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16. Abstract
This report is a compilation of research papers written by students participating in the 2009 Undergraduate Transportation Scholars Program. The ten-week summer program, now in its nineteenth 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) appropriate time-to-clear values for use in developing left-turn lane warrants; 2) driver behavior at freeway interchanges with horizontal signing; 3) quality assurance in speed data collection methods at high speeds; and 4) measuring traveler's willingness-to-pay for time savings.

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COMPENDIUM OF STUDENT PAPERS:

2009 UNDERGRADUATE

TRANSPORTATION SCHOLARS PROGRAM

Participating Students (seated, left to right): George Bogonko,
Christopher Senesi, Jordan Easterling, and Stephanie Everett
Mentors (standing, left to right): Mark Burris, Marcus Brewer,
Gene Hawkins, Program Director (center), Kay Fitzpatrick, and Brooke Ullman

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October 2009

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 2009, the following five students and their faculty/staff mentors were:

STUDENTS	MENTORS
Mr. Christopher Senesi Ohio Northern University, Ada, OH	Dr. Kay Fitzpatrick Mr. Marcus A. Brewer
Ms. Stephanie Everett Duke University, Durham, NC	Ms. Brooke Ullman Ms. Melisa Finley
Mr. Jordan Easterling Texas A&M University, College Station, TX	Dr. Kay Fitzpatrick
Mr. George Bogonko California State Polytechnic University, Pomona, CA	Dr. Mark Burris

Sincere appreciation is extended to the following individuals:

- Mrs. Colleen Dau, who assisted with program administrative matters and in the preparation of the final compendium; and
- Mrs. Cathy Bryan, who assisted with the preparation of the final manuscript of the compendium.

The authors recognize that support was provided by a grant from the U. S. Department of Transportation, University Transportation Centers Program to the Southwest Region University Transportation Center.

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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.

Appropriate Time-to-Clear Values for Use in Developing Left-Turn Lane Warrants

Prepared for
Undergraduate Transportation Scholars Program

by

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August 6, 2009

STUDENT BIOGRAPHY

Christopher Senesi is a senior at Ohio Northern University in Ada, Ohio, and will be graduating in May 2010 with a Bachelor of Science in Civil Engineering. Christopher is a member of numerous honor and professional societies at his university including Tau Beta Pi, Mortar Board, Omicron Delta Kappa, and American Society of Civil Engineers. In addition, he has work experience with two private civil engineering firms, TranSystems and LandDesign, where he served as a summer intern.

Nationally, Christopher volunteers with STANDUP FOR KIDS, a nonprofit organization helping homeless and at-risk youth. Within the organization, he serves as the National Director of Don't Run Away, a preventative program that deters kids from running away and turning to the streets. On campus, Christopher is heavily involved with many student organizations, specifically relating to the university's leadership development office. Additionally, he serves on the steering committee for his campus's American Cancer Society Relay For Life and is the immediate past student body vice-president. He has been recognized for his outstanding leadership both by Ohio Northern University and STANDUP FOR KIDS. Christopher's hobbies include snowboarding, golfing, and traveling. He plans to attend graduate school where he will pursue a master's degree in civil engineering.

ACKNOWLEDGMENTS

The research described in this paper was conducted as part of Project 3-91, sponsored by the National Cooperative Highway Research Program (NCHRP). 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 NCHRP.

The author would like to express his appreciation to Marcus A. Brewer, Assistant Research Engineer, and Dr. Kay Fitzpatrick, Senior Research Engineer, for their guidance and assistance in this project.

SUMMARY

The purpose of this project was to review the current guidelines for left-turn lane warrants at unsignalized intersections and determine updates for the selection of input variables such as the time-to-clear values. The research was primarily based on M. D. Harmelink's 1967 original research on left-turn lane warrants, a method that is still widely accepted and practiced today.

The current version of AASHTO's *Green Book* utilizes Harmelink's left-turn lane warrants. However, with the warrants still based on Harmelink's original assumptions, the question then arises as to whether an update is necessary. In Harmelink's study, three of his five assumptions relate to time to clear and critical gap, and the main focus of this research was to determine if Harmelink's assumptions for time to clear and critical gap were still valid.
The research looked at 18 intersections with the primary goal of recording left-turning vehicles that had to make a gap decision. Sites were selected based on a multitude of characteristics including number of lanes, presence of left-turn lane, and approach speed, which allowed for a more holistic approach. From here, the two time-to-clear values (time to clear the advancing lane and time to clear the opposing lanes) and critical gap were calculated for each site. Average and 85th percentile time-to-clear values were then calculated for the entire data set as well as for the given characteristics (i.e., presence of left-turn lane). This allowed for comparisons between intersection characteristics, which can be seen in this report. Finally, the values found in this research were applied to Harmelink's calculations to determine left-turn lane warrants.

Based on the preliminary findings in this study, it was found that the average time-to-clear values from the study sites were lower than Harmelink's original assumptions. The differences between the values were minor and did not drastically affect the left-turn lane warrants. The research did find that the 85th percentile time-to-clear values closely matched Harmelink's assumptions. The research also found that there was not a major difference between the critical gap values from the study sites and Harmelink's original critical gap assumption.

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INTRODUCTION

Left-turn movements at intersections, especially unsignalized intersections, can negatively influence traffic flow and drivers' safety. One such action to address this issue is the addition of a left-turn lane. A recent study conducted for the Federal Highway Administration in 2002, found that, with the addition of a left-turn lane, accident rates decreased by 10 percent at four-leg signalized intersections and decreased by 27 percent at four-leg unsignalized intersections *(1).* A great deal of research has been conducted on left turns at signalized intersections, but unsignalized intersections have not received as much attention. Therefore, as illustrated in the accident reduction data above, it is just as important to study left-turn movement at unsignalized intersections as it is to study left-turn movement at signalized intersections.

The question then arises as to when it is appropriate to add a left-turn lane at an unsignalized intersection. As the population continues to expand and driver behavior continues to change, the warrants used for left-turn lanes can become outdated; therefore, it is ever more crucial to continue to update these warrants for today's drivers, allowing for increased driver safety and more efficient traffic flow.

Numerous methods exist that can be used to help determine when a left-turn lane should be added and are based on a variety of factors including conflict avoidance, decreased delay, and safety. One such method that is widely accepted is Harmelink's procedure, which uses the conflict avoidance factor. However, in recent years, the assumptions used for Harmelink's procedure have been studied and current research suggests that some of these assumptions may need to be revised.

This research will look at Harmelink's current assumptions, specifically time to clear and critical gap, and provide recommendations for updated left-turn lane warrants based on the new assumptions. By studying traffic flow at a multitude of intersections, the data will include varied geometric characteristics and different driver approach scenarios that will allow for a complete and comprehensive analysis.

BACKGROUND

Left-turn lanes can play a crucial role in the safety of drivers and traffic flow at both three- and four-leg intersections. Functions of left-turn lanes include *(2)*:

- Reducing the number of conflicts and crashes,
- Separating through, turning, and/or queuing traffic,
- Decreasing delay and increasing capacity,
- Providing more operational flexibility, and
- Providing an area for left-turning vehicles to decelerate outside of the through traffic lane.

The addition of a left-turn lane can greatly reduce the problems at an intersection; however, a left-turn lane may not be necessary at a particular intersection. Therefore, left-turn lane warrants are used to aid in deciding when the addition of a left-turn lane is justified. The parent research project, entitled, "Left-Turn Accommodations at Unsignalized Intersections," is looking at factors that might affect the need for a left-turn lane. These factors include *(2)*:

- Type/function of roadway,
- Number of lanes,
- Prevailing speeds,
- Traffic control/operations,
- Turn and other volumes,
- Roadway(s) alignment, and
- Safety (conflict, crash numbers, and crash types/causes).

The parent research project is examining these different factors in an effort to develop updated left-turn accommodations for unsignalized intersections and to provide guidance on the design of these accommodations. The procedures and processes that will be evaluated include *(2)*:

1. Benefit/Cost ratio,
2. Updated Harmelink procedure, and
3. Values selected based on engineering judgment.

Harmelink Procedure

In 1967, M.D. Harmelink published a paper on his findings related to evaluating the need for left-turn lanes at unsignalized intersections. Harmelink created guidelines that could be used to determine whether a left-turn lane would be necessary and were based on the following assumptions *(3)*:

1. Probability of a through vehicle arriving behind a stopped left-turning vehicle should not exceed 0.02 for 40 mph (64 km/h), 0.015 for 50 mph (80 km/h), and 0.010 for 60 mph (96 km/h);
2. Arrival-time and service-time distributions are negative exponential;
3. Average time required for making a left turn is 3.0 sec for two-lane highways and 4.0 sec for four-lane highways as determined from field studies;
4. Critical gap in the opposing traffic stream for a left-turn maneuver is 5.0 sec on two-lane highways and 6.0 sec on four-lane highway as determined from field studies; and
5. Average time required for a left-turning vehicle to clear the advancing lane is 1.9 sec determined from field studies.

From here, Harmelink created left-turn lane warrants that are still included in the AASHTO *Green Book (4)* and are practiced by many states today. To use these guidelines, one must know the following characteristics of the given intersection: opposing volume, advancing volume, percentage of left-turning vehicles, and advancing speed. Using the table provided in the AASHTO *Green Book*, it can be determined if a left-turn lane is warranted. Table 1 shows a sample of the *Green Book* table, which is based on the original Harmelink assumptions.

In a recent study conducted by the Texas Transportation Institute, a variety of left-turn lane guidelines were reviewed, specifically relating to Harmelink's procedure. Based on this review, the research proposed developing a set of updated guidelines and found that Harmelink's assumptions, specifically Assumptions 3 and 5 (time to clear) and Assumption 4 (critical gap), should be modified. The proposed guidelines used the updated assumptions *(3)*.

Table 1. Guide for Left-Turn Lanes on Two-Lane Highways, 2004 *(4)*.

Opposing Volume (vph)	Advancing Volume (vph)			
	5% Left Turns	10% Left Turns	20% Left Turns	30% Left Turns
40 mph (60 km/h) operating speed				
800	330	240	180	160
600	410	305	225	200
400	510	380	275	245
200	640	470	350	305
100	720	515	390	340
50 mph (80 km/h) operating speed				
800	280	210	165	135
600	350	260	195	170
400	430	320	240	210
200	550	400	300	270
100	615	445	335	295
60 mph (100 km/h) operating speed				
800	230	170	125	115
600	290	210	160	140
400	365	270	200	175
200	450	330	250	215
100	505	370	275	240

Time-to-Clear Values

To update the Harmelink procedure, time-to-clear data from a number of unsignalized intersections with left-turn movement will be studied. Figure 1 illustrates the time to clear at a typical three-leg intersection where the major road has two lanes and no left-turn lane. Time to clear is divided into two components: the time necessary to clear the advancing lane (position of Left-Turn Vehicle in Figure 1A to position of Left-Turn Vehicle in 1B) and the time to clear the opposing lane(s) (position of Left-Turn Vehicle in Figure 1A to position of Left-Turn Vehicle in 1C). The time to clear values may be influenced by number of lanes, prevailing speeds, and opposing volumes.

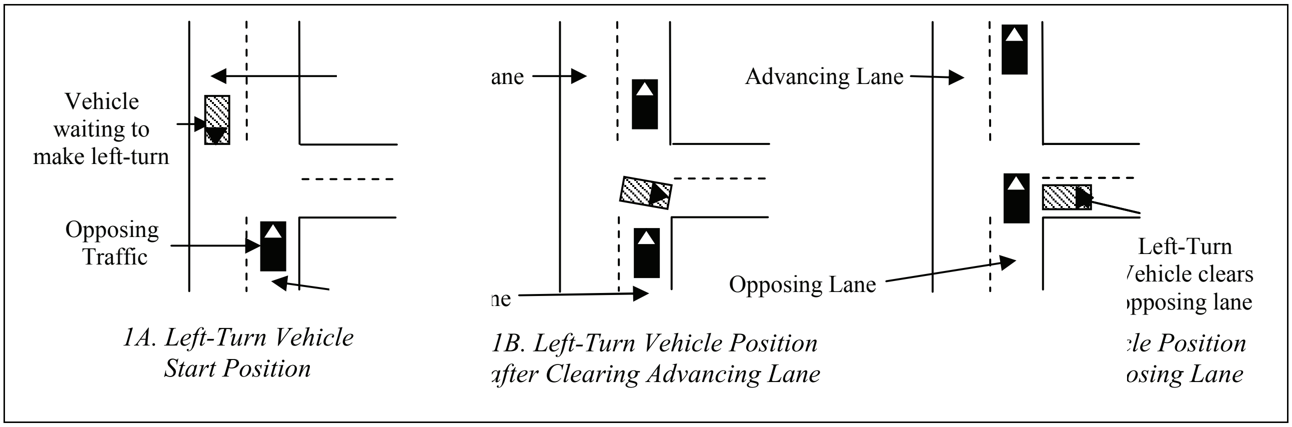

Figure 1. Illustration of Time-to-Clear Values.

Critical Gap

A gap is defined as the time interval between two vehicles passing a given point within an intersection. In relation to a left-turning vehicle, the gap must be long enough for the driver of the vehicle to safely complete a left-turn maneuver. This is the critical gap, defined in a study by Morton S. Raff as the time interval for which the number of shorter accepted gaps is equal to the number of longer rejected gaps *(5)*. To use Raff's Method, gap intervals are assigned for the given data, and the number of accepted and rejected gaps are recorded for each interval. The cumulative percent of accepted gaps and the cumulative percent of rejected gaps are then plotted against the gap intervals, illustrated below in Figure 2. The intersection of the two plotted lines is the critical gap value.

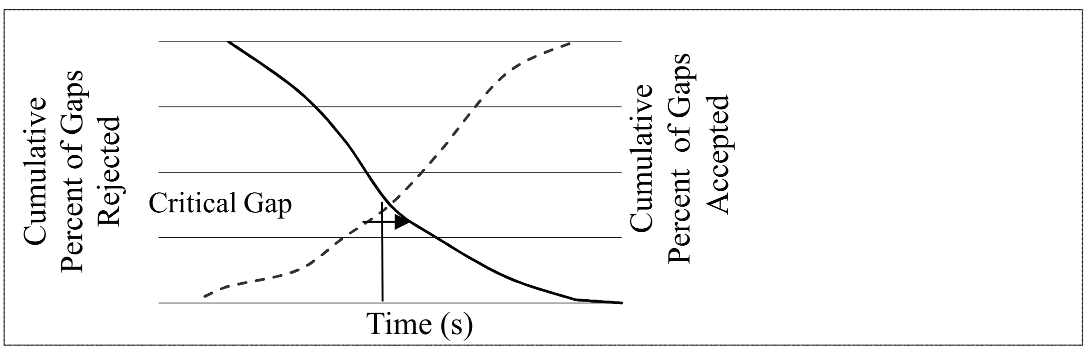

Figure 2. Cumulative Distributions of Accepted and Rejected Gaps in Left-Turn Maneuvers.

Additionally, according to Miller, critical gap is also defined as the median gap, where the percentage of gap acceptance is at or above 50 percent *(6)*. Therefore, another way to determine the critical gap is to determine the percentage of acceptance for each gap interval and then determine when the percentage of acceptance is greater than 50 percent. This will result in only the critical gap interval and not the critical gap value. However, the critical gap interval should correspond to the critical gap value found using Raff's Method.

GOALS AND OBJECTIVES

The overall goal for this project is to develop updated left-turn lane warrants for intersection design. The AASHTO *Green Book* contains left-turn warrants, which are based on Harmelink's original research from 1967, and this research project will assist in recommending new and relevant design warrants for left-turn lanes. The following objectives will need to be considered in accomplishing the overall goal:

1. Determine time-to-clear values, critical gap, and traffic volumes.
2. Use the time to clear values and critical gap to update Harmelink's procedure and calculate new left-turn lane warrant values.

Additionally, the results of this research proposal will assist in the conclusions of the parent research project, "Left-Turn Accommodations at Unsignalized Intersections."

DATA COLLECTION

To conduct this research, left-turn movement was studied at 18 sites that were located in the College Station/Bryan and Houston, Texas, and Phoenix, Arizona, metropolitan areas. The sites were selected based on a variety of intersection arrangements and geometric characteristics, including:

- Number of lanes on major: 2 or 4 lanes;
- Presence of left-turn lane: yes or no;
- Signal coordination: system, random; and
- Approach speed range: 25 to 65 mph.

Table 2 lists the 18 sites used in this study and their corresponding geometric characteristics.

Table 2. Site Characteristics.

	Site Name & Location	Duration of Study Period *(minutes)*	Number of Lanes on Major		Left-Turn Lane		Signal Coordination		Approach Speed *(Posted Speed)*
			4 Lanes	*2 Lanes*	*With*	*Without*	*System*	*Random*	
Texas Sites	TX-1 Wellborn @ Graham	60		X	X			X	45
Texas Sites	TX-2 Univ. @ Copperfield	60		X		X		X	65
Texas Sites	TX-3 Spring Cypress @ Wunsche Loop	60	X			X	X		30
Texas Sites	TX-4 Aldine Westfield @ Lexington Woods	240		X		X		X	35
Texas Sites	TX-5 Cypresswood @ Quail Gate	60	X		X			X	45
Texas Sites	TX-6 Wellborn @ F&B	60	X			X	X		45
Texas Sites	TX-7 University at Veterans Parkway	370		X		X		X	60
Texas Sites	TX-8 Shadow Creek Pkwy at Reflection Dr.	60		X	X			X	40
Texas Sites	TX-9 Fry Rd @ Cannon Fire Dr.-Stockton Falls Dr.	240	X		X			X	45
Texas Sites	TX-10 Broadway @ Garden Rd.	65	X		X		X		40
Texas Sites	TX-11 Boonville at Mohawk	120	X		X		X		55
Arizona Sites	AZ-1 32nd @ Colter	130	X		X		X		40
Arizona Sites	AZ-2 Tatum Blvd @ Pinnacle Vista Dr.	150	X		X			X	45
Arizona Sites	AZ-3 Central Ave. @ Butler Dr.	210	X			X		X	35
Arizona Sites	AZ-4 Stanford Dr. @ PHX County Day School	60		X	X			X	25
Arizona Sites	AZ-6 Campbell Ave. @ Apartment Driveway	195		X		X	X		30
Arizona Sites	AZ-7 Camelback Rd. @ Scottsdale Cullinary Institute	355		X		X	X		35
Arizona Sites	AZ-10 Oak St. @ Costco Driveway	60		X		X	X		25

Equipment

At the Texas sites, the data were collected through the use of one or more video cameras mounted from a data collection trailer, approximately 30 feet high. Figure 3 shows the equipment used at the Texas sites. Equipment at the Phoenix sites included tripods and camcorders.

Figure 3A: Data Recorder

Figure 3B: Video Trailer and Camera

Figure 3. Data Equipment Used for Data Collection.

The video recorded the movement at the intersection for at least four hours, observing the advancing/opposing traffic and left-turn movement. A time stamp was imprinted on the video so that the precise times of each turning movement could be reduced from the video. In addition to the video, site-specific data about the intersection were collected, including the geometric characteristics and measurements as well as detailed photographs of the intersection.

DATA REDUCTION

Following the data collection, each site’s data were reduced to obtain the necessary information for the analysis. The reduction process involved reviewing the site video and obtaining the following information, based on five-minute intervals:

- Number of opposing vehicles,
- Number of advancing vehicles, and
- Number of vehicles making left-turns.

The goal for each site was to obtain data for 100 left-turning vehicles that had to make a decision based on the available gaps in the opposing traffic. In most cases, a one-hour time interval provided the desired sample size. However, some sites did not have 100 left-turning vehicles within the one hour timeframe; therefore, additional hours were reduced for some sites. Once the appropriate timeframe for each site was selected, actions of interest for each left-turning vehicle and opposing vehicle were recorded.

In addition to the time of arrival of the opposing vehicles, the following time for each left-turning vehicle was recorded:

- Time at back of queue,
- Time at front of queue,
- Time at start of left-turn maneuver,
- Time to clear approaching lane,
- Time to clear median and/or median lane (where applicable),
- Time to clear opposing lane 1, and
- Time to clear opposing lane 2 (where applicable).

DATA ANALYSIS

To apply Harmelink's procedure and determine left-turn lane warrants, the following variables from each site had to be determined:

- Advancing volume (veh/h) – all vehicles entering the intersection in the same direction as the left-turning vehicle,
- Opposing volume (veh/h) – all vehicles entering the intersection in the opposite direction as the left-turning vehicle,
- Left-turn volume (veh/h) – all vehicles entering the intersection making a left turn,
- Approach speed (85^{th} percentile),
- Time required for a left-turning vehicle to clear the advancing stream,
- Time taken to complete a left-turn maneuver, and
- Critical gap.

The volumes were obtained from each site's traffic count and the approach or 85^{th} percentile speed was assumed to be equal to the posted speed limit. Additional calculations were required to determine the values for time to clear and critical gap.

Time-to-Clear Values

Harmelink's assumptions for time to clear are based on the average value; therefore, for this study, the average time-to-clear values were calculated for each site as well as the entire data set. In addition to calculating the average value, the 85^{th} percentile value was also calculated, which will allow for the left-turn lane warrants to be designed for 85 percent of drivers, as opposed to only 50 percent. This is illustrated in Figure 4.

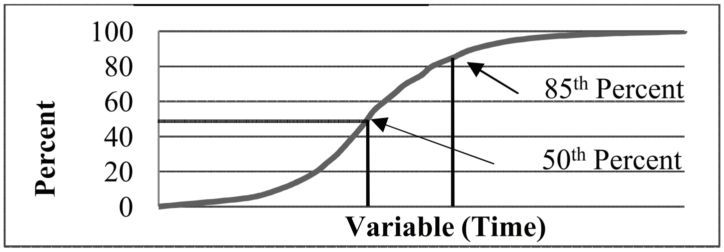

Figure 4. Comparison between Average and 85th Percentile.

Figure 4 displays a typical percentile graph, showing the 85th percentile in relation to the average value and 50th percentile. It is also important to note that the average is different from the 50th percentile, or the median; however, in most cases, the average and median are relatively close to one another. The 50th percentile is only shown for a comparison to the 85th percentile as Harmelink's assumptions are based on the average. The standard deviation was also calculated on a per-site basis to check the variability of the data in relation to the average. In most cases, the standard deviation varied from the mean by 0.5 to 0.75 sec.

In addition to the overall average, per-site values that would be needed for calculating the left-turn lane warrants had to be determined. These values are listed in Table 3.

Table 3. Per-Site Data for Left-Turn Lane Warrant Calculations.

Site		Time	Total Left-Turn (veh)	Avg. Time for vehicle to clear itself from advancing stream (s)	Avg. Time taken to complete a left-turn maneuver (s)	Advancing Volume (veh/h)	Opposing Volume (veh/h)	Left-Turn Volume (veh/h)	Percentage of left-turn volume in advancing volume (%)	Critical Gap (s)
Four-Lane	TX-3	2:09pm - 3:09pm	114	0.80	2.28	764	855	114	14.92%	5.5
	TX-5	3:55pm - 4:55pm	122	1.26	2.58	531	416	122	22.98%	7.0
	TX-6	4:24pm - 5:24pm	100	0.78	2.43	618	470	100	16.18%	6.0
	TX-9	1:05pm - 5:05pm	72	1.87	3.28	903	1017	18	1.99%	7.0
	TX-10	4:55pm - 6:00pm	104	1.34	2.72	1300	1312	96	7.39%	5.0
	TX-11	3:50pm - 5:50pm	98	1.95	3.73	637	601	49	7.69%	5.5
	AZ-1	3:45pm - 5:55pm	102	1.00	2.37	1240	602	47	3.80%	5.0
	AZ-2	3:44pm - 6:14pm	100	0.68	1.95	1294	660	40	3.09%	3.0
	AZ-3	2:45pm - 6:15pm	71	0.89	2.45	657	498	20	3.09%	4.5
Two-Lane	TX-1	4:30pm - 5:30pm	218	0.99	2.01	843	614	218	25.86%	5.0
	TX-2	4:55pm - 5:55pm	340	0.98	2.13	664	160	340	51.20%	5.5
	TX-4	10:09am - 2:09pm	95	0.98	2.40	291	326	24	8.17%	4.0
	TX-7	9:45am - 3:55pm	33	1.15	2.17	297	356	5	1.80%	7.0
	TX-8	4:20pm - 5:20pm	176	1.14	2.02	818	546	176	21.52%	5.0
	AZ-4	7:00am - 8:00am	156	1.34	3.09	308	198	156	50.65%	6.5
	AZ-6	3:06pm - 6:21pm	25	0.98	2.83	176	210	8	4.37%	5.5
	AZ-7	6:58am - 12:53pm	88	0.73	2.79	164	174	15	9.08%	6.0
	AZ-10	3:00pm - 4:00pm	234	1.61	3.80	310	109	234	75.48%	4.0

Time-to-Clear Advancing Stream

The first time-to-clear value that was studied was the time to clear the advancing stream. First, a grand average and an overall 85th percentile were found; this was done by arranging all sites in numerical order, based on the time to clear value. As seen in Figure 5, a cumulative distribution was plotted and the values of average, 50th and 85th percentile, were identified on the plot.

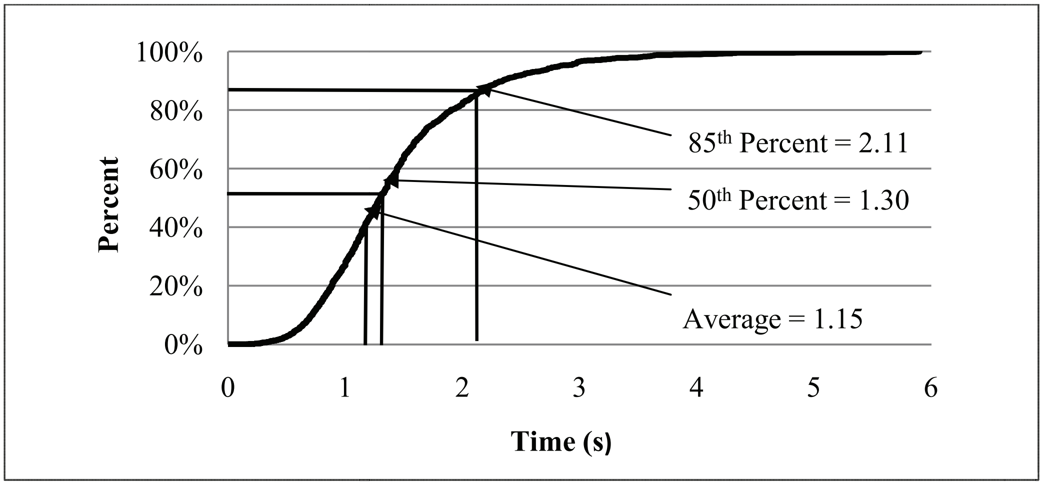

Figure 5. Cumulative Percent Distribution of Time to Clear Advancing Lane.

As mentioned previously, sites were selected based on a number of different characteristics. To help get a better idea of the time-to-clear values found, basic comparisons between site characteristics were reviewed, including the presence of a left-turn lane and the approach speed. Table 4 lists the average values and 85th percentile values for all sites based on presence of a left-turn lane and based on approach speed.

Table 4. Time to Clear Advancing Lane Comparison.

Presence of Left-Turn Lane		
	Left-Turn Lane	No Left-Turn Lane
Average	1.23 sec	1.06 sec
85th Percentile	2.16 sec	2.03 sec
Approach Speed		
	High Approach Speed	Low Approach Speed
Average	1.12 sec	1.17 sec
85th Percentile	2.07 sec	2.12 sec

First, looking at values for the presence of a left-turn lane, both the average and 85th percentile values are higher when a left-turn lane is present than when it is not present; this can be expected, as drivers who are making a left turn from a left-turn lane may be less pressured by other advancing cars, and therefore may take more time in clearing the advancing stream as opposed to drivers making a left turn where no left-turn lane is present. Next, looking at the approach speed, based on the data from this study, it was found that a high approach speed corresponds to a lower time-to-clear value and a low approach speed corresponds to a higher time-to-clear value. Drivers who are making a left turn where the approach speed is higher may feel more pressure to clear that advancing lane faster than if the driver was on a low-speed road.

Time-to-Clear Opposing Lanes

The next time-to-clear value that was studied was the time to clear the opposing lane(s). Again, a grand average and an overall 50th and 85th percentile were found and plotted and can be seen in Figure 6.

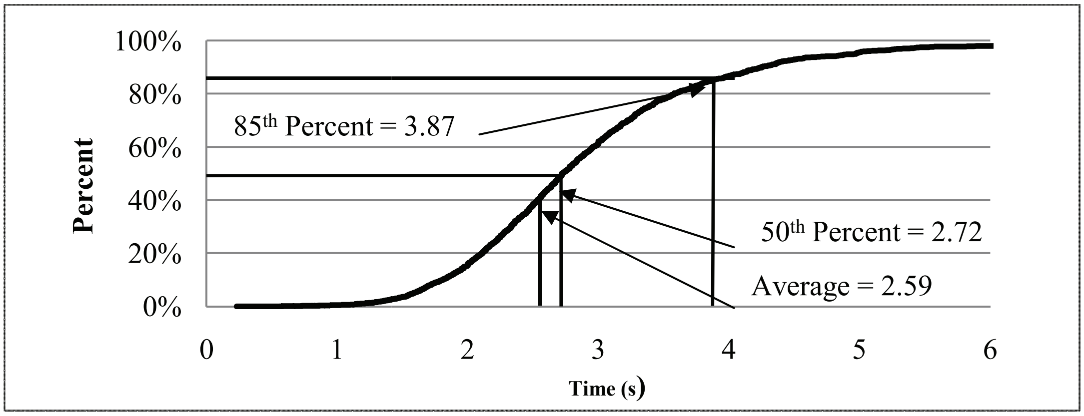

Figure 6. Cumulative Percent Distribution of Time to Clear Opposing Lanes.

Similar to the time to clear the advancing stream, the average and 85th percentile were compared based on the sites' geometric characteristics. Table 5 lists these values.

Table 5. Time to Clear Opposing Lanes Comparison.

Presence of Left-Turn Lane		
	Left-Turn Lane	No Left-Turn Lane
Average	2.53 sec	2.64 sec
85th Percentile	3.74 sec	4.03 sec
Approach Speed		
	High Approach Speed	Low Approach Speed
Average	2.39 sec	2.64 sec
85th Percentile	3.50 sec	4.11 sec

The characteristic that was first compared was the presence of the left-turn lane. In this case, the values were less where a left-turn lane was present. One reason for this result could be because left-turn lanes are usually located on high-volume roadways; therefore, left-turning vehicles may be more apt to complete the left-turn maneuver faster than on a low-volume road, which usually does not have a left-turn lane. Next, the values for a high approach speed were found to be lower than for intersections with low approach speeds. This could be attributed to the fact that drivers making a left-turn lane on a high approach speed road will need to clear the opposing vehicles faster due to the higher speeds.

Critical Gap

The final value determined before using Harmelink's procedure was critical gap. As defined in the background, critical gap is the time interval between two opposing vehicles that is necessary for a left-turning vehicle to safely complete a left-turn maneuver. Calculating critical gap was not a main objective of this research, as the research focused more on the time-to-clear values; however, in order to calculate the left-turn lane warrants, critical gap had to be found. Therefore, a basic yet credible method was used, known as Raff's Method. The critical gap for each site was found, and the overall average was used for Harmelink's procedure.

First, one-second gap intervals were defined (0–0.99, 1.00–1.99, etc.). Then, for each gap interval the number of accepted gaps and the number of rejected gaps were counted, and a cumulative percent was found for both. The cumulative percent of accepted and rejected gaps were then plotted against the gap intervals, as seen in Figure **7**, which is the critical gap accumulation for Texas Site 11. The intersection of the cumulative accepted gaps and the cumulative rejected gaps is the critical gap for each site.

Each site had an average critical gap value around 5 or 6 sec and at most sites, there were no rejected gaps longer than 12 sec. Once the critical gap was determined for each site, a grand average and an overall 85th percentile were calculated for the entire data set as well as for the two geometric characteristics, presence of a left-turn lane and approach speed.

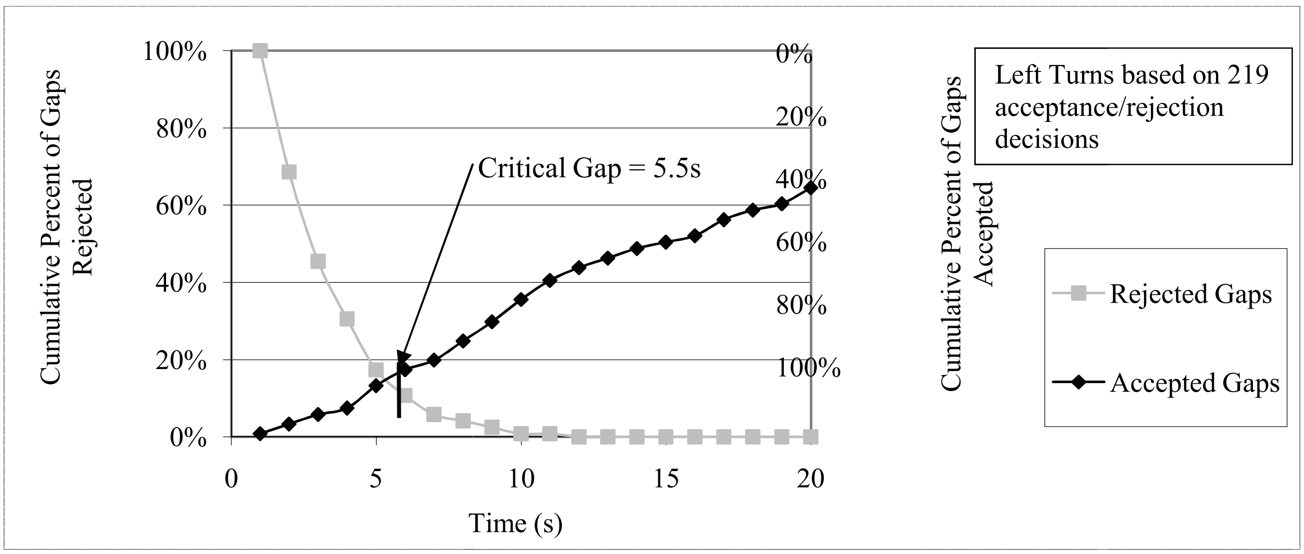

Figure 7. Cumulative Distributions of Accepted and Rejected Gaps in Left-Turn Maneuvers for TX-11.

Table 6 illustrates the average and 85th percentile critical gap as well as the values based on the two site characteristics that were studied.

Table 6. Critical Gap Comparison.

Entire Data Set		
Average	5.4 sec	
85th Percentile	6.8 sec	
Presence of Left-Turn Lane		
	Left-Turn Lane	No Left-Turn Lane
Average	5.4 sec	5.3 sec
85th Percentile	6.9 sec	6.0 sec
Approach Speed		
	High Approach Speed	Low Approach Speed
Average	5.8 sec	5.1 sec
85th Percentile	7.0 sec	5.9 sec

First, looking at the presence of a left-turn lane, the critical gap was less if a left-turn lane was not present, which could be expected, as a driver waiting with no left-turn lane will more likely accept a smaller critical gap to clear itself from the advancing traffic. The critical gap for an intersection with a high approach speed was greater than an intersection with a low approach speed. This can also be expected as drivers on a high-speed roadway will be more cautious, therefore wanting a greater critical gap to complete the left-turn maneuver.

Calculating Left-Turn Lane Warrants

The final step of the analysis was to revise the left-turn lane warrants. Through the use of a Harmelink calculation spreadsheet *(3)*, which was available from previous research, left-turn lane warrant curves were plotted based on approach speed, number of lanes on major, and percent of left-turning vehicles. Next, to plot each curve, based on the research, the two time-to-clear values and the critical gap values were used in place of Harmelink's original assumption values. Then, on the same graph, a curve based on Harmelink's assumptions was plotted and compared to the research's findings. Finally, any site that matched the characteristic of approach speed, number of lanes on major, and percent of left-turning vehicles could be plotted with the curve, based on the site's advancing and opposing volume.

For example, a site with a 35 mph approach speed, a two-lane major, and a 9 percent left-turning volume would use the updated assumptions found in this research:

	Average	85th Percentile
Time to clear advancing lane =	1.2 s	2.1 s
Time to clear the opposing lane(s) =	2.6 s	3.9 s
Critical Gap =	5.4 s	6.8 s

The resulting curve can be seen in Figure 8. Next, a curve based on Harmelink's assumption (current AASHTO's *Green Book*) was plotted:

Time to clear advancing lane =	1.9 s
Time to clear the opposing lane(s) =	3.0 s
Critical Gap =	5.0 s

The resulting curve can also be seen in Figure 8. Finally, the site(s) (in this case, two sites) that match the given criteria were plotted, again, shown in Figure 8.

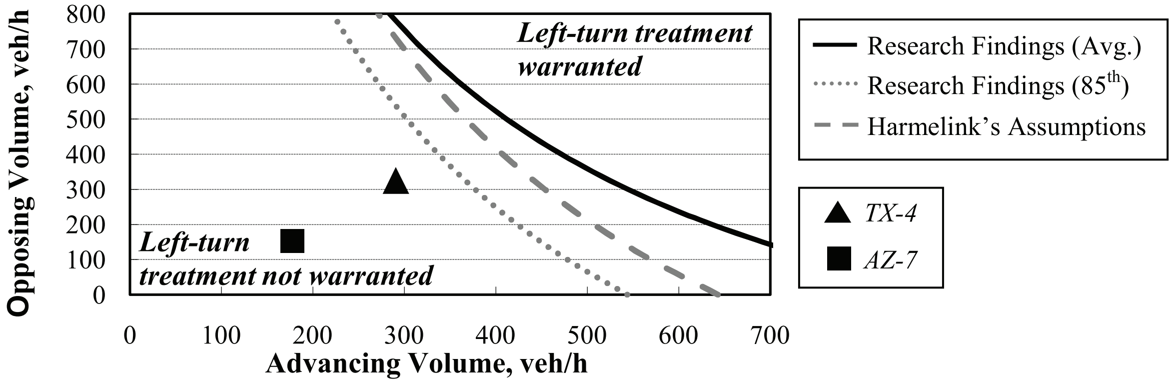

Figure 8. Left-Turn Lane Warrant Curve for Two-Lane Major, 35 mph Approach Speed, and 9 Percent Left-Turning Vehicles.

RESULTS

The activities in this research focused on two issues important to the parent research project: the final left-turn lane warrants based on the research's findings and the resulting time-to-clear values as compared to Harmelink's original assumptions. Key results from those activities are presented in this section.

Left-Turn Lane Warrants

First, looking at the updated left-turn lane warrants, after each site characteristic curve was plotted, the resulting warrants were compared based on the current existence of a left-turn lane, if a left-turn lane was warranted based on the research (average values), and if a left-turn lane was warranted based on Harmelink's original assumptions. The results can be seen in Table 7.

Table 7. Left-Turn Lane Warrant Calculation Results.

Site Name & Location		Current Existence of Left-Turn Lane		Based on Data - Left-Turn Lane Warranted?		Based on Harmelink's Assumptions - Left-Turn Lane Warranted?	
		With	*Without*	*Warranted*	*Not-Warranted*	*Warranted*	*Not-Warranted*
Four-Lane	TX-3		X	X		X	
	TX-5	X		X		X	
	TX-6		X	X		X	
	TX-9	X		X		X	
	TX-10	X		X		X	
	TX-11	X		X		X	
	AZ-1	X		X		X	
	AZ-2	X		X		X	
	AZ-3		X		X	X	
Two-Lane	TX-1	X		X		X	
	TX-2		X	X		X	
	TX-4		X		X		X
	TX-7		X		X		X
	TX-8.	X		X		X	
	AZ-4	X			X		X
	AZ-6		X		X		X
	AZ-7		X		X		X
	AZ-10		X		X		X

Left-turn lane is not present; research warrants left-turn lane

Research does not warrant left-turn lane; Harmelink warrants left-turn lane

Left-turn lane is present; research does not warrant left-turn lane

Of the intersections that do not have a left-turn lane, three were warranted to have left-turn lanes, both by the research results and by Harmelink's assumptions. This could show that these intersections receive a higher volume of traffic now than when they were originally designed. The second finding that should be noted was that of the sites that have a left-turn lane, one site, AZ-4, was not warranted to have a left-turn lane. This could have resulted from the fact that AZ-4 is a school intersection, receiving higher traffic volume during school hours; therefore, the existence of the left-turn lane might be needed for those conditions. Finally, of the 18 sites studied, there was only one site, AZ-3, in which Harmelink's assumptions warranted a left-turn lane and the current research did not warrant a left-turn lane. The overall difference between Harmelink's assumptions and the current research was minimal; therefore, there was no major discrepancy in the final left-turn lane warrants for most sites.

Time-to-Clear Values and Critical Gap

The second result and perhaps the more important result for this study would be the final time-to-clear values and critical gap. As presented in the introduction, the ultimate goal of this research was to compare Harmelink's original assumptions to that of current drivers. The preliminary findings for time to clear and critical gap based on this study can be seen in Table 8.

Table 8. Left-Turn Lane Warrant Assumptions.

	Harmelink		**Current Study**
	2-Lane	***4-Lane***	***(Preliminary)***
Time for vehicle to clear itself from advancing stream (s)	1.9	1.9	1.2 *(Avg.)* 2.1 *(85th)*
Time for vehicle to clear the opposing lanes (s)	3.0	4.0	2.6 *(Avg.)* 3.9 *(85th)*
Critical Gap (s)	5.0	6.0	5.4 *(Avg.)* 6.8 *(85th)*

Harmelink's assumptions are based on average values and are compared to the average values found in this study. The research found that Harmelink's values are higher with the exception of critical gap. However, the 85th percentile time-to-clear values that were found in this research match very close to that of Harmelink's assumptions, which could pose future questions as to which value should be used for Harmelink's calculations. Finally, it should be noted that the current study did not separate time-to-clear values and critical gap based on the number of lanes, as did Harmelink. In this study, after the preliminary results were generated, it was found that some two-lane intersections had a greater pavement width than some four-lane intersections, skewing the results of the time to clear the opposing lane. The 85th percentile for the time to clear the opposing lane of a four-lane major was less than the time-to-clear the opposing lane of a two-lane major, which is opposite of what should be expected. Therefore, future evaluations should consider pavement width as opposed to number of lanes on the major.

CONCLUSIONS

Based on these preliminary findings from this research, it was found that a difference does exist between Harmelink's original assumptions of average time to clear the advancing and opposing lanes compared to the average values found in this research. However, the 85th percentile time-to-clear values found in this study match very closely to that of Harmelink's average value. An interesting finding, this could show that today's drivers make left-turn maneuvers slightly faster than when Harmelink found his average values. Looking at the similarity between the 85th percentile and Harmelink's average could be advantageous and help determine what time-to-clear values and critical gap values are most pertinent for design.

As mentioned in the results, the preliminary outcomes also brought to light the issue of the number of lanes on the major road. The current sites as well as the additional sites that will be used in the parent research will consider the pavement width of the opposing lanes instead of the number of opposing lanes. This will provide a more accurate look at the time-to-clear values and will allow a better comparison to Harmelink's original assumptions, which are based on number of lanes in major. Additionally, the research also recommends that the starting position of each left-turning vehicle be noted, whether the vehicle came to a complete stop prior to starting the left turn or if the vehicle continued into the left-turn maneuver without stopping.

Overall, the results of this study assisted the parent project in providing preliminary findings for both the time-to-clear values and critical gap. Based on this research, the parent project was able to make appropriate changes that will aid in the collection of applicable data.

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Driver Behavior at Freeway Interchanges with Horizontal Signing

Prepared for
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STUDENT BIOGRAPHY

Stephanie Everett is a senior at Duke University in Durham, North Carolina. She will graduate in May 2010 with a Bachelor of Science in Engineering. She is majoring in civil and environmental engineering with an emphasis on structures and a minor in religion. In the fall of 2008, Stephanie completed an undergraduate study abroad program at the University of Sydney in Sydney, Australia.

Outside of class, Stephanie is a member of the Duke University Marching Band and a sister in Chi Omega Fraternity. She is one of the student leaders for Westminster Fellowship, a Presbyterian religious life group on campus. She is also a member of Duke's Chapter of the American Society of Civil Engineers. She was initiated into Chi Epsilon last spring.

Stephanie has had previous work experience with URS Corporation doing transportation planning in Ocoee, Florida, and roadway and bridge design in Morrisville, North Carolina. Following graduation from Duke, Stephanie plans to attend graduate school and continue working toward certification as a Professional Engineer.

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SUMMARY

Traditionally overhead and side-mounted signs have visually provided motorists with necessary information on the nation's freeways. Drivers can only take in this visual information from one source at a time. To attend to the multiple sources of information, drivers continuously scan the entire visual field relying on short glances to the roadway, signs, and mirrors.

When drivers experience stressful driving situations or fatigue, they tend to focus more on the roadway in front of their vehicle with fewer glances to mirrors and signs. Horizontal signs provide drivers with information at the location their eyes are already focused—the pavement in their lanes. Thus during stressful situations, drivers are likely to see the horizontal signing on the roadway sooner than overhead or side-mounted signs.

The study considers the addition of multi-color route shields at an interchange with existing white directional arrows. Video data from the before period were collected at an interchange with directional arrows and text that read "ONLY." Red and blue interstate shields were added to all lanes. Following an adjustment period to account for any novelty effects, video data from the after period were then collected. Both sets of video data were coded for the volume in each of three travel lanes as well as lane change maneuvers between these lanes.

The addition of highway shield symbols had a positive impact on lane change maneuvers in the study area. A change in the lane change distribution approximately 1450 ft upstream of the gore as well as a decrease in the proportion of lane change maneuvers in the segment extending from 900 ft to 1450 ft upstream of the gore indicates that drivers made lane changes further upstream in the after period.

Future research will consider other horizontal signing treatments for freeway exits and interchanges.

TABLE OF CONTENTS

INTRODUCTION

Positive guidance theory states that the driving activity consists of a hierarchy of three smaller tasks: control, guidance, and navigation (*1*). The control task deals with the interactions between the driver and his or her vehicle. Control includes reading gauges, manipulating the steering wheel, and shifting gears. Guidance is the relation between the driver and other motorists. Guidance tasks include maintaining speed relative to other vehicles and placement of the vehicle within the lane to avoid hitting other traffic or objects. The final driving task of navigation includes all decisions necessary to get from origin to destination. Navigation begins pre-departure when the driver chooses a destination and plans the initial route; but it continues throughout the trip as the driver makes additional route decisions based on guide signs, construction, and traffic conditions. These three tasks form a hierarchy in which control is the primary task while the least importance is placed on navigation.

Motorists gather nearly all of the information required for the guidance and navigation tasks from visual clues. According to Alexander, "Drivers scan the environment and sample the information in short glances until a potentially needed source is detected" (*2*). Humans cannot gather and comprehend visual information from multiple sources at the same time. By continuously making short glances at signs and mirrors, drivers can gather individual pieces of information in rapid succession.

Load shedding is the practice of neglecting the least important tasks in situations of fatigue or stress. For drivers in high-stress situations, control and guidance become increasingly more important while navigation is neglected (*1*). To maximize the likelihood of receiving visual information necessary for guidance, drivers will spend more time scanning the roadway directly in front of their vehicles, making fewer glances to vertical signs or mirrors. Horizontal signing can provide drivers with information even when fatigued or stressed by placing the information in the travel lanes where their eyes are already focused (*3*).

A field study of pavement markings on driver behavior at freeway lane drop exits demonstrated that the installation of lane drop arrow markings can cause a shift in motorist lane change locations in advance of a lane drop (*4*). Data demonstrated that fewer drivers moved out of the lane 800 ft immediately upstream of the gore during the after period. For the area between 1700 and 1000 ft upstream of the gore, more drivers left the exit lane in the after period than in the before period. Additionally the number of erratic movements within the entire study area decreased in the after period when lane use arrow markings were installed.

Advances in the availability of large multi-color thermoplastic pavement marking materials allow for horizontal signing in addition to the white directional arrows and text that are already in widespread use. Texas Department of Transportation (TxDOT) project 0-4471, "Evaluation of Horizontal Signing Applications," evaluated the durability of colored marking materials over a three year period on concrete, asphalt, and chip-seal pavements (*5*). For blue and red markings, like those that would be used on an interstate shield, as shown in Figure 1, the colors faded over time and did not stay in the color specifications beyond one year but did still appear blue and red to the naked eye.

Figure 1. Example of Highway Shield Pavement Marking.

Freeway interchanges with lane drops, double lane exits with optional lanes, and other unusual geometries violate driver expectations and may result in late lane changes and erratic movements near the gore. Highway shield horizontal signing has the potential to reiterate the information available on overhead signs, which explains upcoming interchange geometry. Receiving this information early-on and in multiple ways can allow drivers to make better driving decisions and make lane changes further upstream. This has the potential to reduce late lane changes and erratic movements near the gore.

Additionally, the human factors studies performed as a part of TxDOT 0-5890 have shown that drivers prefer to use exit only or through only lanes as opposed to optional lanes. Assuming this is the case in the field, it results in a decreased utilization of available roadway capacity near the gore area. Additional confirmation from horizontal signing that the optional lane can be used for both the exit or through movements could lead to an increased utilization of this lane.

PROJECT BACKGROUND

This research is part of a much larger project sponsored by TxDOT, “Guidelines for the Use of Pavement Marking Symbols at Freeway Interchanges” (Project 0-5890). The purpose of this project is to develop a set of design and application guidelines regarding the use of pavement marking symbols at freeway exits and interchanges.

To date, members of the Texas Transportation Institute (TTI) research team have conducted a state-of-practice review and several human factors evaluations. The human factors evaluations consisted of two tests. Driver surveys were used to determine driver preferences as well as evaluate drivers’ ability to correctly choose appropriate lanes for a given destination. Closed-course tests performed at the Texas A&M Riverside campus were used to determine legibility distances of various pavement marking alternatives.

Currently, TTI researchers are conducting field tests to be used to support or refute the results of the state-of-practice review and human factors studies. Specifically, the research described herein considers one of the unanswered questions coming out of the human factors evaluations:

if arrows are already present at an interchange, what impact (if any) does the addition of highway shields have on driver behavior? Additional field tests in other locations will be used to study the field application of other alternatives.

RESEARCH TASKS

The objective of this research is to evaluate the impact of the addition of highway shield pavement markings on driver behavior at one interchange where directional arrows and text (i.e., “ONLY”) were already present. The evaluation method is a before-and-after comparison of driver behavior. To evaluate the impacts of the additional horizontal signing, data were collected for both the before and after periods, then reduced and analyzed.

Data Collection

To complete the before-and-after comparison within the timeframe of the Undergraduate Transportation Scholars Program, data were collected prior to the start of the program. Data from only one site are considered because collection at other sites with different before-and-after pavement marking scenarios had not occurred at the start of the program.

Data Collection Site

Data were collected in San Antonio at the interchange between I-35S and I-410S as shown in Figure 2. This interchange was ideal as it contains an optional lane and the use of directional arrows is already a common practice in San Antonio. The leftmost lane, considered lane 1, is an exit only lane which exits to I-410S. Lane 2 is the optional lane shared between both freeways. Lanes 3, 4, and 5 on the right continue through to I-35S.

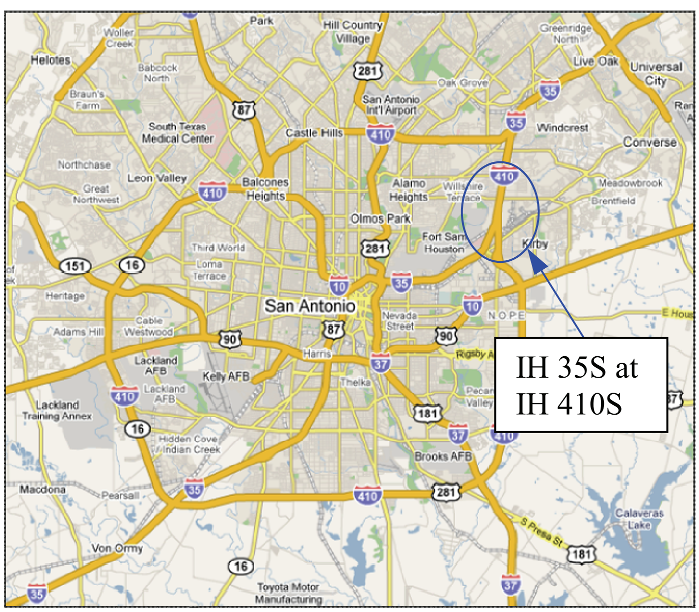

Figure 2. Location of Data Collection Site, San Antonio, TX.

Available Data

Both the before and the after data were collected through use of traffic monitoring cameras at TxDOT's Traffic Management Center (TMC) in San Antonio. TMC cameras provide a unique view of the freeway, but there are some limitations: the cameras may be diverted in emergencies and the field of view is sometimes limited. TTI requested a specific view, but had no control over the view of video ultimately provided from the TMC cameras.

One of the TMC cameras is located at approximately the same location of the cantilever overhead sign shown in the aerial photograph of the site in Figure 3. The camera looks southbound along the roadway towards the sign bridge at the exit ramp. Initially the area between the cantilever overhead sign and the gore was divided into three segments as shown in Figure 3. Segment 1 is between the cantilever overhead sign and the second highway light pole beyond the sign. This segment is approximately 550 ft in length. Segment 2, which is approximately 600 ft in length, is between the second highway light pole beyond the cantilever and the start of a concrete crash barrier upstream of the gore. Segment 3, which is in the immediate vicinity of the gore, is between the crash barrier and the gore. This segment is only 300 ft in length.

Figure 3. Aerial View of Site between Cantilever Overhead Sign and Gore.

Data for the before period were collected between Tuesday, July 8, 2008, and Friday, July 11, 2008. TTI received over 50 hours of before video. Of this, 29 hours were coded in the video reduction phase. The summary of available and usable video included in Appendix A describes which video was removed.

Shields and cardinal direction text were installed in all five lanes on October 12, 2008. Data for the after period were collected on Thursday, April 23, 2009, and Friday, April 24, 2009. For this period, TTI received 24 hours of video and were able to use 12 hours. The summary of available and usable video included in Appendix A documents which video segments were removed.

Video was removed for several reasons. Inclement weather (i.e., rain) and twilight conditions were removed because drivers are already at a disadvantage during these times. Additionally, the night data were removed because the TMC camera did not focus well enough on the vehicles for accurate counts. Any instances where the camera was diverted from the interchange or emergency or traffic control vehicles changed the flow of traffic were also removed.

Pavement Marking Plans

Figure 4 and Figure 5 contain the before and after pavement marking plans, respectively, for the entire study site. For both layouts, the area shown includes roadway that is not visible from the TMC camera. The entire study site must be considered because horizontal signing was added both upstream and downstream of the camera.

For the before period the only information provided to motorists included: directional arrows, text that read “ONLY,” and overhead signs. Drivers pass over four sets of directional arrows and “ONLY” markings before reaching the cantilever sign. The cantilever is an “EXIT ONLY” sign for lane 1. Three additional sets of directional arrows and “ONLY” markings are between the cantilever and the sign bridge. Two of these marking sets are within segment 1, where data reduction counts were made, as shown by the shaded box in Figure 4.

To accommodate the addition of shield symbols, one set of directional arrows as well as one “ONLY” marking had to be relocated further upstream in the after period. One set of shields and cardinal directions was added to all lanes of the freeway in the upstream portion of the study site. This set of shields is upstream of segment 1, which is indicated by the shaded box in Figure 5. An additional set of shields was added immediately upstream of the gore. This set of shields is downstream of segment 1 where counts were made. Although the second set of shield markings is visible from segment 1 of the road, the markings are not legible at this location. Thus the data obtained from segment 1 represents the impact of the addition of the first set of shields.

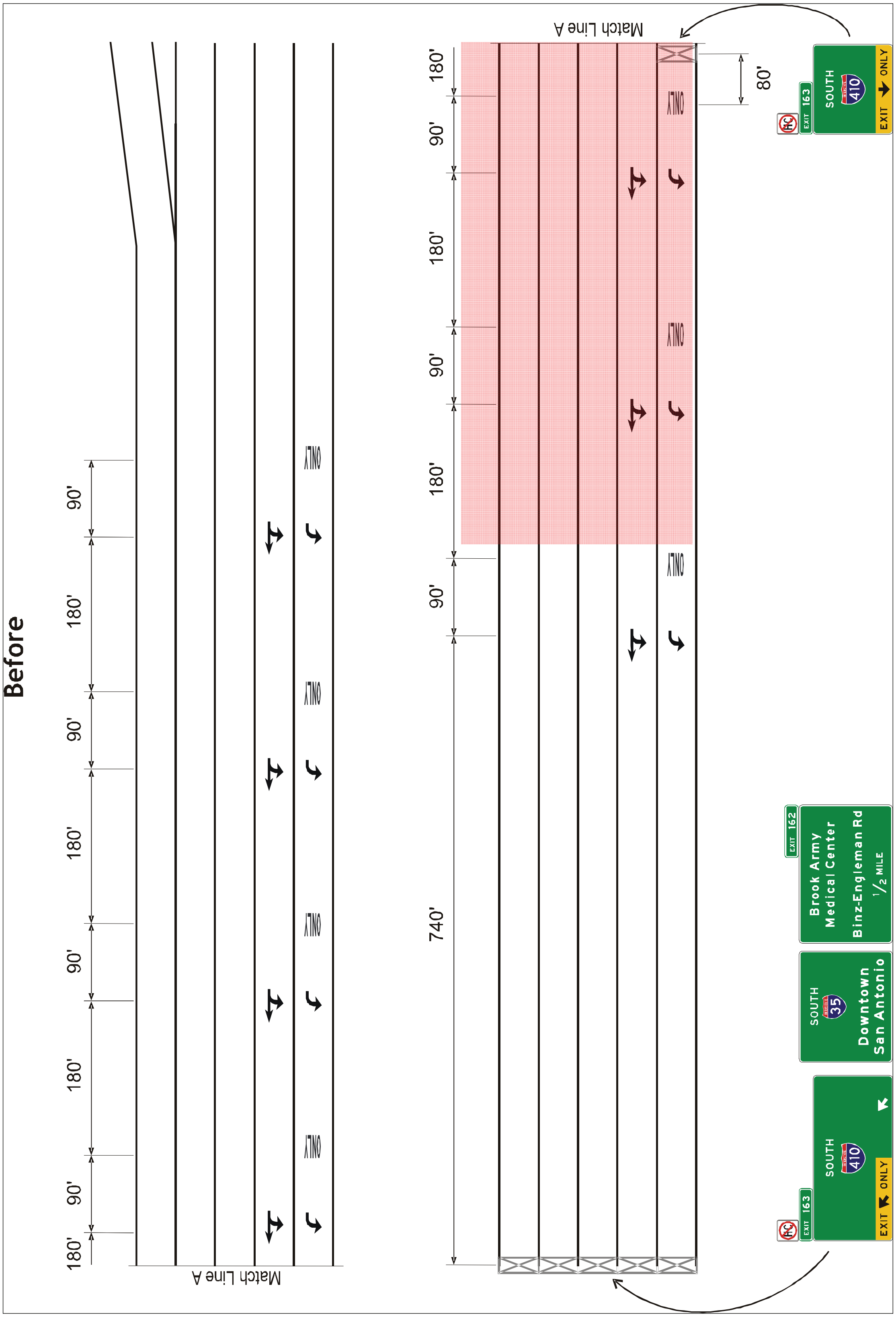

Figure 4. Pavement Marking Plan prior to 10/12/2008.

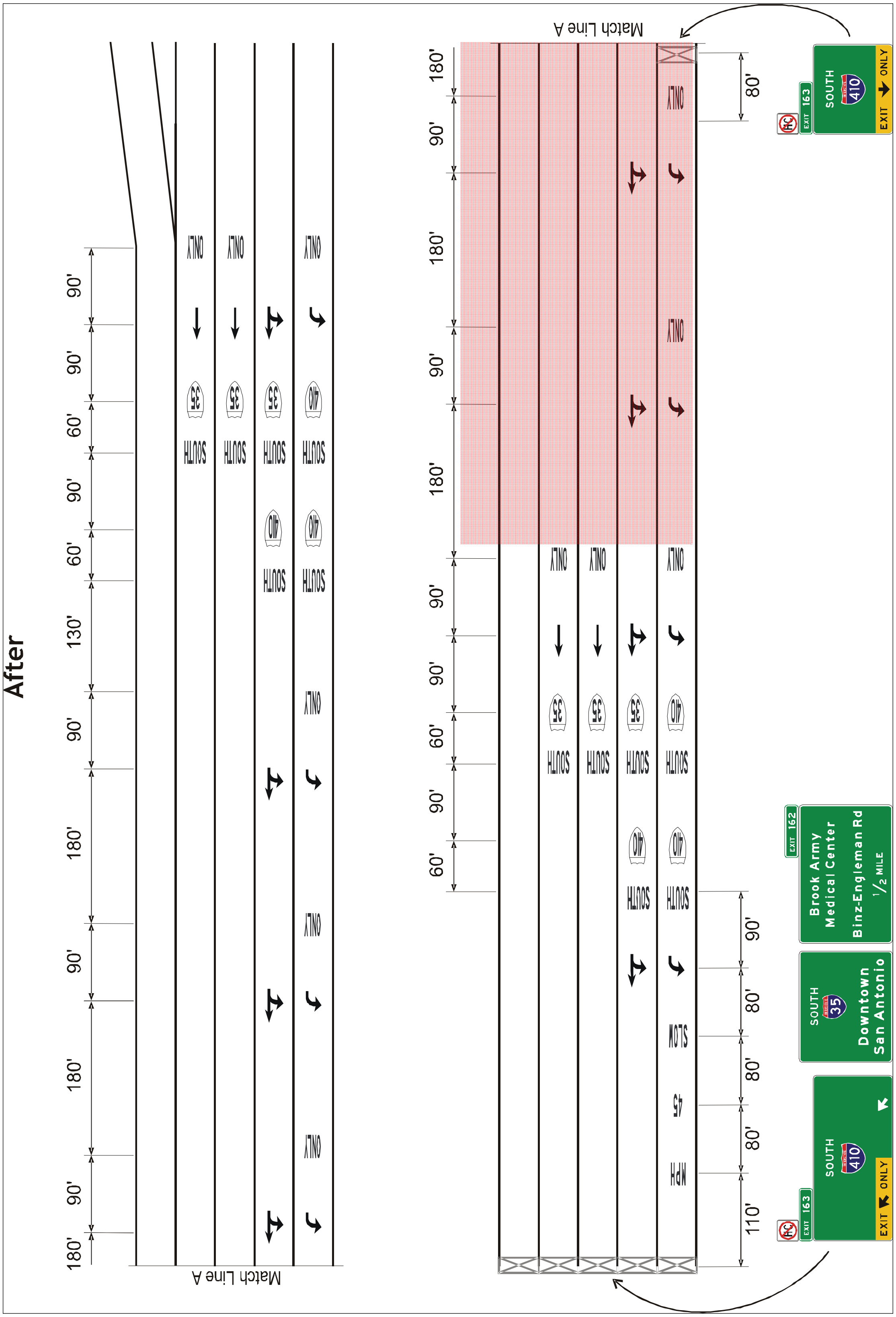

Figure 5. Pavement Marking Plan after 10/12/2008.

Data Reduction

Two measures of effectiveness (MOEs) were considered: the volume of vehicles in each lane and the number of lane changes.

Multiple screenings of each video were required to gather all necessary data. During the first screening of the video, volume counts were made for each of the three leftmost travel lanes at the start of segment 1. A second viewing was necessary to make lane change maneuver counts throughout segment 1. All of the videos were watched in real time in 15-minute intervals. The flow of the traffic and the weather conditions were noted for each 15-minute time period. Appendix B contains sample forms used for data reduction.

Data reduction counts were completed for lanes 1, 2, and 3. Researchers wanted to focus on lane 2 because it is the optional lane as well as the two adjacent lanes. The volumes and lane change maneuvers for lanes 4 and 5 were not considered because it is assumed that the drivers in these two lanes intend to continue through to I-35S. Although some vehicles did move from either of these lanes into the lanes of interest, the proportion of drivers making these movements was extremely small and thus not included in the data analysis.

To verify that the overall traffic volume did not change between the before and after period, counts of the traffic volume in lanes 4 and 5 at the cantilever sign were made for a smaller portion of the data (a 7-hour period).

Data Analysis

To conduct a before-and-after comparison of driver behavior, any possible significant change in the freeway volume must be investigated to account for any confounding effect. To accomplish this, the total volume counts for all five lanes of traffic were used to determine the freeway hourly volumes at the site in each period. A Z-test for differences in means was used to test for significant variation between the two volumes. The formula used was as follows (*6*):

$$Z = \frac{X_1 - X_2}{\sqrt{\frac{s_1^2}{n_1} + \frac{s_2^2}{n_2}}}$$

where:	X_1	=	mean of before volumes
	X_2	=	mean of after volumes
	s_1^2	=	variance of before volumes
	s_2^2	=	variance of after volumes
	n_1	=	before sample size
	n_2	=	after sample size

A Z-statistic of greater than 1.96, which corresponds to a 95 percent level of confidence (alpha = 0.05) indicates a significant change in volume.

Hourly averages for both MOEs were determined from the 15-minute segments. First the raw data were used to determine the hourly average for each hour between 7:00 a.m. and 7:00 p.m. for each day that data were collected. Each of these hours was used to determine an overall hourly average for the before period and the after period.

The hourly averages of both MOEs were then plotted for each day. The patterns over all 12 hours were similar over all data collection days; thus, the data were consolidated and hourly averages for a typical day were computed. These data were further grouped into peak and non-peak times. A morning peak, lunch peak, and evening peak were defined.

The Bernoulli model was used to test for a significant difference in the before and after data for both MOEs for each peak and non-peak time period (*6*). The formula used to determine the test statistic was as follows:

$$Z = \frac{f_1/n_1 - f_2/n_2}{\sqrt{\frac{f_1+f_2}{n_1+n_2}\left(1-\frac{f_1+f_2}{n_1+n_2}\right)\left(\frac{1}{n_1}+\frac{1}{n_2}\right)}}$$

where: f_1 = individual lane volume or number of lane change maneuvers in before period
f_2 = individual lane volume or number of lane change maneuvers in after period
n_1 = total volume in lanes 1-3 or volume of lane in which movement originated in before period
n_2 = total volume in lanes 1-3 or volume of lane in which movement originated in after period

This test compares two proportions of independent random samples. The null hypothesis was that the before and after proportions were equal. The alternate hypothesis was that the before and after proportions were not equal. If the Z statistic was greater than 1.96 or less than -1.96, the null hypothesis was rejected. This value was selected again using a 95 percent level of confidence (alpha = 0.05). Rejection of the null hypothesis indicates that there is a statistically significant difference in driver behavior between the before and after periods.

RESULTS

Statistical analysis showed that there were no differences in before-and-after traffic volumes at the site. Thus, any changes in driver behavior can be attributed to the addition of the first set of shields in the upstream portion of the study segment.

Table 1 contains the hourly averages for the volume of traffic in each of lanes 1, 2, and 3 as well as the number of lane changes coming from each lane. Overall there was a slight increase in the freeway hourly volume, but there was a decrease in the traffic volume in lanes 1, 2, and 3. There was a decrease in the average number of lane changes in an hour coming from all three lanes in the observed segment. The number of lane changes per one million cars also decreased between the before and after period.

Table 1 . Comparison of Before-and-After Data.

Site Characteristics			
Total study length	2700 ft		
Segment length	550 ft		
Segment location	900 ft to 1450 ft upstream of gore		
Before data collection period	7/8/2008 – 7/11/2008		
Shield installation date	10/12/2008		
After data collection period	4/23/2009 – 4/24/2009		
	Before	After	Change
Freeway hourly volume[a]	4952	5242	6%
Average hourly volumes			
Total lanes 1-3	3776	3384	-10%
Lane 1	1080	895	-17%
Lane 2	1317	1229	-7%
Lane 3	1378	1260	-9%
Average hourly lane change maneuvers			
Total lanes 1-3	194	142	-27%
Lane 1	12	6	-55%
Lane 2	81	66	-19%
Lane 3	101	70	-30%
Rate (10^6/ft/veh)[b]			
Total lanes 1-3	93	76	-18%
Lane 1	21	11	-46%
Lane 2	112	98	-13%
Lane 3	133	101	-23%

[a] Freeway hourly volumes were measured at the cantilever overhead sign and represent the average of the time periods used in the comparison. Z = 0.91 for the difference in before and after volumes.

[b] Rates were determined by dividing the number of lane changes in an hour by 550 ft (segment length) and hourly volume, then multiplying by 1,000,000.

Distribution of Traffic

Figure 6 shows the percent of the volume in a given lane out of the total volume in lanes 1, 2, and 3 broken down by peak and non-peak time periods. This plot shows an increase in the proportion of the volume traveling in lane 2 and also in lane 3, which corresponds to a similar decrease in the proportion of the volume traveling in lane 1 between the before and after periods.

Table 2 includes the results of the Bernoulli tests for the distribution of traffic in each lane broken down by peak and non-peak time periods. For the morning peak, there was no significant change in the distribution of traffic. For all other time periods considered, there was a statistically significant difference in the distribution of traffic. The significant increase in the percent of vehicles in lane 2 indicates that drivers are more willing to use the optional lane, which allows for better use of the available roadway capacity.

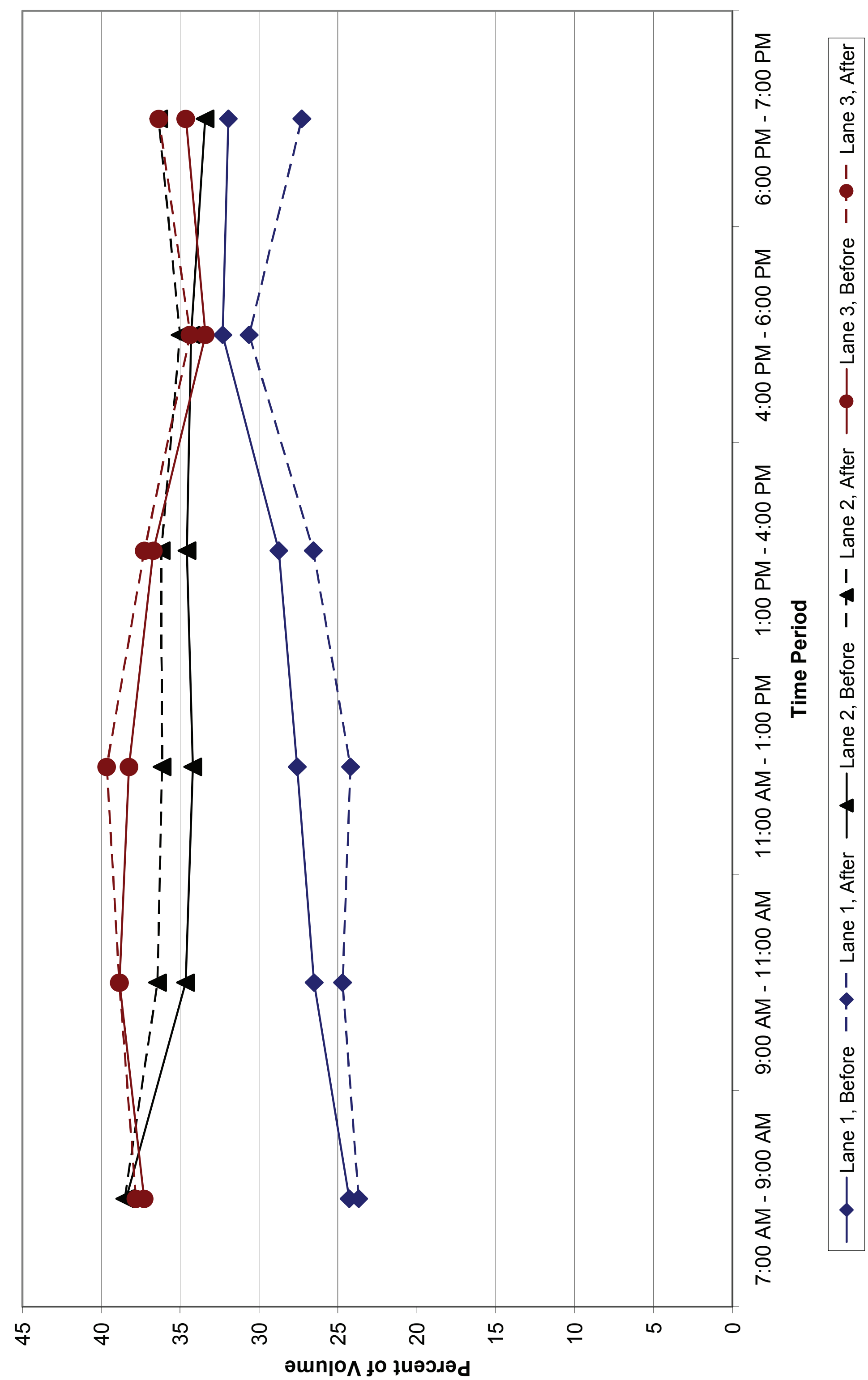

Figure 6. Lane Distribution Plot.

Table 2. Results of the Bernoulli Tests for Volume in Each Lane.

	Volume Distribution across Lanes 1-3								
	Lane 1			Lane 2			Lane 3		
	Before	After	Change	Before	After	Change	Before	After	Change
7:00 AM – 9:00 AM[a]	24.3	23.7	-2%	38.4	38.5	0%	37.3	37.8	1%
Z statistic[b]	0.94			-0.10			-0.73		
9:00 AM – 11:00 AM[a]	26.5	24.7	-7%	34.6	36.4	5%	38.8	38.9	0%
Z statistic[b]	2.82			-2.58			-0.01		
11:00 AM – 1:00 PM[a]	27.6	24.2	-12%	34.2	36.1	6%	38.2	39.7	4%
Z statistic[b]	5.37			-2.88			-2.06		
1:00 PM – 4:00 PM[a]	28.7	26.6	-8%	34.6	36.2	5%	36.7	37.3	2%
Z statistic[b]	4.54			-3.18			-1.10		
4:00 PM – 6:00 PM[a]	32.3	30.5	-5%	34.3	35.0	2%	33.4	34.4	3%
Z statistic[b]	2.97			-1.25			-1.68		
6:00 PM – 7:00 PM[a]	31.9	27.3	-15%	33.4	36.4	9%	34.6	36.3	5%
Z statistic[b]	5.09			-3.12			-1.80		
7:00 AM – 7:00 PM[a]	28.6	26.4	-8%	34.9	36.3	4%	36.5	37.2	2%
Z statistic[b]	8.70			-5.42			-2.74		

[a] Percent of volume in lane for an average day of observations over given time period.
[b] If the calculated Z statistic is greater than 1.96 or less than -1.96, then one can conclude that the difference is significant. Shaded boxes are significantly different.

Lane Change Maneuvers

Figure 7 shows the percent of the volume within a given lane that makes a move into another lane. This plot demonstrates a general trend of fewer movements from all three lanes in the after period for the observed segment. Table 3 includes the results of the Bernoulli tests for lane change maneuvers broken down by peak and non-peak time periods.

The significant decrease in movement in the observed segment can be explained in one of two ways: 1) drivers are making lane changes further upstream of the gore (i.e., before segment 1), or 2) drivers are waiting longer to make lane change maneuvers. Situation 1 represents the intended consequence of the additional shields while the latter situation would be a negative impact. Researchers believe that the former option is occurring because there is a change in traffic distribution at the upstream end of the segment. If drivers were waiting to make lane changes, there would likely not be a significant difference in the volume of cars in each of the three lanes at the overhead cantilever sign. Unfortunately views that would have allowed researchers to verify that lane change maneuvers are happening farther upstream in the after period were not available. A decrease in the number of lane changes results in less erratic movements and thus in fewer conflicts near the gore.

Table 3. Results of the Bernoulli Tests for Lane Change Maneuvers.

	Lane Change Maneuvers Coming from Lanes 1-3								
	Lane 1			Lane 2			Lane 3		
	Before	After	Change	Before	After	Change	Before	After	Change
7:00 AM – 9:00 AM[a]	0.8	0.7	-7%	4.2	5.3	26%	5.9	5.5	-7%
Z statistic[b]	0.21			-2.20			0.69		
9:00 AM – 11:00 AM[a]	1.3	0.7	-46%	5.8	6.0	3%	6.4	5.1	-20%
Z statistic[b]	1.94			-0.28			2.28		
11:00 AM – 1:00 PM[a]	1.4	0.9	-33%	6.7	5.9	-11%	7.4	5.3	-29%
Z statistic[b]	1.45			1.22			3.78		
1:00 PM – 4:00 PM[a]	1.1	0.5	-56%	6.6	5.3	-20%	7.3	5.7	-22%
Z statistic[b]	3.05			3.01			3.51		
4:00 PM – 6:00 PM[a]	1.1	0.5	-61%	6.8	5.2	-23%	8.9	6.2	-30%
Z statistic[b]	3.30			3.19			4.80		
6:00 PM – 7:00 PM[a]	1.0	0.7	-29%	5.9	3.9	-33%	7.3	5.3	-27%
Z statistic[b]	0.81			2.60			2.41		
7:00 AM – 7:00 PM[a]	1.1	0.6	-46%	6.2	5.4	-13%	7.3	5.6	-23%
Z statistic[b]	4.90			3.63			7.45		

[a] Percent of volume in a given lane making a lane change maneuver for an average day of observations of the given time period.

[b] If the calculated Z statistic is greater than 1.96 or less than -1.96, then one can conclude that the difference is significant. Shaded boxes are significantly different.

Lane changes can also be grouped as necessary and unnecessary movements. Unnecessary lane changes reduce the capacity of the roadway and increase the potential for conflicts. Movement out of lane 2 is strictly unnecessary (e.g., 2 to 1 and 2 to 3) because drivers always have an option in this lane to either exit or continue through. By not using lane 2, full capacity of the roadway cannot be reached. Lane change maneuvers across multiple lanes of traffic are also unnecessary (e.g., 3 to 1 and 1 to 3) because they increase the potential for conflicts. Drivers move completely through lane 2 to change their path when they could have simply moved into the optional lane. For this project it is assumed that movements into lane 2 (e.g., 3 to 2 and 1 to 2) are necessary lane change maneuvers, as these lane changes must be made if the driver wants to change his or her freeway options. Figure 8 shows the percent of vehicles making necessary and unnecessary lane change maneuvers. This plot also indicates a general trend of decreased movement in the after period for the observed segment.

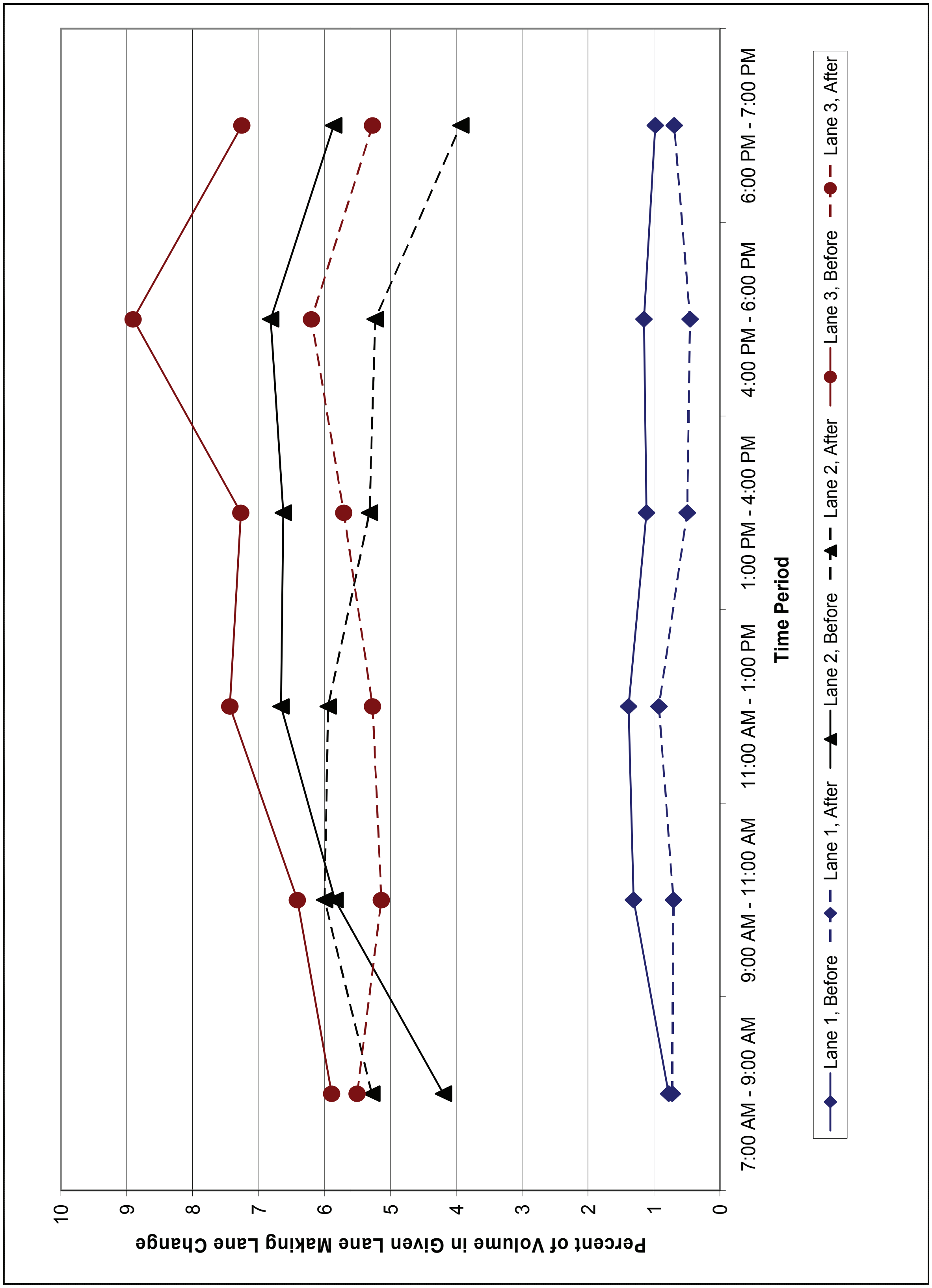

Figure 7. Lane Changing Maneuver Plot.

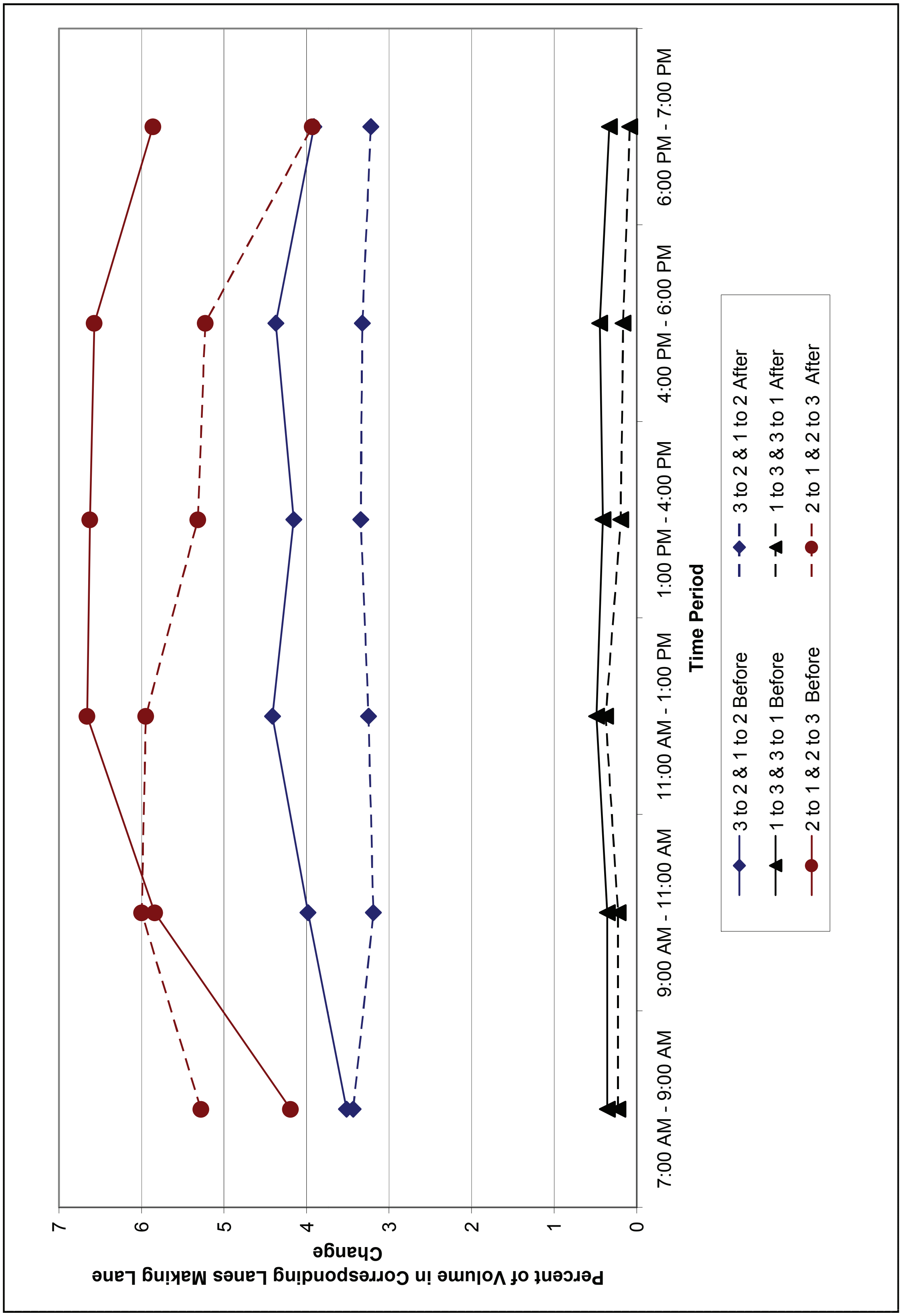

Figure 8. Unnecessary vs. Necessary Lane Change Maneuver Plot.

CONCLUSIONS AND RECOMMENDATIONS

There was an increase in the percent of drivers using the optional lane between the before and after periods. Increased use of the optional lane allows for a more efficient use of the available roadway by taking away additional strain on the capacity of adjacent lanes. There was also a decrease in the number of lane change maneuvers in the observed segment between the before and after periods. The change in the volume distribution indicates that this decrease is due to drivers making lane changes further upstream of the gore.

While there is an observed positive impact at this site, cost and maintenance issues must also be considered before installing shields. The shields used at this site cost approximately 10 times the typical cost of directional arrows. There are also still concerns about the durability of the multi-color thermoplastic materials that were used to create the shields. Additional investigation is necessary to determine when shields can be beneficial and when the lifetime cost of the shields is too prohibitive.

The parent project will look at additional horizontal signing alternatives for freeway interchanges that were not tested as a part of the Undergraduate Transportation Scholars Program. Alternatives include the installation of shields only and the installation of directional arrows only on roads that previously had no horizontal signing.

The results of this field study in conjunction with additional evaluations included in the parent project will be used to develop uniform guidelines for horizontal signing. Guidelines can advance the state-of-the-practice so that standard shield and directional arrow symbols are used at freeway exits and interchanges. This can lead to an overall improvement in safety as drivers learn to identify standard symbols and modify behavior accordingly.

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APPENDIX A

SITE 1, SAN ANTONIO, TX
Intersection: SB 1-35 & SB I-410

AVAILABLE BEFORE VIDEO

Video Number	Date	Time frame	Total Video Collected Start	Total Video Collected End	Total Video Collected Duration	Video Removed Start	Video Removed End	Video Removed Duration	Available Video	Notes	Lane 4 & 5 Volume Counts
1	7/8/2008	Afternoon	14:53:46	21:03:37	6:09:51	14:53:46	15:15:00	0:21:14		remove rain	
	Tuesday					17:15:00	21:03:37	3:48:37		camera leaves interchange at 17:15:11	
							SUM	4:09:51	2:00:00		
2	7/9/2008	Morning	8:24:46	15:30:35	7:05:49	8:24:46	8:30:00	0:05:14			
	Wednesday					11:15:00	11:30:00	0:15:00		sign vehicle driving through segment	
						12:00:00	12:30:00	0:30:00		camera leaves interchange between 12:13 and 13:15:14	
						15:30:00	15:30:35	0:00:35			
							SUM	0:50:49	6:15:00		
3	7/9/2008	Afternoon/	15:31:00	23:43:49	8:12:49	15:31:00	23:43:49	8:12:49		camera pulled back, cannot view gore area	
	Wednesday	Night					SUM	8:12:49	0:00:00		
4	7/10/2008	Morning	7:26:53	15:35:00	8:08:07	7:26:53	7:30:00	0:03:07			
	Thursday					12:45:00	13:00:00	0:15:00		traffic control vehicle in left lane	10:00:00 - 12:00:00
						15:15:00	15:35:00	0:20:00		camera leaves interchange at 15:15:56	13:00:00 - 15:00:00
							SUM	0:38:07	7:30:00		
5	7/10/2008	Afternoon/	15:35:20	23:46:55	8:11:35	15:35:20	15:45:00	0:09:40			
	Thursday	Night				19:00:00	23:46:55	4:46:55		too dark to count/camera not focused	16:00:00 - 19:00:00
							SUM	4:56:35	3:15:00		
6	7/11/2008	Morning	7:53:38	12:31:20	4:37:42	7:53:38	8:00:00	0:06:22			
	Friday					11:00:00	11:15:00	0:15:00		camera leaves interchange between 11:08:20 and 11:09:45	
						12:30:00	12:31:20	0:01:20			
							SUM	0:22:42	4:15:00		
7	7/11/2008	Afternoon/	12:31:42	20:43:03	8:11:21	12:31:42	13:00:00	0:28:18			
	Friday	Night				18:45:00	20:43:03	1:58:03		camera leaves interchange at 18:57:00	
							SUM	2:26:21	5:45:00		
				TOTAL	50:37:14			TOTAL	29:00:00		

AVAILABLE AFTER VIDEO

CD File Number	Date	Time frame	Total Video Collected Start	Total Video Collected End	Total Video Collected Duration	Video Removed Start	Video Removed End	Video Removed Duration	Available Video	Notes	Lane 4 & 5 Volume Counts
1	4/23/2009	Morning/	10:00:00	16:00:00	6:00:00	10:00:00	10:00:00	0:00:00			
	Thursday	Afternoon				16:00:00	16:00:00	0:00:00			10:00:00 - 12:00:00
							SUM	0:00:00	6:00:00		13:00:00 - 15:00:00
2	4/23/2009	Afternoon/	16:00:00	22:00:00	6:00:00	16:00:00	16:00:00	0:00:00		removed twilight data/too dark to count/camera not focused	
	Thursday	Night				19:30:00	22:00:00	2:30:00			16:00:00 - 19:00:00
							SUM	2:30:00	3:30:00		
3	4/24/2009	Night	22:00:00	4:00:00	6:00:00	22:00:00	4:00:00	6:00:00		all night data/too dark to count/camera not focused	
	Friday						SUM	6:00:00	0:00:00		
4	4/24/2009	Morning	4:00:00	10:00:00	6:00:00	4:00:00	7:30:00	3:30:00		removed night and twilight data	
	Friday					10:00:00	10:00:00	0:00:00			
							SUM	3:30:00	2:30:00		
				TOTAL	24:00:00			TOTAL	12:00:00		

APPENDIX B

Project 0-5890 *April 1, 2009*

Site #: 1 *Site Description:* I410S/I35S Interchange in San Antonio, Texas

*Recorder:*_______________ *Page:*______

VOLUME COUNTS

Video #:______ Video Date:_____________ Video Start Time:__________ Video End Time:__________

Location of Count: In line with cantilever overhead sign

Time [a]		**Lane Configuration**					**Flow [b]**	**Weather [c]**
		1-Inside	**2**	**3**	**4**	**5-Outside**		
From	**To**	**Exit Only**	**Shared**	**Thru**				

[a] Time periods should be in 15 minute intervals
[b] Free Flow (FF); Congested (C); Stop & Go (SG)
[c] Sunny (S); Overcast (O); Raining (R)

Project 0-5890 — *April 1, 2009*

Site #: 1 — *Site Description:* I410S/I35S Interchange in San Antonio, Texas

*Recorder:*_______________ — *Page:*______

LANE CHANGE MANEUVER COUNTS

Video #:______ Video Date:_____________ Video Start Time:__________ Video End Time:__________

Location of Count: Segment 1: Between cantilever overhead sign and 2nd light post from cantilever (~550 ft)

Time [a]		**Lane Configuration**						**Flow [b]**	**Weather [c]**
		1-Exit Only		**2-Shared**		**3-Thru**			
From	**To**	**To 2**	**To 3**	**To 1**	**To 3**	**To 1**	**To 2**		

[a] Time periods should be in 15 minute intervals
[b] Free Flow (FF); Congested (C); Stop & Go (SG)
[c] Sunny (S); Overcast (O); Raining (R)

Quality Assurance in Speed Data Collection Methods at High Speeds

Prepared for
Undergraduate Transportation Scholars Program

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Program Sponsored by:
Southwest Region University Transportation Center

August 6, 2009

STUDENT BIOGRAPHY

The proudest member of the "Fighting Texas Aggie Class of 2009," Jordan Easterling is set to graduate from Texas A&M University's Civil Engineering program in December of 2009.

Easterling worked previously as an intern for a Brown and Gay Engineers, a civil engineering consulting firm based out of Houston, TX. During this internship, he was exposed to several different aspects of transportation engineering. Outside of civil engineering, he enjoys several intramural sports and local music. He also is involved in several student organizations including a safe-ride program, CARPOOL, and an arts organization, MSC Townhall. Easterling looks to join the local Engineers without Borders chapter in the short term. His long-term goals include attaining his P.E. and opening his own consulting firm.

ACKNOWLEDGMENTS

The research described in this paper was conducted as part of Project 0-5911, 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 also like to thank his mentor, Dr. Kay Fitzpatrick, for the guidance throughout the duration of his research effort.

SUMMARY

In the past, much research and work was done for design speeds up to 55 mph, as it was the national posted speed limit cap. Today, most states have significantly higher posted speed limits. Texas, specifically, has posted speed limits as high as 80 mph. Therefore, there is a demand for research for driver and vehicle behavior at these higher speeds.

Research was conducted to determine the accuracy of several different devices when used at higher speeds. These devices included: pneumatic tube counters, video footage, a LIDAR gun, and a control vehicle outfitted with devices to monitor speed and location.

Data were collected near Kerrville, TX, in both 70 and 80 mph posted speed limit sections, using the various mediums mentioned beforehand, during two days in June 2009. Once acquired, the data were then formatted to a worksheet for comparisons. Device comparison involved

determining the speed difference between two devices at a time. This difference was then compared by daytime posted speed limit.

Device comparison showed that there was a significant increase in differences as speeds increased from 70 to 80 mph consistently for all devices. Also, the pneumatic tubes showed to be consistently higher in differences than both the LIDAR gun and the control vehicle values. These finding are preliminary, as more work must be done to determine the cause of these differences. As well as this development, more analysis must be done to determine the differences in inter-vehicle gap and classification between devices.

TABLE OF CONTENTS

INTRODUCTION

To further understand the interacting relationships between parameters involved in high speed highways, several factors must be researched. One of these factors includes the vehicle spacing as it relates to highway volume and posted speed limits. Though data has been collected already, there is a critical need for accuracy of both the instruments implemented and the methods of collection used with these instruments. This accuracy will be determined as well as the vehicle spacing as it relates to the posted speed limit. Several other parameters will be explored as they affect the vehicle spacing such as light level, lead vehicle size, lead vehicle speed, and other potential factors.

BACKGROUND INFORMATION

Previous studies have been conducted to relate the vehicle speed and vehicle spacing resulting in the capacity values used in the Highway Capacity Manual (*1*). Due to the limited number of high speed highways in the United States (see Figure 1); little has been done in studying this relationship at speeds on the order of 75 or 80 mph. The Texas Transportation Institute (TTI) is conducting the Texas Department of Transportation's (TxDOT's) 0-5911 project entitled, "*Gap/Headway Traffic Counter: Combined Data Set and Cumulative Distribution Chart.*" As part of the project, TTI is examining the axle gap at high speeds (*2*). The following question is being explored as part of the TxDOT project: is spacing between vehicles greater at high speeds?

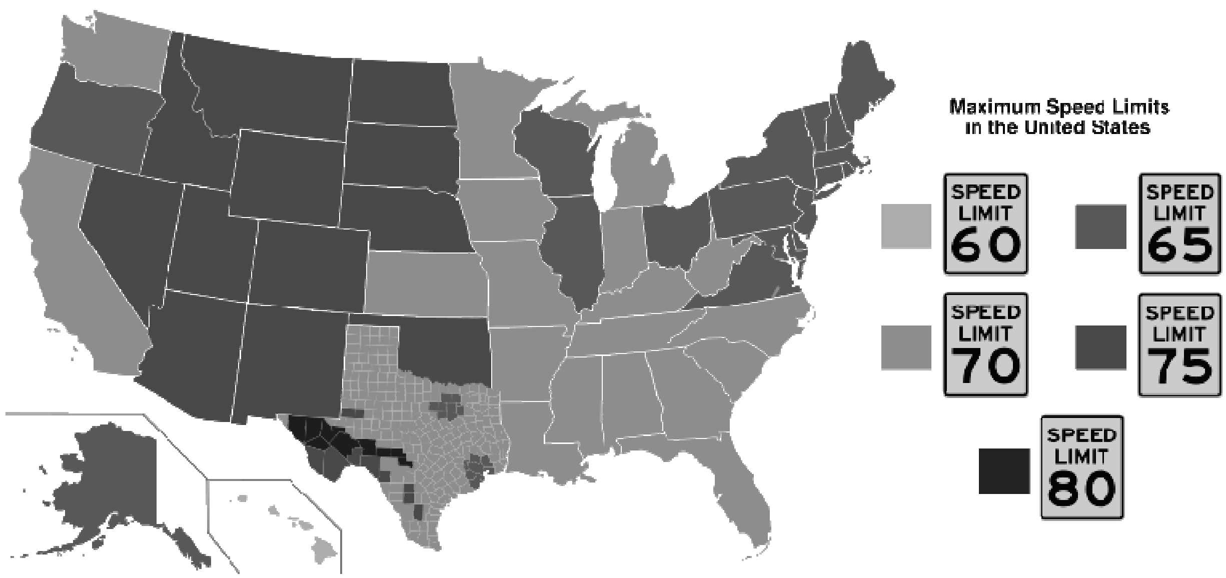

Figure 1. Posted Speed Limits by State (*3*).

The preliminary findings from the parent project, TxDOT's 0-5911, are included in Table 1. These data were collected using the pneumatic tubes that are used often in traffic data collection. Usually, the average speed is relatively close to the Daytime Posted Speed Limit (DPSL). As seen in Table 1, the 60 and 70 mph site data follow this general pattern. At the 11 different collection sites, this is not the case. This deviation from the normal occurrence leads to question whether or not the data are accurate. If the data are not accurate, either the instruments are faulty or the methods are inadequate. Pneumatic tubes have proven accurate in their long use for traffic data collection, though collection is probably rare for speeds as high as 70 or 80 mph posted speed limits. Therefore, the need for a check of accuracy is critical in the progress of understanding this relationship between vehicle spacing and vehicle speed.

Only after the collection methods are confirmed to be accurate or easily adjusted to be accurate, can understanding the relationship between the vehicle spacing and the vehicle speed is determined.

Table 1. Means and Standard Deviations of Collection Sites.

Daytime Posted Speed Limit (mph)	Number of Sites	Passenger Cars	
		Average Speed (mph)	Standard Deviation (mph)
60	5	61.53	7.35
70	17	68.70	8.41
80	11	72.00	8.05

HYPOTHESIS

After the preliminary findings of the TxDOT project, number 0-5911, a hypothesis was developed. The hypothesis was that the *"Accuracy of speed data from different traffic collection devices will be unaffected by higher speeds."* The general aim of the project was to confirm or refute this, though the predicted cause of the findings in 0-5911 could be more readily attributed to several other factors in human behavior rather than the inaccuracy of the detection devices.

DATA COLLECTION

The five main characteristics of the data collection effort are described in detail in the following.

Site Layout

Using pneumatic tubes and plate counters (similar to those being used at other collection sites for TxDOT 0-5911), a LIDAR gun and a camcorder captured vehicle speeds, lane presence, classifications, and other vehicle criteria. The camcorder was set up in line with the first contacted tube, offset by a known distance. The LIDAR gun was operated a known distance from the tubes and the roadway, collecting data from vehicles departing the instrument and approaching the tubes. The typical layout for these sites is shown in Figure 2. The tube counters are offset by 16 ft. The plate counter was located between the tubes and set in the middle of the 12 ft lane. The values for A, B, and C vary according to site and are shown in Table 2. These distances are recorded for two reasons. One, the data set given by the LIDAR gun is relative to its position and velocity. Two, the LIDAR gun's measurements need to be adjusted if recording within a certain angle. Appendix A provides information on needed position for the LIDAR gun to avoid adjustments. For this data collection, adjustments were not recorded.

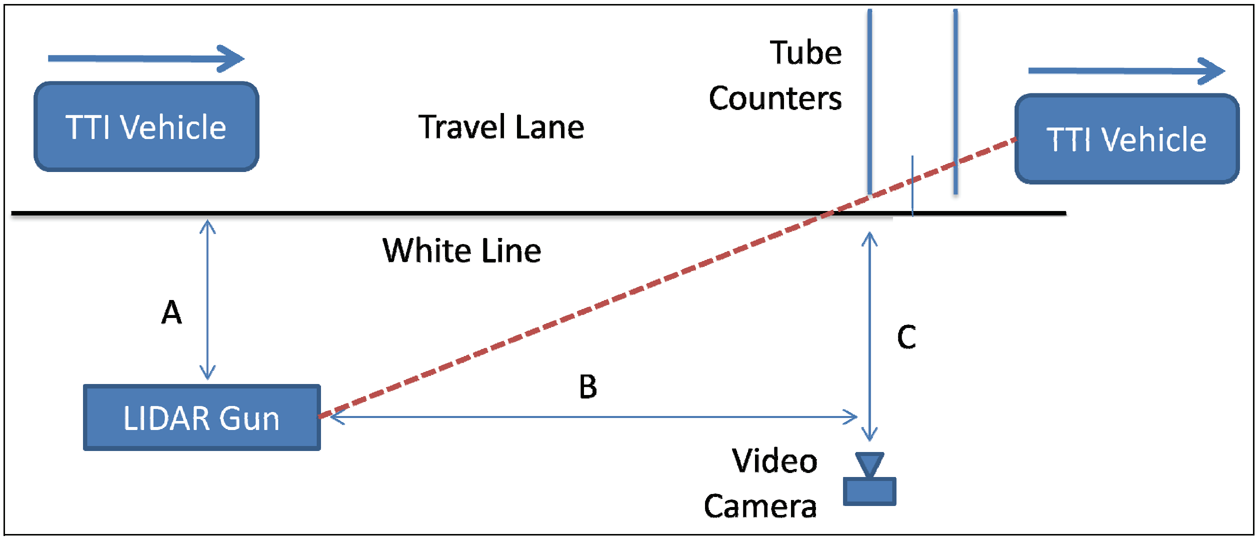

Figure 2. Typical Site Layout.

Table 2. Site Layout Distances.

Site No.	Distance (ft)		
	A	B	C
1	53	823	31
2	37	650	48
3	25	657	25
4	50	685	50

Overview of Devices

Four devices were used to gather data from the collection sites. The operation, parameters gathered, and other information regarding these devices are detailed in the following sections.

Tube Counters

Comparatively, tube counters are the most often employed for traffic data collection purposes due to their resilience, autonomy, and (for lower speeds) their accuracy. These counters are hollow rubber tubes that detect the air displaced when impacted. Depending on several factors, these counters then classify vehicles based on the number of and distance between axles. They also gather speed, count, and time of impact, and lane presence of the aforementioned vehicles.

As mentioned previously, at the sites the tubes were offset 16 ft and were stretched across both lanes of travel. These tubes were then taped down at approximately 6 ft increments, due to the need for tube exposure at the location of tire impact. This was also, in part, due to the time allowed for set up by the technician because of traffic on the interstate. A picture of the tube counters used can be seen in Figure 3.

Figure 3. Tube Counters.

LIDAR Gun

Unlike the other devices used to measure traffic data, the LIDAR gun is the only one that is manually operated. This means there can be more error associated with the operation of the device. To reduce this error, the device used was explained by a knowledgeable technician, and operation was practiced a reasonable amount of time alongside various locations of Texas Highway 6 in College Station, TX.

LIDAR stands for Light Detection and Ranging. The device works by emitting scattered light, which bounces off of the desired object, and bounces back to the device. The time elapsed during this process allows the LIDAR gun to determine speed and distance relative to its own location. The software used by the device also had a comment section for each reading which allowed for vehicle classification. A picture of the LIDAR gun used can be seen in Figure 4.

Video

Camcorders are readily available and used in the private sector and their basic operation is widely understood. These camcorders were mounted on tripods and set up to record vehicles as they impact tubes. Several measurements were then determined from the recorded footage, including classification, time of impact, and inter-vehicle gap time. The times are all accurate up to 1/30 of a second. A picture of the camcorder and tripod used can be seen in Figure 5.

Figure 4. LIDAR Gun.

Figure 5. Tripod Mounted Camcorder.

Control Vehicle

The TTI instrumented vehicle was used as the control vehicle. This particular vehicle is a Toyota Highlander that is outfitted with several extra features to accommodate the higher power drain due to the extra measurement devices on board. The main electronic component set up in the vehicle is the Dewetron system, used to integrate several different collection devices. Though several cameras and other devices are installed, the only device that is relevant to this project is the global positioning system (GPS). This device measures location and speed with the use of communication of the on-board devices, satellites, and communication towers. A picture of the control vehicle used can be seen in Figure 6.

Figure 6. Control Vehicle.

Collection Sites

Data for this study were collected at four open course sites near San Antonio, TX, along I-10. There was a need to get two sites, one with an 80 mph zone and another with a 70 mph zone. These sites needed to be relatively close to each other to prevent too much regional variance in driver behavior. Sites 1 and 2 are located outside of Junction, TX, within 80 mph Daytime Posted Speed Limit (DPSL) zones. Site 1 captured vehicle data along the westbound corridor, while Site 2 is captured along the eastbound corridor. Sites 3 and 4 are located outside of Kerrville, TX, along 70 mph DPSL zones. Site 3 captured vehicle data along the eastbound corridor, while Site 4 captured data along the westbound. Maps depicting the collection zones are shown in Figure 7, Figure 8, and Figure 9.

Figure 7. Sites 1 and 2, 80 mph Sites.

Figure 8. Sites 3 and 4, 70 mph Sites.

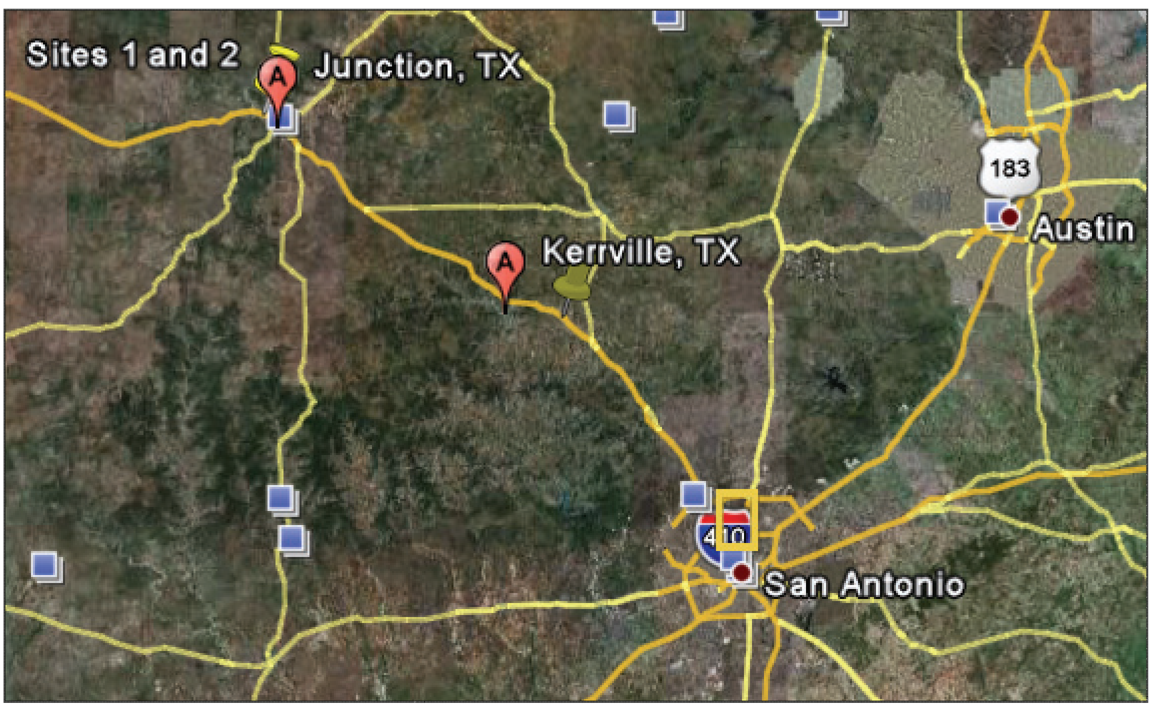

Figure 9. Collection Sites in Relation to Austin and San Antonio.

As can be seen in the figures, the distance from the collection site to the nearest entrance/exit ramps are great enough to allow for free flow speeds. At Sites 1 and 2, the distance is two miles in each direction. At Sites 3 and 4, the ramps are eight and a half miles to the west and three miles to the east. These distances should allow for most vehicles to be travelling at free flow speeds and not be slowing or increasing speed because of an interstate ramp.

Collection Method and Times

After the collection site and the control vehicle's instruments were set up, the control vehicle was to begin the circuit travel setting the cruise control for a known speed. The camcorder captured both the control vehicle as it struck the tubes, while the LIDAR gun was capturing vehicles as they passed the same location.

The video tapes could only collect 90 minutes of footage and therefore had to be changed at these intervals. The LIDAR gun and the control vehicle were operated at these same intervals of time. The times of data collected can be seen in Table 3 and Table 4.

Table 3. Times and Dates of Collected Data – 80 mph Sites.

Sets	Direction	Start Date	Start Time	End Date	End Time
Video	Westbound	6/10/2009	9:10:58 AM	6/10/2009	12:51:10 PM
	Eastbound	6/10/2009	2:46:39 PM	6/10/2009	5:44:43 PM
Tube	Westbound	6/9/2009	11:41:34 AM	6/11/2009	11:13:55 AM
	Eastbound	6/9/2009	12:27:48 PM	6/11/2009	11:22:56 AM
Lidar	Westbound	6/10/2009	9:15:05 AM	6/10/2009	12:50:50 AM
	Eastbound	6/10/2009	2:49:10 PM	6/10/2009	6:06:06 PM
TTI	Westbound	6/10/2009	9:23:56 AM	6/10/2009	6:04:13 PM
	Eastbound	6/10/2009	9:27:42 AM	6/10/2009	6:08:02 PM

Table 4. Times and Dates of Collected Data – 70 mph Sites.

Sets	Direction	Start Date	Start Time	End Date	End Time
Video	Westbound	6/11/2009	9:16:50 AM	6/11/2009	12:04:50 PM
	Eastbound	6/11/2009	2:12:53 PM	6/11/2009	5:32:58 PM
Tube	Westbound	6/9/2009	2:37:38 PM	6/11/2009	10:38:43 AM
	Eastbound	6/9/2009	2:14:51 PM	6/11/2009	11:16:28 AM
Lidar	Westbound	6/11/2009	9:17:08 AM	6/11/2009	12:01:15 PM
	Eastbound	6/11/2009	2:21:27 PM	6/11/2009	5:29:00 PM
TTI	Westbound	6/11/2009	9:24:26 AM	6/11/2009	5:29:27 PM
	Eastbound	6/11/2009	9:39:29 AM	6/11/2009	5:26:05 PM

Personnel

During the course of data collection, three employees were involved directly. One person was to operate the control vehicle, while another was to operate the LIDAR gun. The third person rotated into either of the two positions to prevent error associated with human fatigue associated with the several hours of collection.

APPROACH

Just as each of the devices was different in the variables they measured, they are also unique in how they output the information they collected. The LIDAR gun, tube counters, and the control vehicle all output their information into a workbook format. The video data had to be output manually. This involved classification based on the Federal Highway Association's (FHWA) criteria, which was consistent with the classification criteria the pneumatic tubes software employed. The criteria are included in Appendix B. Along with classification, the video coding allowed for the first and last axle's impacting the tube. This, in turn, allowed for the determination of the inter-vehicle gap in seconds. Again, the timing in the video allows for accuracy to 1/30 of a second. All of these parameters were recorded into a workbook format similar to those from the other devices. The volume of data reduced can be seen in Figure 10.

	Number of Vehicles					Hours of Data Reduced				
	80 mph		70 mph			80 mph		70 mph		
Sets	WB	EB	WB	EB	**Total**	WB	EB	WB	EB	**Total**
Video	847	766		762	**2375**	3.61	2.89		1.83	**8.34**
Tube	801	561	1216	1094	**3672**	3.67	2.98	2.76	2.85	**12.26**
Lidar	371	304	359	384	**1418**	3.43	2.83	2.73	2.85	**11.85**
TTI	24	20	8	8	**60**	3.61	2.89	2.41	2.63	**11.54**

Figure 10. Number Reduced (Left), Hours of Reduced (Right).

After this was achieved, the different device data were combined into a single workbook. An example can be seen in Figure 11. As a single vehicle passed over the collection location, it was captured by the video, the LIDAR gun, the pneumatic tubes, and the control vehicle (if applicable). The datasets were lined up with values in each row being the same captured vehicle to allow for comparison. After combining the datasets, the differences between devices were found regarding speed. These differences were simply comparing two devices against each other at a time, using an absolute difference in some cases and an actual (+/-) difference in others. Once having these differences, they were arranged in a way allowing for the creation of cumulative distribution plots to facilitate analysis, comparing these differences against several factors, but looked mostly at the differences versus the Daytime Posted Speed Limit.

V-Front Time	V-Rear Time	V-Gap	T-Axles	T-Class	T-Time	T-Speed	T-Gap	L-Time	L-Description	L-Speed	C-Time	C-Speed
9:26:43.10 AM	9:26:43.20 AM	9.67	2	2	9:26:32 AM	76.7	9.73					
9:28:23.70 AM	9:28:24.07 AM	100.50	3	8	9:28:18 AM	71.6	105.5					
9:28:26.23 AM	9:28:26.30 AM	2.17	2	2	9:28:21 AM	71.2	2.22	9:28:28 AM	blk car	71		
9:28:31.93 AM	9:28:32.50 AM	5.63	2	3	9:28:48 AM	75.6	27.16					
9:28:53.43 AM	9:28:53.53 AM	20.93			9:28:48 AM			9:28:55 AM	red suv	76		
9:29:12.60 AM	9:29:12.70 AM	19.07	2	2	9:29:07 AM	80.5	19.04	9:29:14 AM	tti	80	9:31:52 AM	81.088783
9:29:14.77 AM	9:29:15.37 AM	2.07	4	8	9:29:09 AM	63.4	2.06					
9:29:21.13 AM	9:29:21.23 AM	5.77	2	2	9:29:15 AM	70.3	5.81					
9:29:50.40 AM	9:29:50.50 AM	29.17	2	2	9:29:45 AM	63.6	29.21	9:29:52 AM	wht car	63		
9:30:16.43 AM	9:30:16.57 AM	25.93	2	2	9:30:11 AM	57.0	25.91	9:30:18 AM	drk car	57		
9:30:42.57 AM	9:30:42.77 AM	26.00	2	4	9:30:37 AM	69.0	25.04	9:30:45 AM	:ving truck (sei	70		
9:30:44.93 AM	9:30:45.10 AM	2.17	2	5	9:30:39 AM	70.3	2.17					
9:31:06.87 AM	9:31:07.53 AM	21.77	4	8	9:31:01 AM	60.5	21.77					
9:31:22.77 AM	9:31:22.87 AM	15.23	2	2	9:31:17 AM	77.5	15.33	9:31:25 AM	tan trk	78		
9:31:27.63 AM	9:31:27.77 AM	4.77	2	5	9:31:22 AM	70.3	4.78					
9:31:41.27 AM	9:31:41.87 AM	13.50	2	5	9:31:36 AM	63.1	13.5	9:31:44 AM	semi	64		
9:32:13.57 AM	9:32:14.17 AM	31.70	2	5	9:32:08 AM	66.5	32.08	9:32:16 AM	semi	67		
9:32:35.30 AM	9:32:35.87 AM	21.13	4	8	9:32:30 AM	62.3	21.57					
9:33:07.27 AM	9:33:07.40 AM	31.40	2	5	9:33:02 AM	81.3	31.43	9:33:09 AM	blk trk	80		
9:33:50.03 AM	9:33:50.13 AM	42.63	2	2	9:33:44 AM	80.9	42.7	9:33:52 AM	drk suv	81		
					9:33:44 AM							
9:34:58.90 AM	9:34:59.00 AM	610.00	2	2	9:34:28 AM	76.3	610					
					9:34:28 AM							
9:39:25.20 AM	9:39:25.27 AM	266.20	2	2	9:34:43 AM	65.6	15.23					
					9:34:43 AM							
					9:34:43 AM							
9:34:49.60 AM	9:34:49.97 AM	59.47	2	4	9:35:09 AM	66.5	84.02	9:34:52 AM	trk w/ trailer	61		
9:35:14.20 AM	9:35:14.67 AM	24.23	2	3	9:35:09 AM	65.9	0.34	9:35:17 AM	semi	66		
9:35:18.30 AM	9:35:18.90 AM	3.63	2	3	9:35:31 AM	83.2	22.22					
9:35:37.03 AM	9:35:37.13 AM	18.13			9:35:31 AM			9:35:39 AM	drk suv	84		

Figure 11. Combined Dataset Example.

RESULTS

The speed difference between the LIDAR gun and the control vehicle (TTI) is shown with the open and closed circles in Figure 12. Higher differences are associated with the 80 mph segments than the 70 mph segments. In fact, in the 70 mph zones, none of the values are over a 1.5 mph difference. At 80 mph, 10 percent of the data are over this 1.5 mph threshold (see curve with open circles).

The tube counter and control vehicle (TTI) difference are shown with open and closed squares in Figure 12. Higher differences are again associated with the 80 mph segments, than the 70 mph segments. In the 70 mph zones, 25 percent of the data are over a 1.5 mph difference. Contrasting that with the 80 mph zone, we see that 75 percent of the data are over the 1.5 mph difference.

The difference between tube counter and the LIDAR gun is shown in Figure 13. The pattern of increased device difference with the higher speed continues. At 70 mph, 5 percent of the data are over a 5 mph difference while, at 80 mph, 15 percent of the data are over a 5 mph difference.

In addition to examining the absolute difference, the actual (+/-) difference between devices was determined. If the differences are predictable and consistent, then correction could be performed by a constant correction factor. The actual differences were set up in a fashion consistent with the other plots, on a cumulative distribution plot. Considering the plot in Figure 14, the differences appear to be random. This prevents correction by a constant factor.

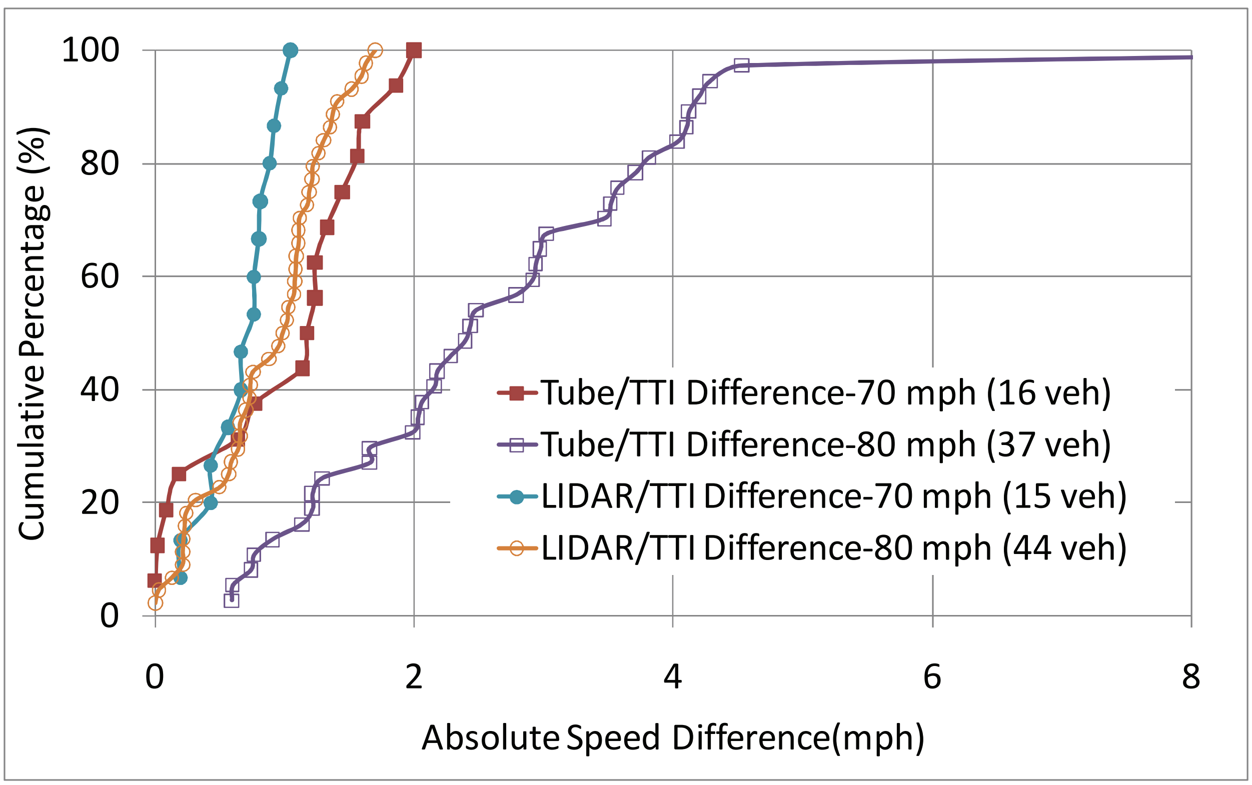

Figure 12. Device Difference with the Control Vehicle.

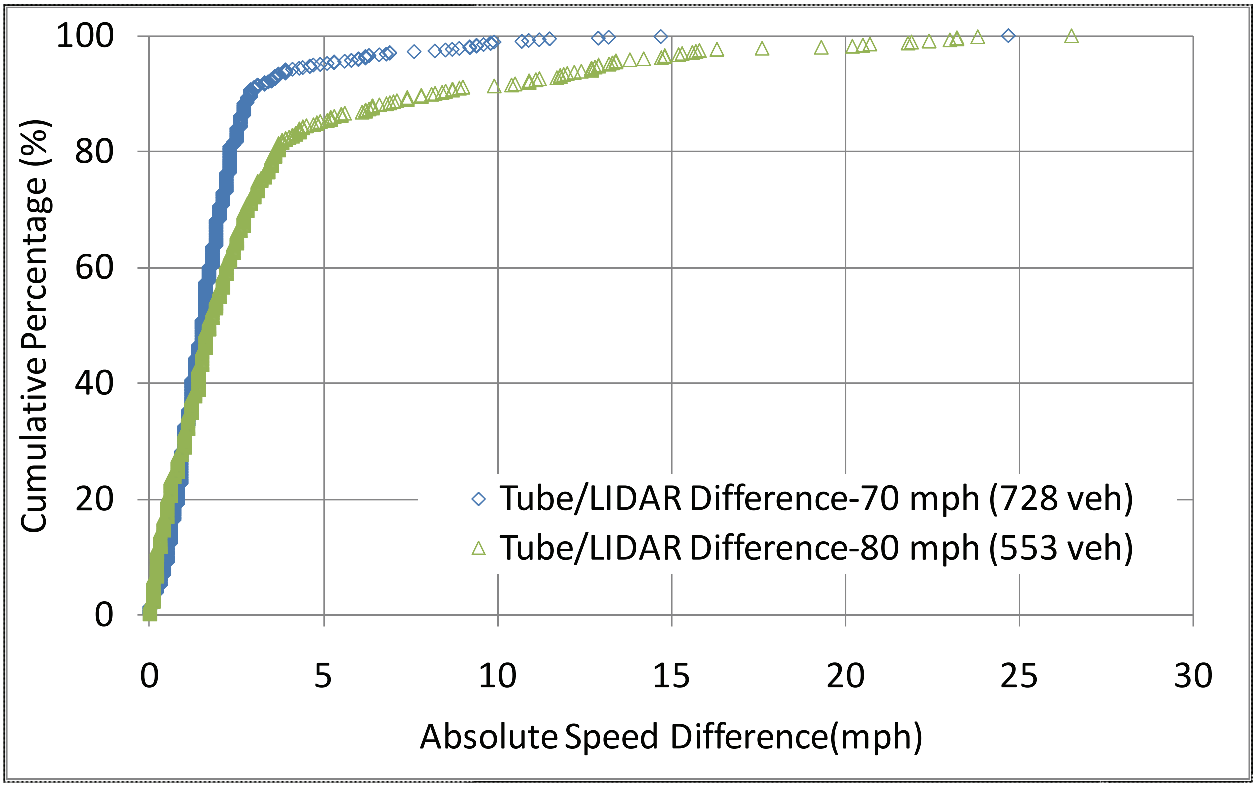

Figure 13. Device Difference, Pneumatic Tubes vs. LIDAR.

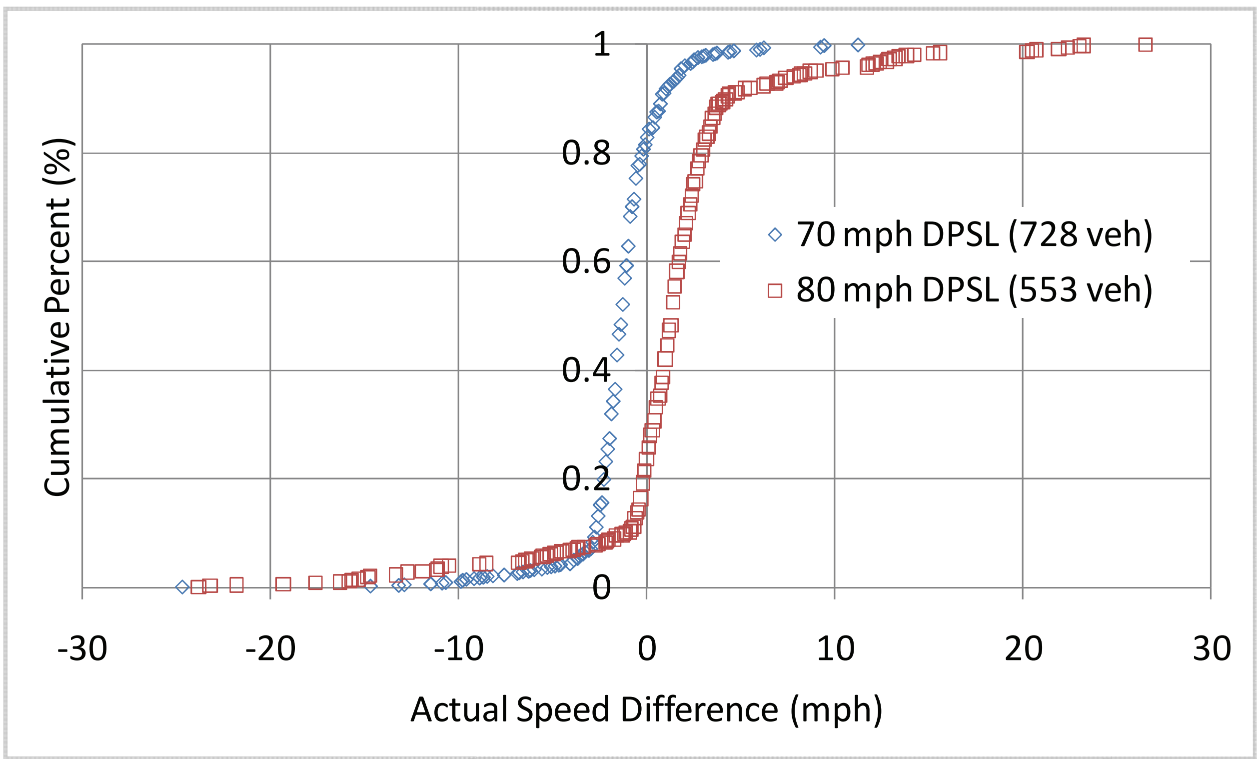

Figure 14. Actual Device Difference.

PRELIMINARY CONCLUSIONS

As can be seen in the datasets, concerns exist with using tubes to collect data at high speeds. The higher speed DOES seem to affect the accuracy of the traffic data collection devices. Though the cause of this needs to be determined, the increase in device differences as speed increases is evident.

POST RESEARCH

Following this work, several tasks are recommended to be carried out. First, the cause of the tube counters higher differences at higher speeds should be determined. This device is used quite often in traffic data collection for their resilience and their ease of use. If they are to be employed at higher speeds, the cause for this error should be eliminated or reduced.

Along with the cause for the tube error, the device difference of gap and classification need to be determined. These devices are commonly used not solely for speeds of vehicles at a location, but also for classification and several other parameters. It is valuable to know whether they are accurate in these measurements as well.

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APPENDIX A

The development of these tables was based on the use of similar triangles of distance of the site area and speed of the vehicles. Knowing the LIDAR gun's accuracy was to 1 mph, we wanted the speed error to be less than half a mph due to the rounding of values by the device. The distance gathered from the LIDAR gun is actually the hypotenuse of the right triangle. With trigonometry, the "B" distance can be found using our known distance from the lane of travel and the hypotenuse (value given by the LIDAR gun). If we set up a similar triangle using speed as the sides of the right triangle we can use the trigonometric properties of these similar triangles to find the difference in actual speed compared to the hypotenuse speed the LIDAR gun yields. If this value is less than a half mph, then the table outputs an "okay." If the value is over, then the table outputs "adjust." This table was developed, as mentioned previously, to ensure the reading would be reasonably accurate according to the site layout.

Adjustment for 80 mph Zone

Speed Limit	80	Does the Lidar Gun Reading Need to Be Adjusted								
From White Line (ft)	Distance From Lidar Gun									
	300	350	400	450	500	550	600	650	700	750
5	Ok	Ok	Ok	Ok	Ok	Ok	Ok	Ok	Ok	Ok
10	Ok	Ok	Ok	Ok	Ok	Ok	Ok	Ok	Ok	Ok
15	Ok	Ok	Ok	Ok	Ok	Ok	Ok	Ok	Ok	Ok
20	Ok	Ok	Ok	Ok	Ok	Ok	Ok	Ok	Ok	Ok
25	Ok	Ok	Ok	Ok	Ok	Ok	Ok	Ok	Ok	Ok
30	Ok	Ok	Ok	Ok	Ok	Ok	Ok	Ok	Ok	Ok
35	Adjust	Ok	Ok	Ok	Ok	Ok	Ok	Ok	Ok	Ok
40	Adjust	Adjust	Ok	Ok	Ok	Ok	Ok	Ok	Ok	Ok
45	Adjust	Adjust	Adjust	Ok	Ok	Ok	Ok	Ok	Ok	Ok
50	Adjust	Adjust	Adjust	Ok	Ok	Ok	Ok	Ok	Ok	Ok
55	Adjust	Adjust	Adjust	Adjust	Ok	Ok	Ok	Ok	Ok	Ok
60	Adjust	Adjust	Adjust	Adjust	Adjust	Ok	Ok	Ok	Ok	Ok
65	Adjust	Adjust	Adjust	Adjust	Adjust	Adjust	Ok	Ok	Ok	Ok
70	Adjust	Adjust	Adjust	Adjust	Adjust	Adjust	Adjust	Ok	Ok	Ok
75	Adjust	Adjust	Adjust	Adjust	Adjust	Adjust	Adjust	Adjust	Ok	Ok
80	Adjust	Adjust	Adjust	Adjust	Adjust	Adjust	Adjust	Adjust	Adjust	Ok
85	Adjust	Adjust	Adjust	Adjust	Adjust	Adjust	Adjust	Adjust	Adjust	Adjust
90	Adjust	Adjust	Adjust	Adjust	Adjust	Adjust	Adjust	Adjust	Adjust	Adjust
95	Adjust	Adjust	Adjust	Adjust	Adjust	Adjust	Adjust	Adjust	Adjust	Adjust
100	Adjust	Adjust	Adjust	Adjust	Adjust	Adjust	Adjust	Adjust	Adjust	Adjust

Adjustment for 70 mph Zone

Speed Limit	70	Does the Lidar Gun Reading Need to Be Adjusted								
From White Line (ft)	Distance From Lidar Gun									
	300	350	400	450	500	550	600	650	700	750
5	Ok	Ok	Ok	Ok	Ok	Ok	Ok	Ok	Ok	Ok
10	Ok	Ok	Ok	Ok	Ok	Ok	Ok	Ok	Ok	Ok
15	Ok	Ok	Ok	Ok	Ok	Ok	Ok	Ok	Ok	Ok
20	Ok	Ok	Ok	Ok	Ok	Ok	Ok	Ok	Ok	Ok
25	Ok	Ok	Ok	Ok	Ok	Ok	Ok	Ok	Ok	Ok
30	Ok	Ok	Ok	Ok	Ok	Ok	Ok	Ok	Ok	Ok
35	Ok	Ok	Ok	Ok	Ok	Ok	Ok	Ok	Ok	Ok
40	Adjust	Ok	Ok	Ok	Ok	Ok	Ok	Ok	Ok	Ok
45	Adjust	Adjust	Ok	Ok	Ok	Ok	Ok	Ok	Ok	Ok
50	Adjust	Adjust	Adjust	Ok	Ok	Ok	Ok	Ok	Ok	Ok
55	Adjust	Adjust	Adjust	Adjust	Ok	Ok	Ok	Ok	Ok	Ok
60	Adjust	Adjust	Adjust	Adjust	Adjust	Ok	Ok	Ok	Ok	Ok
65	Adjust	Adjust	Adjust	Adjust	Adjust	Ok	Ok	Ok	Ok	Ok
70	Adjust	Adjust	Adjust	Adjust	Adjust	Adjust	Ok	Ok	Ok	Ok
75	Adjust	Adjust	Adjust	Adjust	Adjust	Adjust	Adjust	Ok	Ok	Ok
80	Adjust	Adjust	Adjust	Adjust	Adjust	Adjust	Adjust	Adjust	Ok	Ok
85	Adjust	Adjust	Adjust	Adjust	Adjust	Adjust	Adjust	Adjust	Adjust	Ok
90	Adjust	Adjust	Adjust	Adjust	Adjust	Adjust	Adjust	Adjust	Adjust	Adjust
95	Adjust	Adjust	Adjust	Adjust	Adjust	Adjust	Adjust	Adjust	Adjust	Adjust
100	Adjust	Adjust	Adjust	Adjust	Adjust	Adjust	Adjust	Adjust	Adjust	Adjust

APPENDIX B

Figure 1

FHWA VEHICLE CLASSIFICATION

CLASS GROUP	DESCRIPTION	NO. OF AXLES
1	MOTORCYCLES	2
2	ALL CARS CARS CARS W/ 1-AXLE TRAILER CARS W/ 2-AXLE TRAILER	2 3 4
3	PICK-UPS & VANS 1 & 2 AXLE TRAILERS	2, 3, & 4
4	BUSES	2 & 3
5	2-AXLE, SINGLE UNIT	2
6	3-AXLE, SINGLE UNIT	3
7	4-AXLE, SINGLE UNIT	4
8	2-AXLE, TRACTOR, 1-AXLE TRAILER (2&1) 2-AXLE, TRACTOR, 2-AXLE TRAILER (2&2) 3-AXLE, TRACTOR, 1-AXLE TRAILER (3&1)	3 4 4
9	3-AXLE, TRACTOR, 2-AXLE TRAILER (3&2) 3-AXLE, TRUCK W/ 2-AXLE TRAILER	5 5
10	TRACTOR W/ SINGLE TRAILER	6 & 7
11	5-AXLE MULTI-TRAILER	5
12	6-AXLE MULTI-TRAILER	6
13	ANY 7 OR MORE AXLE	7 or more
14	NOT USED	
15	UNKNOWN VEHICLE TYPE	

HEAVY TRUCKS

Figure 15. FHWA Vehicle Classification *(4)*.

Measuring Traveler's Willingness-to-Pay for Time Savings

Prepared for
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George Bogonko is currently pursuing a Bachelor of Science degree in Civil Engineering at California Polytechnic University. He is expected to graduate in December 2009 and is considering an option of attending graduate school. George received an Associate Degree in Architecture from Pasadena City College in 2005.

George is a member and co-founder of Engineers without Borders (EWB) California Polytechnic University. He is also a member of the American Society of Engineers and the Institute of Transportation Engineers. George has previous internship experience with a structural engineering firm in Los Angeles. His career interest is in public policy and urban planning and he hopes to work with international agencies, such as the World Bank, IMF, etc. to formulate policy for urban planning in developing countries.

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SUMMARY

One of the major factors that influence mode and lane choice is the characteristics of drivers, including their willingness to pay for travel time savings. It is important for transportation engineers to understand the willingness to pay for travel time savings and the processes drivers use to determine their value of time in order to effectively manage transportation facilities. In recent years, conversion of high occupancy vehicle (HOV) lanes to high occupancy/toll (HOT) lanes has emerged as one method to more efficiently utilize transportation infrastructure. HOT lanes provide reliable travel time for its users, resulting in travelers willing to pay a toll to use those lanes.

The purpose of this research is to determine the willingness to pay for travel time savings for travelers on the I-394 HOT lanes in Minnesota. This report summarizes the findings of the value of travel time savings for each traveler on the HOT lane in 2008. Preliminary findings indicate

very little travel time savings. However, many travelers have shown a willingness to pay toll to obtain these minimal savings. This would indicate that additional factors, other than just travel time savings, are influencing these drivers to pay to use the HOT lane.

Results and recommendations from this study will benefit future HOV to HOT conversion projects by providing a deeper understanding on traveler's value of travel time savings. More studies need to be done on other factors that influence the use of HOT lanes, such as the income, geometric design, and characteristics of the HOT lanes.

TABLE OF CONTENTS

INTRODUCTION

Transportation facilities are faced with growing challenges of congestion and a limited ability to expand freeway capacity due to construction costs, right-of-way constraints, and environmental and societal impacts. Transportation agencies have tried to solve these challenges through limited capacity expansion, focused planning, and operational strategies to curb congestion.

Transportation engineers and planners often rely on the traveler's value of travel time savings (VTTS) when deciding on alternatives for a transportation facility. In many cases a traveler's VTTS is dependent on two main characteristics: trip characteristics and personal characteristics. Trip characteristics involve the mode of travel, cost of travel, travel time, and the route to name a few. Personal characteristics include the traveler's income level, race, and education level among many others.

Understanding the value of time and the processes drivers apply to determine their value of time allows engineers to choose which operation strategies to implement on different transportation facilities. One of the newest innovative operation strategies being used to reduce traffic congestion involves conversion of high occupancy vehicle (HOV) lanes to high occupancy/toll (HOT) lanes. HOT lanes provide more options to travelers and help to reduce travel time. HOT lanes operate alongside existing highway lanes to allow users easy access. Buses, carpoolers, motorcycles, and emergency vehicles will have free access to HOT lanes. Single occupancy vehicles (SOV) may use the lanes by paying a toll. This toll is modified to maintain a high level of service on the HOT lanes at all times.

Extensive studies are being conducted to evaluate the effectiveness and efficiency of this strategy. The studies have also revealed several aspects that engineers need to resolve in order to fully understand this new mode choice. One of those aspects is traveler's willingness-to-pay for travel time savings on HOT lanes.

BACKGROUND INFORMATION

This research, on measuring traveler's willingness-to-pay for travel time savings, is part of a research project being conducted by the Texas Transportation Institute for the Federal Highway Administration entitled "Tools for HOV to HOT Benefit Analysis." The TTI project involves review and collection of data regarding the impacts of implemented HOV to HOT lane conversion projects such as I-15 in San Diego, Katy and Northwest Freeways in Houston, SR-91 Express lanes in Los Angeles, SR-167 in Seattle, I-25/US 36 in Denver, I-394 in Minnesota, I-15 in Utah, and I-95 in Miami.

The TTI project includes a review of the literature and data collection from implemented projects, and literature review of theoretical impacts of HOV to HOT lane conversion. TTI researchers collected data on how different characteristics have impacted HOT lane usage. Some of the characteristics that influence mode and lane choices include geometric design, characteristics of alternative modes and routes, characteristics of HOT lanes, and characteristics of drivers on HOT lanes. The research described in this paper will focus on one of the main characteristics of drivers that chose the HOT lanes: the value of travel time savings and how those values vary over the traveling public.

Goals and Objectives

The goal of this research is to improve our understanding of driver behavior in HOT lane corridors. The specific objective is to estimate traveler's value of travel time savings including the distribution of these values over the driving population.

Study Location

The study location for this research is I-394 HOT lanes in Minnesota.

Background Information

I-394 is an east-west highway in Hennepin County in the state of Minnesota. It runs for 11 miles (15.8 km) from downtown Minneapolis to the junction of I-494 in the Minneapolis suburb of Minnetonka (see Figure 1).

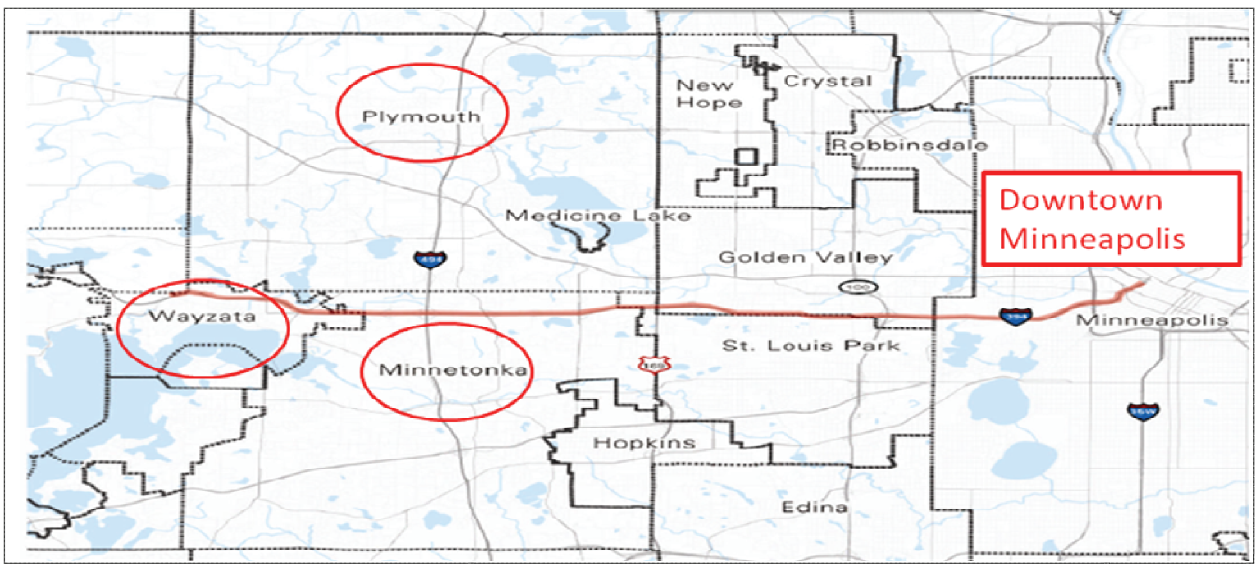

Figure 1. Highway System (I-394).

MnPASS was constructed after the Minnesota State Legislature passed a bill mandating Mn/DOT to conduct a study on the impact of converting HOV lanes on I-394 to HOT lanes and if the conversion would impact the traffic flow and safety on GPLs. The result of the study revealed that based on national averages, the HOV lanes were not operating at their full capacity even though the HOV lanes were moving more people per lane than the general-purpose lanes during peak period. For example, in the 2nd quarter of 2001, the HOV report showed that on eastbound I-394 at Louisiana Ave. during the 7:00 to 8:00 a.m. hour, the HOV lane moved 3,053 people per lane and the general-purpose lanes carried 2076 people per lane. However, there was still space on the HOV lanes for additional vehicles.

Minnesota Managed Lanes

To increase the capacity of the HOV lanes, Mn/DOT embarked a project to convert the I-394 HOV lanes to HOT lanes. The project was authorized by the Minnesota Legislature in 2003, and in 2005 MnPASS became the first managed lanes in Minnesota. MnPASS was developed and completed through a public/private partnership involving the state of Minnesota and service vendor Wilbur Smith Associates. The private firm funded 20 percent of the project's estimated $10 million price tag. Currently the state of Minnesota has two HOV facilities: the East-West facility that runs along I-394 and the North-South facility along I-35 West.

I-394 MnPASS Express Lane contains an 11-mile stretch of carpool lanes between the western Metropolitan area in downtown Minneapolis and the western suburbs (Wayzata area) (see Figure 2 and Figure 3). SOV pay to use the MnPASS lanes while carpoolers, bus riders, emergency vehicles, and motorcyclists use the lanes free of charge. Dynamic pricing ensures continuous free flow by adjusting the toll up or down depending upon the amount of traffic in the lanes. The cost of toll depends on where you enter or exit the MnPASS Express lanes and the volume of vehicles in the toll lanes. The toll is posted on electronic signs located just upstream of entrances to MnPASS lanes. The tolls range from $0.25 cents to $8 and average $1 to $4 during rush hour to ensure free flow traffic on the express lanes.

The layout of I-394 is as follows:

- 4 lanes (2 eastbound, 2 westbound) and 2 HOT lanes (one westbound and one eastbound) from I-494 to US 169,
- 5 lanes (2 eastbound, 3 westbound) and 2 HOT lanes (one westbound and one eastbound)from US 169 to just west of MN 100, and
- 4 lanes and 2 HOT lanes (reversible) from Highway 100 to I-94.

The operation time for the diamond lanes is as follows:

- The non-reversible section is operated Monday through Friday from 2 p.m. to 7 p.m. for westbound and from 6 a.m. to 10 a.m. eastbound. The lanes are open to general traffic the rest of the day and on weekends.
- The reversible diamond section is operated Monday to Friday from 6 a.m. to 1 p.m. eastbound and 2 p.m. to 5 a.m. westbound; between 1 p.m. to 2 p.m. and 5 a.m. to 6 a.m. the lanes are closed to change direction.

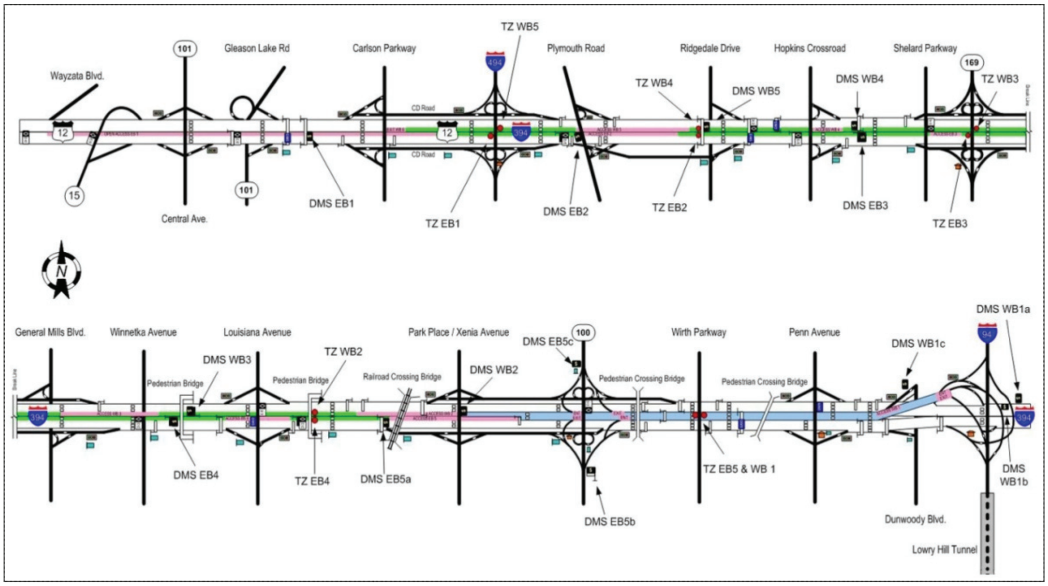

Figure 2. HOT Layout.

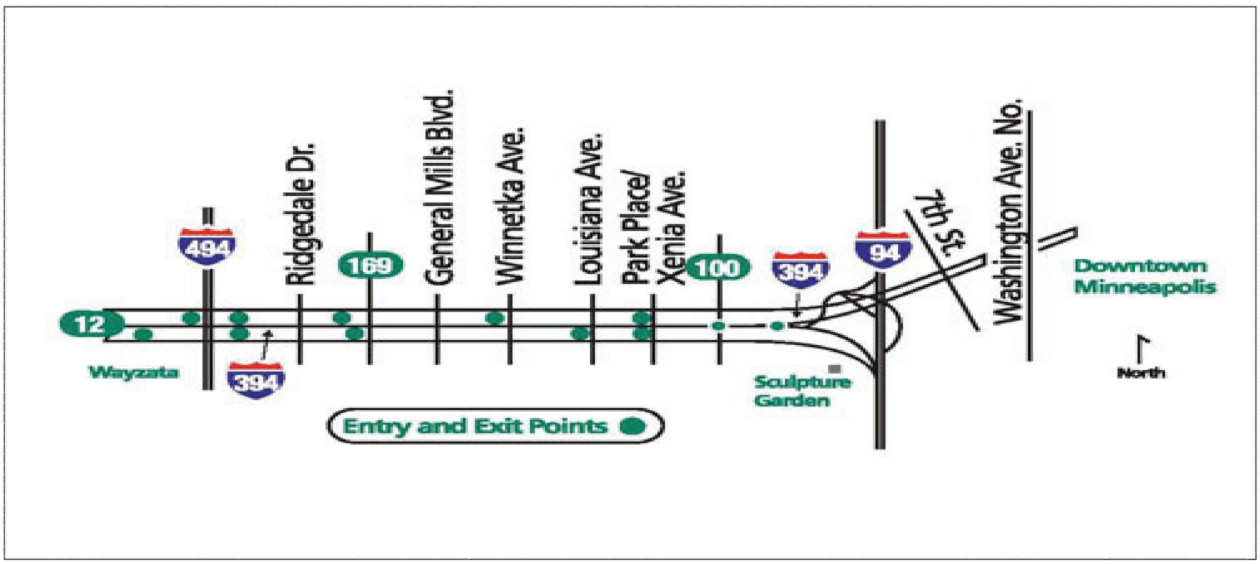

Figure 3. Entry and Exit Points.

RESEARCH TASKS

Four main tasks were performed to achieve the goals of the research project. The first task involved reviewing the parent research project, articles, journals, and other publications on the value of time savings. Task 2 encompassed collecting all the data necessary to complete the research, while task 3 and task 4 included data reduction, analysis, and documenting the findings of the research.

Literature Review

A study conducted in 1999 on SR-91 by Edward Sullivan of Cal Poly State University at San Luis Obispo reveals that the primary reason for using the express lanes is travel time savings. One-third of users gave other reasons such as driving comfort and safety. These two reasons were cited primarily by many of the drivers who pay to use the lane during off-peak periods. About 58 percent of express lane users felt express lanes were safer than the free lanes, while 14 percent felt they were less safe. The 1999 study data also showed that the likelihood to use the express lanes increased significantly with income. Approximately 20 percent of those in the under $40,000 annual income category used the express lanes, compared to 25 percent in the $40,000 to $60,000 category; 40 percent in the $60,000 to $100,000; and 50 percent in the $100,000+ category. A significant drop occurred in usage from 40 percent to 25 percent by the $40,000 to $60,000 group between 1996 and 1999 (*1*).

Another study on I-394 travelers showed a significant increase in the willingness-to-pay a toll for individuals who earn more than $100,000 per year. Younger travelers have higher VOT than older travelers. The value of time also varies depending on the time of the day a trip is made; morning commuters were more willing to pay for time saving compared to afternoon commuters (*2*).

A study conducted by Gunn (1991) in the Netherlands showed that for business travelers and commuters, congestion increases the willingness to pay for travel-time reduction (*3*). Guttman's report in 1979 estimated the value of time during peak hours is $5.17 per hour as compared to a values of $1.97 per hour in off peak time. More recent reviews have suggested that the value of time for work trips is about 50 percent of the wage rate on average (Small, 1992; Waters, 1992) and that varies with income and wage rates but not proportionally (*3*).

Data Collection

For this research we obtained detector data and toll data.

Site Selection

The first step in accomplishing the goal and objective of this research was to identify the area on the HOT lane to measure the amount of travel time savings offered by the HOT lanes over GPLs. The site for this research was determined using the toll data provide by the MnPASS management company. This research will therefore focus on a 6.5-mile stretch between I-494 and Wirth Parkway. The eastbound stations are labeled beginning from 1001 just before I-494 to 1005 at Wirth parkway, whereas westbound stations are labeled beginning from 2001 at Wirth Parkway to 2005 just past I-494 (see Figure 4). Each station indicates the location of transponder sensors, which are used to automatically charge SOV travelers the proper toll.

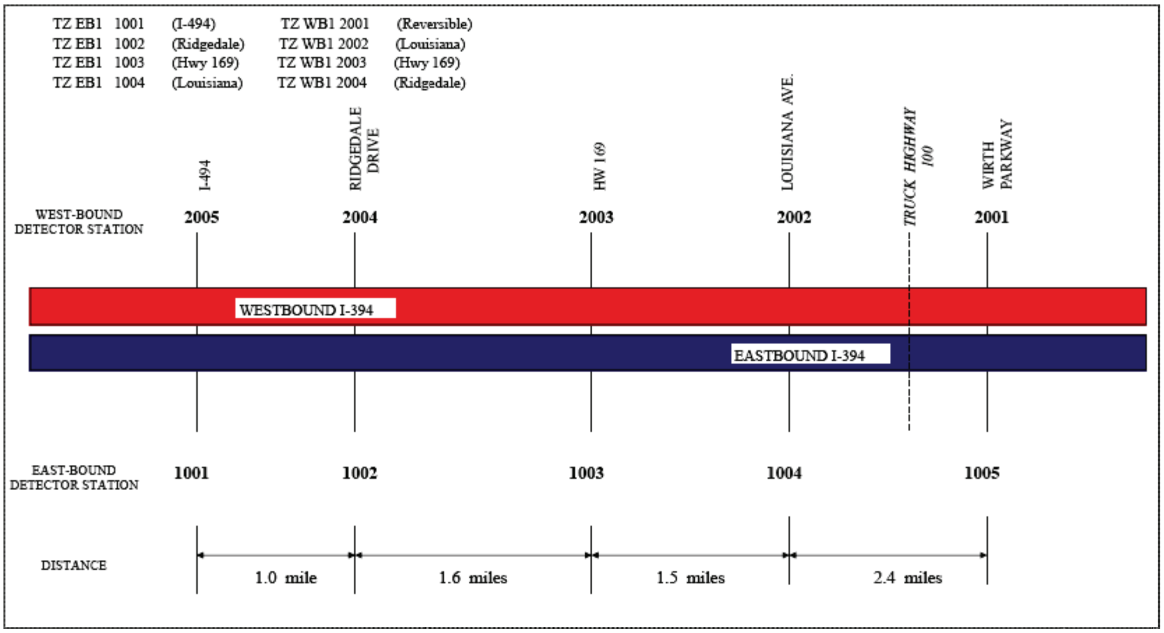

Figure 4. Transponder Sensor Stations.

Loop Detector Data

After the study site was determined, the researcher examined the overall layout and the methods used to collect data at the site. In the case of I-394 lanes, loop detectors were used to collect traffic data. The loop detector data is the property of the Minnesota Department of Transportation. The data are available to the public through the Minnesota Department of Transportation Mn/DOT website (www.dot.state.mn.us/tmc/trafficinfo/developers.html).

There are two types of data available in the website: detector and incident data. Detector data provide the public with real-time detector data in XML format data, which is updated every 30 seconds. The XML files contain volume, occupancy, speed, and flow data for each detector in the Twin Cities Metro area. The incident data are also in XML format and are updated every 30 seconds. It contains road construction information, road conditions, and vehicle crashes.

The "All Detector Report" can be used for locating detectors and stations on all Minnesota highways. When you access the report, it automatically downloads a portable document format (PDF) file onto your computer, which shows the layout of all detectors along the freeway. Numbering of the detectors starts from the right lane to the left lane for all other detectors and the 5000 series detectors refer to HOT detectors. For example in Figure 5, 517 refers to the mainline detector station, 1858 it the outermost right lane detector, 1859 is the outermost left lane detector, and 5675 is the HOT lane detector. Using this procedure, all mainline GPL and HOT lane detectors located in the study section was obtained. Figure 6 and Figure 7 show the GPL and HOT detectors locations in the study sections.

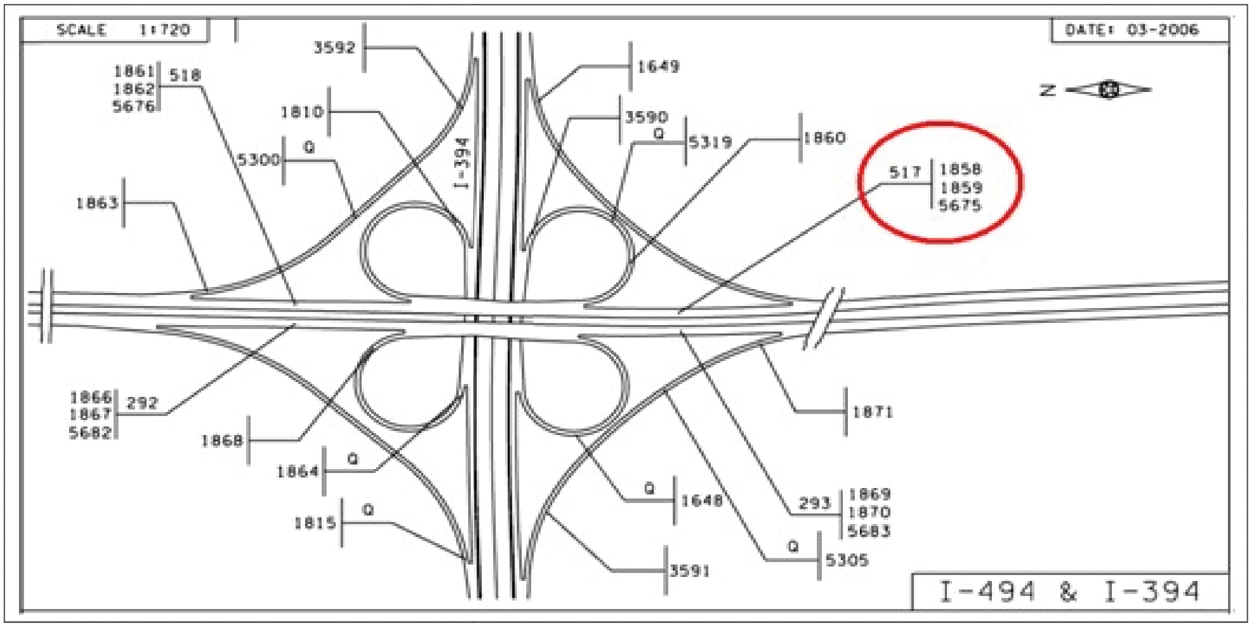

Figure 5. Sample LDS layout (See Appendix A for All Detectors).

The total number detectors on the GPLs is 36 westbound and 38 eastbound. There are 10 detectors on each non-reversible HOT lane and 6 detectors in the reversible section.

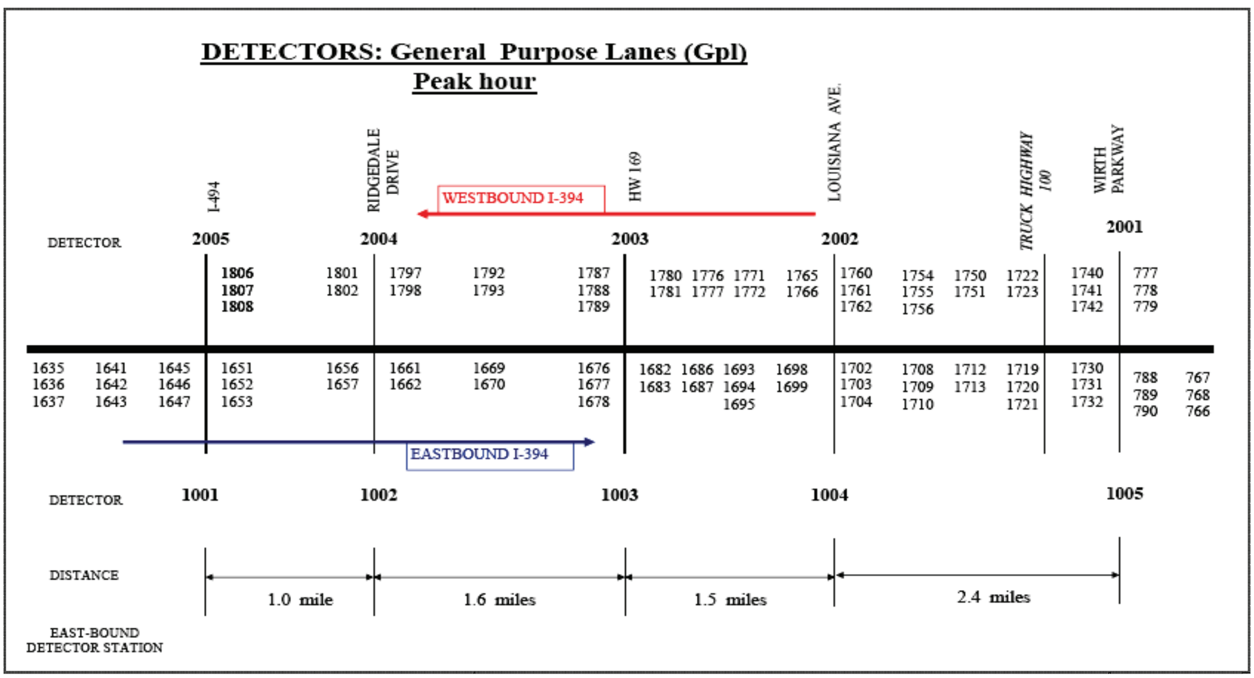

Figure 6. East/Westbound GPL Layout.

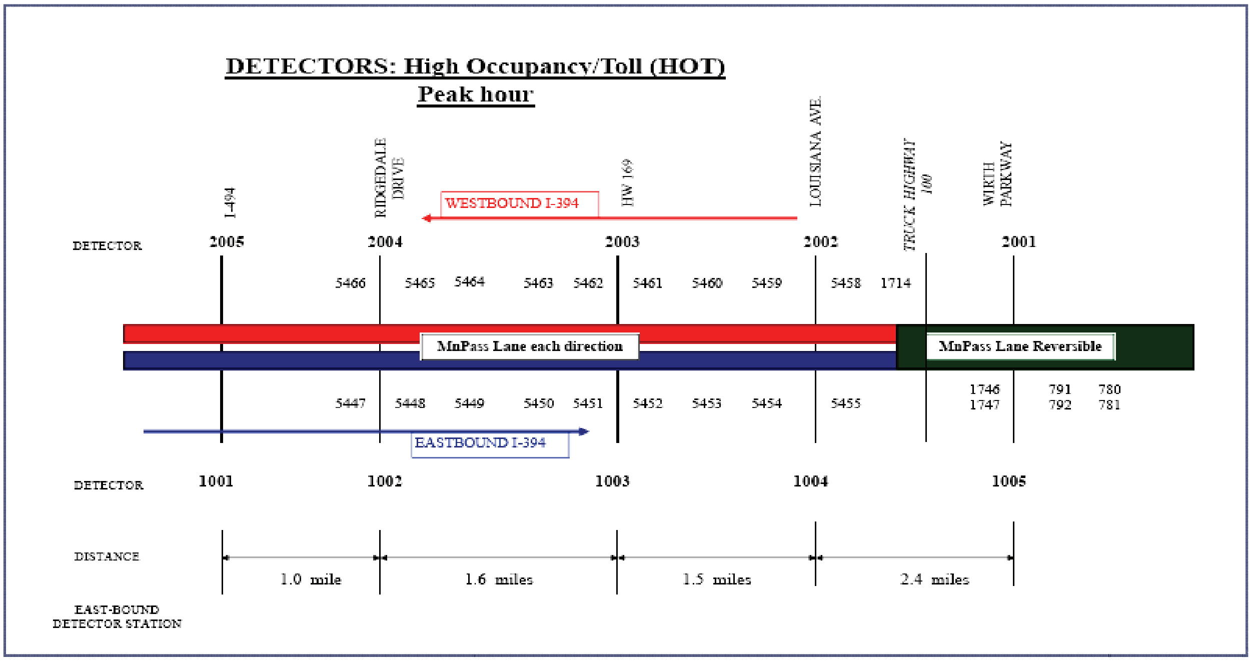

Figure 7. East/Westbound HOT Detectors.

Archived XML volume, occupancy, speed, and flow data for each of the detectors can be found in the resource section on the website via DataExtract and DataPlot tool. DataPlot is a tool for graphing detector data, and DataExtract is a tool for extracting detector data to a comma separated value (csv) file for analysis. To run DataPlot or DataExtract, simply click the link provided.

Procedure for Extracting Data

The steps below document the procedure that should be followed to extract detector data (see Figure 8).

1. Enter the Detector numbers.
2. Select the range of days to extract data.
3. Select type of data Matrix desire (Volume, Speed, Flow, capacity etc.).
4. Select the time of day, and the time interval for the data.
5. Select how the data should be presented (Average, Median, Values, etc.).
6. Create a folder to store data.
7. Click on file, click on extract files.

A sample of a data extract tool is shown in Figure 8; the number D5447 refers to the detector number, the dates selected are July 6 to July 11, and in this case the type of data matrix to extract is the speeds measured by detector 5447. The time of the day required is 6:00 a.m. to 10:00 a.m., and the data will be for 5-minute intervals. In this case actual values of speed will be extracted and store in the file path C:\Documents and Settings\gob9795\Desktop. Figure 8 shows the output values and format for detector 5447. For example, the speed on the lane with detector 5447 on July 9 at 6:35 a.m. is 72.3 mph (see Figure 9).

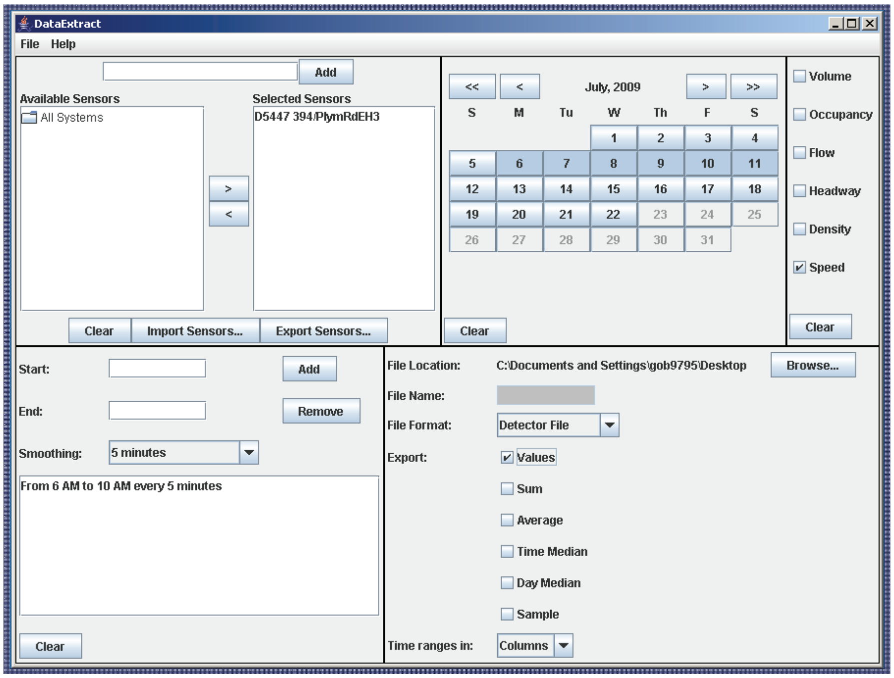

Figure 8. Data Extract Tool.

Detector #	D5447	D5447	D5447	D5447	D5447	D5447
Data Type	Speed	Speed	Speed	Speed	Speed	Speed
Time\Date	7/6/2009	7/7/2009	7/8/2009	7/9/2009	7/10/2009	7/11/2009
6:05 AM	65.6	72.2	69.1	73.1	70.0	76.7
6:10 AM	73.6	70.3	73.1	73.1	78.1	-1.0
6:15 AM	75.0	77.3	79.7	81.0	79.2	102.3
6:20 AM	65.2	75.0	75.4	64.2	66.5	73.1
6:25 AM	71.8	69.5	71.1	75.0	53.8	68.2
6:30 AM	62.5	69.7	69.2	67.6	60.3	75.8
6:35 AM	72.2	69.0	70.5	72.3	79.9	-1.0
6:40 AM	72.8	68.2	69.4	69.4	75.1	68.8
6:45 AM	74.5	70.8	75.4	66.4	70.9	66.0
6:50 AM	75.3	68.9	70.5	69.6	70.4	58.8
6:55 AM	60.4	61.2	65.8	58.9	67.8	-1.0
7:00 AM	68.3	72.2	73.6	64.4	69.1	65.3
7:05 AM	65.0	71.7	66.6	64.4	72.0	-1.0

Figure 9. Sample Output Data Values.

The above procedures were used to extract volume and data matrices for all the GPL detectors in Figure 6 and all the HOT detectors in Figure 7 for every 5 minutes of every day for all of 2008.

Toll Data

The toll data were provided by COFIROUTE, USA. COFIROUTE, USA is a member of a consortium that was awarded a contract from Mn/DOT for conversion of the exiting HOV lanes to HOT lanes.

COFIROUTE is the current operator of MnPASS. The company works closely with Mn/DOT to monitor traffic and the dynamic pricing system on the HOT lanes. COFIROUTE provided data for each traveler who paid to use the MnPASS lanes in 2008 (see Figure 10).

I-394 Data

Start Plaza	End Plaza	AMOUNT	DATE	TIME
1001	1005	0.25	1/2/2008	6/16/2009 5:47:42 AM
1001	1005	0.25	1/2/2008	6/16/2009 5:50:34 AM
1001	1005	0.25	1/2/2008	6/16/2009 5:51:51 AM
1001	1005	0.25	1/2/2008	6/16/2009 5:58:19 AM
1001	1005	0.5	1/2/2008	6/16/2009 6:06:30 AM
1001	1005	0.5	1/2/2008	6/16/2009 6:06:26 AM
1001	1005	0.5	1/2/2008	6/16/2009 6:07:36 AM
1001	1005	0.5	1/2/2008	6/16/2009 6:15:30 AM
1001	1005	0.5	1/2/2008	6/16/2009 6:18:46 AM
1001	1005	0.5	1/2/2008	6/16/2009 6:19:22 AM
1001	1005	0.5	1/2/2008	6/16/2009 6:22:46 AM
1001	1005	0.5	1/2/2008	6/16/2009 6:22:42 AM
1001	1005	0.5	1/2/2008	6/16/2009 6:22:35 AM
1001	1005	0.5	1/2/2008	6/16/2009 6:22:47 AM
1001	1005	0.5	1/2/2008	6/16/2009 6:23:24 AM
1001	1005	0.5	1/2/2008	6/16/2009 6:23:01 AM
1001	1005	0.5	1/2/2008	6/16/2009 6:25:41 AM

Figure 10. Sample of Toll Data.

The toll data includes the Start Plaza, which refers to the point where the traveler was first detected, and the End Plaza, which is the point where a traveler was last detected. In addition, the data also contain information on the cost of the toll for each traveler and the time the traveler was first detected.

Data Reduction

Data reduction involved calculating the average GPL speed, the average GPL volume, and the average HOT lane speed value for each section of the highway. Since 5-minute intervals were used to extract detector data for the morning and the afternoon operating times, the average toll amount and the average number of paying travelers were also computed over a 5-minute interval. One of the biggest challenges in this research was how to manage such large detector data and toll data sets for the entire year of 2008. One of the options that was available to the researcher was to create a program using MatLab to handle the data. The program needed to perform several tasks such as uploading the data, checking the data for errors, removing erroneous data, and then calculating the desired values. The program flow chart in Figure 11 was used for the analysis of the data.

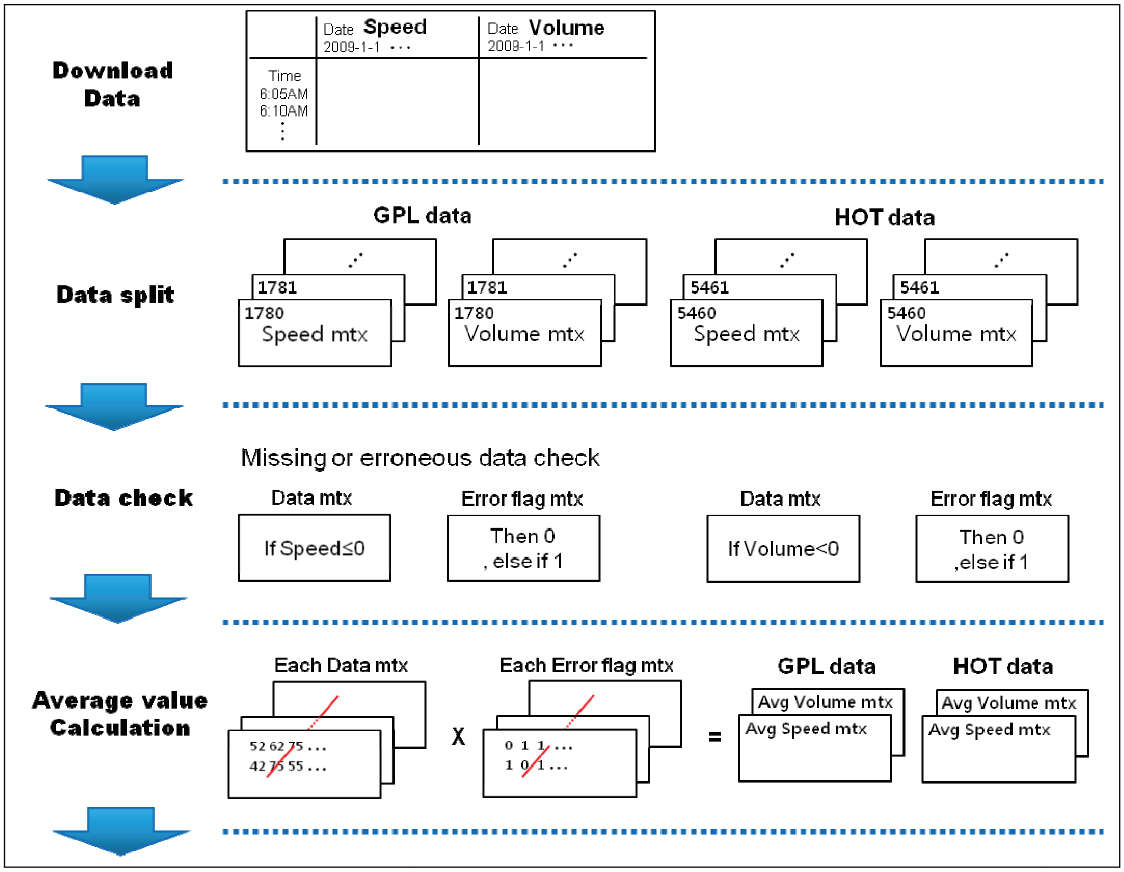

Figure 11. Program Flow Chart.

Program Flow Chart

The first step in the program involves creating a file path that will import the data from the file location into MatLab. The general format of the file imported into MatLab is in a matrix format, and then the matrix was separated into two main categories, volume and speed. In most cases LDS detectors provided reliable data for research; however there are occasions when there was a detector malfunction and erroneous data were recorded. It was important to eliminate the errors in order to analyze conditions that are truly representative of the study site. For speeds, we eliminated readings of less or equal to zero and speed above 100 mph. The assumption was that if the LDS detector recorded a speed of less than zero or speeds more than 100 mph, then the detector had a malfunction. We also treated volume values less than zero as an error in LDS data. The detector data were then averaged for each section of the freeway (for example 1001 to 1002). The matrix outputs include the average GPL speed and volume and the average HOT lane speed and volume. Lastly a similar procedure was used to upload the toll data into MatLab; no clearly erroneous data were found in the toll data set.

Data Analysis

The average GPL speeds and volumes and the average HOT lane speeds and volumes obtained using the program was used to compute the value of travel time savings. The average was over the length of the section of the highway being studied for a given 5-minute period of a single day.

To compute the willingness of travelers to pay for travel time savings, the percentage of the travelers willing-to-pay was determined by dividing the number of paying travelers by the average volume on the GPL lanes plus the number of paying travelers.

$$\%\,WTP = \frac{Number\ of\ \ Paying\ Travelers}{Average\ GPL\ Volume + Number\ of\ \ Paying\ Travelers}$$

The travel time savings were calculated by subtracting the travel time on GPL lanes from the travel time on HOT lanes.

$$\frac{TT\ on\ GPL - TT\ on\ HOT)\ mins}{60\ mins} = Travel\ Time\ Savings\ (hrs)$$

where:

$$TT\ on\ GPL = \frac{Length}{Average\ GPL\ Speed}$$

$$TT\ on\ HOT = \frac{Length}{Average\ Hot\ Speed}$$

The VTTS was calculated by dividing the toll rate with the travel time savings.

$$VTTS = \frac{Cost\ of\ Toll}{Travel\ Time\ Savings}$$

The graph of the VTTS versus the percent willingness-to-pay (WTP) is on key measurement of the willingness of travelers to pay for travel time savings.

RESULTS AND FINDINGS

The eastbound and westbound HOT lanes are presented below.

Morning Operation Period (6 a.m. – 10 a.m.)

The morning operation time for MnPASS is from 6:00 a.m. to 10:00 a.m. In this analysis we examine the travel time required to travel from location 1001 to 1005, a distance of 6.5 miles. If we compare travel time on the GPLs and the HOT lanes during the morning travel period, the travel time on GPLs ranges between 5 minutes and 20 minutes, whereas the travel time on HOT lanes is between 5 minutes and 10 minutes. The median travel time on GPL lanes is 6.1 minutes compared to 5.7 minutes for the HOT lanes. Overall, 85 percent of the travelers in GPL lanes reached station 1005 in less than 8 minutes, while it 95 percent of all travelers in HOT lanes spent less that 8 minutes to reach station (see Figure 12).

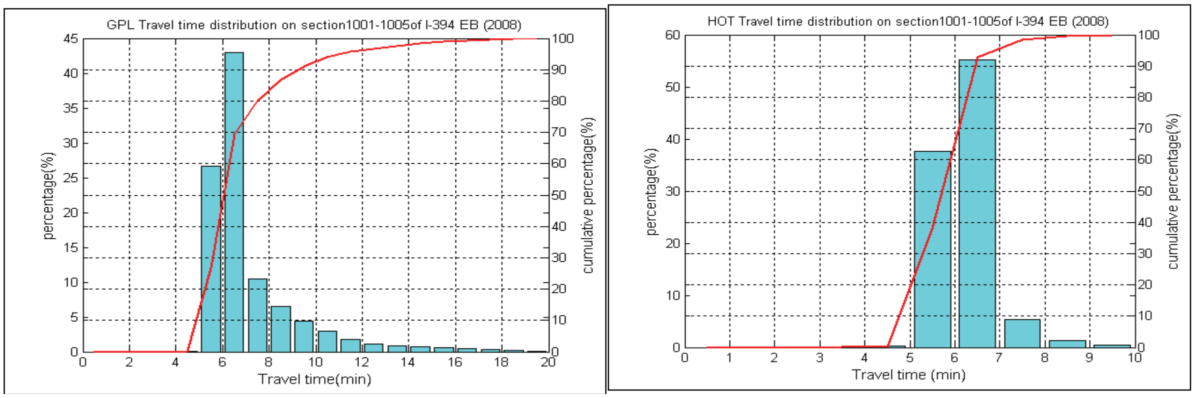

Figure 12. GPL and HOT Travel Time.

During the morning operation period, the median speed on GPL lanes and HOT lanes are 60 mph and 62.5 mph, respectively. The results indicate that 40 percent of the time the speeds on GPLs were faster than 55 mph compared to 57 percent of the time on HOT lanes. The findings support the hypothesis that speeds on HOT lanes are more reliable that the speed on GPL lanes, therefore travelers on HOT lanes can expect more reliable travel time compare to GPL travelers (see Figure 13).

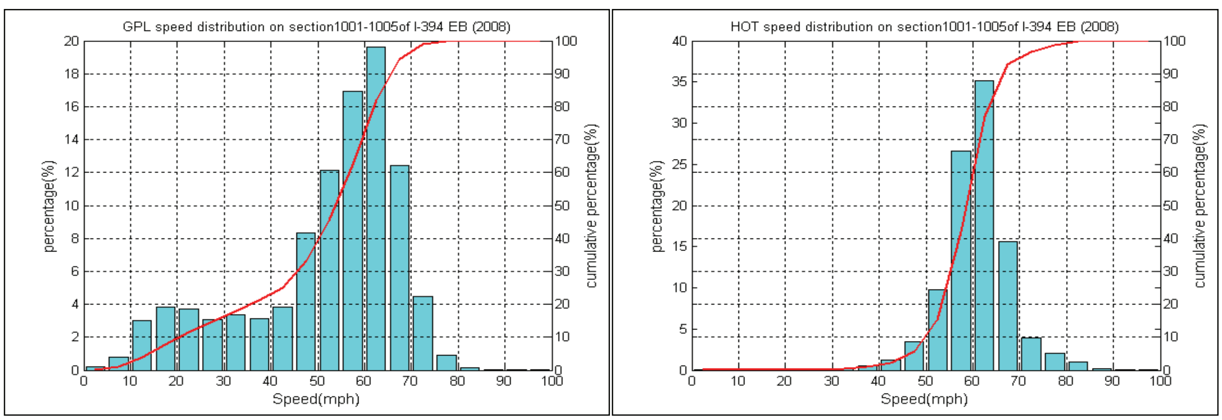

Figure 13. GPL and HOT Speed Distribution.

The small difference between GPL speeds and HOT speeds resulted in very small travel time savings. The total travel time savings for eastbound trips between station 1001 and station 1005 ranges from zero to 3.5 minutes. Over 80 percent travelers on MnPASS paid for an average travel time savings between zero and 0.5 minute (see Figure 14).

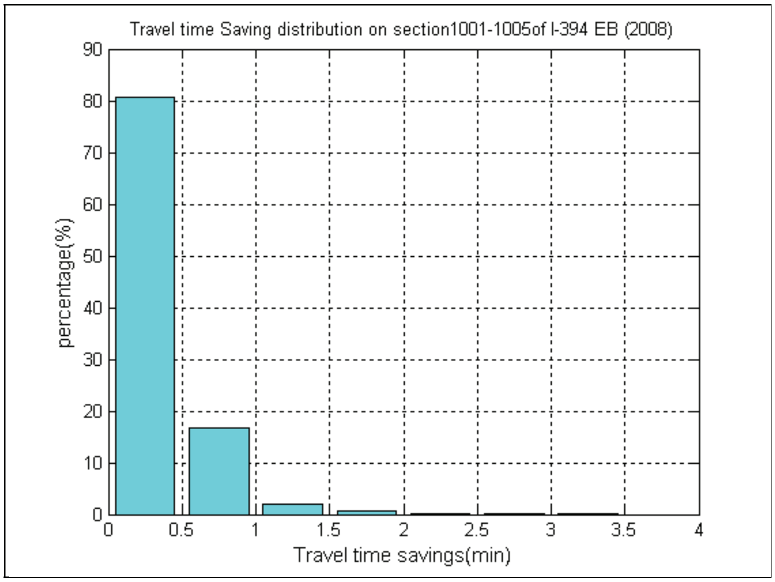

Figure 14. Travel Time Savings (6 a.m. – 10 a.m.).

MnPASS uses a dynamic pricing system to continuously adjust the toll up or down depending upon the amount of traffic in the HOT lanes. The toll ranged between \$0.25 and \$7.95 during this period in 2008. Close to 40 percent of the time the toll rate was \$0.50 (see Figure 15).

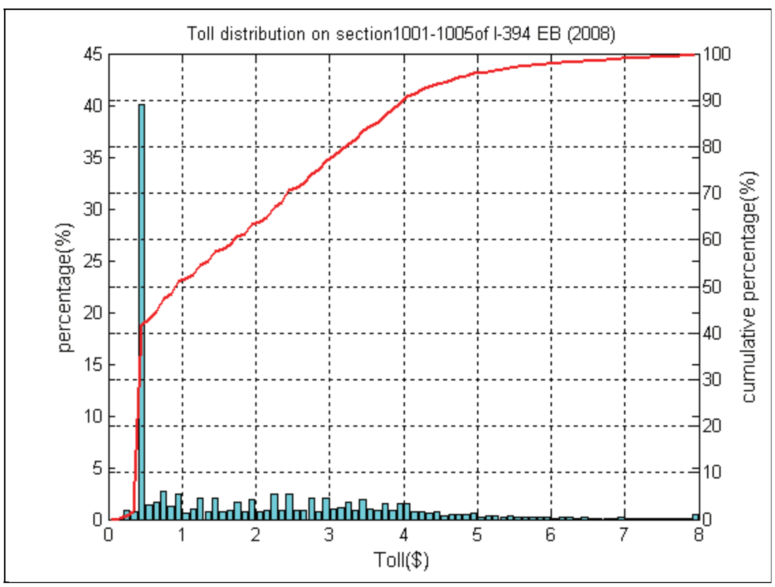

Figure 15. Toll Distribution (6 a.m. – 10 a.m.).

Having established that in the 2008, the travel time differences between the HOT lanes and the GPL lanes were very small, and the average HOT lane toll was approximately \$1.00, the willingness-to-pay results indicate that morning travelers have a wide range of VTTS. This was fairly consistent over a wide range of cost of travel time savings (CTTS), from low values (approximately \$20/hr) to extremely high values (over \$500/hr).

To understand how the WTP percentages varied over the range CTTS, a plot of the average percent WTP for each CTTS was developed (see Figure 16). As expected the percentage of travelers WTP for HOT lanes drops as CTTS increases. The percentage GPL of travelers willing to pay for the HOT lanes ranged from 0 to 8 percent.

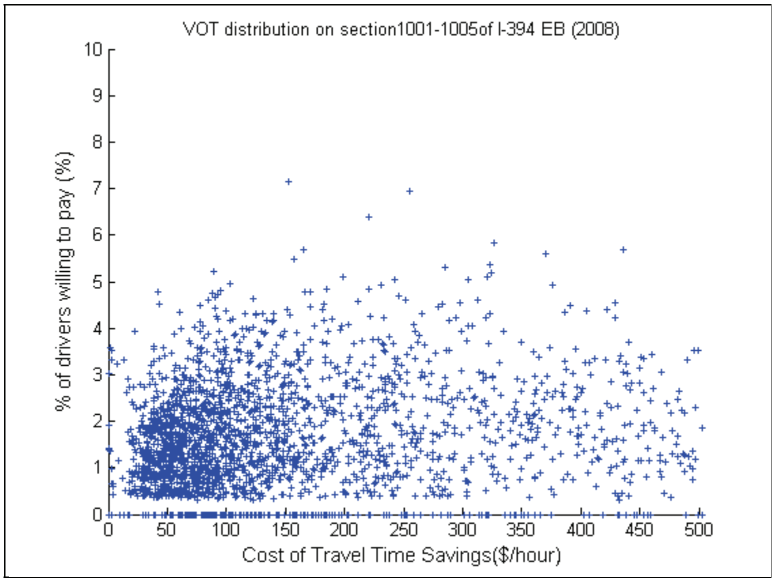

Figure 16. Cost of Travel Time Saving (6 a.m. – 10 a.m.).

Next, the average percentage of GPL travelers willing to pay for a given CTTS was examined (see Figure 17). This was an average of the points in Figure 5. It can be seen that the median VOT was $93 per hour, which is extremely high compared to the literature.

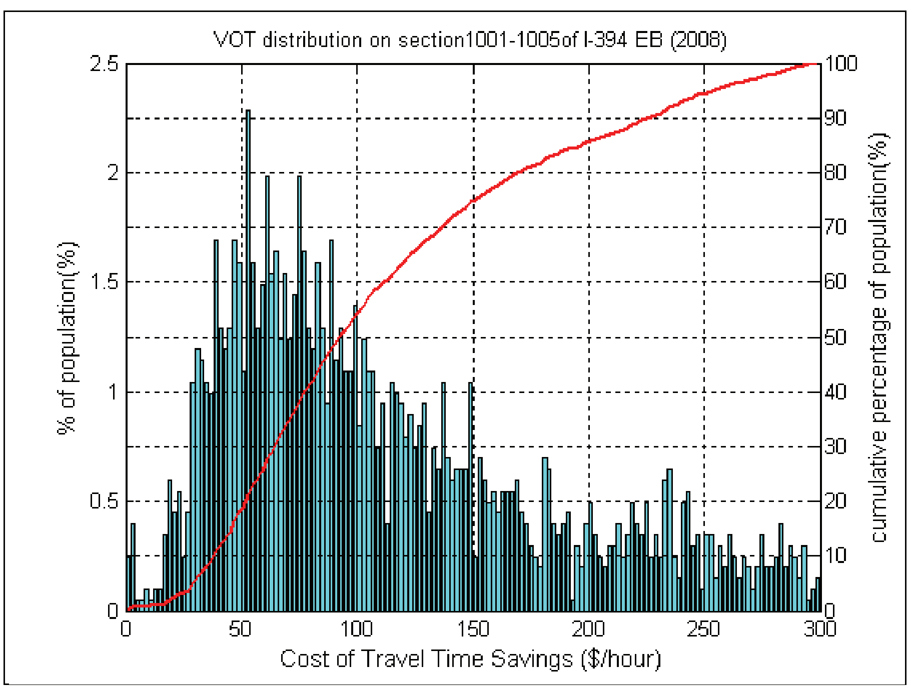

Figure 17. VOT Distribution (6 a.m. – 10 a.m.).

Morning Peak Period (7:30 a.m. – 8:30 a.m.)

During the morning peak period the travel time on GPLs ranges between 5 minutes and 20 minutes, whereas the travel time on HOT lanes is between 5.5 minutes and 10 minutes. Overall, 33 percent of the travelers in GPLs completed the 6.5-mile stretch between station 1001 to 1005 in less than 6.5 minutes, while over 65 percent of travelers in HOT lanes spent less that 6.5 minutes to reach station 1005 (see Figure 18).

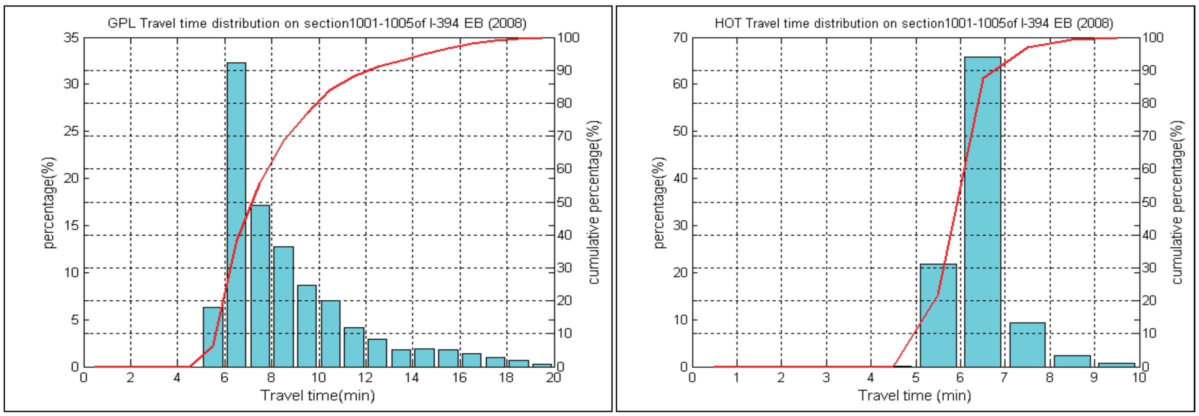

Figure 18. GPL and HOT Travel Time (7:30 a.m. – 8:30 a.m.).

The graphs of GPL speeds is unevenly distributed, with speeds as low 5 mph and as high as 90 mph, while the HOT speeds are more evenly distributed with most data points falling between 40 mph and 85 mph. The median value for GPL speeds and HOT lane speeds was 50 mph and 57.5 mph, respectively. Among the customers who paid to use the HOT lanes, 34 percent of the drivers drove at least 65 mph compared to less that 15 percent of drivers on GPL lanes (see Figure 19).

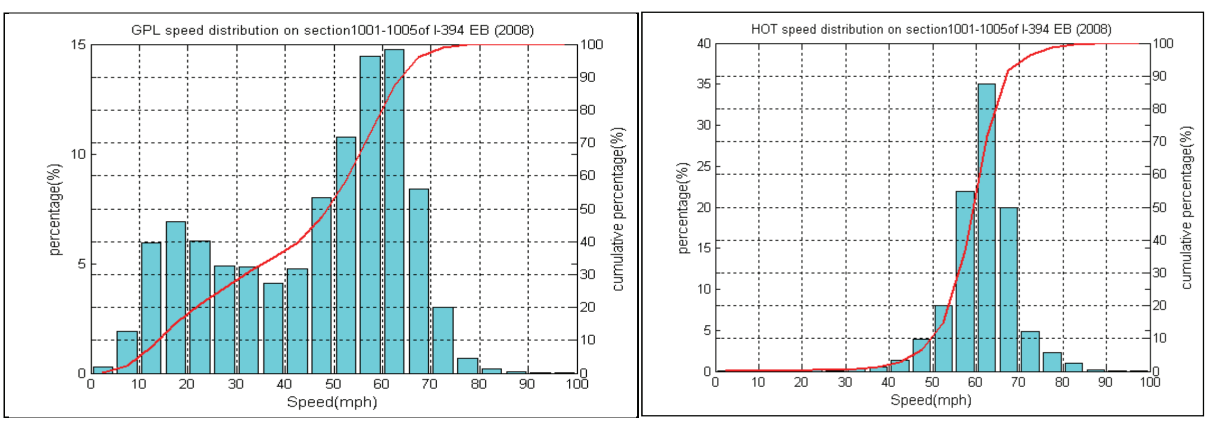

Figure 19. GPL and HOT Speed Distribution (7:30 a.m. – 8:30 a.m.).

Although there is a significant difference between the median speeds, the difference between GPL speeds and HOT speeds still remain small for a large percent of the time. As a result morning peak distribution chart is similar to the chart of the entire morning operation time for GPLs and HOT lanes. The travel time savings during the morning peak period ranges from zero to 10 minutes. Ninety four percent of travelers on MnPASS paid for an average travel time savings between zero and 0.5 minute, 5 percent saved between 0.5 minute and 1 minute. Less than 1 percent of the travelers saved more than a 1.5 minute (see Figure 20).

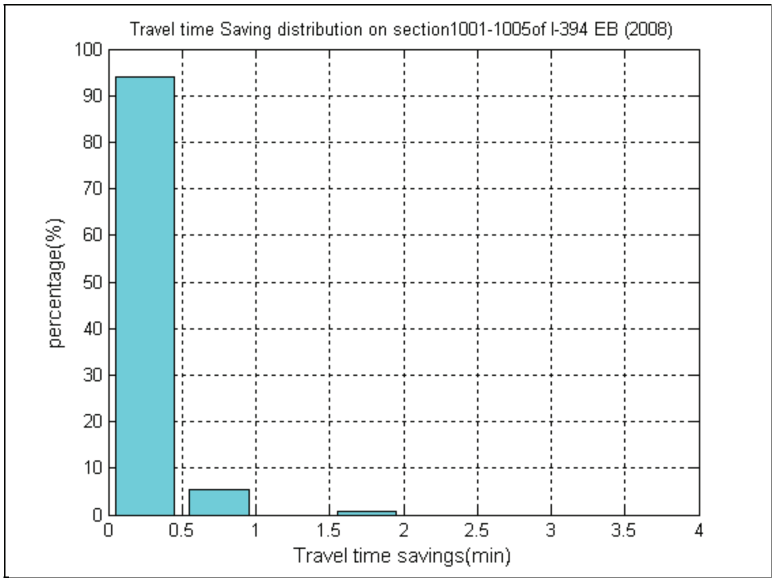

Figure 20. Travel Time Savings (7:30 a.m. – 8:30 a.m.).

The number of paying travelers is almost always less than 5 percent of GPL travelers. The toll ranged between $0.35 and $7.95. The median toll during the morning peak period was $3.45, Fifteen percent of the travelers paid more than $4.00 to use the lanes. When we put all the variations in speeds and toll rates, the travel time differences between HOT lanes and GPL lanes still remain very small even during peak period. According to the data a majority of toll paying commuters paid close to $3.35, for little or no travel time savings. The toll rates also changed more frequently between $1.50 and $5.00 to control the volume of vehicles in HOT lanes (see Figure 21).

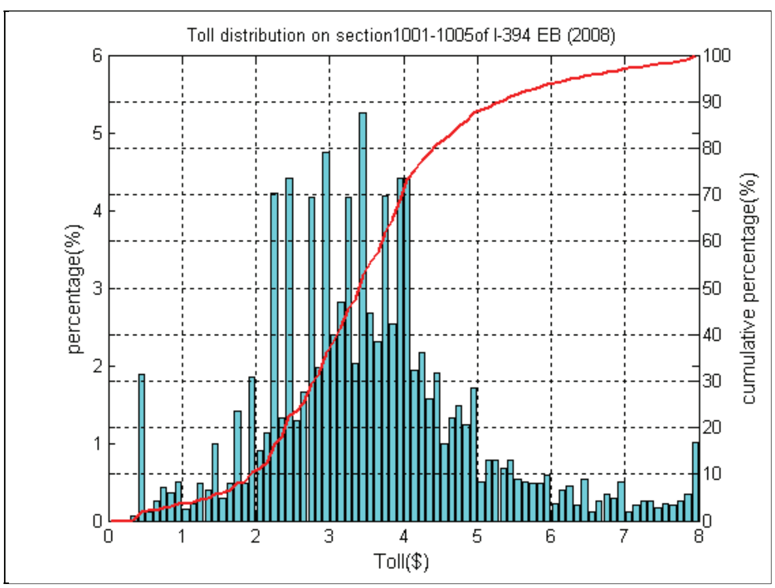

Figure 21. Toll Distribution (7:30 a.m. – 8:30 a.m.).

The percentage of travelers willing to pay during morning peak period ranged between zero and 6 percent (see Figure 11). The percentage of GPL travelers willing to pay is uniform over a wide range of CTTS, from low values (approximately \$5/hr) to extremely high values (over \$500/hr) (see Figure 22).

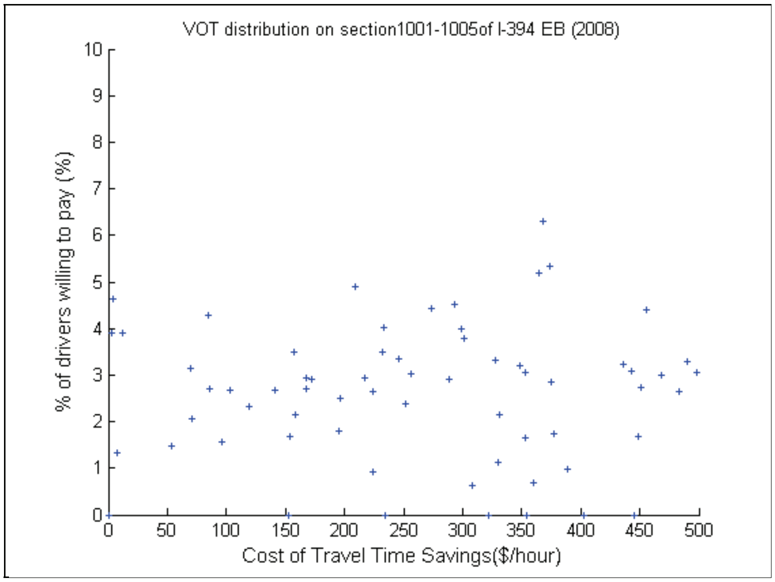

Figure 22. Cost of Travel Time Saving (7:30 a.m. – 8:30 a.m.).

Another way to interpret in willingness-to-pay during morning peak period involves constructing a value of time distribution chart. On I-394, over 5.5 percent of the travelers are willing to pay for travel time savings. The median cost of travel time savings \$167 per hour. Above the 85th percentile level, the value of time was greater than \$253 per hour (see Figure 23). These are extremely high VTTS caused by the small travel time savings.

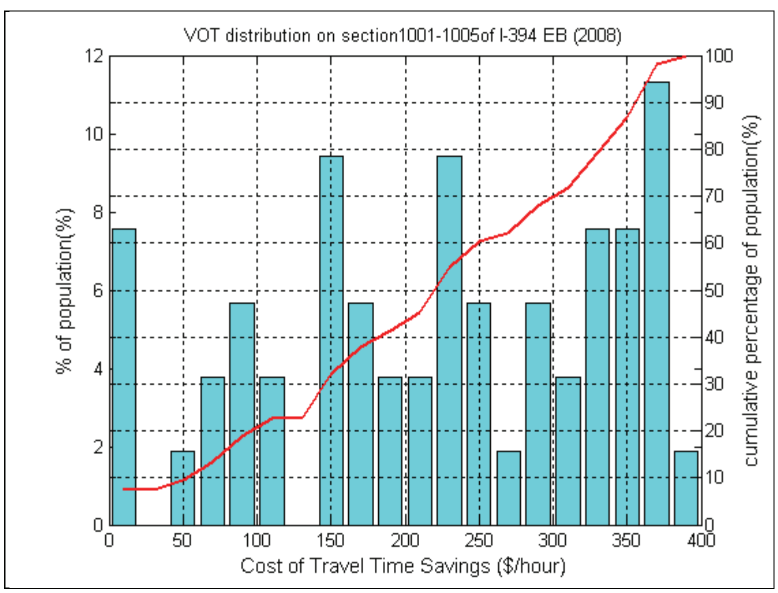

Figure 23. VOT Distribution (7:30 a.m. – 8:30 a.m.).

Afternoon Operation Period (2 p.m. – 7 p.m.)

The westbound HOT lanes between Wirth Parkway and I-494 are open to general traffic on Monday through Friday from 2 p.m. to 7 p.m. The median travel time in 2008 for GPL travelers and HOT travelers was 5.75 minutes and 6 minutes, respectively. The range of travel time GPL lanes is between 5 minutes and 18 minutes, whereas the range travel time on HOT lanes is between 5 minutes and 10 minutes. Over 70 percent of the travelers in GPL lanes and HOT lanes reached station 2005 in less than 6.5 minutes (see Figure 24).

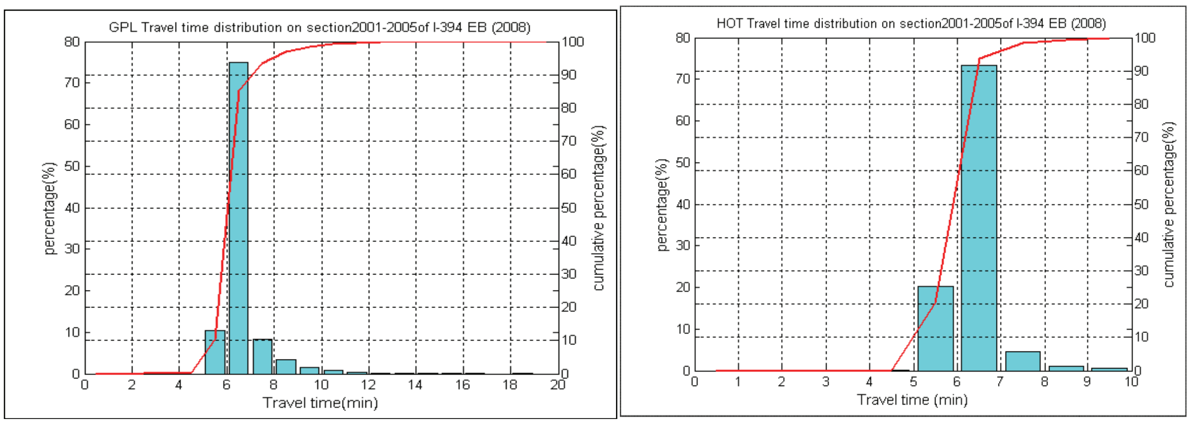

Figure 24. GPL and HOT Travel Time (2 p.m. – 7 p.m.).

HOT lane speeds from station 2001 to 2005 are more evenly distributed compared to GPLs speeds. The range of the speeds on GPL lanes was between 10 mph and 80 mph, while the range on HOT lanes was 40 mph to 80 mph. The median speed on GPL lanes and HOT lanes were 57.5 mph and 60 mph, respectively. The analysis indicates that 40 percent of the time the speeds on HOT lanes we greater than 55 mph compared to 25 percent of the time on GPLs (see Figure 25).

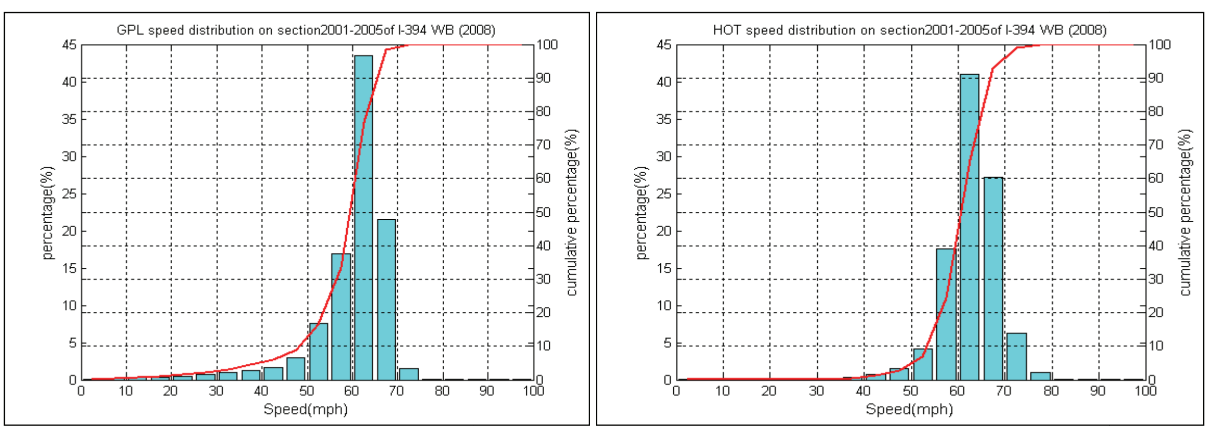

Figure 25. GPL and HOT Speed Distribution (2 p.m. – 7 p.m.).

Since the difference between GPLs speeds and HOT lane speeds is very small, the range of travel time savings was also very small (see Figure 26). In 2008, 77 percent of travelers paid for average travel time savings between zero and 0.5 minute, a little over 10 percent saved between 0.5 minute and 1 minute, 5 percent saved between 1 minute and 1.5 minute, and less than

1 percent of the travelers saved more than 2 minutes on the HOT lanes. The maximum travel time savings in this section is 8 minutes; however a very small number of travelers actually obtained the maximum travel time savings (see Figure 26).

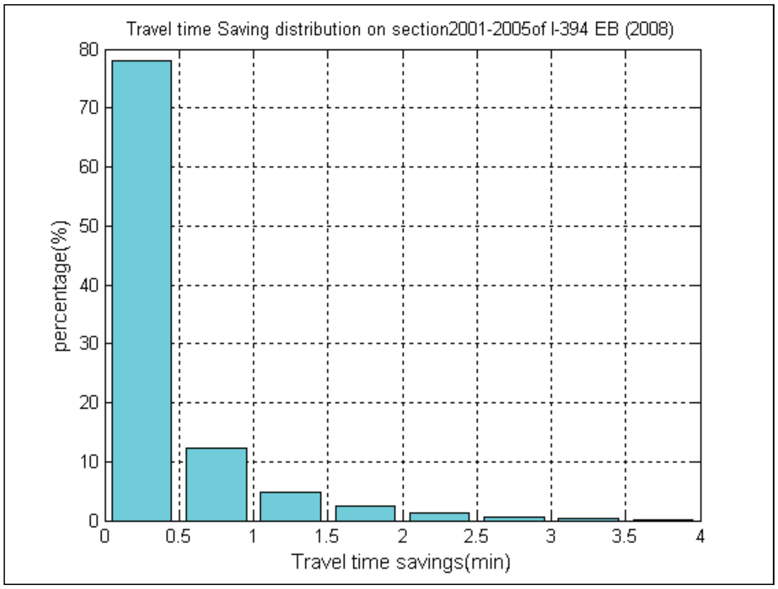

Figure 26. Travel Time Savings (2 p.m. – 7 p.m.).

The range of tolls for westbound HOT lanes was between \$0.35 and \$5.00. Close to 60 percent of travelers paid a \$0.50 toll. Around 10 percent of the travelers paid more than \$2.00 to use the lanes (see Figure 27).

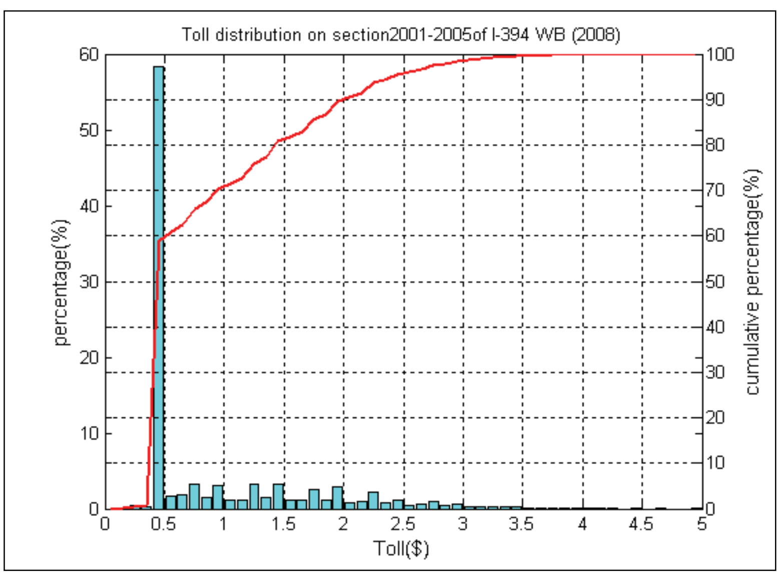

Figure 27. Toll Distribution (2 p.m. – 7 p.m.).

The willingness-to-pay results indicate that afternoon travelers have a wide range of value of time. This was fairly consistent over a wide range of CTTS, from low values (approximately \$25/hr) to extremely high values (over \$500/hr). To understand how the WTP percentages varied over the range CTTS, a plot of the average percent WTP for each CTTS was developed (see Figure 28). As expected the percentage of travelers WTP for HOT lanes drops as CTTS increases. The percentage GPL of travelers willing to pay for the HOT lanes ranged from 0 to 8 percent.

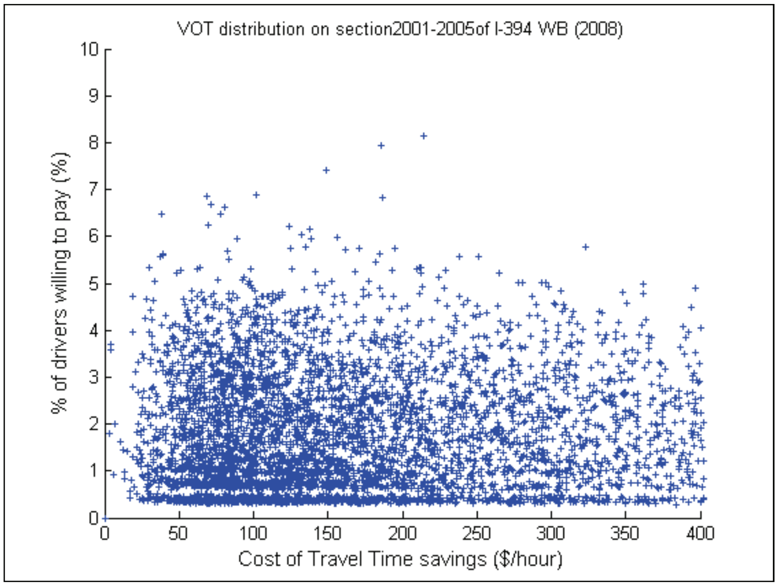

Figure 28. Cost of Travel Time Saving (2 p.m. – 7 p.m.).

By taking an average of the points in Figure 28 to examine the average percentage of GPL travelers willing to pay for a given CTTS (see Figure 29), the median VOT was $135 per hour, which is extremely high compared to the literature.

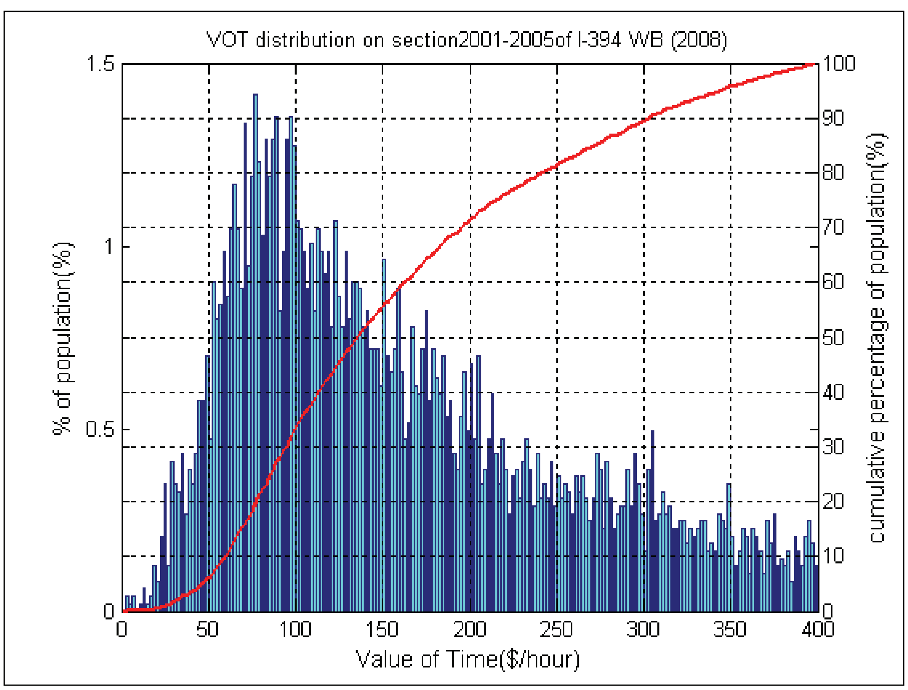

Figure 29. VOT Distribution (2 p.m. – 7 p.m.).

Afternoon Peak Period (5 p.m. – 6 p.m.)

During westbound peak period, between 5 p.m. and 6 p.m., the travel time on GPL lanes ranged between 5 minutes and 18 minutes. The travel time on HOT lanes was between 5.5 minutes and 16 minutes. Approximately, 60 percent of the travelers in GPLs completed the 6.5-mile travel between station 2001 and 2005 in less than 6.5 minutes, while it 93 percent of all travelers in HOT lanes spent less that 6.5 minutes to reach station 2005. The maximum travel time saving in the entire fast lanes was 12 minutes; however less than 0.5 percent of the drivers got the maximum travel time savings (see Figure 30).

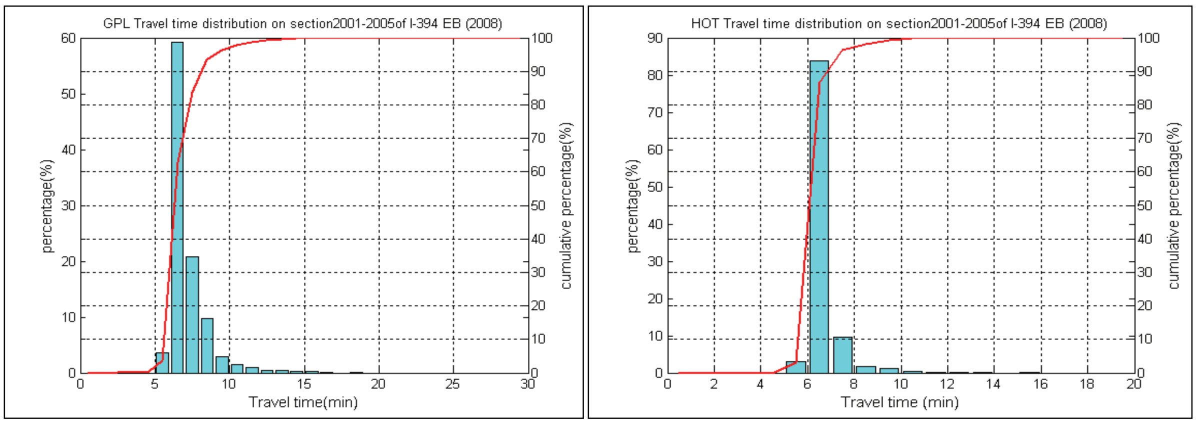

Figure 30. GPL and HOT Travel Time (5 p.m. – 6 p.m.).

The GPL speeds on I-394 vary from as low 5 mph to as high as 85 mph, while the HOT speeds are between 35 mph and 85 mph. The median for GPL speeds and HOT lane speeds is around 57.5 mph and 58 mph, respectively. Seventy five percent of drivers on GPLs drove over at 55 mph or faster, compared to 82 percent for HOT travelers (see Figure 31).

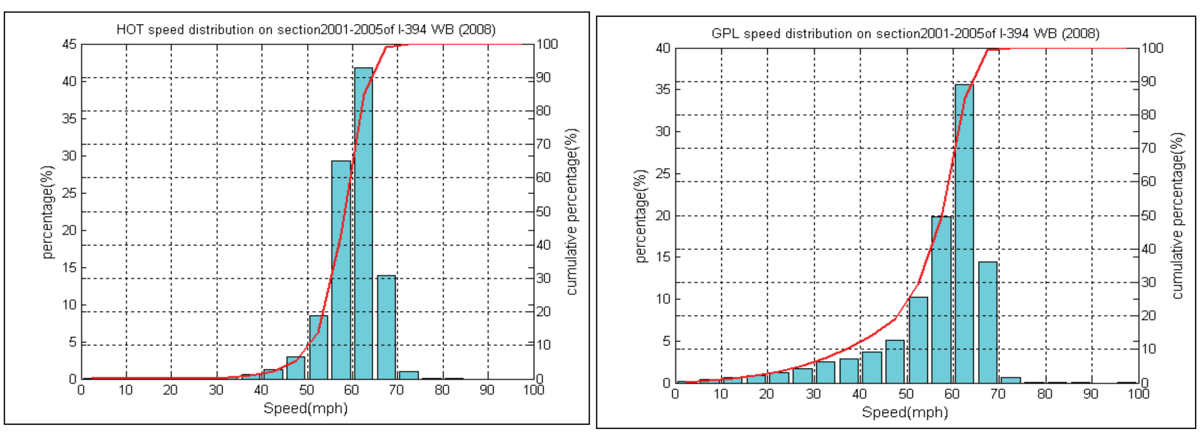

Figure 31. GPL and HOT Speed Distribution (5 p.m. – 6 p.m.).

In the afternoon travel period, the resulting travel time savings are very small. Around 75 percent of travelers on MnPASS paid for an average travel time savings between zero and 1 minute, 10 percent saved between 1 minute and 2 minutes, while less than 1 percent of the travelers saved more than 4.5 minutes (see Figure 32).

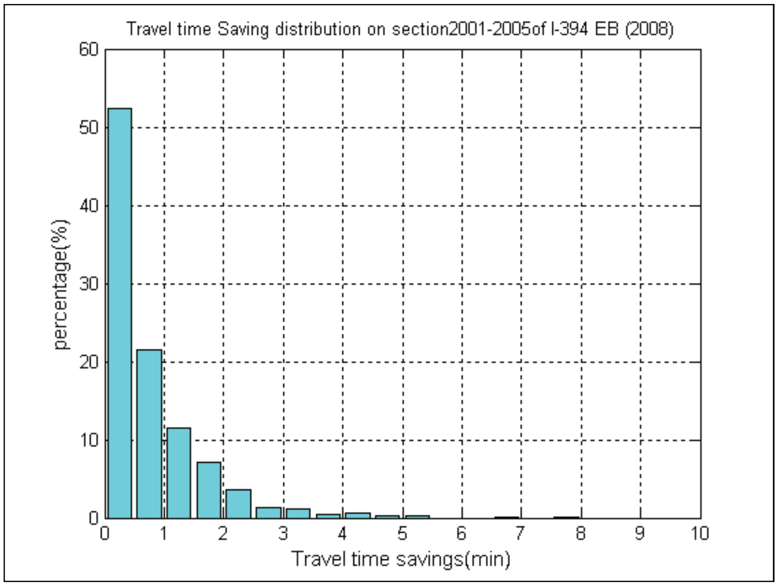

Figure 32. Travel Time Savings (5 p.m. – 6 p.m.).

The toll ranged between $0.35 and $7.95. The median toll during the morning peak period was $1.50, less than 30 percent of the travelers paid more than $2.00 to use the lanes (see Figure 33).

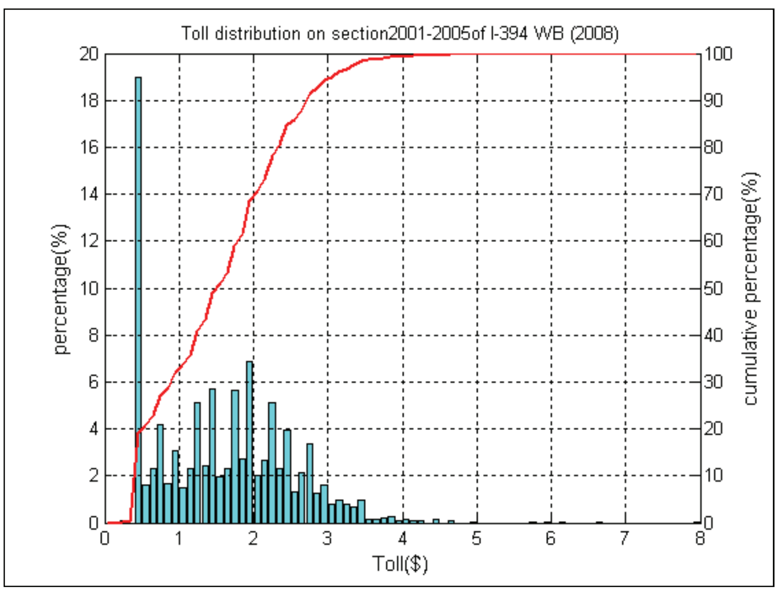

Figure 33. Toll Distribution (5 p.m. – 6 p.m.).

Similar to operation time data, the percentage of travelers willing to pay during peak period ranged between zero and 6 percent (see Figure 11). The percentage of GPL travelers willing to pay is uniform over a wide range of CTTS, from low values (approximately $10/hr) to extremely high values (over $300/hr) (see Figure 34).

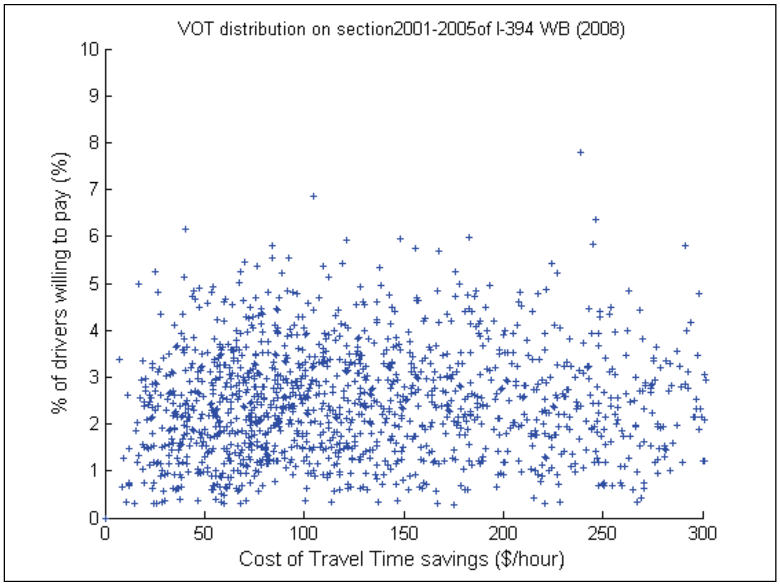

Figure 34. Cost of Travel Time Savings (5 p.m. – 6 p.m.).

At any given time, less than 1.8 percent of the GPLs travelers are willing to pay for travel time savings. The median cost of travel time savings $106 per hour. Above the 85 percentile level, the value of time was greater than $217 per hour (see Figure 35).

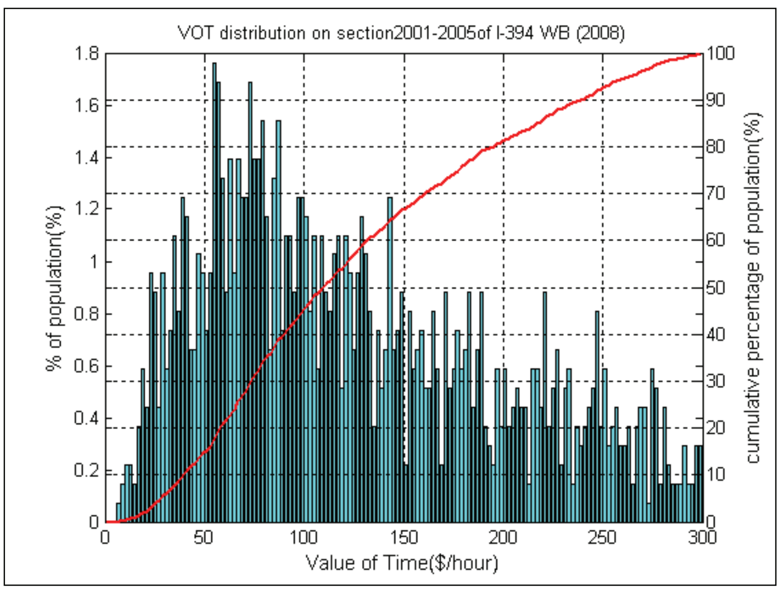

Figure 35. VOT Distribution (5 p.m. – 6 p.m.).

SUMMARY OF FINDINGS

Surprisingly the average peak period travel time on I-394 GPLs was not much higher than on the HOT lanes. The findings of this research indicate that in, both the eastbound and westbound direction, the majority of GPL drivers who paid to use HOT lanes saved less than 1 minute. In both directions, the maximum travel time was less than 20 minutes for GPLs and less than 10 minutes for HOT lanes.

The distributions of GPL speeds in both directions were much more varied than HOT lane speeds. The average GPL and HOT speeds were between 57 mph and 65 mph. This small difference in speeds resulted in the small travel time savings. The average travel time for the 6.5-mile stretch was around 6.5 minutes for both eastbound and westbound and for both GPL and HOT travelers.

The percentage of GPL drivers who were willing to pay to use HOT lanes ranges between zero and 8 percent. Slightly more travelers were willing to pay during the morning than the afternoon period. The value of time for over 50 percent of travelers on MnPASS was more than $90 per hour, much higher than in the literature.

Evidence that more drivers were willing to pay for morning travel time saving can be derived from the toll data. Dynamic pricing is more evident in the morning period than afternoon period. The median toll during the morning peak period was $3.45, while in the afternoon the median toll was $1.50. In both sections the majority the users of the HOT lanes had to pay just $0.50.

Similarities and differences in travel time savings and varying tolls resulted in a varied percent of willing to pay travelers over the range CTTS and very high values of time. As expected the percentage of travelers WTP for HOT lanes drops as CTTS increases.

CONCLUSION

Several reasons can be used to justify why MnPASS travelers use the lanes even though the overall travel time savings are very small. Previous studies on other HOT facilities have shown that there are other reasons why drivers choose to pay a toll to use the lanes. For example, on SR-91 express lanes it has been observed that some toll lane users choose to use the toll lanes under traffic conditions where their expected value of time savings is clearly less than the tolls paid. About 40 percent drivers cited driving comfort and the perception of greater safety as an important supplemental benefit of HOT lanes. Some off-peak toll lane use is also probably due to the availability of company-provided transponders (*4*). On I-25 Denver, drivers said that other than travel time savings HOT lane were more efficiency and more convenience than GPLs (*5*).

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5. Hess, Daniel B., Brown, Jeff and Shuo, Donald. *Waiting for the Bus,* Journal of Public Transportation Vol. 7, No. 4, pp. 67-84. 2004.

APPENDIX

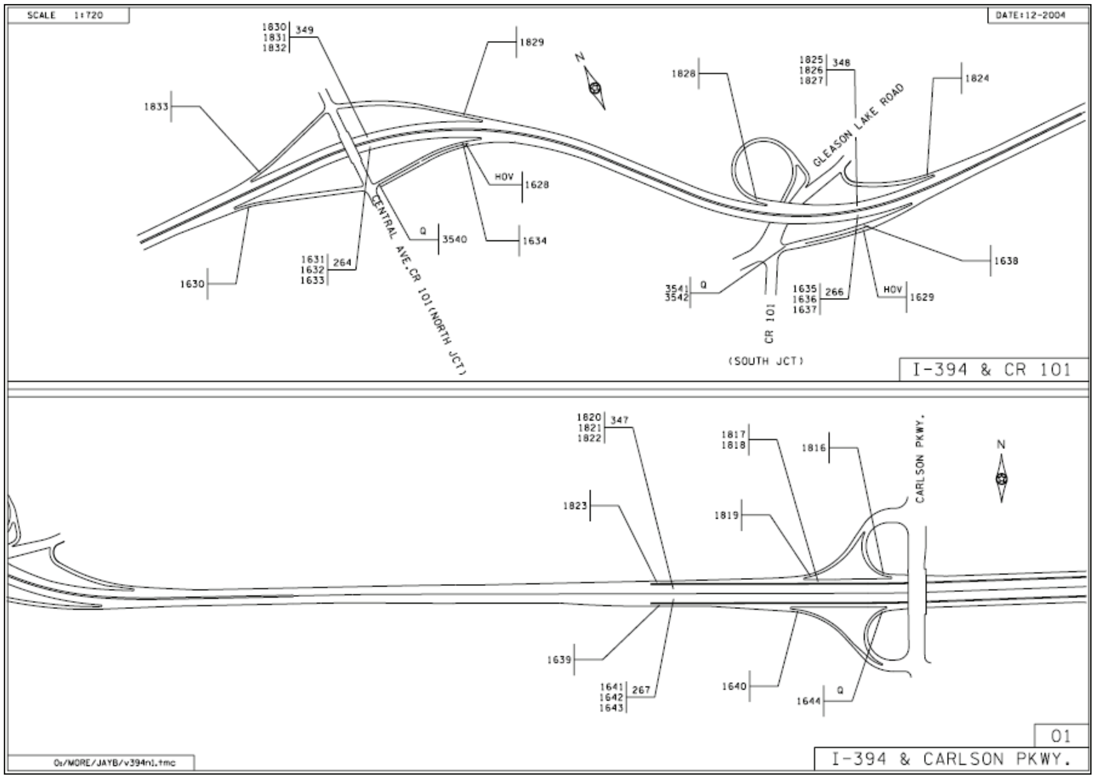

Figure 36. I-394 and Carlson Parkway.

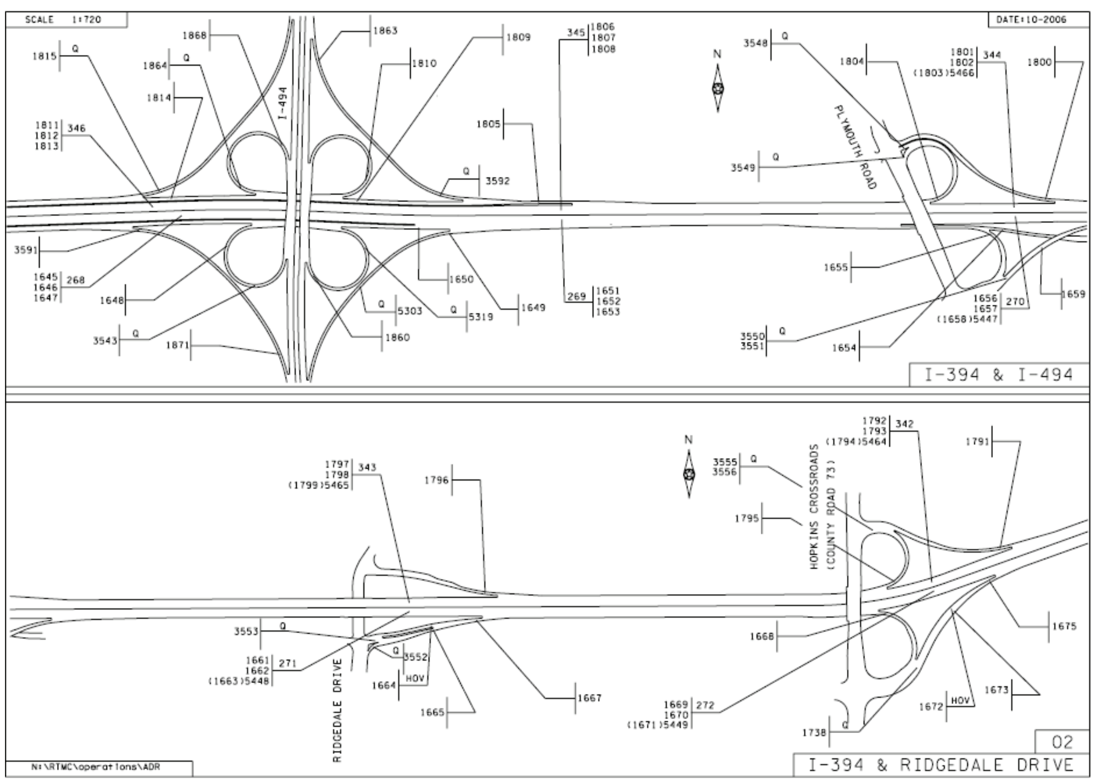

Figure 37. I-394 and Ridgedale Drive.

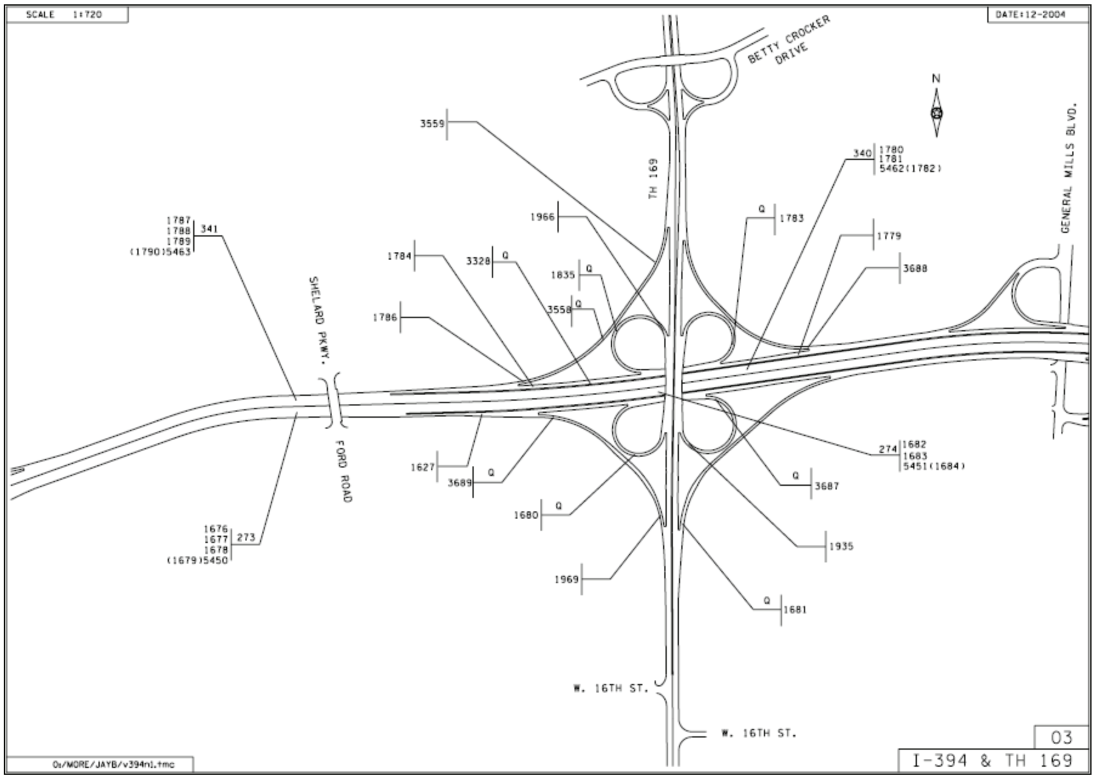

Figure 38. I-394 and Truck Highway 169.

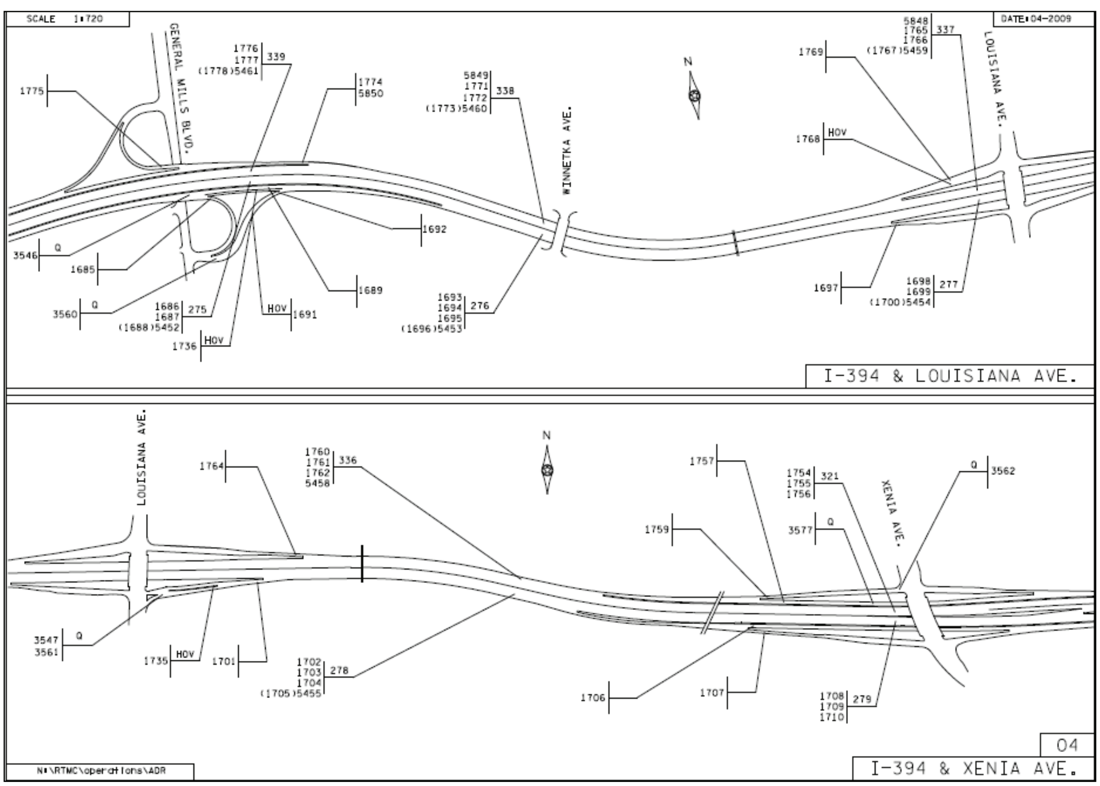

Figure 39. I-394 and Xenia Avenue.

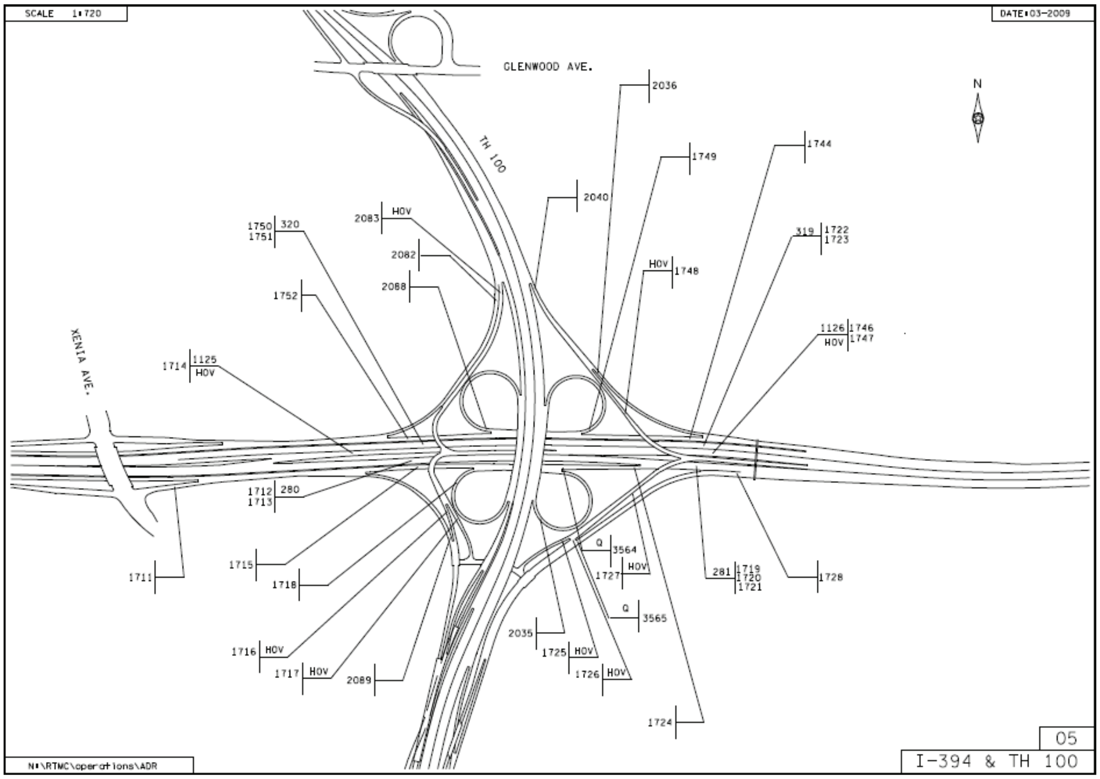

Figure 40. I-394 and Truck Highway 100.

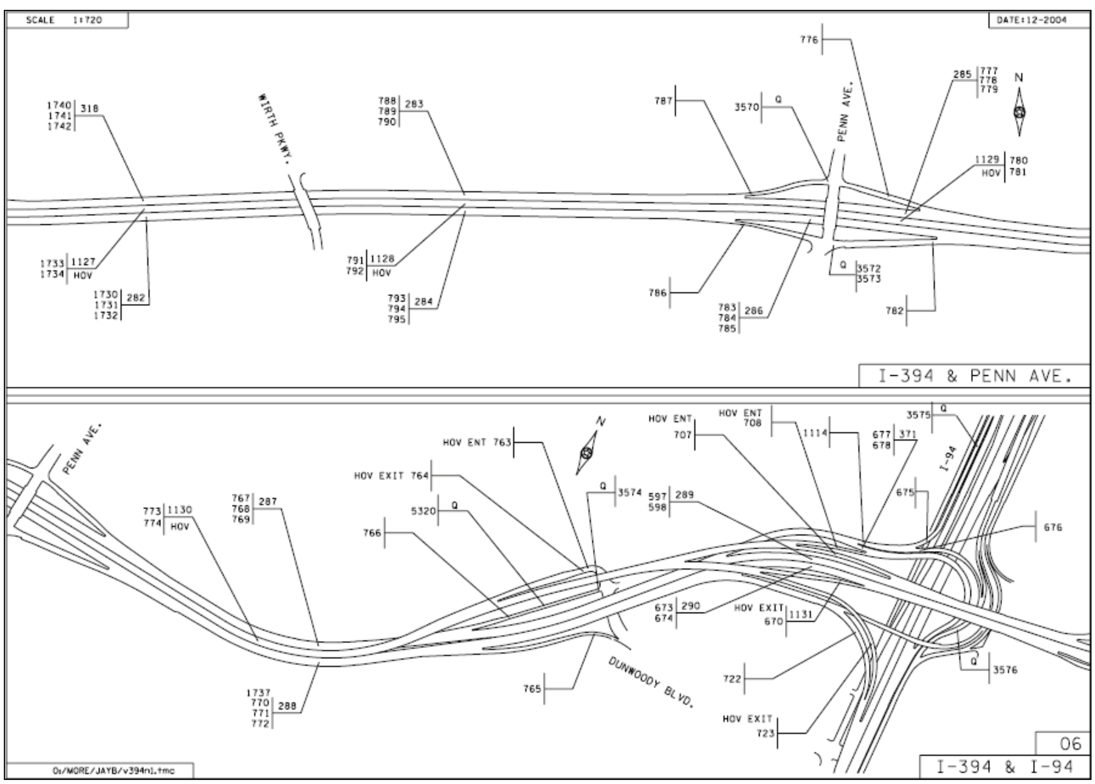

Figure 41. I-394 and I-94.