safety Plan wherein lane departure is one of the critical focus areas.

Wider longitudinal pavement markings may provide a safer environment for drivers by increasing visibility. The safety impact of wider edge lane lines has been examined in at least three states (Kansas, Michigan, and Illinois) and these initial studies have concluded that there is "enough statistical evidence to support the positive safety effects of wider edge lines installed on rural, two-lane highways." The crash types that are likely to be positively impacted by wider pavement marking include run-off-the-road and opposite direction crashes. Historically, these crash types have a high crash severity index.

The Manual on Uniform Traffic Control Devices, or MUTCD, specifies a nominal width of 4 inches for longitudinal pavement markings. For this study, wider pavement markings, including, but not limited to, 6 inches and 8 inches will be implemented to determine if there is a correlation between wider pavement markings, driver behavior, and enhanced user safety. ITD staff will be able to apply the results from the analysis conducted as part of this project study to more accurately assess the expected safety performance of alternative pavement marking practices in Idaho and to accurately determine the cost / benefit ratio of using wider pavement markings in different parts of the state highway system. In addition to the before-and-after safety studies, a driver-simulation based study will be conducted to assess the impact of wider pavement marking on speed and lane position under different geometric and driving conditions.

This project evaluated the safety effects of wider pavement markings on rural two-lane highways in Idaho using two analyses: 1) Observational Before-and-After Studies, and 2) Driver Simulation-Based Studies. The study also examined and modeled the deterioration characteristics of pavement marking using in Idaho highways. Wider pavement markings are implemented at different sites throughout the state. The sites are selected to represent the diverse geometric and operational characteristics in the Idaho state highway system. Two enhanced statistical analyses, namely empirical Bayes before-after analysis and a comparison group safety comparison, are utilized to assess the safety impacts of implementing wider pavement marking in different sites throughout the state.

In addition to the field data analysis, a driver simulator study will be conducted to assess the potential impact of wider pavement markings on speed and lane position under different geometric and driving conditions for different driver groups. Collectively, the results from both studies will provide ITD with a comprehensive understanding of the potential safety benefits of wider pavement marking.

Report Organization

This report is organized in 5 chapters. After the introduction, chapter 2 presents the results of the driver simulator experiment. Chapter 3 documents the analysis and results of the pavement margining deterioration analysis. Chapter 4 includes the results of two before-and-after crash analyses conducted to assess the safety benefits of wider pavement markings: a comparison group and Empirical Bayes analyses. Chapter 5 presents the study summary and conclusions.

Chapter 2
Driver Simulator Experiment

Overview

Single vehicle crashes, particularly those classified as run-off-the-road, are very common on two-lane rural highways. One method to potentially reduce such crashes is to provide additional driver information in the form of wider longitudinal edge line pavement markings. However, since these markings deteriorate over time, the primary objective of this chapter's research was to study the effects of longitudinal edge line pavement markings with varying deterioration levels and widths and to assess a driver's ability to maintain lane position. The University of Idaho's driving simulator was used to examine these effects by incorporating different marking deterioration percentages and roadway geometries on a two-lane rural highway environment. Two different pavement marking widths (4 and 6 inch) and four different deterioration levels (0 percent, 25 percent, 50 percent, and 75 percent) were assessed in daytime and nighttime conditions as part of this study. The results revealed that while wider 6-inch longitudinal edge line pavement markings compared with standard 4-inch edge line markings did not cause any significant changes in driver lane deviation during the day, statistically significant differences were observed in nighttime driving conditions. Drivers consistently maintained a lane position that slightly favored the edge line side throughout the study and increasingly shifted away from the centerline as edge line deterioration worsened.

Background and Literature Review

The inability of the driver to maintain appropriate lane position represents one of the largest causes of single vehicle crashes with drivers failing to maintain lane position contributing to 22 percent of all single vehicle crashes in the State of Idaho.[1] If additional driver cues in the form of wider longitudinal edge line pavement markings can help drivers to maintain lane position and thereby reduce the likelihood of single vehicle crashes, then a change to current operational practices would deserve consideration. For this study, two different edge line widths (4 and 6 inch) were implemented in a roadway simulation track environment to determine how these width variations impacted driver performance. In addition, it was recognized that these markings deteriorate over time due to weather, vehicle tire tracking, and snow removal operations. For these reasons, different edge line deterioration percentages (0 percent, 25 percent, 50 percent, and 75 percent) were also applied to each width and evaluated under different types of roadway geometry (straight and curved horizontal segments).

Many studies have been carried out to investigate the safety impacts of using wider pavement markings on crashes, vehicle lane position, operating speed, and driver behavior, but none have focused on explaining the effect of edge line pavement marking width and deterioration on vehicle lateral position. Previous research efforts associated with the key elements of this study are broadly described in the following topic areas: pavement marking application, safety impacts and evaluation of wider edge lines, and marking retroreflectivity (visibility) and service life (durability).

Industry Practices

Wider edge line markings are very common in the Eastern United States and twenty-two of the twenty-six states located east of the Mississippi River currently use wider markings; west of the Mississippi River only seven of the twenty-four states use them.[2] Based on survey data collected from different state agencies, the main reason for implementing wider markings was visibility improvement (identified by 57 percent of respondents) and as a crash reduction countermeasure for older drivers (19 percent) and crash reduction (14 percent)[2].

Based on the documented safety effects of wider edge lines on rural two-lane highways, individual states such as Michigan and Kansas increased their edge line width from 4 to 6 inches while Illinois increased from four to five inches. Each state performed different statistical analyses based on obtaining crash data (before and after implementing wider edge lines). These states concluded that wider edge lines reduced vehicle crashes with the highest crash reduction percentage in the fatal plus injury category (Kansas: 36.5 percent, Michigan: 15.4 percent, and Illinois: 37.7 percent).[3]

Benefit/Cost

Crash data have been used to analyze the costs associated with fatal and injury crashes, and the benefits from the wider edge lines were quantified while taking into account the assumed service life of standard edge line and installation costs. Specific crash benefits were obtained by calculating the difference between estimated and observed crashes and then multiplying this result by a fatality and injury cost value. In one example, the installation cost of a 4-inch waterborne edge line width was $0.10 per foot, while the installation cost of a 6-inch waterborne edge line width was $0.15 per foot, resulting in a cost difference of $528 per mile. However, the results showed that there was a noticeable benefit to cost ratio improvement for fatal crashes; for every $1 invested in the installation of a 6 iinch edge line an estimated benefit of $55.20 in crash costs was realized.[4]

Marking Retroreflectivity

Retroreflectivity of edge lines are related to their service life. As the deterioration of the edge line increases, retroreflectivity decreases, indicating that the marking needs to be refurbished or replaced.[5] There are many factors that affect the deterioration of an edge line's retroreflectivity including: climate or environment, plowing and snow removal (in select states), vehicle loading, edge line material type, and edge line placement. All of these factors affect how often the edge lines should be maintained in order to provide appropriate roadway visibility. The Michigan DOT restripes 85 percent of their roadways on a yearly basis due to snow plowing activities that occur during a large portion of the year.[6] A comparative test between glass bead and pavement marker materials in nineteen states showed that the service life for a two-lane rural highway edge line with driver speeds of forty-five miles per hour (mph) or greater is in the range of three to five years[7], with epoxy material (five years) lasting longer than typical profiled thermoplastic (three years).

Lateral Position Effects

Several studies have focused on explaining the effect of edge line pavement markings on vehicle lateral position. An early study evaluated the impact of 4 inch edge lines on driver behavior and found that vehicle operators tended to shift toward the roadway centerline when no interference between vehicles was assumed.[8] The tendency and change in magnitude of a motorist to move toward or away from an edge line pavement marking depended on many factors such as lane width, operating speed, time of day, frequency of heavy vehicles, the condition of the pavement, roadway alignment (curvature), edge drop-off, and traffic volume of the opposite direction.[9] During normal operations, drivers often employed a curve-flattening strategy to overcome centrifugal force. For this reason, drivers tended to be closer to the centerline of the road while driving through left-hand curves and closer to the edge line when driving through right-hand curves.[10]

Other Impacts

Wider edge lines helped drivers under sober and alcohol-impaired conditions to better identify the roadway delineation on two-lane rural highways. One such study examined the lateral lane position in order to analyze driver performance and the results showed that an eight inch wide edge line was found to provide benefits when compared with a standard width (4 inch), while no reduction in variability occurred with the 6-inch edge line.[11]

A separate study was conducted to determine if the use of edge lines and wider edge lines benefitted drivers under normal and impaired driving conditions. As part of the participants' driving tasks they encountered curves, obstacles, and road signs. The roadway simulation session was composed of different edge lines widths (none, 4 inch, and 8 inch), spot treatments, and curves. An analysis of variance (ANOVA) was conducted showing that under sober conditions the wider edge lines (8 inch) were associated with greater lateral lane position error than the standard edge line (4 inch); however, neither edge line was significantly different from the no edge line condition. Participants with a high BAC level (0.12 percent) had greater lateral lane position error when there were no edge lines, with the error decreasing as the edge lines widened.[12] Although this research study did not examine the effects of driver sobriety, the results and methodology from these other studies served as helpful references when designing this particular study and assessing specific results.

Driver Simulator Experiment - Methodology

Overview of the Experiment

A sample of 48 licensed participants were tested in a driver simulation environment that replicated a two-lane rural highway in the State of Idaho. Each participant was required to drive for about five minutes before the actual experiment commenced to become familiar with the responsiveness of the driving simulator. This initial drive segment helped the participants become accustom to the sensitivity of the gas pedal, brake pedal, and steering wheel to help mitigate for data anomalies due to the driver's lack of familiarity.

All participants conducted at least two sessions, driving for about 45 to 50 minutes during each session. Exactly half of the participants drove in daytime conditions, while the other half experienced nighttime conditions. Each participant was exposed to a 42½ mile roadway simulation track; within the 42½ miles, 40 miles were driven at speeds close to 60 mph (posted speed limit). One extra mile was placed at the beginning of the track in order for participants to reach the desired travel speed. An extra half mile was placed halfway along the track to accommodate a participant rest period. A final extra mile was placed at the end of the track so that participants could gradually come to a stop when completing the experimental drive.

Scenario Development

Every driving scenario was composed of multiple tiles that displayed the appropriate roadway geometries and surrounding daytime or nighttime environment. The roadway geometry was composed of a paved roadway consisting of straight and horizontal curved segments (gentle and sharp curves), edge lines, gravel shoulder, and varying edge line widths and deterioration percentages.

According to the Highway Capacity Manual (HCM 2010), the base conditions of a two lane highway requires lane widths greater than or equal to twelve feet, shoulder widths up to six feet, zero no-passing zones, all passenger cars in the traffic stream, level terrain, and no impediments to through traffic.[(13)] Based on these conditions, each scenario for this study was composed of: twelve foot lane widths, ten foot shoulders (eight foot gravel shoulder and two foot paved shoulder), zero no-passing zones, level terrain, and no impediments to through traffic. The surrounding daytime environment was composed of trees, mountain hills, and house/building structures. Adjustments were performed on the roadway geometry, and specifically to the edge line width and its deterioration level.

According to the Manual on Uniform Traffic Control Devices (MUTCD), the nominal (standard) edge line is 4-inches wide.[(14)] For this study, scenarios with 4-inch 6-inch edge line widths were developed based on discussions with Idaho Transportation Department staff, and any changes to the width were made on the shoulder side of the roadway. In other words, the lane width between the centerline and the travel side of the edge line was maintained at a constant twelve feet for the duration of the study.

The 3Ds MAX Design program (by Autodesk) was used to make the edge line width and edge line deterioration percentage adjustments (for every single tile) for all study scenarios. The Tile Mosaic Tool (TMT) was used to join multiple tiles together to create the appropriate roadway simulation track for all scenarios. The Interactive Scenario Authoring Tool (ISAT) was used to import all scenarios and add vehicles, speed limit signs, information signs, and triggers (data collection points) to each roadway simulation track. The ISAT was created for the National Advanced Driving Simulator (NADS), which was developed by the National Highway Traffic Safety Administration.[(15)]

Simulated Traffic

According to the Idaho Transportation Department (ITD), typical Annual Average Daily Traffic (AADT) values for two-lane rural highways in this state ranged from 500 to 10,000 vehicles per day.[16] In order to obtain an appropriate AADT value for the scenarios in this study, a sampling of thirty ITD traffic survey and analysis monitoring stations were reviewed. Based on these findings, an AADT of 3,200 vehicles per day was calculated and used as a representative value for all study scenarios.

To obtain the Directional Design Hourly Volume (DDHV) in the oncoming lane, the previous AADT value was used along with a K-value of 0.10 (proportion of AADT occurring in the peak hour) and a D-value of 0.5 (proportion of peak hour traffic in the peak direction). A K-value of 0.10 is recommended by the HCM 2010 for a rural highway[13] while a D-value of 0.5 assumed that the traffic volume experienced in both directions consisted of the same number of vehicles in each lane per hour. A DDHV of 160 vehicles was calculated for the oncoming lane (typical two-lane rural highway traffic volume per hour on one lane) and this DDHV value was implemented for all of the scenarios in this study.

Vehicle, Speed Limit and Information Sign, and Trigger Placement

According to the HCM 2010, the State of Idaho has a default value of 12 percent heavy vehicles on two-lane highways (13). Using this information, a set of regular vehicles (SUVs, sedans, pickups, and vans), two police vehicles, and 12 percent heavy vehicles (semi-trucks and dump trucks) were placed in the oncoming lane. All of these vehicles were placed using the ISAT for all eight scenarios (described later) in order to create the appropriate roadway simulation tracks with realistic traffic volumes and these oncoming vehicles had an approach speed of 60 mph to match the posted speed limit. The experimental vehicle (subject vehicle) was placed near the start of the simulation roadway track. The subject vehicle was accompanied by a vehicle following at a distance of about 1,320 feet (one quarter of a mile) and by another vehicle in front at the same distance. These distances were held constant throughout the experiment so the participant was not able to pass the vehicle in front or be passed by the vehicle behind it.

Two speed limit signs of 60 mph were present along the simulation track, with one at the beginning and another at about halfway of the roadway simulation track in all scenarios. Informational signs included curve warning signs that provided directional information about the ensuing curve.

All participants were required to complete one session for each edge line width. There were eight logs (triggers that create epochs) for each edge line deterioration percentage and since each width scenario was composed of four different edge line deterioration percentages, thirty-two logs (epochs) were generated from each roadway simulation track: four logs representing the straight segments, two logs representing the gentle curved segments (turning left and right), and two logs representing the sharp curved segments. Figure 1 shows a graphical representation of the number of epochs within each edge line width scenario. Each participant drove through 64 triggers during their two sessions so 64 data points were collected for each of the 48 participants, resulting in a cumulative total of 3,072 possible data collection points.

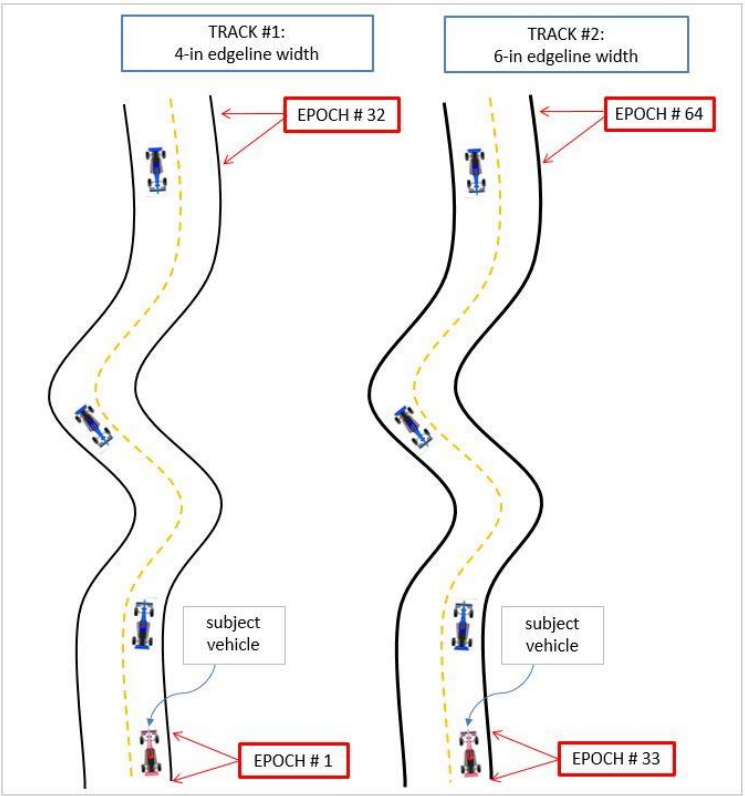


Figure 1 Graphic Description of Epochs Among Scenario Tracks.

Description of Scenarios Created

A total of eight scenarios were created and consisted of four scenarios for each of the two edge line widths. Each scenario had a specific edge line deterioration percentage ordering. Figure 2 shows the representative difference between the 4-inch and 6-inch edge line widths, while Figure 3 shows the difference between the 0 percent, 25 percent, 50 percent, and 75 percent edge line deterioration percentages applied on the 6-inch edge line width. The same design was applied to the 4-inch width but is not separately shown.

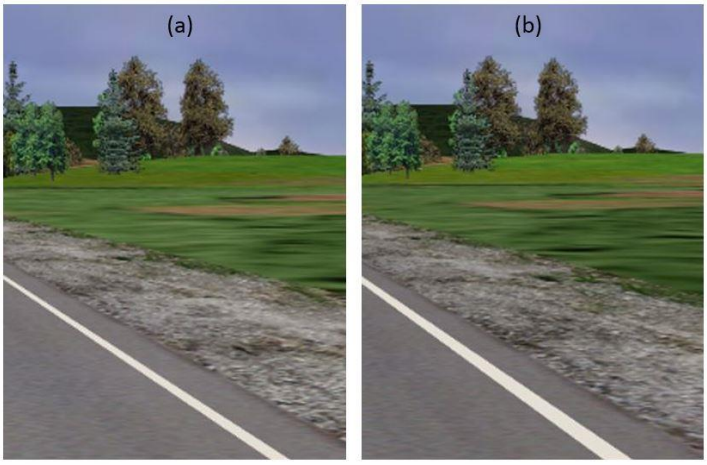


Figure 2 Driver Simulator Graphic of Edge Line Widths: (a) 4 Inch and (b) 6 Inch.

Figure 3 Driver Simulator Graphic of Edge Line Deterioration Percentages (only 6 inch shown): (a) 0 percent, (b) 25 percent, (c) 50 percent, and (d) 75 percent.

Driving Simulator Experiment – Description

The University of Idaho's driver simulator lab was used along with the NADS MiniSim program to display the simulations and to collect and record the data. A 2001 Chevrolet S10 pick-up truck cabin was used by participants to drive all of the scenarios. Forty-eight participants from the local community with unrestricted valid driver's licenses were recruited through online and bulletin board postings and participated in this research.

Each participant's goal was to keep the vehicle centered in the lane and to travel at an appropriate speed, just as in everyday driving. All participants were instructed that they would be completing a 40 mile drive on a rural highway for each session and to imagine themselves returning from a weekend camping trip in rural Idaho. Every participant was assigned and completed sessions that consisted of a unique ordering of two different edge line widths (4-inch 6 inch). Further, each width was composed of four uniquely ordered edge line deterioration percentages to minimize bias effects.

Before a participant began the experiment, the simulator vehicle was reset and centered on the travel lane with no lane deviation. When the vehicle moved toward the center of the roadway, a negative lane deviation value was generated. When the vehicle moved towards the edge line, a positive lane deviation value was generated. These lane deviation values were measured in feet. Accelerator pedal position and steering wheel angle data were also collected but their results did not factor into the final lane deviation results of this study.

Driver Simulator Experiment – Data Analysis and Results

Participant Information

Although forty-eight participants were initially selected for this study, preliminary analysis was only conducted using data from a reduced set of forty-four participants. This was due to the fact that some technical issues were encountered when converting the data format for three daytime participants and one nighttime participant; a fifth participant was identified as an outlier so this participant's data were manually removed from the analysis process.

Final data analysis was performed on the results from forty-three participants; there were twenty-eight male participants, and fifteen female participants. With regard to age, thirty-one participants were between eighteen and thirty years old, eight participants were between thirty-one and forty-nine years old, and four participants were between fifty and seventy years old. The youngest participant was a nineteen year old female, while the oldest participant was a sixty-nine year old male (M = 29.5 years, SD = 12.8 years). The average years of driving experience among the participants was 13.4 years (SD = 12.7 years).

Lane Position

The lateral position of the vehicle within the travel lane depended on driver maneuvering to keep the vehicle in a stable position between the centerline and edge line pavement markings. In this research, the lateral position of the vehicle was analyzed based on lane deviation and measured off of the centerline of the right lane per SAE International recommended practice.[17]

As an example, the average participant performance based on lane deviation for the 4-inch and 6-inch edge line width using the four deterioration scenarios (0 percent, 25 percent, 50 percent, and 75 percent) during nighttime conditions is plotted in Figure 4. The y-axis represents the lane deviation of the vehicle in feet (zero refers to no lateral movement where the vehicle would be in the center of the

lane), while the negative and positive values indicate the vehicle's deviation towards the centerline and edge line, respectively. The x-axis was divided into four sections based on the pavement marking degradation percentage (0 percent, 25 percent, 50 percent, and 75 percent) and each symbol represented a different pavement marking width (4 inch 6 inch). Each section, as described earlier, consisted of eight data collection points that represented a differing roadway geometry option (see Table 1). For example, points 6, 14, 22, and 30 represented the lane deviation of each participant when traveling along the same gentler, left-hand curve segment section. Based on the visual analysis of Figure 4, it can be concluded that all drivers tended to move toward the edge line of the right lane at night and increasingly shifted away from the centerline as edge line deterioration worsened; this could be a factor in run-off-the-road crashes. For all pavement marking degradation levels, a left-turn through the gentle curve segments caused the highest positive values on the lane deviation axis; in other words, drivers deviated the most toward the edge line under this roadway geometric condition versus all of the other roadway geometry options. Similar results were observed for drivers who experienced daytime conditions.

Table 1 Data Collection Points for Specific Geometry Types

Geometry Type	Data Collection Points
Straight Segments	1, 3, 5, 7, 9, 11, 13, 15, 17, 19, 21, 23, 25, 27, 29, and 31
Gentle Curved Segment (Turning Left)	6, 14, 22, and 30
Gentle Curved Segment (Turning Right)	4, 12, 20, and 28
Sharp Curved Segments	2, 8, 10, 16, 18, 24, 26, and 32

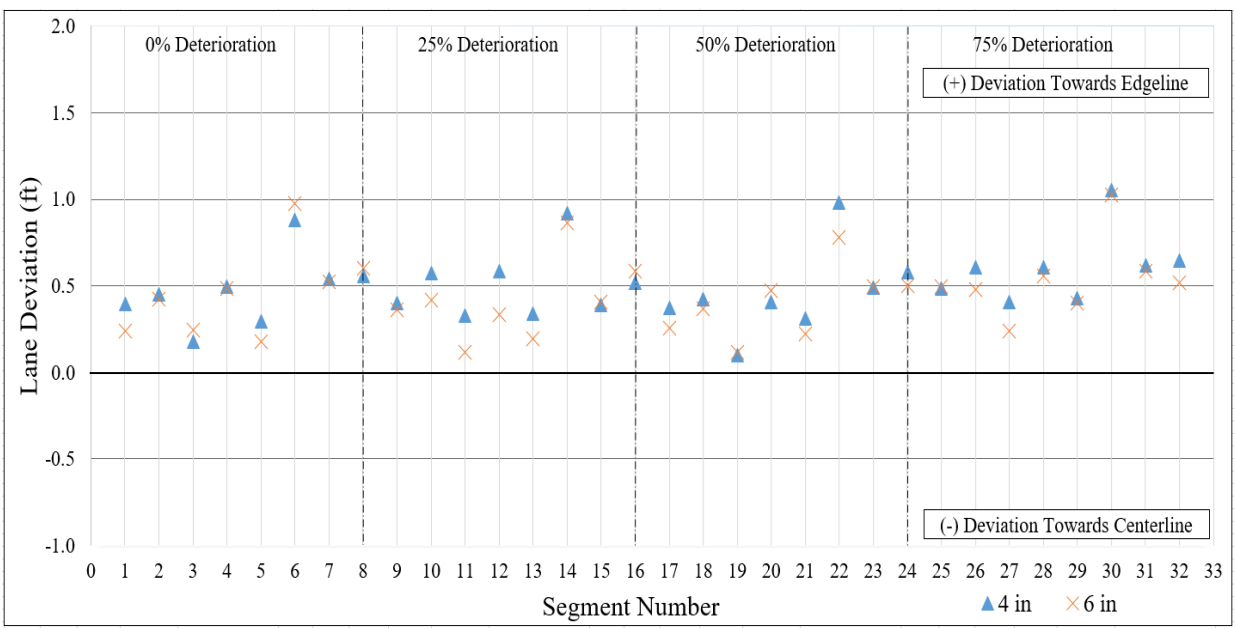


Figure 4 Effects of Pavement Marking Width and Deterioration on Driver Lane Deviation

A three-way analysis of variance (ANOVA) was conducted to understand and statistically describe the effect of edge line width, deterioration percentage, and roadway geometry on vehicle lane deviation. Table 2 summarizes the results when data from all participants were included; due to the value placed on pavement markings for drivers at night, Table 3 isolates the results for this particular segment of the driver population drivers (i.e., only those who experienced nighttime conditions). As shown in Table 2 and Table 3, the three-way ANOVA revealed significant differences only with the edge line deterioration percentage ($F(3, 2752) = 7.35$, $p = 6.6e\text{-}05$ and $F(3, 1472) = 3.30$, $p = 0.02$) and roadway geometry ($F(3, 2752) = 70.64$, $p < 2.2e\text{-}16$ and $F(3, 1472) = 42.27$, $p < 2E\text{-}16$) on lane deviation at the 0.05 significance level (type I error); this implied that the lateral position of the vehicle was impacted independently by edge line deterioration and roadway geometry. When nighttime drivers were isolated, marking width was added to the list of variables with statistically significant results ($F(1, 1472) = 5.52$, $p = 0.02$). For all cases the simultaneous interactions between edge line width and deterioration percentage, width and roadway geometry, and edge line width and deterioration percentages and roadway geometry did not have a significant impact on lane deviation.

Table 2 Effect of Edge Line Width, Deterioration Percentage, and Roadway Geometry on Vehicle Lane Deviation (All Drivers)

Variable	Df	Sum Sq	Mean Sq	F	Pr(>F)
Width (Edgeline)	1	1.29	1.287	2.7891	0.09502 .
Deterioration (Edgeline)	3	10.17	3.391	7.3521	6.61e-05***
Roadway Geometry	3	97.76	32.588	70.6448	< 2.2e-16***
Width : Deterioration	3	0.74	0.246	0.5328	0.65982
Width : Roadway Geometry	3	0.53	0.175	0.3802	0.76732
Deterioration : Roadway Geometry	9	1.3	0.145	0.3134	0.97092
Width : Deterioration : Roadway Geometry	9	0.76	0.084	0.182	0.99597
Residual	2752	1269.49	0.461		

Table 3 Effect of Edge Line Width, Deterioration Percentage, and Roadway Geometry on Vehicle Lane Deviation (Nighttime Drivers Only)

Variable	Df	Sum Sq	Mean Sq	F	Pr(>F)
Width (Edgeline)	1	2.23	2.2318	5.5232	0.0189*
Deterioration (Edgeline)	3	4.00	1.3349	3.3035	0.01961*
Roadway Geometry	3	51.25	17.0824	42.2747	< 2E-16 ***
Width : Deterioration	3	0.37	0.1244	0.3078	0.81974
Width : Roadway Geometry	3	0.04	0.0127	0.0314	0.99254
Deterioration : Roadway Geometry	9	0.45	0.0495	0.1224	0.99916
Width : Deterioration : Roadway Geometry	9	0.93	0.1033	0.2557	0.98567
Residual	1472	594.81	0.4041		

Figure 5 describes the cumulative impacts that edge line deterioration percentage had on lane deviation. As the percentage of edge line deterioration increased, lane deviation increased as well. When participants experienced 0 percent edge line deterioration the corresponding lane deviation ranged from 0.46 to 0.47 feet (14.0 to 14.3 centimeters) while at a 75 percent edge line deterioration the lane deviation increased to between 0.54 and 0.61 feet (16.5 to 18.6 centimeters). This higher edge line deterioration percentage did have an impact on lane deviation and was statistically reliable.

Figure 6 shows a graphical representation of the impact of edge line widths on lane deviation at specific roadway geometries. It can be observed that when the participants drove along the gentle curved segment (turning left) they experienced a higher lane deviation of 0.91 to 0.96 feet (27.7 to 29.3 centimeters) as compared to the other roadway geometries that had lane deviations from 0.32 to 0.54 feet (9.8 to 16.5 centimeters). Since the lane deviation values were universally positive, the results from this study implied that participants moved toward the edge line for all roadway geometry types.

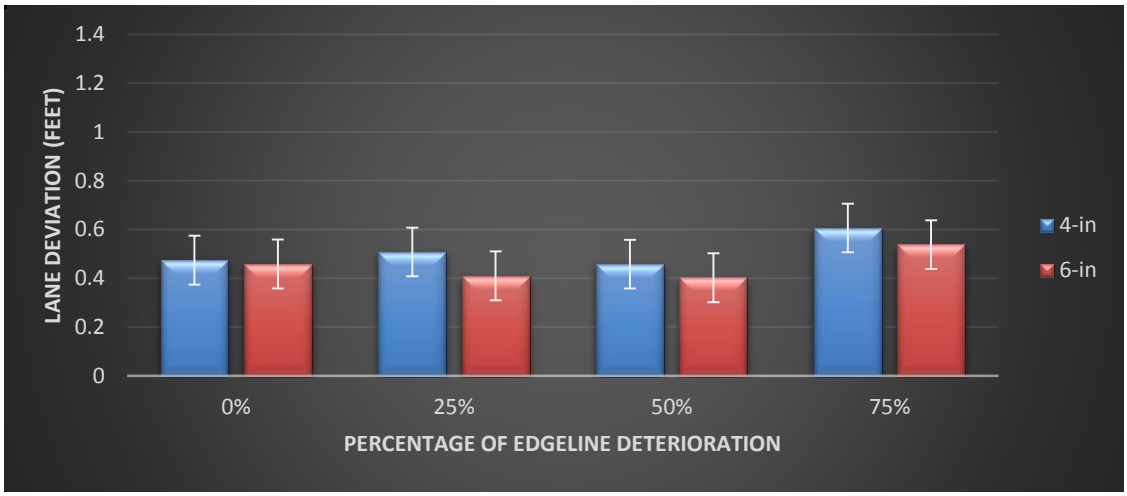


Figure 5 Lane Deviation: Impact of Edge Line Deterioration

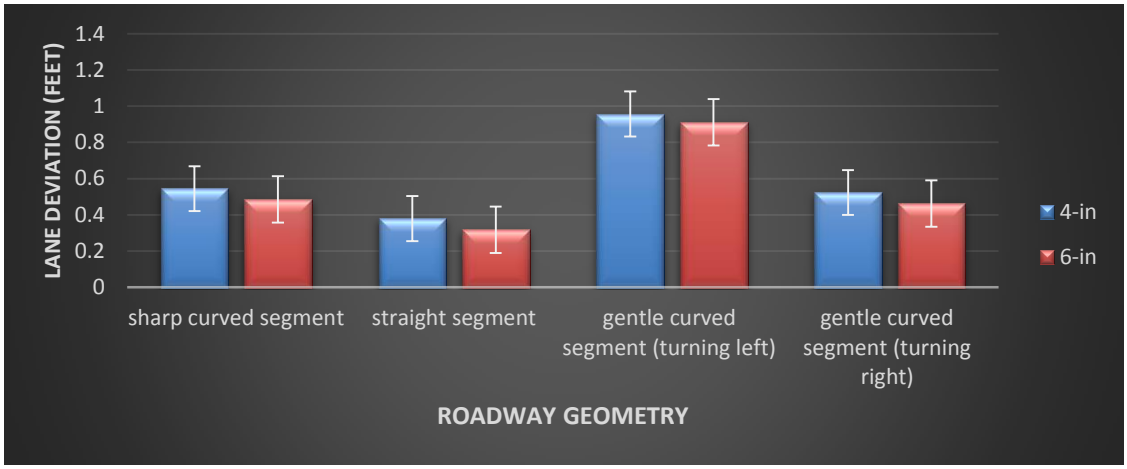


Figure 6 Lane Deviation: Impact of Different Roadway Geometries.

Comparison between Edge Line Widths and Deterioration Levels Based on Lane Deviation

One additional goal of this study was to determine if a driver would maintain similar lane position when encountering either a wider edge line width with a higher deterioration percentage or a narrower edge line width with less deterioration (such as a comparison between a 6 inch edge line with 75 percent deterioration and a 4-inch edge line with 50 percent deterioration). For this reason, an ANOVA with a single factor was conducted to compare specific edge line widths (4 and 6 inch) at different deterioration percentages. The results obtained were based on a 95 percent confidence interval.

Most of the comparisons performed showed that the means between differing conditions were statistically insignificant, which indicated that they resulted in the same lane deviation, regardless of edge line width or deterioration. The only exceptions were the comparisons between a 6-inch edge line with 75 percent deterioration and a 4-inch edge line with either 0 percent or 50 percent deterioration (p-value = 0.036 and 0.057 respectively). These cases implied that driver lane deviation behavior when encountering a 6-inch edge line with 50 percent or less deterioration was similar to the behavior exhibited when encountering a 4- inch edge line with no deterioration. However, these results also implied that drivers reacted similarly when encountering either a 6-inch edge line with 75 percent deterioration or a 4-inch edge line with 25 percent deterioration, so these findings were determined to be inconclusive at this time.

Driver Simulator Experiment Conclusions

The primary objective of this research was to study the effects of longitudinal edge line pavement marking width with varying deterioration levels and to assess the driver's ability to maintain lane position. The study results determined that longitudinal edge line pavement marking width alone does not affect lane deviation, but there is a correlation between deterioration levels and increased lane deviation from the centerline across different roadway geometry types. For this study, drivers consistently maintained a lane position that slightly favored the edge line side and increasingly shifted away from the centerline as edge line deterioration worsened.

The study examined the relationship between driver lane deviation and varying combinations of edge line pavement marking widths and deterioration levels, and the simulated environment encountered by all participants represented daytime and nighttime driving conditions. During real-world conditions with visible light, external factors such as signage, trees, and the presence of guardrail may impact driver behavior, though these elements were not simulated as part of this research. However, for these reasons, testing during nighttime conditions when such cues are not visible and when pavement markings are more heavily relied upon was included.

Edge line deterioration is an expected event that occurs due to weathering, plowing, and wearing from recurrent vehicle tire loading. The results of this study have shown that there are subtle differences in driver behavior that occur at different deterioration levels and different widths. For public agencies who

are responsible for the operations of these facilities, proper maintenance and upkeep of edge line markings, regardless of width, ensures that vehicle operators, under normal driving conditions, will be most likely to maintain lane position when visibility of these markings is highest.

Chapter 3
Pavement Markings Deterioration Models

Overview

Drivers rely on pavement markings to maintain a safe road path especially during nighttime and challenging weather conditions. The objective of this chapter's study was to model the deterioration of waterborne edge-line pavement markings in the field. Retroreflectivity data for the waterborne edge-line pavement markings from thirty-eight sites were collected and analyzed over twelve months in the State of Idaho across six districts with different environmental conditions. The results yielded a logarithmic relationship between retroreflectivity and age, and pavement markings in districts subjected to higher ground snow loads deteriorated faster than those with lower ground snow loads. In the laboratory, a three-wheel polisher device was used to evaluate the performance of waterborne pavement markings and the retroreflectivity, color change, and durability were evaluated at different wearing cycles. This study will benefit transportation agencies, particularly those sited in cold-weather regions, by enabling them to predict the deterioration of waterborne pavement markings and assist in the scheduling of marking maintenance projects.

Background and Literature Review

Pavement Markings Performance Indicators

Pavement markings provide key information to drivers about roadway alignment, vehicle positioning, and other driving-related task.[18] Two criteria are commonly used to evaluate marking performance over time: visibility and durability. The visibility of pavement markings refers to its brightness while durability measures resistance to damage due to traffic and environmental effects, and it is often quantified by the percentage of the remaining marking material on the pavement surface over time.[19]

Most pavement marking materials consist of three major components; binder, pigment, and retroreflective material. The binder refers to the adhesive element that sticks with the pavement surface and provides pavement markings with the ability to resist abrasion caused by traffic and road maintenance activities (e.g., snow plowing). The pigment refers to the color used, and the retroreflective material (such as glass beads) enhances the visibility of the pavement markings.[20]

Pavement marking materials can be classified as either nondurable or durable. Nondurable markings often have an expected service life of less than one year and include conventional solvent-based and water-based paints. Nondurable markings are identified by the solvent used, method of application, and drying time. On the other hand, durable markings include many chemical compounds such as epoxy, thermoplastics, methyl methacrylate, polyurea, polyurethane, and tape.[21],[22],[23] Choosing the appropriate pavement marking relies on several factors including line type (e.g., longitudinal,

transversal, or auxiliary line), pavement surface, traffic volume and speed, road type, snowfall, and lane geometry. Each department of transportation has its own criteria and guidelines when selecting the pavement marking material to meet its operating condition.[23] Approximately 98 percent of pavement markings used in the state of Idaho are waterborne with methyl methacrylate (MMA) and tape representing the remaining two percent.[24]

Waterborne Markings

Although paint has a shorter expected service life than other markings, this material is widely used on rural roads due to its inexpensive cost and eco-friendly attributes. Waterborne paints, in particular, are friendly to the environment and safer than solvent-based paints because they contain less Volatile Organic Compounds (VOC) and are characteristically less than 0.15 kilogram per liter (1.25 pounds per gallon) of VOC. The drying set-to-touch time for most waterborne paints are longer than solvent-based paint times especially when applied during high humidity weather. In practice, after exposure to traffic, weather, and snow plowing, waterborne markings wear off and lose their retroreflectivity faster than other pavement marking materials. For this reason, many studies have recommended that waterborne markings are more suitable for low-volume roads or used as an interim pavement marking material.[25]

Pavement Marking Performance Evaluation

Several studies have been sponsored by the Federal Highway Administration (FHWA) after the US Congress required the Manual of Uniform Traffic Control Devices (MUTCD) to establish a minimum requirement for highway pavement markings retroreflectivity. Until now, no final standards have been published in the MUTCD.[26],[27] The current practice is to construct a “testing deck” by applying different pavement marking materials across a roadway and then monitor the degradation (or wear) over time and under continuous traffic loading. The performance of the pavement material is monitored and reported for a period up to three years depending on the type of pavement marking material according to the National Transportation Product Evaluation Program (NTPEP).[28] This practice has several limitations given the dynamic nature of the pavement marking industry and changes to the chemical composition of pavement marking materials complying with Environmental Protection Agency (EPA) requirements. For this reason, there is value to developing an accelerated laboratory test procedure that can provide a shorter field-representative assessment of pavement marking material performance while maintaining system reliability. An accelerated laboratory test should simulate the environmental and operating conditions including those of the Pacific Northwest where snowplowing is common.

Some accelerated pavement marking wear testing simulators have been proposed and used as a research tool. Accelerated wear testing simulators are used to provide pre-qualifying pavement marking tests in controlled environments over a short time period.[29],[30],[31] These include the German Federal Highway Research Institute (BASt) wear simulator, the Asociacion para el Estudio de las Technologicas de Equipamiento de Carreteras (AETEC) wear simulator, the Model Mobile Load Simulator, third scale (MMLS3), and the Three Wheel Polisher Device (TWPD). The TWPD was originally designed to evaluate skid resistance of asphalt pavements; however, it was recently proposed to evaluate pavement markings

in a research study funded by the Illinois Department of Transportation and preliminary results demonstrated this method produces gradual deterioration of pavement markings. Although the NTPEP test deck provides an evaluation that has live traffic under ideal weather conditions found within the region of the test deck, three years or longer may be required to complete the test. On the other hand, laboratory accelerated wear testing can be achieved in a much shorter time, is less expensive, and is safer because road closure is not necessary.

To describe the degradation performance of pavement markings, mathematical models have been developed and proposed. These models include mathematical relationships of different forms including simple linear, power, exponential, natural logarithmic, quadratic, and multiple linear regression models. In some of these models, retroreflectivity was used as a dependent variable while variables such as age, traffic volume, weather conditions, and roadway geometry were used as independent variables. Other variables such as color retention and durability (material percent lost over time) have been used as performance measures to evaluate the marking degradation. Recent studies, 2012 to present, used the multiple linear regression model to predict pavement marking retroreflectivity degradation in different regions in the United States.[(20),(22)]

Study Goals and Objectives

This study evaluated the field performance of waterborne pavement markings across Idaho by using field data to develop a pavement marking model that describes retroreflectivity degradation over time and simulates an accelerated pavement marking performance evaluation by developing a laboratory-based test procedure. To achieve these objectives, several tasks were completed:
Pavement marking test sections were selected across six districts in Idaho with relatively different environmental conditions (e.g., snowfall). Retroreflectivity data of the pavement markings was collected over time to measure aging effects. Performance deterioration models for pavement markings in each district were developed using the retroreflectivity field data along with other collected information such as the date of marking application and ground snow loads.

Asphalt substrates were prepared, painted, and tested using a three-wheel polisher in a laboratory under different simulated loading conditions (e.g., pneumatic tires, steel tires, and steel plate that simulated snow plow removal). Retroreflectivity, color change, and durability using image analysis techniques were evaluated at different deterioration cycles. Laboratory testing results were compared and correlated with field measurements.

Study Methodology

Retroreflectivity Data Collection in the Field

A total of thirty-eight road segments on rural, two-lane highways in Idaho, corresponding to 552.6 kilometers (343.4 miles), were targeted as test sites. These test sections were chosen based on the: type of pavement markings (waterborne only), color of edge pavement markings (white only), location and climate (per district), pavement surface type (flexible pavements only), and traffic volume (less than 4,000 vehicles per day).

Pavement marking striping activities in Idaho usually start at the end of March and continue through early August. Waterborne white pavement markings in Idaho generally have a service life of one year so data collection replicated this time period though the exact timing of the restriping depended on the pavement marking condition, weather, and maintenance crew schedule. For quality assurance purposes, data collection trips were planned and managed to cover all targeted sites in sequential time periods throughout 2016 and 2017 to monitor the degradation of the pavement markings at the test sites. All retroreflectivity measurements were manually collected using a handheld MX 30 retroreflectometer (Figure 7). The data were collected by averaging three measurements on three different spots at each mile marker along the pavement markings of the targeted sites (nine total readings per marker). The average of these measurements was then entered into the database and assigned to each milepost along the road segments.

Figure 7 Measuring Retroreflectivity in the Field Test Sites Using the MX 30 Retroreflectometer.

Laboratory Procedure

Fifty-centimeter square asphalt slabs with a five-centimeter thickness (50 cm x 50cm x 5cm) were prepared and subsequently striped with pavement markings. The asphalt mixture used in this study was a typical dense graded mix used in Idaho. The mixture was prepared with basalt aggregates and PG 64-34 asphalt binder (5.5 percent by weight). Figure 8 shows the aggregate gradation of the mix. These slabs were prepared using a steel mold and plate compactor (Figures 9a and 9b). The slabs were then transported to the Idaho Transportation Department (ITD) office in Lewiston, ID and painted using the same pavement marking material (10.2 centimeter or 4-inch, white, waterborne paint) and machine used for actual roadway applications. The waterborne paint used in this procedure was one of the ITD preapproved products as identified on the most current Qualified Products List (color of category 707, sub-category No. 14 waterborne), and the striping truck was calibrated per ASTM D713-90. The pavement markings were applied to the slabs at a striping truck speed of approximately 8 kilometers per hour and bead dropping rate (glass bead dosage) of 0.96 kilogram per liter (8 pounds per gallon) to minimize loss of beads. The resulting wet paint thickness was 0.43 millimeters (17 milli-inch), standard for rural Idaho roadways (Figures 9c, 9d and 9e).

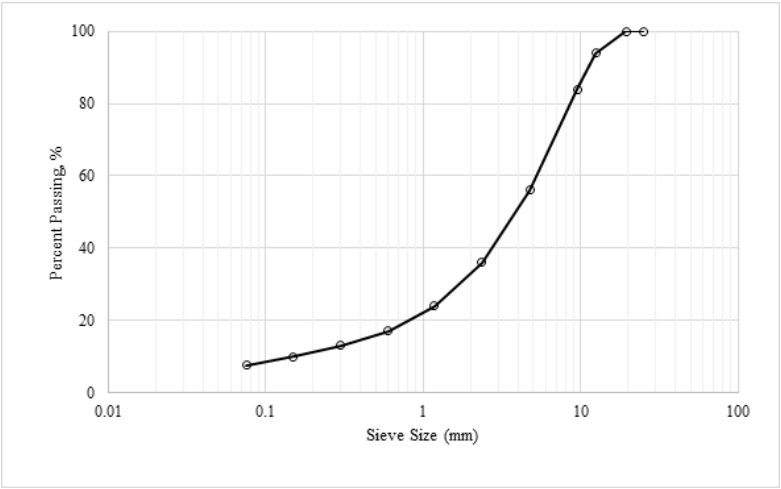


Figure 8 Aggregate Gradation of Asphalt Mixtures

Figure 9 (a, b, c) Sample Preparation, (d, e) Sample Painting, and (f) Laboratory Testing.

The waterborne pavement markings were subsequently subjected to wet accelerated wear testing. Different wheelsets (pneumatic, steel, and a combination of pneumatic wheels and scraper blade) were mounted on a TWPD. To replicate traffic in the laboratory environment, the TWPD was used to accelerate the "wearing out action" of the pavement markings to study degradation (Figure 9f). The TWPD polished a circular path using three caster wheels that tracked in 28 centimeters diameter circle (abrasion wheel assembly) on the top of the prepared slabs. The combination of pneumatic wheels with a scraper blade was proposed and used to simulate snow removal activity in this study.

All polishing tests were conducted at room temperature (25°C, 77°F). All samples were equally subjected to 100,000 cycles with different speeds. The speed was 50 revolutions per minute (rpm) when using the pneumatic tires, 30 rpm when using steel wheels, and 20 rpm when using the combination of the pneumatic tires and the scraper blade. The weight above the wheels was maintained to 15 Kilograms (33 pounds or three standard circular plates No. 10) The tire pressure of the pneumatic tires was maintained at 345 kilopascals (50 psi) during the laboratory testing.

To determine pavement marking performance over time, the surface affected by the wheel path of the TWPD was studied. The MX30 retroreflectometer captured a 10-centimeter by 9-centimeter area. Since the wheel path for the pneumatic wheels only covered an area 4-centimeters by 8-centimeters, retroreflectivity readings were taken with a consistently framed area and then corrected. The reflectivity readings taken from the reduced area were multiplied by an adjustment factor to be corrected. A similar adjustment was made for the steel wheel (6-centimeter by 8-centimeter) and pneumatic with steel wheels and scraper blade (4-centimeter by 8-centimeter) tests, and these correction factors were applied to the dry, recovery, and continuous wetting readings for each wheelset. Table 4 demonstrates the calculation of the correction factors for the 4-centimeter by 8-centimeter reduced area. Three slabs were evaluated at each measurement condition (dry, recovery, and continuous wetting). Five retroreflectivity measurements were conducted and averaged for each sample with and without the reduced area.

Table 4 Estimation of the 513 Reduced Measurement Area Correction Factor

Measurement Condition	*Slab Number*	*Average Full Retroreflectivity Measurement (mcd)*	*Average Reduced Area (4 cm x 8 cm) Retroreflectivity Measurement (mcd)*	*Percent Reduction*	*Average Percent Reduction*	*Correction Factor*
Dry	S1WW	314	140	55.41%	56.60%	1.566
	S2WW	303	141	53.47%		
	S3WW	307	120	60.91%		
Recovery	S1WW	125	90	28.00%	44.10%	1.441
	S9TW	102	48	52.94%		
	S8TW	111	54	51.35%		
Continuous Wetting	S1TW	51	20	60.78%	57.69%	1.577
	S9TW	77	32	58.44%		
	S8TW	78	36	53.85%		

The key factors for a snow plowing mechanism are the cutting angle and attack angle. The cutting angle is the rotation of the blade about the horizontal axis of the road surface and the attack angle is the rotation of the plow from the vertical axis. To simulate snow removal in the laboratory, a scraper blade was installed on the pneumatic wheels on the TWPD's turntable. The cutting angle was reversed to prevent the machine from stopping due to the interaction between the blade and an irregular asphalt texture (Figures 10a and 10b).

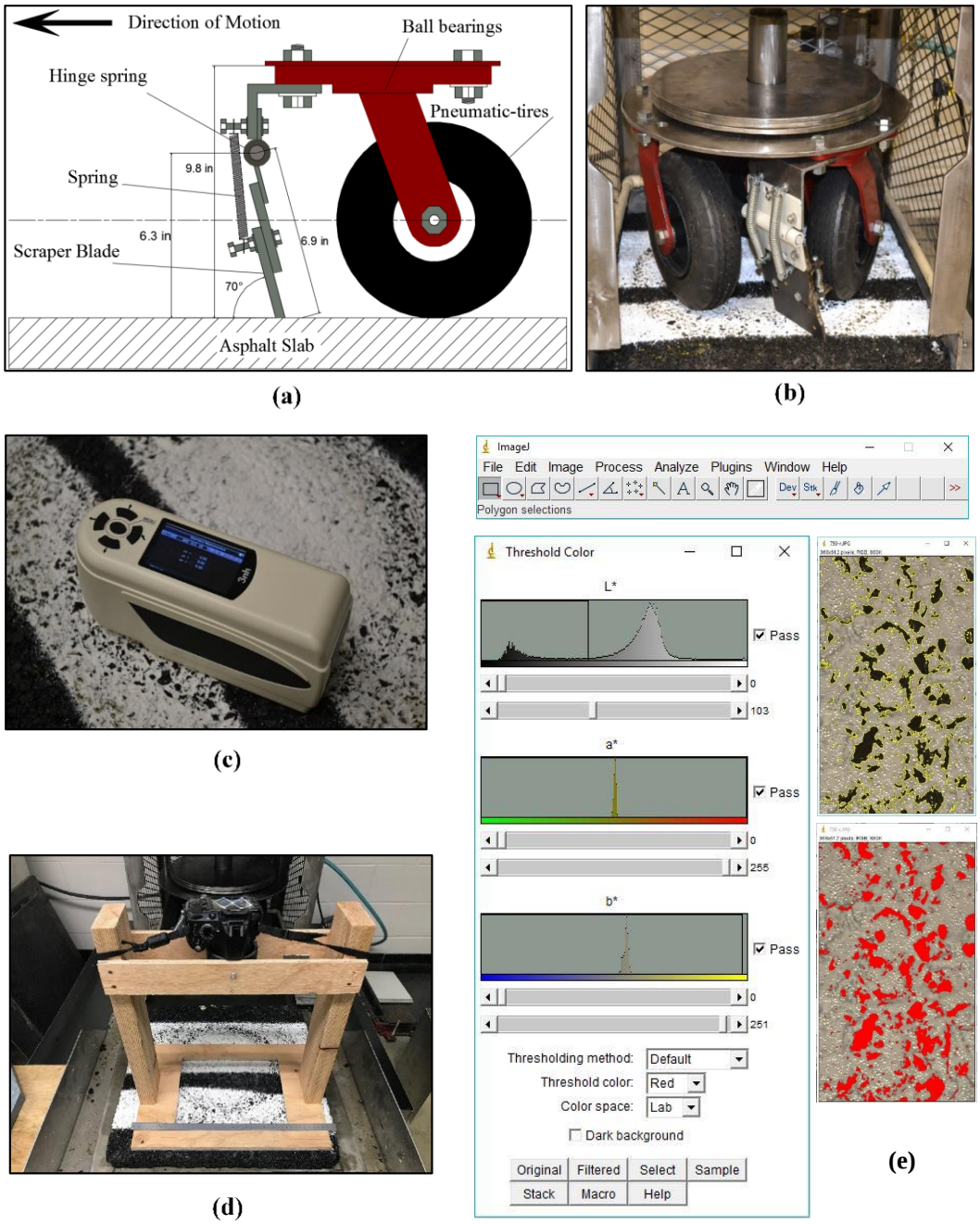


Figure 10 (a, b) Snowplow Simulator, (c) Colorimeter, (d) Camera Stand, and (e) Image Software Interface

Performance Measures

To evaluate pavement marking performance, the retroreflectivity, color retention, and presence performance characteristics were evaluated. These three measures are briefly discussed in this section.

1. Retroreflectivity of the worn markings (mcd/m^2/lx, or mcd) was measured using an MX30 portable retroreflectometer (30-m geometry). Three readings were conducted and averaged at two locations along the wheel path for each interval cycle. One reading was conducted in the middle and used as a reference point. The retroreflectivity was also measured in three conditions: dry (R_L) (ASTM E1710-11), recovery (RL1) (ASTM E-2177-11), and continuous wetting (R_{L2}) (ASTM E2176-08).

2. Color measurements of each surface were conducted using an NR200 high-quality portable colorimeter with an 8 millimeter diameter aperture in accordance with ASTM D-2244 (Figure 10c). The CIELab color coordinates (D65 light source) were used to measure the marking color retention. The CIELab color space was chosen to evaluate the color change because it was designed based on a concept similar to the opponent color processes of human vision. To identify the color difference using the CIELab 1976 coordinates, total change in color (ΔEab) was calculated using the following Euclidean distance equation: ($\Delta E_{ab} = [\Delta_L^2 + \Delta_a^2 + \Delta_b^2]^{½}$) and ($\Delta_L = [\Delta_{L\ Reference}^2 - \Delta_{L\ Sample}^2]^{1/2}$), where Δ_L, Δ_a, and Δ_b represented the differences between the initial and final values of L, a, and b, respectively. An increase in L indicated that the sample was illuminating (+ = lighter, - = darker). A color tended to be redder with an increase in a ($+\Delta_a$) and greener with a decrease ($-\Delta_a$), while a color tended to be more yellow with an increase in b ($+\Delta_b$) and bluer with a decrease (-Δb). The changes in lightness (Δ_L) and total color (Δ_{ab}) of the loaded pavement marking specimen surface at different exposure cycles was also monitored.

3. Presence performance (durability) was used to evaluate laboratory pavement marking performance. An image analysis procedure was applied using a high-resolution camera and the ImageJ v.1.50i software to measure marking material presence after each traffic loading interval using the TWPD. To standardize the imaging environment, the camera was mounted at a constant height in a fluorescent light environment. After each designated number of wearing cycles, images were taken using a high-resolution camera. For consistency, all the images were taken from the same height and have the same resolution. Figures 10d and 10e show the camera stand and image analysis. The images were processed and analyzed using the ImageJ software to quantify the area of pavement markings that got worn due to wearing. Image analysis techniques were used in this study to estimate the percent loss of pavement markings with wearing cycles. ASTM D6359-99 and ASTM D7585 / D7585M were used as guidance to assess this performance measure. A durability rating procedure was used to determine the remaining marking materials percentage (where zero equated to complete material loss and 100 equated to all material still remaining).

Analysis and Results

Retroreflectivity Deterioration Models

Retroreflectivity data from the field test sites were analyzed to study the deterioration of pavement markings in each district of Idaho. Figure 11a shows an example of the data collection along each mile of the test section. In this example, the retroreflectivity measurements were collected right after 1, 5, and 11 months of painting. A logarithmic model was found to provide the highest r-squared for the relationship between pavement marking age and retroreflectivity loss. Such a logarithmic relationship was used to study the gradual change in retroreflectivity with time. Figure 11b illustrates the relationship between pavement marking age and retroreflectivity loss for multiple test sections in District 5 in Idaho. For this data set, the deterioration curves, decreasing retroreflectivity over time did not significantly vary between test sites so the average deterioration curve of all test sites (shown as the dashed line) served as a good representation for the entire district. An average curve was developed using the same method for the other five districts (Figure 11c). Table 5 shows the pavement marking retroreflectivity decay equations for each district graphed in Figure 11c (x represents the number of months after painting).

Table 5 Monthly Retroreflectivity Decay Equation per District

District	*Equation*
1	y = -96.25ln(x) + 258.99
2	y = -112.9ln(x) + 293.4
3	y = -87.22ln(x) + 313.67
4	y = -65.7ln(x) + 288.52
5	y = -81.2ln(x) + 286.50
6	y = -80.89ln(x) + 275.14

Figure 12a shows the normalized ground snow loads for Idaho based on the 2015 snow load map data.[32] Previous research has shown that pavement markings deteriorate at a higher rate in colder climates, and winter maintenance activities and harsh weather are the most dominant factors contributing to pavement marking deterioration.[33],[34] Figure 12b shows a relationship between the deterioration rate of pavement marking retroreflectivity and the weighted average normalized ground snow load (NGSL) for all districts for Idaho. The NGSL was calculated based on the snow load at each measurement site divided by the elevation of the station in feet to normalize the data to pounds per square foot per foot (psf/ft). Districts 1 and 2 had higher NGSLs (3.26 percent to 4.18 percent psf/ft) compared to Districts 3, 4, 5, and 6, (1.28 percent to 1.72 percent psf/ft). This relationship illustrated that a higher NGSL caused greater deterioration (loss) in retroreflectivity, which was likely attributed to the increase in winter maintenance activities (e.g., snowplowing). The slope (which represents the deterioration rate of pavement marking retroreflectivity) were calculated from the logarithmic deterioration equations for each district separately. The normalized ground snow load (NGSL) per each

district was calculated using ArcGIS 10.5.1. The resulting NGSL percentage was weighted for the area and averaged to be compared with other districts.

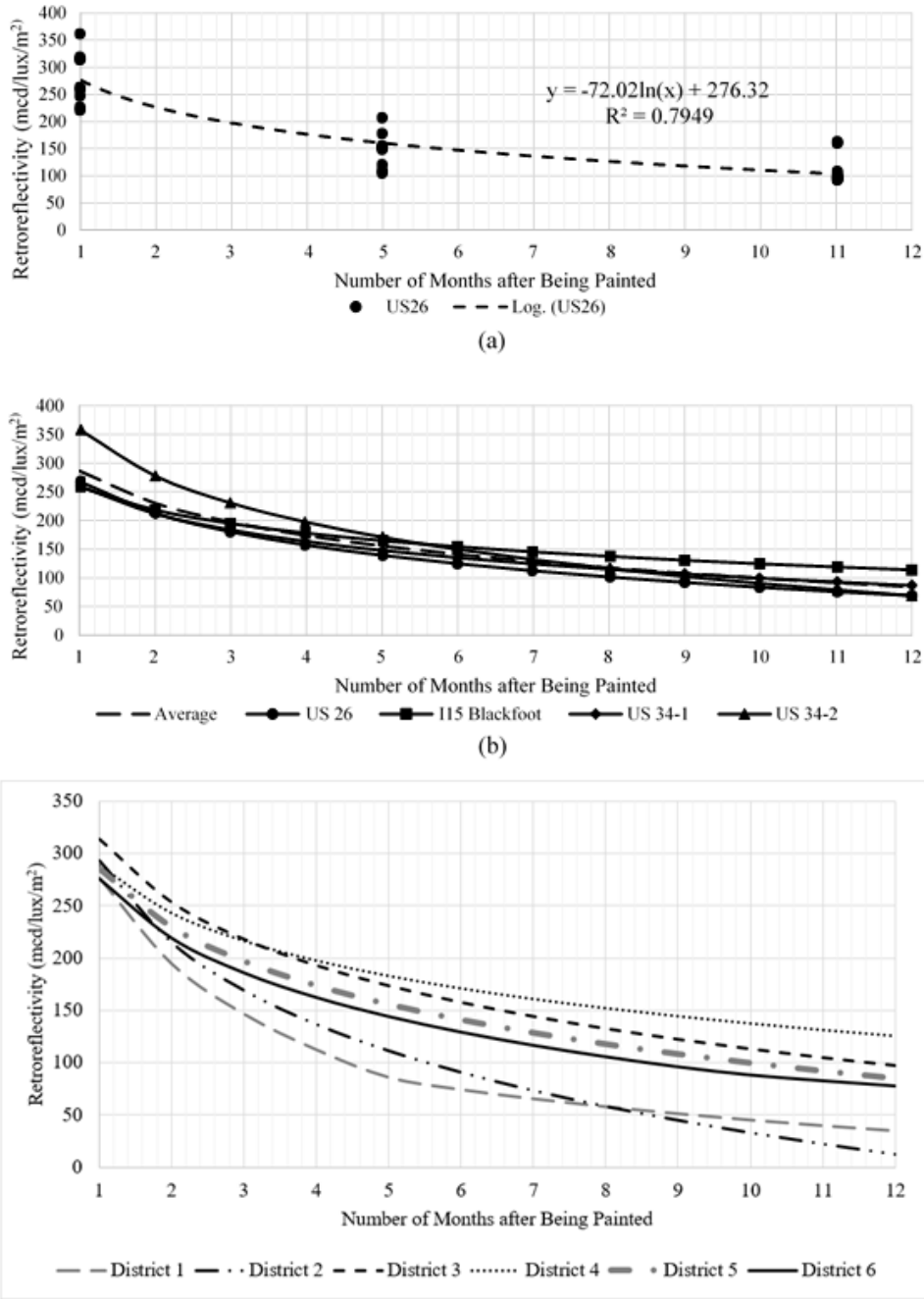
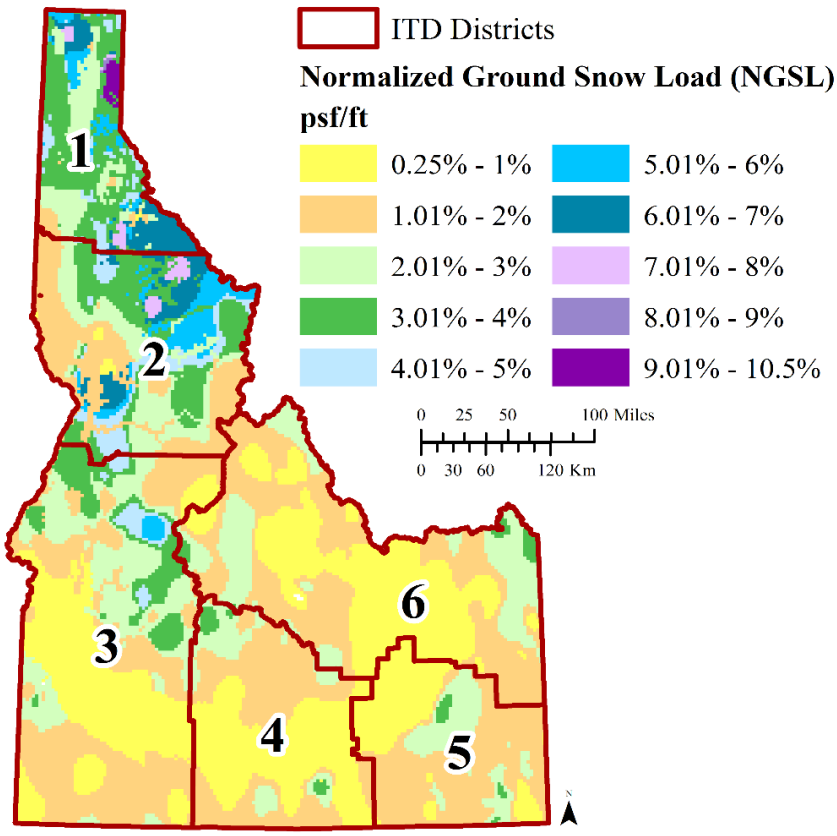
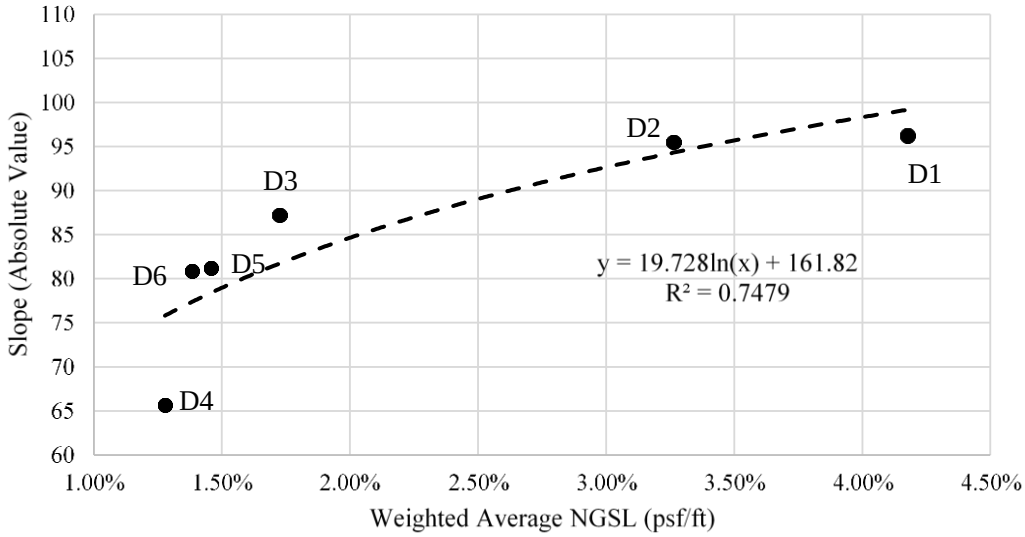


Figure 11 (a) An Example of Data Collection, (b) Change in Retroreflectivity with Time in District 5, and (c) Retroreflectivity Prediction per Month after Paint per District.



(a)

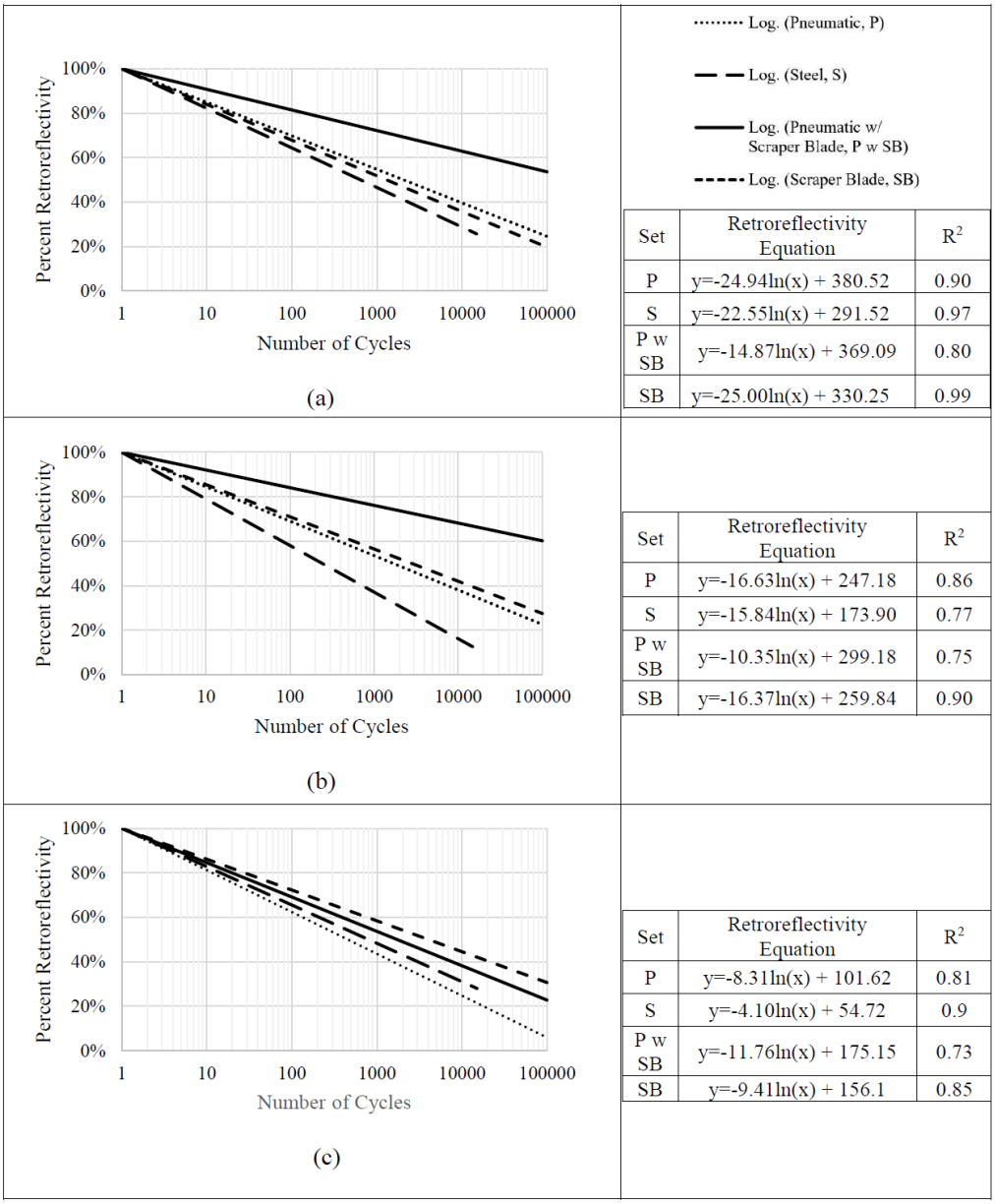


(b)

Figure 12 (a) NGSL for Different Districts in Idaho and (b) Correlation Between NGSLs and Deterioration Rate of Pavement Marking Retroreflectivity.

Evaluation of Pavement Marking Performance under Accelerated Laboratory Testing

Figure 13 shows the R_L retroreflectivity readings for the different loading or blade types versus the number of cycles. The correlation coefficient, R^2, for the pneumatic, steel, pneumatic with scraper blade, and scraper blade were 0.90, 0.97, 0.80, and 0.99 respectively, with the section where the pneumatic and scraper blade passed over having the least deterioration. This correlation is related to how the retroreflectivity equations developed relate to the actual lab data. The graphs display the deterioration based on percent retroreflectivity lost. The pneumatic wheelset deteriorated the pavement marking 75 percent from its initial retroreflectivity after 100,000 cycles.



Set	Retroreflectivity Equation	R^2
P	y=-24.94ln(x) + 380.52	0.90
S	y=-22.55ln(x) + 291.52	0.97
P w SB	y=-14.87ln(x) + 369.09	0.80
SB	y=-25.00ln(x) + 330.25	0.99

Set	Retroreflectivity Equation	R^2
P	y=-16.63ln(x) + 247.18	0.86
S	y=-15.84ln(x) + 173.90	0.77
P w SB	y=-10.35ln(x) + 299.18	0.75
SB	y=-16.37ln(x) + 259.84	0.90

Set	Retroreflectivity Equation	R^2
P	y=-8.31ln(x) + 101.62	0.81
S	y=-4.10ln(x) + 54.72	0.9
P w SB	y=-11.76ln(x) + 175.15	0.73
SB	y=-9.41ln(x) + 156.1	0.85

Figure 13 Percent Retroreflectivity Deterioration Due to Different Loadings and Conditions: (a) Dry R_L, (b) Recovery R_{L1}, and (c) Continuous Wetting R_{L2}.

The R_{L1} percent retroreflectivity readings versus the number of cycles under recovery conditions is shown in Figure 13b. The correlation coefficients, R2, for the pneumatic, steel, pneumatic with scraper blade, and scraper blade were 0.86, 0.77, 0.75, and 0.90 respectively. These correlation coefficients were less than the dry percent readings due to the variability caused by taking readings when the pavement markings were wet. The pneumatic wheel with the scraper blade had the lowest rate of degradation while the steel wheels had the highest rate. Another observation from this data set was that the pneumatic wheels only and scraper blade only have very similar rates of degradation based on the logarithmic decay functions and could be attributed to pavement surface wetness.

Figure 13c shows the R_{L2} percent retroreflectivity readings versus the number of cycles under continuous wetting conditions. The correlation coefficients for the pneumatic, steel, pneumatic with scraper blade, and scraper blade were 0.81, 0.90, 0.73, and 0.85, respectively. Based on laboratory testing observations, the voids on top of the pavement surface were filling with water and affecting the continuous wetting readings for each tested condition. The change in percent retroreflectivity in continuous wetting was relatively close for the different wearing methods.

Figures 14a, 14b, and 14c show the imprints of the pneumatic, steel, and a combination of pneumatic wheels and scraper blade wheelsets used in the experiment, respectively. Figure 8c shows the two paths (A and B) that were evaluated on the pavement marking sample that was polished using the combination of pneumatic tires and the scraper blade. After 1000 cycles, it was observed that the scraper blade had decreased the pavement marking retroreflectivity by 0.194 mcd for the dry, 0.161 for the recovery, and 0.041 for the continuous wetting readings. If the snow plowing event deterioration of a pavement marking retroreflectivity in one pass was available, this field snow pass deterioration value could be divided by 0.194 in order to determine a relationship between the lab and field data. This number could then be multiplied by the number of snow plowing events in a year to predict how much the markings would deteriorate over the winter.

Correlation Between Field and Laboratory Retroreflectivity Deterioration Models

In order to relate the retroreflectivity deterioration field models to the laboratory models, Figure 15 was used to compare the cumulative number of traffic passes (CTP) in the field with the estimated retroreflectivity. The pneumatic, steel, pneumatic with scraper blade, and only scraper blade laboratory tests were completed in an attempt to establish this relationship. A similar trend between retroreflectivity versus CTP and retroreflectivity versus number of cycles from the TWPD with pneumatic wheels was observed. Based on the average field retroreflectivity value for each district after 100,000 CTPs and the dry pneumatic (RL) results in the laboratory after 100,000 cycles, it was determined that 1.58 cycles (one cycle of TWPD has three tire hits) of the TWPD was equivalent to one CTP in the field.

Since the scraper blade caused the pavement markings to deteriorate on the slab, it can be said that the act of snow plowing in the field causes deterioration in pavement markings. However, since field data capturing the number of snow plowing events on each segment was not available, the number of

scraper blade passes in the lab could not be equated to a corresponding number of actual passes in the field at this time.

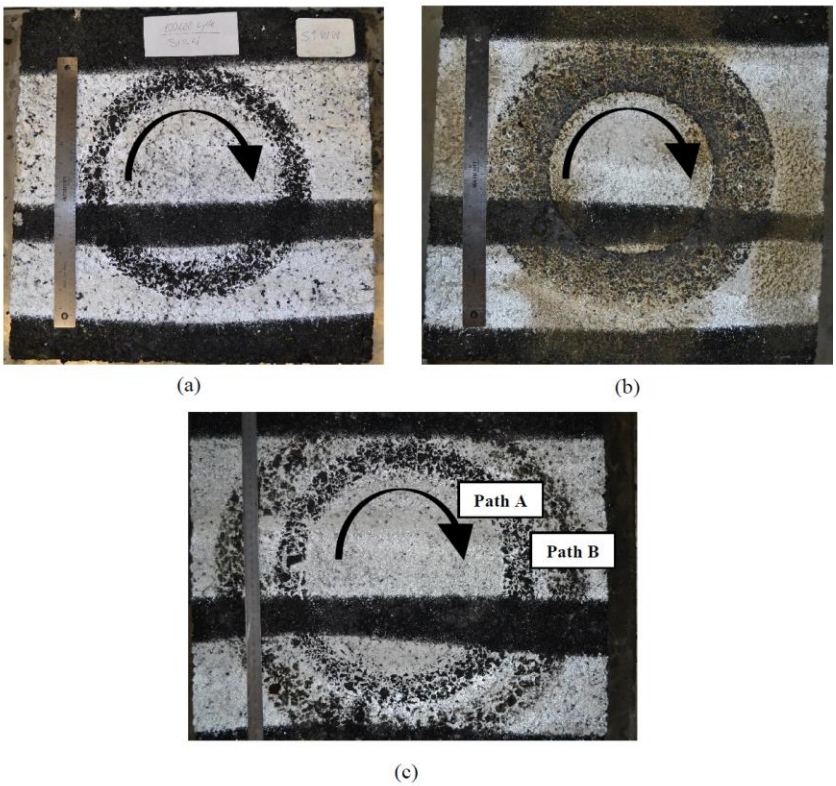


Figure 14 Waterborne Markings Polished Samples, (a) the Pneumatic Wheelset Imprint, (b) Steel Wheelset Imprint, and (c) the Combination of the Pneumatic Wheels and the Scraper Blade Imprint.

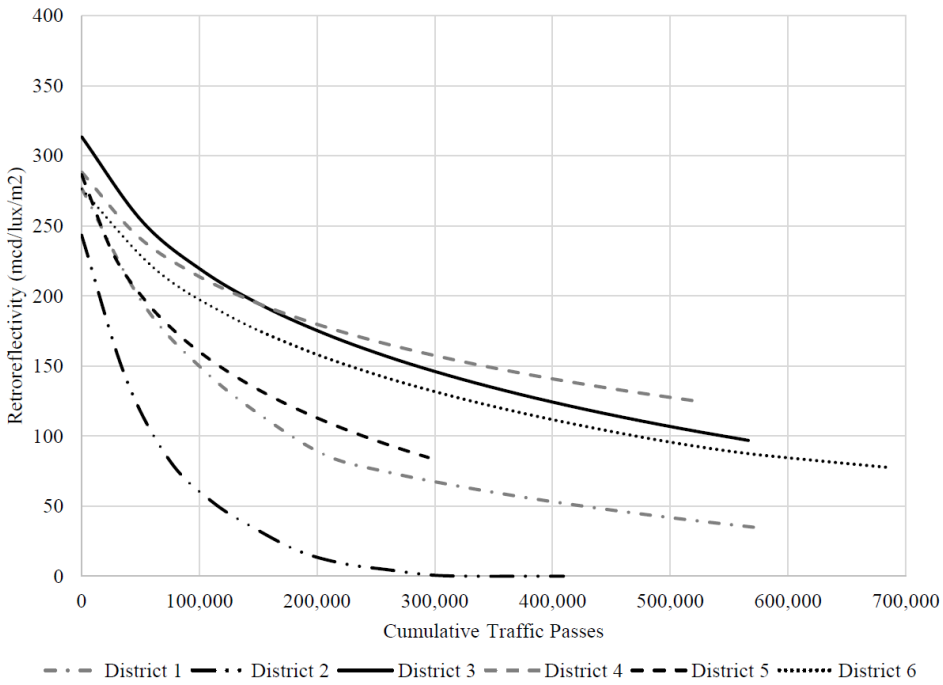


Figure 15 Retroreflectivity Prediction per CTPs.

Color Change Analysis

Figures 16a and 16b show the changes in lightness (Δ_L) and total color (ΔE_{ab}) of the waterborne pavement marking surfaces at different wheelsets and loading intervals for all of the traffic loading types. The steel wheelset had the highest Δ_L and ΔE_{ab} after loading, a likely result caused by compaction of the markings rather than a wearing effect. The Δ_L and ΔE_{ab} results from the pneumatic, pneumatic with scraper blade, and scraper blade only were similar as their ΔE_{ab} ranged from 35 to 40 after applying 100,000 cycles but the steel wheelset reached this level within 10,000 cycles. In other words, the steel wheelset was more abrasive than other wheelsets. A gradual change in Δ_L and ΔE_{ab} was observed throughout the experiment. ΔL for the pneumatic, pneumatic with the scraper blade, and scraper blade followed the same trend as that of ΔE_{ab} at 10,000 to 100,000 cycles of exposure. Both ΔL and ΔE_{ab} increased for all wheelsets up to 10,000 cycles for the steel wheels and 100,000 cycles for the others, and the logarithmic scale showed a drastic increase in Δ_L and ΔE_{ab} between 10,000 and 100,000 cycles of exposure.

The color of the markings darkened when the number of cycles using the pneumatic wheels increased. The main reason the markings lost lightness and changed their color to black was due to the wearing of the pavement markings from the tire simulating traffic and the appearance of the asphalt background. When the tire rotated over the pavement surface, the rubber on the outside of the tire was scraped off and onto the pavement surface. These results were consistent with other trials of the same material under the same conditions. Initial traffic loading of waterborne marking surfaces resulted in color darkening and further traffic loading caused stability in color. The running wheels essentially polished

the pavement markings located on the upper peaks of asphalt texture while the lower peaks still retaining marking material.

Percent Loss Analysis

Figure 16c shows the results of the image analysis for percent loss versus the number of cycles. The correlation coefficients for the pneumatic, steel, pneumatic with scraper blade, and scraper blade were 0.92, 0.98, 0.84, and 0.92, respectively. The pneumatic wheelset did not experience any loss in presence until roughly two hundred cycles from the TWPD. As expected, the steel wheels caused the most rapid deterioration with a 75 percent loss after only 10,000 cycles. The scraper blade, on the other hand, caused the least percent loss and was attributed to the blade dragging along the surface; as the blade drug over the marking, the markings wore off quicker than any other wheelset but after completely removing all of the top markings, a small change in retroreflectivity was observed after 10,000 cycles. The rutting from the pneumatic, steel, and pneumatic with scraper blade resulted in an increase in percent loss compared to just the scraper blade.

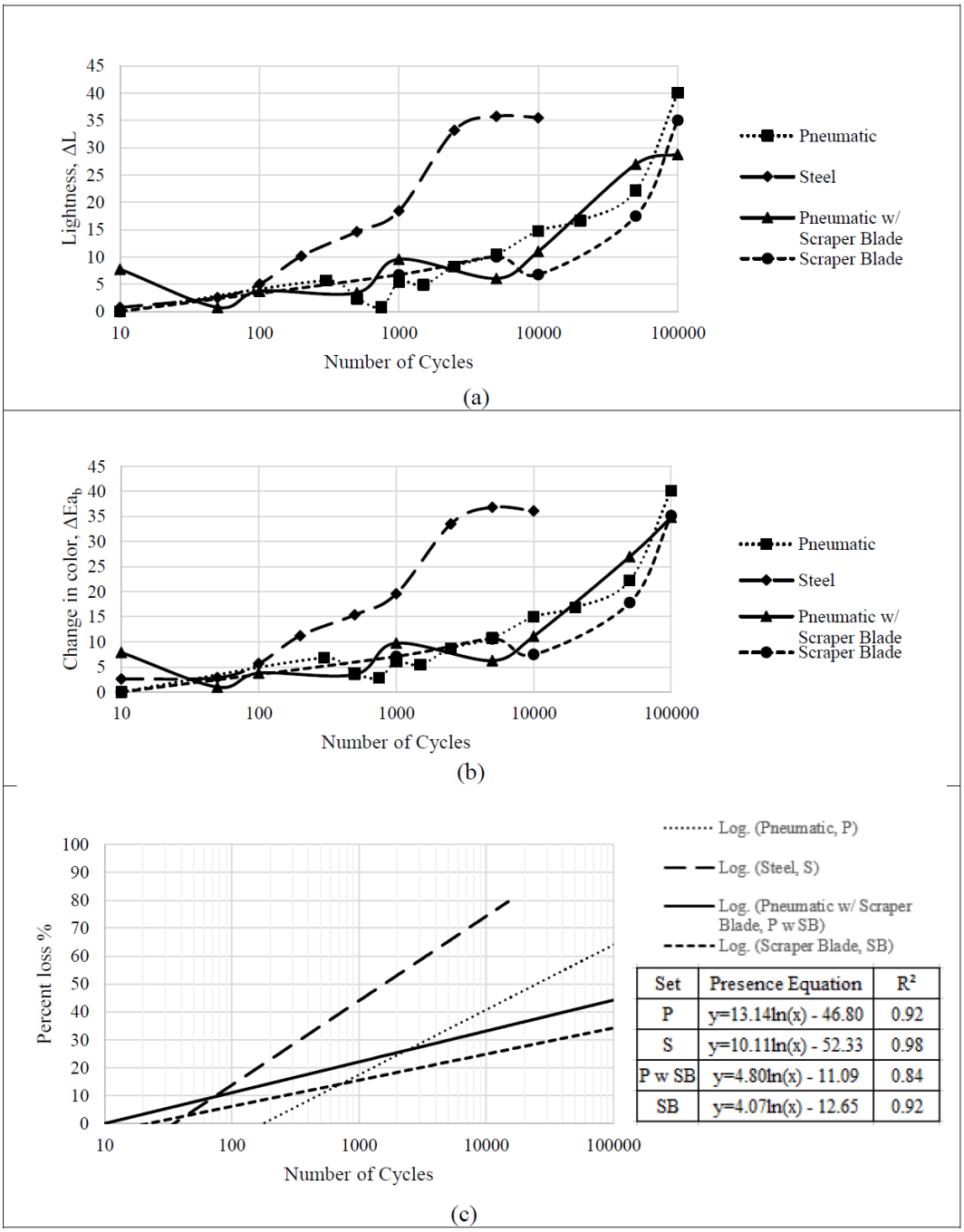


Figure 16 Changes in (a) Lightness, (b) Color Change, and (c) Presence Due to Different Loading Conditions.

Comparison Between the 6-inch and 4-inch Edge Line Retroreflectivity

Figure 17 shows the retroreflectivity comparison for the test sites (with 6-inch wide edge line pavement markings) and the control sites (with standard 4-inch markings) over one year, the typical life span of water-based pavement markings. As seen in the equations on the chart, the 6-inch edge line markings degrade at a relatively slower rate than the 4-inch-wide markings did. However, the difference in

retroreflectivity degradation rate between the 6-inch and the 4-inch edge line markings decreases with time.

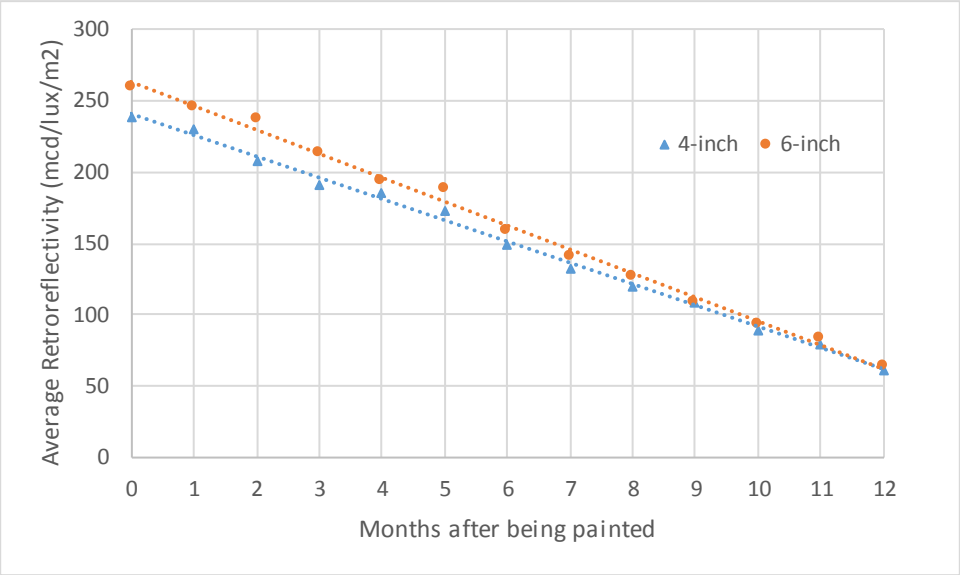


Figure 17 6-inch Versus 4-inch Edge Line Pavement Markings Retroreflectivity Deterioration Trends

Pavement Retroreflectivity Deterioration Study Conclusions

Waterborne marking retroreflectivity deterioration was modeled using field and laboratory data. Field retroreflectivity data for each targeted rural road in Idaho was collected at three different times over twelve months to establish deterioration curves. The endurance of waterborne markings under different conditions were tested and correlated with field data. Retroreflectivity deterioration was modeled in the laboratory using TWPD under four loading scenarios (pneumatic, steel, pneumatic with scraper blade, and scraper blade). Color retention and percent loss due to the same loading scenarios were monitored and used to validate the results from the retroreflectivity data. The following conclusions can be drawn from this data.

- The results show that the best fit curve to predict waterborne pavement markings retroreflectivity deterioration in Idaho was a logarithmic decay function.
- There were strong relationship between the deterioration rate of pavement marking retroreflectivity and the weighted average NGSL for all districts in Idaho. The higher the NGSL, the faster deterioration (loss) in retroreflectivity. This could be attributed to the increase in winter maintenance activities (e.g. snowplowing).
- The TWPD was used successfully to replicate traffic to study and evaluate the waterborne pavement marking degradation. The TWPD can be used to simulate the wearing of traffic similar to the NTPEP test deck currently used in the field to evaluate the wearing of pavement markings. However, the TWPD was much more beneficial since it was less expensive, easier to operate, and took less time to complete.

- Based on the average field retroreflectivity value for each district and the dry pneumatic (RL) results in the laboratory, 1.58 cycles of the TWPD is equivalent to one CTP in the field.
- The recovery and continuous wetting readings were used to observe the variability in retroreflectivity readings when water was present on the surface of the pavement marking. This performance measure data decreased with an increasing number of cycles, similar to the dry readings.
- An increase in lightness (Δ_L), total color change (ΔE_{ab}), and percent loss was directly correlated to a loss in retroreflectivity.
- The 6-inch edge line markings degrade at a relatively slower rate than the 4-inch-wide markings did. However, the difference in retroreflectivity degradation rate between the 6-inch and the 4-inch edge line markings decreases with time.

Due to the success of using the TWPD for simulating traffic loading, it is recommended that future research include the effect of aging and weathering on the pavement marking materials. This could be tested using the laboratory oven and an accelerated weathering machine. Other pavement markings attributes such as color and type of marking (e.g. thermoplastic, tape, MMA) could also be tested and compared to the white waterborne samples that were used in this study. In addition, the effect of anti-icing and de-icing substances on the pavement marking materials as well as climate conditions (test temperature) may also be quantified through the laboratory testing. The TWPD can be equipped with an environmental chamber to simulate the performance of pavement markings in different regions. The addition of snowplow field data to relate to the scraper blade would also be beneficial to equate the number of laboratory scraper blade passes with the number of snowplow passes in the field.

Chapter 4
Before and After Crash Analysis

Overview

The results of a before-and-after study to examine the safety effect of wider pavement markings are presented in this chapter. Two different methods were employed: before-and-after comparison group analysis and Empirical Bayes before-and-after analysis.

Background and Literature Review

The width of the edge line pavement marking can be an effective factor in improving visibility especially at night and during low visibility conditions. According to the Manual of Uniform Traffic Control Devices (MUTCD)[35], the minimum width of pavement markings is 4 inches. However, some agencies are implementing wider pavement markings to provide better visibility, particularly at locations that have high lane departure crashes, such as sharp horizontal curves or complex intersections. Based on survey data collected from different state agencies, the main reason for implementing wider longitudinal pavement markings was improved visibility (identified by 57 percent of respondents). The second reason was as an older driver countermeasure (19 percent), and the third reason was crash reduction (14 percent).[25]

Two survey studies have been conducted to document the state-of-practices for wider pavement markings. The first one was in 2002, and it found that wider longitudinal pavement markings are very common in the Eastern United States. Twenty-two out of twenty-six states from states located east of the Mississippi River are using wider pavement markings to some degree but only seven out of twenty-four sates located west of the Mississippi River are using them. A follow-up study in 2009 found an increase of five states which using wider markings. Benefit-cost studies found that the benefit-cost ratio of wider edge-lines showed that the reduction due to widening the pavement markings will result in Return On Investment (ROI) ranging from $33.00 for every $1.00 invested to $55.00 for every $1 invested.[36]

Earlier studies that investigating the safety improvements of wider pavement markings, such as Cottrell (1986)[37], Hall (1987)[38], and Hughes (1989)[39] reported either no conclusive results or no significant crash reductions as a result of the implementation of wider markings. On the other hand, more recent studies have showed stronger evidences that might support the positive effects of the wider markings usage. A study carried out by Texas Department of Transportation (TxDOT) in 2005 to investigate the effect of edge lines on safety and driver behavior on rural two-lane highways has found that the implementation of edge pavement markings on two-lane rural roadways has reduced crash frequency up to 26 percent. [40]

In 2012, safety effects of wider edge-lines on rural two-lane highways were realistically examined in the states of Illinois, Michigan, and Kansas. The states of Michigan and Kansas increased the edge-line width

from 4 inches to 6 inches during the analysis, while Illinois only increased it from 4 inches to 5 inches. Each state performed different statistical analysis on crash data that were obtained before and after implementing wider edge-lines. Even though the three states conducted different analyses, the study found that wider edge-lines reduce vehicle crashes. The highest crash reduction percentage was the reduction in fatal and severe injury crashes (Kansas (36.5 percent), Michigan (15.4 percent), and Illinois (37.7 percent)[41]. Table 6 lists previous studies that used before and after crash analysis to investigate the safety effect of wider pavement markings. The Table presents summary of their findings as well as the studies limitations.[40]

Wider edge-lines help drivers under alcohol impaired conditions to further identify the roadway delineation on two-lane rural highways. Sixteen male participants (students between 21 and 25 years old) drove over sections of a rural highway in northern New Jersey that was composed of no edge-lines, 4-inch, 6-inch, and 8-inch edge-lines under sober and alcohol impaired conditions. Each participant drove twice, once with a zero Blood Alcohol Concentration (BAC) level and also with levels of either 0.05 or 0.08 BAC. These test drives occurred between midnight and 3:00AM with the help of police officers that controlled traffic by closing these highway sections. During this study, lateral lane position was recorded photographically every 100 feet to analyze driver performance. Wider edge-lines were found to provide benefits when compared with standard width (4 inch), especially the wider 8-inch edge-line, whereas no reduction in variability occurred at some instances with the 6-inch edge-line.[42]

Another study was conducted to confirm if the use of wider edge lines would benefit drivers under normal and impaired conditions. As part of the methodology for the study, twelve male participants with driver licenses and in the age range of 21 to 55 years old were hired. Each participant completed six experimental sessions at BAC levels of 0.00 percent, 0.07 percent, and 0.12 percent (above the legal BAC level to drive in most states). Each participant encountered curves, obstacles, and road signs as part of their driving task. The design of the roadway simulation session was composed of different edge line widths (none, 4 inch, and 8-inch), spot treatments, and curves. For this study an Analysis of Variance (ANOVA) was conducted. The study found that in a sober condition the wider edge-lines (8 inch) were associated with greater lateral lane position error than standard edge-lines (4 inch); however, neither edge line was significantly different from the no edge-line condition. On the other hand, participants with a high BAC level (0.12 percent) had greater lateral lane position error when there was no edge-lines, with the error decreasing as the edge-lines got wider.[43]

Table 6 Summary of the Methodologies, Results, and Limitations of Previous Studies[40]

Year	Author	Methodology	Location	Results	Limitations
1986	Cottrell Jr, Benjamin H	Naïve before–after crash	Virginia	There were no statistically significant differences between the 4- and 8-in. wide edge lines from the analysis of variance of lateral placement, lateral placement variance, encroachments by automobiles and trucks, mean speed, and speed variance.	Hampered by insufficient data and lack of experimental control.
1987	Hall, J. W.	Naïve before–after crash	New Mexico	Compared 8 in to 4 in markings and concluded that wide edge-lines do not have a significant effect on the incidence of ROR accidents. Wider lines have no safety benefit in terms of reducing crashes.	Hampered by insufficient data and lack of experimental control.
1989	Hughes, W E McGee, H W Hussain, S Keegel, J	Before-versus -after experimental design with a control group	Alabama, Maine, Massachusetts, New Mexico, Ohio, South Dakota, and Texas	Rural roads with 5,000 and 10,000 vehicles per day no reduction in crash frequencies have been noticed but rural roads 24-foot-wide rural roadways with less than six-foot shoulders and ADT between 2,000 and 5,000 experienced a relative decrease in total crash rate, total crash frequency and injury/fatal crash rate.	
2009	Paul J. Carlson, Eun Sug Park, and Carl K. Andersen	Empirical Bayes before–after evaluations negative binomial regression model	Illinois, Michigan, and Kansas	No safety improvement has been detected when 4 in markings replaced with 8 in markings on two-lane rural roadways with between 5,000 and 10,000 vpd.	The study is still ongoing during that time, not final results yet.
2012	Eun Sug Parka, Paul J. Carlsona, Richard J. Porterb, Carl K. Andersenc	Empirical Bayes, before-after analysis for Kansas data, a generalized linear segmented regression analysis for Michigan data, and a cross-sectional analysis for Illinois data.	Illinois, Michigan, and Kansas	The safety impact of using wider edge lines was statistically significant and reduction in single-vehicle crashes on rural, two-lane highways has been detected when using wider edge lines.	This study did not include nighttime traffic volumes in nighttime crash models.

Methodology

Crash and Exposure Measures Data

An Idaho vehicle collision report (VCR) must be completed by the local law enforcement agency for every crash in Idaho that involves a motor vehicle, occurs on public property, and results in more than $1,500 in property damage for any 1 person involved (prior to January 1, 2006 - $750), or results in an injury to any person involved. All VCR forms must be sent to ITD's Office of Highway Safety (OHS), which maintains the state's crash database. Crash data, used in this analysis, were obtained from ITD's OHS crash database through a web-based crash analysis interface (WebCARS).[43] This online database, developed and maintained by ITD's OHS, provided the required crash data for each selected segment used in the analysis. In addition to crash data, the geometric characteristics of two-lane rural state highways in Idaho were obtained from ITD's OHS. The data included lane width, shoulder width, and shoulder type. Vehicle exposure data, in the form of AADT, were obtained from ITD's Automatic Traffic Recorders (ATRs).[44]

Lane departure crashes, such as ROR and opposite-direction crashes, have been identified as the type of crashes that are most impacted by low quality of road delineation. The most harmful events that can contribute to the roadway departure crashes vary, but most of them are related to visual deficiencies, loss of control, and misjudgment of reactions. In this study, only crashes that may have occurred as a result of poor edge line pavement markings were includes in the analysis. The criteria for selecting these crashes includes: crashes that occurred on rural two-way two-lane highways (excluding intersection crashes and crashes that occurred within city limits), daytime and nighttime crashes, dry and wet pavement surface, asleep, drowsy, and fatigued crashes, drunk or impaired driving crashes, and careless and distracted driving crashes. Crashes that occurred on icy or snowy surfaces and crashes related to animal collisions were excluded. After the removal of non-targeted crashes, the remaining crashes were primarily hit fixed object, hit other object, and overturn crashes.

Thirty-eight two-lane rural highways in Idaho were selected as test sites for this study. Eighty five percent of the test sites included in the study were randomly selected two-lane rural highway sections. The reminder 15 percent of the test sections were sections that experienced high percentages of lane departure crashes. The test sites, containing 175.39 miles were pained with a wide (6-inch) white edge line pavement markings. The control sites, totaling 168 miles, had regular (4-inch) white edge line pavement markings. Full details of the test and control sections included in this study are presented in Appendix A.

The road segment crash rate can be calculated using the following Equation:

$$R = \frac{100\ million * C}{365 * N * V * L}$$

Figure 18: Crash Rate Equation

Where: R = Crash rate for the road segment expressed as crashes per 100 million vehicle-miles of travel (VMT), C = Total number of crashes in the study period, N = Number of years of data, V = Number of vehicles per day (both directions), and L = Length of the roadway segment in miles.

Before and After Crash Data Analysis

Before and After Comparison Group Analysis

Before and after comparison group analysis incorporates comparison group or control sites in the safety evaluation to estimate the crash frequencies that would have occurred on the test sites if no safety treatment had been implemented. Suitability tests for the control sites can be determined by comparing prior crash counts of the control sites with the test sites using sample odd ratio, a measure of correlation between crash occurrence in the test and control sites.

The expected crash count for the test sites that would have occurred in the after period without treatment ($N_{Observed,T,A}$) is estimated from the following Equation:

$$N_{Expected,T,A} = N_{Observed,T,B} * \frac{N_{Observed,C,A}}{N_{Observed,C,B}}$$

Figure 19: Expected Crash Count for the Test Sites Without Treatment Equation

Where:

- $N_{Observed,T,B}$ = the observed number of crashes in the before period for the treatment group.
- $N_{Observed,T,A}$= the observed number of crashes in the after period for the treatment group.
- $N_{Observed,C,B}$ = the observed number of crashes in the before period in the comparison group.
- $N_{Observed,C,A}$= the observed number of crashes in the after period in the comparison group.

The variance of the estimates of the expected crash counts at the test sites during the after period can be calculated from the following Equation:

$$Variance\ (N_{Expected,T,A}) = {N_{Obs,T,A}}^2 * (\frac{1}{N_{Obs,T,B}} + \frac{1}{N_{Obs,C,B}} + \frac{1}{N_{Obs,C,A}})$$

Figure 20: Variance of the Estimates of the Expected Crash Counts

Empirical Bayes Before-and-After Analysis

The Empirical Bayes method is also used to estimates expected number of crashes in the after period for the treatment group. Like the comparison group method, the summation of the estimates of $N_{expected,T,A}$ for all test sites is used to evaluate the safety effect of the treatment implemented. This statistical method requires AADT and crash data for the before and after periods for test sites. The use of the

Empirical Bayes method addresses and mitigate the bias due to regression-to-the-mean. In the Empirical Bayes method, Safety Performance Functions (SPFs) are used to calculate the expected crash frequency at the test sites had no treatment applied. Several statistical studies have been used to derive the SPFs such as Poisson, Negative Binomial (generalized linear regression models (GLM)), Zero-inflated Poisson, Conway–Maxwell–Poisson, Gamma, and Negative multinomial models.[45]

Safety performance functions (SPFs) are crash prediction models (regression equation) that correlate the expected crash frequency for a specific transportation facility to site physical characteristics. Poisson and Negative Binomial are common models used in developing SPFs because they can prevent the possibility of getting a negative integer crash value over time. The following is the SPF used in this analysis:

$$N_{spf} = L \times \exp\left[\beta_0 + \beta_1 \times ln\,(AADT)\right]$$

Figure 21: Safety Performance Function Equation

Where the AADTs are annual average daily traffic and β_o, and β_1 are model parameters estimated during the SPF development through regression analysis.

A biased estimate of the safety effectiveness (λ) of the treatment can be obtained as A/B where A stands for the actual after-period crashes at the treatment sites and B stands for the expected after-period crashes at the treatment sites had there been no treatment. An unbiased estimate (μ) of safety-effectiveness of the treatment is obtained as follows:

$$\mu = \frac{\lambda}{1 + \left(\frac{Var(\tau)}{B^2}\right)}$$

Figure 22: Unbiased Estimate of Safety Effectivness Equation

Where:

- μ = unbiased estimate of treatment effectiveness,
- λ=A/B,
- B = expected after-period crashes had there been no treatment, and
- Var(τ) = sum of variances of after-period estimate at all treatment sites.

Analysis and Results

Before and After Comparison Group Analysis

A before-and-after comparison group analysis was conducted to examine the potential safety effect of wider pavement marking. Paired t-tests were used to compare the average number of crashes and average crash rates during the before and after periods. The t-test analysis have been conducted for four groups of crashes: total crashes, total night crashes, fatal and severe injury crashes, and fatal and severe injury night crashes. The before period covered the five-year from 2010 to 2014 (before the

implementation of the wide pavement markings treatment) and the after period covered crashes that occurred during 2016.

Table 7 shows the results of the comparison group before-and-after t-test for average crash frequencies (crash/mile/year). A P-value of 0.05 or less indicates that the differences between the before and after crashes values are statistically significant at the 95 percent confidence level. The results show that the average number of crashes decreased by 3.31 percent and the fatal and severe injury crashes decreased by 22.58 percent after the implementation of wider pavement markings. These reductions, however, are not statistically significant at the 95 percent confidence level. While night crashes experienced a marginal increase (1.89 percent) after the implementation of wider pavement markings, fatal and severe injury night crashes showed a statistically significant reduction of 23.53 percent as a result of wider pavement marking implementation.

Table 7 Comparison Group Before-and-After T-Test Results for Average Crash Frequencies

Crash type	Before (2010-2014)	After (2016)	Percent Change	P-Value
Total Crashes	1.21	1.17	-3.31	0.279
Night Crashes	0.53	0.54	1.89	0.173
Fatal and severe injury crashes	0.31	0.24	-22.58	0.126
Fatal and severe injury night crashes	0.17	0.13	-23.53	0.005

Table 8 shows the results of the comparison group before-and-after t-test for crash rates (crash/million mile travelled /year). The results show that the average crash rate for total crashes, night crashes, fatal and severe injury crashes, and fatal and severe injury night crashes decreased after the implementation of wide pavement marking. The percent crash rate reduction for total crashes, night crashes, and fatal and severe injury crashes are 12.22 percent, 8.81 percent, and 18.68 percent, respectively. These crash rate reductions, however, are not statistically significant at the 95 percent confidence level (the fatal and severe injury crash rate reduction of 18.68 percent is statistically significant at the 90 percent confidence level). Again, fatal and severe injury night crashes showed a statistically significant crash rate reduction, at the 95 percent confidence interval, of 39.39 percent as a result of wider pavement marking implementation.

Table 8 Comparison Group Before-and-After T-Test Results for Average Crash Rate

Crash type	Before (2010-2014)	After (2016)	Percent Change	P-Value
Total Crashes	63.84	56.04	-12.22	0.148
Night Crashes	26.46	24.13	-8.81	0.261
Fatal and sever injury crashes	17.67	14.37	-18.68	0.095
Fatal and severe injury night crashes	8.85	5.32	-39.89	0.006

Empirical Bayes Before-and-After Analysis

An important component of the Empirical Bayes before-and-after analysis is the development of Safety performance functions (SPFs) for the test sections. SPFs are used to predict crash frequency for a given set of site conditions. The predicted crashes from the SPF can be used alone or in combination with the site-specific crash history (i.e., Empirical Bayes method) to compare the safety performance of a specific site under various conditions. The Empirical Bayes method is used to estimate the expected long-term crash experience, which is a weighted average of the observed crashes at the site of interest and the predicted crashes from SPFs. In this study, Negative Binomial regression was used to develop the SPF for the test sites using the equation: $Nspf = L \times \exp[\beta 0 + \beta 1 \times ln(AADT)]$, where β0 and β1 are regressions coefficients.

The 2016 crash data for an extended sample of two-lane rural highways control sites were used to develop SPFs. The output of the Negative Binominal regression for the SPF for total crashes and fatal and severe injury crashes are presented in Table 9. The Pseudo R^2 values for the SPF for the night crashes and the fatal and severe injury night crashes were relatively low (0.21 and 0.27, respectively), accordingly, they were not included in the EB analysis.

Table 9 SPF Model Statistics

Model	Parameter	Estimate	Std. Error	95% Confidence Interval		Pseudo R2
				Lower Bound	Upper Bound	
Total Crashes	β0	-5.648	0.830	-6.892	-3.572	0.439
	β1	0.609	0.094	0.799	0.423	
Fatal and Severe Injury Crashes	β0	-9.109	0.930	-10.969	-5.763	0.401
	β1	0.773	0.102	0.977	0.569	

A comparison of the observed and predicted crash frequencies and crash rates for total crashes and fatal and severe injury crashes is presented in Table 10. For crash frequency, the Empirical Bayes unbiased

estimates for the reduction of crashes as a result of the implementation of wider pavement markings are 17 percent and 14 percent for total crashes and fatal and severe injury crashes, respectively. For crash rates, these reductions are 5.53 percent and 12.59 percent, respectively. The reduction in crash rates for total crashes in statistically significant at the 90 percent confidence level. The reduction in crash rates for fatal and severe injury crashes is statistically significant at the 95 percent confidence level.

Table 10 Comparison Between Observed and SPF Estimated Crash Frequencies and Rates

Meaures	Crash Type	Observed Crashes (2016)	SPF Estimated Crshes	Percent Reducation	P-Value
Crash Frequency (crash/mile/year)	Total Crashes	1.17	1.41	-17.26	0.125
	Fatal and Serious Injuries Crashes	0.24	0.28	-14.29	0.103
Crash Rate (crash/MMT/year)	Total Crashes	56.04	59.32	-5.53	0.081
	Fatal and Serious Injuries Crashes	14.37	16.44	-12.59	0.054

Conclusions

The results of a before-and-after study to examine the safety effect of wider pavement markings was presented in this chapter. Two different methods were employed: before-and-after comparison group analysis and Empirical Bayes before-and-after analysis.

The results of the comparison group analysis show that the average crash rate for total crashes, night crashes, fatal and severe injury crashes, and fatal and severe injury night crashes decreased after the implementation of wide pavement marking. The percent crash rate reduction for total crashes, night crashes, and fatal and severe injury crashes are 12.22 percent, 8.81 percent, and 18.68 percent, respectively. These crash rate reductions, however, are not statistically significant at the 95 percent confidence level (the fatal and severe injury crash rate reduction of 18.68 percent is statistically significant at the 90 percent confidence level). Again, fatal and severe injury night crashes showed a statistically significant crash rate reduction, at the 95 percent confidence interval, of 39.39 percent as a result of wider pavement marking implementation.

The results of Empirical Bayes analysis were consistent with the results obtained from the comparison group analysis. For crash frequency, the Empirical Bayes unbiased estimates for the reduction of crashes as a result of the implementation of wider pavement markings are 17 percent and 14 percent for total crashes and fatal and severe injury crashes, respectively. For crash rates, these reductions are 5.53 percent and 12.59 percent, respectively. The reduction in crash rates for total crashes in statistically significant at the 90 percent confidence level. The reduction in crash rates for fatal and severe injury crashes is statistically significant at the 95 percent confidence level.

Chapter 5
Summary and Conclusions

This project evaluated the safety effects of wider pavement markings on rural two-lane highways in Idaho using two analyses: 1) Observational Before-and-After Studies, and 2) Driver Simulation-Based Studies. The study also examined and modeled the deterioration characteristics of pavement marking using in Idaho highways.

The objective of the driver simulator experiment was to study the effects of longitudinal edge line pavement marking width with varying deterioration levels and to assess the driver's ability to maintain lane position. The study results determined that longitudinal edge line pavement marking width alone does not affect lane deviation, but there is a correlation between deterioration levels and increased lane deviation from the centerline across different roadway geometry types. For this study, drivers consistently maintained a lane position that slightly favored the edge line side and increasingly shifted away from the centerline as edge line deterioration worsened.

The driver simulator study examined the relationship between driver lane deviation and varying combinations of edge line pavement marking widths and deterioration levels, and the simulated environment encountered by all participants represented daytime and nighttime driving conditions. During real-world conditions with visible light, external factors such as signage, trees, and the presence of guardrail may impact driver behavior, though these elements were not simulated as part of this research. However, for these reasons, testing during nighttime conditions when such cues are not visible and when pavement markings are more heavily relied upon was included.

Edge line deterioration is an expected event that occurs due to weathering, plowing, and wearing from recurrent vehicle tire loading. The results of this study have shown that there are subtle differences in driver behavior that occur at different deterioration levels and different widths. For public agencies who are responsible for the operations of these facilities, proper maintenance and upkeep of edge line markings, regardless of width, ensures that vehicle operators, under normal driving conditions, will be most likely to maintain lane position when visibility of these markings is highest.

Waterborne marking retroreflectivity deterioration was modeled using field and laboratory data. Field retroreflectivity data for each targeted rural road in Idaho was collected at three different times over twelve months to establish deterioration curves. The endurance of waterborne markings under different conditions were tested and correlated with field data. Retroreflectivity deterioration was modeled in the laboratory using a two wheel polishing device (TWPD) under four loading scenarios (pneumatic, steel, pneumatic with scraper blade, and scraper blade). Color retention and percent loss due to the same loading scenarios were monitored and used to validate the results from the retroreflectivity data. The following conclusions can be drawn from this data.

- The results show that the best fit curve to predict waterborne pavement markings retroreflectivity deterioration in Idaho was a logarithmic decay function.
- There were strong relationship between the deterioration rate of pavement marking retroreflectivity and the weighted average NGSL for all districts in Idaho. The higher the NGSL, the faster deterioration (loss) in retroreflectivity. This could be attributed to the increase in winter maintenance activities (e.g. snowplowing).
- The TWPD was used successfully to replicate traffic to study and evaluate the waterborne pavement marking degradation. The TWPD can be used to simulate the wearing of traffic similar to the NTPEP test deck currently used in the field to evaluate the wearing of pavement markings. However, the TWPD was much more beneficial since it was less expensive, easier to operate, and took less time to complete.
- Based on the average field retroreflectivity value for each district and the dry pneumatic (RL) results in the laboratory, 1.58 cycles of the TWPD is equivalent to one CTP in the field.
- The recovery and continuous wetting readings were used to observe the variability in retroreflectivity readings when water was present on the surface of the pavement marking. This performance measure data decreased with an increasing number of cycles, similar to the dry readings.
- An increase in lightness (Δ_L), total color change (ΔE_{ab}), and percent loss was directly correlated to a loss in retroreflectivity.

Due to the success of using the TWPD for simulating traffic loading, it is recommended that future research include the effect of aging and weathering on the pavement marking materials. This could be tested using the laboratory oven and an accelerated weathering machine. Other pavement markings attributes such as color and type of marking (e.g. thermoplastic, tape, MMA) could also be tested and compared to the white waterborne samples that were used in this study. In addition, the effect of anti-icing and de-icing substances on the pavement marking materials as well as climate conditions (test temperature) may also be quantified through the laboratory testing. The TWPD can be equipped with an environmental chamber to simulate the performance of pavement markings in different regions. The addition of snowplow field data to relate to the scraper blade would also be beneficial to equate the number of laboratory scraper blade passes with the number of snowplow passes in the field.

The results of a before-and-after study to examine the safety effect of wider pavement markings was presented in this chapter. Two different methods were employed: before-and-after comparison group analysis and Empirical Bayes before-and-after analysis.

The results of the comparison group analysis show that the average crash rate for total crashes, night crashes, fatal and severe injury crashes, and fatal and severe injury night crashes decreased after the implementation of wide pavement marking. The percent crash rate reduction for total crashes, night crashes, and fatal and severe injury crashes are 12.22 percent, 8.81 percent, and 18.68 percent, respectively. These crash rate reductions, however, are not statistically significant at the 95 percent confidence level (the fatal and severe injury crash rate reduction of 18.68 percent is statistically significant at the 90 percent confidence level). Again, fatal and severe injury night crashes showed a

statistically significant crash rate reduction, at the 95 percent confidence interval, of 39.39 percent as a result of wider pavement marking implementation.

The results of Empirical Bayes analysis were consistent with the results obtained from the comparison group analysis. For crash frequency, the Empirical Bayes unbiased estimates for the reduction of crashes as a result of the implementation of wider pavement markings are 17 percent and 14 percent for total crashes and fatal and severe injury crashes, respectively. For crash rates, these reductions are 5.53 percent and 12.59 percent, respectively. The reduction in crash rates for total crashes in statistically significant at the 90 percent confidence level. The reduction in crash rates for fatal and severe injury crashes is statistically significant at the 95 percent confidence level.

The 2012 to 2016 Crash data shows a total of 173 fatal and 533 serious injury ROR crashes in two-lane rural highways in Idaho, with an average cost of approximately $382.05 million a year. With an estimated additional cost of $0.04 per lane-foot to increase the width of the right shoulder edgeline pavement marking from 4 inch to 6 inch, the total additional cost of painting the two lane rural highway segments in Idaho (approximately 4,600 miles) with 6-inch edgline pavement marking, instead of the currently used 4-inch markings, is approximately $1.95 M. Wide pavement marking implementation has the potential to reduce fatal and serious ROR crashes by 12.59 percent with an expected cost savings of $48.1 million. The cost to benefit ratio of implementing wide pavement marking is approximately 1:25.

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Appendix A
Characteristics of Test and Control Sites

Table 11 Test Sites

							Right Shoulder Width					
District	Route	Segment	Beg. MP	End MP	Segment Length (mile)	Lane Width	Paved	Unpaved	Total	Number of Horizontal Curves	Degree of Curve Range	AADT
1	SH001	1580	5.58	11.175	5.595	12	2	0	2	11	1.5 - 9	730
1	SH003	1800	114.103	116.894	2.791	12	1	1	2	7	3.0 - 7.5	1642
1	SH200	1610	55.66	62.975	7.315	12	1	1	2	0	NA	964
1	SH041	1630	19.38	22.8	3.42	12	2	0	2	7	1.0 - 12	
1	SH003	1800	92.517	95.331	2.814	11	0	1	1	23	3.0 - 20	1300
1	US095	1540	525.971	527.284	1.313	12	4	0	4	1	2	1200
1	SH097	1790	74.3	81.5	7.2	12	1	0	1	68	4.0 - 45	472
		Total District 1 Miles			**30.448**							
2	SH009	1860	7.26	13.522	6.262	12	3	1	4	11	1.0 - 5.0	1038
2	US095	8605	**263.8**	269.648	5.848	12	4	1	5	5	3.5 - 4.5	3200
	SH128	**6780**	**0**	**2.2**	**2.2**							
2	SH099	1880	0	2.8	2.8	12	1	0	1	24	0.75 - 28	570
2	SH006	1850	17.157	20.286	3.129	12	2	1	3	0	NA	600
2	US012	1910	106	113.8	7.8	12	2	1	3	0	NA	530
		Total District 2 Miles			**28.039**							
3	US095	1540	4.9	7	2.1	12	4	0	4	5	0.50 - 2.0	1500
3	US095	1540	22.71	28	5.29	12	4	0	4	3	1.0 - 10	2034
3	US020	2070	96.05	103.837	7.787	12	2	0	2	9	0.50 - 3.40	1790
3	SH051	2170	19.89	28.9	9.01	12	1	2	3	8	1.0 - 6.0	380
3	SH021	2140	28	34	6	12	2	0	2	31	2.0 - 19	1544
		Total District 3 Miles			**30.187**							
4	SH046	2200	135.625	137.423	1.798	12	2	2	4	0	NA	440
4	SH075	2230	87.248	92.192	4.944	12	4	1	5	0	NA	2800
4	SH025	2270	37.57	42.472	4.902	12	3	2	5	1	2	1269
4	SH081	2310	23.608	26.2	2.592	12	0	1	1	4	1.0 - 1.33	1313
4	SH021	2140	111	116.5	5.5	12	3	0	3	6	0.25 - 2.75	420
4	SH046	2200	121.139	125.674	4.535	12	2	2	4	0	NA	440
4	US030	2040	176.3	177.51	1.21	12	4	0	4	1	1.2	1600
4	SH024	2280	66.916	67.533	0.617	12	5	0	5	1	2	1750
		Total District 4 Miles			**26.098**							
5	US026	2240	272	279.5	7.5	12	4	1	5	2	0.50 - 1.0	1440
5	SH036	2370	0	4.752	4.752	12	1	1	2	11	0.67 - 5.0	695
5	I15 B BlackF	1370	0	2.18	2.18	12	3	1	4	0	NA	4968
5	SH036	2370	29.73	33.926	4.196	12	3	1	4	9	2.0 - 7.0	507
5	SH034	2360	70.47	76	5.53	12	1	2	3	16	2.0 - 14	300
5	SH034	2360	93.3	98.71	5.41	12	3	2	5	6	3.5 - 8	291
		Total District 5 Miles			**29.568**							
6	US093	2220	299	304.3	5.3	12	2	1	3	2	1.0 - 4	2279
6	SH033	2460	8.5	17	8.5	12	2	0	2	0	NA	703
6	SH043	2400	0.3	3.42	3.12	12	4	0	4	0	NA	4100
6	US093	2220	263	268.66	5.66	11	1	0	1	0	NA	809
6	US093	2220	137	144.5	7.5	12	1	0	1	0	NA	530
6	SH031	2450	4.735	5.705	0.97	11	0	2	2	0	NA	1700
		Total District 6 Miles			**31.05**							
		Total Test Section Miles			175.39							

Table 12 Control Sites

							Right Shoulder Width					
District	**Route**	**Segment**	**Beg. MP**	**End MP**	**Segment Length (mile)**	**Lane Width**	**Paved**	**Unpaved**	**Total**	**Number of Horizontal Curves**	**Degree of Curve Range**	**AADT**
1	SH001	1580	0	6	6	12	2	0	2	9	1.5 - 9	730
1	SH003	1800	111	114	3	12	1	1	2	8	3.0 - 7.5	1642
1	SH200	1610	48	55	7	12	1	1	2	2	3.5-4.5	964
1	SH041	1630	24	27	3	12	2	0	2	6	1.0 - 12	
1	SH003	1800	96	99	3	11	0	1	1	16	3.0 - 20	1300
1	US095	1540	528	530	2	12	4	0	4	2	3.0 - 7	1200
1	SH097	1790	83	90	7	12	1	0	1	53	4.0 - 45	472
		Total District 1 Miles			**31**							
2	SH009	1860	1	7	6	12	3	1	4	13	1.0 - 5.0	1038
2	US095	8605	**271**	277	6	12	4	1	5	6	3.5 - 4.5	3200
2	**SH128**	**6780**			**3**							
2	SH099	1880	4	7	3	12	1	0	1	19	0.75 - 28	570
2	SH006	1850	21	24	3	12	2	1	3	0	NA	600
2	US012	1910	115	123	8	12	2	1	3	0	NA	530
		Total District 2 Miles			**29**							
3	US095	1540	8	10	2	12	4	0	4	4	0.50 - 2.0	1500
3	US095	1540	29	34	5	12	4	0	4	3	1.0 - 10	2034
3	US020	2070	104	110	6	12	2	0	2	8	0.50 - 3.40	1790
3	SH051	2170	30	36	6	12	1	2	3	11	1.0 - 6.0	380
3	SH021	2140	35	41	6	12	2	0	2	36	2.0 - 19	1544
		Total District 3 Miles			**25**							
4	SH046	2200	138	140	2	12	2	2	4	0	NA	440
4	SH075	2230	93	98	5	12	4	1	5	0	NA	2800
4	SH025	2270	43	48	5	12	3	2	5	1	2	1269
4	SH081	2310	27	30	3	12	0	1	1	6	1.0 - 1.33	1313
4	SH021	2140	118	123	5	12	3	0	3	5	0.25 - 2.75	420
4	SH046	2200	127	131	4	12	2	2	4	0	NA	440
4	US030	2040	179	182	3	12	4	0	4	1	1.2	1600
4	SH024	2280	65	66	1	12	5	0	5	1	2	1750
		Total District 4 Miles			**28**							
5	US026	2240	281	287	6	12	4	1	5	2	0.50 - 1.0	1440
5	SH036	2370	6	11	5	12	1	1	2	11	0.67 - 5.0	695
5	I15 B	1370	2	4	2	12	3	1	4	0	NA	4968
5	SH036	2370	24	28	4	12	3	1	4	9	2.0 - 7.0	507
5	SH034	2360	77	82	5	12	1	2	3	16	2.0 - 14	300
5	SH034	2360	100	105	5	12	3	2	5	6	3.5 - 8	291
		Total District 5 Miles			**27**							
6	US093	2220	305	310	5	12	2	1	3	2	1.0 - 4	2279
6	SH033	2460	18	24	6	12	2	0	2	0	NA	703
6	SH043	2400	4	7	3	12	4	0	4	0	NA	4100
6	US093	2220	270	276	6	11	1	0	1	0	NA	809
6	US093	2220	146	151	5	12	1	0	1	0	NA	530
6	SH031	2450	7	10	3	11	0	2	2	0	NA	1700
		Total District 6 Miles			**28**							
		Total Test Section Miles			168							