Southwest Region University Transportation Center

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16. Abstract

This report is a compilation of research papers written by students participating in the 2010 Undergraduate Transportation Scholars Program. The 10-week summer program, now in its 20th year, provides undergraduate students in Civil Engineering the opportunity to learn about transportation engineering through participating in sponsored transportation research projects. The program design allows students to interact directly with a Texas A&M University faculty member or Texas Transportation Institute researcher in developing a research proposal, conducting valid research, and documenting the research results through oral presentations and research papers.

The papers in this compendium report on the following topics, respectively: 1) estimating carriers/truckers value of time due to congestion; 2) evaluating retroreflectivity measurement techniques for profiled and rumble stripe pavement markings; 3) analyzing retroreflectivity and color degradation in sign sheeting; 4) evaluating the effectiveness of LED enhanced stop paddles for school crossing guard use; 5) incorporating freight value into the Urban Mobility Report; 6) evaluating ASTM Standard Test Method E2177, retroreflectivity of pavement markings in a condition of wetness; 7) calibrating pavement performance prediction models; and 8) evaluating the effects of concrete curing compounds on hydration.

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COMPENDIUM OF STUDENT PAPERS: 2010 UNDERGRADUATE TRANSPORTATION SCHOLARS PROGRAM

Participating Students (seated, left to right): Brandon Schwenn, Gregory Larson, Jennifer Tovar, and Isaac Almy (standing, left to right): William Huff, Lance Ballard, Bradford Brimley, and Brian Ward

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June 2011

PREFACE

The Southwest Region University Transportation Center (SWUTC), through the Transportation Scholars Program, the Texas Transportation Institute (TTI) and the Zachry Department of Civil Engineering at Texas A&M University, established the Undergraduate Transportation Engineering Fellows Program in 1990. The program design allows students to interact directly with a Texas A&M University faculty member or TTI researcher in developing a research proposal, conducting valid research, and documenting the research results through oral presentations and research papers. The intent of the program is to introduce transportation engineering to students who have demonstrated outstanding academic performance, thus developing capable and qualified future transportation leaders.

In the summer of 2010, the following eight students and their faculty/staff mentors were:

STUDENTS	MENTORS
Isaac Almy University of Maine, Orono, ME	Dr. Bruce Wang
Lance Ballard Texas A&M University, College Station, TX	Mr. Adam Pike
Bradford Brimley Brigham Young University, Provo, UT	Dr. Gene Hawkins Dr. Paul Carlson
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- Mrs. Cathy Bryan, who assisted with program administrative matters and in the preparation of the final compendium.

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CONTENTS

Estimation of Carriers/Truckers Value of Time Due to Congestion

by Isaac Almy 1

Evaluation of Retroreflectivity Measurement Techniques for Profiled and Rumble Stripe Pavement Markings

by Lance D. Ballard 23

Analysis of Retroreflectivity and Color Degradation in Sign Sheeting

by Bradford K. Brimley 81

Evaluating the Effectiveness of LED Enhanced Stop Paddles for School Crossing Guard Use

by William Huff 107

Incorporating Freight Value into the *Urban Mobility Report*

by Gregory W. Larson 123

Evaluation of ASTM Standard Test Method E 2177, Retroreflectivity of Pavement Markings in a Condition of Wetness

by Brandon Schwenn 147

Calibration of Pavement Performance Prediction Models

by Jennifer L. Tovar 193

Evaluating the Effects of Concrete Curing Compounds on Hydration

by Brian Ward 225

DISCLAIMER

The contents of this report reflect the view of the authors, who are responsible for the facts and the accuracy of the information presented herein. This document is disseminated under the sponsorship of the Department of Transportation, University Transportation Centers Program in the interest of information exchange. The U.S. Government assumes no liability for the contents or use thereof.

Estimation of Carriers/Truckers Value of Time Due to Congestion

Prepared for
Undergraduate Transportation Scholars Program

by

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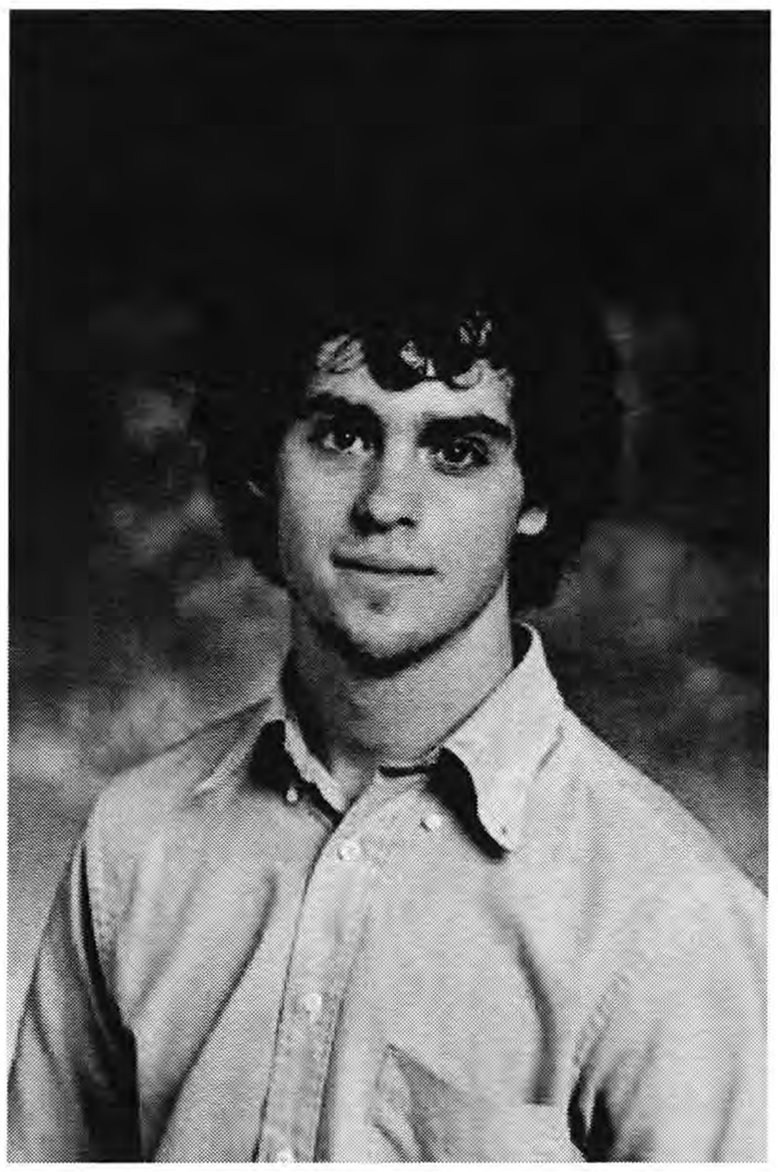

STUDENT BIOGRAPHY

Isaac Almy is a Civil Engineering student at the University of Maine in Orono and expects to graduate in May 2010. Almy also expects to graduate with honors from the University of Maine's Honors College.

Almy is a member of the steel bridge club and competed in the 2010 New England Steel Bridge competition. Almy is also a member of UMaine's Chapter of the American Society of Civil Engineers. Outside of class, Almy enjoys several intramural sports and creates YouTube videos as a hobby. After graduation, Almy plans to apply to graduate school and earn a master's degree focusing in Transportation Engineering.

ACKNOWLEDGMENT

The research described in this paper was funded as independent research through the Southwestern University Transportation Center (SWUTC). The research activities were conducted in support of the Undergraduate Transportation Scholars Program. The findings and recommendations included in this paper are based on the student's summer activities. The contents of this paper reflect the views of the author, who is responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official view or policies of SWUTC.

The author would like to express his appreciation to Bruce Wang and Qing Miao for their guidance this summer. This research is also a task of Dr. Wang's research project funded by the University Transportation Center for Mobility (UTCM) in 2010 with the same project title.

SUMMARY

Past research on the value of time of truckers and carriers has led to low estimations. The current numbers used for the value of time of truckers and carriers is out of date and this research intends to explore the methodologies in order to update the values. Today roadways are more congested and buffer times have increased. This shows a demand for research on the value of time.

One way to get the value of time is through stated preference surveys. There are two scenarios presented to drivers, a scenario where they are late and a scenario where they are on time. The two scenarios will be compared. There will also be a simulation conducted to check results from surveys and to see how a driver's bias affects their decisions. Driver characteristics will be gathered from survey responses and used to create a utility function.

This study uses results from face to face driver interviews to create a utility function. From this utility function, planners and transportation engineers can better predict truck driver decisions

concerning travel time and toll rates. This allows planners to make informed decisions on where to spend money appropriately. The function also provides additional information on toll rates.

The value of time was found to be higher than the current values used. This means that transportation planners and engineers must weigh the value of truck drivers more carefully when making decisions on policy or money use. A range of values will be given according to certain driver characteristics we felt most important and notable.

Future research should consider truck drivers who are too busy to take a survey. Post cards are passed out to drivers to mail back responses. Future work also includes gathering mailed out surveys to include in the data set.

TABLE OF CONTENTS

Student Biography 2
Acknowledgment 2
Summary 2
Introduction 5
Background 6
Goals and Objectives 7
Research Tasks 7
  Literature Review 7
  Data Collection 8
    Houston 9
    Austin/San Marcos 10
    Dallas/Fort Worth 11
  Data Analysis 11
    SAS Software 13
    Simulation 13
Results 15
Conclusion 19
References 20
Appendix 21

INTRODUCTION

Neglecting the importance of freight transportation will be detrimental to the United States economy and to our roadways. The Freight Analysis Framework (FAF) has predicted that 37,211 million tons of freight will be moved in 2035. In 2008, 58.9 million tons of freight was moved per day. Maintaining a state of the art system for trucks will be vital to the United States economy in years to come. As the United States becomes more and more dependent on truck, more trucks will be entering the roadway and causing more delay. The rise of e-commerce in recent years has spurred more deliveries to individual households. The United States has approximately 116 million households according to statistics in 2009. It is vital to the country's economy to keep the shipping costs to a minimum. Transportation planners need to understand the importance and impact that freight transportation has on congestion and vice versa. This is why it is necessary to conduct research on the value of time for truckers and carriers.

While the demand for truck transportation has been on the rise, road facilities are improving at a lower rate. This challenges several aspects in freight transportation, including system capacity, maintenance requirements, and transportation performance. Truck traffic is primarily long distance and therefore uses interstate highways as their primary road. Improvements to roads other than interstate highways will not be a benefit to the freight transportation industry.

The *Urban Mobility Report* from Texas Transportation Institution (TTI) reports that congestion is a problem in the United States' 439 urban areas and continues to get worse. In 2007, an extra 4.2 billion hours and 2.8 billion gallons of fuel were wasted from congestion. The cost of this extra waste is approximately $87.2 billion. The cost of delay increased $100 million from 2006 to 2007. With an almost ubiquitous congestion as predicted, which road section or bottleneck has the largest economic impact or most cost to truckers and carriers? Understanding and answering this question would clearly help freight planners in their decision-making.

The current estimate for the value of time ranges from $28.50 to $41.25 depending on truck size. These numbers are calculated taking into account labor statistics and inflation (*1, 2*). This number is significantly close to the value of time used for commuters. A commuter does not value their time the same way a truck driver does. The consequence for a commuter arriving late is simply not getting to work on time. A truck driver, on the other hand, may experience the loss of goods or possibly a penalty for late arrivals. Operating costs for trucks are higher than the operating costs of commuter vehicles. Since truckers are using their vehicles more often this leads to frequent repairs. Driving and working hours have an impact on the value of time. These reasons make it our expert guess that the current value of time for truckers and carriers is too low. The rich history behind the commuter's value of time gives us insight into how we conduct research for trucker's value of time.

Experienced drivers and trucking companies have the foresight and knowledge to understand that there are traffic delays. The truck driver will plan his schedule to accommodate time spent stuck in traffic. Additionally truck companies and route coordinators will also accommodate time spent in traffic. This extra time allowed in traffic can be called buffer time. This buffer time is a waste of the driver's time and a waste of the company's valuable time. Some inexperienced drivers may not understand that the extra time a company allows in traffic is essentially a waste of time.

It may not be clear to drivers that extra time is just for traffic delays. A roadway with a high buffer index means that the driving time on that section of road is unreliable. Drivers schedule plenty of extra time for that section of roadway to arrive on time. Studying the value of time will help to reduce the buffer index on major highways.

BACKGROUND

The value of time is the maximum amount of money someone will pay to reduce time spent in traffic. The commuter's value of time is rich with information. This information gives insight into a trucker's value of time. The value of time for commuters depends on several characteristics. Examples include income, day of week, time of day, and season. High-income commuters are willing to pay more money. A trucker's value of time also depends on several characteristics. Characteristics that affect the value of time include who pays the toll, how the driver is paid, trip length, type of carrier, cargo, route, and the flexibility of their driving hours. In our study, we are considering two scenarios, late and early.

In a report titled "Perceived value of time for truck operators" by Kawamura the average value of time for interviewed drivers was found to be \$26.8 per hour with a standard deviation of \$43.17 in 1998 dollars (*3*). The author used the switch point method to determine this value of time. This method is not as accurate as using the maximum likelihood model. Kawamura categorized the data and determined that for-hire fleets tend to have higher values of time than private fleets. In a similar study by Water et al., the value of time was found to be between \$6.1 and \$34.6 per hour in 1998 prices (*4*). It is our guess that the value of time be higher than these values.

The switch point method used by Kawamura is inaccurate because the number found is an estimate. In this method, responses from stated preference surveys are analyzed to find a range of acceptable values. Survey responses may indicate that the respondent will accept a value of time of \$30 per hour but will decline \$40 per hour. This means the respondent's value of time is between \$30 and \$40 per hour. A standard deviation of \$43.17 indicates a high degree of uncertainty regarding the final value of \$26.18.

The maximum likelihood method is also a close approximation of the value of time. This approximation, however, is much more accurate than the switch point method. The maximum likelihood method uses a statistical process developed by Newton. This process typically involves using statistical software. In a paper by Adkins et al., it reviews several ways to calculate the value of time. The methods reviewed include the revenue methods, net operating profit methods, cost saving methods, and the willingness to pay method (*5*). For the purpose of this research, we will analyze our data using the maximum likelihood method.

The value of time will be used by transportation planners to determine where to invest money. Investing in the interstate highway will benefit truck operations. State of the art facilities help decrease travel times and reduce congestion. Adjustments to system capacity will improve road conditions and reduce congestion. The value of time also helps create an appropriate toll rate. The value of time could help create congestion-pricing options. A logit model's use is to predict traffic patterns. This logit model determines the percentage of the trucks choosing to use a toll road versus a free road. With this information, we can create a section of road that will benefit all road users.

In order to get the information necessary to create a logit model and find the value of time for truckers we must use a stated preference survey. The stated preference survey asks questions on what the user would prefer. Surveys should be no longer than one page. This keeps the survey simple and less intimidating when the respondent takes it (*6*). Send a detailed survey to the dispatcher or fleet manager because they are able to understand the implications and questions asked (*7*). The survey should also include questions about driver and company characteristics. Information about the driver or company helps create the logit model. Grouping the data and analyzing them in more than one way creates sound research.

GOALS AND OBJECTIVES

The goal of this research effort is to come up with a reasonable number for the value of time. We will also work to create a simple and effective stated preference survey. With results from the stated preference surveys, we will work to create a logit model that predicts driver decisions. We must work to accomplish these two tasks:

1. Analyze stated preference surveys and form a logit model.
   Responses from face to face interviews along with some mailed out surveys will be used to create a logit model for this report. The project must be completed in a 10-week period and waiting for all mail in responses to return will take too long.
2. Compare findings from the stated preference survey to simulated tests.
   A simulation will be used to check results from the survey. The simulation does not have any biases. This characteristic allows us to see what the actual value of time should be. Drivers do not always have an accurate image of the entire operation system. The simulation is based purely on numbers and reasonable assumptions. Results from the survey will also help back the results of the simulation. These two sets of results will confirm each other.

The work plan below outlines each phase of the research. Each task in the work plan needed to be completed to meet the goals outlined above.

RESEARCH TASKS

This portion of the research involves a literature review, data collection, data analysis, and results. Each task in this work plan was worked on thoroughly.

Literature Review

An insight on the study of commuter value of time gives us insight on the trucker's value of time. The trucker's value of time is assumed higher than the current value used by the FHWA. Previous studies suggest that the value of time is characterized by driver and company characteristics. A deeper and more detailed review is mentioned in the Background section of this paper.

Data Collection

The data collected were in three major cities of Texas. Visits made to Dallas, Austin/San Marcos, and Houston gave us the backbone of our data. Mailed out surveys were sent to various major cities within Texas to give the research a diverse geographic distribution. A map in Figure 1 shows the locations of Dallas, Austin/San Marcos, and Houston areas.

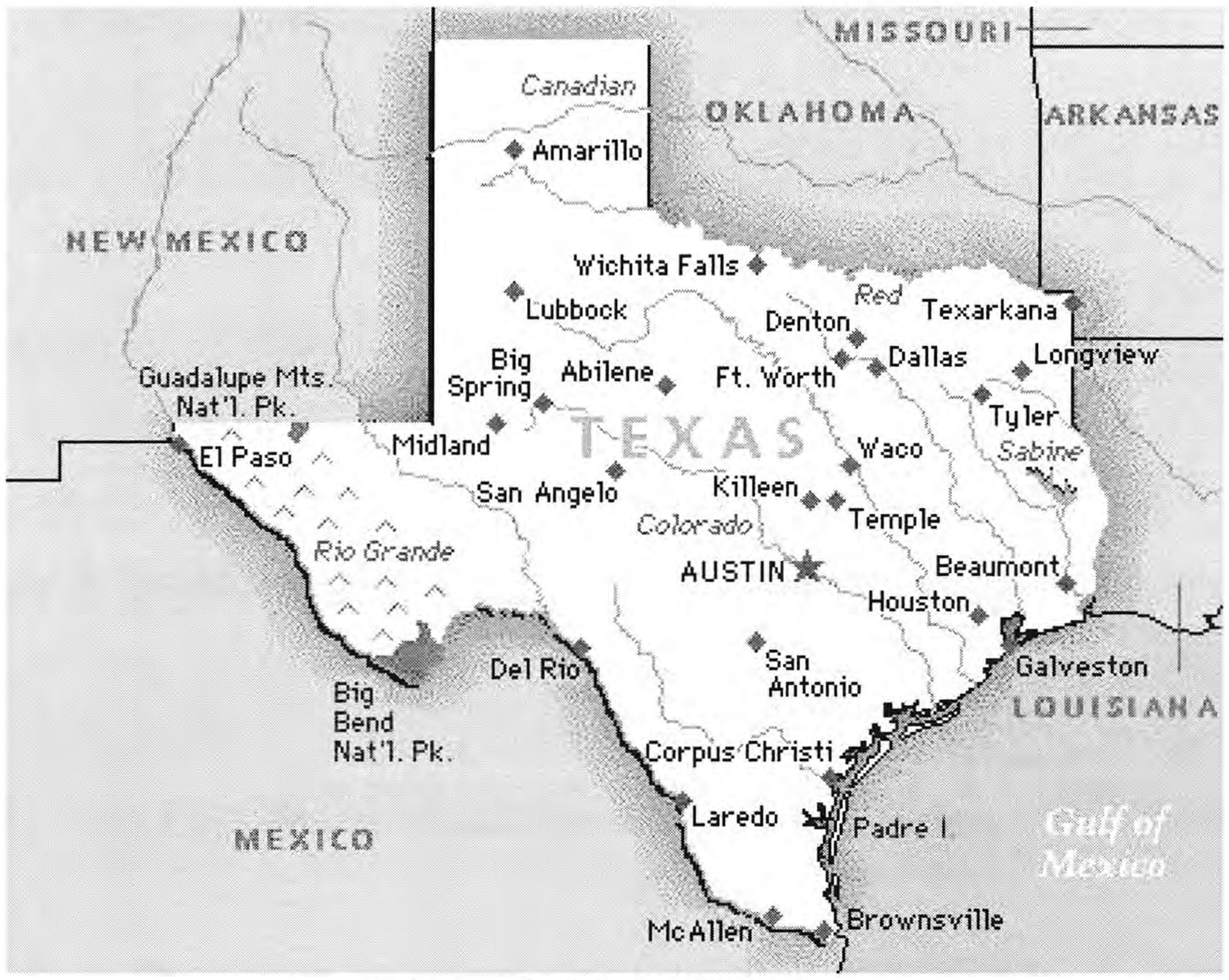

Figure 1. Map of Texas.

Dallas/Ft. Worth area is in the northern area, Austin/San Marcos is near the center, and Houston is in the eastern area. Mailed out surveys were sent as far as El Paso, Amarillo, Midland, and Corpus Christi. The detailed report from each city is documented below.

The questions of the stated preference survey were chosen based on getting the necessary parameters to estimate the value of time accurately. The final design of this survey is in the appendix. The question about truck size was included because the current toll system is based upon the number of axle's a truck has. This characteristic may not have much of an influence on the value of time, but it is relevant to the current system in place. The question of who pays the toll is an important characteristic to consider because drivers who pay the toll feel the financial burden of tolls. Drivers that do not pay the toll do not feel the financial burden of tolls.

The question of how the driver was paid was included because we were interested in the relationship between the pay type and value of time. It is suspected that drivers who are paid by the mile will have a higher value of time. Drivers paid by the mile are not being paid to sit in traffic. The sooner the driver arrives to a destination the more free time he will have. Drivers who are paid by the load are not as concerned about the traffic unless their cargo is time sensitive. Drivers paid from a percentage of revenue are also unconcerned with traffic delay. If they receive a bonus or a penalty, however, their value of time will increase.

A short background section is provided as an optional part of the survey. The responses from this section will help give us a better understanding of what the typical truck driver is. Annual income is particularly interesting because a commuter's increases as income increases. The same could be true for the truck driver's value of time.

The toll rate selections correspond to values of time that range from $31/hour to $120/hour. We used this range because we wanted to give the drivers typical values. A blank space was left for drivers to fill in their own amount if they chose. We were also careful to create amounts that were relatively close together. This is so we can see where a driver will accept an amount and decline an amount. If the driver accepts a value of time of $30 and rejects a value of time of $40, then we know that their value of time is between $30 and $40. Table 1 is the final format of the survey. Figure 8 in the appendix shows the full-length survey.

Table 1. First Scenario Question Format.

Arrival Time: 15 minutes late				**Arrival Time: On time**				**Arrival Time: 15 minutes early**			
$30	$20	$13	Other___	$50	$35	$20	Other___	$68	$45	$23	Other__

We also gave the survey two scenarios. The first scenario is where the driver knows he will be 30 minutes late. The driver then has the option to pay money to save time. In a real world situation, a driver will know ahead of time whether they are running on time, late, or early. The second scenario is the driver is running on time and has options to arrive early. The expected value of time in the first scenario will be higher than the second scenario. A driver is willing to pay more money to arrive on time than to arrive late.

Houston

Two trips were made to Houston. The first trip's purpose was to discover ways to refine the format of the stated preference survey. Five surveys were conducted at different locations. A map of Houston is shown below on the following page. Drivers expressed concern and disinterest when toll roads were brought into discussion. Many of the drivers felt that they should be exempt from the tolls and would refuse to put any amount of money down to save time. This problem encouraged us to revise the survey. Instead of using the words "toll road" on the survey, we gave options on how much they will pay. The words "toll road" were removed from the survey. Confusion also occurred when selecting how much money they are willing to pay. Drivers would often circle options that were not meant to be circled. This forced a change in the survey's design. Replacing many of the columns into just three separate boxes and reducing the number of rows improved the design. The revised version of the design is shown in Figure 2 on the next page.

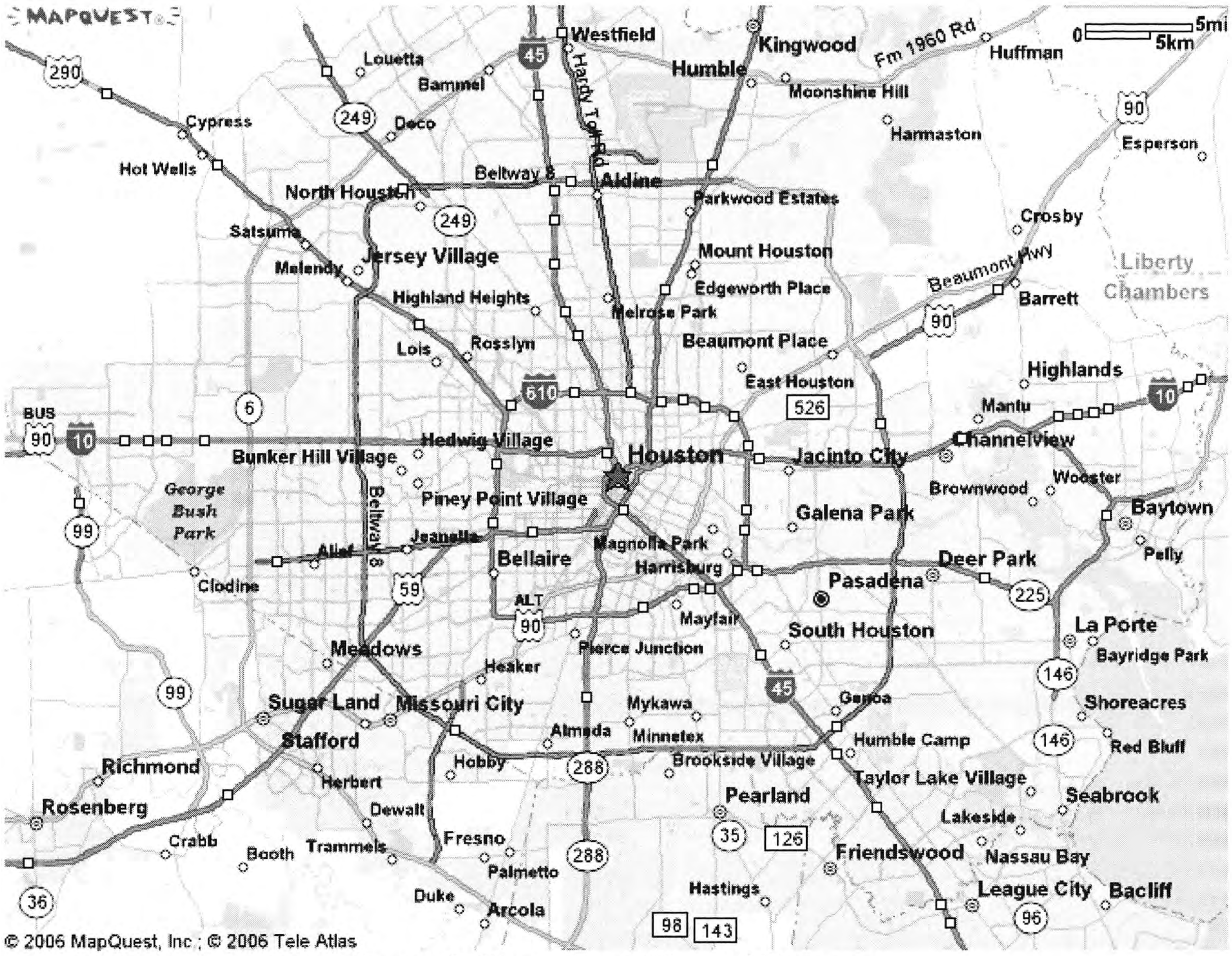

Figure 2. Downtown Map of Houston.

The second trip to Houston proved more valuable than the first. Thirteen interviews were made on this trip. Certain truck stops have more frequent truck drivers than others. We took careful note and took suggestions from drivers on which businesses to interview at. Using the words "toll road" was detrimental to the research. Instead of expressing that we feel the toll rate is inaccurate and we were working to correct it proved much more successful.

Austin/San Marcos

The downtown area of Austin did not have truck stops worth visiting. Instead, we received helpful tips from a truck driver that in San Marcos many truck drivers stopped before Austin. In San Marcos, we interviewed seven drivers. When approaching the driver it is important to use the words "your feedback on this issue is very important to our research." Drivers after hearing this have been more willing to participate in the survey. This longer drive time gave us less time to interview the drivers. Figure 3 is a map of Austin and San Marcos area.

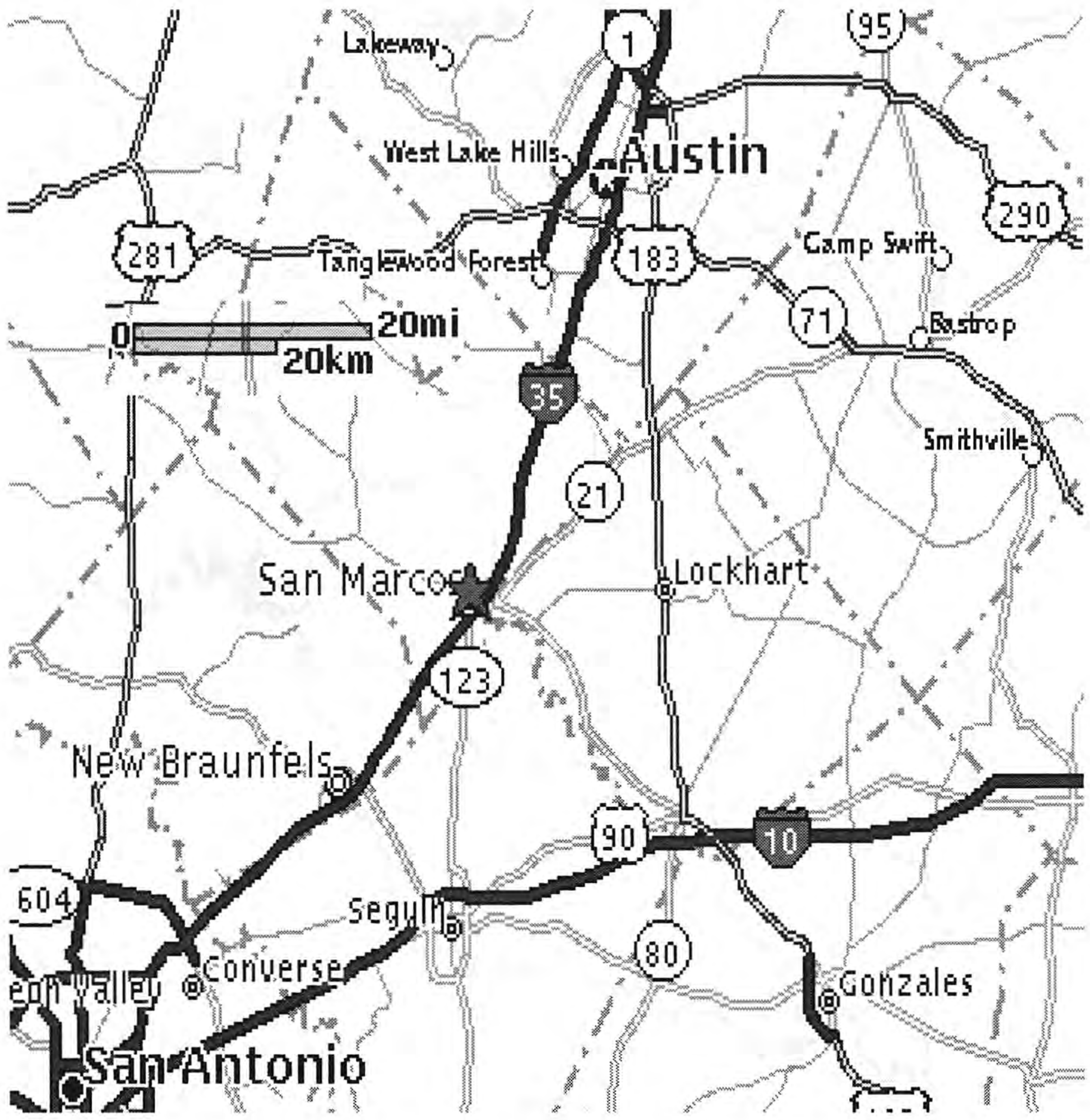

Figure 3. Map of Austin/San Marcos Area.

Dallas/Fort Worth

Two days were spent in the Dallas and Fort Worth area. A total of 22 drivers were surveyed in these two days. Many drivers were helpful on giving us information. Spending the night gave us plenty of time to conduct many surveys in several locations in the area.

Data Analysis

A total of 47 drivers were interviewed face to face. Shown in Table 2 below are the characteristics of drivers. Most drivers were long haul drivers and had an 11+hr drive time. Most drivers decide their own route and pay the toll themselves. This is significant because many of the drivers refused to pay any money for tolls. The number of drivers that pay their own toll accounts for a high response refusing to pay a toll. Drivers are less willing to pay for the toll if they pay for it themselves.

Twenty-seven drivers chose average value cargo. Several observations should be made from this data. Most drivers selected average value and most drivers did not read the footnote explaining the value of their cargo. Many drivers were too busy to take the necessary time to complete the survey properly. It is likely that the real values are more diverse. Some drivers were high value and chose average value and some drivers were bulk and chose average.

Table 2. Driver Characteristics.

Question	Category	Drivers	Question	Category	Drivers
Type of Carrier	Owner Operator	15	Typical route	Regional	14
	For-hire	18		long haul	28
	Private-Carrier	11		Local/delivery	4
Typical cargo	Bulk	10	Who decides route?	Me (the driver)	20
	Average Value	27		Dispatcher/manager	24
	High Value	8		Shipper	1
	Other	0		Other	0
Truck Size	2 axle	14	How are you paid?	By Mile	30
	3 axle	5		By Load	6
	4 axle	19		Percentage of Revenue	7
	Other	5		Other	2
Trip Length	11+ Hours	29	Who pays the toll?	I do	21
	5 to 11 Hours	12		For-hire carrier	16
	2 to 5 Hours	0		Shipper	3
	Less than 2 Hours	1		Other	3
Delivery window	1 day	16	Route changes	Never	4
	Less than 12 hours	9		Occasionally	15
	less than 5 hours	4		Often	17
	less than 3 hours	15		Always	11

From these data we selected the characteristics that we thought are most important when estimating the value of time. We felt that the type of route (regional, long haul, or local/delivery) was an important parameter because it covered many of the other responses. For example, if the driver selected long haul then he would most likely selected 11+ hour drive time. We also found that most long haul drivers are paid by the mile. Choosing the route type as one of the parameters, we felt we were covering many of our bases. This parameter relates to many of the other questions.

The other parameter chosen was the delivery window. The delivery window is an important estimate because it describes how important arriving to their destination on time is. Drivers with a delivery window of more than 12 hours are not as pressured to arrive on time. There is a significant gap in the data. Either drivers selected the largest delivery window (1 day) or the smallest (less than 3 hours) with only a few that selected in between. The delivery window parameter is useful to the estimate because it includes drivers with a small delivery window and a large window.

The truck size was not included in the parameter estimates because we found that many drivers were unsure of their truck size. We felt that their responses were not accurate enough. A two-axle truck is considered a local/delivery truck. Our responses indicate that there were only four local/delivery trucks interviewed and 14 responses tell us that they had a two-axle truck. There were only a few two-axle trucks spotted at truck stops. So, the truck size question was not considered as a reliable parameter.

The question about pay type was not parameter because most long haul drivers are paid by the mile. Since we already included the driver's typical route as a parameter, we felt it would be redundant to include the pay type.

SAS Software

Statistical analysis software (SAS) was used to determine a utility function that characterizes the data. The parameters chosen for the utility were time saved, cost, typical route, and delivery window. The method of maximum likelihood was chosen to estimate the maximum value of time. Breslow developed the specific method used. This method was chosen over Newton's method because it converged on the answer quicker. It is also our understanding that the value is more accurate. The process to estimate the parameters is actual quite complex and using SAS software expedites the process.

Simulation

A simulation was conducted by Qing Miao, a graduate student also working on this project. The purpose of running a simulation is to remove the driver's opinion from the situation. The simulation bases its results purely on the best economic choice for the company. The simulation is designed for the Houston area. In the simulation, truck drivers are given a list of deliveries needed to be made. The simulation chooses the best economical route based on travel time and distance. Throughout the process route, changes can be made to avoid congestion or experience the congestion depending on what the best economical option is.

Five specific details are considered in the simulation:

1. Each demand has two locations, the first is the pickup location and the second is deliver location.
2. Each demand has a time window for the pickup and deliver.
3. Two depots are considered, as well as one central depot.
4. Waiting time between two demands in one route cannot exceed certain hours, such as 4 hours in our case.
5. The entire process is a dynamic process, whenever there are new demands coming in, the schedule will be re-optimized considering the new demands. By that time, the vehicles that are already on their way to pick up loads or deliver loads have to finish that particular demand before they can change their route.

The entire process is dynamic process, whenever there are new demands coming in, the schedule will be re-optimized considering the new demands. By that time, the vehicles that are already on their way to pick up load or deliver load have to finish that particular demand before they can change their route.

The congestion links in Houston can also be edited individually. Where the congestion happens can be changed and how much congestion can be changed. The driver is assumed to be traveling at 65 mph in uncongested traffic. The corresponding value of time will be calculated as
Value of delay = Total cost of congestion/Total vehicle time facing congestion.

The degree of congestion, number of congestion areas, location of congestion, location of depots, demand distribution pattern, demand size, time window, etc. are also simulated for the purpose of testing to different values of demand. The process for the simulation is shown in Figure 4.

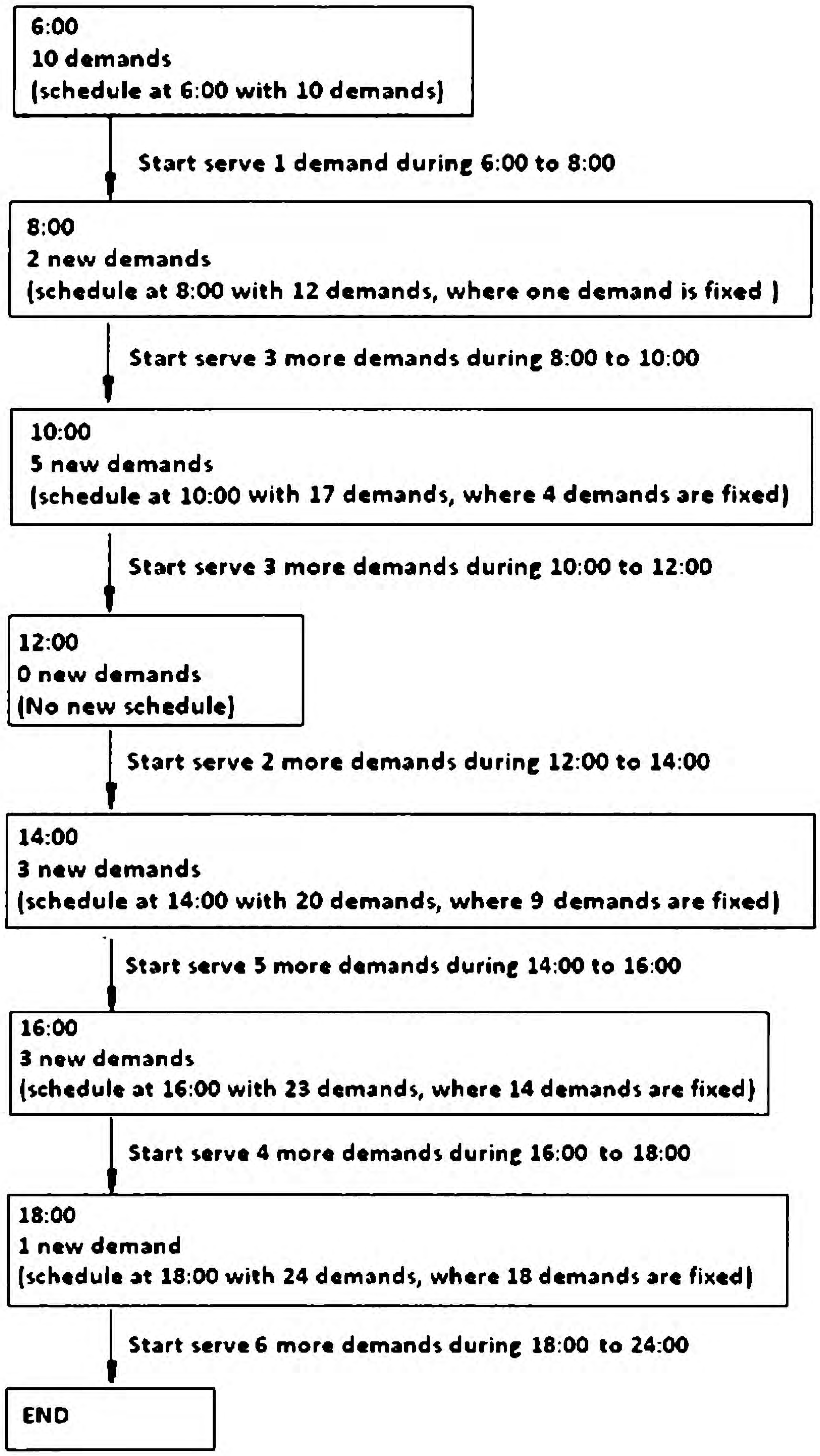

Figure 4. Simulation Process.

Each truck starts out with demands at 6:00 AM and between 6:00 and 8:00 AM, some demands are served. At 8:00 AM, new demands will come in. Between 8:00 and at 10:00 AM, more demands are served. This process continues until at the end of the day all demands are served. A view of the simulation map is shown in Figure 5.

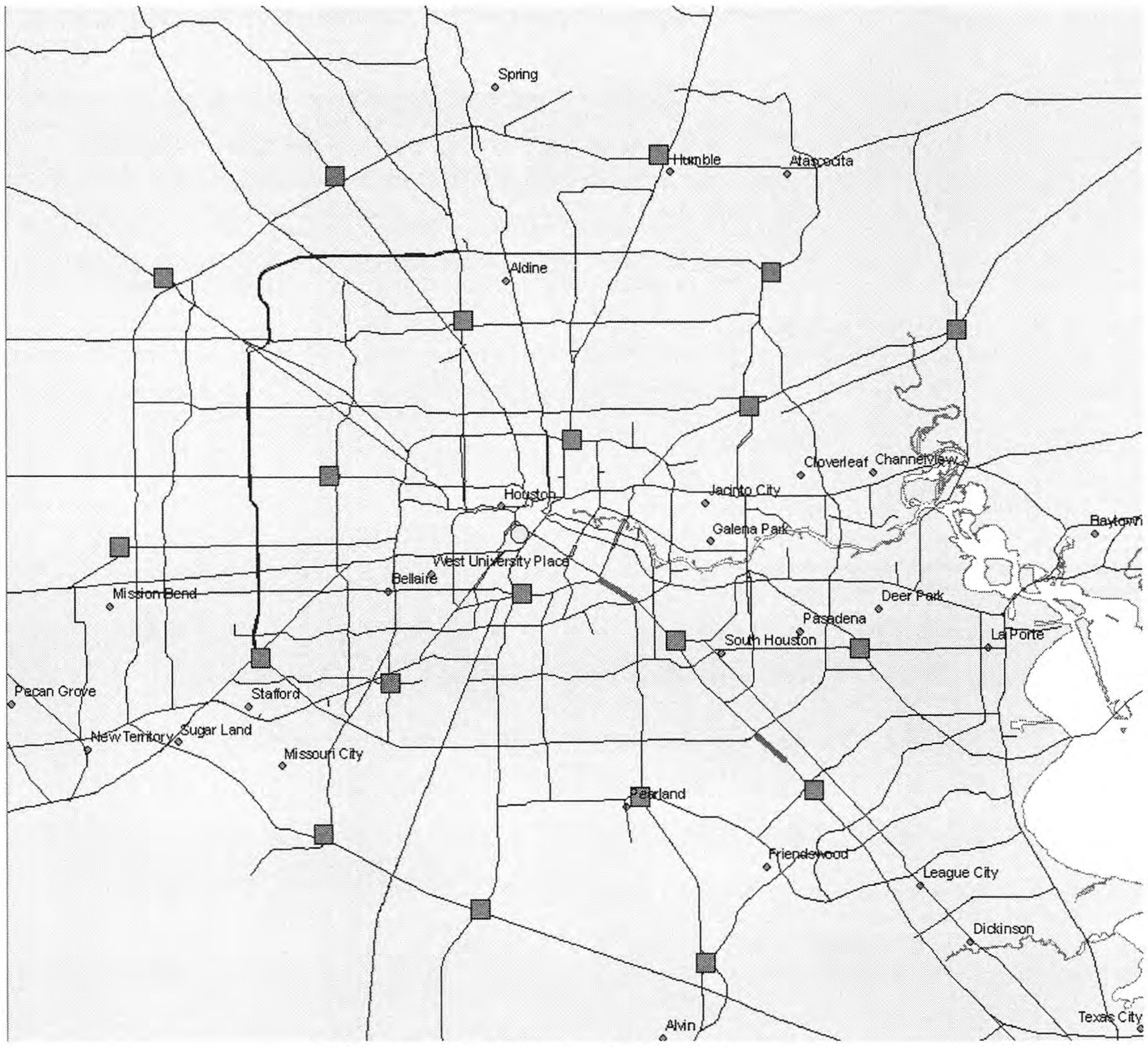

Figure 5. Map of Simulation.

The pink squares represent potential pickup or delivery locations, the light blue dot is the central depot location, and red road segments are locations of congestion.

RESULTS

Statistical analysis software (SAS) was used to determine a utility function that characterizes the data. The parameters chosen for the utility were time saved, cost, typical route, and delivery window. The process to estimate the parameters is actual quite complex and using SAS software expedites the process. Figure 6 is the output file from SAS software.

The output shows a relatively low standard error for each of the estimates. This shows that our data were accurate. Our sample size was large enough and consistent enough to make sense statistically. The software also ran a Chi-Square test. Parameter T corresponds to travel time savings. LogP is the log size of the payment. Using the log of payment does not affect the overall result for value of time. The log is purely to make the model fit more appropriately. The column "Parameter Estimates" are coefficients in the utility function. The coefficient for parameter T is negative because as a driver pays more money more time is saved. While the left hand side of the equation remains the same, meaning he chooses to stay on the toll road. A positive value would mean that as drivers paid more money travel time also increased. This, however, is counter intuitive because more money paid corresponds to a decrease of time in traffic.

Analysis of Maximum Likelihood Estimates

Parameter	DF	Parameter Estimate	Standard Error	Chi-Square	Pr > ChiSq	Hazard Ratio	Label
T	1	-0.01624	0.01042	2.4313	0.1189	0.984	T
LogP	1	0.04835	0.20060	0.0581	0.8095	1.050	LogP
R1	1	-0.03167	0.49745	0.0041	0.9492	0.969	R1
R2	1	0.11995	0.28854	0.1728	0.6776	1.127	R2
R3	0	0	.	.	.	.	R3
F1	1	0.01883	0.64591	0.0009	0.9767	1.019	F1
F2	1	0.05706	0.71100	0.0064	0.9360	1.059	F2
F3	1	0.12359	0.64410	0.0368	0.8478	1.132	F3
F4	1	-0.00871	0.65414	0.0002	0.9894	0.991	F4

Figure 6. SAS Output File.

The general form of the utility function is as follows:

$$U_n = a_i *LogP_n+b_i*T_i+c1_i*R1_n+c2_i*R2_n+c3_i*R3_n+d1_i*F1_n+d2_i*F2_n+d3_i*F3_n+d4_i*F4_n+E_{in}$$

The function is for an individual n choosing alternative i.

where: I = alternative choice, 1 if accepts toll road, 2 otherwise;
- n = individual index;
- P_n = payment specified by individual n;
- T_i = travel time saved, measured by 15, 30, and 45 minutes;
- $R1_n$ = 1 if local, 0 otherwise;
- $R2_n$ = 1 if regional, 0 otherwise;
- $R3_n$ = 1 if long haul, 0 otherwise;
- $F1_n$ = 1 if flexibility of delivery hours is less than 3 hrs, 0 otherwise;
- $F2_n$ = 1 if flexibility of delivery hours is from 3 hrs to 5 hrs, 0 otherwise;
- $F3_n$ = 1 if flexibility of delivery hours is from 5 hrs to 12 hrs, 0 otherwise;
- $F1_n$ = 1 if flexibility of delivery hours is more than 12 hrs (such as 1 day), 0 otherwise;
- E_{in} = unobserved stochastic portion of utility.

The variable E_{in} corresponds to an unpredictable amount of error corresponding to the process that is used to solve for the parameters. The purpose of this utility function is to predict probabilities. For example, this function can be used to determine the number of long haul drivers that will pay $5 to save 15 minutes in travel time.

The corresponding value of time from this utility function can be found using the formula:

VOT=e^(T/LogP)=e^(-.01624/.04835)=.7417 $/min = 42.88 $/hr

Our best estimate from our research for the value of time is 42.88 $/hr. This estimate is taking into account the driver characteristics that we felt were most important to the value. Using an exponential function in this case is necessary only because log size of P proves a better fitting model, judged by maximum likelihood ratio test. If the software were to use P on its own then the equation would have simply been T/P. More values are obtained shown in Table 3 below. Figure 7 is a bar graph of Table 3 and compares the first and second scenarios with each other.

Table 3. Values of Time.

	coef. T	**coef. logP**	**VOT/min**	**VOT/hr**
All 1	-0.0175	0.11929	0.86326	51.7957
All 2	-0.0229	0.1232	0.83051	49.8307
Owner Operator 1	-0.0196	0.04615	0.65354	39.2124
Owner Operator 2	-0.0217	-0.32085	1.07001	64.2003
For Hire 1	-0.0152	0.12914	0.8891	53.3459
For Hire 2	-0.0191	0.20612	0.91168	54.7006
Private Carrier 1	-0.0063	0.09159	0.93353	56.0117
Private Carrier 2	-0.0048	0.02157	0.79901	47.9404
Bulk 1	-0.0058	0.23859	0.97582	58.5492
Bulk 2	-0.0128	0.88586	0.98568	59.1406
Average Value 1	-0.0193	0.21586	0.91434	54.8606
Average Value 2	-0.0225	0.15217	0.86266	51.7599
High Value 1	-0.01	0.09267	0.89771	53.8625
High Value 2	-0.0122	0.14	0.91668	55.0006
*1,2 corresponds to first and second scenarios				

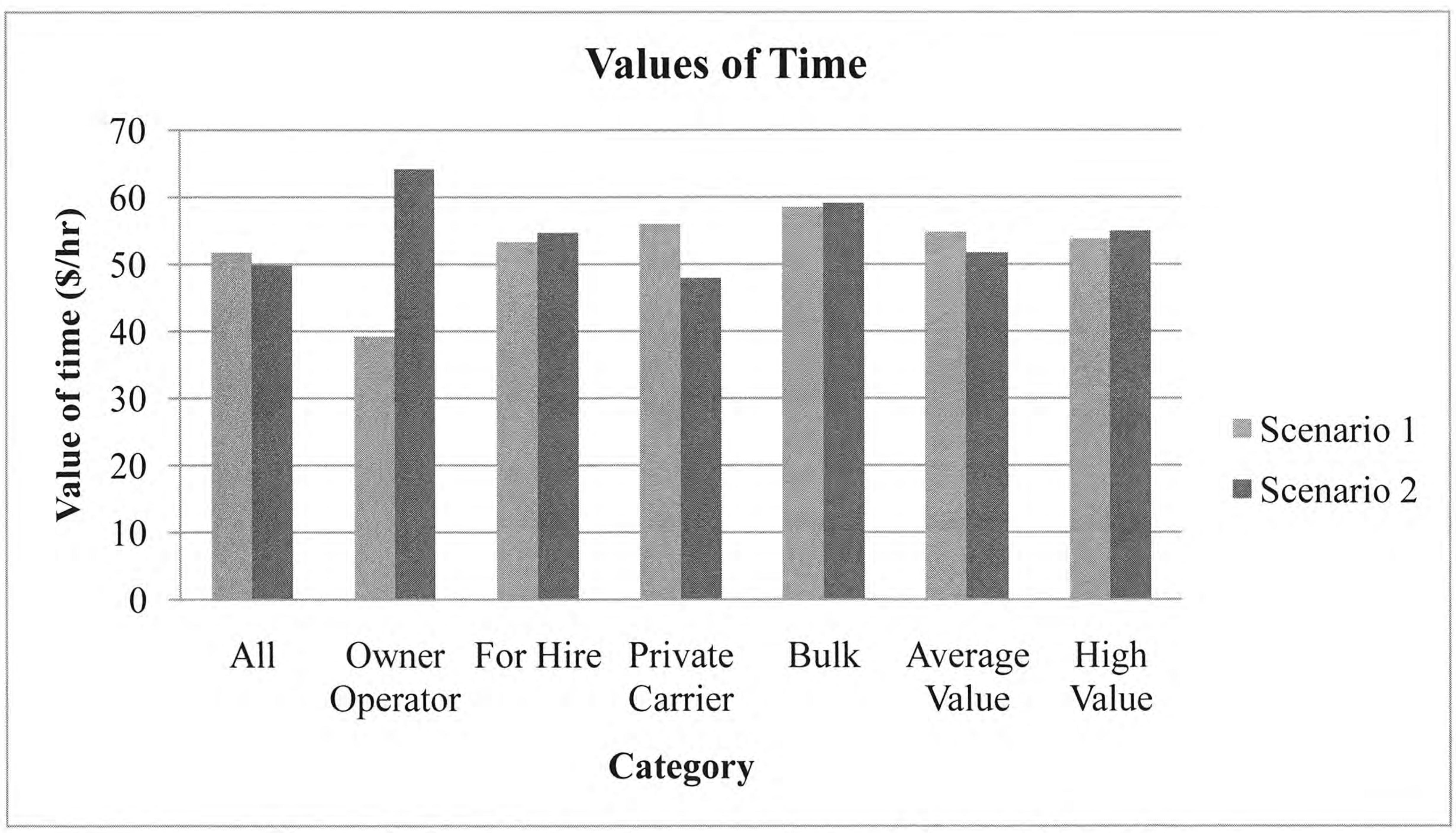

Figure 7. Comparison between First and Second Scenarios.

It was expected that the value of time be higher in the first scenario than the second scenario simply because the driver was running late. This causes a bit more urgency to arrive on time. If we look at the values individually by driver characteristic we see that only two categories where the first scenario is greater than the second. This is in the private carrier and in the average value category. The numbers on average value or more reliable because that is where most of the data are. The majority of the drivers selected average value as their cargo type.

The second scenario for owner operators was much higher than the first scenario. This could be because many drivers did not mind being 30 minutes late. When creating the survey we were assuming that there would be a penalty for arriving late. After talking to many drivers, we found that there was no penalty for being late. The penalty might be given to the drivers company and not the driver. The response from many drivers was that they did not care if they arrived late. They only cared if they were to arrive early or on time.

The simulation was ran 10 cases for 2 scenarios and 1000 cases for the third scenario. The results vary by window size. Window size is the same as delivery window. The delivery window or window size describes how much extra time a driver may have to make their delivery. The following tables are the partial results from the simulation.

Table 4. Case 1: Two Depots (values in $/hr).

Congestion No. 12350	Demand size 25	Demand size 50	Demand size 100
Window size 1 hrs	78.85	87.70	98.26
Window size 1.5 hrs	76.50	81.26	95.89
Window size 2 hrs	77.50	83.66	96.03
Window size 2.5 hrs	67.82	77.79	96.74
Window size 5 hrs	72.19	74.20	76.71
Note*Each number is the average of **10** cases.			

Table 5. Case 2: One Central Depot (values in $/hr).

Congestion No. 12350	Demand size 25	Demand size 50	Demand size 100
Window size 1 hrs	98.43	102.95	102.13
Window size 1.5 hrs	98.62	99.35	99.59
Window size 2 hrs	96.89	99.61	99.55
Window size 2.5 hrs	97.66	97.48	99.70
Window size 5 hrs	88.23	94.04	95.96
Note*Each number is the average of **10** cases.			

Table 6. Case 3: One Central Depot (values in $/hr).

Congestion No. 12202	Demand size 25	Demand size 50	Demand size 100
Window size 1 hrs	102.61	117.26	120.89
Window size 1.5 hrs	101.36	117.30	119.79
Window size 2 hrs	101.40	117.06	118.82
Window size 2.5 hrs	101.91	117.30	120.48
Window size 5 hrs	99.75	116.55	118.24
Note*Each number is the average of **1000** cases.			

In case 3 we find our most reliable results. The results were averaged over 1000 different cases. The most significant trend we find is the relationship between window size and the value of time. As the window, size increases the value of time decreases. This is to be expected. Drivers with a small window size are in a hurry to arrive on time chose to take a toll road. Drivers with a large window do not have the same pressures and will choose to experience congestion. As demand size increases, we notice that the value of time also increases in most cases. The biggest increases are from a demand size of 25 to a demand size of 50. Between 50 and 100, the increase is significantly smaller than before.

CONCLUSION

The results clearly indicate that the value of time for truckers and carriers much higher than the current values. An increase in value of time will also mean an increase in toll rates. Drivers and companies are willing to pay more money to decrease time in traffic. This decreased time in traffic saves the truck driver money as well as the company.

Truck drivers do not see the extra costs associated with experiencing congestion. Extra costs for the driver include repairing trucks more often. Air conditioning, tires, windows, and engines are affected by extra time in traffic. The driver does not consider these when filling out a stated preference survey. This explains why there is such a lower number for a truck driver's value of time. The values from the first scenario were higher than the second scenario as expected. The values from the simulation were also higher as expected.

A higher value of time thus results in higher toll rates. The results have shown that drivers are willing to pay more money in order to save time. A higher toll rate will reduce time in traffic because fewer cars will take the road. This less time in traffic becomes appealing to truck drivers and other commuters. Transportation planners should also note that since the value of time has increased repair and upgrades should be made that affects truck drivers the most. These areas include interstate highways, known truck routes, and transportation hubs. With growing demand for shipping and an increase in the value of time, it is important to invest money appropriately.

Future work should include trying to include drivers that were too busy to take the survey. These drivers could be handed a simple postcard or a pre-stamped and pre-addressed envelope. Incorporating these truck drivers would potentially result in a higher value of time than what was found. The future work should also include a more detailed result from simulations and should include mail in surveys. Mailed in surveys were not included in this report because they are still coming back in the mail.

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APPENDIX

Truck Driver Value of Time Survey

Measurement	Options (Choose at least one option from each row)			
Type of carrier	Owner-operator	For-hire	Private Carrier	
Typical route	Regional	Long Haul	Local Delivery	
Typical cargo [1]	Bulk	Average value	High value	Other:
Truck Size	2 axle	3 axle	4 axle	Other:
Trip Length	11+ hours	5 to 11 hours	2 to 5 hours	Less than 2 hours
Who decides Route	Me (the driver)	Dispatcher or the fleet manager	Shipper	Other:
How are you paid	By mile	By load	Percentage of revenue	Other:
Who pays the toll	I do	For-hire carrier	Shippers	Other:
How often do you change route to avoid congestion	Never	Occasionally	Often	Always
Flexibility of delivery hours on an average trip	1 day	Less than 12-hours	Less than 5 hours	Less than 3 hours

You are running **30 minutes late**. Please select the maximum you are willing to pay for each scenario:

Arrival Time: 15 minutes late				Arrival Time: On time				Arrival Time: 15 minutes early			
$30	$20	$13	Other $5	$50	$35	$20	Other___	$68	$45	$23	Other___

You are running **on time**. Please select the maximum you are willing to pay for each scenario:

Arrival Time: 15 minutes early				Arrival Time: 30 minutes early				Arrival Time: 45 minutes early			
$30	$20	$13	Other___	$50	$35	$20	Other___	$68	$45	$23	Other___

Background (optional) Affiliation:________ Phone #: __________
ethnicity_______ age_______ family size_____ annual income_________

[1] **Bulk commodity:** agricultural products, fertilizer, coal and other minerals, oil products, sand, gravel, logs and rough wood, waste and scrap; **Average value:** wood products, paper print, paper board, textile products, base metal, chemical products, machinery, vehicles, office equipment, and mixed freight; **high value:** electronic equipment, precision instruments, perishable products such as seafood, fashion items.

Figure 8. Final Stated Preference Survey Filled Out.

Evaluation of Retroreflectivity Measurement Techniques for Profiled and Rumble Stripe Pavement Markings

Prepared for
Undergraduate Transportation Scholars Program

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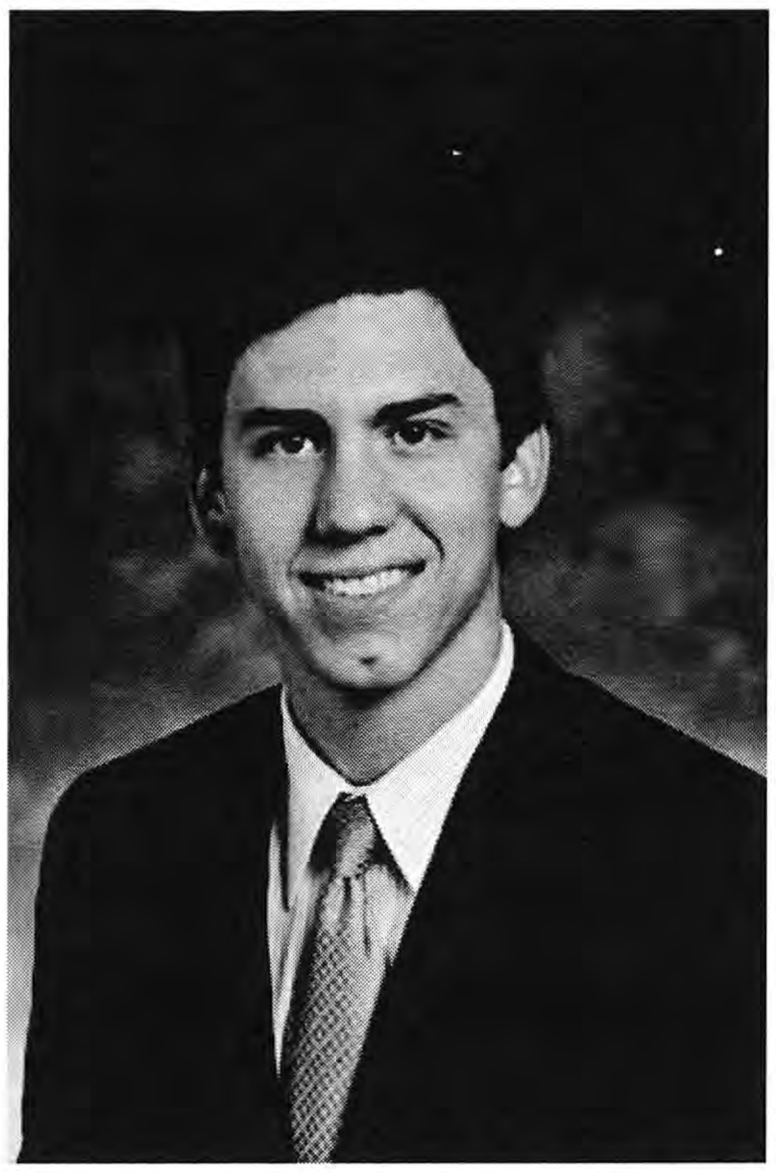
Southwest Region University Transportation Center

August 6, 2010



STUDENT BIOGRAPHY

Lance Ballard is a senior at Texas A&M University in College Station, Texas. He plans to graduate in May 2011 with a Bachelor of Science degree in Civil Engineering and a minor in Business. He is a Terry Foundation Scholar and a Look College of Engineering Distinguished Student.

Lance enjoys the opportunity to mentor other young Christian leaders in their faith while serving as a small group coordinator at his local church. He is a member of the American Society of Civil Engineers, the Texas A&M chapter of the Institute of Transportation Engineers, and the National Society of Collegiate Scholars. Some of Lance's hobbies include road cycling, mountain biking, sports, guitar, and reading.

Lance has previously worked for the Energy Systems Laboratory of the Texas Engineering Experiment Station, and he also worked as an intern for Goodwin & Marshall Inc., a Civil Engineering firm in Grapevine, Texas.

Lance plans to attend graduate school to earn a master's degree in Civil Engineering and become a licensed Professional Engineer.

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SUMMARY

The objectives of this research were to evaluate the influence of stepping distance on average dry retroreflectivity measurements of profiled and rumble stripe pavement markings using a portable retroreflectometer and to compare these retroreflectivity measurements to measurements made with a mobile retroreflectometer and a CCD photometer.

The retroreflectivity of flat, profiled, and rumble stripe thermoplastic pavement markings were evaluated several ways by using multiple portable retroreflectometers, a mobile retroreflectometer, and a CCD photometer. Stepping distance was found to have no practical influence on averaged retroreflectivity measurements. Structural additions to the portable retroreflectometer and/or hand-leveling the device were suitable ways to maintain the retroreflectometer in the plane defined by the tops of the pavement marking profiles for proper measurement.

The vertical structure of the profiled and rumble stripe pavement markings that were evaluated did not increase the dry retroreflectivity measurements of the markings. The flat segments between the depressions of the rumble stripe were found to produce the highest retroreflectivity readings of any part of the rumble stripe.

The use of a properly calibrated mobile retroreflectometer operated by an experienced user will result in dry retroreflectivity measurements that are not practically different from portable retroreflectometer measurements. This validates the ability of the portable retroreflectometer to accurately measure profiled and rumble stripe pavement markings.

CCD photometer images were taken using a 30 m field geometry in order to simulate what the driver sees. This resulted in significantly lower retroreflectivity values than the portable and mobile retroreflectometers. The lower readings could be a result of the 30 m field geometry, transmission losses through the vehicle's windshield, or other factors. If the lower CCD photometer retroreflectivity values were due only to the 30 m field geometry as opposed to the coplanar 30 m geometry used by the handheld retroreflectometer, then the coplanar geometry may not be representative of the actual marking visibility.

TABLE OF CONTENTS

Student Biography 24
Acknowledgment 24
Summary 24
Table of Contents 26
Introduction 28
Background 28
  Retroreflectivity 28
  Handheld Retroreflectometers 29
  Pavement Markings 30
    Profiled Pavement Markings 31
    Rumble Stripes 31
  ASTM E 1710 – 05 32
  CEN EN 1436 33
  Mobile Retroreflectometer 33
  CCD Photometer 34
  Technical Note: RS 104 – DELTA 35
  Evaluation of Rumble Stripes – North Dakota DOT 35
  Effect of Rumble Strips and Pavement Marking on Lateral Placement of Vehicles 35
Goals and Objectives 35
Data Collection 36
  Test Locations 36
  Handheld Retroreflectometer Modification 36
  Height Tolerance Test 37
  Handheld Retroreflectometer Data Collection 38
  Mobile Retroreflectometer Data Collection 39
  CCD Photometer Data Collection 40
Data Analysis 40
Results 41
  Stepping Distance 41
  Rumble Stripe Retroreflectivity 43
  Profiled Pavement Marking Retroreflectivity 44
  Comparison of Handheld Retroreflectometers 46
  Mobile Retroreflectometer Results 47
  CCD Photometer Results 47
  Comparison of Methods 49
Findings 51
Recommendations 51
References 52
Appendix A: Comparison of Stepping Distances for Flat Pavement Marking 53
Appendix B: Comparison of Stepping Distances for Profiled Pavement Marking 55
Appendix C: Comparison of Stepping Distances for Rumble Stripe Pavement Marking 57
Appendix D: Comparison of Handheld Retroreflectometers for Flat Pavement Marking 59
Appendix E: Comparison of Handheld Retroreflectometers for Profiled Pavement Marking 61

Appendix F: Comparison of Handheld Retroreflectometers for Rumble Stripe Pavement Marking 63
Appendix G: Comparison of Speed for Flat Pavement Marking 65
Appendix H: Comparison of Speed for Profiled Pavement Marking 67
Appendix I: Comparison of Speed for Rumble Stripe Pavement Marking 69
Appendix J: Comparison of Photo Distance for CCD Photometer (Line Method) on Flat Pavement Marking 71
Appendix K: Comparison of Photo Distance for CCD Photometer (Line Method) on Profiled Pavement Marking 73
Appendix L: Comparison of Photo Distance for CCD Photometer (Line Method) on Rumble Stripe Pavement Marking 75
Appendix M: CCD Photometer Calibration Panel Correction 77
Appendix N: Handheld, Mobile, and CCD Photometer Comparison 78

INTRODUCTION

Retroreflective luminance, R_L (which is referred to as retroreflectivity in this paper), is an important characteristic of pavement markings because it is a surrogate measure of nighttime roadway visibility. Pavement markings with higher retroreflectivity are assumed to provide higher levels of roadway visibility during nighttime conditions. ASTM E 1710 – 05 defines a standard of measure of retroreflectivity for dry pavement markings using a portable (which is referred to as handheld in this paper) retroreflectometer. This allows engineers and transportation professionals to monitor the quality of pavement markings on the country's roads.

However, pavement markings are not all the same. As opposed to standard, flat pavement markings, profiled and rumble stripe pavement markings (which will be collectively referred to as structured pavement markings) have intermittent vertical surfaces created by peaks and depressions that are formed during installation. While these two types of markings are typically used to create an audible and/or tactile warning when crossed by the driver, the raised or depressed faces are thought to improve visibility in both dry and wet conditions.

The uneven surfaces of structured pavement markings can make it difficult to measure the retroreflectivity of the markings using the handheld retroreflectometer. Little information exists concerning standard measurement protocol of these types of pavement makings, and as a result, concerns exist regarding the reliability of retroreflectivity measurements made on these marking types. Additional details are needed for a standard protocol to ensure that all pavement markings are measured in the same manner every time.

This research intends to analyze retroreflectivity measurements of profiled and rumble stripe pavement markings to develop recommendations for a standard test method. For these same markings, this research also compares retroreflectivity measurements of handheld retroreflectometers, a mobile retroreflectometer, and a CCD photometer.

BACKGROUND

This section provides brief summaries of topics necessary to understand various aspects of this research project.

Retroreflectivity

The ability of an object to reflect light back in the general direction of its source is called retroreflectivity. Figure 1 shows how pavement markings achieve retroreflectivity through the use of small glass beads embedded in the surface of the marking material. First the emitted light is refracted when entering the glass bead. Next the light reflects off the pavement marking material. Then the light is refracted again when exiting the bead before finally returning toward the source of emission.

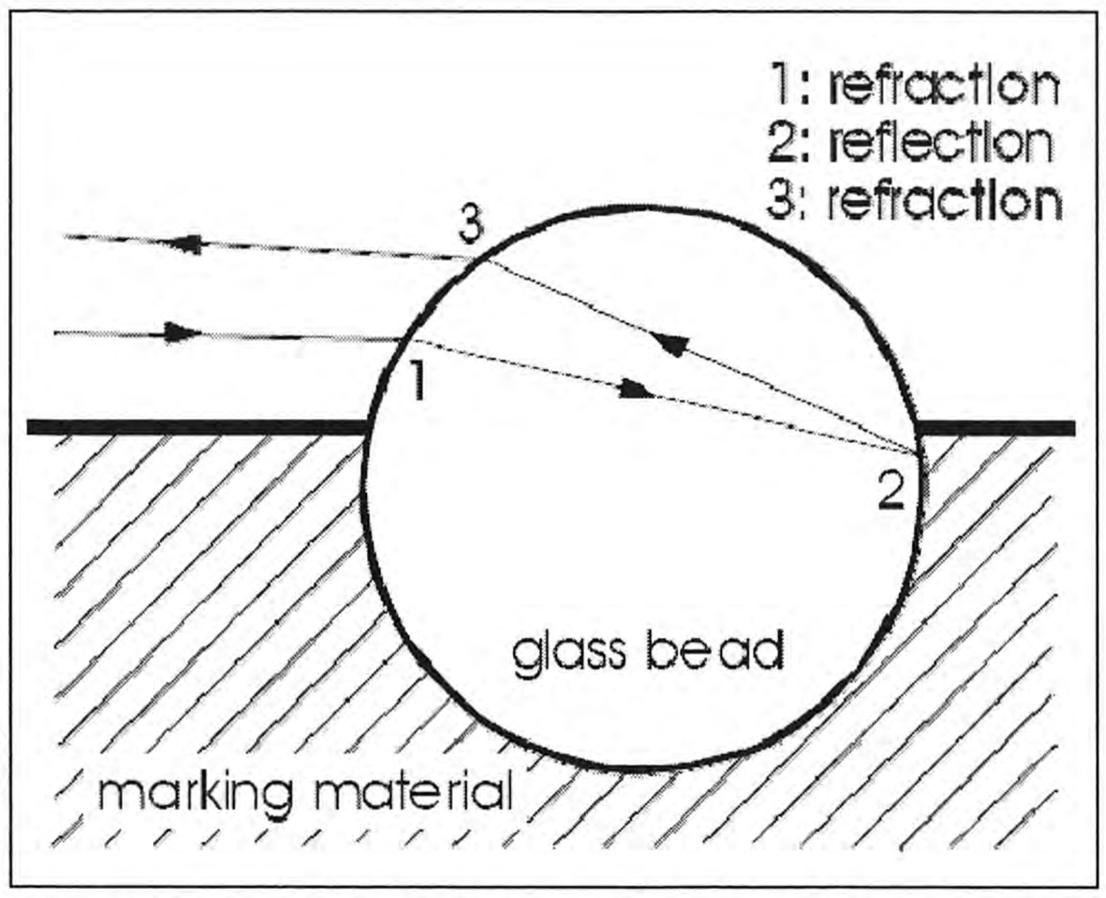

Figure 1. Glass Bead Retroreflection (1).

A retroreflectometer is a device that measures the coefficient of retroreflective luminance, R_L (referred to as retroreflectivity in this paper), of an object. *Illuminance* is the intensity of light emitted from the source that reaches the object, and *luminance* is the amount of light per unit area that is reflected from the surface of the illuminated object. In general terms, retroreflectivity is the ratio of luminance to illuminance.

Handheld Retroreflectometers

Handheld retroreflectometers measure retroreflectivity of pavement markings by illuminating the pavement marking surface and then measuring the retroreflected luminance. They do so using the standard geometry set forth by ASTM E 1710. This geometry is meant to simulate what a driver in an average U.S. automobile would see at night (*2*). The illumination (entrance) angle and the reception (observation) angle are defined by an illumination distance of 30 m, a driver height of 1.2 m, and a headlamp height of 0.65 m (see Figure 2).

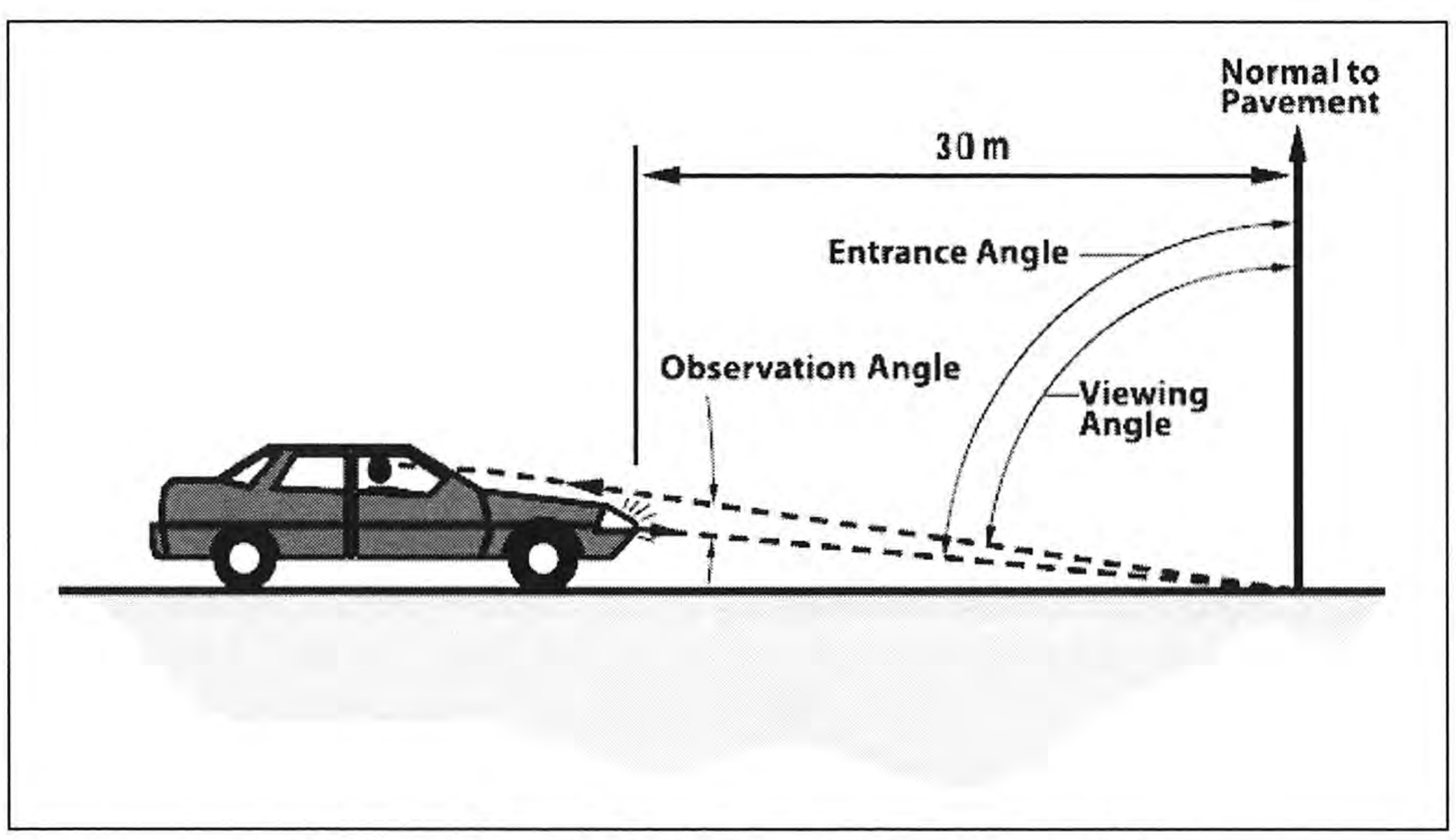

Figure 2. 30 Meter Geometry (*2*).

While handheld retroreflectometers take measurements using the same angles defined by the 30 m geometry, they do so at a much smaller scale in order to be portable. Two handheld retroreflectometer models were used for this research. Handheld Retroreflectometer 1 had an approximate scale of 1:90, and Handheld Retroreflectometer 2 had a scale of 1:112.

Handheld retroreflectometers may have one of two optical measurement configurations. For arrangement A the length of the receptive field is fully included within the length of the illumination field. Figure 3 shows an example of arrangement B where the illumination field is fully included within the reception field (*3*). The measurement field is where the illumination and reception fields overlap. It is recommended to use a handheld retroreflectometer of arrangement B especially when measuring structured markings (*3, 4*).

Handheld retroreflectometers are portable due to the scaled version of the 30 m geometry. However, it also causes the device to be sensitive to how it is placed on the pavement marking, which makes it particularly challenging to place the device on some types of pavement markings such as profiled and rumble stripe pavement markings.

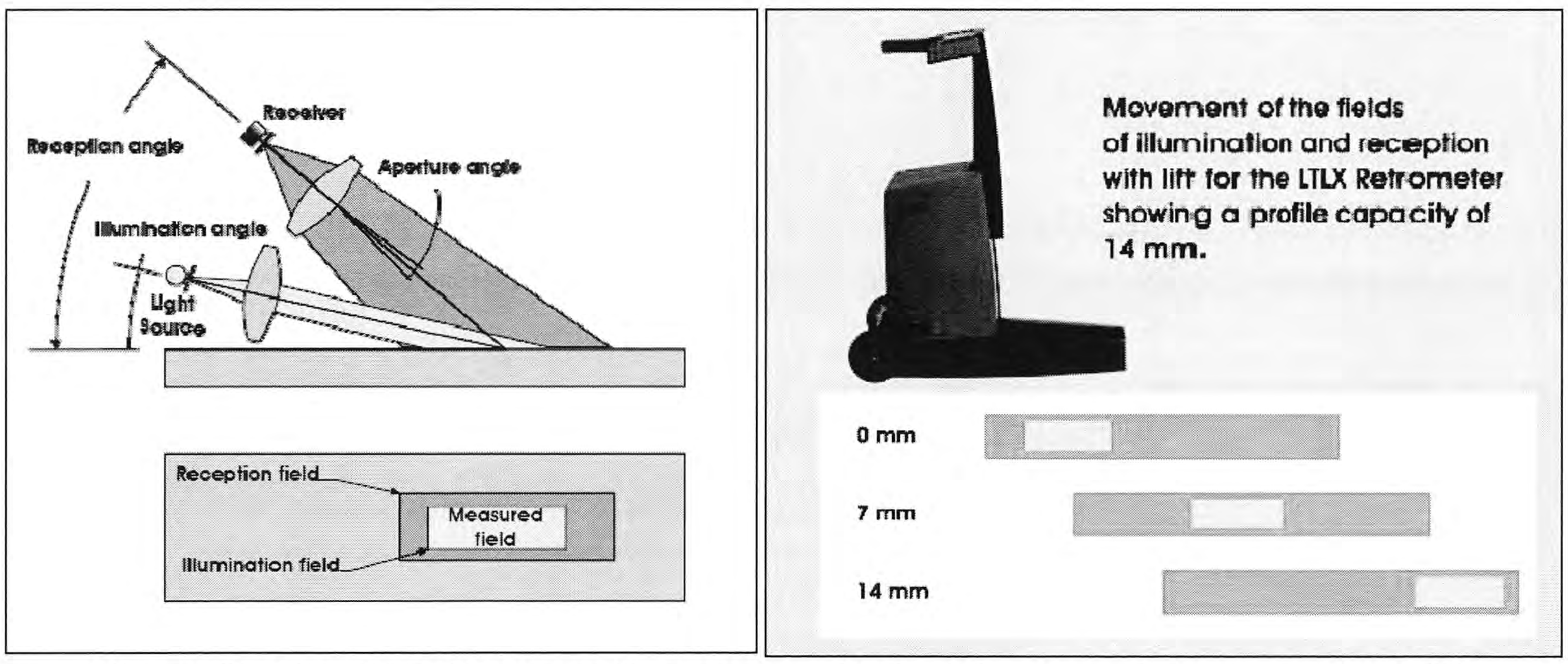

Figure 3. Handheld Retroreflectometer Optical System, Arrangement B (*5*).

As the retroreflectometer is tilted or placed above a depressed area of the pavement marking surface, the illumination field moves away from the device at a faster rate than the reception field (see Figure 3). Eventually, the illumination field will no longer be fully included within the receptive field causing erroneous readings when measuring above a certain height. The height to which the retroreflectometer can be lifted with a measurement change of less than 10 percent is called the height tolerance (*3*, *4*).

A retroreflectometer with arrangement B is considered capable of measuring a structured pavement marking if the height differences in the marking are less than the height tolerance (H) and/or if the gaps between the structured parts of the markings are less than 46 times the height tolerance (*5*).

Pavement Markings

Most pavement markings are flat, but some markings are structured to provide a means of creating an audible or tactile warning and/or to improve night and wet-night visibility characteristics. However, since some markings have a structured profile, consistently placing the handheld retroreflectometer on the marking for measurement can be difficult. This study focused on a flat marking and two types of structured markings, a profiled marking and a rumble stripe marking. The two following sections describe profiled and rumble stripe pavement markings in greater detail.

Profiled Pavement Markings

The type of profiled (also called raised-profile) pavement marking evaluated in this research was a flat pavement marking with short, raised sections of thicker marking material at uniform spacings. The profiled pavement markings were white edge lines with a raised profile height of 0.25 in and a length of 3 in, repeated every 12 in. An example can be seen in Figure 4.

Figure 4. Profiled Pavement Marking.

The profiled markings were approximately two years old and were made of thermoplastic material with type II beads. There are other types of profiled markings, but usually with much more closely spaced vertical surfaces such as inverted profile markings, some types of preformed tape, and some splatter markings (see Section 3A.04 of the MUTCD) (*6*).

Rumble Stripes

Rumble stripes are pavement markings that have been applied over a milled rumble strip. A milled rumble strip is made by a machine that mills the pavement at uniform spacings creating depressions. In Texas, typical milled rumble strips have depressions that are 7 in long (longitudinal to the centerline) and have a 5 in gap between each depression. They have a width of 16 in and a depth that can vary between 1/2–5/8 in. The rumble stripes used for this research had a depth of 1/2 in. The rumble stripes were approximately three years old and were made of thermoplastic material with type II beads. An example of the rumble stripe markings from the study can be seen in Figure 5.

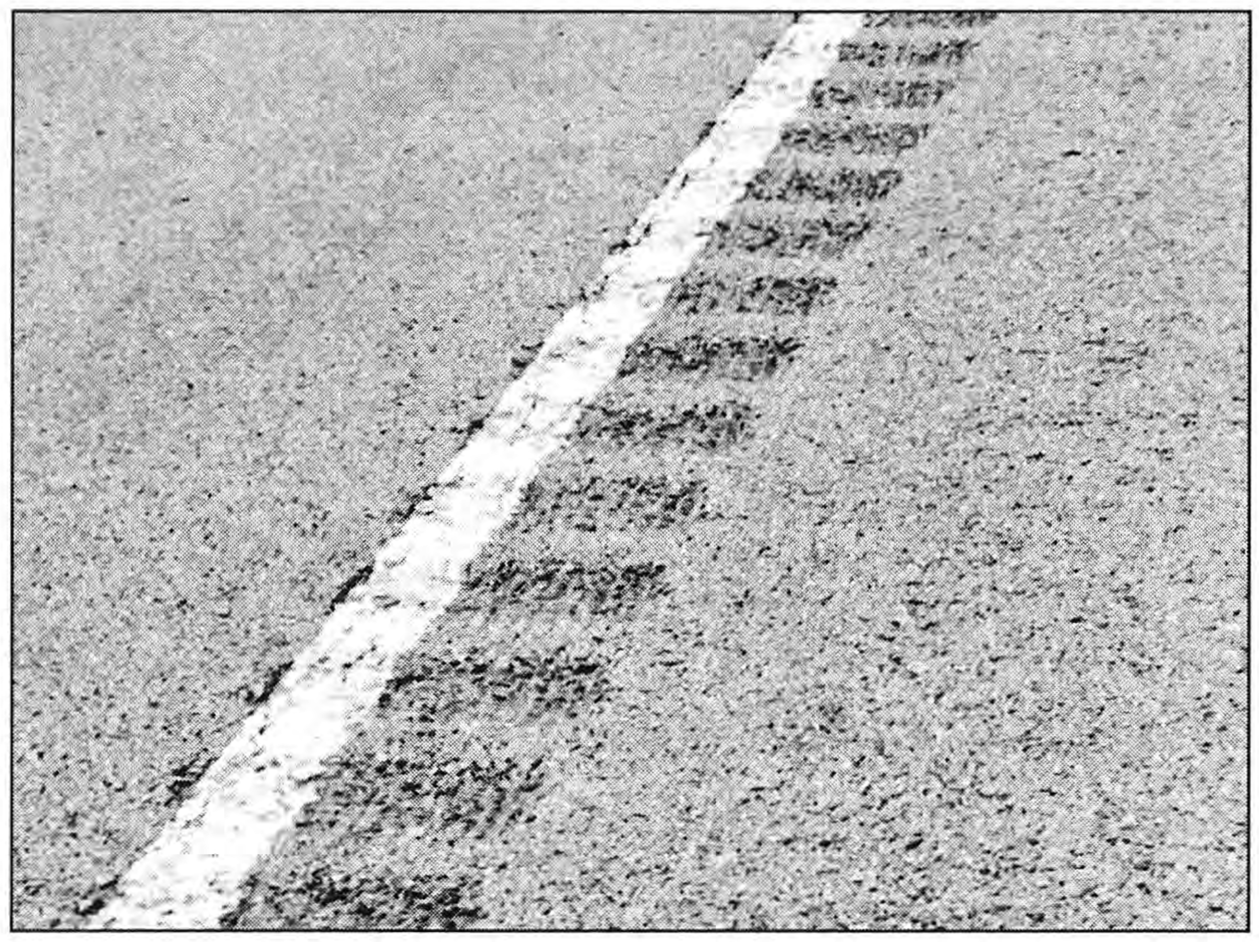

Figure 5. Rumble Stripe Pavement Marking.

ASTM E 1710 – 05

ASTM Designation E 1710 is the standard test method for the measurement of retroreflective pavement marking materials using a portable retroreflectometer. In reference to structured markings, Section 6.4.7.4 states,

> For fixed-aim instruments, when measuring profiled pavement markings, move the instrument laterally using sufficiently small steps, while maintaining it essentially in the plane defined by the tops of the profiles, take and average the readings at each location covering in total one or more profile spacings (*3*).

At the end of Section 6.4.7.4, Note 11 goes on to state the following:

> The stepping distance should be at most the length of the measurement area [of the handheld retroreflectometer]. For markings with regularly spaced profiles, the stepping distance D should be selected so that D × N, where N is an integral number, equals a small integral number of profile spacings, for example one or two. Readings are taken at N locations and the average is used to represent the R_L of the profiled pavement marking (*3*).

In summary, the stepping distance (D) should be less than the measurement length of the retroreflectometer and a number that is a divisor of the structured pavement marking segment length (S). Figure 6 gives a visual representation of this principle for a rumble stripe.

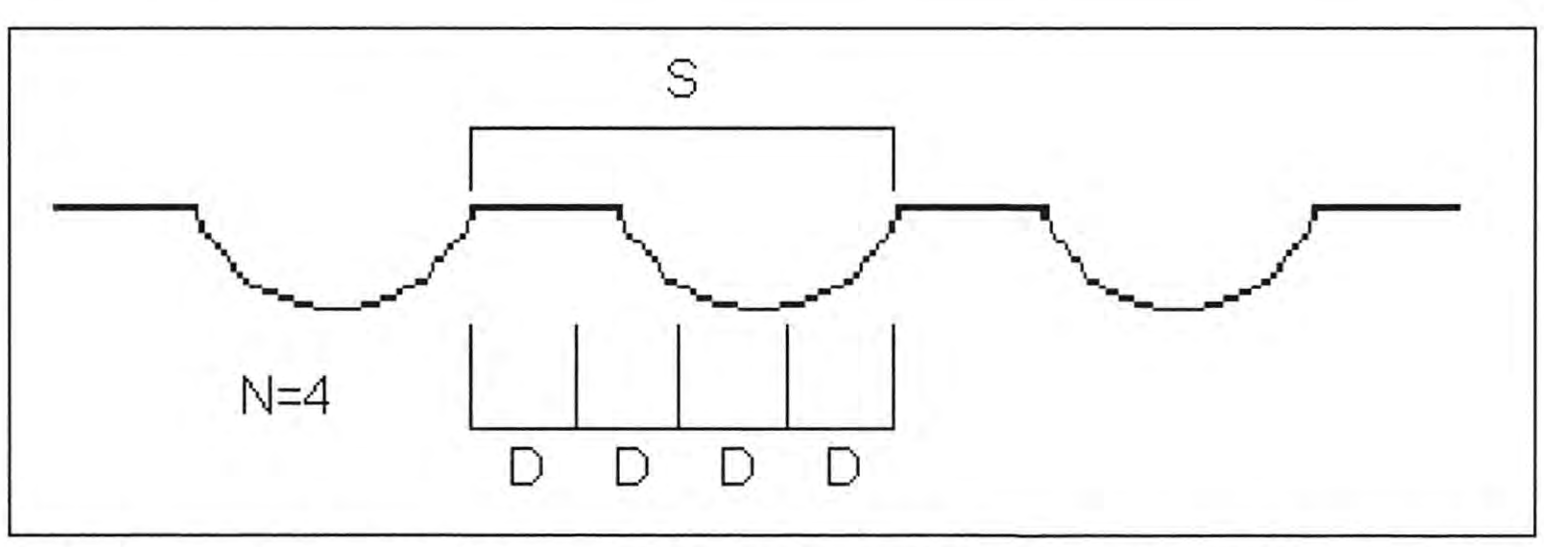

Figure 6. Example of Stepping Distance for Rumble Stripe Segment.

For example, given a typical rumble stripe spacing of 12 in and a handheld retroreflectometer measurement length of 20 cm (approx. 8 in), a stepping distance (D) of 1, 2, 3, 4, or 6 in would be acceptable for one spacing. For two spacings, three readings at 8 in apart would also be acceptable.

CEN EN 1436

In reference to structured markings, the European standard for road marking performance states,

> When measuring a structured road marking with a portable instrument, it is necessary to establish if the instrument in question is able to measure the structured road marking with the actual height of structures and gaps between these. The R_L value is established as the average of a number of readings taken with shifts of the instrument in steps along the marking, in total covering one or more spacing of structures (*7*).

It also goes on to state that "portable instruments may be tilted and shifted in height" for structured road markings.

Mobile Retroreflectometer

Another device used for measuring the retroreflectivity of pavement markings is the mobile retroreflectometer. This technology uses a retroreflectometer attached to the side of a vehicle (see Figure 7). The device transmits light to the pavement marking, and a receiver captures the subsequent returning light (*8*). This device measures retroreflectivity at a 1:3 scale of the 30 m geometry. The retroreflectometer takes continuous readings at approximately 18 Hz while the vehicle is in motion. These values are then averaged over a user specified distance. The distance over which these measurements are averaged is called the acquire frequency. The minimum acquire frequency is 0.005 miles (26.4 ft).

Figure 7. Mobile Retroreflectometer Outfitted on Vehicle.

The mobile retroreflectometer has a number of advantages and disadvantages for its application to this research.

Advantages

- As compared to the handheld retroreflectometer, the larger scale of the mobile retroreflectometer makes it more representative of what the driver sees.
- There is no device placement issue measuring structured markings since the retroreflectometer is not placed on the uneven marking surface to take measurements.

Disadvantages

- Calibration of the mobile retroreflectometer is much more complex and time consuming than for the handheld retroreflectometer.
- The 10 m geometry is still susceptible to errors due to things like road elevation changes.
- It requires a much longer sample for measurement than the handheld retroreflectometer.
- The retroreflectometer can get dirty from bugs or dirt from the roadway, which requires inspection and cleaning periodically.

CCD Photometer

The CCD (Charge-Coupled Device) photometer is a camera used to analyze the luminance of objects. The CCD photometer takes a digital photograph of the study object. The resulting image can be analyzed to obtain luminance values. By measuring the illuminance at various points in the image, retroreflectivity can be calculated. An example of a CCD photometer image of a profiled pavement marking can be seen in Figure 8.

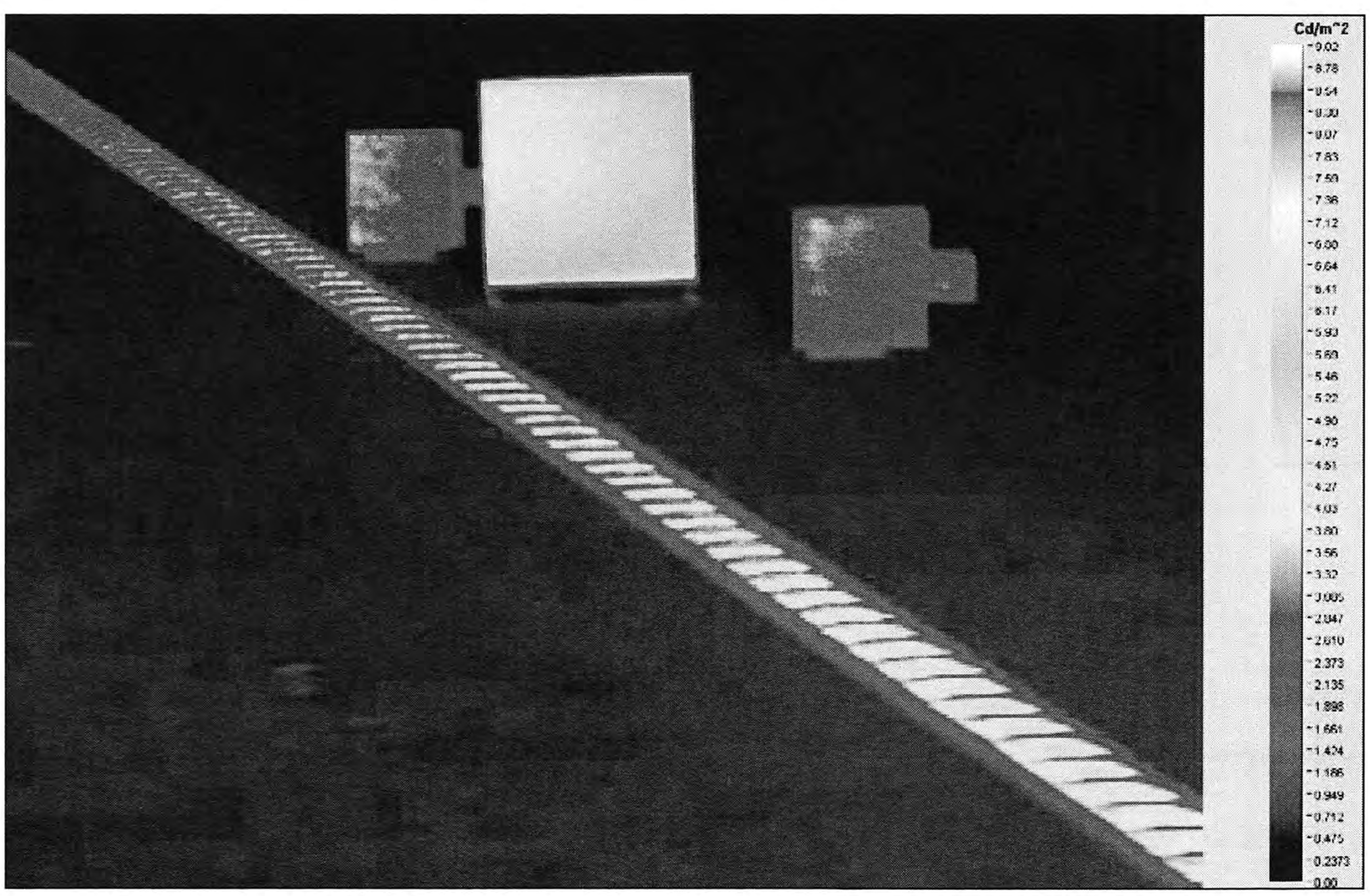

Figure 8. CCD Photometer Image of Profiled Pavement Marking.

One of the greatest advantages of the CCD photometer is that it is most representative of driver visibility because an actual image can be taken from the driver's perspective. A major disadvantage is that the CCD photometer needs to be used at night so that the only source of illumination is the vehicle headlamps. The CCD photometer is also vulnerable to errors due to things like a dirty windshield, poor calibration, poor photo exposure, and other environmental factors.

Technical Note: RS 104 – DELTA

DELTA, a manufacturer of retroreflectometers and other optical equipment, published Technical Note: RS 104 with information and recommendations regarding the use of handheld retroreflectometers to measure structured pavement markings. This document also stresses the need for sufficient height tolerance when measuring structured pavement markings.

It is recommended that if the retroreflectometer cannot span over the gaps in a structure, it should be supported so that it remains in the plane defined by the top of the profiles. This can be done by attaching rails to the retroreflectometer or by placing objects under the device to level it (*4*).

Evaluation of Rumble Stripes – North Dakota DOT

The purpose of the study conducted by the North Dakota Department of Transportation was to determine if rumble stripes improved visibility in comparison to flat pavement markings. A stepping distance of 1 in was used to determine if the retroreflectivity varied along the length of the rumble stripe. This study concluded that "there were no obvious patterns to the variations in retroreflectivity readings of the rumble stripes and no area of the rumble stripe appeared to have unusually high retroreflectivity" (*9*). While the dry retroreflectivity levels of each pavement marking were not significantly different, "the wet-night readings appeared to show that rumble stripes provided higher retroreflectivity readings than nearby flat markings" (*9*). The study did not describe the type of handheld retroreflectometer used.

Effect of Rumble Strips and Pavement Marking on Lateral Placement of Vehicles

While the objective of this study was to determine the effectiveness of rumble stripes to keep vehicles in the traveled lane, the researchers cite a presentation about rumble stripe pavement markings on I-75 in Michigan, which showed that both dry and wet rumble stripe markings had greater retroreflectivity values than standard flat edge lines (*10, 11*). There was no mention of how the study measured the retroreflectivity of these markings.

GOALS AND OBJECTIVES

The goal of this research was to evaluate retroreflectivity measurement techniques and their results regarding retroreflected luminance of profiled and rumble stripe pavement markings. This goal was reached by completing the following objectives:

1. Evaluating the influence of stepping distance on average dry retroreflectivity measurements of profiled and rumble stripe pavement markings using a handheld retroreflectometer.
2. Evaluating the ability of handheld retroreflectometers to accurately measure the dry retroreflectivity of structured pavement markings as opposed to other measurement techniques.

DATA COLLECTION

The following sections describe the data collection procedure for each aspect of this project. The height tolerance test was completed in the lab at the Texas Transportation Institute State Headquarters and Research Building. Since each test location required a night lane closure, the handheld and CCD data at each site were collected on separate nights. However, the mobile data were collected at all three sites in one day.

Test Locations

Three separate test locations were selected, one for each pavement marking type (flat, profiled, and rumble stripe). Locations were selected based on a number of criteria:

1. Pavement marking type and condition (based on daytime visual assessment).
2. Reasonable proximity to TTI headquarters.
3. Multiple lanes to maintain traffic flow during night lane closure.
4. Good pavement marking condition with low variability in retroreflectivity measurements.
5. Limited or no horizontal or vertical curvature of roadway.

The following locations were selected and can be seen in Figure 9:

1. Flat – SH 47 near Silver Hill Road, west of Bryan, Texas.
2. Profiled – SH 21 between FM 1624 and FM 2440.
3. Rumble Stripe – SH 6 north of US 79 overpass in Hearne, Texas.

Handheld Retroreflectometer Modification

Two models of handheld retroreflectometers were used in this research. They are referred to as Handheld Retroreflectometers 1 and 2. In order to properly measure retroreflectivity of structured pavement markings using the handheld retroreflectometer according to ASTM E 1710, measurements must be taken "while maintaining [the device] essentially in the plane defined by the tops of the profiles" (*3*). For this research, Handheld Retroreflectometer 1 was modified by replacing the bottom plate with one that extended 8 in out from the rear of the device (see Figure 10). This allowed the device to span the gap between the raised sections of the profiled markings and across the depressions of the rumble stripes.

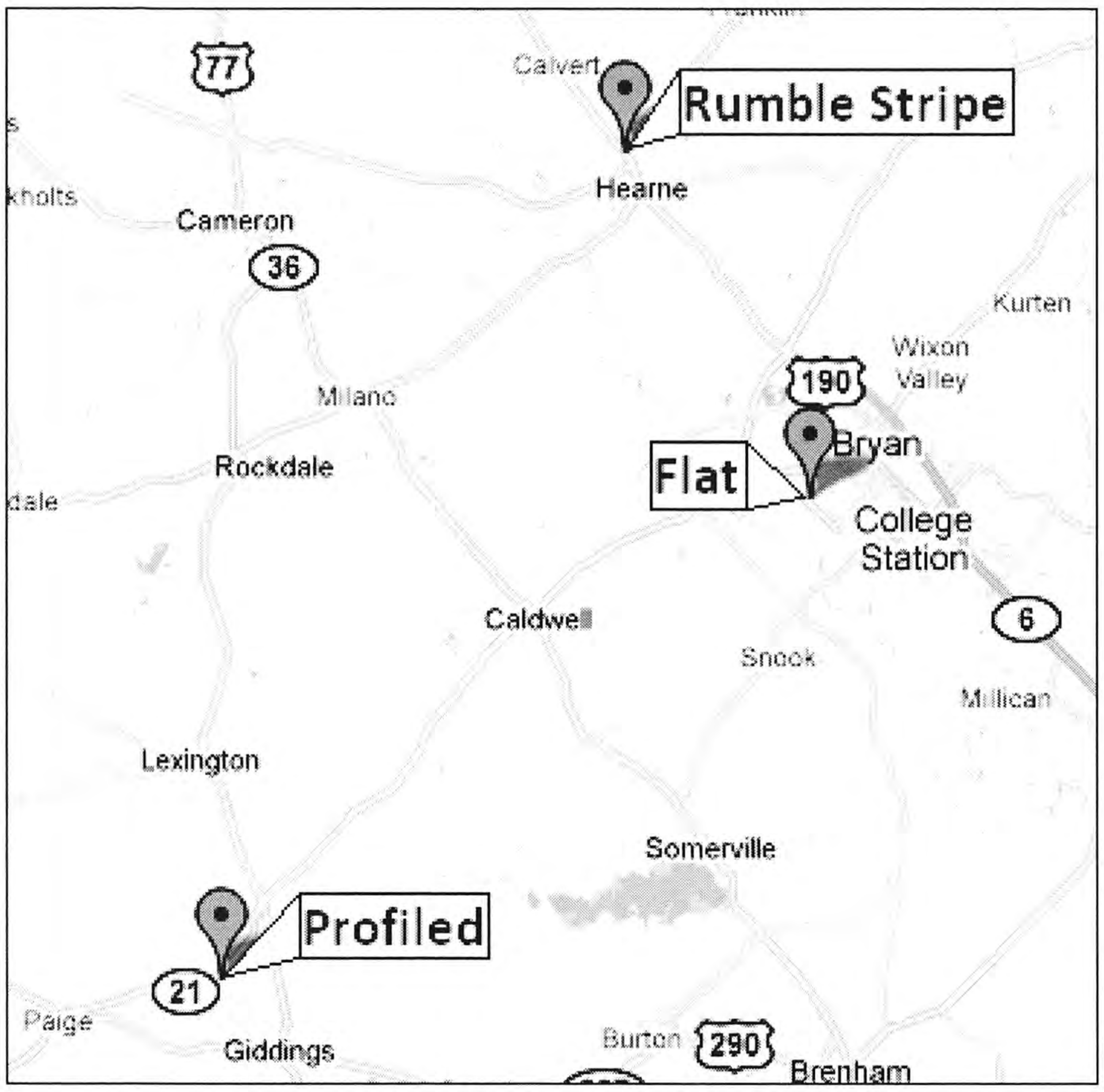

Figure 9. Test Location Map.

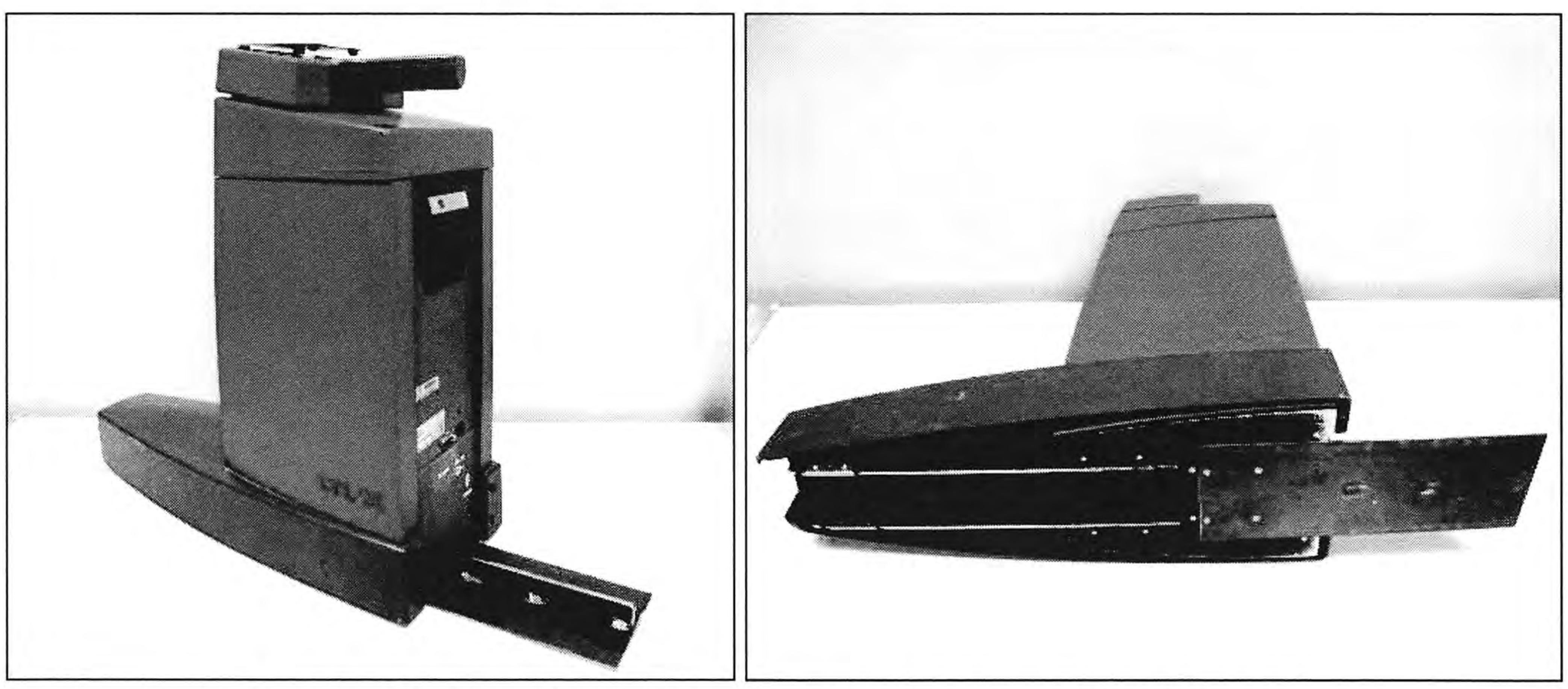

Figure 10. Handheld Retroreflectometer Modification.

Height Tolerance Test

To confirm that the handheld retroreflectometers used for data collection were capable of measuring the structured markings, the researcher completed a height tolerance test on Handheld Retroreflectometer 1 and 2. First, a R_L measurement was taken on the surface of flat pavement marking tape panel. The next R_L measurement was taken after raising the instrument 0.08 in by placing shims under each of the three contact points. Each time the instrument was raised 0.08 in, the device was moved backwards 3.68 in to keep the measured field in the same location. The researcher repeated these steps until the readings decreased by at least 10 percent. The test was performed on two pavement marking panels. The results of the height tolerance tests can be seen in Figure 11.

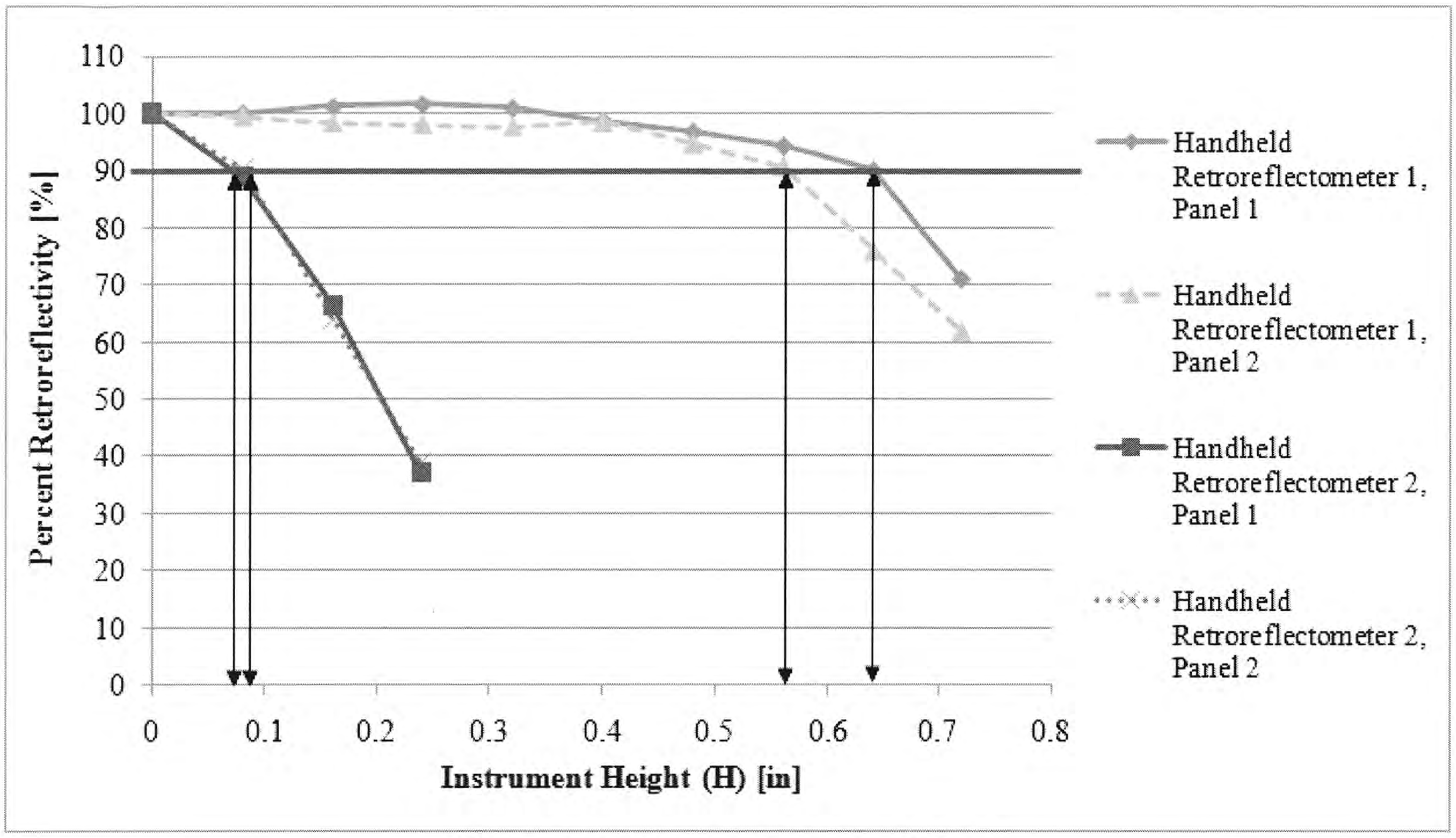

Figure 11. Height Tolerance Test.

As seen in Figure 11, the height tolerance of Handheld Retroreflectometer 1 exceeded 0.5 in for both trials. Since the pavement markings to be measured had a maximum height difference of less than 0.5 in and a maximum span between structures of less than 46 × 0.5 in, Handheld Retroreflectometer 1 was found adequate to measure the structured markings. While Handheld Retroreflectometer 2 did not have an acceptable height tolerance for this project, the researcher still decided to collect additional data with the device for further analysis.

Handheld Retroreflectometer Data Collection

In order to have a sample segment long enough to measure using the mobile retroreflectometer, a sample length of 170 ft was used. To reduce the handheld measurements for the 170 ft segment, three 10 ft segments (one each at the beginning, middle, and end of the 170 ft segment) were measured using the handheld retroreflectometer. Each of these three segments could then be averaged and compared to the mobile data for the whole 170 ft segment. Prior to taking the test measurements, several spot checks along the 170 ft length were taken to ensure the R_L values were similar.

At the beginning of each of the three data collection areas, the handheld retroreflectometers were calibrated and periodically checked throughout the data collection procedure. The starting position of the center of the retroreflectometer measurement field was documented for each segment in order to analyze how retroreflectivity values change along the length of the segment in relation to the structure of the markings.

To analyze the influence of stepping distance on averaged handheld retroreflectometer measurements, readings were taken using different stepping distances on the same pavement marking sample. Since both structured markings had uniform segment lengths of approximately

12 in and the measurement field of Handheld Retroreflectometer 1 was approximately 8 in long, stepping distances of 1, 2, 3, 4, 6, and 8 in were used in accordance with ASTM E 1710. Readings using these stepping distances were taken on each pavement marking type using the modified Handheld Retroreflectometer 1 (see Figure 10). Figure 12 shows the researcher using Handheld Retroreflectometer 1 to measure a 10 ft segment of rumble stripe.

Figure 12. Handheld Retroreflectometer Data Collection on Rumble Stripe.

Readings using other devices and methods were taken for comparison by using only select stepping distances in order to reduce the amount of data collection because of time constraints. These readings were taken using a standard, unmodified Handheld Retroreflectometer 1 and Handheld Retroreflectometer 2. No additional measurements were taken using the standard Handheld Retroreflectometer 1 on the flat pavement marking since there would not have been a difference measuring the flat pavement marking using the modified or standard Handheld Retroreflectometer 1.

For the readings using the standard Handheld Retroreflectometer 1, one set of measurements were taken leaving the device free to tilt on the structured markings while the other was hand-leveled. For the free-to-tilt readings, the standard Handheld Retroreflectometer 1 was held in level in position and then released and allowed to rest naturally on the structured marking. The hand-leveled measurements were taken while an experienced handheld retroreflectometer user held the device level in the plane defined by the tops of the profiles of the pavement marking.

Mobile Retroreflectometer Data Collection

Prior to any mobile data collection, the mobile retroreflectometer was calibrated by an experienced mobile retroreflectometer user. The mobile retroreflectometer was calibrated to a section of flat edge line by adjusting the values to measurements taken using a calibrated handheld retroreflectometer. The mobile retroreflectometer was calibrated to within 3 percent of the handheld retroreflectometer readings on the flat line calibration section.

The researcher then used the mobile retroreflectometer to measure the retroreflectivity of the 170 ft test section. Readings were taken at driving speeds of 40, 50, 60, and 70 mph. After completing the measurements at a site, the researcher measured the calibration section on the flat marking once again to confirm the accuracy of the recently collected data.

CCD Photometer Data Collection

A 2004 Ford Taurus was used to take the CCD photometer images since it would closely simulate the 30 m geometry headlight and driver height. This geometry differs from the ASTM specified 30 m geometry. While the illumination distance and observation angles are the same at a distance of 30 m, the luminance is measured from a horizontal offset position rather than directly over the source of illumination as specified by the ASTM E 1710 because the camera was placed to view the road from the driver's perspective. The car was positioned in the middle of the traveled lane. Figure 13 shows the CCD photometer configuration used to create the 30 m field geometry.

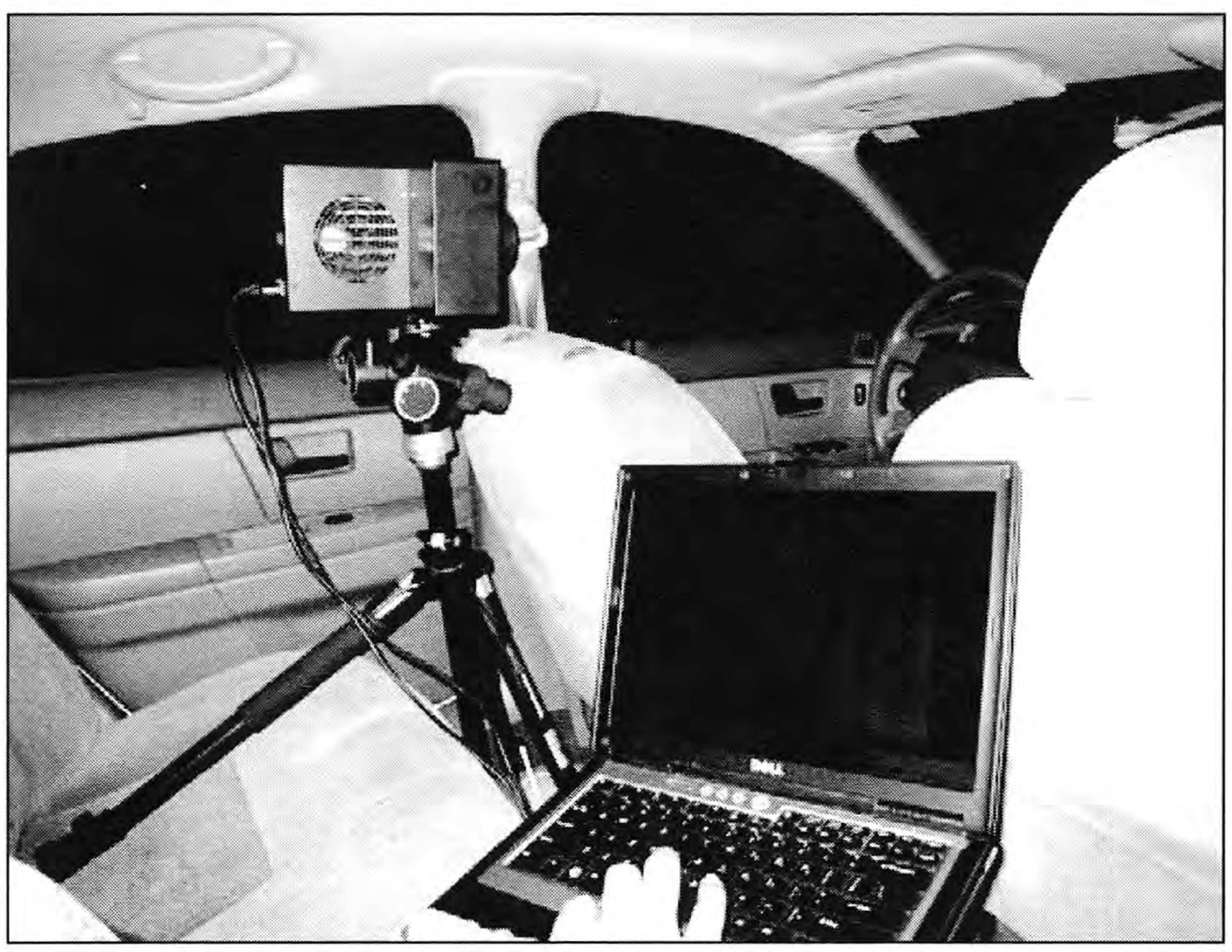

Figure 13. CCD Photometer Set-Up.

CCD photometer images were taken at a distance of 80, 100, and 160 ft from the center of each of the three 10 ft segments that were measured by the handheld retroreflectometer. The 100 ft distance was used to simulate the 30 m field geometry. For each of these images, illuminance readings were taken at the front, center, and rear of each 10 ft measurement section. Once the images were analyzed for luminance values, the illuminance readings were used to calculate retroreflectivity.

DATA ANALYSIS

The handheld retroreflectometer data were analyzed by comparing the mean and variance of readings taken at different stepping distances to see if any of the stepping distances yielded significantly different averages. This statistical analysis was completed through the use of Oneway ANOVA and Kruskal-Wallis tests. Once a significant difference was established, Tukey-Kramer HSD and Student's t tests were used to analyze which stepping distances were significantly different from one another. The Tukey-Kramer HSD test is more conservative than the Student's t test.

Measurements taken using the mobile retroreflectometer resulted in a small number of readings (usually 3–6) over the length of the 170 ft measurement section. The averages for each section were compared to the averaged values of the handheld and CCD measurements. These values were then statistically analyzed for the influence of speed on retroreflectivity readings for each pavement marking type using the Tukey-Kramer HSD test.

The CCD photometer images were analyzed in two ways using the Radiant Imaging ProMetric 9.1 software. In the first method of analysis, the researcher outlined a complete 10 ft segment. The software then averaged the luminance from that segment and output a single luminance value. The researcher then calculated the retroreflectivity of the segment using the average illuminance readings over that segment. This method was referred to as the area method.

The second method of CCD photometer analysis was referred to as the line method. In this method, the researcher used the cross-section tool within the software to measure the luminance values along the length of a line running down the middle of the 10 ft segment. The software output luminance and position values for each pixel on the line. Since the length of the line was known to be 10 ft, the position values were scaled to the 10 ft line segment. Illuminance readings for the same segment were used to create a linear regression that related position to illuminance. Using the scaled position value and the illuminance regression, illuminance values were calculated for the whole length of the line. These corresponding luminance and illuminance values were then used to calculate retroreflectivity. This resulted in a retroreflectivity profile for the whole length of the 10 ft segment. The line method data were then analyzed using the Tukey-Kramer HSD test to observe the effect of photo distance on retroreflectivity values for each pavement marking type.

The handheld, mobile, and CCD photometer data for each pavement marking type were compared using a Least Squares Means Differences Tukey HSD test. This test indicated which measurement method and pavement type pairings were significantly different from one another.

RESULTS

The results from the previously mentioned data collection and data analysis are described in detail in the following sections.

Stepping Distance

As seen in Table 1, the average retroreflectivity values of a particular pavement marking type varied little due to stepping distance. The values in Table 1 were taken using only the modified Handheld Retroreflectometer 1.

Table 1. Retroreflectivity vs. Stepping Distance.

Pavement Marking Type	Stepping Distance [in]						Avg
	1	*2*	*3*	*4*	*6*	*8*	
Flat	-	170	-	168	171	172	*170*
Profiled	151	148	148	147	147	146	*148*
Rumble Stripe	162	161	161	160	160	160	*161*
	Retroreflectivity (R_L) [mcd/m²/lux]						

The Tukey-Kramer HSD test (see Appendix A) revealed that the 4 in and 8 in stepping distances were significantly different from the 2 in and 6 in stepping distances for the flat pavement marking sample. For the profiled pavement marking data, the Tukey-Kramer HSD test found that the 1 in and 2 in stepping distance were significantly different than the rest. The less conservative Student's t test found that only the 1 in readings for the profiled marking were significantly different than the rest (see Appendix B). There were no significant differences in retroreflectivity among stepping distances for the rumble stripe (see Appendix C). These relationships are shown in Figure 14. The stepping distances that are not statistically significantly different from one another are shown in the same group. Rumble stripe data are not shown because there were no significant differences among stepping distances for the rumble stripe.

While statistically significant differences between stepping distances were found for the profiled pavement markings, statistically significant differences were also identified for the flat markings. In both cases, these differences were not considered practically significant. Small changes in exact measurement location between stepping distances could have caused the small changes in readings. Also, the average retroreflectivity for the structured pavement markings did not vary more than the flat pavement marking. For these reasons, even though some retroreflectivity differences between stepping distances for some pavement marking type were statistically significantly different, these differences were not considered practically significant.

In order to further understand these differences, findings from a recent ASTM precision and bias test were used. In February 2010, a group of interested parties conducted a set of handheld measurements on dry pavement marking panels in order to build a data set that could be used for a precision and bias statement in ASTM E 1710. ASTM intends to ballot the results during the fall of 2010. The results of the ASTM testing from a flat thermoplastic panel with type II beads were 16 mcd/m^2/lux for their repeatability limit and 65 mcd/m^2/lux for their reproducibility limit. Using these limits to test the practicality of the findings presented in Table 1, one can conclude that there are no practical differences among stepping distances for flat, profiled, or rumble stripe pavement markings (at least of the variety tested herein).

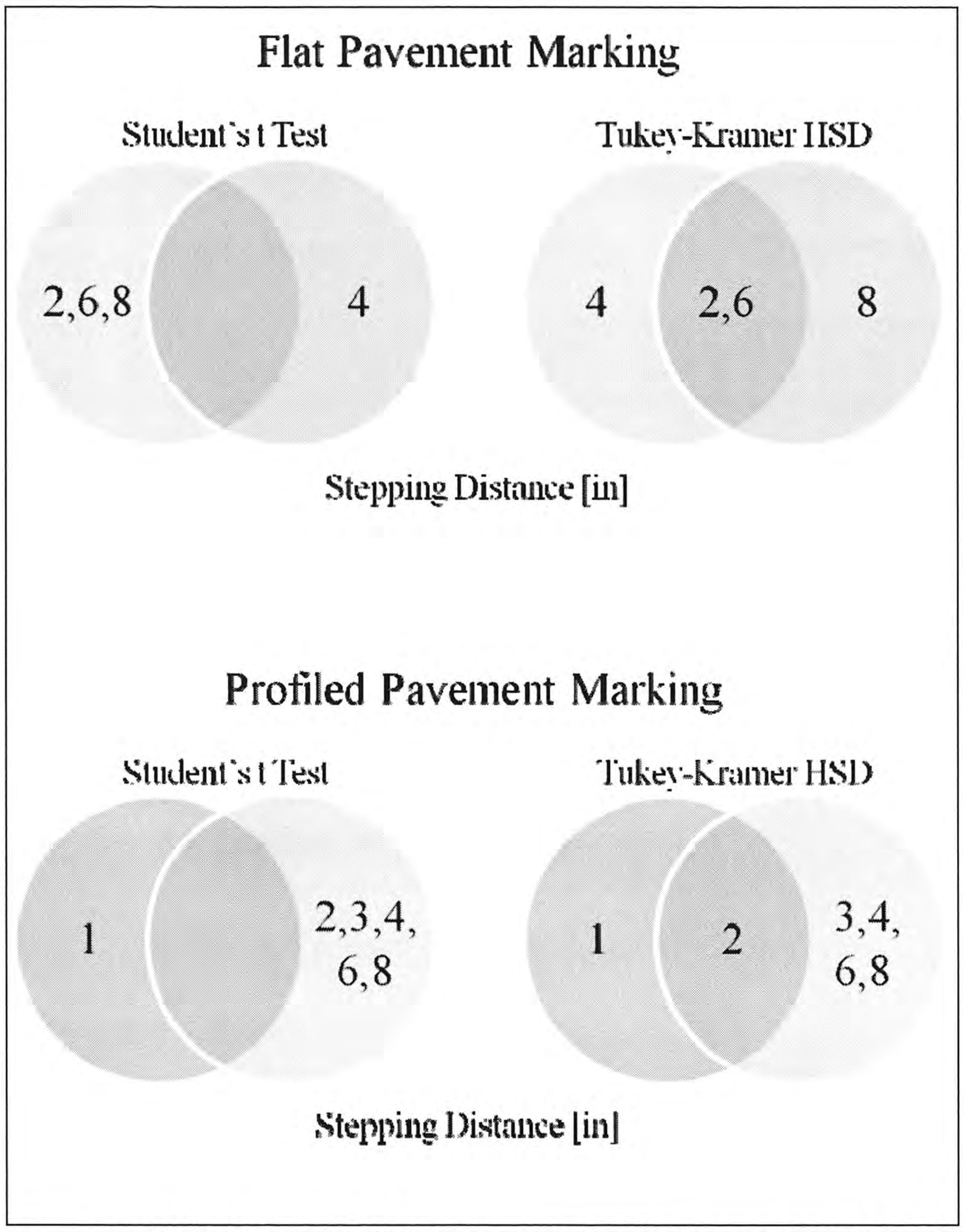

Figure 14. Significant Differences among Stepping Distances.

Rumble Stripe Retroreflectivity

While the variation of retroreflectivity measurements between different stepping distances was found to be small, the retroreflectivity readings did vary along the pavement marking profile. Figure 15 shows the retroreflectivity values for a 2 ft section of rumble stripe at each stepping distance using the modified Handheld Retroreflectometer 1. The first reading was taken with the center of the measurement field at the beginning of the depression of the rumble stripe. The data series "Pavement Profile" represents the corresponding pavement marking profile of the rumble stripe.

The readings for the rumble stripe varied outside the bounds of the repeatability limit of the ASTM E 1710 precision and bias statement. However, the average readings for each stepping distance did not. Therefore it is necessary to average several incremental readings over one or more repeated pavement marking segments to get a representative value of the marking as a whole.

The results seen in Figure 15 are typical of the readings from the rumble stripe. As the center of the measurement field moved from the flat section into the depression, the retroreflectivity began to decline. Once the middle of the measurement field moved back to the flat section, the retroreflectivity increased.

One would have assumed that the retroreflectivity values would have increased as the vertical face of the rumble stripe began to enter the measurement field of the device. However, the values did not increase until the center of the measurement field reached the beginning of the flat section between the depressions.

This relationship between handheld retroreflectivity readings and rumble stripe profile position could have been the result of many factors. The depressions of the rumble stripe may actually be detrimental to the retroreflectivity of the rumble stripe as a whole. The handheld retroreflectometer may read the flat part of the marking as having the higher retroreflectivity than the vertical face of the rumble stripe depression even though previous research has shown that both dry and wet rumble stripe markings have greater retroreflectivity values than standard flat edge lines (*10, 11*). The physical condition of the markings studied may also have impacted the readings as the vertical face of the markings may have experienced more wear than the top of the marking during its three years of service.

Another possibility is that certain parts of the handheld retroreflectometer measurement field have greater influence on the retroreflectivity readings than others. The parts of the measurement field farthest from the luminance receptor could return light of less intensity to the device than the closer areas. This could cause objects in the measurement field that are nearer to the optical device to have greater influence on the reading.

Height tolerance issues could also explain the rumble stripe retroreflectivity profile. Although Handheld Retroreflectometer 1 had a height tolerance deemed capable of measuring rumble stripes, the depression still might have negatively affected the readings.

Profiled Pavement Marking Retroreflectivity

While the rumble stripe showed a strong correlation between retroreflectivity readings and measurement position, the profiled pavement marking did not. Figure 16 represents a typical 2 ft segment of profiled pavement marking measured using the modified Handheld Retroreflectometer 1.

While the raised profiles of the marking seemed to influence the readings to a degree, variations in retroreflectivity values of the flat sections between the profiles varied to a similar magnitude. Overall, the retroreflectivity values of the profiled pavement markings seemed to be most influenced by variability in the pavement marking quality along the length of the marking rather than the structure of the marking itself.

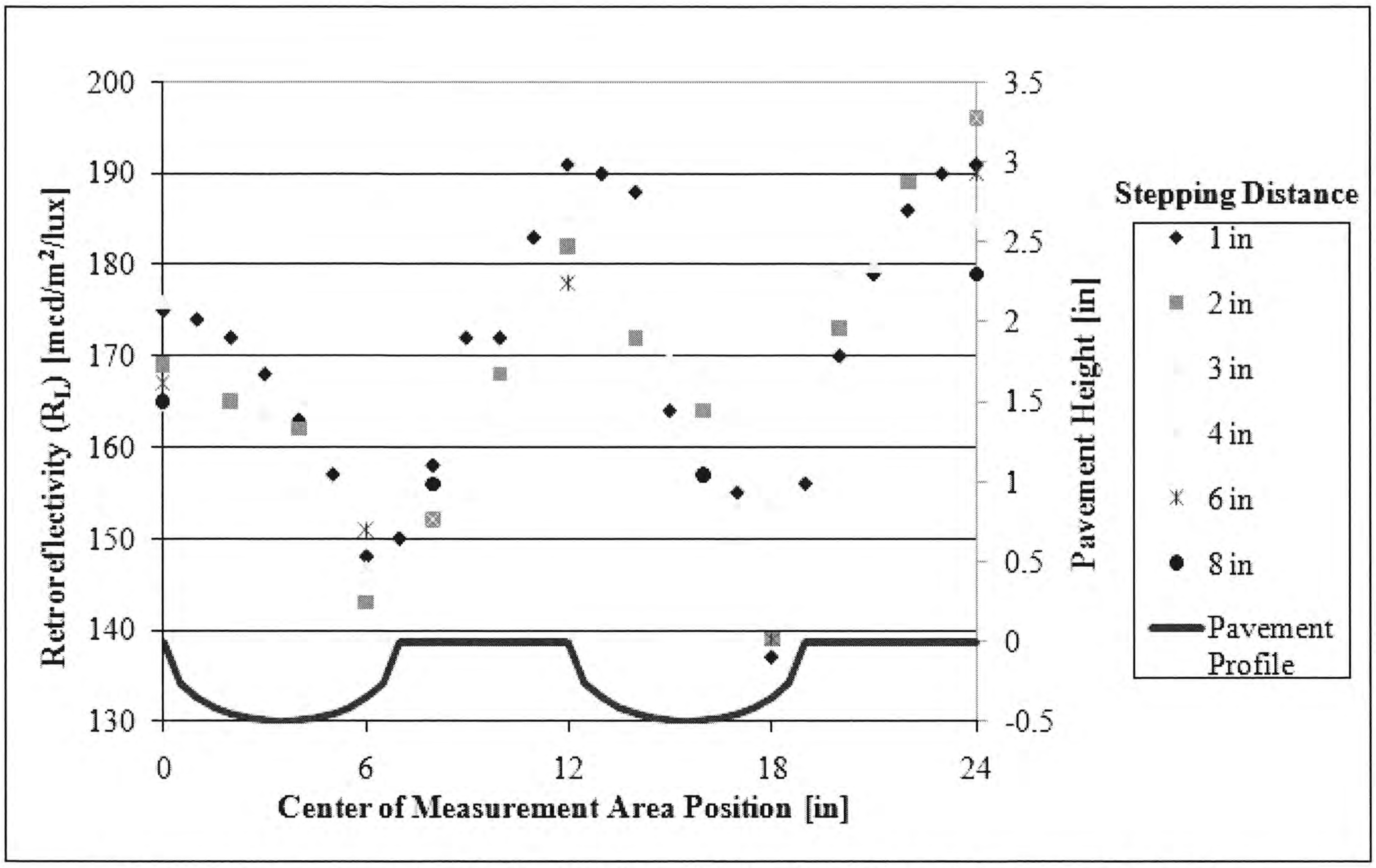

Figure 15. Retroreflectivity Profile of Rumble Stripe Pavement Marking.

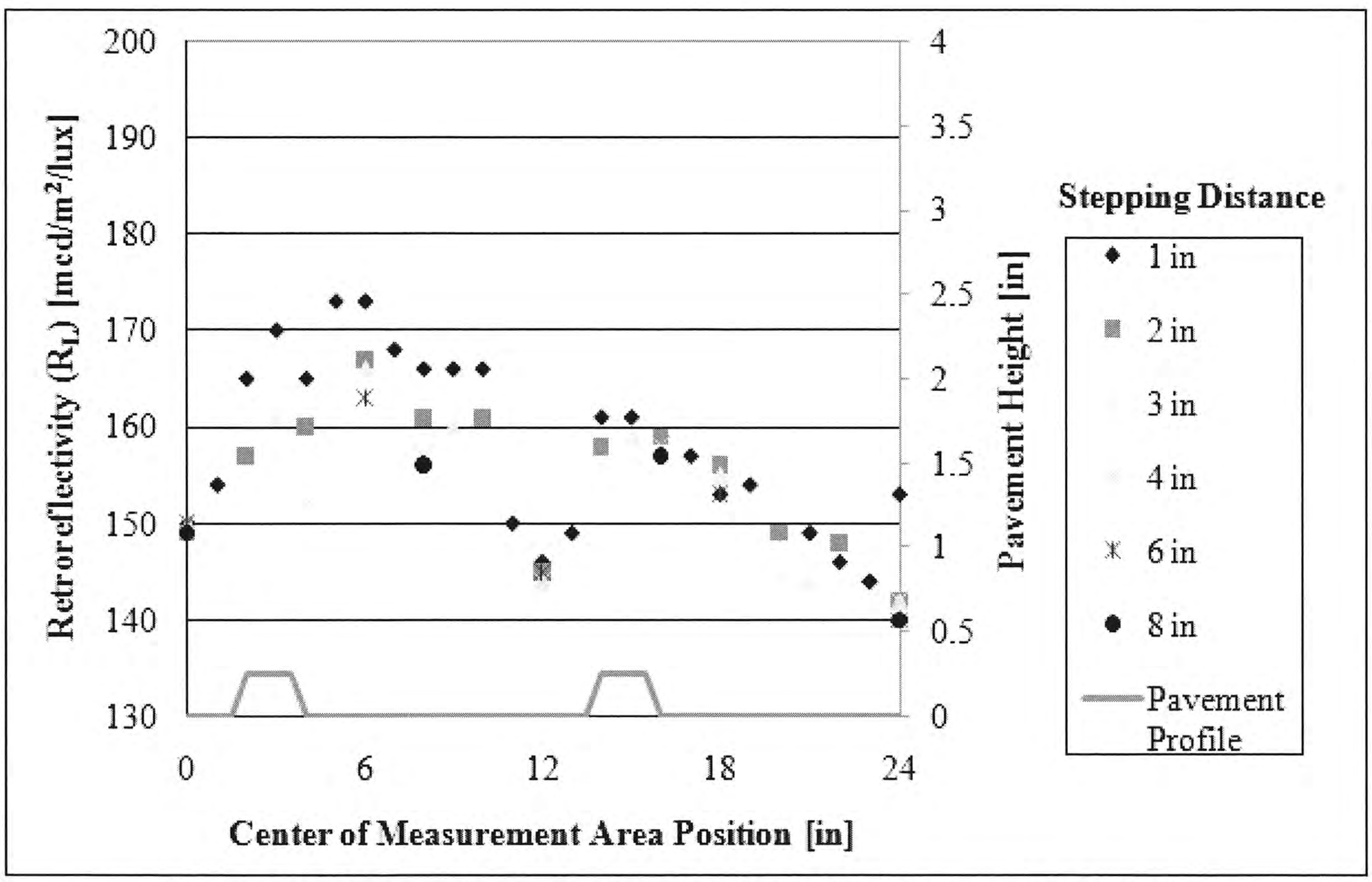

Figure 16. Retroreflectivity Profile of Profiled Pavement Marking.

Comparison of Handheld Retroreflectometers

A comparison of handheld retroreflectometers and measurement methods can be seen in Table 2. For the flat pavement marking, only modified Handheld Retroreflectometer 1, and Handheld Retroreflectometer 2 were compared. This is because the modified and standard Handheld Retroreflectometer 1 would measure the flat pavement marking in the same manner. The modified Handheld Retroreflectometer 1 readings did not differ significantly from Handheld Retroreflectometer 2 (see Appendix D).

Table 2. Comparison of Handheld Retroreflectometers and Methods.

Handheld Retroreflectometer	Method	Pavement Marking Type		
		Flat	Profiled*	Rumble Stripe
Modified Handheld Retroreflectometer 1	N/A	171	144	160
Standard Handheld Retroreflectometer 1	Free-to-Tilt	-	103	98
Standard Handheld Retroreflectometer 1	Hand-Leveled	-	149	165
Handheld Retroreflectometer 2	Hand-Leveled	169	151	152
*only segments 1 and 2		Retroreflectivity (R_L) [$mcd/m^2/lux$]		

The use of the standard Handheld Retroreflectometer 1 while leaving it free-to-tilt resulted in much lower retroreflectivity values than that of the modified Handheld Retroreflectometer 1. This was caused by many of the readings being zero due to dipping and unlevel placement on the pavement marking. The free-to-tilt measurements were found to be significantly different than all other measurement devices and methods (see Appendices E-F).

The hand-leveled, standard Handheld Retroreflectometer 1 measurements were not significantly different than the modified Handheld Retroreflectometer 1 readings for any marking type (see Appendices E-F). Even though Handheld Retroreflectometer 2 did not have a sufficient height tolerance to measure the structured pavement markings, it too was found to be not significantly different from the modified Handheld Retroreflectometer 1 measurements for any marking type (see Appendices D-F).

For each of the four handheld retroreflectometer and method combinations, the mean measurement for each stepping distance was found for each of 10 ft measurement sections. These stepping distance means were then averaged to obtain the value in Table 2.
For the profiled data in Table 2, only segments 1 and 2 were compared because Handheld Retroreflectometer 2 experienced problems during the profiled pavement marking data collection and was unable to measure the third segment.

Mobile Retroreflectometer Results

While research has previously shown that vehicle travel speed does not influence the mobile retroreflectometer measurements of flat pavement markings (*8*), retroreflectivity measurements of both the profiled and rumble stripe pavement markings seem to decrease as vehicle travel speed increases (see Table 3). This could be a result of the pavement marking structure and the nature of the scanning laser. However, the differences among speeds for each pavement marking type were not significant (see Appendices G-I).

Table 3. Mobile Retroreflectometer Results.

Speed [mph]	Pavement Marking Type		
	Flat	**Profiled**	**Rumble Stripe**
40	182	153	152
50	173	153	150
60	181	146	145
70	172	144	144
Avg	*177*	*149*	*148*
	Retroreflectivity (R_L) [mcd/m²/lux]		

CCD Photometer Results

The CCD photometer was used to obtain luminance values. Using the area and line methods, retroreflectivity values were calculated from the illuminance readings recorded for each image. The retroreflectivity values for each of the three 10 ft segments were then averaged for each photo distance. These average values can be seen in Table 4 and Table 5.

Table 4. CCD Photometer Results – Area Method.

Area Method		Pavement Marking Type		
		Flat	**Profile**	**Rumble Stripe**
Photo Distance [ft]	**80**	92	76	33
	100	92	81	41
	160	103	87	49
	Avg	*95*	*82*	*41*
		Retroreflectivity (R_L) [mcd/m²/lux]		

Table 5. CCD Photometer Results – Line Method.

Line Method		Pavement Marking Type: Flat	Pavement Marking Type: Profile	Pavement Marking Type: Rumble Stripe
Photo Distance [ft]	**80**	106	80	35
Photo Distance [ft]	**100**	103	84	43
Photo Distance [ft]	**160**	128	86	51
	Avg	*112*	*83*	*43*
		Retroreflectivity (R_L) [mcd/m^2/lux]		

Photo distance was found to have an effect on retroreflectivity calculations from the CCD photometer imagery. As photo distance increased, so increased the retroreflectivity. Except for the 80 ft and 100 ft photo distances for the flat pavement marking, all of the photo distances for each pavement marking were found to be significantly different (see Appendices J-L).

The retroreflectivity values calculated using the CCD photometer data were notably lower than the values obtained using the modified Handheld Retroreflectometer 1 and the mobile retroreflectometer. Upon noticing this trend after the flat and profiled pavement marking data collection, a calibration pavement marking panel of known retroreflectivity was placed next to the measurement area for the rumble stripe data collection. The calculated CCD photometer retroreflectivity values for the panel were also low. On average, the CCD photometer calculated retroreflectivity values were approximately 20 percent of the known retroreflectivity of the panel (see Appendix M). The CCD calculated values and the known retroreflectivity value were used to create a correction factor for the rumble stripe measurements (see Figure 17). The CCD photometer area method calculations were corrected and then compared to the average handheld and mobile retroreflectivity for the rumble stripe. This comparison is shown in Table 6.

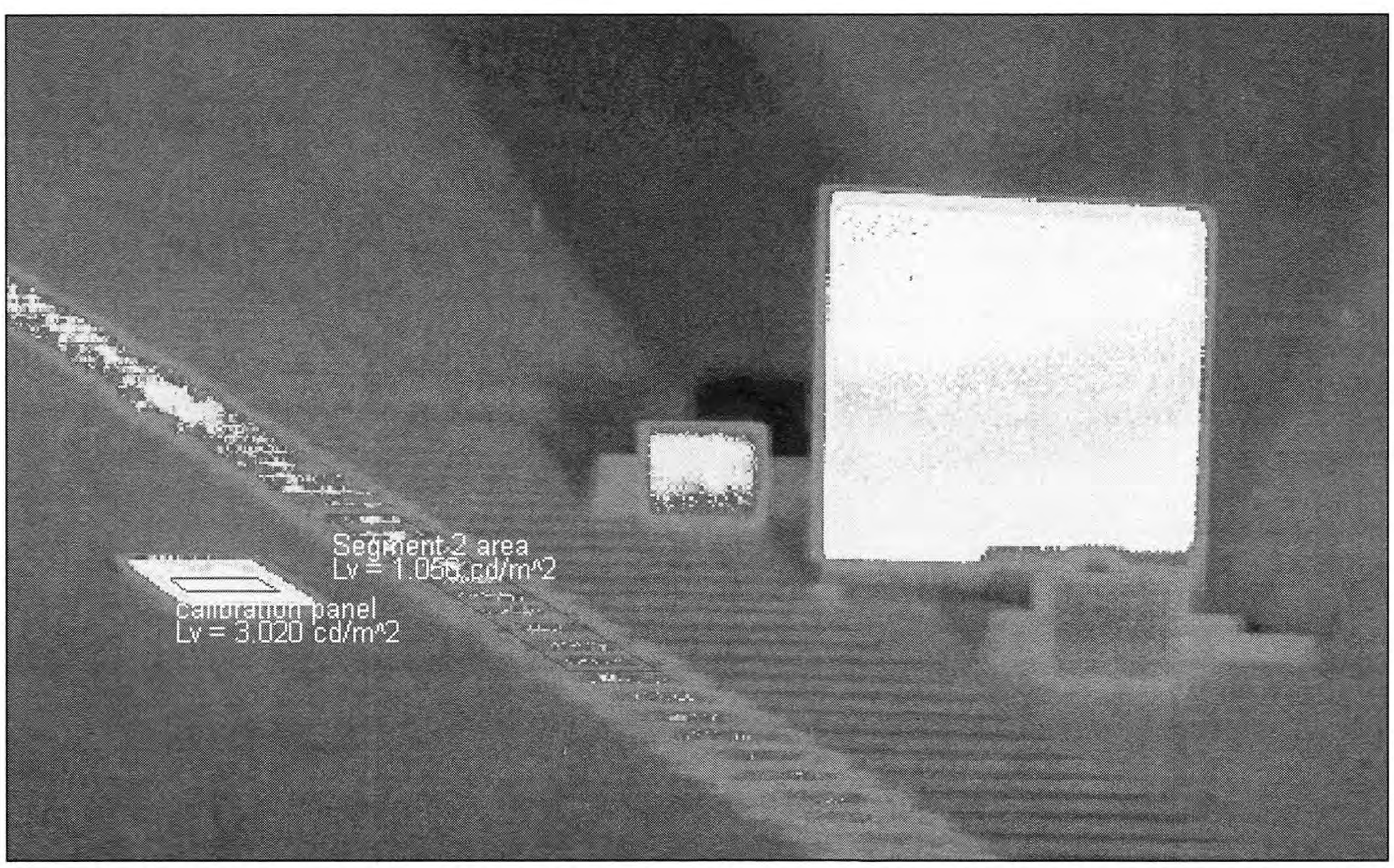

Figure 17. CCD Photometer Calibration Panel Correction Measurement (Rumble Stripe).

Table 6. CCD Photometer Retroreflectivity Correction.

Segment	Photo Distance [ft]	Rumble Stripe Retroreflectivity: CCD Area Method	Corrected	Corrected CCD Avg	Handheld	Mobile
1	80	31.1	148	165	160	148
	100	38.9	145			
	160	48.2	211			
2	80	30.8	157			
	100	42.8	180			
	160					
3	80	36.1	159			
	100	42.5	158			
	160					

There are no retroreflectivity values for the 160 ft photo distances for segments two and three because the calibration panel was placed next to the segment that had a photo distance of 80 ft in the same image.

Table 6 shows that the corrected CCD photometer average is comparable to the handheld and mobile retroreflectometer measurements. Therefore, rather than being a result of the pavement marking structure, it appears that these CCD photometer biases were more likely caused by poor calibration of the CCD photometer, luminance losses through the windshield of the vehicle, especially due to dirt and grime on the windshield, and/or the horizontally offset observation angle of the 30 m field geometry setup for the CCD photometer.

Since the calibration panel was only used for the rumble stripe, there was no way to create a correction factor for the profiled or flat pavement marking measurements. Therefore, only the uncorrected CCD photometer values were used for the comparison of methods analysis.

Comparison of Methods

Table 7 shows the total averages for each pavement marking material using each measurement method. The handheld values are same as the "Avg" values for the modified Handheld Retroreflectometer 1 in Table 1. The mobile values are the averages for each pavement marking type seen in the row "Avg" in Table 3. The CCD photometer values are averages for each pavement marking type from Table 5.

Table 7. Handheld, Mobile, and CCD Photometer Means.

Pavement Marking Type	Total Averages		
	Hand	Mobile	CCD (Line)
Flat	170	177	112
Profile	147	149	83
Rumble Stripe	161	148	43
	Retroreflectivity (R_L) [mcd/m^2/lux]		

A Least Squares Means Differences Tukey HSD test was used to compare the three measurement methods for each pavement marking type. The results of this test are shown in Table 8 and Appendix N. Combinations not connected by the same letter were found to be significantly different.

Table 8. Comparison of Pavement Marking Type and Measurement Method.

Pavement Marking Type	Measurement Method							Least Sq Mean
Flat	Mobile	A						178.58379
Flat	Handheld	A						169.98708
Rumble Stripe	Handheld		B					161.47995
Profiled	Mobile			C				149.87759
Profiled	Handheld			C				148.83734
Rumble Stripe	Mobile			C				148.30692
Flat	CCD Photometer-Line				D			108.04543
Flat	CCD Photometer-Area				D	E		95.46175
Profiled	CCD Photometer-Line					E		82.48515
Profiled	CCD Photometer-Area					E		81.50375
Rumble Stripe	CCD Photometer-Area						F	40.90055
Rumble Stripe	CCD Photometer-Line						F	40.03247

The handheld and mobile retroreflectivity data were statistically the same for both the flat and profiled pavement marking samples. However, the handheld, mobile, and CCD photometer averages were all significantly different for the rumble stripe pavement marking sample. While this difference is statistically significant, it is not practically significant.

Using the ASTM E 1710 precision and bias limits for repeatability and reproducibility to test the practicality of the findings presented in Table 7, one can conclude that there are no practical differences between results produced from handheld and mobile retroreflectometers for flat, profiled, or rumble stripe pavement markings (at least of the variety tested herein).

The two types of CCD photometer analysis (area method and line method) were statistically the same for each pavement marking type. However, the CCD photometer data were significantly different than handheld and mobile data for all pavement marking samples.

FINDINGS

The following are findings derived from the results of this research and pertain to profiled and rumble stripe pavement markings represented by those described and tested herein:

- Stepping distance has no practical influence on averaged handheld retroreflectometer readings for flat, profiled, or rumble stripe pavement markings as long as the device has a sufficient height tolerance and is properly used in accordance with ASTM E 1710–05.
    - Retroreflectivity data should be collected across the entire length of a marking segment and averaged to get an accurate retroreflectivity measurement. At a minimum for a handheld retroreflectometer with an 8 in measurement field, three longitudinally adjacent readings should be taken spanning two marking segments for profiled and rumble stripe pavement markings with 12 in spacings. Using a stepping distance shorter than the measurement field is not needed.
- Physical modification of the handheld retroreflectometer can be used as a means to maintain the instrument in a plane defined by the tops of the pavement marking profiles.
- Hand-leveling a handheld retroreflectometer by an experienced user on profiled or rumble stripe pavement markings is a suitable means to maintain the instrument in a plane defined by the tops of the pavement marking profiles.
- The vertical structure of the rumble stripe and profile pavement markings did not appear to increase the dry retroreflectivity measurements of the markings tested. If there was any influence, it seemed to lower retroreflectivity of the rumble stripe marking as a whole.
- The use of a properly calibrated mobile retroreflectometer operated by an experienced user will result in the same retroreflectivity levels as handheld retroreflectometer readings measured in accordance with ASTM E 1710–05.
- Use of the CCD photometer at a 30 m field geometry resulted in lower calculated retroreflectivity values than either the handheld or mobile retroreflectometer measurements. This could imply that the current coplanar geometry used to measure retroreflectivity of pavement markings may not be representative of what the driver actually sees.

RECOMMENDATIONS

Further research in to the following may be necessary to better understand retroreflectivity measurement characteristics:

- Would newer or older markings show similar characteristics to these markings that have been in service for 2–4 years?
- Do certain parts of the handheld retroreflectometer measurement field have greater influence on the retroreflectivity readings than others?
- Would recovery or continuous wetting measurements on profiled and rumble stripe pavement markings show similar characteristics?
- How would coplanar CCD luminance measurements compare to the field 30 m geometry CCD measurements and the other retroreflectivity measurement techniques?

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APPENDIX A: COMPARISON OF STEPPING DISTANCES FOR FLAT PAVEMENT MARKING

Oneway ANOVA Analysis of Retroreflectivity (R_L) by Stepping Distance for Flat Pavement Marking

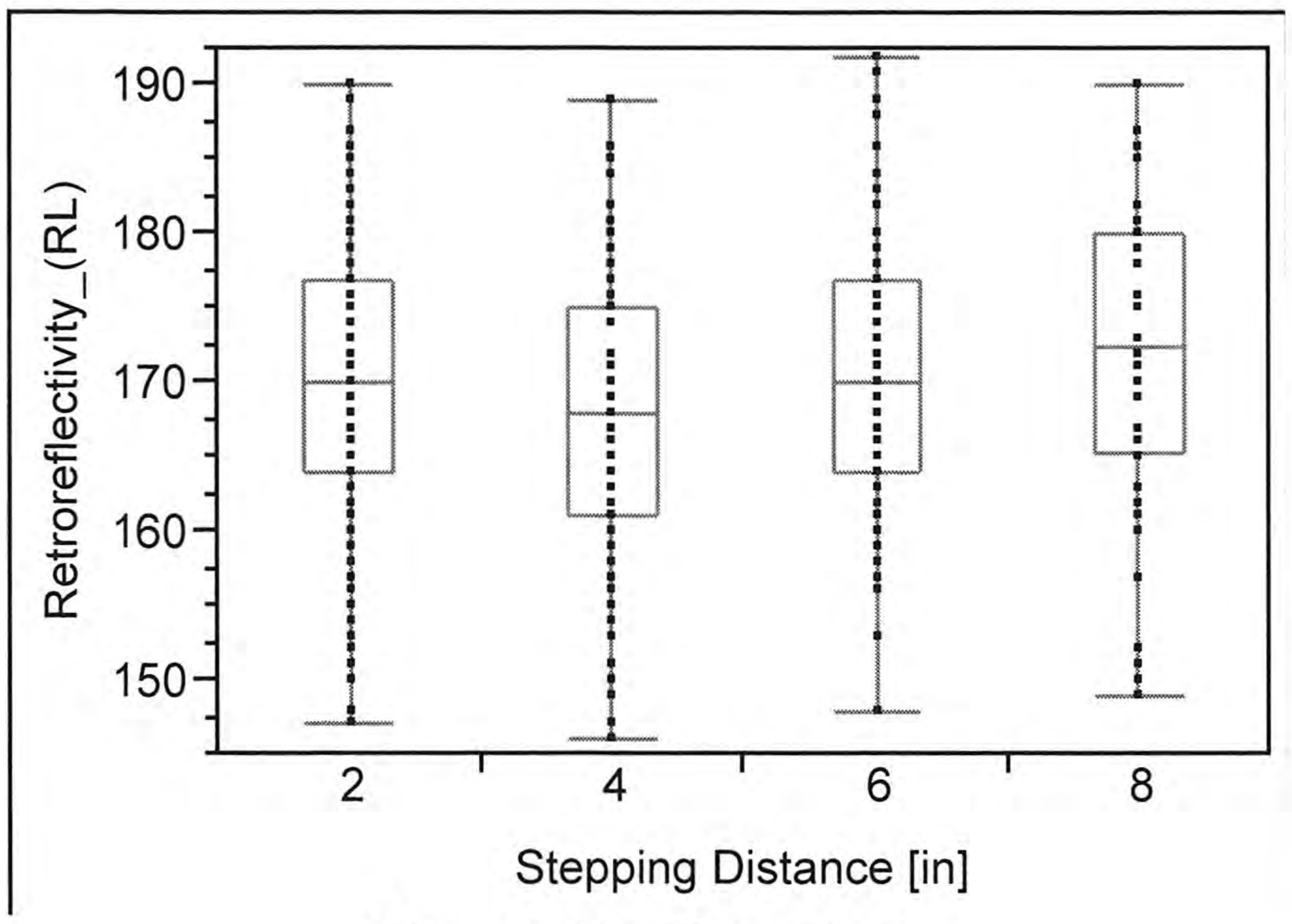

Figure 18. Box Plot.

Table 9. Summary of Fit

Rsquare	0.022825
Adj Rsquare	0.015171
Root Mean Square Error	9.703775
Mean of Response	169.9871
Observations (or Sum Wgts)	387

Table 10. Analysis of Variance.

Source	DF	Sum of Squares	Mean Square	F Ratio	Prob > F
Stepping Distance [in]	3	842.408	280.803	2.9821	0.0313*
Error	383	36064.527	94.163		
C. Total	386	36906.935			

Table 11. Means for Oneway ANOVA.

Stepping Distance [in]	n	Mean	Std Error	Lower 95%	Upper 95%
2	183	170.279	0.7173	168.87	171.69
4	93	167.581	1.0062	165.60	169.56
6	63	171.095	1.2226	168.69	173.50
8	48	172.083	1.4006	169.33	174.84

Std Error uses a pooled estimate of error variance.

Means Comparisons

Table 12. Student's t Test for All Pairs.

Stepping Distance [in]			Mean
8	A		172.08333
6	A		171.09524
2	A		170.27869
4		B	167.58065

Levels not connected by same letter are significantly different.

Table 13 Tukey-Kramer HSD Test for All Pairs.

Stepping Distance [in]			Mean
8	A		172.08333
6	A	B	171.09524
2	A	B	170.27869
4		B	167.58065

Levels not connected by same letter are significantly different.

Table 14. Wilcoxon/Kruskal-Wallis Tests (Rank Sums).

Stepping Distance [in]	N	Score Sum	Score Mean	(Mean-Mean0)/Std0
2	183	36014.0	196.798	0.466
4	93	15667.0	168.462	-2.527
6	63	12832.5	203.690	0.751
8	48	10564.5	220.094	1.727

Table 15. Oneway Test, Chi-Square Approximation.

Chi Square	DF	Prob>ChiSq
8.0560	3	0.0449*

APPENDIX B: COMPARISON OF STEPPING DISTANCES FOR PROFILED PAVEMENT MARKING

Oneway ANOVA Analysis of Retroreflectivity (R_L) by Stepping Distance for Profiled Pavement Marking

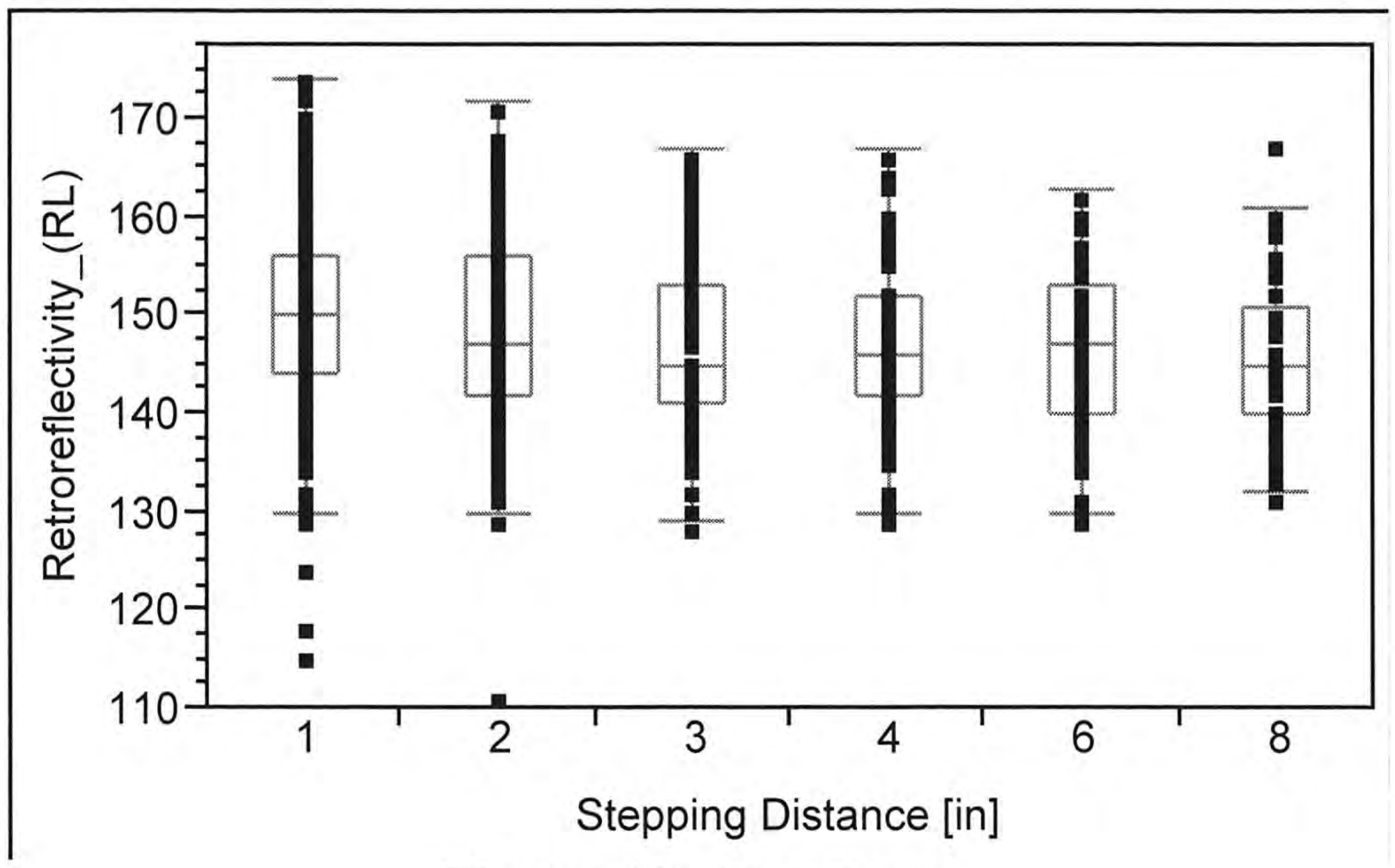

Figure 19. Box Plot.

Table 16. Summary of Fit.

Rsquare	0.029113
Adj Rsquare	0.023514
Root Mean Square Error	9.19766
Mean of Response	148.8373
Observations (or Sum Wgts)	873

Table 17. ANOVA.

Source	DF	Sum of Squares	Mean Square	F Ratio	Prob > F
Stepping Distance [in]	5	2199.341	439.868	5.1996	0.0001*
Error	867	73345.561	84.597		
C. Total	872	75544.903			

Table 18. Means for Oneway ANOVA.

Stepping Distance [in]	n	Mean	Std Error	Lower 95%	Upper 95%
1	363	150.612	0.4828	149.66	151.56
2	183	148.322	0.6799	146.99	149.66
3	123	147.683	0.8293	146.06	149.31
4	93	147.140	0.9538	145.27	149.01
6	63	146.889	1.1588	144.61	149.16
8	48	146.188	1.3276	143.58	148.79

Std Error uses a pooled estimate of error variance.

Means Comparisons

Table 19. Student's t Test for All Pairs.

Stepping Distance [in]			Mean
1	A		150.61157
2		B	148.32240
3		B	147.68293
4		B	147.13978
6		B	146.88889
8		B	146.18750

Levels not connected by same letter are significantly different.

Table 20. Tukey-Kramer HSD Test for All Pairs.

Stepping Distance [in]			Mean
1	A		150.61157
2	A	B	148.32240
3		B	147.68293
4		B	147.13978
6		B	146.88889
8		B	146.18750

Levels not connected by same letter are significantly different.

Table 21. Wilcox/Kruskal-Wallis Tests (Rank Sums).

Stepping Distance [in]	N	Score Sum	Score Mean	(Mean-Mean0)/Std0
1	363	176137	485.226	4.770
2	183	77488.0	423.432	-0.819
3	123	49034.5	398.654	-1.821
4	93	36453.5	391.973	-1.823
6	63	24893.5	395.135	-1.369
8	48	17494.5	364.469	-2.051

Table 22. Oneway Test, Chi Square Approximation.

ChiSquare	DF	Prob>ChiSq
25.3572	5	0.0001*

APPENDIX C: COMPARISON OF STEPPING DISTANCES FOR RUMBLE STRIPE PAVEMENT MARKING

Oneway ANOVA Analysis of Retroreflectivity (R_L) by Stepping Distance for Rumble Stripe Pavement Marking

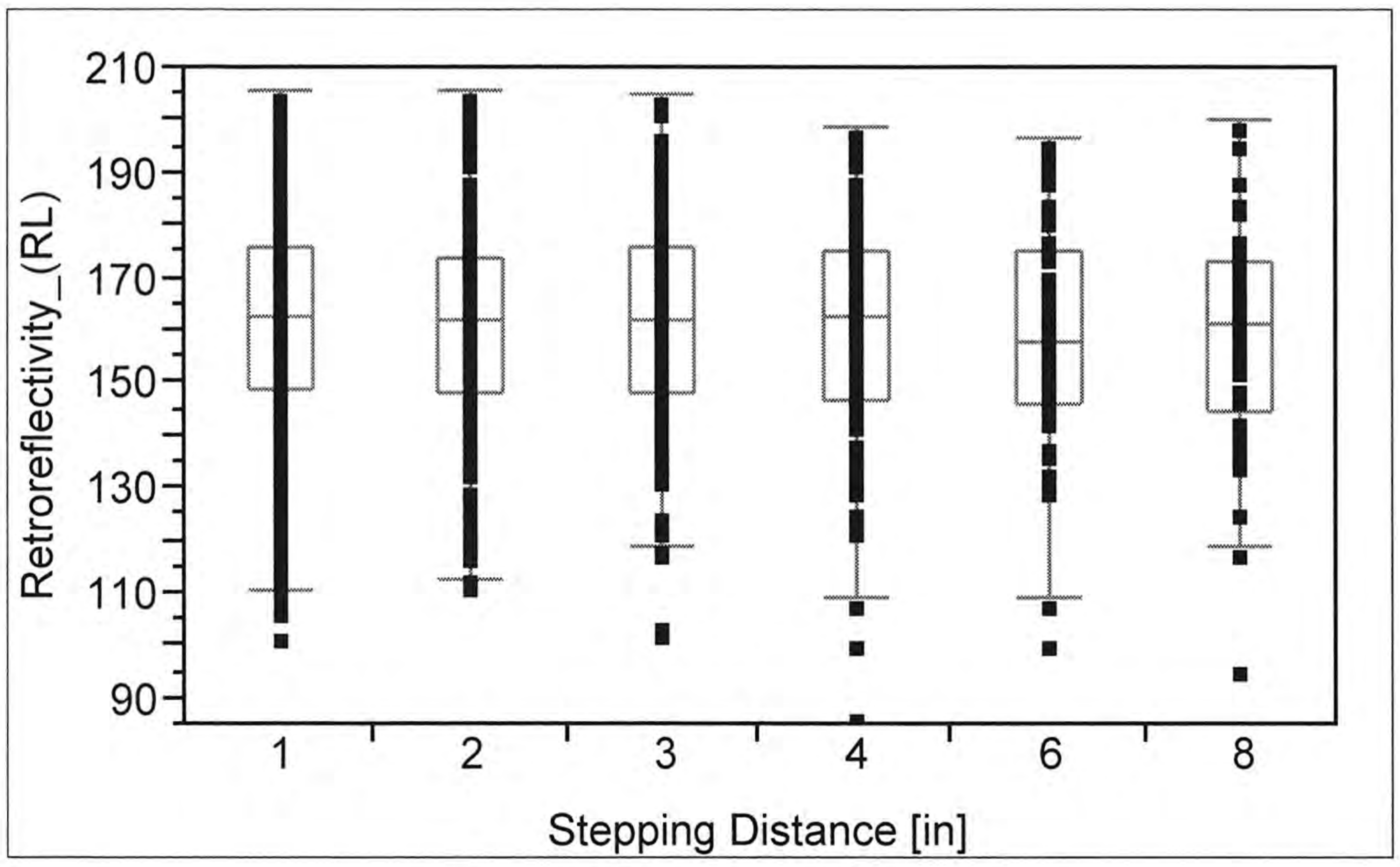

Figure 20. Box Plot.

Table 23. Summary of Fit.

Rsquare	0.002142
Adj Rsquare	-0.00361
Root Mean Square Error	20.70477
Mean of Response	161.48
Observations (or Sum Wgts)	873

Table 24. ANOVA.

Source	DF	Sum of Squares	Mean Square	F Ratio	Prob > F
Stepping Distance [in]	5	797.90	159.580	0.3723	0.8678
Error	867	371672.00	428.687		
C. Total	872	372469.90			

Table 25. Means for Oneway ANOVA.

Stepping Distance [in]	N	Mean	Std Error	Lower 95%	Upper 95%
1	363	162.444	1.0867	160.31	164.58
2	183	161.426	1.5305	158.42	164.43
3	123	161.171	1.8669	157.51	164.83
4	93	160.097	2.1470	155.88	164.31
6	63	159.952	2.6086	154.83	165.07
8	48	159.875	2.9885	154.01	165.74

Std Error uses a pooled estimate of error variance.

Table 26. Wilcoxon/Kruskal-Wallis Tests (Rank Sums).

Stepping Distance [in]	n	Score Sum	Score Mean	(Mean-Mean0)/Std0
1	363	162745	448.332	1.120
2	183	79142.5	432.473	-0.273
3	123	53482.5	434.817	-0.103
4	93	39744.0	427.355	-0.390
6	63	26127.5	414.722	-0.728
8	48	20260.0	422.083	-0.421

Table 27. Oneway Test, Chi Square Approximation.

Chi Square	DF	Prob>ChiSq
1.5976	5	0.9015

APPENDIX D: COMPARISON OF HANDHELD RETROREFLECTOMETERS FOR FLAT PAVEMENT MARKING

Oneway ANOVA Analysis of Retroreflectivity by Measurement Device for Flat Pavement Marking

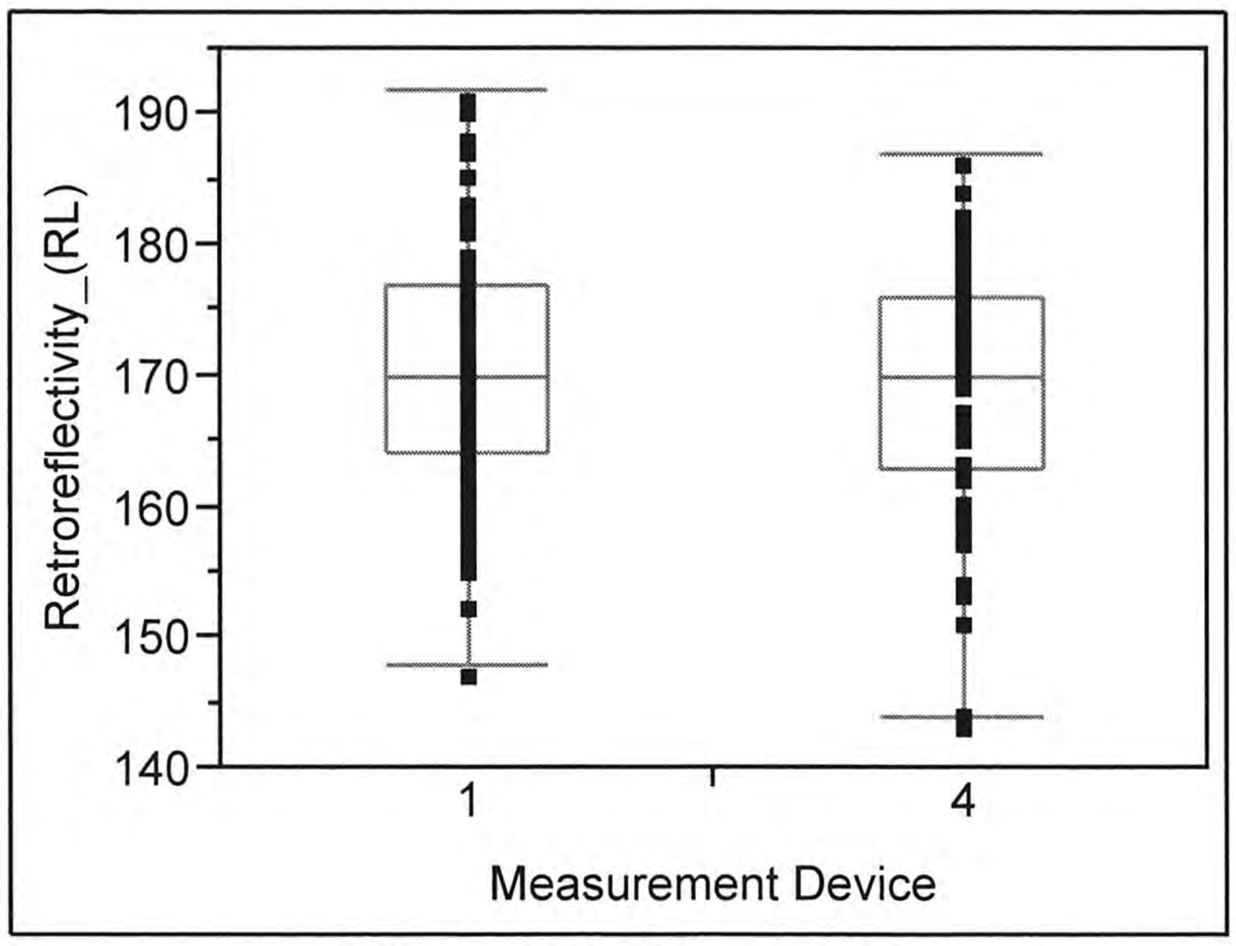

Figure 21. Box Plot.

Table 28. Summary of Fit.

Rsquare	0.009887
Adj Rsquare	0.001902
Root Mean Square Error	9.607265
Mean of Response	170.1429
Observations (or Sum Wgts)	126

Table 29. t-Test: Handheld Retroreflectometer 1 - 2.

Difference	−1.9048	**t Ratio**	−1.11275
Std Err Dif	1.7118	**DF**	124
Upper CL Dif	1.4833	**Prob > |t|**	0.2680
Lower CL Dif	−5.2928	**Prob > t**	0.8660
Confidence	0.95	**Prob < t**	0.1340

Assuming equal variances

Table 30. ANOVA.

Source	DF	Sum of Squares	Mean Square	F Ratio	Prob > F
Measurement Device	1	114.286	114.286	1.2382	0.2680
Error	124	11445.143	92.300		
C. Total	125	11559.429			

Table 31. Means for Oneway ANOVA.

Handheld Retroreflectometer	n	Mean	Std Error	Lower 95%	Upper 95%
1	63	171.095	1.2104	168.70	173.49
2	63	169.190	1.2104	166.79	171.59

Std Error uses a pooled estimate of error variance.

Means Comparisons

Table 32. Tukey-Kramer HSD Test for All Pairs.

Handheld Retroreflectometer		Mean
1	A	171.09524
2	A	169.19048

Levels not connected by same letter are significantly different.

APPENDIX E: COMPARISON OF HANDHELD RETROREFLECTOMETERS FOR PROFILED PAVEMENT MARKING

Oneway ANOVA Analysis of Retroreflectivity by Measurement Device for Profiled Pavement Marking

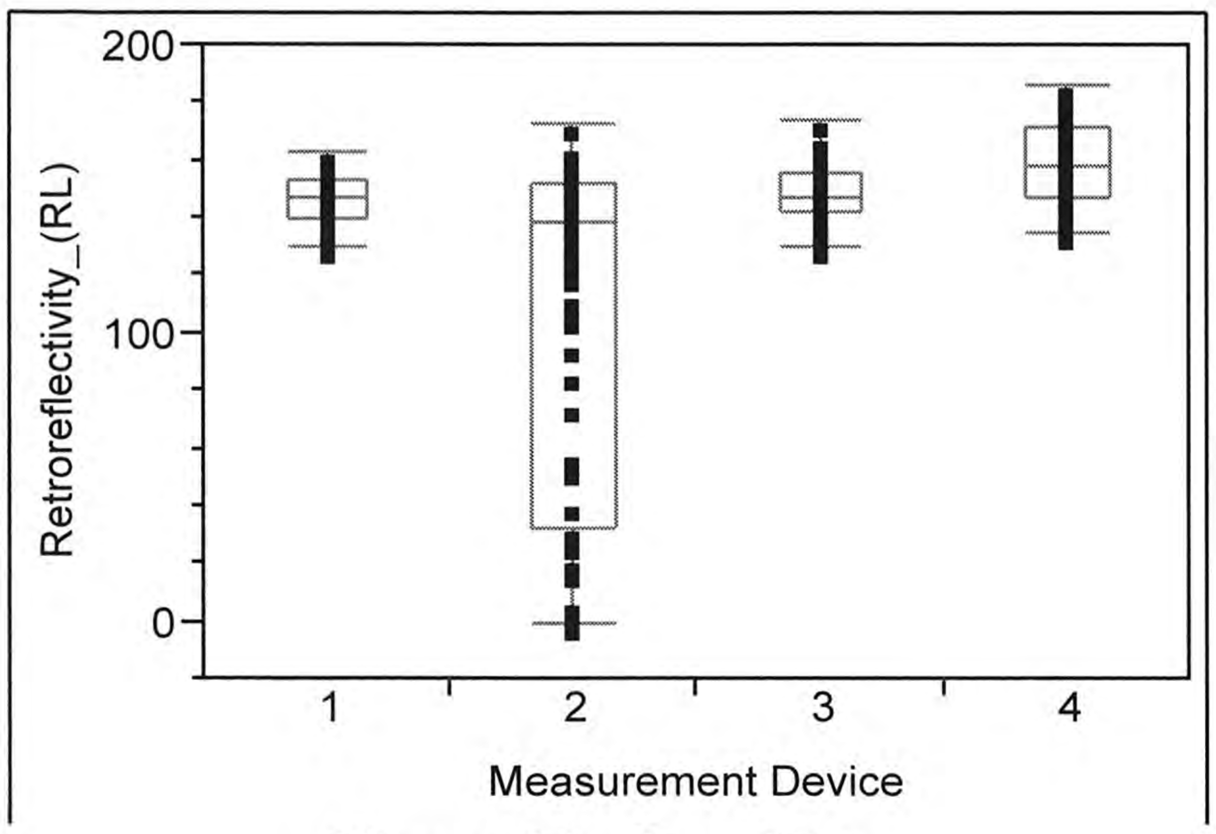

Figure 22. Box Plot.

Table 33. Summary of Fit.

Rsquare	0.314554
Adj Rsquare	0.306262
Root Mean Square Error	32.35763
Mean of Response	139.5119
Observations (or Sum Wgts)	252

Table 34. ANOVA.

Source	DF	Sum of Squares	Mean Square	F Ratio	Prob > F
Measurement Device	3	119158.90	39719.6	37.9360	<.0001*
Error	248	259660.06	1047.0		
C. Total	251	378818.96			

Table 35. Means for Oneway ANOVA.

Measurement Device	n	Mean	Std Error	Lower 95%	Upper 95%
Modified Handheld 1	63	146.889	4.0767	138.86	154.92
Standard Handheld 1 (free-to-tilt)	63	102.762	4.0767	94.73	110.79
Standard Handheld 1 (hand-leveled)	63	148.952	4.0767	140.92	156.98
Handheld 2	63	159.444	4.0767	151.42	167.47

Std Error uses a pooled estimate of error variance.

Means Comparisons

Table 36. Tukey-Kramer HSD Test for All Pairs.

Measurement Device			Mean
Handheld 2	A		159.44444
Standard Handheld 1 (hand-leveled)	A		148.95238
Modified Handheld 1	A		146.88889
Standard Handheld 1 (free-to-tilt)		B	102.76190

Levels not connected by same letter are significantly different.

APPENDIX F: COMPARISON OF HANDHELD RETROREFLECTOMETERS FOR RUMBLE STRIPE PAVEMENT MARKING

Oneway ANOVA Analysis of Retroreflectivity by Measurement Device for Rumble Stripe Pavement Marking

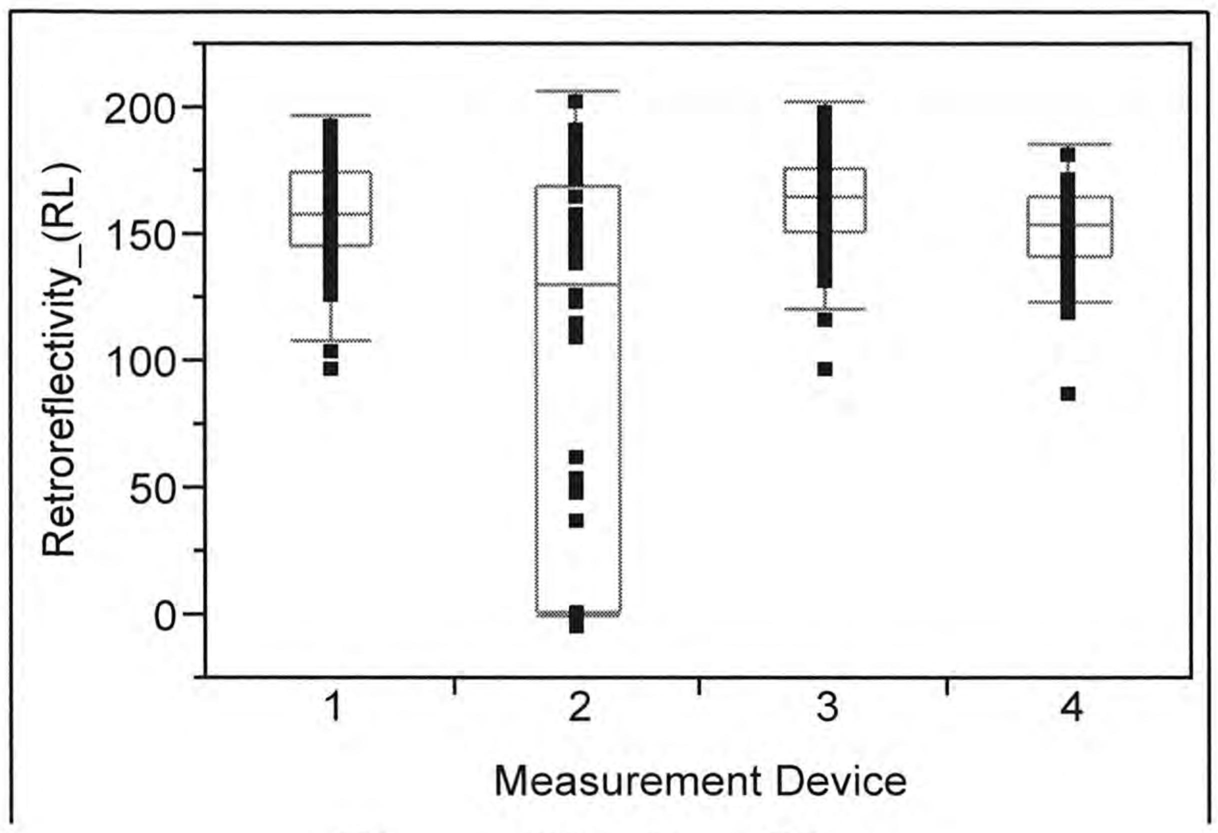

Figure 23. Box Plot.

Table 37. Summary of Fit.

Rsquare	0.277928
Adj Rsquare	0.269193
Root Mean Square Error	43.52941
Mean of Response	143.7103
Observations (or Sum Wgts)	252

Table 38. ANOVA.

Source	DF	Sum of Squares	Mean Square	F Ratio	Prob > F
Measurement Device	3	180871.00	60290.3	31.8187	<.0001*
Error	248	469912.86	1894.8		
C. Total	251	650783.85			

Table 39. Means for Oneway ANOVA.

Measurement Device	n	Mean	Std Error	Lower 95%	Upper 95%
Modified Handheld 1	63	159.952	5.4842	149.15	170.75
Standard Handheld 1 (free-to-tilt)	63	97.937	5.4842	87.13	108.74
Standard Handheld 1 (hand-leveled)	63	164.635	5.4842	153.83	175.44
Handheld 2	63	152.317	5.4842	141.52	163.12

Std Error uses a pooled estimate of error variance.

Means Comparisons

Table 40. Tukey-Kramer HSD Test for All Pairs.

Measurement Device			Mean
Standard Handheld 1 (hand-leveled)	A		164.63492
Modified Handheld 1	A		159.95238
Handheld 2	A		152.31746
Standard Handheld (free-to-tilt)		B	97.93651

Levels not connected by same letter are significantly different.

APPENDIX G: COMPARISON OF SPEED FOR FLAT PAVEMENT MARKING

Oneway ANOVA Analysis of Retroreflectivity by Speed for Flat Pavement Marking

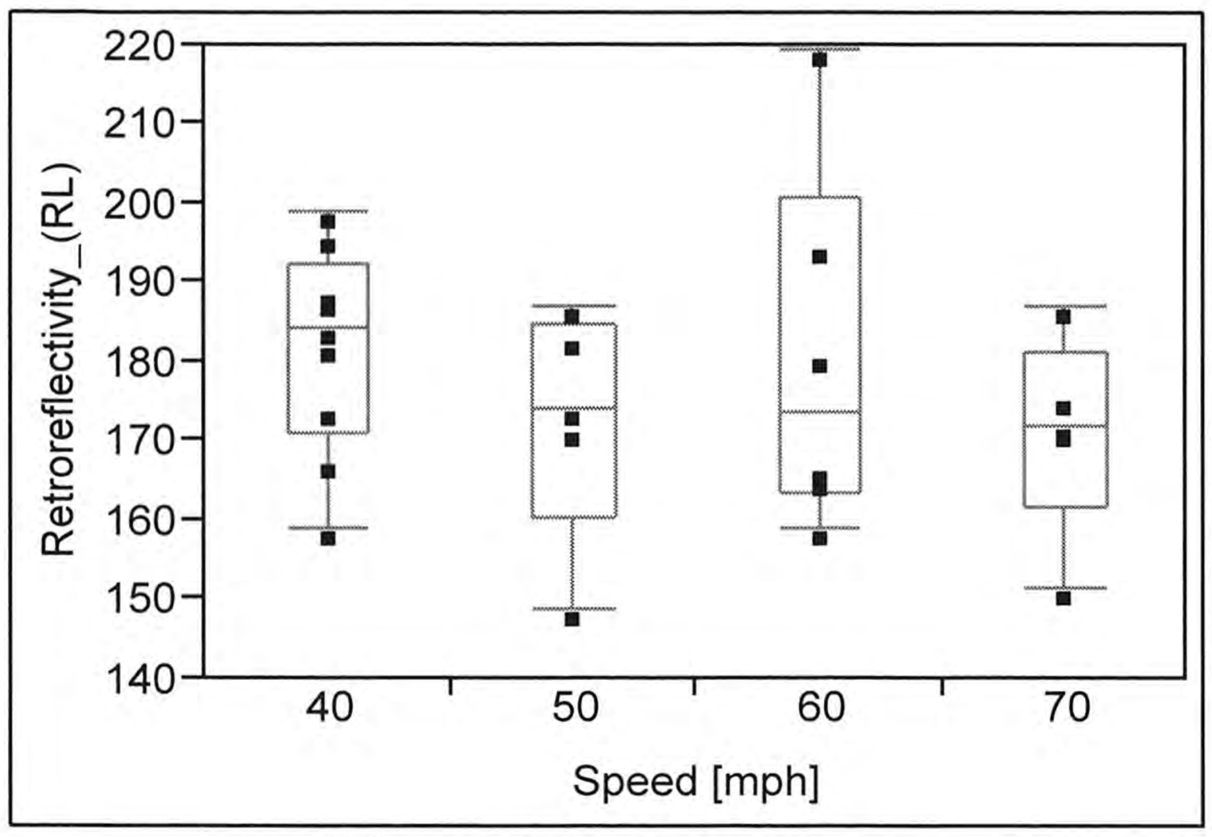

Figure 24. Box Plot.

Table 41. Summary of Fit.

Rsquare	0.087085
Adj Rsquare	-0.02246
Root Mean Square Error	16.02894
Mean of Response	178.5838
Observations (or Sum Wgts)	29

Table 42. ANOVA.

Source	**DF**	**Sum of Squares**	**Mean Square**	**F Ratio**	**Prob > F**
Speed [mph]	3	612.7215	204.240	0.7949	0.5083
Error	25	6423.1750	256.927		
C. Total	28	7035.8965			

Table 43. Means for Oneway ANOVA.

Speed [mph]	**n**	**Mean**	**Std Error**	**Lower 95%**	**Upper 95%**
40	13	182.283	4.4456	173.13	191.44
50	5	172.996	7.1684	158.23	187.76
60	6	181.032	6.5438	167.55	194.51
70	5	171.616	7.1684	156.85	186.38

Std Error uses a pooled estimate of error variance.

Means Comparisons

Table 44. Tukey-Kramer HSD Test for All Pairs.

Speed [mph]		Mean
40	A	182.28308
60	A	181.03167
50	A	172.99600
70	A	171.61600

Levels not connected by same letter are significantly different.

APPENDIX H: COMPARISON OF SPEED FOR PROFILED PAVEMENT MARKING

Oneway ANOVA Analysis of Retroreflectivity by Speed for Profiled Pavement Marking

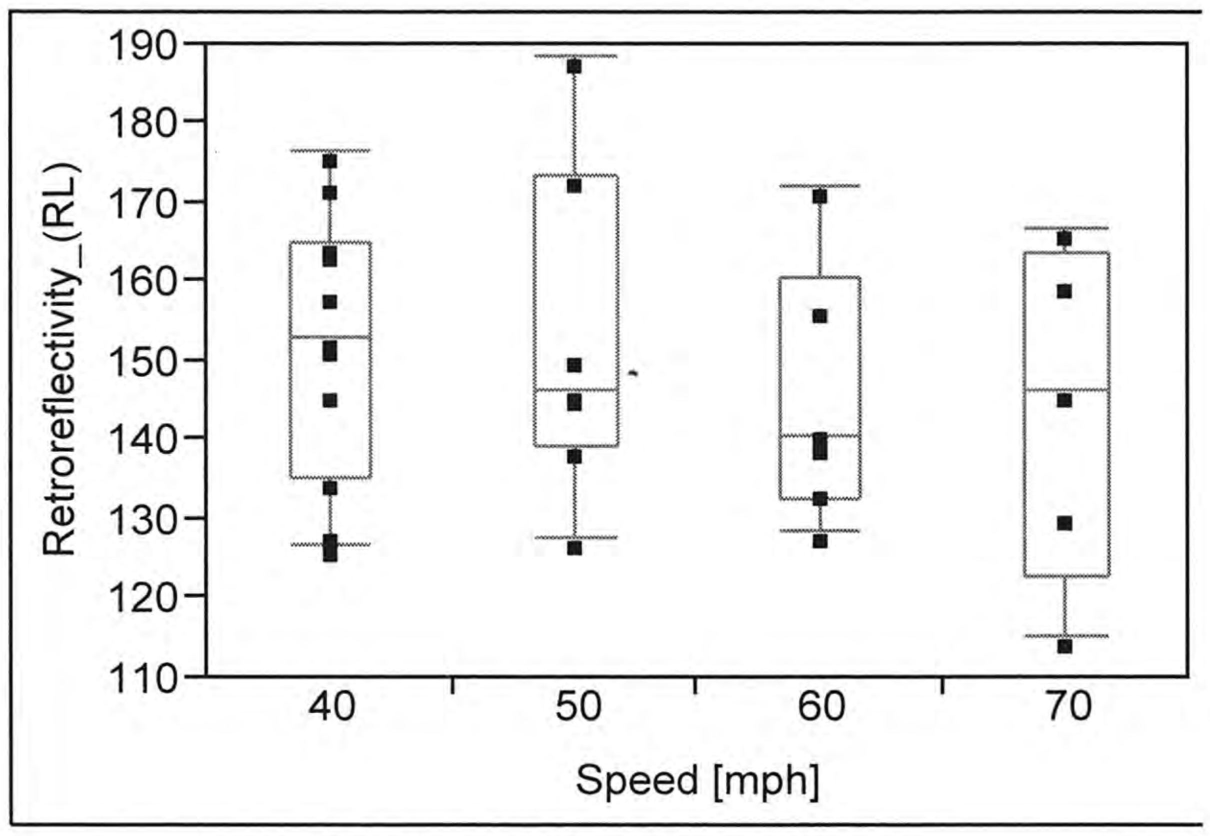

Figure 25. Box Plot.

Table 45. Summary of Fit.

Rsquare	0.050326
Adj Rsquare	−0.06363
Root Mean Square Error	18.52564
Mean of Response	149.8776
Observations (or Sum Wgts)	29

Table 46. ANOVA.

Source	DF	Sum of Squares	Mean Square	F Ratio	Prob > F
Speed [mph]	3	454.6788	151.560	0.4416	0.7253
Error	25	8579.9851	343.199		
C. Total	28	9034.6639			

Table 47. Means for Oneway ANOVA.

Speed [mph]	n	Mean	Std Error	Lower 95%	Upper 95%
40	11	152.737	5.5857	141.23	164.24
50	7	153.269	7.0020	138.85	167.69
60	6	145.603	7.5631	130.03	161.18
70	5	143.968	8.2849	126.90	161.03

Std Error uses a pooled estimate of error variance.

Means Comparisons

Table 48. Tukey-Kramer HSD Test for All Pairs.

Speed [mph]		Mean
50	A	153.26857
40	A	152.73727
60	A	145.60333
70	A	143.96800

Levels not connected by same letter are significantly different.

APPENDIX I: COMPARISON OF SPEED FOR RUMBLE STRIPE PAVEMENT MARKING

Oneway ANOVA Analysis of Retroreflectivity by Speed for Rumble Stripe Pavement Marking

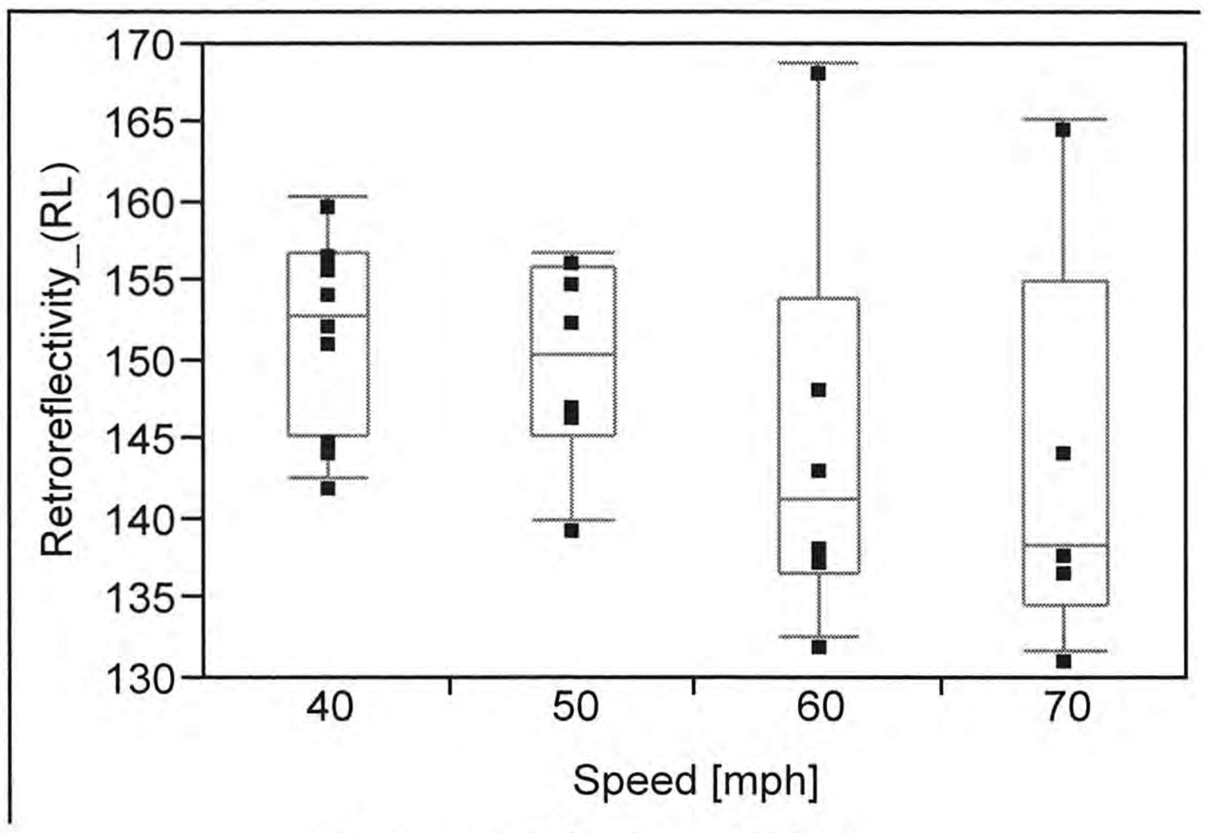

Figure 26. Box Plot.

Table 49. Summary of Fit.

Rsquare	0.130386
Adj Rsquare	0.011802
Root Mean Square Error	9.565001
Mean of Response	148.3069
Observations (or Sum Wgts)	26

Table 50. ANOVA.

Source	DF	Sum of Squares	Mean Square	F Ratio	Prob > F
Speed [mph]	3	301.7839	100.595	1.0995	0.3704
Error	22	2012.7635	91.489		
C. Total	25	2314.5474			

Table 51. Means for Oneway ANOVA.

Speed [mph]	n	Mean	Std Error	Lower 95%	Upper 95%
40	9	151.852	3.1883	145.24	158.46
50	6	150.058	3.9049	141.96	158.16
60	6	145.163	3.9049	137.07	153.26
70	5	143.596	4.2776	134.72	152.47

Std Error uses a pooled estimate of error variance.

Means Comparisons

Table 52. Tukey-Kramer HSD Test for All Pairs.

Speed [mph]		Mean
40	A	151.85222
50	A	150.05833
60	A	145.16333
70	A	143.59600

Levels not connected by same letter are significantly different.

APPENDIX J: COMPARISON OF PHOTO DISTANCE FOR CCD PHOTOMETER (LINE METHOD) ON FLAT PAVEMENT MARKING

Oneway ANOVA Analysis of Retroreflectivity by Photo Distance for Flat Pavement Marking

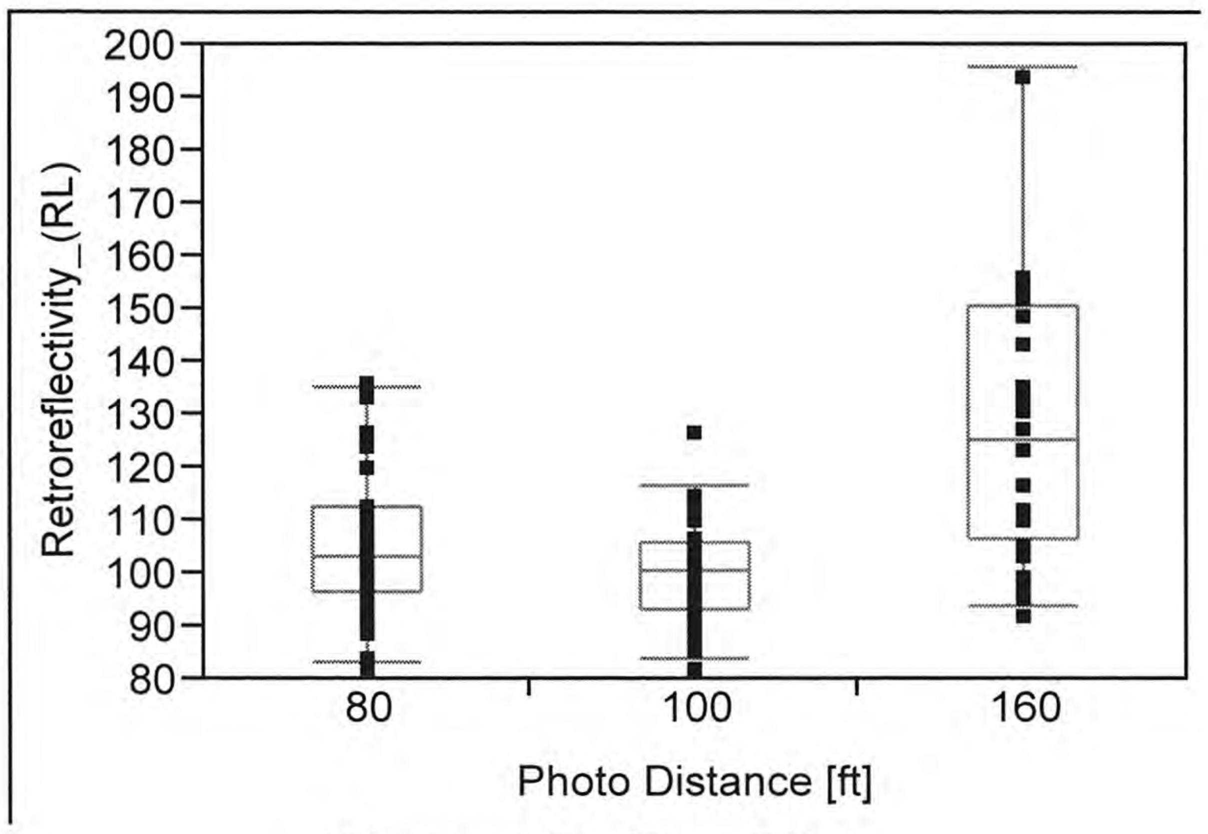

Figure 27. Box Plot.

Table 53. Summary of Fit.

Rsquare	0.296943
Adj Rsquare	0.284925
Root Mean Square Error	15.09394
Mean of Response	108.0454
Observations (or Sum Wgts)	120

Table 54. ANOVA.

Source	DF	Sum of Squares	Mean Square	F Ratio	Prob > F
Photo Distance [ft]	2	11258.348	5629.17	24.7081	<.0001*
Error	117	26655.778	227.83		
C. Total	119	37914.127			

Table 55. Means for Oneway ANOVA.

Photo Distance [ft]	n	Mean	Std Error	Lower 95%	Upper 95%
80	58	105.153	1.9819	101.23	109.08
100	39	100.844	2.4170	96.06	105.63
160	23	127.551	3.1473	121.32	133.78

Std Error uses a pooled estimate of error variance.

Means Comparisons

Table 56. Tukey-Kramer HSD Test for All Pairs.

Photo Distance			Mean
160	A		127.55077
80		B	105.15284
100		B	100.84408

Levels not connected by same letter are significantly different.

APPENDIX K: COMPARISON OF PHOTO DISTANCE FOR CCD PHOTOMETER (LINE METHOD) ON PROFILED PAVEMENT MARKING

Oneway ANOVA Analysis of Retroreflectivity by Photo Distance for Profiled Pavement Marking

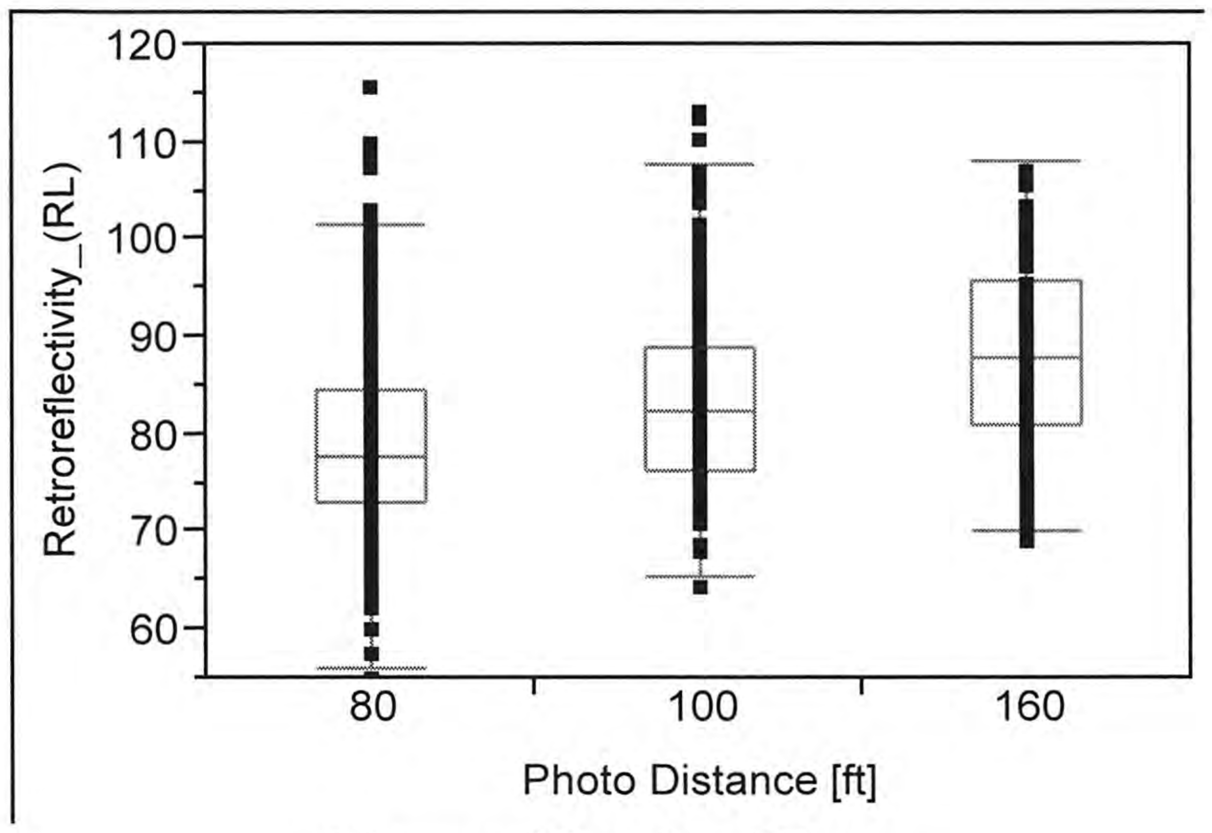

Figure 28. Box Plot.

Table 57. Summary of Fit.

Rsquare	0.083162
Adj Rsquare	0.079389
Root Mean Square Error	10.10074
Mean of Response	82.48515
Observations (or Sum Wgts)	489

Table 58. ANOVA.

Source	DF	Sum of Squares	Mean Square	F Ratio	Prob > F
Photo Distance [ft]	2	4497.525	2248.76	22.0413	<.0001*
Error	486	49584.133	102.02		
C. Total	488	54081.658			

Table 59. Means for Oneway ANOVA.

Photo Distance [ft]	n	Mean	Std Error	Lower 95%	Upper 95%
80	246	79.7019	0.6440	78.437	80.967
100	170	84.2204	0.7747	82.698	85.743
160	73	87.8231	1.1822	85.500	90.146

Std Error uses a pooled estimate of error variance.

Means Comparisons

Table 60. Tukey-Kramer HSD Test for All Pairs.

Photo Distance [ft]				Mean
160	A			87.823130
100		B		84.220410
80			C	79.701947

Levels not connected by same letter are significantly different.

APPENDIX L: COMPARISON OF PHOTO DISTANCE FOR CCD PHOTOMETER (LINE METHOD) ON RUMBLE STRIPE PAVEMENT MARKING

Oneway ANOVA Analysis of Retroreflectivity by Photo Distance for Rumble Stripe Pavement Marking

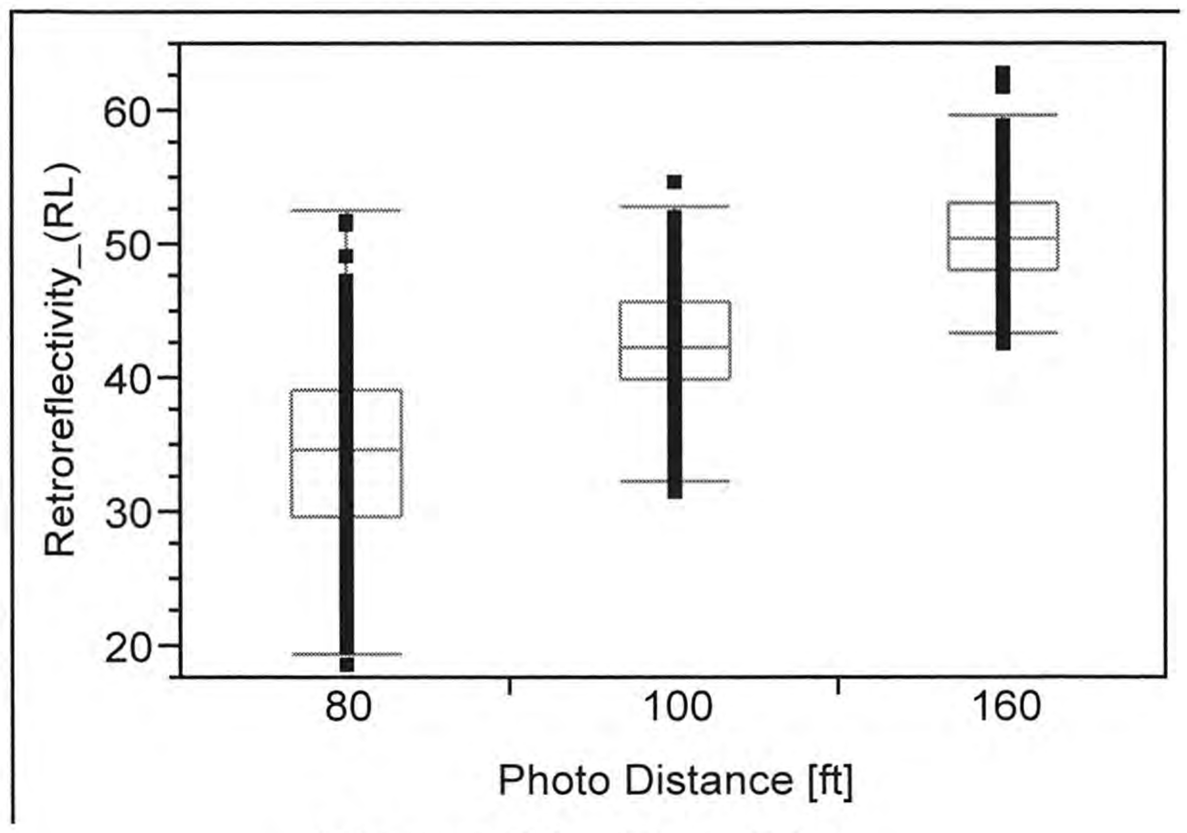

Figure 29. Box Plot.

Table 61. Summary of Fit.

Rsquare	0.529017
Adj Rsquare	0.527335
Root Mean Square Error	5.678555
Mean of Response	40.03247
Observations (or Sum Wgts)	563

Table 62. ANOVA.

Source	DF	Sum of Squares	Mean Square	F Ratio	Prob > F
Photo Distance [ft]	2	20282.778	10141.4	314.5008	<.0001*
Error	560	18057.752	32.2		
C. Total	562	38340.529			

Table 63. ANOVA.

Photo Distance [ft]	n	Mean	Std Error	Lower 95%	Upper 95%
80	279	34.6204	0.33997	33.953	35.288
100	196	42.8087	0.40561	42.012	43.605
160	88	51.0077	0.60534	49.819	52.197

Std Error uses a pooled estimate of error variance.

Means Comparisons

Table 64. Tukey-Kramer HSD Test for All Pairs.

Photo Distance [ft]				Mean
160	A			51.007703
100		B		42.808700
80			C	34.620412

Levels not connected by same letter are significantly different.

APPENDIX M: CCD PHOTOMETER CALIBRATION PANEL CORRECTION

Table 65. CCD Photometer Calibration Panel Correction.

	Calibration Panel					
Segment	Photo Distance [ft]	Illuminance [lux]	Luminance [mcd/m2]	Retro-reflectivity [mcd/m^2/lux]	% of Known Retro-reflectivity	Correction Factor
1	80	26.30	2.431	92	21.0	4.7602
	100	20.62	2.429	118	26.8	3.7352
	160	15.99	1.606	100	22.8	4.3808
2	80	34.90	3.02	87	19.7	5.0848
	100	29.90	3.137	105	23.8	4.1938
	160	15.32				
3	80	28.90	2.898	100	22.8	4.3879
	100	25.76	3.046	118	26.9	3.7211
	160	14.83				

APPENDIX N: HANDHELD, MOBILE, AND CCD PHOTOMETER COMPARISON

ANOVA for Measurement Method

Table 66. Summary of Fit.

RSquare	0.922711
RSquare Adj	0.922459
Root Mean Square Error	13.68267
Mean of Response	125.5344
Observations (or Sum Wgts)	3398

Table 67. ANOVA.

Source	DF	Sum of Squares	Mean Square	F Ratio
Model	11	7567872.1	687988	3674.850
Error	3386	633911.2	187	**Prob > F**
C. Total	3397	8201783.3		0.0000*

Table 68. Effect Tests.

Source	Nparm	DF	Sum of Squares	F Ratio	Prob > F
Measurement Method	3	3	3741314.4	6661.338	0.0000*
Pavement Marking Type	2	2	34678.4	92.6165	<.0001*
Pavement Marking Type*Measurement Method	6	6	588208.8	523.6473	0.0000*

Effect Details

Table 69. Least Squares Means Table: Measurement Method

Measurement Method	Least Sq Mean	Std Error	Mean
Handheld	160.10146	0.3184444	157.849
Mobile	158.92277	1.4948799	159.302
CCD Photometer - Area	72.62202	4.5608887	72.622
CCD Photometer - Line	76.85435	0.5028268	64.709

Table 70. Least Squares Means Table: Pavement Marking Type.

Pavement Marking Type	Least Sq Mean	Std Error	Mean
Flat	138.01951	2.1051233	156.244
Profiled	115.67596	2.0835385	125.438
Rumble Stripe	97.67997	2.0939301	114.327

Means Comparison

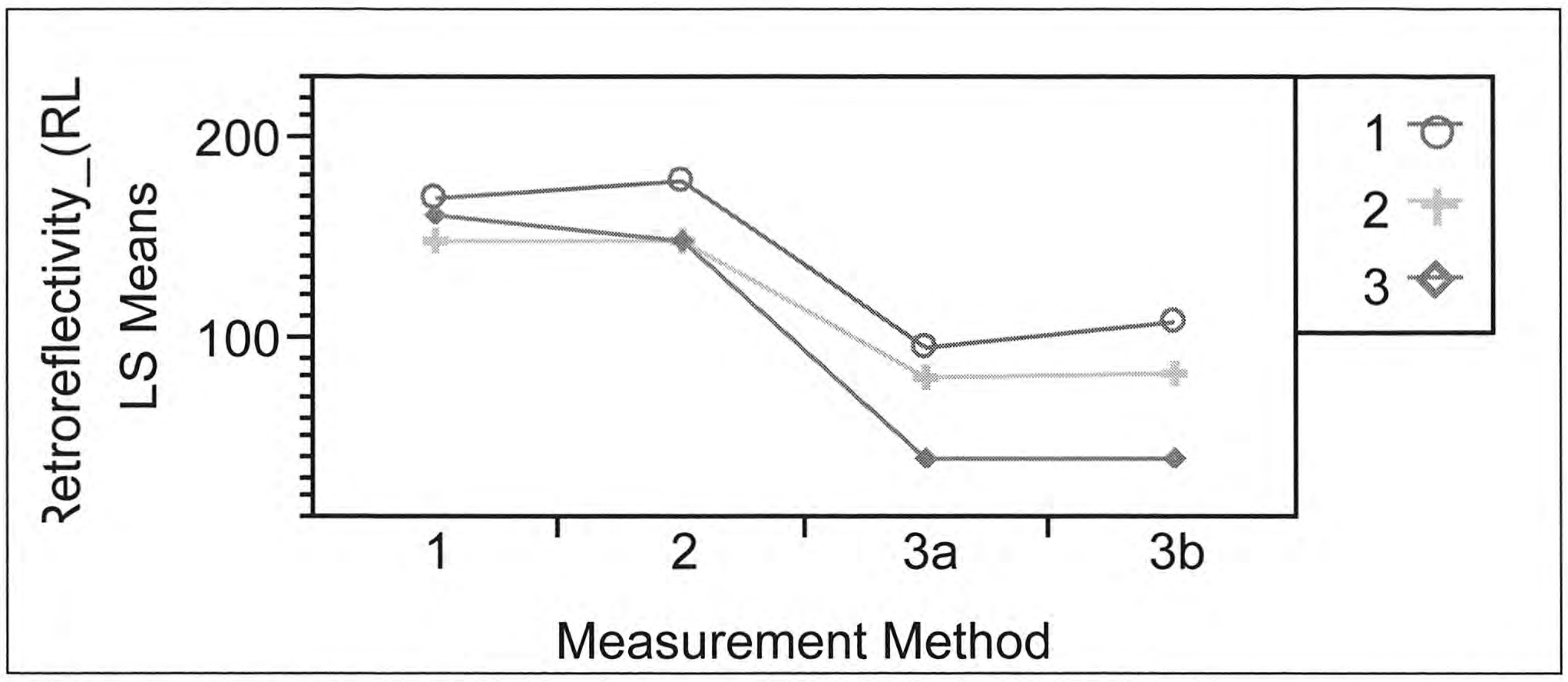

Figure 30. LS Means Plot: Pavement Marking Type by Measurement Method.

Table 71. LS Means Differences Tukey HSD Test.

Pavement Marking Type	Measurement Method							Least Sq Mean
Flat	Mobile	A						178.58379
Flat	Handheld	A						169.98708
Rumble Stripe	Handheld		B					161.47995
Profiled	Mobile			C				149.87759
Profiled	Handheld			C				148.83734
Rumble Stripe	Mobile			C				148.30692
Flat	CCD Photometer-Line				D			108.04543
Flat	CCD Photometer-Area				D	E		95.46175
Profiled	CCD Photometer-Line					E		82.48515
Profiled	CCD Photometer-Area					E		81.50375
Rumble Stripe	CCD Photometer-Area						F	40.90055
Rumble Stripe	CCD Photometer-Line						F	40.03247

Levels not connected by same letter are significantly different.

Analysis of Retroreflectivity and Color Degradation in Sign Sheeting

Prepared for
Undergraduate Transportation Scholars Program

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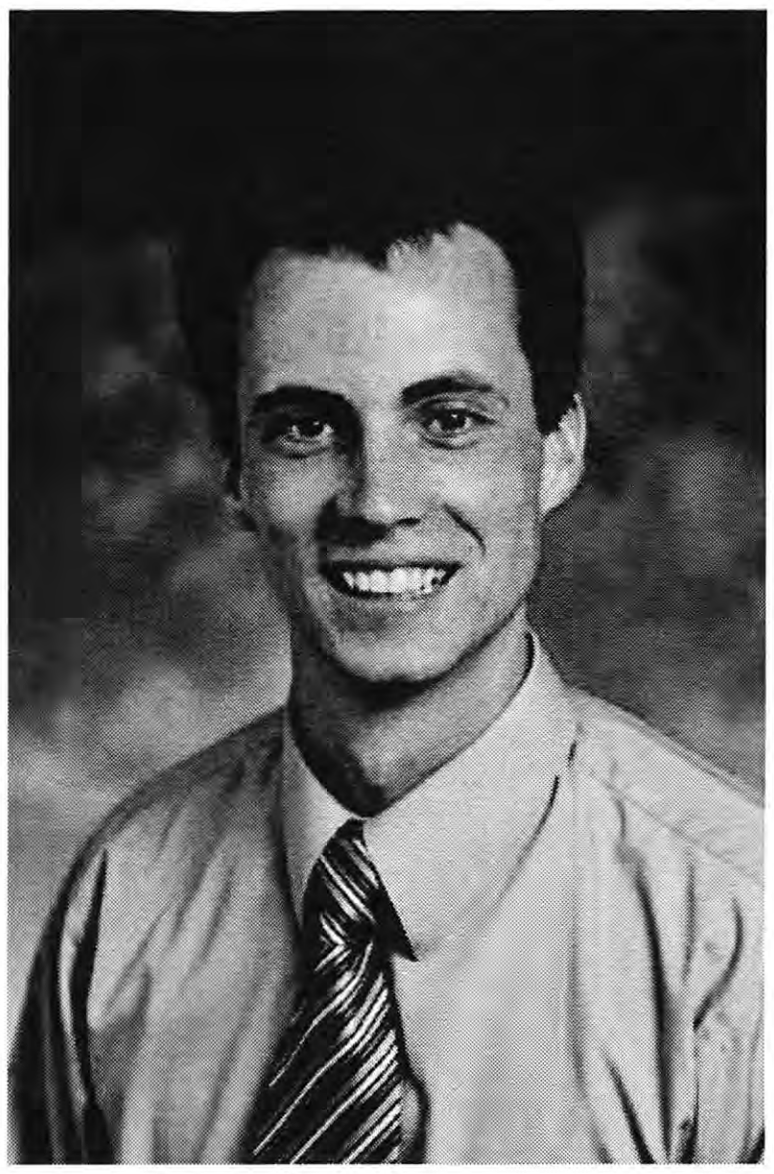

STUDENT BIOGRAPHY

Bradford Brimley is a senior at Brigham Young University in Provo, Utah, and will graduate in April 2011 with both a Bachelor of Science and a Master of Science degree in Civil Engineering. The research for his thesis focuses on the development of calibrated safety performance functions for rural two-way highways in Utah. He is a Heritage Scholarship recipient at BYU.

Brad is an officer in the Institute of Transportation Engineers student chapter at BYU. He has been involved in the University's marching and pep bands for four years and served as the trombone section leader for the past two years. He enjoys hiking, camping, fishing, and participating in the social dance club on campus. He hopes to pursue further graduate studies in transportation engineering.

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SUMMARY

The 2009 *Manual on Uniform Traffic Control Devices* (MUTCD) requires agencies to maintain traffic signs above a specified minimum level of retroreflectivity. Because of degradation from weathering, signs must be replaced that do not meet these minimums. Agencies responsible for the maintenance and replacement of traffic signs may benefit from an evaluation of the performance of retroreflective sign sheeting through long-term exposure. This study evaluated the performance of sign sheeting against retroreflectivity and color criteria for a period of over 20 simulated years.

Nine different sign sheeting products were weathered—one color sample of each product. The samples were placed on weathering racks facing south at a 45 degree elevation angle. This orientation provides an increase in the degradation of sign sheeting that research has shown to be approximately twofold. Researchers used the MUTCD's minimum levels of retroreflectivity for

in-service signs and the color specifications for new signs to evaluate how degradation affects the service life of sign sheeting

Most sign sheeting met the retroreflectivity requirements for at least 15 simulated years. However, most orange and red sheeting, and some yellow sheeting, did not meet the color specifications for as long as the retroreflectivity requirements. Some products fell outside the color specifications in less than five simulated years. Green and white sheeting never degraded outside of the color specifications and mostly maintained acceptable levels of retroreflectivity throughout the study. Based on the findings, end of service life for sign sheeting due to degradation is more often the result of changes in color than failure in retroreflectivity.

TABLE OF CONTENTS

Student Biography 82
Acknowledgment 82
Summary 82
Introduction 85
Background 85
  Observation and Entrance Angles 86
  MUTCD Retroreflectivity Requirements 86
  FHWA Color Specifications 87
  Accelerated Outdoor Weathering Research 88
  Samples 89
Methodology 89
  Placement 89
  Measurements 89
  Failure Criteria 90
Results 90
  White Sheeting 92
  Green Sheeting 94
  Orange Sheeting 95
  Red Sheeting 96
  Yellow Sheeting 97
Conclusions 98
Limitations 98
References 99
Appendix 100

INTRODUCTION

In 2007, the Federal Highway Administration (FHWA) revised the *Manual on Uniform Traffic Control Devices* (MUTCD) to include requirements for maintaining retroreflectivity of traffic signs. With these new in-service sign retroreflectivity requirements, agencies responsible for maintaining signs need to have a better understanding of the service life of the signs they are responsible for so that the signs can be replaced before they degrade below the minimum levels. With a variety of options for selecting new sign materials, agencies should understand how they perform under long-term exposure. Sign sheeting degrades through weathering stresses—a result of ultraviolet light, heat, water, and other climatic elements—which change the retroreflective and color properties of each product at a unique rate (*1*). The public domain contains only limited and somewhat dated information concerning these changes and their effect on the usability of sign sheeting. Although most manufacturers guarantee their products will last for a specific length of time, they do not publish information about the tested service life based on MUTCD minimum retroreflectivity criteria.

The purpose of this research was to assess the service life of sign sheeting that has been exposed for over 10 years and provide insight into the degradation of retroreflectivity and color. The weathering protocol for the experiment was assumed to have provided a 2:1 accelerated degradation rate (due to a 45 degree, south-facing exposure), resulting in a simulated exposure time of over 20 years of service. Researchers evaluated the durability of the materials by comparing their retroreflectivity to the MUTCD minimum in-service retroreflectivity levels and color performance to FHWA color specifications. The intent was to gain a sense of the long-term performance of sign sheeting with respect to minimum retroreflectivity and color.

BACKGROUND

Retroreflectivity—the reflecting of light back to its source—and color are two properties necessary for traffic signs to function effectively. Drivers rely on a sign's color to identify and respond to its message. They also depend on retroreflective sheeting to be able to see signs at night when the only light source is the vehicle's headlights.

Sheeting construction consists of three types: glass beads, microprisms, and full-cube corner (full-cube corner sheeting was not available at the time this weathering effort began and thus is not included in this study). Prismatic sheeting is more efficient at retroreflecting light than beaded sheeting, though it is also more expensive. The coefficient of retroreflectivity (R_A) is used to describe the performance of a retroreflective material. R_A is the ratio of a sign's luminance (the light reflected back to the driver) to its illuminance (the light reaching the sign from the headlights) expressed in SI units of candelas per lux per square meter ($cd/lx/m^2$). R_A is dependent upon four angles—entrance, observation, rotation, and orientation—which describe the position of a light source and receiver to a retroreflective object. An ideal sign placement (perfectly straight and facing the direction of travel) makes orientation and rotation angles unnecessary when comparing retroreflectivity. The handheld retroreflectometer used measures signs with orientation and rotation angles of 0 degrees.

Observation and Entrance Angles

The observation angle, α, is the angle between the line formed by the beam of light from the source (or headlights) to the sign and the line formed by the light from the sign to the observer (or driver). The entrance angle, β, is the angle between the line from the source to the sign and a line perpendicular to the sign. Figures 1 and 2 illustrate the observation and entrance angles. When measuring the retroreflectivity of sign sheeting, the observation and entrance angles must be noted to make correct comparisons among materials.

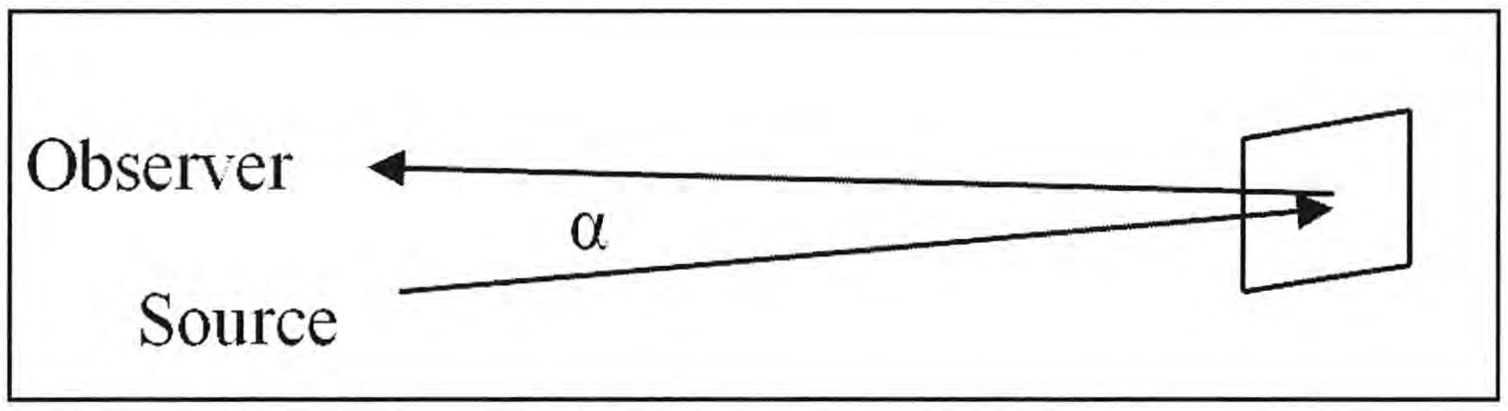

Figure 1. Observation Angle.

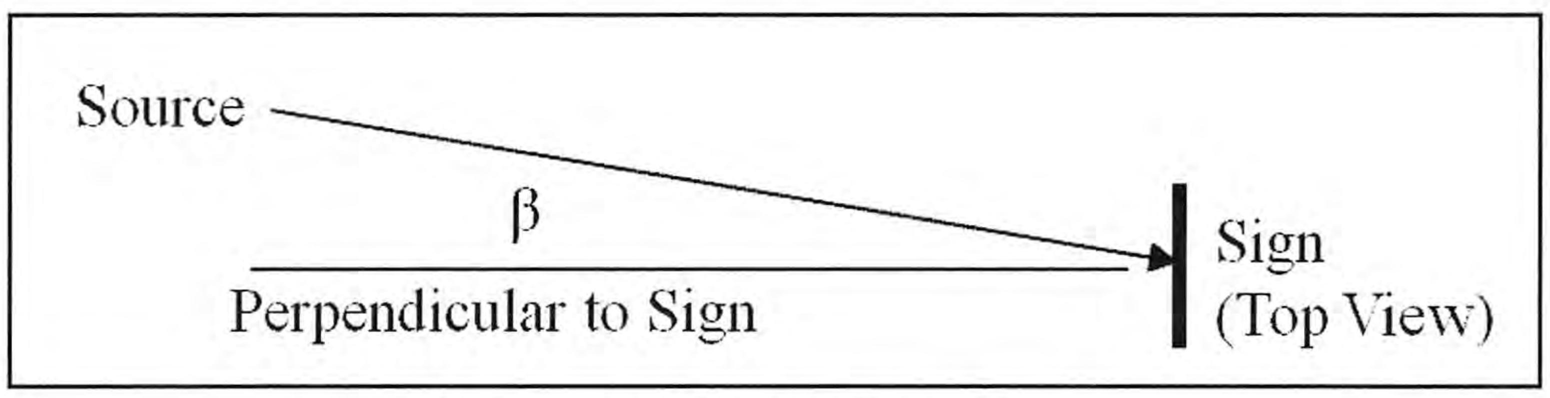

Figure 2. Entrance Angle.

Retroreflective sheeting is often classified by its performance at four measurement geometries: observation entrance angles of 0.2 and 0.5 degrees and entrance angles of −4 and +30 degrees. ASTM International classifies nine types of retroreflective sheeting (two of the nine are not typically used for permanent rigid signs) based on the retroreflective performance and durability of the material (ASTM D4956-09) at these geometries and some additional geometries for certain sheeting types. The minimum retroreflectivity values in the MUTCD are based only on a single-measurement geometry: observation angle of 0.2 degrees and entrance angle of −4 degrees.

MUTCD Retroreflectivity Requirements

In 2003, the FHWA published research recommending updated minimum retroreflectivity requirements. These recommended minimums were developed to update previous research recommendations based on changes that had occurred, such as different headlight patterns and intensity, vehicle size, driver height, and new MUTCD legibility requirements (*2*). The new retroreflectivity requirements from Table 2A-3 of the MUTCD are reproduced in Table 1. As indicated in this table, Type I sheeting should not be used for yellow or orange signs. Therefore, the results for these colors of Type I sheeting are not reported in this paper.

Table 1. Minimum Maintained Retroreflectivity Levels (cd/lx/m²).

Sign Color	Sheeting Type (ASTM D4956)				Additional Criteria
	Beaded Sheeting			Prismatic Sheeting	
	I	II	III	III, IV, VI, VII, VIII, IX, X	
White on Green	W*; G≥7	W*; G≥15	W*; G≥25	W≥250; G≥25	Overhead
	W*; G≥7	W≥120; G≥15			Ground-mounted
Black on Yellow or Black on Orange	Y*; O*	Y≥50; O≥50			Size ≥ 48 in
	Y*; O*	Y≥75; O≥75			Size < 48 in
White on Red	W≥35; R≥7				Contrast ≥ 3:1
Black on White	W≥50				—

* Indicates a sheeting type that should not be used for this color and application
Source: 2009 MUTCD Table 2A-3 (*3*)

Agencies are responsible for maintaining signs above the minimum standards given in the MUTCD (*3*) and can use a variety of maintenance methods to do so. The MUTCD guidelines allow most types of retroreflective sheeting to be used. For example, any type of sheeting may be used on Speed Limit signs, as long as the R_A for the white color is maintained at a retroreflectivity level of at least 50 cd/lx/m^2 throughout its service life. Using a less expensive sign sheeting may provide opportunity for agencies to minimize the initial expenditure on sign sheeting, but the less expensive sign sheeting materials are often perceived to have shorter service lives and may have a greater life cycle cost. On the other hand, costs may be minimized if a more expensive sheeting material meets the MUTCD criteria longer than one that is less expensive.

FHWA Color Specifications

The MUTCD refers to FHWA 23 CFR Part 655, which defines appropriate color tones using the International Commission on Illumination's (CIE) 1931 Chromaticity Diagram and luminance factors. Chromaticity is indicated by two values (x and y, unitless) and the luminance factor is represented by one value (Y, %). Four x and y chromaticity coordinates form a box of acceptable values for each color, as shown in Figure 3. The luminance factor has maximum and minimum limitations. The FHWA does not require "traffic control materials to maintain the color and luminance factors throughout the service life," but these values are required at initial placement (23 CFR Part 655).

As sign sheeting weathers, the x and y chromaticity coordinates usually fade toward white and the daytime luminance factor, Y, increases. One may hypothesize that the values of x, y, and Y change more quickly and noticeably than R_A because retroreflectivity is a physical property and color is a chemical property (from the color dyes). Chemical changes are facilitated by the ultraviolet radiation from the sun and the oxygen in moisture received by the exposed samples (*4*).

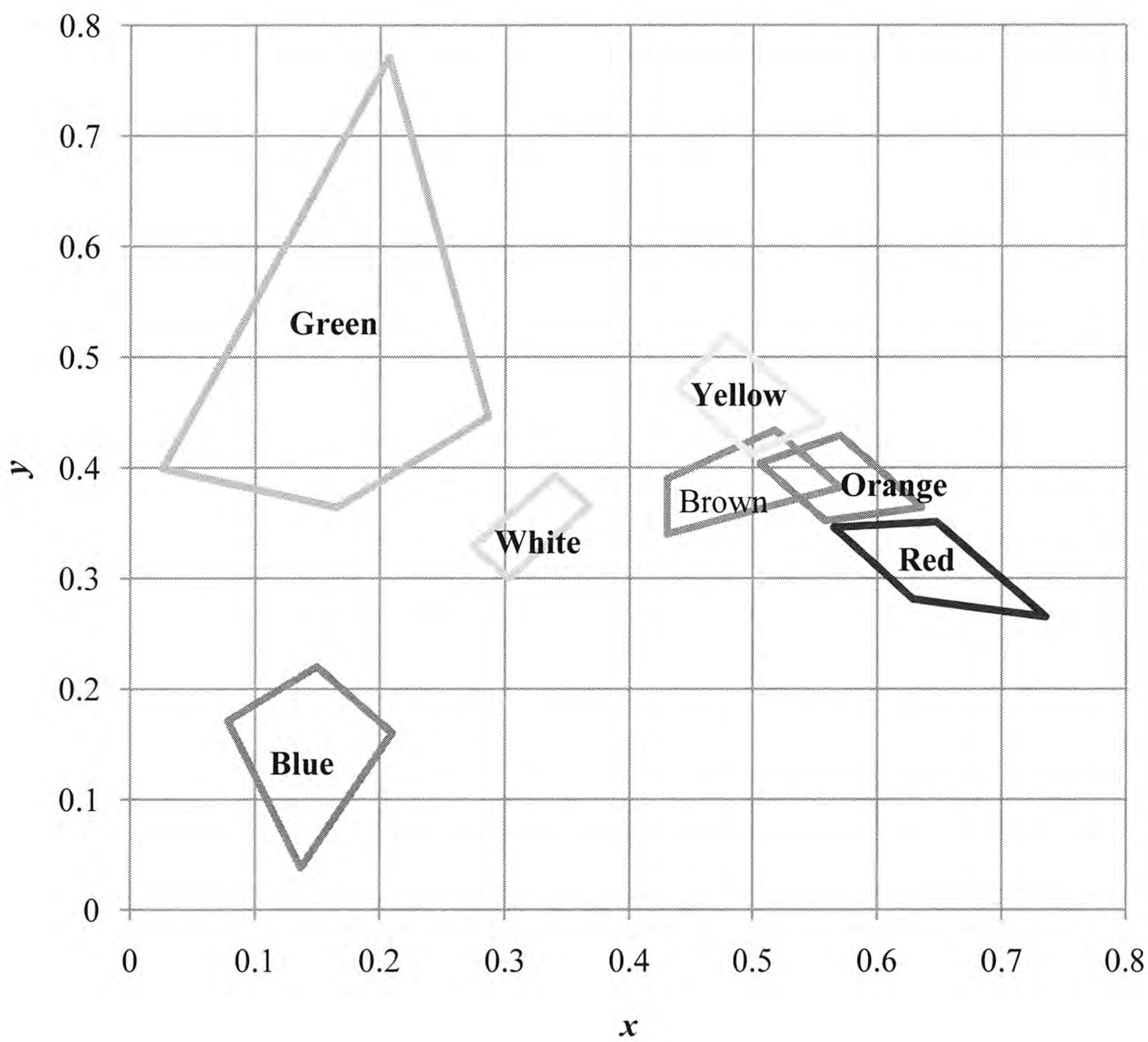

Figure 3. Acceptable Color Coordinate Boxes (Plotted from 23 CFR Part 655).

Accelerated Outdoor Weathering Research

A 1989 paper indicated that sheeting degradation increases twofold when a sign is positioned at a 45 degree elevation angle facing the equator (*4*). At the time of that research, sheeting was almost entirely beaded materials. This research effort assumes that the 2:1 weather ratio also applies to prismatic material, but could not find data to support or refute that assumption. The National Transportation Product Evaluation Program (NTPEP), sponsored by AASHTO, conducts three-year exposure tests of sign sheeting, taking advantage of this 2:1 degradation rate. The three-year exposure simulates six years of actual use. Testing products made by all major manufacturers, NTPEP monitors the performance of the sign sheeting to approve products for use by transportation agencies (*5*). However, these studies have one overriding limitation: six years is generally not long enough to determine the time required for a sign to fail. The NTPEP six-year simulation is insufficient for selecting products expected to provide long-lasting service.

Hawkins et al. investigated the long-term durability of fluorescent orange signs through an outdoor exposure similar to the NTPEP program (*6*). The researchers compared the performance of sign sheeting to standards established by ASTM and found that fluorescent orange signs generally fail in color before retroreflectivity.

Samples

In this project, researchers tested a total of nine sign sheeting products from four manufacturers. Some of the products were of the same ASTM Type and others were unique to that manufacturer. One color of each sign sheeting product was tested, including fluorescent materials. Fluorescent colors are not included in this paper because of the specialized equipment required to measure fluorescent properties. Other products of different types were tested on the weathering racks but are also not included in this study. Table 2 identifies the sheeting Types that were tested with the number of manufacturers whose products were included in the study.

Table 2. Number of Tested Products of Each Sheeting Type.

Type of Sheeting	ASTM Type	Number of Manufacturers
Beaded	I	2
	II	1
	III	3
Prismatic	IV	1
	VIII	1
	IX	1

METHODOLOGY

In 1999, researchers at the Texas Transportation Institute (TTI) obtained retroreflective sign sheeting products for long-term weathering tests. The testing lasted for over 10 years (to produce a 20-plus year simulation) and concluded in 2010.

Placement

The samples were placed on aluminum substrates to simulate the in-field placement of aluminum signs. To take advantage of the 2:1 accelerated deterioration rate, the researchers placed the samples on weathering racks located on the Texas A&M University Riverside Campus. The racks faced south and the samples were oriented at a 45 degree elevation angle. Figure 4 shows the samples on the weathering racks.

Measurements

At approximately yearly intervals, researchers removed each sample from the weathering racks to clean and measure its retroreflectivity and color properties. R_A was measured with a handheld DELTA RetroSign retroreflectometer using a −4.0 degree entrance angle and 0.2 degree observation angle. Color measurements were made with different instruments. Measurements early in the weather process were made with a BYK Gardner ColorGuide, and later measurements were made with a HunterLab MiniScan. Color measurements were not made until the fifth year of simulated exposure. Both used CIE Standard Illuminant D65. The retroreflectometer and colorimeter were calibrated before each testing period using calibration tiles. Four measurements of retroreflectivity and color were used to determine the average values of R_A, x, y, and Y of each sample.

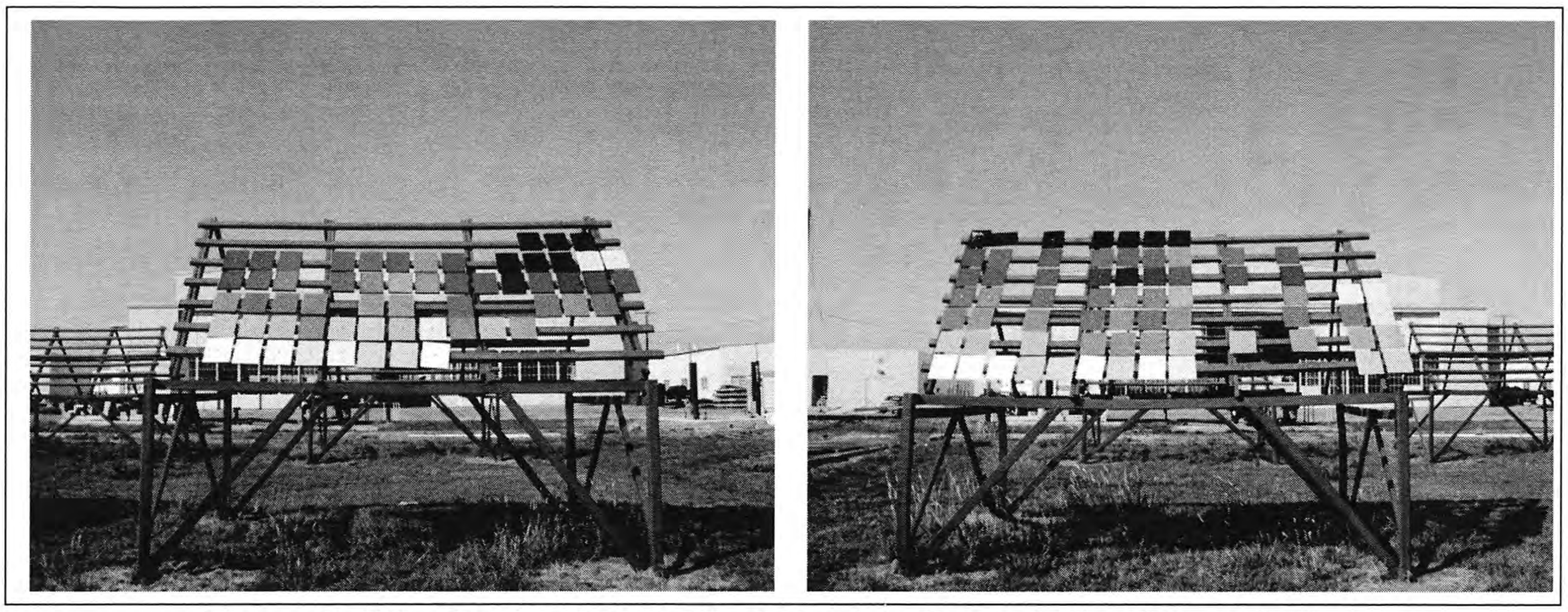

Figure 4. Weathering Racks Used for Accelerated Sign Degradation.

Failure Criteria

The samples were able to fail in one of four criteria: retroreflectivity, chromaticity, luminosity, or surface defects. Failure in retroreflectivity occurred if R_A fell below the MUTCD minimum level for that ASTM Type, as indicated in Table 1. Failure in chromaticity occurred if the chromaticity coordinates were recorded outside of the defined boxes shown in Figure 3. Failure in luminosity occurred if the daytime luminous factor was measured outside of the maximum or minimum levels given in 23 CFR Part 655. Retroreflectivity is the only criteria specifically addressed in a quantifiable manner in the MUTCD.

Surface defects introduced a subjective element to the failure criteria. Sometimes the surface of a sample would deteriorate while the sheeting maintained adequate levels of measured retroreflectivity and color. When the surface of a sample experienced significant cracking, peeling, or flaking such that further measurements may be affected by these defects, the sample was retired from the racks and considered failed. In most instances, failure in retroreflectivity or color accompanied these defects. Other samples were removed before the end of the study period if the sheeting had degraded far below the minimum level of retroreflectivity or color such that further testing was unnecessary. Often, not only were the chromaticity coordinates far outside of the color box, but the color of the sheeting could not be visually identified.

RESULTS

Table 3 displays the performance of the weathered samples over the exposure period. If a sample failed during the 20+ years of simulated exposure, the time of failure is indicated in the table. Sheeting types made by multiple manufacturers are distinguished by the letters x, y, and z in parentheses. Note that failures for green and white sheeting were rarer than for orange, red, and yellow. Because Type I sheeting should not be used for yellow and orange signs (there are no minimum retroreflectivity criteria), the performance for these samples are not included in the results. If a sample was removed for surface defects alone, it is indicated by the number of years to reach that failure point.

Table 3. Simulated Time for Failure Modes.

Color	Sample (by ASTM Type)	Min. R_A (cd/lx/m^2)	Years to Failure			
			Retroreflectivity	Chromaticity	Luminosity	Surface Defects[1]
White	I (y)	50	16	+	+	
	I (z)	50	7	+	+	
	II	120	15	+	+	
	III-B (x)	120	+	+	+	
	III-B (y)	120	+	+	+	
	III-B (z)	120	+	+	+	
	IV	250	21	+	+	
	VIII	250	+	+	+	20
	IX	250	+	+	+	
Green	I (y)	7	+	+	+	
	I (z)	7	7	+	+	
	II	15	+	+	+	
	III-B (x)	25	+	+	+	16.5
	III-B (y)	25	+	+	+	
	III-B (z)	25	+	+	+	
	IV	25	+	+	+	
	VIII	25	+	+	+	
	IX	25	+	+	+	
Orange	II	75	16	9.5	+	
	III-B (x)	75	+	<5	+	
	III-B (y)	75	+	7	+	
	III-B (z)	75	+	<5	9	
	IV	75	+	6.5	13	
	VIII	75	+	20	+	
Red	I (y)	7	+	6.5	10	
	I (z)	7	13	14.5	+	
	II	7	+	9.5	12	
	III-B (x)	7	+	7	16	
	III-B (y)	7	+	7	+	
	III-B (z)	7	+	4.5	+	
	IV	7	+	12	14	
	VIII	7	+	<5	+	
	IX	7	+	<5	+	
Yellow	II	75	16	+	+	
	III-B (x)	75	+	+	+	16.5
	III-B (y)	75	+	9.5	+	
	III-B (z)	75	+	+	+	
	IV	75	20	12	18	
	VIII	75	20	+	15	
	IX	75	+	+	+	

Notes: Samples were exposed for a simulated period of 21 years.
1. Indicates simulated year the sample was removed due to surface defects.
+ Sample did not fail during the exposure period.
- No MUTCD minimum for this sheeting type and color

For sheeting types that have multiple retroreflectivity criteria (as indicated in Table 1), the minimum R_A value in Table 3 is the highest minimum value required by the MUTCD. Sheeting types that have multiple uses (for example, overhead or ground-mounted signs for white or green sheeting) may have multiple criteria. The higher value is indicated as the minimum R_A because a sample that exceeds the higher value also exceeds the lower value.

Overall, most products failed in color chromaticity before retroreflectivity, especially for orange, red, and yellow sheeting. Considering retroreflectivity alone, each product lasted at least as long as the typical warranty for that type of sheeting. Most of the orange, red, and yellow samples failed in color chromaticity within 10 simulated years, and some even before five simulated years, the time when chromaticity was first measured.

White Sheeting

Figure 5 is a plot of R_A for white Type I and Type II sheeting throughout the length of the study. No color changes occur in white sheeting, so only retroreflectivity measurements are essential. In Figure 5, the two Type I samples have notably different initial R_A measurements. Type I (z) fell below the minimum level of 50 cd/lx/m^2 for black on white signs after seven years, while Type I (y) lasted beyond 15 years. (For white on red signs, the minimum R_A is 35 cd/lx/m^2, which extends the failure times of the samples.)

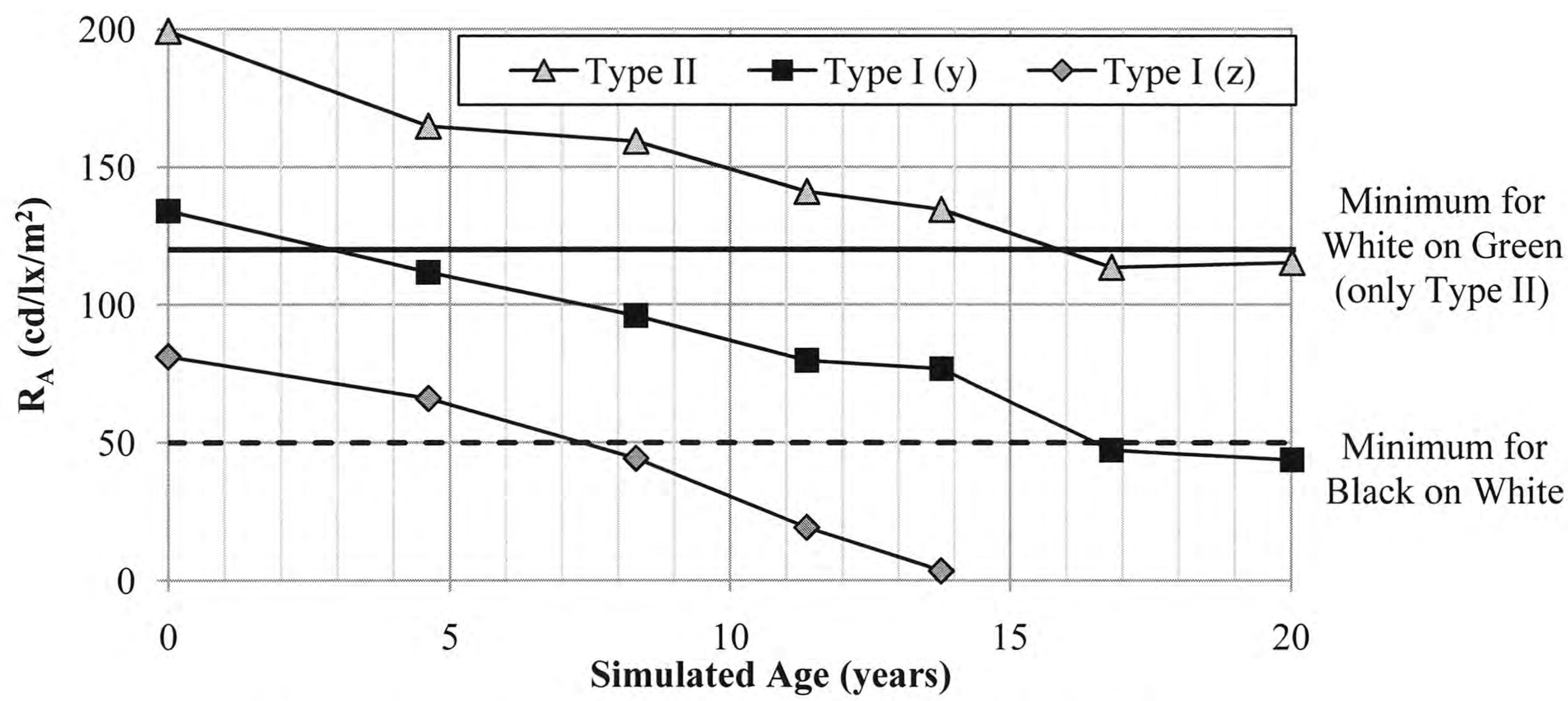

Figure 5. Retroreflectivity of White Types I & II Sheeting.

White Type II sheeting can be used for both black on white signs and ground-mounted white on green signs, whose minimum R_A is 120 cd/lx/m^2. This minimum level for Type II sheeting was maintained for more than 15 years.

Figures 6 and 7 are plots of the percentage of initial retroreflectivity retained in each white sample throughout the study. The most noticeable samples are Type I (z) and Type IV, which degraded by 50 percent within about nine years. The amount of initial retroreflectivity retained in sign sheeting is not addressed by the MUTCD.

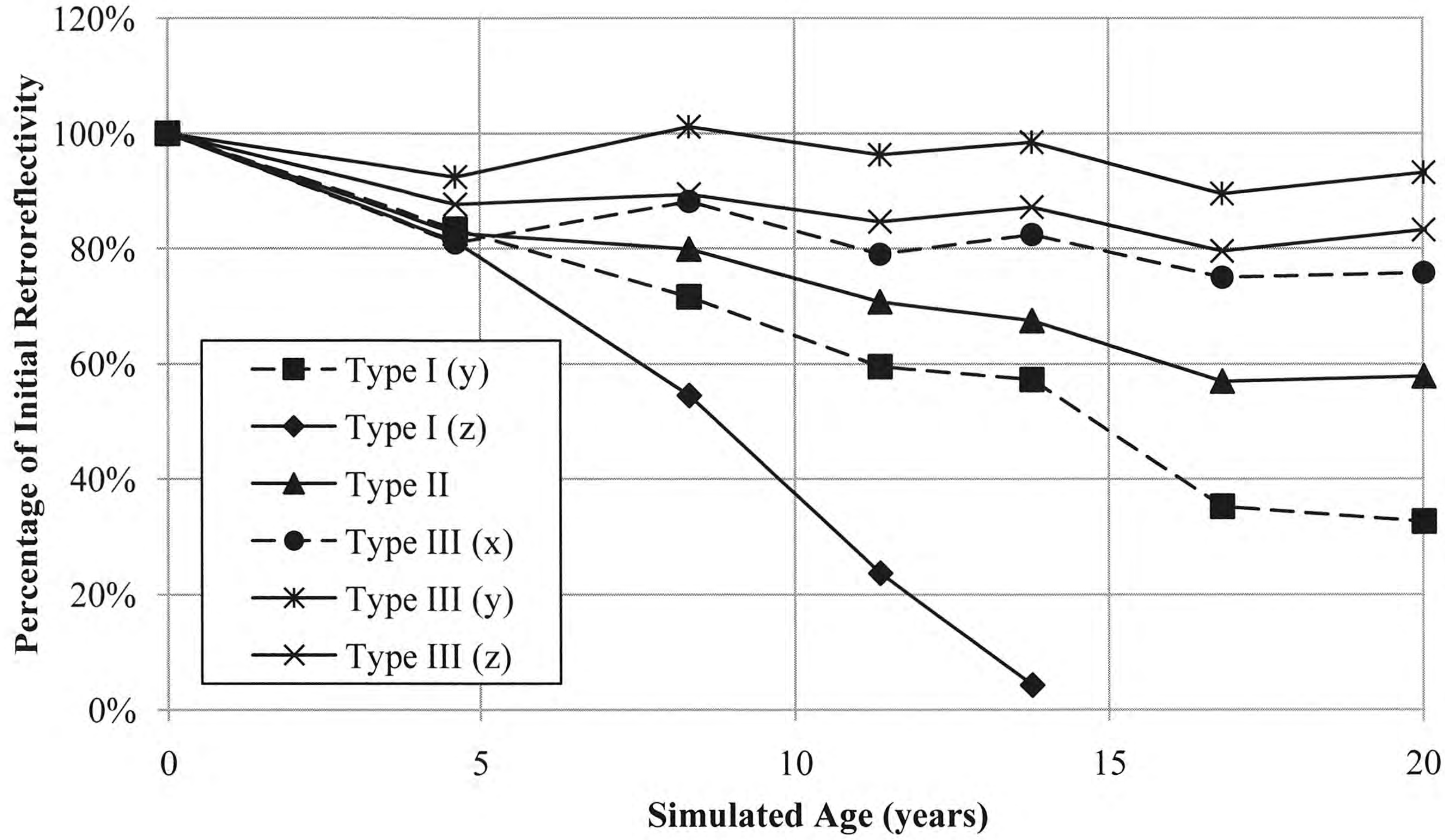

Figure 6. Percentage of Initial Retroreflectivity for White Beaded Sheeting.

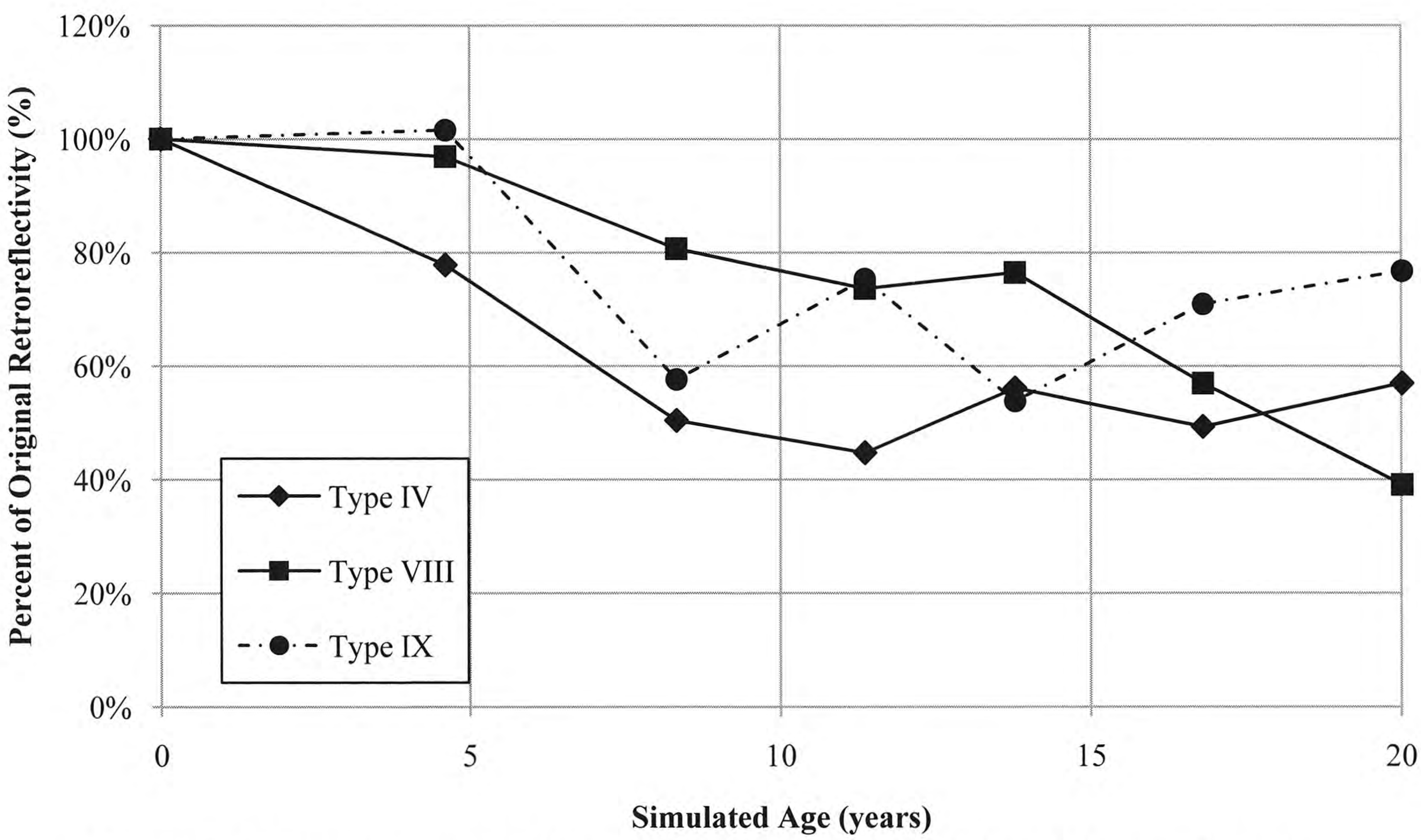

Figure 7. Percentage of Initial Retroreflectivity for White Prismatic Sheeting.

Green Sheeting

Similar to white, no green sheeting samples failed in color chromaticity or luminosity. Figure 8 is a plot of R_A for green Type I and Type II sheeting. The only product to fail was the Type I (z) sheeting, which failed at seven years.

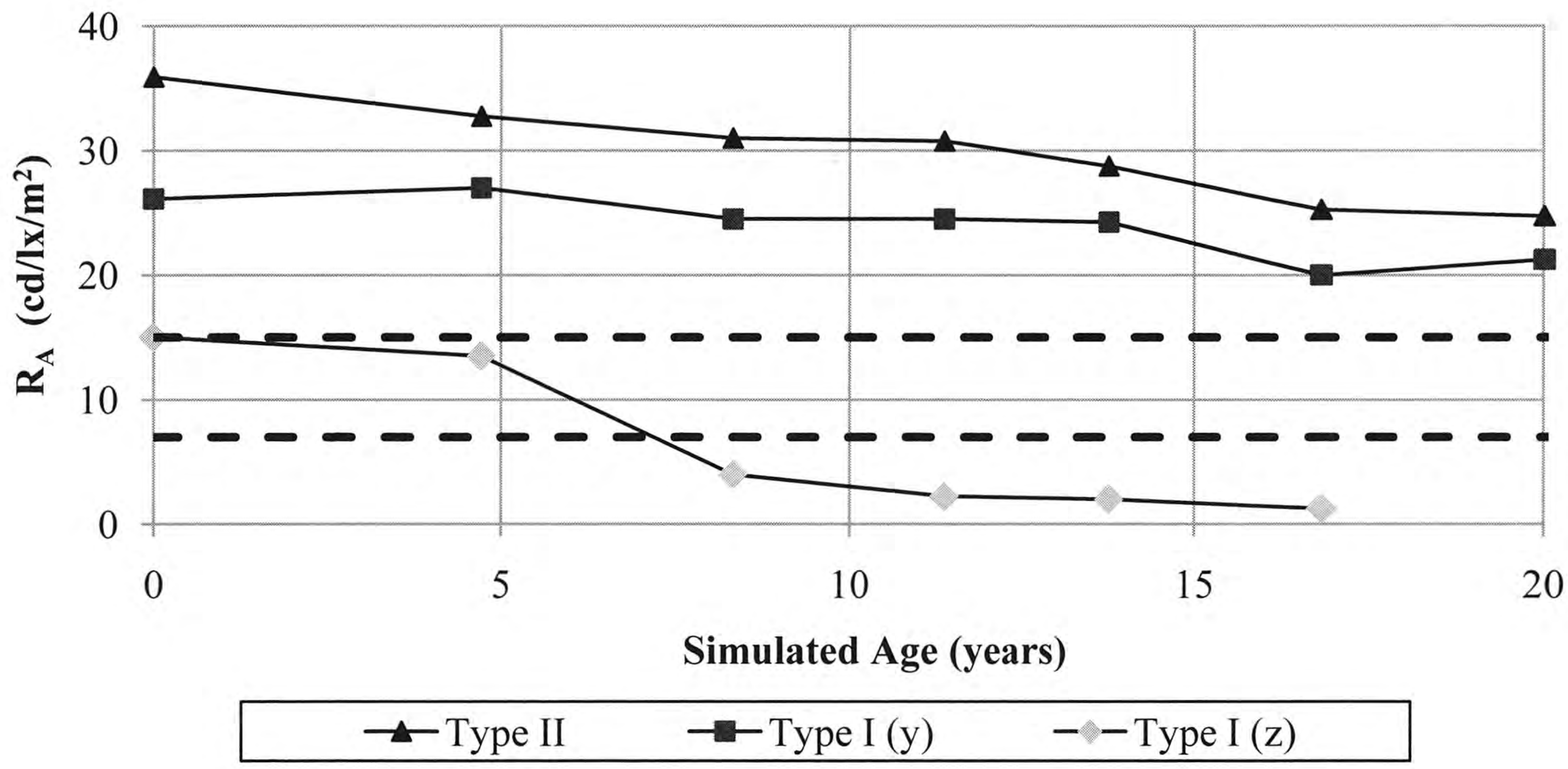

Figure 8. Retroreflectivity of Green Type I and Type II Sheeting.

Figure 9 is a plot of R_A for green Types III, IV, and VIII sheeting. The minimum value of R_A for these ASTM Types is 25 cd/lx/m^2. No sheeting failed in retroreflectivity, though one Type III sample was removed from the weathering racks because of surface deterioration.

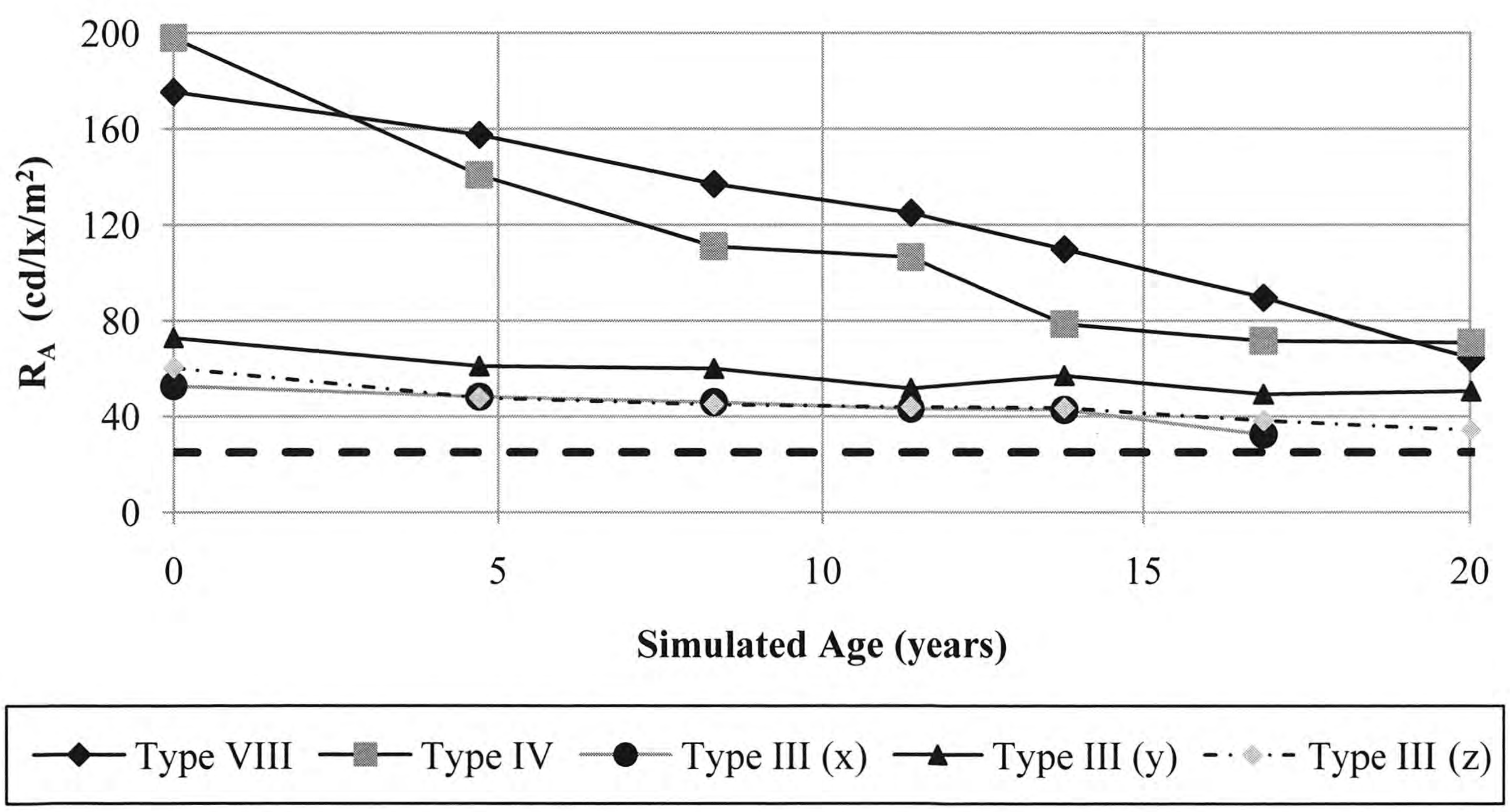

Figure 9. Retroreflectivity of Green Types III, IV, and VIII Sheeting.

Orange Sheeting

The only orange sample to fall below the MUTCD retroreflectivity minimum level for small orange signs was Type II, which occurred after 16 years. Figure 10 shows R_A for all orange sheeting samples. The minimum retroreflectivity requirement for large black on orange signs is 50 cd/lx/m^2, which the Type II sample exceeded throughout the study.

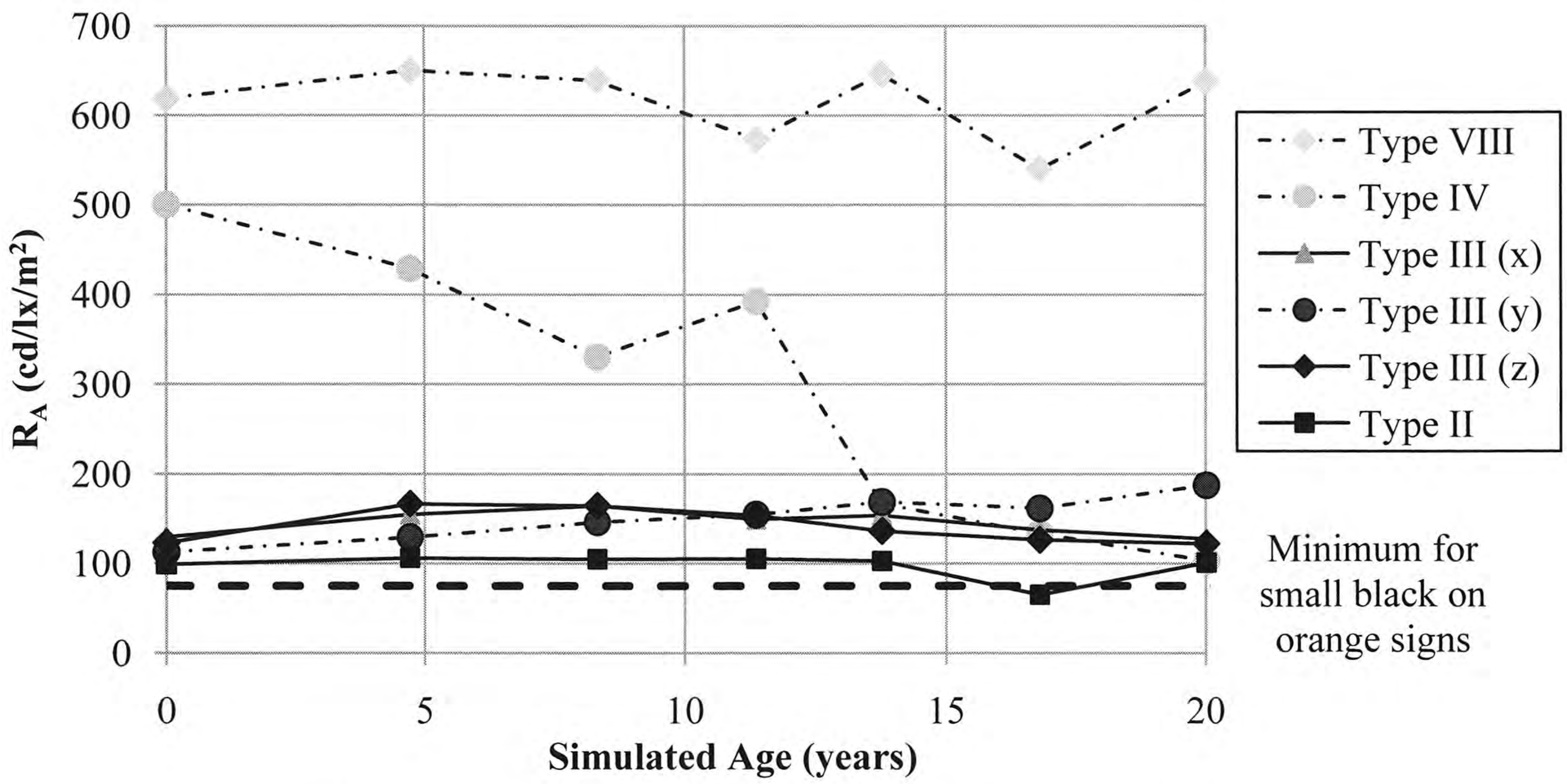

Figure 10. Retroreflectivity of All Orange Sheeting.

Figure 11 is a plot of the chromaticity coordinates for all of the orange sheeting samples. The plot shows that the samples faded toward yellow or white. Type VIII sheeting was the only sample to remain within the orange box throughout most of the testing period.

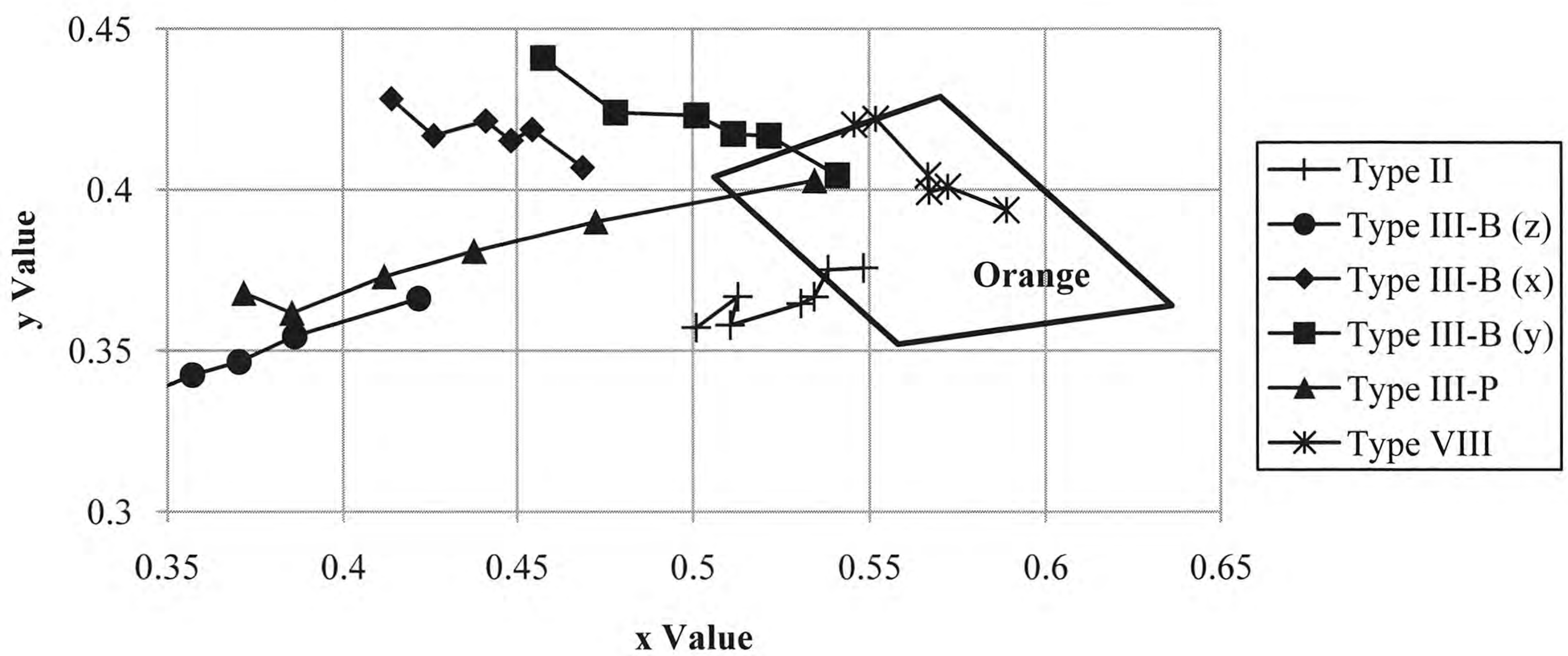

Figure 11. Chromaticity Coordinates for All Tested Orange Sheeting.

Red Sheeting

Figure 12 is a plot of R_A for red Type I and Type II sheeting. Other sheeting samples are not displayed in the plot because they are sufficiently above the MUTCD minimum value. The only sample that failed in retroreflectivity was the Type I (z) product, although the Type II product was removed after 16 years for excessive loss of color.

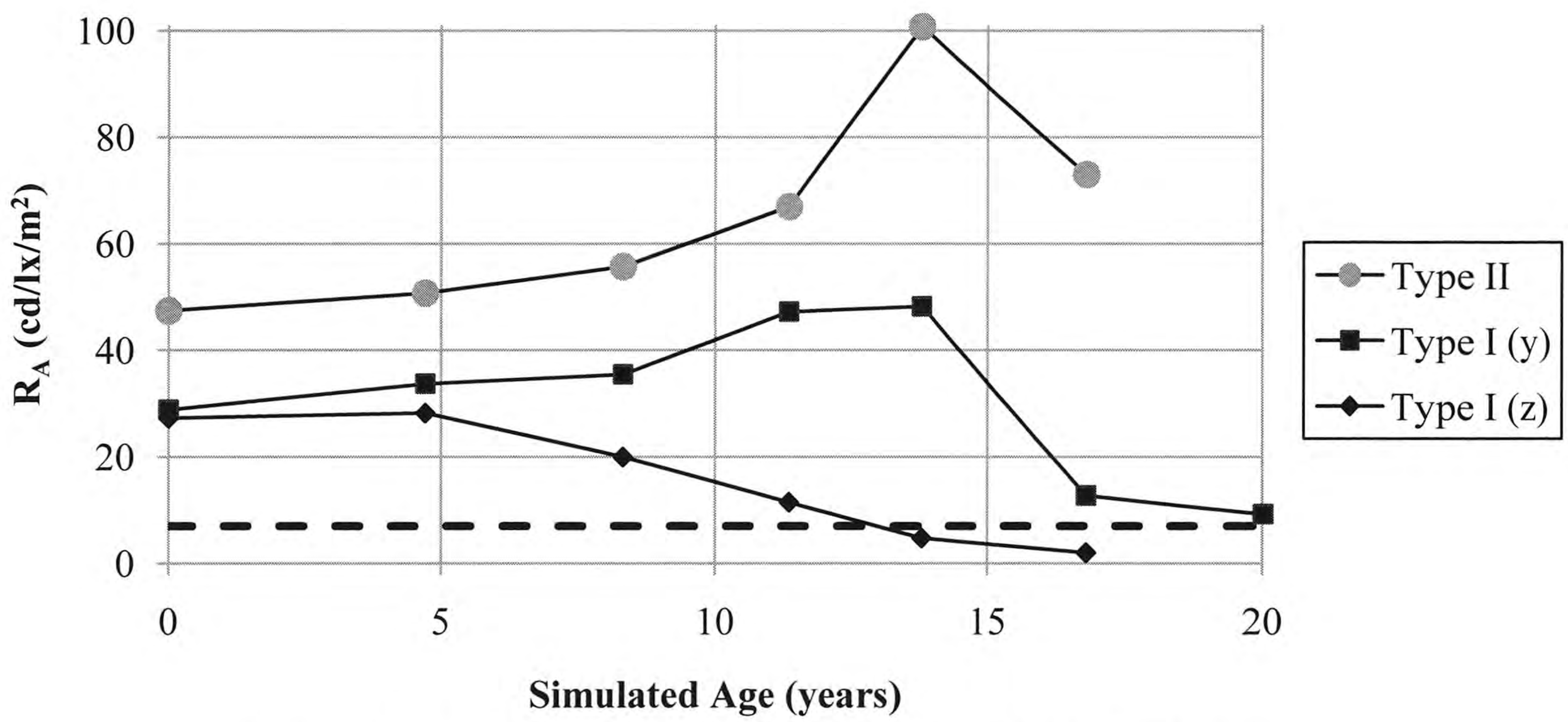

Figure 12. Retroreflectivity of Red Types I & II Sheeting.

Figure 13 shows the chromaticity changes in the tested red sheeting samples. Note that the overall direction of the chromaticity changes is much more uniform for red than for orange sheeting.

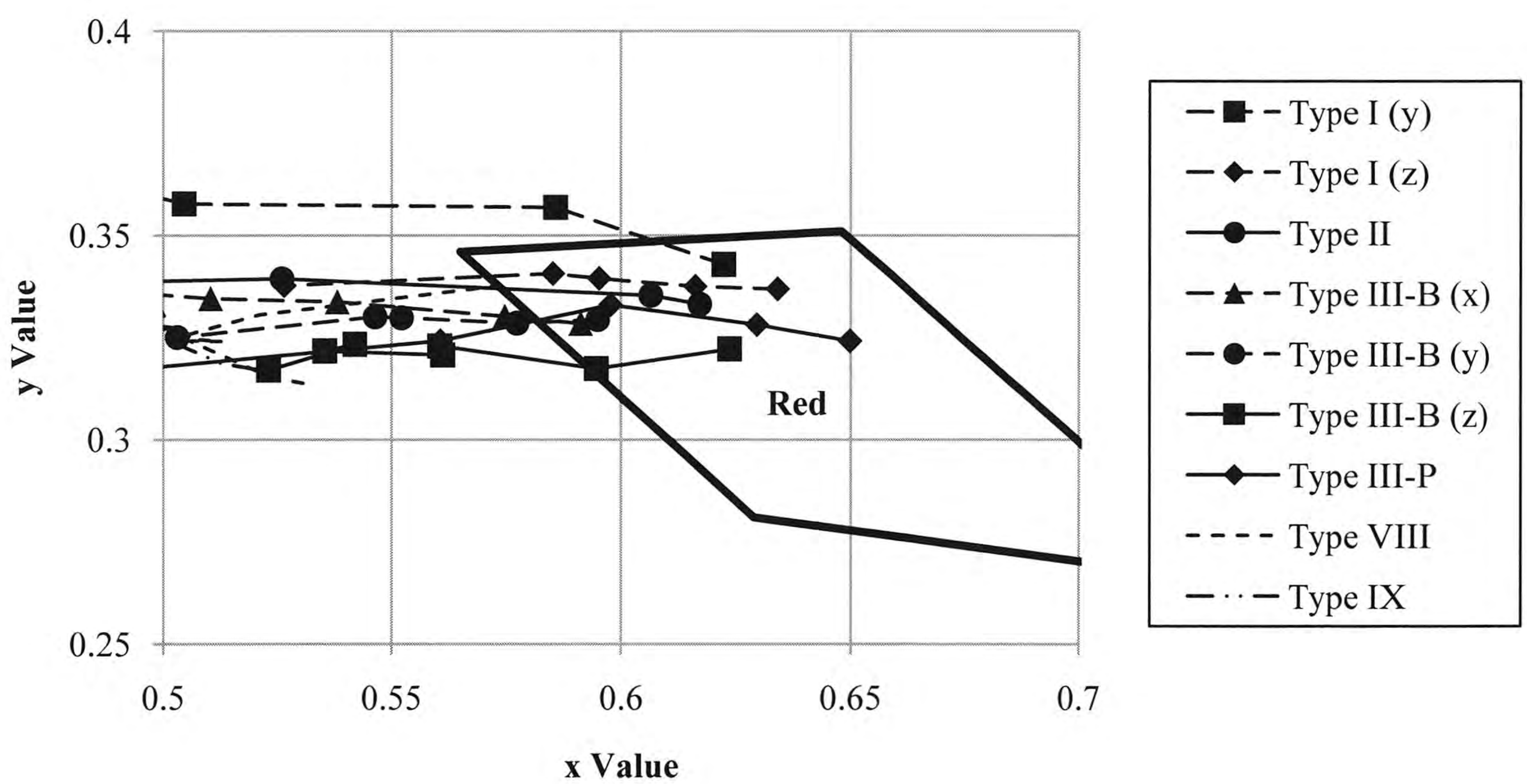

Figure 13. Chromaticity Coordinates for All Tested Red Sheeting.

Yellow Sheeting

Only one yellow sample failed in retroreflectivity within 20 simulated years. The Type II sample failed at 16 years and three samples were removed because of color loss and surface deterioration. Figure 14 shows R_A for all the yellow sheeting in the study. The failure line is for small black on yellow signs.

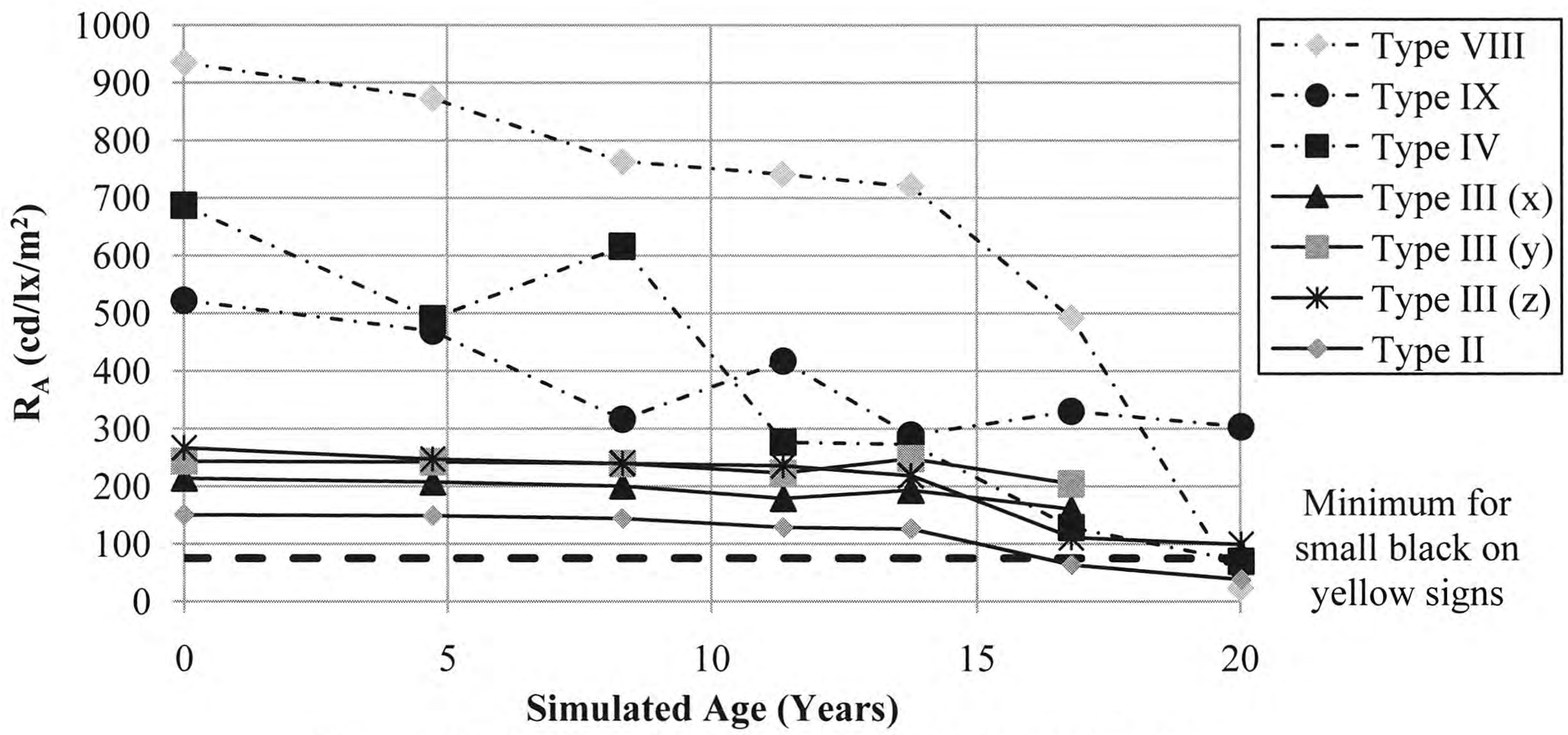

Figure 14. Retroreflectivity of All Yellow Sheeting.

Overall, the yellow sheeting samples maintained their color better than orange and red sheeting. Only two samples, Type III (y) and Type IV, were ever measured outside of the defined chromaticity box for yellow. Figure 15 shows the chromaticity plot for the yellow samples.

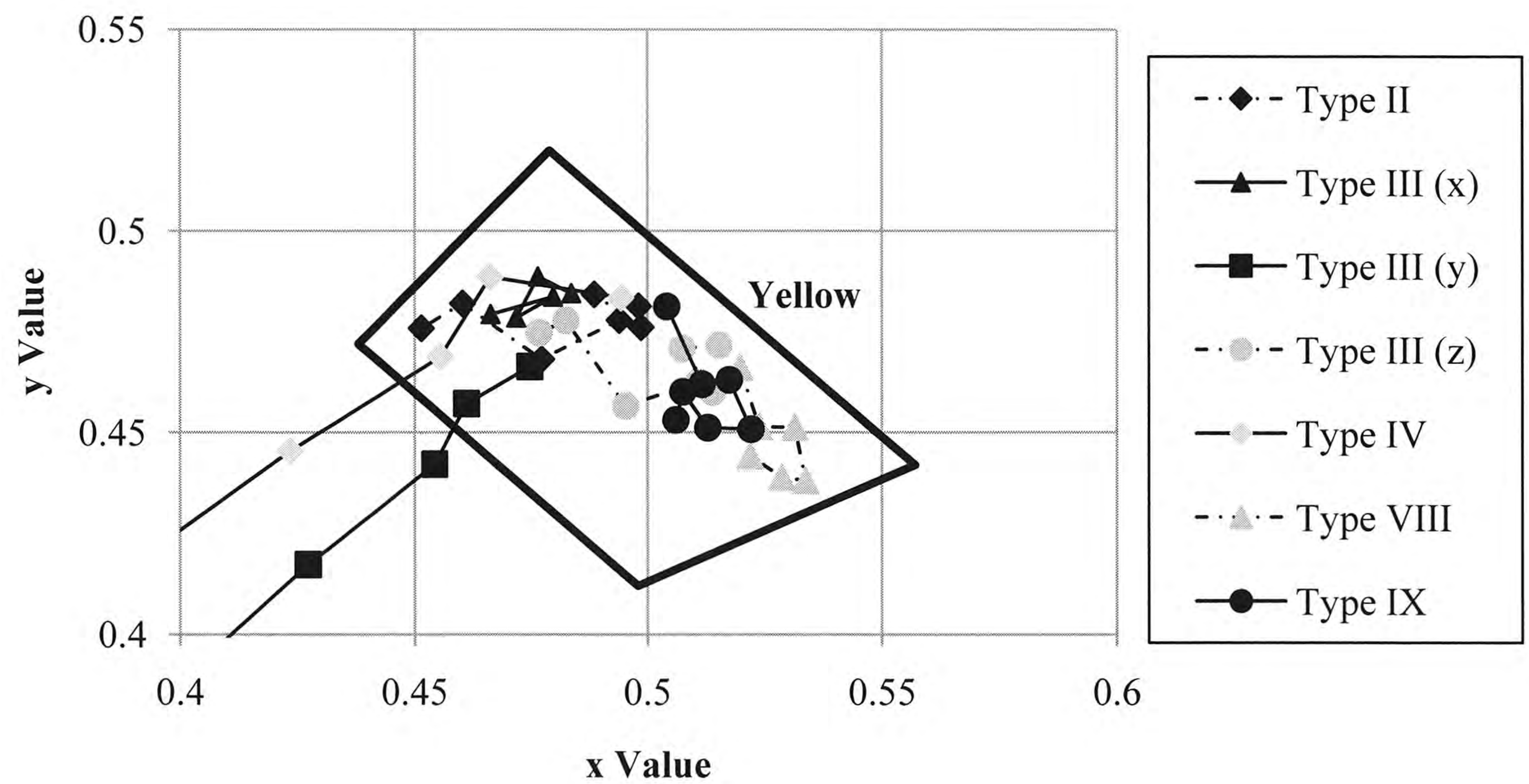

Figure 15. Chromaticity Coordinates for All Tested Yellow Sheeting.

CONCLUSIONS

The following conclusions can be made regarding the degradation of the samples' retroreflectivity and color:

- Almost all of the samples met the minimum retroreflectivity requirements for 15 simulated years or more. The only consistent exception was the Type I engineering grade products of one specific manufacturer, which generally lasted seven simulated years for white and green sheeting and 13 years for red sheeting.
- The chromaticity coordinates for red, orange, and yellow sheeting consistently faded outside of the defined color boxes. Chromaticity for white and green sheeting did not fall outside of the boxes.
- The colors of the samples consistently faded outside of the color boxes before the samples reached the minimum levels of retroreflectivity.
- Failures in luminosity were almost always accompanied by failures in chromaticity.
- If the color boxes defined in the CFR were to be included in determining a sign's useable life, some materials (with current warranties of 10 or more years) would have a useable life less than five years.

LIMITATIONS

This research effort was an unfunded and limited attempt to assess the long-term performance of retroreflective sign sheeting. There were a number of limitations associated with this undertaking (see below). A more thorough effort is needed to better define the long-term retroreflective and color performance of sign sheeting.

- Only one sample of each sheeting Type and color by a manufacturer was tested. This limits the ability to make statistical inferences regarding the performance of the samples.
- Measurements did not occur at the same time each year and for some years were skipped entirely. This affects the ability to accurately determine when a sample reached a defined measurement in retroreflectivity and color.
- Times to reach failure were estimated based on the two data points nearest to the failure criteria. An assumption was made that the degradation rate between two points is constant.
- The retroreflectivity and color measurements alone do not describe the amount of surface deterioration that each sample experienced. The handheld retroreflectometer and colorimeter were used to take one measurement in each of the samples' four quadrants. Care was taken to avoid the worst cracking and peeling so that they would have as little effect as possible on the tests, even though such measurements may produce better results than would otherwise occur.
- Throughout the study, different people were responsible for removing, cleaning, measuring, and replacing the samples. Reasonable care was taken to ensure that consistency was achieved in each of these processes, but small year-to-year variability may have been introduced.
- This study only included one-geometry retroreflectivity measurements (−4.0 degrees entrance angle with 0.2 degrees observation angle). Although the MUTCD

retroreflectivity criteria are for one-geometry, this oversimplifies the variable situations experienced by drivers. Further research should be conducted that shows the degradation in retroreflectivity at multiple observation and entrance angles.

- The results of this analysis must be interpreted with consideration for the climate in which the sign sheeting weathered. Degradation patterns in other climates may be significantly different from those discussed in this paper.

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APPENDIX

Table A-1. Retroreflectivity and Color Data for White Samples.

Sample (by ASTM Type)	Date	Retroreflectivity (cd/lx/m²)	Color Measurements			Actual Age (years)	Simulated Age (years)
			Y (%)	x	y		
I (y)	9/1/1999	134.03	—	—	—	0.00	0.00
	12/21/2001	111.75	—	—	—	2.31	4.61
	10/29/2003	96.00	48.04	0.312	0.334	4.16	8.32
	5/6/2005	79.75	45.47	0.313	0.335	5.68	11.36
	7/19/2006	76.75	44.71	0.313	0.336	6.88	13.77
	1/24/2008	47.25	41.76	0.313	0.332	8.40	16.81
	9/1/2009	43.75	42.83	0.313	0.335	10.01	20.02
	6/25/2010	45.00	40.88	0.309	0.328	10.82	21.64
I (z)	9/1/1999	81.20	—	—	—	0.00	0.00
	12/21/2001	66.00	—	—	—	2.31	4.61
	10/29/2003	44.25	45.46	0.323	0.346	4.16	8.32
	5/6/2005	19.25	44.33	0.319	0.343	5.68	11.36
	7/19/2006	3.50	38.51	0.324	0.347	6.88	13.77
II	9/1/1999	199.25	—	—	—	0.00	0.00
	12/21/2001	164.75	—	—	—	2.31	4.61
	10/29/2003	159.25	45.34	0.312	0.334	4.16	8.32
	5/6/2005	141.00	42.67	0.313	0.336	5.68	11.36
	7/19/2006	134.50	42.44	0.314	0.336	6.88	13.77
	1/24/2008	113.50	40.32	0.313	0.333	8.40	16.81
	9/1/2009	115.25	41.64	0.312	0.334	10.01	20.02
	6/25/2010	116.75	38.21	0.309	0.328	10.82	21.64
III (x)	9/1/1999	292.25	—	—	—	0.00	0.00
	12/21/2001	236.75	—	—	—	2.31	4.61
	10/29/2003	257.75	31.58	0.307	0.328	4.16	8.32
	5/6/2005	231.00	33.33	0.307	0.330	5.68	11.36
	7/19/2006	241.00	33.05	0.307	0.330	6.88	13.77
	1/24/2008	219.25	32.44	0.306	0.325	8.40	16.81
	9/1/2009	221.50	33.01	0.307	0.328	10.01	20.02
	6/25/2010	229.75	33.13	0.302	0.321	10.82	21.64
III (y)	9/1/1999	344.00	—	—	—	0.00	0.00
	12/21/2001	318.00	—	—	—	2.31	4.61
	10/29/2003	348.00	37.07	0.307	0.328	4.16	8.32
	5/6/2005	331.25	36.16	0.307	0.331	5.68	11.36
	7/19/2006	338.75	36.10	0.307	0.329	6.88	13.77
	1/24/2008	308.00	34.98	0.306	0.326	8.40	16.81
	9/1/2009	320.50	35.90	0.306	0.327	10.01	20.02
	6/25/2010	318.25	35.64	0.303	0.321	10.82	21.64
III (z)	9/1/1999	314.75	—	—	—	0.00	0.00
	12/21/2001	276.00	—	—	—	2.31	4.61
	10/29/2003	281.50	34.84	0.307	0.330	4.16	8.32
	5/6/2005	266.50	36.26	0.309	0.334	5.68	11.36
	7/19/2006	274.50	36.07	0.308	0.332	6.88	13.77
	1/24/2008	250.50	35.15	0.307	0.328	8.40	16.81
	9/1/2009	262.00	36.06	0.307	0.330	10.01	20.02
	6/25/2010	252.50	35.45	0.303	0.323	10.82	21.64

Note: — indicates no data were obtained at this time

Table A-1 (continued). Retroreflectivity and Color Data for White Samples.

Sample (by ASTM Type)	Date	Retroreflectivity (cd/lx/m^2)	Color Measurements Y (%)	x	y	Actual Age (years)	Simulated Age (years)
IV	9/1/1999	659.75	—	—	—	0.00	0.00
	12/21/2001	513.50	—	—	—	2.31	4.61
	10/29/2003	332.50	53.33	0.312	0.332	4.16	8.32
	5/6/2005	295.00	54.31	0.313	0.339	5.68	11.36
	7/19/2006	370.25	54.14	0.312	0.335	6.88	13.77
	1/24/2008	325.25	52.39	0.313	0.337	8.40	16.81
	9/1/2009	376.00	53.94	0.313	0.338	10.01	20.02
	6/25/2010	234.50	55.54	0.307	0.327	10.82	21.64
VIII	9/1/1999	1130.50	—	—	—	0.00	0.00
	12/21/2001	1095.50	—	—	—	2.31	4.61
	10/29/2003	912.00	52.85	0.306	0.352	4.16	8.32
	5/6/2005	832.75	55.00	0.307	0.326	5.68	11.36
	7/19/2006	864.75	56.20	0.310	0.350	6.88	13.77
	1/24/2008	644.00	51.93	0.310	0.327	8.40	16.81
	9/1/2009	442.00	50.55	0.333	0.328	10.01	20.02
IX	9/1/1999	653.25	—	—	—	0.00	0.00
	12/21/2001	663.50	—	—	—	2.31	4.61
	10/29/2003	376.50	49.57	0.304	0.313	4.16	8.32
	5/6/2005	492.25	53.90	0.311	0.354	5.68	11.36
	7/19/2006	351.50	53.01	0.307	0.323	6.88	13.77
	1/24/2008	463.50	50.85	0.313	0.349	8.40	16.81
	9/1/2009	501.50	54.16	0.313	0.352	10.01	20.02
	6/25/2010	483.00	51.13	0.315	0.339	10.82	21.64

Note: — indicates no data were obtained at this time

Table A-2. Retroreflectivity and Color Data for Green Samples.

Sample (by ASTM Type)	Date	Retroreflectivity (cd/lx/m^2)	Color Measurements Y (%)	x	y	Actual Age (years)	Simulated Age (years)
I (y)	9/1/1999	26.13	—	—	—	0.00	0.00
	1/8/2002	27.00	4.89	0.127	0.416	2.36	4.71
	10/29/2003	24.50	4.92	0.127	0.418	4.16	8.32
	5/6/2005	24.50	5.38	0.139	0.457	5.68	11.36
	7/19/2006	24.25	5.45	0.134	0.462	6.88	13.77
	1/24/2008	20.00	5.88	0.158	0.435	8.40	16.81
	9/1/2009	21.25	5.31	0.135	0.412	10.01	20.02
	6/25/2010	23.25	5.57	0.138	0.427	10.82	21.64
I (z)	9/1/1999	15.03	—	—	—	0.00	0.00
	1/8/2002	13.50	5.92	0.141	0.413	2.36	4.71
	10/29/2003	4.00	8.18	0.182	0.394	4.16	8.32
	5/6/2005	2.25	9.37	0.198	0.422	5.68	11.36
	7/19/2006	2.00	9.79	0.197	0.426	6.88	13.77
	1/24/2008	1.25	9.50	0.211	0.408	8.40	16.81
	9/1/2009	0.00	9.93	0.200	0.397	10.01	20.02

Note: — indicates no data were obtained at this time

Table A-2 (continued). Retroreflectivity and Color Data for Green Samples.

Sample (by ASTM Type)	Date	Retroreflectivity (cd/lx/m^2)	Color Measurements			Actual Age (years)	Simulated Age (years)
			Y (%)	x	y		
II	9/1/1999	35.90	—	—	—	0.00	0.00
	1/8/2002	32.75	4.36	0.130	0.411	2.36	4.71
	10/29/2003	31.00	4.32	0.127	0.411	4.16	8.32
	5/6/2005	30.75	4.91	0.140	0.452	5.68	11.36
	7/19/2006	28.75	4.98	0.140	0.452	6.88	13.77
	1/24/2008	25.25	5.06	0.151	0.436	8.40	16.81
	9/1/2009	24.75	4.55	0.129	0.409	10.01	20.02
	6/25/2010	24.75	4.86	0.138	0.421	10.82	21.64
III (x)	9/1/1999	52.75	—	—	—	0.00	0.00
	1/8/2002	48.25	6.76	0.149	0.394	2.36	4.71
	10/29/2003	46.00	7.15	0.153	0.393	4.16	8.32
	5/6/2005	43.25	8.03	0.168	0.424	5.68	11.36
	7/19/2006	42.75	8.27	0.169	0.422	6.88	13.77
	1/24/2008	32.50	8.99	0.183	0.408	8.40	16.81
III (y)	9/1/1999	72.80	—	—	—	0.00	0.00
	1/8/2002	61.00	5.98	0.149	0.401	2.36	4.71
	10/29/2003	60.00	6.14	0.153	0.399	4.16	8.32
	5/6/2005	51.75	6.98	0.170	0.427	5.68	11.36
	7/19/2006	57.00	6.79	0.161	0.427	6.88	13.77
	1/24/2008	49.25	7.49	0.182	0.410	8.40	16.81
	9/1/2009	50.50	6.75	0.161	0.395	10.01	20.02
	6/25/2010	51.75	7.28	0.169	0.407	10.82	21.64
III (z)	9/1/1999	60.28	—	—	—	0.00	0.00
	1/8/2002	47.75	7.49	0.141	0.421	2.36	4.71
	10/29/2003	45.00	7.78	0.147	0.416	4.16	8.32
	5/6/2005	44.00	8.80	0.162	0.448	5.68	11.36
	7/19/2006	43.50	8.85	0.162	0.444	6.88	13.77
	1/24/2008	38.25	9.14	0.171	0.430	8.40	16.81
	9/1/2009	34.25	8.49	0.157	0.410	10.01	20.02
	6/25/2010	36.50	9.05	0.163	0.428	10.82	21.64
IV	9/1/1999	198.00	—	—	—	0.00	0.00
	1/8/2002	140.75	8.76	0.154	0.409	2.36	4.71
	10/29/2003	111.00	9.23	0.158	0.406	4.16	8.32
	5/6/2005	106.50	10.15	0.172	0.435	5.68	11.36
	7/19/2006	78.50	10.02	0.170	0.436	6.88	13.77
	1/24/2008	71.50	10.59	0.178	0.424	8.40	16.81
	9/1/2009	70.75	9.83	0.164	0.410	10.01	20.02
	6/25/2010	65.50	10.31	0.169	0.415	10.82	21.64
VIII	9/1/1999	175.45	—	—	—	0.00	0.00
	1/8/2002	157.50	6.94	0.132	0.427	2.36	4.71
	10/29/2003	137.00	7.85	0.150	0.394	4.16	8.32
	5/6/2005	125.00	9.59	0.164	0.460	5.68	11.36
	7/19/2006	109.75	9.95	0.169	0.439	6.88	13.77
	1/24/2008	89.50	10.94	0.176	0.438	8.40	16.81
	9/1/2009	64.25	11.19	0.169	0.411	10.01	20.02
	6/25/2010	58.25	11.47	0.172	0.432	10.82	21.64

Note: — indicates no data were obtained at this time

Table A-3. Retroreflectivity and Color Data for Orange Samples.

Sample (by ASTM Type)	Date	Retroreflectivity (cd/lx/m²)	Color Measurements			Actual Age (years)	Simulated Age (years)
			Y (%)	x	y		
II	9/1/1999	99.25	—	—	—	0.00	0.00
	1/8/2002	106.25	19.49	0.548	0.376	2.36	4.71
	10/29/2003	105.00	21.04	0.538	0.375	4.16	8.32
	5/6/2005	105.50	19.07	0.534	0.367	5.68	11.36
	7/19/2006	103.00	19.10	0.531	0.365	6.88	13.77
	1/24/2008	65.25	19.02	0.511	0.358	8.40	16.81
	9/1/2009	101.25	20.74	0.513	0.367	10.01	20.02
	6/25/2010	93.25	19.93	0.501	0.357	10.82	21.64
III (x)	9/1/1999	129.60	—	—	—	0.00	0.00
	1/8/2002	155.00	16.90	0.469	0.407	2.36	4.71
	10/29/2003	164.50	20.35	0.454	0.419	4.16	8.32
	5/6/2005	149.50	20.57	0.448	0.415	5.68	11.36
	7/19/2006	154.00	22.34	0.441	0.421	6.88	13.77
	1/24/2008	137.50	23.97	0.426	0.417	8.40	16.81
	9/1/2009	127.75	27.64	0.414	0.428	10.01	20.02
III (y)	9/1/1999	113.00	—	—	—	0.00	0.00
	1/8/2002	129.50	15.86	0.541	0.404	2.36	4.71
	10/29/2003	146.25	18.16	0.522	0.417	4.16	8.32
	5/6/2005	155.00	18.18	0.512	0.417	5.68	11.36
	7/19/2006	169.00	19.55	0.501	0.423	6.88	13.77
	1/24/2008	161.75	21.09	0.478	0.424	8.40	16.81
	9/1/2009	187.50	24.86	0.457	0.441	10.01	20.02
III (z)	9/1/1999	123.18	—	—	—	0.00	0.00
	1/8/2002	166.75	23.40	0.422	0.366	2.36	4.71
	10/29/2003	164.50	29.83	0.386	0.355	4.16	8.32
	5/6/2005	154.00	30.98	0.370	0.347	5.68	11.36
	7/19/2006	136.50	33.66	0.357	0.342	6.88	13.77
	1/24/2008	126.50	35.88	0.337	0.333	8.40	16.81
	9/1/2009	122.25	40.31	0.330	0.336	10.01	20.02
IV	9/1/1999	500.50	—	—	—	0.00	0.00
	1/8/2002	429.00	18.15	0.534	0.403	2.36	4.71
	10/29/2003	330.50	23.77	0.472	0.390	4.16	8.32
	5/6/2005	392.75	27.13	0.438	0.381	5.68	11.36
	7/19/2006	167.25	31.14	0.412	0.373	6.88	13.77
	1/24/2008	133.50	33.23	0.386	0.362	8.40	16.81
	9/1/2009	103.25	39.89	0.372	0.368	10.01	20.02
VIII	9/1/1999	619.00	—	—	—	0.00	0.00
	1/8/2002	650.50	16.27	0.589	0.401	2.36	4.71
	10/29/2003	639.00	20.00	0.572	0.403	4.16	8.32
	5/6/2005	573.50	19.57	0.567	0.404	5.68	11.36
	7/19/2006	646.25	21.24	0.567	0.406	6.88	13.77
	1/24/2008	539.75	21.13	0.552	0.411	8.40	16.81
	9/1/2009	638.50	24.78	0.546	0.428	10.01	20.02
	6/25/2010	659.00	26.64	0.544	0.421	10.82	21.64

Note: — indicates no data were obtained at this time

Table A-4. Retroreflectivity and Color Data for Red Samples.

Sample (by ASTM Type)	Date	Retroreflectivity (cd/lx/m^2)	Color Measurements Y (%)	x	y	Actual Age (years)	Simulated Age (years)
I (y)	9/1/1999	28.88	—	—	—	0.00	0.00
	1/8/2002	33.75	8.74	0.622	0.343	2.36	4.71
	10/29/2003	35.50	10.86	0.586	0.357	4.16	8.32
	5/6/2005	47.25	13.54	0.505	0.358	5.68	11.36
	7/19/2006	48.25	22.59	0.443	0.375	6.88	13.77
	1/24/2008	12.75	34.93	0.359	0.372	8.40	16.81
	9/1/2009	9.25	43.44	0.344	0.372	10.01	20.02
I (z)	9/1/1999	27.33	—	—	—	0.00	0.00
	1/8/2002	28.25	6.90	0.634	0.337	2.36	4.71
	10/29/2003	20.00	7.46	0.616	0.338	4.16	8.32
	5/6/2005	11.50	7.17	0.595	0.339	5.68	11.36
	7/19/2006	4.75	7.66	0.585	0.341	6.88	13.77
	1/24/2008	2.00	8.11	0.527	0.338	8.40	16.81
II	9/1/1999	47.48	—	—	—	0.00	0.00
	1/8/2002	50.75	7.41	0.617	0.333	2.36	4.71
	10/29/2003	55.75	7.90	0.606	0.335	4.16	8.32
	5/6/2005	67.00	10.30	0.526	0.339	5.68	11.36
	7/19/2006	100.75	16.13	0.440	0.338	6.88	13.77
	1/24/2008	73.00	34.52	0.323	0.324	8.40	16.81
III (x)	9/1/1999	46.53	—	—	—	0.00	0.00
	1/8/2002	63.00	6.72	0.591	0.328	2.36	4.71
	10/29/2003	69.75	7.51	0.575	0.330	4.16	8.32
	5/6/2005	75.25	8.20	0.538	0.334	5.68	11.36
	7/19/2006	86.25	9.57	0.510	0.335	6.88	13.77
	1/24/2008	90.50	13.28	0.454	0.339	8.40	16.81
III (y)	9/1/1999	48.53	—	—	—	0.00	0.00
	1/8/2002	63.75	6.61	0.595	0.329	2.36	4.71
	10/29/2003	68.75	6.93	0.577	0.329	4.16	8.32
	5/6/2005	71.25	6.73	0.552	0.330	5.68	11.36
	7/19/2006	79.75	6.82	0.546	0.330	6.88	13.77
	1/24/2008	78.75	7.96	0.503	0.325	8.40	16.81
	9/1/2009	99.25	9.01	0.499	0.332	10.01	20.02
	6/25/2010	110.75	9.96	0.468	0.327	10.82	21.64
III (z)	9/1/1999	63.30	—	—	—	0.00	0.00
	1/8/2002	74.00	3.92	0.624	0.322	2.36	4.71
	10/29/2003	71.25	4.37	0.595	0.317	4.16	8.32
	5/6/2005	71.25	4.57	0.561	0.323	5.68	11.36
	7/19/2006	73.25	4.61	0.561	0.321	6.88	13.77
	1/24/2008	72.50	5.25	0.535	0.322	8.40	16.81
	9/1/2009	77.50	6.07	0.542	0.323	10.01	20.02
	6/25/2010	81.25	6.38	0.523	0.317	10.82	21.64
IV	9/1/1999	195.50	—	—	—	0.00	0.00
	1/8/2002	228.75	6.87	0.650	0.324	2.36	4.71
	10/29/2003	263.50	8.73	0.630	0.328	4.16	8.32
	5/6/2005	224.50	9.79	0.598	0.333	5.68	11.36
	7/19/2006	240.25	11.89	0.561	0.324	6.88	13.77
	1/24/2008	239.50	15.78	0.497	0.318	8.40	16.81
	9/1/2009	134.00	21.03	0.460	0.320	10.01	20.02

Note: — indicates no data were obtained at this time

Table A-4 (continued). Retroreflectivity and Color Data for Red Samples.

Sample (by ASTM Type)	Date	Retroreflectivity ($cd/lx/m^2$)	Color Measurements			Actual Age (years)	Simulated Age (years)
			Y (%)	x	y		
VIII	9/1/1999	239.25	—	—	—	0.00	0.00
	1/8/2002	289.25	7.68	0.531	0.314	2.36	4.71
	10/29/2003	280.75	9.05	0.523	0.315	4.16	8.32
	5/6/2005	277.50	9.38	0.506	0.324	5.68	11.36
	7/19/2006	285.50	9.95	0.513	0.324	6.88	13.77
	1/24/2008	252.75	10.42	0.503	0.325	8.40	16.81
	9/1/2009	281.50	11.87	0.521	0.331	10.01	20.02
	6/25/2010	248.75	11.28	0.575	0.338	10.82	21.64
IX	9/1/1999	152.08	—	—	—	0.00	0.00
	1/8/2002	196.50	8.05	0.521	0.317	2.36	4.71
	10/29/2003	135.50	9.69	0.513	0.319	4.16	8.32
	5/6/2005	185.75	10.26	0.497	0.326	5.68	11.36
	7/19/2006	147.75	11.09	0.502	0.328	6.88	13.77
	1/24/2008	151.25	12.49	0.488	0.329	8.40	16.81
	9/1/2009	175.25	14.85	0.496	0.342	10.01	20.02

Note: — indicates no data were obtained at this time

Table A-5. Retroreflectivity and Color Data for Yellow Samples.

Sample (by ASTM Type)	Date	Retroreflectivity (cd/lx/m2)	Color Measurements			Actual Age (years)	Simulated Age (years)
			Y (%)	x	y		
II	9/1/1999	150.50	—	—	—	0.00	0.00
	1/8/2002	148.75	31.52	0.498	0.481	2.36	4.71
	10/29/2003	144.25	35.12	0.489	0.484	4.16	8.32
	5/6/2005	128.50	32.14	0.499	0.476	5.68	11.36
	7/19/2006	126.00	33.08	0.494	0.478	6.88	13.77
	1/24/2008	63.75	30.69	0.477	0.468	8.40	16.81
	9/1/2009	38.25	36.43	0.460	0.482	10.01	20.02
	6/25/2010	25.25	35.41	0.451	0.476	10.82	21.64
III (x)	9/1/1999	213.75	—	—	—	0.00	0.00
	1/8/2002	207.25	21.57	0.484	0.485	2.36	4.71
	10/29/2003	200.50	24.26	0.477	0.489	4.16	8.32
	5/6/2005	178.75	22.78	0.472	0.478	5.68	11.36
	7/19/2006	193.25	23.45	0.480	0.484	6.88	13.77
	1/24/2008	160.25	23.63	0.466	0.479	8.40	16.81
III (y)	9/1/1999	243.25	—	—	—	0.00	0.00
	1/8/2002	242.25	22.13	0.475	0.466	2.36	4.71
	10/29/2003	239.50	24.18	0.461	0.457	4.16	8.32
	5/6/2005	222.75	22.67	0.454	0.442	5.68	11.36
	7/19/2006	248.00	23.37	0.427	0.417	6.88	13.77
	1/24/2008	204.33	23.41	0.409	0.398	8.40	16.81

Note: — indicates no data were obtained at this time

Table A-5 (continued). Retroreflectivity and Color Data for Yellow Samples.

Sample (by ASTM Type)	Date	Retroreflectivity (cd/lx/m2)	Color Measurements			Actual Age (years)	Simulated Age (years)
			Y (%)	x	y		
III (z)	9/1/1999	266.50	—	—	—	0.00	0.00
	1/8/2002	247.00	19.72	0.515	0.472	2.36	4.71
	10/29/2003	239.75	21.62	0.507	0.471	4.16	8.32
	5/6/2005	235.50	20.30	0.514	0.460	5.68	11.36
	7/19/2006	218.50	20.78	0.511	0.462	6.88	13.77
	1/24/2008	111.00	21.54	0.495	0.457	8.40	16.81
	9/1/2009	99.50	24.21	0.483	0.478	10.01	20.02
	6/25/2010	77.00	24.57	0.477	0.475	10.82	21.64
IV	9/1/1999	686.75	—	—	—	0.00	0.00
	1/8/2002	490.25	33.82	0.494	0.483	2.36	4.71
	10/29/2003	616.00	38.89	0.466	0.489	4.16	8.32
	5/6/2005	276.25	39.76	0.456	0.469	5.68	11.36
	7/19/2006	273.00	41.85	0.424	0.445	6.88	13.77
	1/24/2008	128.50	42.01	0.390	0.417	8.40	16.81
	9/1/2009	70.00	49.35	0.365	0.403	10.01	20.02
VIII	9/1/1999	934.50	—	—	—	0.00	0.00
	1/8/2002	873.25	26.50	0.520	0.466	2.36	4.71
	10/29/2003	763.50	28.17	0.524	0.451	4.16	8.32
	5/6/2005	741.00	26.13	0.531	0.451	5.68	11.36
	7/19/2006	720.00	25.52	0.534	0.438	6.88	13.77
	1/24/2008	492.75	21.63	0.529	0.439	8.40	16.81
	9/1/2009	23.75	25.70	0.522	0.444	10.01	20.02
IX	9/1/1999	522.25	—	—	—	0.00	0.00
	1/8/2002	469.00	29.84	0.504	0.481	2.36	4.71
	10/29/2003	316.25	31.27	0.512	0.462	4.16	8.32
	5/6/2005	417.25	30.21	0.517	0.463	5.68	11.36
	7/19/2006	289.00	29.72	0.522	0.451	6.88	13.77
	1/24/2008	330.25	25.85	0.513	0.451	8.40	16.81
	9/1/2009	303.50	24.97	0.508	0.460	10.01	20.02

Note: — indicates no data were obtained at this time

Evaluating the Effectiveness of LED Enhanced Stop Paddles for School Crossing Guard Use

Prepared for
Undergraduate Transportation Scholars Program

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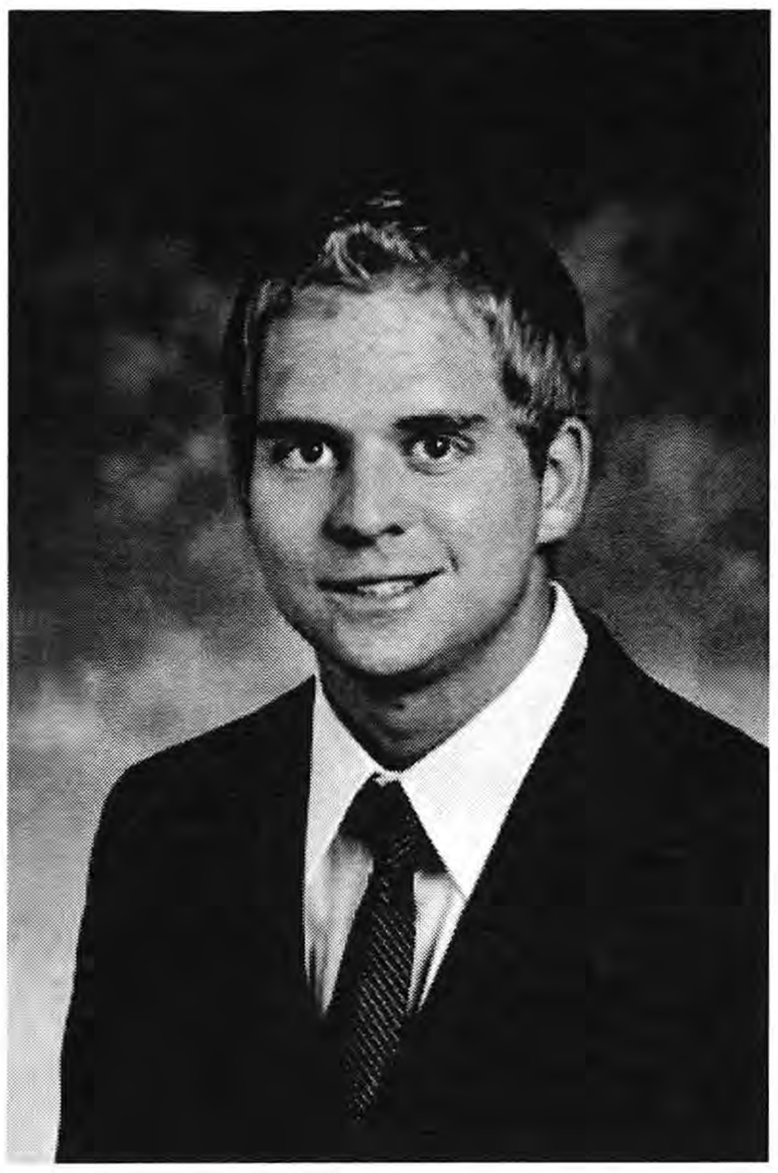

STUDENT BIOGRAPHY

William Huff is a senior at the University of Arkansas in Fayetteville, Arkansas. He will graduate in December 2010 with a Bachelor of Science in civil engineering. William currently serves as President of the University of Arkansas Chapter of Chi Epsilon, and is also a member of the American Society of Civil Engineers.

William has previous experience working as an intern for a variety of civil engineering firms including Terracon, Jones & Carter, and Garver Engineers. He has also worked as a teaching assistant for the University of Arkansas Department of Civil Engineering. William is one of the founding members of the University of Arkansas Trap and Skeet Team and has competed in tournaments at several different universities. His other hobbies include golfing, hunting, and fishing. William plans to attend graduate school, where he will pursue his master's of science in civil engineering and continue working toward certification as a Professional Engineer.

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The United States Government and the State of Texas do not endorse products or manufacturers. Trade or manufacturers' names appear herein solely because they are considered essential to the object of this report.

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SUMMARY

School crossing guards are the key to effective traffic control in school crosswalks, and their ability to stop traffic is critical to keeping school children safe. In an effort to increase the

conspicuity of crossing guards, several manufacturers have begun marketing stop paddles with embedded light emitting diodes (LEDs). While previous research has been done to evaluate LED lighting on larger stop signs and flagger stop/slow paddles, there has been little effort to examine the effect of LED lighting on stop paddles in school crossing circumstances. In addition, there is concern that these attempts to increase conspicuity might negatively impact a driver's ability to identify the stop sign.

The objective of this study was to evaluate the effectiveness of LED enhanced stop paddles in a school crossing application compared to a traditional non-lighted stop paddle. Specifically, this research assessed the ability of drivers to recognize the three main characteristics of a stop paddle: shape, background color, and stop legend. The stop paddles were presented before human subjects in a simulated school crossing situation.

The measures of effectiveness used in evaluating the stop paddles were the recognition distance of the shape of the sign, the recognition distance of the background color of the sign and the legibility distance of the text on the sign. Subjects approached each stop paddle in a test vehicle and were asked to state when they could identify each of these sign attributes. These distances as well as driver commentary were recorded and analyzed.

The results show that the addition of LEDs to stop paddles does not significantly affect the recognition distance or legibility distance of a stop paddle compared to a standard, non-lighted stop paddle. The stop paddles with a full string of red LEDs around the border (Treatments 3a and 3b) were found to have greater mean shape recognition distances and greater mean background color recognition distances than those of the other LED enhanced stop paddles. There were no significant differences in the mean text legibility distances between any of the treatments. Overall, the LED enhanced stop paddles with a full string of red LEDs forming the border of the sign were preferred by subjects.

TABLE OF CONTENTS

Student Biography 108
Acknowledgments 108
Summary 108
Table of Contents 110
Introduction 111
Background Information 111
  Evaluation of Effectiveness of Automated Flagger Assistance Devices 111
  Previous Research 112
Goals and Objectives 112
Treatments 113
Study Approach 114
  Location 114
  Time Period 115
  Subjects 116
  Equipment 116
Data Collection 117
Data Reduction and Analysis 117
Results 118
  Shape Recognition 118
  Background Color Recognition 118
  Text Legibility 118
  Standard Treatment 119
  Stopping Sight Distance 120
  Additional Subjective Data 120
Conclusions 120
References 121
Appendix 122

INTRODUCTION

The safety of crossing guards is essential as they are responsible for the safe and efficient crossing of school children within crosswalks. Over the past few decades, there has been an increase in children being dropped off or picked up for school in private vehicles. This has resulted in an increase in traffic through school zones and an increased potential for conflicts occurring while pedestrians are attempting crossing maneuvers. There are concerns regarding the safety of school crossing guards in their task of stopping drivers both before children enter the crosswalk, and while children are crossing. Drivers are often unresponsive and do not yield to crossing guards. This can be attributed to three main problems:

- driver non-compliance due to lack of attention or disregard of the crossing guard,
- poor visibility of the crossing guard due to visual clutter in the surrounding area, and
- need of better crossing guard training.

There are many possible solutions to the problem regarding school crossing guard safety. The focus of this project was to assess the effectiveness of the various handheld stop paddles with embedded light emitting diodes (LEDs) that are currently allowed by the Texas Manual on Uniform Traffic Control Devices (MUTCD) (*1*).

BACKGROUND INFORMATION

This research is a component of a larger project sponsored by the Texas Department of Transportation (TxDOT), "Evaluation of Effectiveness of Automated Flagger Assistance Devices" (Project No. 0-6407). The goal of this project is to improve the safety of roadway flaggers as well as school crossing guards, by evaluating the effectiveness of automated flagger assistance devices (AFADs) and innovative crosswalk devices.

Evaluation of Effectiveness of Automated Flagger Assistance Devices

Researchers with the Texas Transportation Institute (TTI) have begun investigating the issues facing school crossing guard safety. To date, TTI researchers have conducted telephone interviews with TxDOT personnel, performed field observations at problem school crossing locations, and conducted focus groups with crossing guards. From the data obtained, the research team was able to identify several strategies for improving the safety and effectiveness of school crossing guards. Upon review of all potential strategies, one of the easiest and most inexpensive solutions was to increase the conspicuity of the crossing guard by using LED enhanced handheld stop paddles. Also, a large number of crossing guards were interested in using these devices. However, there were some concerns regarding the effectiveness of the following wide range of light configurations currently approved in the Texas MUTCD (*1*):

- two white or red lights centered vertically above and below the STOP legend;
- two white or red lights centered horizontally on each side of the STOP legend;
- one white or red light centered below the STOP legend;
- a series of eight or more small white or red lights having a diameter of 1/4 inch or less along the outer edge of the paddle, arranged in an octagonal pattern at the eight corners

of the STOP paddle (more than eight lights may be used only if the arrangement of the lights is such that it clearly conveys the octagonal shape of the STOP paddle); or

- a series of white lights forming the shapes of the letters in the legend.

Previous Research

In another study performed by TTI (*2*), researchers investigated the impacts of various higher conspicuity sign materials on traffic operations and driver behavior. Several different high-conspicuity signs were examined, including a stop sign with eight LEDs embedded in the corners of the sign. Traffic operations data were collected and analyzed at field sites before and after the specified signs were installed. The research concluded that the red flashing LED stop signs were beneficial in reducing stop sign violations in both daytime and nighttime occurrences. However, the signs tested in this study were normal static stop signs and not handheld stop paddles.

Arnold and Lantz (*3*) also investigated the use of stop signs with LEDs embedded at each corner. Similar to the TTI study (*2*), this study collected data on vehicles approaching the stop sign both before and after the LED signs were installed. Researchers concluded that flashing LED stop signs were effective in reducing the speeds of vehicles approaching an intersection. A recommendation was made that these devices should be used as a potential safety countermeasure when addressing accident problems at stop sign intersections. The results of this study found the effect of LEDs on compliance to be inconclusive. Again, the study used normal static signs and not handheld paddles.

A study by Agent and Hibbs (*4*) evaluated the use of various work zone devices developed by the Strategic Highway Research Program (SHRP). One of the devices studied was a flashing handheld stop/slow paddle. Six different models of flashing paddles were evaluated with some having better results than others. Researchers found the overall experience with the flashing stop/slow paddles to be very positive and that drivers were able to observe the flashing paddle sooner than a typical paddle. A recommendation was made that expanded use of flashing stop/slow paddles in work zones was warranted. The paddles were used in a work zone setting.

Schrock (*5*) also evaluated the usefulness and workability of several technology-enhanced flagging devices, including handheld LED enhanced stop/slow paddles. Researchers used responses from transportation maintenance and emergency services personnel in focus groups as well as field surveys of motorists who viewed the devices in active work zones to gauge the usefulness of each device. Three LED enhanced stop/slow paddles were tested as well as a standard non-lighted paddle. Overall, the results revealed that there were few differences between the standard and flashing stop/slow paddles when tested in work zones. The paddles were 24 inches in diameter and were used in a work zone setting.

GOALS AND OBJECTIVES

Although prior research exists concerning the use of LEDs to enhance normal static stop signs and 24-inch work zone stop/slow paddles, limited research had been done to assess the effectiveness of LEDs in handheld stop paddles used in school zones, which are typically 18 inches in diameter. The purpose of the research presented herein was to determine the effectiveness of the different LED patterns that are currently available, compared to a standard, non-lighted stop paddle. To accomplish this goal, the following objectives were considered:

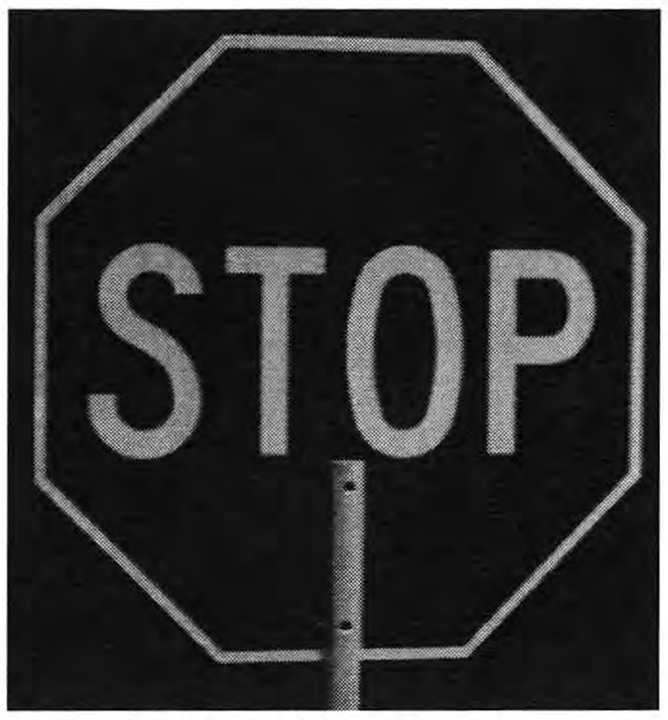
1. identify and obtain available LED enhanced stop paddles that meet the current Texas MUTCD standards, and
2. test the effectiveness of the LED enhanced stop paddles compared to a standard, non-lighted stop paddle.

TREATMENTS

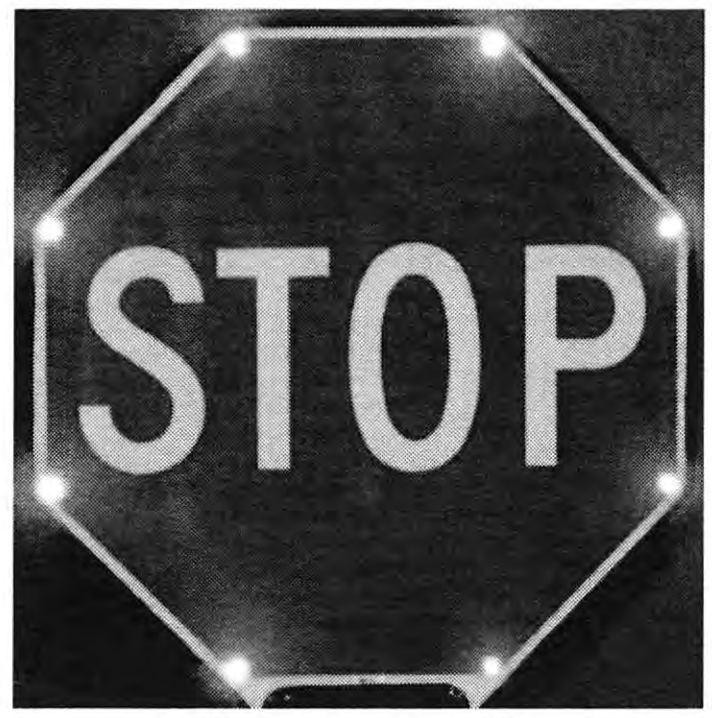
Due to time and budget constraints as well as availability, not all of the light configurations listed in the Texas MUTCD were evaluated in the study. A search was conducted as to light configurations currently available "off the shelf." After an extensive search, several LED enhanced stop paddles as well as a standard, non-lighted stop paddle were identified and ordered. The following stop paddles were tested in this study:

- a standard non-lighted stop paddle,
- a stop paddle containing eight flashing red LEDs, one in each corner;
- a stop paddle containing a full string of LEDs around the border that was set to steady burn;
- a stop paddle containing a full string of red LEDs around the border that was set to flashing mode;
- a stop paddle with two sets of flashing red LEDs, one above and one below the stop legend, and
- a stop paddle with a series of white steady burning LEDs forming the letters of the stop legend.

Figure 1 displays these six stop paddle designs. All paddles were 18 inches in diameter (the minimum size required by the Texas MUTCD) and were fabricated with high intensity sheeting. Unfortunately, due to the numerous malfunctions that occurred during the testing, an insufficient amount of data was collected for the stop paddle containing flashing red LEDs in each corner. Thus, this treatment was not included in the results of this study.



Treatment 1:
Standard
Non-Lighted



Treatment 2:
8 Flashing Red
Border Lights

Treatment 3a:
Steady-Burn Red
Full Border Lights

Treatment 3b:
Flashing Red
Full Border Lights

Treatment 4:
Flashing Red
Vertically Centered

Treatment 5:
Steady-Burn White
Forming Legend

Figure 1. Stop Paddles Tested.

STUDY APPROACH

Data were collected from human subjects approaching a simulated school crosswalk. The measures of effectiveness that were studied for each stop paddle were the shape recognition distance, the background color recognition distance, and the text legibility distance. Subjective data including driver opinions and comments were also examined.

Location

The data were collected at the Riverside Campus of Texas A&M University. The road network provided an urban atmosphere, while remaining relatively free of traffic. Figure 2 shows the location of the road that was used. The road chosen was ideal for the study due to the fair pavement conditions, lack of stop signs at the cross streets, low volume of traffic in the surrounding area, and also the direction in relation to the sun.

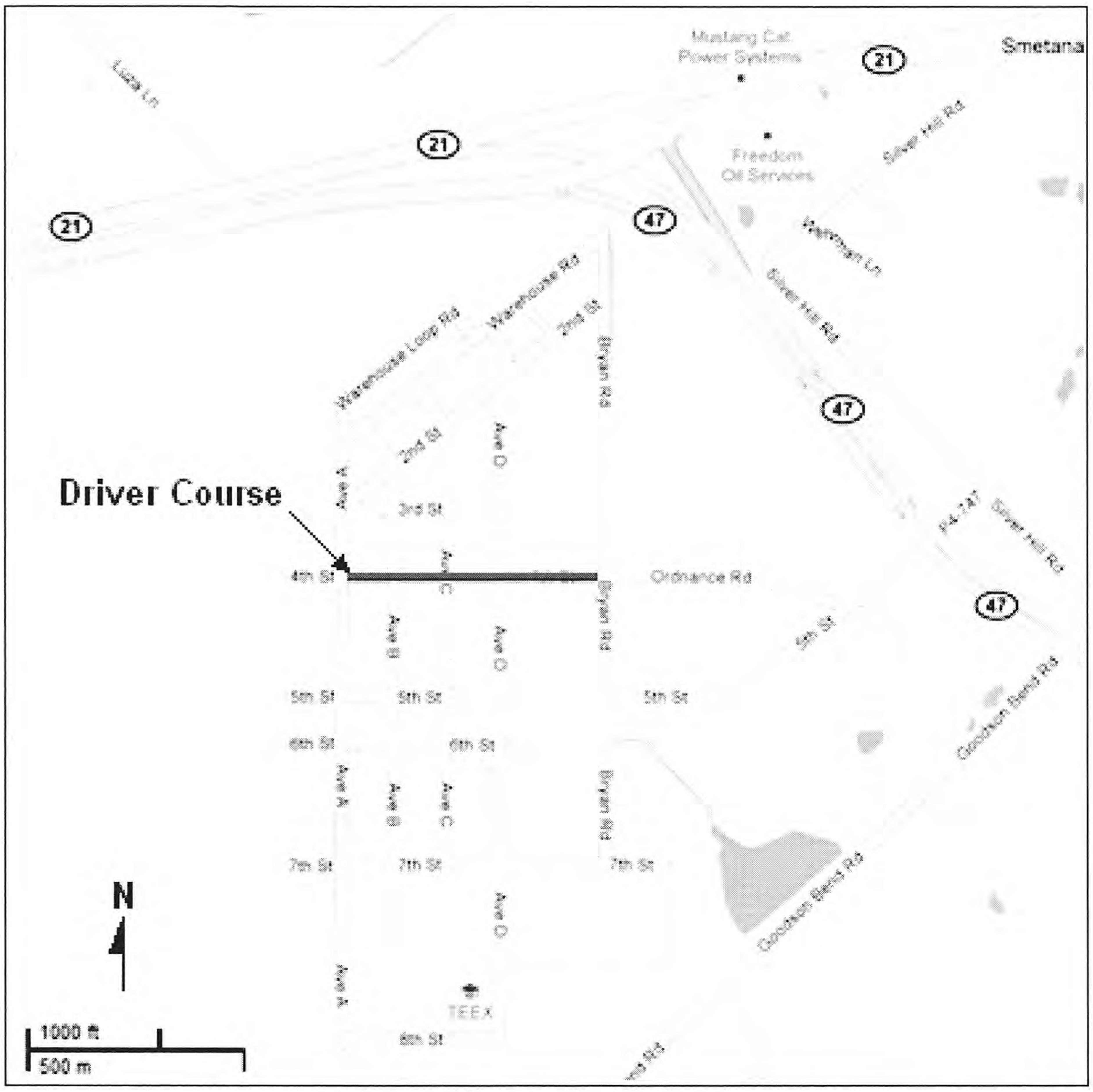


Figure 2. Study Location.

Time Period

Based on the telephone interviews with TxDOT personnel, field observations at problem school crossing locations, and focus groups with crossing guards, researchers found that crossing guards working during early morning hours and those located directly in front of the sun in the morning (i.e., backlit) experience more driver non-compliance. Researchers believe this is due to the low visibility conditions before sunrise and the sun "blinding" drivers when it is near the horizon early in the morning. Thus, the study was performed during the lowest light conditions that would occur during school crossing hours. In order to complete the study in a timely manner, data were collected during both morning and evening time periods. Based on the previously conducted field observations, the earliest school zone time started at 7:00 a.m. Researchers compared this time period to the sunrise times for the entire year (*6*) to determine the proportion of time school crossing guards were active during twilight and early morning time periods. This information along with sunrise/sunset times was then used to determine the study time period. Based on this information, the morning portion was executed 30 minutes before and after sunrise (i.e., from 6:00 a.m. to 7:00 a.m.). The evening portion was executed 30 minutes before and after sunset (i.e., from 8:00 p.m. to 9:00 p.m.). Every attempt was made to collect data during the same time periods each day. During each time period, subjects drove toward the sun (i.e., east in the morning and west in the evening). However, for the majority of the data collection periods, there were overcast conditions.

Subjects

A total of 36 subjects were recruited for this study. Research subjects were required to be over the age of 18 and have a current Texas driver's license. Subjects were selected in a demographic sampling procedure based on the age and gender of the Texas driving population (*7*). Only people in a younger age group (18–39) and older age group (55+) were employed in the study. Table 1 summarizes the overall demographic distribution obtained. Before the test began, each subject was given a standard static visual acuity test, a contrast sensitivity test, and a color blindness test. These screenings provided comparison information for data reduction and ensured that all participants had at least minimal levels of acceptable vision prior to beginning the study. No participant had to be disqualified from study participation based on the visual screening. Subjects were also given ample time before the test to become familiar with the test vehicle.

Table 1. Test Subject Demographics.

Sample	Gender		Age	
	Male	**Female**	**18-39**	**55+**
Study Sample	50%	50%	62%	38%
Texas Data (*7*)	50%	50%	44%	25%

Equipment

A TTI instrumented vehicle was used as the control vehicle. The vehicle was a Ford Taurus that was equipped with a distance measuring instrument (DMI). To increase the efficiency of the project, two vehicles were used. An assumption was made that the vehicles were identical. A picture of the test vehicles can be seen in Figure 3.

Figure 3. Test Vehicles.

The order in which the treatments were presented to the subjects was randomized, as well as the side of the road in which the crossing guard was positioned, to reduce learning effects that could occur through multiple runs. Because researchers were concerned with the safety of school crossing guards when they first enter the travel lanes and begin to stop traffic, the crossing guard was positioned on the left or right side of the road, instead of in the middle of the roadway.

DATA COLLECTION

Data were collected by a team of two researchers, while another research assistant dressed in a standard fluorescent yellow-green safety vest, played the role of the crossing guard and held the stop paddles. Subjects drove down the road and were presented with each of the five treatments. The subject was asked to state out loud when they could clearly identify the background color of the sign, the shape of the sign, and could completely read the text on the sign. A researcher accompanied the subject in the vehicle and used a DMI to measure and record these distances. After each treatment was presented, the driver was asked a series of follow-up questions about the treatment they had just viewed as well as their opinion of it. The subjects were also asked to rate their overall ability to recognize and read each treatment. The rating scale ranged from one (very well) to three (not well). Researchers recorded this subjective data as well.

Upon completion of the driving portion of the study, subjects were presented with an image containing all of the treatments. If the treatment was flashing, a video clip was shown. The subjects were then asked to rank the treatments in order of their preference (one being the best and five being the worst) as well as provide any other additional comments or suggestions.

DATA REDUCTION AND ANALYSIS

Following the data collection, each subject's raw data were screened and reduced into a fully formatted data set to obtain the necessary information for analysis. During the data screening process, any anomalous data (e.g., misidentifications, malfunctioning treatment, etc.) was eliminated from the set. The following measures of effectiveness were analyzed from each subject:

- recognition distance of the shape of the sign,
- recognition distance of the background color of the sign,
- legibility distance of the sign text, and
- subjective data from the follow-up questions.

Upon completion of the data formatting procedures, a statistical analysis was performed, computing the mean, standard deviation, minimum and maximum distance for each treatment (see Appendix). Researchers used analysis of variance and Tukey's Honestly Significant Difference (HSD) procedure to determine if there were significant differences among the mean distances for each treatment. A 95 percent level of confidence was used for all statistical analyses. Researchers were able to determine that the time period the data were collected did not significantly impact the mean distances.

The subjective data provided by the subjects were also analyzed. An average rating was computed for each treatment, with the lowest rating corresponding to the most effective treatment. In addition, based on the subject ranking of the five treatments, a ranking score for each treatment was computed. One point was assigned each time a treatment was ranked first (best), two points for second place rankings, etc. with five points being assigned each time a treatment was ranked fifth (worst). Thus, the treatment perceived to be best would have the lowest score.

RESULTS

The results of the analysis on the mean shape recognition distance, mean background color recognition distance, and mean text legibility distance data are summarized below. Any significant differences that were found between the treatments were reported. Trends found in the subjective data were also reported.

Shape Recognition

As shown in Figure 4, the stop paddle with a full string of flashing red border lights (Treatment 3b) was found to have the greatest mean shape recognition distance (585 ft); however, the difference was not statistically significant from the other treatments, with the exception of the stop paddle with steady-burning white LEDs forming the legend (Treatment 5) (365 ft). The stop paddle with a full string of steady-burning red border lights (Treatment 3a) (518 ft) was also found to have a mean shape recognition distance that was significantly greater than that of Treatment 5 (365 ft).

An examination of the subjective data for Treatment 3b revealed that 92 percent of subjects thought the flashing red border lights made the sign easier to recognize. Of the subjects who thought that the lights helped them recognize the sign, 73 percent said it was specifically due to the flashing of the lights. The subjective data from Treatment 5 showed that 61 percent of subjects thought that the white LEDs forming the legend made the sign harder to recognize, with 14 percent of subjects specifically stating that the lights made it harder to identify the shape.

Background Color Recognition

As shown in Figure 4, the stop paddle with a full string of steady-burning red border lights (Treatment 3a) was found to have the greatest mean background color recognition distance (848 ft). The mean color recognition distances of the standard stop paddle as well as both of the stop paddles containing a full string of LEDs around the border (Treatments 1, 3a and 3b) (804 ft, 848 ft and 797 ft, respectively) were all found to be significantly greater than those of the stop paddle with the flashing red vertically centered LEDs and the stop paddle with steady-burning white LEDs forming the legend (Treatments 4 and 5) (595 ft and 496 ft, respectively).

The subjective data from Treatment 4 showed that 36 percent of subjects thought the sign was harder to recognize because of the flashing lights above and below the stop legend. Of the subjects who thought the lights made the sign harder to recognize, most thought that the flashing was distracting, while some stated that the lights looked yellow. As mentioned previously, the subjective data from Treatment 5 showed that 61 percent of subjects thought that the white LEDs forming the legend made the sign harder to recognize, with 28 percent of subjects specifically stating that the lights made it harder to distinguish the red background color.

Text Legibility

Treatment 5 was anticipated to have the greatest legibility distance because of the white LEDs forming the stop legend; however, there were no significant differences found between any of the treatments with regard to mean text legibility distance. Figure 4 shows that Treatment 5 was found to have the greatest mean text legibility distance (404 ft), although only by a small margin. In the subjective questioning, 42 percent of subjects stated that Treatment 5 was harder to read because of the white LEDs forming the stop legend. Of the subjects who thought the lights made the sign harder to read, some thought that the lights were distracting and others thought that the glow from the lights made it harder to read.

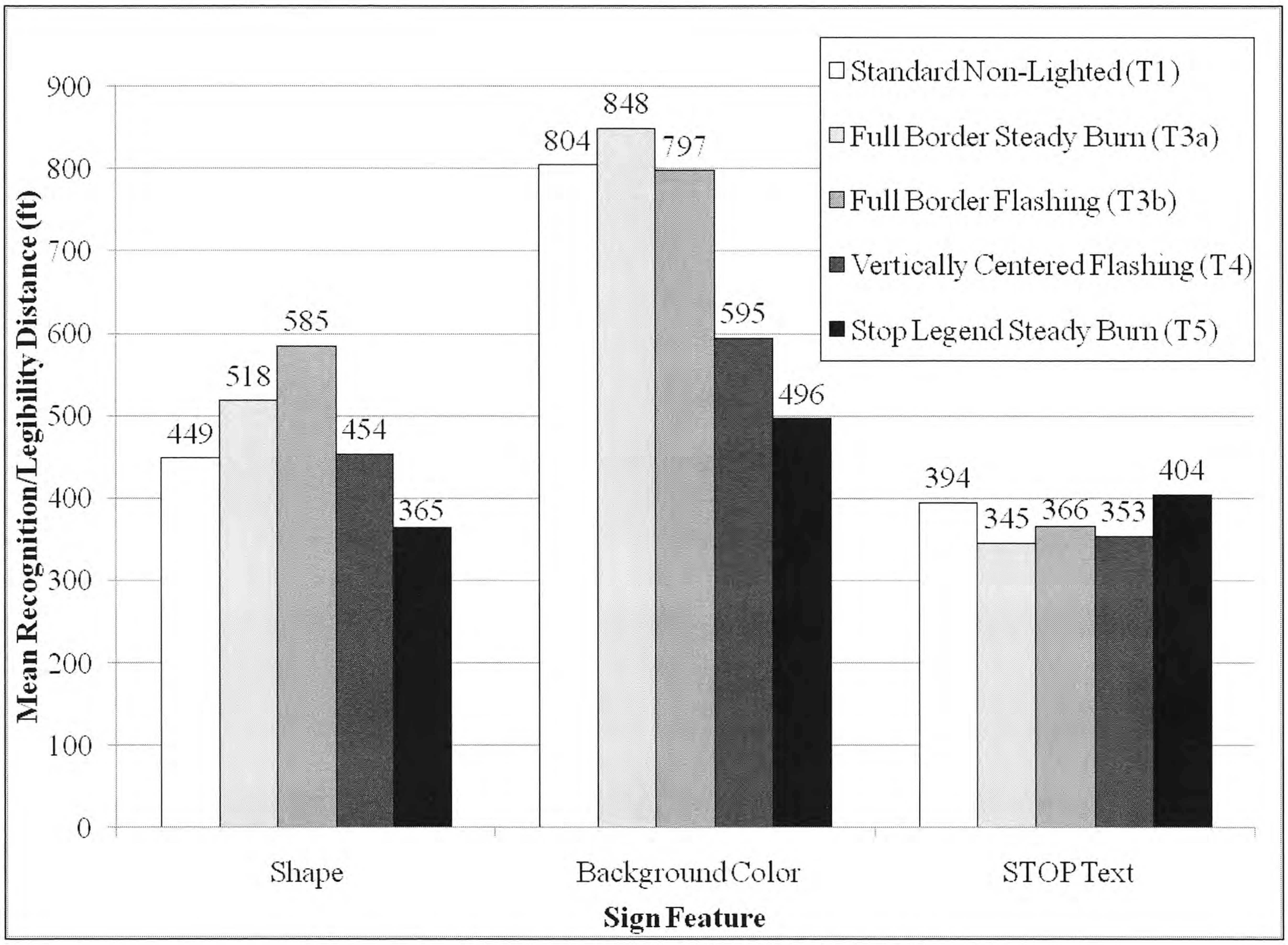


Figure 4. Comparison of Mean Distance Data.

Standard Treatment

The standard treatment in this experiment was Treatment 1, a typical non-lighted stop paddle. With regard to shape recognition, Treatment 1 had a smaller mean shape recognition distance (449 ft) than all other treatments with the exception of the stop paddle with white LEDs forming the letters in the legend (Treatment 5) (365 ft), although it was not significantly different from any of the other treatments. Treatment 1 was found to have a mean color recognition distance (804 ft) that was significantly greater than those of the stop paddle with red flashing vertically centered LEDs above and below the stop legend (Treatment 4) and Treatment 5 (595 ft and 496 ft, respectively). Although there were no significant differences found in the mean text legibility distances of any of the treatments, Treatment 1 was found to have the second greatest mean text legibility distance (394 ft).

Stopping Sight Distance

Typically, in Texas the maximum speed limit in a school zone on an urban road is 30 mph. According to the *Policy on Geometric Design of Highways and Streets*, the stopping sight distance required for a vehicle traveling at 30 mph is 200 ft (*8*). While several significant differences were found among the treatments, the mean recognition and legibility distances for all treatments were found to be greater than the required stopping sight distance. Although a small percentage of subjects did have recognition or legibility distances that were below the required stopping sight distance, a test of proportions between all treatments determined that there were no significant differences.

Additional Subjective Data

As mentioned previously, the treatments were rated by the subjects according to their overall ability to recognize and read each sign. The lowest rating corresponded with the highest preference. The mean rating for each treatment is shown in Table 2. The stop paddles with a full string of red border lights (Treatments 3a and 3b) were found to be rated the best.

Also, the subjects were asked to rank the five treatments in order of their preference. Based on the subject ranking of the five treatments, a ranking score for each treatment was computed by assigning points for each ranking place. Again, the treatment perceived to be best would have the lowest score. Table 2 shows the ranking scores for each treatment. The stop paddles with a full string of red border lights (Treatments 3a and 3b) were also found to have the best ranking score.

Overall, the majority of subjects stated that the stop paddle with the full string of flashing red border lights was helpful in getting their attention and helping them recognize the sign. Most of the subjects thought that the stop paddle with white lights forming the stop legend made it easier to read but harder to recognize the shape and background color of the sign. Also, while most subjects thought that the stop paddle with flashing red lights above and below the stop legend helped get their attention, the majority of subjects thought that these lights were distracting and made it harder to read the sign.

Table 2. Subjective Data.

Treatment	Mean Rating	Ranking Score
1	1.50	141
3a	1.33	88
3b	1.47	70
4	1.64	124
5	2.08	117

CONCLUSIONS

Overall, there were no significant differences found in the recognition or legibility distances when comparing the LED enhanced stop paddles to the standard, non-lighted stop paddle. The stop paddles with a full string of red LEDs forming the border of the sign (Treatments 3a and 3b) were found to have greater mean shape recognition distances and significantly greater mean background color recognition distances than those of the other LED enhanced stop paddles. Although the stop paddle with white LEDs forming the letters of the legend (Treatment 5) had the greatest mean text legibility distance, there were no significant differences in the mean text legibility distances between any of the treatments. The LED enhanced stop paddles with a full string of LEDs forming the border of the sign received the most positive feedback from the subjects.

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APPENDIX

Table 3. Shape Recognition Distance Data.

Treatment	N	Mean (ft)	Std Dev (ft)	Min (ft)	Max (ft)
1	36	449	155.28	157	804
3a	36	518	279.73	151	1215
3b	35	585	307.67	184	1313
4	36	454	194.78	161	1184
5	35	365	137.34	128	686

Table 4. Background Color Recognition Distance Data.

Treatment	N	Mean (ft)	Std Dev (ft)	Min (ft)	Max (ft)
1	35	804	270.40	330	1342
3a	30	848	352.08	169	1469
3b	32	797	331.14	256	1417
4	35	595	240.09	182	1184
5	35	496	211.15	20	1134

Table 5. Text Legibility Distance Data.

Treatment	N	Mean (ft)	Std Dev (ft)	Min (ft)	Max (ft)
1	36	394	105.00	161	714
3a	36	345	107.81	169	556
3b	36	366	116.56	184	738
4	36	353	104.65	146	577
5	35	404	138.70	118	763

Table 6. Subjective Data.

Treatment	Helped Recognition	Impaired Recognition	Helped Legibility	Impaired Legibility
1	81%	33%	44%	14%
3a	97%	19%	58%	17%
3b	97%	36%	50%	31%
4	92%	36%	25%	53%
5	64%	75%	78%	42%

Incorporating Freight Value into the *Urban Mobility Report*

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Undergraduate Transportation Scholars Program

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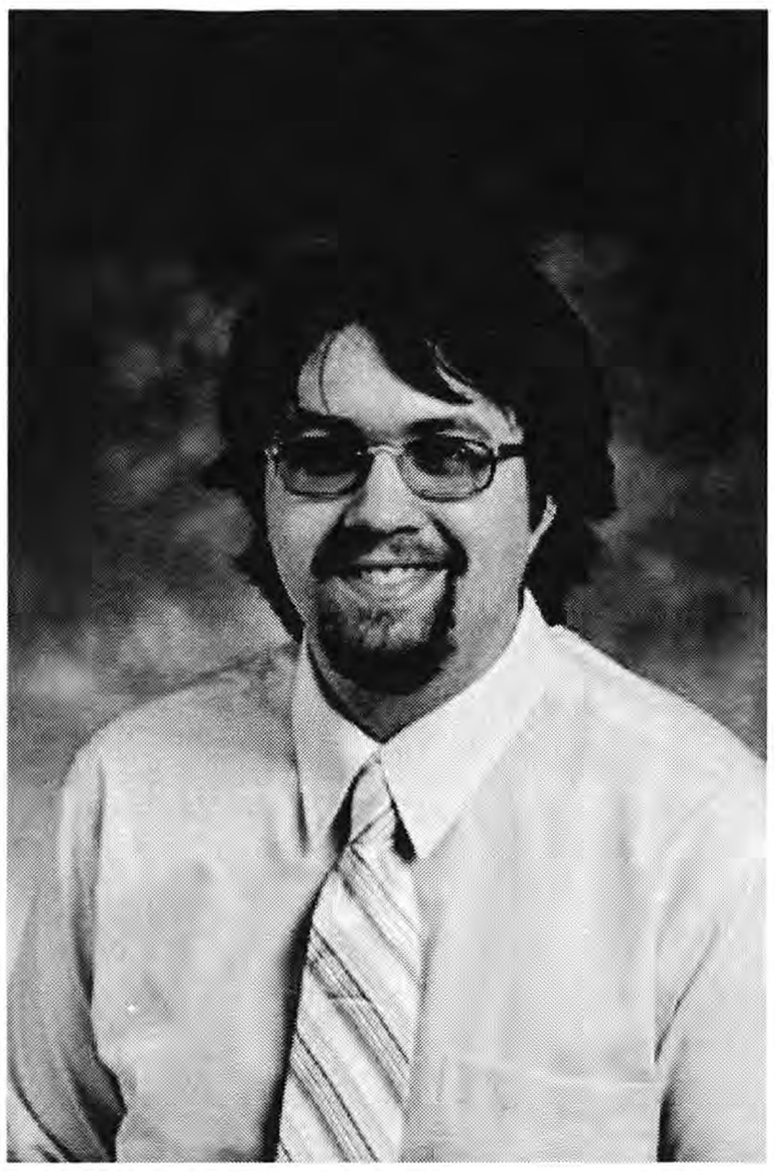

STUDENT BIOGRAPHY

Gregory (Greg) Larson is a senior at the University of Utah where he is pursuing a Bachelor of Science Degree in Civil and Environmental Engineering. He is expected to graduate in May of 2011. Greg has been on the Dean's list at Utah for multiple semesters, while achieving a GPA of 3.5 or higher.

Greg is a member of the University of Utah Chapter of the American Society of Civil Engineers. He has spent three previous summers as an intern for the construction company Watts Constructors, LLC, located in Gig Harbor, Washington. There he completed multiple estimation tasks, such as quantity take-offs and using the AgTEK software to estimate earth moving volumes. Greg also managed the subcontractor database for the company during his employment. During the 2009–10 school year Greg was an undergraduate research assistant for Dr. R.J. Porter at the University of Utah, with whom he studied the correlation of vehicle speed and roadway safety. Greg and Dr. Porter studied multiple roadway design theories to determine the extent of this connection. They also studied the validity of implementing "self-explaining, self-enforcing" roadway design in the United States. Upon graduating with an undergraduate degree, Greg plans to attend a graduate transportation program. Future career goals include employment in the research field of transportation engineering. He intends to focus on geometric design or mobility, as he feels these are becoming major concerns facing U.S. transportation.

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The author would like to express great appreciation to Dr. David Schrank and Dr. William Eisele for their help in teaching the author basic analysis concepts that were applied to the FAF and HPMS datasets. He would also like to thank these two for assisting him in the development and review of the method presented in this research report.

The author would also like to express his gratitude to Ms. Sherie Ruter for her assistance in completing the mileage matrix used in the analysis portion of this research.

SUMMARY

The goal of this research was to develop a method used to calculate the value of freight transported by tractor-trailer through U.S. states and urban areas. The author used data provided by the Freight Analysis Framework (FAF) and the Highway Performance Monitoring System (HPMS). The basis of the research was the integration of commodity tonnage and value supplied by FAF and the truck vehicle-miles of travel percentages calculated using HPMS roadway inventory and characteristic data. It was determined that a value adjustment method provided justifiable estimates of freight value. This method was developed after examining the previously calculated commodity values for both Austin, Texas, and Denver, Colorado, from prior Texas Transportation Institute research.

The developed method, or adjustment factor value method, is a four-step process used to estimate the value of freight commodities. The steps are as follows:

1. Calculate the HPMS truck VMT percents and the national commodity value.
2. Estimate the state and urban commodity values using the HPMS truck percents.
3. Calculate the adjustment factor based on known Austin and Denver commodity values.
4. Estimate the adjusted truck commodity values through all HPMS regions.

The adjustment factor method produced freight values and rankings expected for the states and urban areas of interest, considering population densities and roadway networks. The most valued city was New York, New York, at an estimated $702.4 billion dollars based on 2008 data. The highest value state was Texas with $3.3 trillion of freight. Upon comparing these values to those throughout the nation, the adjustment factor method was found to be a viable process to estimate urban freight values for commodities transported via tractor-trailer.

The adjustment factor method produced satisfactory results. However, further investigation is required. Future research involving the finalized method would determine an adjustment factor based on more cities. This analysis will lessen the amount of assumptions included in the adjustment factor method. The procedure developed during this research will serve as an initial evaluation of the value of commodities traveling through urban areas included in the *Urban Mobility Report*.

TABLE OF CONTENTS

Student Biography 124
Acknowledgments 124
Summary 125
Table of Contents 126
Introduction 127
Background Information 128
Adjustment Factor Methodology for Estimating Truck Freight Value 129
  Methodology Flowchart 129
  Step 1: Calculate the HPMS Truck VMT Percents and the National Commodity Value 129
  Step 2: Estimate State and Urban Commodity Values Using the HPMS Truck Percents 131
  Step 3: Calculate the Adjustment Factor Based on Known Austin and Denver Commodity Values 131
  Step 4: Estimate the Adjusted Truck Commodity Values through All HPMS Regions 132
Research Findings 132
  Adjustment Factor Method (Finalized Method) Discussion and Analysis 133
  Local Commodities Method (Alternate Method) Discussion 138
Conclusion and Recommendations 139
References 139
Appendix 141

INTRODUCTION

Urban congestion is a common concern of commuters in the United States. To analyze the full effects of congestion on travel time and the economy at the urban level, the Texas Transportation Institute (TTI) has published the *Urban Mobility Report* (UMR) since 1982. The UMR examines the effects of urban congestion on passenger cars in terms of time and cost (*1*). The findings included in the UMR identify that congestion is becoming a major issue nationally and is not limited to a few areas. In 2007, Americans traveled an estimated 4.2 billion more hours and used 2.8 billion more gallons of gasoline because of congestion. The additional delay and fuel consumption cost approximately $87.2 billion, based on congestion prices alone (*1*). This number is calculated using the value of time and fuel only. The value of delay on freight commodities in trucks is not currently reported in the UMR (*1*).

The trucking industry is vulnerable to the same extraneous costs that passenger cars encounter during a commute. As an example, it has been reported that a 15 minute delay for a passenger car will result in a $4 cost, based on fuel and time. The same 15 minutes for a freight truck produces a loss of $26, again calculated considering only fuel consumption and lost time (*2*). The difference in costs does not include money spent for late delivery or, in some cases, lost cargo (e.g., spoiled produce). External circumstances will cause the lost revenue to increase, placing a greater strain on the trucking industry. The lost time and money incurred by trucks is determined in the UMR. However, it is only incorporated into the total cost of congestion. The only contrast made between the tractor-trailers and passenger cars is that trucks take the place of 2 to 3 standard vehicles. This lack of distinction between passenger cars and tractor-trailers may cause different estimates of the true economic effects of congestion on the trucking industry.

The economy of the United States is not only dependent on the production and sale of goods, but also the movement of these products. Nearly 79 percent of items transported through the U.S. are moved using trucks at some point during shipment (*3*). Knowing the effects of congestion on the freight industry will help the U.S. understand how to improve the overall shipment of these products and minimize the impact placed on costs to the industry. Congestion must be handled differently by the trucking industry due to delivery of goods becoming a just-in-time service. Items transported via tractor-trailer are dependent on this on-time delivery. Proper use of the just-in-time delivery concept is expected to reduce the overall cost to the trucking industry. This is predicted due to less time required to move and wait on the road. However, to effectively utilize this concept, the trucking industry must understand how to better anticipate and deal with congestion on the nation's roadways. Understanding the value of commodities on U.S. roads is the first step to comprehending the effects of congestion on the movement of freight.

The value of freight transported through urban areas is important. Determining these values will allow users of the UMR to better understand how congestion is affecting goods' movement in urban areas. This report describes a methodology that will allow researchers to provide estimates of commodity values and begins to find the effect of congestion on the goods' movement.

BACKGROUND INFORMATION

The research sponsor for this project is an FHWA Pooled Fund Research Project conducted by the Texas Transportation Institute. The research in this report is part of two larger parent projects: the annual *Urban Mobility Report* (UMR) and *Mobility Measures for Urban Transportation* (MMUT). The UMR is sponsored by the University Transportation Center for Mobility-Texas A&M University, American Road & Transportation Builders Association-Transportation Builders Foundation, and the American Public Transportation Association. The MMUT is sponsored by 12 state Departments of Transportation (DOTs), 2 Metropolitan Planning Organizations (MPOs), and the FHWA. The purposes of these projects are to analyze the effects of congestion, measure mobility, and monitor mobility in urban areas throughout the United States.

Researchers at TTI previously developed a method using FAF and HPMS information to estimate the value and delay of freight in urban regions, called the freight box concept. This analysis method is accepted based on feedback from outside sources regarding the findings of this method. Also, there is a lack of alternate methods. This freight box was tested using the Austin, Texas, and Denver, Colorado, metropolitan areas. The procedure takes approximately 7-10 days to complete for each of the analyzed regions. It is a useful tool at the metropolitan area level when more detailed information and results are desired. However, this detailed method is not practical for the 101 cities included in the UMR. A flow chart representing the previous method of linking FAF and HPMS data is provided as Figure 1. After reviewing the need for a less labor-intensive and stream-lined estimation procedure, the author researched and developed the method discussed in this report.

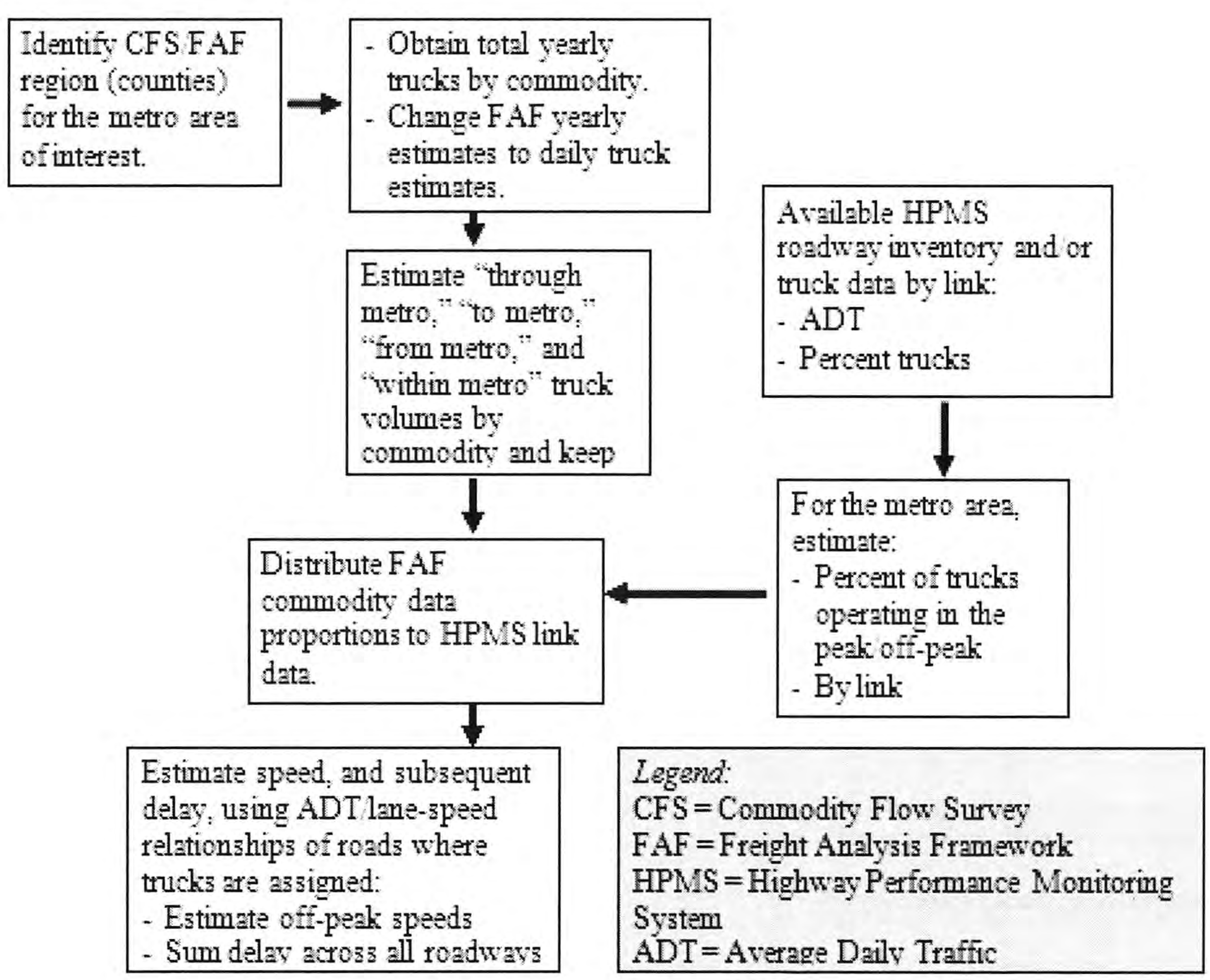

Figure 1. Flowchart of Previous Method (Adapted from Reference *2*).

TTI mobility analysts are studying the effect of congestion on the trucking industry in the U.S. The analysis reviews the relationship of congestion and the on-time delivery of goods, along with approaches to better estimate the travel time of freight through urban regions. Specifically, the information presented in this report discusses the developed method and the findings for estimating the value of freight traveling on freeways and principle arterials through the UMR urban areas.

ADJUSTMENT FACTOR METHODOLOGY FOR ESTIMATING TRUCK FREIGHT VALUE

The adjustment factor methodology developed through this research is described in the following paragraphs, along with the reasoning to support the presented process. This section describes the four-step process of integrating FAF and HPMS data to estimate the value of truck freight. The methodology can be applied to all regions included in the HPMS database and summarized specifically for those regions included in the UMR. The method proposed in this section provides an initial analysis tool to evaluate the value of urban areas in the U.S. and the true economic impacts of urban congestion on the trucking industry.

The four steps of the methodology are as follows:

1. Calculate the HPMS truck VMT percents and the national commodity value.
2. Estimate the state and urban commodity values using the HPMS truck percents.
3. Calculate the adjustment factor based on known Austin and Denver commodity values.
4. Estimate the adjusted truck commodity values through all HPMS regions.

Methodology Flowchart

Figure 2 visually represents the method developed to estimate the value of freight traveling through urban regions. Figure 2 displays the connections between the steps and data of the adjustment factor method.

Step 1: Calculate the HPMS Truck VMT Percents and the National Commodity Value

The objective of the first step is to determine HPMS truck VMT percents, for both the state and urban regions, and the national value of commodities transported by truck. Highway data systems are used to analyze the value of freight transportation. The FAF system contains freight commodity movement information. The FAF dataset is based on the commodity flow through 114 regions of the country, including dollars and tons of goods moved. The division of these regions is based on the Commodity Flow Survey (*4*). The HPMS dataset is a second example of a database used in the freight value estimate methodology. The HPMS system contains inventory data for the U.S. roadway network (*5*). It provides information such as average daily traffic (ADT) and percent trucks by functional classification. The truck percentage is based on national and state vehicle-miles of travel (VMT). Full analysis of the freight value is dependent on the data provided by these two systems. Linking these datasets will allow further evaluation of the total cost of commodity flow through the country.

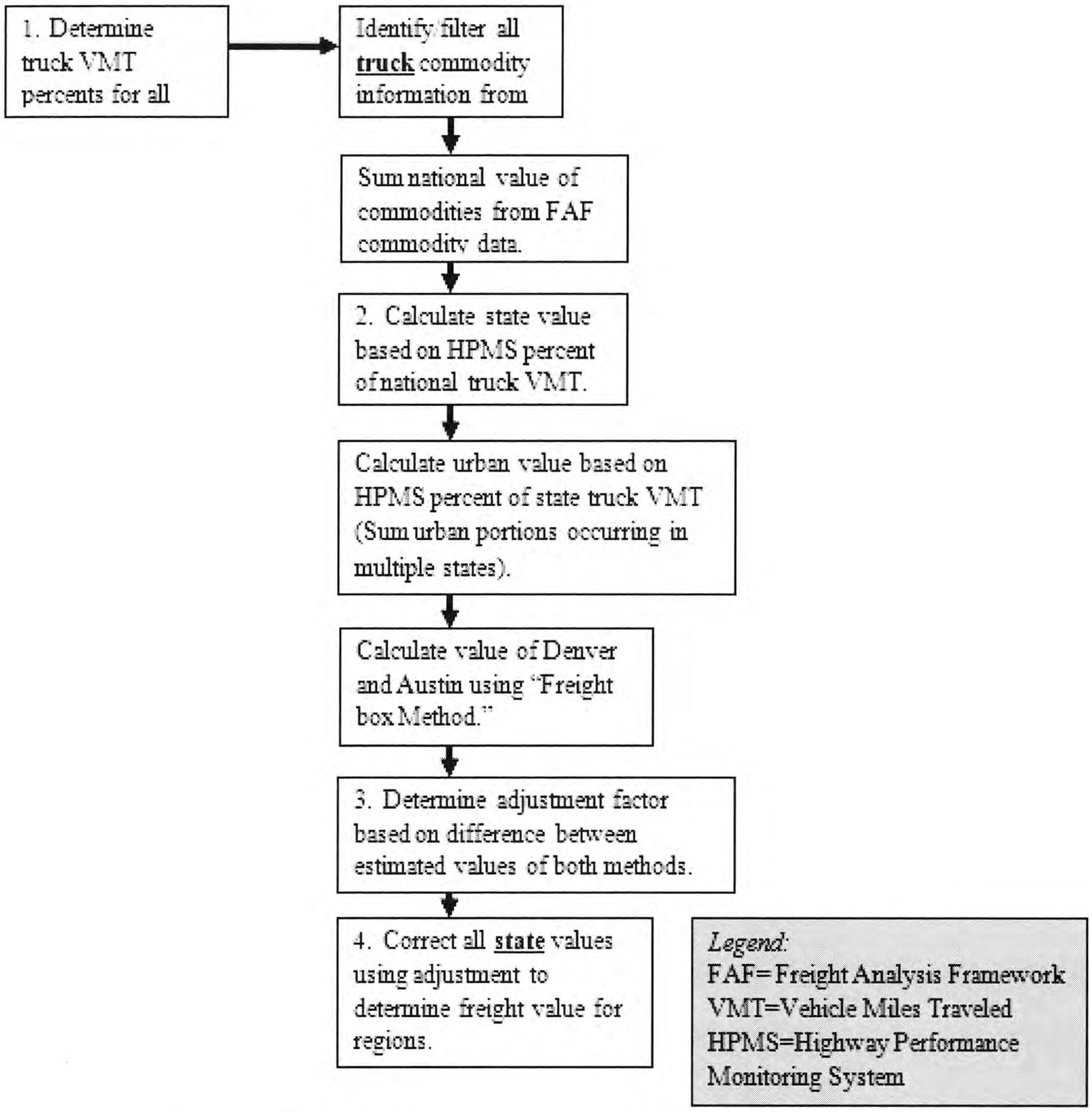

Figure 2. Flowchart of Adjustment Factor Method.

Researchers have developed a two-part method to complete this step of calculating the truck VMT percents and the national commodity value. First, the HPMS state and urban percents of truck VMT are calculated based on the national amount of truck VMT, as shown for states in Equation 1.

$$\text{State Truck VMT Percentage} = \frac{\text{State Truck VMT}}{\text{National Truck VMT}} \tag{1}$$

Equation 1 uses the estimated national truck VMT and the estimated state truck VMTs to calculate the state percents. This HPMS percentage will be used to approximate total commodity value at the state level.

The urban percents are calculated similarly, but with respect to the state mileage. The equations used for the urban percentages are given as Equation 2.

$$\text{Urban Truck VMT Percentage} = \frac{\text{Urban Truck VMT}}{\text{State Truck VMT}} \tag{2}$$

Secondly, the FAF data are provided in Microsoft Excel® format. All modes of freight traveling between city pairs are included in the file. Using the SAS program, researchers keep only truck data and sum the commodity values accordingly. Also, using basic Excel sorting functions (i.e., filter and sort), the data are arranged to distinguish only truck commodities from all other modes. All truck commodity values are summed to estimate the national truck value. Summing the value in Excel is more commonly used as a check of the SAS output. The mathematical programming is preferred because the same program may be used over several years of service, while using Excel must be done annually and is more time consuming compared to an already programmed SAS output. The total commodity value provides a national commodity value used in further steps of the developed method and is the number used to produce the state and urban values of truck freight.

Upon calculating the state/urban truck VMT percents and the national commodity value, researchers and analysts can use these numbers to evaluate the commodity value at the state and urban level. Integration of the data calculated in this step provides the desired freight values.

Step 2: Estimate State and Urban Commodity Values Using the HPMS Truck Percents

This step begins with the output from Step 1. The initial state commodity values are approximated using the national commodity value and the state percents of the truck VMT. It is assumed that the VMT percent corresponds directly with the percent of the national commodity value. This step is used to estimate the base value of commodities traveling through or within each state as explained by Equation 3.

$$\text{State Value (Dollars)} = (\text{National Commodity Value} \times \text{State Truck VMT Percentage}) \quad (3)$$

The urban commodity values are intended for presentation in the UMR. Using similar theory to the state value calculation, the urban values are determined. The state values estimated in Equation 3 are used to estimate the urban commodity value based on HPMS percents. Only the state containing the particular urban area is used at this time. This is presented as Equation 4.

$$\text{Urban Value (Dollars)} = (\text{State Value} \times \text{Urban Truck VMT Percentage}) \quad (4)$$

The urban values estimated in this step provide a general assessment of the urban commodity value. This approximation is based on an average commodity value, and therefore must be adjusted to account for values differing by commodity.

Step 3: Calculate the Adjustment Factor Based on Known Austin and Denver Commodity Values

This step calculates the factor used to adjust the value of commodities for states found in Step 2. The freight box concept previously developed by TTI (shown in Figure 1) estimated the value of freight based on roadway links within the metropolitan areas of Austin, Texas, and Denver, Colorado. The adjustment factor is the comparison between known commodity values of Austin, Texas, and Denver, Colorado, and the values estimated in Step 2 for these two cities. Equation 5 presents this step (Ex. Austin, Texas).

$$\text{Adjustment Factor} = \frac{\text{Austin Commodity Value Using Link Method}}{\text{Austin Commodity Value Using HPMS Percentage}} \quad (5)$$

The calculated adjustment factor will be rounded down to the nearest whole number to provide a more conservative estimate of the adjusted values found in the next step.

Step 4: Estimate the Adjusted Truck Commodity Values through All HPMS Regions

This step uses the outputs from Steps 2 and 3. Specifically, this procedure is used to convert the HPMS value for all states, calculated in Step 2, to an adjusted value of freight commodity. The Step 2 value is altered using the rounded adjustment factor calculated in Step 3. This information will grow the state and urban values of commodities to an accepted estimation of freight commodity value assuming single trucks are counted more than once in the national truck VMT. To estimate this adjusted value, the products of these steps are multiplied together as in Equation 6.

$$\text{Adjusted Value} = \text{Adjustment Factor} \times \text{HPMS State Value} \quad (6)$$

The same adjustment factor is used for all states. This assumption is based on available data and should be noted for continued use of the procedure. The urban values are not directly adjusted using the adjustment factor. The urban values are altered with the change in state value. The re-evaluation of all values is necessary because freight trucks affect the urban values of multiple cities. The freight value estimated in Step 2 assumes a truck will only influence the value of one urban area. The estimated adjustment factor will consider the effects trucks have on commodity values in multiple cities and adapt the values through all UMR urban regions.

Given these adjusted values the desired connection between the FAF and HPMS databases is created. The data produced will be an estimate of the commodity value through the urban areas in question. Upon finalizing these values, a cost of congestion on the trucking industry can be estimated for inclusion in future publications of the UMR.

RESEARCH FINDINGS

The adjustment factor method is a viable procedure to estimate U.S. urban freight values for the UMR. This was determined after a comparison of the approaches researched to estimate the necessary freight values. Based on the datasets and collection methods currently available, this previously described adjustment factor method, or finalized method, produces defensible values of commodities transported via tractor-trailer. Although this research did not consider a count of trucks, the explanations quantify the findings based on a percentage of trucks and the corresponding values.

The following section discusses two primary discoveries found during the research and development of the adjustment factor method:

- First, this section provides details regarding the reasoning for utilizing the adjustment factor method and the values produced. Interpretations of the overall results will be given as well. The results produced by the adjustment factor method are based on the 2008 HPMS truck VMT percents and the 2008 national commodity value.

- Second, this section discusses the details regarding an alternate method. The alternate method incorporates local commodities for each analyzed region. This other procedure was not used to produce any published values, but explains the further reasoning for the adjustment factor method.

Adjustment Factor Method (Finalized Method) Discussion and Analysis

The adjustment factor method used to estimate truck freight values was developed to create a link between the FAF (freight commodities) and HPMS (freight travel) databases. It also established the relationship between a simple, stream-lined estimation process (shown as Step 2: Estimate State and Urban Commodity Values Using the HPMS Truck Percents) and the freight box methodology. The connection between these two estimation methods ensures estimates of freight values included in the UMR are a representative value for each region. This model value is anticipated because the adjusted factor method is based on the accepted freight box estimates.

The HPMS based "average commodity value" method (Step 2: Estimate State and Urban Commodity Values Using the HPMS Truck Percents) was initially researched as a final method of analysis. This estimation process became the foundation from which to build the finalized adjustment factor method. The average commodity value methodology used the truck VMT provided by the HPMS database. The author made an assumption to link the provided truck VMT measures and national commodity value. It was assumed that the state or city percent of truck VMT equals the percent of the national or state commodity value assigned to each state and city. This pathway is based on the possible ways to link the HPMS VMT and FAF value datasets. The simple procedure produced values that provide a general idea of the freight value through each state and urban region.
The average commodity value method assigns each particular truck to one spot on the roadway network. The spot assignment implies each truck has an effect on only one city. The author understands the single effect assumption is not accurate for a UMR analysis. This single effect treatment will result in each state and city receiving a freight value based on an average commodity value. The commodity values of all trucks affect multiple cities in the U.S. An adjustment factor is required to increase the freight values calculated by the average commodity value method to achieve the values calculated in the freight box method. Value estimates similar to those calculated using the freight box method are desirable for inclusion in the UMR. Although the average commodity value method produced values that can be used to represent certain growth and demand trends, it was different when compared to the accepted freight box method.

The freight box method incorporates all truck freight movements in the U.S. It determines which trucks may travel through a given city (e.g., Austin), as opposed to only considering the originating and destination city for the measured freight. Thus, each truck causes an effect on freight values in multiple urban areas. This provides a broader representation of the truck travel through the U.S., captured by the national truck VMT. This economic effect is incorporated in the freight box method by approximating the number of trucks traveling through each city, increasing the number of times a truck affects urban values. The resulting freight values generated for the trucks in this methodology show the value of the system freight value on a particular city, as opposed to an average value.

The freight box and average commodity value methodologies produced very different results when compared for both the cities of Austin, Texas, and Denver, Colorado. For example the freight box method produced a total freight value for the Austin roadways of $78.9 billion in 2010, while the unadjusted HPMS method produced a total freight value of about $24 billion. The author understands that the average commodity value is based on 2008 percents and national commodity value. The 2008 data were the information available at the time of analysis. The author understands that the U.S. Gross Domestic Product (GDP) was estimated to grow about 0.09 percent between 2008 and 2010 and about 1 percent between 2009 and 2010 (*6*). Therefore, the large commodity value difference of over $50 billion is not due purely to a difference in the years of the data collected. Instead the disparity is primarily caused by how the different methods analyze the urban area in question.

A comparison of the values estimated for Austin and Denver that the freight box method produced an area-wide freight value that was approximately 3.3 times greater than the freight value calculated by the average commodity value method for each city. However, in order to err on the conservative side until further research proves or disproves the relationship, a factor of 3.0 is used to convert all results of the average commodity value methodology to adjustment factor method values that can be provided in the UMR. More research is needed to ensure that this relationship is standard across the country and in cities of all sizes. Until more analysis is done to compare the freight box method to the adjusted value method, all states will be assumed to require the same adjustment factor.

Each of the resulting values from the freight box and average commodity value methods is accurate depending on the questions being answered. A procedure similar to the average commodity value method was needed for the UMR due to ease of calculation and lower time requirement compared to the freight box method. The average commodity values were weighted to reflect the true commodity appraisal traveling through each city, such as $71.9 billion dollars using the adjustment factor method versus $24 billion using the average commodity value method. This was done to convey the specific regional values necessary for the information conveyed to the public and policymakers by the UMR. The finalized method value provides an overall idea of the commodity value traveling through Austin. General, or average, freight value estimates do not convey the impact of congestion on the goods' movement necessary for the UMR.

The adjustment factor value method was analyzed using an interpolated national truck commodity value for the year 2008. The national commodity values provided by FAF were reported for both 2002 and 2010. Table 1 provides the values and years given for this freight value analysis.

Table 1. Total Truck Commodity Value Report by FAF.

Year	Total Truck Commodity Value (Trillion Dollars)
2010	$11.7
2002	$9.6
2008*	$11.2

*Estimated using linear interpolation

The total commodity value for the year 2008 is a sum of all freight transported via truck through the U.S. The $11.2 trillion dollars accounts for about 79 percent of the total commodity value moved using all modes of freight transportation. This large percentage of the U.S. freight value makes the analysis and findings of this procedure crucial to assist in determining the effects of congestion on the trucking industry.

The goal of this research was to develop a method used to estimate the values of freight traveling through U.S. state and urban regions. Exact values for urban regions are not currently known. However, based on geographic location of the urban areas and the reported population densities, assumptions can be made as to how the different cities will be ranked for freight commodity value. Final analysis of the UMR cities provided urban rankings expected based on the characteristics of the cities. The top 5 city highest valued of the UMR urban regions are provided in Table 2. Please refer to Table A-1 in the Appendix for a full listing of the UMR cities' values.

Table 2. The Top 5 UMR Urban Regions in Terms of Commodity Values.

Urban Region	Total Commodity Value (Million Dollars)	Port City?	Population Rank
New York, NY	$702,400	Yes	1
Chicago, IL	$654,400	No	3
Los Angeles, CA	$574,200	Yes	2
Dallas-Ft. Worth, TX	$429,800	No	9/17
Houston, TX	$363,800	Yes	4

The values in Table 2 are considered to be defensible because of the method's dependency on the freight box concept already used by TTI researchers. The author believes the values and rankings in Table 2 are also justified based on the information included in the last two columns of the table. Three of the top five cities listed are port cities. Major port cities were expected to have greater truck traffic due to the large amounts of goods entering and exiting the region caused by the maritime trade. Both Chicago and Dallas-Fort Worth are major multimodal hubs. Although Chicago and Dallas-Ft. Worth do not have ports, they do have multiple interstates and airports routed through the metropolitan areas. The non-port based urban areas are a major convergence of interstates and are located near the middle of the east and west coasts. Creating this link between coasts will increase the overall truck traffic through both areas. Easy access to major roadways increases the amount of trucks traveling through the urban region because trucks commonly take major routes between the commodities' origin and destination.

The population rank of the cities is a second verification of the values and rankings estimated by the finalized method. Four of the five listed cities are the top 4 most populated cities in the U.S. (*7*). Higher populated cities need larger amounts of trucks to deliver the goods required to satiate demand and to export the goods produced by the local industry. Again, Dallas-Ft. Worth is a region not in the top 5 most populated cities. However, this urban area is a major economic center for the nation. A large amount of business and trade occurs in the greater metropolitan region. This economic activity will inflate the amount of truck travel through the region.

The adjustment factor method does not only estimate the values of commodities in urban regions. It also estimates freight values for the states. Each of the 50 states, Washington, D.C., and Puerto Rico is separated into inter-regional (i.e., rural) truck travel and urban truck travel. The inter-regional travel is measured outside of any metropolitan areas and the urban travel is the sum of all travel within the listed HPMS urban regions. Based on the previously created link between truck travel and freight value, there is an estimated rural commodity value and urban commodity value. Further assumptions were made about the separation of rural and urban truck commodity value. The finalized method also calculated the state values and ranked them in terms of highest to lowest total commodity value. The same expectations regarding the city values were made for the state values and rankings. Similar to the urban rankings, this analysis provided an order that is expected considering the number of roadways and the types of industries based in the states. The 10 highest freight value states are listed in Table 3 in order from highest to lowest commodity value. Please refer to Table A- 2 in the Appendix for a full listing of U.S. states and territories.

Table 3. Top 10 U.S. States in Terms of Commodity Value.

State	Urban Commodity Value (Million Dollars)	Inter-Regional* Commodity Value (Million Dollars)	Total Commodity Value (Million Dollars)
Texas	$1,383,400	$1,868,100	$3,251,500
California	$1,696,000	$1,184,600	$2,880,600
Illinois	$730,000	$626,500	$1,356,500
Pennsylvania	$566,400	$638,600	$1,205,000
Ohio	$523,600	$678,000	$1,201,600
Wisconsin	$442,400	$715,400	$1,157,800
Indiana	$432,200	$712,200	$1,144,400
Georgia	$484,000	$619,800	$1,103,800
North Carolina	$481,700	$548,000	$1,029,800
New York	$626,500	$373,200	$999,800

*Inter-Regional travel is rural travel

All states listed in Table 3 are states that have a high level of industry and/or high urban concentration. The ranking of the different states is support for use of the adjustment factor method. The top two most valuable states, Texas and California, are the second and third largest states in the nation. The only state larger is Alaska. However, Alaska does not have the population compared to Texas and California. A heavier population will necessitate a larger amount of trucks to feed and please the people of the area. The estimated freight value is directly proportional to the amount of trucks, creating the ranking of Texas and California being the top two states in terms of freight value. All other states on the list have both a large land area and population, comparatively. This link between land size, population density, and freight value provides the necessary expectations for the findings of the adjustment factor method.

Table 3 provides insight into how the method accurately divides the rural and urban freight values for urbanized states. Considering the concentration of population and roadways in the urban sector of California (e.g., Los Angeles and San Francisco-Oakland), it is expected that the urban commodity value would be greater than the rural freight value. According to the FHWA, in 2008 California had 83.5 thousand miles of road in land classified as rural. This value is compared to 89 thousand miles in urban land (*8*). Also, California had 173.4 thousand rural lanes-miles and 213.2 thousand urban lane-miles (*9*). Both roadway measurements support the expectation of a greater amount California truck travel occurring in the urban areas. States such as California have a majority of the truck deliveries travel in and out of the urban centers. The adjustment factor method interprets the disparity between the urban and rural commodity value.

The adjustment factor method also correctly analyzes states with large rural areas. States such as Texas have multiple large urban centers. However, due to the overall area of Texas and the number of interstate highways located in the state, a majority of their roadways route through rural land. In 2008, the FHWA reported that Texas had 213 thousand miles of rural roadway and 93.4 thousand miles of urban roadway (*8*). Texas also measured at 440.7 thousand lane-miles in rural land, compared to 214.2 thousand urban lane-miles (*9*). The trend shown in Texas is the opposite as California. The finalized method estimates that Texas has a majority of the truck value traveling through the rural portion of this state. The freight value finding for Texas support the roadway measurements and support the adjustment factor estimation method.

A secondary objective completed through this research was the creation of a format to calculate values that is easy to follow and is distributable to third parties. To easily distribute the freight value findings, a programmed Excel spreadsheet was created. The spreadsheet developed is one that requires only two inputs, the national commodity value and the adjustment factor. The file will then calculate all freight values and present them in four separate summaries. The summaries include:

1. A UMR urban summary (101 cities)
2. State summary (including Washington, D.C., and Puerto Rico)
3. A full urban summary (all U.S. regions classified as urban)
4. A full state/urban summary

The program will calculate the values for each summary based on the yearly percents that are also used in the file. Providing TTI with a simple spreadsheet will not only allow other organizations (i.e., DOTs, MPOs, and private contractors) with a way to determine the commodity value through their respective regions, but also provide an easy to explain method used to estimate all values published in the UMR.

As was stated before, the author does not have exact freight values for the states and urban areas of the U.S. All findings estimated by the adjustment factor method are based on the assumed link between truck VMT percents and the distribution of freight value. The only information used to support the findings of this finalized procedure is external information. There is currently little research and information regarding value estimates of freight traveling through urban regions. The provided method is reported in the UMR because it estimates values that are expected for the different states and urban regions, as well as producing the anticipated trends for the different areas. The adjustment factor method is dependent on the freight box concept, an accepted estimate method. This relationship between the two procedures creates an assurance of the values calculated using the adjustment factor method.

Local Commodities Method (Alternate Method) Discussion

The finalized procedure evolved after multiple attempts to provide defensible estimates of freight values. The author feels one alternative method should be discussed to explain other procedures tested to estimate freight value. An explanation of the alternate method will solidify the finalized procedure as well as the findings reported.

Although the local commodity method was considered to be a good reflection of urban commodity values, it is not viable for use in the UMR analysis. Two dilemmas were encountered during the development and use of this method. The first reason to not use the local commodity method is due to time and data requirements noticed during development. This local commodity method, although less labor-intensive than the previous TTI freight box version, requires a large amount of annual data input. Information such as summing all local commodities and updating truck VMT percentages will need to be updated regularly, as economic effects and transportation improvements affect both the mode and traveled route. Doing so requires time that is not allotted for the UMR analysis and provides only a difference of a few percentage points when compared to the finalized method. This method of incorporating local commodities into the value estimate was expected to produce accurate results. However, it did not produce results that justified the time requirement when compared to the ease of the finalized method.

The second consideration deeming this method unfit for use in the UMR is the final information produced by the consolidation of national commodity values and national miles between all FAF regions. Commodity value data combined with the developed mileage matrix estimates a result with "dollar-mile" units. The researchers do not know what this unit determines and how to properly use it to analyze the value of truck commodities using the rest of the provided information. All attempts to avoid creating this variable also produced unusable information. The integration of the two created units, ton-miles and dollar-miles, causes further development of this process to halt. Due to the lack of research in this area of urban mobility, there are no coefficients available to allow a merger of these units. Also, there are no developed conversions to change the dollar-mile unit into one that can be used in a fashion that produces acceptable numbers. It is therefore determined that while this approach is valid, until a better technique to estimate local and national values is created, this procedure cannot be used for truck value estimates.

The use of local commodities was considered to be the most reflective method in terms of actual value traveling. However, the freight values were not dissimilar to those calculated via the simpler finalized method. The adjustment factor method is the viable method to estimate freight value based on the time and data readily available for the analysis.

CONCLUSION AND RECOMMENDATIONS

As with many transportation issues, the calculation of freight value estimates is impeded due to lack of usable information and accepted techniques. There are currently no nationally accepted methods to estimate the value of freight for all urban areas in the U.S. Cities and other MPOs have developed regional methods to estimate freight value, but these are not used on the national scale for inclusion in the UMR. It is concluded that the method discussed the preceding report is one that provides justifiable freight value estimates. The presented research provides the base to estimate freight value for the UMR. This procedure will assist TTI researchers to better calculate, present, and explain truck freight value estimates used to express the effects of congestion on the trucking industry. The ease of the procedure will assist in explaining exactly how researchers are producing the final conclusions regarding the effects of congestion on freight movement.

The method and products of this research achieve all goals and objectives set forth by the purpose of this research. The technique produces values defensible for states and urban regions included in the UMR. Although there is a lack of in-depth research for procedures used to estimate the value of freight, the values calculated using the developed methodology provides numbers that would be expected. Also, this procedure can be reproduced easily to be applied to all regions requesting access to this information and use of the program design.
Similar to all scientific research, the analysis and development of the finalized method is not complete. There is a need for future work to improve the technique used. The future advancements will increase the accuracy of the values produced using this program. One area of further research is to use the freight box method in other cities. Doing so will confirm the presented adjustment factor or produce a more accurate factor to be used in future UMR analysis. Second, completing the analysis based on commodity type will allow cities and states to see what types of commodities are traveling through the region. This commodity division will also allow policy makers and the public to see what commodities are affected the most by high congestion. Last, further analysis is required as improvements to the quality and quantity of data increase. This will also provide more support for this first estimation version, or assist in providing more accurate estimates. The improved estimates will advance the overall analysis and require less allocation of the commodity values. It has therefore been determined that the method developed and discussed in this report will provide strong value estimates for current UMR publications and create a base from which to improve freight value estimates for future editions.

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APPENDIX

Table A-1. *Urban Mobility Report* Summary of Freight Values.

Urban Region	Commodity Value (Million Dollars)
New York-Newark, NY	$702,354.9
Chicago, IL	$654,445.8
Los Angeles-Long Beach-Santa Ana, CA	$574,170.9
Dallas-Ft. Worth, TX	$429,802.3
Houston, TX	$363,803.9
Philadelphia, PA	$345,611.9
Atlanta, GA	$328,965.4
Miami, FL	$314,059.9
Phoenix, AZ	$293,046.6
St. Louis, MO	$266,493.1
Seattle, WA	$222,174.2
Detroit, MI	$221,478.2
Washington DC	$215,038.1
San Fran-Oak, CA	$201,953.7
Boston, MA	$200,721.9
Milwaukee, WI	$179,452.8
Baltimore, MD	$175,690.1
San Diego, CA	$174,584.8
Indianapolis, IN	$158,370.8
Riverside-San Bernadino, CA	$152,689.7
Nashville-Davidson, TN	$152,688.0
Kansas City, MO-KS	$148,478.7
Tampa-St. Petersburg, FL	$139,672.3
Orlando, FL	$136,265.7
Cincinnati, OH	$134,208.0
Minneapolis-St. Paul, MN	$130,083.8
Denver-Aurora, CO	$127,472.1
Pittsburgh, PA	$116,873.2
Charlotte, NC	$111,045.7
San Antonio, TX	$106,962.9
Memphis, TN	$106,257.1
Cleveland, OH	$101,888.3
Louisville, KY-IN	$100,600.6
Portland, OR-WA	$100,386.9
San Juan, PR	$94,656.3
Sacramento, CA	$93,918.6
Oklahoma City, OK	$89,366.0

Table A-1. *Urban Mobility Report* Summary of Freight Values (Continued).

Urban Region	Commodity Value (Million Dollars)
Columbus, OH	$88,671.6
Salt Lake City, UT	$87,461.7
Tucson, AZ	$85,630.5
Jacksonville, FL	$83,219.4
Birmingham, AL	$82,731.0
Raleigh, NC	$81,050.6
San Jose, CA	$80,378.2
Austin, TX	$71,850.5
Baton Rouge, LA	$70,801.7
Madison, WI	$65,297.7
Richmond, VA	$65,048.6
Jackson, MS	$63,097.9
Dayton, OH	$62,959.3
Albuquerque, NM	$62,060.7
Provo/Orem, UT	$61,340.1
Hartford, CT	$61,283.2
New Orleans, LA	$56,347.9
Tulsa, OK	$56,306.2
Bridgeport-Stamford, CT	$55,249.4
Virginia Beach-Norfolk-Newport News, VA	$54,822.6
Little Rock, AR	$53,451.2
Allen/Beth/Easton, PA-NJ	$52,182.0
Buffalo, NY	$50,878.5
Providence, RI-MA	$50,313.4
Las Vegas, NV	$49,503.3
Poughkeepsie-Newburgh, NY	$45,880.6
Columbus, SC	$45,353.6
El Paso, TX	$45,228.7
Albany-Schenectady, NY	$43,981.4
Bakersfield, CA	$43,502.2
Toledo, OH-MI	$41,722.8
Stockton, CA	$41,197.4
Grand Rapids, MI	$40,978.0
New Haven/Merid, CT	$40,807.0
Knoxville, TN	$40,628.3
Charleston, SC	$38,580.3
Greensboro, NC	$38,311.1

Table A-1. *Urban Mobility Report* Summary of Freight Values (Continued).

Urban Region	Commodity Value (Million Dollars)
Fresno, CA	$38,028.4
Akron, OH	$37,487.2
Oxnard-Ventura, CA	$36,876.0
Worcester, MA	$34,349.5
Springfield, MA	$33,916.0
Beaumont, TX	$33,486.9
Winston-Salem, NC	$32,543.8
McAll-Edin-Miss, TX	$32,511.5
Sarasota-Bradenton, FL	$30,984.2
Rochester, NY	$30,887.0
Spokane, WA	$26,989.9
Pensacola, FL	$25,864.2
Colorado Springs, CO	$25,460.0
Omaha, NE-IA	$25,203.5
Cape Coral, FL	$24,333.2
Indio-Cathedral City-Palm Springs, CA	$21,895.1
Corpus Christi, TX	$18,206.5
Salem, OR	$15,129.6
Boise, ID	$14,403.3
Eugene/Springfield, OR	$14,323.9
Anchorage, AK	$12,838.0
Panama City, FL	$12,328.8
Lancaster/Palmd, CA	$10,947.6
Brownsville, TX	$10,078.6
Laredo, TX	$10,078.6
Hattiesburg, MS	$9,841.0
Boulder, CO	$3,187.9

Table A- 2. Summary of the States' Urban and Inter-Regional Freight Values.

STATE	Urban Commodity Value (Million Dollars)	Inter-Regional Commodity Value (Million Dollars)	Total Commodity Value (Million Dollars)
Texas	$1,383,400	$1,868,100	$3,251,500
California	$1,696,000	$1,184,600	$2,880,600
Illinois	$730,000	$626,500	$1,356,500
Pennsylvania	$566,400	$638,600	$1,205,000
Ohio	$523,600	$678,000	$1,201,600
Wisconsin	$442,400	$715,400	$1,157,800
Indiana	$432,200	$712,200	$1,144,400
Georgia	$484,000	$619,800	$1,103,800
North Carolina	$481,700	$548,100	$1,029,800
New York	$626,500	$373,200	$999,700
Missouri	$387,800	$608,400	$996,200
Tennessee	$354,000	$531,300	$885,300
Arizona	$407,000	$457,800	$864,800
Michigan	$447,100	$374,100	$821,200
Oklahoma	$149,700	$654,700	$804,400
Louisiana	$311,500	$422,200	$733,700
Alabama	$210,200	$500,100	$710,300
Virginia	$275,900	$434,200	$710,100
Kentucky	$171,500	$508,400	$679,900
Washington	$337,000	$342,900	$679,900
Maryland	$354,600	$224,400	$579,000
Mississippi	$127,200	$451,600	$578,800
Arkansas	$104,000	$458,100	$562,100
South Carolina	$184,300	$357,500	$541,800
New Jersey	$480,700	$57,800	$538,500
Utah	$217,300	$290,900	$508,200
New Mexico	$87,200	$404,200	$491,400
Minnesota	$154,900	$302,800	$457,700
Oregon	$125,900	$321,700	$447,600
Colorado	$189,300	$241,500	$430,800
Iowa	$68,700	$341,900	$410,600
Kansas	$75,100	$308,600	$383,700
West Virginia	$95,900	$250,800	$346,700
Massachusetts	$291,300	$35,100	$326,400
Nebraska	$34,500	$251,600	$286,100
Nevada	$70,200	$174,600	$244,800

Table A- 2. Summary of the States' Urban and Inter-Regional Freight Values (Continued).

STATE	Urban Commodity Value (Million Dollars)	Inter-Regional Commodity Value (Million Dollars)	Total Commodity Value (Million Dollars)
Connecticut	$204,900	$37,400	$242,300
Wyoming	$11,000	$201,000	$212,000
Idaho	$33,100	$141,900	$175,000
Puerto Rico	$143,900	$14,300	$158,200
South Dakota	$15,600	$129,100	$144,700
Montana	$7,200	$130,800	$138,000
North Dakota	$11,900	$116,000	$127,900
Maine	$23,800	$100,700	$124,500
New Hampshire	$40,100	$60,900	$101,000
Delaware	$58,700	$32,200	$90,900
Vermont	$6,600	$57,300	$63,900
Alaska	$15,700	$34,800	$50,500
Rhode Island	$35,900	$7,800	$43,700
Hawaii	$17,700	$15,900	$33,600
District of Columbia	$23,600	$0	$23,600

Evaluation of ASTM Standard Test Method E 2177, Retroreflectivity of Pavement Markings in a Condition of Wetness

Prepared for
Undergraduate Transportation Scholars Program

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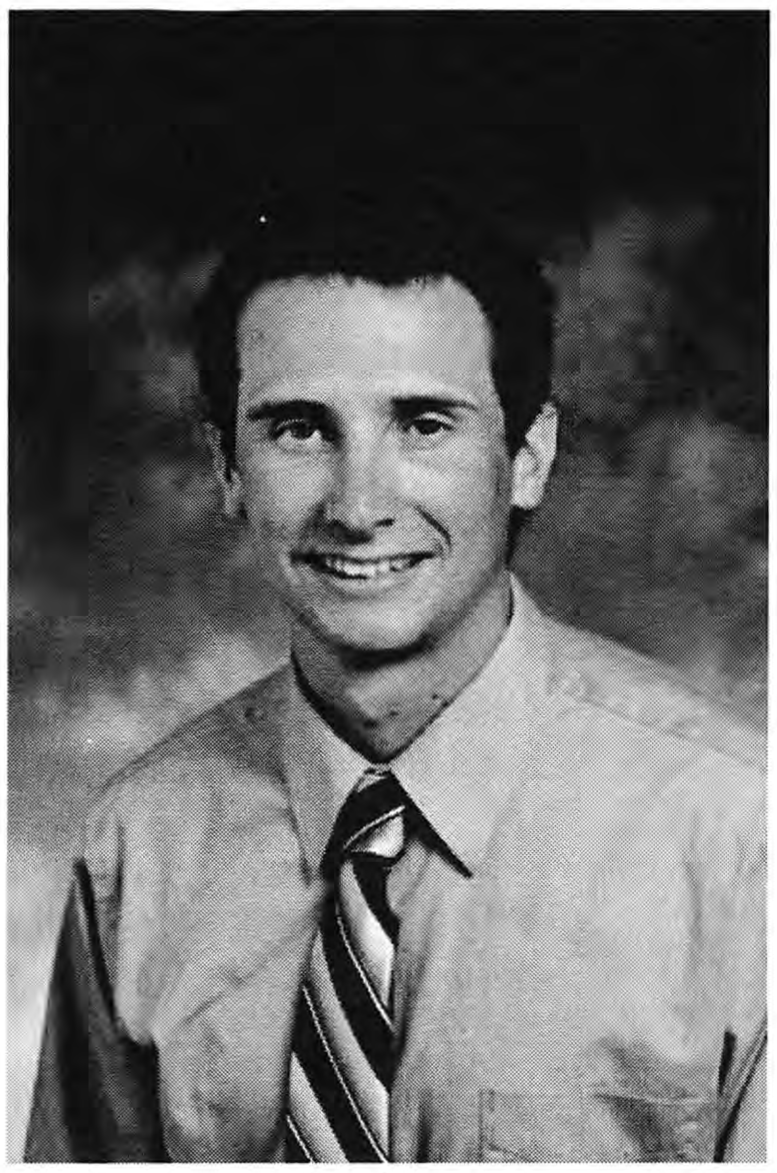

STUDENT BIOGRAPHY

Brandon Schwenn is a senior at the University of Wisconsin – Platteville located in Platteville, Wisconsin, and will be graduating in May 2011 with a Bachelor of Science in Civil Engineering. He will complete the Civil Engineering program at UW-Platteville with emphasis in transportation and construction engineering.

Outside of class Brandon is heavily involved in the club sport organizations; holding the presidency of the Sports Club Advisory Board (SCAB) as well as the current president of the UW-Platteville Lacrosse Club. While also playing many intramural sports, he also is involved as a team member of the UW-Platteville Wisconsin Asphalt Pavement Association Asphalt Design Competition Team. Also outside of the classroom Brandon works as a student tutor for the UW-Platteville Writing and Tutoring Resource Center mostly focusing on helping students in engineering and math courses.

After graduation Brandon plans to enter the job market by pursuing a career in the transportation field. He plans to either work for a department of transportation or with a private consulting firm.

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The author would also like to extend his thanks to the TTI statistics help desk for its assistance.

SUMMARY

The purpose of this project was to determine the sensitivity of standard test method ASTM E 2177 to various factors. The standard test method is a way of determining an accurate representation of the retroreflective performance of a pavement marking in a recovery condition. Having a test method that is consistent over the allowable range of factors is very important to the overall goal, and any inconsistencies within the test method could produce unequal results.

Many factors within the test were examined and they included: water application method, water volume, recovery time, pavement marking slope, and pavement marking type. These factors were evaluated in a controlled lab setting. Additional field testing on three different marking types was also conducted to examine the variability of the test method in the field. Following the lab data collection the results were statistically examined for the effect of each factor and any interactions between factors.

The results of the research showed a significant impact for all factors tested. Each marking type was impacted differently by each factor, and recommendations were made based on the significance of each impact. The research showed that ASTM E 2177 can produce repeatable results; however a narrower range of variables within the test could create more consistent results and reduce significant differences within the test.

TABLE OF CONTENTS

Student Biography ... 148
Acknowledgment ... 148
Summary ... 148
Table of Contents ... 150
Introduction ... 151
Background Information ... 151
  ASTM E 2177-01 ... 152
  European CEN EN 1436 B.6 ... 153
  ASTM vs. European Test ... 153
  Previous Research ... 153
Goals and Objectives ... 155
Data Collection ... 155
  Laboratory Testing ... 155
    Water Application Method ... 156
    Volume of Water ... 157
    Recovery Time ... 157
    Pavement Marking Cross Slope ... 158
    Type of Pavement Marking ... 158
    Laboratory Setup ... 159
    Problems Encountered ... 161
  Field Testing ... 161
Data Analysis ... 162
  Laboratory Data Analysis ... 162
    Analysis 1 ... 163
    Analysis 2 ... 164
    Analysis 3 ... 165
  Field Data Analysis ... 170
    State Highway 47 West of Bryan ... 170
    State Highway 21 West of Caldwell ... 171
    State Highway 6 North of Hearn ... 171
    Site Comparison ... 172
Findings ... 173
Recommendations ... 174
References ... 174
Appendix A: Analysis 1 ... 176
Appendix B: Analysis 2 ... 179
Appendix C: Analysis 3 Marking Type 1 ... 180
Appendix D: Analysis 3 Marking Type 2 ... 183
Appendix E: Analysis 3 Marking Type 3 ... 186
Appendix F: Analysis 3 Marking Type 4 ... 188

INTRODUCTION

Almost every pavement marking and roadway sign on today's roads and highways are retroreflective. Retroreflectivity is the reflection of light rays back to the original light source. What this means for drivers is that when their headlights shine on a pavement marking, a fraction of that light is reflected back toward the driver making the marking appear bright. Having greater visibility of markings and signs causes the driver to be able to more easily navigate the roadway, and in theory improves safety on roadways (*1*).

The most ideal condition for retroreflectivity is when roadway conditions are at their best, i.e., dry weather. However, retroreflectivity is affected when adverse weather conditions are introduced, and more specifically when water is on top of the pavement markings. Generally when water is introduced to a pavement marking there is an overall drop in retroreflectivity, and in some cases an extreme reduction in retroreflectivity is observed. Water on a roadway surface can cause issues for drivers that are trying to see pavement markings, and it can become even more of an issue at night (*2*).

In order to determine how markings perform under wet conditions there are two ASTM test methods that measure wet retroreflectivity of pavement markings. The main focus of this research was the ASTM E 2177 recovery test method (*3*). This method involves wetting the pavement marking and then taking a retroreflectivity reading with a handheld retroreflectometer after a specified recovery time.

The ASTM E 2177 test method was examined through many factors that could cause inconsistent or different results. Lab testing was the primary data collection procedure; however some field testing was done for more input on what factors have an effect in the field.

BACKGROUND INFORMATION

Microspheres (beads) of glass are most commonly used in pavement markings in order to achieve retroreflection. Light enters the glass beads, refracts on entry, reflects off the back of the bead when it hits the pavement marking that it is embedded in, then refracts again on exit, and is reflected toward the original source. Figure 1 shows the path of light being reflected though a glass bead. The glass beads are either placed directly as the markings are being applied, or can be placed on a wet marking. The optimal bead placement has the top layer of beads embedded 60 percent of the beads diameter (*4*).

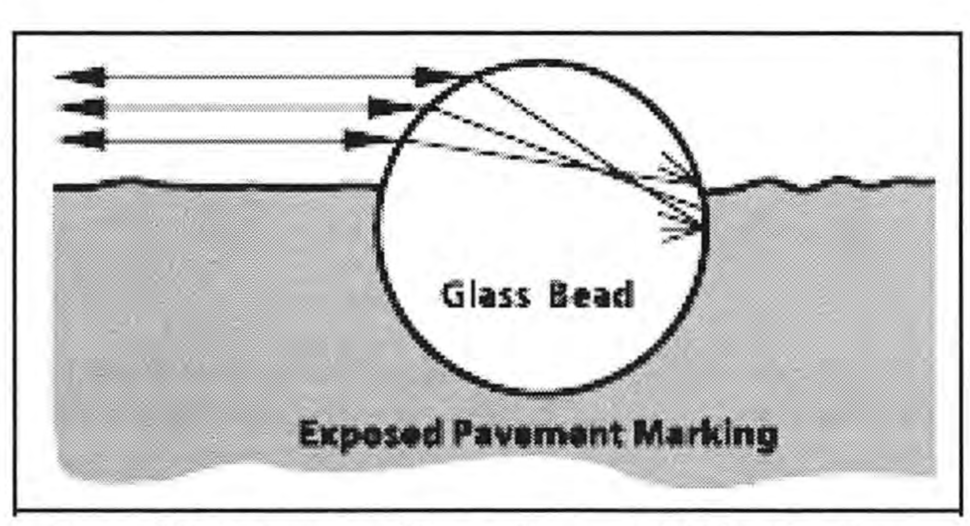

Figure 1. Example of Retroreflection of Light by a Glass Bead (*1*).

Using retroreflective materials in pavement markings allows for increased nighttime visibility of the markings. Necessary markings can be seen by drivers, reducing the risk of a driver losing the ability to safely navigate the roadway. However, just like any other material, pavement markings, and the glass beads, can deteriorate over time and reduce the overall retroreflectivity (*1*). In order to keep the necessary level of safety for drivers, pavement markings must be tested in order to check their performance.

In order to test the retroreflectivity of a pavement marking, a standard instrument must be used, and this instrument is a retroreflectometer. This instrument gives a standardized reading for the retroreflectivity of pavement markings. The measurement of retroreflectivity uses a 30 meter geometry, which utilizes a measuring distance of 30 meters from the cars headlights to the pavement marking. This distance produces standard angles for the path light would travel from the headlights, to the marking, and back to the driver (*5*). Figure 2 shows a diagram of the 30 meter geometry. Using these angles will produce the same test being applied to every marking. A retroreflectometer, however, is not 30 meters long, so it has to apply the geometry on a very small scale. The angles are still kept the same, but any slight movement of the instrument changes the field of measurement by a large margin.

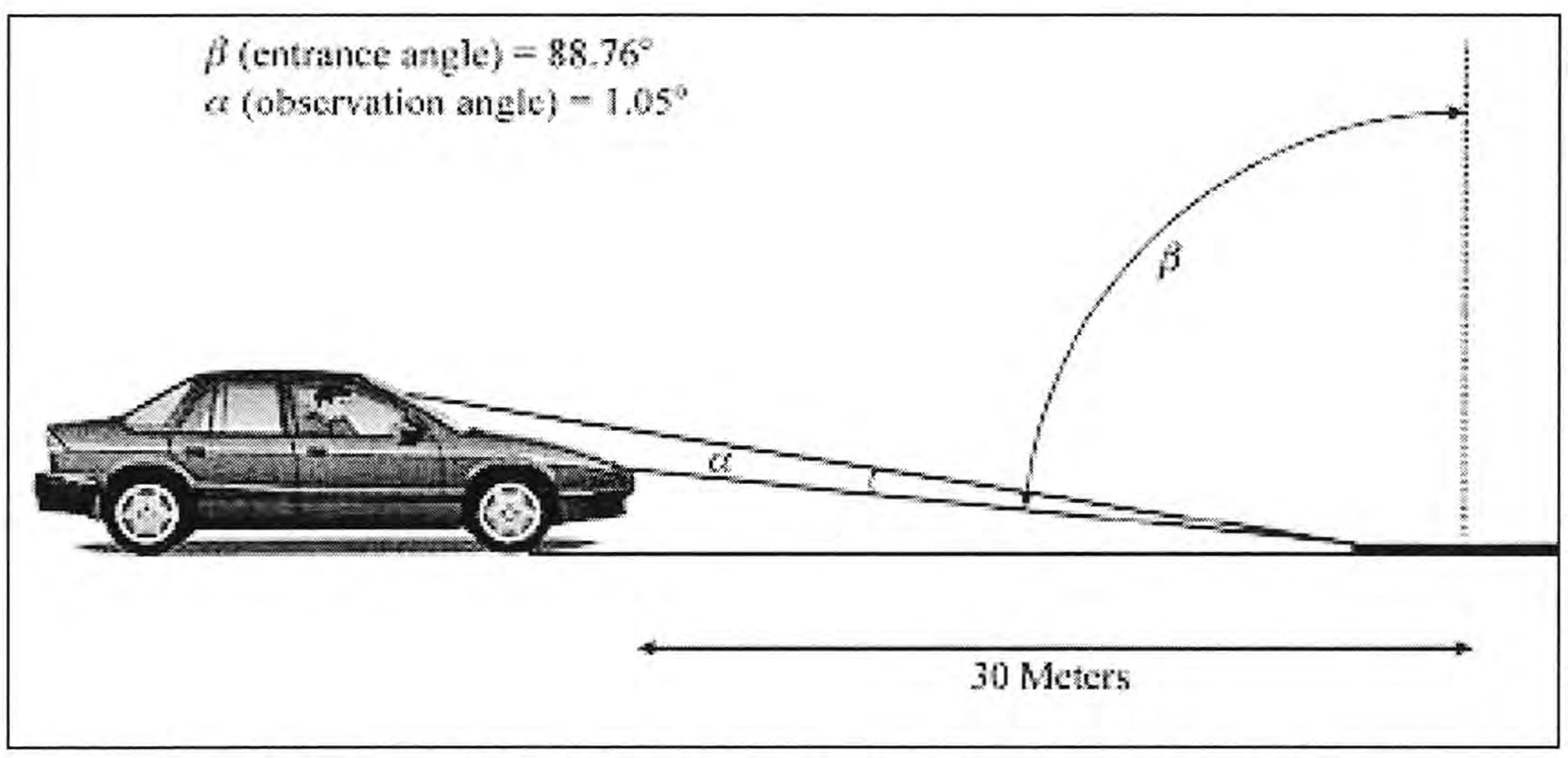

Figure 2. 30 Meter Observation Geometry (*6*).

The theory of retroreflectivity has had a major impact on highway visibility; however the visibility can be easily affected when adverse weather conditions are introduced. Having a condition of wetness or rain on the surface of the pavement marking will typically cause a decrease in the overall retroreflectivity of the pavement markings. Water on the surface of the markings increases specular reflection and changes the refraction of the light rays to and from the glass beads in the markings, typically resulting in a reduction of retroreflected light to the driver. During wet conditions the roadway markings are typically not as visible and sometimes almost invisible to drivers, which can result in a decrease in safety (*4*).

ASTM E 2177-01

Standard tests are available for testing the reflectivity of pavement markings, and ASTM E 2177-01 is the standard test for testing marking retroreflectivity recovery after a rain event. This test was developed to simulate the conditions after a rain event has occurred and the marking and pavement surface are still wet. In order to try and accurately represent a rain event the test gives two options to wet the marking under testing. A garden sprayer can be used for

30 seconds in order to wet the marking and surrounding area, or a bucket can be used to pour 2 to 5 liters of water over the marking and surrounding area. Both techniques should be equivalent in their ability to represent a wet condition; however this may not be the case. After the wetting of the surface occurs, a portable retroreflectometer is used to measure the retroreflectance of the marking. The measurement is then taken 45 (±5) seconds after the wetting has occurred and recorded in millicandelas per square meter per lux (mcd/m^2/lx). A dry and a wet measurement of the marking are taken for comparison and the number of trials is determined by the user. The specification also indicates to describe the slope of the marking as it could have an effect on the drainage of the water off of the top of the marking (*3*).

European CEN EN 1436 B.6

There is also a European Standard test for determining the retroreflectivity recovery of a pavement marking under a wet condition. This test differs slightly from the ASTM test used in the United States. For this test method a height of 0.3 meters is used while pouring water onto the marking with a bucket, and a minimum of 3 liters of water is used. The condition of wetness can also be achieved by clean water being sprayed over the making with an average intensity of 20 (±2) millimeters per hour over an area twice the width of the measured area, minimum 0.3 meters, and at least 25 percent longer than the measured area. This is the same method that is used for the condition of rain test in the European standard. The condition of wetness does not specifically give a time for the spray; however it refers to the condition of rain specification, which water should be sprayed for 5 minutes. Spraying water with the specified rates for 5 minutes would result in 1.7 millimeters of water over the application area. The measurements after the simulated rain event are taken at 60 (±5) seconds (*4*).

ASTM vs. European Test

The ASTM test and the European test differ in multiple ways, while testing the same condition. The water application method differs in both tests, and includes different standards for volume and rate of water flow. Also the recovery time after the application of the water differs by 15 seconds, which could produce very different results. The difference between the two tests shows evidence that further study must be completed in order to solidify that results are consistent under the standardized test.

Previous Research

Previous research has been done on the correlation between measured retroreflectivity and the detection distance of pavement markings (*2*, *7*, *8*). Detection distance is the distance from a marking that a driver can first see a given marking. Detection distance is a measurement of how far away a driver can detect a marking. Test courses have been used to collect data on detection distances of markings, while also comparing the values found on the same markings with the ASTM retroreflectivity test methods.

A specific study was done at the University of Iowa to evaluate detection distances and the standardized ASTM tests for retroreflectivity of pavement markings. This study was done on multiple types of markings under dry, simulated rain, and recovery conditions. The three conditions were created on a test track in Cottage Grove, MN. The simulated rain condition was created by spray nozzles above the roadway to create a 1 in/hr rainfall. The recovery condition used the same spray nozzles above the roadway; however they were turned off and a recovery time of 45 seconds was waited before the subjects entered that test section. The research showed that there was a significant correlation between the measured retroreflectivity and detection distances in each of the three conditions.

Also included in the study was an examination of how well the three ASTM tests represented each condition and how effective the field testing was. The ASTM tests were conducted with three different retroreflectometers (where applicable) and for each pavement condition, five readings were taken at various locations and then averaged for a total retroreflectivity for that section. The research did not state the volume or recovery time used for their ASTM E 2177 testing; however it was stated that measurements were conducted using the wet recovery test method according to ASTM E 2177. The results were found to be that ASTM E 1710 was accurate in a dry condition, ASTM E 2176 was accurate in a continuous rain condition, and ASTM E 2177 was accurate in a recovery condition. It was also found that ASTM E 2177 did not represent true conditions of a continuous rain event, and ASTM E 2176 did not accurately represent the results in a recovery condition (*9*). The results provided evidence that the tests performed to the condition they are standardized for and give accurate results based on field data. All of the accuracy results were based on the correlation between the measured detection distances and retroreflectivity values.

Previous research has been conducted on the ASTM E 2176 standard condition of wetness test. The condition of wetness test is different from the recovery method test in that the pavement marking must be continuously wetted during testing. Variables within the test have been known to cause inconsistent results, or make the test hard to repeat (*10*). Studies on the methods within the test have been conducted to produce more consistent results and the ASTM standards have been adjusted based on previous research (*8*). The ASTM E 2177 recovery test method differs from the ASTM E 2176 test, but some of the same variables and inconsistencies arise during testing. The changes and previous research on the ASTM E 2176 test give an insight that further research should also be conducted on the ASTM E 2177 to ensure consistent testing results.

Included within the ASTM E 2176 research (*8*), tests were also conducted with the ASTM E 2177 recovery method. The relationship between retroreflectivity and cross slope, and retroreflectivity and marking material type were evaluated. It was found that for the recovery test method, increasing the cross slope resulted in an increase in retroreflectivity across four different marking types. These results showed a definite effect of cross slope in the recovery method and gave more evidence that looking at cross slope was important in this research. The results also showed that the four marking types were impacted differently by the wet condition, indicating that different marking types should also be studied to make sure the results are equitable across common types of makings. Table 1 shows the results from the research on cross slope and marking type.

Table 1. Effect of Cross Slope on Retroreflectivity (*8*).

	Retroreflectivity			% Change	
	0%	2%	4%	2%	4%
Thermo, Type III	266	379	425	42.5%	59.8%
Structured Tape	604	620	624	2.6%	3.3%
Thermo, Mixed	104	125	152	20.2%	46.2%
Thermo, Type II	36	48	57	33.3%	58.3%

GOALS AND OBJECTIVES

This research centers on the evaluation of the effectiveness and consistency of ASTM E 2177-01. Some factors within the test have the possibility to cause error or inconsistency during testing. There are options within the test that result in a range of acceptable parameters and may not result in equivalent measuring practices. This problem was addressed by testing all of the different factors for equivalency and consistency. To meet the goal of the project the following objectives were established:

1. **Test the different factors and methods used in ASTM E 2177-01.** The factors including type of water application, volume of water, recovery time, pavement slope, and marking type were all examined. The examination looked at effectiveness and consistency of the test to be performed in a lab setting and a field setting. Multiple values of each variable were tested within and outside the range of recommended values to verify the standardization of each in the ASTM test.
2. **Determine the overall effectiveness of the ASTM E 2177-01.** Once all variables were tested for effectiveness and consistency, they were compared to the current standards of the ASTM test method and recommendations were made regarding the current standards.

Some factors, which include environmental factors such as pavement and ambient temperature, wind, humidity, and other weather conditions were not examined in this project. The main focus in this project was the factors that are given within the ASTM standard.

DATA COLLECTION

In order to determine the effect that the various factors have on ASTM E 2177, there needed to be an in depth data collection plan to examine all of the factors. Almost all of the data were collected in a lab setting for optimal control over most of the factors. Field testing data were also taken at three locations on three different pavement marking types to determine the effect of some of the factors in the field.

Laboratory Testing

All of the main factors being tested in this project were testing in the laboratory setting, as it was the most convenient to control each of the factors being examined. The lab used was the Visibility Research Lab located in the Texas Transportation Institute State Headquarters and Research Building. Inside the laboratory there was access to water and a drain in the floor to facilitate drainage of the large amount of water that was used. The temperature in the lab was held constant; there was no wind or sunlight, or any other environmental conditions that could cause possible inconsistencies in the testing. As mentioned in earlier sections the factors being examined are explained in detail below.

Water Application Method

The water application method is designed to simulate a rain event and make the marking and surrounding area covered in enough water to have an accurate simulation. As stated in ASTM 2177-01 there are two water application methods that can be used that are assumed to produce the same result: the bucket and hand sprayer application methods. The two methods were used throughout the laboratory testing in combination with the different pavement slopes and markings.

Before conducting the testing with both application methods, the two sprayers were tested for flow rate. Included in sprayer 1 were three different tips for the nozzle: a red fan tip, a yellow fan tip, and an adjustable cone to straight spray tip. Sprayer 2 only included one tip for the nozzle; however the tip was adjustable from a cone setting to a straight spray setting. For the flow rate calculations all three tips were tested on sprayer 1 and two different levels were tested on the sprayer 2 tip. It was not known at the best way to use the sprayers to return a consistent flow rate for multiple trials. Multiple different strategies were used to find consistent results, and after multiple tests it was found that keeping the water level in the sprayer between 1 and 1.5 gallons, while pumping the sprayer 15 times, and releasing the pressure after each trial produced the sought after results. The flow rate was calculated by determining the time it would take to fill up a container to a volume of 590 ml. Table 2 illustrates the final times and calculated flow rates.

Table 2. Sprayer Flow Rate Results.

	Average Time (s)	Flow Rate (ml/s)
Sprayer 1 Normal	85	7.0
Sprayer 1 Fan (Red)	68	8.6
Sprayer 1 Fan (Yellow)	108	5.5
Sprayer 2 Low	95	6.2
Sprayer 2 High	66	8.9

After the completion of the flow rate testing, it was determined that both sprayers would be used, but only one of the tips on each was tested. The red fan tip from sprayer 1 and the low setting for sprayer 2 were selected to give a different value for flow rates among the two sprayers. The two sprayers are shown below in Figure 3.

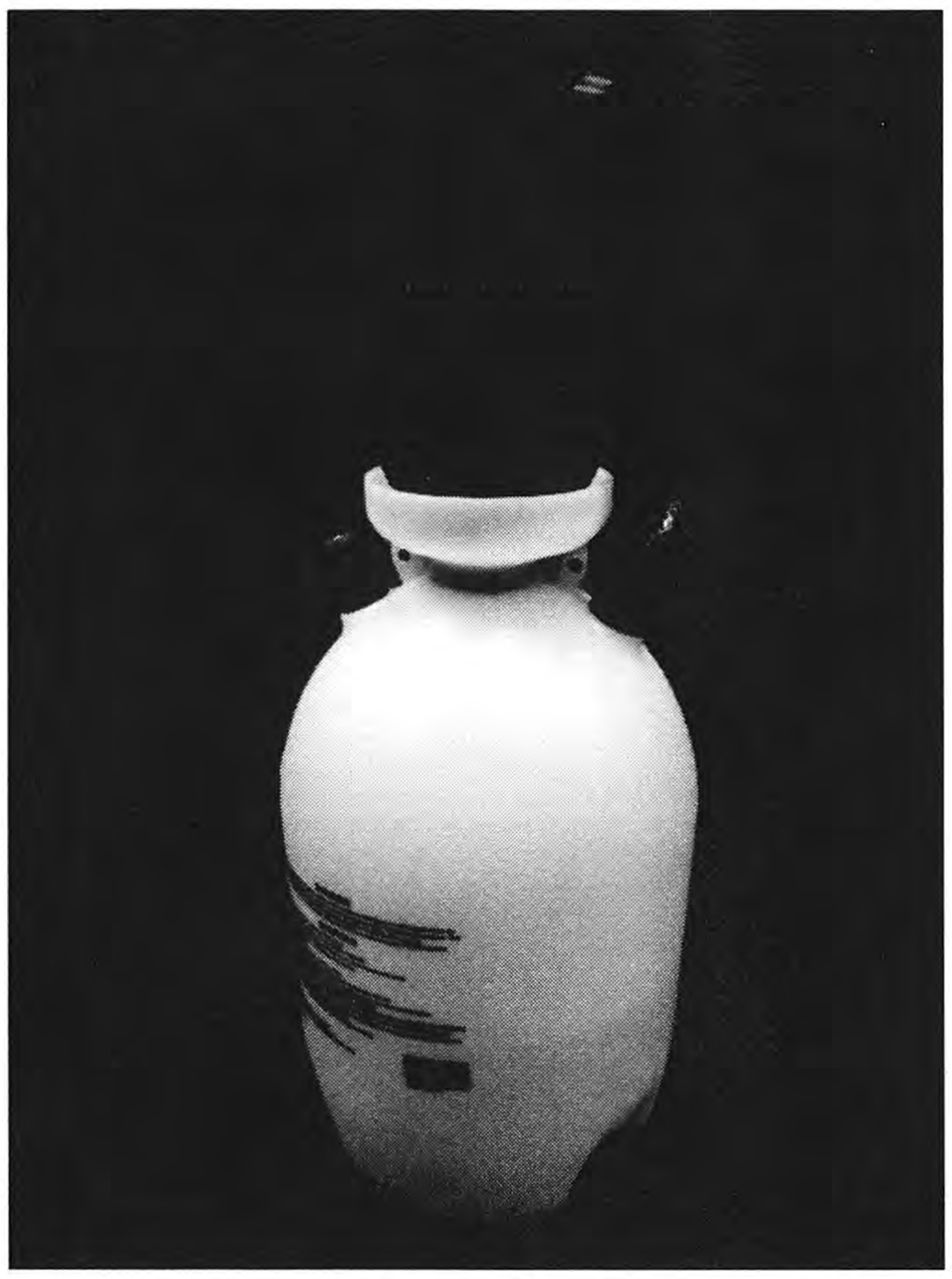 

Figure 3. Sprayer 1 (Left) and Sprayer 2 (Right).

During testing the two sprayers were used a certain way to best keep the results as consistent as possible. Water was applied from each sprayer at a height of 10–12 inches and a back and forth pattern across the measurement area was utilized. Also to keep the pressure as consistent as possible, before each trial the sprayer was pumped 15 times, then the marking was wetted for 30 seconds, after wetting the pressure in the sprayer was then released completely, and the next trial started with 15 pumps again. This kept the pressure inside the sprayer to be as equivalent across each trial as possible. Also the water inside the sprayer was kept between 1 and 1.5 gallons at all times to minimize the effect of water loss inside the tank. This same procedure was used in the flow rate calculations and then applied to the data collection.

Volume of Water

In order to keep a consistent simulation of rain from test to test, a volume of water applied to the marking is given in the standard. The volume of water is stated to be between 2 and 5 liters from a bucket or 30 seconds of spraying with a sprayer. Volumes of water from 1 to 6 liters at 1 liter increments from the bucket method were tested for their effect on the results and overall consistency. The total volume of water sprayed in 30 seconds from the sprayers was approximately 0.19 liters and 0.26 liters, clearly less than the 2 to 5 liters being poured out of a bucket. Following the specifications of the test the two sprayers were used and only sprayed for 30 seconds, and were compared to the bucket method in the data analysis.

Recovery Time

Once the marking and surrounding area are wet, a recovery time must be waited until a measurement is taken. This is to simulate a drying of a marking that would occur after a rain event and is the basis of the ASTM E 2177 test. The recovery time after wetting in the ASTM standard is 45 (±5) seconds, and in the European standard it is 60 (±5) seconds. A recovery time range of 30 to 65 seconds with 5 second intervals was tested to give a wide range of data surrounding the recovery time. The recovery time readings were taken in succession starting at 30 seconds, this allowed for eight readings to be taken during one run of the test that drastically reduced the total time for testing.

Pavement Marking Cross Slope

The slope at which a marking is sitting could have an effect on the drainage of water off of the top and could change the results of the test. Pavement marking slopes were added during lab testing with a range of 1 to 2.5 degrees at 0.5 degree intervals. Figure 4 shows the measurement of the marking slope at 2.5 degrees.

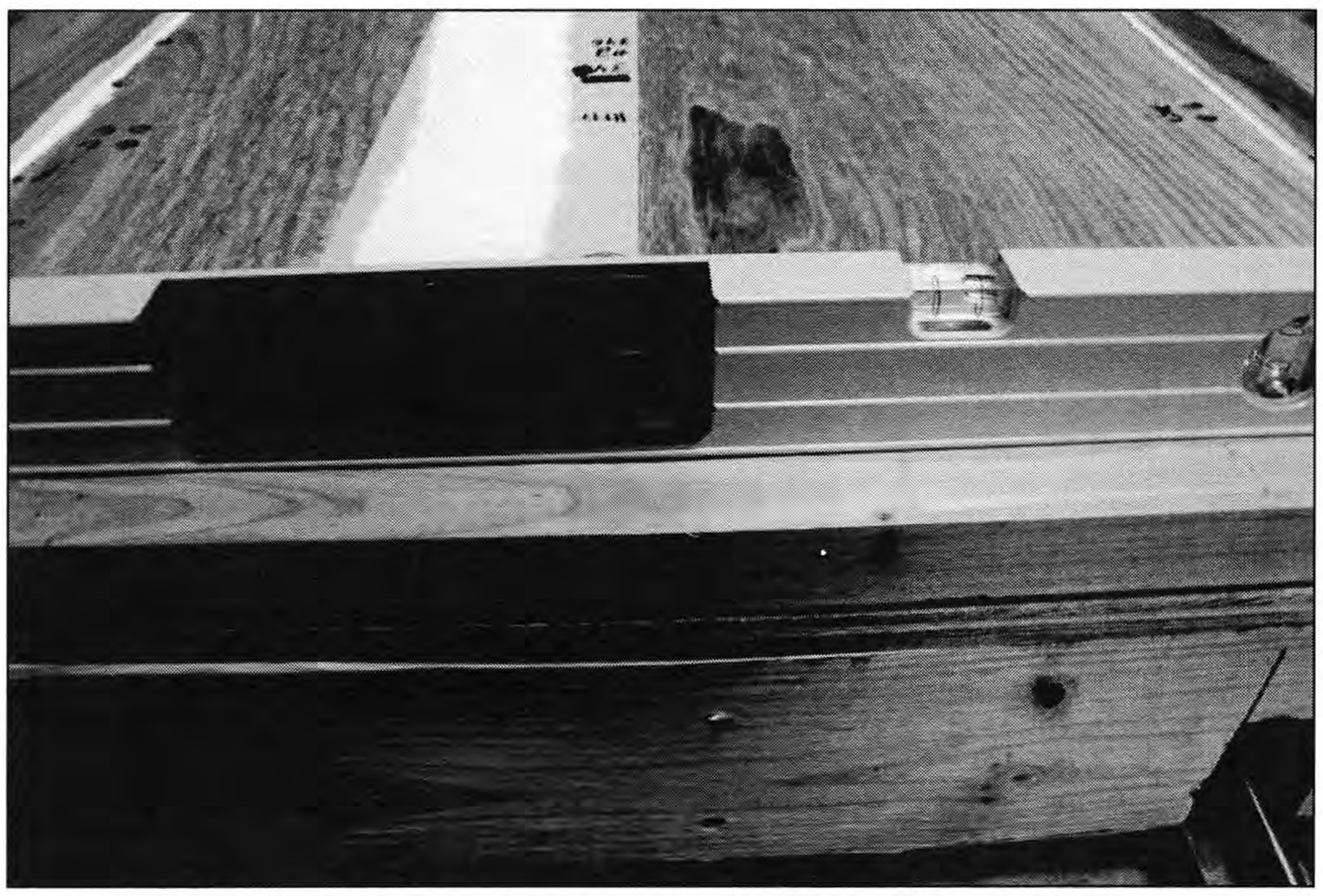

Figure 4. Measurement of Marking Slope.

Type of Pavement Marking

The type of marking could have an effect on the consistency of the test, based on that marking's properties. Two standard marking types (thermoplastic with type 2 beads and paint with type 3 beads), and two profiled (inverted profiled thermoplastic and wet reflective tape), were used throughout the laboratory testing. Two different samples of each of the four marking types were included in the testing. Figure 5 shows the four markings types. Each of the pavement markings were placed on a fiberglass substrate panel to allow for portability and the ability to change the panels out between tests. Dry retroreflectivity readings of each marking type were taken prior to any recovery readings being taken. A total of 10 dry readings were taken on each marking panel, at the center of designated measurement field where the testing was to be conducted. The retroreflectometer was moved a few inches around that center point in all directions to get to the 10 readings that were averaged for each panel. The dry readings are shown in Table 3.

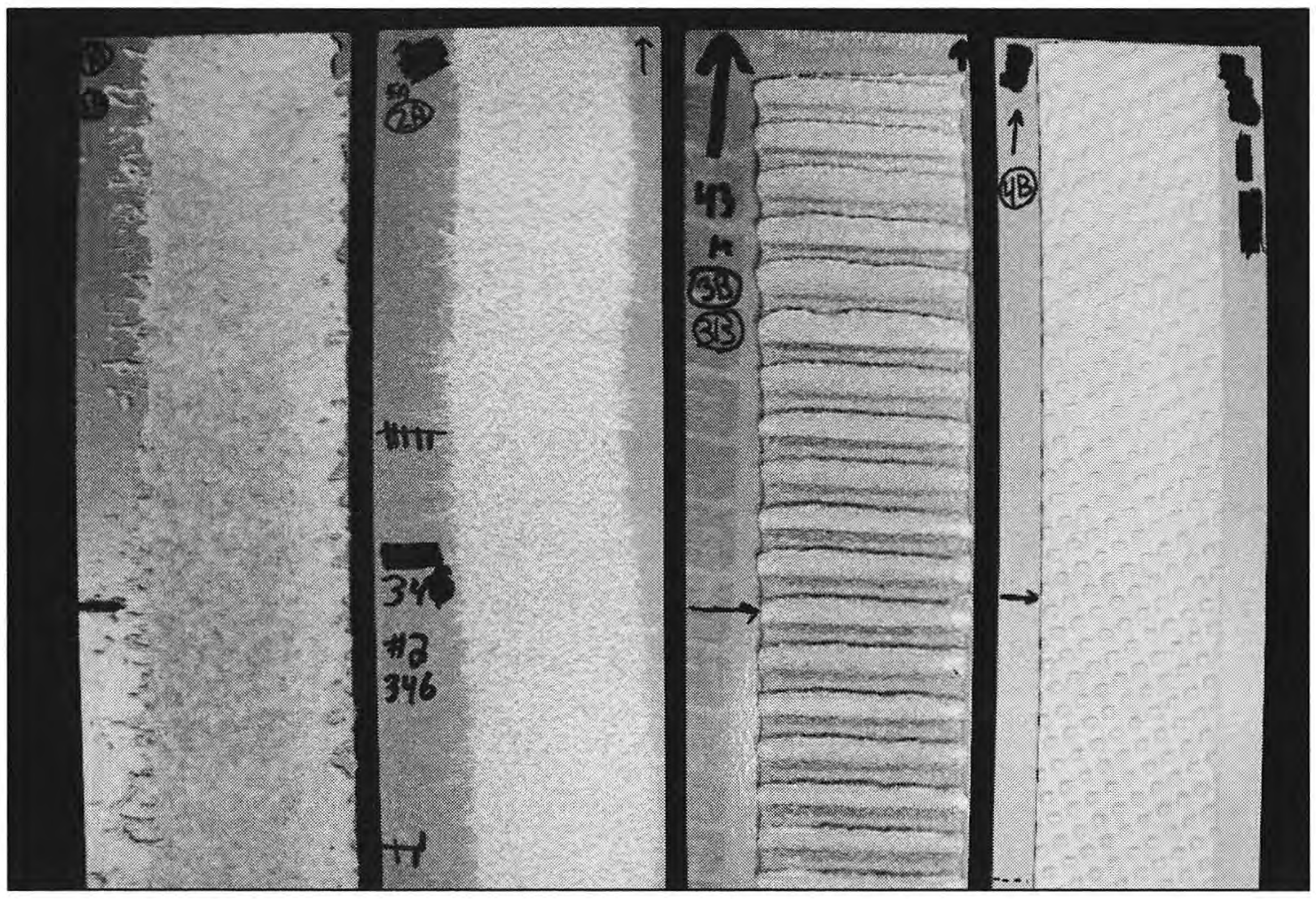

Figure 5. All Four Marking Types.

Table 3. Average Dry Retroreflectivity Values.

Panel	Average Dry Retroreflectivity ($mcd/m^2/lux$)
1A	291
1B	315
2A	361
2B	377
3A	609
3B	605
4A	1427
4B	1648

Laboratory Setup

Before data collection was started, a system to control the water being poured on the marking was needed given the conditions in the laboratory. A platform was created out of wood materials in order to create a control for water to make it from the marking to the drain in the floor. Figure 6 shows the testing platform created. The platform also made it easier to control the cross slope of the pavement marking by raising one side. The marking panels were all placed in the exact same location for every run of the test.

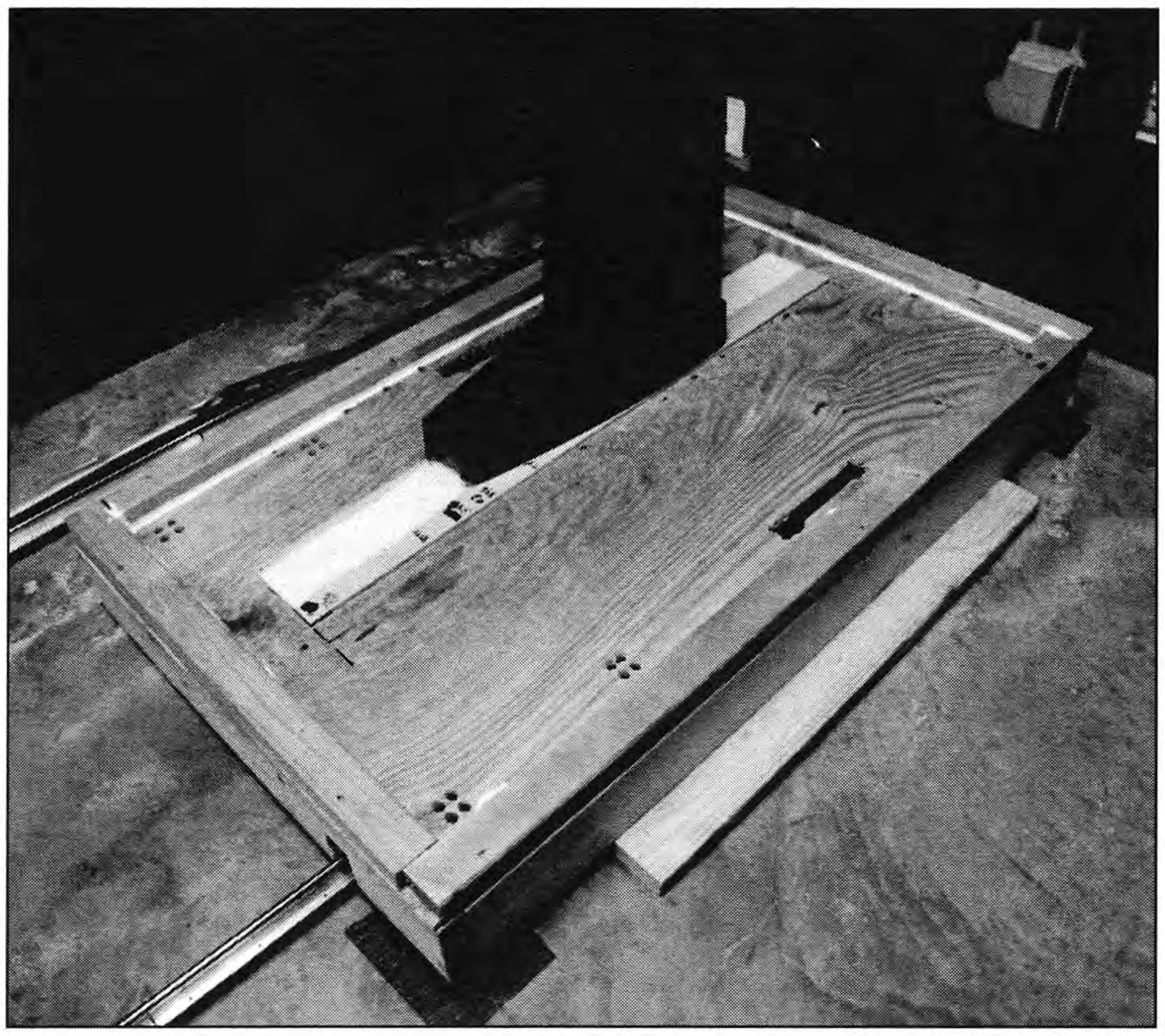

Figure 6. Testing Platform Made to Control Water Flow.

The testing was done by starting at a given slope, i.e., 1 degree, and then running the test over each of the four marking types at each volume of water and each recovery time. There were two different marking panels for each marking type so a total of eight marking panels were tested at each slope, volume, and recovery time. One run of the test would include one panel at a given slope and a given volume of water and then retroreflectivity readings were taken at each of the recovery times starting at 30 seconds and ending at 65 seconds. Table 4 shows the data table used for data collection. A complete set of data was developed at each marking slope, volume, recovery time, and marking type. Multiple trials were also conducted for several combinations of the variables to increase the size of the data set. Some runs were done with three separate trials while others only had one or two trials given time constraints for data collection.

Table 4. Lab Data Sheet for One Trial Set.

Retroreflectivity, Lab, 3 Liters:Bucket, 1° Slope								
	Trial 1							
Panel	**30s**	**35s**	**40s**	**45s**	**50s**	**55s**	**60s**	**65s**
1A								
1B								
2A								
2B								
3A								
3B								
4A								
4B								

Problems Encountered

During lab testing, a few problems were encountered. The first problem was that after multiple trials had been conducted during the day, and at higher water volumes, a small amount of fog was noticed building up on the glass lens while taking readings at each recovery time. Fog is a common problem faced with this type of testing, and within the ASTM E 2177 specification fog is mentioned as a precaution. "Verification must be made that there is no moisture on the instrument's lens when the instrument is being used for wet readings" (*3*). In order to counter this and prevent the fog buildup, the front of the retroreflectometer was tilted up between recovery time readings and then set back down as close as possible to the previous reading location. The second issue that was found was there was a slight warping of the plywood in the platform used for testing. This was easily countered with a checking of the slope before every trial and ensuring that the desired slope was met.

Field Testing

The second set of testing was conducted in the field to evaluate some of the factors in ASTM E 2177. Field testing was completed for evidence of any sensitivity in the field to any of the tested factors. The field testing took place at three separate road locations. The first field location was SH 21 west of Caldwell on a section of road that has profiled thermoplastic pavement markings that were approximately 2 years old. The second location was north of Hearne on SH 6 and has rumble stripe thermoplastic pavement markings that were approximately 3 years old. The third location was on SH 47 west of Bryan that had standard (flat) thermoplastic pavement markings that were approximately 4 years old. All three locations had standard type II beads on the markings. In order to perform these tests, water needed to be transported out to the field. Two 32 gallon trash cans were filled with water for ease of transport. This allowed for enough water to be available to complete the testing, as well as an easy way to fill the buckets required for each run.

The field testing was done to look at some of the factors that were examined in lab testing, but with some limitations. The field location determined the type of marking and the marking cross slope; however testing was done with different volumes of water and recovery times. The field data set differs from the lab set; fewer volumes and recovery times were tested in the field based on time constraints. Field test volumes included 1, 3, and 5 liters, and field test recovery times included 40, 45, and 50 seconds.

For the profiled and rumble stripe pavement markings another factor was added in addition to the previously mention factors: a small stepping distance. There is not currently a standardized measurement technique to measure profiled or rumble stripe pavement markings. ASTM E 1710 states that when measuring profiled pavement markings sufficiently small steps should be used in measurement (*11*). A recent paper titled, "Evaluation of Retroreflectivity Measurement Techniques for Profiled and Rumble Stripe Pavement Markings" is a more in depth look at the measurement of structured pavement markings (*12*). For this project a stepping distance of 3 inches was included to determine if the location of the retroreflectometer had any effect on the wet recovery retroreflectivity readings. Three trials were taken at the zero location, and then three trials were also taken at the +3 inch and +6 inch locations. These stepping distances will measure the recovery properties of the marking over a span that is slightly longer than the frequency of the profiled sections. Table 5 shows the data sheet used in the field.

Table 5. Field Data Sheet.

Location 1 (3 L of Water)									
Dry:	**Trial 1**			**Trial 2**			**Trial 3**		
(+0 in)	**40 s**	**45 s**	**50 s**	**40 s**	**45 s**	**50 s**	**40 s**	**45 s**	**50 s**
Retroreflectivity									
Dry:	**Trial 1**			**Trial 2**			**Trial 3**		
(+3 in)	**40 s**	**45 s**	**50 s**	**40 s**	**45 s**	**50 s**	**40 s**	**45 s**	**50 s**
Retroreflectivity									
Dry:	**Trial 1**			**Trial 2**			**Trial 3**		
(+6 in)	**40 s**	**45 s**	**50 s**	**40 s**	**45 s**	**50 s**	**40 s**	**45 s**	**50 s**
Retroreflectivity									

DATA ANALYSIS

The data analysis was broken down into two sections, one for the lab data, and one for the field data. The two analysis sections are shown below.

Laboratory Data Analysis

The lab data analysis was completed by the Short Term Statistics Help Desk on all data that were taken. Retroreflectivity was used of the dependant variable, and a split-plot model with marking type as a whole plot factor and all other factors as subplot factors was applied to the data. Also included in the analysis were the two-way interaction effects between factors. This allowed for more than just analysis on each factor separately and to see if the combination of any factors produced significant results. The analysis was obtained by the restricted maximum likelihood (REML) method implemented in the JMP statistical package. Table 6 shows the factors that were considered in the analysis. For water application volume the six bucket levels represent the six levels of water poured from the bucket, whereas S1 and S2 represent the two levels for the garden sprayers.

Table 6. Factors Considered in the Analysis.

Factor	**Levels**
Marking Type	Marking type 1, Marking type 2, Marking type 3, Marking type 4
Recovery Time	30 sec, 35 sec, 40 sec, 45 sec, 50 sec, 55 sec, 60 sec, 65 sec
Marking Slope	1 degree, 1.5 degree, 2 degree, 2.5 degree
Water Application Volume	Bucket_1, Bucket_2, Bucket_3, Bucket_4, Bucket_5, Bucket_6, S1, S2

Analysis 1

The first analysis that was run on the given data was a complete run of all of the factors being considered for statistical significance, and all two-way interactions also being considered. With the completion of the analysis it was found that individually all of the individual factors created a statistically significant effect on retroreflectivity. Along with all of the factors individually, it was also found that multiple two way interactions were statistically significant. These two-way interactions included; marking type and pavement marking slope, marking type and recovery time, marking type and water application volume, and pavement marking slope and water application volume. The results from the two-way interactions between marking type and the other factors can be seen in Figure 7 through Figure 9. The figures give a graphical representation of the least square mean of the data, which is an overall representation of all of the data collected over all of the factors. The full numerical analysis can be seen in Appendix A. After noticing all of the two way interactions in analysis 1, and the fact that marking type was included in all but one of them, it was decided to further the analysis by separating each marking type. Since the marking types were going to act individually, rerunning the analysis for each marking was justified, and the effect of the factors could more easily be determined.

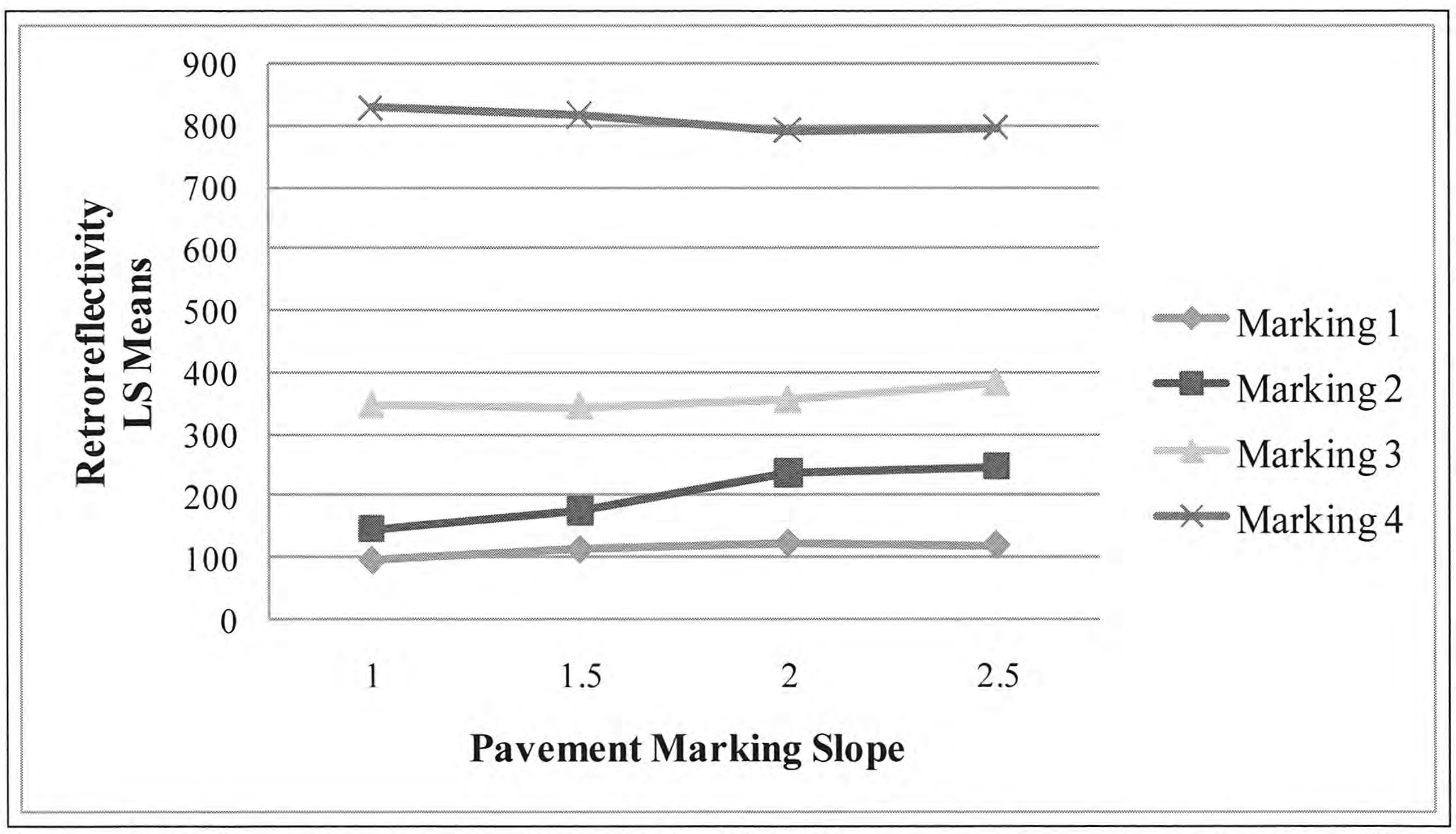

Figure 7. Interaction Plot between Marking Type and Pavement Marking Slope.

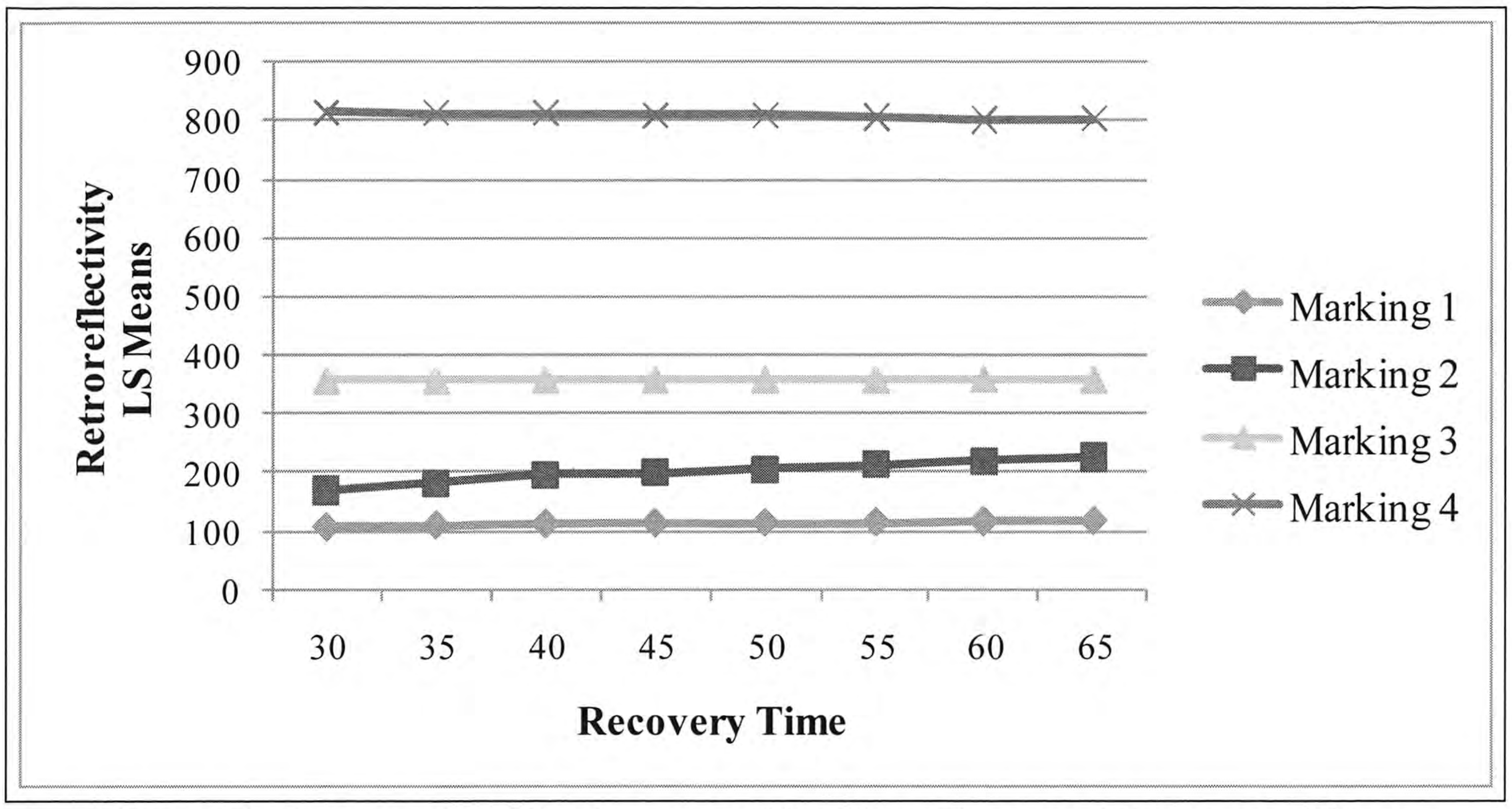

Figure 8. Interaction Plot between Marking Type and Recovery Time.

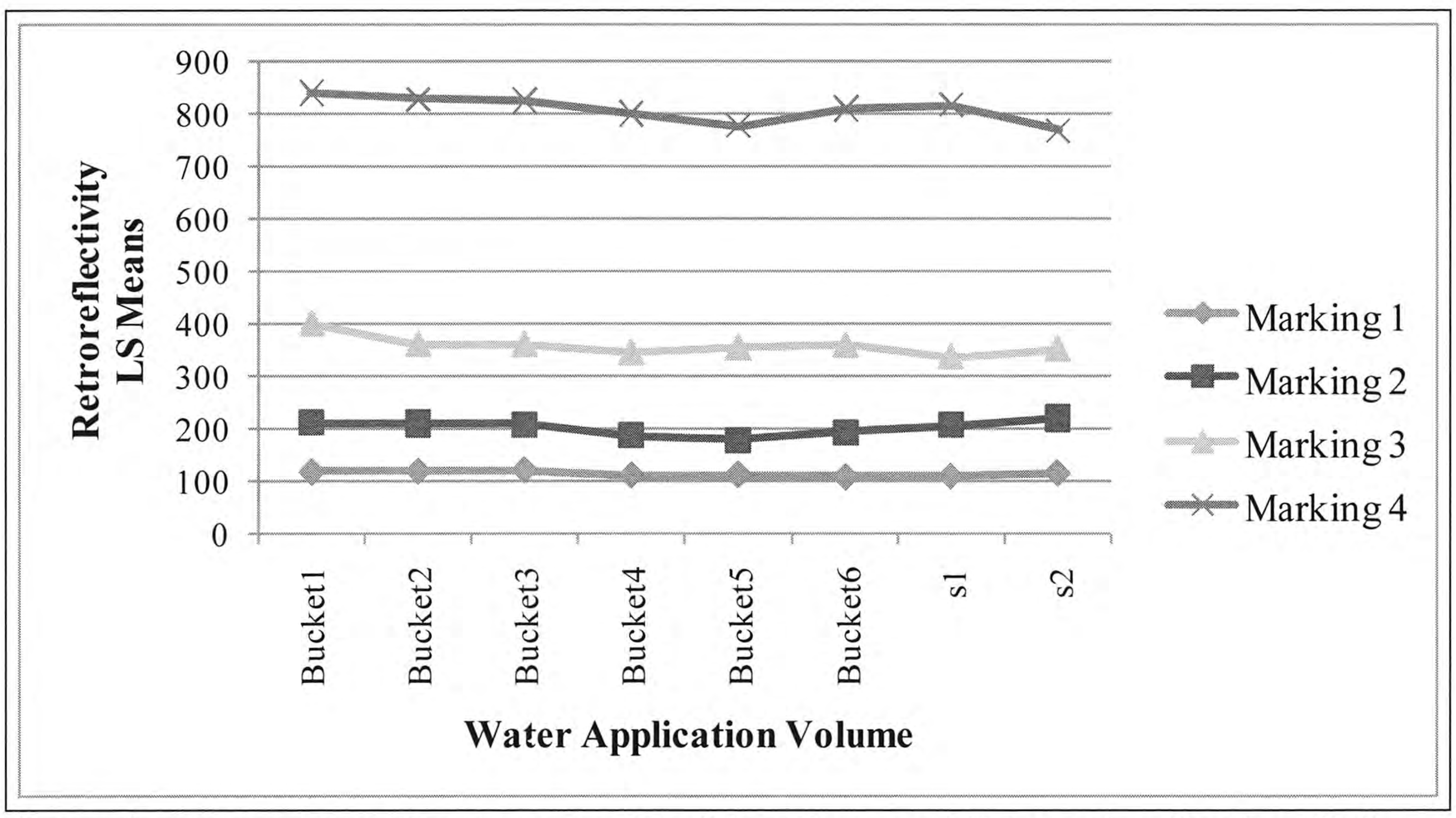

Figure 9. Interaction Plot between Marking Type and Water Application Volume.

Analysis 2

Before completing a complete analysis for each of the four marking types, an intermediate second analysis was added to test for the statistical significance of possible three-way interactions between factors. This analysis was completed the same way as the first, with adding the three-way interactions. After completion it was again found that all factors individually are statistically significant, as well as the same two-way interactions as before. In addition to these, there was also one three-way interaction that was found to be statistically significant: pavement marking type, recovery time, and water application volume. The full numerical analysis can be found in Appendix B.

Analysis 3

The third analysis as previously mentioned evaluated each factor for each of the marking types individually. This allowed for easier conclusions and findings to be made for each marking type. Again the results showed both single and two-way analysis on each of the factors within each marking type. The analysis for each marking type will be provided below in separate sections.

Marking Type 1

Marking type 1 was a thermoplastic marking with type II beads. The analysis showed a statistical significance of all of the single factors along with a two-way interaction of pavement marking slope and water application volume. The full numerical analysis can be found in Appendix C.

Since there was only one statistically significant two-way interaction between marking slope and water application volume, the recovery time is left as being statistically significant without any other interaction. The output of this data is shown in a Tukey chart showing statistical differences and is shown in Table 7. The levels that are not connected by the same letter are significantly different. Looking at the table, the analysis shows that from 50 to 65 seconds there is no significant difference, and also 40 to 50 seconds are not significantly different. This shows that the range accepted within the ASTM test method there is no significant difference between the accepted values for marking type 1. However, looking at the European CEN test, the recovery time of 65 seconds is significantly different than any recovery time in the ASTM test. The times of 60 and 55 seconds were significantly different than the 40 second recovery time.

Table 7. Tukey Analysis of Recovery Time for Marking Type 1.

Level						Least Sq Mean
65	A					115.04274
60	A	B				114.49772
55	A	B				113.59526
50	A	B	C			112.61278
45		B	C	D		111.61868
40			C	D		110.16065
35				D	E	108.48551
30					E	105.34110

The output of the data for the two-way interaction of pavement marking slope and water application volume was shown in an LSMeans Differences Tukey HSD table shown in Appendix C and provided below in Figure 10. Looking at the table and figure, it is expected that the higher slopes and lower water volumes would have lower retroreflectivity levels, whereas the lower slopes and higher water volumes would have higher retroreflectivity levels relative to one another. This however is not exactly the results that are produced in the analysis; the highest least square mean value was actually 1 liter at a slope of 2 degrees. This does not exactly fit the expected results, but generally looking at the values and the significance, the higher slopes of 2 and 2.5 degrees show higher values, where as the lower slopes of 1 and 1.5 degrees show lower retroreflectivity values. Also in general the lower bucket application volumes tended to produce higher retroreflectivity values compared to the higher wetting rates. The retroreflectivity values produced by the sprayer method seemed to fall in line with the values produced by the bucket method for marking type 1.

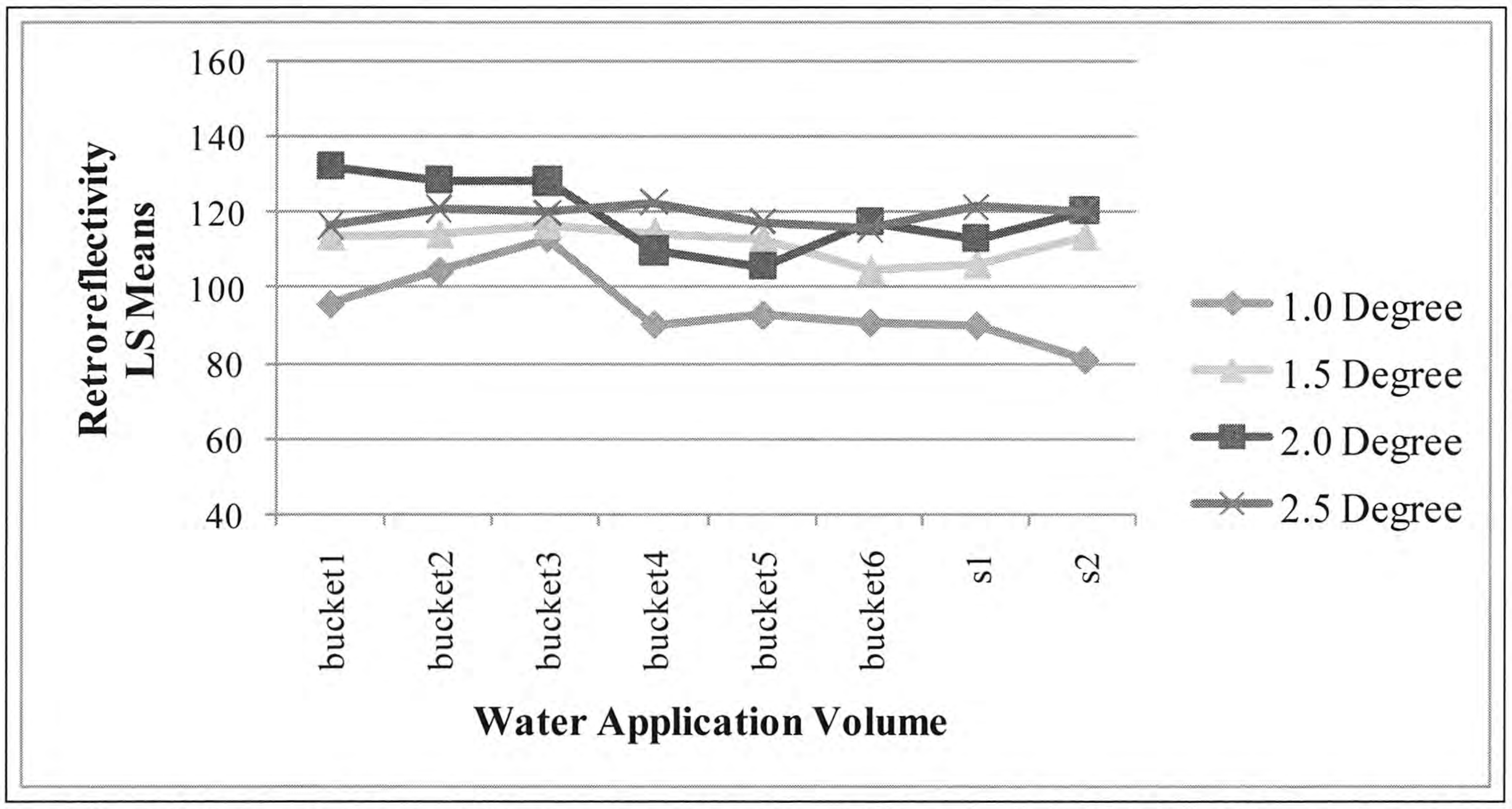

Figure 10. LS Means Plot of Water Application Volume vs. Marking Slope for Marking Type 1.

Marking Type 2

Marking type 2 was a paint marking with type III beads. The analysis showed a statistical significance of all of the single factors along with a two-way interaction of pavement marking slope and water application volume. The full numerical analysis can be found in Appendix D.

Again there was only one statistically significant two-way interaction between marking slope and water application volume, the recovery time is left as being statistically significant without any other interaction. The output of this data is shown in a Tukey chart showing statistical differences, and is shown in Table 8. The levels that are not connected by the same letter are significantly different. Looking at the table, the analysis shows that from 55 to 65 seconds, 50 to 55 seconds, 45 to 50 seconds, and 40 to 45 seconds are not significantly different. With this marking type however, since the times are mostly grouped in sets of two, it shows that there is a significant difference between 40 seconds and 50 seconds. This means that within the ASTM specification there could be some significantly different values if measurements are taken on opposite end of the acceptable time range of 45 (±5) seconds. Also with this marking type there is a significant difference between the European standard and the ASTM standard, other than the 50 and 55 second readings, which represent the highest level of the ASTM standard and the lowest level of the European standard.

Table 8. Tukey Analysis of Recovery Time for Marking Type 2.

Level							Least Sq Mean
65	A						224.87538
60	A						218.14773
55	A	B					212.36171
50		B	C				203.64665
45			C	D			197.70736
40				D	E		189.84204
35					E	F	180.21881
30						F	167.87959

The output of the data for the two-way interaction of pavement marking slope and water application volume was shown in an LSMeans Differences Tukey HSD table shown in Appendix D and displayed below in Figure 11. Again the expected result is that the lower volumes and higher slopes would produce the highest retroreflectivity values, and the lower slopes and higher volumes would be lower. Generally looking at the values and the significance, the higher slopes of 2 and 2.5 degrees show higher values, where as the lower slopes of 1 and 1.5 degrees show lower retroreflectivity values. Also in general the lower bucket application volumes tended to produce higher retroreflectivity values compared to the higher wetting rates. For marking type 2 a large difference in retroreflectivity values exists when the slopes drop below 2 degrees. The retroreflectivity values produced by the sprayer method seemed to fall in line with the values produced by the bucket method for marking type 2.

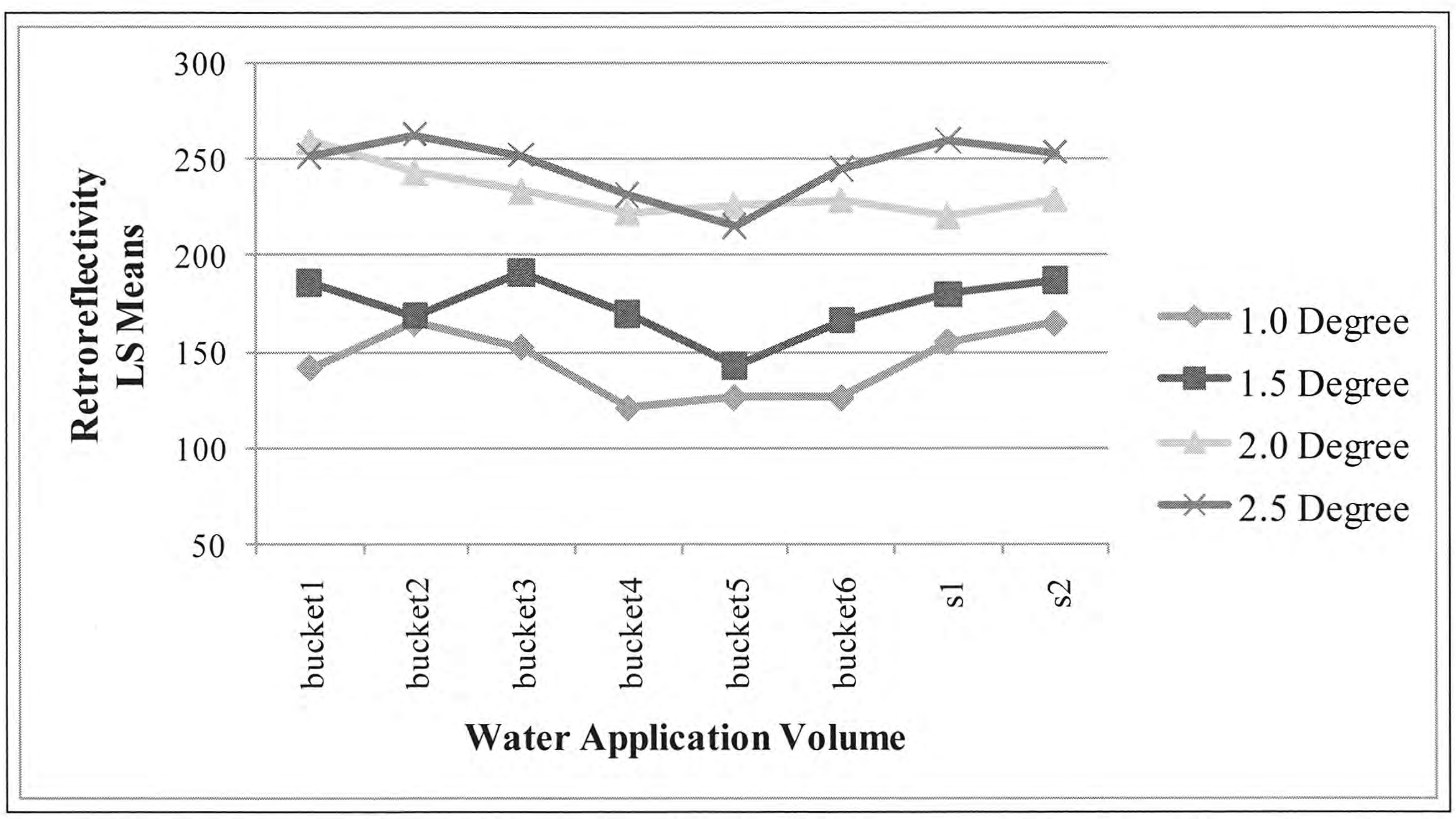

Figure 11. LS Means Plot of Water Application Volume vs. Marking Slope for Marking Type 2.

Marking Type 3

Marking type 3 was an inverted profiled thermoplastic marking that had type I and type IV beads. The analysis showed a statistical significance of pavement marking slope, and water application volume as single factors along with a two-way interaction between the two. The full numerical analysis can be found in Appendix E.

Since there was only one two-way interaction among two significant factors the only significant result was the two-way interaction. The output of the data for the two-way interaction of pavement marking slope and water application volume was shown in an LSMeans Differences Tukey HSD table shown in Appendix E and displayed below in Figure 12. Since the same two-way interaction is being examined the expected results are the same as before. Marking type 3 shows some different results than both marking types 1 and 2. For marking type 3 there is more of a mixing of the lower slope values within the higher slope values across the different water application volumes. In general the 2 and 2.5 degree slopes still hold near the top, as do the lower water volumes, but it is much more mixed. The retroreflectivity values produced by the sprayer method seemed to fall in line with the values produced by the bucket method for marking type 3.

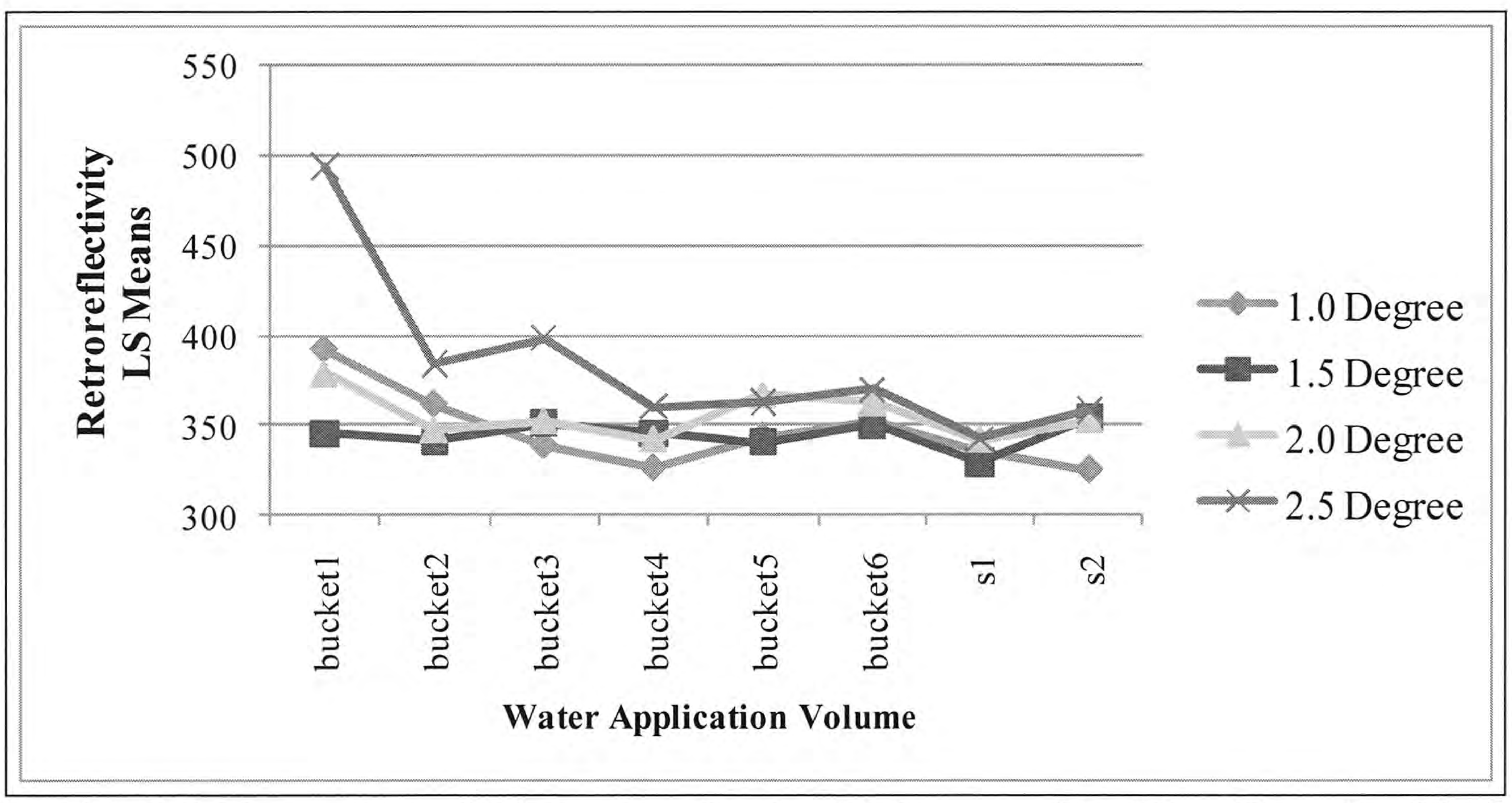

Figure 12. LS Means Plot of Water Application Volume vs. Marking Slope for Marking Type 3.

Marking Type 4

Marking type 4 was a structured wet reflective tape marking. The analysis showed a statistical significance of pavement marking slope, and water application volume as single factors along with a two-way interaction of pavement marking slope and water application volume, along with recovery time and water application volume. The full numerical analysis can be found in Appendix F.

The output of the data for the two-way interaction of pavement marking slope and water application volume was shown in an LSMeans Differences Tukey HSD table shown in Appendix F and is also presented below in Figure 13. For marking type 4, the expected results were not shown in the actual results. The top eight values for least square means were either from 1 or 1.5 degree slopes. Also five of the bottom 6 are from 2 or 2.5 degree slopes. The trend followed for this marking type is that at lower volumes the retroreflectivity values were generally higher, but higher slopes seemed to negatively impact the retroreflectivity. It is thought that with the structure of the marking any water that sits in the low areas, which is more likely with lower slopes, will cause specular reflection of the light that will then hit the face of a structure and be reflected back toward the retroreflectometer. The retroreflectivity values produced by the sprayer method seemed to fall in line with the values produced by the bucket method for marking type 4, except for the variable data for the 1 degree slope.

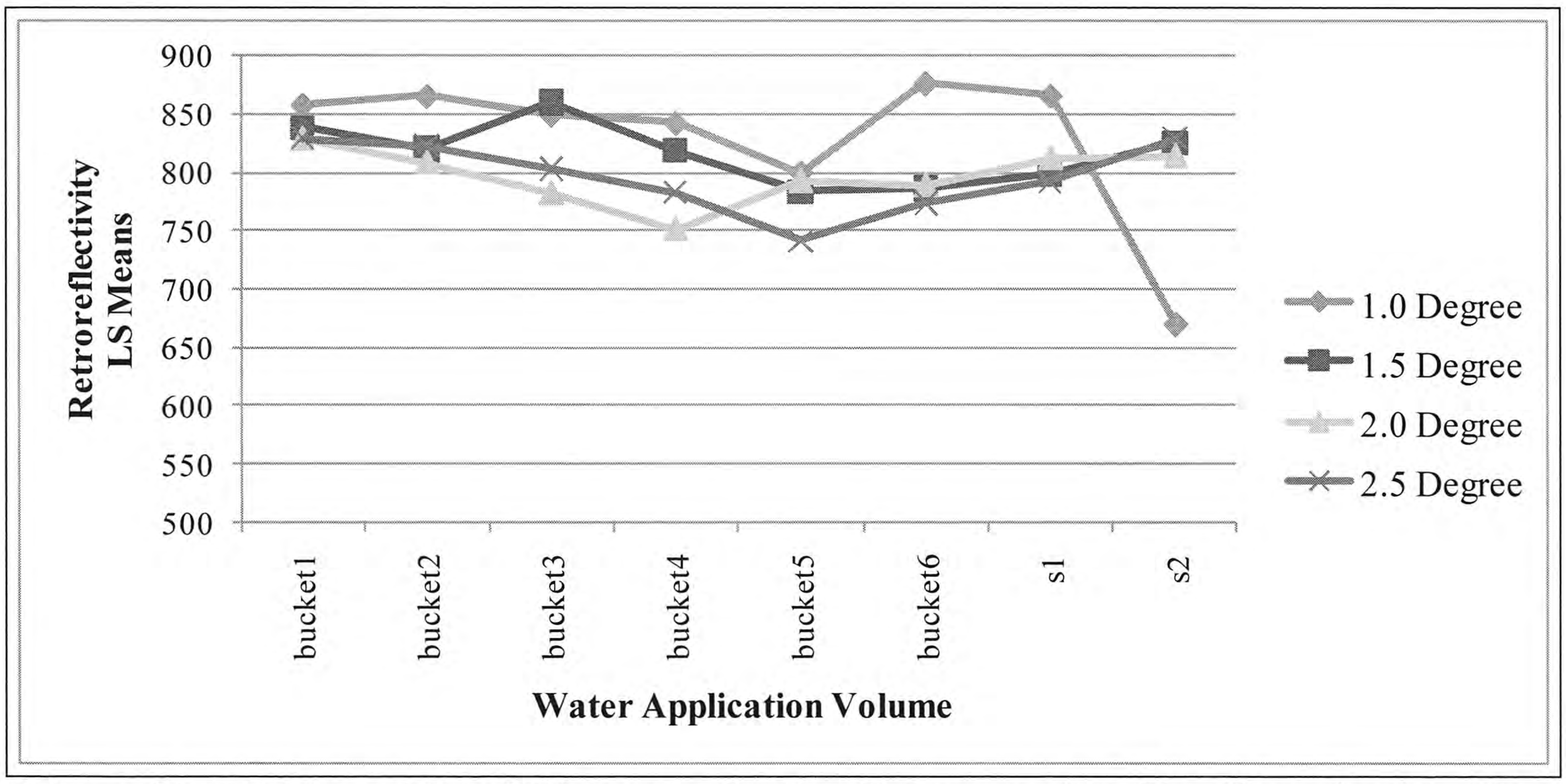

Figure 13. LS Means Plot of Water Application Volume vs. Marking Slope for Marking Type 4.

Marking type 4 was also different in that fact that there were two statistically significant two-way interactions. The second two-way interaction was recovery time and water application volume. The output of the data for the two-way interaction of recovery time and water application volume was shown in an LSMeans Differences Tukey HSD table shown in Appendix F and is also presented below in Figure 14. For this type of interaction it was somewhat expected that the effect of recovery time would have the same effect across each of the water volumes. This expected result was true over all but two of the water volumes tested. There was a significant difference noticed at the 5 liter volume along with the sprayer 2 volume. All of the other volume values are generally of the same significance. Looking at the LS Means Plot in Figure 14 a dip can be seen in the plot at 5 liters and with the second sprayer. Other than those two different dips, the plots of recovery times are basically right on top of one another. It is hard to determine the cause of this, but the values are significantly different.

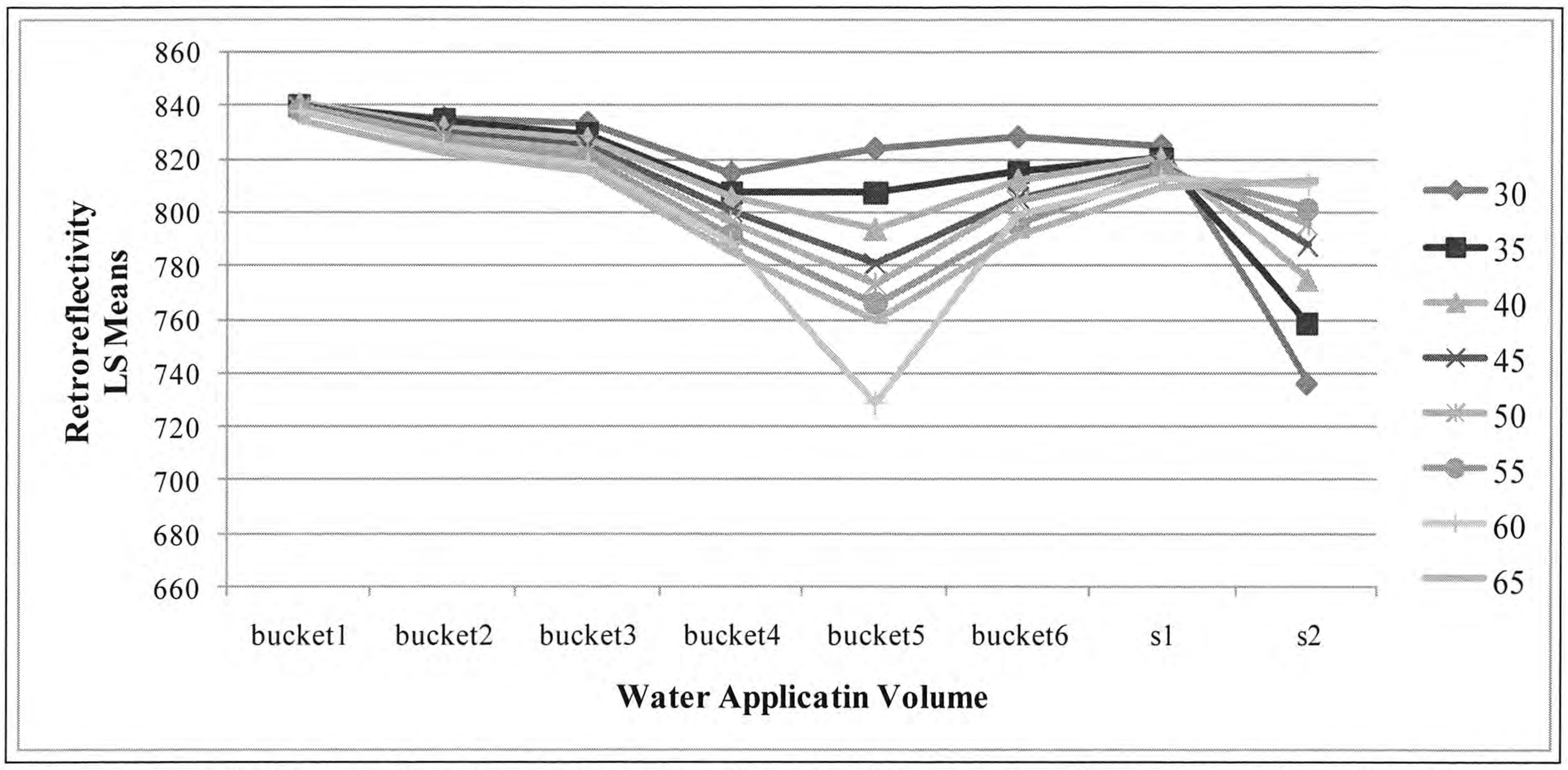

Figure 14. LS Means Plot for Water Application Volume vs. Recovery Time for Marking Type 4.

Field Data Analysis

There were three locations for the field data collection and each location was analyzed separately and then compared. The locations are as follows.

State Highway 47 West of Bryan

The first location where field testing occurred was on SH 47 West of Bryan, Texas, on flat thermoplastic pavement markings with type II beads. A total of 6 locations along the stretch of marking were tested with volumes of 3 and 5 liters, and recovery times of 40, 45, and 50 seconds. The locations had dry retroreflectivity values that ranged from 194 to 266 mcd/m^2/lux. Since there was not a large data set taken from the field, a full statistical analysis could not be completed; however general analysis was completed.

Taking a look at the data there were a few trends that could be determined by the field testing. The dry retroreflectivity reading did not necessarily predict the results of the recovery test method. For example, location 1 had a dry reading of 255 mcd/m^2/lux and a recovery reading of 53 mcd/m^2/lux at 45 seconds; however location 4 had a dry reading of 194 mcd/m^2/lux and a recovery reading of 76 mcd/m^2/lux at 45 seconds. Also the data showed that retroreflectivity increases with recovery time, which was the expected result, but the increase was not substantial over the 10 second window being used. The highest increase was only 9 mcd/m^2/lux, and most of the locations showed an increase of around 5 mcd/m^2/lux.

Two different water volumes were tested at 3 of the locations, and the data show an increase in the retroreflectivity values of about 10 to 20 mcd/m^2/lux with the 5 liter volume compared to the 3 liter volume. Since there was a limited data set for analysis these trends might not always be the case, however for these data an increase occurred. Table 9 summarizes the final averages of all of the retroreflectivity readings at the SH 47 site.

Table 9. Summary of Average Retroreflectivity at SH 47.

	Retroreflectivity (mcd/m²/lux)				Slope (degrees)
	40s	**45s**	**50s**	**Dry**	
Location 1 (3 liters)	50	51	53	255	2.2
Location 2 (3 liters)	58	61	63	266	2.0
Location 3 (3 liters)	49	51	55	226	2.3
Location 4 (3 liters)	74	76	79	194	2.2
Location 2 (5 liters)	69	72	79	266	2.0
Location 3 (5 liters)	71	74	77	226	2.3
Location 4 (5 liters)	84	88	89	194	2.2
Location 5 (5 liters)	56	59	61	246	2.4
Location 6 (5 liters)	62	67	71	244	2.3

State Highway 21 West of Caldwell

The second field location was on SH 21 west of Caldwell, Texas, that featured profiled thermoplastic pavement markings that had type II beads. Three different locations were tested along this stretch of markings, and three different volumes of water were also tested. As stated earlier the addition of a small stepping distance created more readings at each location, making only three locations possible given the time for data collection. Again only three recovery times were looked at during testing, and a general analysis was utilized on the small set of data.

Three volumes of water were tested, 3 liters at location 1, 5 liters at location 2, and 1 liter at location 3. The three different water volumes did not appear to show a trend of having an effect on the retroreflectivity values. The values were consistently around 20 or 30 mcd/m^2/lux. The recovery time did not seem to affect the values of retroreflectivity over the 10 second window being tested. The greatest difference from 40 seconds to 50 seconds was 3 mcd/m^2/lux, with most staying very consistent over the 3 readings.

The addition of the stepping distance seemed to have mixed results over the three locations. For the second (5 liters) and third (1 liter) location the zero location has the highest retroreflectivity followed by the 3 inch location and then the 6 inch location. For the first location (3 liters) the three steps had similar retroreflectivity readings, with the 3 inch step having the highest reading. Table 10 shows a summary of the averaged data taken at SH 21.

State Highway 6 North of Hearne

The third field location was on SH 6 North of Hearne, Texas, that featured rumble stripe thermoplastic pavement markings with type II beads. The same testing procedure was followed for the rumble stripe markings as for the profiled markings at SH 21. Again a general data analysis was used to determine any trends.

Table 10. Summary of Average Retroreflectivity at SH 21.

	+0" Step				
	Retroreflectivity (mcd/m²/lux)				Slope (degrees)
	40s	45s	50s	Dry	
Location 1 (3 liters)	24	25	26	139	1.6
Location 2 (5 liters)	32	32	31	127	1.5
Location 3 (1 liter)	36	36	35	119	1.5
	+3" Step				
	Retroreflectivity (mcd/m²/lux)				Slope (degrees)
	40s	45s	50s	Dry	
Location 1 (3 liters)	24	27	27	131	1.6
Location 2 (5 liters)	29	29	29	132	1.5
Location 3 (1 liter)	26	26	26	122	1.5
	+6" Step				
	Retroreflectivity (mcd/m²/lux)				Slope (degrees)
	40s	45s	50s	Dry	
Location 1 (3 liters)	21	21	22	135	1.6
Location 2 (5 liters)	20	20	21	137	1.5
Location 3 (1 liter)	18	18	18	122	1.5

The same three volumes were tested; 1, 3, and 5 liters, and three different locations were tested. The volume of water did not show any significant difference on the retroreflectivity values. For all three volumes the values were consistently around 40 to 50 mcd/m^2/lux. The recovery time showed a slight increase in numbers over the 10 second window being tested; however the increases were only 3 or 4 mcd/m^2/lux, and in one case the increase was 6 mcd/m^2/lux. The stepping distance also did not seem to have a consistent trend with the retroreflectivity values. For the three locations tested there was no step that was consistently higher than the others, the data varied at each location. The data showed fairly consistent results over the 3 factors being tested. Table 11 shows a summary of the averaged data taken at SH 6.

Site Comparison

The three sites showed how ASTM E 2177 performed on three different pavement markings. After looking at the results from these three sites, some trends can be seen. Comparing the two profiled pavement markings, the dry readings of both locations were similar; however, the rumble stripe markings at SH 6 showed higher recovery readings than the profiled markings at SH 21. The retroreflectivity values of the two profiled sections were more consistent than the flat markings. Also the data typically showed a slight increase in retroreflectivity at each site with increased recovery time, which is expected, however the increase was not substantial over the 10 second window being used. The data taken at these field locations was not a large data set; however the data do show some basic trends and evidence of the affect of some factors.

Table 11. Summary of Average Retroreflectivity at SH 6.

	+0" Step				
	Retroreflectivity ($mcd/m^2/lux$)				Slope (degrees)
	40s	45s	50s	Dry	
Location 1 (1 liter)	42	44	46	157	2.4
Location 2 (3 liters)	49	51	52	124	2.5
Location 3 (5 liters)	40	42	46	124	2.5
	+3" Step				
	Retroreflectivity ($mcd/m^2/lux$)				Slope (degrees)
	40s	45s	50s	Dry	
Location 1 (1 liter)	45	46	47	164	2.4
Location 2 (3 liters)	39	39	41	116	2.5
Location 3 (5 liters)	37	37	39	105	2.5
	+6" Step				
	Retroreflectivity ($mcd/m^2/lux$)				Slope (degrees)
	40s	45s	50s	Dry	
Location 1 (1 liter)	57	59	60	169	2.4
Location 2 (3 liters)	39	41	43	149	2.5
Location 3 (5 liters)	46	48	49	132	2.5

FINDINGS

The findings from this research cover the impact of the factors tested and how they relate to ASTM 2177. All factors tested showed significant impacts across the ranges tested. Additional analysis broke the factors down by marking type to determine if the impacts were the same for each of the marking types tested. The findings are summarized below.

The recovery time factor can have a significant impact on the results, however it is variable by each marking type. The significance was shown in both pavement marking types 1 and 2, but there was no significant difference over the range within the ASTM spec for marking type 1, and only a slight difference at 40 seconds for marking type two. Both markings created a significant difference to the upper level of the accepted range in the European CEN standard. Marking types 3 and 4 did not see a significant impact on recovery time for the times studied. This likely indicates that the markings do most of their recovery quickly after the water is applied. This makes sense as both of these markings are designed for wet conditions.

The water application volume factor and more specifically the sprayer method versus the bucket method, it was found that in general the sprayer is not significantly different than the volumes within the range of the specification, even though the sprayers used a considerably lower volume than the bucket method and takes longer to apply the water. Across each marking type and slope the sprayers showed similarities in results when compared to water volumes of 2–5 liters.

Pavement marking slope has a significant impact on the results for all marking types. There was typically an increase in retroreflectivity from 1 degree to the 2.5 degree readings. This was a major finding given that the ASTM spec only mentions to note the slope used during testing.

The interaction effect of pavement marking slope and water application method had a significant impact on the results across each pavement marking type. The general finding across all of the data was that higher marking slopes with lower water volumes produced higher retroreflectivity than lower slopes with higher volumes. There were some overall exceptions from this general finding but that was more based on marking type having a slightly different effect with water volume.

The field data showed some similarities to the laboratory testing. It was found that in the field recovery time did not have an effect on the retroreflectivity within the range specified in the ASTM standard. The field data showed a less significant impact on water volume than the lab data showed. There are limitations with the field data, based on the small data set and the testing being done on worn markings; however the field data gave a sample of what happens when not all factors can be controlled.

RECOMMENDATIONS

The recommendation after the analysis of the data is that ASTM E 2177 is a test that can produce repeatable results; however a reduction in the range of acceptable volumes could produce a more consistent standardized test. Also a possible expansion on the importance of marking slope could help the consistency. Creating a range of accepted marking slopes for the test is a possible suggestion to be made and would reduce the significant difference between the lower and upper end of tested slopes.

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APPENDIX A: ANALYSIS 1

JMP output for the analysis of retroreflectivity data

Table A1. Response Retroreflectivity Summary of Fit.

RSquare	0.984194
RSquare Adj	0.983614
Root Mean Square Error	35.66847
Mean of Response	371.2178
Observations (or Sum Wgts)	4578

Table A2. REML Variance Component Estimates.

Random Effect	Var Ratio	Var Component	Std Error	95% Lower	95% Upper	Pct of Total
Whole Plots [marking type]	1.5899489	2022.796	1431.9187	-783.7646	4829.3565	61.389
Residual		1272.2396	27.090398	1220.7693	1327.0512	38.611
Total		3295.0355				100.000

-2 LogLikelihood = 45038.672828

Table A3. Fixed Effect Tests.

Source	Nparm	DF	DFDen	F Ratio	Prob > F
marking type	3	3	4.001	94.7298	0.0004*
pavement marking slope	3	3	4411	195.7837	<.0001*
recovery time	7	7	4411	9.4546	<.0001*
water app_vol	7	7	4411	64.5185	<.0001*
marking type*pavement marking slope	9	9	4411	169.4679	<.0001*
marking type*recovery time	21	21	4411	10.8969	<.0001*
marking type*water app_vol	21	21	4411	22.7935	<.0001*
pavement marking slope*recovery time	21	21	4411	0.6372	0.8943
pavement marking slope*water app_vol	21	21	4411	21.1817	<.0001*
recovery time*water app_vol	49	49	4411	0.9148	0.6425

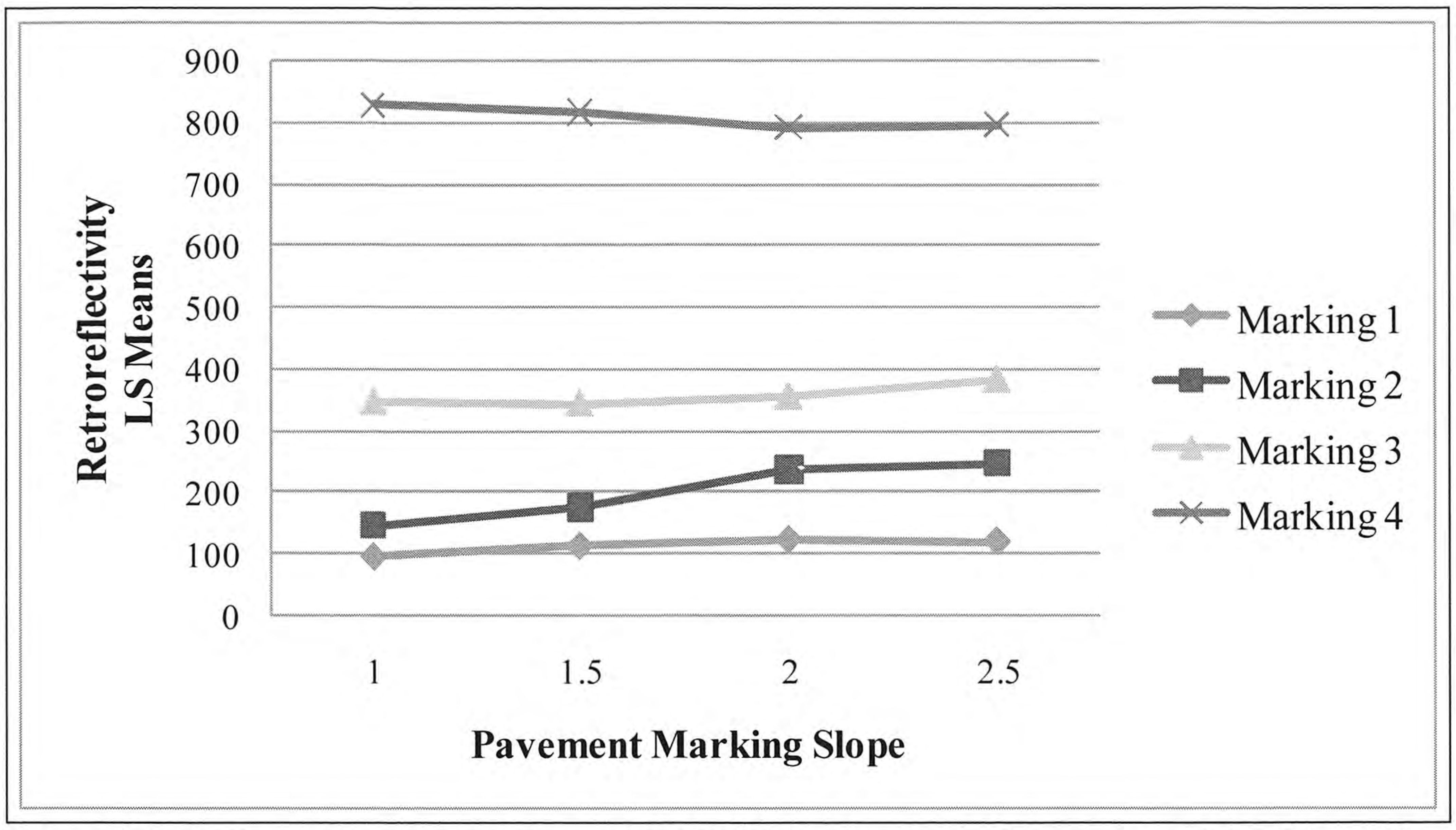

Figure A1. Interaction Plot between Marking Type and Pavement Marking Slope.

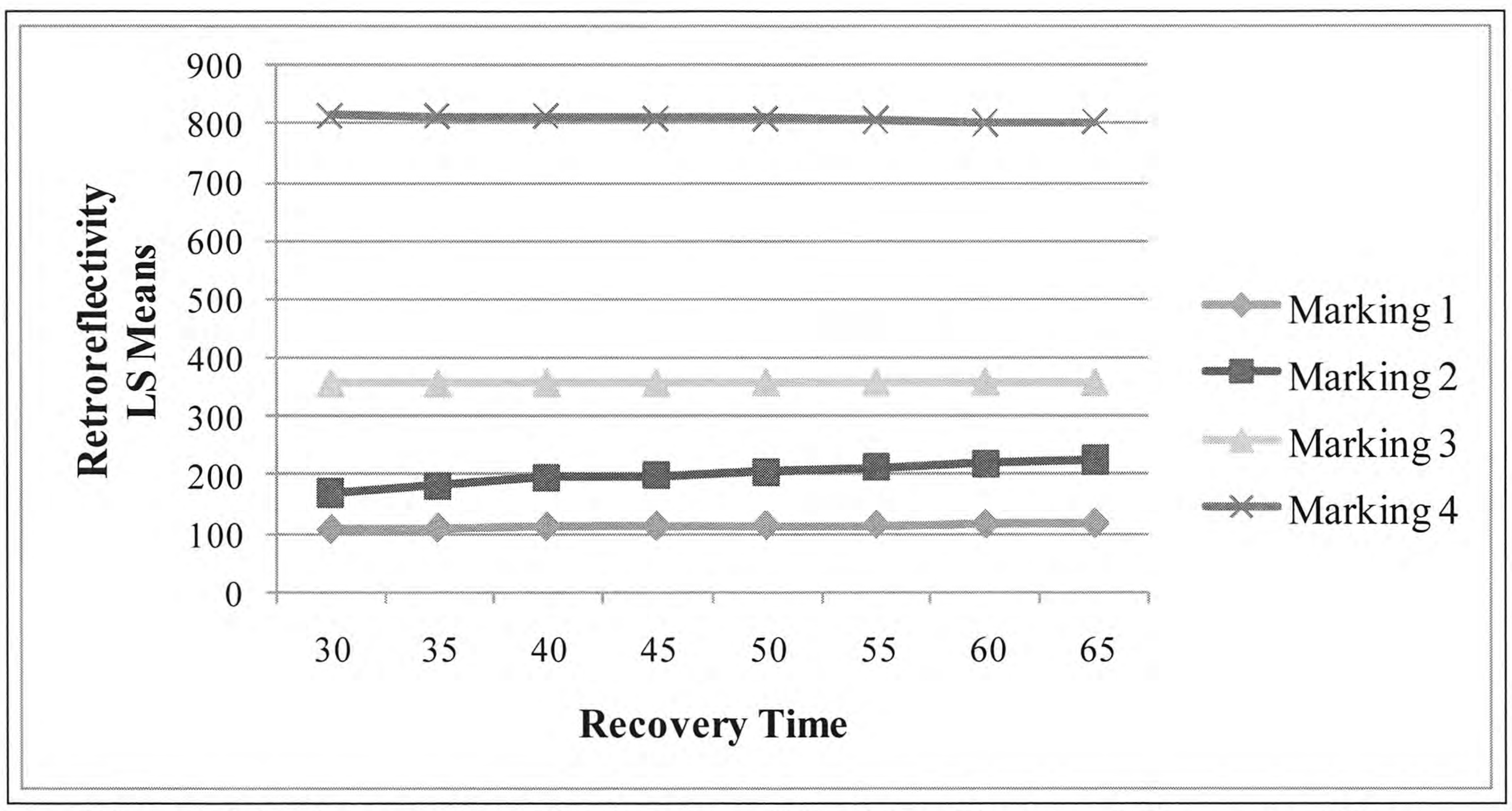

Figure A2. Interaction Plot between Marking Type and Recovery Time.

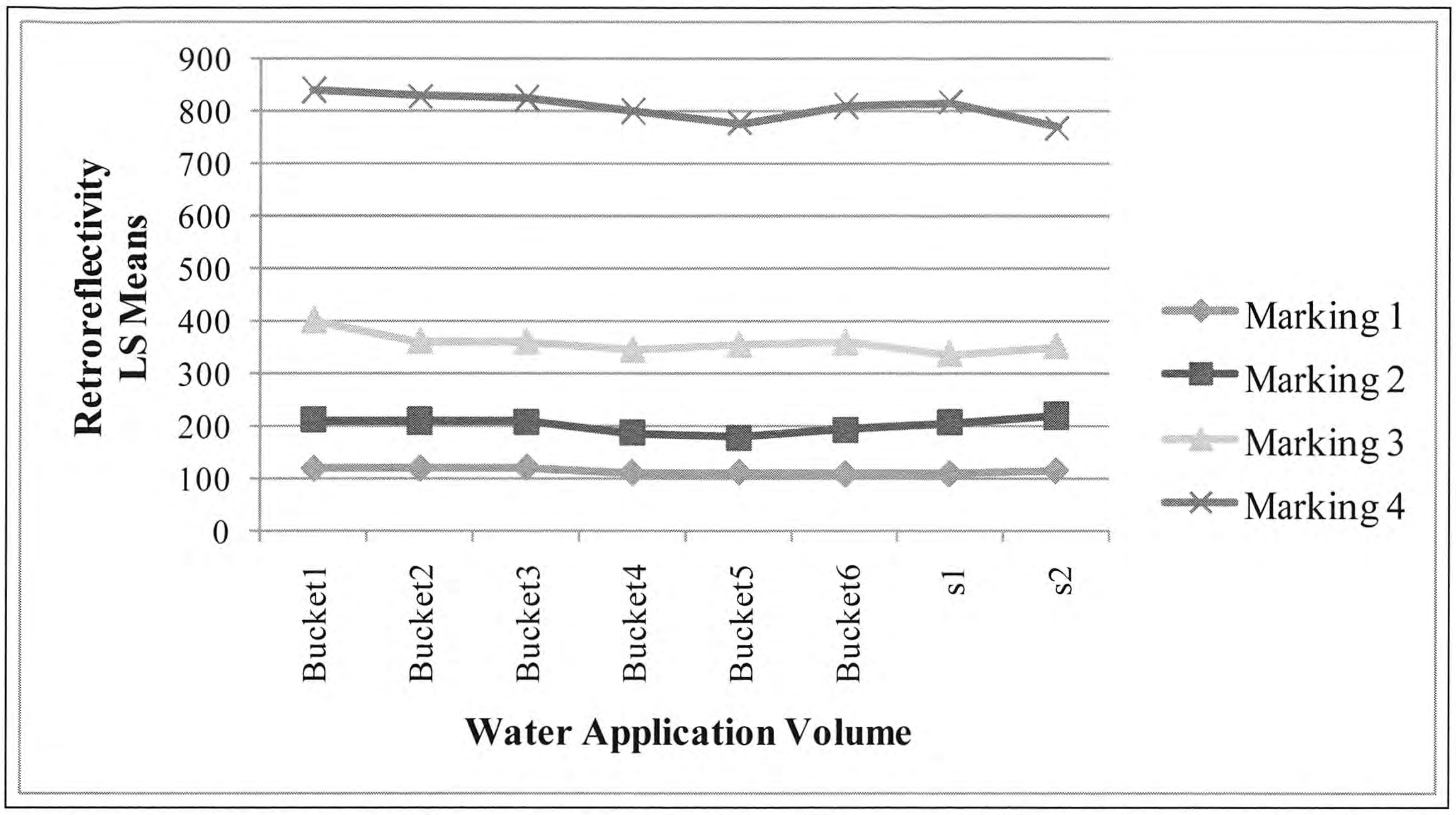

Figure A3. Interaction Plot between Marking Type and Water Application Volume.

APPENDIX B: ANALYSIS 2

JMP output for the analysis with three-way interactions

Table B1. Response Retroreflectivity Summary of Fit.

RSquare	0.988395
RSquare Adj	0.987177
Root Mean Square Error	31.55471
Mean of Response	371.2178
Observations (or Sum Wgts)	4578

Table B2. REML Variance Component Estimates.

Random Effect	Var Ratio	Var Component	Std Error	95% Lower	95% Upper	Pct of Total
Whole Plots [marking type]	2.027337	2018.6194	1428.6241	-781.4838	4818.7227	66.968
Residual		995.7	21.890126	954.1514	1040.0363	33.032
Total		3014.3194				100.000

-2 LogLikelihood = 42473.720809

Table B3. Fixed Effect Tests.

Source	Nparm	DF	DFDen	F Ratio	Prob > F
marking type	3	3	4.001	95.5013	0.0004*
pavement marking slop	3	3	4138	249.2198	<.0001*
recovery time	7	7	4138	11.8974	<.0001*
water app_vol	7	7	4138	82.1928	<.0001*
marking type*pavement marking slop	9	9	4138	199.3908	<.0001*
marking type*recovery time	21	21	4138	13.7965	<.0001*
marking type*water app_vol	21	21	4138	18.9830	<.0001*
pavement marking slop*recovery time	21	21	4138	0.8065	0.7146
pavement marking slop*water app_vol	21	21	4138	26.7803	<.0001*
recovery time*water app_vol	49	49	4138	1.2157	0.1451
marking type*pavement marking slop*water app_vol	63	63	4138	20.4936	<.0001*
marking type*recovery time*water app_vol	147	147	4138	1.0941	0.2104
marking type*pavement marking slop*recovery time	63	63	4138	0.6813	0.9746

APPENDIX C: ANALYSIS 3 MARKING TYPE 1

Table C1. Response Retroreflectivity marking type=1 Summary of Fit.

RSquare	0.924802
RSquare Adj	0.916948
Root Mean Square Error	8.584665
Mean of Response	109.3981
Observations (or Sum Wgts)	1143

Table C2. REML Variance Component Estimates.

Random Effect	Var Ratio	Var Component	Std Error	95% Lower	95% Upper	Pct of Total
Whole Plots	17.266037	1272.4462	1799.6635	-2254.894	4799.7866	94.525
Residual		73.696482	3.242736	67.732019	80.489177	5.475
Total		1346.1427				100.000

-2 LogLikelihood = 7838.1990151

Table C3. Fixed Effect Tests.

Source	Nparm	DF	DFDen	F Ratio	Prob > F
pavement marking slop	3	3	1033	578.8942	<.0001*
recovery time	7	7	1033	19.4085	<.0001*
water app_vol	7	7	1033	44.2076	<.0001*
pavement marking slop*recovery time	21	21	1033	1.5332	0.0586
pavement marking slop*water app_vol	21	21	1033	18.6689	<.0001*
recovery time*water app_vol	49	49	1033	0.2792	1.0000

Table C4. LSMeans Differences Tukey HSD.

Level						Least Sq Mean
65	A					115.04274
60	A	B				114.49772
55	A	B				113.59526
50	A	B	C			112.61278
45		B	C	D		111.61868
40			C	D		110.16065
35				D	E	108.48551
30					E	105.34110

Levels not connected by same letter are significantly different.

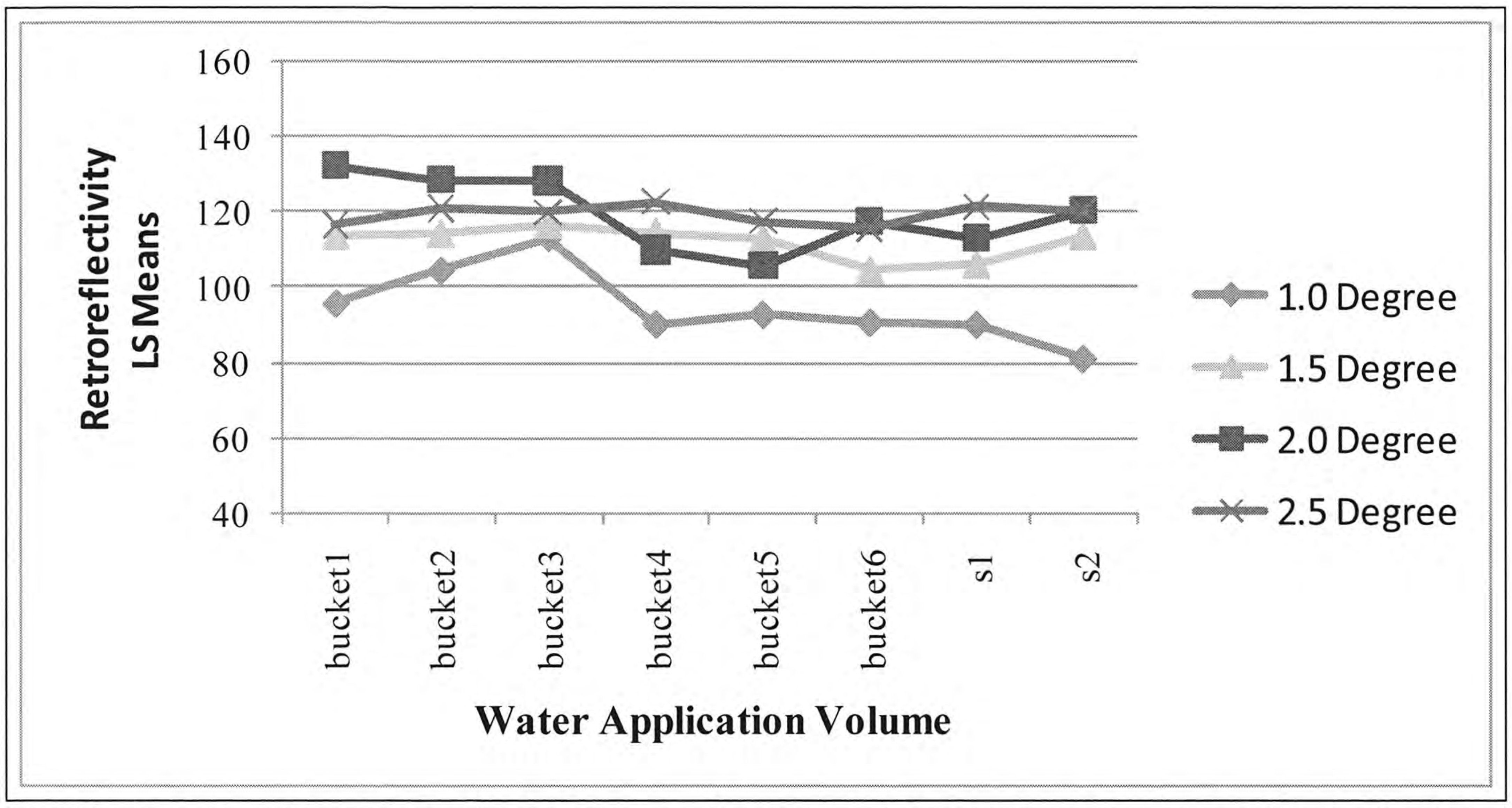

Figure C1. Interaction Plot of Water Application Volume vs. Marking Slope for Marking Type 1.

Table C5. LSMeans Differences Tukey HSD.

Level														Least Sq Mean
2,bucket1	A													132.28125
2,bucket2	A	B												128.71875
2,bucket3	A	B												128.31250
2.5,bucket4		B	C											122.75000
2.5,s1		B	C	D										121.50000
2.5,bucket2		B	C	D	E									121.09375
2.5,s2		B	C	D	E	F								120.59375
2,s2		B	C	D	E	F	G							120.50000
2.5,bucket3			C	D	E	F	G							120.12500
2,bucket6			C	D	E	F	G	H						117.50000
2.5,bucket5			C	D	E	F	G	H						117.46875
2.5,bucket1			C	D	E	F	G	H						116.65625
1.5,bucket3			C	D	E	F	G	H						116.47917
2.5,bucket6			C	D	E	F	G	H						115.84375
1.5,bucket4				D	E	F	G	H	I					114.44448
1.5,bucket2					E	F	G	H	I					114.04167
1.5,bucket1						F	G	H	I	J				113.43750
1.5,s2				D	E	F	G	H	I	J				113.43750
2,s1			C	D	E	F	G	H	I	J	K			113.18750
1,bucket3							G	H	I	J				113.14583
1.5,bucket5						F	G	H	I	J				112.90625
2,bucket4								H	I	J	K			110.00000
1.5,s1										J	K			106.09375
2,bucket5									I	J	K			105.87500
1.5,bucket6											K			104.50000
1,bucket2											K			104.43750
1,bucket1												L		95.56703
1,bucket5												L		92.70833
1,bucket6												L		90.64583
1,bucket4												L		90.22917
1,s1												L		90.06250
1,s2													M	80.87500

Levels not connected by same letter are significantly different.

APPENDIX D: ANALYSIS 3 MARKING TYPE 2

Table D1. Response Retroreflectivity marking type=2 Summary of Fit.

RSquare	0.78813
RSquare Adj	0.765017
Root Mean Square Error	34.61097
Mean of Response	192.8235
Observations (or Sum Wgts)	1099

Table D2. REML Variance Component Estimates.

Random Effect	Var Ratio	Var Component	Std Error	95% Lower	95% Upper	Pct of Total
Whole Plots	2.7725459	3321.2859	4699.2578	-5889.259	12531.831	73.493
Residual		1197.9192	53.869677	1098.9703	1310.9315	26.507
Total		4519.2051				100.000

-2 LogLikelihood = 10278.549223

Table D3. Fixed Effect Tests.

Source	Nparm	DF	DFDen	F Ratio	Prob > F
pavement marking slop	3	3	989	537.1984	<.0001*
recovery time	7	7	989	40.1551	<.0001*
water app_vol	7	7	989	15.9912	<.0001*
pavement marking slop*recovery time	21	21	989	0.0786	1.0000
pavement marking slop*water app_vol	21	21	989	3.0879	<.0001*
recovery time*water app_vol	49	49	989	0.1224	1.0000

Table D4. LSMeans Differences Tukey HSD.

Level							Least Sq Mean
65	A						224.87538
60	A						218.14773
55	A	B					212.36171
50		B	C				203.64665
45			C	D			197.70736
40				D	E		189.84204
35					E	F	180.21881
30						F	167.87959

Levels not connected by same letter are significantly different.

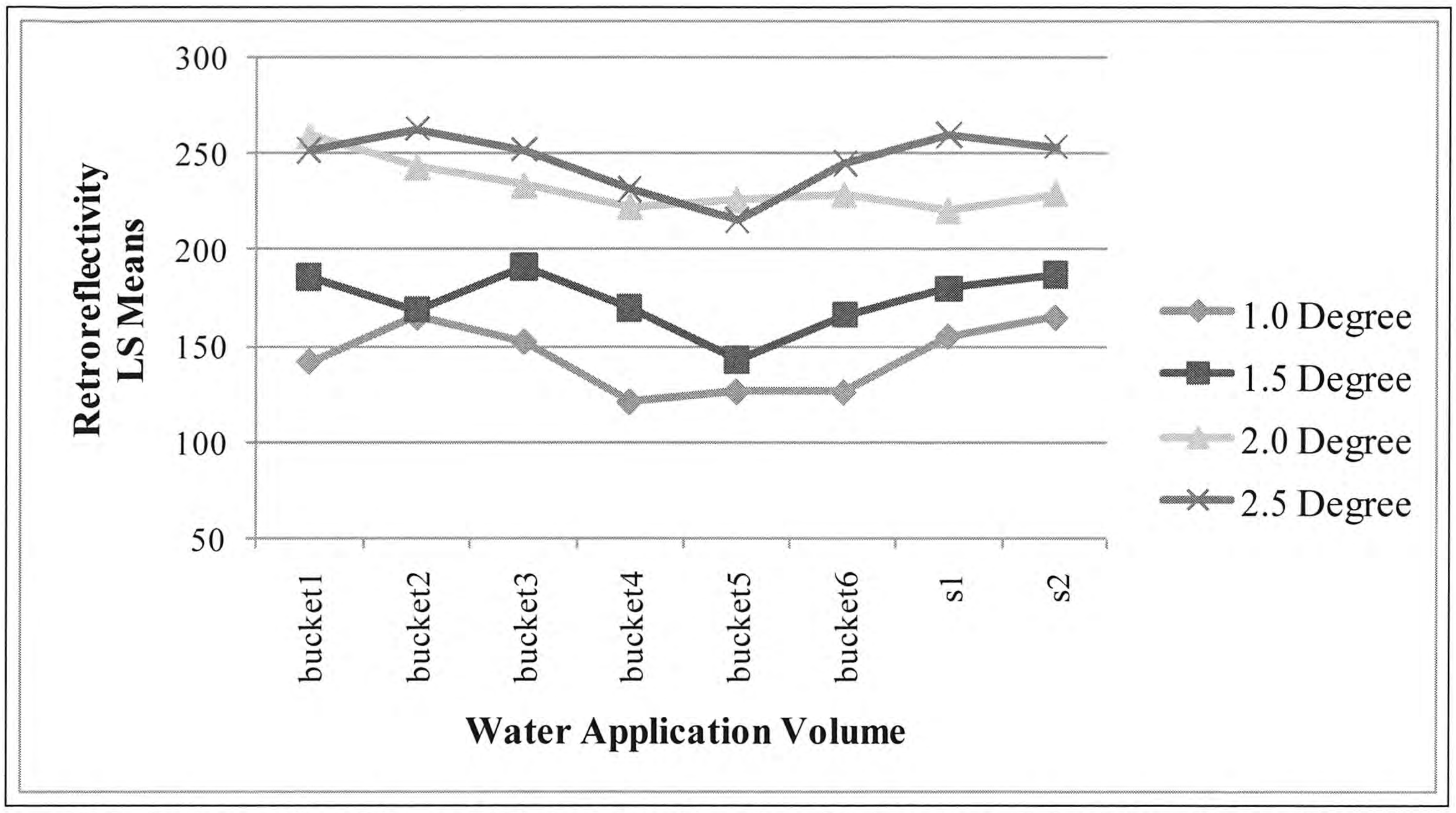

Figure D1. Interaction Plot of Water Application Volume vs. Marking Slope for Marking Type 2.

Table D5. LSMeans Differences Tukey HSD.

Level														**Least Sq Mean**
2.5,bucket2	A													263.18750
2.5,s1	A	B												259.71875
2,bucket1	A	B												259.37500
2.5,s2	A	B	C											253.53125
2.5,bucket3	A	B	C											252.28125
2.5,bucket1	A	B	C											251.87500
2.5,bucket6	A	B	C	D										245.15625
2,bucket2	A	B	C	D										243.31250
2,bucket3	A	B	C	D										233.62500
2.5,bucket4	A	B	C	D										231.40625
2,s2	A	B	C	D	E									229.44976
2,bucket6	A	B	C	D	E									228.68750
2,bucket5	A	B	C	D	E	F								225.93750
2,bucket4			C	D										222.21875
2,s1		B	C	D	E	F	G							220.37500
2.5,bucket5				D	E	F	G							215.53125
1.5,bucket3					E	F	G	H						191.16667
1.5,s2						F	G	H						187.03125
1.5,bucket1							G	H						185.81250
1.5,s1								H	I					179.90625
1.5,bucket4								H	I	J				169.78244
1.5,bucket2								H	I	J				168.81250
1.5,bucket6								H	I	J				166.04459
1,bucket2								H	I	J				164.97675
1,s2								H	I	J				164.91667
1,s1									I	J	K			155.04167
1,bucket3									I	J	K	L		152.29167
1.5,bucket5										J	K	L	M	142.78125
1,bucket1										J	K	L	M	141.85175
1,bucket5											K	L	M	126.03125
1,bucket6												L	M	125.91392
1,bucket4													M	120.68750

Levels not connected by same letter are significantly different.

APPENDIX E: ANALYSIS 3 MARKING TYPE 3

Table E1. Response Retroreflectivity Marking Type=3 Summary of Fit.

RSquare	0.843714
RSquare Adj	0.827895
Root Mean Square Error	17.65582
Mean of Response	356.3469
Observations (or Sum Wgts)	1176

Table E2. REML Variance Component Estimates.

Random Effect	Var Ratio	Var Component	Std Error	95% Lower	95% Upper	Pct of Total
Whole Plots	4.3935071	1369.5794	1937.5057	-2427.932	5167.0905	81.459
Residual		311.72804	13.50244	286.86851	339.98216	18.541
Total		1681.3074				100.000

-2 LogLikelihood = 9614.7071746

Table E3. Fixed Effect Tests.

Source	Nparm	DF	DFDen	F Ratio	Prob > F
pavement marking slop	3	3	1066	287.5617	<.0001*
recovery time	7	7	1066	0.7435	0.6351
water app_vol	7	7	1066	183.3963	<.0001*
pavement marking slop*recovery time	21	21	1066	0.0261	1.0000
pavement marking slop*water app_vol	21	21	1066	53.7572	<.0001*
recovery time*water app_vol	49	49	1066	0.0264	1.0000

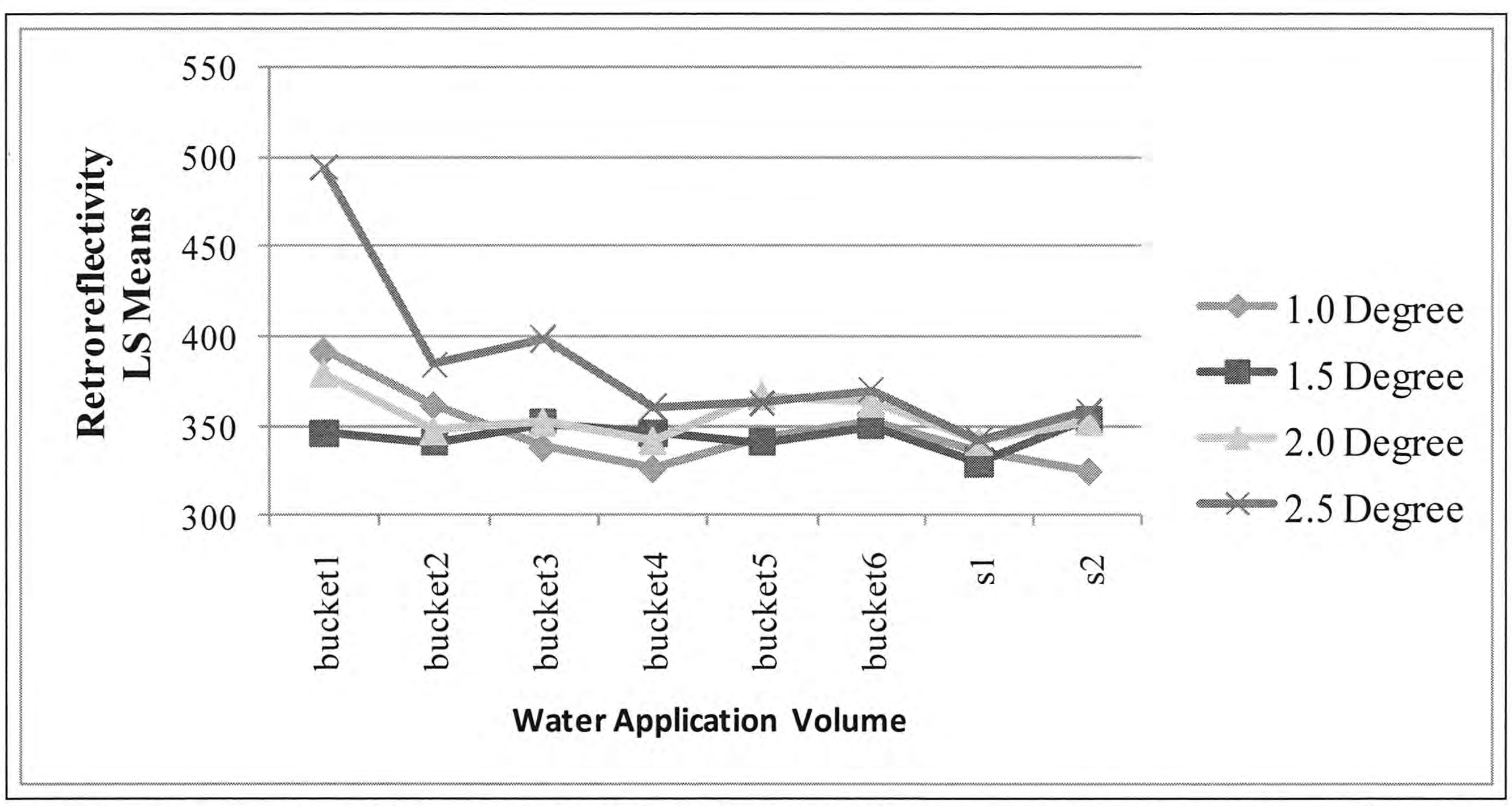

Figure E1. Interaction Plot of Water Application Volume vs. Marking Slope for Marking Type 3.

Table E4. LSMeans Differences Tukey HSD.

Level																	Least Sq Mean
2.5,bucket1	A																494.15625
2.5,bucket3		B															398.46875
1,bucket1		B	C														391.72917
2.5,bucket2		B	C	D													384.25000
2,bucket1			C	D	E												379.31250
2.5,bucket6				D	E	F											369.81250
2,bucket5					E	F	G										367.15625
2,bucket6					E	F	G	H	I	J							362.75000
2.5,bucket5					E	F	G	H									362.68750
1,bucket2						F	G	H									361.72917
2.5,bucket4						F	G	H	I								360.34375
2.5,s2						F	G	H	I	J	K						358.34375
1.5,s2						F	G	H	I	J	K	L					353.65625
2,s2						F	G	H	I	J	K	L	M				352.81250
2,bucket3							G	H	I	J	K	L					352.36715
1,bucket6							G	H	I	J	K	L					352.10417
1.5,bucket3								H	I	J	K	L					351.50000
1.5,bucket6								H	I	J	K	L	M				350.09375
2,bucket2								H	I	J	K	L	M				347.46875
1.5,bucket1									I	J	K	L	M				345.89583
1.5,bucket4									I	J	K	L	M				345.79167
1,bucket5										J	K	L	M	N			343.58333
2.5,s1										J	K	L	M	N			342.68750
2,bucket4											K	L	M	N			342.09375
2,s1									I	J	K	L	M	N	O	P	342.00000
1.5,bucket2												L	M	N			341.16667
1.5,bucket5												L	M	N	O		340.68750
1,bucket3												L	M	N	O	P	338.62500
1,s1													M	N	O	P	335.72917
1.5,s1														N	O	P	329.56250
1,bucket4															O	P	326.35417
1,s2																P	325.25000

Levels not connected by same letter are significantly different.

APPENDIX F: ANALYSIS 3 MARKING TYPE 4

Table F1. Response Retroreflectivity Marking Type=4 Summary of Fit.

RSquare	0.628406
RSquare Adj	0.590184
Root Mean Square Error	46.19726
Mean of Response	813.3667
Observations (or Sum Wgts)	1159

Table F2. REML Variance Component Estimates.

Random Effect	Var Ratio	Var Component	Std Error	95% Lower	95% Upper	Pct of Total
Whole Plots	0.9493018	2025.9874	2870.3945	-3599.986	7651.9607	48.700
Residual		2134.1869	93.187939	1962.702	2329.2899	51.300
Total		4160.1744				100.000

-2 LogLikelihood = 11485.114392

Table F3. Fixed Effect Tests.

Source	Nparm	DF	DFDen	F Ratio	Prob > F
pavement marking slop	3	3	1049	32.7262	<.0001*
recovery time	7	7	1049	1.6208	0.1256
water app_vol	7	7	1049	27.5457	<.0001*
pavement marking slop*recovery time	21	21	1049	1.1365	0.3022
pavement marking slop*water app_vol	21	21	1049	31.2288	<.0001*
recovery time*water app_vol	49	49	1049	1.8343	0.0005*

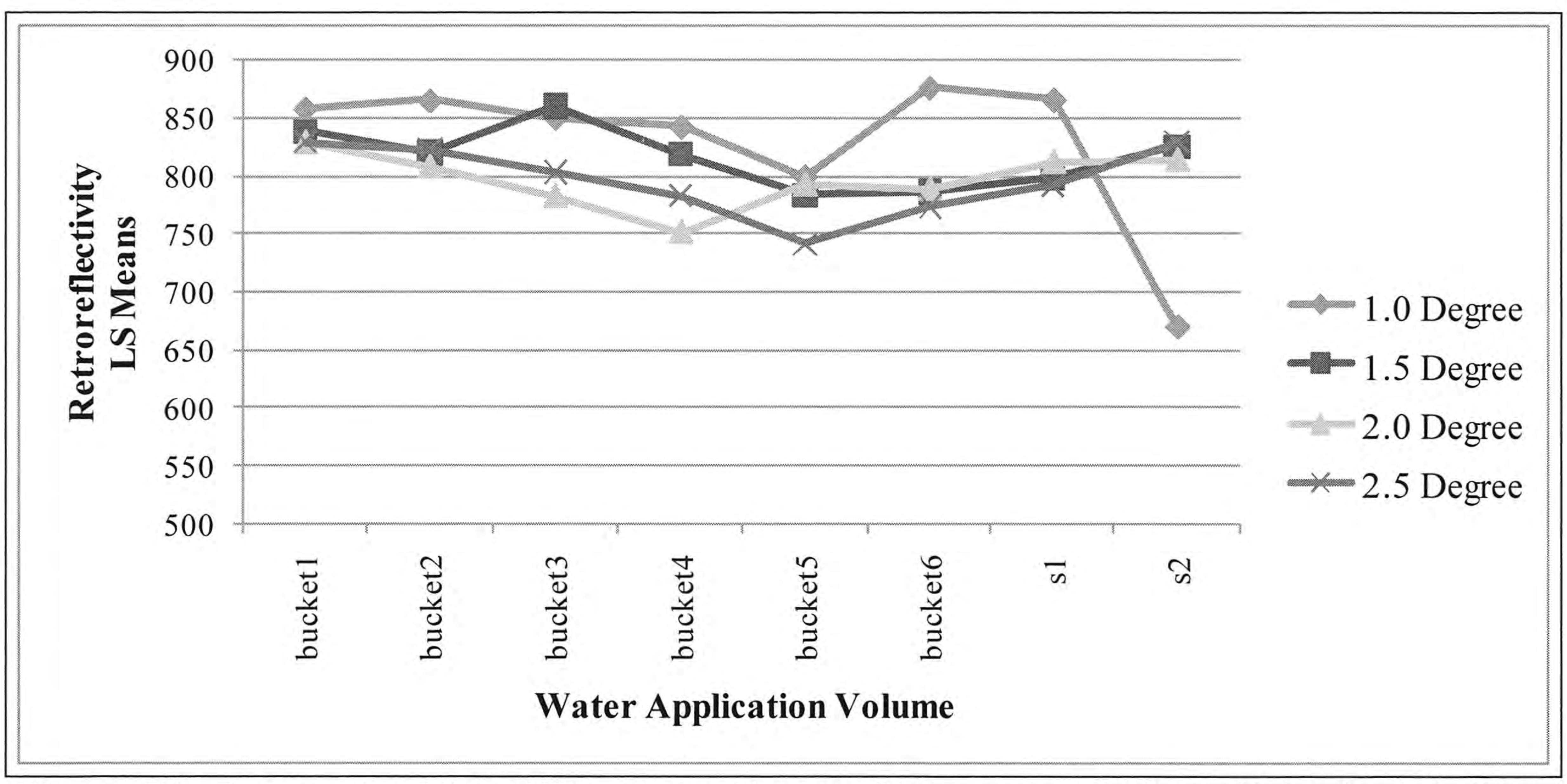

Figure F1. Interaction Plot of Water Application Volume vs. Marking Slope for Marking Type 4.

Table F4. LSMeans Differences Tukey HSD.

Level															Least Sq Mean
1,bucket6	A														876.56250
1,s1	A	B													866.20833
1,bucket2	A	B	C												865.66667
1.5,bucket3	A	B	C	D											860.60417
1,bucket1	A	B	C	D											858.54167
1,bucket3	A	B	C	D	E										850.29167
1,bucket4	A	B	C	D	E	F									842.70833
1.5,bucket1		B	C	D	E	F	G								838.81250
2.5,bucket1		B	C	D	E	F	G	H							829.56250
2.5,s2		B	C	D	E	F	G	H							829.28125
2,bucket1		B	C	D	E	F	G	H							828.34375
1.5,s2			C	D	E	F	G	H	I						826.06250
2.5,bucket2				D	E	F	G	H	I	J					822.90625
1.5,bucket2					E	F	G	H	I						820.14583
1.5,bucket4					E	F	G	H	I	J					818.85417
2,s2				D	E	F	G	H	I	J	K				813.50000
2,s1				D	E	F	G	H	I	J	K				811.68750
2,bucket2						F	G	H	I	J	K				807.96875
2.5,bucket3						F	G	H	I	J	K				803.18750
1,bucket5								H	I	J	K				799.79167
1.5,s1							G	H	I	J	K				798.96875
2,bucket5						F	G	H	I	J	K	L	M		792.37500
2.5,s1								H	I	J	K	L			792.31250
2,bucket6							G	H	I	J	K	L	M		788.68750
1.5,bucket6								H	I	J	K	L			786.65625
1.5,bucket5									I	J	K	L	M		783.37500
2.5,bucket4									I	J	K	L	M		782.71875
2,bucket3										J	K	L	M		782.23473
2.5,bucket6											K	L	M		773.53125
2,bucket4												L	M		751.31250
2.5,bucket5													M		741.32908
1,s2														N	669.81250

Levels not connected by same letter are significantly different.

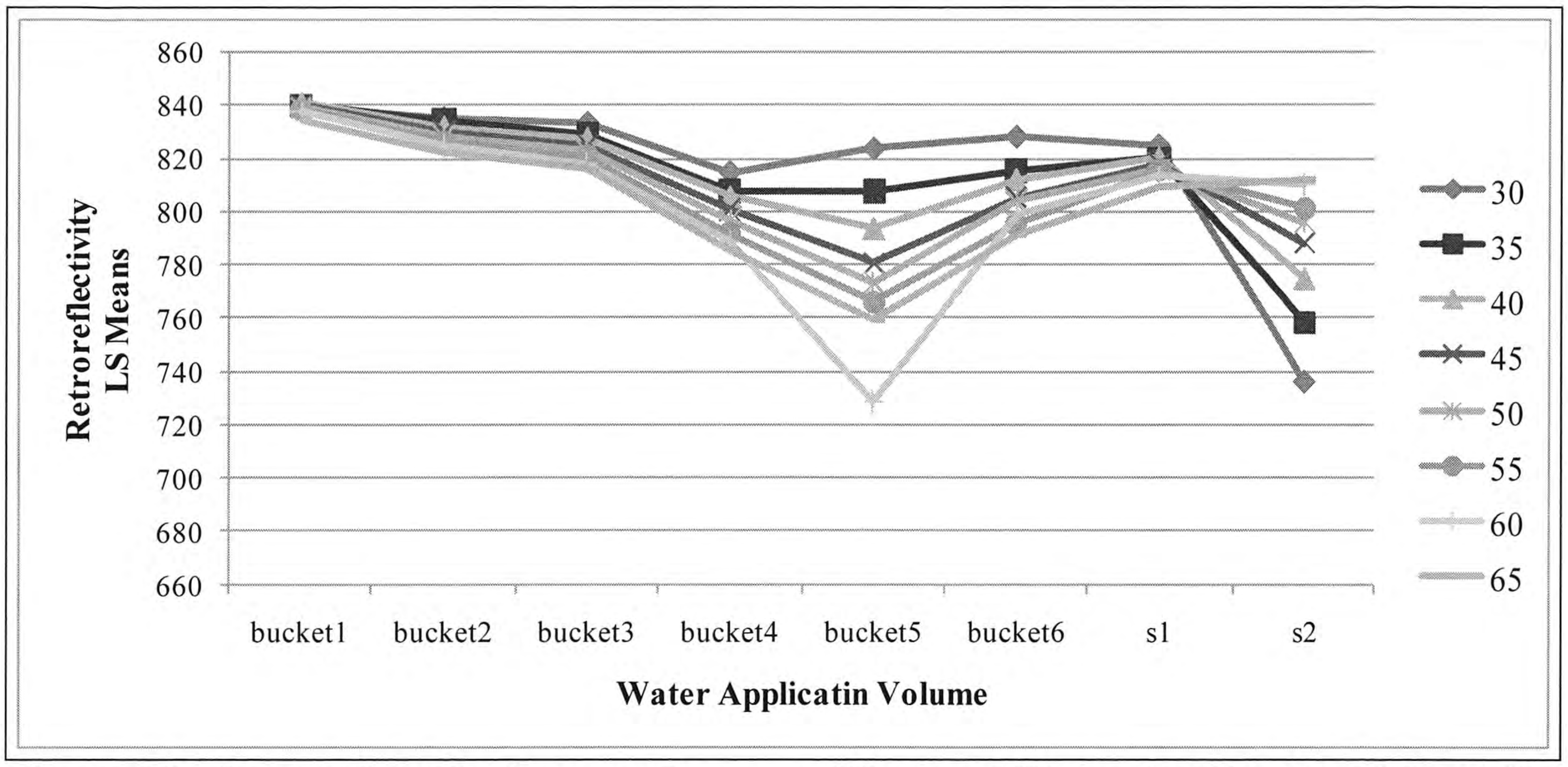

Figure F2. Interaction Plot between Recovery Time and Water Application Volume for Marking Type 4.

Table F5. LSMeans Differences Tukey HSD.

Level									Least Sq Mean
40,bucket1	A								840.94995
35,bucket1	A								840.16880
45,bucket1	A								840.08582
30,bucket1	A	B							839.49963
50,bucket1	A	B							839.26557
55,bucket1	A	B							838.33783
60,bucket1	A	B	C						837.58301
30,bucket2	A	B	C						835.17515
65,bucket1	A	B	C						834.63023
35,bucket2	A	B	C						834.29432
30,bucket3	A	B	C						833.32581
40,bucket2	A	B	C						832.07547
45,bucket2	A	B	C	D					830.21134
35,bucket3	A	B	C	D					829.23708
50,bucket2	A	B	C	D					828.39109
30,bucket6	A	B	C	D	E				828.06564
40,bucket3	A	B	C	D					827.71534
55,bucket2	A	B	C	D					826.36335
30,s1	A	B	C	D	E	F			824.92241
45,bucket3	A	B	C	D	E				824.86905
60,bucket2	A	B	C	D	E				824.85853
30,bucket5	A	B	C	D	E	F			823.68543
50,bucket3	A	B	C	D	E				822.90683
65,bucket2	A	B	C	D	E	F			822.00575

Table F5 (Continued). LSMeans Differences Tukey HSD.

Level									Least Sq Mean
40,s1	A	B	C	D	E	F			820.45542
55,bucket3	A	B	C	D	E	F			820.45325
35,s1	A	B	C	D	E	F			820.29348
60,bucket3	A	B	C	D	E	F			818.27095
45,s1	A	B	C	D	E	F			818.16704
50,s1	A	B	C	D	E	F			816.03730
65,bucket3	A	B	C	D	E	F			815.85780
55,s1	A	B	C	D	E	F			815.62969
35,bucket6	A	B	C	D	E	F			815.31171
30,bucket4	A	B	C	D	E	F			814.74546
60,s1	A	B	C	D	E	F			813.27961
40,bucket6	A	B	C	D	E	F			812.34865
65,s2	A	B	C	D	E	F			811.84526
60,s2	A	B	C	D	E	F			810.36815
65,s1	A	B	C	D	E	F			809.56922
35,bucket4	A	B	C	D	E	F			807.56463
35,bucket5	A	B	C	D	E	F			807.55650
40,bucket4	A	B	C	D	E	F			805.64578
45,bucket6	A	B	C	D	E	F			805.49777
50,bucket6	A	B	C	D	E	F	G		803.93052
55,s2	A	B	C	D	E	F	G		801.46823
45,bucket4	A	B	C	D	E	F	G		800.73165
60,bucket6	A	B	C	D	E	F	G		799.23534
50,bucket4	A	B	C	D	E	F	G		796.51140
50,s2	A	B	C	D	E	F	G	H	795.87584
55,bucket6	A	B	C	D	E	F	G	H	795.46042
40,bucket5	A	B	C	D	E	F	G	H	793.96845
55,bucket4	A	B	C	D	E	F	G	H	791.53367
65,bucket6	A	B	C	D	E	F	G	H	791.02495
60,bucket4	A	B	C	D	E	F	G	H	789.17884
45,s2	A	B	C	D	E	F	G	H	788.06808
65,bucket4	A	B	C	D	E	F	G	H	785.27606
45,bucket5	A	B	C	D	E	F	G	H	780.80506
40,s2		B	C	D	E	F	G	H	774.98147
50,bucket5			C	D	E	F	G	H	773.30032
55,bucket5				D	E	F	G	H	765.95521
65,bucket5					E	F	G	H	759.80291
35,s2						F	G	H	758.31952
30,s2							G	H	736.38596
60,bucket5								H	728.66763

Levels not connected by same letter are significantly different.

CALIBRATION OF PAVEMENT PERFORMANCE PREDICTION MODELS

Prepared for
Undergraduate Transportation Scholars Program

by

Jennifer L. Tovar
Civil and Environmental Engineering
Brigham Young University

Professional Mentor
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Program Sponsored by:
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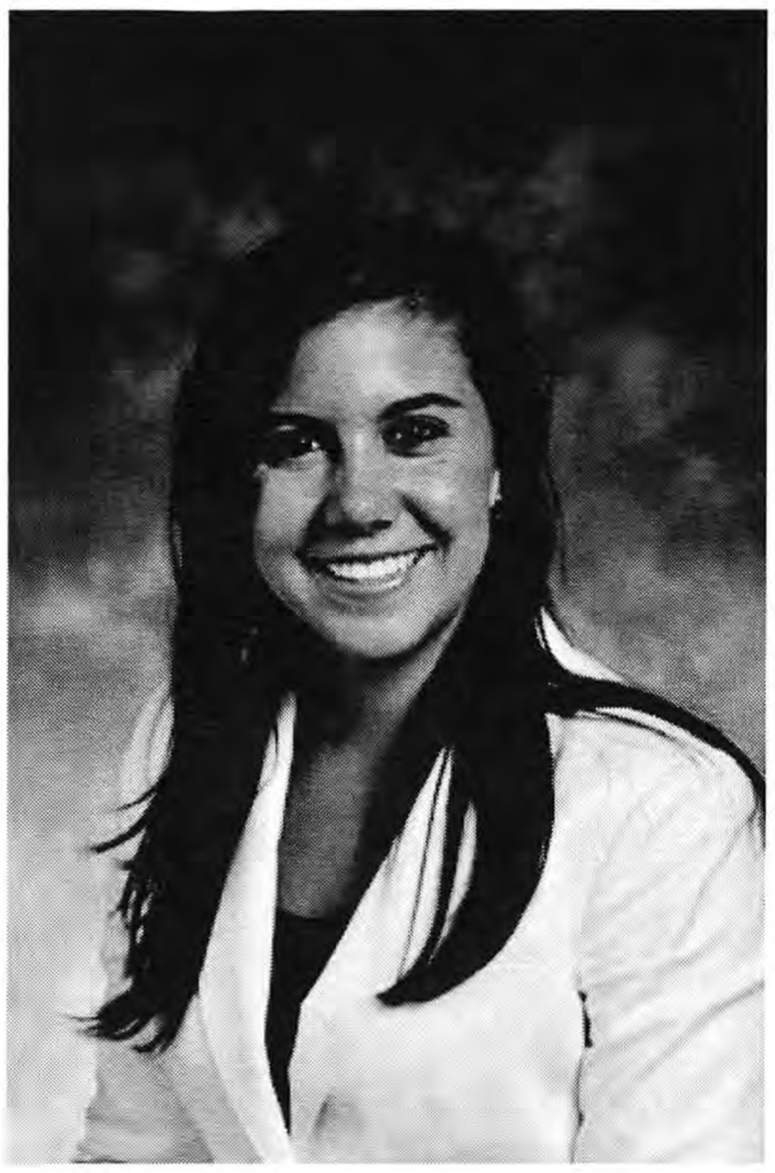
August 6, 2010



STUDENT BIOGRAPHY

Jennifer L. Tovar is a fourth-year student attending Brigham Young University in Provo, Utah. She will graduate in April 2012 with a degree in Civil Engineering and a minor in Music and Business Management. Jennifer has been awarded the Department of Civil & Environmental Engineering Scholarship Society Undergraduate Scholarship Award and BYU Academic Scholarships. Jennifer is an active member of the American Society of Civil Engineers (ASCE), and currently serves as the president-elect for the Society of Women Engineers (SWE), and the secretary for Earthquake Engineering Research Institute (EERI).

Jennifer participates in a wide range of activities at BYU. As an athlete, she is a member of the BYU Racquetball Team and participates in various intramural teams, including girls soccer, co-ed soccer, girls football, and volleyball. As a musician, Jennifer took piano lessons at BYU for a year after participating in the Texas Christian University (TCU) program for 7 years, was the first chair clarinetist in the University Orchestra her freshman year, sang for a year in BYU's University Chorale, took an organ class, and served as choir director for her church. In addition to her school activities, Jennifer enjoys spending time with her family, playing sports, camping, reading, and shopping. After obtaining her undergraduate degree, Jennifer plans on getting her master's degree in civil engineering.

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Jennifer L. Tovar would like to express her appreciation to Siamak Saliminejad, a PhD student, for familiarizing her with the research project, developing the computer program used in this project, and for his continued assistance as they worked together to complete the research project.

SUMMARY

Prediction of pavement condition in the future is a key component of pavement management systems and processes. Accurate prediction models are necessary for aiding decision makers in

estimating remaining service life of pavements and planning future maintenance and rehabilitation activities. The Texas Department of Transportation (TxDOT) developed its existing prediction models in 1990s based on engineering judgment due to the lack of field performance data at that time. These models use S-shaped relationships between pavement age and performance. This report presents the process and results of calibrating these models for asphalt roads in Texas using actual field data. Calibrating new prediction models using data collected minimizes the error between actual data and prediction models.

Calibration was completed on four zones derived from data for different Texas counties. These new prediction models provide more accurate information about when distresses (e.g., alligator cracking) occur in asphalt pavement. This research project is a part of a parent project focused on calibrating prediction models for all roads in Texas. A computer program was developed to calibrate new coefficients used to calculate new prediction models. The mathematical calculations and equations for the prediction models are based on prediction models developed by TxDOT engineers in the 1980s and1990s.

The first part of this project was to divide the data provided by TxDOT to narrow the amount of data used in iterative calculations. The data were then organized and prepared for processing in the updated program. Next, the data were processed in the program producing calibration coefficients. Using these coefficients, calculations were made producing calibrated prediction models. To evaluate the calibrated models, graphs of the corresponding prediction models were prepared and reviewed. After reviewing preliminary results, changes were made and new coefficients were produced along with corresponding distress and condition score graphs. Individual distress graphs were also prepared for one zone.

The calibrated prediction models were found to be more accurate than the original prediction models. In most of the calibrated models, the model error has decreased 10–15 percent from the original prediction models. Minimizing this error makes these prediction models very useful in planning future maintenance and rehabilitation projects and in planning for project funding. Still, there is room for improvement. Some prediction models lack adequate data and can be improved with more pavement data collection.

TABLE OF CONTENTS

Student Biography 194
Acknowledgments 194
Summary 194
Table of Contents 196
Introduction 197
Research Process 200
- Literature Review 200
- Traffic Loading Analysis 201
  - Data Organization: Five Divisions 202
  - First Division: Four Climate & Subgrade Zones 203
  - Second Division: Three Pavement Families 204
  - Third Division: Four Treatment Types 204
  - Fourth Division: Traffic Loading Levels 205
  - Fifth Division: Eight Distress Types & Scores 205
- Data Preparation 205
- Run Calibration Analysis 205

Results 206
- Coefficient Tables 206
- Individual Distress Graphs 208
- Combined Distress Graphs 208
- Comparing Results 210
- Discussion of Results 211

Conclusion 211
Recommendations 211
References 212
Appendix: Coefficient Tables 213

INTRODUCTION

Roadway pavements develop distresses as a result of environmental effects and traffic loading. If remained untreated, these distresses eventually lead to structural and functional failures. Such distresses include, but are not limited to fatigue (alligator) cracking, block cracking, rutting, and bleeding. To keep roads properly maintained, it is helpful to predict when different distresses will occur in the pavement. This information is particularly useful to the Texas Department of Transportation (TxDOT) because it allows them to make more accurate plans for future repairs. Furthermore, pavement performance prediction models can help TxDOT justify funding for maintenance and rehabilitation (M&R) projects.

In the 1980s and 1990s, TxDOT developed prediction models for use in their Pavement Management Information System (PMIS). More specifically, these predication models were developed as sigmoidal functions, a function that, represented as a curve, shows exponential patterns at the beginning and end of the curve and linear patterns in the middle. These developed models predict the density of the distress in a 0.5-mile section of pavement at any age (i.e., years after construction). As shown in Equation 1, these models have the following general form (*1*):

$$L_i = \alpha e^{-\left[\left(\frac{\chi \varepsilon \sigma \rho}{Age_i}\right)^{\beta}\right]} \tag{1}$$

where:

L	=	level of distress
χ	=	traffic loading
Age_i	=	age of pavement (years since construction or last M&R activity)
ε	=	climate region
β	=	slope factor
σ	=	subgrade type
ρ	=	prolongation factor
α	=	maximum loss factor

Equation 1 is depicted graphically in Figure 1.

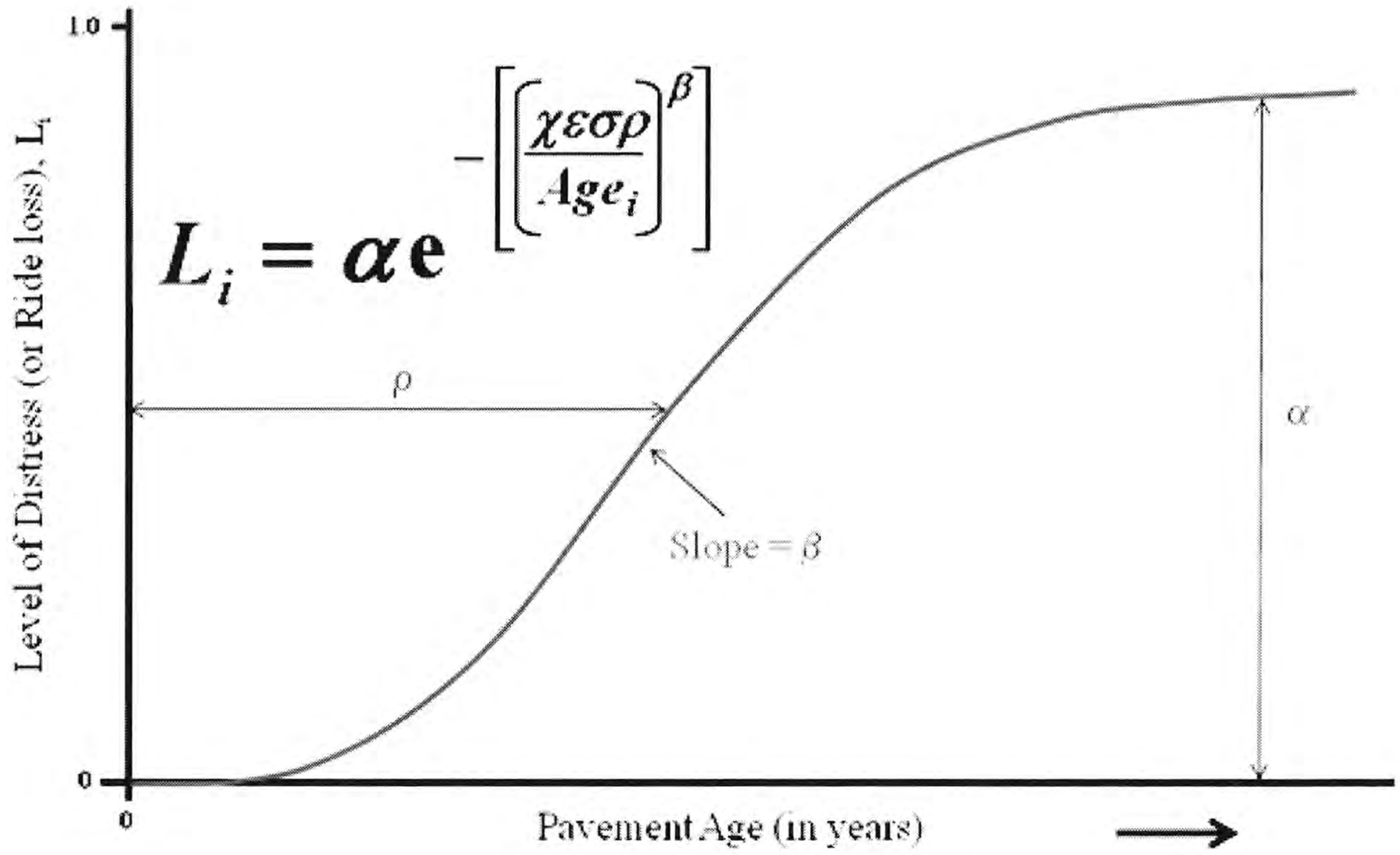

Figure 1. PMIS's Pavement Performance Prediction Model (*1*).

In the 1980s-1990s there was no sufficient data available on pavement factors and distresses to develop and validate these models. Thus, TxDOT's engineers and researchers used their best judgment to create the original pavement prediction models. Subsequently, extensive data have been collected on pavement distresses. Comparing recent data to the original prediction models revealed large discrepancies. This created a need for the calibration of new prediction models and thus, a need for this project.

To simplify the calibration process and reduce the number of calibration coefficients, the factors of traffic loading (χ), climate region (ε), and subgrade type (σ) were grouped together to form a new coefficient, A. Making this adjustment requires the data to be divided into uniform families based on these factors. The ultimate purpose of the calibration is to minimize the error between the measured data and the predicted distress values. A computer program has been developed to automate this calibration process. The computer program generates new coefficients (A,α, and β) for the pavement prediction models. These new coefficients provide TxDOT necessary tools to accurately predict when distresses will occur in asphalt pavement.

After coefficients are calibrated by the computer program, further calculations are necessary to give TxDOT graphical representations of the prediction models. First, L_i is calculated for each distress using a modified version of Equation 1. Then, the L_i values are converted to 0-1 utility values as shown in Equation 2. U_i ranges between zero and 1.0 and represents the quality of a pavement in terms of overall usefulness (e.g., a U_i of 1.0 indicates that distress type *i* is not present and thus is most useful).

$$U_i = \begin{cases} 1.0 & when\ L_i = 0 \\ 1 - \alpha e^{-\left(\frac{\rho}{L_i}\right)^{\beta}} & when\ L_i > 0 \end{cases} \tag{2}$$

Next, a 0–100 distress score (DS) (with 100 representing distress-free pavement) and a 0–100 condition score (CS) (with 100 representing perfectly smooth pavement) are computed. Distress score is calculated by multiplying 100 by all the U values for each distress type (Equation 3). Distress score is a numerical number given to each section of pavement to represent what percentage of the pavement is distressed. A "perfect" distress score is 100 and lowers over time as distresses develop in pavement.

$$DS = 100 \times \prod_{i=1}^{n} U_i \tag{3}$$

Condition score is calculated by multiplying the distress score by ride score (Equation 4). Condition score is useful because it produces a more accurate function of the roads deterioration because the roughness of the road (ride score) is accounted for.

$$CS = U_{ride} \times DS \tag{4}$$

Again, these equations are used to produce values used in graphical representations of prediction models. These graphical models are presented to TxDOT to help in future M&R projects and budget planning.

As stated earlier, the purpose of the calibration process is to determine a new set of values for the model coefficients to minimize the difference between predicted and observed performance. This can be expressed as an objective function as shown in Equation 5.

$$Minimize\ x\ \sum_{g \in G}\left(P_p(c_g) - P_a\right)^2 \tag{5}$$

where:

c_g = set of coefficient values that minimize the difference P_p and P_a
P_p = predicted performance
P_a = actual performance

Because of the large number of model coefficients (more than 1000 distress coefficients are used for various combinations of distress types, pavement types, and M&R types) and the massive data that exists in PMIS, it was necessary to automate the calibration process (to the maximum possible extent).

A computer program has been developed to read PMIS data for any given zone, main division of data, and search for the model coefficient values that achieve the above objective function. Figure 2 shows the main input screen of the model calibration software tool.

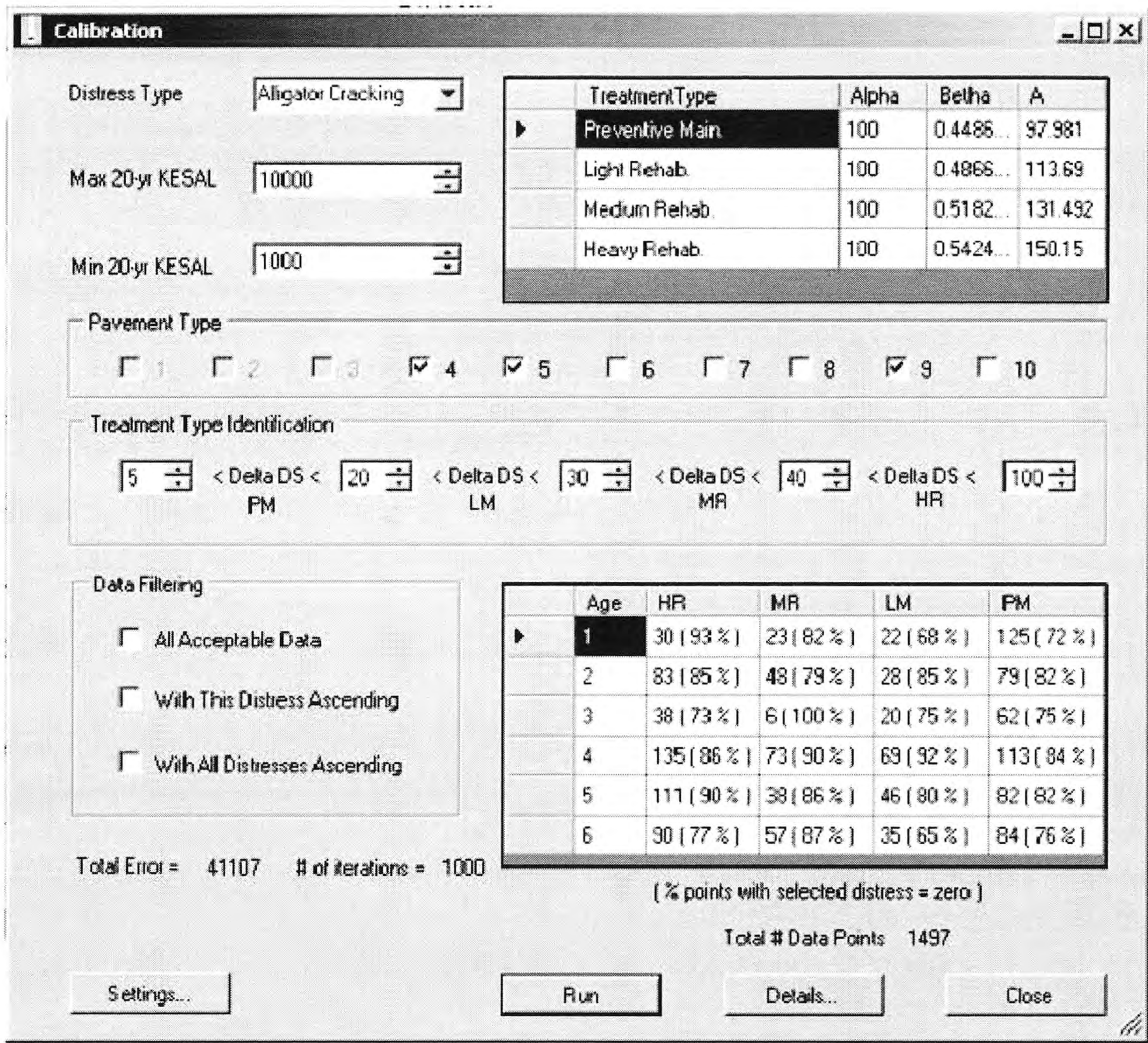

Figure 2. Main Input Screen for the PMIS Model Calibration Tool.

This computer program, developed at Texas A&M University, is exclusive to flexible pavements. Other universities were responsible for calibrating coefficients for other pavement structures, such as rigid pavements.

RESEARCH PROCESS

This section discusses the process implemented to complete this research project. Different tasks were accomplished that led to the completion of the research project.

Literature Review

A literature review was completed to obtain a greater understanding of how pavement performance prediction models operate.

Shahin discussed uses of prediction models (*2*). Prediction models are used both on the network and project levels. On the network level prediction models are used for condition forecasting, budget planning, inspection scheduling, and work planning. Studying the effects of various budget levels on future pavement condition ranks top among important network uses of prediction models. On the project level prediction models are used to select specific rehabilitation alternatives to meet expected traffic and climate conditions. Both levels use prediction models to analyze pavement condition and determine maintenance and rehabilitation requirements.

Hudson, Haas, and Uddin discussed performance modeling techniques (*3*). There are several types of prediction models, varying in complexity. For example, regression analysis for performance modeling, which is used in this project, utilizes a relationship between an independent variable and historical data. Regardless of the modeling technique, these models mimic the deterioration process of a pavement. Pavement deteriorates because of five major categories: load, environment, material degradation, construction quality, and interaction affects. A higher rate of deterioration indicates a poorer performance. Consistent with this logic, when looking at the treatment types used in this study heavy rehabilitation has the slowest rate of deterioration, followed by medium and light rehabilitation; preventive maintenance has the highest rate of deterioration.

Lytton discussed different types of models, input data, and the uses and limitations of different types of models (*4*). Two primary types of performance models were discussed: deterministic and probabilistic. Deterministic models give a single number as a prediction for the life of a pavement whereas probabilistic models predict a statistical distribution of such events. Deterministic models include primary response, structural performance, functional performance, and damage models. On the other hand, probabilistic models include survivor curves and Markov process models. In this research project, probabilistic models, specifically survivor curves, were used.

Lytton also explains the basic data types required as inputs to the prediction models (*4*). These basic data types are inventory (does not change with time), monitoring (changes with time or traffic), and costs. This research project primarily uses inventory data such as age of the pavement, traffic loading, climate, and subgrade type. Lastly, Lytton's paper discussed the importance of continually updating and calibrating prediction models.

Traffic Loading Analysis

The first task was to complete a traffic loading analysis. Traffic loading is one characteristic used to divide the data provided by TxDOT. As mentioned previously, dividing the data narrows the amount of data used in each iterative calculation performed by the computer program. A traffic loading analysis was necessary to help decide appropriate traffic levels for data division.

One measurement used to describe traffic loading levels is ESAL (Equivalent Single Axle Load). One ESAL is approximately equal to the distress caused by one loading cycle of a typical truck. The expected 20-year ESALs applied to each pavement section was obtained from the PMIS database. The traffic levels include different ranges of ESALs. The traffic loading analysis began by first, separating the raw data from TxDOT into climate and subgrade zones (discussed later). Then, the traffic loading data from each zone was used to plot histograms. Several histograms were created for each zone using different traffic loading ranges (i.e., 100–1,000; 1,000–10,000; and 10,000–50,000). The various histograms showed which ranges would create better, more equally distributed divisions. Another factor affecting which ranges of traffic loading were chosen was the roadway classification. For example, generally Farm-to-Market

(FM) roads have less traffic than Interstate Highways (IH). Therefore, it is expected that most FM roads would be in the "light traffic" level.

After histograms were made for each zone, possible traffic levels were developed. A column graph was made displaying the number of data points (sections of road) that fit in each ESAL level. Figure 3 shows this column graph.

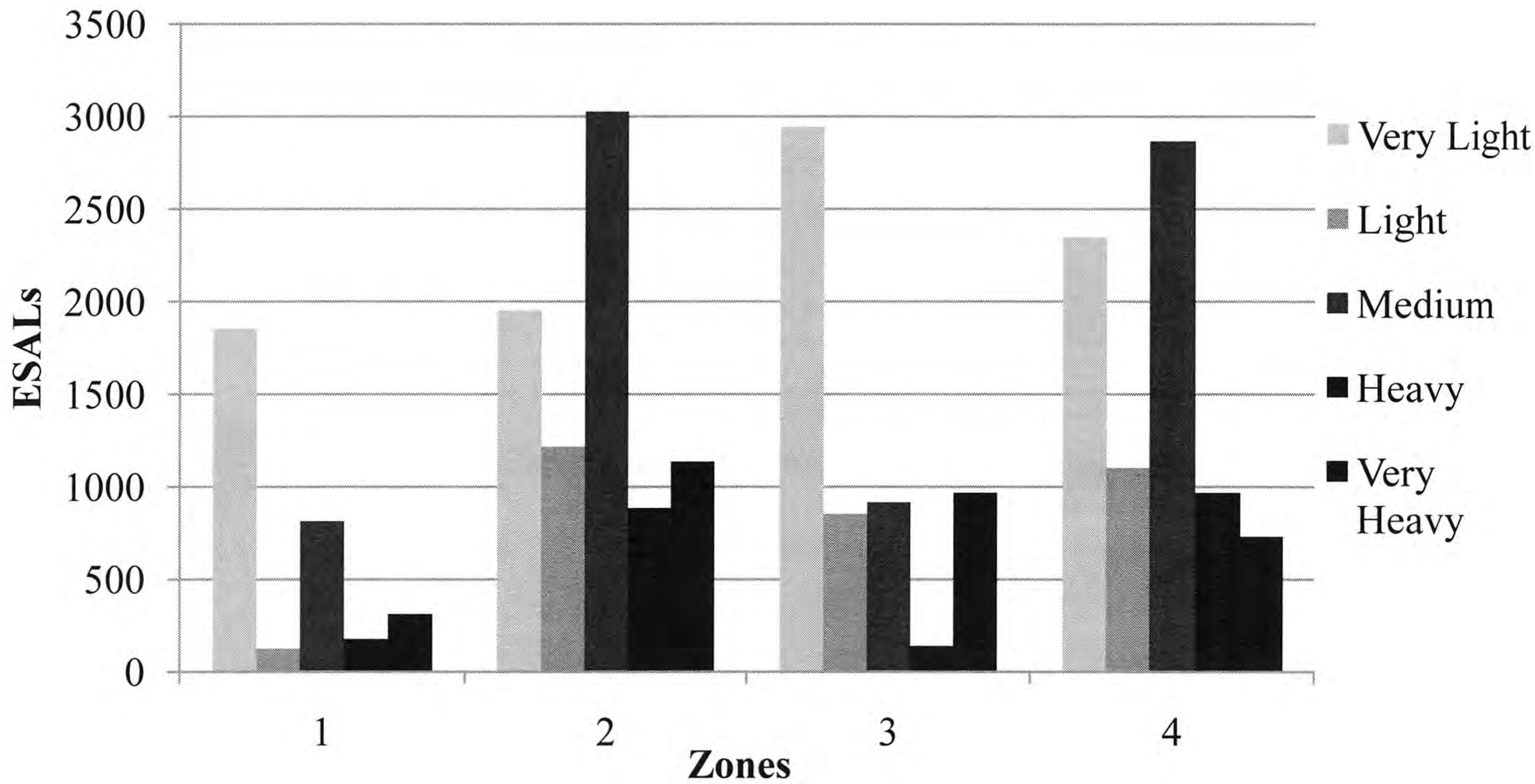

Figure 3. Column Graph Displaying Five Possible Traffic Loading Levels.

Notice the column graph displays five levels of traffic loading including very light, light, medium, heavy, and very heavy. After further analysis, including matching roadway classifications with ESAL amounts, it was decided that only three levels were necessary for this project. Table 1 shows the final levels used in the project: low, medium, and high traffic.

Table 1. Traffic Families.

Level	20-year 18-kip ESAL (in thousands)
Low	ESAL< 1000
Medium	1000 < ESAL < 10,000
High	ESAL > 10,000

Data Organization: Five Divisions

In order to predict pavement performance accurately, families of pavement sections with uniform characteristics were created. These characteristics included climate, subgrade quality, pavement type, maintenance and rehabilitation type, traffic volume, and distress type. Grouping these characteristics create a tree-like division, as shown in Figure 4, that divides the data into uniform families making it easier for the computer program to iteratively calculate the calibrated model coefficients.

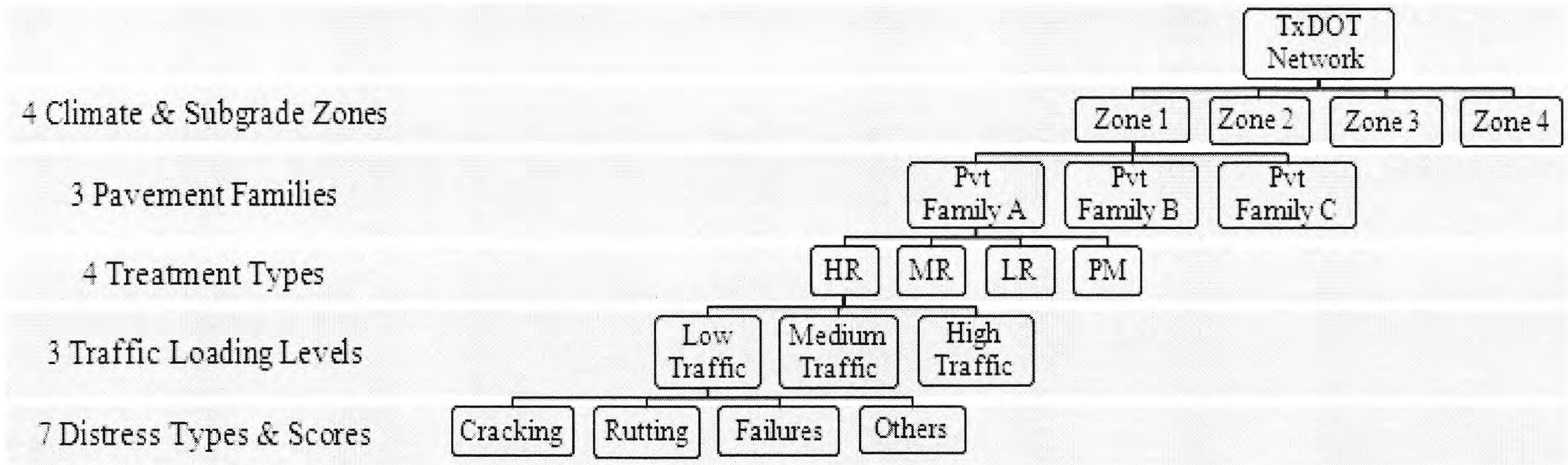

Figure 4. Data Groupings.

First Division: Four Climate & Subgrade Zones

The first division has four general zones. These zones are based on the climate and subgrade conditions, as follows:

- Zone 1: This zone is classified as wet, cold, and poor. Wet and cold refer to physical parameters of the climate experienced in specific regions. Poor refers to subgrade type.
- Zone 2: This zone is classified as wet and warm climate, and poor subgrade type.
- Zone 3: This zone is classified as dry and cold climate, and good subgrade type.
- Zone 4: This zone is classified as dry and warm climate, and good subgrade type.

Due to freeze-thaw phenomenon, the wet and cold zone was considered the worst zone for a pavement section and by contrast, dry-warm was considered best. The expected distress development curves for the various zones are shown in Figure 5. Not all the counties in Texas were specifically placed in these zones, but were a combination of these zones. If counties were plotted as points on Figure 5, some counties would be on the curves representing designated zones, while other counties would lie between these curves.

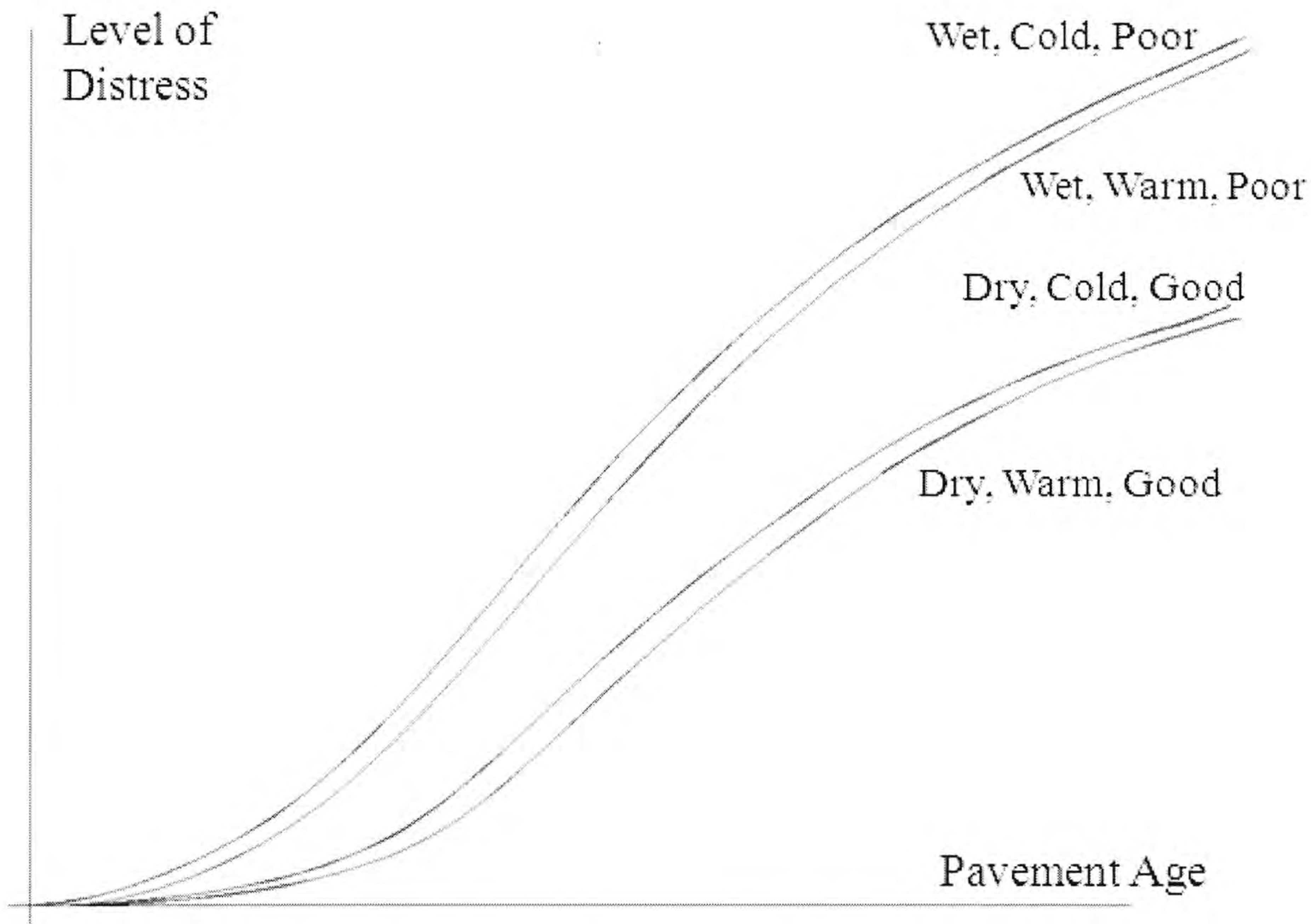

Figure 5. Distress Development Curves.

Second Division: Three Pavement Families

This particular project concentrates only on flexible pavements. There are seven general types of pavement. For this project, these seven pavement types were grouped into three broader families, each consisting of similar pavement types. Table 2 shows the three pavement families used in this study and their similarities based on the PMIS pavement types (*5*).

Table 2. Three Pavement Families.

Family	Pavement Type	Main Similarity in Family
A	4- Thick asphalt concrete	Asphalt only
A	5- Intermediate asphalt concrete	Asphalt only
C	6- Thin asphalt concrete	Thin flexible pavement
B	7- Composite	Has a concrete layer
B	8- Concrete overlaid	Has a concrete layer
A	9- Flexible overlaid	Asphalt only
C	10- Thin-surfaced flexible base	Thin flexible pavement

Third Division: Four Treatment Types

The third division divides the data into four treatment (maintenance and rehabilitation) types. These treatment types range from preventive maintenance, minimal treatments, to heavy rehabilitation, maximum treatments. Roads treated with preventive maintenance degrade significantly faster than those treated with heavy rehabilitation. Table 3 provides examples of various treatments that would occur on asphalt pavements.

Table 3. Treatment Type Examples (*6*).

Treatment Type	Example 1	Example 2
Preventive Maintenance (PM)	Crack Seal	Surface Seal, No Patching
Light Rehabilitation (LR)	Thin Asphalt Overlay	Surface Seal, Light/Medium Patching
Medium Rehabilitation (MR)	Thick Asphalt Overlay	Surface Seal, Heavy Patching
Heavy Rehabilitation (HR)	Remove Asphalt Surface	Replace and Rework Base

Fourth Division: Traffic Loading Levels

The fourth division categorizes traffic loadings into various levels. This division was previously discussed in the traffic loading analysis section. See Table 1 for the traffic loading levels.

Fifth Division: Eight Distress Types & Scores

TxDOT uses eight key distress types for asphalt pavement. These include shallow rutting, deep rutting, failures, block cracking, alligator cracking, longitudinal cracking, patching, and transverse cracking. The "failures" distress type consist of other apparent distresses in the asphalt pavement that cannot be classified as one of the other seven types. Ride score, although technically not a distress, is also included in this division. In general terms, ride score describes the roughness of the road. Ride score is a function of International Roughness Index (IRI).

Data Preparation

Originally the data from TxDOT was organized by districts and then counties within each district. The first step in data preparation was to organize the data into the four zones and to eliminate unorganized, incomplete, and outlier data.

The next part of data preparation was to identify unusable data and removed it. Generally, these data are incomplete or difficult to interpret. For example, data with ESAL values of zero were eliminated because it does not make sense that no traffic exists on a road. Finally, the database needed to be compatible with the calibration tool. In order to increase the speed and efficiency of data analysis, a preliminary analysis (indexing) was first conducted on the data by another module of the computer program. In this step, different rows of data were flagged, highlighting the data to be used by the computer program. This increases the speed of extraction and mining for the program.

Run Calibration Analysis

After the database was uploaded to the computer program, different options could be selected to specify the data desired. Different divisions, as discussed earlier, were chosen to calibrate the coefficients in a more effective manner. After the program cycles, it produces three coefficients (A, α, and β) that were recorded. The results were analyzed and compared to the original data.

Based on recommendations from TxDOT engineers, the program was modified from the preliminary results and the coefficients were recalibrated.

The calibration analysis was performed on each uniform group of pavement sections. The coefficients produced by the computer program were organized in standard tables. An example of one of these tables is presented in the results section.

The program uses Genetic Algorithm (GA) to calibrate the model. The calibration can be more accurate spending more running time. The GA settings form (Figure 6) of the computer program lets the user control the accuracy versus processing time of calibration depending on necessity. Some of these options include changing the number of iterations, population size, and termination criteria. The number of iterations is the number of times the computer program modifies the coefficients by comparing prediction models produced by the coefficients to the actual data trend. Population size is how many random coefficient values are generated before iterations begin. The termination criterion is a selected number of iterations at which the computer program stops if those iterations have consecutively produce the same value or the error is not being significantly reduced.

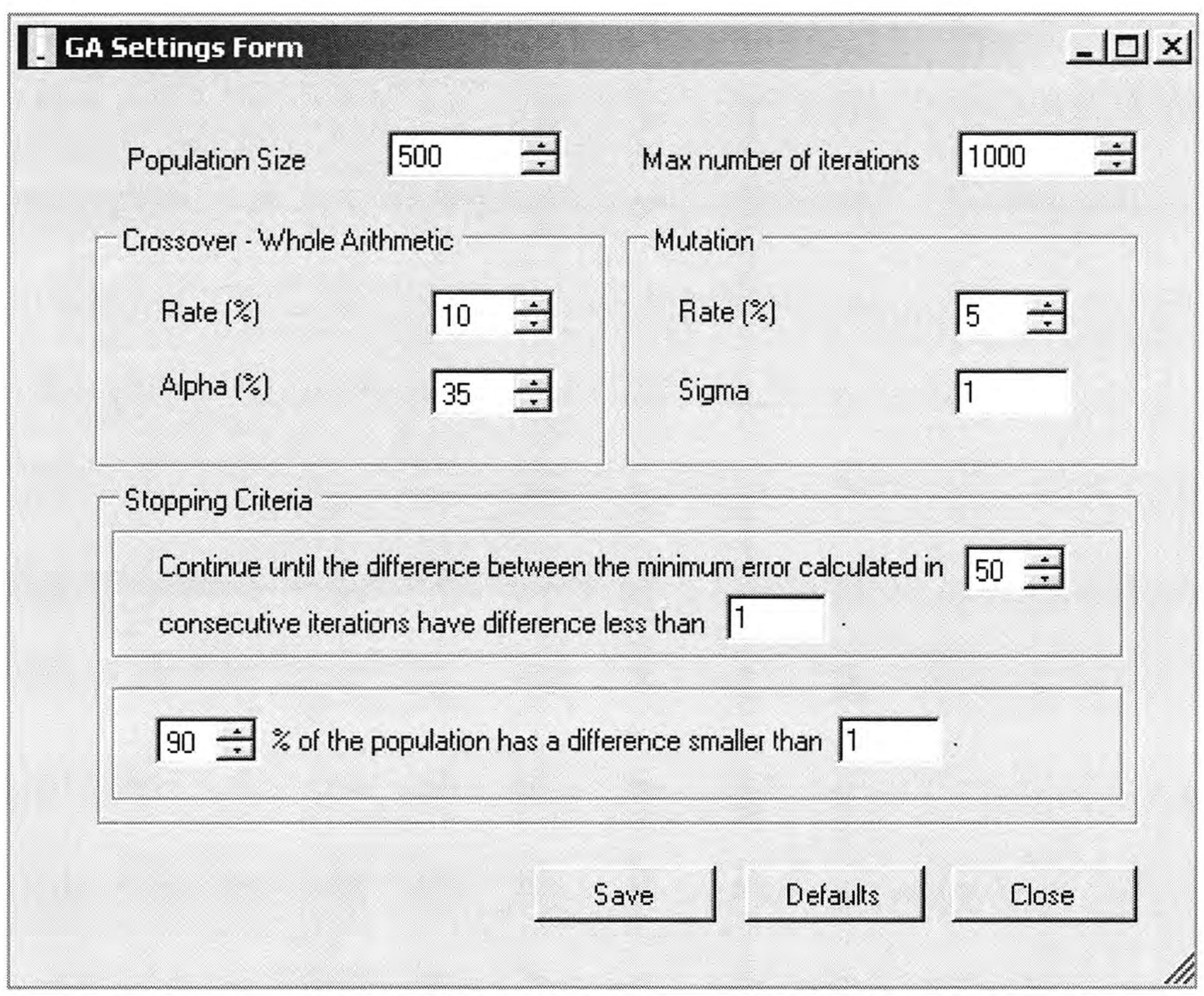

Figure 6. Screen of GA Settings Form of the Computer Program.

RESULTS

This section describes the calibrated prediction models, particularly for zone 3, pavement type A, and medium traffic.

Coefficient Tables

Table 4 shows the coefficients for zone 3, pavement family A. All other coefficient tables produced in this research project can be found in the appendix.

Table 4. Coefficients for Zone 3, Pavement Family A.

		Low Traffic			Medium Traffic			High Traffic		
Distress Type	**Treatment Type**	**α**	**β**	**A**	**α**	**β**	**A**	**α**	**β**	**A**
Shallow Rutting	Preventive Maintenance	100	0.4108	91.755	100	0.5356	78.582	100	0.5936	42.285
	Light Rehabilitation	100	0.4649	101.532	100	0.5592	94.000	100	0.6009	45.134
	Medium Rehabilitation	100	0.4905	104.919	100	0.6015	95.909	100	0.6080	48.520
	Heavy Rehabilitation	100	0.5781	121.010	100	0.6671	101.005	100	0.6150	52.905
Deep Rutting	Preventive Maintenance	100	0.5936	74.711	100	0.7096	87.173	100	3.4344	9.858
	Light Rehabilitation	100	0.6038	86.357	100	0.7739	91.199	100	3.7545	10.348
	Medium Rehabilitation	100	0.6168	99.268	100	0.9034	102.324	100	4.1972	10.594
	Heavy Rehabilitation	100	0.6329	115.077	100	0.9423	102.224	100	4.7957	11.840
Failures	Preventive Maintenance	20	9.7215	7.173	20	1.5075	20.904	20	1.1999	62.403
	Light Rehabilitation	20	9.9637	7.175	20	1.5684	21.869	20	1.3375	62.684
	Medium Rehabilitation	20	10.5148	7.327	20	1.6467	23.069	20	1.3438	69.627
	Heavy Rehabilitation	20	11.4166	7.479	20	1.7459	24.587	20	1.4727	83.475
Block Cracking	Preventive Maintenance	100	5.8098	7.961	100	7.0305	69.900	100	1.9890	21.769
	Light Rehabilitation	100	6.8925	8.763	100	7.9356	78.728	100	2.0414	22.437
	Medium Rehabilitation	100	7.2811	10.464	100	8.8623	88.021	100	2.0674	22.168
	Heavy Rehabilitation	100	8.2023	10.165	100	9.7911	97.074	100	2.4799	25.900
Alligator Cracking	Preventive Maintenance	100	0.5525	98.073	100	0.5356	83.868	100	0.6846	78.414
	Light Rehabilitation	100	0.6442	113.062	100	0.5770	86.721	100	0.7099	92.172
	Medium Rehabilitation	100	0.7217	125.110	100	0.6590	95.329	100	0.7518	109.977
	Heavy Rehabilitation	100	0.7761	132.498	100	0.6647	110.577	100	0.8131	112.457
Longitudinal Cracking	Preventive Maintenance	500	0.4277	71.873	500	0.3609	62.006	500	0.2780	92.543
	Light Rehabilitation	500	0.4302	81.623	500	0.4134	65.503	500	0.3177	104.450
	Medium Rehabilitation	500	0.4308	92.734	500	0.4479	78.790	500	0.3426	109.959
	Heavy Rehabilitation	500	0.5164	105.433	500	0.4580	89.813	500	0.3478	131.227
Transverse Cracking	Preventive Maintenance	20	0.4107	82.919	20	0.3448	65.325	20	0.3528	83.707
	Light Rehabilitation	20	0.4463	85.297	20	0.4027	70.675	20	0.3820	87.538
	Medium Rehabilitation	20	0.5172	92.650	20	0.4540	74.334	20	0.4424	98.580
	Heavy Rehabilitation	20	0.6352	107.534	20	0.4940	74.596	20	0.4575	118.349
Ride Score	Preventive Maintenance	100	2.2931	9.645	100	20.8333	8.852	100	39.3914	6.228
	Light Rehabilitation	100	3.2680	11.062	100	22.8218	10.910	100	51.5386	7.459
	Medium Rehabilitation	100	9.7364	15.533	100	28.0828	15.392	100	52.0487	9.607
	Heavy Rehabilitation	100	22.0614	17.128	100	44.0405	16.420	100	66.3958	14.159
Patching	Preventive Maintenance	100	0.4857	105.722	100	0.5024	89.971	100	0.2699	106.778
	Light Rehabilitation	100	0.5374	113.511	100	0.5136	103.946	100	0.3025	116.641
	Medium Rehabilitation	100	0.5383	133.309	100	0.5301	121.024	100	0.3127	117.764
	Heavy Rehabilitation	100	0.5914	141.676	100	0.5512	141.318	100	0.3595	132.576

Individual Distress Graphs

The coefficients produced by the computer program are used in the prediction equation to calculate the value L_i (Equation 1) for the eight pavement distresses and ride score. For zone 3, three graphs for each division of traffic (low, medium, and high) are prepared for each of the eight distresses and ride score. These graphs plot the treatments types using L versus age, showing the individual prediction models for each traffic division in each distress type. Figure 7 gives an example of one of these graphs; this graph represents prediction models for alligator cracking under medium traffic in zone 3, pavement type A.

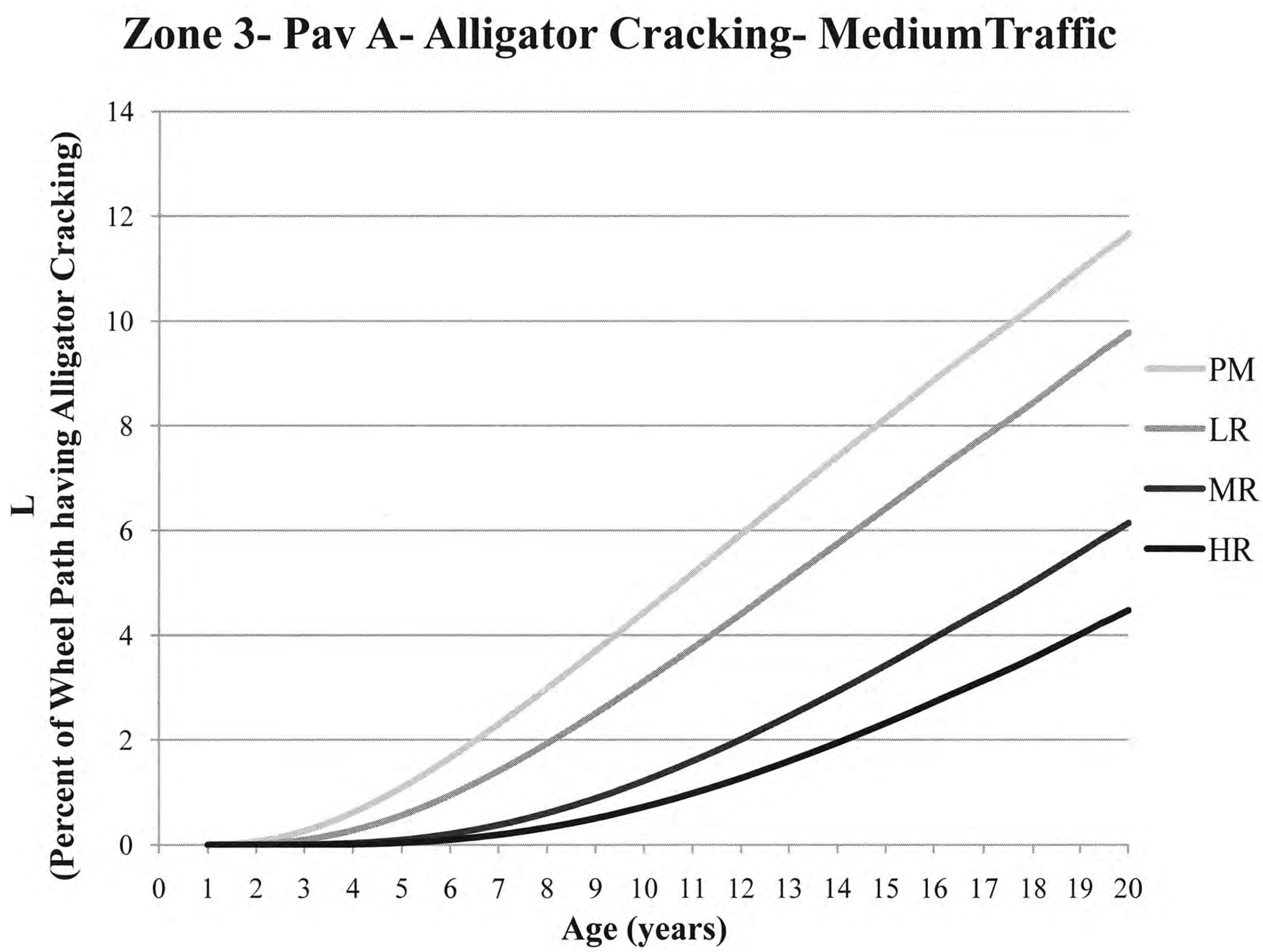

Figure 7. Prediction Model for 3A Alligator Cracking with Medium Traffic.

Combined Distress Graphs

From the calculated L_i values for each pavement distress type and ride score, a corresponding U value (Equation 2) is calculated. By multiplying the U values together, a distress score value is calculated (Equation 3). Three graphs representing each traffic division were prepared from the distress scores calculated. Figure 8 shows the combined distress score graph of medium traffic in zone 3, pavement type A.

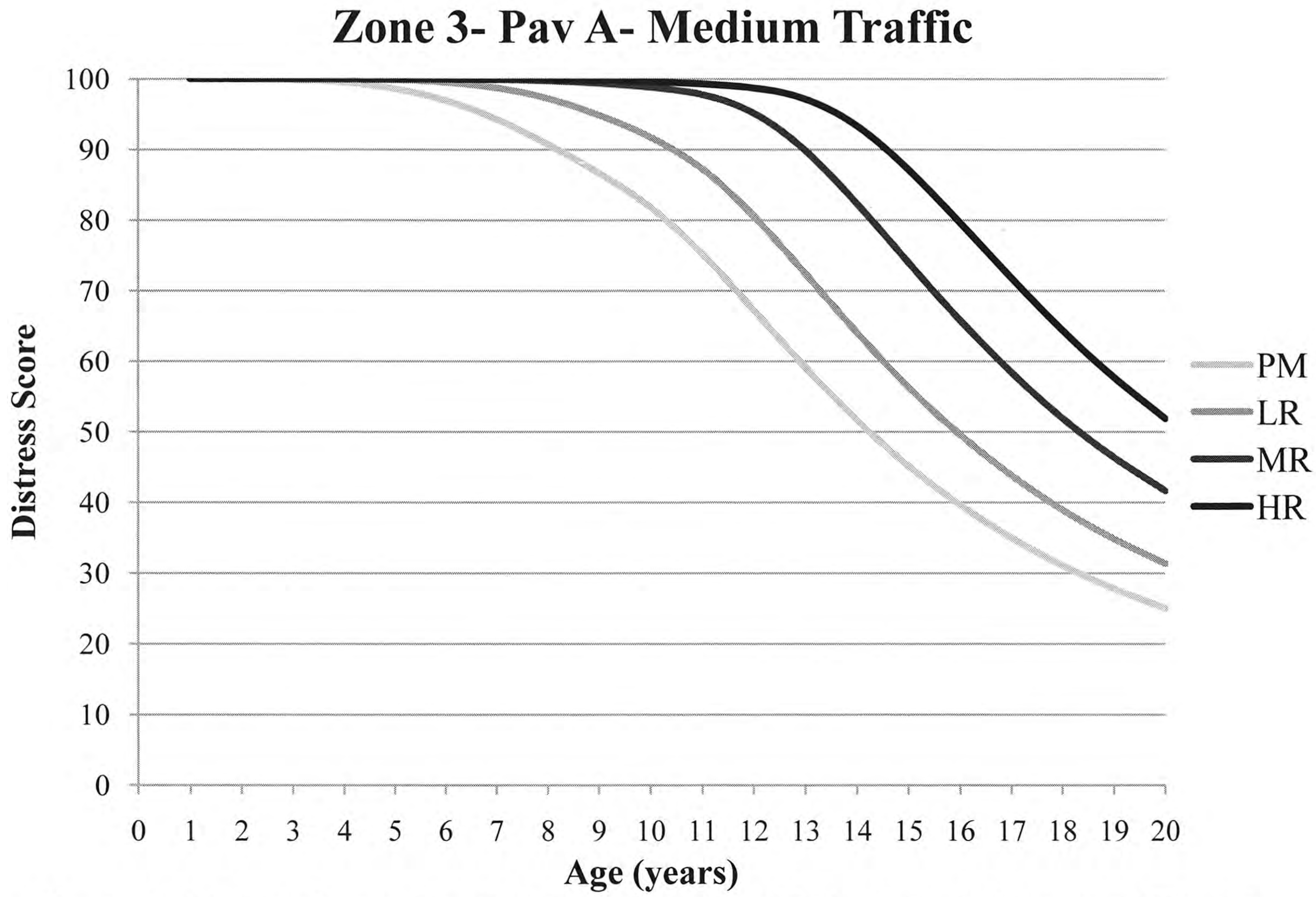

Figure 8. Distress Score Graph for Medium Traffic in Zone 3, Pavement Type A.

Similar to the three distress score graphs prepared for each coefficients table, three condition score graphs were prepared. These graphs relate condition score to age. Figure 9 shows the condition score graph of medium traffic in zone 3, pavement type A, which corresponds the distress score graph above (Figure 8).

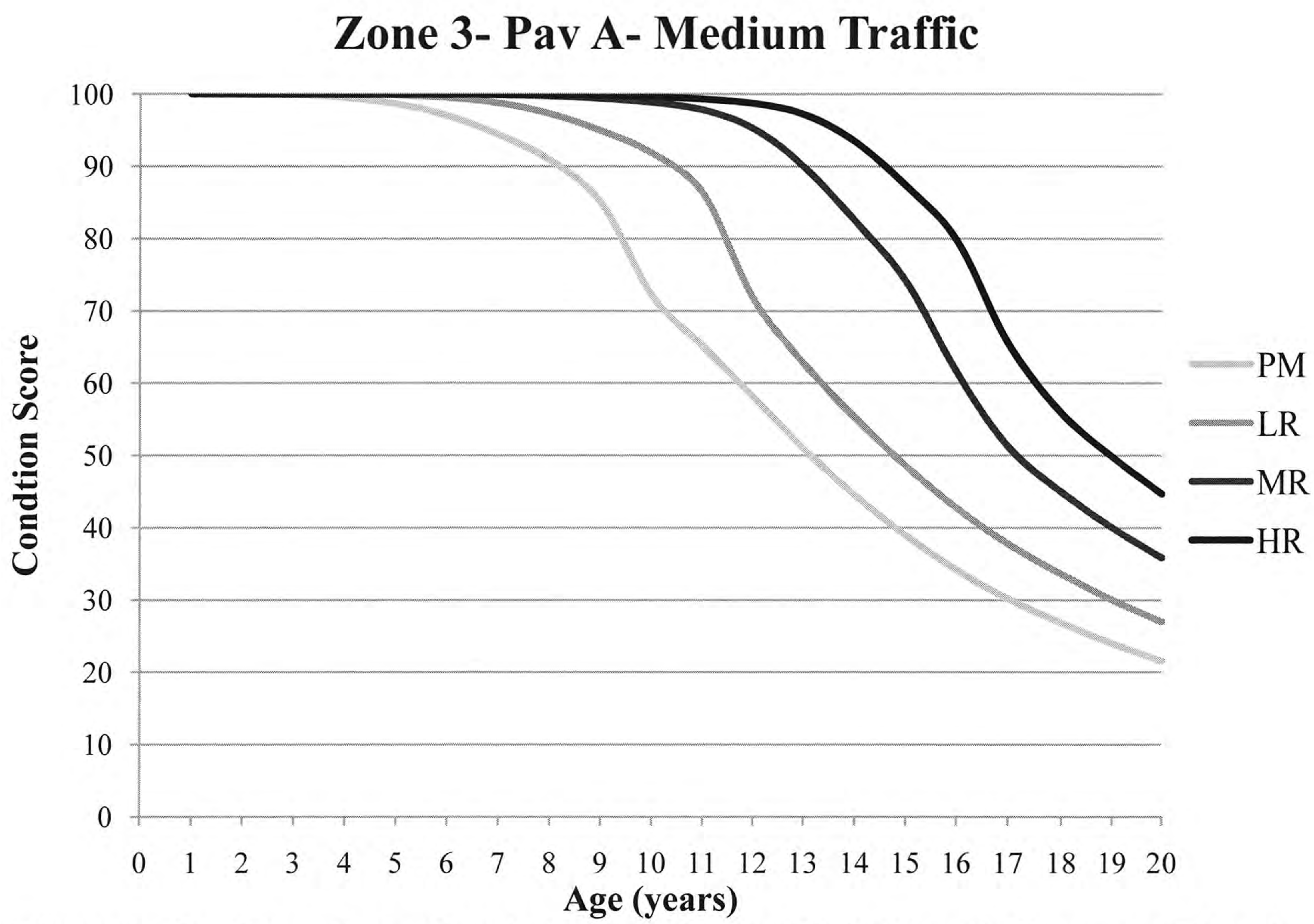

Figure 9. Condition Score Graph for Medium Traffic in Zone 3, Pavement Type A.

Comparing Results

The main purpose of this research project was to improve the prediction models for asphalt pavements in Texas. This improvement was measured by a decrease in the difference between actual and predicted data points. The error was calculated based on the measurement of this difference between actual data and predicted values. Specifically, analyzing preventive maintenance on alligator cracking for zone 3, pavement family A in medium traffic, error values for the original prediction and calibrated prediction can be compared. The error values were calculated using Equation 6.

$$S = \sqrt{\frac{\sum_{i=1}^{n}(X_{i-predicted} - X_{i-actual})^2}{n-1}} \qquad (6)$$

where:

S	=	the sample standard deviation
N	=	the total number of data points
$X_{i\text{-predicted}}$	=	the predicted value for i[th] section
$X_{i\text{-actua}}$	=	the actual value for i[th] section

The standard deviation value calculated for the original prediction was 3.1287 whereas the calibrated prediction value was 2.7077. This decrease in the error value reduced the error between actual data and predicted data by 15.55 percent. Similar improvement in calibrated prediction models were found thus validating the need to for calibration.

Discussion of Results

Most of the coefficients were found to produce reasonable results and follow logical trends. Tables of coefficients shown in this paper (Table 4 and Table 5–Table 15) will be used for planning future projects and funding purposes.

When analyzing the results, particularly the distress and condition score graphs, insufficient data were found in every zone for high traffic in pavement family C (6 and 10). The computer program needs a minimum amount of data points to iteratively calibrate the coefficients. Finding insufficient data in the same grouping in each zone (high traffic in pavement family C) was expected because pavement family C includes thin flexible pavements that are not used in areas of high traffic. There are a few other sets of coefficients that indicated insufficient data. Because the majority of the distresses in these sets provided sufficient data, the prediction models still showed improvements and were deemed reasonable. Additional research can be conducted to determine particular reasons for insufficient data in different cases. This can lead to sufficient data collection for the needed areas.

CONCLUSION

Calibrated prediction models were found to be more accurate than original prediction models. These prediction models estimated the occurrence of distresses in pavement over time. These models will be useful in scheduling future maintenance and repair projects and budget allocation. The computer program used to obtain these calibrated prediction models showed logical results. The computer program produced reasonable coefficients that minimized the error between actual data and calibrated prediction models. Some statistical analysis was performed on individual distress types in zone 3 and showed the calibrated models had decreased the error between actual data and predicted models by 10–15 percent. The divisions of data used to filter the data also proved to be sufficient for calibration purposes.

Largely, the calibrated prediction models were an improvement over the original prediction models. Still, there was some error in these models. Both the computer program and prediction model calculations can be continuously improved by changing settings on the computer program, methodology of the program design itself, and/or by changing the written code of the program. Further work on the computer program can result in further minimization of the error between actual data and predictions.

RECOMMENDATIONS

Future research for this project is recommended. Prediction models can constantly be improved. The computer program used in this project is a great start for improving the original prediction models. The coefficients produced were more accurate than the original prediction models, thus fulfilling the main purpose of this project. Still, further research may find appropriate settings for the computer program to produce the most accurate results.

The settings can be modified both for the computer program and for the families used to separate the data. The computer program gives the user the freedom to change different families (traffic, pavement type, etc.). This gives opportunity for experimentation on which divisions produce the best results. The settings options for the computer program can also be modified. These modifications change the speed of calibration and the accuracy. Experimentation can be conducted on the settings options to find the optimal settings for calibration.

In addition to these setting and family changes, the computer program itself can be modified. Statistical analysis can be added to more precisely show the reduction in error of the results. This can be accomplished by first producing the error for original and calibrated prediction models and then finding the percent error improved based on the difference in error. Perhaps statistical graphs can also be made to accompany this error.

Additional data can also be collected to improve the accuracy of the prediction models. Furthermore, the most recent repair and construction dates of pavement roads can be collected and recorded. This project estimated the most recent repair dates due to the lack of information about them. As new data are collected, it is recommended to recalibrate the coefficients and update prediction models continually.

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APPENDIX: COEFFICIENT TABLES

Table 5. Coefficients for Zone 1, Pavement Family A.

		Low Traffic			Medium Traffic			High Traffic		
Distress Type	**Treatment Type**	**α**	**β**	**A**	**α**	**β**	**A**	**α**	**β**	**A**
Shallow Rutting	Preventive Maintenance	100	0.4857	52.459	100	0.4857	68.003	100	1.0832	14.502
	Light Rehabilitation	100	0.5250	60.525	100	0.5726	75.098	100	1.1011	15.225
	Medium Rehabilitation	100	0.6042	63.467	100	0.6569	80.263	100	1.1139	15.150
	Heavy Rehabilitation	100	0.6170	70.261	100	0.7330	83.116	100	1.1215	15.076
Deep Rutting	Preventive Maintenance	100	0.6017	66.580	100	0.6526	89.070	100	0.5875	79.753
	Light Rehabilitation	100	0.7009	72.360	100	0.7361	103.688	100	0.5909	89.972
	Medium Rehabilitation	100	0.7834	74.471	100	0.7876	119.689	100	0.8876	98.273
	Heavy Rehabilitation	100	0.8384	88.110	100	0.8055	136.276	100	1.1677	103.920
Failures	Preventive Maintenance	20	6.7728	7.403	20	1.2660	21.287	20	3.1349	9.453
	Light Rehabilitation	20	7.7497	7.069	20	1.3321	22.223	20	3.4571	9.494
	Medium Rehabilitation	20	8.9363	7.113	20	1.4373	23.762	20	3.9503	9.894
	Heavy Rehabilitation	20	10.3832	7.157	20	1.5897	26.875	20	4.6721	11.294
Block Cracking	Preventive Maintenance	100	2.9022	96.719	100	7.6449	18.686	100	2.6534	71.789
	Light Rehabilitation	100	3.0856	115.833	100	9.0260	20.131	100	2.9602	72.226
	Medium Rehabilitation	100	3.2969	116.391	100	9.1017	21.686	100	3.4878	76.610
	Heavy Rehabilitation	100	3.5392	116.983	100	9.4309	24.241	100	3.6307	86.637
Alligator Cracking	Preventive Maintenance	100	0.5686	93.093	100	0.6846	39.042	100	3.1349	9.453
	Light Rehabilitation	100	0.6445	103.233	100	0.8135	40.823	100	3.4571	9.494
	Medium Rehabilitation	100	0.6808	106.148	100	0.9449	41.780	100	3.9503	9.894
	Heavy Rehabilitation	100	0.8022	121.986	100	1.0730	49.737	100	4.6721	11.294
Longitudinal Cracking	Preventive Maintenance	500	0.6846	61.608	500	0.7263	38.651	500	0.3677	99.091
	Light Rehabilitation	500	0.7741	64.137	500	0.8713	40.412	500	1.1918	117.148
	Medium Rehabilitation	500	0.8101	73.884	500	1.0087	47.990	500	1.9877	135.961
	Heavy Rehabilitation	500	0.9421	79.139	500	1.1272	56.210	500	2.7257	156.017
Transverse Cracking	Preventive Maintenance	20	0.7265	83.310	20	0.5437	57.033	20	0.4607	73.289
	Light Rehabilitation	20	0.7738	84.348	20	0.6017	57.305	20	0.4669	84.056
	Medium Rehabilitation	20	0.8643	89.683	20	0.6037	62.586	20	0.4747	96.841
	Heavy Rehabilitation	20	1.0089	100.608	20	0.6633	75.843	20	0.4846	111.699
Ride Score	Preventive Maintenance	100	19.0707	6.236	100	20.1904	6.340	100	5.6529	7.611
	Light Rehabilitation	100	20.2624	7.167	100	21.2202	7.082	100	6.0633	11.579
	Medium Rehabilitation	100	21.4548	8.174	100	28.8614	9.894	100	6.5668	15.566
	Heavy Rehabilitation	100	29.6932	14.692	100	38.9295	17.321	100	36.5640	17.643
Patching	Preventive Maintenance	100	0.4519	75.720	100	0.4189	110.204	100	4.2976	8.289
	Light Rehabilitation	100	0.5199	80.864	100	0.4745	121.739	100	4.4924	9.650
	Medium Rehabilitation	100	0.5682	81.537	100	0.5006	125.658	100	4.5125	11.243
	Heavy Rehabilitation	100	0.5877	93.886	100	0.5908	145.281	100	5.2519	11.836

Table 6. Coefficients for Zone 1, Pavement Family B.

		Low Traffic			Medium Traffic			High Traffic		
Distress Type	**Treatment Type**	**α**	**β**	**A**	**α**	**β**	**A**	**α**	**β**	**A**
Shallow Rutting	Preventive Maintenance	100	0.3690	91.434	100	0.7345	46.301	100	0.4269	71.468
	Light Rehabilitation	100	0.4175	100.515	100	0.7580	50.667	100	0.4282	80.388
	Medium Rehabilitation	100	0.4408	103.948	100	0.7947	57.025	100	0.5127	90.264
	Heavy Rehabilitation	100	0.5196	120.091	100	0.8467	64.551	100	0.6105	101.024
Deep Rutting	Preventive Maintenance	100	0.6596	68.630	100	0.7844	78.337	100	0.8674	36.043
	Light Rehabilitation	100	0.7758	75.738	100	0.8090	91.641	100	1.0074	36.328
	Medium Rehabilitation	100	0.8828	80.391	100	0.8478	108.797	100	1.1134	41.368
	Heavy Rehabilitation	100	0.9711	81.668	100	0.9021	109.303	100	1.1681	44.896
Failures	Preventive Maintenance	20	0.4957	64.606	20	0.7178	85.520	20	0.7675	50.876
	Light Rehabilitation	20	0.5842	69.377	20	1.1511	96.710	20	0.8132	58.013
	Medium Rehabilitation	20	0.6473	69.645	20	1.3421	104.836	20	0.8967	69.037
	Heavy Rehabilitation	20	0.6741	80.575	20	2.5544	109.092	20	1.0280	70.935
Block Cracking	Preventive Maintenance	100	0.7345	87.647	100	6.9886	98.693	100	6.3243	57.033
	Light Rehabilitation	100	0.8026	92.671	100	7.7727	115.967	100	6.8026	60.229
	Medium Rehabilitation	100	0.9394	105.332	100	8.4306	132.342	100	6.9719	60.473
	Heavy Rehabilitation	100	0.9845	105.811	100	8.9120	146.889	100	8.1849	69.424
Alligator Cracking	Preventive Maintenance	100	0.4688	70.841	100	0.4688	89.146	100	3.0418	9.851
	Light Rehabilitation	100	0.5616	79.842	100	0.5220	96.242	100	3.3714	10.517
	Medium Rehabilitation	100	0.6655	89.123	100	0.5303	113.461	100	3.8802	11.932
	Heavy Rehabilitation	100	0.7806	98.157	100	0.5933	123.019	100	4.6287	13.348
Longitudinal Cracking	Preventive Maintenance	500	0.3359	80.862	500	1.3159	15.213	500	0.7096	37.542
	Light Rehabilitation	500	0.3580	82.494	500	1.3316	15.105	500	0.8349	39.258
	Medium Rehabilitation	500	0.4029	88.212	500	1.5946	17.889	500	0.9520	46.777
	Heavy Rehabilitation	500	0.4765	100.296	500	1.8859	21.563	500	1.0499	54.082
Transverse Cracking	Preventive Maintenance	20	0.3859	27.445	20	0.3690	88.435	20	8.5170	6.692
	Light Rehabilitation	20	0.4346	31.146	20	0.4105	95.242	20	9.1131	7.516
	Medium Rehabilitation	20	0.4545	33.534	20	0.4180	112.603	20	10.6596	8.029
	Heavy Rehabilitation	20	0.5286	34.107	20	0.4692	123.364	20	11.3993	9.025
Ride Score	Preventive Maintenance	100	13.7623	6.527	100	5.3160	7.663	100	35.2232	8.852
	Light Rehabilitation	100	17.0410	9.617	100	5.8721	11.762	100	40.0648	10.419
	Medium Rehabilitation	100	17.6695	12.447	100	8.2686	13.998	100	44.6861	12.161
	Heavy Rehabilitation	100	28.2886	17.983	100	48.5960	18.198	100	48.2214	18.953
Patching	Preventive Maintenance	100	0.6347	16.215	100	0.5187	100.463	100	3.4755	5.807
	Light Rehabilitation	100	0.6560	16.599	100	0.5588	103.296	100	3.7019	6.985
	Medium Rehabilitation	100	0.6911	17.905	100	0.6398	113.491	100	3.9272	7.963
	Heavy Rehabilitation	100	0.7421	19.616	100	0.6472	132.188	100	4.1493	9.884

Table 7. Coefficients for Zone 1, Pavement Family C.

Distress Type	Treatment Type	Low Traffic α	Low Traffic β	Low Traffic A	Medium Traffic α	Medium Traffic β	Medium Traffic A	High Traffic α	High Traffic β	High Traffic A
Shallow Rutting	Preventive Maintenance	100	0.4590	49.851	100	0.2780	74.788	-	-	-
	Light Rehabilitation	100	0.4843	57.777	100	0.2863	87.826	-	-	-
	Medium Rehabilitation	100	0.5352	59.001	100	0.3016	105.559	-	-	-
	Heavy Rehabilitation	100	0.6199	63.669	100	0.3254	108.015	-	-	-
Deep Rutting	Preventive Maintenance	100	0.5437	80.074	100	0.4857	85.283	-	-	-
	Light Rehabilitation	100	0.5723	95.966	100	0.5285	88.989	-	-	-
	Medium Rehabilitation	100	0.6260	99.906	100	0.6157	100.235	-	-	-
	Heavy Rehabilitation	100	0.7107	108.265	100	0.6427	100.275	-	-	-
Failures	Preventive Maintenance	20	0.6186	74.788	20	8.2432	25.632	-	-	-
	Light Rehabilitation	20	0.6289	86.293	20	8.2478	27.978	-	-	-
	Medium Rehabilitation	20	0.6414	99.109	20	8.5473	31.446	-	-	-
	Heavy Rehabilitation	20	0.6561	114.789	20	9.1628	35.517	-	-	-
Block Cracking	Preventive Maintenance	100	5.6101	57.270	100	7.5040	12.849	-	-	-
	Light Rehabilitation	100	6.2031	62.696	100	8.6254	13.237	-	-	-
	Medium Rehabilitation	100	6.8189	69.311	100	9.9080	13.228	-	-	-
	Heavy Rehabilitation	100	7.4518	75.343	100	11.3757	13.218	-	-	-
Alligator Cracking	Preventive Maintenance	100	0.5936	93.331	100	0.7345	30.757	-	-	-
	Light Rehabilitation	100	0.6731	103.281	100	0.8323	36.185	-	-	-
	Medium Rehabilitation	100	0.7111	106.201	100	0.8753	39.200	-	-	-
	Heavy Rehabilitation	100	0.8379	122.039	100	1.0242	39.402	-	-	-
Longitudinal Cracking	Preventive Maintenance	500	0.6242	91.831	500	0.4963	44.412	-	-	-
	Light Rehabilitation	500	0.7616	93.472	500	1.7364	50.881	-	-	-
	Medium Rehabilitation	500	0.9781	97.742	500	2.2365	52.142	-	-	-
	Heavy Rehabilitation	500	1.2935	106.348	500	3.3337	56.639	-	-	-
Transverse Cracking	Preventive Maintenance	20	5.3444	32.180	20	8.1683	24.920	-	-	-
	Light Rehabilitation	20	6.1879	35.246	20	9.7945	26.941	-	-	-
	Medium Rehabilitation	20	6.2430	40.572	20	10.1357	29.834	-	-	-
	Heavy Rehabilitation	20	6.6170	41.409	20	10.8508	34.472	-	-	-
Ride Score	Preventive Maintenance	100	5.4283	6.736	100	20.8643	5.167	-	-	-
	Light Rehabilitation	100	6.0763	8.563	100	27.7967	7.383	-	-	-
	Medium Rehabilitation	100	6.4730	12.818	100	39.8947	9.886	-	-	-
	Heavy Rehabilitation	100	7.3218	17.531	100	41.7462	12.936	-	-	-
Patching	Preventive Maintenance	100	0.3609	67.988	100	0.1950	45.736	-	-	-
	Light Rehabilitation	100	0.4021	69.156	100	0.1959	49.225	-	-	-
	Medium Rehabilitation	100	0.4106	77.645	100	0.1971	52.626	-	-	-
	Heavy Rehabilitation	100	0.4635	80.439	100	0.1983	57.034	-	-	-

-: not sufficient data

Table 8. Coefficients for Zone 2, Pavement Family A.

		Low Traffic			Medium Traffic			High Traffic		
Distress Type	**Treatment Type**	**α**	**β**	**A**	**α**	**β**	**A**	**α**	**β**	**A**
Shallow Rutting	Preventive Maintenance	100	0.4358	86.137	100	0.4715	103.231	100	1.1001	21.280
	Light Rehabilitation	100	0.4386	97.754	100	0.5490	122.473	100	1.1450	21.945
	Medium Rehabilitation	100	0.4716	98.723	100	0.6548	122.010	100	1.2136	23.323
	Heavy Rehabilitation	100	0.5059	99.441	100	0.7938	123.764	100	1.3094	25.128
Deep Rutting	Preventive Maintenance	100	0.8094	49.459	100	0.6471	116.603	100	0.8013	59.682
	Light Rehabilitation	100	0.9707	52.035	100	0.8380	117.085	100	0.8439	67.938
	Medium Rehabilitation	100	1.1485	54.205	100	1.0622	117.850	100	0.9204	79.867
	Heavy Rehabilitation	100	1.3393	55.512	100	1.3246	119.685	100	1.0381	80.540
Failures	Preventive Maintenance	20	0.5665	107.917	20	0.5936	77.350	20	0.7764	87.952
	Light Rehabilitation	20	0.6398	116.342	20	0.6847	82.076	20	0.7774	99.743
	Medium Rehabilitation	20	0.6452	135.859	20	0.7505	82.764	20	0.9236	111.669
	Heavy Rehabilitation	20	0.7229	145.388	20	0.7783	95.870	20	1.0842	123.320
Block Cracking	Preventive Maintenance	100	0.7674	51.746	100	0.6166	78.683	100	0.8674	108.102
	Light Rehabilitation	100	0.7734	55.483	100	1.3306	85.943	100	0.9568	115.171
	Medium Rehabilitation	100	0.9556	58.192	100	1.7935	89.861	100	1.1376	132.373
	Heavy Rehabilitation	100	1.1598	60.784	100	1.9408	90.366	100	1.2231	136.805
Alligator Cracking	Preventive Maintenance	100	0.3859	107.435	100	0.3359	89.860	100	0.4607	74.294
	Light Rehabilitation	100	0.4318	116.391	100	0.3458	105.316	100	0.5268	78.080
	Medium Rehabilitation	100	0.4450	116.085	100	0.3639	125.241	100	0.5674	93.063
	Heavy Rehabilitation	100	0.5080	128.884	100	0.3912	126.920	100	0.5732	104.607
Longitudinal Cracking	Preventive Maintenance	500	0.5465	63.978	500	0.3859	119.853	500	0.6097	56.904
	Light Rehabilitation	500	0.6034	74.835	500	0.4568	141.099	500	0.6383	63.846
	Medium Rehabilitation	500	0.7151	77.311	500	0.5297	162.268	500	0.6896	74.316
	Heavy Rehabilitation	500	0.7212	84.121	500	0.6021	182.480	500	0.7693	88.470
Transverse Cracking	Preventive Maintenance	20	0.6516	55.857	20	0.5018	70.849	20	0.5599	98.842
	Light Rehabilitation	20	0.6775	62.308	20	0.5637	72.958	20	0.6095	116.782
	Medium Rehabilitation	20	0.7222	70.456	20	0.5847	83.607	20	0.6802	139.131
	Heavy Rehabilitation	20	0.7885	81.703	20	0.6730	88.665	20	0.7743	139.990
Ride Score	Preventive Maintenance	100	31.1803	6.280	100	5.5406	7.566	100	5.6529	7.611
	Light Rehabilitation	100	46.2971	8.881	100	5.9464	11.529	100	6.0633	11.579
	Medium Rehabilitation	100	68.3312	13.877	100	6.4165	12.141	100	6.5668	15.566
	Heavy Rehabilitation	100	102.001	19.943	100	7.4835	15.066	100	36.5640	17.643
Patching	Preventive Maintenance	100	0.6097	50.505	100	0.4607	96.740	100	4.0319	8.576
	Light Rehabilitation	100	0.6179	54.998	100	0.4887	98.002	100	4.2633	10.829
	Medium Rehabilitation	100	0.6266	59.444	100	0.5429	104.155	100	4.4086	12.597
	Heavy Rehabilitation	100	0.6362	63.899	100	0.6314	115.818	100	4.4569	15.046

Table 9. Coefficients for Zone 2, Pavement Family B.

		Low Traffic			Medium Traffic			High Traffic		
Distress Type	**Treatment Type**	**α**	**β**	**A**	**α**	**β**	**A**	**α**	**β**	**A**
Shallow Rutting	Preventive Maintenance	100	0.4770	105.333	100	0.4189	110.204	100	0.7265	74.007
	Light Rehabilitation	100	2.0336	124.630	100	0.4745	121.739	100	0.8216	76.734
	Medium Rehabilitation	100	4.0004	149.927	100	0.5006	125.658	100	0.8609	88.468
	Heavy Rehabilitation	100	4.0931	151.783	100	0.5908	145.281	100	1.0019	94.242
Deep Rutting	Preventive Maintenance	100	0.6596	96.925	100	0.6412	105.815	100	5.8340	84.508
	Light Rehabilitation	100	0.6951	115.991	100	0.7375	126.102	100	6.8929	85.260
	Medium Rehabilitation	100	0.7598	120.593	100	0.8706	126.946	100	7.2117	90.767
	Heavy Rehabilitation	100	0.8613	130.597	100	1.0461	129.961	100	8.0126	102.937
Failures	Preventive Maintenance	20	1.0832	16.502	20	0.5437	88.619	20	8.0684	31.309
	Light Rehabilitation	20	1.1011	17.225	20	0.5518	102.417	20	8.1853	33.967
	Medium Rehabilitation	20	1.1139	17.150	20	0.5620	117.273	20	8.7390	39.322
	Heavy Rehabilitation	20	1.1215	17.076	20	0.5742	134.985	20	9.7952	39.209
Block Cracking	Preventive Maintenance	100	9.6217	50.986	100	6.5071	53.088	100	7.5620	32.457
	Light Rehabilitation	100	10.1517	58.130	100	7.7479	60.925	100	7.8703	36.118
	Medium Rehabilitation	100	11.3466	58.314	100	8.1437	61.274	100	8.8956	36.939
	Heavy Rehabilitation	100	13.3928	62.071	100	9.0521	64.902	100	9.0726	41.073
Alligator Cracking	Preventive Maintenance	100	0.7514	52.135	100	0.4689	115.168	100	1.6129	13.067
	Light Rehabilitation	100	0.7630	56.140	100	0.6321	124.827	100	1.8572	15.687
	Medium Rehabilitation	100	0.7758	60.600	100	0.6543	147.413	100	2.0955	18.163
	Heavy Rehabilitation	100	0.7896	66.069	100	0.8206	157.608	100	2.3096	20.187
Longitudinal Cracking	Preventive Maintenance	500	0.4358	51.274	500	0.8175	16.789	500	0.3029	111.927
	Light Rehabilitation	500	0.4464	55.787	500	0.8406	17.494	500	0.3475	125.823
	Medium Rehabilitation	500	0.4637	62.107	500	0.8757	17.681	500	0.3769	134.171
	Heavy Rehabilitation	500	0.4891	69.579	500	0.9230	19.157	500	0.3864	134.685
Transverse Cracking	Preventive Maintenance	20	0.4358	63.691	20	0.3690	86.610	20	0.7176	71.803
	Light Rehabilitation	20	0.4746	75.839	20	0.3731	99.374	20	0.8037	73.516
	Medium Rehabilitation	20	0.5552	80.666	20	0.3785	113.924	20	0.8243	83.536
	Heavy Rehabilitation	20	0.5822	92.382	20	0.3859	131.583	20	0.9319	86.773
Ride Score	Preventive Maintenance	100	6.5480	7.813	100	11.0338	6.691	100	25.5350	7.693
	Light Rehabilitation	100	7.2460	11.699	100	13.8112	9.891	100	29.5048	10.423
	Medium Rehabilitation	100	9.9820	13.805	100	20.7165	12.294	100	32.1232	12.340
	Heavy Rehabilitation	100	45.8631	17.715	100	34.0580	19.631	100	32.8876	14.641
Patching	Preventive Maintenance	100	0.3359	83.461	100	0.8593	67.034	100	2.5366	8.668
	Light Rehabilitation	100	0.4016	93.681	100	0.9358	78.699	100	2.8353	9.733
	Medium Rehabilitation	100	0.4761	104.913	100	1.0869	83.166	100	3.3584	11.402
	Heavy Rehabilitation	100	0.5588	115.928	100	1.1233	93.555	100	3.5313	11.072

Table 10. Coefficients for Zone 2, Pavement Family C.

Distress Type	Treatment Type	Low Traffic α	Low Traffic β	Low Traffic A	Medium Traffic α	Medium Traffic β	Medium Traffic A	High Traffic α	High Traffic β	High Traffic A
Shallow Rutting	Preventive Maintenance	100	0.3609	90.434	100	1.0252	19.178	-	-	-
	Light Rehabilitation	100	0.3723	105.540	100	1.0588	19.742	-	-	-
	Medium Rehabilitation	100	0.3928	126.660	100	1.1072	20.991	-	-	-
	Heavy Rehabilitation	100	0.4240	129.607	100	1.1713	22.241	-	-	-
Deep Rutting	Preventive Maintenance	100	0.7526	50.125	100	0.7514	70.951	-	-	-
	Light Rehabilitation	100	1.0232	52.728	100	0.8371	71.959	-	-	-
	Medium Rehabilitation	100	1.2005	54.685	100	0.8480	79.890	-	-	-
	Heavy Rehabilitation	100	1.2702	55.370	100	0.9432	80.938	-	-	-
Failures	Preventive Maintenance	20	3.2017	28.735	20	8.8245	36.661	-	-	-
	Light Rehabilitation	20	3.8285	34.190	20	8.9327	40.177	-	-	-
	Medium Rehabilitation	20	4.3163	37.666	20	9.3610	45.060	-	-	-
	Heavy Rehabilitation	20	4.5719	39.459	20	10.1440	52.299	-	-	-
Block Cracking	Preventive Maintenance	100	6.0337	64.645	100	1.0832	55.376	-	-	-
	Light Rehabilitation	100	6.4552	68.438	100	1.1042	59.699	-	-	-
	Medium Rehabilitation	100	6.5778	68.145	100	1.1232	64.997	-	-	-
	Heavy Rehabilitation	100	7.6839	78.587	100	1.1401	70.273	-	-	-
Alligator Cracking	Preventive Maintenance	100	0.5517	100.842	100	0.4528	116.797	-	-	-
	Light Rehabilitation	100	0.5949	104.362	100	0.6259	128.241	-	-	-
	Medium Rehabilitation	100	0.6812	114.560	100	0.6761	128.140	-	-	-
	Heavy Rehabilitation	100	0.6891	134.264	100	0.8885	141.572	-	-	-
Longitudinal Cracking	Preventive Maintenance	500	0.4358	114.982	500	0.4769	117.557	-	-	-
	Light Rehabilitation	500	0.5040	130.979	500	0.5569	136.527	-	-	-
	Medium Rehabilitation	500	0.5566	142.261	500	0.6274	152.260	-	-	-
	Heavy Rehabilitation	500	0.5857	146.802	500	0.6808	162.752	-	-	-
Transverse Cracking	Preventive Maintenance	20	0.6017	101.991	20	0.7925	83.554	-	-	-
	Light Rehabilitation	20	0.6512	105.862	20	0.9337	92.407	-	-	-
	Medium Rehabilitation	20	0.7497	117.399	20	1.0646	97.509	-	-	-
	Heavy Rehabilitation	20	0.7664	137.527	20	1.1744	100.351	-	-	-
Ride Score	Preventive Maintenance	100	13.1644	6.288	100	25.3864	5.219	-	-	-
	Light Rehabilitation	100	15.8976	8.418	100	36.6219	7.719	-	-	-
	Medium Rehabilitation	100	17.2942	10.313	100	53.8623	11.815	-	-	-
	Heavy Rehabilitation	100	30.0751	17.969	100	63.1329	14.880	-	-	-
Patching	Preventive Maintenance	100	0.5180	114.501	100	0.4607	80.693	-	-	-
	Light Rehabilitation	100	0.7016	118.848	100	0.5422	88.913	-	-	-
	Medium Rehabilitation	100	1.0009	128.179	100	0.6204	94.957	-	-	-
	Heavy Rehabilitation	100	1.4513	143.538	100	0.6888	98.179	-	-	-

-: not sufficient data

Table 11. Coefficients for Zone 4, Pavement Family A.

		Low Traffic			Medium Traffic			High Traffic		
Distress Type	**Treatment Type**	**α**	**β**	**A**	**α**	**β**	**A**	**α**	**β**	**A**
Shallow Rutting	Preventive Maintenance	100	0.4108	95.067	100	0.5018	98.456	100	0.5606	94.600
	Light Rehabilitation	100	0.4735	107.712	100	0.5871	113.689	100	0.6413	106.485
	Medium Rehabilitation	100	0.5189	116.581	100	0.6639	127.458	100	0.6899	111.581
	Heavy Rehabilitation	100	0.5402	119.554	100	0.7242	136.646	100	0.6955	132.396
Deep Rutting	Preventive Maintenance	100	0.7184	87.341	100	0.7972	65.325	100	0.7015	97.278
	Light Rehabilitation	100	0.7837	91.275	100	0.9275	68.505	100	0.8103	109.823
	Medium Rehabilitation	100	0.9152	102.451	100	1.0014	81.660	100	0.8873	118.327
	Heavy Rehabilitation	100	0.9564	102.418	100	1.2207	90.624	100	0.9192	119.971
Failures	Preventive Maintenance	20	1.3078	84.969	20	0.6435	89.230	20	0.7184	90.653
	Light Rehabilitation	20	1.3769	101.571	20	0.7120	95.363	20	0.7988	98.036
	Medium Rehabilitation	20	1.4861	104.156	20	0.8537	110.586	20	0.8071	115.605
	Heavy Rehabilitation	20	1.6434	109.057	20	0.9330	117.319	20	0.8939	124.500
Block Cracking	Preventive Maintenance	100	5.7599	83.156	100	4.5303	88.282	100	4.9377	75.820
	Light Rehabilitation	100	6.1242	92.568	100	5.1859	90.655	100	5.1837	83.509
	Medium Rehabilitation	100	6.1930	98.757	100	6.2176	98.531	100	5.1882	88.347
	Heavy Rehabilitation	100	7.1798	99.863	100	6.5482	112.785	100	5.9760	88.195
Alligator Cracking	Preventive Maintenance	100	0.6516	77.787	100	0.6186	65.241	100	0.7096	83.860
	Light Rehabilitation	100	0.6739	91.907	100	0.7148	69.562	100	0.7591	85.829
	Medium Rehabilitation	100	0.7106	109.501	100	0.7859	70.948	100	0.8541	92.546
	Heavy Rehabilitation	100	0.7642	111.658	100	0.8193	81.983	100	1.0085	105.588
Longitudinal Cracking	Preventive Maintenance	500	0.3939	47.089	500	0.4688	72.340	500	0.4197	61.462
	Light Rehabilitation	500	0.4138	52.886	500	0.4721	81.726	500	0.4787	64.017
	Medium Rehabilitation	500	0.4523	62.514	500	0.4738	92.890	500	0.5128	75.811
	Heavy Rehabilitation	500	0.5132	63.987	500	0.4744	105.664	500	0.5144	84.486
Transverse Cracking	Preventive Maintenance	20	0.3528	48.282	20	0.4028	44.258	20	0.5606	74.084
	Light Rehabilitation	20	0.3737	55.165	20	0.4157	48.789	20	0.5688	85.109
	Medium Rehabilitation	20	0.4152	65.752	20	0.4391	55.511	20	0.5785	97.819
	heavy Rehabilitation	20	0.4829	69.100	20	0.4739	64.470	20	0.5892	113.350
Ride Score	Preventive Maintenance	100	10.4722	7.125	100	3.2094	9.548	100	2.2680	8.740
	Light Rehabilitation	100	11.3419	8.072	100	4.3653	10.204	100	2.3867	11.523
	Medium Rehabilitation	100	11.5088	8.918	100	9.1232	11.271	100	4.0318	13.072
	Heavy Rehabilitation	100	14.0200	15.866	100	18.2589	14.657	100	40.3059	17.865
Patching	Preventive Maintenance	100	4.8049	7.918	100	0.7426	82.406	100	0.4438	110.778
	Light Rehabilitation	100	4.9157	9.320	100	0.8719	89.513	100	0.5037	123.014
	Medium Rehabilitation	100	5.6531	9.589	100	0.9881	94.790	100	0.5337	127.310
	Heavy Rehabilitation	100	6.0879	11.858	100	1.0798	96.190	100	0.6319	146.967

Table 12. Coefficients for Zone 3, Pavement Family C.

		Low Traffic			Medium Traffic			High Traffic		
Distress Type	**Treatment Type**	**α**	**β**	**A**	**α**	**β**	**A**	**α**	**β**	**A**
Shallow Rutting	Preventive Maintenance	100	0.5018	79.913	100	0.4857	83.233	-	-	-
	Light Rehabilitation	100	0.5285	80.059	100	0.5222	85.521	-	-	-
	Medium Rehabilitation	100	0.5795	83.501	100	0.5953	94.043	-	-	-
	Heavy Rehabilitation	100	0.6597	91.378	100	0.5977	109.161	-	-	-
Deep Rutting	Preventive Maintenance	100	0.5855	93.415	100	0.7096	78.888	-	-	-
	Light Rehabilitation	100	0.6650	103.394	100	0.7370	93.371	-	-	-
	Medium Rehabilitation	100	0.7037	106.432	100	0.7828	93.355	-	-	-
	Heavy Rehabilitation	100	0.8311	122.377	100	0.8504	95.643	-	-	-
Failures	Preventive Maintenance	20	0.6355	84.105	20	0.7933	73.533	-	-	-
	Light Rehabilitation	20	0.6825	86.382	20	0.7939	83.714	-	-	-
	Medium Rehabilitation	20	0.7749	93.608	20	0.9422	93.756	-	-	-
	Heavy Rehabilitation	20	0.9278	108.344	20	1.1042	103.502	-	-	-
Block Cracking	Preventive Maintenance	100	1.0671	46.622	100	0.5669	98.777	-	-	-
	Light Rehabilitation	100	1.0885	50.891	100	0.9919	106.569	-	-	-
	Medium Rehabilitation	100	1.1097	55.266	100	2.1416	123.165	-	-	-
	Heavy Rehabilitation	100	1.1299	59.630	100	2.2042	127.028	-	-	-
Alligator Cracking	Preventive Maintenance	100	0.5767	74.390	100	0.5356	53.407	-	-	-
	Light Rehabilitation	100	0.5857	85.230	100	0.5811	62.652	-	-	-
	Medium Rehabilitation	100	0.5970	98.038	100	0.6726	66.191	-	-	-
	Heavy Rehabilitation	100	0.6104	113.705	100	0.6941	73.975	-	-	-
Longitudinal Cracking	Preventive Maintenance	500	0.5437	74.077	500	0.4769	45.353	-	-	-
	Light Rehabilitation	500	0.5518	85.181	500	0.4940	50.004	-	-	-
	Medium Rehabilitation	500	0.5620	97.982	500	0.5247	56.872	-	-	-
	Heavy Rehabilitation	500	0.5742	113.639	500	0.5714	66.003	-	-	-
Transverse Cracking	Preventive Maintenance	20	0.5026	86.866	20	0.4108	66.266	-	-	-
	Light Rehabilitation	20	0.5515	92.036	20	0.4809	71.933	-	-	-
	Medium Rehabilitation	20	0.6530	105.153	20	0.5448	75.750	-	-	-
	Heavy Rehabilitation	20	0.6993	108.469	20	0.5968	77.191	-	-	-
Ride Score	Preventive Maintenance	100	3.1563	8.733	100	4.1016	9.742	-	-	-
	Light Rehabilitation	100	4.1974	11.957	100	4.1038	11.834	-	-	-
	Medium Rehabilitation	100	7.0883	13.102	100	6.7724	14.673	-	-	-
	Heavy Rehabilitation	100	36.5778	16.960	100	26.8085	17.096	-	-	-
Patching	Preventive Maintenance	100	0.4938	99.509	100	0.3198	99.518	-	-	-
	Light Rehabilitation	100	0.5303	102.783	100	0.3457	103.833	-	-	-
	Medium Rehabilitation	100	0.6025	112.586	100	0.3992	115.789	-	-	-
	Heavy Rehabilitation	100	0.6032	130.797	100	0.4103	137.416	-	-	-

-: not sufficient data

Table 13. Coefficients for Zone 4, Pavement Family A.

		Low Traffic			Medium Traffic			High Traffic		
Distress Type	**Treatment Type**	**α**	**β**	**A**	**α**	**β**	**A**	**α**	**β**	**A**
Shallow Rutting	Preventive Maintenance	100	0.4857	68.003	100	0.5402	91.831	100	0.7015	58.533
	Light Rehabilitation	100	0.5726	75.098	100	0.5403	101.785	100	0.7790	59.522
	Medium Rehabilitation	100	0.6569	80.263	100	1.0075	105.464	100	0.7837	66.283
	Heavy Rehabilitation	100	0.7330	83.116	100	1.2240	121.859	100	0.8636	66.048
Deep Rutting	Preventive Maintenance	100	0.7345	53.560	100	0.6979	84.021	100	0.8674	71.629
	Light Rehabilitation	100	0.7914	63.067	100	0.7263	95.613	100	1.0289	79.633
	Medium Rehabilitation	100	0.9023	64.787	100	1.1251	106.155	100	1.1885	86.225
	Heavy Rehabilitation	100	0.9043	70.857	100	1.5209	114.103	100	1.3355	89.960
Failures	Preventive Maintenance	20	0.6097	90.248	20	0.7675	69.418	20	6.4741	86.782
	Light Rehabilitation	20	0.6796	97.316	20	0.9028	75.749	20	6.8467	95.521
	Medium Rehabilitation	20	0.6907	114.396	20	1.0263	80.291	20	8.1177	98.691
	Heavy Rehabilitation	20	0.7722	123.769	20	1.1267	81.441	20	9.0408	114.640
Block Cracking	Preventive Maintenance	100	3.4006	35.018	100	0.6516	89.383	100	9.4220	77.940
	Light Rehabilitation	100	3.5831	36.706	100	0.7215	95.435	100	10.2113	81.912
	Medium Rehabilitation	100	3.7232	38.200	100	0.7215	110.709	100	12.0410	93.541
	Heavy Rehabilitation	100	3.8149	38.794	100	0.7901	117.514	100	12.9380	97.763
Alligator Cracking	Preventive Maintenance	100	0.7426	42.117	100	0.5018	69.655	100	1.2379	20.025
	Light Rehabilitation	100	0.7465	44.720	100	0.5961	78.005	100	1.2929	20.881
	Medium Rehabilitation	100	0.8919	47.604	100	0.6951	85.408	100	1.3679	22.131
	Heavy Rehabilitation	100	1.0562	50.387	100	0.7956	91.021	100	1.4656	23.380
Longitudinal Cracking	Preventive Maintenance	500	0.5602	57.347	500	0.9503	17.263	500	0.5054	67.766
	Light Rehabilitation	500	0.6022	60.668	500	0.9875	17.693	500	0.7462	80.308
	Medium Rehabilitation	500	1.1056	62.390	500	1.0463	19.333	500	1.1311	81.561
	Heavy Rehabilitation	500	1.5452	72.779	500	1.1303	20.742	500	1.7079	87.056
Transverse Cracking	Preventive Maintenance	20	1.7563	20.660	20	0.6365	57.821	20	0.6347	90.722
	Light Rehabilitation	20	1.8064	21.529	20	0.7478	66.725	20	0.7087	98.541
	Medium Rehabilitation	20	1.8461	22.400	20	0.9273	78.677	20	0.7226	117.711
	Heavy Rehabilitation	20	1.8750	22.270	20	1.1918	80.109	20	0.8113	128.853
Ride Score	Preventive Maintenance	100	8.3416	7.057	100	22.0963	6.317	100	22.2087	8.119
	Light Rehabilitation	100	9.4114	9.485	100	22.2358	6.671	100	23.9609	8.848
	Medium Rehabilitation	100	9.5308	12.124	100	29.1865	9.570	100	28.6356	12.984
	Heavy Rehabilitation	100	10.2518	16.933	100	34.9859	14.895	100	49.4240	15.671
Patching	Preventive Maintenance	100	0.3609	78.016	100	0.3609	90.434	100	0.4358	89.767
	Light Rehabilitation	100	0.4209	84.903	100	0.3723	105.540	100	0.4468	104.718
	Medium Rehabilitation	100	0.4738	88.875	100	0.3928	126.660	100	0.4652	123.864
	Heavy Rehabilitation	100	0.5144	89.440	100	0.4240	129.607	100	0.4915	148.472

Table 14. Coefficients for Zone 4, Pavement Family B.

		Low Traffic			Medium Traffic			High Traffic		
Distress Type	**Treatment Type**	**α**	**β**	**A**	**α**	**β**	**A**	**α**	**β**	**A**
Shallow Rutting	Preventive Maintenance	100	0.4688	72.340	100	0.4943	71.078	100	0.3359	83.386
	Light Rehabilitation	100	0.4721	81.726	100	0.7567	74.331	100	0.3633	86.395
	Medium Rehabilitation	100	0.4738	92.890	100	0.8675	85.705	100	0.4201	95.763
	Heavy Rehabilitation	100	0.4744	105.664	100	1.2212	91.486	100	0.4332	114.028
Deep Rutting	Preventive Maintenance	100	2.4955	10.876	100	1.0002	40.067	100	6.1746	40.067
	Light Rehabilitation	100	2.8385	11.997	100	1.1815	41.411	100	7.2516	44.993
	Medium Rehabilitation	100	2.9090	13.871	100	1.3504	48.870	100	7.4368	52.912
	Heavy Rehabilitation	100	3.2351	15.117	100	1.4909	56.910	100	8.0058	54.475
Failures	Preventive Maintenance	20	9.1651	42.912	20	1.3239	19.076	20	0.4269	36.120
	Light Rehabilitation	20	9.8326	50.621	20	1.3716	19.782	20	0.5033	37.275
	Medium Rehabilitation	20	11.4292	53.944	20	1.4342	20.971	20	0.5790	37.258
	Heavy Rehabilitation	20	12.0225	62.164	20	1.5128	22.160	20	0.6494	43.240
Block Cracking	Preventive Maintenance	100	3.9409	38.651	100	3.1598	9.927	100	4.6881	62.793
	Light Rehabilitation	100	4.1573	40.877	100	3.4909	10.519	100	5.5204	63.350
	Medium Rehabilitation	100	4.2859	41.920	100	4.0015	11.934	100	5.8582	68.793
	Heavy Rehabilitation	100	4.3163	49.964	100	4.7531	13.349	100	6.7074	81.099
Alligator Cracking	Preventive Maintenance	100	0.5284	75.499	100	0.4688	72.340	100	0.8175	14.739
	Light Rehabilitation	100	0.8295	81.883	100	0.4721	81.726	100	0.8406	15.494
	Medium Rehabilitation	100	1.0704	84.811	100	0.4738	92.890	100	0.8757	15.681
	Heavy Rehabilitation	100	1.2135	100.172	100	0.4744	105.664	100	0.9230	16.868
Longitudinal Cracking	Preventive Maintenance	500	0.4857	52.459	500	0.3029	69.502	500	0.2361	30.520
	Light Rehabilitation	500	0.5250	60.525	500	0.3626	78.750	500	0.2717	31.022
	Medium Rehabilitation	500	0.6042	63.467	500	0.4294	88.137	500	0.2987	35.432
	Heavy Rehabilitation	500	0.6170	70.261	500	0.5037	97.308	500	0.3125	38.317
Transverse Cracking	Preventive Maintenance	20	0.5150	61.057	20	0.4335	60.506	20	0.2860	84.411
	Light Rehabilitation	20	0.6620	69.745	20	0.5721	71.200	20	0.3117	88.383
	Medium Rehabilitation	20	0.8816	82.145	20	0.8170	72.622	20	0.3661	100.236
	Heavy Rehabilitation	20	1.1916	82.100	20	1.2019	78.725	20	0.3875	102.034
Ride Score	Preventive Maintenance	100	10.2113	5.578	100	8.0444	7.282	100	23.2921	9.808
	Light Rehabilitation	100	11.6209	8.061	100	9.3007	9.844	100	24.7355	11.391
	Medium Rehabilitation	100	14.1653	11.463	100	10.2826	14.196	100	35.0390	13.061
	Heavy Rehabilitation	100	14.5377	14.629	100	14.2588	18.383	100	43.3708	13.904
Patching	Preventive Maintenance	100	0.3690	67.794	100	3.2428	6.474	100	0.4688	41.625
	Light Rehabilitation	100	0.4108	69.028	100	3.3896	6.914	100	0.5500	43.066
	Medium Rehabilitation	100	0.4185	77.412	100	3.5879	7.980	100	0.6244	51.532
	Heavy Rehabilitation	100	0.4704	78.924	100	3.8444	8.092	100	0.6854	59.628

Table 15. Coefficients for Zone 4, Pavement Family C.

		Low Traffic			Medium Traffic			High Traffic		
Distress Type	**Treatment Type**	**α**	**β**	**A**	**α**	**β**	**A**	**α**	**β**	**A**
Shallow Rutting	Preventive Maintenance	100	0.5464	58.770	100	0.4189	73.365	-	-	-
	Light Rehabilitation	100	0.6680	68.025	100	0.4253	84.293	-	-	-
	Medium Rehabilitation	100	0.8667	81.134	100	0.4342	97.364	-	-	-
	Heavy Rehabilitation	100	1.1622	82.988	100	0.4463	112.568	-	-	-
Deep Rutting	Preventive Maintenance	100	1.4906	20.897	100	0.6347	66.894	-	-	-
	Light Rehabilitation	100	1.5518	21.888	100	0.7401	72.403	-	-	-
	Medium Rehabilitation	100	1.6322	23.108	100	0.8276	74.517	-	-	-
	Heavy Rehabilitation	100	1.7345	24.659	100	0.8867	89.159	-	-	-
Failures	Preventive Maintenance	20	1.6646	15.290	20	0.4325	90.018	-	-	-
	Light Rehabilitation	20	1.9944	17.801	20	0.5276	105.137	-	-	-
	Medium Rehabilitation	20	2.3202	21.037	20	0.5987	121.527	-	-	-
	Heavy Rehabilitation	20	2.6185	23.340	20	0.6433	139.593	-	-	-
Block Cracking	Preventive Maintenance	100	8.3760	22.633	100	0.3435	89.146	-	-	-
	Light Rehabilitation	100	9.8528	24.148	100	0.4211	91.339	-	-	-
	Medium Rehabilitation	100	11.6766	25.368	100	0.8965	97.735	-	-	-
	Heavy Rehabilitation	100	13.9413	26.588	100	1.8197	110.368	-	-	-
Alligator Cracking	Preventive Maintenance	100	1.3409	17.738	100	0.3440	61.371	-	-	-
	Light Rehabilitation	100	1.3769	18.536	100	0.3931	64.243	-	-	-
	Medium Rehabilitation	100	1.4139	19.536	100	0.4237	76.314	-	-	-
	Heavy Rehabilitation	100	1.4519	20.536	100	0.4295	85.363	-	-	-
Longitudinal Cracking	Preventive Maintenance	500	0.4769	87.724	500	0.2611	60.109	-	-	-
	Light Rehabilitation	500	0.5261	94.121	500	0.2968	62.904	-	-	-
	Medium Rehabilitation	500	0.6296	109.377	500	0.3177	74.752	-	-	-
	Heavy Rehabilitation	500	0.6869	115.129	500	0.3188	83.490	-	-	-
Transverse Cracking	Preventive Maintenance	20	4.5633	33.282	20	3.4586	59.634	-	-	-
	Light Rehabilitation	20	5.4581	38.752	20	3.4822	63.154	-	-	-
	Medium Rehabilitation	20	6.0090	41.767	20	3.9848	75.357	-	-	-
	Heavy Rehabilitation	20	6.0435	48.960	20	4.3110	84.574	-	-	-
Ride Score	Preventive Maintenance	100	37.3475	6.303	100	36.8222	6.220	-	-	-
	Light Rehabilitation	100	47.1466	7.187	100	46.2955	7.182	-	-	-
	Medium Rehabilitation	100	58.9653	8.097	100	68.4628	9.547	-	-	-
	Heavy Rehabilitation	100	62.1132	19.726	100	88.8356	14.680	-	-	-
Patching	Preventive Maintenance	100	0.5175	42.291	100	1.1250	9.437	-	-	-
	Light Rehabilitation	100	1.2835	43.086	100	1.3254	10.633	-	-	-
	Medium Rehabilitation	100	1.8314	51.525	100	1.5030	12.190	-	-	-
	Heavy Rehabilitation	100	2.0950	59.587	100	1.6376	12.746	-	-	-

-: not sufficient data

Evaluating the Effects of Concrete Curing Compounds on Hydration

Prepared for
Undergraduate Transportation Scholars Program

by

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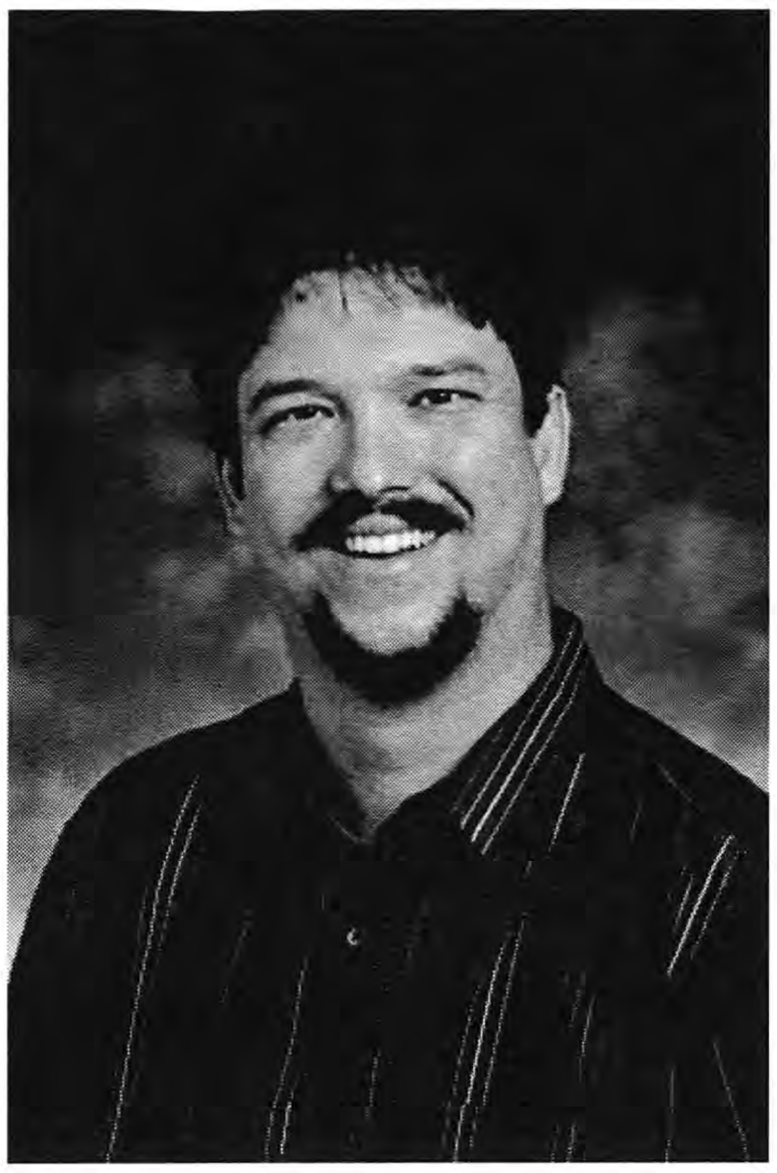
August 6, 2010



STUDENT BIOGRAPHY

Brian Ward is a senior at Colorado State University in Fort Collins, Colorado, and will be graduating in May 2011 with a Bachelors of Science in Civil Engineering. Brian is the president of the Tau Beta Pi honor society and is currently active with the adult and veterans services program at Colorado State University. In addition he has work experience with Stantec Engineering as a project coordinator and Holcim cement as a mine planning engineer. Brian plans to attend graduate school where he will pursue a master's degree in civil engineering.

Brian has served on the board of trustees for Christ United Methodist Church where he is a current member. Additionally, he volunteers in various roles inside the Windsor RE-4 school district where his wife works. Brian enjoys working with high school age youth to encourage youth to learn social development through sports. These roles include volleyball official, official book/announcer for the boys/girls basketball programs, ticket sales booth for football games, concession stand, and knowledge bowl reader. In addition to sports involvement Brian works with the Tau Beta Pi honor society and local schools on programs to encourage elementary aged youth to stay focused on hard science related fields.

ACKNOWLEDGMENT

The research described in this paper was conducted as part of Project 0-5106, sponsored by the Texas Department of Transportation (TxDOT). The research activities were conducted in support of the Undergraduate Transportation Scholars Program. The findings and recommendations included in this paper are based on the student's summer activities. They should be considered preliminary and not as representative of the findings and recommendations of the parent project. This paper has not been reviewed or approved by the sponsor. The contents of this paper reflect the views of the author, who is responsible for the facts and the accuracy of these data presented herein. The contents do not necessarily reflect the official view or policies of TxDOT.

The author would like to express his appreciation to Dr. Marvin Criswell for his support in the understanding of building materials, his wife Carol Ward for her support in the understanding that these building materials can be interesting, and his daughter Maddigan Ward's support in the understanding that these building materials can be made of bright colored plastic and contain pictures of Sesame Street characters.

SUMMARY

The purpose of this project was to evaluate various concrete curing compounds for their relative effectiveness to increase water available for hydration, study effectiveness of these curing compounds as a function of compressive strength, and evaluate the optimal time after casting to apply lithium based curing compounds. The research was based on Dr. Dan Zollinger, Dr. Anol Mukhopadhyay, and Dan Ye's original research from technical report 0-5106-3. The current

ASTM standard for evaluating concrete curing compounds has several deficiencies that this research looks to overcome. The curing compound effectiveness research evaluated five different curing compounds as a function of differing performance indicators including moisture loss, di-electric constant, relative humidity, and compressive strength. The application time research evaluated one lithium based compound at differing application times as a function of compressive strength.

Based on the preliminary findings the lithium curing compound seems ineffective in stopping moisture loss in concrete during the curing process and does not increase compressive strength. A clear trend in the time of application of the lithium compound places the optimal application time at 2.5–3.0 hours after casting under the testing curing conditions. Differential hardening of the concrete specimens was observed and offers evidence that the lithium compound should not be added to concrete prior to one hour after casting. Based on Electron Scanning Microscope images the lithium compound is seen to cause a clear effect of the surface of the concrete surface. It is unclear is this layer is retaining moisture.

The Eco-Cure (clay based) compound seemed ineffective as well in all parameters. The Sinak Relay compound tested approximately the same in regards to moisture loss and compressive strength as the WRM 1250 compound, which is commonly used in Texas as a curing compound. The WRM 2255 compound performed as expected from previous work and was used as the standard for all other compounds. The results of this report are consistent with previous work reported in technical report 0-5601-3.

TABLE OF CONTENTS

Student Biography 226
Acknowledgment 226
Summary 226
Table of Contents 228
Introduction 229
Background 229
Data Collection 230
  Disc Samples 231
  Compressive Strength Cube Samples 232
  Application Rate Cube Samples 233
Results 233
  Disc Samples 233
  Compressive Strength Cube Samples 235
  Application Rate Cube Samples 236
Errors 237
  Disc Samples 237
  Compressive Strength Cube Samples 238
  Application Rate Cube Samples 238
Conclusions 238
  Disc Samples 238
  Compressive Strength Cube Samples 238
  Application Rate Cube Samples 239
References 239
Appendix A: Results of Disc Tests 240
Appendix B: Results of Cube Tests 246
Appendix C: Results of Lithium Time Tests 248

INTRODUCTION

Excessive early-age water evaporation from the surface of concrete pavement may induce detrimental impacts, i.e., high porosity, delamination (leading to spalling), and loss of strength on long-term performance of the pavement. Spalling involves a breakdown or dislodging of concrete segments along a crack or joint in a concrete slab within 0.6 m (2 ft) of a crack or joint (*1*), and it affects the quality of concrete pavement smoothness and riding quality.

The Texas Department of Transportation (TxDOT) has recently experienced cases of spalling and delamination failures, which may be related to excessive early age evaporation worsened under the influence of certain field conditions such as high temperature, low relative humidity, high solar radiation, and high wind speed. To mitigate early-age unexpected water loss, application of curing compounds in concrete paving has been widely used to minimize evaporation. However, the TxDOT standard specifications for pavement construction (Item 360) only defines the use of the membrane curing in terms of key characteristics such as percent solids, density, viscosity, color, and the application rate, but does not specify curing performance or limits on the rate of evaporation. The current laboratory curing membrane effectiveness evaluation method ASTM C 156 has some intrinsic deficiencies, i.e., irrelevance of the test conditions to field performance, mortar's hardening effect, and questionable basis for the moisture loss limits. Therefore, a new laboratory test protocol was created to evaluate the curing membrane effectiveness in controlling evaporation (*2*).

The previously created test protocol was created to study resin and wax based membrane barrier compounds. Lithium based compounds, which stop water loss by differing methods, was examined in this testing for both performance characteristics and for verification that the newly created testing method is applicable to these types of compounds.

BACKGROUND

Previous studies have been done into the effectiveness of concrete curing compounds as well as into validating the suggested test protocol as reported in technical report 0-5106-3 "Laboratory and Field Evaluation of Concrete Paving Curing Effectiveness." Little was done during this previous work to investigate lithium based curing compounds, or to validate the test protocol to these types of compounds. The purpose of the current project is three fold. The first objective is to perform testing using the proposed testing protocol from 0-5106-3 in order to classify lithium compounds in terms of effectiveness of curing compounds using the same indicator parameters as the previous work, and to validate the testing protocols for lithium type hardener compounds. The second objective is to compare the compressive strength of the five differing compounds at 1, 3, 7, and 28 days as a function of compressive strength. The third objective is to evaluate the application time of the lithium compound as a function of 3 day compressive strength. This objective is examining if a trend exist between time of application of the lithium compound and the compressive strength of the concrete at three days to establish an optimal time to apply the compound after casting. If the suggested test protocol is applicable to lithium style compounds then similar results should be obtained in reference to technical report 0-5106-3.

DATA COLLECTION

Data were collected for each part of the research investigation including disc, porosity, and cube samples. All mortar specimens used for these experiments were cast from a 0.40 *w/c* ratio design mix as outlined in Table 1.

Table 1. Mixture Proportion.

Mixture	W/C	Unit Weight (lb/ft^3)		
		Water	Cement	Sand
	0.40	13.94	34.84	95.81

The mortar was mixed using the same protocol used in previous experiments according to ASTM C 305, *Standard Practice for Mechanical Mixing of Hydraulic Cement Pastes and Mortars of Plastic Consistency,* using the following sequence:

1. The total amount of water was first placed in the mixing bowl.
2. The cement is introduced and mixed at a slow speed for 30 s.
3. The required amount of aggregate is added to the mixer over a period of 30 s while the mixer continues to operate.
4. The resulting mortar is allowed to mix for an additional 30 s at a medium speed.
5. After a minute rest period, the mixing is continued for an additional minute until a homogeneous mortar with no lumps is obtained.

A method of identifying the samples and tests was created for these experiments and is outlined in Tables 2 and 3 for the disc and cube samples respectively.

Table 2. Disc Annotation.

First #	Test # from above
Second Letter	Disc Sample
2-B	Test 2 (Sinak Relay at 60) second disc tested

Table 3. Cube Annotation.

First #	Test # from above
Second #	Sample ID
Third #	Day Sample was tested
2 1 3	Test 2 (Sinak Relay at 60) - Sample 1 - Tested on Day 3

Disc Samples

Mortar disc were poured using a .40 w/c ratio as outlined in Table 1. All discs used for these experiments measured 12 inches in diameter by 4 inches tall. A square chamber assembly was attached to the concrete specimen by use of commercial silicon adhesive and curing compound was added to the surface of the disc immediately after casting in amounts outlined in Table 4 corresponding to an application rate of 180 ft^2/gallon. The testing schedule for the specific curing compounds studied is included as Table 5.

Table 4. Application Rate (180 ft^2/gallon).

Specific Weight	Area (ft^2)	Mass of Compound (Grams)
1.00	0.785	16.500
1.06	0.785	17.500
1.00	0.028	0.584
1.06	0.028	0.619

Table 5. Test Combinations.

Test #	Curing Compound	Application Rate (ft^2/gallon)	Comments
1	Sinak Relay	180	Lithium-Based with Resin component
2	Sinak Relay	60	Lithium-Based with Resin component
3	Sinak Lithium	180	Lithium-Based
4	Sinak Lithium	60	Lithium-Based
5	Eco-Cure	180	Clay-Based
6	Eco-Cure	60	Clay-Based
7	WRM 1250	180	Normal Resin-Based
8	WRM 1250	60	Normal Resin-Based
9	WRM 2255	180	High Reflective
10	WRM 2255	60	High Reflective

The mortar discs were then placed on a scale to measure moisture loss in an environmental chamber kept at conditions outlined in Table 6.

Table 6. Laboratory Standard Testing Conditions.

Wind Speed	Ambient Relative Humidity	Temperature
10 mph	30 percent	104 ±5 °F

Figure 1 shows the entire test setup for the current testing protocol.

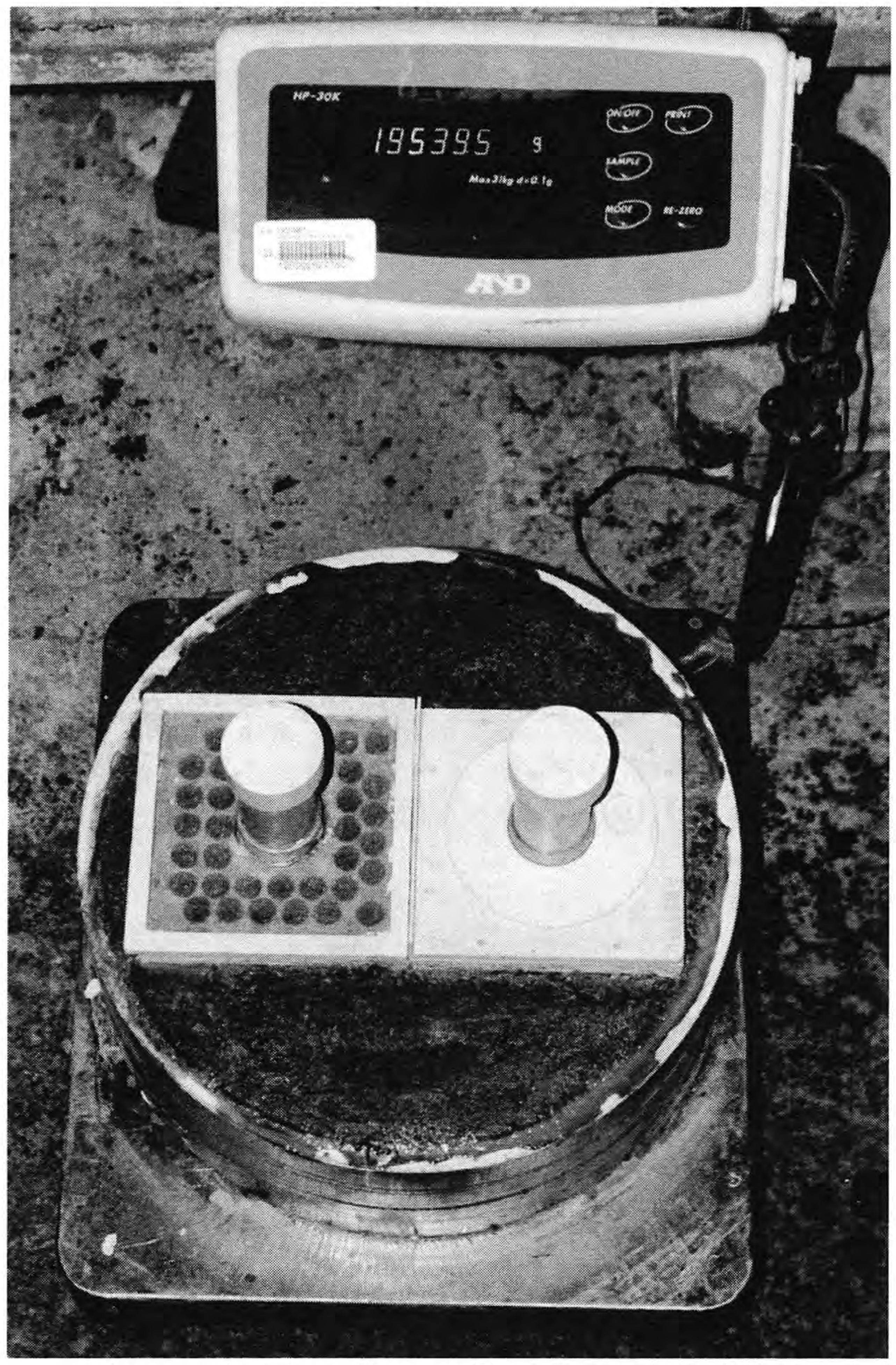


Figure 1. Test Setup for Concrete Disc.

Readings were taken every 30 minutes for the first five hours, every hour for the next five hours, and then at consistent intervals up to 48 hours for the following parameters:

1. Weight Reading – Taken by reading digital scale.
2. Filtered Chamber Relative Humidity and Temperature – Taken by inserting handheld sensor into filtered chamber assembly.
3. Sealed Chamber Relative Humidity and Temperature – Taken by inserting handheld sensor into sealed chamber assembly.
4. Ambient Relative Humidity and Temperature – Taken by placing handheld sensor in ambient position in the environmental chamber.
5. Dielectric Constant – Taken by collecting three reading with a Percometer and averaging the three readings.

Compressive Strength Cube Samples

Two by two inch cubes were cast by the following procedure:

1. Mortar was placed into each 2×2 inch opening to a volume of approximately ½ the depth of the mold (1 inch).
2. With a rubber tapping device the mortar was compacted 16 times, four times in four differing directions.
3. The entire brass mold (3 – 2×2 inch cubes) was tapped approximately 10 times.
4. Mortar was placed into each 2×2 inch mold to fill the mold to the top (2 inches).
5. With a rubber tapping device the mortar was compacted 16 times, four times in four differing directions.
6. The entire brass mold (3 – 2×2 inch cubes) was tapped approximately 10 times.
7. The tops of the 2×2 inch molds were smoothed (finished) with a metal trowel to the top of the brass mold assembly.

After the 2×2 inch cubes were created curing compounds were applied three hours after casting occurred. The application rates of the compounds of the cube samples were consistent Table 4 for an area of 0.028 ft^2. These samples were cured under the conditions outlined in Table 6 for the appropriate amount of time, i.e., 1, 3, 7, or 28 days. The specimens were then compressed to failure using a hydraulic compression machine and ultimate compressive strength data were collected. Each compound was added to a minimum of two cubes and the final compressive strength values were averaged to obtain a final value.

Application Rate Cube Samples

Two by two inch cubes were cast using the same procedure as described in the compressive strength cube sample procedure and lithium compound was added to saturation (Lithium compound was added to the cubes until the cubes would no longer hold the compound) to three cubes beginning at casting and for every 30 minutes thereafter until five hours.

These cubes were cured for three days consistent with the environmental conditions outlined in Table 6. The cubes were then compressed to failure by use of a compression machine to establish an ultimate compressive strength for each cube. The three ultimate compressive strengths were then averaged for an overall compressive strength of the test.

RESULTS

Results obtained for the testing are outlined in the following sections for each of the tests accordingly.

Disc Samples

Graphs outlining the raw collected data of the disc samples are included in Appendix A. Graphs include relative humidity (ambient, filtered chamber, and sealed chamber), moisture loss, and dielectric constant vs. time for each test performed. Figures 2, 3, and 4 display relative humidity, moisture loss, and dielectric vs. time respectively for each of the compounds.

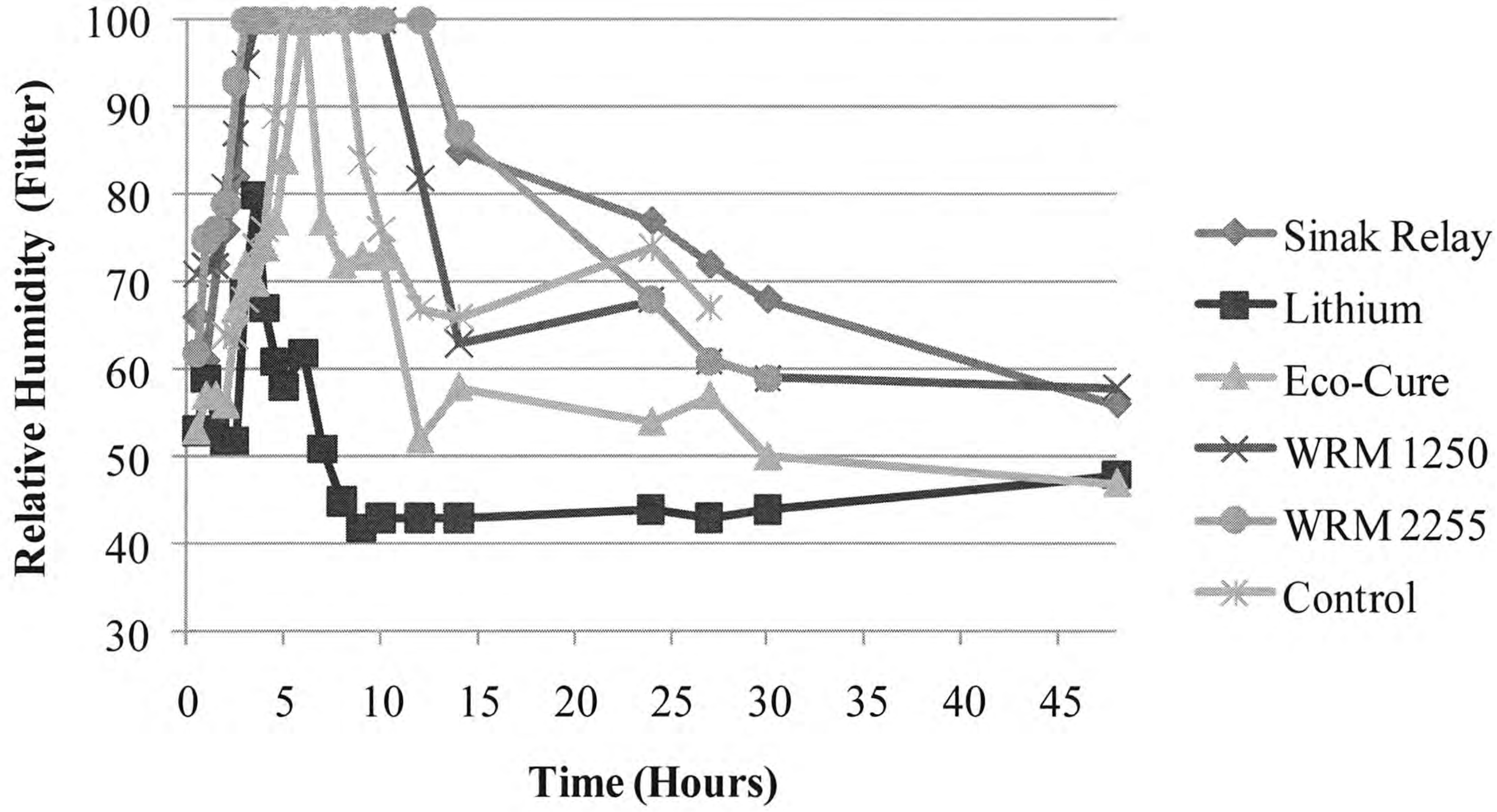


Figure 2. Relative Humidity vs. Time of Curing Compounds.

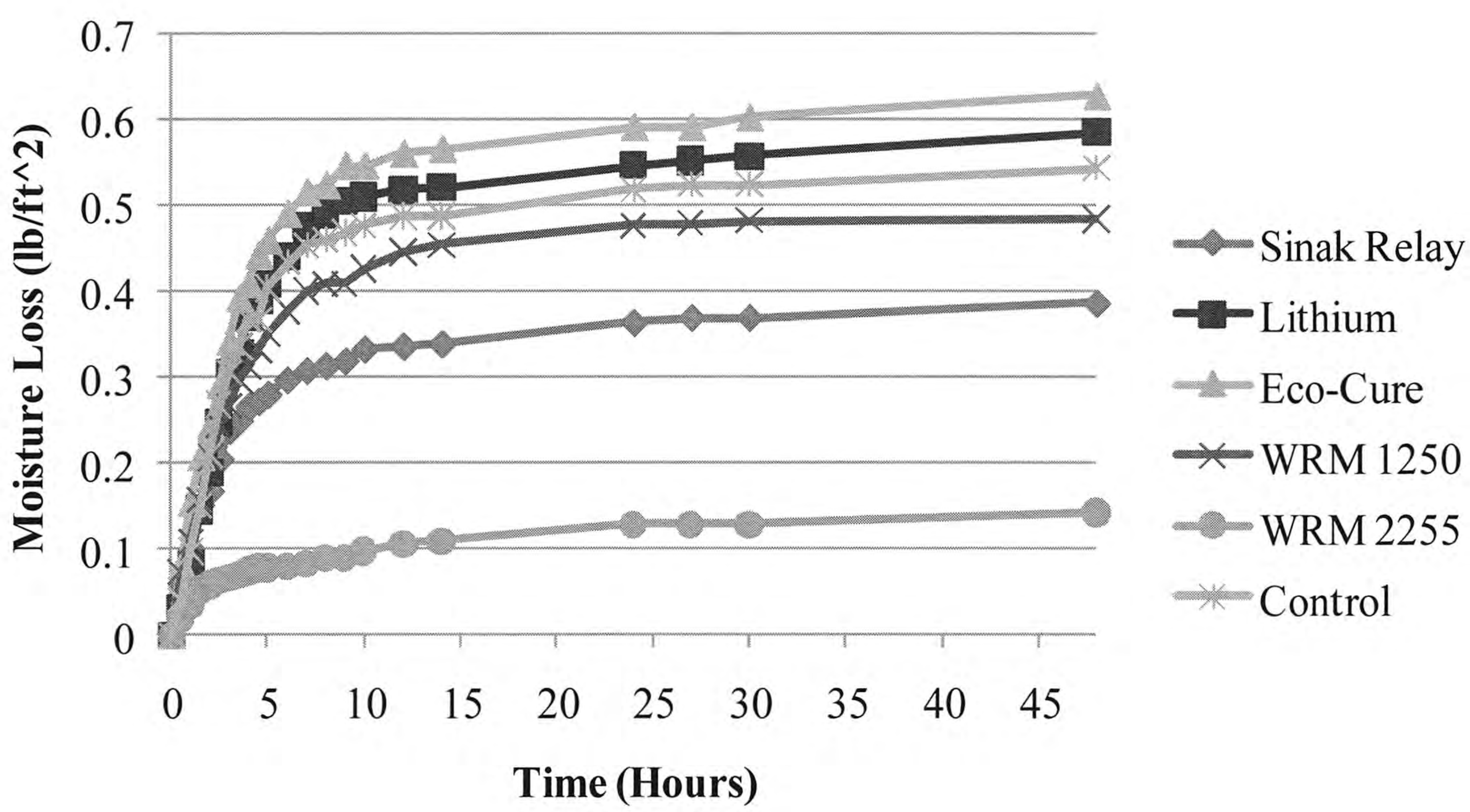


Figure 3. Moisture Loss vs. Time of Curing Compounds.

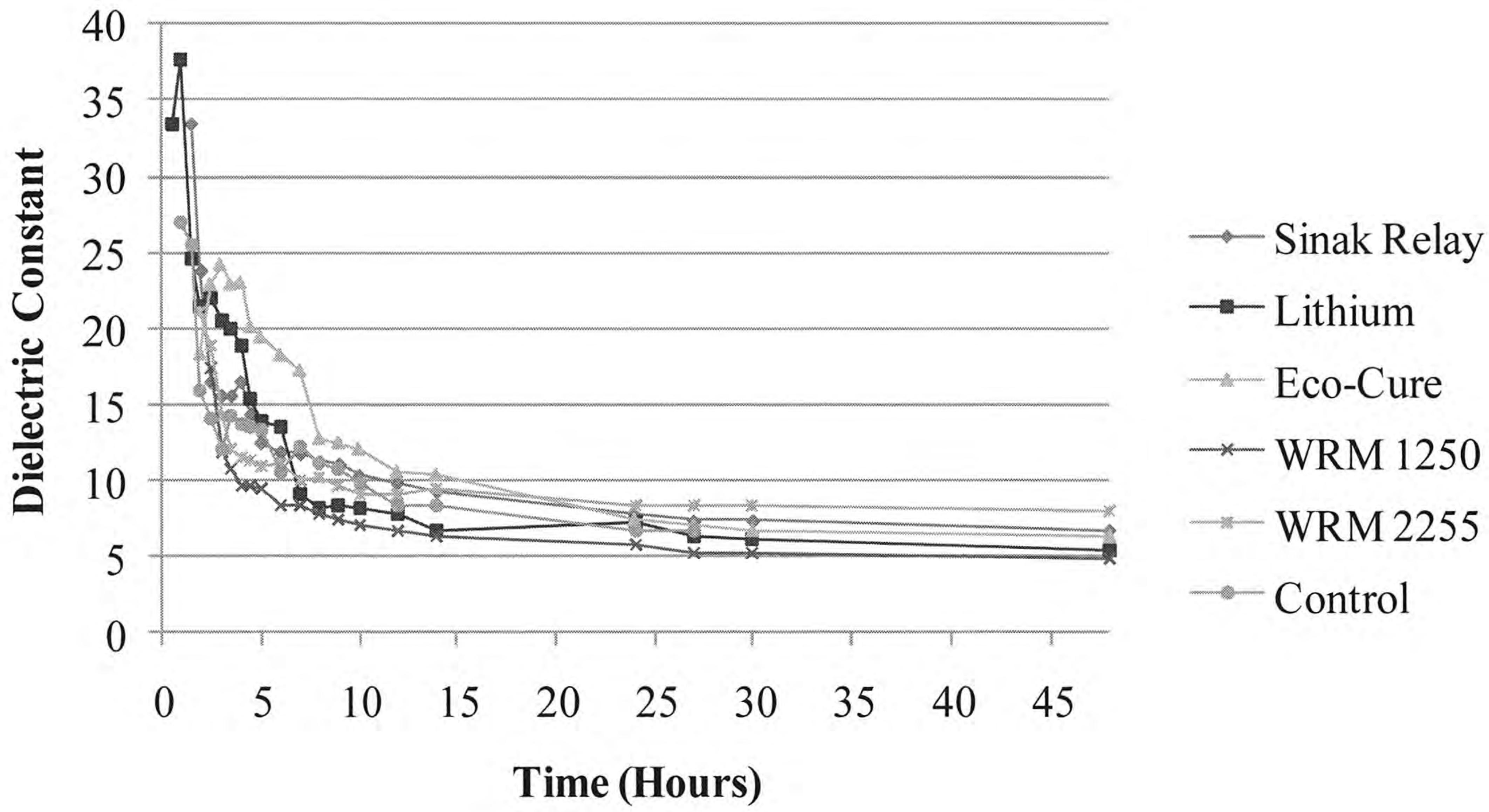


Figure 4. Dielectric Constant vs. Time of Curing Compounds.

Compressive Strength Cube Samples

Ranking for the compressive strengths of the cubes has been calculated and appears in Table 8. The compressive strength for each test as well as calculated extended statistics appears in Appendix B. Figure 5 displays the compressive strengths of all tests in bar graph form.

Table 8. Rankings for Compressive Strength Cube Samples.

Break Day				Product	Application Rate
1	3	7	28		
Ranking					
9	5	1	3	Sinak Relay	180
8	6	3	N/A	Sinak Relay	60
10	10	10	5	Sinak Lithium	180
4	7	9	N/A	Sinak Lithium	60
7	8	8	4	Eco-Cure	180
3	9	7	N/A	Eco-Cure	60
2	2	4	2	WRM 1250	180
6	1	5	N/A	WRM 1250	60
1	4	6	1	WRM 2255	180
5	3	2	N/A	WRM 2255	60

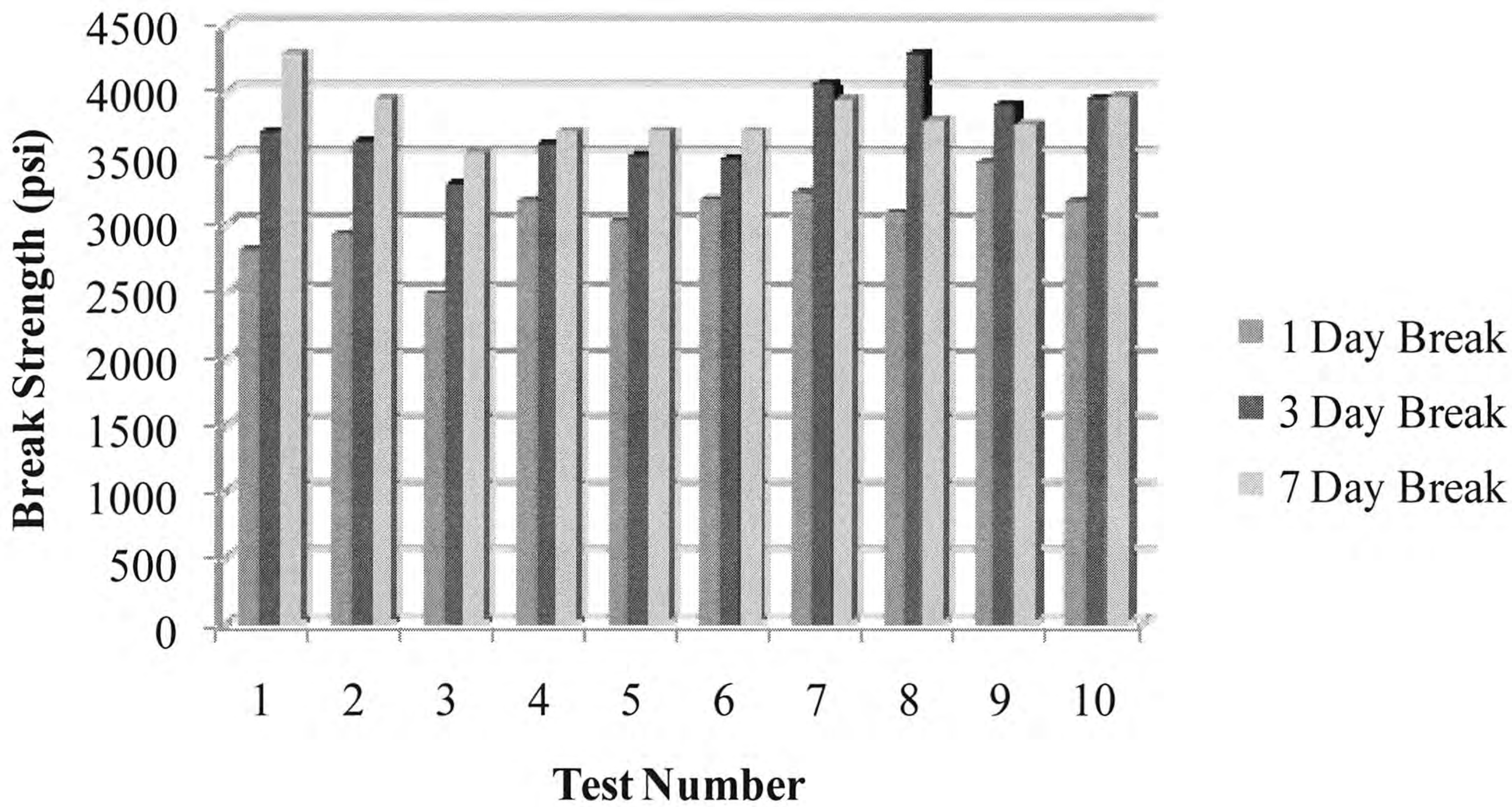


Figure 5. Strength vs. Test Number of Compressive Strength Cube Samples.

Application Rate Cube Samples

Figure 6 is a photo of cube samples 101-1-3 and 102-1-3 showing the delamination of the surface concrete. Figure 7 contains a graph of compressive strength vs. time at which the lithium compound was added.

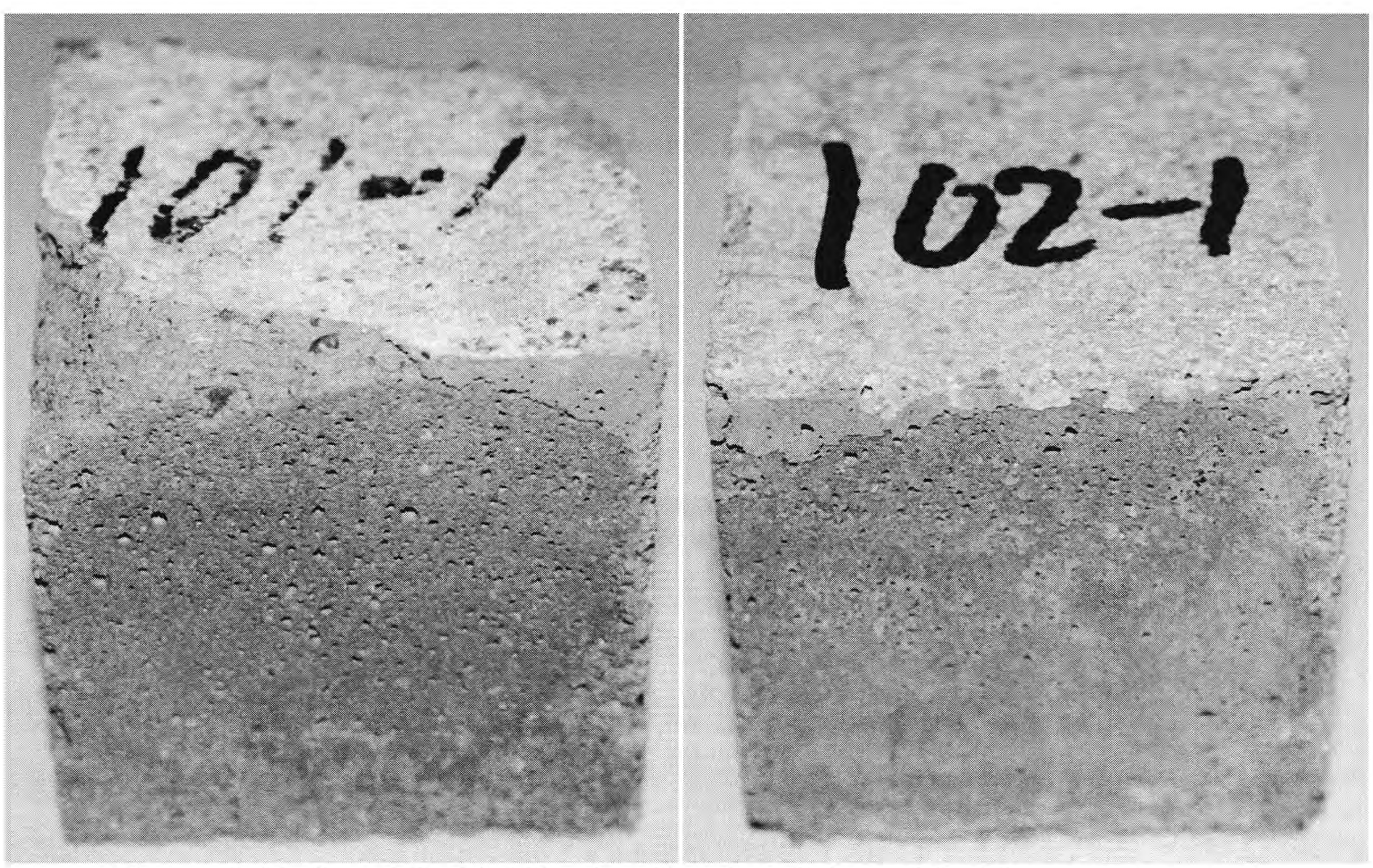


Figure 6. T=0 and T=30 Min. Application Cubes of Lithium Compound.

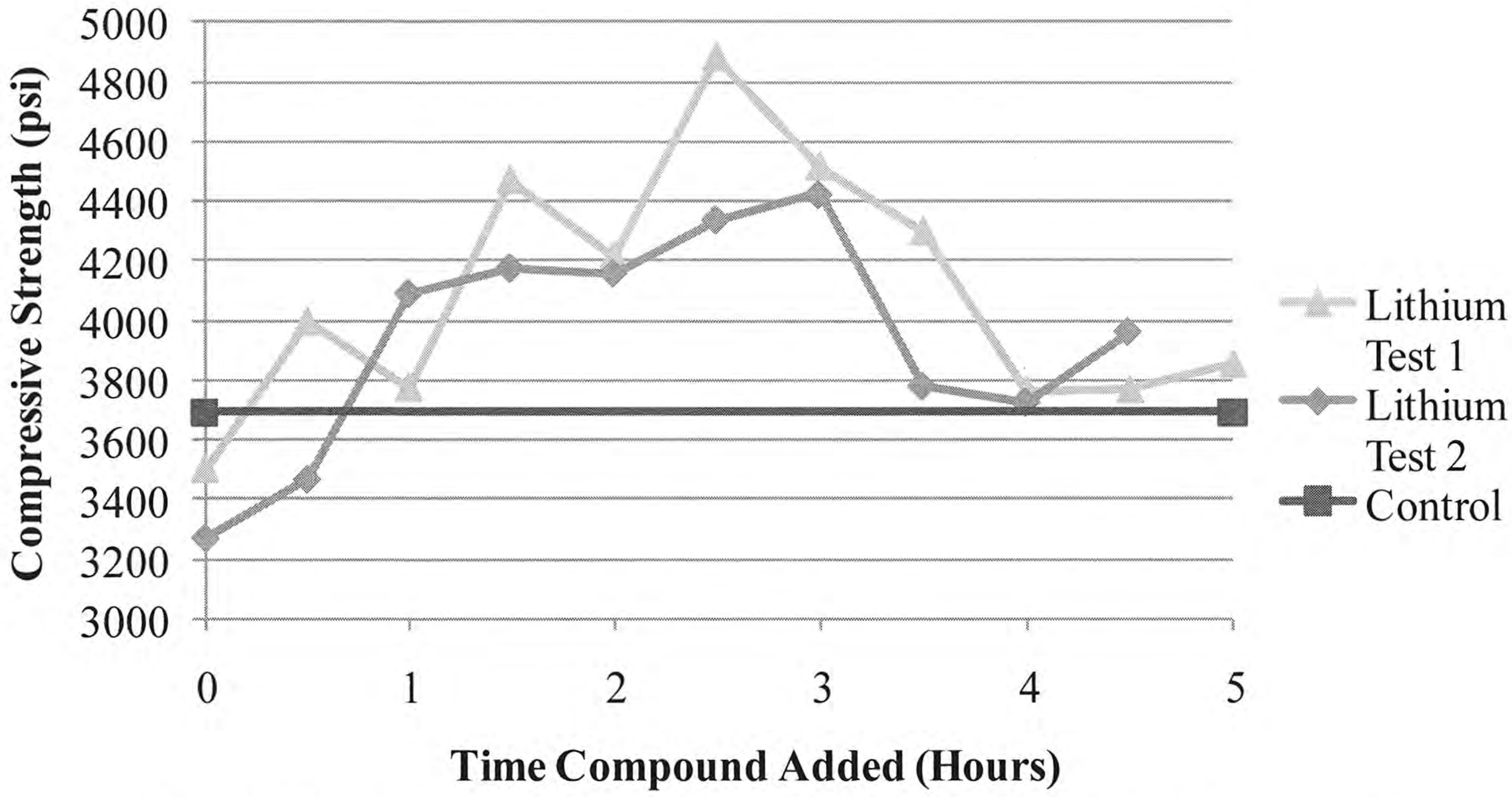


Figure 7. Strength vs. Time of Application of Lithium Compound.

ERRORS

Errors were observed in the testing and are reported in the following sections for each of the tests accordingly. No errors were observed that would serve to invalidate the experiments although some errors made the series of tests incompatible with previous results for direct comparison.

Disc Samples

The most difficult portion of this testing series to control was the application rate of the curing compound itself. With varying consistencies and weights, an exact measure of the true application rate was in question. It was attempted to use an application rate consistent with all tests; however this was not achievable by use of scales.

Wind speed was additionally very hard to exactly control and these errors were minimized by using an identical setup in each testing series. A calculated wind speed of 6.8 mph was used; however it is important to note this wind speed was an average of wind speed across the top of the disc. Differing wind speed factors in minimally into the total potential of evaporation calculation as the air directly above the disc was exchanged constantly.

The most important error between the current testing and previous work performed during the 0-5106-3 report was in the type of sand used for the concrete samples. In the 0-5106-3 reported work "Playground Sand" was used that has a uniform consistency and very small grain size. For the current work, concrete sand was used with a less uniform particle size distribution and larger grain size. This difference caused very large discrepancies between exact values of compressive strength between the two differing testing procedures. The relative strengths and characteristics between the samples is believed consistent; however the current work cannot be directly compared to the 0-5106-3 work without normalizing the results to factor in this difference.

Compressive Strength Cube Samples

It was noticed while performing the calculations for the compressive cube strengths that the seven day break strengths were sometimes below the three day break strengths. It is believed a possible error in the batch mixing of these seven day samples may have been introduced. All batches were mixed by the procedure described in the Data Collection section. Additionally the lithium compound application rate data for three day breaks had a average break strength of 4170 psi (not counting the T=0 and T=30 min data), which is much higher than the reported 3592 psi for the 60 ft^2/gallon data reported in Appendix B. It is believed that only two samples may not have been enough samples to properly determine the true compressive strength of the concrete for the compressive strength cube samples testing.

Application Rate Cube Samples

No reportable errors were observed in the application rate testing.

CONCLUSIONS

Conclusions from the observed results are reported in the following sections for each of the tests accordingly.

Disc Samples

The disc samples showed basic trends consistent with previous work. Samples that had less moisture loss also had longer times of relative humidity above the required 80 percent necessary for hydration to occur in concrete. Additionally samples that showed lower moisture loss also displayed signs of higher dielectric constants as expected based on results from previous work.

The primary objective of classifying and characterizing the lithium compound was achieved. In regards to moisture loss the lithium compound was ineffective. The relatively high moisture loss of the lithium compound can be seen in Figure 3. These results are validated by the WRM 1250 and WRM 2255 results, which are consistent with previous work.

The dielectric constant readings seem to indicate that good performing compounds (low moisture loss) returned relatively low dielectric constant readings. This is consistent with previous work and further validates the data.

Compressive Strength Cube Samples

The compressive strength of the cube samples is another indicator of performance of the curing compound. The general trend of the samples tested is consistent with the expected trends of previous work; however the magnitude of the compressive values differs by approximately a magnitude of two times. The lithium cube strengths displayed poor cube strength characteristics in this testing series but good strength characteristics in the lithium time test studies. This is an indicator that, even though the trends in this bar graph do follow general trends, the absolute values are in question. The general trend shows that the lithium compound is poor at retaining strength at 1, 3, 7, and 28 days.

The strait lithium compound was consistently the lowest in regards to compressive strength. The Sinak Relay product displayed trends of gaining strength as curing time developed with the Relay beginning ranked 9 at 1 day compressive strength and ending at 1 for 7 day compressive strength.

Application Rate Cube Samples

The data outlined in Figure 7 show a trend in the time of application of the lithium compound and the compressive strength of the concrete sample at three days. Based on the characteristics of the graph the optimal time to apply the lithium compound is approximately 2.5–3.0 hours after casting. Conditions for this time of application are relative to environmental conditions and actual optimal conditions should be adjusted for field conditions based on equivalent time calculations.

As an observation, referring to Figure 6 that shows samples 101-1-3 and 102-1-3 from the application time cube test, is was noted that the lithium when added at very early times can cause differential hardening of the concrete, ultimately leading to delamination of the surface of the concrete. This can be seen in Figure 6 as a light colored layer at the top of the cube that could be easily flaked off and had separated from the main body of the cube. This effect was the most evident in the T=0 (after casting) sample and was still visible in the T=30 minute sample. The T=60 minute sample did not display signs of differential hardening. Further studies should be performed to understand the effects of adding lithium compound very early after casting.

Using a scanning electron microscope it was observed that the lithium compound formed a very tight layer that could reduce evaporation from the specimen. However it is was also observed that this layer contained multiple fissures and holes that were larger than the effective pore size of evaporation of water.

REFERENCES

1. Miller, J.S. and W.Y. Bellinger (2003). Distress Identification Manual for the Long-Term Pavement Performance Program. McLean, Virginia, Federal Highway Administration.
2. Zollinger, D.G., A.K. Mukhopadhyay, and D. Ye (2009). Laboratory and Field Evaluation of Concrete Paving Curing Effectiveness, Texas Transportation Institute, College Station, Texas.

APPENDIX A: RESULTS OF DISC TESTS

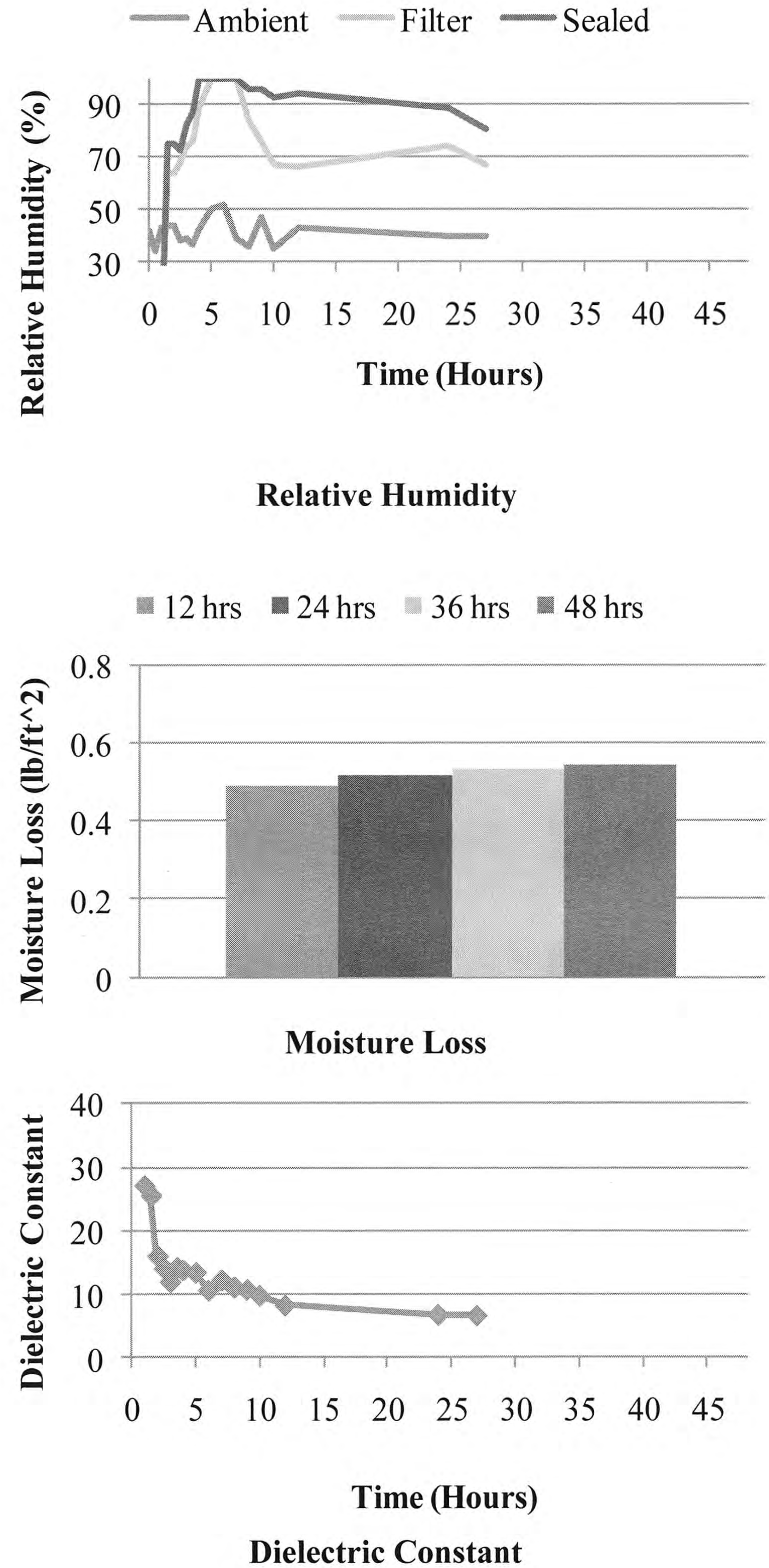


Figure A-1. Control: 1st Trial.

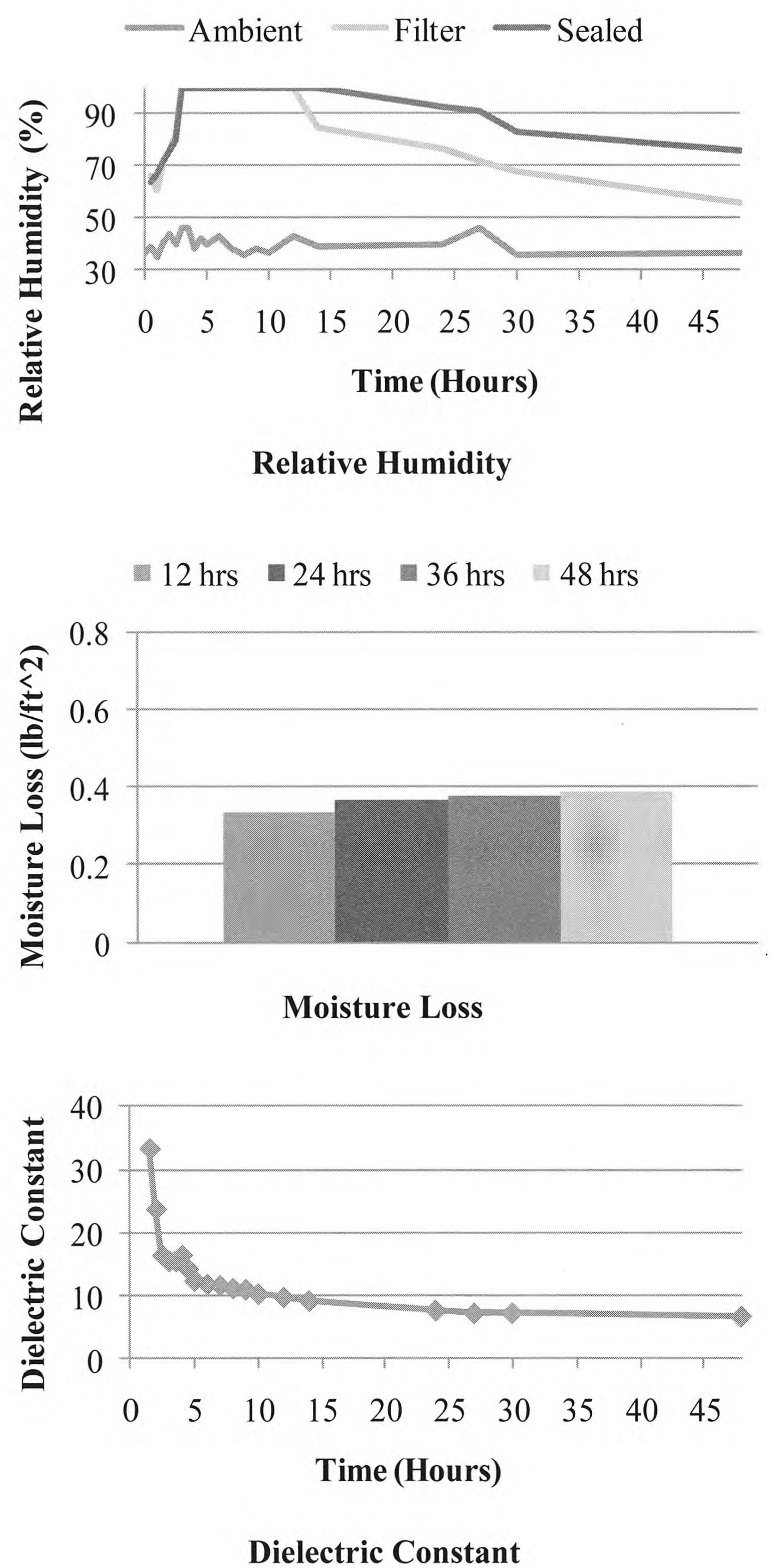
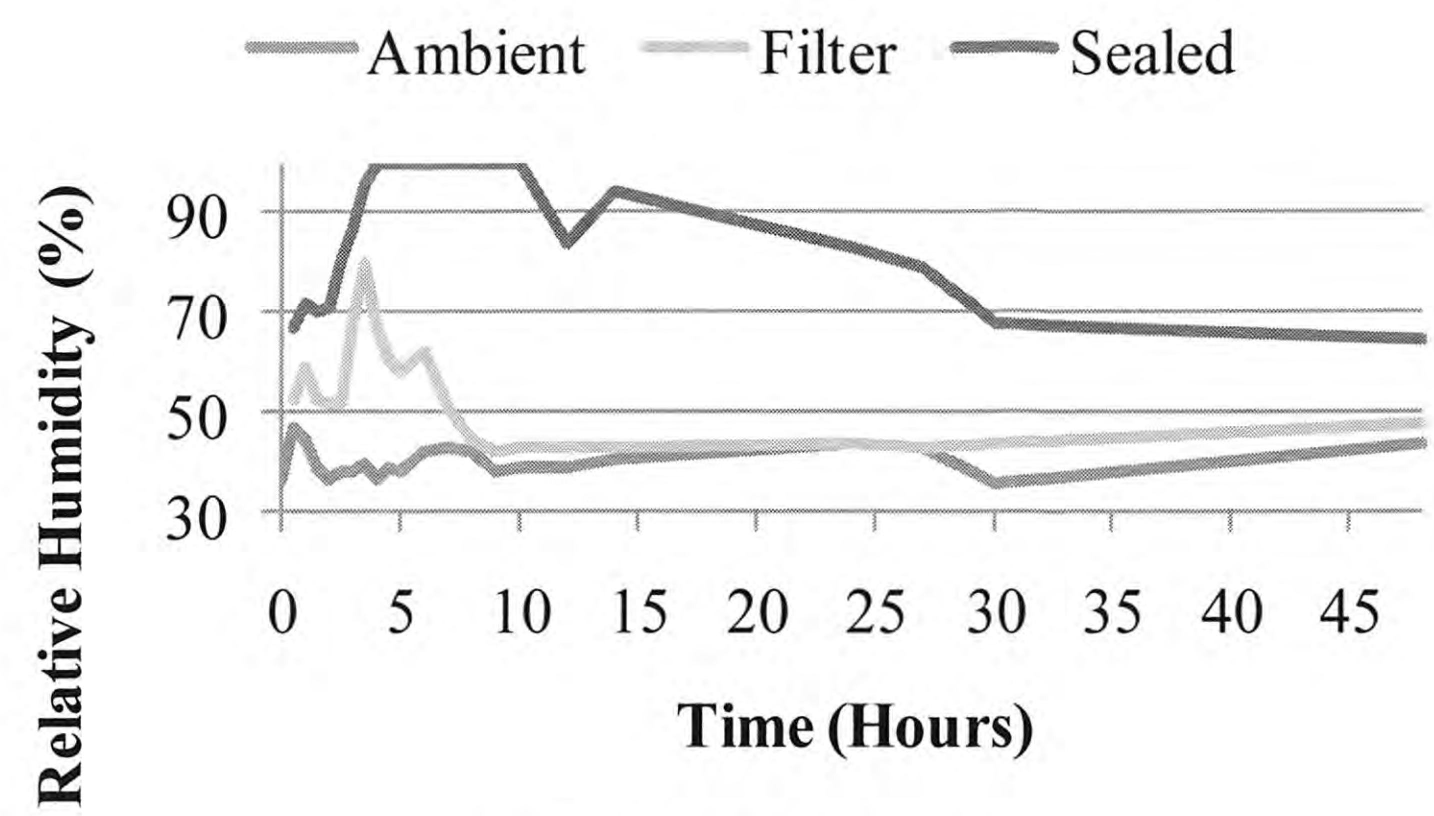
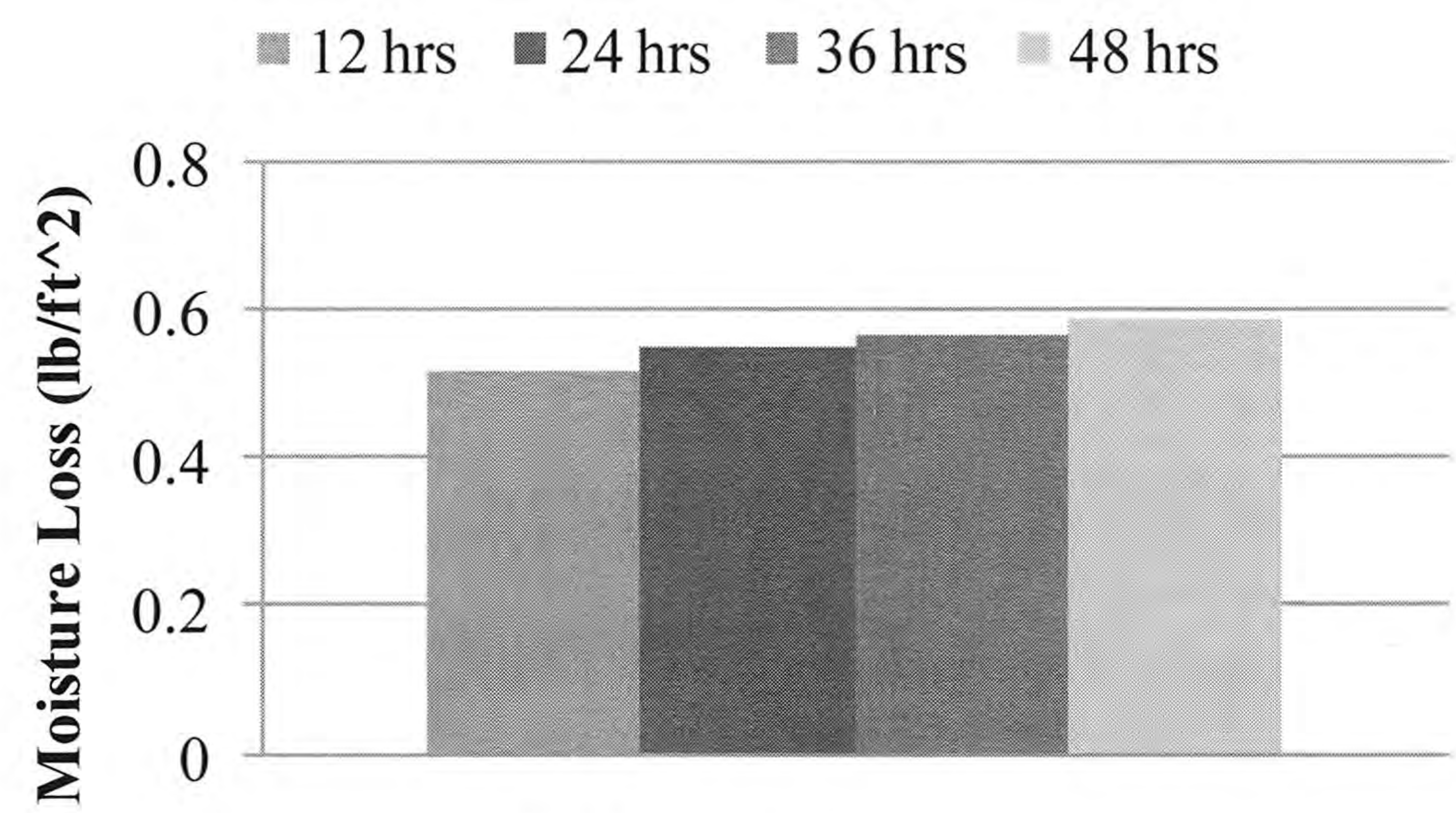


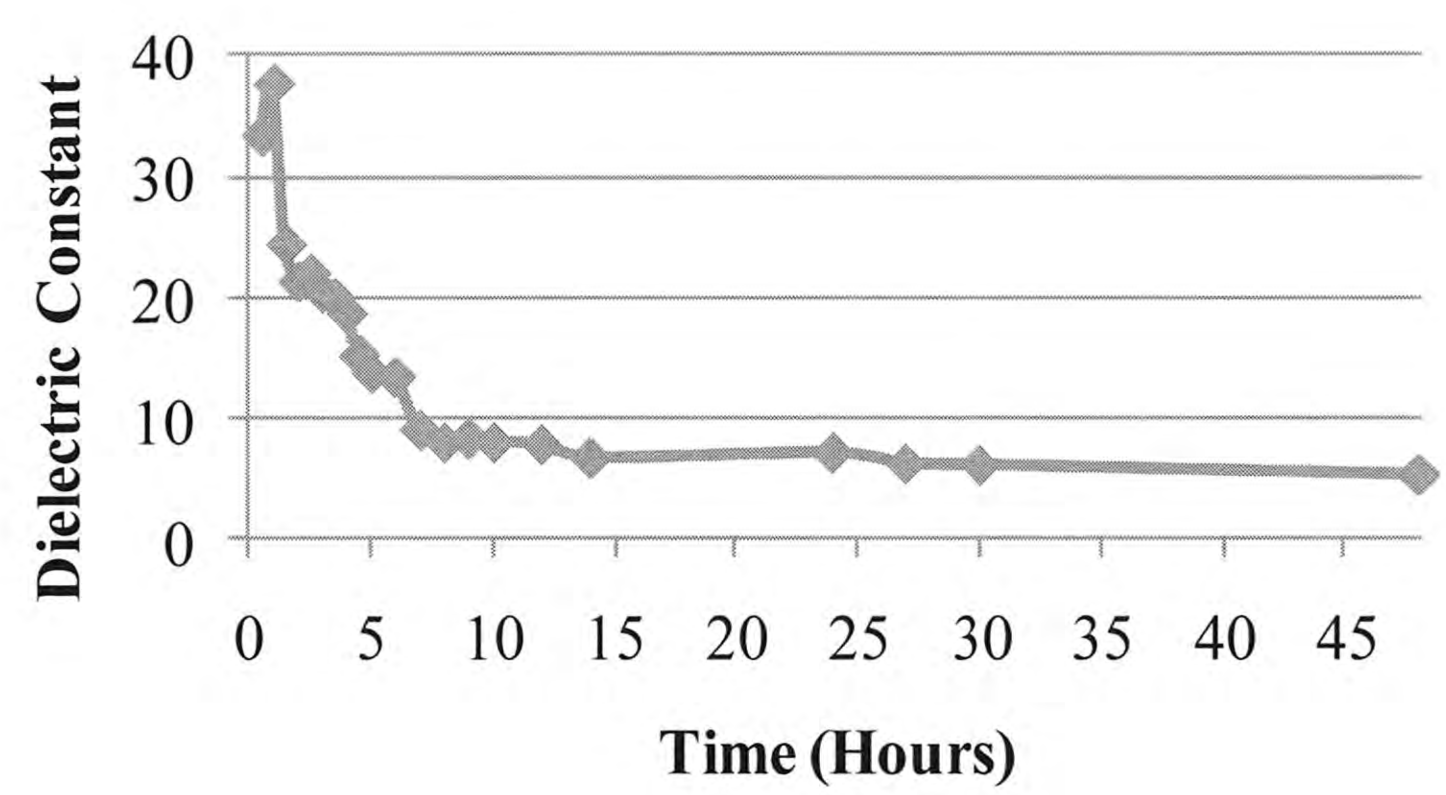
Figure A-2. Sinak Relay (1-C): 1st Trial.



Relative Humidity

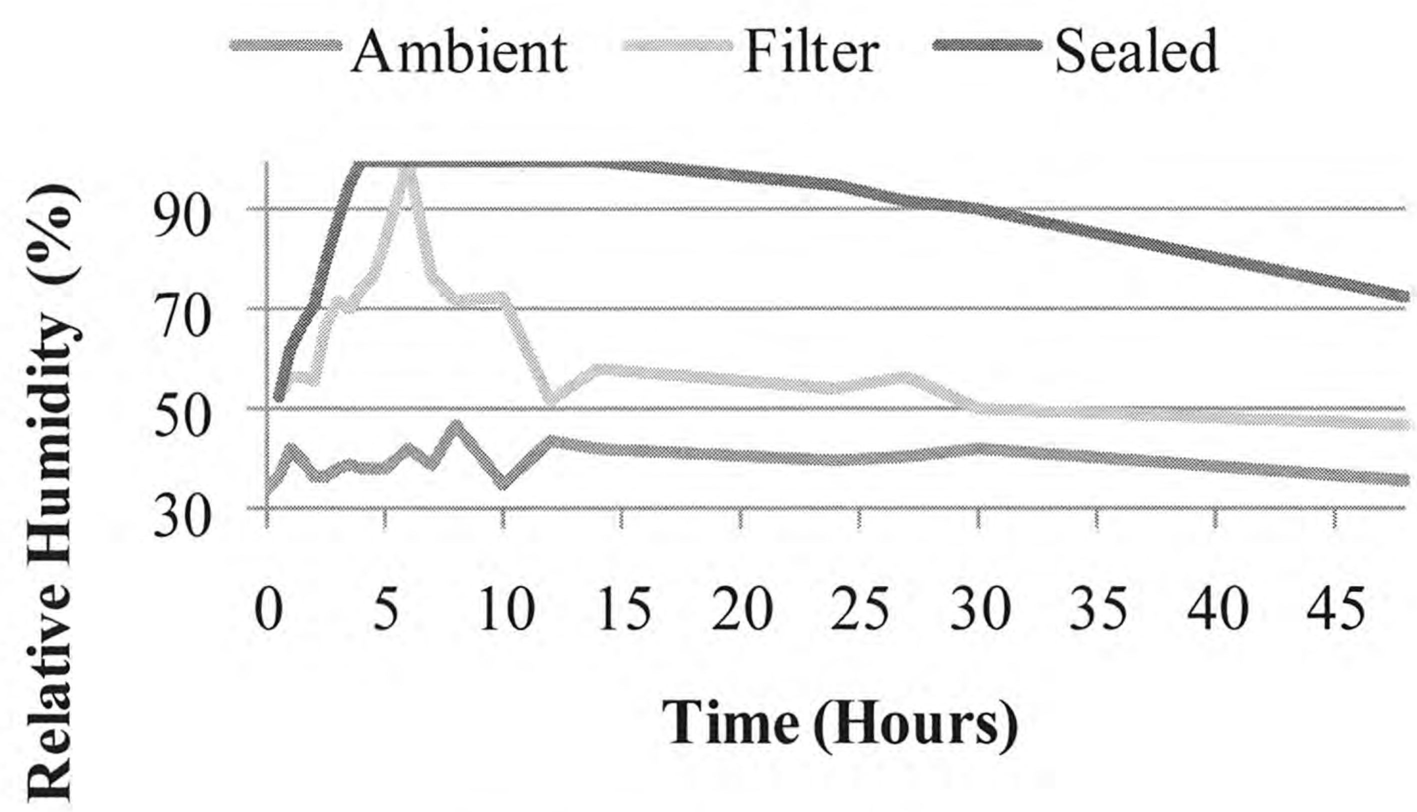


Moisture Loss

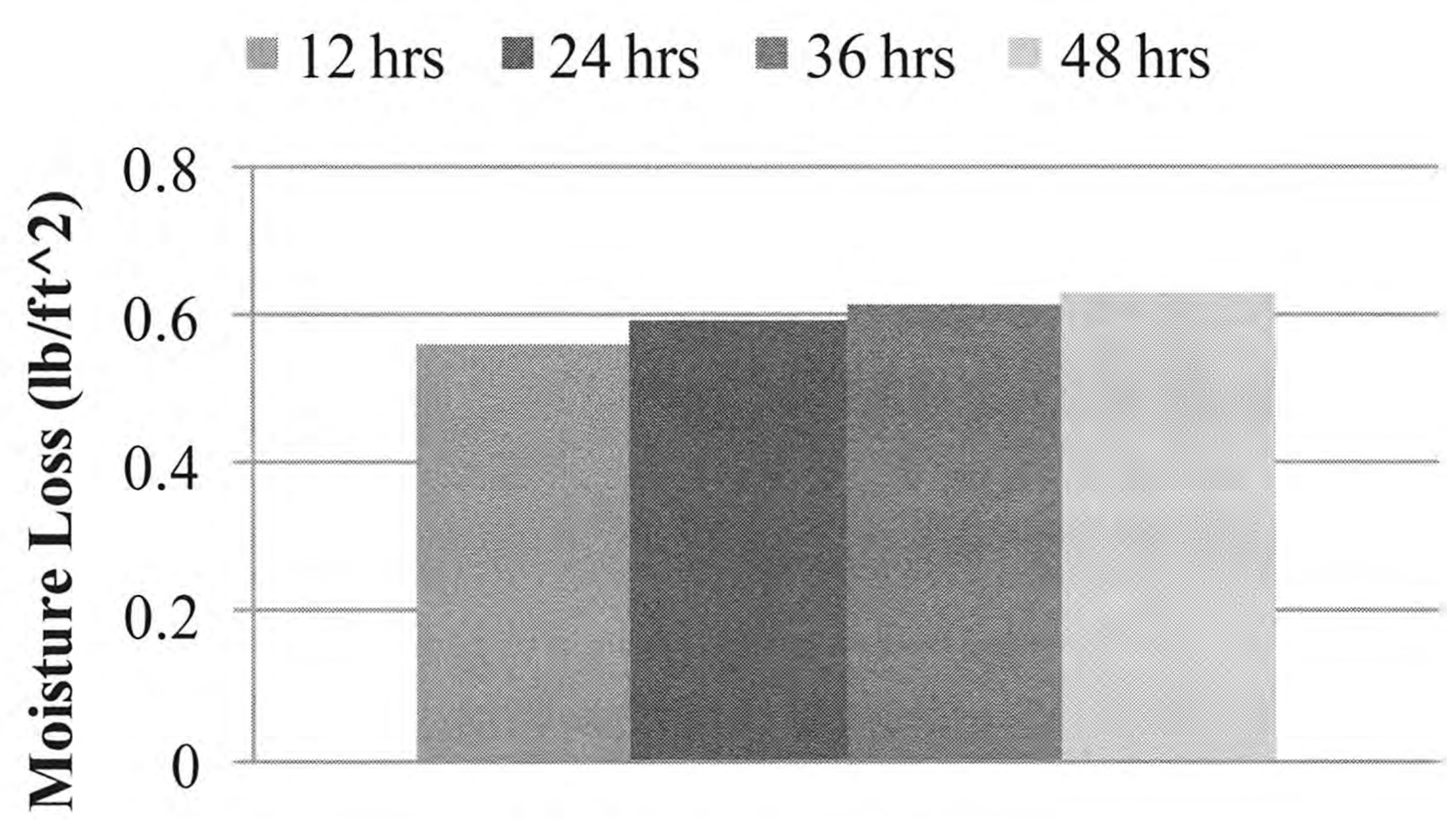


Dielectric Constant

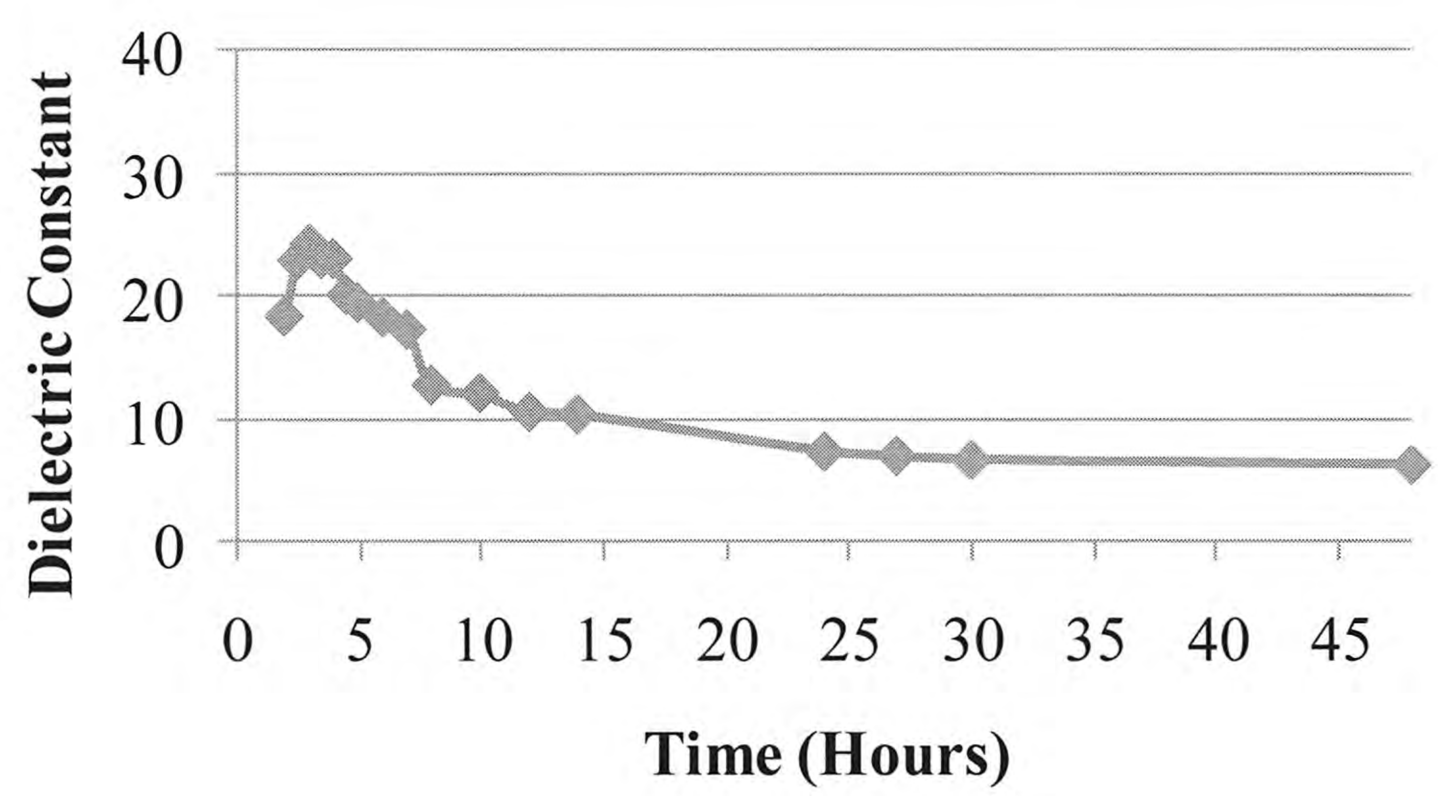
Figure A-3. Lithium (3-B): 1st Trial.



Relative Humidity

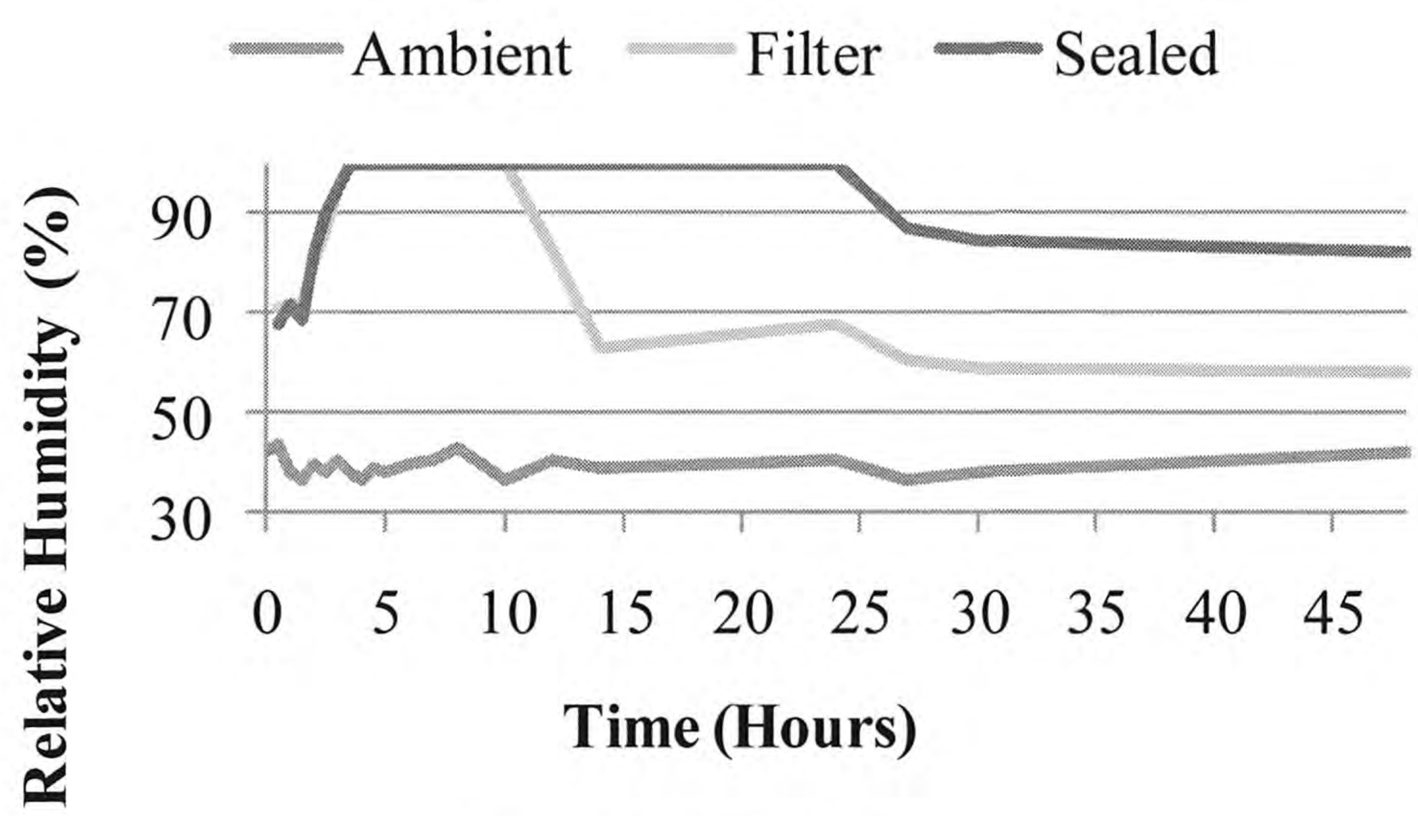


Moisture Loss

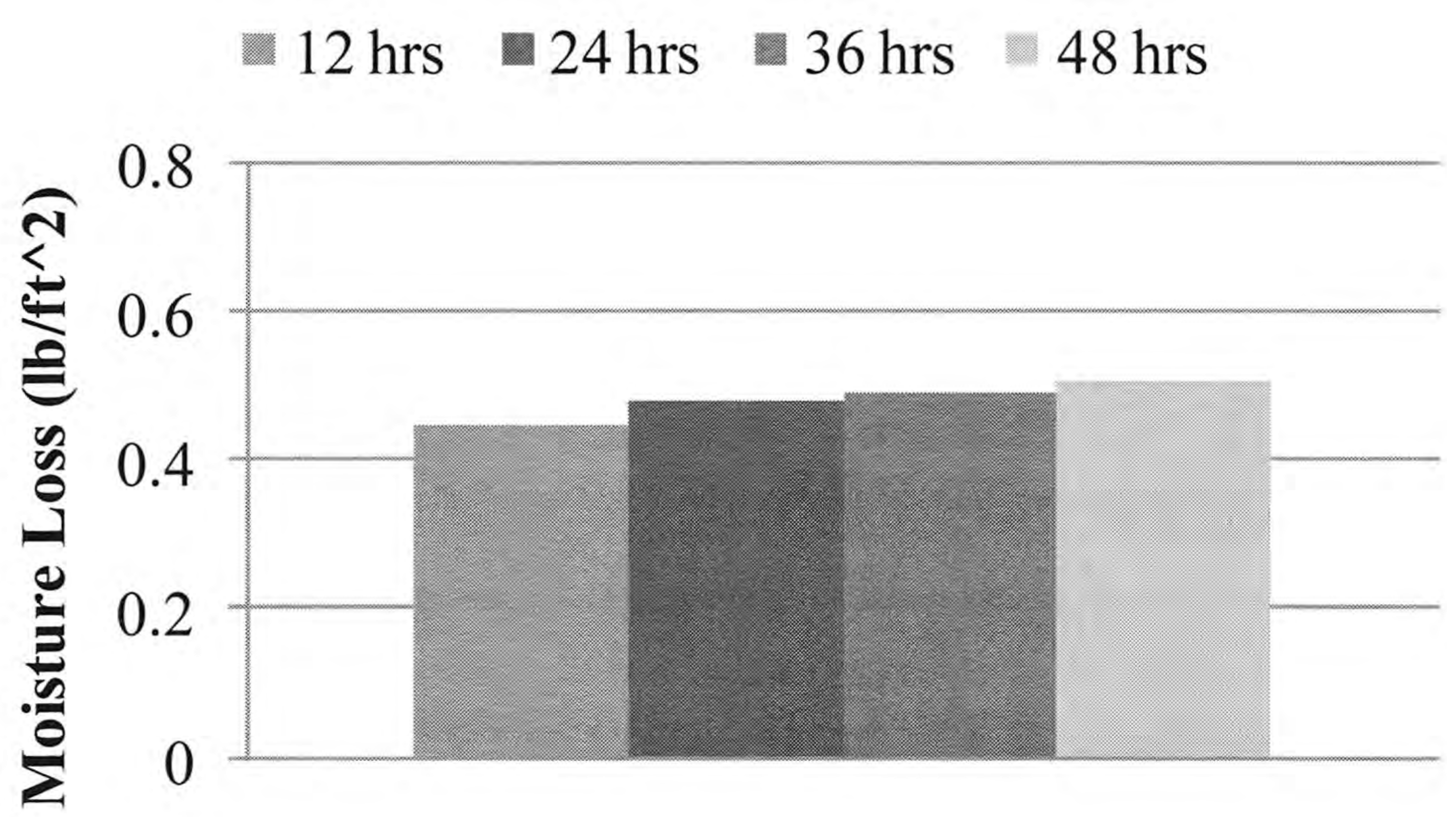


Dielectric Constant

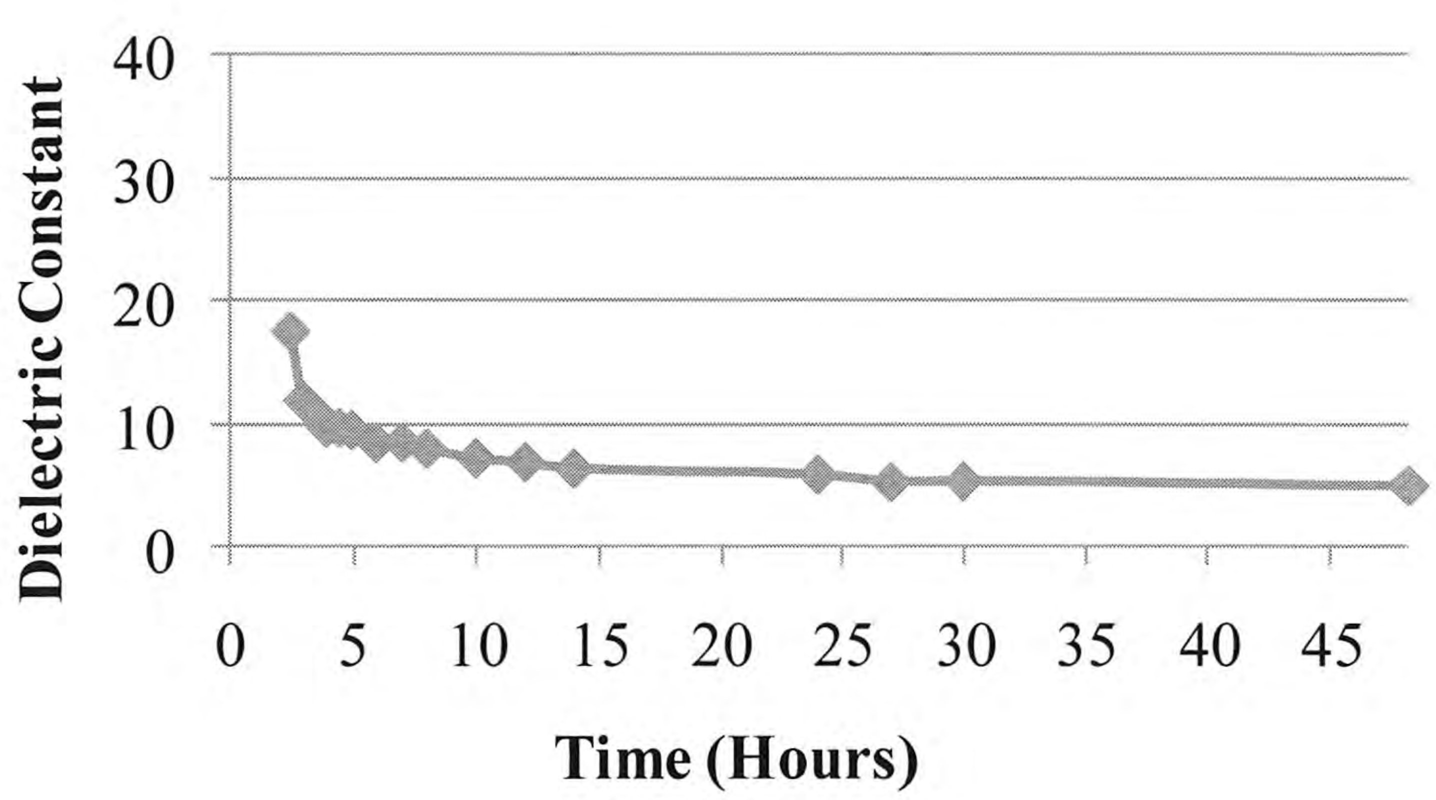
Figure A-4. Eco-Cure (5-B): 1st Trial.



Relative Humidity

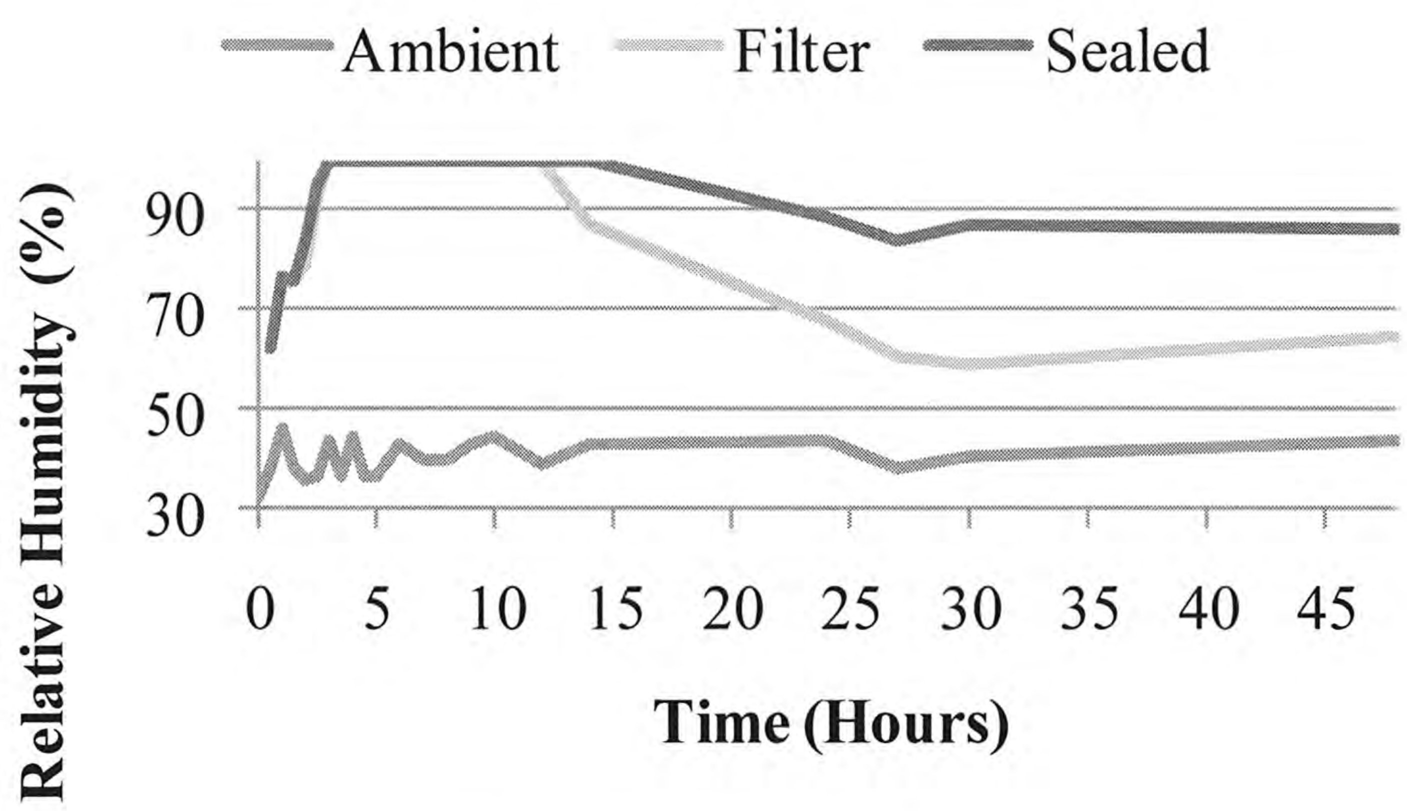


Moisture Loss

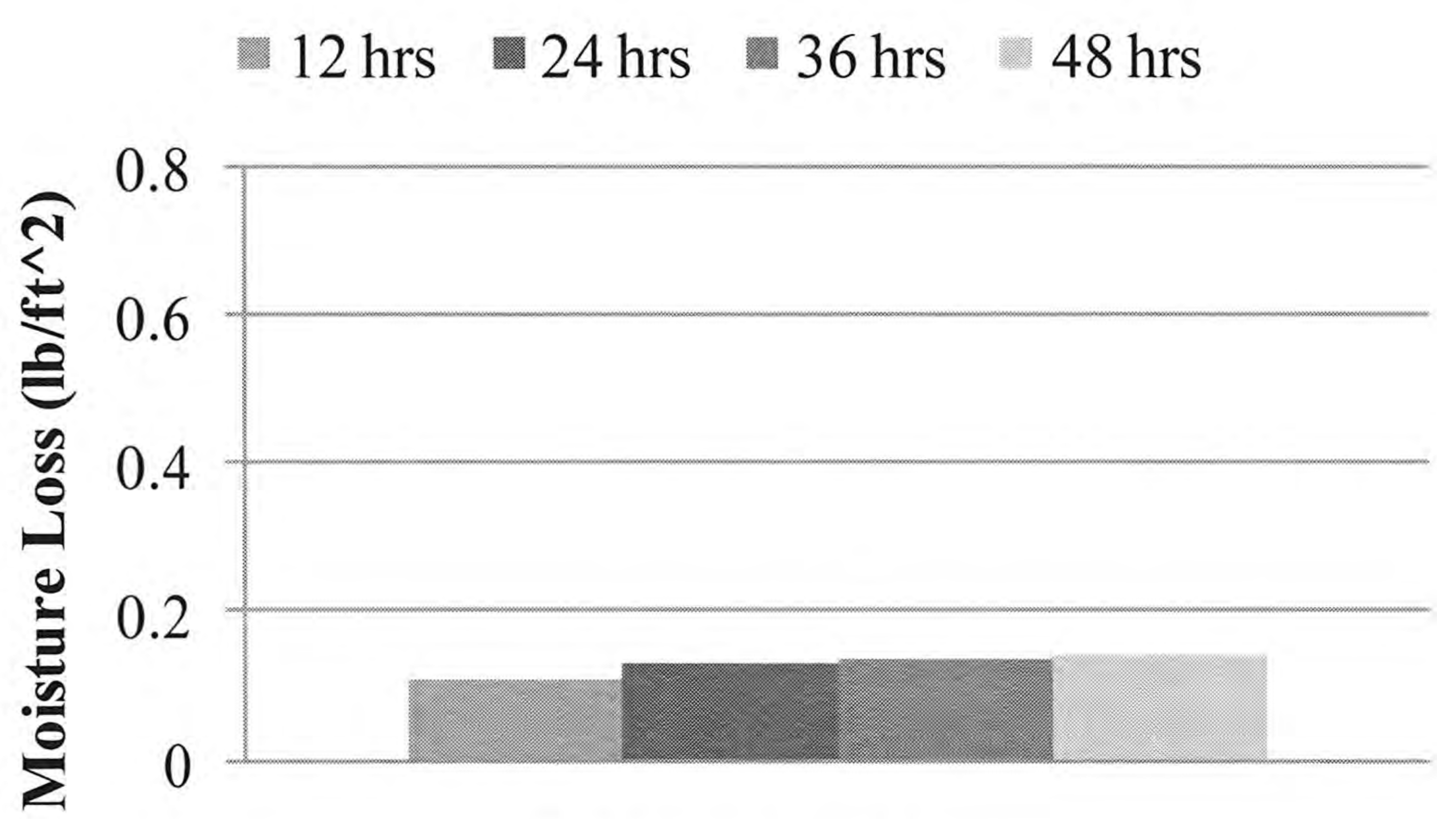


Dielectric Constant

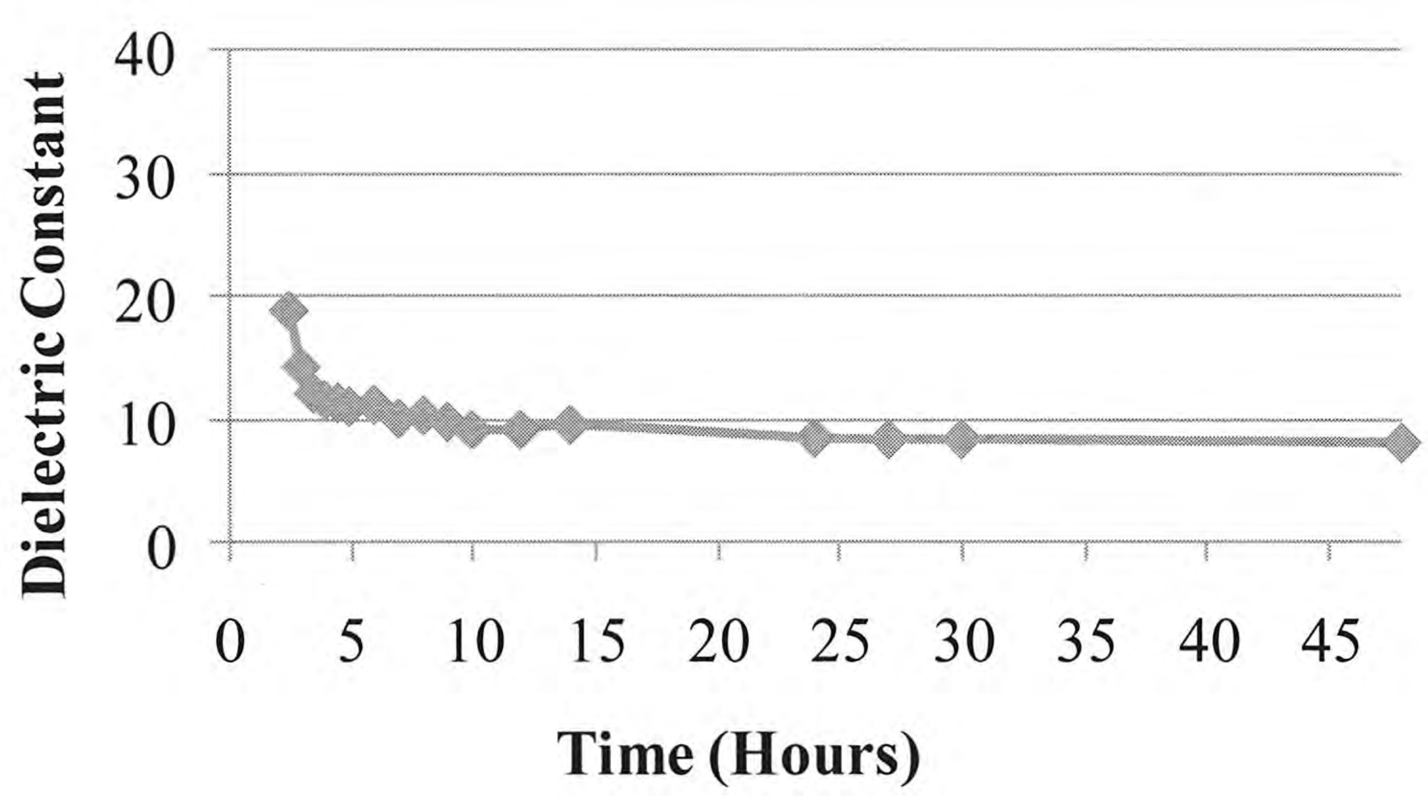
Figure A-5. WRM 1250 (7-A): 1st Trial.



Relative Humidity



Moisture Loss



Dielectric Constant

Figure A-6. WRM 2255 (9-C): 1st Trial.

APPENDIX B: RESULTS OF CUBE TESTS

Table B-1. 1 Day Compressive Strength.

Sample ID	Average (psi)
Test 1	2805
Test 2	2919
Test 3	2473
Test 4	3183
Test 5	3012
Test 6	3188
Test 7	3245
Test 8	3082
Test 9	3463
Test 10	3179

Table B-2. 3 Day Compressive Strength.

Sample ID	Average (psi)
Test 1	3686
Test 2	3615
Test 3	3304
Test 4	3592
Test 5	3509
Test 6	3487
Test 7	4055
Test 8	4272
Test 9	3903
Test 10	3947

Table B-3. 7 Day Compressive Strength.

Sample ID	Average (psi)
Test 1	4277
Test 2	3942
Test 3	3531
Test 4	3688
Test 5	3695
Test 6	3703
Test 7	3939
Test 8	3788
Test 9	3759
Test 10	3961

Table B-4. 28 Day Compressive Strength.

Sample ID	Average (psi)
Test 1	4267
Test 3	4072
Test 5	4253
Test 7	4281
Test 9	4371

Table B-5. Calculated Characteristics of Compressive Strength Cube Samples.

Break Day	Average	Standard Deviation
1	3055	274
3	3737	298
7	3828	209
28	4249	109

APPENDIX C: RESULTS OF LITHIUM TIME TESTS

Table C-1. Strength of Application Time Test Series 1.

Test #	Time Compound Added (Hours)	Strength (psi)
101	0.0	3501
102	0.5	3994
103	1.0	3774
104	1.5	4470
105	2.0	4215
106	2.5	4881
107	3.0	4515
108	3.5	4297
109	4.0	3763
110	4.5	3766
111	5.0	3851

Table C-2. Strength of Application Time Test Series 2.

Test #	Time Compound Added (Hours)	Strength (psi)
2000	0.0	3268
2030	0.5	3468
2060	1.0	4092
2090	1.5	4177
2120	2.0	4159
2150	2.5	4340
2180	3.0	4425
2210	3.5	3780
2240	4.0	3727
2270	4.5	3963