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16. Abstract

This report is a compilation of research papers written by students participating in the 2008 Undergraduate Transportation Scholars Program. The ten-week summer program, now in its eighteenth 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) selecting warning signs using decision theory and systems engineering concepts; 2) estimating corridor travel time from arterial traffic volume; 3) evaluating the effectiveness of life cycle variables in travel demand modeling; 4) estimating cross median crashes on horizontal curves; and 5) driver workload and visual studies.

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COMPENDIUM OF STUDENT PAPERS:

2008 UNDERGRADUATE

TRANSPORTATION SCHOLARS PROGRAM

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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 2008, the following five students and their faculty/staff mentors were:

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

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SELECTING WARNING SIGNS USING DECISION THEORY AND SYSTEMS ENGINEERING CONCEPTS

Prepared for
Undergraduate Transportation Scholars Program

by

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ACKNOWLEDGMENT

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

SUMMARY

Traffic control devices are a vital component of the transportation system as they communicate important information to road users. Warning signs are one type of traffic control devices that serve an important role as they provide road users with advance notice of potentially hazardous conditions which are not self-evident and are located on, or adjacent to, a roadway.

There are a large number of warning signs in the *Manual on Uniform Traffic Control Devices*. The MUTCD offers guidelines on the intended use of warning signs and the different situations they should be used to address; however, it lacks a systematic procedure for selecting them. As a result of the large number of warning signs and the lack of specific installation requirements, it can be difficult when trying to determine whether to install a warning sign.

The primary purpose of this project was to address the lack of a systematic process for the selection of warning signs, and to develop a conceptual framework for a selection process for warning signs. In trying to develop a system for the selection of warning signs, three concepts were investigated: systems engineering, decision theory, and multi-criteria decision methods.

Although these concepts may be different, they share many similarities. They address a problem and develop a method for coming up with a solution to the problem. The project determined that there is a basis for using decision theory for warning sign selection, but identified the need for additional development of the concept before it is a viable process for practitioners.

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INTRODUCTION

Traffic control devices are very important for the safe and efficient transport of people and goods (*1*). They provide road users with a variety of information which include: traffic laws, traffic regulations, traffic control features, potential hazards in or near the road, distances, and directions to various destinations. Some examples of the most commonly known traffic control devices include regulatory signs, guide signs, pavement markings, traffic signals, and warning signs.

Warning signs are used to call attention to potentially hazardous conditions which are not self-evident and are located on, or adjacent to, a roadway (*1).* The purpose of a warning sign is to prevent users from doing something they would otherwise do or to induce users to do something they would not normally do *(2).* Aside from this, warning signs provide a convenient and useful means of informing users of the potential hazards associated with the environment they are traveling in. The *Manual on Uniform Traffic Control Devices* (MUTCD) contains an abundance of information on warning signs which include standards (requirements) on the design, and application of not only warning signs but other traffic control devices *(1).* The MUTCD also offers guidance (recommendations), options, and support (background) provisions *(1).*

The *MUTCD* is one of the key documents in the field of transportation engineering because it contains standards and warrants for the design and application of traffic control devices *(2).* The purpose of the MUTCD is to provide uniformity amongst signs in order to promote safety and efficiency on streets and highways. Uniformity is achieved by traffic engineers adhering to the principles of the MUTCD and being consistent with the common practices of traffic control devices. Uniformity has been created through many years of revision, research, collaborative engineering efforts, and implementation of standards. This uniformity has aided in increasing driving comprehension of warning signs by providing information in a consistent matter, therefore reducing crashes and improving the efficiency of transportation systems *(2).*

Although the MUTCD promotes uniformity amongst warning signs, it has yet to develop a functional process that links many considerations to the various factors associated with the selection of traffic control devices. The current process for selecting traffic control devices consists primarily of the use of either individual engineering judgment or previous traffic studies (Section 1A.09) *(1)*. Current decisions regarding traffic control devices are often made on a device by device basis and not as part of a uniform system. The purpose of this project is to address the missing process for the selection of traffic control devices, more specifically warning signs.

This research developed a framework for a conceptual process which ties several different concepts together, including systems engineering, decision theory, and multi-criteria decision methods. Systems engineering is the integration of many engineering disciplines, which focuses on the development and organization of complex systems such as transportation management systems *(3)*. The use of Decision Theory further supplements the implementation of a System Engineering approach. Decision Theory is also an interdisciplinary area of study which concerns mathematicians, statisticians, managers and anyone else interested in analytical techniques used in decision making *(4)*. There are two types of decision theory, normative and descriptive *(4)*.

This research project used the application of normative decision theory since it is aimed at finding tools and methodologies to help promote the selection of better decisions. Multi-criteria decision methods was also incorporated into the development of the conceptual framework since it offers support for decision makers who are faced with making numerous and conflicting evaluations such as those associated with warning signs.

GOAL AND OBJECTIVES

The goal of this research effort was to develop a conceptual framework for the selection of warning signs using systems engineering and decision theory concepts. As this framework is developed through future research efforts it will evolve into one that links many considerations to the various factors associated with traffic control device decision.

In order to begin developing the conceptual framework for all traffic control devices, the author created a preliminary one for the selection of warning signs. This framework ties together many factors such as the intended purpose, targeted audience, costs, individual and system performance, risk management, and other factors that are applied in the selection, design, installation, operation, maintenance, and related activities of warning signs. Quintessentially the research combines normative decision theory principles with traffic engineering and creates a conceptual method, such as the Analytic Hierarchy Process, which considers:

- Installation and life-cycle costs of the device,
- Effectiveness of the device,
- Promotion of traffic safety by the device,
- Operational benefit of device,
- Method of communication by the device,
- Alternatives for communication by the device,
- Risks associated with the device,
- Tort concerns associated with the device,
- Political considerations associated with the use or non-use of the device.

In order to complete the goal of developing a quantifiable model that defines relationships between the various factors, a series of measurable objectives were defined to achieve the overall goal. The first measurable objective was to receive input from traffic engineers as to what they take into consideration when dealing with traffic control devices. Based upon the input from participants, the development of a list of survey questions was the next measurable objective. This survey was constructed as a result of expert input and quantifiable relationships. Based upon the results received, the next task was to determine how the concepts of systems engineering, decision theory, and multi-criteria decision analysis could be applied in developing the conceptual framework. This stage of the project involved an extensive amount of literature review because of the level of complexity of these subjects. However, once the literature review was completed the next step of the project was to incorporate these concepts in creating the conceptual framework. The development of the framework involved a collaborative effort among the researcher and those providing factors that should be incorporated into the process. Once the framework was developed all results and conclusions were prepared for this paper.

PROCEDURE

In developing the conceptual framework, there were a number of tasks that needed to be completed. The first task was to define the relationship between various factors involved in the decisions made on warning signs. For this task to be complete, input from traffic engineers was needed so that a list of common factors could be determined. A series of interviews were conducted with a variety of traffic engineers. The traffic engineers selected for the interviews included those who have or are currently serving on the Regulatory and Warning Sign Technical Committee of the National Committee on Uniform Traffic Control Devices. These engineers were selected because of their high level of expertise and extensive knowledge of factors that are commonly associated with the selection of warning signs. They were able to provide expert input and establish some of the main factors involved in the decisions made on warning signs.

After some of the main factors were identified, the next objective was to determine how important these factors were in relation to each other. The survey results would be used to determine how important the factors previously established are in relation to each other. In order to determine this, a series of surveys were conducted. A wide variety of transportation engineers participated in the survey and was instructed to rank and assign a percentage to each of the factors to indicate the relative importance of the factor relative to the other factors. For a sample of the survey given refer to Appendix A.

Once the survey was conducted the next object was to determine how the concepts of systems engineering and decision theory could be applied to developing a system for the selection of warning signs. This stage of the project involved an extensive amount of literature review.

After the literature review was completed, the conceptual framework was developed. In order to create the conceptual framework there were many things that needed to be integrated. This involved the incorporation of the results received from the interviews and surveys so that the criteria of the analytic hierarchy process could be determined. Following the establishment of the criteria needed for the development of the framework, the process was carried out by asking several engineers to participate in the conceptual process that was created. A step by step process will be explained later in this paper.

RESULTS

The following sections will describe the results received throughout the research process. These results help contribute to the development of the conceptual framework that was applied for the selection of warning signs.

Interview Results

The interviews conducted during the course of this research were conducted over the phone. Interviews consisted of informal discussions about the factors related to the selection of warning signs. Although the number of participants was limited to five, those who participated dealt with warning signs on a daily basis. Participants were asked to state the factors they considered when making decisions on the installation of signs and why they considered them to be important.

Based upon the responses received from the interviews, the most important factors traffic engineers consider when making decisions on warning signs were identified. The following sections describe what each factor means in relation to warning signs.

- **Benefits.** Like all traffic control devices, warning signs are installed because of the benefits they provide to road users. Some of the benefits that warning signs have to offer are the ability to improve traffic operations and most importantly the safety of not only motorist but pedestrians as well. The reason warning signs are able to improve traffic is that they provide the road user with adequate time to assess the potential hazard and respond properly. If a warning sign were not in place to warn of the oncoming hazard, motorists would be unaware of the situation and as a result traffic would be affected. Not only do warning signs provide an operational benefit they also provide safety benefits for motorists. By installing warning signs drivers are aware of hazards on the road and can safely commute on the roads.
- **Potential Effectiveness** represents how effective a particular warning could be if it is installed. Potential effectiveness is composed of many things, such as how well does the device warn motorists of the hazard. Is the warning sign difficult for motorist to understand, which leads to a decrease in motorist comprehension? Does the sign command respect from road users or has its integrity gone down due to the misuse of this sign? These are all factors that are related to potential effectiveness and should be thought about when deciding when, where, and how a warning sign should be used.
- **Cost** is often an important factor when dealing with the installation of traffic control devices. More often than not, cost issues affect the selection of warning signs. However, the Analytic Hierarchy Process enables decision makers to include this criterion in the decision making process so that appropriate priority is assigned to this factor. Although decision makers may decide not to place great priority on cost issues, it will still offer input to the global priority of the alternatives available.
- **Constraints** are always a part of any engineering process or decision. Constraints often have a great influence on decisions because of how much they affect the intended outcome. Although there may not seem like there are many constraints associated with warning signs there are a few that will be addressed later on in this report.
- **Need** may be the criteria that requires the most analysis. The MUTCD states that all traffic control devices must meet five basic requirements so that they are effective, and one of those requirements is that the device must fulfill a need. There are several types of needs that warning signs are meant to address, so for that reason this criterion will be the most complex and will require the most comparison.
- **Correct Practice** is a criterion that decision makers often overlook when making decisions on warning signs. Engineers and practitioners have worked many years on creating the uniformity that the MUTCD has established today. Due to this great effort, it is important that traffic engineers take in to consideration the consistency with the local practices. This will ensure that road users are familiar with warning signs and it will help increase driver comprehension of these signs.

After these factors were identified, there was a need to state a few key factors that fell under these general categories. The reason for this is that there needs to be a set of sub criteria so that the analysis of the hierarchy can take place.

- **Traffic Operations** is a sub criterion of benefits. Traffic engineers are always concerned with how specific traffic control devices impact traffic operations, more importantly their ability to improve traffic operations. Although warning signs may not have a great influence on improving traffic operations they are still important.
- **Safety** is another sub criterion of the possible benefits that warning signs can provide. This sub criterion refers to the amount of safety a warning sign contributes to road users. For example, if there is a potential hazard ahead and it is not apparent or expected by a driver, there are safety concerns associated with it. In order to promote safer travel for road users and to prevent injury, the potential safety benefit of warning signs should be considered in the analytic hierarchy process.
- **Initial** costs are an important factor on decisions made concerning hazard signs. Although costs should not be an issue when trying to improve safety and traffic operations, it often influences the decision of traffic engineers.
- **Life Cycle** costs are the costs required to maintain the traffic control device. For signs, life cycle cost would entail replacing the poles that they are mounted on if they are damaged and replacing the signs once they no longer meet retroreflectivity standards.
- **Installation** constraints involve installing warning signs so that they meet the standards stated in the MUTCD. There may be other installation constraints such as whether appropriate response distances are given for the driver to respond.
- **Maintenance** constraints entail how often a warning sign will need maintenance. This includes that the warning signs meet retroreflectivity standards. It would also entail how often signs need to be replaced. Although there may not be many maintenance issues pertaining to warning signs, there may be many more with other traffic control devices.
- **Operational** constraints are those that are involved with keeping the traffic control device operating correctly. Not only are there operational constraints but others as well. These may be whether or not the device does what it is intended to do and in what ways is it limited.
- **Warning Presence** criteria deal with how often the warning is present. The warning presence may either be fixed, present all the time, or transient, present only part of the time. For those warnings that are present only part of the time, the decision maker would need to determine how often the hazard is present and if it is severe enough to warrant the installation of the sign.
- The **Impacts** associated with a warning sign installation are those such as the risk of an accident or a legal claim being filed. Since warning signs often deal with hazardous situations, there is a possibility of these two things happening.
- **Consequences** fall into three categories that are based upon their severity. The first consequence is a minor consequence. A minor consequence involves minor damage caused to the vehicle as a result of an accident. The second consequence would be any major ones. This may involve great damage to a vehicle and some physical injury caused to the motorist. Finally the last major category of a consequence is a severe consequence. A severe consequence would involve a fatally as a result of an accident.
- **Political** needs are often the ones that are the least stated. Signs are sometimes installed for other reason than what they are intended for, such as political influences. A local politician may want a sign installed because of complaints received from the public or to reduce the risk of potential lawsuits.

- **MUTCD** correct practice is the most important practice that should be followed. The reason of the MUTCD is to great uniformity among all traffic control devices and therefore all warning signs should be consistent with the correct practice stated in the MUTCD. This would include following all design, installation, placement and all other standards stated in the current issue of the MUTCD.
- **Local Practices** entails state MUTCD's as well as common practices in the area. It is important that signs follow the correct practice.

Survey Results

The survey conducted listed several factors that are involved in decisions made on the installation of warning signs. The fifteen engineers who participated in the survey were asked to assign a percentage to the factors provided to indicate their relative importance in relation to each other. The figure below represents the total percentage each factor received.

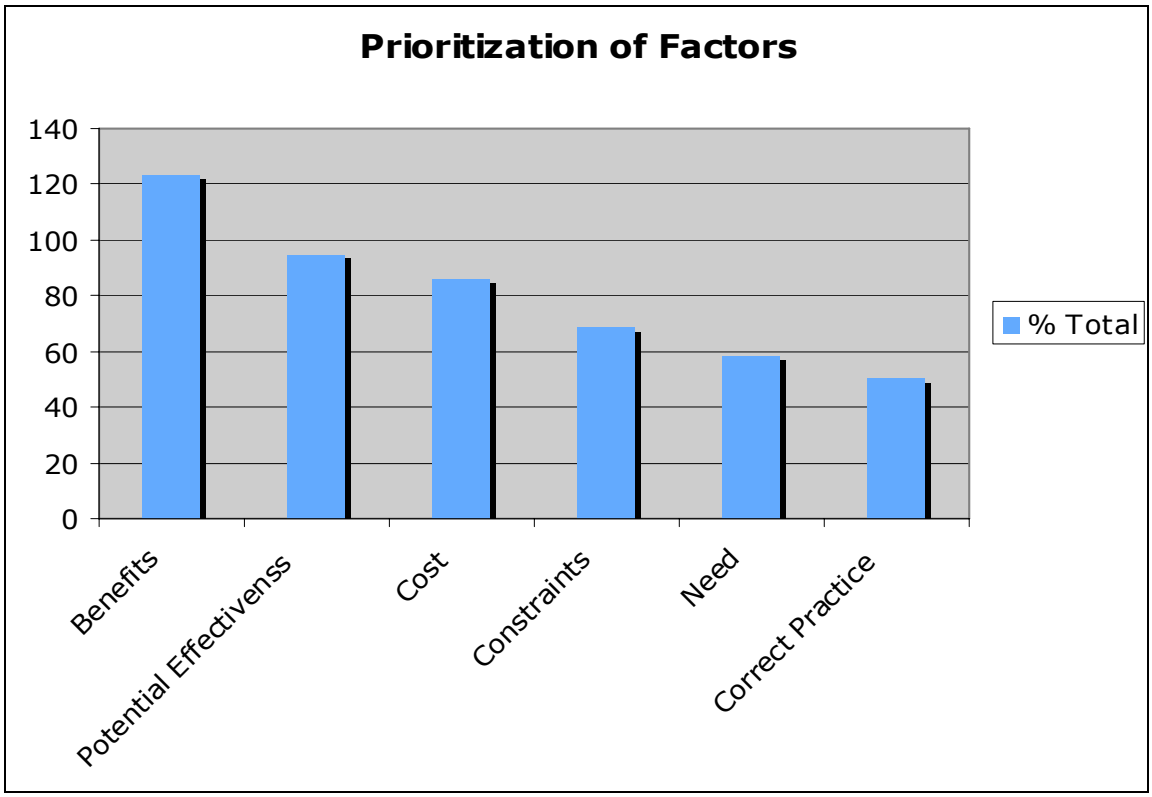

Figure 1. Survey Results

Literature Review

Once the survey was conducted, the next objective was to determine how the concepts of systems engineering and decision theory could be applied to developing a system for the selection of warning signs. This stage of the project involved an extensive amount of literature review because of the level of complexity of these subjects. The following sections will describe the concepts used in this research project in detail.

Manual on Uniform Traffic Control Devices

The MUTCD is one of the key documents in the field of transportation engineering because of the standards and warrants it contains for the design and application of traffic control devices. Although it may offer guidance on the use and application of devices, it offers limited guidelines for selecting warning signs. The MUTCD states that the decision to use a particular device at a particular location should be made on the basis of either an engineering study or the application of engineering judgment (Section 1A.09) *(1)*. By allowing this and not having a systematic selection process, it opens the door to inconsistency and misuse of warning signs.

Although the MUTCD may lack a defined process for selecting traffic control devices it has accomplished great uniformity. In the early years of highway signs, there was no consistency in their appearance or use. Signs were typically hand painted and took whatever appearance the creator thought would be most effective *(5)*. However as automobile users began to increase, traffic engineers began to realize the need for a consistent and uniform signing system. This eventually led to the development of the first national manual that addressed the appearance and application of traffic control devices *(5)*. Through many years of revision, research, collaborative engineering efforts, and implementation, the MUTCD has created a uniform standard on TCDs that provides excellent transportation conditions for motorist. By creating such great uniformity it has aided in the safe and efficient transportation of people and goods. The uniformity of traffic control devices is especially important for warning signs. It helps establish a level of familiarity and establishes a prior knowledge of warning signs, so that motorists are able to recognize and perform the desired operation.

Systems Engineering

Systems engineering is an interdisciplinary field of engineering that focuses on how complex engineering projects should be designed and managed *(6)*. It deals with work-processes and tools to handle this and overlap with both technical fields like control engineering and with project management *(7)*. Systems engineering's main responsibility is creating and executing an interdisciplinary process to ensure that the needs of the problems are met *(6)*. The process usually involves seven steps.

1. **State the problem**: This part of the process involves the description of the problem being addressed. The problem statement should express the needs that are being addressed and this information should be input by all those involved in the process.
2. **Investigate alternatives**: The alternatives are determined and evaluated based on overall performance on solving the problem. No alternative is likely to be the best so multicriteria decision-aiding techniques should be used to reveal the preferred alternatives. By incorporating this, it eliminates the possibility of decision makers choosing their predetermined solution.
3. **Model the system**: In this part of the process, a model is developed for the alternative designs. Then the model for the preferred alternative is expanded and used to help manage the system throughout its entire life cycle. For the application of this project, the model will be a preliminary flow chart, which should include the major criteria and sub criteria concerning the factors that are associated with warning signs.

4. **Integrate**: This part of the process is where everything that has been done for the project is brought together so that they work as a whole. This allows decision makers to see how everything ties together and allows for minor changes to be made so that the system works well together.
5. **Launch the system**: This step is meant for systems that require operation. In this step the system is running and producing outputs. In a manufacturing environment this means actually making things. However for the application of this project it would entail using the proposed flowchart for selecting a warning sign. This would be done by completing all the steps that are involved in the analytic hierarchy process.
6. **Assess performance**: For this set the decision maker would have to evaluate how well the analytic hierarchy process worked in selecting a warning sign. Did the process lead to a reasonable conclusion or is it one that was unexpected? The decision maker would have to decide what changes should be made so that a reasonable conclusion is reached through the process.
7. **Re-evaluate**: This is arguably the most important of the seven steps. For a century, engineers have used feedback to help control systems and improve their performance and effectiveness. Re-evaluate means observing the output and using the information received to modify the system and the inputs so that the process is a logical one *(6)*.

Like all processes, the systems engineering process is not one that is applicable to all processes. The above description of the Systems Engineering process is just one of the many that have been proposed and should be modified according to its application.

Since Systems Engineering is such a large and complex field, there are many techniques and tools that can be used. Perhaps the most fundamental technique is the flowchart, a graphical display composed of boxes representing individual components or subsystems of the complete systems, plus arrows from box to box to show how they interact *(7)*. This representation is very useful in the initial parts of a study and is essentially qualitative. A more effective approach to the flow chart would be the incorporation of a mathematical model, which consists of a set of some type of equations that would help describe the interactions quantitatively *(7)*. When it comes to systems engineering tools there is a great deal of diversity. The incorporation of different mathematical, statistical, logical, and decision methods are endless, which makes the discipline useful and unique.

Decision Theory

Decision theory is an interdisciplinary area of study that concerns mathematicians, statisticians, economists, philosophers, managers, politicians, psychologists and anyone else interested in analyses of decisions and their consequences *(8)*. It has developed since the middle of the 20th century through the contribution of several academic disciplines. There are two types of decision theories: normative and descriptive *(8)*. A normative decision theory is a theory about how decisions should be made, and descriptive theory is a theory about how decisions are actually made *(8)*. Decision theory can apply to conditions of certainty, risk, or uncertainty. It also recognizes that the ranking produced by using a criterion has to be consistent with the decision maker's objectives and preferences. Decision theory offers a vast realm of techniques and procedures that introduce decision makers into models of decision. The theory is not meant

to define objectives, design alternatives or assess consequences but define a simple procedure for selecting a choice.

There have been many stages of the evolution of decision theory. The first general stage was started by the great enlightenment philosopher Condorcet in 1793 *(8)*. He divided the decision process into three stages: discuss the principles that will serve as the basis for the decision, the decision is reduced to a choice between a manageable set of alternatives, and the third part is the actual choice between these alternatives *(8)*. After this stage of evolution decision it was further evolved by the efforts of John Dewey, Herbert Simon, and Brim et al. Their proposals were all sequential in the sense that they divided the decision process into parts that were always in the same order. Their decision process consisted of these steps:

1. Identification of the problem,
2. Obtaining necessary information,
3. Production of possible solutions,
4. Evaluation of such solutions, and
5. Selection of a strategy for performance *(8)*.

Another influential evolution of decision theory was contributed by Mintzberg, Raisinghani, and Theoret (1976). They viewed the decision process of consisting of distinct phases, but these phases did not have a sequential relationship. Their decision process consisted of three major phases which were: identification, development and selection *(8)*. Although they felt that their process was not sequential, it had many similarities to the previous processes.

Most people often make choices out of habit or tradition, without going through the decision-making process steps systematically *(8)*. Decisions may be made under social pressure or time constraints which interfer with a careful consideration of the options and consequences. Decisions may also be influenced by decision maker's emotions, which often lead to unwanted results. For this reason decision theory is used by decision makers when faced with difficult and conflicting decisions such as decisions made on traffic control devices. One example to be looked at is the practical application of the normative decision theory, which is called decision analysis. Decision analysis is aimed at finding tools, methodologies and software to help people make better decisions. One discipline of this is the Multi-Criteria Decision Analysis (MCDA), or Multi Criteria Decision Making (MCDM).

Multi-Criteria Decision Methods

Multi-Criteria Decision Methods (MCDM) is a discipline that helps decision makers make a decision when faced with a problem that has numerous and conflicting criteria *(9)*. MCDM finds a way to evaluate each conflict and derives a way to come to a compromise where each conflicting criteria is taken into consideration *(9)*. The Analytic Hierarchy Process (AHP) is a MCDM method that was applied in developing the conceptual framework for the selection of warning signs. The Analytic Hierarchy Process is a basic approach to decision making, that it is designed to cope with both the rational and intuitive to select the best from a number of alternatives evaluated with respect to several criteria *(10)*. It was developed by Dr. Thomas L.

Saaty in 1970s, and this process has been used to assist in numerous corporate and government decisions *(10)*.

Part of the reason the analytic hierarchy process is an effective way to determine the best alternative for any decision lies in the structure. When constructing hierarchies the decision maker must be sure to include enough relevant detail so that the problem is represented as thoroughly as possible *(10)*. However it should not be so thorough that it loses sensitivity to change in certain elements. When constructing the hierarchy the decision maker should first ask the question: "Can I compare the elements on a lower level in terms of some or all of the elements on the next higher level?" *(11)*. This will help make comparisons reasonable and possible. It will also help in selecting the set of the major criteria that needs to be addressed as well as the set of sub criteria that needs to be taken into consideration in the decision making process. By arranging the goals, attributes, and issues in a hierarchy, it provides the decision maker with an overall view of the complex relationships present in the situation and judgment process. Aside from this, it allows the decision maker to assess whether he or she is comparing issues of the same order of magnitude and if it is consistent with the overall goal *(11)*.

By developing the hierarchy for the problem being addressed, it allows for the elements to be compared two by two *(11)*. The decision makers compare the set of criteria against each other and then the same pairwise comparison process is applied to the sub criteria. The comparison is performed by determining how important an element is when compared to one another and how important it is with respect to the goal. Things change a bit when we get to the alternatives row. Here, the alternatives are compared pair-by-pair with respect to the covering criterion of the group, which is the node directly above them in the hierarchy *(12)*. What we are doing here is evaluating the models under consideration with respect to the set of sub criteria. In order for the comparison to be quantified each comparison will need to be assigned an intensity value. These values will help assign priorities values to each set of criteria and help in calculating the significance of each element to the overall goal of the decision.

Assigning intensity values to each set of pairwise comparison allows for the priority vector, also known as the normalized Eigen vector, to be computed *(12)*. The Eigen value is very important for many applications in science and engineering because of its numerous applications. The normalized principal Eigen vector is applied in the Analytic Hierarchy Process so that all the relative weights among the things that are compared are shown. Aside from illustrating the relative weights of each set of criteria in relation to the overall goal, Eigen values help check the consistency of the alternative selected *(12)*.

Although the Analytic Hierarchy Process may seem like it is very complex, it is rather simple. One of the most important parts of the process is that the following three steps are accomplished:

- State the objective,
- Define the criteria, and
- Pick the alternatives.

ANALYTIC HIERARCHY PROCESS EXAMPLE

In order to better understand the AHP process, the following section is dedicated to the explanation of the process that was developed as a result of this research. As mentioned earlier, there are three main steps that need to be determined to complete the AHP process.

- State the objective (should a warning sign be installed?),
- Define the criteria (benefits, potential effectiveness, cost, constraints, need, correct practice), and
- Pick the alternatives (install or do not install).

The following example provides a step by step process for the use of the analytic hierarchy process. The following created scenario is one that could be faced by a practitioner. The basic scenario is whether a series of Slippery When Wet signs should be installed on a six-mile stretch of roadway where the pavement friction value on the road has dropped below the agency threshold value. The agency has programmed the road for resurfacing, but it will be eighteen months before this is completed. The following process can be used by a practitioner in helping them decide whether to install the sign. The figure below was developed for the selection of warning signs using the research results obtained through this project.

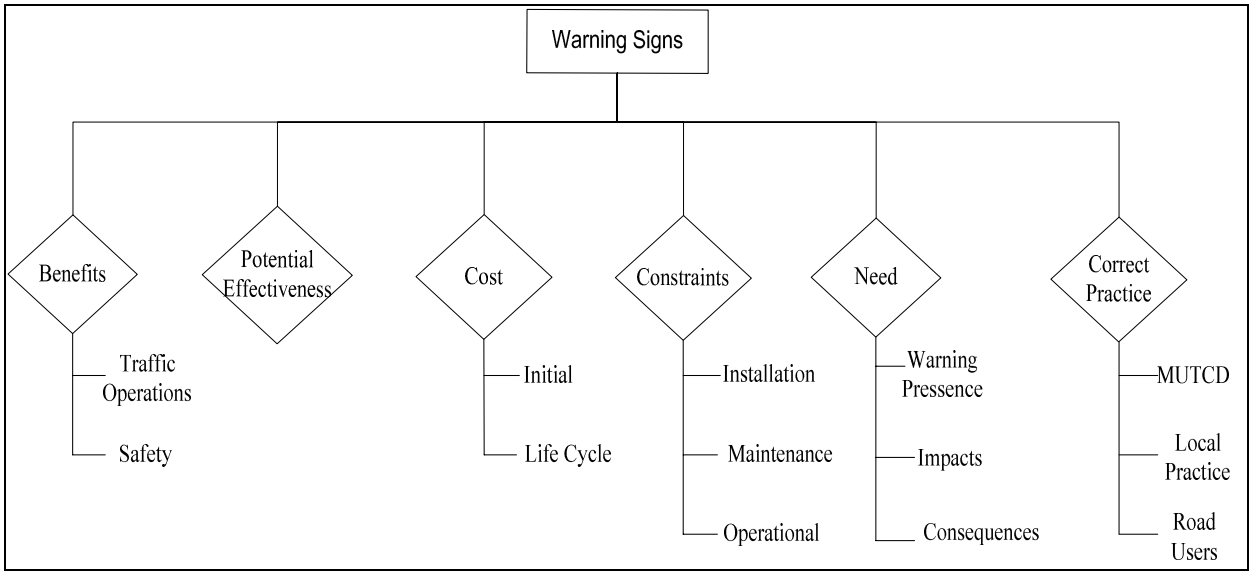

Figure 2. AHP Hierarchy for Slippery When Wet Sign

The next step of the Analytic Hierarchy Process is to synthesize all of the criteria established in the previous step *(12)*. This is done to determine the relative ranking of alternatives. To determine the relative importance of the criteria, a series of pairwise comparisons were performed. In performing pairwise comparisons, the decision maker who participated in the research was asked to evaluate each criterion in pairs and assign an intensity value to the comparison. The intensity values found in Table 1 are those developed by Dr. Thomas L. Saaty.

Table 1. Fundamental Scale for Pairwise Comparisons *(12)*

Intensity of Importance	Definition of Importance	Explanation
1	Equal Importance	Two elements contribute equally to the objective
3	Moderate Importance	Experience and judgment slightly favor one element over the other
5	Strong Importance	Experience and judgment strongly favor one element over the other
7	Very Strong Importance	One element is favored very strongly over another, its dominance is demonstrated in practice
9	Extreme Importance	The evidence favoring one element over another is of the highest possible order of affirmation
Intensities of 2, 4, 6, and 8 can be used to express intermediate values. Intensities 1.1, 1.2, 1.3, etc. can be used for elements that are very close in importance.		

Table 2 contains the values assigned by the engineer making the decision on the installation of the Slippery When Wet sign. An "A" in the "More Important" column indicates that the benefit under Criteria A is more important in a pairwise comparison than the benefit under Criteria B. The intensity indicates the importance of the relationship.

Table 2. Level One Pairwise Comparison

Criteria		More Important	Intensity
A	B		
Benefits	Potential Effectiveness	A	6
Benefits	Cost	A	3
Benefits	Constraints	A	2
Benefits	Need	A	5
Benefits	Correct Practice	A	6
Potential Effectiveness	Cost	A	4
Potential Effectiveness	Constraints	A	3
Potential Effectiveness	Need	A	7
Potential Effectiveness	Correct Practice	A	4
Cost	Constraints	A	5
Cost	Need	A	7
Cost	Correct Practice	A	4
Constraints	Need	A	8
Constraints	Correct Practice	A	6
Need	Correct Practice	A	7

The intensity values from Table 2 are inserted in a matrix that establishes a numerical relationship between any two criteria. Table 3 is a representation of the decision maker's pairwise comparison values in matrix form. A value greater than 1 in a cell indicates that the criteria in the first column is more important than the criteria in the first row. All values in the

upper right portion of the matrix diagonal (the diagonal is defined by a cell value of 1) have values greater than 1. The cell values in the lower left portion of Table 3 were established by using the reciprocal values of the upper diagonal *(12)*. If A_{ij} is the element of row ***i*** column ***j*** of the matrix, then the lower diagonal is filled using the formula below. The sum of each of the columns is included in this table because it will be used later in the process to compute the Priority vector.

$$A_{ij} = \frac{1}{A_{ij}}$$

Table 3. Complete Comparison Matrix

Criteria	Benefits	Potential Effectiveness	Cost	Constraints	Need	Correct Practice
Benefits	1	6	3	2	5	6
Potential Eff.	1/6	1	4	3	7	4
Cost	1/3	1/4	1	5	7	4
Constraints	1/2	1/3	1/4	1	8	6
Need	1/5	1/7	1/7	1/8	1	7
Correct Practice	1/6	1/4	1/4	1/6	1/7	1
Sum	2.367	7.976	8.643	11.292	28.143	28.000

The next step is to compute the priority vector. To do this first one divides each cell of the matrix by the sum of its column. (i.e., 1.00/2.367 = 0.423) *(12)*. By doing this, we have the normalized relative weight, and the result is that the sum of each column should be 1. Once this done the priority vector can be computed by averaging the values across the rows *(12)*.

Table 4. Normalized Relative Weight with Priority Vector Values

Criteria	Benefits	Potential Effectiveness	Cost	Constraints	Need	Correct Practice	Priority Vector
Benefits	0.423	0.752	0.347	0.177	0.178	0.214	34.85%
Potential Eff.	0.070	0.125	0.463	0.266	0.249	0.143	21.93%
Cost	0.141	0.031	0.116	0.443	0.249	0.143	18.70%
Constraints	0.211	0.042	0.029	0.089	0.284	0.214	14.48%
Need	0.085	0.018	0.017	0.011	0.036	0.250	6.93%
Correct Practice	0.070	0.031	0.029	0.015	0.005	0.036	3.10%
Sum	1.000	1.000	1.000	1.000	1.000	1.000	100.00%

At this point, all the comparisons for the level one criterion have been made, and the priority vectors have been derived. The next step is to compute the local priority vectors of the subcriteria found in level two *(12)*. The items in each group of subcriteria should be pairwise compared and then the manipulation of these results will provide the local priorities. Table 6 provides the pairwise comparison values assigned by the engineer for each of the subcriteria.

Table 5. Level Two Pairwise Comparisons

Criteria		More Important	Intensity
A	**B**		
Traffic Operations	Safety	B	8
Initial	Life Cycle	A	4
Installation	Maintenance	B	4
Installation	Operational	B	5
Maintenance	Operational	A	3
Warning Presence	Impacts	A	5
Warning Presence	Consequences	B	7
Impacts	Consequences	A	2
MUTCD	Local Practice	A	7
MUTCD	Road Users	A	3
Local Practice	Road Users	A	1

Once the decision maker has determined the intensity values, the calculation of the local priorities is now possible. Local priorities are the overall priorities of each factor with respect to its specific level *(12)*. The priorities in each group should always total 100 percent. The calculation is the same as the previous example (Table 5) except that the comparisons are made for a smaller number of subcriteria. Table 7 illustrates the calculation of the priority vectors for traffic operations and safety. Table 8 contains all the values for each set of subcriteria.

Table 6. Subcriteria Comparison Maxtrix for Traffic Operations and Safety

Subcriteria	Traffic Operations	Safety		Subcriteria	Traffic Operations	Safety	Priority Vector
Traffic Operations	1	1/8		Traffic Operations	1/9=0.1111	0.125/1.125=0.1111	11.11%
Safety	8	1		Safety	8/9=0.8888	1/1.125=0.8888	88.89%
Sum	9	1.125		Sum	1	1	100%

The next step in determining whether the sign should be installed is to calculate the global priority of each subcriteria *(12)*. This will show us the priority of each subcriteria with respect to the goal. The way to calculate the global priority is to multiply the priority vector from level one by the local priority vector from level two (i.e., $0.3485 \times 0.111 = 0.0387$) *(12)*. Table 9 indicates the global priorities.

Based on the judgments entered by the engineer, the AHP has derived the priorities for the factors against which each of the two alternatives will be compared *(12)*. They are shown, from highest to lowest, in Table 10. Notice that Benefits, Cost, Constraints, Need, and Correct Practice will not be evaluated directly, but that each of their Subcriteria will be evaluated on its own.

Table 7. Subcriteria Local Priorities

Subcriteria	Local Priority		Subcriteria	Local Priority
Traffic Operations	11.11%		Warning Presence	31.22%
Safety	88.89%		Impacts	27.15%
Sum	100.00%		Consequences	41.63%
			Sum	100.00%
Initial	80.00%			
Life Cycle	20.00%		MUTCD	68.51%
Sum	100.00%		Local Practice	13.60%
			Road Users	17.90%
Installation	10.18%		Sum	100.00%
Maintenance	58.20%			
Operational	31.62%			
Sum	100.00%			

Table 8. Subcriteria Global Priorities

Subcriteria	Global Priority		Subcriteria	Global Priority
Traffic Operations	3.87%		Operational	4.58%
Safety	30.98%		Warning Presence	2.16%
Potential Effectiveness	21.93%		Impacts	1.88%
Initial	14.96%		Consequences	2.89%
Life Cycle	3.74%		MUTCD	2.12%
Installation	1.47%		Local Practices	0.42%
Maintenance	8.43%		Road Users	0.55%

Table 9. AHP Global Priorities

Criteria	Subcriteria	Priority
Benefits	Safety	30.98%
Potential Effectiveness	none	21.93%
Cost	Initial	14.96%
Constraints	Maintenance	8.43%
Constraints	Operational	4.58%
Benefits	Traffic Operations	3.87%
Cost	Life Cycle	3.74%
Need	Consequences	2.89%
Need	Warning Presence	2.16%
Correct Practice	MUTCD	2.12%
Need	Impacts	1.88%
Constraints	Installation	1.47%
Correct Practice	Road Users	0.55%
Correct Practice	Local Practices	0.42%

The next step is to evaluate each of the alternatives with respect to these factors. In the technical language of AHP, we will pairwise compare the alternatives with respect to their covering criteria and use this information to calculate local and global priorities for each of the alternatives *(12)*. Table 11 indicates these intensities.

Table 10. Pairwise Comparison of Alternatives with Respect to the Factors

Factors	Alternative		Intensity
	A (Install)	B (Do Not Install)	
Safety	X		6
Initial		X	3
Potential Effectiveness	X		3
Maintenance		X	4
Operational		X	8
Traffic Operations	X		3
Life Cycle		X	7
Consequences		X	2
Warning Presence	X		4
MUTCD	X		2
Impacts	X		6
Installation		X	1
Road Users	X		1
Local Practices		X	2

The calculations of the local priorities are computed in the same fashion as before. The global priorities are calculated using the local priorities for this table and the global priorities from Table 9. The result is the global priorities shown in Table 12. For any one factor, the sum of the "install" and "do not install" priority percentages is equal to global priority percentage from Table 9.

At the end of all these calculations, all the global priorities are arranged for each of the alternatives. Their grand total is 100 percent, which is identical to the priority of the goal. Each alternative has a global priority corresponding to its fit to the engineer's judgments about all the aspects involved. Table 13 provides a summary of the global priorities of all the alternatives.

Table 11. Local and Global Priority of Alternatives with Respect to the Factors

Factors	Alternative	Local Priority	Global Priority
Traffic Operations	Install	75.00%	2.90%
	Do Not Install	25.00%	0.97%
Safety	Install	85.72%	26.56%
	Do Not Install	14.29%	4.43%
Potential Effectiveness	Install	75.00%	16.45%
	Do Not Install	25.00%	5.48%
Initial	Install	25.00%	3.74%
	Do Not Install	75.00%	11.22%
Life Cycle	Install	12.50%	0.47%
	Do Not Install	87.50%	3.27%
Installation	Install	50.00%	0.74%
	Do Not Install	50.00%	0.74%
Maintenance	Install	20.00%	1.69%
	Do Not Install	80.00%	6.74%
Operational	Install	11.11%	0.51%
	Do Not Install	88.89%	4.07%
Warning Presence	Install	80.00%	1.73%
	Do Not Install	20.00%	0.43%
Impacts	Install	85.71%	1.61%
	Do Not Install	14.29%	0.27%
Consequences	Install	33.33%	0.96%
	Do Not Install	66.67%	1.93%
MUTCD	Install	66.67%	1.41%
	Do Not Install	33.33%	0.71%
Local Practices	Install	33.33%	0.14%
	Do Not Install	66.67%	0.28%
Road Users	Install	50.00%	0.28%
	Do Not Install	50.00%	0.28%

Table 12. Decision to Install a Slippery When Wet Sign

Alternatives	Benefits		Potential Effectiveness	Cost		Constraints			Need			Correct Practice			Total
	Traffic Operations	Safety		Initial	Life Cycle	Instl.	Maint.	Oper.	Warn. Pres.	Impacts	Conseq.	MUTCD	Local Pract.	Road Users	
Install	2.90%	26.56%	16.45%	3.74%	0.47%	0.74%	1.69%	0.51%	1.73%	1.61%	0.96%	1.41%	0.14%	0.28%	59.19%
Do Not Install	0.97%	4.43%	5.48%	11.22%	3.27%	0.74%	6.74%	4.07%	0.43%	0.27%	1.93%	0.71%	0.28%	0.28%	40.82%
TOTALS	3.87%	30.99%	21.93%	14.96%	3.74%	1.48%	8.43%	4.58%	2.16%	1.88%	2.89%	2.12%	0.42%	0.56%	100.00%
	34.86%			18.70%		14.49%			6.93%			3.10%			100.00%
	100.00%														100.00%

The decision that was reached based on the engineer's judgments was to install the Slippery When Wet sign. This alternative had a global priority of 59.19 percent which contributed most to the goal of the hierarchy.

After the decision is reached to install the Slippery When Wet signs, it is now time to compute the consistency of the engineer's judgment. To calculate the consistency of the judgment, we will need to go back to the level one comparison matrices. In order to compute the consistency of the decision maker's answers we will need to obtain the Principal Eigen value *(12)*. This value is calculated using the summation of the products from the sum of the columns from the complete comparison matrix in Table 4 and its corresponding priority vector in Table 5 *(12)*. The calculation is shown below.

$$\lambda_{\max} = 2.360 \times 0.3485 + 7.976 \times 0.2196 + 8.643 \times 0.1870 + 11.292 \times 0.1448 + 28.143 \times 0.0693 + 28.00 \times 0.0310 = 8.6437$$

Once we have calculated the Principal Eigen value, we can now calculate the Consistency Index, which is one step closer to measuring the overall consistency of the decision makers answers for the level one pairwise comparisons *(12)*. To compute the Consistency Index we use the formula below where n is the number of critieria. In this case, there are six criteria (benefits, potential effectiveness, cost, constraints, need, correct practice).

$$CI = \frac{\lambda_{\max} - n}{n-1} = \frac{8.6437 - 6}{6-1} = 0.5287$$

Knowing the Consistency Index, the new question is how do we use this index? Dr. Saaty proposed that we use this index by comparing it with an appropriate one. The appropriate

Consistency Index is called Random Consistency Index (RI) *(12)*. Dr. Saaty developed the values in this index by performing many calculations using various sample size.

Table 13. Random Consistency Index (RI) *(12)*

N	1	2	3	4	5	6	7	8	9	10
RI	0	0	.58	.9	1.12	1.24	1.32	1.41	1.45	1.49

After Dr. Saaty developed this table, he proposed what is called the Consistency Ratio, which is a comparison between Consistency Index and Random Consistency Index *(12)*. The formula for this is:

$$CR = \frac{CI}{RI} = \frac{0.5287}{1.24} = 42.64\%.$$

If the value of the Consistency Ratio is smaller or equal to 10%, the inconsistency is acceptable *(12)*. If the Consistency Ratio is greater than 10%, we need to revise the subjective judgment *(12)*. Therefore with this knowledge, we can conclude that the decision maker involved in this example was not consistent in his/her judgment to install the Slippery When Wet sign.

CONCLUSIONS

After the development of the conceptual framework for the selection of warning signs, there are several conclusions that were reached. There is a need to further develop this concept and continue to investigate the incorporation of a system for the selection of warning signs. In investigating the application of these concepts, decision theory concepts seemed to have the most application for creating the framework. Decision theory offers a variety of analytical tools, such as statistical analysis, which can be applied in creating optimal decision making methods for practitioners to use in the selection of warning signs.

Although the application of systems engineering was useful in developing the framework for the conceptual model, it lacked the tools necessary for evaluating the selection of warning signs. It provided a great set of steps to follow for the development of a system but offered limited analytical tools. The incorporation of systems engineering seems to have more of an application for the development of a system that produces tangible objects. Systems engineering concepts still should be used in creating logical steps to develop a process for the selection of warning signs.

RECOMMENDATIONS

As the project is developed, the future researcher should continue to investigate the concepts of systems engineering and decision theory. Future research should investigate the application of other normative decision theory methods such as the one used in this project. In doing so the investigator should look at incorporating methods which involve statistical tools for the analysis of a decision. By incorporating statistical tools, it will better define interrelationships between factors affecting the decisions made on warning signs.

Future researchers should look at finding ways to improve the analytic hierarchy process used in this effort. The process provided a great way to evaluate each criterion in relation to each other but there were many problems associated with the calculation of the consistency index. Many times the consistency ratio would state that the judgmental process was inconsistent when it may have not been. Due to this, future investigators should look at other ways of calculating the consistency of judgments made or look at another method to replace the analytic hierarchy process.

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APPENDIX A

SURVEY OF PROFESSIONALS
SYSTEMS ENGINEERING APPROACH FOR
SELECTING TRAFFIC CONTROL DEVICES

Note to Participants: This survey is being conducted as part of an exploratory research project investigating the potential to develop a systems engineering approach for the selection of traffic control devices. Your responses will help the researcher identify critical factors that should be considered in selecting devices and the relative importance of those factors. Participation in the survey is voluntary. Individual survey responses will not be included in any reports and respondents will not be identified by name in any reporting of the results. Please return the survey to Gene Hawkins at 979-845-9294, fax 979-845-6481, or gene-h@tamu.edu. You can contact Gene Hawkins if you have any questions.

Name: Phone:
Organization: Email:

1. What type of organization do you work for?
   - ❑ Government – federal
   - ❑ Government – state
   - ❑ Government – city
   - ❑ Government – county
   - ❑ Government – other (please identify) _____
   - ❑ Consultant
   - ❑ Industry
   - ❑ Research
   - ❑ Other (please identify) _____

2. How familiar are you with issues related to the selection and use of traffic control devices?
   - ❑ Very
   - ❑ Somewhat
   - ❑ Vaguely
   - ❑ Not much
   - ❑ Not at all

3. Do you believe that the *Manual on Uniform Traffic Control Devices* provides adequate guidance for practitioners to make informed and appropriate decisions regarding whether a particular device should be used in a specific situation?

4. To what extent does the decision making process change as a function of the type of devices you are considering for installation?

5. Do you believe that there would be a benefit to developing a systematic procedure to guide practitioners through the decision making process relative to the need for a traffic control device?

6. Please assign a percentage to each of the following factors to indicate the relative importance of the factor relative to the other factors. The total should add up to 100 percent. You can assign 0 percent to a factor if you think it is not pertinent.

Selection Factor	**Relative Importance (Percent)**	**Comments**
Initial cost of the device		
Life cycle cost of the device		
Installation demands of the device		
Operational demands of the device		
Maintenance demands of the device		
Potential of the device to improve safety		
Potential of the device to improve operations/mobility		
Significance of situation the device is addressing		
Lack of other alternatives to address the need		
Similarity in use of the device with related devices		
Ability of road users to understand the device		
Likelihood of device attaining the desired intent		
Compliance with state/national standards/MUTCD		
Potential of device to reduce tort claims/lawsuits		
Desires of elected officials to install the device		
Other factors (list below and assign percentage)		
Total	100%	

Estimating Corridor Travel Time from Arterial Traffic Volume

Prepared for
Undergraduate Transportation Scholars Program

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August 7, 2008

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SUMMARY

Dealing with congestion has become routine for the average person. With growing urban areas and population, congestion not only increases in size but in density thus increasing travel time. Travel time is of high importance to travelers. Travelers tend to base their travel plans on the time it takes to get to their destination and attempt to avoid obstructions, such as traffic congestion, to make it in a timely manner. Many cities and state agencies provide travelers with travel time estimates and information displayed on message boards regarding traffic conditions to assist with commute.

The scope of this project was to estimate corridor travel time from arterial traffic volumes. In doing this, researchers are able to predict travel time at various traffic volumes and can make changes to signals timings and roadways to meet different levels of demand. From this analysis, researchers were able to understand the flow on University Drive and are now able to predict travel times for various events and traffic volumes. The results and models obtained from this study can be used to predict flow conditions for signalized corridors in mid-sized urban communities. With additional research and more recent data collection, the models generated can be used to identify threshold demand level when congestion level increases drastically and mitigation becomes necessary.

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INTRODUCTION

Traveling is the change in location from point A to point B and to make that change in location takes time. Many state departments of transportation (DOTs) have message signs located on major highways to inform travelers of traffic conditions, attempting to ease congestion and persuading motorists to take alternate routes to their destinations, if necessary. However, there are many causes of congestion and delay that can impede free-flow travel—too many vehicles, construction, population growth, crashes, traffic control, vehicle breakdown, etc. All of these factors can cause congestion, and that costs the motoring public time, fuel, and money. The *2007 Urban Mobility Report (145)* states in 2005, the most current data available, that congestion wasted 2.9 billion gallons of fuel, 4.2 billion hours of time, and $78 billion of delay and fuel cost. The transportation system of an urban area includes both uninterrupted (freeways) and interrupted (arterial) facilities. Typically, as major freeways in an urban area began to build traffic volume and become congested, arterial roads become congested also. With increases in traffic volume and incidents, recurring and non-recurring congestion will increase—increasing costs to the motoring public. As transportation agencies struggle with limited financial resources, the efficient management of existing facilities is imperative. There is a need for planning tools to better understand traffic congestion impacts. Planning tools and models that estimate arterial travel time are especially valuable because of the multitude of factors that can contribute to congestion in the arterial environment (e.g., traffic signals, access density).

BACKGROUND

The motivation of this project is monitoring mobility as well as transportation system performance measurement. Transportation professionals need to understand how their system is operating. A common concern is identifying the level of congestion on a roadway. Travel time data are often used to answer this question. Transportation professionals can use travel time to identify locations that may be congested, and to prioritize where to allocate resources for roadway improvements. Some estimation models use historic traffic data while others rely on real-time traffic information *(2)*. Traffic volume is one variable representing demand that can be used to estimate travel time, and practitioners can obtain volume data from permanent roadway instrumentation (e.g., loops), which collect real-time data, or from temporary tube counters. As an indicator of mobility, travel time is often presented relative to the free-flow travel time and realizing it increases with demand.

Travel time is easily understood by a large range of stakeholders—both technical and non-technical audiences. Communicating congestion levels to the general public in terms of travel time is effective. There are numerous research projects and references accessible to assist with a more in depth understanding of travel time such as the Federal Highway Administration's (FHWA's*) Travel Time Data Collection Handbook (2)*, the Texas Transportation Institute's (TTI's) *Urban Mobility Report (1)*, and National Cooperative Highway Research Program Report 398, *Quantifying Congestion (3).*

The Bureau of Public Roads (BPR) is the former name of FHWA and was developed in 1918 *(4)*. The Bureau produced a travel time function; however, it is for freeway segment and does not consider signal density, vehicle progression, or cross-street effects (e.g., side-street volume). The

Bureau's model is commonly used for planning purposes. The proposed work establishes relationships between travel time and traffic volume that can be used in models researchers are developing for the mobility study of signalized corridors.

Limitations of Existing Models

There are several limitations of existing models. The following common limitations are discussed below:

- Limited field data;
- Need for extensive inputs;
- Link-based analysis units;
- Estimating congested conditions; and
- Transferability.

The first limitation is limited field data. Some models use micro-simulation to calibrate their model, with limited *(5)*, or no *(6,8)*, actual field data. There is a need for models that are calibrated with extensive field data. The proposed work develops a model using extensive field data.

Some models require practitioners to collect extensive inputs. This can be difficult if practitioners do not have the data readily available. The *Highway Capacity Manual* lists, among others, signal control, speed, delay, saturation flow rate and lost time, and queuing as parameters for interrupted traffic flow *(7)*. Related models in the literature require extensive inputs that may not be readily available to practitioners *(8,9).*To simplify the process, there is a need to investigate relationships that rely on fewer, and less complicated, inputs.

Another limitation is that many models use link-based analysis procedures. A link may be defined as a section of roadway from signal to signal. Such models estimate traffic operations for each link, and then corridor travel time is estimated by "adding up" each link. While this does make some models more transferable, there is still a question of whether this "addition" really equates to actual travel time through the corridor. Study of the correlations between arterial through link travel times and flow values reveal that detector data have limited significance to the process of forecasting arterial link travel times *(5)*. In addition, researchers have developed link-based methods for dynamic assignment on signalized networks, which include extensive inputs *(9)*. This suggests a corridor approach may be more effective.

Another limitation is estimating congestion conditions. Many models are acceptable for estimating travel time for non-congested (free-flow) conditions, but have increased error during congested conditions. A previous distribution-free model was applied to free- flow conditions, but has the ability to be converted to a time-dependent form to account for congestion *(6)*. Congested conditions are of primary interest for practitioners who want to report congested conditions. There is a need to investigate and incorporate travel time estimates in the arterial environment for both congested and non-congested conditions.

A final limitation is transferability. All models are calibrated and developed for specific operating conditions. This means that the models cannot be applied to conditions beyond those

for which the model was calibrated, without introducing some error. While this proposed work investigates only one signal density for the corridor discussed below, future work will investigate other signal densities, therefore increasing transferability.

GOAL AND OBJECTIVES

The goal reached for this research was to estimate corridor travel time from arterial traffic volumes. The goal was reached by completing the following two objectives:

1. Developing a relationship between corridor travel time and average traffic volume.
2. Investigating variability of travel time estimates for varying traffic volume.

The work plan below highlights the key tasks performed that satisfied these objectives.

RESEARCH TASKS

This portion of the research is composed of five parts. Each part needed an extensive amount of time dedicated to it so that the information and results were accurate and presentable.

Literature Review

Previous work includes many existing models for estimating arterial travel time. During this task, researchers performed a literature review to identify previous relevant research on estimating travel time in an arterial environment. Researchers gained a better understanding of the limitations of existing models mentioned in the "BACKGROUND" of this report. Researchers also reviewed literature on the related topic of travel time data collection. A review of more literature will be an advantage to get a better understanding of limitations and existing models previously discussed as well as travel time data.

Data Collection

The test corridor for this study is University Drive (FM 60), located in College Station, Texas. The roadway is 2.55 miles from SH 6 East Frontage Road to Ireland Street. There are 12 signalized intersections, and the signal density is 4.7 signals per mile. The signalized intersections do not have uniform spacing, and they are closer on the west end of the corridor. There are three through lanes both in the eastbound and westbound direction along the corridor and left-and right-hand turn lanes at the major signalized intersections. The median treatment varies along the corridor with raised medians near SH 6 and no medians near campus. The land use along the corridor includes several shopping centers, restaurants, and hotels. The west end of the corridor is along the north side of the Texas A&M University campus. With the population growth of the community and the university, recurring congestion is growing.

Travel time runs were previously collected on a Wednesday and Thursday morning during the hours of 6:00 a.m. to 10:00 a.m., and on a Saturday before a football game during the hours of 10:00 a.m. and 1:00 p.m. This equates to 11 hours of travel time data. The travel time runs were collected using global positioning system (GPS). Researchers collected 283 runs eastbound and 280 runs westbound

GPS-instrumented test vehicles used the "floating car" driving method. The "floating car" test vehicle driving style is when the driver "floats" with the traffic by attempting to safely pass as many vehicles as pass the test vehicle *(2)*. There were 40 locations of tube counts to collect average daily traffic (ADT) between major signalized intersections and at major cross streets. Video was recorded at major intersections to obtain queue data at signals. This will be used for the CORSIM model calibration.

During this task, the primary researcher became familiar with the previous data collection to facilitate understanding for data reduction performed in the next task.

Data Reduction

The travel time data used in this study are based on both field observations (GPS runs) and traffic simulation. Travel time estimation was performed using a calibrated CORridor SIMulation (CORSIM) model. CORSIM is a well-known, widely used micro-simulation tool. Researchers calibrated the CORSIM model with field data, including travel time runs, signalized intersection queue data, traffic volumes, and signal timing plans. This was done by matching queue lengths, at intersections, travel time values from simulation with field observations through adjusting simulation parameters. A calibrated CORSIM model was developed to allow analysis of corridor travel time given varying traffic volume inputs.

Selected travel time runs that were collected for the entire corridor for both directions were used to calibrate this model. The travel time run data were inserted into PC-travel, a travel time and delay data analysis software, created by JAMAR. The travel time data were summarized in Excel. The ADTs, average daily traffic counts, were also entered into Excel. With 11 hours of data, there are twenty-two 30-minute periods. A peak period from 8:00 a.m. to 8:30 a.m. was selected from Wednesday, November 8, 2006, to initially calibrate the model. From that 30-minute period, 21 other cases were created by using average daily traffic volumes, as well as signal timings.

Using these 22 cases, researchers ran each case eleven times to produce 242 cases. Each run will include both the eastbound and westbound directions to obtain 484 observations of traffic volume and travel time. Four extreme cases were created to fill gaps, if necessary, using the data from the 8:00-8:30 a.m. peak period as the baseline condition. These four extreme cases were run eleven times in both directions to produce 88 extreme cases. Researchers then went into CORSIM's output screen and pulled out corridor travel time data, and used the data to plot the relationship between travel time and volume for all 572 total cases.

The four extreme cases, as stated above, were created using the data from the 8:00-8:30a.m. peak period. Extreme case one was formed by making a 20% volume increase for major inputs and a 0% for minor inputs. Extreme case two kept the volume increase for major inputs at 20% and increased the minor inputs to 10%. Extreme case three increased the volume for major inputs to 30% with the minor inputs remaining at 10%. Lastly, extreme case four had a 40% volume increase for major inputs and remained 10% for minor inputs. Major and minor inputs for the extreme cases are as follows:

- Major inputs- include Texas northbound and southbound entrance to University Drive and east and west end entrances to University Drive
- Minor inputs- include all other streets on the twelve signalized corridor.

Researchers defined the corridor travel time along University Drive as the amount of time spent in the entire corridor from entrance to exit which is the sum of the eleven segments. The corridor volume was defined as the average through volume on links. An example of what links were used to get the average corridor volumes and corridor travel times are shown in Table 1.

Table 1. Example of the Links Used to Calculate Average Corridor Volume and Corridor Travel Time for Both Directions

Wed 6:00-6:30am							
Original			**Link Volume (vehicles)**		**Free Flow Time (distance/free flow speed)**	**Travel Time (vehicle-minutes)**	**Link Travel Time (seconds)**
	All	Left turn	Through	Right Turn	Through	Through	Through
East to Ireland	1498	76	1353	69	14.29	895.70	39.7
East to Spence	1399	36	1333	30	19.51	384.72	17.3
East to College	1445	132	1040	273	12.94	592.80	34.2
East to Polo	1269	0	1243	26	37.30	670.98	32.4
East to Texas	1250	169	683	398	28.50	471.00	41.4
East to Tarrow W	1314	0	1262	52	58.62	1701.25	80.9
East to Tarrow E	1523	313	1195	15	16.46	1376.47	69.1
East to Spring Loop	1212	27	1175	10	50.74	1484.73	75.8
East to Forest	1502	178	1324	0	17.70	478.33	21.7
East to Glenhaven	1347	134	1105	108	23.70	730.88	39.7
East to WFR	1181	0	669	512	13.32	467.05	41.9
East to EFR	999	554	445	0	21.86	313.40	42.3
East to Out	690	0	690	0	10.00	115.48	10.0
Total Eastbound	**16629**	**1619**	**13517**	**1493**	**324.9**	**9682.79**	**546.4**
			Link Volume (vehicles)		**Free Flow Time (distance/free flow speed)**	**Travel Time (vehicle-minutes)**	**Link Travel Time (seconds)**
	All	Left turn	Through	Right Turn	Through	Through	Through
West to EFR	574	0	501	73	10.00	334.02	40.0
West to WFR	1627	252	1375	0	21.86	508.45	22.2
West to Glenhaven	2158	186	1873	99	13.32	1556.12	49.8
West to Forest	2027	45	1926	56	23.70	1460.88	45.5
West to Spring Loop	2081	198	1679	204	17.70	1269.42	45.4
West to Tarrow E	1697	17	1557	123	50.74	2236.10	86.2
West to Tarrow W	1534	45	1489	0	16.46	436.73	17.6
West to Texas	1880	408	1205	267	58.62	1876.83	93.5
West to Polo	1928	127	1801	0	28.50	1052.33	35.1
West to College	1826	45	1626	155	37.30	1728.80	63.8
West to Spence	1965	33	1910	22	12.94	491.55	15.4
West to Ireland	1977	281	1604	92	19.51	794.43	29.7
West to Out	1659	0	1659	0	14.29	328.18	11.9
Total Westbound	**22933**	**1637**	**20205**	**1091**	**324.9**	**14073.84**	**556.0**

When traveling east, there are nine segments used to get the average volume which extends from Ireland Street to Glenhaven Drive. When traveling west, there are eleven segments used to get the average volume which extends from SH 6 West Frontage Road to Ireland Street. The difference in the length of these two corridor volumes is because when traveling east, a significant portion of the total traffic does not continue to flow through the corridor but rather turns right to get onto the West Frontage Road traveling south.

The 572 cases that were entered into Excel were summarized into tables of each time period with its eleven runs with the average corridor volume and corridor travel time. The average of the eleven runs average corridor volumes were calculated and documented as well as the corridor travel time mean and standard deviation for the eleven runs. These tables are located in Appendix A.

Data Analysis/Statistical Modeling

The first objective was to develop a relationship between average corridor travel time and traffic volume (see #1 in Figure 1). Using Excel, researchers investigated the shape of the relationship—linear, non-linear, and slope changes. The second objective was to investigate the variability of travel time over the range of traffic volumes (see #2 in Figure 1).

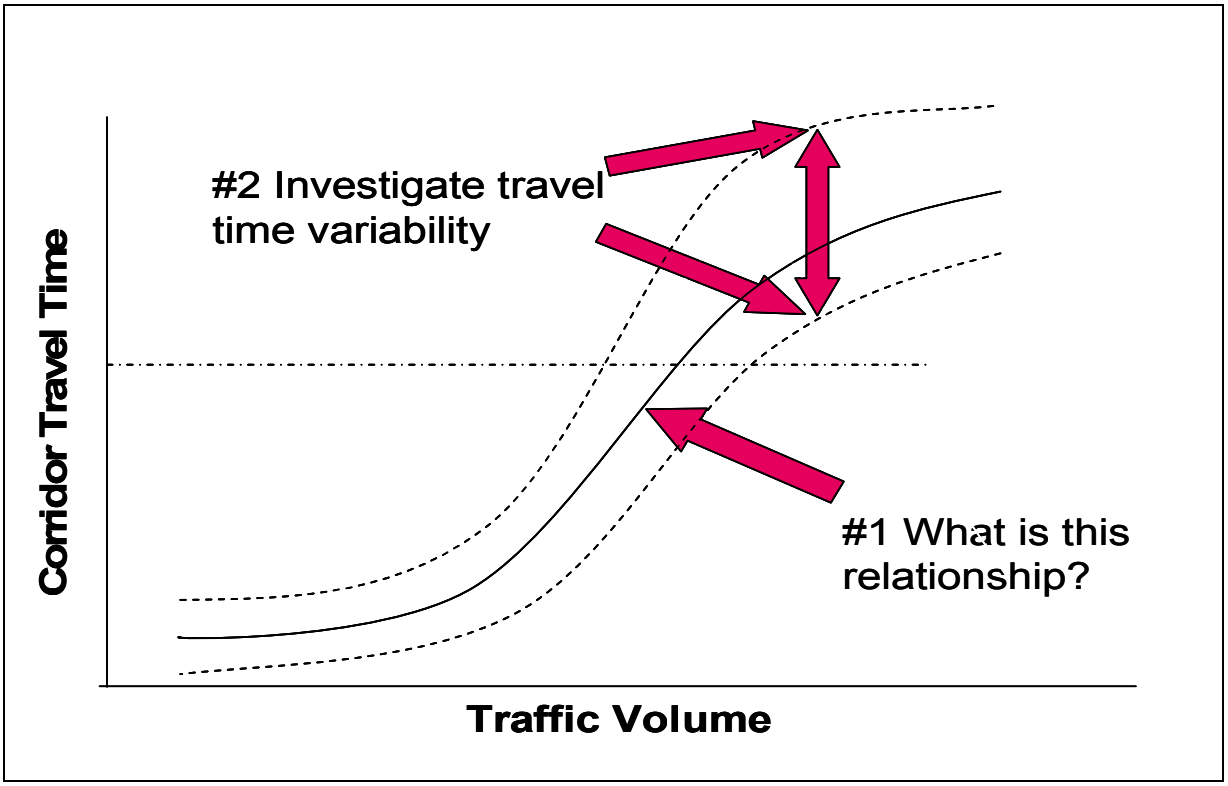

Figure 1. Traffic Volume vs. Corridor Travel Time

After calibrating the model and plotting the corridor travel time versus the traffic volume, the relationship was investigated (see Figures 2, 3, and 4). Figure 2 is a graph of the 286 cases collected for the eastbound direction showing a relationship between the average corridor volumes versus the corridor travel time. The graph illustrates a slight curve as both travel time and volume increases. A small cluster of data points appears as outliers because of a gap. Micro-simulation was used to fill gaps in volume range shown by the model but since there are no observations or simulation cases for low to moderate conditions or p.m. conditions, a gap is present.

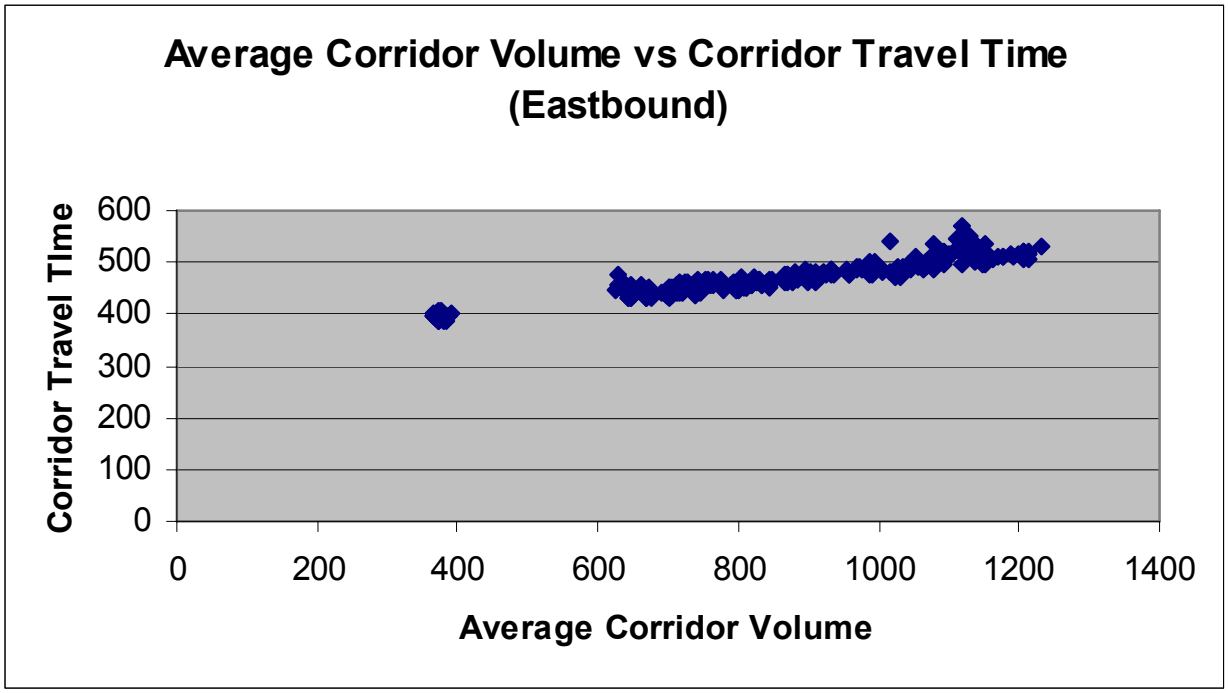

Figure 2. Average Corridor Volume vs. Corridor Travel Time for Eastbound Direction

The graph of the 286 cases collected for the peak flow direction is shown in Figure 3. This illustration has a gap for the same reasons as the non-peak direction graph. There is also a curve viewable in this graph but it has a more rapid increase than the eastbound direction.

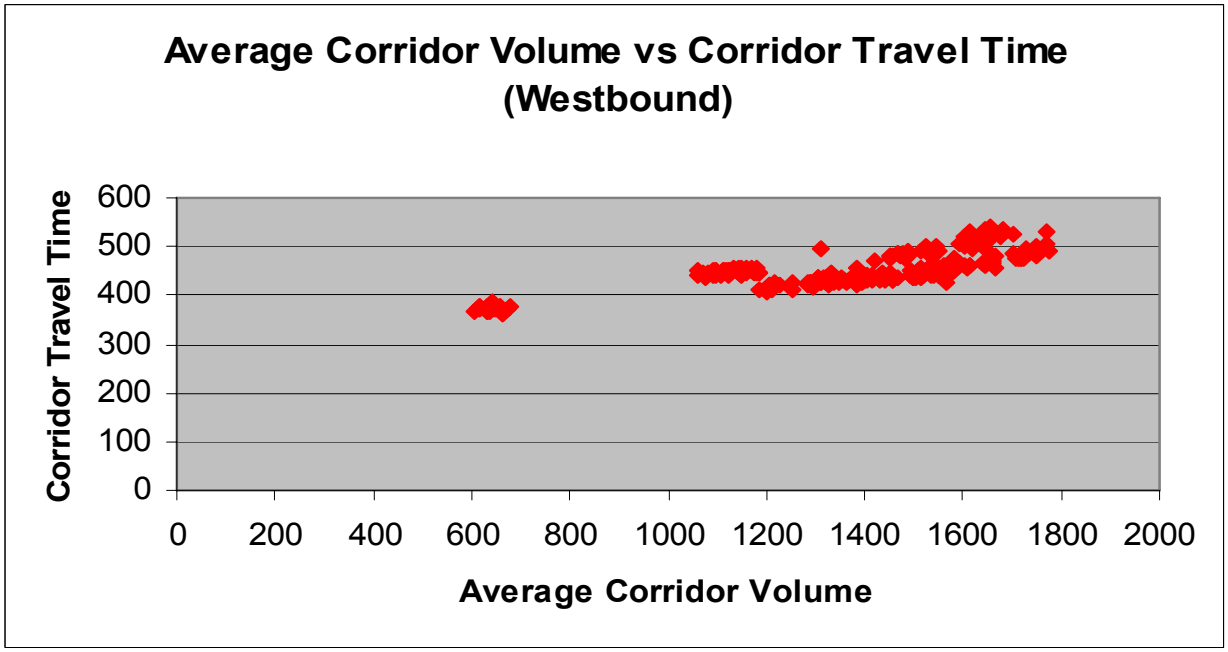

Figure 3. Average Corridor Volume vs. Corridor Travel Time for Westbound Direction

Both the peak and non-peak conditions were plotted, in Figure 4, for comparison. The eastbound direction has smaller volumes and higher travel times than the westbound direction. This is because signal timing favors peak direction by giving more green time and better offset considerations.

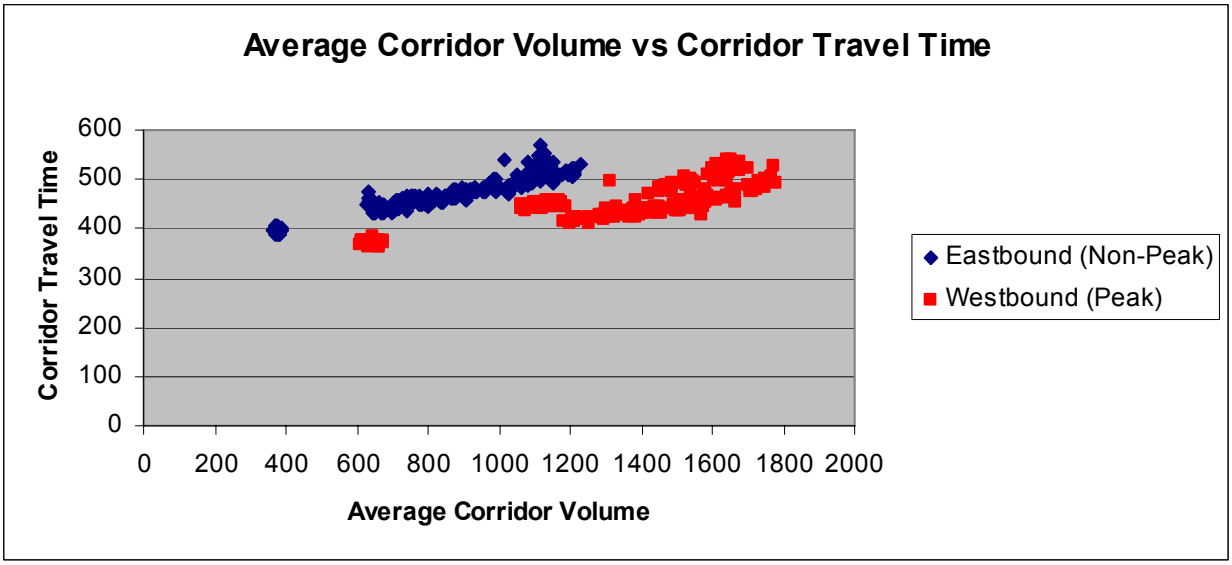

Figure 4. Average Corridor Volume vs. Corridor Travel Time for Comparison of Both Eastbound and Westbound Directions

Researchers used regression analysis to estimate the relationship between traffic volume and travel time and to investigate travel time variability. Using Excel and running an analysis of variance (ANOVA), a linear model for the eastbound and westbound directions were developed. The analysis is shown in Table 2, and the linear regression model is shown below in Figure 5 for the non-peak direction.

Table 2. Statistical Analysis Summary Output of the Linear Model for the Eastbound Direction

SUMMARY OUTPUT

Eastbound (Non-Peak)

Regression Statistics	
Multiple R	0.946352349
R Square	0.895582768
Adjusted R Square	0.895215102
Standard Error	11.54956229
Observations	286

ANOVA

	df	*SS*	*MS*	*F*	*Significance F*
Regression	1	324924.8629	324924.8629	2435.857585	2.3138E-141
Residual	284	37883.43851	133.3923891		
Total	285	362808.3014			

	Coefficients	*Standard Error*	*t Stat*	*P-value*	*Lower 95%*	*Upper 95%*	*Lower 95.0%*	*Upper 95.0%*
Intercept	337.0463649	2.778485439	121.3057878	1.1194E-246	331.5773273	342.5154026	331.5773273	342.5154026
X Variable 1	0.155680214	0.003154333	49.35440796	2.3138E-141	0.149471377	0.161889051	0.149471377	0.161889051

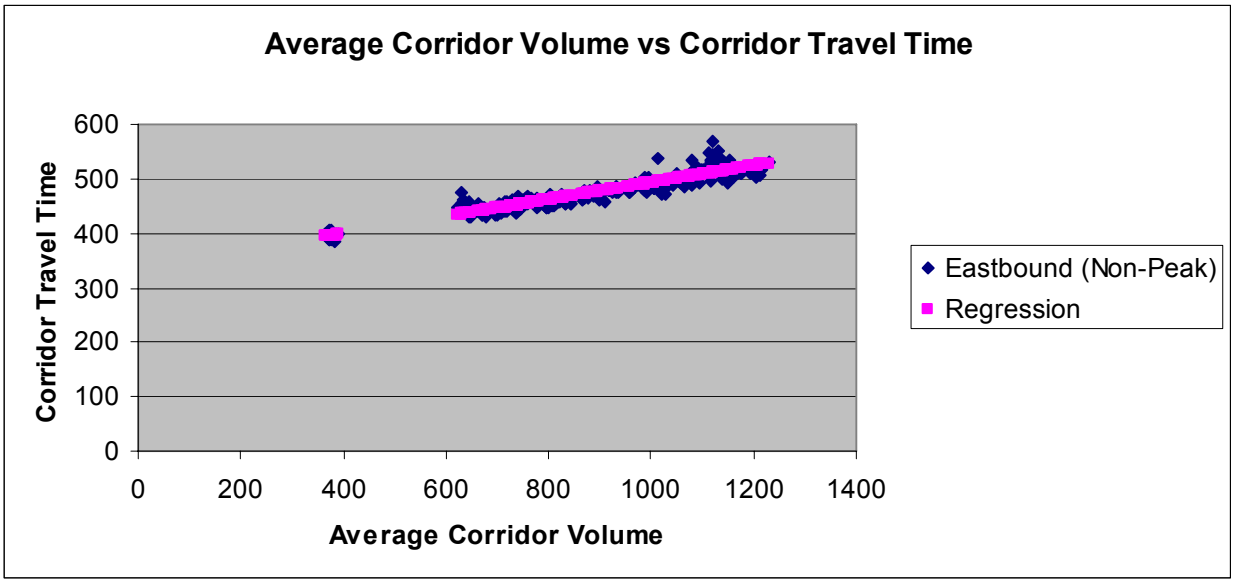

Figure 5. Linear Regression Model of the Average Corridor Volume vs. Corridor Travel Time for the Eastbound Direction

The model for the eastbound regression is as follows: TT= 337.05+0.15568*CV. The model resulted in a R^2 value of 0.895 which is a strong correlation. Below in Table 3 and Figure 6 are the analysis and linear regression model for the westbound direction.

Table 3. Statistical Analysis Summary Output of the Linear Model for the Westbound Direction

SUMMARY OUTPUT

Westbound (Peak)

Regression Statistics	
Multiple R	0.779453872
R Square	0.607548338
Adjusted R Square	0.606166466
Standard Error	21.53649932
Observations	286

ANOVA

	df	*SS*	*MS*	*F*	*Significance F*
Regression	1	203921.5995	203921.5995	439.6560011	1.25826E-59
Residual	284	131725.1081	463.8208031		
Total	285	335646.7076			

	Coefficients	*Standard Error*	*t Stat*	*P-value*	*Lower 95%*	*Upper 95%*	*Lower 95.0%*	*Upper 95.0%*
Intercept	315.9292428	6.460382709	48.90255841	2.3972E-140	303.2129349	328.6455506	303.2129349	328.6455506
X Variable 1	0.097091073	0.004630446	20.96797561	1.25826E-59	0.087976726	0.106205421	0.087976726	0.106205421

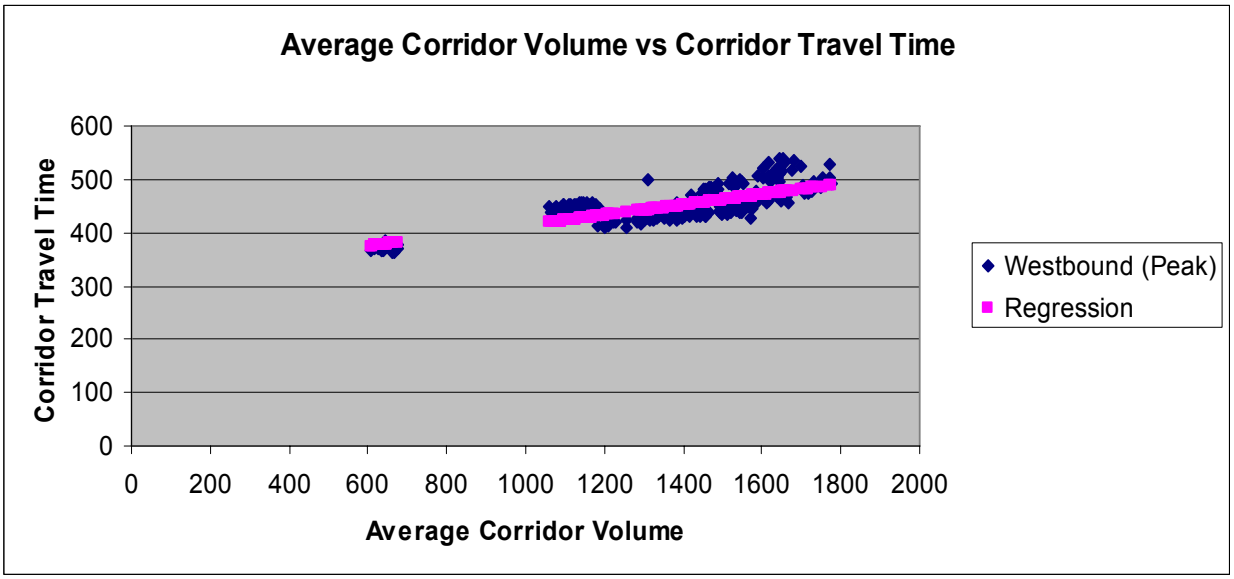

Figure 6. Linear Regression Model of the Average Corridor Volume vs. Corridor Travel Time for the Westbound Direction

A moderate R^2 value of 0.608 was the output from the model of TT=315.93+0.09709*CV. The same analysis was performed again but with another model to see how results varied. The results are presented in Table 4 and Figure 7 for the non-peak condition.

Table 4. Statistical Analysis Summary Output of the Non-Linear Model for the Eastbound Direction

SUMMARY OUTPUT

Eastbound (Non-Peak)

Regression Statistics	
Multiple R	0.946820739
R Square	0.896469513
Adjusted R Square	0.896104969
Standard Error	11.50041645
Observations	286

ANOVA

	df	*SS*	*MS*	*F*	*Significance F*
Regression	1	325246.5811	325246.5811	2459.153317	6.8889E-142
Residual	284	37561.72029	132.2595785		
Total	285	362808.3014			

	Coefficients	*Standard Error*	*t Stat*	*P-value*	*Lower 95%*	*Upper 95%*	*Lower 95.0%*	*Upper 95.0%*
Intercept	356.5760433	2.385612188	149.4694088	4.9769E-272	351.8803186	361.2717681	351.8803186	361.2717681
X Variable 1	0.034153362	0.000688717	49.58985095	6.8889E-142	0.032797725	0.035508999	0.032797725	0.035508999

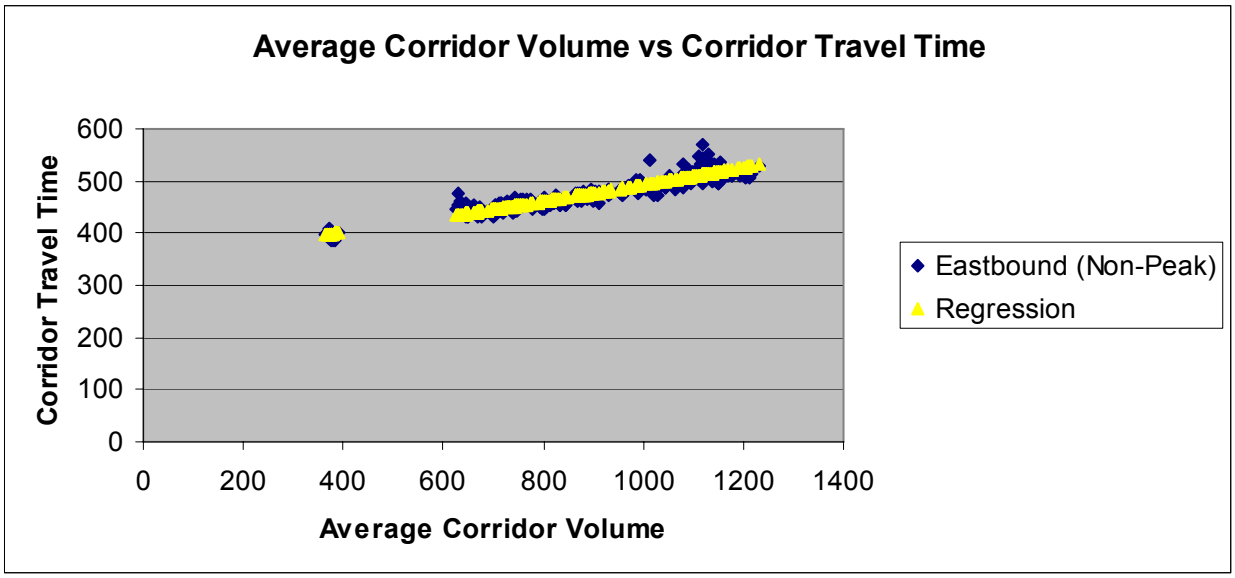

Figure 7. Non-Linear Regression Model of the Average Corridor Volume vs. Corridor Travel Time for the Eastbound Direction

Even though the regression appears to be linear, the model $TT=356.58+0.03415*(CV)^{1.2}$ has an R^2 value of 0.896, which is comparable to the linear model. Table 5 and Figure 8 display the analysis and regression for the peak condition.

Table 5. Statistical Analysis Summary Output of the Non-Linear Model for the Westbound Direction

SUMMARY OUTPUT

Westbound (Peak)

Regression Statistics	
Multiple R	0.779729803
R Square	0.607978566
Adjusted R Square	0.606598209
Standard Error	21.52469132
Observations	286

ANOVA

	df	*SS*	*MS*	*F*	*Significance F*
Regression	1	204066.004	204066.004	440.4501842	1.0764E-59
Residual	284	131580.7036	463.3123366		
Total	285	335646.7076			

	Coefficients	*Standard Error*	*t Stat*	*P-value*	*Lower 95%*	*Upper 95%*	*Lower 95.0%*	*Upper 95.0%*
Intercept	333.9687642	5.61451963	59.48305219	2.3643E-162	322.9174127	345.0201156	322.9174127	345.0201156
X Variable 1	0.019691062	0.000938255	20.98690506	1.0764E-59	0.017844247	0.021537878	0.017844247	0.021537878

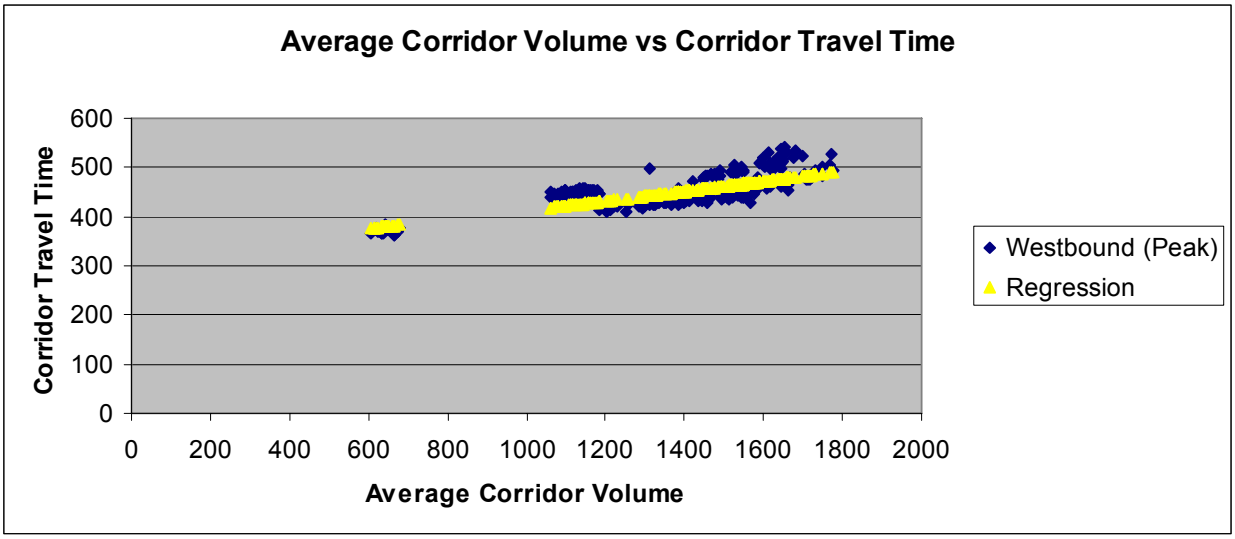

Figure 8. Non-Linear Regression Model of the Average Corridor Volume vs. Corridor Travel Time for the Westbound Direction

When this analysis was applied to the westbound direction, the westbound model of TT= 333.97+0.01969 *$(CV)^{1.2}$ produced a R^2 value of 0.608. This value was also comparable to the linear model for the peak condition.

After reviewing the relationship between travel time and traffic volume, which was the first objective, the travel time variability can be evaluated from the models and data. Summary tables showing variance for the peak and non-peak conditions can be seen in Table 6 and Table 7. Note that the volume range intervals for the directions are of the same size. As the travel time range increases so does the variance.

Table 6. Travel Time Variability for Eastbound Direction

Average Corridor Volume Range	Travel Time Range (sec.)	Variance (sec.)
365-582	386.5-407.3	32.29
583-800	430.6-475.7	100.24
801-1027	447.4-539.0	180.88
1028-1245	484.8-570.3	303.11

Table 7. Travel Time Variability for Westbound Direction

Average Corridor Volume Range	Travel Time Range (sec.)	Variance (sec.)
606-899	361.7-384.9	31.80
900-1193	412.3-456.4	50.70
1194-1487	408.5-498.1	350.67
1488-1781	428.7-539.7	763.96

After completing and analyzing the data from graphs and summary outputs produced in Excel, results can be concluded.

Results

All of the 572 cases that were plotted are located in Appendix B in summary tables. The average of the averaged corridor volumes were calculated for the 572 cases, as well as the mean and standard deviation of the corridor travel time are summarized in Table 8 and Table 9 below.

Table 8. Summary of Eastbound Corridor Volumes and Travel Time Mean and Standard Deviation for Various Time Periods

Time Period	Average of Averaged Corridor Volumes	Average Travel Time (sec) mean	Average Travel Time (sec) standard deviation
Wed 6:00-6:30am	1150	504.8	7.113
Wed 6:30-7:00am	1022	482.3	6.585
Wed 7:00-7:30am	377	397.9	5.599
Wed 7:30-8:00am	814	458.8	6.195
Wed 8:00-8:30am	697	439.7	7.600
Wed 8:30-9:00am	663	440.0	8.198
Wed 9:00-9:30am	755	458.4	3.365
Wed 9:30-10:00am	643	451.9	2.698
Thur 6:00-6:30am	1204	515.9	7.264
Thur 6:30-7:00am	1069	494.2	5.427
Thur 7:00-7:30am	375	394.0	5.318
Thur 7:30-8:00am	865	468.0	6.515
Thur 8:00-8:30am	741	447.3	7.727
Thur 8:30-9:00am	661	440.6	12.636
Thur 9:00-9:30am	735	458.6	4.717
Thur 9:30-10:00am	732	456.8	4.496
Sat 10:00-10:30am	888	473.3	6.710
Sat 10:30-11:00am	974	487.7	9.451
Sat 11:00-11:30am	1076	511.7	10.440
Sat 11:30-12:00pm	1116	522.8	14.461
Sat 12:00-12:30pm	1113	533.0	16.500
Sat 12:30-1:00pm	1110	524.5	14.405
Extreme Case 1	808	455.1	6.420
Extreme Case 2	821	457.7	6.832
Extreme Case 3	869	468.2	7.028
Extreme Case 4	920	476.0	6.355

From the time periods on Saturday, November 11, 2006, and the first hour for the weekdays, the data illustrate that there were more volume during these periods and less volume during the extreme cases and the other three hours of the weekdays.

Table 9. Summary of Westbound Corridor Volumes and Travel Time Mean and Standard Deviation for Different Time Periods

Time Period	Average of Averaged Corridor Volumes	Average Travel Time (sec)	Average Travel Time (sec)
		mean	standard deviation
Wed 6:00-6:30am	1626	510.5	10.723
Wed 6:30-7:00am	1463	480.1	4.764
Wed 7:00-7:30am	657	371.8	5.615
Wed 7:30-8:00am	1318	433.1	5.685
Wed 8:00-8:30am	1330	427.8	2.232
Wed 8:30-9:00am	1464	437.3	6.291
Wed 9:00-9:30am	1156	449.4	4.707
Wed 9:30-10:00am	1094	447.7	4.279
Thur 6:00-6:30am	1651	522.5	12.242
Thur 6:30-7:00am	1512	488.8	12.322
Thur 7:00-7:30am	625	372.4	5.916
Thur 7:30-8:00am	1296	425.9	7.482
Thur 8:00-8:30am	1378	440.8	19.416
Thur 8:30-9:00am	1406	434.9	5.085
Thur 9:00-9:30am	1149	450.8	4.557
Thur 9:30-10:00am	1110	446.7	3.056
Sat 10:00-10:30am	1202	419.8	7.128
Sat 10:30-11:00am	1412	431.3	5.804
Sat 11:00-11:30am	1517	444.9	6.926
Sat 11:30-12:00pm	1539	456.0	9.103
Sat 12:00-12:30pm	1582	455.4	6.641
Sat 12:30-1:00pm	1547	446.9	6.976
Extreme Case 1	1547	453.4	6.501
Extreme Case 2	1570	455.5	5.046
Extreme Case 3	1671	471.2	8.751
Extreme Case 4	1742	492.0	15.200

From the extreme cases and the time periods on Saturday, November 11, 2006, the data illustrate that there were more volume during these periods and less volume during the weekdays.

When comparing and relating travel time to traffic volume, they are proportional. When volume increases, travel time generally increases and vice versa. When comparing and relating the peak and non-peak conditions, the peak condition generally had the upper hand because of the favoritism that the timing plan shows.

CONCLUSION

Based on the results of the study, several conclusions were made by the researchers.

The following is a summary of conclusions made from the study:

- Westbound traveling has more volume but shorter travel time because of the timing plan favoring peak direction.
- Westbound has a larger range in travel time.
- Relationship appears to be linear mostly, then goes up non-linearly as demand increases.
- Eastbound data have a stronger linear correlation than westbound data.

From the results and conclusions presented in the paper and with additional research and more recent data collection, future model generation will be better at estimating traffic volumes. The results and models obtained from this study can be used to predict flow conditions for signalized corridors in mid-sized urban communities.

FUTURE RESEARCH

In the future, the following procedures can be carried out to produce better results and accuracy of the data:

- PM + low to moderate volume data to fill the gap;
- Investigate relationship models considering multiple factors as the same time (i.e., intersection density, major cross street, etc.); and
- Identify a threshold demand level.

With the continuation of research for this project and with data relative to the city's population gradual increase, researchers and transportation engineers will have the opportunity to limit, if not remove, the build-up of congestion.

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APPENDIX A

Table A1. Summary of Time Period Wed. 6:00-6:30am Eleven Cases for Eastbound

Wed 6:00-6:30am	Average Corridor Volume	Corridor Travel Time(seconds)	
Original	1151	496.6	
Case 1	1149	494.1	
Case 2	1158	512.5	
Case 3	1144	507.9	**Average of Averaged Corridor Volumes**
Case 4	1168	508.5	1150
Case 5	1162	504.5	
Case 6	1177	510.7	**Corridor Travel Time(seconds)**
Case 7	1138	499.5	mean=504.8
Case 8	1150	508.4	standard deviation=7.113
Case 9	1140	514.0	
Case 10	1117	495.8	

Table A2. Summary of Time Period Wed. 6:00-6:30am Eleven Cases for Westbound

Wed 6:00-6:30am	Average Corridor Volume	Corridor Travel Time(seconds)	
Original	1640	504.1	
Case 1	1649	513.2	
Case 2	1601	521.1	
Case 3	1626	508.1	**Average of Averaged Corridor Volumes**
Case 4	1614	531.8	1626
Case 5	1634	518.4	
Case 6	1620	494.7	**Corridor Travel Time(seconds)**
Case 7	1591	507.7	mean=510.5
Case 8	1645	496.0	standard deviation=10.723
Case 9	1625	509.9	
Case 10	1643	510.3	

Table A3. Summary of Time Period Wed. 6:30-7:00am Eleven Cases for Eastbound

Wed 6:30-7:00am	Average Corridor Volume	Corridor Travel Time(seconds)	
Original	1030	471.8	
Case 1	1022	472.9	
Case 2	1028	489.3	
Case 3	1019	482.1	**Average of Averaged Corridor Volumes**
Case 4	1044	487.9	1022
Case 5	1035	489.6	
Case 6	1042	488.7	**Corridor Travel Time(seconds)**
Case 7	1004	483.3	mean=482.3
Case 8	1018	481.2	standard deviation=6.585
Case 9	1006	483.7	
Case 10	991	474.8	

Table A4. Summary of Time Period Wed. 6:30-7:00am Eleven Cases for Westbound

Wed 6:30-7:00am	Average Corridor Volume	Corridor Travel Time(seconds)	
Original	1485	478.0	
Case 1	1485	476.8	
Case 2	1421	471.5	
Case 3	1470	484.2	**Average of Averaged Corridor Volumes**
Case 4	1450	479.7	1463
Case 5	1475	480.3	
Case 6	1450	474.0	**Corridor Travel Time(seconds)**
Case 7	1453	482.8	mean=480.1
Case 8	1457	481.7	standard deviation=4.764
Case 9	1468	486.6	
Case 10	1478	485.7	

Table A5. Summary of Time Period Wed. 7:00-7:30am Eleven Cases for Eastbound

Wed 7:00-7:30am	Average Corridor Volume	Corridor Travel Time(seconds)	
Original	375	394.6	
Case 1	375	393.8	
Case 2	390	399.8	
Case 3	384	396.7	**Average of Averaged Corridor Volumes**
Case 4	386	395.1	377
Case 5	371	407.3	
Case 6	367	401.9	**Corridor Travel Time(seconds)**
Case 7	378	397.0	mean=397.9
Case 8	367	397.8	standard deviation=5.599
Case 9	375	405.2	
Case 10	380	387.2	

Table A6. Summary of Time Period Wed. 7:00-7:30am Eleven Cases for Westbound

Wed 7:00-7:30am	Average Corridor Volume	Corridor Travel Time(seconds)	
Original	664	363.1	
Case 1	664	361.7	
Case 2	647	376.9	
Case 3	675	370.4	**Average of Averaged Corridor Volumes**
Case 4	638	366.3	657
Case 5	661	375.9	
Case 6	652	373.6	**Corridor Travel Time(seconds)**
Case 7	648	373.2	mean=371.8
Case 8	659	376.5	standard deviation=5.615
Case 9	648	375.0	
Case 10	677	376.8	

Table A7. Summary of Time Period Wed. 7:30-8:00am Eleven Cases for Eastbound

Wed 7:30-8:00am	Average Corridor Volume	Corridor Travel Time(seconds)	
Original	796	451.5	
Case 1	794	449.5	
Case 2	818	457.3	
Case 3	816	461.9	**Average of Averaged Corridor Volumes**
Case 4	819	460.9	814
Case 5	826	463.3	
Case 6	824	471.0	**Corridor Travel Time(seconds)**
Case 7	820	456.2	mean=458.8
Case 8	820	462.6	standard deviation=6.195
Case 9	826	460.0	
Case 10	797	452.8	

Table A8. Summary of Time Period Wed. 7:30-8:00am Eleven Cases for Westbound

Wed 7:30-8:00am	Average Corridor Volume	Corridor Travel Time(seconds)	
Original	1307	437.1	
Case 1	1311	432.5	
Case 2	1290	426.6	
Case 3	1340	430.3	**Average of Averaged Corridor Volumes**
Case 4	1300	422.4	1318
Case 5	1333	444.2	
Case 6	1309	432.3	**Corridor Travel Time(seconds)**
Case 7	1318	437.2	mean=433.1
Case 8	1319	434.1	standard deviation=5.685
Case 9	1329	433.8	
Case 10	1338	433.5	

Table A9. Summary of Time Period Wed. 8:00-8:30am Eleven Cases for Eastbound

Wed 8:00-8:30am	Average Corridor Volume	Corridor Travel Time(seconds)	
Original	677	430.9	
Case 1	677	434.6	
Case 2	699	444.0	
Case 3	701	431.8	**Average of Averaged Corridor Volumes**
Case 4	706	435.8	697
Case 5	699	436.8	
Case 6	702	452.8	**Corridor Travel Time(seconds)**
Case 7	690	442.1	mean=439.7
Case 8	704	452.3	standard deviation=7.6
Case 9	714	441.5	
Case 10	698	434.1	

Table A10. Summary of Time Period Wed. 8:00-8:30am Eleven Cases for Westbound

Wed 8:00-8:30am	Average Corridor Volume	Corridor Travel Time(seconds)	
Original	1323	426.0	
Case 1	1325	423.8	
Case 2	1313	425.0	
Case 3	1355	430.4	**Average of Averaged Corridor Volumes**
Case 4	1328	429.7	1330
Case 5	1350	426.6	
Case 6	1303	430.5	**Corridor Travel Time(seconds)**
Case 7	1334	428.5	mean=427.8
Case 8	1324	428.2	standard deviation=2.232
Case 9	1326	427.5	
Case 10	1355	429.5	

Table A11. Summary of Time Period Wed. 8:30-9:00am Eleven Cases for Eastbound

Wed 8:30-9:00am	Average Corridor Volume	Corridor Travel Time(seconds)	
Original	644	433.2	
Case 1	645	430.6	
Case 2	673	435.6	
Case 3	674	435.8	**Average of Averaged Corridor Volumes**
Case 4	669	435.9	663
Case 5	669	444.4	
Case 6	672	443.1	**Corridor Travel Time(seconds)**
Case 7	673	439.2	mean=440.0
Case 8	672	438.8	standard deviation=8.198
Case 9	635	461.2	
Case 10	671	441.8	

Table A12. Summary of Time Period Wed. 8:30-9:00am Eleven Cases for Westbound

Wed 8:30-9:00am	Average Corridor Volume	Corridor Travel Time(seconds)	
Original	1457	429.8	
Case 1	1457	430.6	
Case 2	1444	434.1	
Case 3	1493	450.7	**Average of Averaged Corridor Volumes**
Case 4	1452	436.1	1464
Case 5	1493	439.7	
Case 6	1444	431.5	**Corridor Travel Time(seconds)**
Case 7	1467	438.8	mean=437.3
Case 8	1449	437.8	standard deviation=6.291
Case 9	1450	445.1	
Case 10	1496	436.5	

Table A13. Summary of Time Period Wed. 9:00-9:30am Eleven Cases for Eastbound

Wed 9:00-9:30am	Average Corridor Volume	Corridor Travel Time(seconds)	
Original	755	457.8	
Case 1	757	455.7	
Case 2	757	461.8	
Case 3	750	456.0	**Average of Averaged Corridor Volumes**
Case 4	789	457.2	755
Case 5	764	461.5	
Case 6	776	464.3	**Corridor Travel Time(seconds)**
Case 7	737	456.1	mean=458.4
Case 8	742	457.6	standard deviation=3.365
Case 9	749	461.1	
Case 10	728	453.0	

Table A14. Summary of Time Period Wed. 9:00-9:30am Eleven Cases for Westbound

Wed 9:00-9:30am	Average Corridor Volume	Corridor Travel Time(seconds)	
Original	1186	445.0	
Case 1	1186	445.6	
Case 2	1123	442.2	
Case 3	1148	455.8	**Average of Averaged Corridor Volumes**
Case 4	1158	447.5	1156
Case 5	1167	452.6	
Case 6	1127	451.3	**Corridor Travel Time(seconds)**
Case 7	1138	452.9	mean=449.4
Case 8	1149	443.9	standard deviation=4.707
Case 9	1157	453.0	
Case 10	1179	454.1	

Table A15. Summary of Time Period Wed. 9:30-10:00am Eleven Cases for Eastbound

Wed 9:30-10:00am	Average Corridor Volume	Corridor Travel Time(seconds)	
Original	645	451.6	
Case 1	645	450.5	
Case 2	647	457.1	
Case 3	637	449.5	**Average of Averaged Corridor Volumes**
Case 4	673	449.1	643
Case 5	642	451.9	
Case 6	662	454.4	**Corridor Travel Time(seconds)**
Case 7	630	454.8	mean=451.9
Case 8	637	452.4	standard deviation=2.698
Case 9	632	451.9	
Case 10	624	448.0	

Table A16. Summary of Time Period Wed. 9:30-10:00am Eleven Cases for Westbound

Wed 9:30-10:00am	Average Corridor Volume	Corridor Travel Time(seconds)	
Original	1125	452.2	
Case 1	1125	449.4	
Case 2	1062	439.6	
Case 3	1093	445.4	**Average of Averaged Corridor Volumes**
Case 4	1087	445.9	1094
Case 5	1097	453.1	
Case 6	1060	449.9	**Corridor Travel Time(seconds)**
Case 7	1079	447.8	mean=447.7
Case 8	1093	443.4	standard deviation=4.279
Case 9	1096	452.8	
Case 10	1116	445.0	

Table A17. Summary of Time Period Thur. 6:00-6:30am Eleven Cases for Eastbound

Thur 6:00-6:30am	Average Corridor Volume	Corridor Travel Time(seconds)	
Original	1212	507.4	
Case 1	1206	504.4	
Case 2	1213	520.3	
Case 3	1206	521.3	**Average of Averaged Corridor Volumes**
Case 4	1207	521.6	1204
Case 5	1214	515.1	
Case 6	1231	529.6	**Corridor Travel Time(seconds)**
Case 7	1193	511.5	mean=515.9
Case 8	1199	516.7	standard deviation=7.264
Case 9	1189	516.4	
Case 10	1175	510.1	

Table A18. Summary of Time Period Thur. 6:00-6:30am Eleven Cases for Westbound

Thur 6:00-6:30am	Average Corridor Volume	Corridor Travel Time(seconds)	
Original	1679	534.1	
Case 1	1701	523.3	
Case 2	1617	507.9	
Case 3	1659	532.2	**Average of Averaged Corridor Volumes**
Case 4	1644	537.5	1651
Case 5	1640	519.9	
Case 6	1644	515.1	**Corridor Travel Time(seconds)**
Case 7	1604	501.8	mean=522.5
Case 8	1675	518.6	standard deviation=12.242
Case 9	1653	539.7	
Case 10	1650	517.0	

Table A19. Summary of Time Period Thur. 6:30-7:00am Eleven Cases for Eastbound

Thur 6:30-7:00am	Average Corridor Volume	Corridor Travel Time(seconds)	
Original	1079	487.6	
Case 1	1078	490.3	
Case 2	1068	491.8	
Case 3	1065	484.8	**Average of Averaged Corridor Volumes**
Case 4	1085	500.9	1069
Case 5	1077	494.3	
Case 6	1089	501.6	**Corridor Travel Time(seconds)**
Case 7	1058	493.3	mean=494.2
Case 8	1067	499.7	standard deviation=5.427
Case 9	1057	497.5	
Case 10	1041	494.2	

Table A20. Summary of Time Period Thur. 6:30-7:00am Eleven Cases for Westbound

Thur 6:30-7:00am	Average Corridor Volume	Corridor Travel Time(seconds)	
Original	1551	492.0	
Case 1	1543	499.6	
Case 2	1489	481.5	
Case 3	1534	491.5	**Average of Averaged Corridor Volumes**
Case 4	1515	490.5	1512
Case 5	1524	503.3	
Case 6	1488	492.6	**Corridor Travel Time(seconds)**
Case 7	1384	456.2	mean=488.8
Case 8	1527	486.6	standard deviation=12.322
Case 9	1531	487.8	
Case 10	1547	494.7	

Table A21. Summary of Time Period Thur. 7:00-7:30am Eleven Cases for Eastbound

Thur 7:00-7:30am	Average Corridor Volume	Corridor Travel Time(seconds)	
Original	374	387.4	
Case 1	373	388.4	
Case 2	386	395.1	
Case 3	377	390.1	**Average of Averaged Corridor Volumes**
Case 4	385	398.8	375
Case 5	370	392.7	
Case 6	366	397.8	**Corridor Travel Time(seconds)**
Case 7	376	396.9	mean=394.0
Case 8	365	397.7	standard deviation=5.318
Case 9	371	402.6	
Case 10	382	386.5	

Table A22. Summary of Time Period Thur. 7:00-7:30am Eleven Cases for Westbound

Thur 7:00-7:30am	Average Corridor Volume	Corridor Travel Time(seconds)	
Original	633	365.9	
Case 1	634	364.7	
Case 2	617	374.4	
Case 3	642	374.2	**Average of Averaged Corridor Volumes**
Case 4	606	365.1	625
Case 5	625	370.2	
Case 6	615	371.7	**Corridor Travel Time(seconds)**
Case 7	613	374.3	mean=372.4
Case 8	633	375.0	standard deviation=5.916
Case 9	618	376.4	
Case 10	644	384.9	

Table A23. Summary of Time Period Thur. 7:30-8:00am Eleven Cases for Eastbound

Thur 7:30-8:00am	Average Corridor Volume	Corridor Travel Time(seconds)	
Original	843	453.7	
Case 1	842	461.5	
Case 2	870	477.4	
Case 3	871	466.4	**Average of Averaged Corridor Volumes**
Case 4	872	471.2	865
Case 5	876	470.3	
Case 6	881	470.1	**Corridor Travel Time(seconds)**
Case 7	871	475.9	mean=468.0
Case 8	871	467.8	standard deviation=6.515
Case 9	868	468.0	
Case 10	848	465.9	

Table A24. Summary of Time Period Thur. 7:30-8:00am Eleven Cases for Westbound

Thur 7:30-8:00am	Average Corridor Volume	Corridor Travel Time(seconds)	
Original	1294	418.0	
Case 1	1291	424.7	
Case 2	1255	411.1	
Case 3	1312	428.2	**Average of Averaged Corridor Volumes**
Case 4	1283	422.1	1296
Case 5	1303	434.3	
Case 6	1290	424.6	**Corridor Travel Time(seconds)**
Case 7	1285	426.2	mean=425.9
Case 8	1301	427.2	standard deviation=7.482
Case 9	1303	438.7	
Case 10	1335	430.3	

Table A25. Summary of Time Period Thur. 8:00-8:30am Eleven Cases for Eastbound

Thur 8:00-8:30am	Average Corridor Volume	Corridor Travel Time(seconds)	
Original	718	443.4	
Case 1	719	439.5	
Case 2	745	444.3	
Case 3	746	448.3	**Average of Averaged Corridor Volumes**
Case 4	745	448.2	741
Case 5	746	454.0	
Case 6	742	450.8	**Corridor Travel Time(seconds)**
Case 7	740	445.1	mean=447.3
Case 8	744	443.6	standard deviation=7.727
Case 9	765	465.7	
Case 10	739	437.5	

Table A26. Summary of Time Period Thur. 8:00-8:30am Eleven Cases for Westbound

Thur 8:00-8:30am	Average Corridor Volume	Corridor Travel Time(seconds)	
Original	1384	427.7	
Case 1	1380	430.3	
Case 2	1367	433.7	
Case 3	1414	442.4	**Average of Averaged Corridor Volumes**
Case 4	1376	434.1	1378
Case 5	1400	436.8	
Case 6	1352	436.7	**Corridor Travel Time(seconds)**
Case 7	1311	498.1	mean=440.8
Case 8	1378	433.7	standard deviation=19.416
Case 9	1380	438.7	
Case 10	1412	436.4	

Table A27. Summary of Time Period Thur. 8:30-9:00am Eleven Cases for Eastbound

Thur 8:30-9:00am	Average Corridor Volume	Corridor Travel Time(seconds)	
Original	648	431.8	
Case 1	648	430.8	
Case 2	665	439.5	
Case 3	671	433.3	**Average of Averaged Corridor Volumes**
Case 4	671	440.4	661
Case 5	670	432.0	
Case 6	664	439.7	**Corridor Travel Time(seconds)**
Case 7	662	446.9	mean=440.6
Case 8	674	440.1	standard deviation=12.636
Case 9	630	475.7	
Case 10	672	435.9	

Table A28. Summary of Time Period Thur. 8:30-9:00am Eleven Cases for Westbound

Thur 8:30-9:00am	Average Corridor Volume	Corridor Travel Time(seconds)	
Original	1401	432.2	
Case 1	1401	434.1	
Case 2	1379	437.4	
Case 3	1433	430.9	**Average of Averaged Corridor Volumes**
Case 4	1397	427.2	1406
Case 5	1431	437.8	
Case 6	1389	437.3	**Corridor Travel Time(seconds)**
Case 7	1405	438.4	mean=434.9
Case 8	1396	430.7	standard deviation=5.085
Case 9	1401	432.4	
Case 10	1436	445.8	

Table A29. Summary of Time Period Thur. 9:00-9:30am Eleven Cases for Eastbound

Thur 9:00-9:30am	Average Corridor Volume	Corridor Travel Time(seconds)	
Original	732	455.3	
Case 1	734	454.7	
Case 2	742	467.9	
Case 3	737	455.2	**Average of Averaged Corridor Volumes**
Case 4	759	455.8	735
Case 5	742	456.7	
Case 6	758	466.8	**Corridor Travel Time(seconds)**
Case 7	717	458.9	mean=458.6
Case 8	726	455.6	standard deviation=4.717
Case 9	728	460.9	
Case 10	713	456.3	

Table A30. Summary of Time Period Thur. 9:00-9:30am Eleven Cases for Westbound

Thur 9:00-9:30am	Average Corridor Volume	Corridor Travel Time(seconds)	
Original	1178	445.1	
Case 1	1176	446.4	
Case 2	1117	448.5	
Case 3	1142	455.5	**Average of Averaged Corridor Volumes**
Case 4	1147	443.6	1149
Case 5	1157	454.9	
Case 6	1114	451.8	**Corridor Travel Time(seconds)**
Case 7	1135	456.4	mean=450.8
Case 8	1142	448.8	standard deviation=4.557
Case 9	1169	455.4	
Case 10	1160	452.3	

Table A31. Summary of Time Period Thur. 9:30-10:00am Eleven Cases for Eastbound

Thur 9:30-10:00am	Average Corridor Volume	Corridor Travel Time(seconds)	
Original	733	452.3	
Case 1	733	452.5	
Case 2	738	461.5	
Case 3	733	453.0	**Average of Averaged Corridor Volumes**
Case 4	765	458.6	732
Case 5	741	461.7	
Case 6	755	464.7	**Corridor Travel Time(seconds)**
Case 7	713	455.0	mean=456.8
Case 8	717	455.7	standard deviation=4.496
Case 9	724	458.7	
Case 10	707	451.4	

Table A32. Summary of Time Period Thur. 9:30-10:00am Eleven Cases for Westbound

Thur 9:30-10:00am	Average Corridor Volume	Corridor Travel Time(seconds)	
Original	1143	448.1	
Case 1	1144	446.4	
Case 2	1078	443.1	
Case 3	1097	442.7	**Average of Averaged Corridor Volumes**
Case 4	1108	442.4	1110
Case 5	1119	450.4	
Case 6	1072	446.2	**Corridor Travel Time(seconds)**
Case 7	1091	450.1	mean=446.7
Case 8	1112	447.4	standard deviation=3.056
Case 9	1110	450.9	
Case 10	1130	446.2	

Table A33. Summary of Time Period Sat. 10:00-10:30am Eleven Cases for Eastbound

Sat 10:00-10:30am	Average Corridor Volume	Corridor Travel Time(seconds)	
Original	865	463.8	
Case 1	871	463.5	
Case 2	895	484.2	
Case 3	895	479.6	**Average of Averaged Corridor Volumes**
Case 4	897	471.4	888
Case 5	904	479.0	
Case 6	898	470.9	**Corridor Travel Time(seconds)**
Case 7	890	471.8	mean=473.3
Case 8	897	480.2	standard deviation=6.710
Case 9	883	472.1	
Case 10	873	469.8	

Table A34. Summary of Time Period Sat. 10:00-10:30am Eleven Cases for Westbound

Sat 10:00-10:30am	Average Corridor Volume	Corridor Travel Time(seconds)	
Original	1210	413.7	
Case 1	1211	415.9	
Case 2	1183	412.3	
Case 3	1230	420.5	**Average of Averaged Corridor Volumes**
Case 4	1201	408.5	1202
Case 5	1223	420.3	
Case 6	1075	434.5	**Corridor Travel Time(seconds)**
Case 7	1208	421.8	mean=419.8
Case 8	1207	421.4	standard deviation=7.128
Case 9	1215	424.3	
Case 10	1255	425.1	

Table A35. Summary of Time Period Sat. 10:30-11:00am Eleven Cases for Eastbound

Sat 10:30-11:00am	Average Corridor Volume	Corridor Travel Time(seconds)	
Original	959	473.6	
Case 1	956	475.2	
Case 2	986	501.3	
Case 3	973	490.3	**Average of Averaged Corridor Volumes**
Case 4	975	487.3	974
Case 5	993	501.4	
Case 6	987	493.7	**Corridor Travel Time(seconds)**
Case 7	970	486.1	mean=487.7
Case 8	988	478.2	standard deviation=9.451
Case 9	970	492.6	
Case 10	954	485.3	

Table A36. Summary of Time Period Sat. 10:30-11:00am Eleven Cases for Westbound

Sat 10:30-11:00am	Average Corridor Volume	Corridor Travel Time(seconds)	
Original	1388	428.5	
Case 1	1385	428.6	
Case 2	1365	424.6	
Case 3	1429	439.1	**Average of Averaged Corridor Volumes**
Case 4	1385	430.3	1412
Case 5	1413	429.9	
Case 6	1403	430.6	**Corridor Travel Time(seconds)**
Case 7	1384	423.9	mean=431.3
Case 8	1399	440.8	standard deviation=5.804
Case 9	1406	439.1	
Case 10	1569	428.7	

Table A37. Summary of Time Period Sat. 11:00-11:30am Eleven Cases for Eastbound

Sat 11:00-11:30am	Average Corridor Volume	Corridor Travel Time(seconds)	
Original	1055	496.3	
Case 1	1051	499.4	
Case 2	1080	533.5	
Case 3	1089	505.4	**Average of Averaged Corridor Volumes**
Case 4	1076	509.1	1076
Case 5	1089	518.6	
Case 6	1098	518.1	**Corridor Travel Time(seconds)**
Case 7	1074	507.9	mean=511.7
Case 8	1092	520.1	standard deviation=10.440
Case 9	1081	511.6	
Case 10	1051	509.2	

Table A38. Summary of Time Period Sat. 11:00-11:30am Eleven Cases for Westbound

Sat 11:00-11:30am	Average Corridor Volume	Corridor Travel Time(seconds)	
Original	1512	435.9	
Case 1	1506	437.3	
Case 2	1492	442.7	
Case 3	1562	450.1	**Average of Averaged Corridor Volumes**
Case 4	1509	452.0	1517
Case 5	1513	440.0	
Case 6	1519	442.7	**Corridor Travel Time(seconds)**
Case 7	1497	438.4	mean=444.9
Case 8	1523	451.6	standard deviation=6.926
Case 9	1513	445.9	
Case 10	1541	456.8	

Table A39. Summary of Time Period Sat. 11:30-12:00pm Eleven Cases for Eastbound

Sat 11:30-12:00pm	Average Corridor Volume	Corridor Travel Time(seconds)	
Original	1094	493.9	
Case 1	1091	508.9	
Case 2	1119	544.8	
Case 3	1131	530.9	**Average of Averaged Corridor Volumes**
Case 4	1126	520.4	1116
Case 5	1128	521.8	
Case 6	1141	531.9	**Corridor Travel Time(seconds)**
Case 7	1114	523.3	mean=522.8
Case 8	1129	530.4	standard deviation=14.461
Case 9	1118	535.6	
Case 10	1087	508.6	

Table A40. Summary of Time Period Sat. 11:30-12:00pm Eleven Cases for Westbound

Sat 11:30-12:00pm	Average Corridor Volume	Corridor Travel Time(seconds)	
Original	1529	453.0	
Case 1	1525	446.9	
Case 2	1528	448.8	
Case 3	1564	461.4	**Average of Averaged Corridor Volumes**
Case 4	1524	448.6	1539
Case 5	1546	450.9	
Case 6	1541	465.1	**Corridor Travel Time(seconds)**
Case 7	1514	455.7	mean=456.0
Case 8	1535	455.6	standard deviation=9.103
Case 9	1543	451.9	
Case 10	1583	477.8	

Table A41. Summary of Time Period Sat. 12:00-12:30pm Eleven Cases for Eastbound

Sat 12:00-12:30pm	Average Corridor Volume	Corridor Travel Time(seconds)	
Original	1105	517.3	
Case 1	1102	517.1	
Case 2	1119	570.3	
Case 3	1139	533.1	**Average of Averaged Corridor Volumes**
Case 4	1125	523.0	1113
Case 5	1145	525.7	
Case 6	1153	535.0	**Corridor Travel Time(seconds)**
Case 7	1123	531.7	mean=533
Case 8	1015	539.0	standard deviation=16.5
Case 9	1130	552.5	
Case 10	1092	517.2	

Table A42. Summary of Time Period Sat. 12:00-12:30pm Eleven Cases for Westbound

Sat 12:00-12:30pm	Average Corridor Volume	Corridor Travel Time(seconds)	
Original	1563	448.8	
Case 1	1557	453.6	
Case 2	1572	451.1	
Case 3	1616	459.9	**Average of Averaged Corridor Volumes**
Case 4	1571	457.7	1582
Case 5	1581	452.0	
Case 6	1574	445.0	**Corridor Travel Time(seconds)**
Case 7	1571	451.3	mean=455.4
Case 8	1591	466.9	standard deviation=6.641
Case 9	1590	460.1	
Case 10	1616	463.1	

Table A43. Summary of Time Period Sat. 12:30-1:00pm Eleven Cases for Eastbound

Sat 12:30-1:00pm	Average Corridor Volume	Corridor Travel Time(seconds)	
Original	1087	501.4	
Case 1	1090	512.4	
Case 2	1111	547.8	
Case 3	1122	526.0	**Average of Averaged Corridor Volumes**
Case 4	1115	521.7	1110
Case 5	1123	532.6	
Case 6	1135	530.8	**Corridor Travel Time(seconds)**
Case 7	1109	519.2	mean=524.5
Case 8	1119	541.3	standard deviation=14.405
Case 9	1115	531.0	
Case 10	1086	504.9	

Table A44. Summary of Time Period Sat. 12:30-1:00pm Eleven Cases for Westbound

Sat 12:30-1:00pm	Average Corridor Volume	Corridor Travel Time(seconds)	
Original	1549	440.8	
Case 1	1522	439.1	
Case 2	1543	438.9	
Case 3	1575	455.0	**Average of Averaged Corridor Volumes**
Case 4	1544	455.8	1547
Case 5	1549	438.9	
Case 6	1542	448.5	**Corridor Travel Time(seconds)**
Case 7	1549	444.7	mean=446.9
Case 8	1545	449.5	standard deviation=6.976
Case 9	1548	448.2	
Case 10	1552	456.9	

Table A45. Summary of Extreme Case 1 Eleven Cases for Eastbound

Extreme Case 1	Average Corridor Volume	Corridor Travel Time(seconds)	
Original	779	450.9	
Case 1	778	447.0	
Case 2	795	459.6	
Case 3	807	451.6	**Average of Averaged Corridor Volumes**
Case 4	812	452.2	808
Case 5	810	456.3	
Case 6	803	470.0	**Corridor Travel Time(seconds)**
Case 7	795	454.0	mean=455.1
Case 8	807	457.4	standard deviation=6.420
Case 9	910	458.9	
Case 10	795	448.5	

Table A46. Summary of Extreme Case 1 Eleven Cases for Westbound

Extreme Case 1	Average Corridor Volume	Corridor Travel Time(seconds)	
Original	1536	443.4	
Case 1	1534	451.5	
Case 2	1525	450.9	
Case 3	1576	466.9	**Average of Averaged Corridor Volumes**
Case 4	1529	448.6	1547
Case 5	1569	458.9	
Case 6	1528	452.6	**Corridor Travel Time(seconds)**
Case 7	1544	459.6	mean=453.4
Case 8	1543	447.5	standard deviation=6.501
Case 9	1556	455.1	
Case 10	1579	452.7	

Table A47. Summary of Extreme Case 2 Eleven Cases for Eastbound

Extreme Case 2	Average Corridor Volume	Corridor Travel Time(seconds)	
Original	797	446.2	
Case 1	801	447.4	
Case 2	813	453.4	
Case 3	833	455.5	**Average of Averaged Corridor Volumes**
Case 4	834	455.0	821
Case 5	831	462.0	
Case 6	829	464.9	**Corridor Travel Time(seconds)**
Case 7	821	460.9	mean=457.7
Case 8	829	467.2	standard deviation=6.832
Case 9	823	462.7	
Case 10	819	459.1	

Table A48. Summary of Extreme Case 2 Eleven Cases for Westbound

Extreme Case 2	Average Corridor Volume	Corridor Travel Time(seconds)	
Original	1564	450.0	
Case 1	1564	453.8	
Case 2	1549	452.1	
Case 3	1610	458.0	**Average of Averaged Corridor Volumes**
Case 4	1549	449.9	1570
Case 5	1592	462.9	
Case 6	1547	455.2	**Corridor Travel Time(seconds)**
Case 7	1563	449.2	mean=455.5
Case 8	1555	457.3	standard deviation=5.046
Case 9	1576	459.2	
Case 10	1601	463.3	

Table A49. Summary of Extreme Case 3 Eleven Cases for Eastbound

Extreme Case 3	Average Corridor Volume	Corridor Travel Time(seconds)	
Original	846	464.7	
Case 1	843	455.0	
Case 2	865	471.7	
Case 3	877	463.1	**Average of Averaged Corridor Volumes**
Case 4	886	466.5	869
Case 5	882	474.2	
Case 6	880	478.6	**Corridor Travel Time(seconds)**
Case 7	867	461.4	mean=468.2
Case 8	880	472.8	standard deviation=7.028
Case 9	865	475.5	
Case 10	870	467.0	

Table A50. Summary of Extreme Case 3 Eleven Cases for Westbound

Extreme Case 3	Average Corridor Volume	Corridor Travel Time(seconds)	
Original	1662	475.7	
Case 1	1665	455.0	
Case 2	1640	467.7	
Case 3	1716	474.1	**Average of Averaged Corridor Volumes**
Case 4	1647	461.6	1671
Case 5	1704	487.3	
Case 6	1653	471.6	**Corridor Travel Time(seconds)**
Case 7	1661	466.3	mean=471.2
Case 8	1659	469.3	standard deviation=8.751
Case 9	1666	479.2	
Case 10	1709	475.6	

Table A51. Summary of Extreme Case 4 Eleven Cases for Eastbound

Extreme Case 4	Average Corridor Volume	Corridor Travel Time(seconds)	
Original	901	468.9	
Case 1	900	463.0	
Case 2	925	474.8	
Case 3	923	479.1	**Average of Averaged Corridor Volumes**
Case 4	930	485.1	920
Case 5	935	474.7	
Case 6	931	484.3	**Corridor Travel Time(seconds)**
Case 7	917	477.6	mean=476.0
Case 8	932	474.3	standard deviation=6.355
Case 9	917	477.8	
Case 10	910	479.5	

Table A52. Summary of Extreme Case 4 Eleven Cases for Westbound

Extreme Case 4	Average Corridor Volume	Corridor Travel Time(seconds)	
Original	1731	486.7	
Case 1	1717	477.7	
Case 2	1730	494.3	
Case 3	1777	492.9	**Average of Averaged Corridor Volumes**
Case 4	1724	478.0	1742
Case 5	1770	503.6	
Case 6	1725	487.5	**Corridor Travel Time(seconds)**
Case 7	1715	477.4	mean=492.0
Case 8	1748	483.3	standard deviation=15.200
Case 9	1751	502.7	
Case 10	1773	528.3	

EVALUATING THE EFFECTIVENESS OF LIFE CYCLE VARIABLES IN TRAVEL DEMAND MODELING

Prepared for
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SUMMARY

In order to prioritize and plan for future transportation projects, survey data are analyzed and used to estimate trip rate. Currently, in Texas, socioeconomic variables such as income and household size are used in estimation. These variables may not be the best trip rate estimators because many trips are necessary and not a luxury that can be foregone with economic difficulty. Travel has continued to increase over the past few decades while income level remains relatively constant, adding to the idea that socioeconomic variables may be becoming more obsolete in their trip estimation capabilities. It is hypothesized that life cycle variables, which consider the stage of life of a household, may be better variables to use in future travel demand modeling. As part of an Interagency Agreement funded by the Texas Department of Transportation, household travel survey data from the Texas urban areas of Tyler, Longview, Austin, and San Antonio were analyzed. The effectiveness of the socioeconomic variable of income level was compared

against that of four life cycle variables, which include households with no children, households with children under 16-years old, number employed in the household, and the head of household's age.

Based on statistical analysis, the head of household's age estimates trips significantly better than income level, yielding less than 1% error for the overall trips made in the four tested Texas urban areas. The number employed per household is also a better trip estimator than income level, but by less than 1% error. Though the variable of households with no children does not yield the most accurate trip estimations, a one-to-one comparison against income level shows comparable effectiveness. The number of kids under 16 yields a poorer estimation than income level.

Further research of life cycle variables is recommended. Though the results show some of the tested life cycle variables are more effective estimators than income level, all of the variables give somewhat comparable results. Thus, desired accuracy level, time constraints, and data availability should all be considered when deciding upon which variable to use in travel demand modeling. When the highest level of effectiveness is desired, the head of household's age variable is recommended.

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INTRODUCTION

Trip generation is the first step in the travel demand modeling process. In order to ensure accurate information is available for use in the steps that follow, great care should be taken in the gathering and analysis of data used in trip generation. For organized analysis, trip rate estimation is carried out in areas of similar geographic location, often called Traffic Analysis Zones (*1*). For analysis purposes, a trip is produced at the home and attracted to the business, regardless of whether the home is the trip's starting or ending point (*2*).

Information about the travel of an area is often obtained through travel surveys, containing questions not only about the number of trips made but also when, for what purpose, and by whom. Planners are then able to analyze the data. Socioeconomic variables such as income and household size are currently used in Texas for travel demand forecasting. However, socioeconomic variables may not be the most accurate means of obtaining estimates of future trip rate because many trips are necessary and not a luxury that can be foregone due to economic difficulty. Also, over the past few decades, travel has continued to increase while income level remains relatively constant. Thus, the effectiveness of socioeconomic variables in trip rate estimation may be decreasing with time. Life cycle variables, which consider the stage of life, may be more effective in travel demand modeling.

Research of specific life cycle variables has been done in the past. One existing model relates age of an individual and the average age of their household, claiming that distinct, repetitive trends occur that may be useful in predicting trip rate (*3*). Other research considers the effect historic events (period effects) and generation differences (cohort effects) play in travel demand modeling (*4*). Though there are a number of life cycle variables with the potential to accurately predict future travel, in order to be useful in real-world application the variable must be able to be accurately projected 20 or more years into the future.

It is hypothesized that some, if not all, life cycle variables may be more effective in travel demand modeling than the socioeconomic variable of income level. In order to test this hypothesis, statistical analysis was performed on household survey data obtained through the Texas Department of Transportation Travel Survey Program. The research was funded by the Texas Department of Transportation as part of Interagency Contract F, Travel Demand Model Data and Training.

BACKGROUND

Travel Survey Program

In an effort to more accurately and reliably estimate travel throughout Texas, the Travel Survey Program was established in 1990. The information obtained through these surveys is used to prioritize future road projects and improvements. The four main facets of the survey program include external surveys, household surveys, work place surveys, and commercial vehicle surveys. Each region in Texas is surveyed in a rotational manner, allowing for widespread and continual travel demand modeling improvements (*5*). Because Texas is one of the few states

with an organized travel survey program in place, the results are sometimes borrowed by other states and adapted for use in transportation project planning.

Texas Urban Areas

Four of the Texas urban areas recently surveyed as part of the Travel Survey Program include Tyler, Longview, Austin and San Antonio. Individual organization of each area was performed, and then the data were combined for analysis. Though individual area comparisons may be indicative of unique transportation environments, often a low number of households are included in survey substratum when only considering one area, leading to somewhat skewed and random trip rate estimates. Thus, in many respects, the combination of urban area results is more accurate and meaningful for future model use. For this reason, as well as time constraints, the percent error in estimating total trips made was only calculated for the combined survey data from all four urban areas.

It is important to note that although the household travel surveys were not performed in the same year for all four areas, the 2.5-year time span of fall 2003 to spring 2006 is considered insignificant for the purposes of the research. Background information on each area, obtained from technical reports compiled by the Texas Department of Transportation, follows. Figure 1 shows the general geographic location within Texas of each analyzed urban area.

Figure 1. General Location of Urban Areas

Tyler/Smith County

- Household travel survey conducted on a school year weekday in fall 2003
- Households: 69,600 (estimated)
- Average Number of Vehicles Per Household: 1.9 vehicles
- Population: 178,000 (estimated)
- Average Number of Trips per Person: 3.6 trips
- Average Trip Distance: 6.1 miles
- Average Trip Duration: 9 minutes
- Percent of Vehicles that are Pick-up Trucks: 28% (higher than national statistic of 18%) (*6*)

Longview/Gregg County

- Household travel survey data collected during a school year weekday in fall 2003
- Households: 42,782 (estimated)
- Average Number of Vehicles per Household: 1.9 vehicles
- Population: 113,624 (estimated)
- Average Number of Trips per Person: 4.0 trips
- Average Trip Distance: 4.6 miles
- Average Trip Duration: 7.3 minutes
- Percent of Vehicles that are Pick-Up Trucks: 29% (*7*)

Austin/Capitol Area

- Household travel survey data conducted during a school year weekday in fall 2005 and spring 2006
- Included Areas: Bastrop, Caldwell, Hays, Travis, and Williamson counties
- Home of University of Texas (contributing to 24% of population being students)
- Households: 574,225 (estimated)
- Average Number of Vehicles per Household: 1.88 vehicles
- Population: 1,484,934 (taken from expanded survey – Tyler and Longview technical reports make no mention of expanded version use)
- Average Trip Distance: 7.8 miles
- Average Trip Duration: 12.8 minutes
- Home of University of Texas (contributing to 24% of population being students) (*8*)

San Antonio

- Household travel survey data conducted during a school year weekday in fall 2005 and spring 2006
- Included Areas: Kendall, Comal, Guadalupe, Wilson, and Bexar counties
- Number of Households Surveyed: 2,000
- Households: 641,487 (expanded and estimated)
- Population: 1,791,418
- Average Number of Person Trips: 3.31 trips
- Similar to Austin, 25% of population is students (*9*)

RESEARCH PROCESS

Literature Review

In order to become more familiar with trip generation and life cycle variables, a literature review was performed. Information on travel demand modeling, household travel surveys, previous research involving life cycle variables, as well as the Texas urban areas of interest was reviewed. As the research developed, there was a continual need to glean more information related to the

project. Literature review was particularly concentrated near the beginning and end of the project.

Proposal and Variable Selection

Although not followed in its entirety, an original proposal of the research project was created, outlining how to organize and analyze the data. This served to guide the sequence of steps involved in the research project. Also, prior to the start of the project, life cycle variables of particular interest were selected for study. Small adjustments to the substratum breakdown and variable classifications were made throughout the project. Selected life cycle variables, along with income level, were analyzed on a household basis and include the following:

- Income Level
  - Level 1 (0-$19,999)
  - Level 2 ($20,000-$34,999)
  - Level 3 ($35,000-$49,999)
  - Level 4 ($50,000-$74,999)
  - Level 5 ($75,000 +)
- Number of Kids Under 16
  - 0
  - 1
  - 2
  - 3+
- No Children (Under 18)
- Head of Household's Age (in years)
  - 18-24
  - 25-34
  - 35-49
  - 50-64
  - 65+
- Number Employed
  - 0
  - 1
  - 2+

Note that the substratums are set-up such that each household can only fit into one substratum per variable. This is a critical component in the analysis, ensuring that no double counting of households occurs and that results are not confusing and misleading. Also notice that there may be differences in the number of households with 0 kids under 16 and households with no children, as "no children" is defined as households with no members under 18, not 16. The variable of the number of kids under 16 was of particular interest because prior to 16, kids are unable to legally drive without an adult. Thus, it was hypothesized that households with kids under 16 make more person trips because every time a kid makes a trip in a vehicle so does an adult.

For consistency throughout the analysis, the head of household is defined as the oldest member of the household. The head of household's age substratum breakdown was chosen in an attempt to roughly capture distinct stages of life. For example, 18-24 is college age, 25-34 is young adult life, 35-49 is child-rearing years, 50-64 is middle-age, and 65+ is retired. While not everyone's situation fits succinctly into these categories, it yields a general picture of travel during different stages of life.

Data Organization

The survey data files for each urban area had to be properly formatted and organized prior to any analysis. The data were imported into Excel and given column labels. Tabulation columns were added for more efficient analysis. Because much of the data was in code, it took time to become familiar with the set-up and meaning. Filters and the sort option within Excel were utilized to more quickly and accurately tabulate trip rates for each variable's substratum. Initially the filtering option posed difficulties, allowing for the summing of only information lining up perfectly with the filtered row. The fact that blank cells indicate that no trip was made by that household member was clarified mid-tabulation, and proper corrections applied. Due to the steep learning curve associated with such difficulties, data organization and manipulation were done one urban area at a time so that the same mistakes and difficulties were not repeatedly encountered.

Statistical Analysis

Multiple statistical methods were utilized in analysis. Linear regression was performed, yielding graphs, correlation coefficients, and equations for each variable. In addition, cross classification matrices were created showing the average person trips per household, number of households, and 90% Confidence Interval associated with the person trips per households made in each substratum. While these two methods give a rough idea of the effectiveness of each tested variable, a quantitative method was needed for solid statistical results. Thus, the percent error was calculated using subsets consistent with a one-to-one comparison between the life cycle variables and income level.

RESULTS

Multiple urban areas, life cycle variables, and statistical methods pertain to this research project, generating a large volume of figures and tables. For the purposes of the paper, only the combined overall data from all four urban areas are included and discussed within the body of the text. Refer to the Appendix for partial results obtained for each individual area. Note that some tables in the Appendix contain cells where either not enough information was available (nei) or the available data were limited in scope and not applicable to the cell (NA). For Tyler and Longview in particular, the equations and correlation coefficient values obtained from the line forced through the origin are not applicable to for those households with 3 or more kids under 16.

Linear Regression

The linear regression graphs created allow trends between household size and the average person trips per household to be seen; a positive trend existing for each tested variable. Figures 2-6 show the graphs and their associated trends. Notice that the graphs include only those lines that were forced through the origin. This was done to ensure that practical results were represented. A household with 0 persons would be expected to make 0 trips.

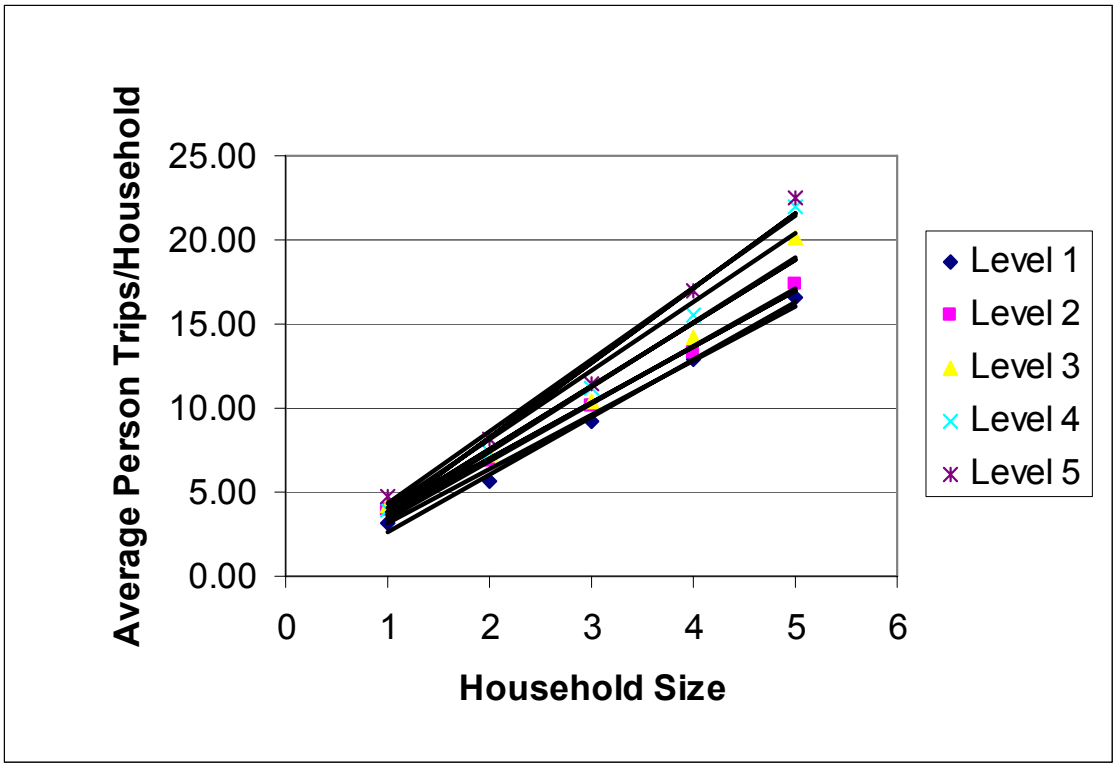

Figure 2. Linear Regression Forced through Origin for Income Level

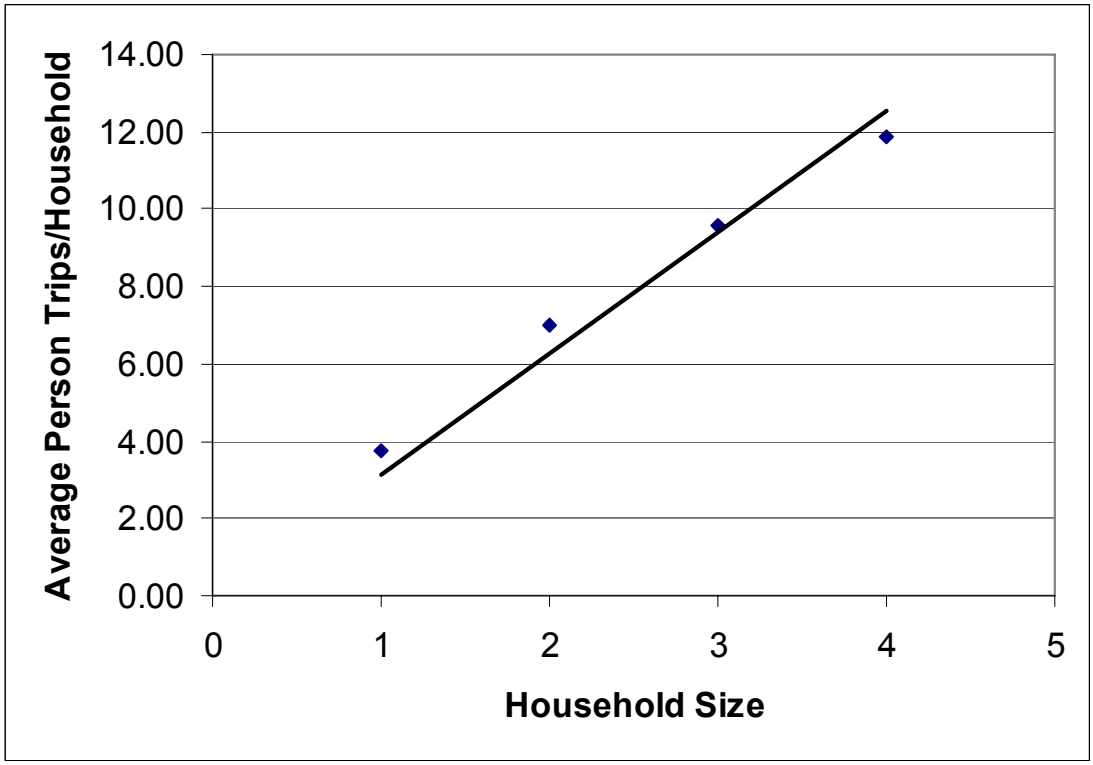

Figure 3. Linear Regression Forced through Origin for No Children

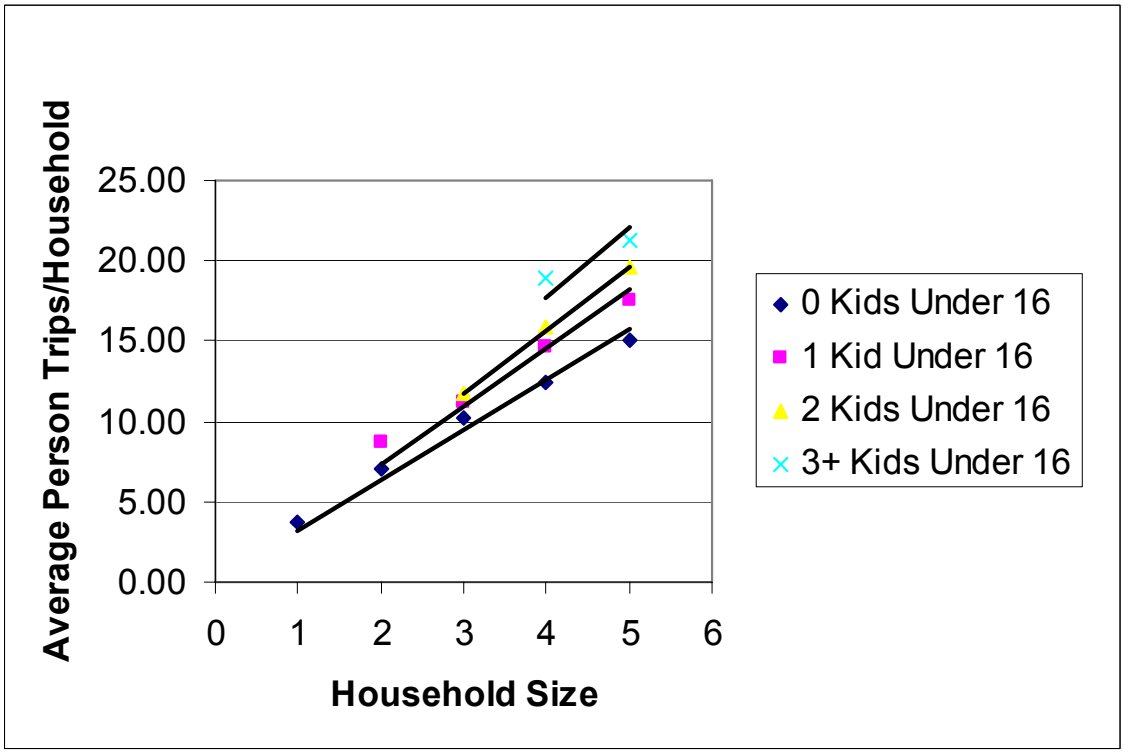

Figure 4. Linear Regression Forced through Origin for Number of Kids Under 16

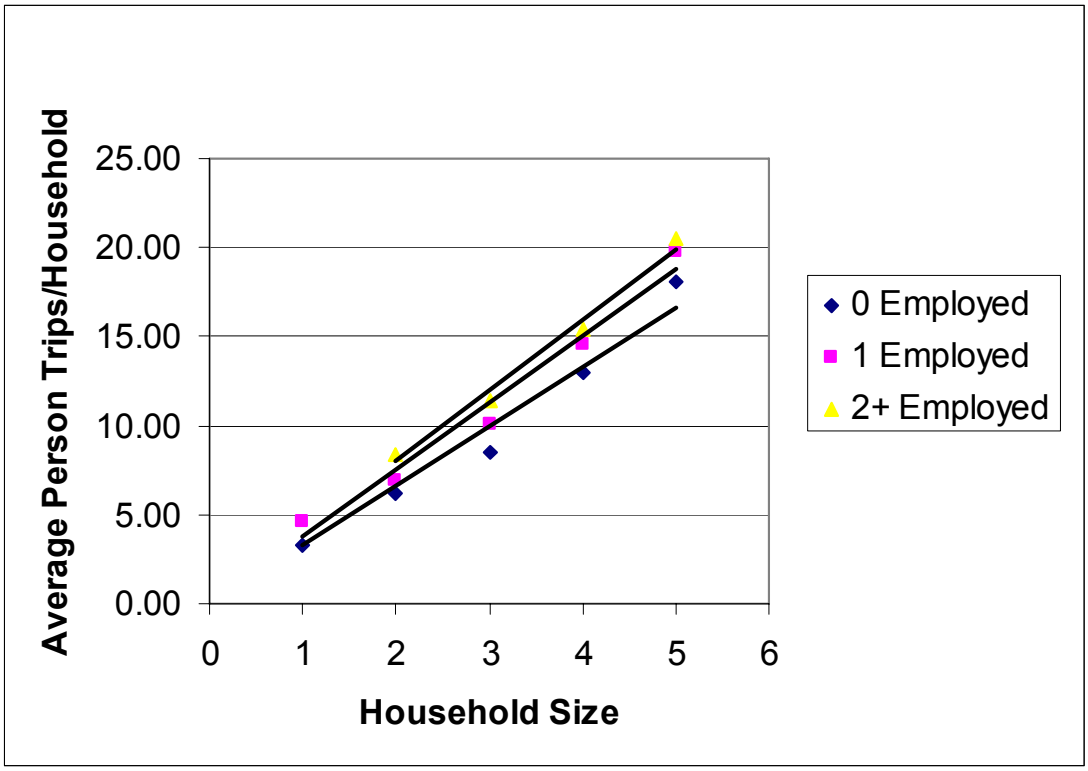

Figure 5. Linear Regression Forced through Origin for Number Employed

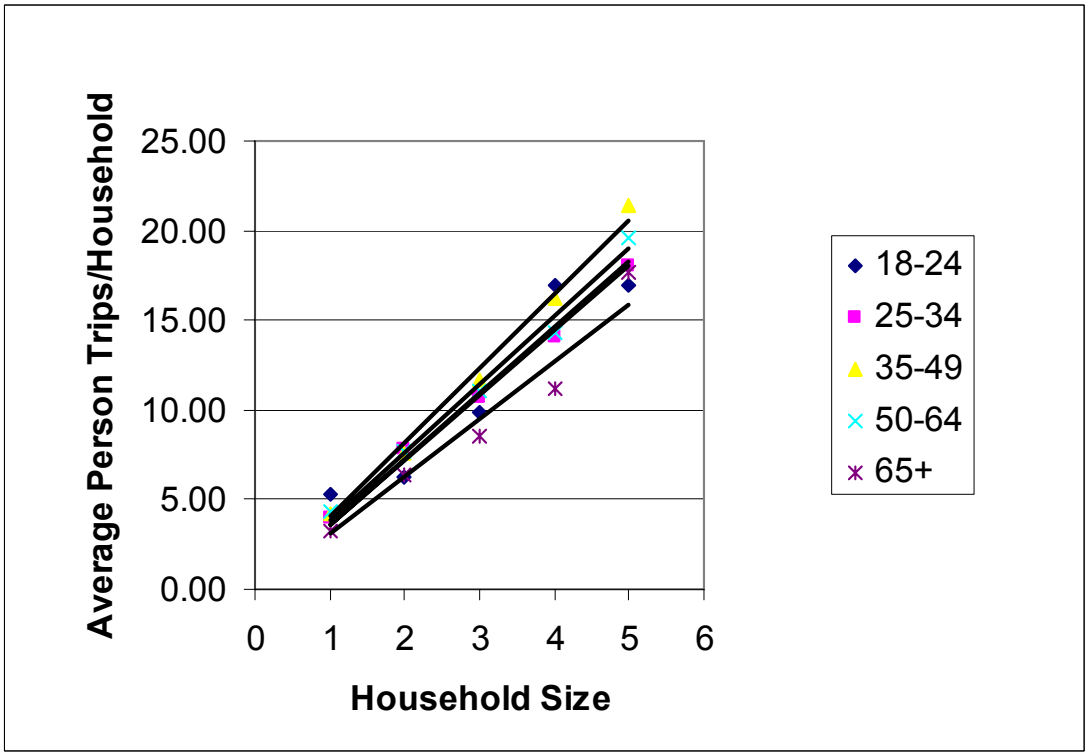

Figure 6. Linear Regression Forced through Origin for Head of Household's Age

Although more closely modeling real-life, the lines forced through the origin do not cross through the plotted points as accurately as the line of best fit. This difference can be seen in how the correlation coefficients move farther away from magnitude 1 when the line is forced through the origin. Table 1 shows the correlation coefficients and equations associated with both type of lines. Because the values are rounded to the nearest two decimal places, small adjustments to the correlation coefficients are not seen, though still present. Note that when significant changes in the slope of the line occur, changes also occur in the correlation coefficient. The fact that little, if any, noticeable change occurs in correlation coefficient values for income level indicates that little adjustment is needed to ensure real-life modeling.

Table 1. Linear Regression Equations and Correlation Coefficients

	Income Level				
	1	**2**	**3**	**4**	**5**
Line of Best Fit					
Equation	y = 3.43x - 0.81	y = 3.32x + 0.36	y = 3.86x - 0.36	y = 4.40x - 1.23	y = 4.44x - 0.57
r^2 Value	0.99	1.00	0.98	0.98	0.98
Forced through Origin					
Equation	y = 3.21x	y = 3.42x	y = 3.77x	y = 4.07x	y = 4.28x
r^2 Value	0.99	1.00	0.98	0.97	0.98

The correlation coefficient for the variable of no children varies more than the coefficients for income level do when forced to meet practical limitations. However, the difference is still considered minor, with the coefficient staying in the 0.90-1.00 range. Notice that the slope difference between the two lines is more drastic than any differences existing for the income level variable, explaining the greater change in correlation coefficient value as seen in Table 2.

Table 2. Linear Regression Equations and Correlation Coefficients

No Children	
Line of Best Fit	
Equation	y = 2.71x + 1.29
r^2 Value	0.99
Forced through Origin	
Equation	y = 3.14x
r^2 Value	0.96

As seen in Table 3, for the number of kids under 16 variable, the correlation coefficients obtained from the line of best fit yield correlation coefficients equal, or nearly equal to 1.00. However, this apparent high level of correlation disappears for the substratum of 3 or more kids under 16 when the line is forced through the origin. One of the reasons for this drastic decrease is the fact that only households in the 4 and 5 or more household size substratums ever have 3 or more kids under 16. Thus, for the line of best fit equation, the line goes right through these two points, yielding a correlation coefficient of 1.00. Because there are not more household sizes represented on the graph, when forced through the origin the slope of the line greatly increases, causing the correlation to steeply decrease.

Table 3. Linear Regression Equations and Correlation Coefficients

	Number of Kids Under 16			
	0	**1**	**2**	**3+**
Line-of-Best-Fit				
Equation	y = 2.81x + 1.25	y = 3.02x + 2.45	y = 3.89x + 0.17	y = 2.35x + 9.46
r^2 Value	0.99	1.00	1.00	1.00
Forced through Origin				
Equation	y = 3.15x	y = 3.65x	y = 3.93x	y = 4.43x
r^2 Value	0.98	0.95	1.00	0.21

For the number employed variable, there is no noticeable change in any of the correlation coefficients (when rounded to two decimal places), as seen in Table 4. This indicates that the lines of best fit represent real-life limitations fairly accurately, even prior to ensuring the lines pass through the origin.

Table 4. Linear Regression Equations and Correlation Coefficients

	Number Employed		
	0	**1**	**2+**
Line of Best Fit			
Equation	y = 3.64x - 1.12	y = 3.82x - 0.22	y = 4.04x - 0.18
r^2 Value	0.97	0.98	0.99
Forced through Origin			
Equation	y = 3.33x	y = 3.75x	y = 3.99x
r^2 Value	0.97	0.98	0.99

For the head of household's age, very little change in the correlation coefficients is seen between the two types of generated lines. However, the 18-24-year-old substratum correlation coefficient values obtained from both line type cases is lower than the coefficients of the other age category substratum. This lower coefficient, though still indicating high correlation, may be partially explained by the fact that so few 18-24 year-old households participated in the survey. The surveys are completed on a voluntary basis, with no one being forced to fill-out and return the survey. The younger generation is not as likely to return the survey, causing under-representation and making it difficult to accurately model their travel. Another component that may contribute to the low number of younger generation surveys is that the head is defined to be the oldest person within the household. If an older spouse, parent, or grandparent is living in the same household, their age trumps the 18-24-year-old. The head of household's age results can be seen in Table 5.

Table 5. Linear Regression Equations and Correlation Coefficients

	Head of Household's Age				
	18-24	**25-34**	**35-49**	**50-64**	**65+**
Line of Best Fit					
Equation	y = 3.41x + 0.85	y = 3.45x + 0.58	y = 4.31x - 0.72	y = 3.72x + 0.24	y = 3.35x - 0.63
r^2 Value	0.91	1.00	0.99	0.99	0.95
Forced through Origin					
Equation	y = 3.65x	y = 3.61x	y = 4.11x	y = 3.79x	y = 3.18x
r^2 Value	0.91	0.99	0.99	0.99	0.95

Cross Classification Matrices

In addition to linear regression analysis, cross classification matrices were created as a quick means to see and explain trends. Matrices showing the average person trips per household, the number of households, and the 90% confidence intervals associated with the average person trips per household for each substratum were created. While it is desirable to have at least 30 households in each substratum to better ensure data are representative of the population, even after combining the results from all four urban areas there are still substratum with low household counts. This shortcoming contributes to some of the estimation error.

The matrices containing the 90% confidence intervals for the average person trips per household give a general idea of how reliable the tabulated average person trips per household value is. Generally, the smaller the interval, the more accurately the average person trips per household represents the average of the population and not just the sample. In essence, the interval is saying that we would expect the average person trips per household to fall within the given interval 90% of the time when sampling. A built-in Excel function was used to calculate the desired confidence interval, following the calculation of the standard deviation using another built-in Excel function.

The income level matrices are shown in Table 6. True to intuition, as household size increases, the average person trips per household also increase. Vertically moving down the chart for a given household size, as income level increases so does the average person trips made per household. A minor exception to this overall trend occurs between income levels 3 and 4 for households of 1 person.

Table 6. Cross Classification Matrices for Income Level

Average Person Trips/Household					
	Household Size				
Income Level	**1**	**2**	**3**	**4**	**5+**
1	3.11	5.60	9.23	12.88	16.64
2	3.94	6.89	10.12	13.31	17.33
3	4.21	7.28	10.40	14.16	20.10
4	4.01	7.31	11.17	15.47	21.94
5	4.68	8.15	11.46	16.94	22.48
Number of Households in Each Substratum					
	Household Size				
Income Level	**1**	**2**	**3**	**4**	**5+**
1	372	371	124	74	77
2	228	401	177	144	124
3	218	391	215	197	114
4	103	399	216	192	109
5	95	410	294	246	158
90% Confidence Interval					
	Household Size				
Income Level	**1**	**2**	**3**	**4**	**5+**
1	(2.88,3.34)	(5.25,5.95)	(8.34,10.13)	(11.31,14.45)	(15.05,18.22)
2	(3.66,4.22)	(6.51,7.27)	(9.40,10.84)	(12.23,14.38)	(16.01,18.64)
3	(3.87,4.55)	(6.90,7.67)	(9.75,11.05)	(13.35,14.96)	(18.55,21.65)
4	(3.65,4.36)	(6.89,7.72)	(10.46,11.87)	(14.57,16.37)	(20.33,23.54)
5	(4.20,5.17)	(7.79,8.51)	(10.87,12.06)	(16.10,17.77)	(20.80,24.16)

Looking at the matrices found in Tables 6-10, it is interesting to note that the most common surveyed household size is 2 persons, with households of 5 or more persons being the least common household size. This is consistent with the trends discussed in each area's technical summary (*6-9*).

Table 7 shows the matrices for households with no children. While it is interesting to note the increase in person trips per household with increase in household size, the sudden drop in surveyed households as you increase to households with 4 or more persons is stark. It is for this reason that the no children variable was grouped into household size substratum differently than the other variables. The estimations deviated from their linear nature when the household size substratum went up to 5 or more persons.

Table 7. Cross Classification Matrices for No Children

Average Person Trips/ Household				
Household Size				
	1	2	3	4+
No Children	3.74	7.00	9.59	11.90
Number of Households in Each Substratum				
Household Size				
	1	2	3	4+
No Children	1056	1993	449	106
90% Confidence Interval				
Household Size				
	1	2	3	4+
No Children	(3.60,3.88)	(6.83,7.16)	(9.12,10.07)	(10.81,12.98)

The life cycle variable of number of kids under 16 has increasingly fewer and fewer household sizes represented as the number of kids in the household increases because it is not possible for there to be more kids under 16 than there are persons in the household. Although theoretically possible, no households exist where the number of kids under 16 equals the household size. Kids under 16 are not yet independent and need the support and care of an adult. This dependency appears to transfer over into trips. Although able to use other forms of transportation independently, kids under 16 cannot make a trip in a vehicle without a licensed driver accompanying them. As can be seen in Tables 8, as the number of kids under 16 increases so does the average person trips per household for a given household size. There are very few households with 4 persons and 3 or more kids under 16, as this may indicate a single parent of multiple children. Likewise, there are also only a small number of households with no kids under 16 and 5 or more household members.

Table 9 shows that roughly half of the households with no one employed have 2 persons, a possible indication of retired couples. Also, there are very few large households with no one employed. It makes sense that as the number of mouths to feed increases, an income must be earned through employment.

Table 8. Cross Classification Matrices for Number of Kids Under 16

Average Person Trips/ Household					
	Household Size				
# Kids Under 16	**1**	**2**	**3**	**4**	**5+**
0	3.74	7.01	10.19	12.42	15.09
1	none	8.66	11.17	14.65	17.56
2	NA	none	11.76	15.91	19.55
3+	NA	NA	none	18.87	21.22
Number of Households in Each Substratum					
	Household Size				
# Kids Under 16	**1**	**2**	**3**	**4**	**5+**
0	1056	2012	546	148	22
1	none	53	461	202	72
2	NA	none	38	494	150
3+	NA	NA	none	15	343
90% Confidence Interval					
	Household Size				
# Kids Under 16	**1**	**2**	**3**	**4**	**5+**
0	(3.60,3.88)	(6.84,7.17)	(9.75,10.63)	(11.47,13.37)	(12.39,17.79)
1	none	(7.34,9.98)	(10.72,11.62)	(13.81,15.49)	(15.73,19.38)
2	NA	none	(10.22,13.31)	(15.32,16.50)	(18.14,20.95)
3+	NA	NA	none	(15.95,21.79)	(20.23,22.20)

Table 9. Cross Classification Matrices for Number Employed

Average Person Trips/ Household					
	Household Size				
# Employed	**1**	**2**	**3**	**4**	**5+**
0	3.24	6.25	8.48	12.93	18.09
1	4.58	6.97	10.12	14.62	19.83
2+	NA	8.38	11.46	15.43	20.52
Number of Households in Each Substratum					
	Household Size				
# Employed	**1**	**2**	**3**	**4**	**5+**
0	667	873	113	27	34
1	393	640	357	330	224
2+	NA	554	575	504	329
90% Confidence Interval					
	Household Size				
# Employed	**1**	**2**	**3**	**4**	**5+**
0	(3.07,3.41)	(6.00,6.50)	(7.56,9.39)	(10.10,15.76)	(15.66,20.52)
1	(4.34,4.81)	(6.67,7.27)	(9.64,10.60)	(13.89,15.35)	(18.57,21.08)
2+	NA	(8.06,8.71)	(11.02,11.89)	(14.88,15.98)	(19.79,21.25)

While Table 10 shows an increase in average person trips per household with increase in household size, there is not an obvious trend among the different head of household age categories. Once again, there are a very small number of households in the 18-24-year-old range, possibly skewing the results.

Table 10. Cross Classification Matrices for Head of Household's Age

Average Person Trips/ Household					
	Household Size				
Head's Age	**1**	**2**	**3**	**4**	**5+**
18-24	5.27	6.31	9.87	17.00	17.00
25-34	4.00	7.78	10.65	14.12	18.07
35-49	4.19	7.53	11.71	16.19	21.41
50-64	4.29	7.74	11.11	14.34	19.61
65+	3.28	6.42	8.55	11.24	17.62
Number of Households in Each Substratum					
	Household Size				
Head's Age	**1**	**2**	**3**	**4**	**5+**
18-24	11	13	15	5	3
25-34	31	81	115	137	88
35-49	140	193	354	459	312
50-64	308	726	330	189	122
65+	570	1053	231	68	61
90% Confidence Interval					
	Household Size				
Age of Head	**1**	**2**	**3**	**4**	**5+**
18-24	(3.87,6.68)	(4.76,7.86)	(7.56,12.17)	(8l76,25.24)	(14.49,19.51)
25-34	(3.43,4.57)	(6.85,8.70)	(9.77,11.53)	(13.06,15.17)	(16.40,19.74)
35-49	(3.86,4.52)	(7.03,8.03)	(11.16,12.26)	(15.59,16.78)	(20.40,22.43)
50-64	(3.00,4.58)	(7.45,8.04)	(10.57,11.65)	(13.45,15.22)	(18.04,21.17)
65+	(3.10,3.46)	(6.20,6.65)	(7.94,9.16)	(9.74,12.73)	(15.27,10.98)

Percent Error

Although linear regression analysis and cross classification matrices serve as quick tools to observe general statistical results, it became necessary to quantitatively determine which variable type (life cycle or socioeconomic) serves as a more effective forecaster in travel demand modeling. The percent error in trip estimation was found based on the linear regression equation of the line forced through the origin. The percent error equation is as follows:

$$\%Error = \frac{100 \cdot (\varepsilon - \alpha)}{\alpha} \quad \text{(Equation 1)}$$

where

$\varepsilon =$ Estimated Number of Trips

$\alpha =$ Actual Number of Trips

Thus, the closer the estimated number of trips is to the actual number of trips made, the smaller the percent error. The value may be positive or negative depending on whether the number of trips was over or under estimated; magnitude is what is generally important. However, if all estimates are significantly off in the same direction, the cause should be further investigated. In order for a one-to-one comparison to be made between income level and the tested life cycle variables, appropriate subsets of households were formed and used in finding the percent error. Life cycle variables were sorted such that only those households counted in a previous income level analysis were counted for the percent error calculation. The same substratum was used for all variables, except households with no children, as all households with children are excluded in the total trip tabulation for households without children. It is important to distinguish between trip rate and total trips made, as both terms are used in analysis. Trip rate, for the purposes of the research project, is person trips per household. By multiplying trip rate by the number of surveyed households, the total trips made are calculated. Though the created matrices show trip rates, the percent error calculation deals with total trips made, which is ultimately what engineers and planners consider when planning for future projects. Tables 11 and 12 show the percent error calculated using the equation for the line forced through the origin.

Table 11. Percent Error Based on Linear Equation Forced through Origin

Variable	Actual Trips	Estimated Trips	% Error
Income Level	53191	54638.54	2.72
# Kids Under 16	53193	51091.25	-3.95
# Employed	53193	54147.33	1.79
Head's Age	53193	53400.64	0.39

Table 12. Percent Error Based on Linear Equation Forced through Origin

Variable	Actual Trips	Estimated Trips	% Error
Income Level	22572	24630.33	9.12
No Children	22572	20573.28	-8.85

In looking at the overall percent error results, it is clear that all tested variables exhibit an ability to estimate trips, with all variables estimating the actual number of trips with less than 10% error. However, the head of household's age is the most accurate estimator, with less than 1% error in estimating the actual number of trips made by the selected subset of households. The number employed is also more effective than income level, though not by as large a percentage. Surprisingly, the number of kids under 16 is less effective than income level in trip estimation. Using the equations of the lines forced through the origin, households by the number of kids under 16 and households with no children underestimate trips made, while all other variables overestimate. Figure 7 gives a visual representation of the overall percent error magnitude for each variable.

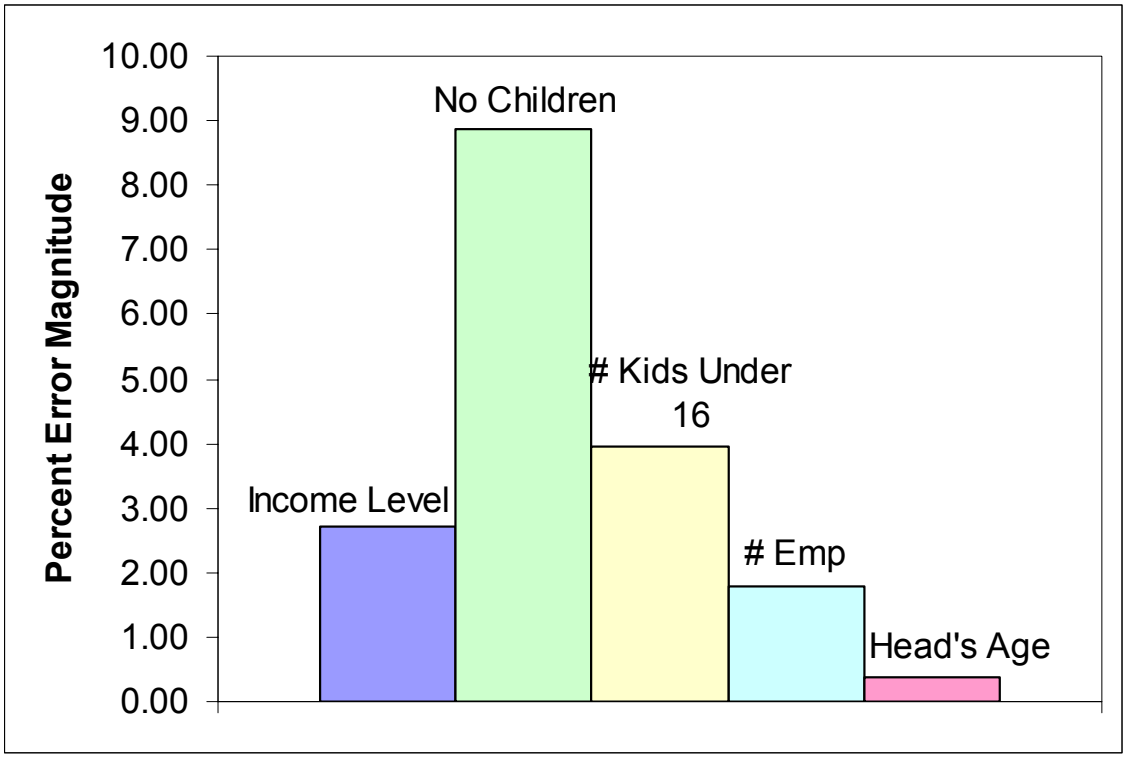

Figure 7. Overall Percent Error Magnitude

Note that income level was able to predict the actual number of trips made with only 2.72% error when calculated for all substratum for which the equation was created. However, when the same equation was applied only up until the 4 or more persons per household substratum, the estimation accuracy greatly decreased to 9.12% error. This adjustment was nonetheless needed to allow for a one-to-one comparison with households with no children, but undoubtedly contributed to the larger percent error for income level than previously calculated. Looking at the data valid for a one-to-one comparison, the magnitude of percent error is nearly identical for income level and households with no children. Thus, while the no children variable appears to be the least effective trip estimator overall, it is a slightly better estimator than income level.

However, when looking at the range that exists between the two percent errors, there is nearly an 18% difference, with income level overestimating and the households with no children variable underestimating the actual number of trips made. Figure 8 shows the difference in percent error between each life cycle variable and the appropriate one-to-one comparison results for income level. Those life cycle variables plotted above the x-axis indicate a more effective trip estimate than income level, while the number of kids under 16 variable is less effective.

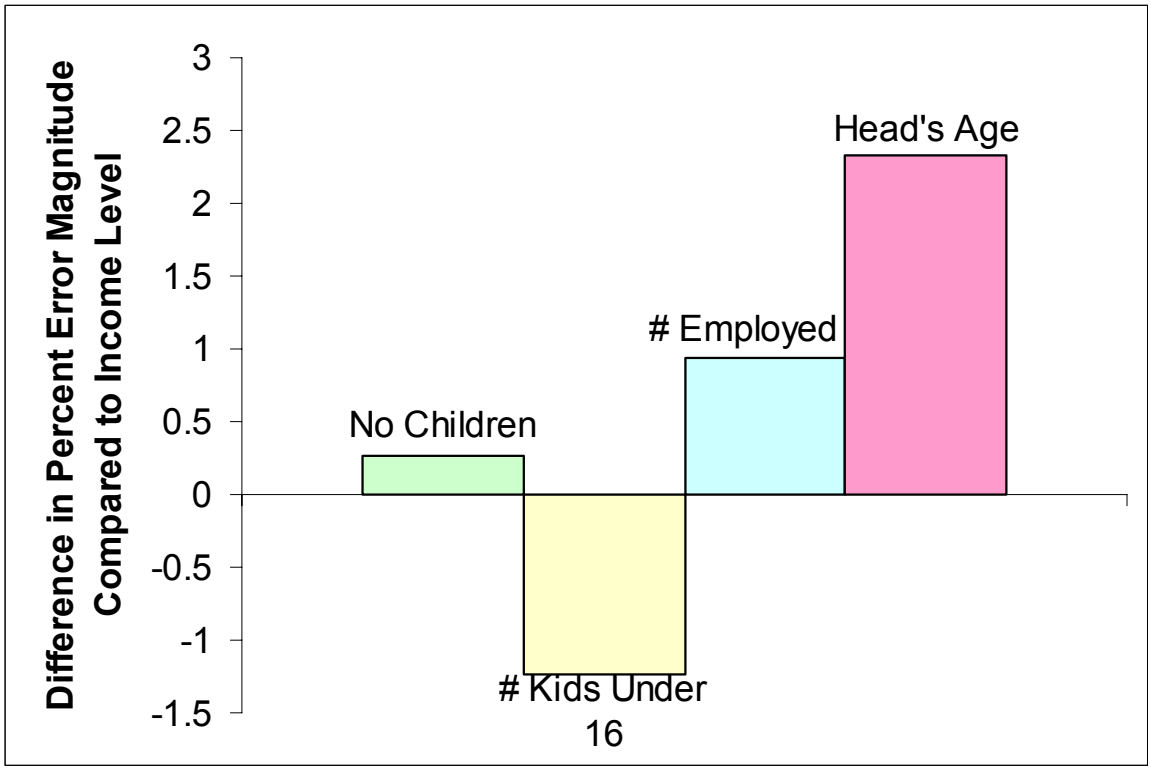

Figure 8. Percent Error Magnitude Compared to Income Level

CONCLUSION

Based upon the results, the hypothesis that at least one of the tested life cycle variables is a better estimator of trip rate than income level is correct. The head of household's age is significantly more effective, estimating the actual person trips made with only 0.39% error. The number employed is also more effective than income level, though differing by less than 1% error.

The life cycle variable of no children, although more effective than income level in a one-to-one comparison, is the worst tested variable, underestimating the actual number of trips made by well over 8%. Surprisingly, the number of kids under 16 is not more effective than income level, as was anticipated during the variable selection process. Overall, no tested variable should be considered a terrible trip estimator. All tested variables correlate to trip indication, predicting the actual number of trips made with relative accuracy.

BENEFITS

The research results are exciting in the future of survey data analysis. While the commonly used socioeconomic variable of income level performed relatively well, three of the four tested life cycle variables performed more effectively in trip estimation. With more effective variable options available, engineers and planners will be able to more efficiently plan for and implement future transportation projects. Effective and efficient trip generation analysis will lead to success in the steps that follow in the travel demand modeling process, benefiting all who travel.

RECOMMENDATIONS

Use of these life cycle variables is recommended in future travel survey data analysis. However, consideration as to the data and time available, as well as the desired level of accuracy in trip estimation should be taken in selecting variables for use in transportation planning. Time is money, and thus whatever data are most readily accessible may be the best variable to use in modeling. When data and time permit, it is recommended that the head of household's age be selected for trip estimation.

Further research of life cycle variables is recommended. The results obtained in this research project could be expounded upon. For instance, the head of household age groupings could be altered or considered dually with other variable subsets such as gender. Graphs, equations, and correlation coefficients could be made using exponential or quadratic settings to better explain deviance in linear regression analysis. Regression analysis of multiple life cycle variables at a time could be performed, combining the factors associated with such variables as head of household's age and the number employed. It would also be interesting to complete the analysis started on each individual urban area, allowing for comparison. For this process, it is recommended that a larger number of households be surveyed in those substratums that are under-represented. It is possible that individual urban areas would yield different results that may be explained by the demographics of an area.

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APPENDIX

Tyler

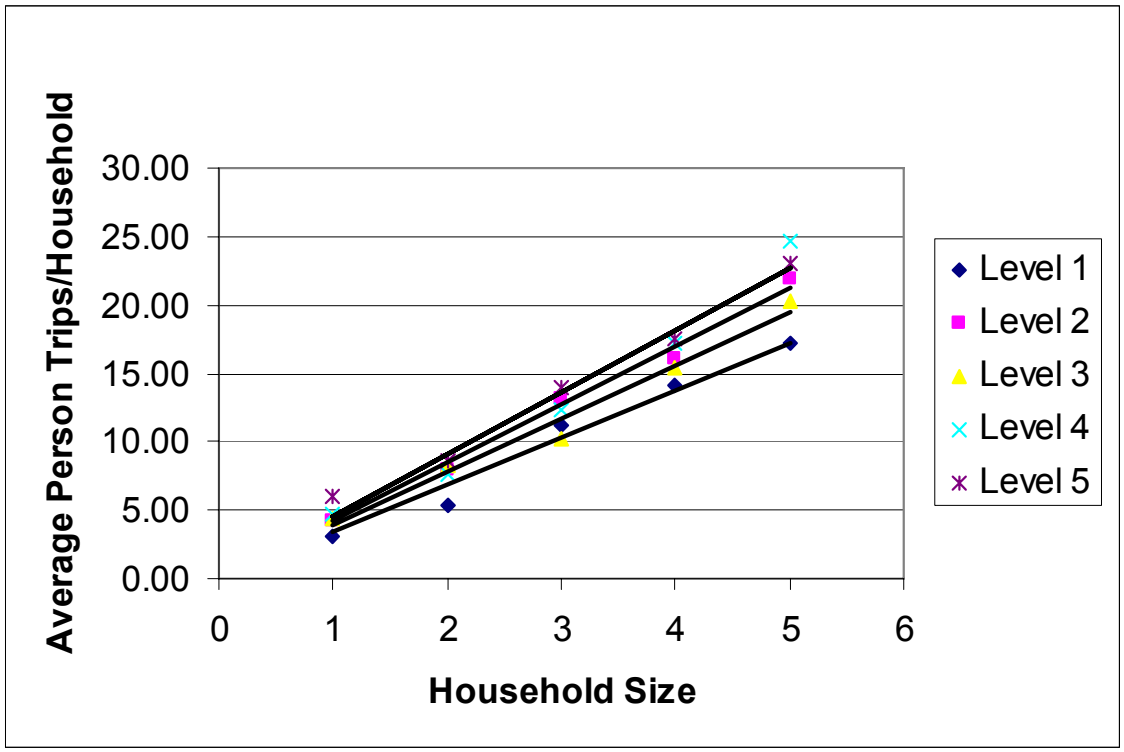

Figure A1. Tyler Linear Regression Forced through Origin for Income Level

Table A1. Tyler Linear Regression Equations and Correlation Coefficients

	Income Level				
	1	**2**	**3**	**4**	**5**
Line of Best Fit					
Equation	y = 3.72x - 0.97	y = 4.34x - 0.37	y = 3.90x + 0.01	y = 4.97x - 1.61	y = 4.31x + 0.91
r^2 value	0.98	0.99	0.98	0.97	0.99
Forced through Origin					
Equation	y = 3.45x	y = 4.24x	y = 3.90x	y = 4.53x	y = 4.55x
r^2 value	0.97	0.99	0.98	0.97	0.98

Table A2. Tyler Cross Classification Matrices

Average Person Trips/HH					
	Household Size				
Income Level	**1**	**2**	**3**	**4**	**5+**
1	3.04	5.30	11.24	14.06	17.25
2	4.27	7.89	13.19	16.04	21.92
3	4.39	8.29	10.16	15.39	20.33
4	4.67	7.60	12.26	17.26	24.67
5	6.00	8.57	13.97	17.53	23.05
Number of Households in Each Substratum					
	Household Size				
Income Level	**1**	**2**	**3**	**4**	**5+**
1	96	82	25	18	8
2	48	81	26	26	25
3	62	105	38	31	12
4	21	91	39	31	9
5	23	94	38	30	20
90% Confidence Interval					
	Household Size				
Income Level	**1**	**2**	**3**	**4**	**5+**
1	(2.59,3.50)	(4.55,6.06)	(9.20,13.28)	(10.35,17.76)	(13.01,21.49)
2	(3.62,4.93)	(6.94,8.84)	(11.46,14.93)	(13.07,19.01)	(19.04,24.80)
3	(3.82,4.95)	(7.44,9.13)	(8.82,11.49)	(12.88,17.90)	(13.44,27.22)
4	(3.66,5.67)	(6.61,8.60)	(10.69,13.82)	(14.74,19.77)	(20.54,28.80)
5	(4.80,7.20)	(7.78,9.37)	(12.25,15.69)	(14.50,20.56)	(18.23,27.87)

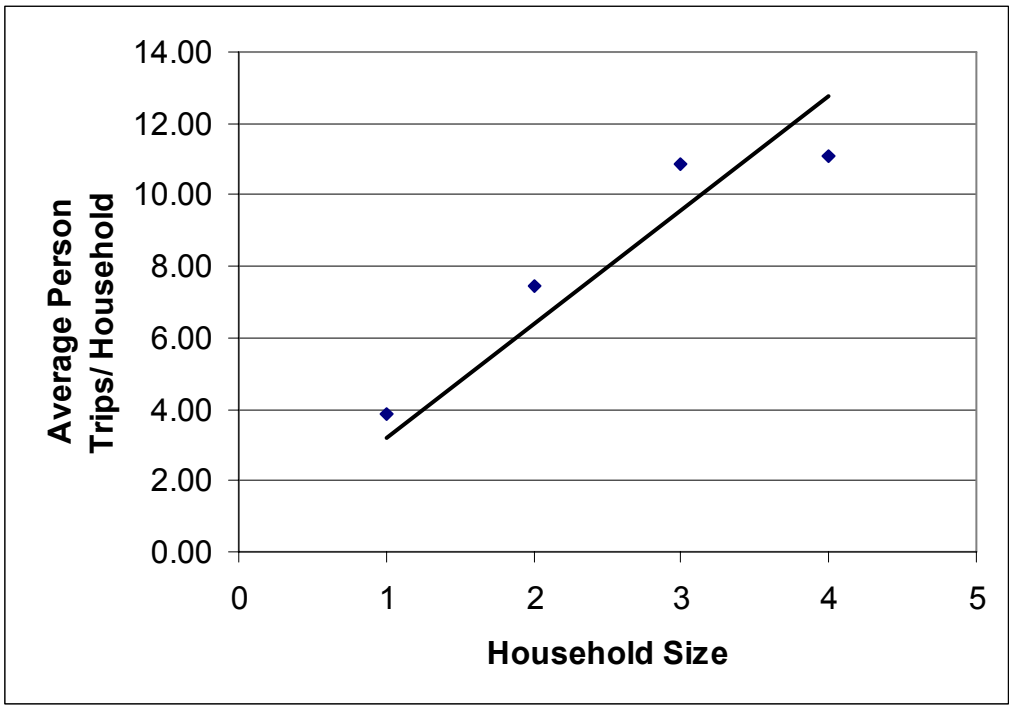

Figure A2. Tyler Linear Regression Forced through Origin for No Children

Table A3. Tyler Linear Regression Equations and Correlation Coefficients

No Children	
Line of Best Fit	
Equation	y = 2.51x + 2.02
r^2 Value	0.91
Forced through Origin	
Equation	y = 3.19x
r^2 Value	0.83

Table A4. Tyler Cross Classification Matrices

Average Person Trips/HH				
Household Size				
	1	**2**	**3**	**4+**
No Children	3.84	7.44	10.84	11.09
Number of Households in Each Substratum				
Household Size				
	1	**2**	**3**	**4+**
No Children	282	482	61	11
90% Confidence Interval				
Household Size				
	1	**2**	**3**	**4+**
No Children	(3.56,4.13)	(7.07,7.81)	(9.67,12.00)	(6.96,15.22)

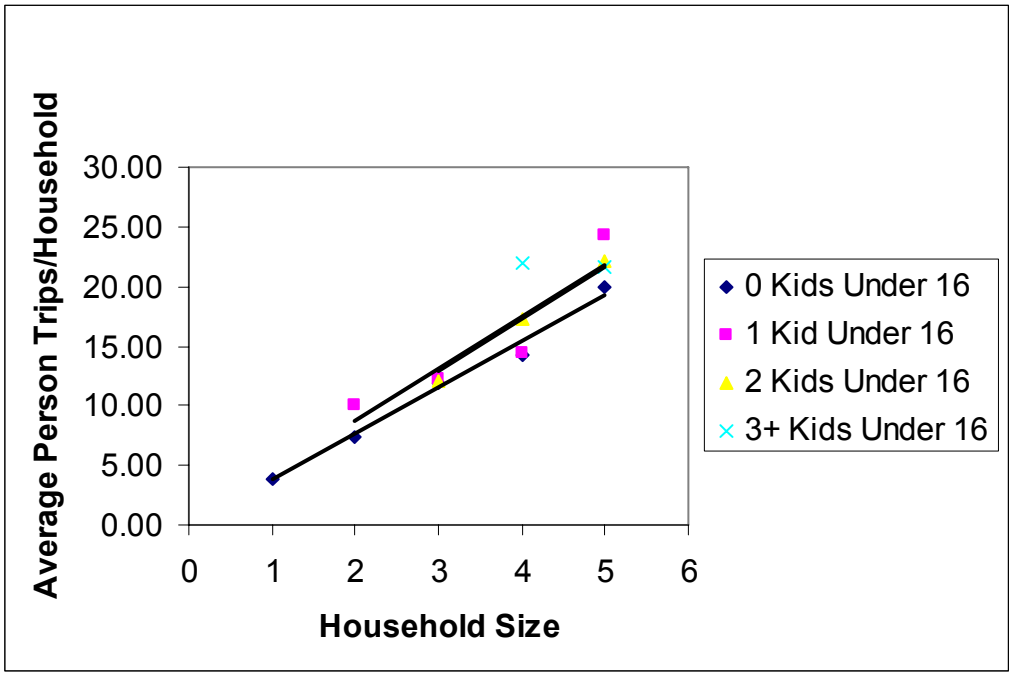

Figure A3. Tyler Linear Regression Forced through Origin for # of Kids Under 16

Table A5. Tyler Linear Regression Equations and Correlation Coefficients

	Number of Kids Under 16			
	0	**1**	**2**	**3+**
Line of Best Fit				
Equation	y = 3.91x - 0.25	y = 4.45x - 0.33	y = 5.10x - 3.25	y = -0.31x + 23.24
r^2 Value	0.99	0.85	1.00	1.00
Forced through Origin				
Equation	y = 3.85x	y = 4.36x	y = 4.32x	NA
r^2 Value	0.99	0.85	0.98	NA

Table A6. Tyler Cross Classification Matrices

Average Person Trips/HH					
	Household Size				
# Kids Under 16	**1**	**2**	**3**	**4**	**5+**
0	3.84	7.44	11.92	14.27	20.00
1	none	10.13	12.19	14.38	24.22
2	NA	None	12.00	17.25	22.20
3+	NA	NA	none	22.00	21.69
Number of Households in Each Substratum					
	Household Size				
# Kids Under 16	**1**	**2**	**3**	**4**	**5+**
0	282	489	79	26	1
1	none	15	88	21	9
2	NA	None	9	89	25
3+	NA	NA	none	2	42
90% Confidence Interval					
	Household Size				
# Kids Under 16	**1**	**2**	**3**	**4**	**5+**
0	(3.56,4.13)	(7.07,7.81)	(10.79,13.05)	(11.42,17.12)	nei
1	none	(7.06,13.20)	(11.14,13.25)	(11.61,17.15)	(18.37,30.07)
2	NA	None	(8.73,15.27)	(15.65,18.85)	(18.95,25.45)
3+	NA	NA	none	(10.49,33.51)	(18.75,24.63)

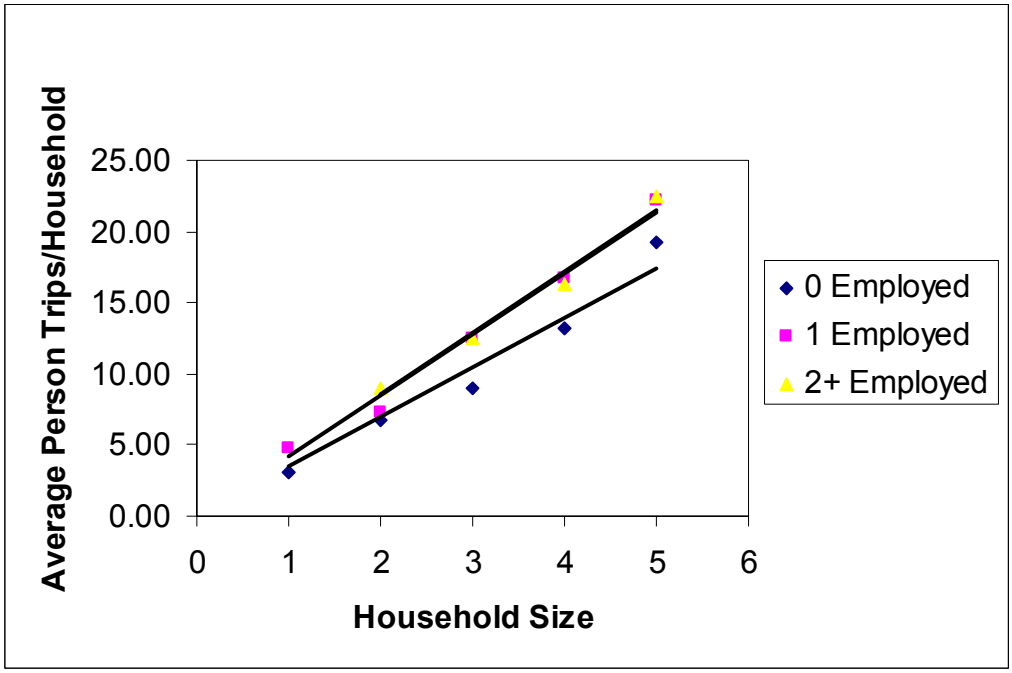

Figure A4. Tyler Linear Regression Forced through Origin for # Employed

Table A7. Tyler Linear Regression Equations and Correlation Coefficients

	Number Employed		
	0	**1**	**2+**
Line of Best Fit			
Equation	y = 3.86x - 1.36	y = 4.41x - 0.57	y = 4.44x - 0.48
r^2 Value	0.97	0.99	0.98
Forced through Origin			
Equation	y = 3.49x	y = 4.26x	y = 4.31x
r^2 Value	0.96	0.99	0.98

Table A8. Tyler Cross Classification Matrices

Average Person Trips/HH					
	Household Size				
# Employed	**1**	**2**	**3**	**4**	**5+**
0	3.13	6.69	8.95	13.17	19.20
1	4.79	7.28	12.46	16.70	22.15
2+	NA	8.99	12.43	16.24	22.50
Number of Households in Each Substratum					
	Household Size				
# Employed	**1**	**2**	**3**	**4**	**5+**
0	160	210	19	6	5
1	122	149	57	50	34
2+	NA	145	100	82	38
90% Confidence Interval					
	Household Size				
# Employed	**1**	**2**	**3**	**4**	**5+**
0	(2.74,3.51)	(6.11,7.26)	(6.87,11.02)	(9.08,17.25)	(12.33,26.07)
1	(4.41,5.16)	(6.59,7.97)	(11.03,13.88)	(14.54,18.86)	(18.86,25.43)
2+	NA	(8.34,9.63)	(11.49,13.37)	(14.61,17.88)	(19.75,25.25)

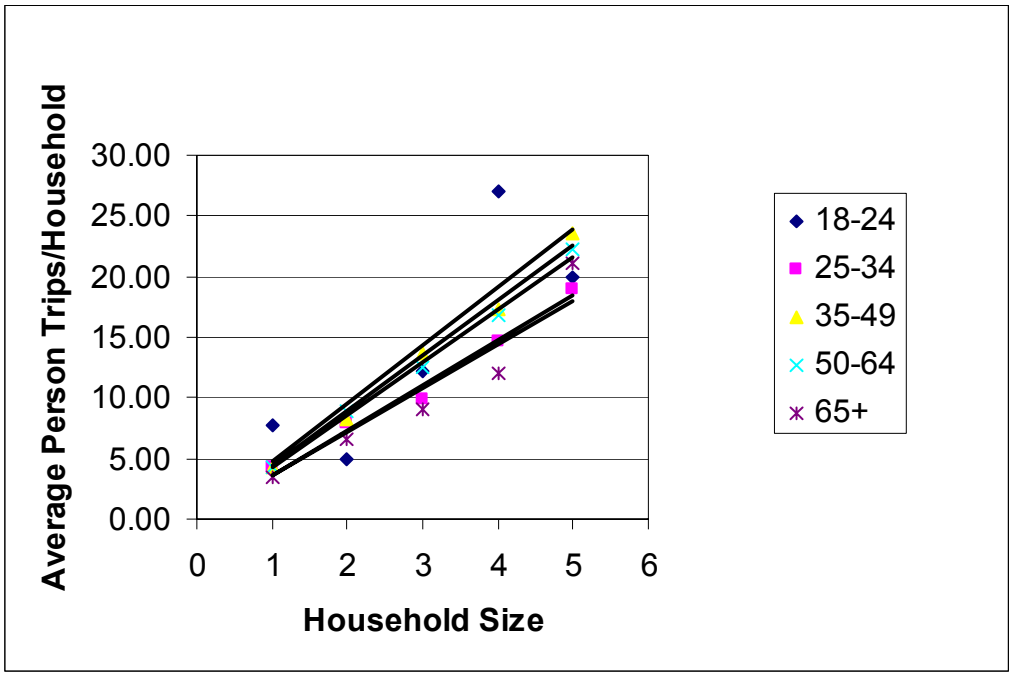

Figure A5. Tyler Linear Regression Forced through Origin for Head's Age

Table A9. Tyler Linear Regression Correlation Coefficients and Equations

	Age of Household's Head				
	18-24	**25-34**	**35-49**	**50-64**	**65+**
Line of Best Fit					
Equation	y = 4.65x + 0.44	y = 3.62x + 0.27	y = 4.78x - 0.96	y = 4.39x - 0.23	y = 4.10x - 1.85
r^2 Value	0.66	0.98	0.99	0.99	0.92
Forced through Origin					
Equation	y = 4.77x	y = 3.70x	y = 4.52x	y = 4.33x	y = 3.59x
r^2 Value	0.66	0.98	0.99	0.99	0.90

Table A10. Tyler Cross Classification Matrices

Average Person Trips/HH					
	Household Size				
Age of Head	**1**	**2**	**3**	**4**	**5+**
18-24	7.75	5.00	12.20	27.00	20.00
25-34	4.25	7.90	9.94	14.64	19.00
35-49	4.24	8.16	13.60	17.33	23.54
50-64	4.28	8.91	12.52	16.74	22.33
65+	3.39	6.52	9.09	12.08	21.10
Number of Households in Each Substratum					
	Household Size				
Age of Head	**1**	**2**	**3**	**4**	**5+**
18-24	4	6	5	2	1
25-34	8	20	16	33	13
35-49	42	50	70	69	35
50-64	72	169	52	23	18
65+	157	258	33	12	10
90% Confidence Interval					
	Household Size				
Head's Age	**1**	**2**	**3**	**4**	**5+**
18-24	(5.93,9.57)	(3.30,6.70)	(6.38,18.02)	(13.84,40.16)	nei
25-34	(2.83,5.67)	(5.92,9.88)	(7.48,12.40)	(12.00,17.27)	(13.17,24.83)
35-49	(3.75,4.73)	(7.02,9.30)	(12.43,14.77)	(15.53,19.14)	(20.48,26.61)
50-64	(3.74,4.82)	(8.25,9.56)	(11.26,13.78)	(14.15,19.33)	(18.36,26.31)
65+	(2.99,3.80)	(6.03,7.02)	(7.48,10.70)	(8.01,16.16)	(16.45,25.75)

Longview

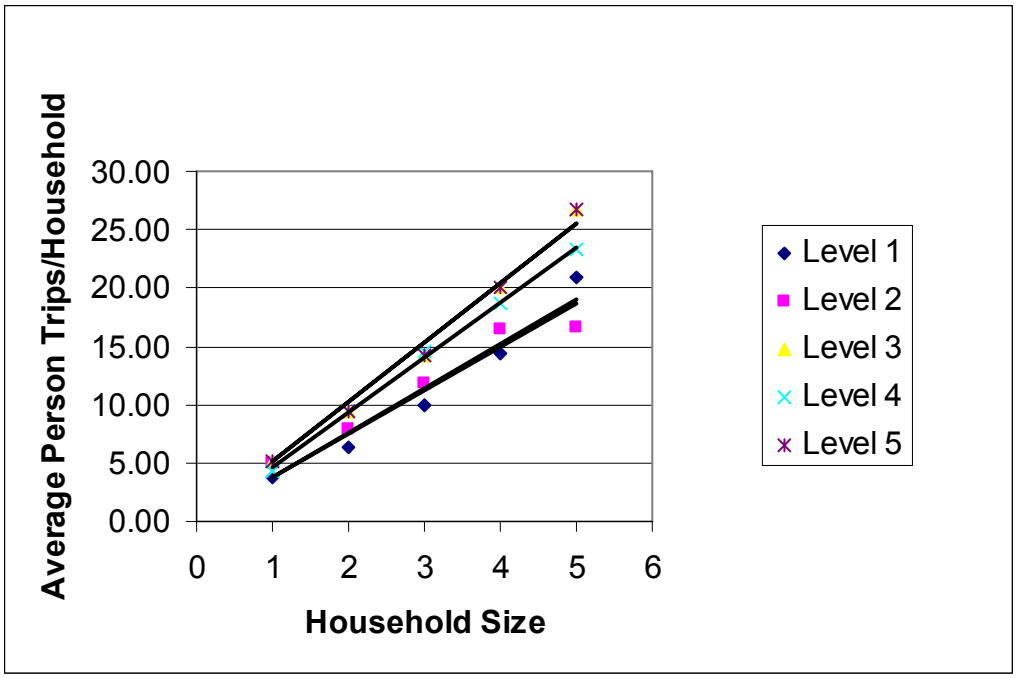

Figure A6. Longview Linear Regression Forced through Origin for Income Level

Table A11. Longview Linear Regression Equations and Correlation Coefficients

	Income Level				
	1	**2**	**3**	**4**	**5**
Line of Best Fit					
Equation	y = 4.26x - 1.72	y = 3.19x + 1.99	y = 5.41x - 1.09	y = 4.74x - 0.15	y = 5.41x - 1.09
r^2 Value	0.97	0.95	0.99	1.00	0.99
Forced Through Origin					
Equation	y = 3.79x	y = 3.73x	y = 5.11x	y = 4.70x	y = 5.11x
r^2 Value	0.96	0.92	0.99	1.00	0.99

Table A12. Longview Cross Classification Matrices

Average Person Trips/Household					
	Household Size				
Income Level	**1**	**2**	**3**	**4**	**5+**
1	3.70	6.35	9.92	14.38	21.00
2	4.97	7.83	11.88	16.48	16.58
3	4.04	7.96	14.31	15.00	23.06
4	4.27	9.50	14.50	18.76	23.37
5	5.08	9.42	14.29	20.00	26.82
Number of Households in Each Substratum					
	Household Size				
Income Level	**1**	**2**	**3**	**4**	**5+**
1	53	84	26	16	11
2	37	52	25	25	19
3	25	54	51	45	31
4	11	52	36	37	19
5	12	43	51	41	17
90% Confidence Interval					
	Household Size				
Income Level	**1**	**2**	**3**	**4**	**5+**
1	(2.92,4.48)	(5.47,7.22)	(7.89,11.95)	(10.48,18.27)	(14.62,27.38)
2	(4.17,5.78)	(6.57,9.09)	(9.60,14.16)	(13.55,19.41)	(13.17,19.99)
3	(3.04,5.04)	(6.74,9.18)	(12.81,15.82)	(13.27,16.73)	20.57,25.56)
4	(3.00,5.55)	(8.05,10.95)	(12.14,16.86)	(16.38,21.13)	(18.45,28.28)
5	(3.84,6.32)	(8.19,10.65)	(12.53,16.06)	(18.18,21.82)	(22.24,31.41)

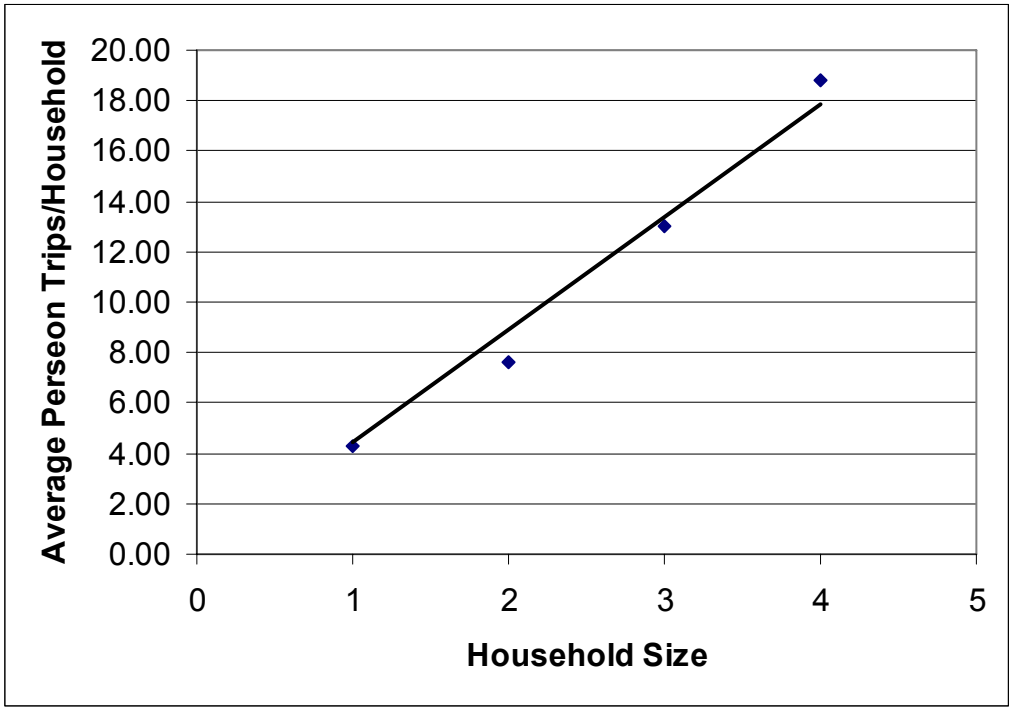

Figure A7. Longview Linear Regression Forced through Origin for No Children

Table A13. Longview Linear Regression Equations and Correlation Coefficients

No Children	
Line of Best Fit	
Equation	y = 4.90x - 1.33
r^2 Value	0.99
Forced Through Origin	
Equation	y = 4.46x
r^2 Value	0.98

Table A14. Longview Cross Classification Matrices

Average Person Trips/Household				
	Household Size			
	1	2	3	4+
No Children	4.27	7.59	13.05	18.80
Number of Households in Each Substratum				
	Household Size			
	1	2	3	4+
No Children	147	304	58	10
90% Confidence Interval				
	Household Size			
	1	2	3	4+
No Children	(3.85,4.70)	(7.09,8.09)	(11.31,14.79)	(14.63,22.97)

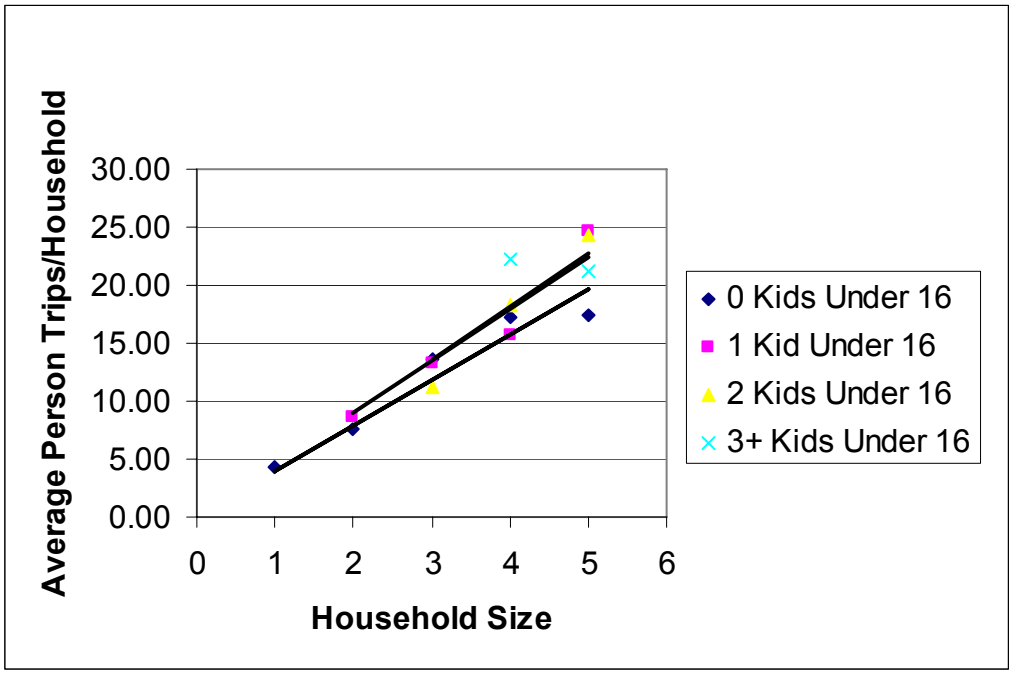

Figure A8. Longview Linear Regression Forced through Origin for # of Kids Under 16

Table A15. Longview Linear Regression Equations and Correlation Coefficients

	Number of Kids Under 16			
	0	**1**	**2**	**3+**
Line of Best Fit				
Equation	y = 3.61x + 1.22	y = 5.02x - 2.03	y = 6.56x - 8.33	y = -0.92x + 25.87
r^2 Value	0.94	0.94	1.00	1.00
Forced through Origin				
Equation	y = 3.94x	y = 4.49x	y = 4.56x	NA
r^2 Value	0.93	0.93	0.90	NA

Table A16. Longview Cross Classification Matrices

Average Person Trips/Household					
	Household Size				
# Kids Under 16	**1**	**2**	**3**	**4**	**5+**
0	4.27	7.63	13.57	17.29	17.50
1	none	8.65	13.25	15.69	24.57
2	NA	none	11.14	18.34	24.27
3+	NA	NA	none	22.20	21.28
Number of Households in Each Substratum					
	Household Size				
# Kids Under 16	**1**	**2**	**3**	**4**	**5+**
0	147	309	84	14	2
1	none	20	107	58	7
2	NA	none	7	91	30
3+	NA	NA	none	5	60
90% Confidence Interval					
	Household Size				
# Kids Under 16	**1**	**2**	**3**	**4**	**5+**
0	(3.85,4.70)	(7.13,8.13)	(12.16,14.98)	(13.21,21.36)	(5.16,29.84)
1	none	(6.36,10.94)	(12.13,14.37)	(14.09,17.29)	(19.68,29.46)
2	NA	none	(8.26,14.02)	(16.89,19.79)	(20.68,27.85)
3+	NA	NA	none	(15.49,27.91)	(19.03,23.54)

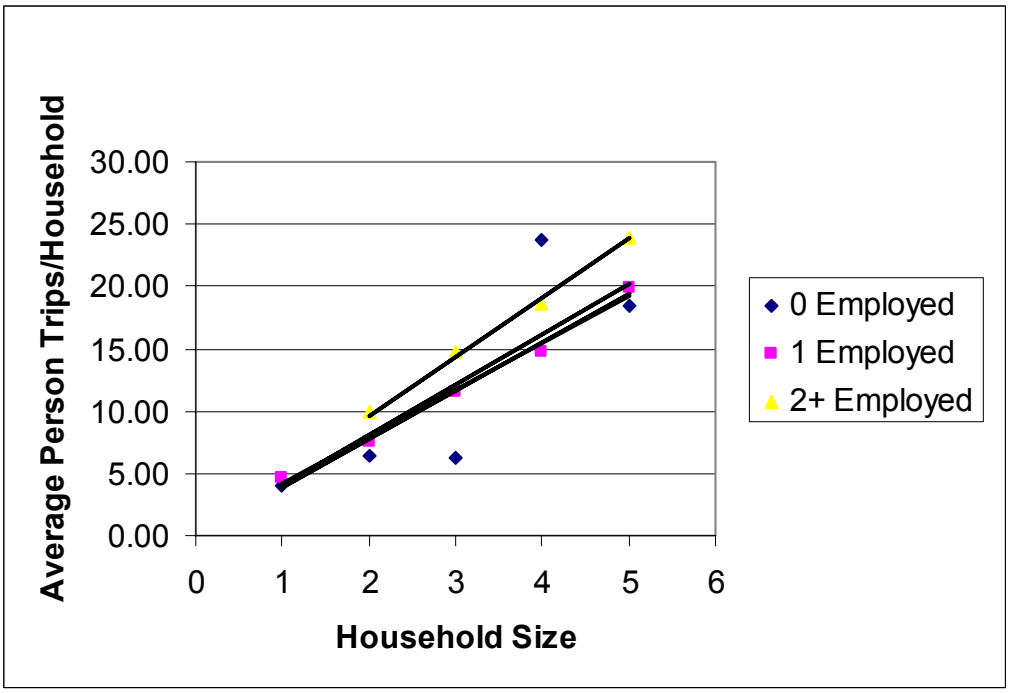

Figure A9. Longview Linear Regression Forced through Origin for # Employed

Table A17. Longview Linear Regression Equations and Correlation Coefficients

	# Employed		
	0	**1**	**2+**
Line of Best Fit			
Equation	y = 4.64x - 2.14	y = 3.79x + 0.33	y = 4.58x + 0.76
r^2 Value	0.70	0.99	1.00
Forced through Origin			
Equation	y = 4.06x	y = 3.88x	y = 4.78x
r^2 Value	0.68	0.99	0.99

Table A18. Longview Cross Classification Matrices

Average Person Trips/Household					
	Household Size				
# Employed	**1**	**2**	**3**	**4**	**5+**
0	3.94	6.46	6.25	23.75	18.50
1	4.66	7.53	11.54	14.83	19.97
2+	NA	9.93	14.75	18.62	23.92
Number of Households in Each Substratum					
	Household Size				
# Employed	**1**	**2**	**3**	**4**	**5+**
0	88	140	12	4	4
1	62	104	57	60	34
2+	NA	85	129	105	61
90% Confidence Interval					
	Household Size				
# Employed	**1**	**2**	**3**	**4**	**5+**
0	(3.36,4.53)	(5.70,7.21)	(3.38,9.12)	(20.72,26.78)	(8.13,28.87)
1	(4.09,5.23)	(6.75,8.31)	(10.26,12.83)	(13.03,16.64)	(16.79,23.15)
2+	NA	(8.98,10.87)	(13.67,15.83)	(17.38,19.86)	(21.74,26.10)

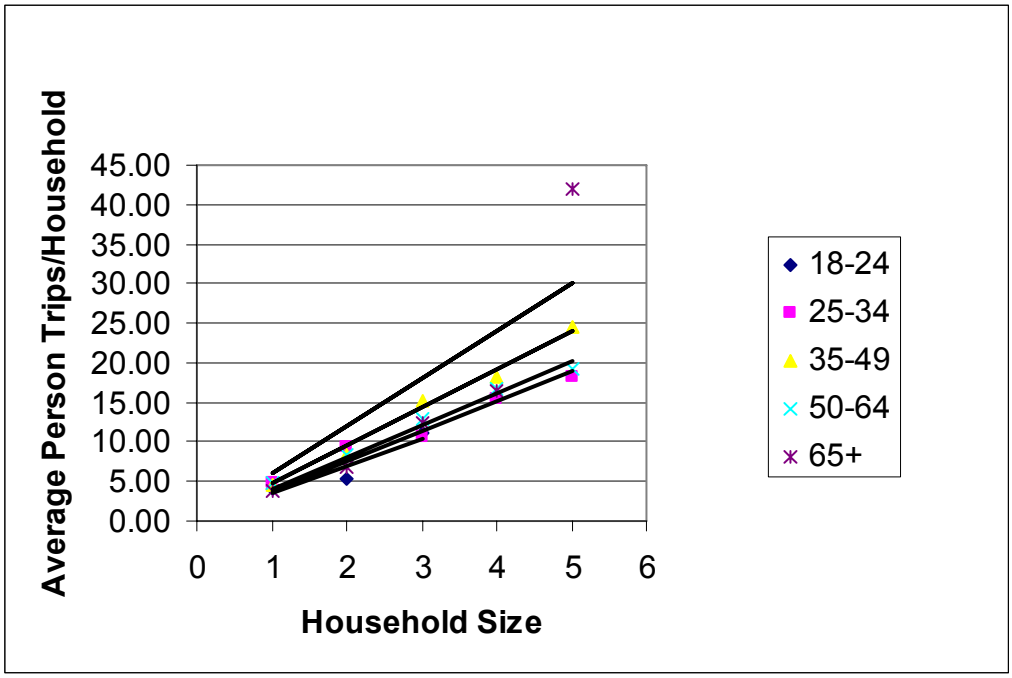

Figure A10. Longview Linear Regression Forced through Origin for Head's Age

Table A19. Longview Linear Regression Equations and Correlation Coefficients

	Age of Household's Head				
	18-24	**25-34**	**35-49**	**50-64**	**65+**
Line of Best Fit					
Equation	y = 3.63x - 0.39	y = 3.26x + 1.89	y = 5.00x - 0.79	y = 3.74x + 1.18	y = 8.61x - 9.53
r^2 Value	0.88	0.98	0.99	0.99	0.80
Forced through Origin					
Equation	y = 3.46x	y = 3.78x	y = 4.78x	y = 4.06x	y = 6.01x
r^2 Value	0.88	0.95	0.99	0.98	0.71

Table A20. Longview Cross Classification Matrices

Average Person Trips/Household					
	Household Size				
Age of Head	**1**	**2**	**3**	**4**	**5+**
18-24	4.00	5.33	11.25	none	none
25-34	4.83	9.35	10.69	15.41	18.12
35-49	4.62	8.29	15.22	18.27	24.62
50-64	4.75	8.34	12.92	16.65	19.27
65+	3.75	6.89	12.34	16.44	42.00
Number of Households in Each Substratum					
	Household Size				
Age of Head	**1**	**2**	**3**	**4**	**5+**
18-24	4	3	4	none	none
25-34	6	20	35	27	25
35-49	21	34	78	105	58
50-64	48	119	52	26	15
65+	71	153	29	9	1
90% Confidence Interval					
	Household Size				
Head's Age	**1**	**2**	**3**	**4**	**5+**
18-24	(1.77,6.23)	(2.43,8.23)	(8.29,14.21)	none	none
25-34	(3.84,5.82)	(7.17,11.53)	(8.95,12.42)	(13.26,17.55)	(14.92,21.32)
35-49	(3.69,5.55)	(6.93,9.65)	(13.77,16.67)	(16.97,19.56)	(22.22,27.02)
50-64	(3.93,5.57)	(7.51,9.16)	(11.63,14.21)	(13.77,19.54)	(15.76,22.77)
65+	(3.14,4.36)	(6.18,7.59)	(9.57,15.12)	(10.31,22.58)	nei

Austin

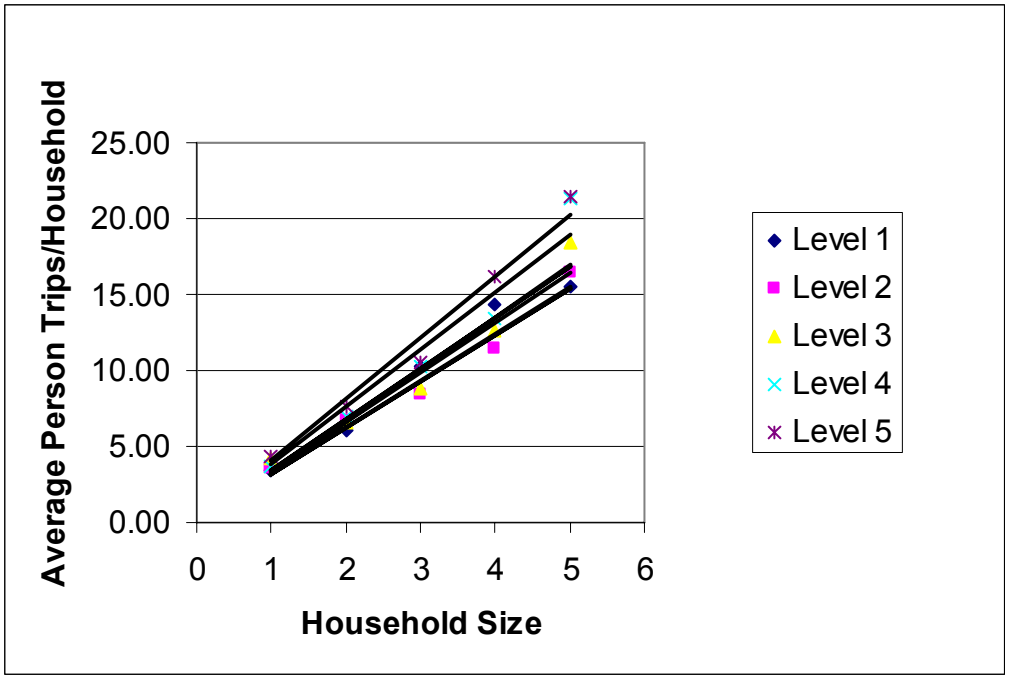

Figure A11. Austin Linear Regression Forced through Origin for Income Level

Table A21. Austin Linear Regression Equations and Correlation Coefficients

	Income Level				
	1	**2**	**3**	**4**	**5**
Line of Best Fit					
Equation	y = 3.26x + 0.12	y = 3.05x + 0.17	y = 3.44x - 0.24	y = 4.16x - 1.35	y = 4.28x - 0.81
r^2 Value	0.98	0.97	0.96	0.95	0.98
Forced Through Origin					
Equation	y = 3.29x	y = 3.10x	y = 3.38x	y = 3.79x	y = 4.06x
r^2 Value	0.98	0.97	0.96	0.95	0.98

Table A22. Austin Cross Classification Matrices

Average Person Trips/Household					
	Household Size				
Income Level	**1**	**2**	**3**	**4**	**5+**
1	3.36	6.03	10.26	14.29	15.53
2	3.56	6.75	8.39	11.43	16.47
3	4.16	6.57	8.81	12.58	18.37
4	3.73	6.91	10.28	13.44	21.26
5	4.37	7.63	10.51	16.19	21.51
Number of Households in Each Substratum					
	Household Size				
Income Level	**1**	**2**	**3**	**4**	**5+**
1	111	91	31	14	34
2	64	115	56	42	34
3	58	107	52	55	35
4	26	99	46	41	39
5	19	104	89	86	49
90% Confidence Interval					
	Household Size				
Income Level	**1**	**2**	**3**	**4**	**5+**
1	(2.96,3.77)	(5.39,6.68)	(8.30,12.21)	(10.55,18.03)	(13.46,17.60)
2	(3.09,4.04)	(6.08,7.42)	(7.37,9.42)	(9.75,13.10)	(13.78,19.16)
3	(3.71,4.60)	(5.95,7.19)	(7.71,9.90)	(11.33,13.83)	(15.72,21.03)
4	(3.00,4.46)	(6.20,7.62)	(9.03,11.54)	(12.04,14.84)	(18.71,23.80)
5	(3.55,5.19)	(7.04,8.23)	(9.69,11.32)	(14.83,17.54)	(18.65,24.37)

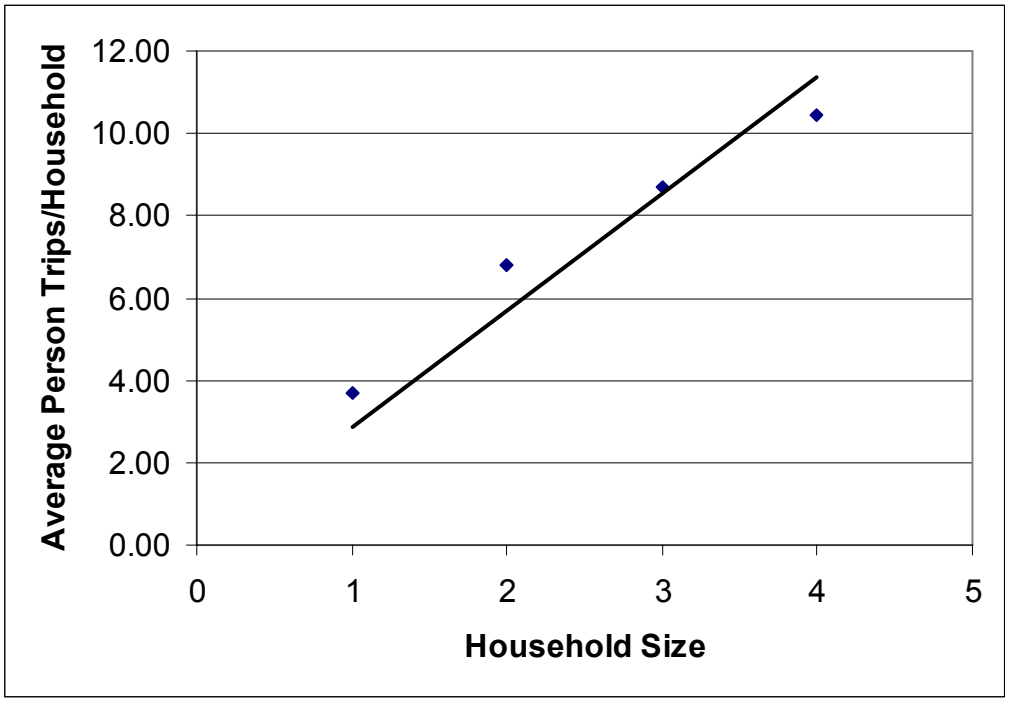

Figure A12. Austin Linear Regression Forced through Origin for No Children

Table A23. Austin Linear Regression Equations Correlation Coefficients

No Children	
Line of Best Fit	
Equation	y = 2.22x + 1.86
r^2 Value	0.98
Forced Through Origin	
Equation	y = 2.84x
r^2 Value	0.89

Table A24. Austin Cross Classification Matrices

Average Person Trips/ Household				
Household Size				
	1	2	3	4+
No Children	3.68	6.82	8.71	10.46
Number of Households in Each Substratum				
Household Size				
	1	2	3	4+
No Children	277	458	109	28
90% Confidence Interval				
Household Size				
	1	2	3	4+
No Children	(3.45,3.92)	(6.52,7.12)	(7.92,9.50)	(8.97,11.96)

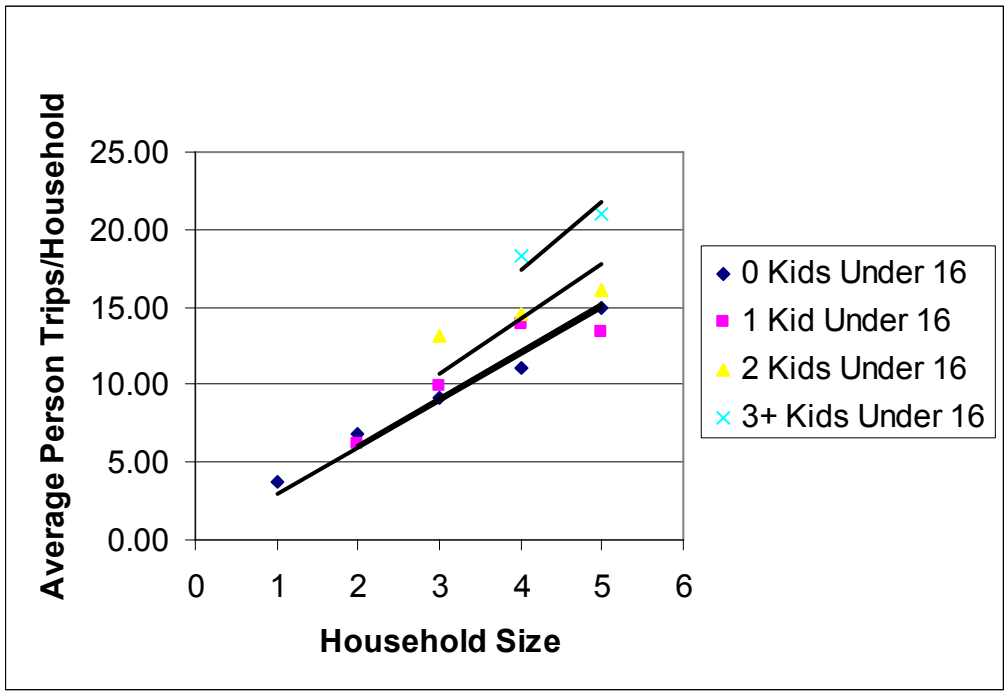

Figure A13. Austin Linear Regression Forced through Origin for # of Kids Under 16

Table A25. Austin Linear Regression Equations and Correlation Coefficients

	Number of Kids Under 16			
	0	**1**	**2**	**3+**
Line-of-Best-Fit				
Equation	y = 2.69x + 1.08	y = 2.59x + 1.78	y = 1.50x + 8.59	y = 2.69x + 7.57
r^2 Value	0.99	0.86	1.00	1.00
Forced through Origin				
Equation	y = 2.98x	y = 3.05x	y = 3.57x	y = 4.35x
r^2 Value	0.97	0.83	-0.95	0.61

Table A26. Austin Cross Classification Matrices

Average Person Trips/ Household					
	Household Size				
# Kids Under 16	**1**	**2**	**3**	**4**	**5+**
0	3.68	6.81	9.17	11.08	15.00
1	none	6.13	9.92	13.86	13.43
2	NA	none	13.09	14.62	16.10
3+	NA	NA	none	18.33	21.02
Number of Household in Each Substratum					
	Household Size				
# Kids Under 16	**1**	**2**	**3**	**4**	**5+**
0	277	461	132	39	5
1	none	8	131	50	23
2	NA	none	11	146	40
3+	NA	NA	none	3	123
90% Confidence Interval					
	Household Size				
# Kids Under 16	**1**	**2**	**3**	**4**	**5+**
0	(3.45,3.92)	(6.52,7.11)	(8.47,9.86)	(9.81,12.34)	(10.53,19.47)
1	none	(3.81,8.44)	(9.20,10.64)	(12.14,15.58)	(11.38,15.49)
2	NA	none	(9.47,16.71)	(13.65,15.59)	(13.42,18.48)
3+	NA	NA	none	(14.05,22.62)	(19.49,22.56)

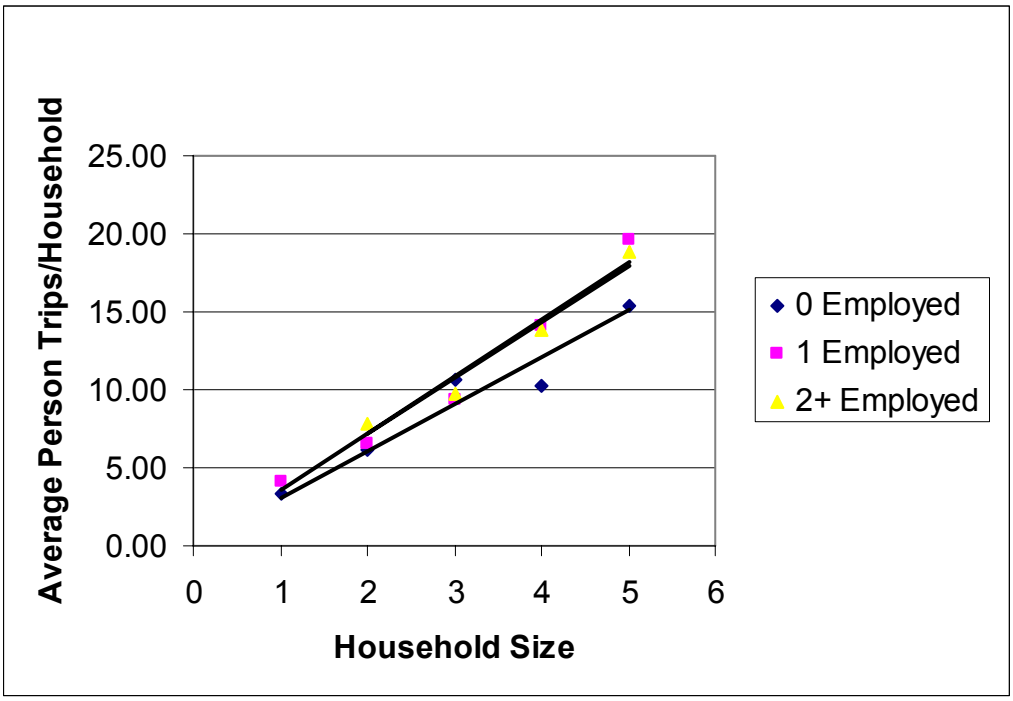

Figure A14. Austin Linear Regression Forced through Origin for # Employed

Table A27. Austin Linear Regression Equations and Correlation Coefficients

	Number Employed		
	0	1	2+
Line of Best Fit			
Equation	y = 2.83x + 0.69	y = 3.85x - 0.80	y = 3.73x - 0.49
r^2 Value	0.93	0.97	0.97
Forced Through Origin			
Equation	y = 3.02x	y = 3.63x	y = 3.60x
r^2 Value	0.93	0.97	0.97

Table A28. Austin Cross Classification Matrices

Average Person Trips/ Household					
	Household Size				
# Employed	**1**	**2**	**3**	**4**	**5+**
0	3.33	6.21	10.65	10.25	15.44
1	4.13	6.50	9.42	14.16	19.56
2+	NA	7.79	9.72	13.89	18.82
Number of Households in Each Substratum					
	Household Size				
# Employed	**1**	**2**	**3**	**4**	**5+**
0	158	203	23	4	9
1	120	149	102	90	68
2+	NA	164	149	144	114
90% Confidence Interval					
	Household Size				
# Employed	**1**	**2**	**3**	**4**	**5+**
0	(3.00,3.65)	(5.76,6.65)	(8.54,12.77)	(5.30,15.20)	(10.37,20.52)
1	(3.81,4.45)	(5.97,7.03)	(8.58,10.26)	(12.72,15.59)	(17.19,21.93)
2+	NA	(7.24,8.33)	(9.06,10.37)	(13.06,14.71)	(17.41,20.22)

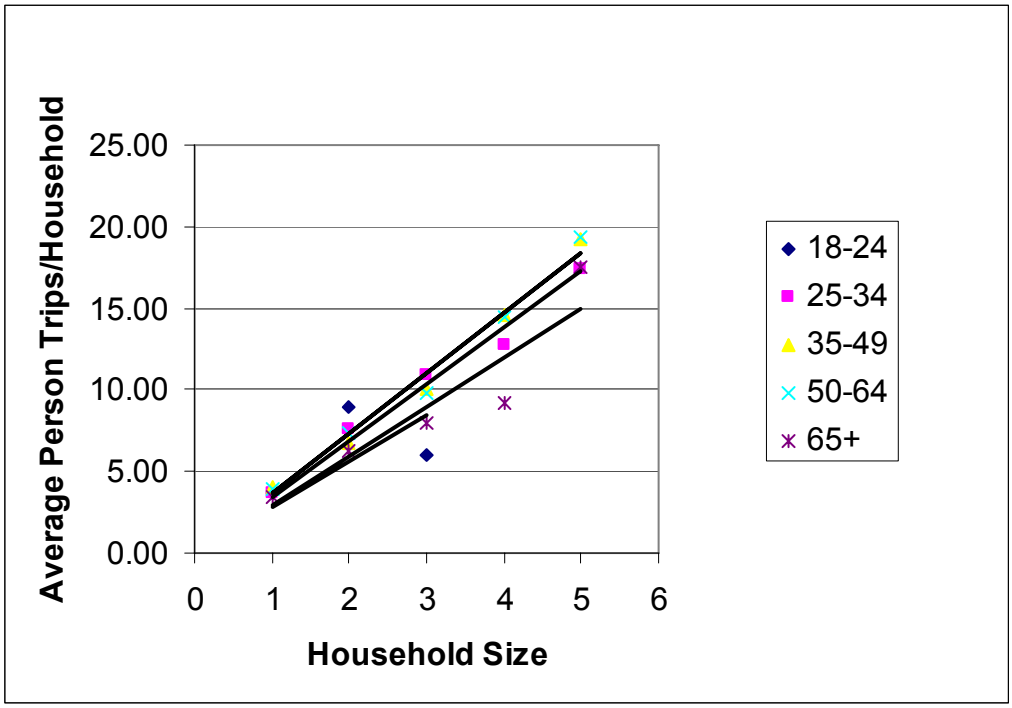

Figure A15. Austin Linear Regression Forced through Origin for Head's Age

Table A29. Austin Linear Regression Equations and Correlation Coefficients

	Age of Household's Head				
	18-24	**25-34**	**35-49**	**50-64**	**65+**
Line of Best Fit					
Equation	y = 1.17x + 3.89	y = 3.26x + 0.71	y = 3.82x - 0.54	y = 3.80x - 0.41	y = 3.12x - 0.52
r^2 Value	0.19	0.99	0.99	0.98	0.87
Forced through Origin					
Equation	y = 2.83x	y = 3.45x	y = 3.67x	y = 3.69x	y = 2.98x
r^2 Value	-0.26	0.98	0.98	0.98	0.86

Table A30. Austin Cross Classification Matrices

Average Person Trips/ Household					
	Household Size				
Age of Head	**1**	**2**	**3**	**4**	**5+**
18-24	3.67	9.00	6.00	none	none
25-34	3.69	7.62	10.92	12.79	17.38
35-49	4.06	6.69	10.01	14.57	19.20
50-64	3.89	7.41	9.82	14.44	19.36
65+	3.37	6.21	7.98	9.19	17.50
Number of Household in Each Substratum					
	Household Size				
Age of Head	**1**	**2**	**3**	**4**	**5+**
18-24	3	4	3	none	none
25-34	16	34	36	33	21
35-49	50	59	106	132	114
50-64	85	180	83	57	39
65+	124	239	46	16	16
90% Confidence Interval					
	Household Size				
Age of Head	**1**	**2**	**3**	**4**	**5+**
18-24	(2.22,5.12)	(5.78,12.22)	nei	nei	nei
25-34	(2.94,4.43)	(6.36,8.87)	(9.40,12.43)	(11.10,14.47)	(14.80,19.97)
35-49	(3.52,4.60)	(5.89,7.50)	(9.19,10.83)	(13.55,15.59)	(17.80,20.61)
50-64	(3.44,4.34)	(6.89,7.92)	(8.88,10.76)	(12.88,16.00)	(16.18,22.53)
65+	(3.02,3.72)	(5.80,6.61)	(6.97,8.99)	(7.36,11.02)	(11.10,23.90)

San Antonio

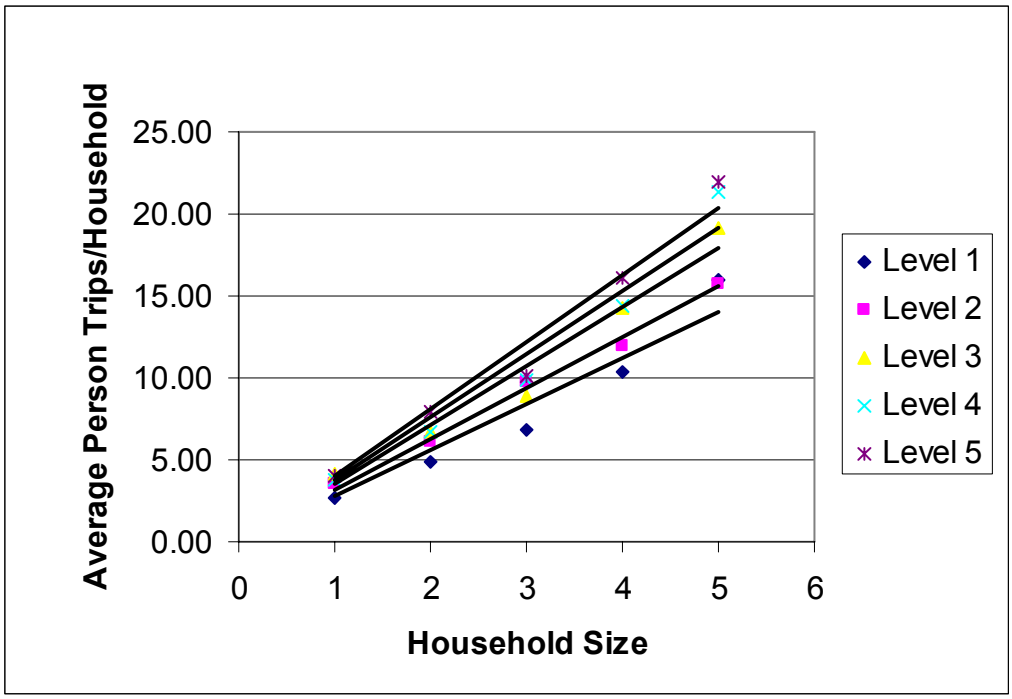

Figure A16. San Antonio Linear Regression Forced through Origin for Income Level

Table A31. San Antonio Linear Regression Equations and Correlation Coefficients

	Income Level				
	1	**2**	**3**	**4**	**5**
Line of Best Fit					
Equation	y = 3.22x - 1.52	y = 3.02x + 0.36	y = 3.75x - 0.59	y = 4.27x - 1.62	y = 4.41x - 1.22
r^2 Value	0.95	0.99	0.97	0.96	0.97
Forced Through Origin					
Equation	y = 2.81x	y = 3.12x	y = 3.59x	y = 3.83x	y = 4.08x
r^2 Value	0.93	0.99	0.96	0.95	0.96

Table A32. San Antonio Cross Classification Matrices

Average Person Trips/Household					
	Household Size				
Income Level	**1**	**2**	**3**	**4**	**5+**
1	2.65	4.86	6.86	10.38	16.00
2	3.56	6.15	9.73	11.90	15.78
3	4.16	6.76	8.95	14.32	19.14
4	3.80	6.66	9.88	14.34	21.33
5	3.98	7.92	10.13	16.06	21.96
Number of Households in Each Substratum					
	Household Size				
Income Level	**1**	**2**	**3**	**4**	**5+**
1	112	114	42	26	24
2	79	153	70	51	46
3	73	125	74	66	36
4	45	157	95	83	42
5	41	169	116	89	72
90% Confidence Interval					
	Household Size				
Income Level	**1**	**2**	**3**	**4**	**5+**
1	(2.28,3.02)	(4.30,5.42)	(5.65,8.06)	(8.53,12.24)	(13.41,18.59)
2	(3.11,4.00)	(5.60,6.70)	(8.56,10.89)	(10.33,13.47)	(13.89,17.68)
3	(3.42,4.91)	(6.15,7.37)	(7.99,9.90)	(12.94,15.69)	(16.41,21.87)
4	(3.39,4.21)	(6.11,7.20)	(8.97,10.80)	(7.26,21.42)	(18.82,23.85)
5	(3.30,4.65)	(7.34,8.49)	(9.24,11.02)	(14.73,17.38)	(19.34,24.58)

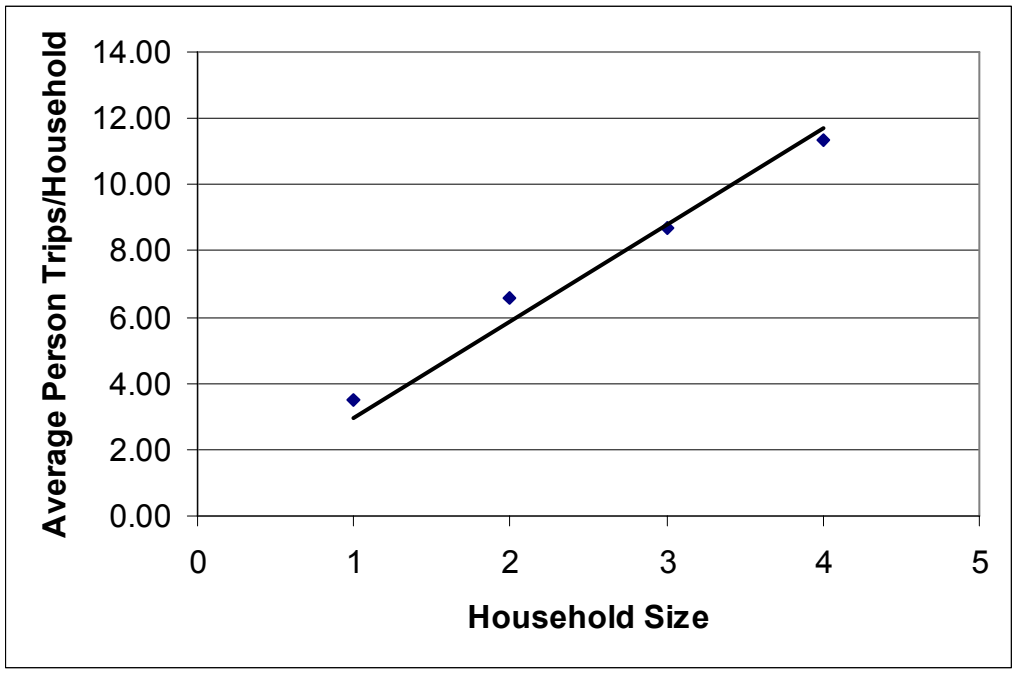

Figure A17. San Antonio Linear Regression Forced through Origin for No Children

Table A33. San Antonio Linear Regression Equations and Correlation Coefficients

No Children	
Line of Best Fit	
Equation	y = 2.57x + 1.09
r^2 Value	1.00
Forced through Origin	
Equation	y = 2.93x
r^2 Value	0.97

Table A34. San Antonio Cross Classification Matrices

Average Person Trips/Household				
	Household Size			
	1	**2**	**3**	**4+**
No Children	3.47	6.55	8.71	11.32
Number of Households in Each Substratum				
	Household Size			
	1	**2**	**3**	**4+**
No Children	350	704	221	57
90% Confidence Interval				
	Household Size			
	1	**2**	**3**	**4+**
No Children	(3.23,3.72)	(6.29,6.82)	(8.05,9.36)	(9.89.12.74)

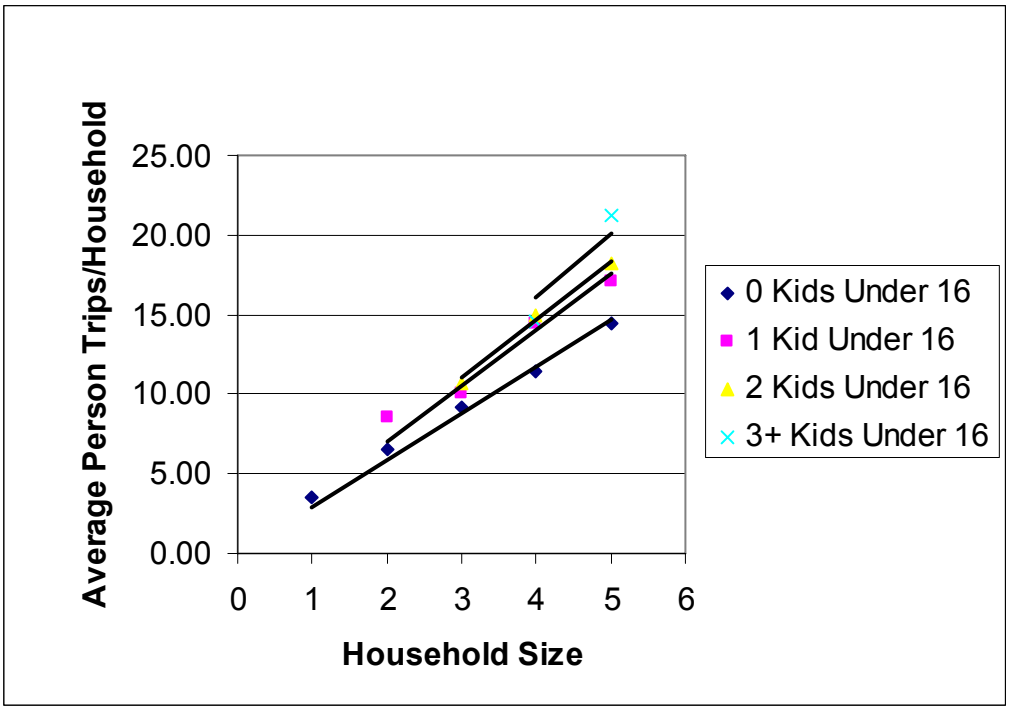

Figure A18. San Antonio Linear Regression Forced through Origin for # of Kids Under 16

Table A35. San Antonio Linear Regression Equations and Correlation Coefficients

	Number of Kids Under 16			
	0	**1**	**2**	**3+**
Line of Best Fit				
Equation	y = 2.68x + 0.96	y = 3.03x + 1.94	y = 3.82x - 0.64	y = 6.62x - 11.88
r^2 Value	1.00	0.97	0.99	1.00
Forced through Origin				
Equation	y = 2.95x	y = 3.53x	y = 3.67x	y = 4.01x
r^2 Value	0.99	0.94	0.99	0.84

Table A36. San Antonio Cross Classification Matrices

Average Person Trips/Household					
	Household Size				
# Kids Under 16	**1**	**2**	**3**	**4**	**5+**
0	3.47	6.56	9.11	11.49	14.43
1	none	8.50	10.06	14.45	17.12
2	NA	none	10.64	15.00	18.27
3+	NA	NA	none	14.60	21.22
Number of Households in Each Substratum					
	Household Size				
# Kids Under 16	**1**	**2**	**3**	**4**	**5+**
0	350	708	252	69	14
1	none	10	135	73	33
2	NA	none	11	168	55
3+	NA	NA	none	5	118
90% Confidence Interval					
	Household Size				
# Kids Under 16	**1**	**2**	**3**	**4**	**5+**
0	(3.23,3.72)	(6.30,6.82)	(8.51,9.72)	(10.22,12.77)	(10.68,18.17)
1	none	(6.68,10.32)	(9.35,10.77)	(13.11,15.79)	(14.32,19.92)
2	NA	none	(8.12,13.15)	(14.05,15.95)	(16.30,20.25)
3+	NA	NA	none	(10.65,18.55)	(19.40,23.04)

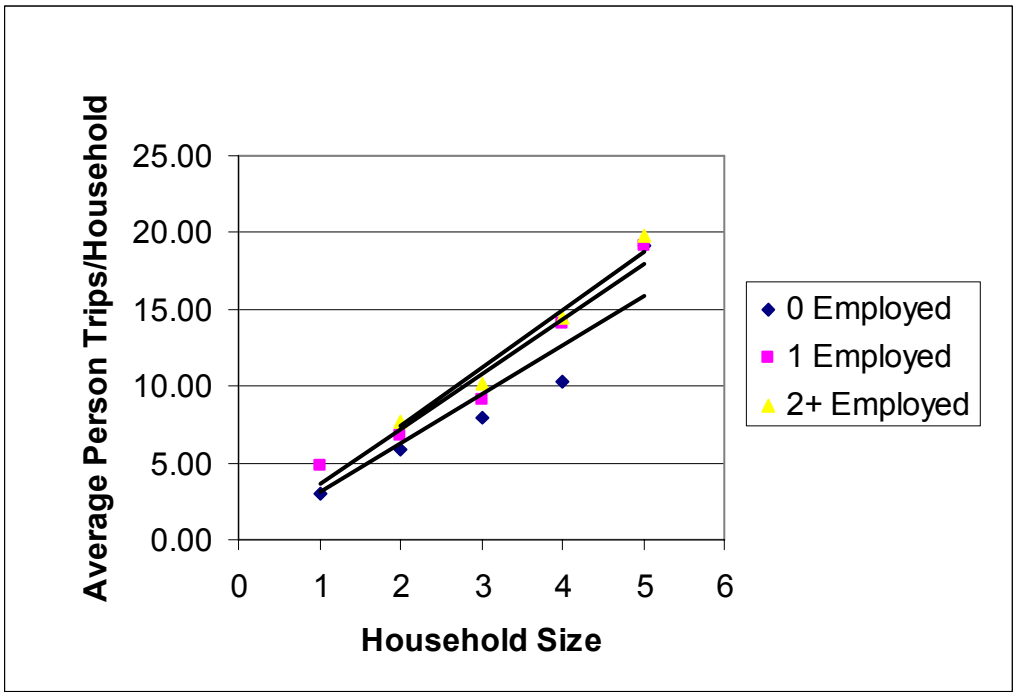

Figure A19. San Antonio Linear Regression Forced through Origin for # Employed

Table A37. San Antonio Linear Regression Equations and Correlation Coefficients

	Number Employed		
	0	**1**	**2+**
Line of Best Fit			
Equation	y = 3.66x - 1.74	y = 3.57x + 0.05	y = 4.07x - 1.25
r^2 Value	0.89	0.96	0.98
Forced through Origin			
Equation	y = 3.19x	y = 3.59x	y = 3.74x
r^2 Value	0.87	0.96	0.97

Table A38. San Antonio Cross Classification Matrices

Average Person Trips/Household					
	Household Size				
# Employed	**1**	**2**	**3**	**4**	**5+**
0	3.02	5.89	7.93	10.31	19.13
1	4.82	6.82	9.11	14.05	19.08
2+	NA	7.63	10.12	14.42	19.75
Number of Households in Each Substratum					
	Household Size				
# Employed	**1**	**2**	**3**	**4**	**5+**
0	261	320	59	13	16
1	89	238	141	130	88
2+	NA	160	197	172	116
90% Confidence Interval					
	Household Size				
# Employed	**1**	**2**	**3**	**4**	**5+**
0	(2.80,3.23)	(5.52,6.26)	(6.69,9.17)	(5.83,14.79)	(16.03,22.22)
1	(4.16,5.48)	(6.34,7.30)	(8.46,9.76)	(13.03,15.06)	(17.13,21.03)
2+	NA	(7.08,8.18)	(9.42,10.81)	(13.54,15.31)	(18.04,21.46)

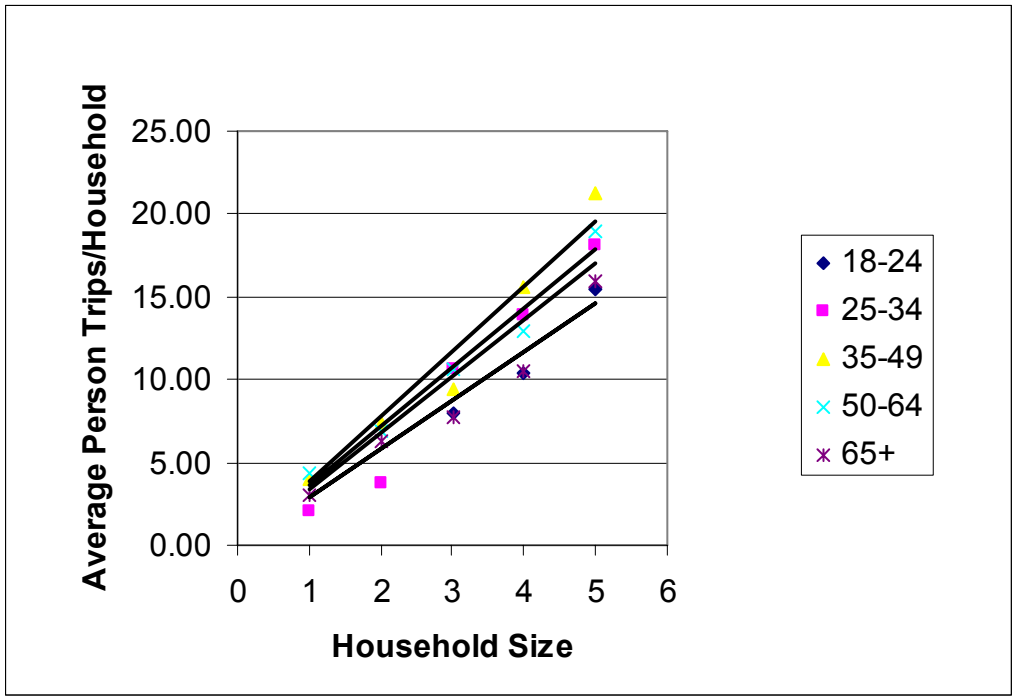

Figure A20. San Antonio Linear Regression Forced through Origin for Head's Age

Table A39. San Antonio Linear Regression Equations and Correlation Coefficients

	Age of Household's Head				
	18-24	**25-34**	**35-49**	**50-64**	**65+**
Line of Best Fit					
Equation	y = 3.01x - 0.34	y = 4.24x - 3.04	y = 4.27x - 1.27	y = 3.50x + 0.27	y = 3.01x - 0.34
r^2 Value	0.95	0.97	0.96	0.97	0.95
Forced Through Origin					
Equation	y = 2.91x	y = 3.41x	y = 3.92x	y = 3.57x	y = 2.91x
r^2 Value	0.95	0.93	0.96	0.97	0.95

Table A40. San Antonio Cross Classification Matrices

Average Person Trips/Household					
	Household Size				
Age of Head	**1**	**2**	**3**	**4**	**5+**
18-24	none	none	8.00	10.33	15.50
25-34	2.00	3.71	10.68	13.93	18.10
35-49	4.04	7.36	9.44	15.61	21.26
50-64	4.40	6.94	10.69	12.88	18.92
65+	2.98	6.32	7.72	10.45	15.94
Number of Households in Each Substratum					
	Household Size				
Age of Head	**1**	**2**	**3**	**4**	**5+**
18-24	none	none	3	3	2
25-34	1	7	28	44	29
35-49	27	50	100	154	105
50-64	103	258	143	83	50
65+	219	403	123	31	34
90% Confidence Interval					
	Household Size				
Age of Head	**1**	**2**	**3**	**4**	**5+**
18-24	none	none	(4.20,11.80)	(6.05,14.62)	(14.68,16.32)
25-34	nei	(2.93,4.49)	(8.96,12.40)	(12.04,15.82)	(15.16,21.05)
35-49	(3.13,4.94)	(6.56,8.16)	(8.68,10.20)	(14.66,16.56)	(19.31,23.21)
50-64	(3.82,4.98)	(6.47,7.40)	(9.81,11.56)	(11.67,14.09)	(16.58,21.26)
65+	(2.74,3.22)	(5.97,6.66)	(7.01,8.43)	(8.49,12.41)	(13.57,18.31)

Examining Cross Median Crashes on Horizontal Curves

Prepared for
Undergraduate Transportation Scholars Program

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Research Funded by:
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Incorporating Safety into the Highway Design Process
0-4703

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Southwest Region University Transportation Center

August 8, 2008

MATTHEW MULKERN

Matthew Mulkern is a fourth-year Civil Engineering Major from the University of Maine in Orono, Maine. Originally from Houston Texas, he will complete his Bachelor of Science degree in May 2009. His forthcoming goal is to attend graduate school for a Masters in Transportation Engineering. Mr. Mulkern has previously attended Bishop's University in Sherbrooke, Québec, in the spring of 2008. Last summer he attended the Research Experience for Undergraduates-National Science Foundation (REU-NSF) Summer 2007 Civil Engineering Program at the University of Houston. He graduated cum laude from Cypress Ridge High School, a school in the Houston suburbs, in 2005. His career interests include Access Management, Roadway Design and Public Transportation Systems.

Mr. Mulkern is a member of the American Society of Civil Engineers (ASCE), the Institute of Transportation Engineers (ITE), Pi Mu Epsilon honour society, and Chi Epsilon honour society. He is also a University of Maine Presidential Scholar and a Member of the National Honour Society. He is Vice-President of the Canadian Club at the University of Maine. He previously has worked as a Disc Jockey for radio stations WMEB 91.9FM, Orono, Maine, and CJMQ 88.9FM, Sherbrooke, Québec, and for University of Maine home athletic events. In addition, he has worked in the field of Sports Management for University of Maine sports marketing auxiliary services.

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The author would like to thank Dr. Kay Fitzpatrick, Byung-Jung Park and especially Dr. Dominique Lord for their help with this paper and research.

SUMMARY

The purpose of this project was to analyze crash characteristics involving medians on horizontal curve and tangent freeway segments, and establish relationships between crashes and geometric design characteristics. A secondary objective was to develop Accident Modification Factors (AMF) for medians (if time permitted).

In this project, the author analyzed left (inside) shoulder width and median width and their impact on the number and severity of cross-median collisions on curved and tangent freeway sections. In addition, the researchers collected data for horizontal curves and an adjacent tangent freeway sections, confirmed the TRM data with Google Earth, analyzed data using descriptive statistics, ANOVA and regression analysis by means of a negative binomial model (if time permitted).

This study is a part of a larger project associated with project TxDOT 0-4703 by TXDOT [Incorporating Safety into the Highway Design Process] lead by Dr. James Bonneson (TTI) and sponsored by the Texas Department of Transportation (TxDOT). Some of the findings of this report will complement the larger project, including a few of the recommendations and future areas of study.

Ultimately, not all of the objectives of this project were achieved due to the time constraints of the summer work session and errors inherent in the precision of crash location identification.

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INTRODUCTION

Medians are part of all freeways, expressways and principle arterials separating two flows of opposing traffic. According to the American Association of State Highway and Transportation Officials (AASHTO) *Roadside Design Guide (1)***,** a median is the portion of the divided roadway, including inside shoulders, that separates the traveled way for through traffic in opposing directions of travel. Some freeways have medians with concrete positive barriers while others just have a grassy center in the middle. In addition to dividing traffic, a median also provides a recovery area for wayward vehicles, a stopping location for emergencies, glare reduction for oncoming headlights and space for future lane expansion.

Cross-median crashes are usually violent collisions between two vehicles traveling in opposite directions. These crashes often involve high traveling speeds. Furthermore, although cross-median crashes do not happen frequently, they are more likely to be scrutinized by the media since they often lead to serious injuries. For instance, a North Carolina study *(2)* found that 40 % of the injuries sustained in cross-median collisions resulted in incapacitating and fatal injuries. In addition, horizontal curves are associated with a disproportionate number of severe crashes *(3)*. Furthermore, a Pennsylvania Study *(4)* found that nearly 15% of cross median collisions involve fatalities and 72% involve non-fatal injuries. There is a need for a comprehensive study on median width and safety to help prevent these types of collisions.

Agencies are seeking a better understanding of roadway or roadside features that affect safety. The objectives of this study were to analyze cross median crash characteristics for urban and rural, four or more lanes divided limited access highways and to determine the effect median type and width of horizontal curves have on crashes as compared to adjacent tangent sections. Data available for use in the evaluation included 169.889 centerline miles of highways. A secondary objective was to use negative binomial regression models to determine the effects of independent variables on crashes, if time permitted. Variables considered in developing the base models included lane width, outside shoulder width, insider shoulder width, median width (which excluded inside shoulder width), median type, segment length, and annual average daily traffic. Five years (1997-2001) of highway segment crashes were examined.

The product of this research could be useful in the course of developing the Highway Safety Manual (HSM). The HSM is envisioned to become a nationwide predictive tool available to evaluate the safety performance of streets and highways. Additional information is available on the HSM website (http://www.highwaysafetymanual.org/). The HSM is being developed under the direction of the Transportation Research Board Highway Safety Manual Task Force. The manual's goal is to provide the best available safety knowledge in a condensed and widely usable form for designers and practitioners *(5)*. With such a tool, agencies can identify potential areas of concern on streets and highways that can lead to safety performance improvement associated with those facilities. The first edition of the HSM is expected for public release in the summer of 2009.

BACKGROUND

This section describes background information on the project with additional information about previous research involving median width and median barriers.

Previous Research

For the past decades, limited research has been conducted on crashes occurring within medians of high-speed highways. Within the past few years, however, greater attention has been focused on median-related crashes and, in particular, cross-median crashes. The latter category of crashes tends to involve high speeds and results in multiple injuries and fatalities. The research into cross-median crashes typically provides advice on when to install a median barrier for a given AADT and median width. For example, a Texas study *(6)* developed improved guidelines for the use of median barriers on new and existing high-speed, multilane, divided highways. As part of their research, they reviewed existing guidelines for the installation need of median barriers. States developing policy on installing median barriers include North Carolina, California, Washington, Florida, and Pennsylvania.

Miaou, et al. as summarized by Fitzpatrick et al. *(7)* notes that median barriers could reduce cross-median crashes by keeping errant vehicles from reaching the other side of the traffic lanes. However, median barriers do not prevent crashes and may, in fact, increase the number of crashes. Barriers are also obstacles on the roadside and it is possible they will be struck because of their close proximity to the moving vehicles. Miaou et al. summarized the median-related crash rates from several studies as shown in Table 1 *(8)*. They also provided the distribution of severity of the crashes used in their analysis (see Table 2).

The Missouri DOT (MDOT) started installing cable barriers on their Interstate Highways in 1999 on small high-volume segments located in the metropolitan districts of Kansas City and St. Louis (i.e., Interstates 44, 70 and 435) *(9)*. Following the successful outcome these installations, MDOT decided that cable barriers would be installed on all Missouri Interstates with medians less than 60 feet wide due to their over-representation in cross-median crash numbers. A great reduction in cross-median fatalities was noticed as the number of miles with barrier increased. A graph of the decline of cross-median fatalities on I-70 as more miles of cable median were installed is shown in Figure 1.

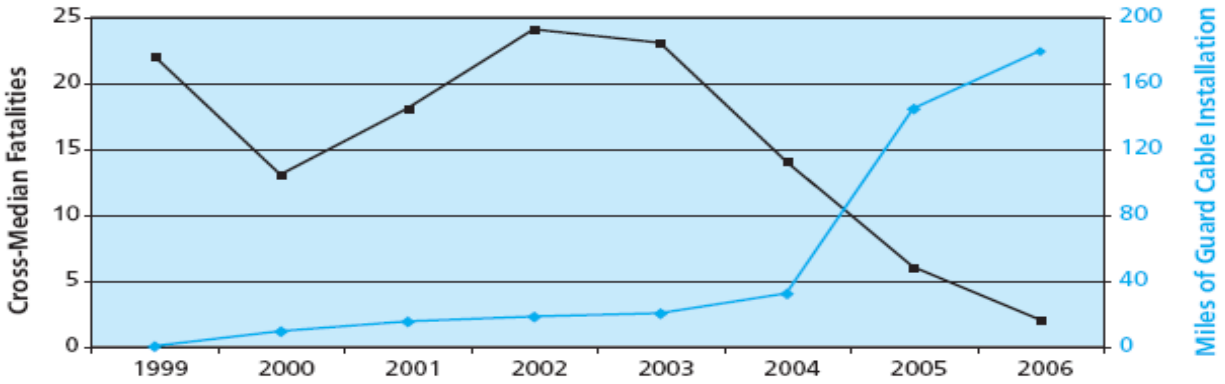


Figure 1. Cross-Median Fatalities on I-70 in Missouri as Miles of Cable Barriers Were Installed *(9)*

Table 1. Median Related Crash Rates of Previous Studies as Summarized by Miaou et al. *(6) (8)*
Table 2. Number and Distribution of Crashes by Severity of Injury for 2006 Miaou et al. *(6) (8)*

Table 1. Median-Related Crash Rates.

Study	Crash rates (crashes/MVM)			
	With Barrier	Without Barrier	Subsets of Without Barrier crashes	
			Cross Median	Other Median Related
Previous Studies (as summarized by Miaou et al. 2006)				
Miaou, 1998-1999 data	0.108	0.087	0.009	0.078
PA, 1994-1998 data, Interstate	---	---	0.004	---
PA, 1994-1998 data, Freeways	---	---	0.007	---
CA, 1984-188 data, Freeways	---	---	0.007	---
Washington, Sites where barriers were added due to crash history	---	---	0.021	---
Current Study (TX, 1997-2001 data)				
With barriers, all crashes With barriers, KABC* crashes	0.105 0.059	---	---	---
Without barrier, rural, all crashes Without barrier, rural, KABC crashes	---	0.068 0.024	---	---
Without barrier, urban, all crashes Without barrier, urban, KABC crashes	---	0.065 0.036	---	---

--- = crash rate not identified in the study
* KABC crashes includes fatal (K), incapacitating injury (A), non incapacitating injury (B), and possible injury (C)

Table 2. Number and Distribution of Crashes by Severity of Injury for 2006 Miaou Study.

Barrier and Crash Type	Total Number of Crashes	Severity Type				
		Fatal (K)	Incapacitating Injury (A)	Non Incapacitating Injury (B)	Possible Injury (C)	Property Damage Only (PDO)
No barrier						
Cross-median crashes	346 100%	73 21.1%	73 21.1%	82 23.7%	58 16.8%	60 17.3%
Other median-related crashes	3064 100%	71 2.3%	272 8.9%	639 20.9%	734 23.9%	1348 44.0%
With Longitudinal Barrier						
All median-related crashes (including hit-median-barrier crashes)	3672 100%	36 1.0%	190 5.2%	681 18.5%	1098 29.9%	1667 45.5%
Hit-median-barrier crashes	2714 100%	13 0.5%	128 4.7%	490 18.0%	835 30.8%	1248 46.0%

Median Width

Previous research on this topic has produced many conflicting results on the impact of median width and safety along with the uncertainty about the role of the median barrier. Although the standards state the median should "be as wide as possible," economic conditions dictate otherwise. It is unknown what the optimal width should be for the median. Hauer *(10)* summarized previous studies in his report on median width and safety**.** He concluded that previous studies showed that total accident rates by median width are not associated with each other. However, he notes that the results of these studies are questionable because of the omission of the influence of some important variables (rural vs. urban, speed limit, highway type, right shoulder width, etc.). He also mentions that the previous research has visibly shown that medians wider than 50 feet show a clear reduction in cross-median collisions than with medians less than 50 feet wide. Finally, he remarks that there is a probability the accident rates of median-related crashes for medians without-barriers increase with median width up to 30 feet then decline as the median gets wider *(11)*.

Median Barriers

A study conducted by the Texas Transportation Institute on median barriers reported that the presence of a median barrier "does not eliminate crashes occurring in medians but alters the character of those crashes" *(12)***.** In other words, the overall amount of median-related crashes would probably increase but the overall severity of cross-median collisions would be reduced. As the report notes however, the median width is already established or constrained by right-of-way restrictions.

DATA COLLECTION

This section describes the data collection processes to obtain the appropriate sample size for developing the statistics in this paper.

Data Collection Process

Many variables were considered as input data. The researchers' (the ones associated with TxDOT Project 0-4703) approach was to determine if a difference in roadway elements influence the number and severity of median-related and cross-median crashes. To do this, they collected sections of roadways in pairs. The sections were a horizontal curved segment with an adjacent tangent segment separated by a tenth of a mile buffer. The attributes within each pair needed to be identical except for the difference in the attribute(s) of interest. By selecting pairs of matched sections, the effect of the selected attribute(s) on safety is isolated and other factors are better controlled.

Selection Procedure

For this study, controlled access divided highways were selected to be adjacent to each other on the same roadway to ensure that the pairs are as nearly matched as possible. Sites were eliminated if the horizontal curve was longer than 1.5 miles or less than one-tenth of a mile or

degree of curvature information was not available for the curve. In addition, the curves were eliminated if the aerial photos showed the road under construction or were not clear enough. The tangent prior to or following the horizontal curve was required to be equal to the curve length plus 0.1 of a mile. The 0.1 mile represents the "buffer zone" that was eliminated from the evaluation to ensure that curve-related crashes are not inadvertently placed on the tangent segment and vice versa. The length of the buffer reflects recognition of the precision of crash location in the Texas Department of Public Safety (DPS) crash database. The database locates crashes to the nearest 0.1 mi.

Because data for only a sample of the curves could be collected, the researchers selected districts in different regions of Texas. Geometric and crash data were collected on the curves within ten districts that match the project criteria. Freeways and divided highways located in the following Texas Department of Transportation (TxDOT) districts were considered for inclusion for the sample size (number of curves from each district in parenthesis):

- Amarillo (35)
- Austin (50)
- Beaumont (26)
- Bryan (34)
- El Paso (42)
- Ft. Worth (71)
- Houston (84)
- Laredo (10)
- Odessa (87)
- San Antonio (151)

The dataset for the study included 590 curves in total. Table 3 shows the location and number of curves collected for this study.

Types of Roadway Used

The TxDOT Reference Marker Database (TRM) was used to identify potential highway segments. The roadway geometric characteristics for segments were used when the following conditions were met:

- Main-lanes
- Highway design type = freeway or expressway with no high-occupancy vehicle (HOV) lanes, no railroad crossings, no at grade intersections and no tolls
- Median Type = no median, grass, or flush
- Barrier Type = none, concrete, or W-beam
- Total number of lanes is four or greater

Researchers then located each horizontal curve segment within an aerial photograph using previously found GIS coordinates. See Figure 2 for an example of an aerial photograph of a horizontal curve. Preference was to use Google's Google Earth® program for the aerial photographs of the roadways. The degree of curvature geometric information was obtained from

the TxDOT TRM database. If a curve could not be found or the photo was unreliable (i.e., showing the road under construction or the resolution too small) the data entry was deleted. If the quality of the view available on the photograph was sufficient for data collection, the following segment characteristics were identified for the horizontal curve and for the associated tangent section:

- Lane width (ft) [for increasing and decreasing milepoints],
- Outside {Right} shoulder width (ft) [for increasing and decreasing milepoints],
- Inside {Left} shoulder width (ft) [for increasing and decreasing milepoints],
- Median type (e.g., none, grass, flush),
- Barrier type (rigid [includes both concrete barrier or W-beam] or no barrier),
- Median width (ft) (without inside shoulders),
- Distance to Barrier (ft) {if applicable}[for increasing and decreasing milepoints],
- Number of Lanes (both directions),

In addition, the following values were collected from the TRM database:

- Speed Limit (Miles per Hour)
- AADT
- Curve Length
- Right of Way (ROW) Width (ft)

Table 3. Location and Number of Curves Collected for This Study

District Number	District Name	All Curves	Collected
2	Ft. Worth	192	71
4	Amarillo	79	35
20	Beaumont	55	26
17	Bryan	80	34
22	Laredo	41	10
15	San Antonio	254	151
12	Houston	281	84
24	El Paso	76	42
14	Austin	135	50
6	Odessa	119	87
Total			590

Tables and Figures in the Appendix show the distribution of the section attributes that were the same for both the tangent and horizontal curve sections.

Figure 2. Sample Aerial Photo/Google Maps® Image
The horizontal curve segment of the highway is outlined in this sample image.

Crash Data

Crash data for each section were extracted from the DPS electronic database. A total of five years of crash data (1997 to 2001) was used. Crashes for all severity levels were extracted. Analyses were performed using only fatal (K), incapacitating-injury (A), non-incapacitating injury (B), and minor injury (C) [KABC Crashes]. Due to their unreliability (i.e., under-reporting), property damage only (PDO) crashes were not used in the analysis.

The values listed in the tables in the Appendix are based on crashes that occurred during the five-year period of 1997 to 2001 for the 590 segments.
Crashes were grouped into three categories: Median-Related Crashes (MRC), Median Crossover Crashes Type 1 (MCC1) and Median Crossover Crashes Type 2 (MCC2). MRC are described as when a vehicle is traveling in the main-lanes, crashes and ends up in the median area (excluding shoulders). See Figure 3 for more details by lane number. MRC can be single or multi-vehicle

accidents. MCC1 are depicted as when a vehicle is traveling in the main-lanes and crosses over the median and collides with another vehicle traveling in the opposite flow of traffic thus ending up on the opposite side on the median but still on the main-lanes of the freeway. MCC1 are multi-vehicle collisions only. See Figure 3 for more details by lane number. MCC2 are described as when a vehicle is traveling in the main-lanes, crosses the median and crashes as a single vehicle accident in the opposing main-lanes of travel or continues over all the way to the opposing frontage road and beyond. This type of collision could involve a single vehicle or multiple vehicles (i.e., on the frontage opposite frontage road). Consequently, MCC2 can be single or multi-vehicle crashes. See Figure 3 for more details by lane number.

Due to data inconsistencies, the researchers limited the data for with-barrier MCC1 and with-barrier MCC2 to two (2) or less crashes per section for each the horizontal curve segment and tangent segment. The breakdown of curves for Crash Data analysis is presented in Table 4.

Roadway Statistics

The researchers computed various figures from the roadway dataset. The charts of the roadway statistics are displayed in Appendix A. Additionally, graphical summaries of the distribution of variables per number of miles for with and without barrier segments in the freeway dataset are displayed in Appendix D and Appendix E. Table A-1 shows the summary of statistics of the roadway characteristics for all 590 segments. The researchers divided the 590 segments into the 225 with-barrier and the 365 without barrier segments. For each of the roadway characteristics listed above, the researchers recorded the average (arithmetic mean), minimum, maximum and standard deviation for the with-barrier segments, without-barrier segments and the whole dataset.

To compute the standard deviations of the data, the following equation was used:

$$\sigma = \frac{\sqrt{\sum (x - \bar{x})^2}}{(n-1)}$$

Where,

x = Individual Value
$\bar{x}$ = Sample Mean
n = Sample Size

Accident Statistics

The author plotted the relative frequency of the data compared to crash rate and crashes per mile for the six sets of data. As shown in Figure 4, Figure 5, Figure 6, Figure 7, Figure 8, and Figure 9 only the data for Median Related Crashes with-barrier follows a normal pattern. As these figures illustrate, nearly all of the crash rates are below 0.02 Crashes/mvm and the majority of the segments are under one crash per mile. This is because for about 80% of the Median Crossover Crash segments, Types 1 and 2, and Median-Related Crashes without-barrier there were no crashes at all thus their values are zero. For Median Related Crashes with-barrier the data are

more spread out among the variables, and only about 30-35% of the segments have no crashes on them.

Table 4. Breakdown of Segments for Crash Statistics

Breakdown of Segments for Crash Statistics	
Total Curves	**590**
With Barrier	225
Without Barrier	365
Median Related Crashes	
With Barrier	
Horizontal Curve	225
Tangent	225
Total	**450**
Without Barrier	
Horizontal Curve	365
Tangent	365
Total	**730**
Median Crossover Crashes 1	
With Barrier	
Horizontal Curve [2 or Less Crashes]	223
Tangent [2 or Less Crashes]	222
Total	**445**
Without Barrier	
Horizontal Curve [All]	365
Tangent [All]	365
Total	**730**
Median Crossover Crashes 2	
With Barrier	
Horizontal Curve [2 or Less Crashes]	210
Tangent [2 or Less Crashes]	210
Total	**420**
Without Barrier	
Horizontal Curve [All]	365
Tangent [All]	365
Total	**730**

In Appendix A, the tables summarizing the crash characteristics for horizontal curve and tangent segments are shown in Table A-2 and Table A-3, respectively. Again, the segments were divided into with-barrier and without-barrier sections. The data were separated into horizontal curve sections and tangential sections for crash comparisons. After each crash was separated into HC and TAN segments, they were further divided into the three types of crashes, as described above: Median Related Crashes (MRC), Median Crossover Crashes Type 1 (MCC1) and Median Crossover Crashes Type 2 (MCC2). Once more, for Median Crossover Crashes types 1 and 2 with-barrier, the data were limited to two crashes per segment for each of the horizontal curve and tangent segments. For each subset, average crashes per segment, maximum number crashes

on an individual segment, minimum number of crashes on an individual segment, the total number of crashes, the standard deviation, the crash count and the crash rate were calculated.

To compute the crash rate, the following equation was used *(13)*:

$$\text{Crash Rate/MEV} = \frac{N}{DEV} \times \frac{1{,}000{,}000}{5yr \times 365 days / year}$$

Where,

MEV = Million Vehicle Miles
DEV= Length of Segment*AADT
AADT = Annual Average Traffic per Day in vehicle per day

To compute the crash count per mile, the following equation was used:

$$\text{Crash Count 5-yr} = \frac{N}{L \times 5yr}$$

Where,

N = Total Number of Crashes for five year period
L = Length of Segment
5yr = Five year period of crash data

DATA ANALYSIS

This section of the paper describes the data analysis and the various techniques the author used to evaluate the data.

Analysis of Variance

The author calculated the Analysis of Variance (ANOVA) for Median-Related Crashes with and without-barrier, Median Crossover Crashes Type 1 without-barrier and Median Crossover Crashes Type 2 without-barrier comparing the crash rate and crashes per mile of the tangent and horizontal curve segments. A summary of the results is shown in Table 5, and the full calculations are shown in Tables B-1 – B-4 in Appendix B. First, the Fisher F-Test was calculated on an Excel spreadsheet, and then the p-value was calculated using an internet math program *(14)*. The F-test is a statistical test in which the test statistic has an F-distribution if the null hypothesis is true. The hypothesis shows that the averages of multiple normally distributed populations with the same standard deviation are equal and thus that they are of comparable origin. The p-value is the probability of obtaining a value of the test statistic at least as extreme as the one that was actually observed, given that the null hypothesis is true. The null hypothesis can be rejected if the p-value is smaller than or equal to the significance level, (in this case $\alpha = 0.05$) [i.e., degree of confidence equals 95%] *(15)*. A null hypothesis is a scenario set up to be nullified or disproved statistically in order to support an alternative hypothesis. The null hypothesis is assumed true until statistically proved otherwise.

As shown, the F-value varies significantly from the critical value of 3.84. Additionally, the p-values are smaller than α when the F-test is less than the critical value, thus those values the null hypothesis is true. For median related crashes and cross median crashes type 1, there is a statistical difference between tangent and horizontal curves, but not for cross median crashes type 2.

Table 5. Analysis of Variance Calculations Summary for Comparison Tangents – Horizontal Curves

ANOVA Results for Comparison Tangents – Horizontal Curves		
Crash Rate		
Variables	Barrier	No Barrier
Median-Related	F=0.12 (p<0.73)	F=6.43 (p<0.01)
Cross-Median Type 1	---	F=3.74 (p<0.05)
Cross-Median Type 2	----	F=1.92 (p<0.17)
	c.v. = 3.84	α = .05
Crashes per Mile		
Variables	Barrier	No Barrier
Median-Related	F=0.39 (p<0.53)	F=5.22 (p<0.02)
Cross-Median Type 1	---	F=8.19 (p<0.004)
Cross-Median Type 2	----	F=0.45 (p<0.50)
	c.v. = 3.84	α = .05

To compute variance, the following equation was used:

$$s^2 = \frac{\sum (x - \bar{x})^2}{(n-1)}$$

Where,

$x =$ Individual Value
$\bar{x} =$ Sample Mean
$n =$ Sample Size

Unfortunately, the data were not without errors. Due to time restraints, the data could only be rechecked and verified a limited number of times before the dataset was finalized for analysis. During the analysis of the data, inconsistencies were found. There were a large number of cross-median collisions on roadways with positive barriers, which is counterintuitive. The roadway segments with barriers that have experienced an unusually large number of crashes will have to be checked individually to verify the validity of the statistics and to confirm whether a barrier was installed since the period of the crash data [1997-2001]. Measures will be taken to correct the data for the larger TxDOT study [Incorporating Safety into the Highway Design Process 0-4703]. However, due to time constraints the author for this summer project did not have the time or resources to complete that task. Consequently, for each of the two types of roadway with

barrier [Horizontal Curve and Tangential] and for each Median Crossover division [Median Crossover Type 1 and Median Crossover Type 2] the data were limited to a maximum of two crashes per segment. This split the data into four sets as shown in Table 4.

Statistical Modeling

The author planned to use the Statistical Analysis Software (SAS) program to model the data. Subsequently, the author intended to derive AMFs from the developed models. However, due to time limitations and data error, neither goal was accomplished.

RESULTS

This section of the paper displays the results of the research compiled by the author.

Figures

The following are the Figures referenced earlier in the text.

Refer to Figure 1 for the following explanations of crash categories:

*Median Related Crashes (MRC)***:** the vehicle is traveling in lanes 10, 11, 12, 80 or 81 for decreasing Milepoint or 16, 17, 18, 82 or 83 for increasing Milepoint and ends up in 63 [the median].

*Median Crossover Crashes (MCC1***):** the first vehicle is traveling in lanes 10, 11, 12, 80 or 81, crosses the median and collides with a vehicle traveling in lanes 16, 17, 18, 82, or 83 [for decreasing Milepoint], or the first vehicle is traveling in lanes 16, 17, 18, 82 or 83 crosses the median and collides with a vehicle traveling in lanes 10, 11, 12, 80 or 81.

Median Crossover Crashes Type 2 (MCC2):
Case 1 [Single Vehicle Accident Scenario]: The vehicle originates in lanes 10, 11, 12, 80 or 81 crosses the median and comes to stop in lanes 6, 7, 8, 16, 17, 18, 26, 27, 28, 29, 58, 64, 82 or 83, [for decreasing Milepoint] or the vehicle originated in lanes 16, 17, 18, 82, 83 crosses the median and comes to stop in lanes 1, 2, 3, 10, 11, 12, 20, 21, 22, 23 52, 62, 80 or 81 [for increasing Milepoint] without striking another vehicle.

Case 2 [Multi-Vehicle Accident Scenario]: The first vehicle originates in lanes 10, 11, 12, 80 or 81 crosses the median and collides with a vehicle in lanes 26, 27, 28 or 29 [for decreasing Milepoint] or the first vehicle originates in lanes 16, 17, 18, 82, 83 crosses the median and collides with a vehicle in lanes 20, 21, 22, 23 [for Increasing Milepoint].

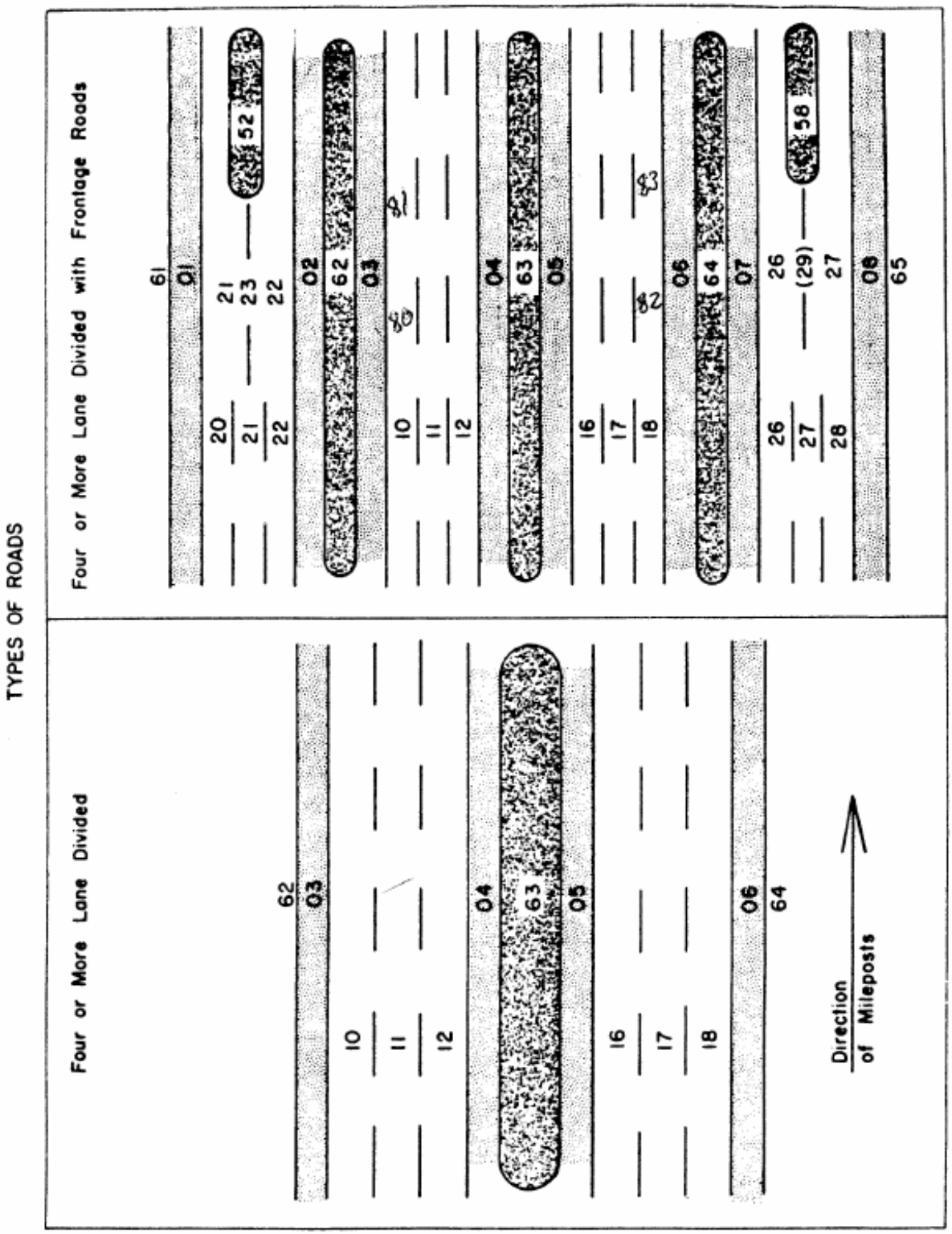
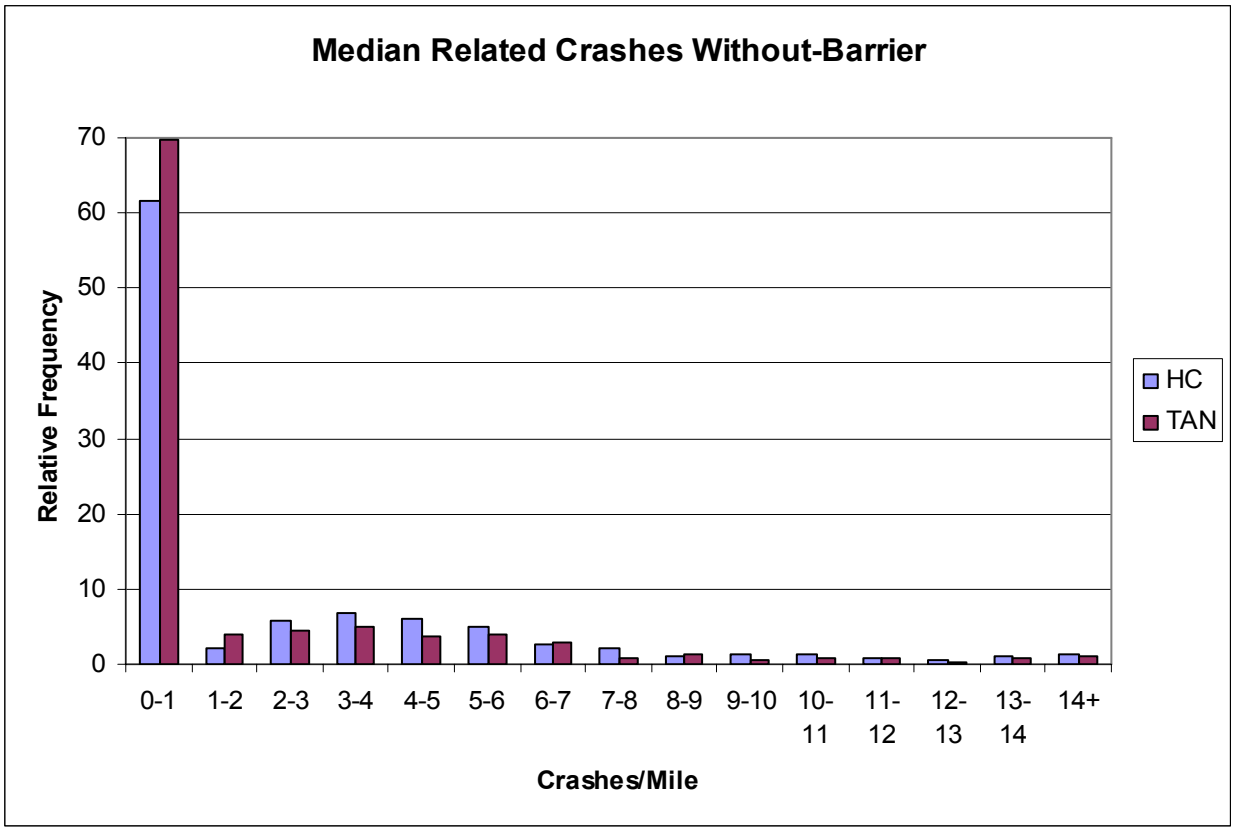


Figure 3. Lane Descriptions [Lanes 80, 81 or 82, 83 are the Fourth and Fifth Main-Lanes in Each Direction if Applicable] *Adapted from Accident and Roadway Inventors (16)*



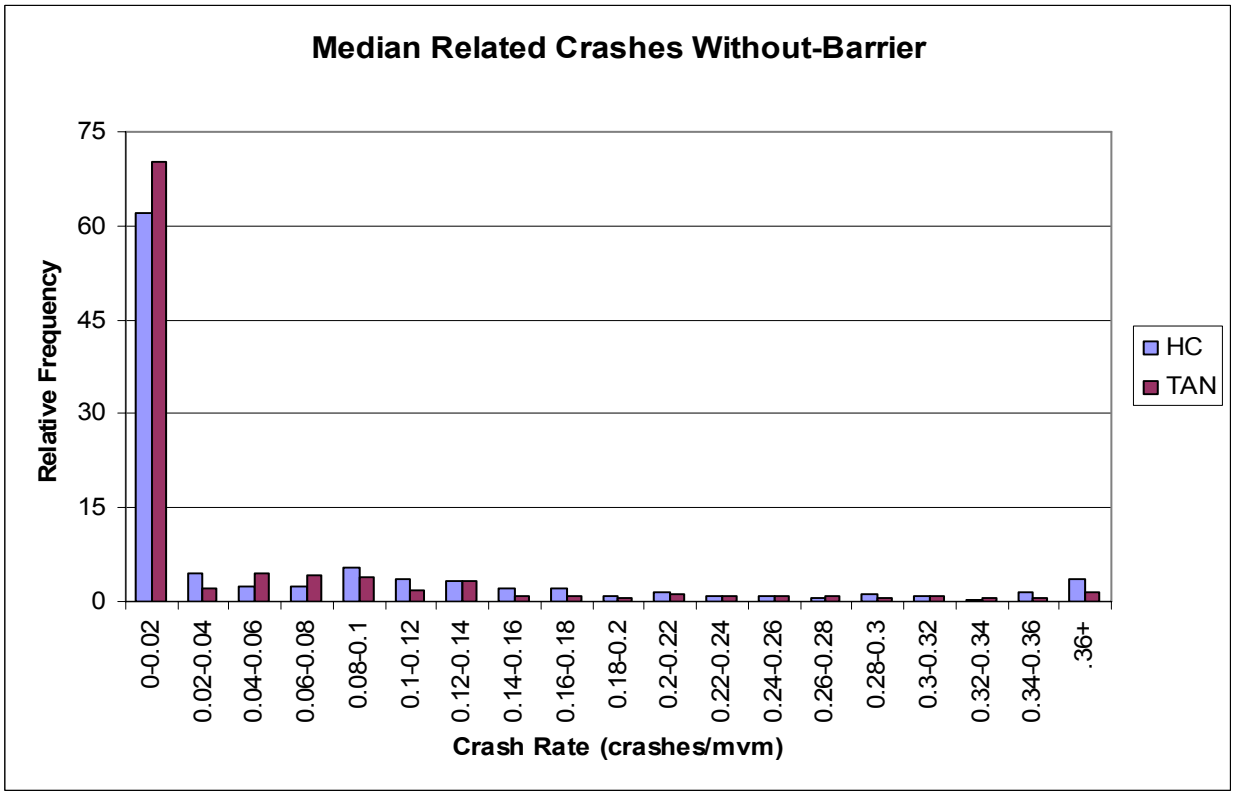
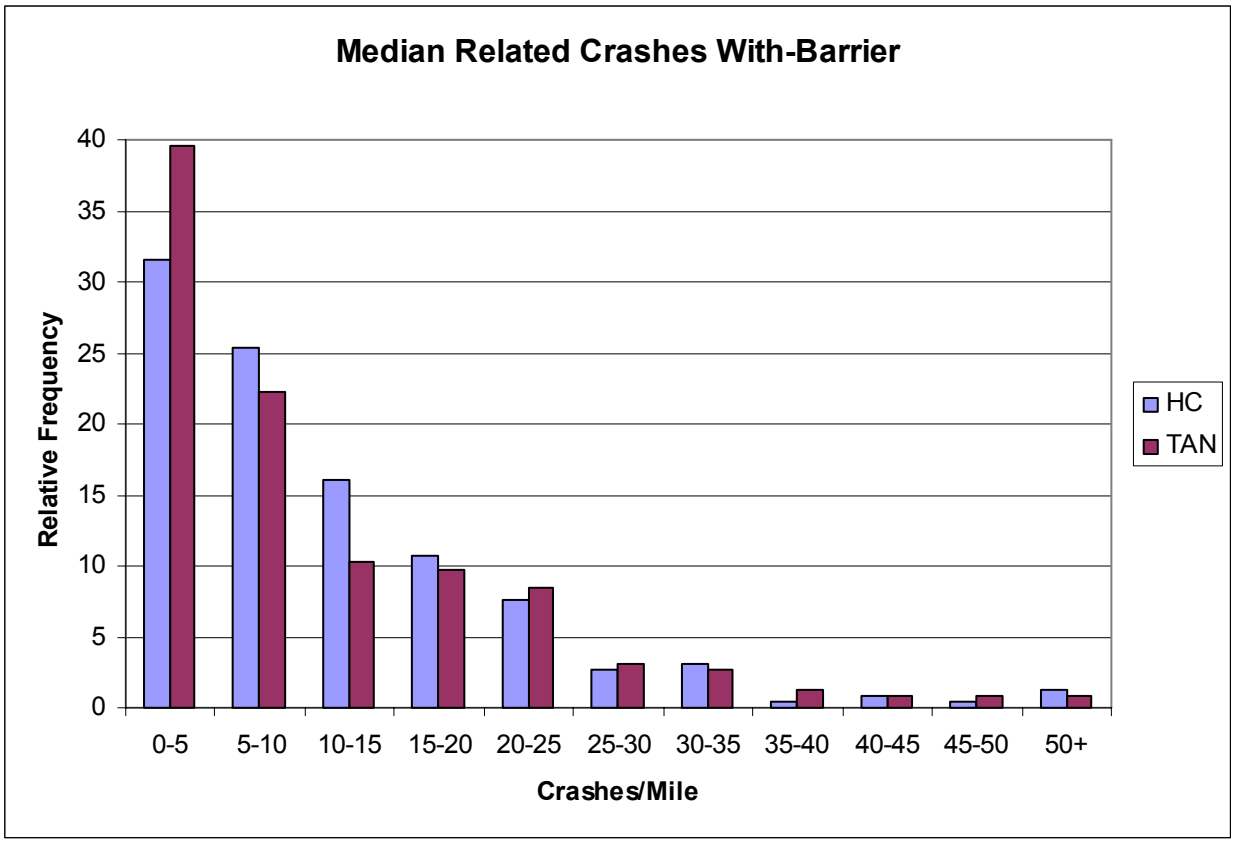


Figure 4. MRC Without-Barrier Crash Statistics



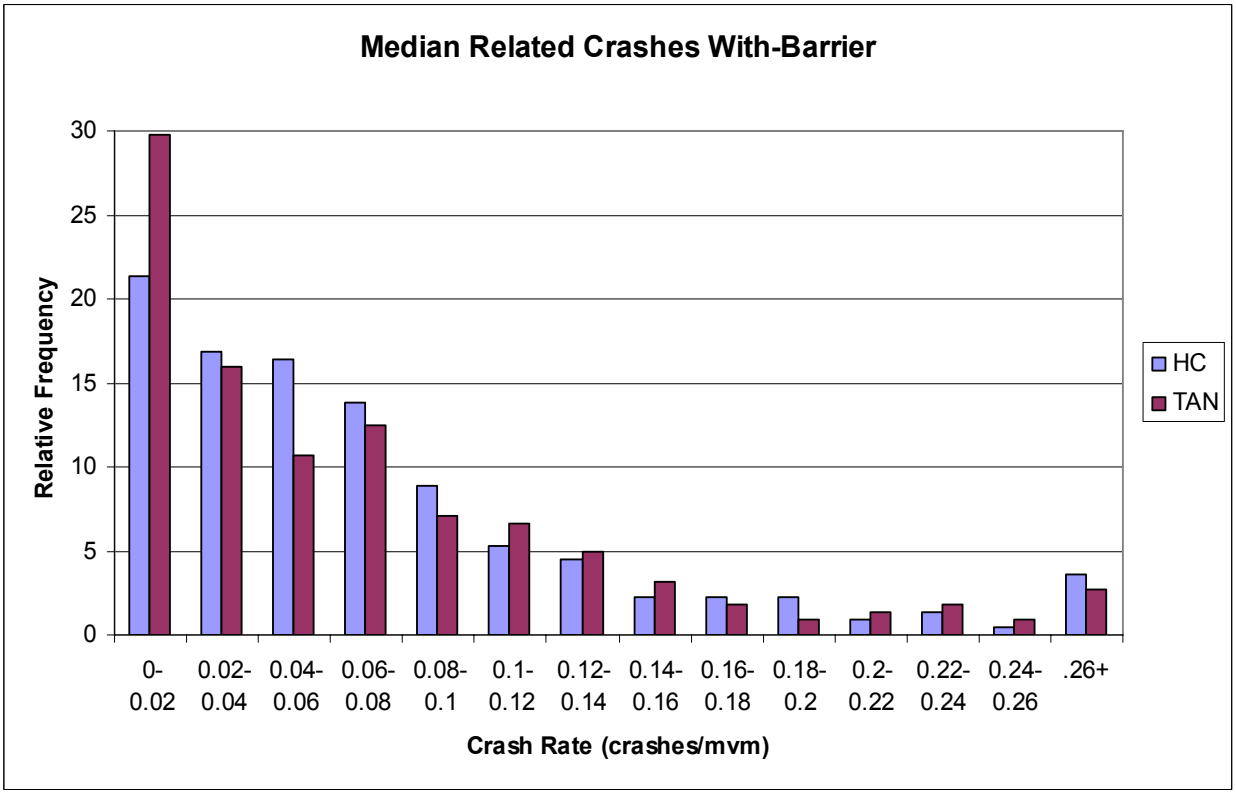


Figure 5. MRC With-Barrier Crash Statistics

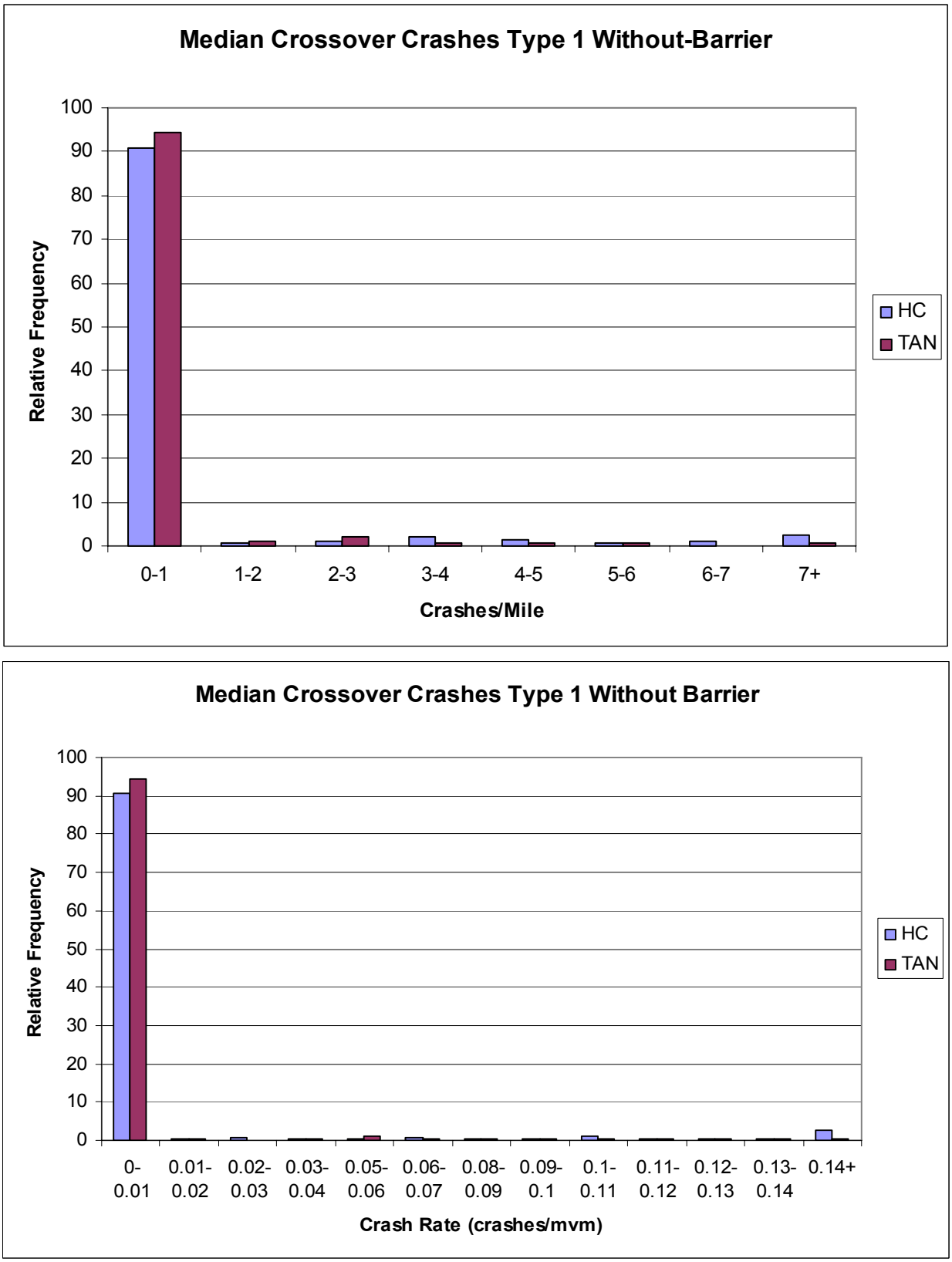
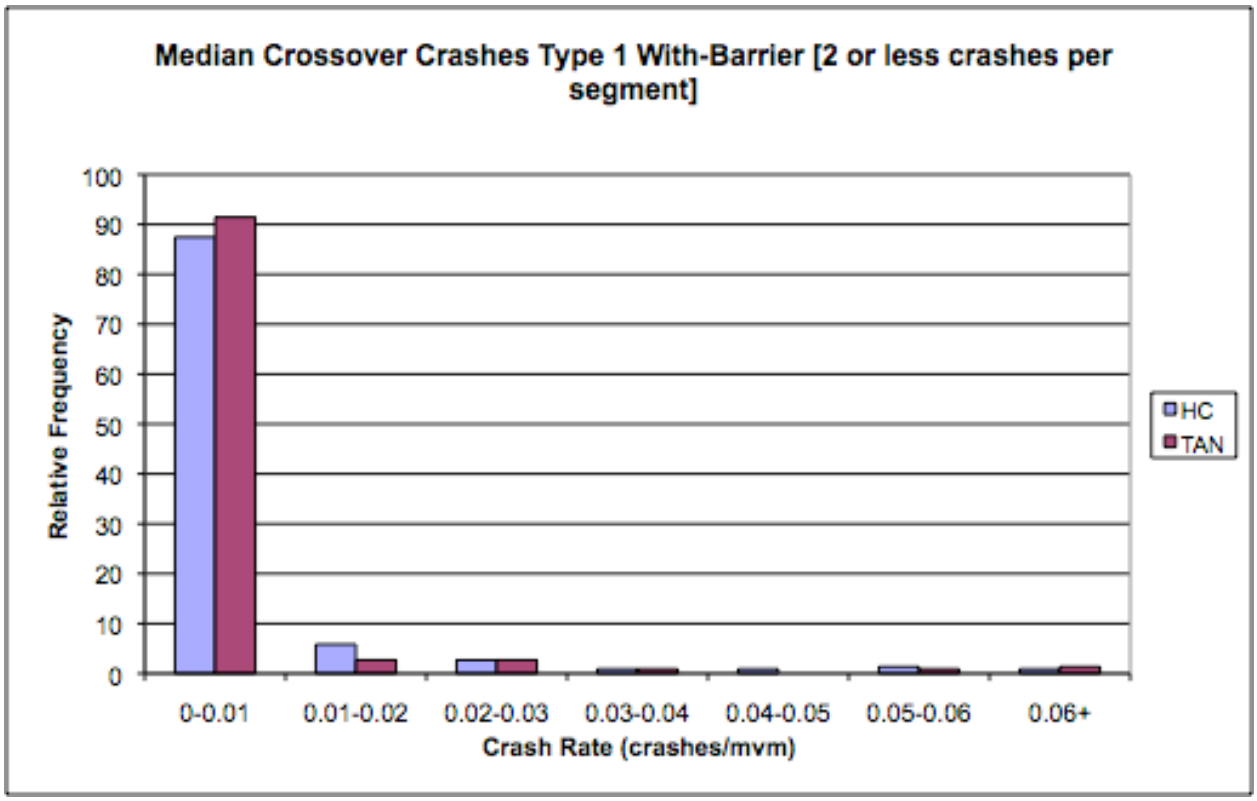


Figure 6. MCC Type 1 Without-Barrier Crash Statistics



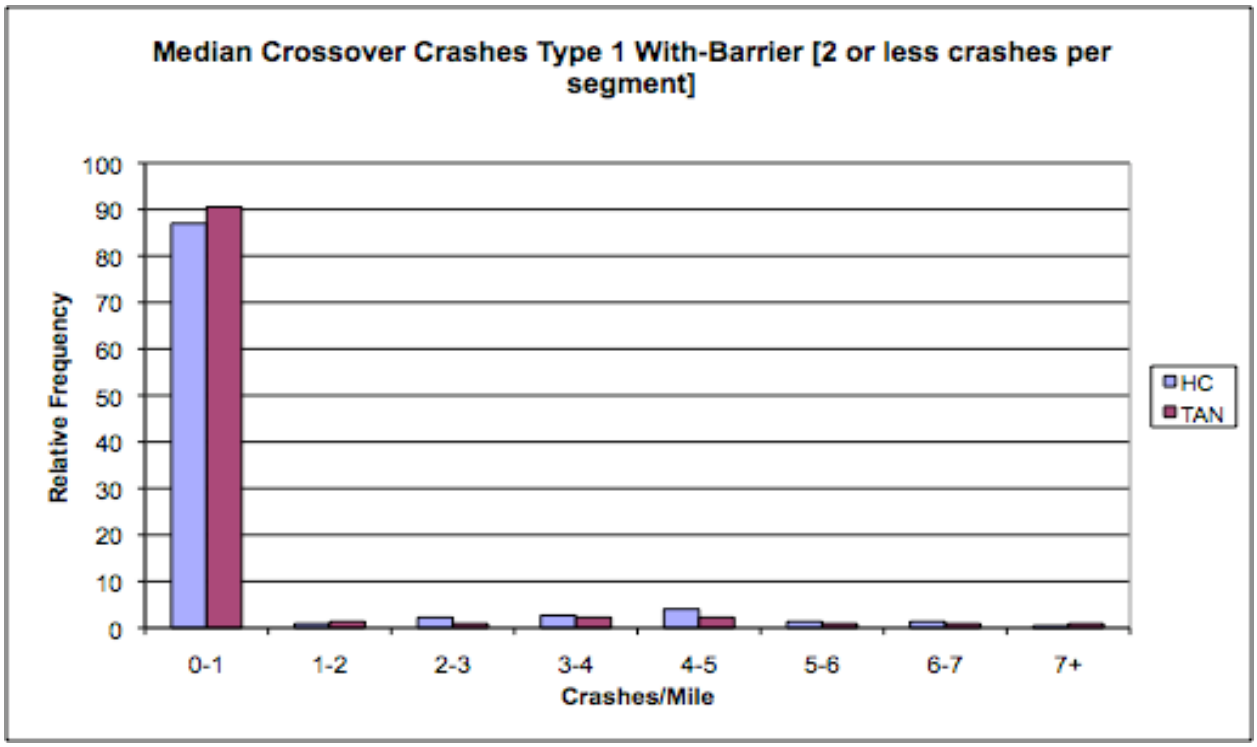
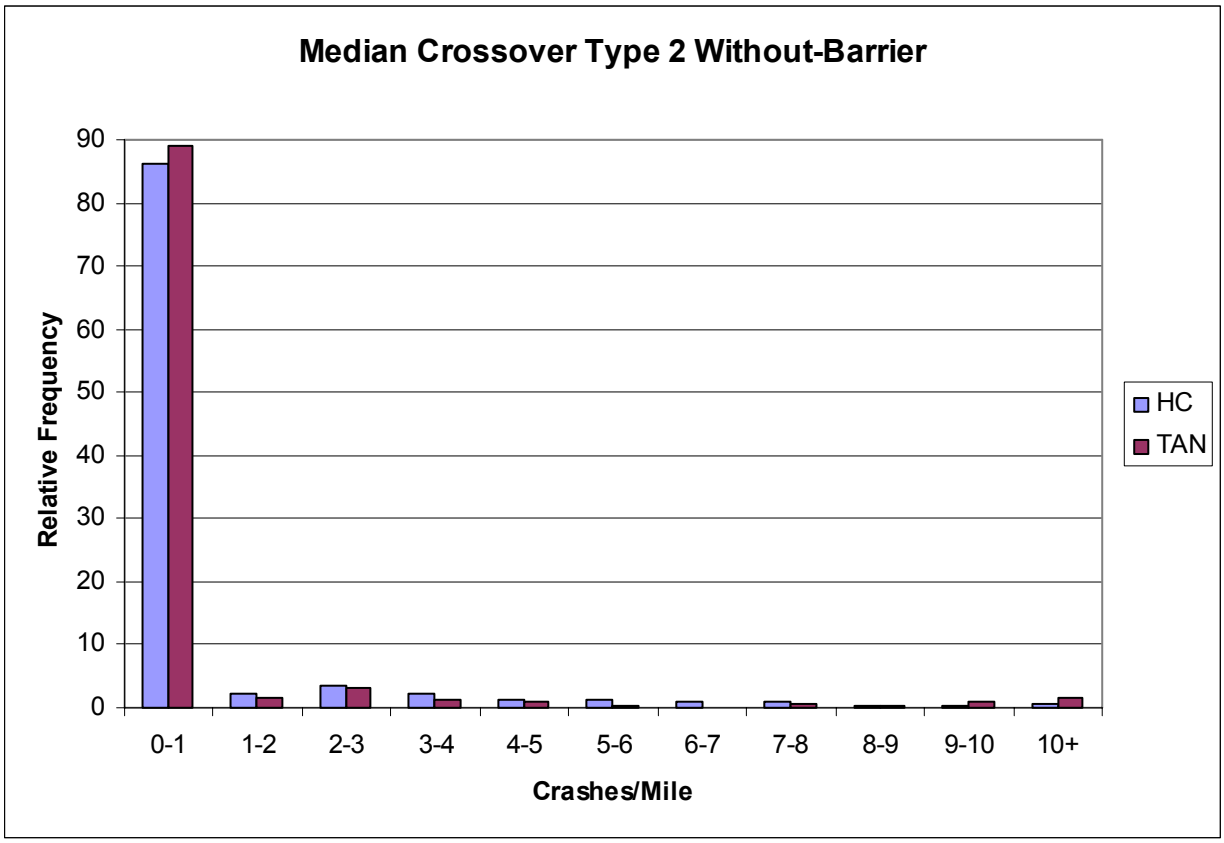


Figure 7. MCC Type 1 With-Barrier Crash Statistics



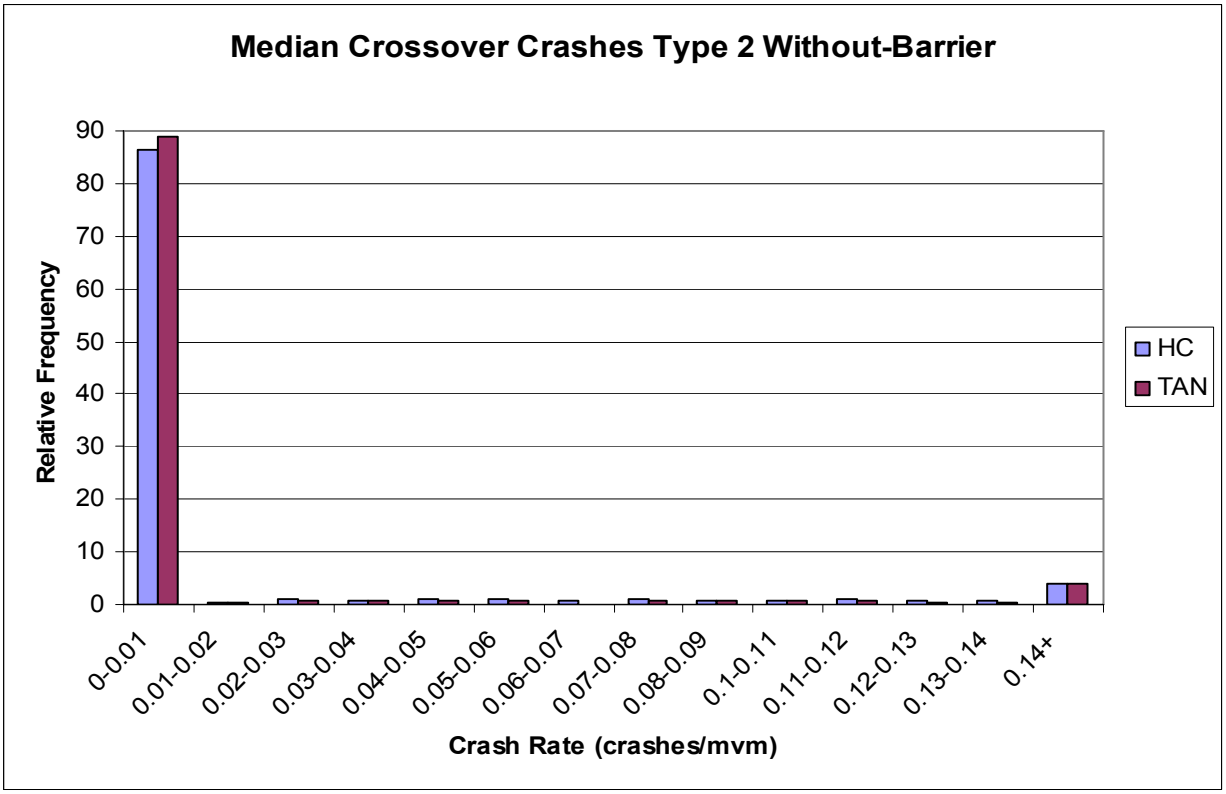
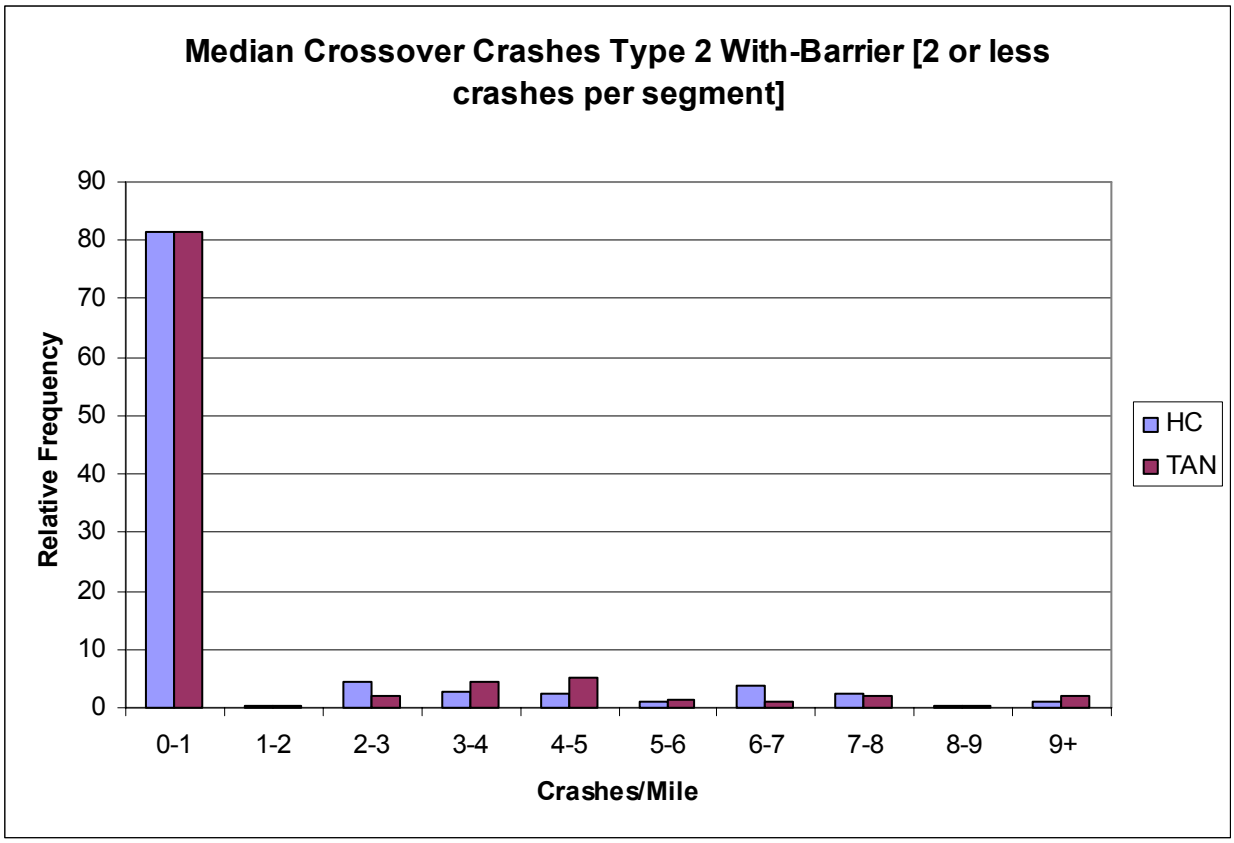


Figure 8. MCC Type 2 Without-Barrier Crash Statistics



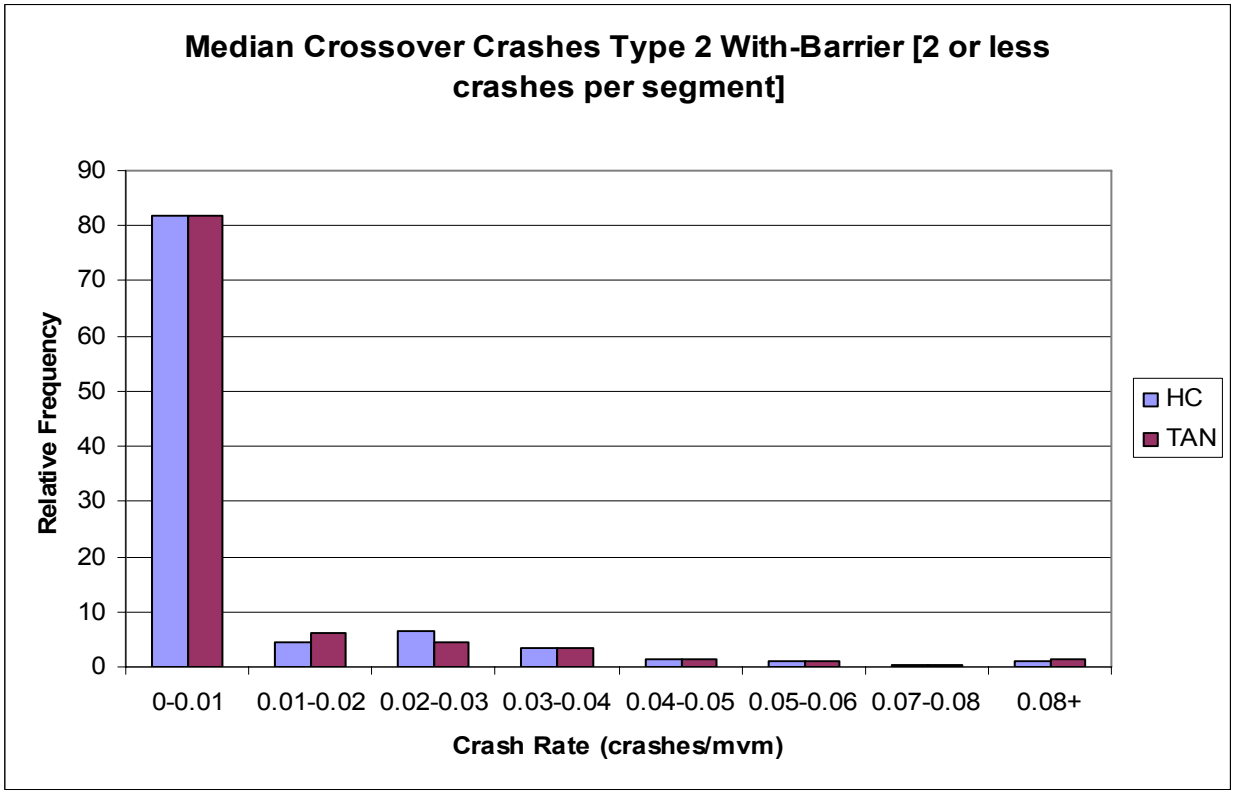


Figure 9. MCC Type 2 With-Barrier Crash Statistics

RECOMMENDATIONS

Due to time constraints, the author was not able to finish the requirements by the paper deadline. However, the next step would involve the estimation of predictive models using statistical software programs, such as SAS or Gentstat to predict and derive Accident Modification Factors. Additionally, information is needed for the development of predictive models using Negative Binomial regression models. This would necessitate a better estimate about the actual influence of median width on cross-median collisions. These tasks are scheduled to be completed later. The author was unable to include these analyses in this work. The final goal will be to incorporate the results into the larger TxDOT Project [0-4703] and the Highway Safety Manual.

CONCLUSIONS

This report examined the relationships and results of cross-median crashes on horizontal curves. The researchers found that segments with barriers have higher crash rates than segments without barriers. Furthermore, it was discovered that larger median widths have fewer cross-median collision crashes although their rate of reduction is not very sizeable in comparison to median width. Finally, it was learned that horizontal curve segments experience more median-related and cross-median type 1 crashes per mile than tangential segments for the same exposure (length and traffic flow).

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APPENDICES

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APPENDIX A [ROADWAY STATISTICS CHARTS]

Table A-1: Roadway Characteristics- Summary of Statistics for the 590 Segments

Roadway Characteristics					
Summary of Statistics					
	Variable			**Condition**	
			Total	With Barrier	Without Barrier
	Total Sites		590	225	365
	Sum of Segment Lengths (miles)		169.889	56.713	113.176
	Average Curve Length (miles)	Average	0.288	0.252	0.310
		Minimum	0.101	0.101	0.102
		Maximum	0.982	0.944	0.982
		SD**	0.162	0.162	0.162
	AADT for All Sites (2003^)	Average	55,714	102,861	26,650
		Minimum	4,020	16,530	4,020
		Maximum	265,280	265,280	151,220
		SD**	54866.35	54082.76	55055.07
	Average AADT for All Sites ('97-'01)		50,282	93,643	23,554
	Speed Limit (Miles Per Hour)	Average	65.90	60.53	69.21
		Minimum	40	40	55
		Maximum	75	70	75
		SD**	6.538	6.538	6.539
	Number of Lanes {TRM Data}*	Average	4.95	6.07	4.24
		Minimum	4	4	4
		Maximum	11	11	9
		SD**	1.524	1.524	1.523
	Right of Way [ROW] Width (ft)	Average	340.41	314.66	353.58
		Minimum	120	120	200
		Maximum	785	520	785
		SD**	61.31	67.02	61.50
	Right Shoulder Width [Increasing Milepoint] (ft)	Average	10.17	10.50	10.05
		Minimum	3.5	3.5	4
		Maximum	20	20	15
		SD**	1.498	1.909	1.502
	Left Shoulder Width [Increasing Milepoint] (ft)	Average	6.85	9.61	5.14
		Minimum	0	0	0
		Maximum	24	20	24
		SD**	3.863	4.113	3.870
	Lane Width [Increasing Milepoint] (ft)	Average	11.65	11.66	11.64
		Minimum	10	10	10
		Maximum	13	13	13
		SD**	0.647	0.533	0.648

Roadway Characteristics					
Summary of Statistics					
	Variable			**Condition**	
			Total	With Barrier	Without Barrier
	Distance to Barrier [Increasing Milepoint] (ft) *{If Applicable}*	Average	12.03	12.03	
		Minimum	0.5	0.5	
		Maximum	66	66	
		SD**	8.659	8.659	
	Median Width without Shoulder (ft)	Average	34.82	8.43	51.08
		Minimum	2	2	7
		Maximum	142	70	142
		SD**	27.254	13.858	27.233
	Distance to Barrier [Decreasing Milepoint] (ft) *{If Applicable}*	Average	10.78	10.78	
		Minimum	0.5	0.5	
		Maximum	40	40	
		SD**	5.375	5.375	
	Left Shoulder Width [Decreasing Milepoint] (ft)	Average	6.77	9.41	5.14
		Minimum	0	0	0
		Maximum	25	25	24
		SD**	3.677	3.676	3.682
	Lane Width [Decreasing Milepoint] (ft)	Average	11.65	11.66	11.64
		Minimum	10	10	10
		Maximum	13	13	13
		SD**	0.633	0.503	0.634
	Right Shoulder Width [Decreasing Milepoint] (ft)	Average	10.17	10.37	10.05
		Minimum	3	3	3
		Maximum	22	17	22
		SD**	1.544	1.800	1.548
	^ Latest Year Data is Available * Total Number of Lanes in Both Directions ** Standard Deviation				

Table A-2: Crash Characteristics for Horizontal Curve Portion of Roadway

Crash Characteristics					
Horizontal Curve Portion of Roadway (Two or less Crashes per segment with barrier)					
Variable				Crash Characteristics KABC* Crashes	
				Condition	
			Total	With Barrier	Without Barrier
Total Sites			590	225	365
Sum of Segment Lengths (miles)			169.889	56.713	113.176
Median Related Crashes (MRC) in Five Years (1997 to 2001)	Per Segment	Average	1.454	2.73	0.67
		Minimum	0	0	0
		Maximum	17	17	6
		SD**	2.188	2.191	2.189
	All Segments	Sum	858	614	244
		Crash Count (crashes/mile-yr)	1.010	2.165	0.431
		Crash Rate (crashes/mvm)^	0.055	0.063	0.051
Total Sites			588	223	365
Sum of Segment Lengths (miles)			168.461	55.285	113.176
Median Crossover Crashes 1 (MCC1) in Five Years (1997 to 2001)	Per Segment	Average	0.16	0.15	89.75
		Minimum	0	0	0
		Maximum	7	2	7
		SD**	0.384	0.418	0.611
	All Segments	Sum	94	34	60
		Crash Count (crashes/mile-yr)	0.112	0.123	0.106
		Crash Rate (crashes/mvm)	0.006	0.004	0.012
Total Sites			575	210	365
Sum of Segment Lengths (miles)			165.589	52.413	113.176
Median Crossover Crashes 2 (MCC2) in Five Years (1997 to 2001)	Per Segment	Average	0.21	0.23	0.20
		Minimum	0	0	0
		Maximum	5	2	5
		SD**	0.432	0.513	1.182
	All Segments	Sum	120	48	72
		Crash Count (crashes/mile-yr)	0.145	0.183	0.127
		Crash Rate (crashes/mvm)	0.008	0.006	0.015

* Fatal (K), incapacitating injury (A), non-incapacitating- injury (B), and minor injury (C) crashes

** Standard Deviation ^ Crash rate has units of yearly crashes per million vehicle miles

Table A-3: Crash Characteristics for Tangent Portion of Roadway

Tangent Portion of Roadway (two or less Crashes per segment with barrier)					
Variable				Crash Characteristics KABC* Crashes	
				Condition	
			Total	With Barrier	Without Barrier
Total Sites			590	225	365
Sum of Segment Lengths (miles)			169.889	56.713	113.176
Median Related Crashes (MRC) in Five Years (1997 to 2001)	Per Segment	Average	1.27	2.50	0.52
		Minimum	0	0	0
		Maximum	16	16	14
		SD**	2.128	2.132	2.134
	All Segments	Sum	751	563	188
		Crash Count (crashes/mile-yr)	0.884	1.985	0.332
		Crash Rate (crashes/mvm)^	0.048	0.057	0.039
Total Sites			587	222	365
Sum of Segment Lengths (miles)			168.391	55.215	113.176
Median Crossover Crashes 1 (MCC1) in Five Years (1997 to 2001)	Per Segment	Average	0.09	0.11	48.54
		Minimum	0	0	0
		Maximum	2	2	2
		SD**	0.319	0.313	0.719
	All Segments	Sum	50	25	25
		Crash Count (crashes/mile-yr)	0.059	0.091	0.044
		Crash Rate (crashes/mvm)	0.003	0.003	0.005
Total Sites			575	210	365
Sum of Segment Lengths (miles)			165.522	52.346	113.176
Median Crossover Crashes 2 (MCC2) in Five Years (1997 to 2001)	Per Segment	Average	0.24	0.24	6.36
		Minimum	0	0	0
		Maximum	11	2	11
		SD**	0.426	0.537	1.192
	All Segments	Sum	139	50	89
		Crash Count (crashes/mile-yr)	0.168	0.191	0.157
		Crash Rate (crashes/mvm)	0.010	0.006	0.018

* Fatal (K), incapacitating injury (A), non-incapacitating- injury (B), and minor injury (C) crashes

** Standard Deviation ^ Crash rate has units of yearly crashes per million vehicle miles

APPENDIX B [ANOVA ANALYSIS]

Table B-1: Analysis of Variance (ANOVA) for Median Related Crashes (MRC) With-Barrier

Median Related Crashes With-Barrier Analysis

Summary Statistics

	HC	TAN	Total
T	15.88882	15.12637	31.01519
N	225	225	450
X Bar	0.07062	0.06723	0.06892
s^2	0.00473	0.00601	

CRASH RATE

Sigma I Sigma J **X i j**	31.01519	961.94207	2.13765
Sigma I Sigma J **X i j** Squared	4.545829		

(SS Treat)

SS Total	2.40818
SS Sites	0.001292
SS Error	2.406888
MS Sites	0.001292
MS Error	0.010793

Source of Variation	Sum of Squares	df	MS	F	p Value
Segments	0.001292	1	0.001292	0.11969	0.729631
Error	2.406888	448	0.005373		
Total	2.408180	449			α = .05
				c.v. = 3.84	

Summary Statistics

	HC	TAN	Total
T	2560.391335	2402.482034	4962.87337
N	225	225	450
X Bar	11.37951704	10.67769793	11.02860749
s^2	143.6909957	140.7005831	

Crashes per Mile

Sigma I Sigma J **X i j**	4962.873	24630112	54733.58
Sigma I Sigma J **X i j** Squared	118492.7		

(SS Treat)

SS Total	63759.13
SS Sites	55.41
SS Error	63703.71
MS Sites	55.41188
MS Error	142.1958

Source of Variation	Sum of Squares	df	MS	F	p Value
Segments	55.4	1	55.41188	0.389687	0.532827
Error	63703.7	448	142.1958		
Total	63759.1	449			α = .05
				c.v. = 3.84	

Table B-2: Analysis of Variance (ANOVA) for Median Related Crashes (MRC) Without-Barrier

Median Related Crashes Without-Barrier Analysis

Summary Statistics

	HC	TAN	Total	
T	23.78600732	16.04699916	39.83300647	**CRASH RATE**
N	365	365	730	
X Bar	0.065167143	0.043964381	0.054565762	
s^2	0.016474147	0.009042104		

Sigma I Sigma J **X i j**	39.83301	1586.668	2.173518
Sigma I Sigma J **X i j** Squared	11.54348		

SS Total	9.36996
SS Sites	0.082044
SS Error	9.287916
MS Sites	0.082044
MS Error	0.012758

Source of Variation	Sum of Squares	df	MS	F	p Value
Segments	0.001292	1	0.001292	6.430739	0.011424
Error	2.406888	728	0.003306		
Total	2.408180	729			α = .05
				c.v.=3.84	

Summary Statistics

	HC	TAN	Total	**Crashes per Mile**
T	821.8185888	603.7858326	1425.604421	
N	365	365	730	
X Bar	2.251557777	1.654207761	1.952882769	
s^2	14.24440257	10.69142609		

Sigma I Sigma J **X i j**	1425.604	2032348	2784.038
Sigma I Sigma J **X i j** Squared	11925.8		

	SS Total	9141.763
(SS Treat)	SS Sites	65.12
	SS Error	9076.642
	MS Sites	65.12094
	MS Error	12.46791

Source of Variation	Sum of Squares	df	MS	F	p Value
Segments	65.1	1	65.12094	5.223082	0.022577
Error	9076.6	728	12.46791		
Total	9141.8	729			α = .05
			c.v. =	3.84	

Table B-3: Analysis of Variance (ANOVA) for Median Crossover Crashes Type 1 (MCC1) Without-Barrier

Median Crossover Crashes Type 1 Without-Barrier Analysis

Summary Statistics

	HC	TAN	SUMMARY
T	4.507294	2.181582	6.688875
N	365	365	730
X Bar	0.01234875	0.005976936	0.009162843
s^2	0.002674097	0.001284613	

CRASH RATE

Sigma I Sigma J **X i j**	6.688875	44.74105	0.061289
Sigma I Sigma J **X i j** Squared	1.509669		

SS Total	1.44838
SS Sites	0.00741
SS Error	1.44097
MS Sites	0.00741
MS Error	0.001979

Source of Variation	Sum of Squares	df	MS	F	p Value
Segments	0.007410	1	0.00741	3.743393	0.053407
Error	1.440970	728	0.001979		
Total	1.448380	729			α = .05
			c.v. =	3.84	

Summary Statistics

	HC	TAN	
T	201.822891	76.65553681	278.4784278
N	365	365	730
X Bar	0.552939427	0.210015169	0.381477298
s^2	4.236219107	1.003697757	

Crashes per Mile

Sigma I Sigma J **X i j**	278.4784	77550.23	106.2332
Sigma I Sigma J **X i j** Squared	2035.024		

	SS Total	1928.791
(SS Treat)	SS Sites	21.46
	SS Error	1907.33
	MS Sites	21.46146
	MS Error	2.619958

Source of Variation	Sum of Squares	df	MS	F	p Value
Segments	21.46	1	21.46146	8.191527	0.00433
Error	1907.33	728	2.619958		
Total	1928.79	729			α = .05
			c.v. =	3.84	

Table B-4: Analysis of Variance (ANOVA) for Median Crossover Crashes Type 2 (MCC2) Without-Barrier

Median Crossover Crashes Type 2 Without-Barrier Analysis

Summary Statistics

	HC	TAN	SUMMARY
T	6.6490171	4.6384890	11.2875060
N	365	365	730
X Bar	0.018216485	0.012708189	0.015462337
s^2	0.003935894	0.001812514	

CRASH RATE

Sigma I Sigma J **X i j**	11.28751	127.4078	0.174531
Sigma I Sigma J **X i j** Squared	2.272489		

SS Total	2.097958
SS Sites	0.005537
SS Error	2.09242
MS Sites	0.005537
MS Error	0.002874

Source of Variation	Sum of Squares	df	MS	F	p Value
Segments	0.005537	1	0.005537	1.926548	0.165566
Error	2.092420	728	0.002874		
Total	2.097958	729			α = .05
			c.v. =	3.84	

Summary Statistics

	HC	TAN	
T	211.259026	255.176304	466.435330
N	365	365	730
X Bar	0.578791853	0.69911316	0.638952507
s^2	2.924479621	8.879708764	

Crashes per Mile

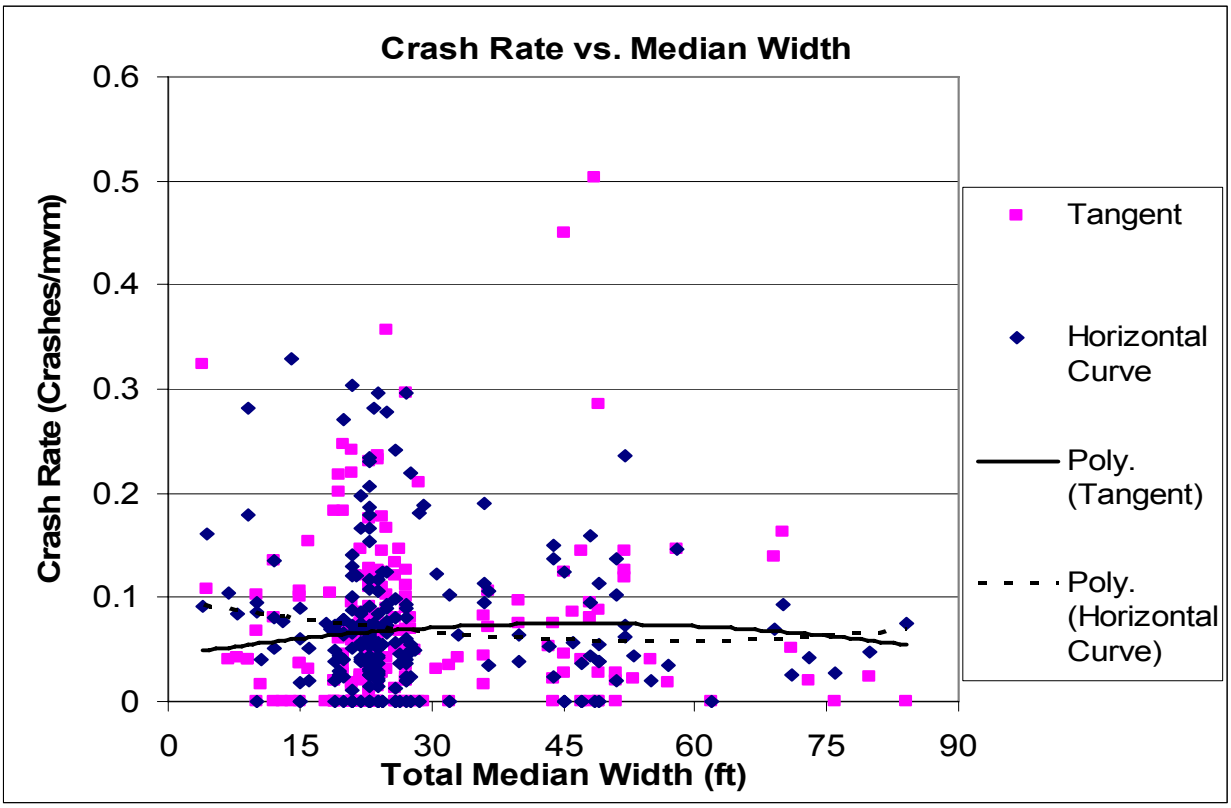
Sigma I Sigma J **X i j**	466.4353	217561.9	298.03
Sigma I Sigma J **X i j** Squared	4597.397		

(SS Treat)

SS Total	4299.367
SS Sites	2.64
SS Error	4296.725
MS Sites	2.642092
MS Error	5.902094

Source of Variation	Sum of Squares	df	MS	F	p Value
Segments	2.64	1	2.642092	0.447653	0.503689
Error	4296.72	728	5.902094		
Total	4299.37	729			α = .05
			c.v. =	3.84	

APPENDIX C [GRAPHS]



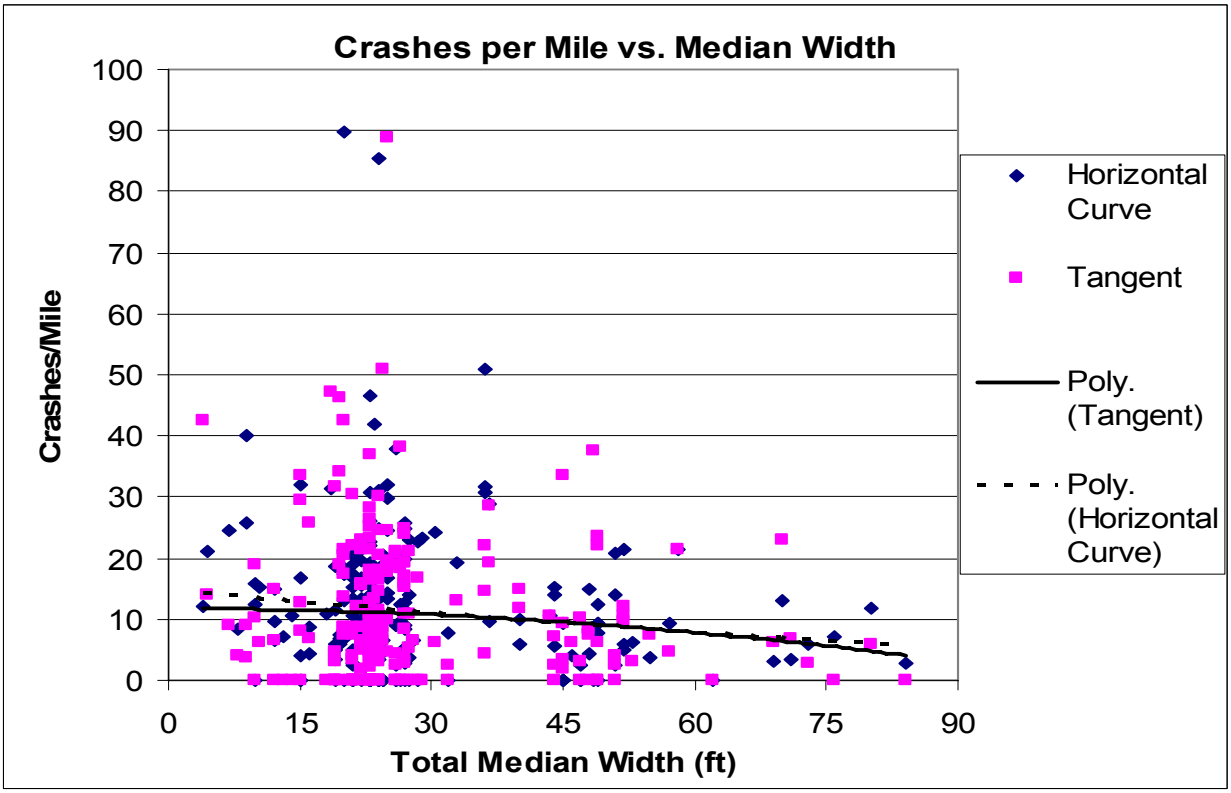
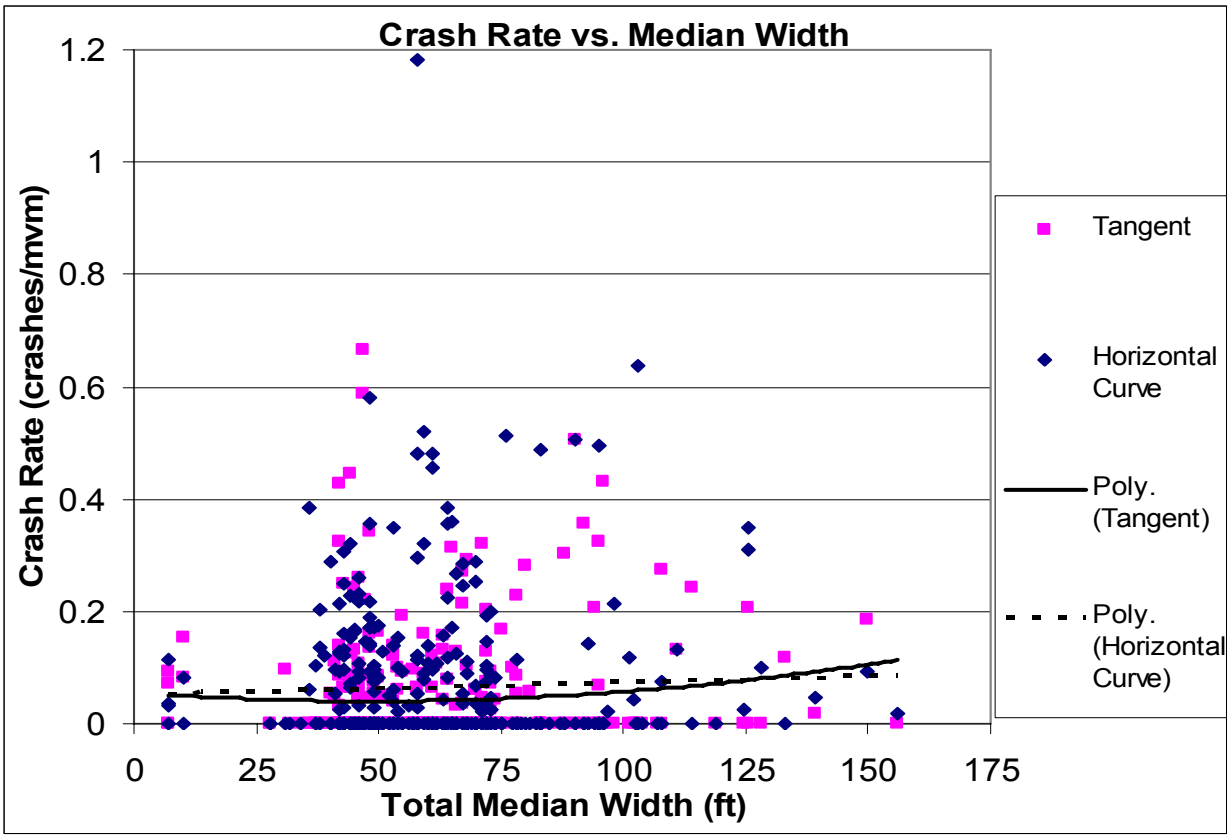


Figure C-1: Median Related Crashes With-Barrier Graphs



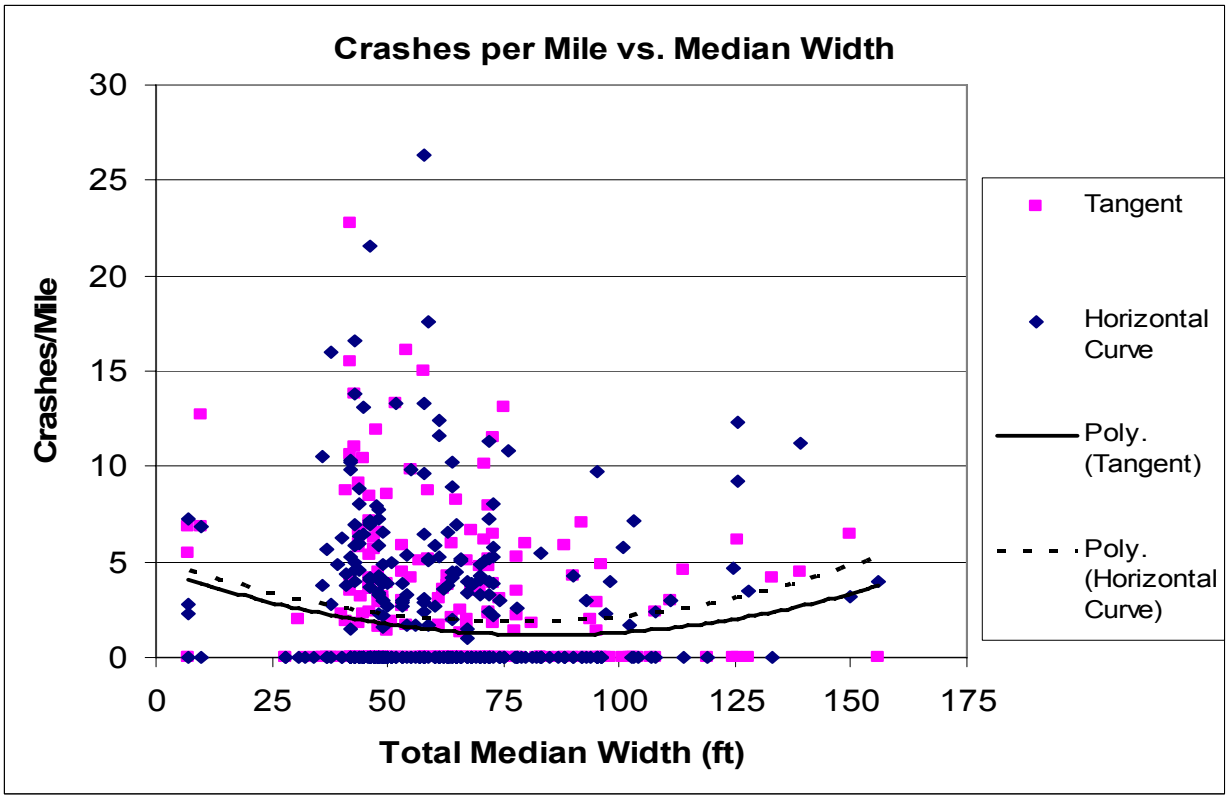
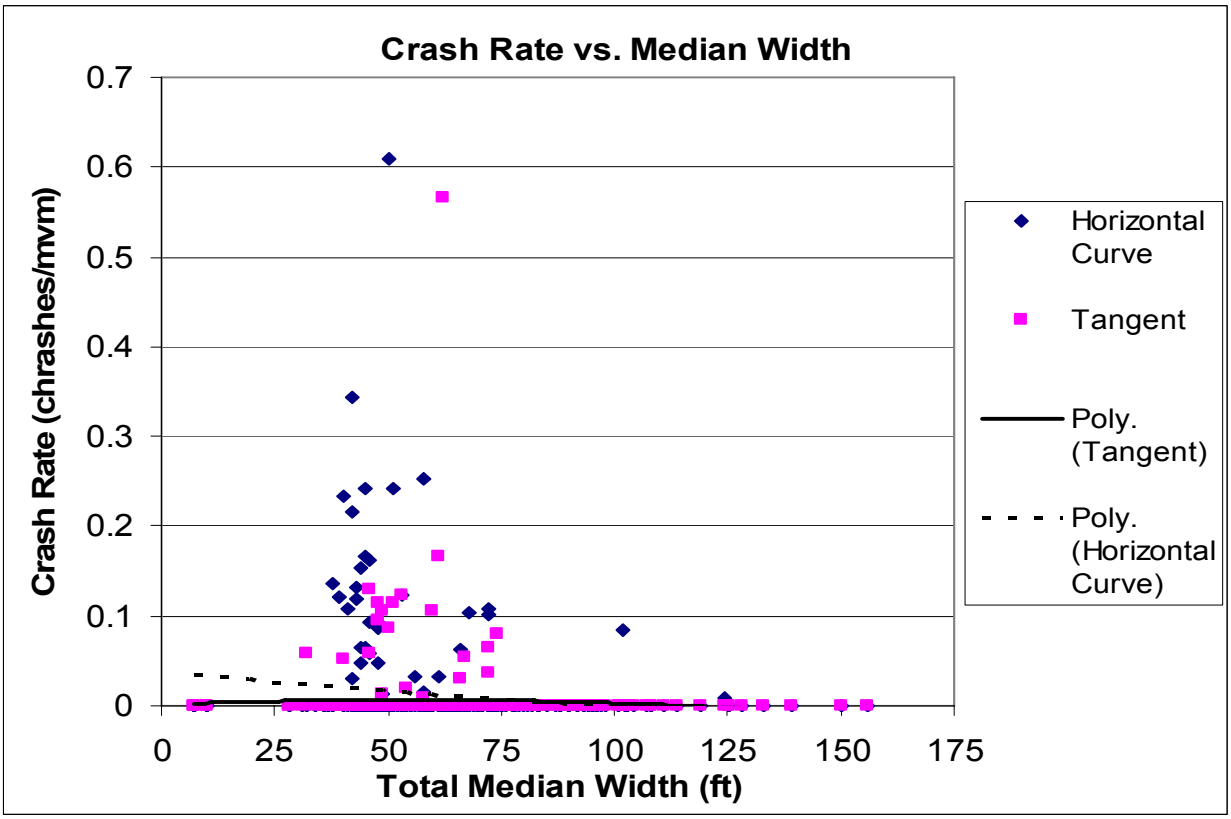


Figure C-2: Median Related Crashes Without-Barrier Graphs



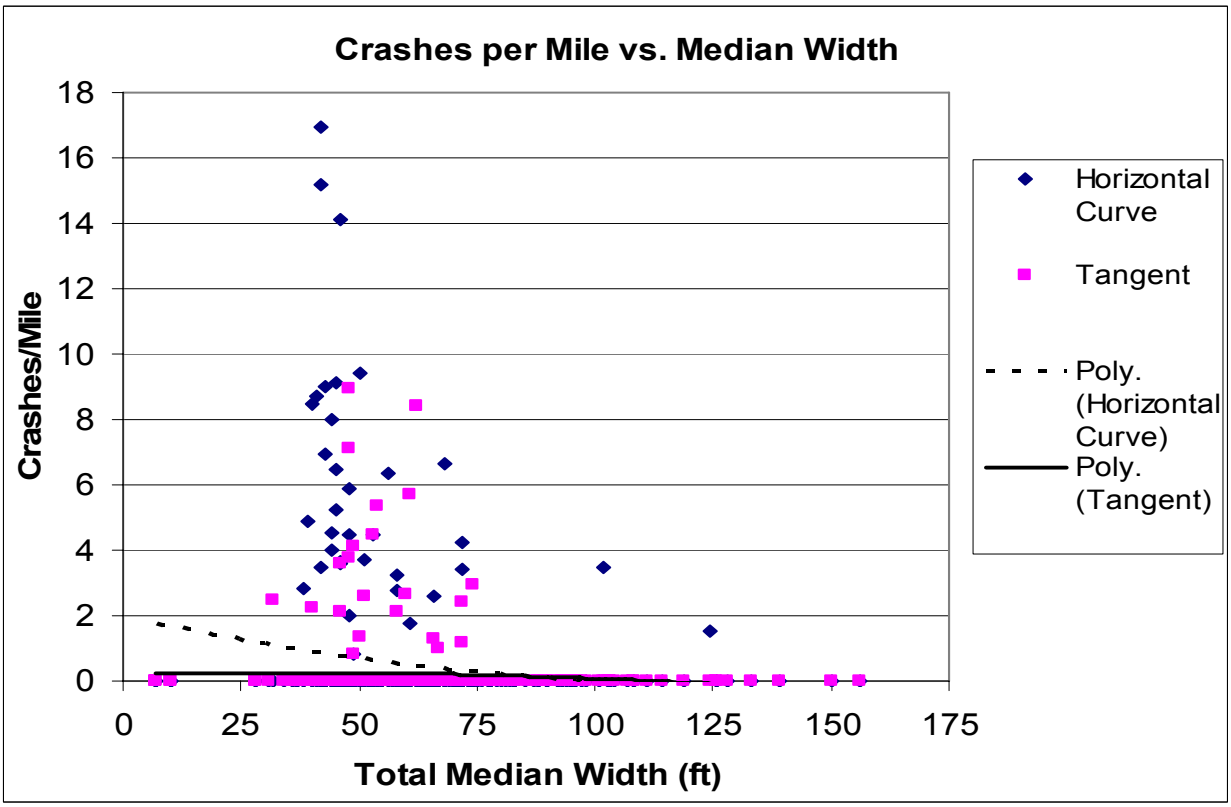
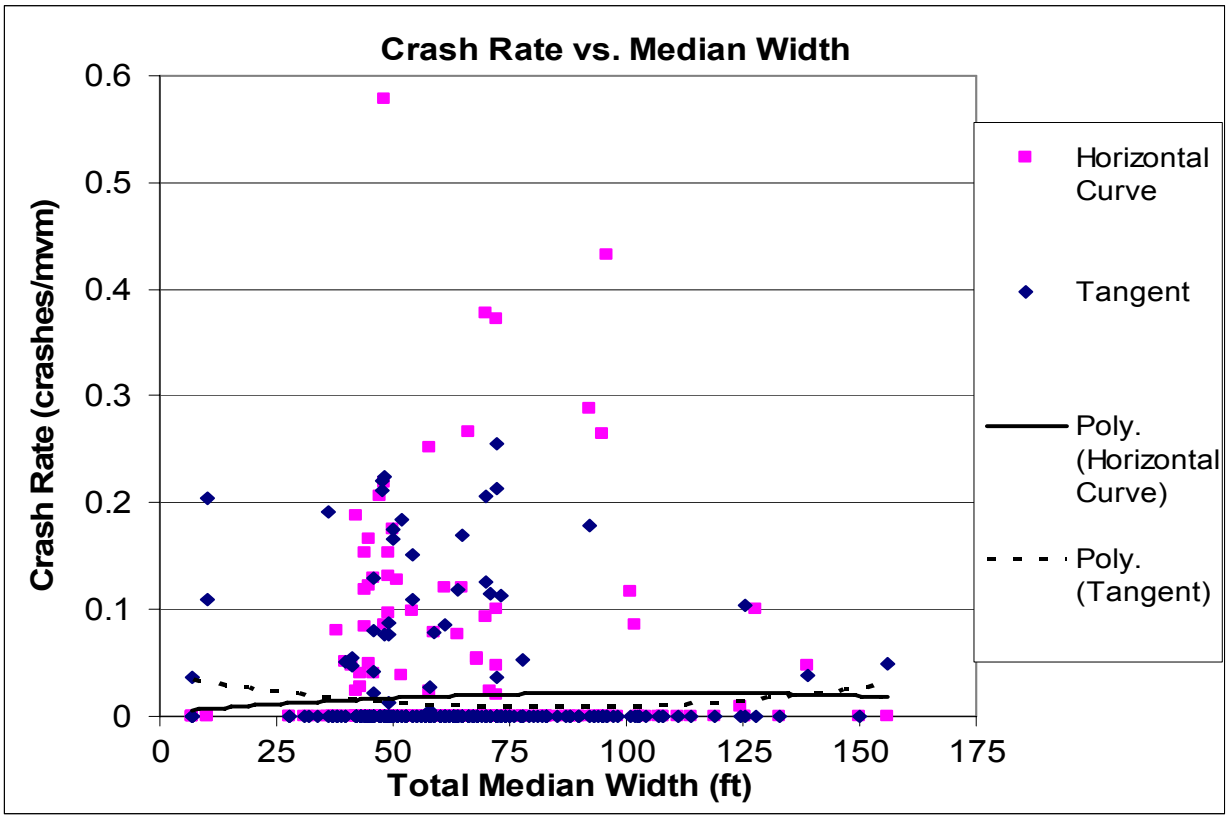


Figure C-3: Median Crossover Crashes Type 1 Without-Barrier Graphs



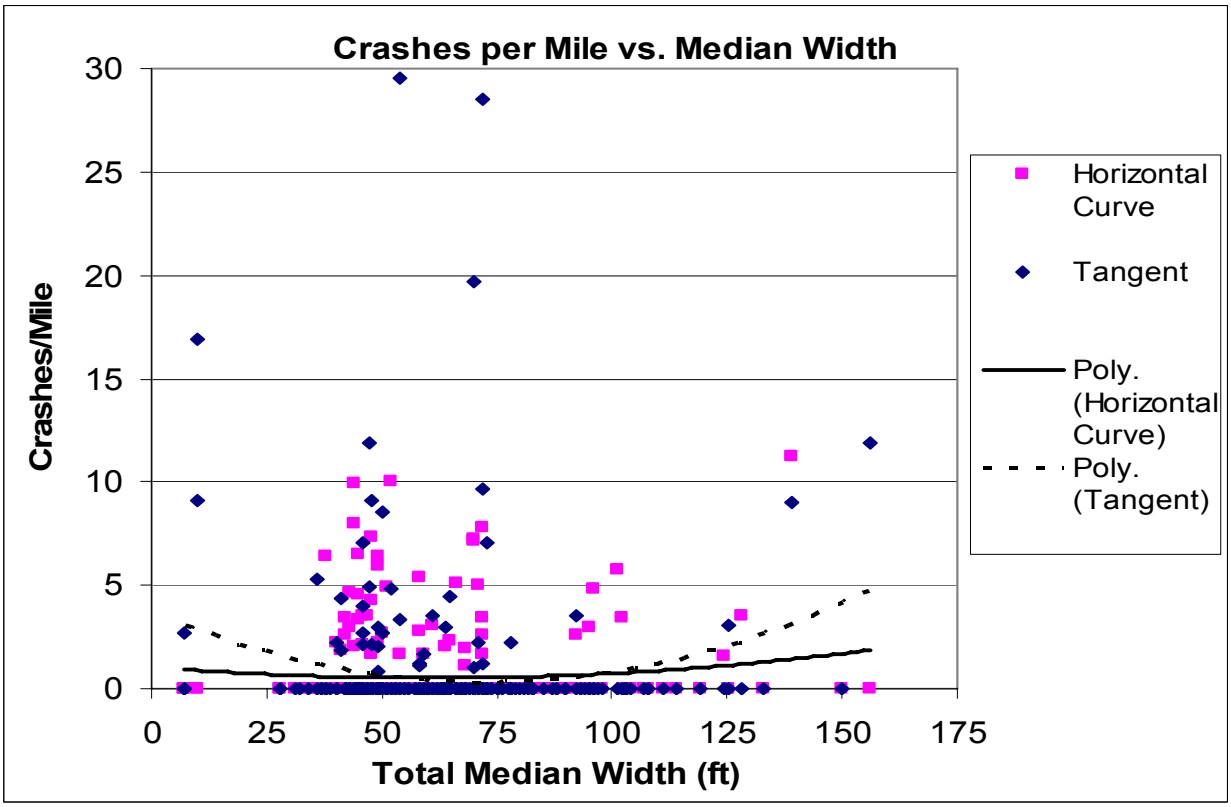


Figure C-4: Median Crossover Crashes Type 2 Without-Barrier Graphs

APPENDIX D [ROADWAY STATISTICS GRAPHS WITH-BARRIER]

Distribution of variables per number of miles (y-axis is in mile) for with-barrier segments in the freeway dataset graphical summary

[Range of variables are listed as x-y with x being inclusive and y being exclusive]

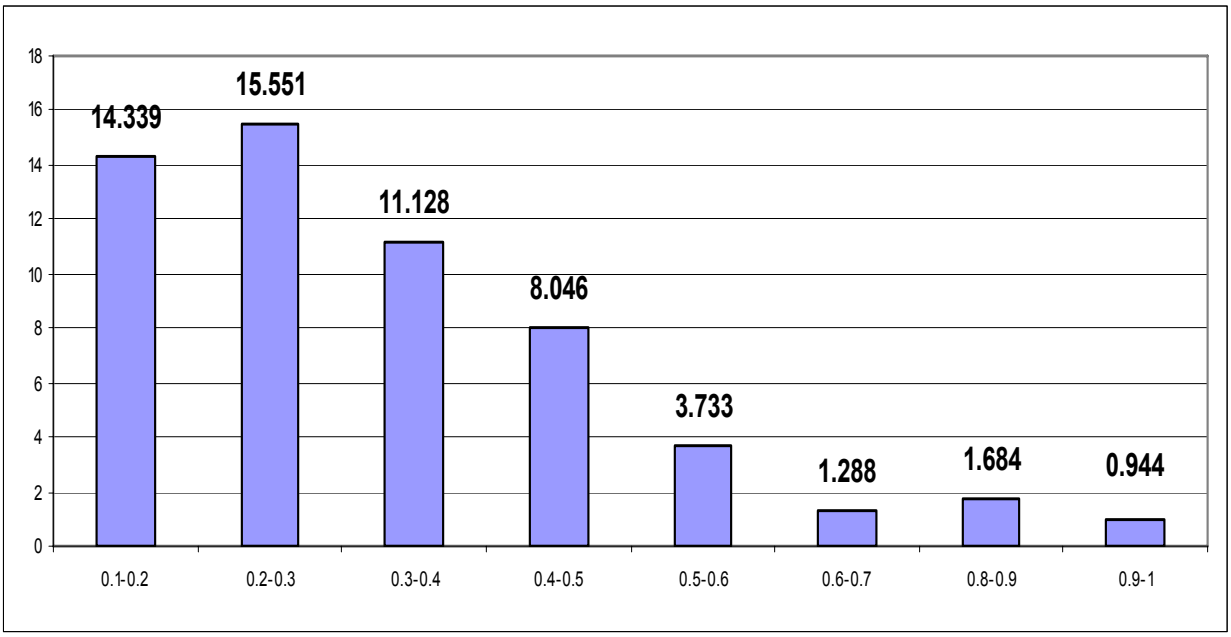


Figure D-1: Section Length (in Miles)

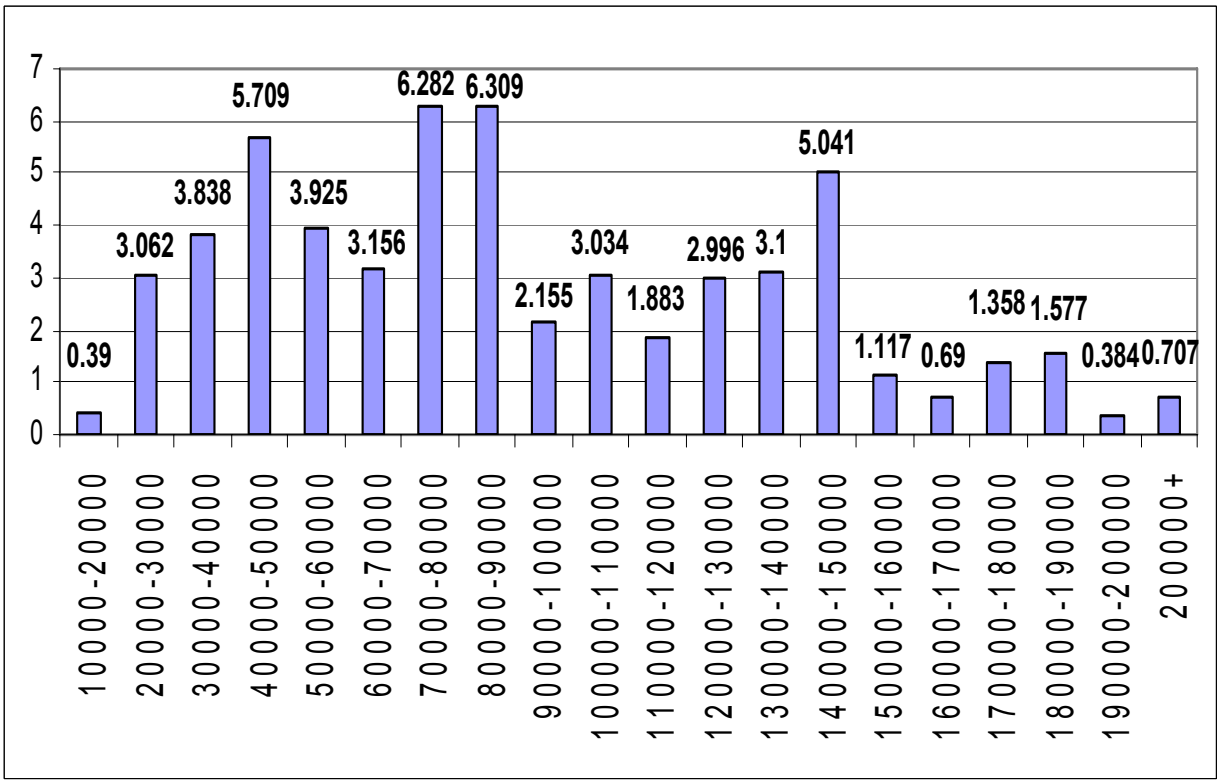


Figure D-2: Annual Average Daily Traffic (AADT)

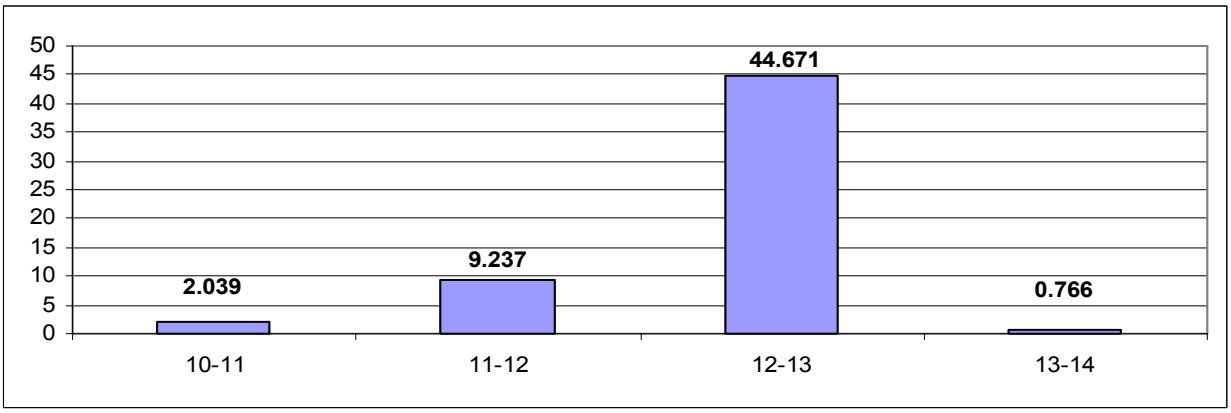


Figure D-3: Lane Width (ft) [Decreasing Milepoint]

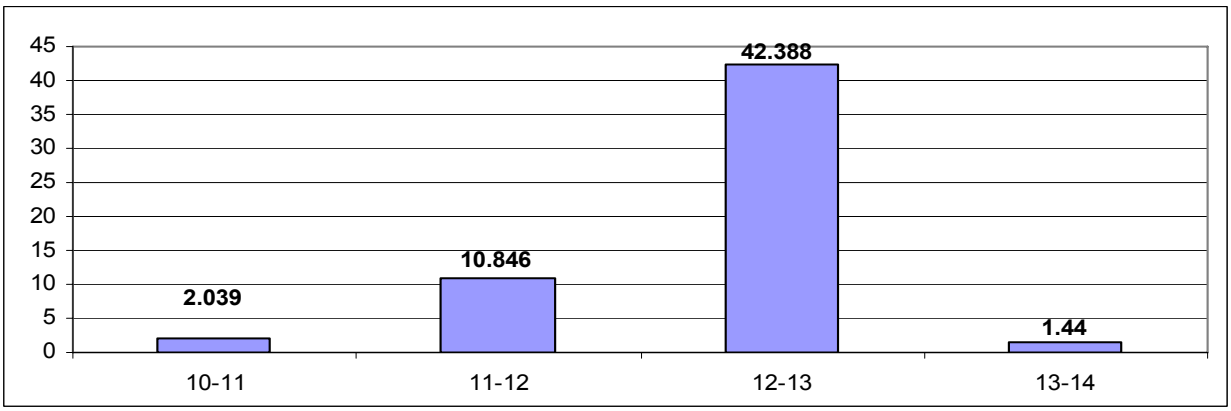


Figure D-4: Lane Width (ft) [Increasing Milepoint]

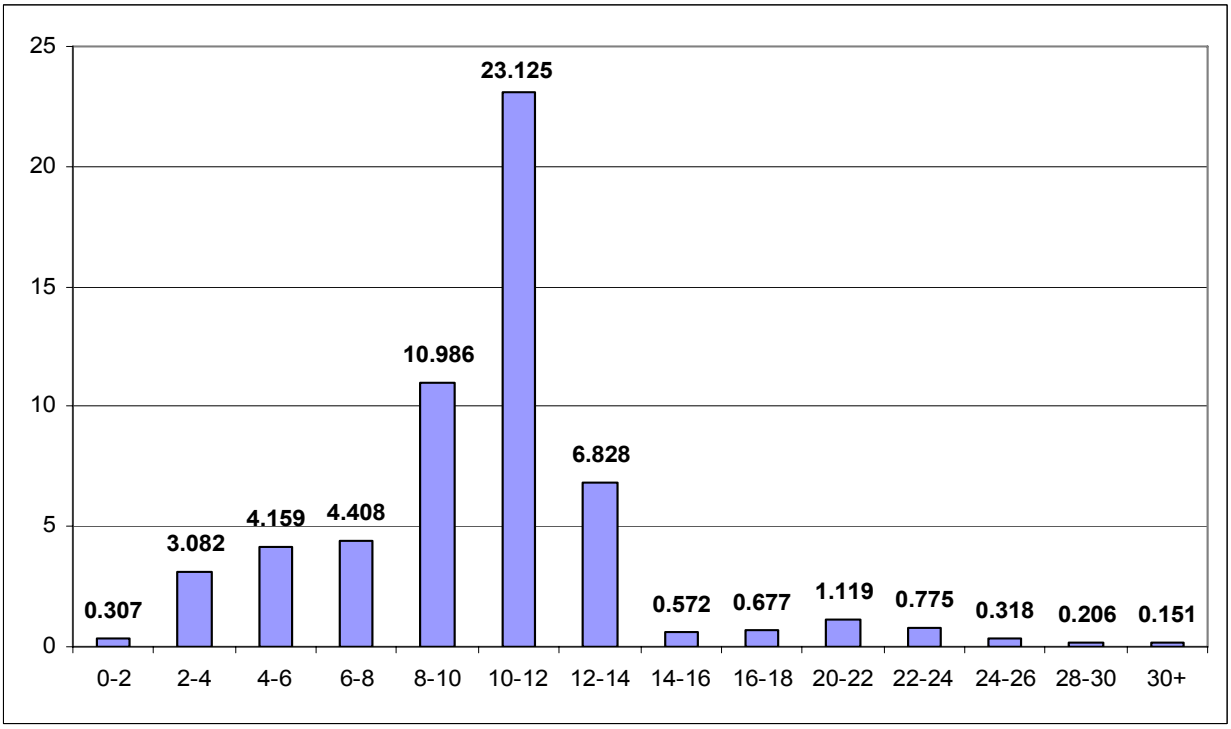


Figure D-5: Inside Shoulder Width (ft) [Increasing Milepoint]

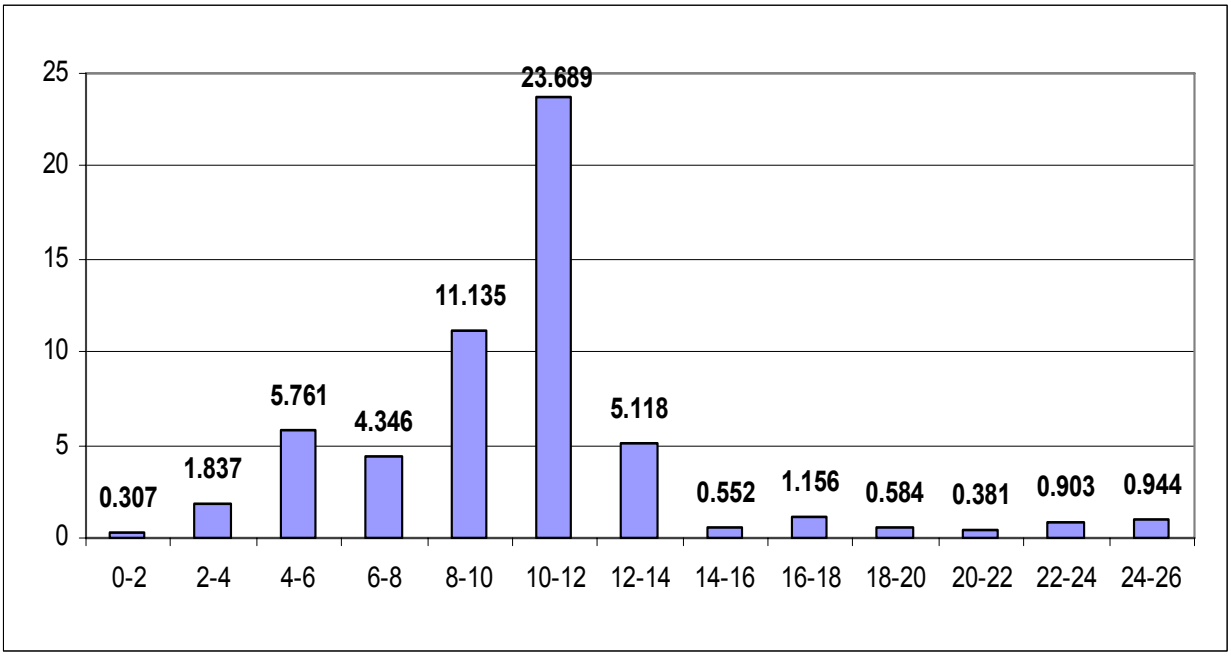


Figure D-6: Inside Shoulder Width (ft) [Decreasing Milepoint]

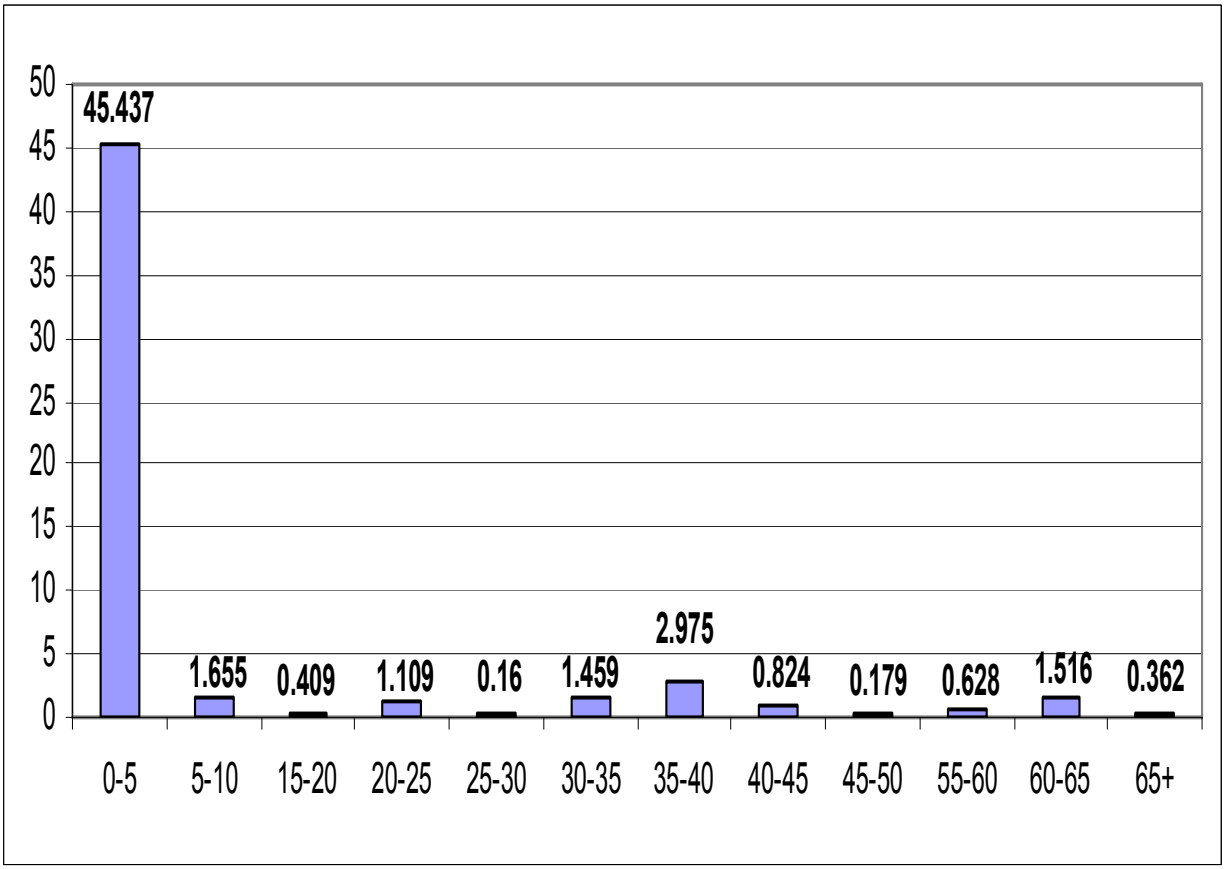


Figure D-7: Median Width (ft) [Not Including Inside Shoulders]

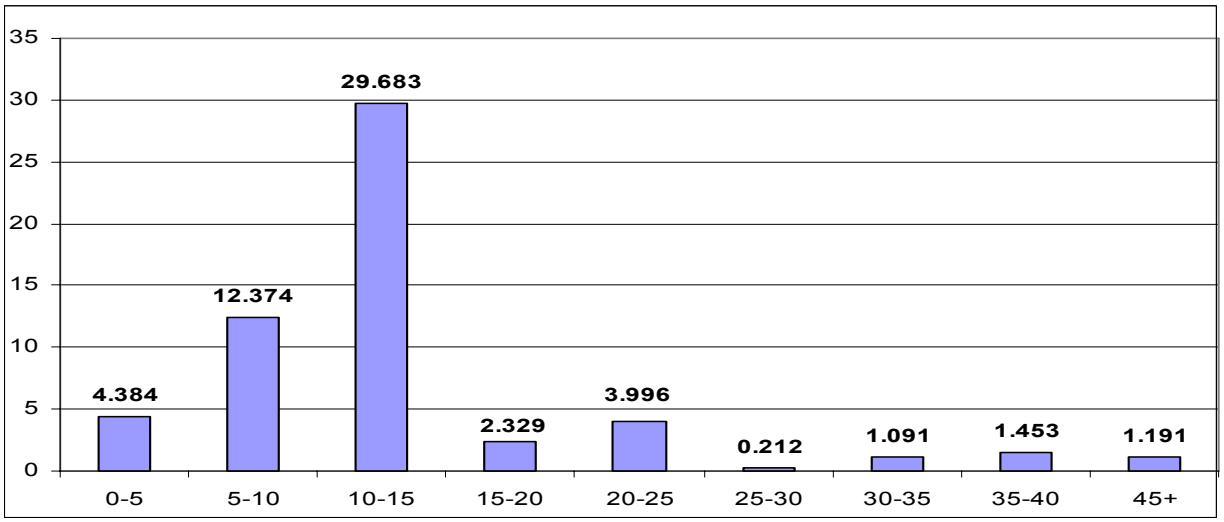


Figure D-8: Total Barrier Offset (ft) [Increasing Milepoint]

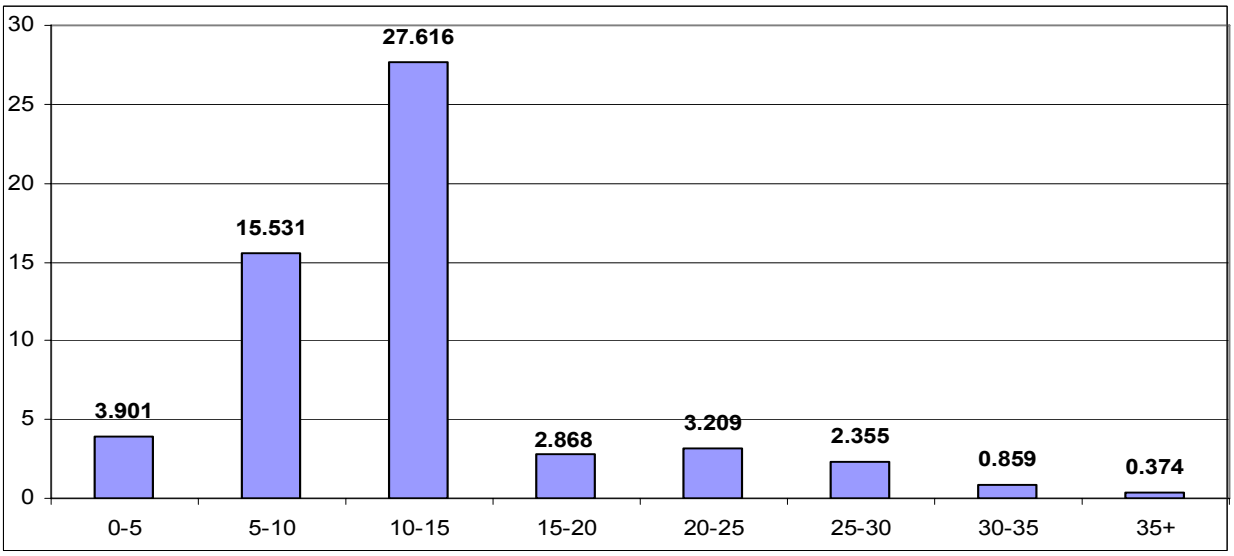


Figure D-9: Total Barrier Offset (ft) [Decreasing Milepoint]

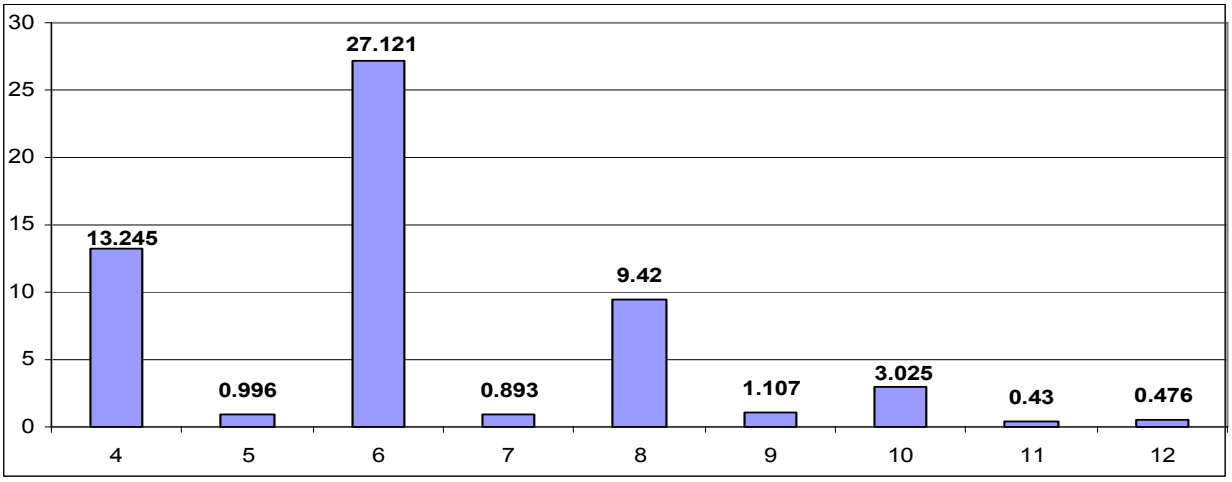


Figure D-10: Total Number of Lanes (Both Directions)

APPENDIX E [ROADWAY STATISTICS GRAPHS WITHOUT-BARRIER]

Distribution of variables per number of miles (y-axis is in mile) for without-barrier segments in the freeway dataset graphical summary

Range of variables are listed as x-y with x being inclusive and y being exclusive

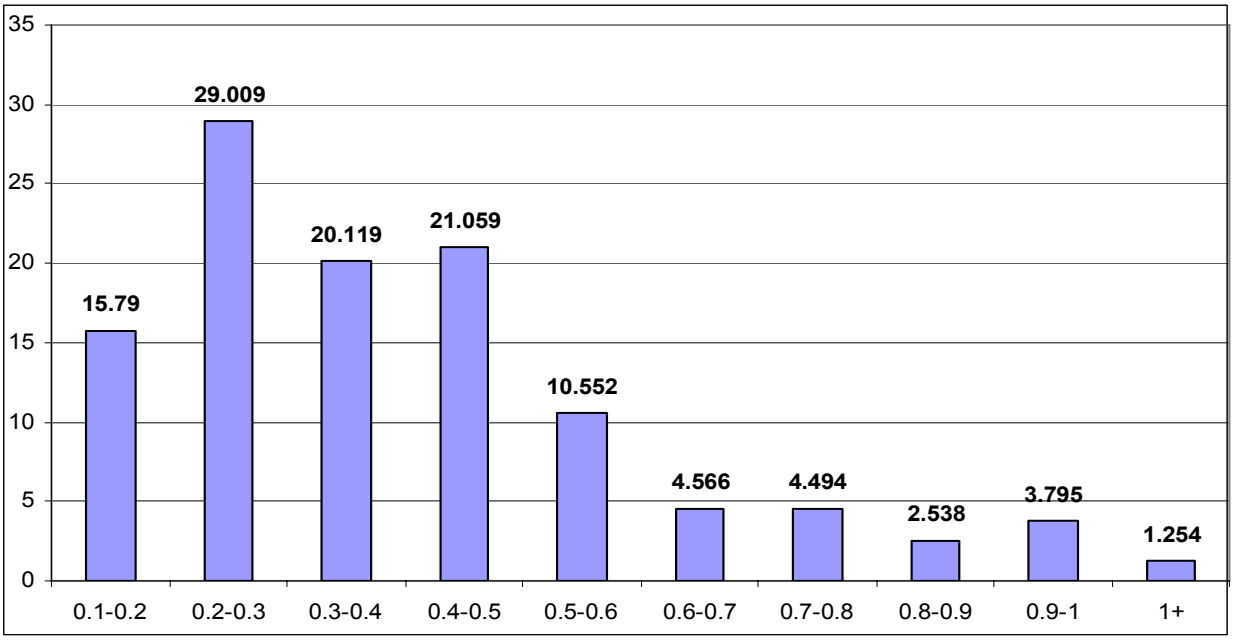


Figure E-1: Section Length (in Miles)

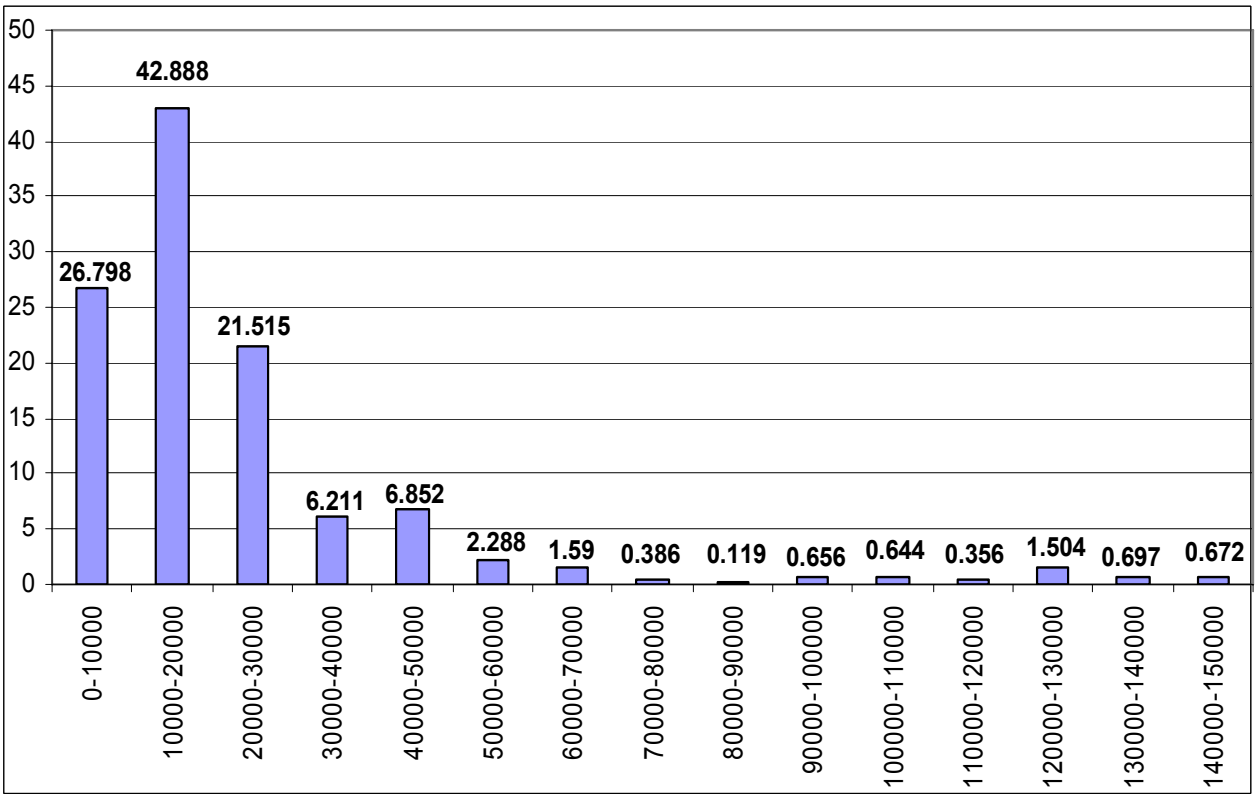


Figure E-2: Annual Average Daily Traffic (AADT)

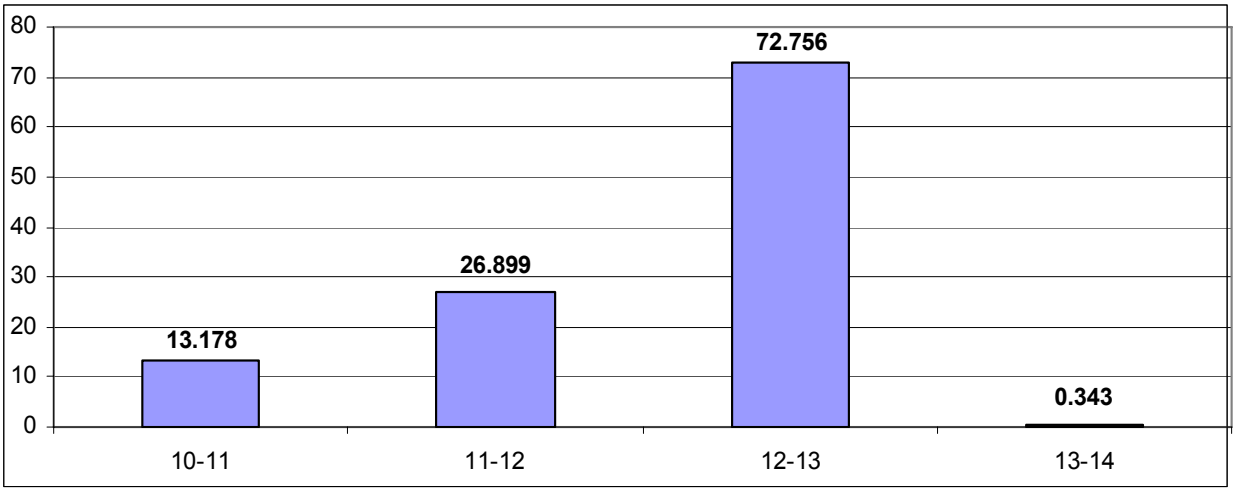


Figure E-3: Lane Width (ft) [Decreasing Milepoint]

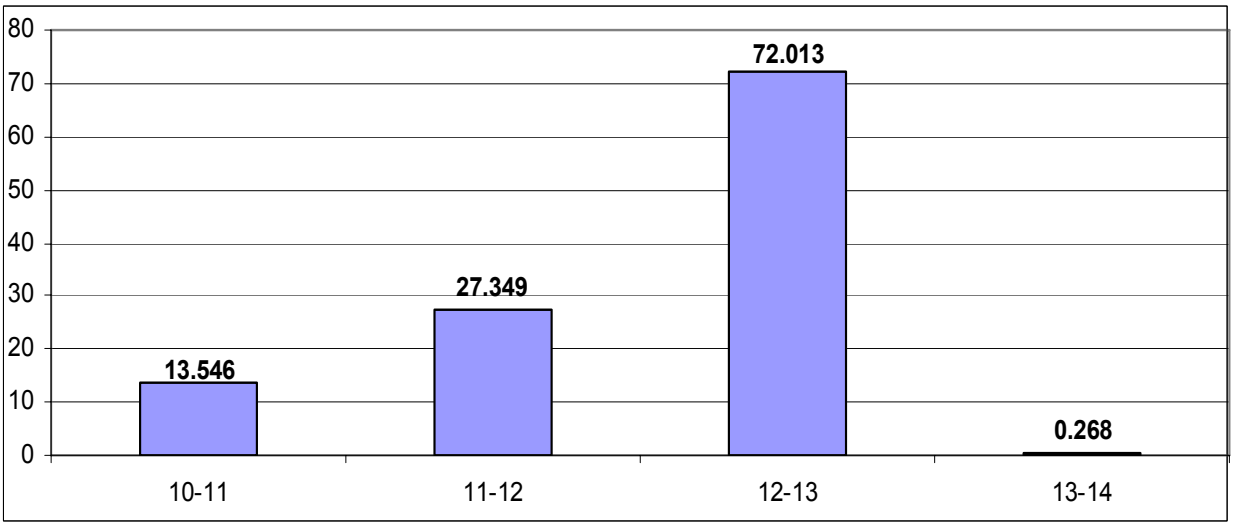


Figure E-4: Lane Width (ft) [Increasing Milepoint]

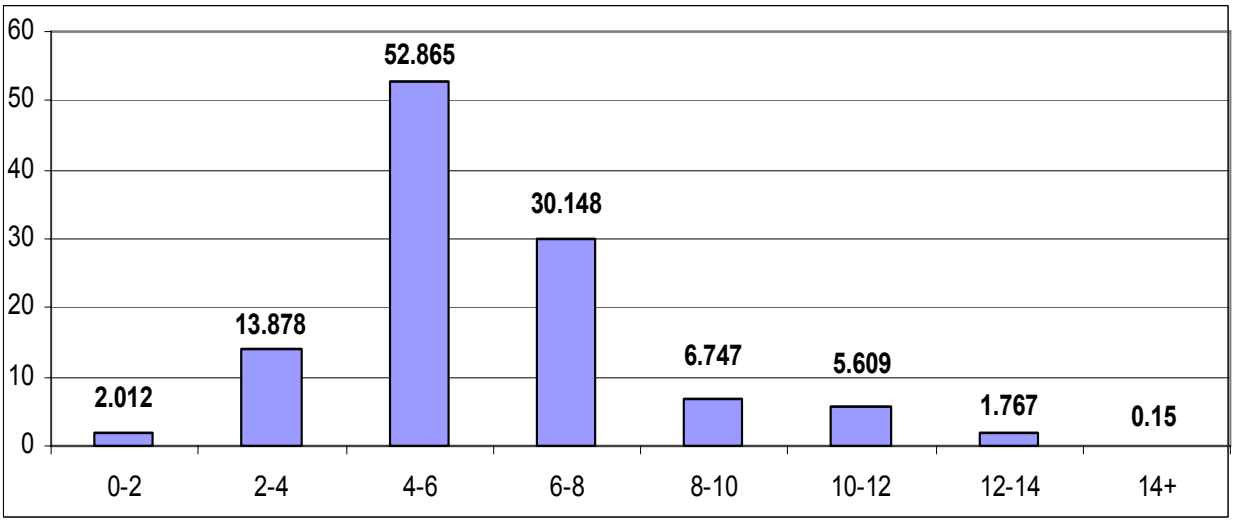


Figure E-5: Inside Shoulder Width (ft) [Increasing Milepoint]

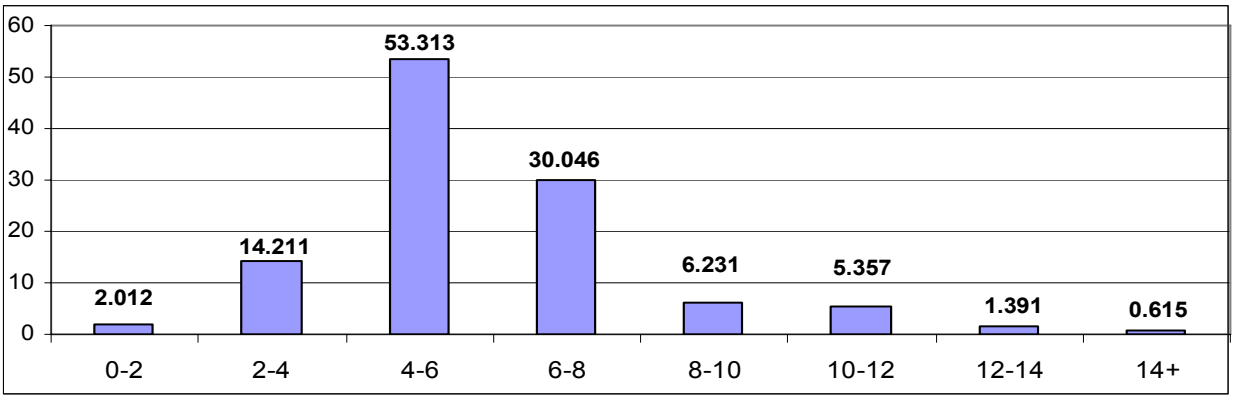


Figure E-6: Inside Shoulder Width (ft) [Decreasing Milepoint]

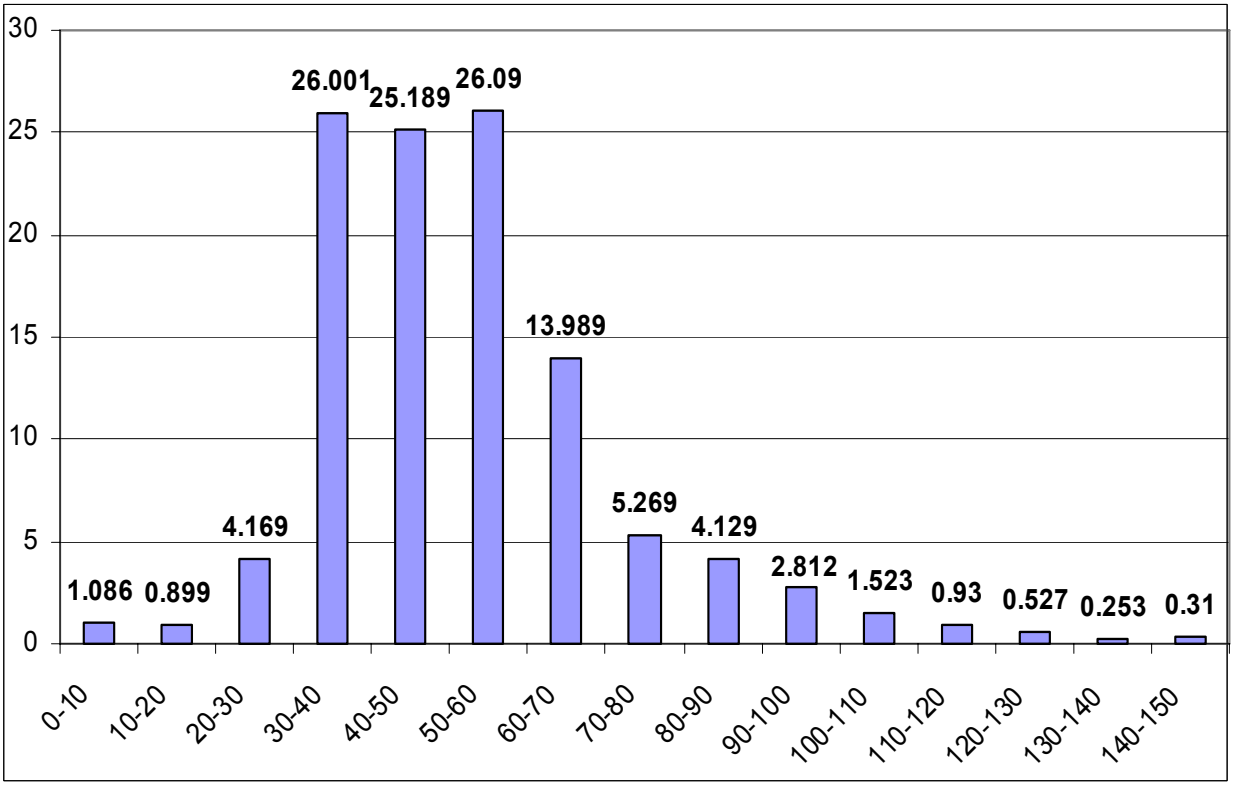


Figure E-7: Median Width (ft) [Not Including Inside Shoulders]

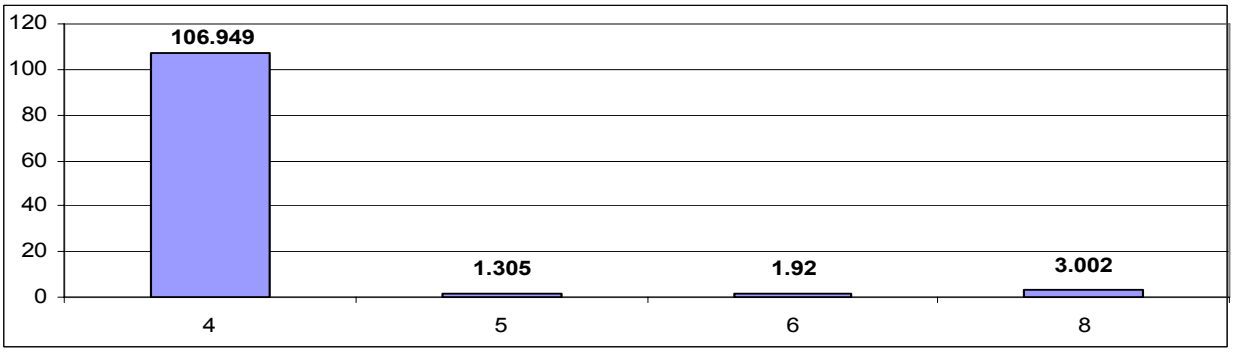


Figure E-8: Total Number of Lanes (Both Directions)

Driver Workload and Performance on an Interstate with Posted Speed Limits of 70 and 80 mph

Prepared for
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August 7, 2008

JAMES ROBERTSON

James Robertson is currently attending Michigan State University pursuing a Bachelor of Science in Civil Engineering. He plans to graduate in December 2008 and will pursue a career in transportation consulting or attend graduate school. Mr. Robertson received his first degree, a Bachelor of Arts in Psychology, from The University of Notre Dame in May 2007.

During his undergraduate career James devoted his free time to helping others through organizations such a Circle K International and the Knights of Columbus. Some of his fondest memories involve tutoring underserved students in the South Bend community and his participation in research on motivation in middle school math classrooms. During his last semester at Michigan State University James will serve as the lead tutor for the college of Engineering ROSES program, a community living option for first and second year undergraduate engineering students that provides support in all aspects of the college experience.

ACKNOWLEDGMENTS

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The author would like to express his special thanks to Marcus Brewer, Dillon Funkhouser, Kristin Landua and Jesse Stanly for their support in this endeavor.

SUMMARY

In this preliminary investigation of driver workload and visual capabilities at high speeds, fourteen subjects drove the Texas Transportation Institute instrumented vehicle along a section of interstate in west Texas where the speed limit changed from 70 to 80 mph. Four of the subjects were female, and the average age for the total was 48 years with the minimum of 20 and maximum of 71. A Dewetron 5000 computer synchronized three video cameras, sub meter GPS, lane tracking, and forward radar unit every tenth of a second. The forward viewing camera, driver viewing camera, and other synchronized devices were used to code the presence of vehicles around the instrumented vehicle along with driver non-forward glance rates and hand placement. These data were further reduced using computer macros to investigate the differences in lane geometry, vehicle velocity, lateral offset, driver non-forward glance rates, and driver

hand positions in the two speed conditions. Vehicles traveling in the left lane were found to offset further to the left than vehicle traveling in the right lane in the 70 mph condition. This same difference was not found in the 80 mph condition presenting the possibility in difference between the two speed conditions and driver's lane maintenance. No measured practical differences in roadway geometry, vehicle velocity, driver non-forward glance rates and driver hand placement between the two available speed conditions were found. Recommendations were developed for the next step in this two year study.

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INTRODUCTION

In Project 0-5544 Texas Department of Transportation (TxDOT) sought to develop design criteria for high speed facilities. High speed facilities were defined as 70 to 100 mile per hour. One of Project 0-5544's conclusions was the need for research to be conducted related to driver performance, workload, and visual capabilities at these speeds. TxDOT Project 0-5911 was created to investigate those question posed by Project 0-5544.

TxDOT Project 0-5911 is a two year study interested in driver workload and visual capabilities at high speeds. Due to a lack of collective knowledge of driver behavior at these speeds an expansive study is being conducted which includes a preliminary research stage. This preliminary research stage includes a test track following distance study, open road driving study, and driving simulator study. These three preliminary studies are being conducted to evaluate the most effective way for the second half of Project 0-5911 to be conducted.

This paper encompasses the reduction and analysis of the preliminary open road data collection study using the Texas Transportation Institute (TTI) instrumented vehicle. These data were collected as the instrumented vehicle drove from the test track study to study headquarters and back along a 70 mile stretch of interstate, which had a posted speed limit changing from 70 to 80 mph along its route. This section of interstate was selected due to its proximity to the test track and change in posted speed limit along the route. Specific questions to be answered by this portion of Project 0-5911 were:

1. Is vehicle lateral offset different for horizontal curves than tangent sections of the road at high speeds? Does lane of travel affect these values?
2. Is the lateral offset different between speed conditions? Does lane of travel affect these values?
3. What is the relationship between speed limits and actual velocity of vehicles at these speeds?
4. Is there a change in driver glance rates between speed conditions? Does lane of travel affect these values?
5. Does driver hand placement and number of hands on top of the steering wheel vary between speed conditions? Does lane of travel affect driver hand placement?
6. How can data collection methods be changed in the final study to provide relevant data more efficiently?

These questions were answered through the performance of a literature review and the development of data reduction methods to be used with the TTI instrumented vehicle.

LITERATURE REVIEW

In TxDOT Project 0-5544, design criteria were identified for design speeds of 85 to 100 mph (*1*). Engineering judgment and extrapolation were used in the absence of previous research on driver performance at these speeds (*1*). A specific area of interest was driver workload and visual acuity. Previous comprehensive research was conducted at speeds below 60 mph (*2*).

Driver Characteristics

According to the Virginia Tech Transportation Institute 100 car study, 93% of vehicle collisions and 68% of near collisions with lead a vehicle were found to have driver inattention as a contributing factor (*3*). Driver inattention encompassed secondary task engagement, fatigue, non-specific eye glances, and driving-related inattention to the forward roadway. Classical understanding only looked at secondary task engagement and fatigue. Task engagement and fatigue are part of a single construct referred to as driver arousal level (*4*). According to the Yerkes-Dodson law driver's quality of performance is based upon their arousal level. Driver performance level has been found to be poor at both low and high levels of arousal. Figure 1 shows the effect that arousal level has on quality of performance when conducting simple and complex tasks. This theory suggests simple tasks such as lane keeping are less affected by arousal than a complex task such as avoiding an unexpected object in the roadway.

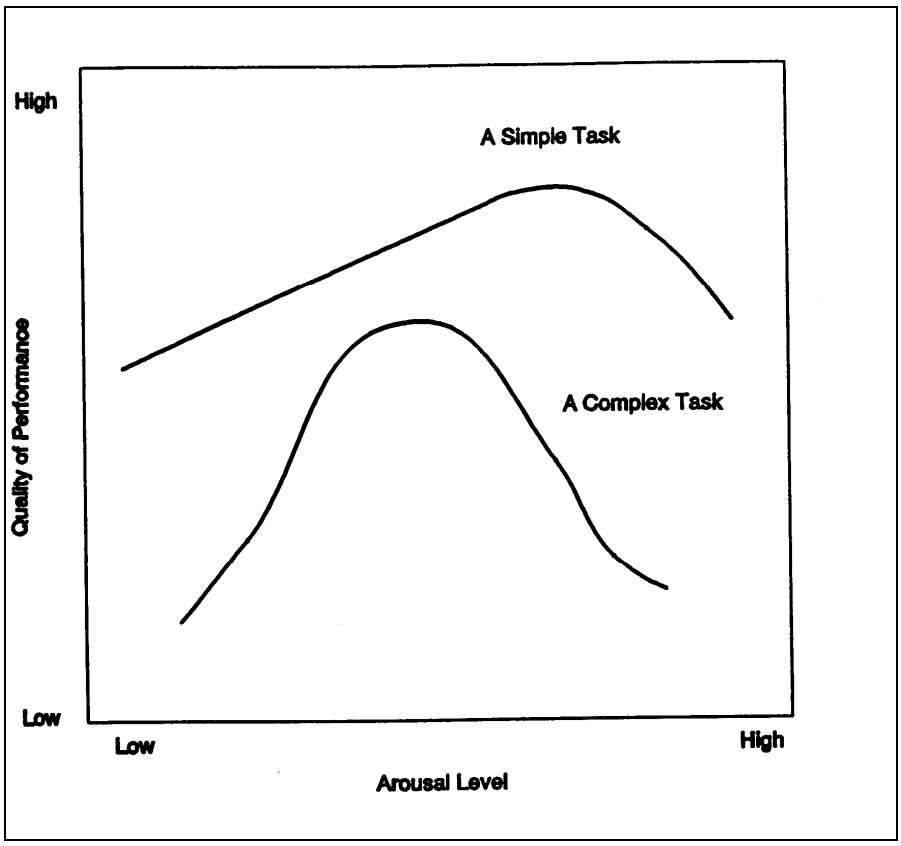

Figure 1. Effect on Task Performance as Arousal Level Increases

Simonov's (*5*) "Information Theory of Emotions" work describes a similar pattern when looking at workload. When experiencing extremely high or low levels of workload, performance declines. At intermediate levels performance is generally at an improved level. Hancock's illustration of this phenomenon could be considered more realistic, when considering driving, view of performance (*6*). Figure 2 shows Hancock's theory which includes a wide "comfort zone" that covers a workload range which is found during more normative driving. As the driver becomes more or less stressed their ability to adapt, perform, becomes compromised. Also illustrated is instability in the system at the extreme ends of stress level. This suggests it is

difficult to accurately predict how people will perform in hypo stressed and hyper stressed situations. In the middle levels drivers can adjust physically and physiologically to deal with stress so it does not affect their performance.

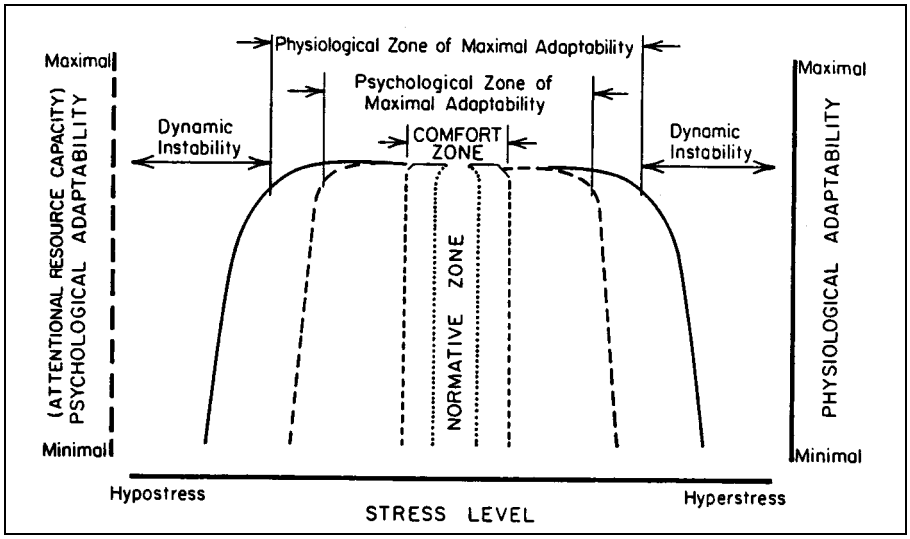

Figure 2. Physiological and Psychological Adaptability as a Function of Stress

Mental Workload

The primary method for measuring driver stress, fatigue, engagement, and arousal level is through the construct of mental workload (*7*). Mental workload refers to the amount of effort and limited processing capacity required to interpret all of the stimuli provided to perform necessary driving and non-driving tasks. Mental workload is primarily measured using subjective, performance, and psycho physiological methodologies.

Subjective Measures

These measures obtain a rating of mental workload based on the driver's assessment of the experience. Their scales are primarily defined by three different approaches. First, these assessments can occur immediately, retrospectively, or at both times. Second, subjective measures are either unidimensional or multidimensional. Third, subjective measures provide either an absolute or relative measurement.

The NASA-Task Loading Index (TLX) and the Subjective Workload Assessment Technique (SWAT) are widely used subjective assessments. The TLX and SWAT are measured immediately, are multidimensional, and provided an absolute rating. This type of measure is extensively used in a wide range of areas (*7*). The NASA TLX is widely used when assessing aircraft pilot mental workload (*8*).These measures are used because of to their ease of use, face validity, and driver acceptance.

Performance Measures

Performance measures use deterioration or erratic performance as an indication that workload is reaching an unacceptable level (*7*). This assumes that drivers have limited workload capacity and that as driver capacity is overloaded performance diminishes (*4*). Performance can be evaluated through the use of primary and secondary tasks. In the primary task method the driver's ability to perform the primary task of driving is measure directly (*7*). Using the secondary task method driver workload is measured indirectly through a secondary task. Drivers are instructed to focus on the primary task of driving and do the best they can on the secondary task. The driver's ability to perform the secondary task without jeopardizing the primary task performance provides a measure of the excess workload capacity remaining to the driver (*7*).

Secondary task measures are widely used in measuring driver workload. If driver performance on the primary task of driving were to falter then the testing environment would become dangerous. Through the use of secondary task performance measures a safe testing environment can be maintained. The use of performance measures can be intrusive and thus change the way the task of driving is performed. The use of word games and easy to reach secondary controls can elevate some of this intrusion.

With improvements in technology two types of performance measure are becoming more prevalent in the measurement of visual workload. These measures involve the use of eye tracking technologies and occlusion devises. These methods measure visual workload directly by looking at driver gaze patterns and visual need. These measures have been shown to be highly correlated with visual demand and the technologies making these measures possible have made the psychophysiological measure of blink rate more viable in the natural driving environment.

Psychophysiological Measures

Psychophysiological measures look at changes in driver physiology associated with cognitive task demand (*7*). Heart rate, blink rate, and brain activity have been used and associated with certain cognitive demands. Eye blink rate has been used extensively in driver workload measures of visual demand. Heart rate and brain activity have also been used while measuring workload in pilots.

Heart rate and brain activity measures have been shown to contain high variability, lag time, and the equipment is generally intrusive. This is to say the changes in heart rate and brain activity both tend to lag behind the actual increase in demand and there seems to be large viabilities between subjects making it difficult to measure. The equipment used in these measures can also interfere with the driver's ability to perform the task of driving, and in some instances raise the workload of participants not use to such equipment or uncomfortable with it.

Sensory Input

The five traditional senses are vision, hearing, smell, taste, and touch. Proprioception or kinesthesia describes driver's perception of body parts in relation to each other. Driver situational awareness is a function of these senses (*9*). For many years the visual sensory input

level was assumed to account for 90% of the sensory input received by the driver. This number has been shown to have no empirical validation yet is still widely accepted (*10*). The research does suggest that visual input is the most important sensory input to the driver but until empirical evidence is found the value of 90% should not be used (*10*).

Besides visual input hearing, touch, and proprioception are considered inputs that are used by drivers while performing the task of driving. These senses combine to provide feedback to the driver about vehicle performance and the driving environment. Drivers have high sensitivity to changes in this feedback whether it is tactile, auditory, or vibratory (*9*). While drivers have been shown to have high sensitivity to changes in feedback they do not seem to be aware of this sensitivity (*9*). It is believed advances in vehicle technology could inadvertently remove situational feedback without drivers compensating for this change.

Perception Reaction Time

Perception reaction time is the amount of time it takes for a driver to detect a danger, recognize it as danger, decide on a course of action, and begin to take action. These judgments are a function of perceived following distance, perceived time to contact, and driver experience (*11*, *12*). Fambro and his colleagues confirmed that a 2.5s minimum AASHTO stopping sight distance encompasses most of the driving population (*13*). This time includes both the Perception Reaction Time and the time for vehicles to slow to a stop. These studies were conducted at speeds of 55 mph or lower.

Perceived Risk and Risk Homoeostasis

Part of perception reaction time is the driver's judgment of a dangerous situation. The driver's judgment is a result of their recognition of the perceived risk. Kruysse suggests a majority of dangerous judgments are made at the onset of the conflict (*14*). Risk homoeostasis theory implies that drivers have a level of risk in their driving which they are willing to accept and that as technologies improve vehicle safety drivers will perform more dangers driving maneuvers (*15*).

While risk homoeostasis is under debate, Fuller suggests there are three basic uses of the term risk (*16*). According to Fuller risk has been used to describe objective risk, subjective risk, and a feeling of risk (*16*). Objective risk is the statistical probability of being involved in an accident. Subjective risk is the driver's estimate of the objective risk. The driver's feeling of risk is an emotional response to threat such as a feeling of anxiety. These assessments of risk have been related to driver's feeling of control. This feeling of control is assumed to be the inverse of the difference between the demanded ability and capability of the driver (*16*). Thus, as demand increases or capability decreases the feeling of control decreases. This feeling of control is associated with the perceived risk of the situation.

Driver Trait and State Factors

Driver trait factors are related to the driver in general and state factors are related to the particular driving experience in question. A strong correlation between reckless driving behavior

and the trait factors of aggressiveness and sensation seeking has been found (*17*). Angry drivers are also correlated with reckless driving behavior. Adolescent boys tend to have high ratings of sensation seeking, aggressiveness, and episodes of anger than adolescent girls (*17*). Driver age is another trait factor of concern. Drivers over the age of 65 have higher crash ratio's than drivers of other ages when performing left turns, gap acceptance, and lane changes (*18*).

Speed Adaptation

As driver's experience higher speeds for longer periods of time their estimation of lower speeds becomes less accurate (*19*). Schmidt and Tiffin found that as vehicles traveled at 70 mph for 20, 40, and 60 minutes total the driver's estimation of when the vehicle was traveling at 40 mph became less accurate. Drivers having traveled 70 mph for 20 minutes estimated 44.5 mph as 40mph, those going for 40 minutes estimated 50.5 mph as 40 mph, and those going 60 minutes estimated 53.4 mph as 40 mph (*19*). Speed adaptation is important to consider in the design of higher speed facilities because the effects at those speeds are unknown.

Driver -Vehicle Interaction

The modern vehicle has many components that compete for driver attention. Radios have been in vehicles for quite some time but recently MP3 players, blue tooth technology, navigation devices, and other intelligent transportation systems are beginning to find their way into the cab. These devices compete with other components of the vehicle, steering wheel and feedback instrumentation, necessary for proper vehicle operations. Vehicle size and driver placement within the cab also affect driver's ability to perform the primary task of driving such as maintaining proper following distance.

Vision Occlusion

Driver's judgment of following distance is based on the amount of road visible in front of the vehicle (*11*). The amount of road visible is affected by the size of the following vehicle's hood and driver eye height. One study looked at lead vehicles affect on this perception and no difference was found. Raising the driver's eye height thus allowing them to view more of the roadway did increase subjects' rating of following distance.

While larger vehicles may be on the conservative side of following distances this is not true of their blind spots. Larger vehicles have larger blind spots (*20*). Blind spots are locations around the vehicle that the driver cannot view with the use of mirrors and must change their gaze pattern to detect the presence of a vehicle. Even when the driver looks, some vehicles create blind spots so large that detecting another vehicles presence may be difficult or impossible (*20*).

Driver Hand Placement

Modern vehicles also contain more advanced devices for driver comfort such as electronically controlled mirrors, and seats. Many steering wheels now contain radio controls along with the cruise controls. While many of these features are intended to make driving more enjoyable, they are still competing for driver attention. The steering wheel is the primary method for vehicle

trajectory control (*21*). A hand position of ten and two has been assumed to provide maximum vehicle controllability.

Survey data confirmed that drivers perceive a hand position of ten and two as providing more vehicle control than other hand positions (*22*). In a study comparing perceived risk and driver hand placement it was found that in situations where there is a higher perceived risk drivers place more hands on the top of the steering wheel (*22*). There does seem to be a difference between observed and reported typical hand placement. Drivers tend to overestimate themselves having more hands on top of the steering wheel.

Driver-Vehicle Interaction with the Driving Environment

As mentioned previously drivers primary method for interacting with the driving environment is through vision. Vision occurs through windows in the vehicle and the mirrors contained on the outside. Blind spots are dangerous points surrounding the vehicle where other vehicles are difficult to detect. Through the use of frequent mirror checks surrounding vehicles can be tracked and their presence in blind spots can be more easily discovered. On the German autobahn the use of the rearview mirror is strongly encouraged due to the approach of vehicles traveling at high velocities (*23*). Drivers of the autobahn are also encouraged to perform passing maneuvers quickly to limit their time next to other vehicles and passing on the right is never allowed (*23*). The right side of a vehicle has the largest blind spot (*20*).

Mirror Check and Gaze Patterns

Mirror checks are only a portion of a driver's gaze pattern. Driver gaze patterns involve all the objects inside and outside the vehicle that the driver fixates upon. Fewer mirror checks and longer fixations are often found in instances of higher workload (*24*,*25*,*26*,*27*). A report conducted for Transport Canada found the drivers spend 78% of their time looking forward when performing no cognitive tasks. As tasks of increasing difficulty were added the percentage of time drivers spend looking forward increased (*24*). In the same conditions the percentage of time checking mirrors decreased indicating a drop in this secondary task as cognitive demand increases. In another study it was found that as vehicle speed increases driver gaze patterns become narrower indicating an increase in driver workload (*27*).

Perception of Forward Objects

Forward objects are any obstacles that present themselves in front of the driver. The first step in avoiding a collision is to recognize the presence of an object, first step in perception reaction time. These objects must be detectable in both daylight and night time conditions. In most cases this object is another vehicle (*28*). Non-object characteristics of the roadway such as signs and curvatures also need to be recognizable to the driver (*29*, *30*, *31*, *32*). The number of cues available to drivers at night is relatively small in relation to the amount available during the day (*29*). During the nighttime hours, sign and pavement marking reflectivity are important for they provide the primary cues for roadway curvature and direction of drivers (*29*).

Visual Demand

Visual demand is a portion of driver workload related to the amount of time a driver must fixate on an object or objects. Some roadway geometries require a longer fixation and thus increase visual demand (*30*,*31*,*32*). There is an inverse relationship between curve radius and visual demand (*32*).

Sight Distance

As noted in the AASHTO *Green Book* (*33*), a driver's ability to see ahead is of the utmost importance in the safe and efficient operation of vehicle on the highway. Sight distance is the length of the roadway ahead visible to the driver. This distance should be sufficiently long to enable a vehicle traveling at or near the design speed to stop before reaching a stationary object in the path of travel. Stopping sight distance includes the distance travel by the vehicle during perception reaction time and the distance required by the vehicle to come to a complete stop. The 2004 *Green Book* uses values for driver eye height and object height identified in a mid-1990s study (*34*). Horizontal and vertical curve designs are based on these driver eye heights and object heights along with stopping sight distance.

Time-To-Contact Estimations

When driving on multilane freeways the most common object encountered by vehicles is other vehicles (*35*). Time-to-Contact estimation is the driver's assessment of the time it will take for their vehicle to contact the lead vehicle if their velocities remain constant. It has been shown that drivers underestimate their time to contact judgments and that as closing speeds increase these estimates become more accurate (*35*,*36*). These studies paced a subject driver closely following a lead vehicle which suddenly braked. In these situations, as opposed to studies of object avoidance, drivers chose to brake rather than performing a lane change maneuver. Given a greater amount of time to choose their method of avoidance drivers will exhibit uncertainty and take more time to make their decision (*35*,*36*).

DATA COLLECTION

All data collection procedures were approved by the Texas Transportation Institute Internal Review Board (IRB). All TTI personal involved in the collection and reduction of these data completed training required by the IRB for work with human subjects.

Subjects

Fourteen subjects participated in an initial study of driver workload and visual capabilities at 70mph and greater speeds. The subjects had an average age of 48 years with a minimum of 20 and maximum of 71. The average number of years with a driver's license was 33 and four of the fourteen subjects were women.

These subjects were part of a test track following distance study and also drove the instrumented vehicle to or from the test track. An experimenter was responsible for shuttling the subject in

whichever direction they were not responsible for driving. The intention was to have an equal number of subjects in both directions; however due to cancellations and no shows, ten subjects drove to the test track with only four driving the return trip.

The route chosen for the subjects between the test track and study headquarters was always the same and covered a stretch of interstate in west Texas were the posted speed limit changes from 70mph to 80mph in the west bound direction. When heading eastbound the speed changed from 80mph to 70mph in the same location

TTI Instrumented Vehicle

For the purpose of data collection the TTI Instrumented Vehicle was used. The Instrumented Vehicle is a Toyota Highlander equipped with multiple data collection systems synchronized by an onboard DEWE5000 made by Dewetron. This study used three synchronized camera views. Also synchronized with the cameras were an Assist Ware SafeTRAC device, Global Positioning System (GPS), Eaton Vorad Radar unit, and accelerometer. Data from each device were recorded every tenth of a second while subjects were operating the instrumented vehicle.

The available three camera views were out the front windshield, in cockpit, and of the driver's feet. These camera views can be seen in Figure 3. The Assist Ware SafeTRAC device collected data related to the instrumented vehicles position in the lane of travel. Lane potion was measured as offset from center of the lane of travel. Negative values are associated with an offset left of center and positive values an offset right of center.

Figure 3. Cockpit (Upper Right), Driver's Feet (Lower Left), and Forward (Lower Right) Camera Views

The Global Positioning System provided vehicle velocity and GPS coordinates with sub meter accuracy. The Eaton Vorad Radar provided distance and relative velocity for up to two different lead vehicles simultaneously. The radar had a maximum range error of 5% of the total +/- 3 feet and a maximum relative velocity error of 1% of the total +/- 0.2 mph.

DATA REDUCTION

Dewesoft version 6.3 Rev. 7 was used to view the synchronized video, GPS, radar, and lateral offset data. All data reduced and analyzed in this paper are from the open road driving between the test track and study headquarters. The primary focus in this study was driver workload and behavior at high speeds, therefore non-interstate data were not reduced or analyzed. Two video coding runs were conducted and computer macros were used to reduce the collected data. The following explains the video coding methods used and assumptions made during data reduction.

First Run Coding

The purpose of the first run coding was to determine the location of the Instrumented Vehicle on the interstate and the location of other vehicles in relation to the Instrumented Vehicle. The following are the event times recorded during the first run coding:

- Instrumented Vehicle Merges With The Interstate or Video Beginning
- Instrumented Vehicle Leaving the Interstate or Video Ending
- Other Vehicles around the Instrumented Vehicle
    - When neighboring vehicle appears in view
        - Vehicle Type
        - Vehicle Location
        - Vehicle Lane
    - When neighboring vehicle departs from view
- Instrumented Vehicle Lane Changes

If at any time one of the above events occurred all vehicles considered present at that time had their locations, lane, and type rerecorded. For example if a vehicle was already present and another vehicle became present, details about the first vehicle were recorded at the time the second vehicle arrived. In the same situation if the first vehicle left the presence of the instrumented vehicle details for the second vehicle were recorded at the time of the first vehicle's departure.

These event times do not cover the entire event but are only usable as reference markers. This was done so that future detailed analysis can be conducted on these data using consistent reference points. These coding methods were used to maintain high inter- and intra-rater reliability. What follows is a more detailed explanation of the codes used and event definitions.

Lane Location Codes

The same lane location codes were used for both the instrumented vehicle and non-instrumented vehicles.

- 1 = Vehicle located in the right lane
- 2 = Vehicle located in the left lane
- 3 = Vehicle performing a right to left lane change
- 4 = Vehicle performing a left to right lane change
- 5 = Vehicles located on an off or on ramp

Vehicle Type Codes

Vehicle codes only apply to non-instrumented vehicles.

- 1 = Motorcycle
- 2 = Car
- 3 = SUV or Van
- 4 = Pick-up Truck
- 5 = Flatbed truck
- 6 = RV
- 7 = Semi Truck
- 8 = Emergency Vehicle

Any Vehicle pulling a trailer other than a Semi Truck was given an additional coding of 1 in front of the number defined above. For example a motorcycle with a trailer was given a code of 11 and a car with a trailer a code of 12. All vehicles were given the coded they most closely resembled above unless it never came close enough to recognize. In these cases the vehicle received a vehicle code of 0.

Vehicle Location Codes

- 1 = Vehicle located in front of instrumented vehicle
- 2 = Vehicle located to the side of instrumented vehicle
- Vehicles behind the instrumented vehicle were undetectable and not coded

Instrumented Vehicle Merging With and Exiting the Interstate

The instrumented vehicle was considered as merging with the highway when the white line to the left of the vehicle was no longer visible through the forward viewing camera. An instance of this event can be seen in Figure 4. The first view shows the ramp gore line still present and the second view is the first instance where the line is no longer visible. The time where this second view occurs is the time recorded and all relevant event data were also be recorded such as the SUV present in front of the instrumented vehicle in Figure 4.

The instrumented vehicle leaving the interstate is coded similarly. When exiting the highway the time coded was the time when the end of the white line was no longer visible. Figure 5 shows this occurrence. The first frame is with the end of the gore markings still visible and the second is the first frame where it is no longer visible. The time where the second frame occurs was recorded along with any relevant data about other vehicles.

If these events do not occur then the first/last available synchronized video will be coded to mark what was occurring at the beginning/end of the available video.

Figure 4. Instrumented Vehicle Merging with Interstate

Figure 5. Instrumented Vehicle Exiting Interstate

Presence of Other Vehicles

Vehicles were considered present around the instrumented vehicle if they were visible through the driver side window, passenger side window, front window, or were picked up by the radar. The radar's maximum range is 500 ft.; however, accurate readings normally occurred within 450 ft. When the instrumented vehicle was not in the same lane as a leading vehicle there was a good chance, as the gap got smaller, that the radar would no longer register the lead vehicle. For this reason any vehicle within 160 ft, measured by the center skip line, was considered present even if the radar was not registering it.

Vehicles were defined as being present the first instance that any of the above occurred and times were also recorded when they changed locations around the instrumented vehicle. There are only three defined locations due to limited camera views.

- in front of the instrumented vehicle
- beside the instrumented vehicle (Left or Right)
- no longer present

Lane changes by other vehicles were not coded. If a vehicle changed lanes the change in location would not be coded until another event occurred. Vehicles present on on-ramps were only coded if the instrumented vehicle performed a lane changing maneuver or the vehicle on the on-ramp changed location around the instrumented vehicle. Vehicles were no longer considered present the first instant they were no longer visible through the passenger or driver side window or when they outdistanced the radar.

Figure 6 shows the instrumented vehicle passing another vehicle. In the left image we see the car in front of the instrumented vehicle. The instant the entire rear bumper was no longer visible to the front camera view that vehicle was considered to the side of the instrumented vehicle. The right image shows the car outside the passenger's side window. When no portion of the vehicle was present through that window the vehicle was no longer considered present.

Figure 6. Instrumented Vehicle Passing a Car

Vehicles approaching from the rear were considered first present when any portion of their vehicle was viewable through the passenger or driver side window. Vehicles are considered to be no longer to the side of the vehicle when the entire rear bumper is visible to the forward-viewing camera as seen in the left part of Figure 6. If this event occurred on the left side of the instrumented vehicle then the lane location for both the instrumented vehicle and vehicle present were given different codes but the qualifications for vehicle presence remained the same.

Lane Changing

Lane changes were only a coded event when the instrumented vehicle performed the lane change. Lane changes by vehicles around the instrumented vehicle were not coded. For example if a car passed the instrumented vehicle then changed lanes to be in front of the instrumented vehicle, this event was not coded. Non-instrumented vehicles changing lane during the coding of another event were given a lane location code for their destination lane. For example if a car moved from the left to right lane when another event occurred the vehicle changing lanes was given a lane location of right. During other coded events vehicles present have their location, lane of travel, and type recoded. At these times changes in lane by other vehicles would be accounted for.

Lane change time marks for the instrumented vehicle were obtained by placing the cursor at the highest or lowest peak that occurred during a lane change event. Figure 7 shows a lane change event demonstrated on the timeline by a spike in lateral offset and the cursor obtaining the time

of this spike. The upper timeline demonstrates a right-to-left lane change event and the bottom timeline demonstrates a left-to-right lane change. These spikes occur at the point the lane tracker determines the lane of travel by the instrumented vehicle has changed.

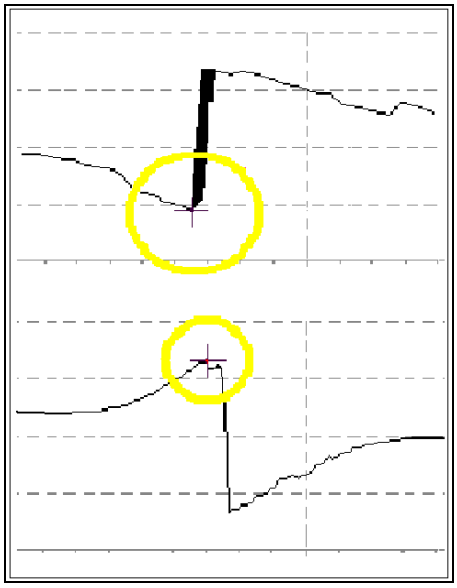

Figure 7. Lane Changes Shown by Lateral Offset on Timeline

Second Run Coding

The purpose of second run coding was to record the number and direction of non-forward glances made by drivers along with the location of their hands in 5 second intervals. This was a very time consuming endeavor and for this preliminary study only two 5 minute segments were coded. The segments were all tangents located in each of the two speed conditions allowing for comparison. These segments occurred approximately 15 minutes into each of the speed conditions.

Coding Glance Rate & Head Movement

Driver glance rates were primarily coded using driver head movement due to the video data quality. The coding of glance duration would have been difficult to achieve using the video available thus was not sought during this preliminary study. The direction of the head movement was coded from the driver's perspective and each glance was given only one direction. If a driver looked down and right it was given a code of down or right not both. The coded given was dependent on what the driver was perceived to be looking at. The following were the only movements recorded as glances during these coding runs:

- Glances at one of the three mirrors
- Glance at an object in the vehicle including other passengers
- Glances at the instrumentation panel or steering column. The only event coded as down.
- Blind spot checks

If a driver looks at the driver side mirror and then checks the blind spot, both events were coded even if their eyes never returned to a forward view between the two events. The same was true if a driver checks the rear view mirror and then proceeds to check the passenger side mirror in the same movement. In both cases if the coder can perceive two intentions by the driver than two events were coded.

In some instances drivers only moved their eyes and not their head. In the cases where these movements were visible they were given the proper coding that a head movement in that direction would be given. In the case of drivers with sun glasses, these types of coding were not possible.

Driver Hand Placement

Driver hand placement was recorded at the beginning and end of each five second segment. Each hand was coded separately and the following four codes were given:

- A driver's hand is considered on the upper half of the steering wheel if any portion of the hand is visible above the center of the steering wheel. The center of the steering wheel is defined by the top of the uppermost spoke connected to the hub. These events were given a code of (1).
- A driver's hand is given a code of (-1) if it is clearly visible to the driver view camera and is not grasping the steering wheel or another object.
- A driver's hand is given a code of (-2) if the hand is clearly visible to the driver view camera and it is grasping something other than the steering wheel.
- If a driver's hand does not meet any of the above criteria than it is given a coding of (0). A driver resting his/her hand on the shifter would be given such a code because the hand is not clearly visible to the available cameras.

Dewesoft Exports

The synchronized data from the Dewesoft program were exported into spreadsheets that contained all recorded data over the duration of the recording. These data from the coding were then incorporated into data files using the designed macros. Using the GPS coordinates from the file along with accelerometer data, the beginning and ending of horizontal curves were determined and everything in between was considered a tangent segment of roadway. These geometric data were incorporated into the spreadsheet files as well. The final spreadsheets in their raw form contained more than 60 variables and 40,000 synchronized data points.

Final Data Reduction

Final data reduction was conducted using macros that sorted these data files into separate worksheets by zone, lane, and geometry in order to develop exploratory results for this project in the form of charts and tables. If the Assist Ware SafeTRAC had a confidence of less than 90% those data points were eliminated from final analysis. These data were then made into five tables and nine charts exploring driver hand placement, lane position, and glace rates. To eliminate some bias due to lane changing events:

- Vehicles spending less than 30 seconds in a particular lane of travel and speed condition had their data for that segment removed before final analysis.
- Vehicles spending less than 15 seconds in a lane of travel within the zones were hand placement and eye coding took place had that section of data removed before final analysis.

These times were chosen because of the length of time a lane changing maneuver took and the proportion of the overall segment being analyzed. The results for this study are broken down into categories which correspond to the research questions being answered. All data are provided in both table and graph form.

Lateral Offset & Lane Geometry

Table 1 contains the lateral offset associated with the three geometric characteristic being broken down by lane of travel and speed zone. Table 1 also contains the standard deviation and total observations for each lateral offset. Figure 8 is a graphical representation of these data in Table 1. The first and last five minutes of observations were not included in these data. As noted previously negative values are offsets to the left of center in the lane of travel and positive values are to the right of center.

Table 1. Average Lateral Offset by Lane and Lane Geometry

	Left Lane			Right Lane		
Lane Geometry	Mean (in)	S.D. (in)	n	Mean (in)	S.D. (in)	n
70mph Left	-10.79	15.78	4828	-0.88	11.05	10776
80mph Left	-3.99	15.08	3581	-3.55	11.22	13786
70mph Tangent	-9.49	15.69	33871	-0.96	10.43	104826
80mph Tangent	-5.71	13.62	60910	-2.34	10.75	178481
70mph Right	-8.42	16.13	4839	1.67	13.09	11045
80mph Right	-5.88	17.28	4887	-0.69	13.56	15847

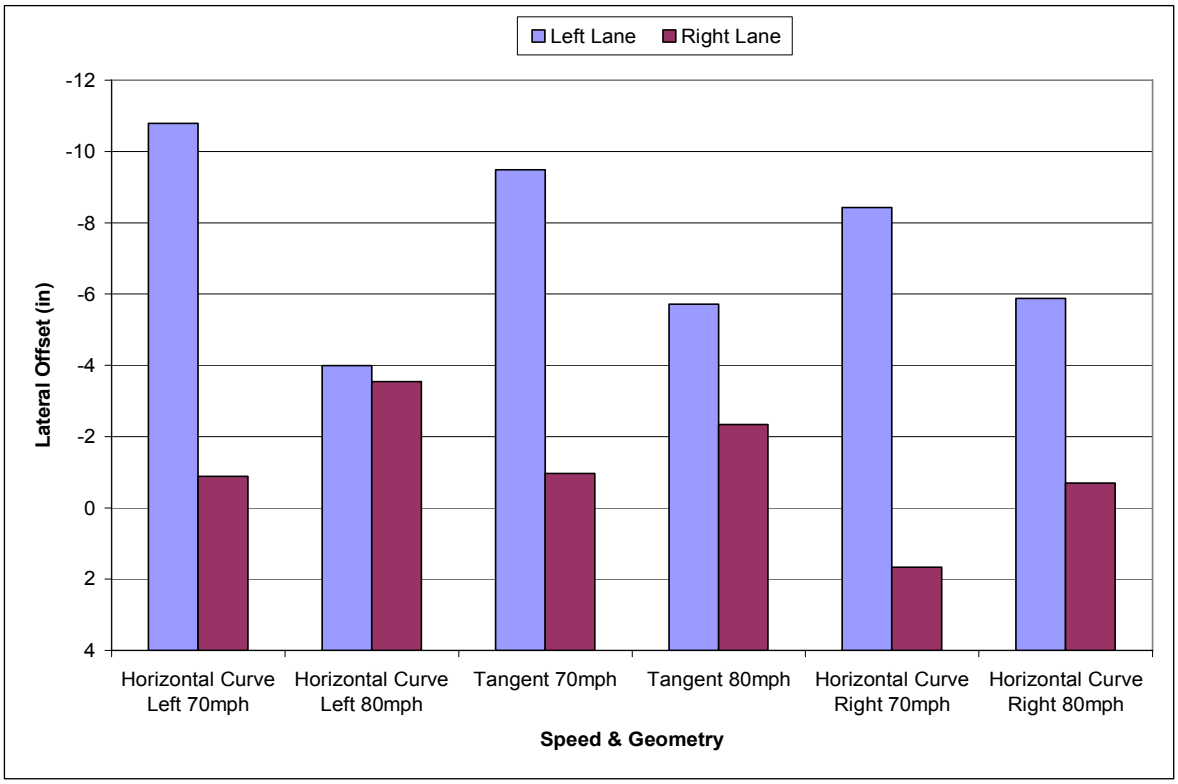

Figure 8. Average Lateral Offset by Lane & Lane Geometry

Velocity & Lateral Offset

Table 2 contains the average velocity and lateral offset for the zones hand placement and head movements were coded. Table 2 also contains the standard deviation and number of observations for each measurement. Figure 9 is a graphical representation of the velocity data. Figure 10 is a graphical representation of the variance in velocity measure by standard deviation. Figure 11 is a graphical representation of the lateral offset broken by lane in each zone. Figure 12 is a graphical representation of the variance in lateral offset measured by standard deviation broken down by zone and zone by lane.

Table 2. Average Velocity and Lateral Offset

	Velocity			Lateral Offset		
Zone by Lane	Mean (mph)	S.D. (mph)	n	Mean (in)	S.D. (in)	N
70mph Left Lane	73.11	2.41	29442	-8.49	15.63	29442
70mph Right Lane	72.13	1.78	7462	-1.17	8.91	7462
80mph Left Lane	79.69	1.42	25487	-7.89	16.59	25487
80mph Right Lane	78.70	1.46	9075	-2.00	10.89	9075

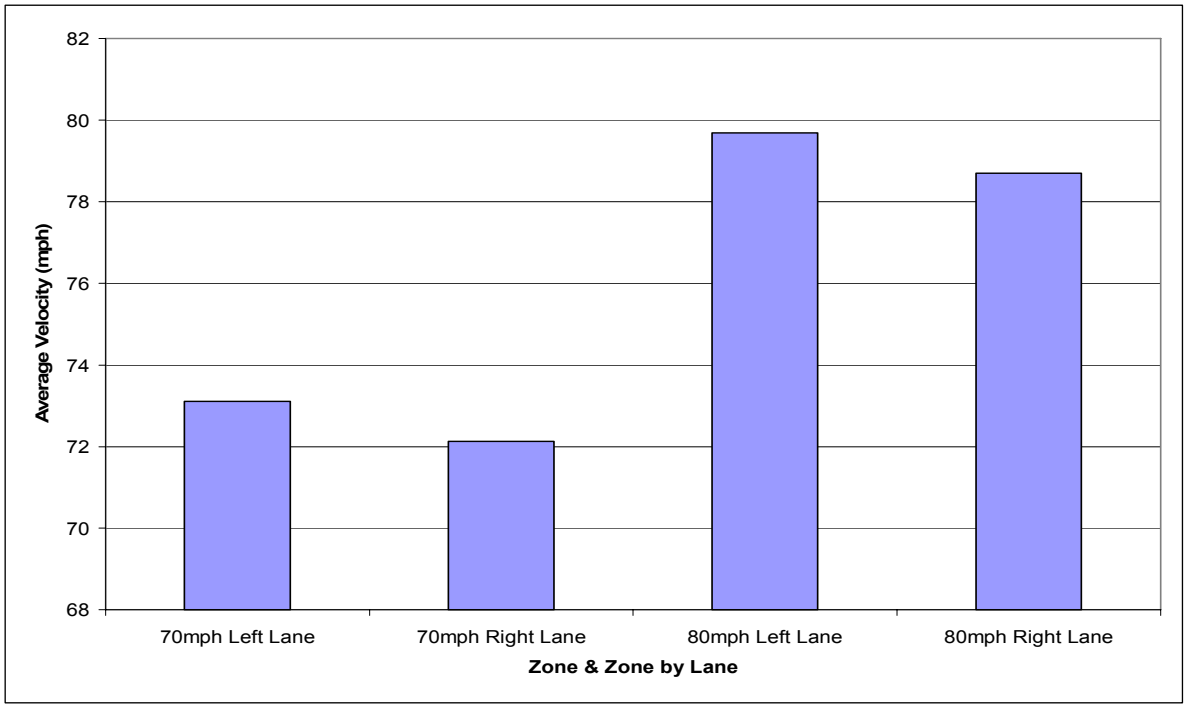

Figure 9. Average Velocity by Zone & Lane

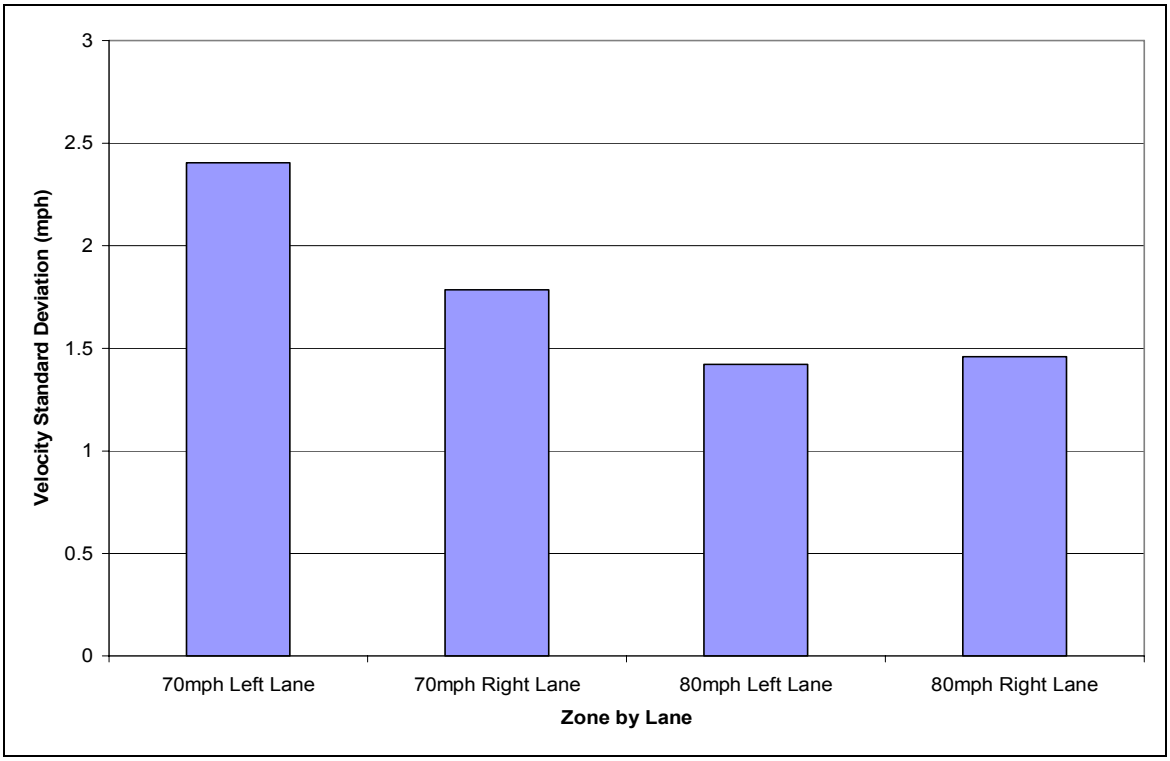

Figure 10. Variance in Velocity by Zone & Lane

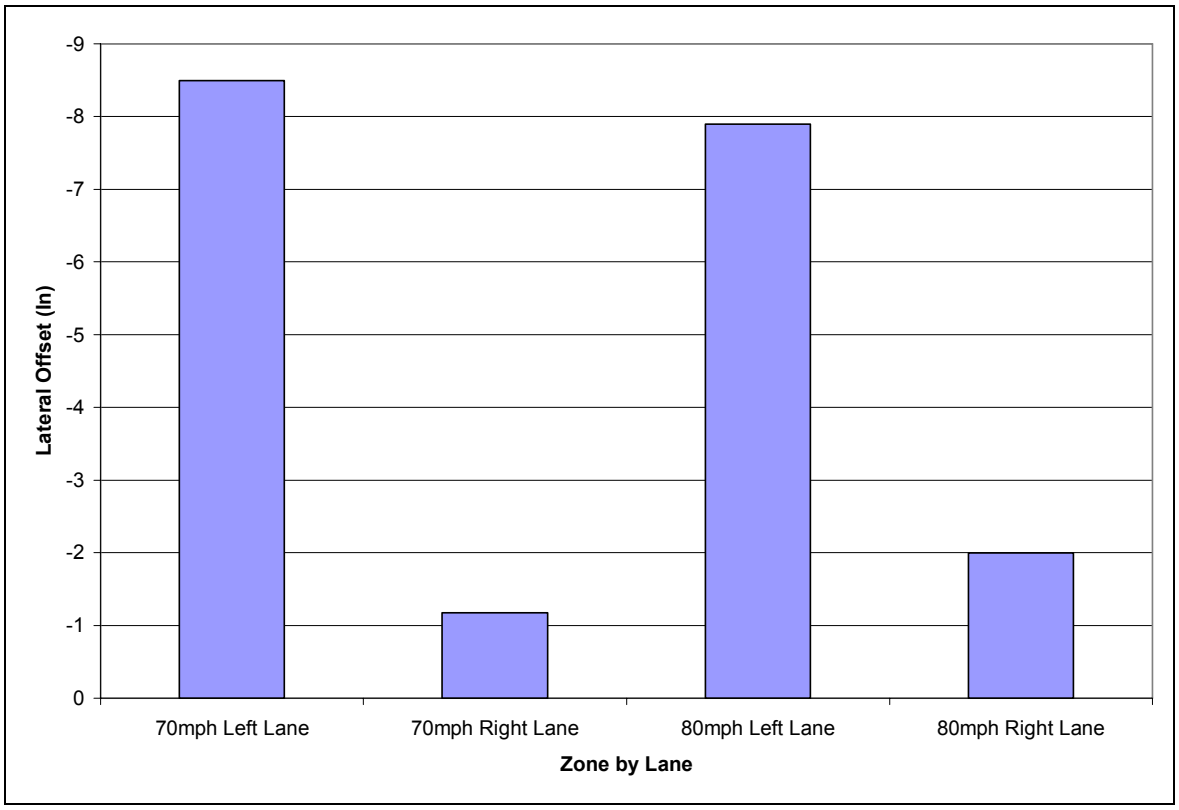

Figure 11. Average Lateral Offset by Zone & Lane

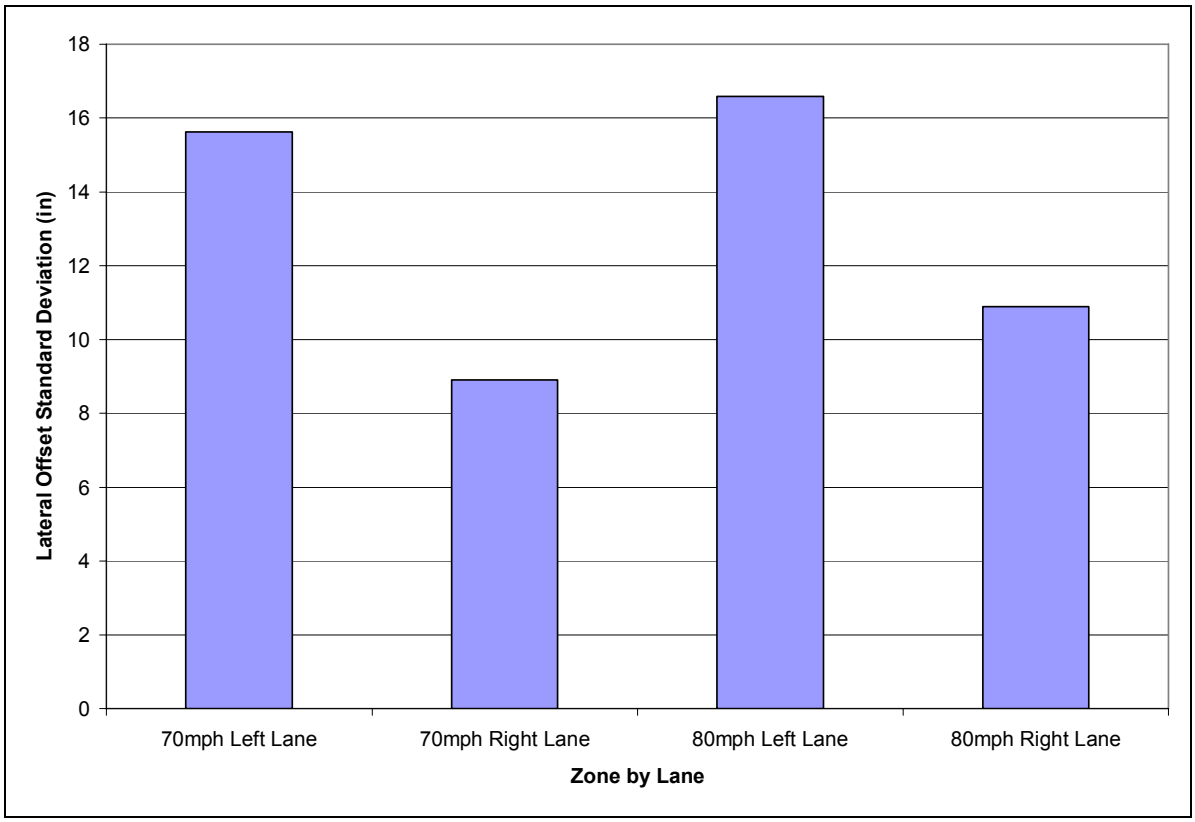

Figure 12. Variance in Lateral Offset by Zone & Lane

Driver Glance Rate

Table 3 contains the driver glance rate broken down by direction and zone by lane. Figure 13 is a graphical representation of these data. Forward glances were not taken into account in these observations.

Table 3. Non-Forward Glance Rates by Zone & Lane

	Glances Per Minute		
Zone by Lane	Left	Down	Right
70mph Left Lane	2.85	1.32	7.85
70mph Right Lane	3.35	2.32	7.51
80mph Left Lane	2.09	1.54	7.88
80mph Right Lane	3.29	2.98	6.60

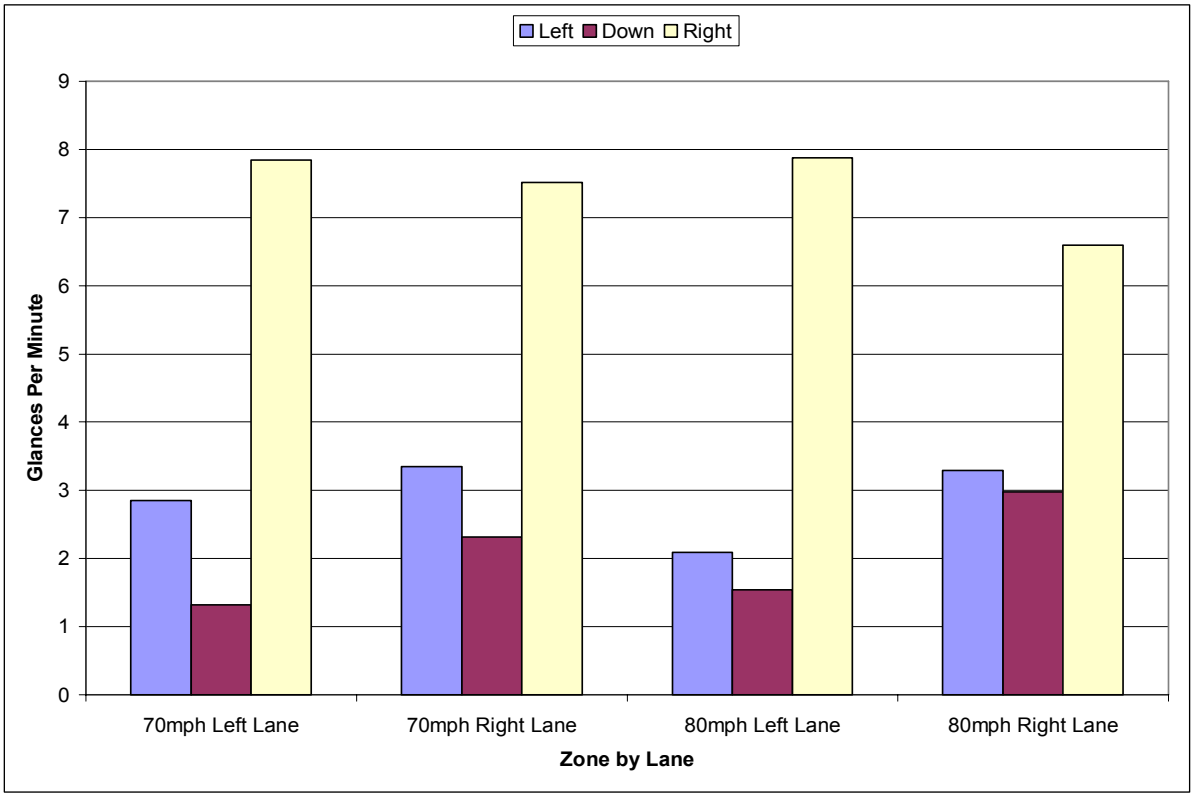

Figure 13. Non-Forward Glance Rates by Zone & Lane

Driver Hand Placement

Table 4 contains the percentage of time drivers' hands spent at the four coding location described above split by zone and lane. Figure 14 and Figure 15 are graphical representations of drivers' left and right hand placement, respectively, as a percent of total time. Table 5 contains the percent of total time that the driver had two, one, or zero hands on the top half of the steering wheel. Figure 16 is a graphical representation of Table 5.

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Table 4. Drivers' Individual Hand Placement by Zone & Lane

	Left Hand				Right Hand			
Zone by Lane	Top	Below	Off	Object	Top	Below	Off	Object
70mph Left Lane	73.92%	22.30%	3.76%	0.00%	45.21%	49.08%	5.71%	0.00%
70mph Right Lane	77.97%	17.98%	4.04%	0.00%	30.21%	62.64%	6.99%	0.17%
80mph Left Lane	67.63%	25.62%	6.75%	0.00%	28.76%	66.22%	5.02%	0.00%
80mph Right Lane	63.17%	30.70%	3.97%	2.16%	47.86%	45.65%	5.63%	0.86%

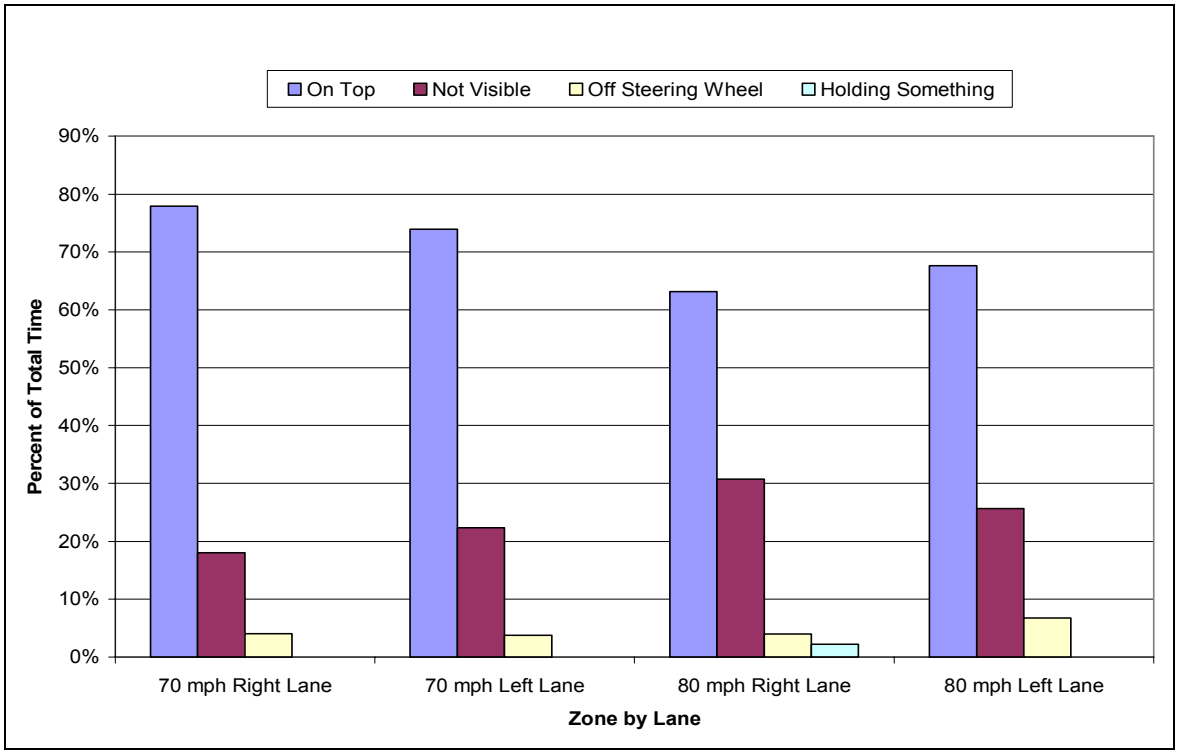

Figure 14. Drivers' Left Hand Placement by Zone & Lane

Table 5. Number of Hands on Top Half of Steering Wheel by Zone & Lane

	Hands on Top		
Zone by Lane	2	1	0
70mph Left Lane	36.43%	46.27%	17.30%
70mph Right Lane	31.26%	50.35%	20.74%
80mph Left Lane	26.15%	44.10%	29.75%
80mph Right Lane	32.67%	45.53%	21.81%

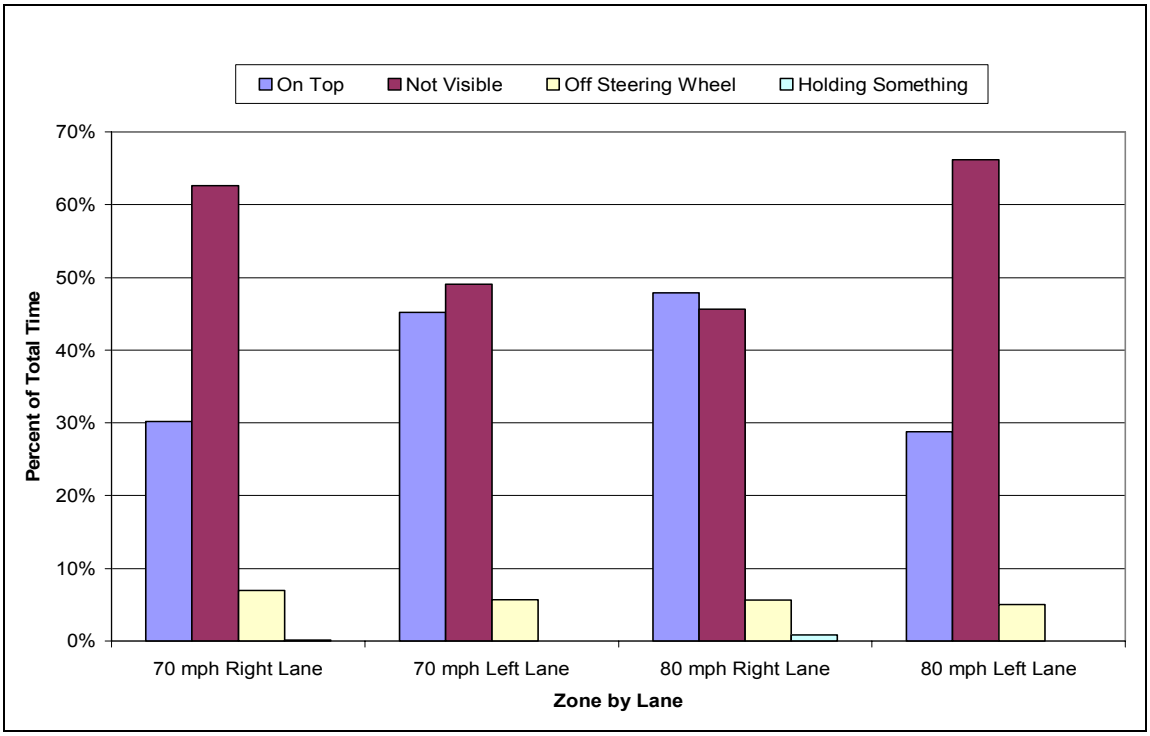

Figure 15. Drivers' Right Hand Placement by Zone & Zone by Lane

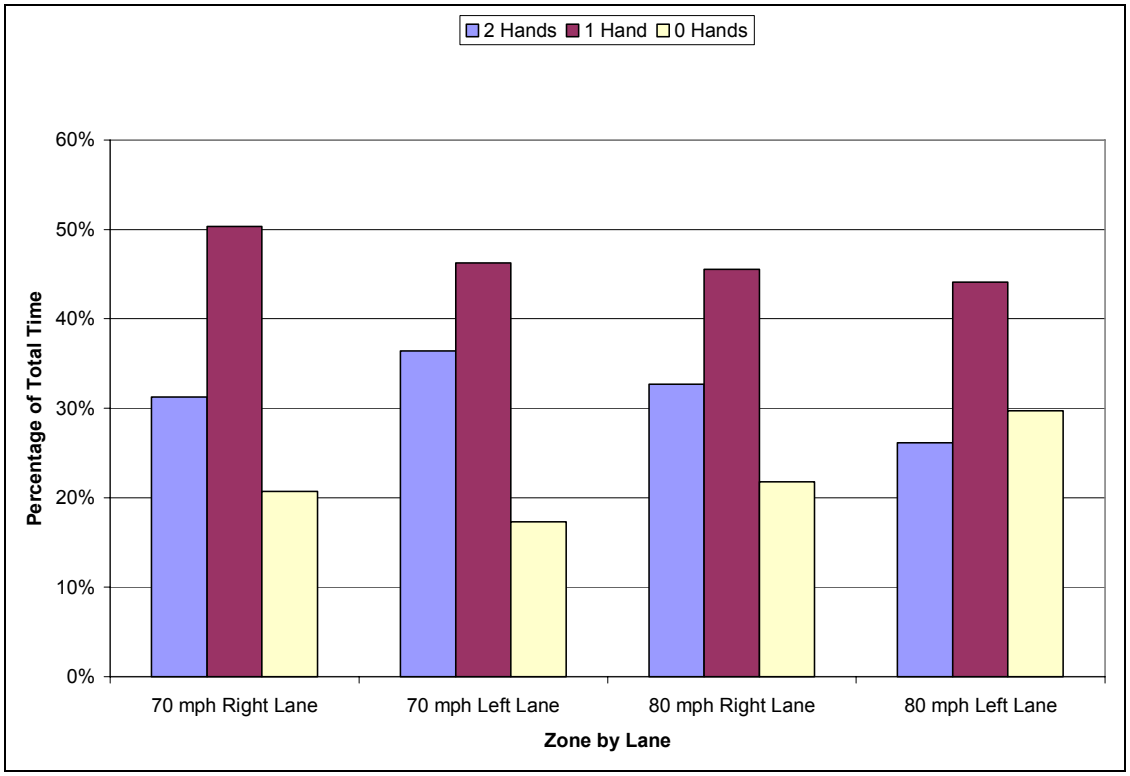

Figure 16. Number of Hands on Top Half of Steering Wheel by Zone and Zone by Lane

CONCLUSIONS

For this study velocity results were considered to have practical implications if the difference in speed is greater than 2 mph. Lateral offset results were considered to have potential practical implications if the difference was greater than 6 inches or half a foot. Practical implications for hand placement and glance rates will be discussed in their respective sections.

Lane Geometry and Lane of Travels Effect on Lateral Offset

Table 1 demonstrates practical difference between lateral offset and lane of travel. Within the 70 mph speed condition there were significant practical differences between vehicle lateral offset in the left lane verse the right lane in all geometric features investigated. This difference is not seen in any of the 80 mph geometries. However, there is a practical difference between lateral offset and negotiating a left horizontal curve in the two speed conditions.

These results suggest drivers going 70 mph are more willing to offset further to the left when traveling in the left lane than drivers at 80 mph in all geometries considered. These results suggest at 70 mph drivers may be more willing to travel close to the road's edge at these speeds than at 80 mph. More detailed analysis taking into account radius of the curves and remove of transition zones is desired to verify these results. The presence of other vehicles could be a contributing factor that was not considered in the development of these results.

Lane of Travel and Posted Speed Limits Effect on Vehicle Velocity

While there seems to be statistical difference between vehicle velocity and variance in velocity they are not greater than the practical implication thresholds for velocity. One aspect of note is the average traveling speed for all drivers in the 80 mph condition was under the posted speed limit. This suggests drivers in the experiment may not have been driving as they normally do on such facilities.

While drivers not exceeding the speed limit could be an indication of unwillingness to travel at those speeds there are likely other reasons. They may have been driving differently because it was not their vehicle they were driving. Another factor in west Texas would be the enforcement of the 80 mph speed limit. As a prelude to getting permission to have posted speeds limits of 80 mph, stricter enforcement of these speeds was required. A speed study along the same corridor the subjects are driving would be desirable for comparison. Further analysis is desired taking into account other possible variables.

Lane of Travel and Posted Speed Limits Effect on Lateral Offset

The lateral offset difference found in 70 mph tangents was also seen in the 70 mph tangent studied. Again the presence of other vehicles was not taken into account in these results. In the 80 mph coded segment there also seems to be a practical difference between offset and lane of travel. These differences suggest further analysis of other factors affect on lateral offset in the lane.

In both speed conditions the variance in lateral offset was also practically different. Vehicles traveling in the left lane seemed to have greater variation in their speed though this difference is smaller in the 80 mph condition. Further analysis needs to take into account the affect other vehicle have on these variances.

Lane of Travel and Posted Speed Limits Effect on Driver Glance Rates

Drivers had higher glance rates to the right in all speed conditions and all lanes than the other two categories coded. There do not seem to be any practical differences in driver glance rates using the current methods. Future studies and analysis should use methods that obtain glance duration and can differentiate between a subject looking at a passenger in the vehicle and passenger side mirror.

Lane of Travel and Posted Speed Limits Effect on Driver Hand Placement

Some general observations above driver hand placement can be made using data from Table 4 and Table 5. Data in Table 4 suggest drivers prefer having their left hand on the upper half of the steering wheel rather than their right. These data also suggest the right hand is normally in the driver's lap or gripping the lower portion of the steering wheel. The right hand also seems to be the hand most likely to clearly be off the steering wheel in all conditions.

Contrary to what would be expected drivers tend to place their hands on the upper half of the steering column less often in the 80 mph condition than they did in the 70 mph condition. This contradicts the idea that drivers would feel more at risk at these speeds and thus try to exert more control over the vehicle. As mentioned previously a hand position of ten and two has been shown to be associated with more vehicle control. This association may be true since in the 80 mph speed condition there was greater variance in lateral offset than the 70 mph condition as well.

So the question this raises is why did drivers exert less control over the vehicle in a condition that would seem to have a higher risk? One answer might be the driver's familiarity with the vehicle. Most observations had the driver going through the 70 mph section after having driving the vehicle for 15 minutes where in the 80 mph condition they had been driving for 45 minutes total. As driver familiarity with the vehicle became greater they would likely feel more comfortable in the vehicle no matter the small change in risk they perceive between the two speed conditions measured.

RECOMMENDATIONS FOR FUTURE STUDIES

- Use of an eye tracking device that can obtain both glance rate and duration without further coding to reduce data reduction effort.
- Inclusion of a camera showing the entire steering wheel to differentiate between a hand in the driver's lap and a hand on the lower half of the steering wheel.
- Possible inclusion of steering wheel grip sensors to evaluate the intensity at which the driver's grip the steering wheel as a measure of workload.
- Development of more efficient computer macros to decrease data reduction effort.

- Inclusion of a camera showing the roadway behind the instrumented vehicle. On high speed facilities what is going on behind the vehicle can be just as important as what is going on in front of it (23).
- Obtaining roadway geometry data to measure the effects such as super elevation and curve radii in relation to lateral lane placement.
- Use the instrumented vehicles GPS system to obtain accurate GPS location for significant roadside objects such as Off Ramps, On Ramps, and Speed Limit Changes.

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