Examining Dynamic Variable Speed Limit Strategies For The Reduction Of Real-time Crash Risk On Freeways

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EXAMINING DYNAMIC VARIABLE SPEED LIMIT STRATEGIES FOR THE REDUCTION OF REAL-TIME CRASH RISK ON FREEWAYS

by

RYAN J. CUNNINGHAM
B.Sc. University of Central Florida, 2006

A thesis submitted in partial fulfillment of the requirements for the degree of Master of Science in the Department of Civil and Environmental Engineering in the College of Engineering and Computer Science at the University of Central Florida Orlando, Florida

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ABSTRACT

Recent research at the University of Central Florida involving crashes on Interstate-4 in Orlando, Florida has led to the creation of new statistical models capable of determining the crash risk on the freeway (Abdel-Aty et al., 2004; 2005, Pande and Abdel-Aty, 2006). These models are able to calculate the rear-end and lane-change crash risks along the freeway in real-time through the use of static information at various locations along the freeway as well as the real-time traffic data obtained by loop detectors. Since these models use real-time traffic data, they are capable of calculating rear-end and lane-change crash risk values as the traffic flow conditions are changing on the freeway. The objective of this study is to examine the potential benefits of variable speed limit implementation techniques for reducing the crash risk along the freeway.

Variable speed limits is an ITS strategy that is typically used upstream of a queue in order to reduce the effects of congestion. By lowering the speeds of the vehicles approaching a queue, more time is given for the queue to dissipate from the front before it continues to grow from the back. This study uses variable speed limit strategies in a corridor-wide attempt to reduce rear-end and lane-change crash risks where speed differences between upstream and downstream vehicles are high. The idea of homogeneous speed zones was also introduced in this study to determine the distance over which variable speed limits should be implemented from a station of interest. This is unique since it is the first time a dynamic distance has been considered for variable speed limit implementation.

Several VSL strategies were found to successfully reduce the rear-end and lane-change crash risks at low-volume traffic conditions (60% and 80% loading conditions). In every case,
the most successful treatments involved the lowering of upstream speed limits by 5 mph and the raising of downstream speed limits by 5 mph. In the free-flow condition (60% loading), the best treatments involved the more liberal threshold for defining homogeneous speed zones (5 mph) and the more liberal implementation distance (entire speed zone), as well as a minimum time period of 10 minutes. This treatment was actually shown to significantly reduce the network travel time by 0.8%. It was also shown that this particular implementation strategy (lowering upstream, raising downstream) is wholly resistant to the effects of crash migration in the 60% loading scenario.

In the condition approaching congestion (80% loading), the best treatment again involved the more liberal threshold for homogeneous speed zones (5 mph), yet the more conservative implementation distance (half the speed zone), along with a minimum time period of 5 minutes. This particular treatment arose as the best due to its unique capability to resist the increasing effects of crash migration in the 80% loading scenario. It was shown that the treatments implementing over half the speed zone were more robust against crash migration than other treatments. The best treatment exemplified the greatest benefit in reduced sections and the greatest resistance to crash migration in other sections. In the 80% loading scenario, the best treatment increased the network travel time by less than 0.4%, which is deemed acceptable.

No treatment was found to successfully reduce the rear-end and lane-change crash risks in the congested traffic condition (90% loading). This is attributed to the fact that, in the congested state, the speed of vehicles is subject to the surrounding traffic conditions and not to the posted speed limit. Therefore, changing the posted speed limit does not affect the speed of vehicles in a desirable manner. These conclusions agree with Dilmore (2005).
To the glory of God

and to his Son, Jesus Christ my Savior, through whom and for whom all things were created, on heaven and on earth, and by whom all things have their being and are held together. It is by his shed blood that my sins are covered, and I am reconciled to God. Though once I was an enemy of God, living in rebellion and sin, Christ has come to seek and save the lost. By his sacrifice to God on the cross, Christ has paid the ransom for my sins and covered me with his righteousness. Though this cost him so much, I have received it freely by his grace through faith. Therefore, it is unto the Lord that I labor endlessly, and it is to his glorious praise that I dedicate every work that I have done.
ACKNOWLEDGMENTS

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# LIST OF ACRONYMS/ABBREVIATIONS

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<th>Description</th>
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<tbody>
<tr>
<td>AADT</td>
<td>Annual Average Daily Traffic</td>
</tr>
<tr>
<td>API</td>
<td>Application Programmer Interface</td>
</tr>
<tr>
<td>CATSS</td>
<td>Center for Advanced Transportation Systems Simulation</td>
</tr>
<tr>
<td>DDHV</td>
<td>Directional Design Hourly Volume</td>
</tr>
<tr>
<td>FDOT</td>
<td>Florida Department of Transportation</td>
</tr>
<tr>
<td>FHWA</td>
<td>Federal Highway Administration</td>
</tr>
<tr>
<td>FTI</td>
<td>Florida Traffic Information</td>
</tr>
<tr>
<td>GUI</td>
<td>Graphic User Interface</td>
</tr>
<tr>
<td>HOV</td>
<td>High Occupancy Vehicle</td>
</tr>
<tr>
<td>ITS</td>
<td>Intelligent Transportation Systems</td>
</tr>
<tr>
<td>OD</td>
<td>Origin-Destination</td>
</tr>
<tr>
<td>ORCI</td>
<td>Overall Risk Change Index</td>
</tr>
<tr>
<td>RCI</td>
<td>Roadway Characteristics Index</td>
</tr>
<tr>
<td>VHT</td>
<td>Vehicles Hours Traveled</td>
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<tr>
<td>VSL</td>
<td>Variable Speed Limits</td>
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CHAPTER 1. INTRODUCTION

Transportation systems are a major component in the successful economy of the United States. The vast majority of people in America travel on the nation’s Interstate System for business and pleasure. This system, officially called the National System of Interstate and Defense Highways, is a 42,500-mile national system of multilane, limited-access facilities (Roess et al, 2004). While the efficiency of highways is very important to the growth and sustainability of the economy, the primary concern for transportation engineers is safety. In recent years, fatalities on U.S. highways have ranged between 40,000 and 43,000 per year. As reported by Roess et al (2004), more Americans have died on U.S. highways than in all of the wars in United States history. That is a staggering fact, and it emphasizes that safety should still be a top priority.

Recent research at the University of Central Florida has focused on increasing safety on the Interstate-4 corridor through Orlando, Florida. The first part of this research used logistic regression modeling to develop crash prediction models based on real-time traffic conditions (Abdel-Aty et al., 2004). Through the use of micro-simulation, Dilmore (2005) explored the benefits of variable speeds limits at a single location on Interstate-4 and found that variable speed limits were beneficial in reducing the crash risk at high speed scenarios. Dhindsa (2006) explored the benefits of ramp metering, as well as the combination of ramp metering and variable speed limits, on Interstate-4 using micro-simulation.

Pande and Abdel-Aty (2006) improved their crash prediction models through the use of neural networks and the inclusion of off-line variables including the time of day and location on the network. Using these newer models, Gayah (2006) expanded on the research through his
investigation of ramp metering and route diversion on a larger network. Gayah’s (2006) new network expanded the boundaries of the older one for a total of 36.25 miles of highway. Gayah investigated the use of coordinated and un-coordinated ramp metering, as well as two separate implementation methods.

This study expands on previous research by investigating the benefits of corridor-wide variable speed limit implementation, based on several important factors. Though Dilmore (2005) investigated variable speed limits previously, this study greatly expands on his work. This study utilizes the newer crash risk models developed by Pande and Abdel-Aty (2006) as well as the larger network used by Gayah (2006). Also, variable speed limits in this study are implemented in response to increasingly risky traffic conditions, as opposed to implementing at a fixed time. Prominently different from Dilmore (2005), this study intends to find the best corridor-wide variable speed limit implementation strategy. Every location on the network is capable of variable speed limit implementation, as opposed to testing variable speed limits at only a single location. Three implementation techniques are examined: decreasing upstream speeds by 10 mph, decreasing upstream speeds by 5 mph, and simultaneously decreasing upstream speeds and increasing downstream speeds by 5 mph. The magnitudes of the speed change are considered more reasonable than the recommendations made by Dilmore (2005). Furthermore, the design speeds along the freeway were considered to ensure that the VSL implementation would not exceed them. Perhaps most importantly, this study includes a dynamic distance over which speed limits should be changed, based on the traffic conditions on the network. In other studies on variable speed limits, several fixed distances have been tested, such as 1 mile, 2 miles, etc. For this study, however, the distance over which variable speed limits are implemented is dynamically dependent on the traffic conditions of the network, in real time, based on
homogeneous speed zones. As far as the author knows, this approach is unique as it is the first time dynamic implementation distances have been included in research for variable speed limits.

Therefore, the objectives of this research are as follows:

- Examine a variety of corridor-wide Variable Speed Limits implementation strategies and their effects on the Rear-End and Lane-Change Crash Risk at various volume loading scenarios
- Consider the different aspects of defining and implementing over dynamically-defined homogeneous speed zones and their effects on crash risk
- Compare different time periods over which to implement variable speed limits
- Examine the impacts of variable speed limits on the operational capabilities of the Interstate-4 system
- Make recommendations for variable speed limits to be implemented along Interstate-4 at different volume loading scenarios
CHAPTER 2. LITERATURE REVIEW

2.1 Traffic Simulation

Traffic Simulation has become increasingly more popular and more effective as a tool for analyzing dynamic problems in transportation networks that could not be analyzed by any other feasible means. This increase is due in large part to the ever-increasing advances in computer processing technology. The behavior of vehicles on a transportation network can be accurately described with traffic simulation through the integration of individual system component’s behavior and the interactive behavior of all system components on a network (Lieberman and Rathi, 1997). After the traffic simulation user is able to specify a certain “scenario”, the simulation will describe the system’s operations graphically and statistically. From there, the user may attempt to explain why the system behaves the way that it does.

Traffic Simulation is used for a wide range of activities, including the evaluation of alternative traffic engineering treatments. In this capacity, traffic simulation can save engineers and municipalities thousands of dollars, in terms of time and capital. Many different alternatives, for example, traffic demand management strategies, can be tested using traffic simulation without the need of constructing and maintaining new infrastructure. The savings in public confidence is also considered when using traffic simulation for the testing of alternative treatments. Many different alternatives can be analyzed without exposing the public to the risk of potentially harmful treatments.

Another advantage of traffic simulation is its use in the testing of new design that could otherwise not be tested in the field. The cost of adding lanes or reconfiguring intersections on a
transportation system can be significant, not to mention the time that is employed in construction and in the closing down of traffic lanes. Traffic simulation can be used to analyze new designs, new configurations, etc. without the cost of any infrastructure and with the minimal time needed to create the new design using traffic simulation software. This can also be implemented towards new technologies. Researchers can use traffic simulation to analyze the application of new technologies to existing transportation systems. Based on the statistical and graphical output, then, the researcher can attempt to explain the advantages and disadvantages of the new technology and its effect on the existing system.

2.1.1 Classifications of Traffic Simulation Models

There are various ways to classify traffic simulation models. Most traffic simulation models are used to describe dynamical systems, where time is always a basic independent variable (Lieberman and Rathi, 1997). There exist two classes of traffic simulation models with respect to time: continuous and discrete. Continuous models describe how elements change continuously over time in response to continuous stimuli. Discrete models, however, describe how the states of elements change abruptly in response to events.

Traffic simulation models may also be classified according to their levels of detail. There are three levels by which a traffic simulation model may be classified. These include macroscopic (low fidelity), mesoscopic (mixed fidelity) and microscopic (high fidelity). Macroscopic models have a low level of detail and perform modeling based on the basic elements of volume, density and speed of the traffic stream. These models are the easiest to build and execute. Results can be obtained quickly, however, they are not as accurate as the low fidelity models.
Microscopic models operate at a high level of detail. They are able to describe the system in terms of its individual elements and the interaction of its elements. For instance, the lane change of a specific vehicle would affect the leader-follower relationship of the vehicle behind it, so that lane change would affect the behavior of the traffic stream. The advantage of microscopic models is their ability to produce very accurate results. Some disadvantages include the time necessary to construct microscopic networks, as well as the time needed to run the model in comparison to macroscopic models.

A mesoscopic model typically will represent the individual elements at a high level of detail but describes their interactions at a low level of detail. The event of a lane change, for instance, could be represented as an event caused by traffic stream densities rather than the detailed interaction of the traffic elements. Ultimately, the type of model used is left to the discretion of the user. If the detailed behavior and interaction of elements is important to the purpose of the simulation, microscopic or mesoscopic should be used. However, if only general relationships need to be analyzed and time for model construction is limited, macroscopic models may also be used.

Another classification used to describe traffic simulation models addresses the processes in the model, whether they are stochastic or deterministic. In deterministic models, the interactions between elements are defined by exact relationships. These relationships can be logical, mathematical or statistical. There are no random variables involved in deterministic relationships. Stochastic models, however, exercise the use of probability functions to describe the interaction between elements in the model. For example, a car-following model may be stochastic or deterministic depending on the whether the reaction time is defined by a random variable or a constant value, respectively.
2.1.2 Applications of Traffic Simulation Models

There have been many various applications of traffic simulation to research and to professional design projects and traffic operations studies. Mahmassani and Jayakrishnan (1991) used traffic simulation modeling to analyze the driving behavior of route choice along three major arterials. The study involved the analysis of traffic system performance with the utilization of in-vehicle information, and thus the potential benefits of in-vehicle information systems to improve corridor-wide congestion situations. Experimental factors included the percentage of drivers provided with in-vehicle information and the minimum percentage of travel time savings required for drivers to choose a route change. Their findings included a framework for further study in the area and hope that in-vehicle information could potentially decrease network travel times.

Traffic simulation has also been used as part of the design process of new transportation infrastructure and as a means of validation for projected use. In 1996, Korve Engineers (1996) utilized WATSim simulation software to help in alternative designs of SR242, a 20-mile freeway connection in California. Their objective was to increase capacity on SR242 while maintaining a balanced flow of traffic on the surrounding freeways, SR4 and I-680. The design alternatives considered the projected traffic in 2000, 2010, and 2020 and included geometric changes, widening, HOV lanes, and the use of ramp metering. The use of simulation in this project proved to be very useful and gave them a statistical basis with which to move forward with the final design.

Gardes et al (2002) used the PARAMICS software to test proposed congestion management strategies on I-680 in California in the San Francisco Bay Area. This freeway is
considered one of the most congested freeways in the United States. The first section of the paper deals intensively with the PARAMICS program itself and the model developed. The authors explain in depth the procedure developed to calibrate two critical driver behavior parameters: mean target headway and mean reaction time. It is strongly recommended that no model applications be recorded before model calibration is carried out. After calibration, the model was applied to test the effectiveness of three strategies on I-680: implementation of ramp metering, addition of auxiliary lanes at various locations, and addition of an HOV lane. The results from the simulation helped Caltrans (the governing body of the California freeway system) determine the most cost-effective means of improving the I-680 situation.

Another example of recent applications of traffic simulation is the work done in by Portland State University in Oregon. Bertini et al (2002) used PARAMICS to simulate an existing diamond interchange in the city of Wilsonville, Oregon. The network included the diamond interchange at the intersection of Wilsonville Road and I-5, as well as the surrounding intersections of Wilsonville Road with Boones Ferry Road, Parkway Avenue and Town Center Loop West. Bertini discovered the importance of including the nearby intersections in the model to properly observe the delay at the diamond interchange. Traffic demand variations and geometric changes to the interchange were included to analyze their potential to reduce travel time and average vehicle delay. It was found that, at high traffic demands, an additional left turn lane would significantly decrease vehicle delay at the intersection.

Chu et al (2003) also used PARAMICS to construct and calibrate a network including a 6-mile section of freeway I-405, a 3-mile section of freeway I-5, a 3-mile section of freeway SR-133 and adjacent surface streets, located in the city of Irvine, California. After construction, calibration, and validation, PARAMICS was used to evaluate the effectiveness of a number of
ITS strategies on the network. The evaluated ITS strategies included incident management, local adaptive ramp metering and coordinated ramp metering, traveler information systems, corridor control, and the combination of traveler information systems, corridor control, and adaptive ramp metering. These strategies were implemented through the use of PARAMICS Application Programmer Interface (API), which is used to override default PARAMICS values for the implementation of various ITS strategies. Chu et al. found that the implementation of the various ITS strategies were beneficial to the network as a whole and to its individual component and that the combination of traveler information systems, corridor control, and adaptive ramp metering generated the greatest benefits for the network.

2.1.3 Safety Applications of Traffic Simulation

Traffic simulation models have more recently been used to observe the affects that different scenarios would have on safety in the field. Although safety cannot be measured directly, and most simulation packages do not simulate accidents, performance measures pertinent to safety in the field can be outputted from the simulation to estimate a measure of safety. In this way, different scenarios or management tactics can be tested using simulation, and the safety effect can be estimated indirectly.

Drummond (2002) uses simulation modeling (Synchro/SimTraffic) to create prediction models correlating the safety and operational impacts of increased traffic signal density along two corridors in Virginia. The primary focus of this research is to predict the safety effects of proposed signal installations by correlating crash rates with simulated operational measures taken with increasing signal density on the two Virginia corridors. Crash data from 1990 to 2000 was used to observe any correlation between crash rate (number of crashes per 100 million
vehicle miles traveled, VMT) and operational performance measures (delay, stops per vehicle, and speed) outputted from the simulation model. The corridors were chosen based on their significant demand growth throughout the 1990s, the availability of traffic, crash and geometric data, and the lack of major geometric changes at the sites throughout the decade. From the analysis, there were found to be increasing relationships between mainline delay per vehicle and crash rates, and mainline stops per vehicle and crash rates.

Park and Yadlapati (2003) used VISSIM to evaluate the safety benefits that could be obtained through the use of variable speed limits (VSL) in work zones. They proposed a surrogate crash measure, the minimum safe distance equation (MSDE), and a method used to maximizes MSDE (safety measure) and travel time. VISSIM was used to evaluate a proposed VSL control logic (logic 3) against two other common control logics used in work zones. Park and Yadlapati used varying compliance rates (70, 80, 100%) and two demand conditions (undersaturated and oversaturated). Their recommendations include a case study of field implementation of VSL at work zone using 45 mpg for the base case and deployment of the proposed VSL logic 3.

Studies by Lee et al (2004, 2005) used PARAMICS to determine the safety benefits of variable speed limits and ramp metering on freeways. Lee (2003) used historical loop data, collected from a 10 km stretch of the Gardiner Expressway in Toronto, Canada, to develop a log-linear model that identified several real-time crash precursors. This model allowed the crash risk along the freeway to be determined in real-time. By simulating a 2.5 km portion of this freeway in PARAMICS and applying the model, they found that the individual applications of variable speed limits and ramp metering successfully reduced the crash risk along the freeway. This
simulation, however, was rather simplistic, barring the use of real traffic data for model calibration and including only a small portion of the freeway (2.5 km).

Abdel-Aty et al. (2005) expanded on Lee’s work by simulating approximately 9 miles of Interstate-4 in Orlando, FL in PARAMICS and using real traffic flow data to calibrate the model. A sophisticated crash prediction algorithm developed by Abdel-Aty et al (2004), based on historical data from the same stretch of Interstate-4, was utilized in this study. Abdel-Aty argued that the accuracy of a real-time crash prediction model would be increased if the model considered information from both crash cases and matched non-crash cases with similar offline factors (location, time of day, etc.). Following this concept, their model was built as a matched case controlled logistic regression model that implicitly accounted for location and geometry of the freeway. Two separate models that could determine crash risk were created for low-speed (less than 37.5 mph) and high-speed (greater than 37.5 mph) flow regimes.

Using these crash risk models as the basis for his research, Dilmore (2005) found that the application of variable speed limits along the freeway could successfully reduce the crash risk at particular locations in the high speed regime. However, he did not find the implementation of variable speed limits in the low speed regime to have as considerable an effect due to congestion limitations. Abdel-Aty and Dhindsa (2007) further expanded on Dilmore’s research by examining the potential benefits of ramp metering and variable speed limits. Dhindsa used the same PARAMICS network and crash prediction models as Dilmore (2005) but focused more intently on the improvement of real time crash risk in the low speed regime. Dhindsa also used a more sophisticated variable speed limit implementation strategy and considered the simultaneous implementation of variable speed limits and ramp metering. He found that feedback ramp metering has significant potential to reduce real time crash risk in the low speed regime and that
using variable speed limits on a corridor-wide level (as opposed to a fixed location) has a significant effect in reducing crash risk potential on the freeway.

Gayah (2006) expanded this collection of research significantly by reconstructing the Interstate-4 simulation model, in PARAMICS, to a 36.25-mile section. Gayah tested the effect that implementing route diversion and ramp metering, separately, had on the freeway’s real time crash risk. He found that both route diversion and ramp metering decreased the overall crash risk in the network during low volume loading conditions. During extreme congestion, Gayah found that the ALINEA ramp metering strategy worked well to reduce crash risk in the downtown area.

2.1.4 Selection of a Traffic Simulation Software

The objective of this project is to determine the effects of variable speed limits on the safety of an urban freeway. Therefore, a microscopic simulator is deemed the most pertinent choice of simulation software. While the freeway corridor to be modeled is rather simplistic and could warrant a macroscopic model for operational studies, the implementation of variable speed limits will certainly include the involvement of car-following and lane-changing models. The best method, then, to capture this vehicle interaction and the effect on the traffic safety is through the use of a microscopic simulation software. Additionally, a stochastic software would be best used since it more accurately models the behavior of vehicles on the freeway and, furthermore, a discrete time model should be used since there are no discrete events on the freeway that could be used to reduce the runtime of the simulation.

As can be shown from a review of the literature, there is a plethora of simulation packages that fit the given criteria. Boxill and Yu (2000) presented an evaluation of existing
traffic simulation models to identify which models could be applied in ITS equipped networks. Their evaluation occurred mainly in two steps: (1) an initial screening that generated a shorter list of traffic simulation models based on some pre-determined criteria, and (2) an in-depth evaluation which identified which specific models are most suitable for specific ITS applications. Of the 76 simulation models considered, they ranked CORSIM, INTEGRATION, AIMSUN2 and PARAMICS as the best simulation models for real world application.

Table 2-1 Comparison of Multiple Micro-simulation Packages for ITS Purposes (Boxill and Yu, 2000)

<table>
<thead>
<tr>
<th>ITS Features Modeled</th>
<th>AIMSUN 2</th>
<th>CONTRAM</th>
<th>CORFLO</th>
<th>CORSIM</th>
<th>FLEXYT II</th>
<th>HUTSIM</th>
<th>INTEGRATION</th>
<th>PARAMICS</th>
<th>VISSIM</th>
</tr>
</thead>
<tbody>
<tr>
<td>Traffic devices</td>
<td>X</td>
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<td>X</td>
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<tr>
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<tr>
<td>Driver behavior</td>
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<td>Vehicle interaction</td>
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<tr>
<td>Congestion pricing</td>
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<td>Ramp metering</td>
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<td>Coordinated traffic signals</td>
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<td>Adaptive traffic signals</td>
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<td>Interface w/other ITS algorithms</td>
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<tr>
<td>Network flow pattern predictions</td>
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<td>Route guidance</td>
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<td>Integrated simulation</td>
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<td>Other Properties</td>
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<tr>
<td>Runs on a PC</td>
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<tr>
<td>Graphical Network Builder</td>
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<td>X</td>
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<td>X</td>
<td>X</td>
<td>X</td>
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<tr>
<td>Graphical Presentation of Results</td>
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<td></td>
<td>X</td>
<td>X</td>
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<td>X</td>
<td>X</td>
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<tr>
<td>Well Documented</td>
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</tbody>
</table>
As seen above, these four models rank among the best simulation packages for ITS purposes. It can be seen that PARAMICS actually ranks the best among the other models. Also, what is lacking in PARAMICS - “Interface with other ITS algorithms” and “Route guidance” - can be made up for with its Application Programmer Interface (API) feature. The Application Programmer Interface allows the user to edit select built-in functions within the PARAMICS code that can be used to control the behavior of the vehicles. By creating an API one can change the behavior of the vehicles while the simulation is running, so potentially any ITS strategy can be applied.

Shaw and Nam (2002) worked extensively using the CORSIM 4.2 software for the construction of the metropolitan Milwaukee freeway system. However, they noted that there were some severe limitations. One of the main drawbacks of CORSIM 4.2 was a maximum of 500 nodes allowed per network. One of the freeway’s complex interchanges alone required the use of 430 nodes, which limited the rest of their model to 70 nodes. Additional problems with CORSIM included cumbersome network editing, older traffic algorithms, and unrealistic simulation results. The authors looked to two newer simulation packages, PARAMICS (PARAllel MICroscopic Simulation) and VISSIM, for comparison. A comparison of the three microscopic simulation software packages was performed and the results are summarized below in Table 2-2. As shown, both PARAMICS and VISSIM were found to be far superior to the CORSIM package.
Table 2-2  Comparison of CORSIM, PARAMICS, and VISSIM Packages (Shaw and Nam, 2002)

<table>
<thead>
<tr>
<th>Evaluation Criteria</th>
<th>CORSIM</th>
<th>PARAMICS</th>
<th>VISSIM</th>
</tr>
</thead>
<tbody>
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<td>A. Model Capability</td>
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<tr>
<td>1 Network Size Limit</td>
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<td>3 Traffic Flow Representation</td>
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<tr>
<td>4 Detail of Output</td>
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<tr>
<td>5 Network Merge</td>
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<td>⭐⭐⭐⭐</td>
<td>⭐</td>
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<tr>
<td>6 3-D Modeling</td>
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<td>7 Traffic Composition</td>
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<tr>
<td>8 Animation</td>
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<tr>
<td>B. Ease of Use</td>
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<tr>
<td>9 Input Data Requirements</td>
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<td>10 Network Coding / Editing</td>
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<td>11 Input / Output Review</td>
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<tr>
<td>C. FSOA Application Requirements</td>
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<tr>
<td>12 VISTA (GIS) Interface</td>
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<td>13 Economic Analysis Interface</td>
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<td>15 Actuated Signal Control Devices</td>
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<td>16 User-Defined Traffic Control &amp; API</td>
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<tr>
<td>D. Other</td>
<td></td>
<td></td>
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<tr>
<td>18 Calibration Results</td>
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<td>⭐⭐⭐⭐</td>
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<tr>
<td>19 Program Integrity</td>
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<tr>
<td>21 Documentation</td>
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<td>⭐⭐⭐⭐</td>
<td>⭐⭐⭐⭐</td>
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<tr>
<td>22 Record of Large-Scale Freeway Applications</td>
<td>⭐⭐⭐⭐</td>
<td>⭐⭐⭐⭐</td>
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<tr>
<td>23 Software Cost per Copy</td>
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<td>FINAL RATING</td>
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</table>

It is clear from this study, and from Boxill and Yu (2000), that PARAMICS is ranked as one of the best micro-simulation models for ITS purposes in real world applications. With the support of the literature, proven results in previous studies, and the flexibility provided by its
Application Programmer Interface (API), the PARAMICS micro-simulation software was selected for use in this study.

2.2 Variable Speed Limits

Variable speed limits are used in micro-simulation studies and have been implemented all over the world. There are many reasons for the use of variable speed limits. In terms of safety, variable speed limits have often been used during inclement weather when dangerous roadway conditions are detected to state a reduced safe speed limit for drivers (McLawhorn, 2003). Speed limits can be reduced when visibility decreases, when heavy precipitation approaches, or when high speed winds are present (Sisiopiku, 2001). Variable speed limits have also been used to increase safety in work zones. Park and Yadlapati (2003) found that the implementation of variable speed limits in work zones could increase safety by reducing speeds and increasing headways.

Variable speed limits have also been shown in aggregate studies to decrease the variation in speeds and increase headways in high crash risk scenarios (Borrough, 1997). Accident prone conditions are often characterized by high frequencies of short headways and high variation in speeds among the traffic flow. Variable speed limits can be used to increase average headways and decrease variation in speeds in the flow. This would promote a smoother, calmer flow of traffic. Lee (2004) and Dilmore (2005) used variable speed limits on simulated freeways to decrease the risk of crash occurrence during hazardous periods.

Variable speed can be used to decrease the intensity of traffic congestion. Van de Hoogen (1994) reports that variable speed limits, when used to decrease speed differences in traffic flow, reduced the number and severity of shockwaves. Zhichai et al. (2004) used
simulation to test different variable speed limit scenarios with the goal of relieving demand driven congestion and supply driven congestion. The simulation results indicated that the variable speed limits, for a freeway with demand-driven congestion in two lanes, are most effective when the traffic volume is equal to or greater than 2800 vehicles per hour. Benefits from variable speed limit implementation included an increase in the served traffic volume, travel time reduction and reduction of speed deviation. When applied to supply-driven congestion, the results indicated that variable speed limits were most effective for traffic volumes greater than or equal to 2000 vehicles per hour. Benefits from variable speed limits included reduction the queue time, reduction of the number of stops, avoidance of congestion and reduced travel time. Their findings also showed that variable speed limits could delay the occurrence of congestion at higher volumes but could not prevent it altogether.

2.2.1 Real World Applications of Variable Speed Limits

Variable speed limits have been implemented in real-world networks in areas all over the world, including the United Kingdom, the Netherlands, Germany, Australia, New Zealand and even the United States. According to Wilmot and Khanal (1999), Sydney, Australia uses variable message signs coupled with a fog warning and speed advisory system to provide recommendations to drivers in real time, based on the visibility and speed of the preceding vehicles. In the United Kingdom, mandatory variable speed limits are used to improve traffic flow and safety on the M25 London Orbital Motorway. Their incident detection system, Motorway Incident Detection and Automatic Signaling (MIDAS), monitors traffic flow and speeds to adjust the speed limit in order to avoid or delay congestion when the system is operating near capacity. High levels of compliance are obtained through the use of enforcement
cameras. MIDAS is also used to detect queuing behavior and prevent secondary accidents at the back of queues as they form. Also on the M25 in England, Borrough (1997) reported that enforcement of Variable Speed Limits effectively decrease the number of crashes. Speed limits were adjusted in response to the level of congestion on the M25, which is one of the most congested freeways in England. Congestion was detected by measuring traffic density and speed from loop, and speed limits were lowered in increments as congestion increased. Speed cameras were also used to enforce the speed limits. The study found that motorists were less inclined to switch lanes when a "faster lane" no longer existed. They were also more inclined to keep to the inside lane and to keep proper distances between successive vehicles, which resulted in smoother traffic flow. Results show that traffic crashes decreased by 28% during the 18 months of operation.

In the Netherlands, Van de Hoogen (1994) used variable speed limits to homogenize the traffic flow on a 20-km section of the 6-lane A2 Motorway connecting Amsterdam and Utrecht. The primary function of the control was to decrease not average speed, but rather speed differences, on the freeway. According to the authors, decreasing speed differences imply more regulated headways and a reduction in the number and severity of shockwaves. The authors found that the implementation of variable speed limits provided calmer traffic and improved the distribution of the traffic over the available road space.

In Finland, Rämä (1999) used two roadways with similar speed limits, roadway geometries and weather to test the effects of variable speed limits. On one road she used static speed limit signs, and on the other she used variable message signs. She found that the use of variable speed limits led to lower average speeds and decreased the variability in speed.
In the United States, variable speed limits are used in Washington State, New Jersey, New Mexico and Wisconsin in order to properly identify and implement a safe speed limit when inclement weather converges on vulnerable freeways (McLawhorn, 2003). The systems in these states have been successful in detecting dangerous roadway conditions and suggesting a safe speed to the drivers. They have been effective in reducing average speed in dangerous conditions and decreasing the risk of crashes on the road.

2.2.2 Micro-simulation and Variable Speed Limits

Micro-simulation has often been used to test the safety effect that variable speed limits can have in many different scenarios. Researchers have simulated instances such as freeways, work zones and inclement weather conditions to test variable speed limits and their effects on performance measures used to estimate crash risk.

Park and Yadlapati (2003) used VISSIM to evaluate the safety benefits that could be obtained through the use of variable speed limits (VSL) in work zones. They proposed a surrogate crash measure, the minimum safe distance equation (MSDE), and a method used to maximizes MSDE (safety measure) and travel time. VISSIM was used to evaluate a proposed VSL control logic (logic 3) against two other common control logics used in work zones. Park and Yadlapati used varying compliance rates (70, 80, 100%) and two demand conditions (undersaturated and oversaturated). Their recommendations included a case study of field implementation of VSL at work zone using 45 mpg for the base case and deployment of the proposed VSL logic 3.

Studies by Lee et al (2004, 2006) used PARAMICS to determine the safety benefits of variable speed limits and ramp metering on freeways. Lee (2003) used historical loop data,
collected from a 10 km stretch of the Gardiner Expressway in Toronto, Canada, to develop a log-linear model that identified several real-time crash precursors. By simulating a 2.5 km portion of this freeway in PARAMICS and applying the model, they found that the individual applications of variable speed limits and ramp metering successfully reduced the crash risk along the freeway.

Abdel-Aty (2005) expanded on Lee’s work by simulating approximately 9 miles of Interstate-4 in Orlando, FL in PARAMICS and using real traffic flow data to calibrate the model. A sophisticated crash prediction algorithm developed by Abdel-Aty et al (2004), based on historical data from the same stretch of Interstate-4, was utilized in this study. Using these crash risk models as the basis for his research, Dilmore (2005) found that the application of variable speed limits along the freeway could successfully reduce the crash risk, as well as the travel time, at particular locations in the high speed regime. However, he did not find the implementation of variable speed limits in the low speed regime to have as considerable an effect due to congestion limitations. Abdel-Aty and Dhindsa (2007) further expanded on Dilmore’s research by examining the potential benefits of ramp metering and variable speed limits. Dhindsa used the same PARAMICS network and crash prediction models as Dilmore (2005) but focused more intently on the improvement of real time crash risk in the low speed regime. Dhindsa also used a more sophisticated variable speed limit implementation strategy and considered the simultaneous implementation of variable speed limits and ramp metering. He found that using variable speed limits on a corridor-wide level (as opposed to a fixed location) has a significant effect in reducing crash risk potential on the freeway.
2.2.3 Variable Speed Limit Implementation Techniques

The previous sections have dealt primarily with the potential uses of variable speed limits, as well as their applications, both in the field and in micro-simulation. The techniques by which variable speed limits were implemented will here be summarized, and the specific differences between those techniques and this study’s will be discussed.

Park and Yadlapati (2003) used VISSIM to evaluate the safety benefits that could be obtained through the use of variable speed limits (VSL) in work zones. They used loop detectors to detect the presence of vehicles and to collect speed and headway information. Loop detectors were placed upstream of the work zone, at the beginning of the work zone, and halfway through the work zone. Compliant and non-compliant vehicle classes were defined within the simulation. The Application Programmer Interface (API) for VISSIM was used to implement one of three variable speed limit logics over a fixed cycle time of 5 minutes. Logic 1 tries to reduce the speed differences between the upstream work zone area and the activity area by posting the average speed at the activity area as the speed limit upstream of the work zone. Logic 2 tries to reduce the speed differences between the early warning area and the merge area by gradually changing the speed limit between these two points. Essentially the early warning area is assigned a speed limit 10 mph higher than the average speed of the merge area. Logic 3 uses both vehicle speed and headway data to choose the best speed for the work zone. When volume is low, the speed limit is raised to increase mobility. When the volume gets high, the speed limit is lowered to increase safety. Speed change increments are constrained to 5 mph per cycle. The three logics are compared against one another, and the safety measure MSDE and travel time are maximized for the best technique.
Lee (2004) used the PARAMICS micro-simulation software to model variable speed limits on a 2.5-km stretch of freeway in Canada. Loop detectors collect traffic flow information in real time, and that information is used to calculate crash potential. A 3-hr morning peak period is modeled from 7 AM to 10 AM. Three variables comprise the experimental design of this study: (1) the threshold of crash potential at which variable speed limit intervention occurs, (2) the duration of the intervention, and (3) the extent of the intervention (change in speed limit). The best combination of these factors is sought to minimize crash potential.

The crash potential threshold is considered over two levels: high and low. Depending on the geometry of the road, merge/diverge or straight section, different crash potential values were considered to be high or low. The duration of the intervention was considered over four levels: 2 minutes, 5 minutes, 10 minutes and the whole duration (3 hours). In the case of 3 hours, a single speed limit was enforced throughout the whole simulation. In terms of the extent of the intervention, Lee considered the design speed limit (90 km/hr) and six cases of reduced speed limit (80, 70, 60, 55, 50 and 40 km/hr). A control strategy was also put in place to prohibit excessively abrupt reduction in speeds in time. This strategy basically only allowed the reduced speed limit to be one interval below the current speed limit (i.e. it would take two steps of implementation to change the speed limit from 90 km/hr to 70 km/hr).

In later research, Lee (2006) used PARAMICS to model variable speed limits on a 4.7-km stretch of the Gardiner Expressway in Toronto, Canada. Lee once again addressed his three main questions through the use of a crash potential threshold variable, a duration variable, and a variable describing the extent of the intervention. In this case, Lee used four levels for the crash potential threshold, once again specifying separately for merge/diverge and straight sections. Four durations of intervention time were considered (2, 5, 10 and 15 min). Three strategies were
considered for the extent of the intervention: (1) set intervention speed limit equal to a fixed value (80, 70, 60, 50, 40, 30 and 20 km/hr); (2) intervention speed limit is set equal to approximate average speed downstream (500 m); and (3) intervention speed limit is set equal to “transition speed”, or the average of observed speeds upstream and downstream. Again, combinations of these factors are analyzed to minimize crash potential.

Dilmore (2005) applied variable speed limits to test the effect that variable speed limit implementation could have at three stations on the Interstate-4 corridor in Orlando, FL. Dilmore considered several variables. He first considered the application of variable speed limits in terms of magnitude and space. This included upstream lowering, downstream raising, and the combination of both. The downstream raising of speed limits is an intuitive strategy based on the intrinsic characteristics of a developing queue. As the queue is forming backwards, the upstream vehicles are slowed to keep them from approaching the queue too quickly, while the front of the queue is expected to dissipate swiftly with raised speed limits in place. He also tested for the maximum speed limit change (10 and 15 mph) and whether it should be implemented gradually or abruptly (over space). For scenarios that showed promise, he focused second on the spatial extent of the implementation. He used the three values for upstream (2, 4 and 6 mi) and three values for downstream (2, 4 and 5.5 mi).

Thirdly, Dilmore focused on the temporal application of variable speed limits. He tested whether the new speed limits should be applied abruptly or whether they should be changed in intervals of 5 or 10 mph, every 5 or 10 minutes. Finally, Dilmore took used what appeared to have results and tested the best gap distance. The gap is the space between the lowering of speeds upstream and the raising of speed downstream. Essentially, it is the space between the
two segments that maintains the original speed limit. Dilmore tested for four gap distances (0, 1, 2 and 3 mi).

There are some significant drawbacks to Dilmore’s design. Two drawbacks are that he only tested three stagnant locations for a stagnant period of time. There is no trigger, as in Lee’s study, that warrants the use of variable speed limits. Because of the time constraint, the implementation of variable speed limits is limited in its effect. It cannot be said whether the speeds are able to be restored at any point in the study, since there is no set of criteria to warrant it. Because of the spatial constraint, it is hard to say whether variable speed limits would have the resulting effects at all. This study was primarily intended to show the effects of variable speed limits downtown and out of downtown, but there is no doubt that, if variable speed limits are to be applied, it would involve a corridor-wide strategy. This relates directly to the experienced crash migration problem. A corridor-wide strategy could have the capability to alleviate crash migration.

Dhindsa (2006) expanded on Dilmore’s work by creating an API in PARAMICS that is able to test for criteria and implement variable speed limits if warranted. Dhindsa, rather than using a specified location, observed all 17 locations over a 9-mile stretch of freeway. His intentions were to smooth the speeds over the whole network rather than focus on a single location. Dhindsa implemented variable speed limits based on the speed difference from station to station. If the difference in speed is negative from the upstream and downstream stations, variable speed limits were considered. He considered two speed change magnitudes (5 and 10 mph). Dhindsa considered four implementation strategies: (1) Decrease speed limits upstream and downstream, (2) Decrease speed limits upstream and Increase downstream, (3) Lower speed limits upstream only, and (4) Raise speed limits downstream only. He considered only changing
the speeds by a maximum of 5 mph per change in location (i.e. a 10 mph change would occur over two locations). These strategies were tested alone and in conjunction with ramp metering.

This study builds and expands on ideas from Lee (2004, 2006), Dilmore (2005), and Dhindsa (2006). Like Dhindsa’s, this study is intended to evaluate a corridor-wide variable speed limit strategy that can effectively reduce the crash risk at unsafe locations. This network, like Dilmore’s and Dhindsa’s, is a PARAMICS model of Interstate-4, but has been expanded from 9 miles to 36.25 miles. It covers the whole of downtown, and it stretches from Disney to Lake Mary. In this study, vehicle speeds are collected by loop detectors at every station on the network, and 5-minute average speeds are calculated. Every five minutes, the difference in average speeds from one detector to the next is analyzed, and, if the difference is above the threshold, variable speed limits will be implemented. Three implementation strategies are tested in this study: (1) Decrease speed limits upstream by 10 mph, (2) Decrease speed limits upstream by 5 mph, and (3) Decrease speed limits upstream by 5 mph and Raise speed limits downstream by 5 mph.

Perhaps the most unique thing about this study is the introduction of homogeneous speed zones. This will later be discussed in depth. In short, it is the idea of collecting similar, contiguous segments of road into homogeneous groups and distinguishing them from other homogenous groups. The similarity of these groups is determined by their average speeds, from one station to another. The entire network, then, is not made up of 70 stations, but of a much smaller number of homogeneous speed zones. This study includes two thresholds for defining the speed zones.

The spatial extent, or the distance over which the variable speed limits should be applied, is another important factor. All studies reviewed here, and to the knowledge of the author, have
used a number of set distances and applied them to their study. A fixed distance is hard to justify as the best option, considering the dynamic characteristics and vibrancy of traffic flows on freeways. For instance, a backward-forming queue may necessitate decreased speeds upstream and increased speeds downstream. It may do no good to increase speed limits 2 miles downstream if the queue is, say, 5 miles long. For this reason, a dynamic distance was considered for variable speed limit application in this study. Since the average speed is a good indicator of an existing queue (or congestion), and homogeneous speed zones are already being applied to the network, the applied implementation distances were chosen as all of or part of the homogeneous speed zone in which variable speed limits were applied. This causes the spatial extent of implementation to depend directly on the current speed profile of the network.

The length of time over which variable speeds limits were applied is the last variable considered in this study. This study allowed for an implementation period of 5 minutes or 10 minutes. However, speed differences problem were reassessed at the end of the implementation period, and, if needed, variable speed limits could be immediately reapplied. So the length of 5 or 10 minutes is really a minimum implementation time, and the maximum depends wholly on the needs of the network.

In comparison with previous studies, this study is exceptional in that it uses a corridor-wide implementation strategy of variable speed limits for a network that is considerably larger than any other evaluated in previous research. Not only this, but it evaluates the system based on network-defined homogeneous speed zones rather than arbitrary lengths of freeway. Furthermore, this study assesses the effectiveness of three separate strategies over different lengths and periods of time. All of this is repeated for off-peak volumes, approaching-peak volumes, and peak volumes of Interstate-4.
CHAPTER 3. METHODOLOGY

3.1 PARAMICS Micro-Simulation Software

PARAMICS (PARAllel MICroscopic Simulation) is the simulation software of choice used in this project (Section 2.1.4). PARAMICS is a micro-simulation modeling program used to model the movement and behavior of individual vehicles on urban and highway road networks. The PARAMICS suite consists of the Modeler, Processor and Analyzer (Quadstone Limited, 2005). The PARAMICS Modeler is used to build the network. It creates a visualization of the road network and traffic demands using a graphical user interface (GUI). The speed of the simulation depends upon the computer processing power, the size of the network and the number of vehicles on the network.

The PARAMICS Processor is used to run multiple simulation scenarios in batch mode without the visualization provided by the Modeler. Without the high computer processing demand due to the GUI for visualization, the processes are able to run much quicker in the Processor.

The PARAMICS Analyzer, which was not used for this project, allows the user to read the output from the simulation model and compare post processing simulation results. This allows for the comparison of processed data to real data, as well as the comparison of different simulation scenarios.
3.2 Area of Study

3.2.1 Interstate-4 Study Corridor

Interstate-4 is a major freeway that runs across the state of Florida from Tampa upwards to Daytona, bisecting the city of Orlando along the way. It is used excessively by tourists traveling between high-demand tourist destinations such as the beaches of Tampa Bay, Busch Gardens (Tampa), Walt Disney World Resorts (Orlando), Sea World (Orlando), International Drive (Orlando), Universal Studios Florida (Orlando) and the famous shores of Daytona Beach.

The study corridor used for this study is a 36.25 section of Interstate-4, running from SR 192 in the southwestern end all the way up to Lake Mary Boulevard in the northeastern end. A map of the study corridor is shown in Figure 3-1. Though the direction of traffic is described as east-west, the center of this section of Interstate-4 actually runs north-south along the western perimeter of Orlando’s downtown metropolitan area.

The majority of this section of freeway is 6-lane and is divided by a concrete, median wall. The corridor is equipped with loop detectors in each lane, every half mile. As will be discussed later, these loop detectors are essential to providing information for the creation of the crash risk models, as well as the calibration and validation of the simulation network. Their presence is assumed for the implementation of variable speed limits in this study and is a necessity if variable speed limits are to be implemented in the field. In 2003, FDOT approximated that the composite AADT on Interstate-4 through the Orlando area was 183,000 vehicles per day. The composite AADT is an average AADT taken from different locations throughout Orlando (FDOT, 2003).
3.2.2 Speed Limit Stratification

Throughout the 36.25 mile stretch of freeway, three different speed limits are assigned to Interstate-4 in five different speed limit areas. The corridor begins in the east with a high speed limit of 65 mph. It is then reduced to 55 mph as it approaches downtown and then to 50 mph as it enters the downtown area. Because of the speed variation, and the sign of the speed difference that this causes, these first three speed limit sections are the most dangerous for the eastbound direction. Therefore, it is in these areas that variable speed limits are expected to have the
greatest benefit. As the corridor leaves downtown, the speed limit is raised to 55 mph, and then it is raised again to 65 mph in the west.

Figure 3-2  Stratification of Speed Limits

The speed limit stratification of the study corridor is shown in Figure 3-2. In this figure, the 65-mph speed limit zones are represented by the blue color. They are located at the extents of the study corridor. The 55-mph speed limit zones are represented by the green color. These zones buffer the high-speed extents and the low-speed downtown area. The 50-mph speed limit zone, located throughout the downtown area, is represented by the red color in Figure 3-2. This area is generally where congestion occurs most frequently and where the crash risk is the highest.
3.3 Network Building

The simulated network of Interstate-4 used in this study was built by Gayah, with the help of the present author, in 2006. The detailed processes involved in the network construction, as well as the network calibration and validation, are thoroughly reviewed by Gayah (2006). These steps will be summarized here, but see Gayah (2006) for a further explanation. Chronologically, the steps involved include:

- Overlay Generation
- Coding of Nodes and Links
- Coding of Zones and Vehicular Demand
- Coding of Loop Detectors
- Network Calibration
- Creation of Origin-Destination Matrix
- Network Validation

This section will deal exclusively with overlay generation and the coding of nodes and links, zones and vehicular demand, and loop detectors.

3.3.1 Overlay Generation

An overlay of Interstate-4 was used to accurately code the geometry of the freeway in PARAMICS. Previously, Dilmore (2005) used a combination of aerial photography obtained from the Seminole County Property Appraiser’s Office and the Orange County Property Appraiser’s Office as the base for the overlay to create a smaller section of the same network. This aerial photograph was also combined with AutoCad drawings obtained from the Orange...
County Transportation GIS department. A sample of this hybrid overlay is shown below in Figure 3-3.

The study section for this research, however, is 36 miles long compared to Dilmore’s 20 miles. The remaining geometry was obtained from aerial imagery software: the Google Earth Aerial Mapping program. The aerial images in the Google Earth program available for Orlando, Florida were taken during April 2002. Individual pieces of the aerial were obtained from the Google Earth program and then assembled into one master file using the Adobe Photoshop program. Great care was taken by Gayah (2006) when extracting the aerials from Google Earth to ensure that the graphical scales of all individual pieces were the same.
Since PARAMICS only accepts *.dxl and *.bmp files as overlays, the master aerial was then saved as a *.bmp file for import into the simulation program. However, this file was too large to be imported directly (greater than 100 MB) so the master overlay had to be split into seven distinct segments to be imported into PARAMICS. A sample of one of the 7 segments is shown below in Figure 3-4. Each box in the sample grids shown in the figure represented a 500 ft x 500 ft square and was used in the scaling of the aerial in PARAMICS.

Once each segment was imported, it was scaled individually (using the grid boxes as shown in Figure 3-4) and then reassembled in the PARAMICS program to form the master aerial. This was checked once again with a single, low-resolution image to ensure that there were no scaling errors. Once this was completed, the next step was to draw the roadway network.
Figure 3-4 One of Seven Distinct Sections of Aerial Used in Network Construction from Gayah (2006)
3.3.2 Network Geometry

It was determined to code the network as two one-way roads (one eastbound, one westbound) as opposed to one two-way freeway. This was done for a number of reasons. First of all, the two directions had different speed limits and geometries that could only be properly modeled if the two directions were coded independently of one another. Secondly, the merging of links, such as at on-ramp and off-ramps, forces the creation of an additional node. If a two-way road was constructed, then the addition of ramps on either side would effectively split the links on both sides and create a large number of short links on the network. The maximum value for the awareness distance of any ramp cannot be greater than the length of the nearest link. Therefore, having a single roadway with many shorter links would result in improper vehicular behavior at the on-ramps, since will not be sufficient gaps for vehicles to merge onto the freeway. Separating the two directions of traffic would reduce the number of short links since each direction would only have to split links for its own on-ramps or off-ramps. Refer to Gayah (2006) for a more detailed summary.

After determining to code both directions of the freeway separately, the nodes that defined the links for each direction were first coded into the simulation. A new node was created at every location where the roadway changed geometry (curved section to straight section), number of lanes, speed limits, or a ramp entered the mainline. Each of the nodes was then connected using links, which represented the roadway that the simulated vehicles would use. Link categories were created that contained information about the number of lanes and speed limit on the link. Each link was assigned to a category. The information from the number of lanes and curvature of each link was obtained visually from the Google Earth aerial maps. The
speed limit for each link was determined from the placement of speed limit signs that were found from watching a video stream of I-4 obtained from the Roadway Characteristics Information (RCI) obtained from the Mainframe Database operated by the Florida Department of Transportation (FDOT). Once the links were created, the next step was to adjust the kerb points in the links to match the real roadway geometry. Kerb points are locations on each link that define the shape of the link more specifically than simply using the nodes and curvature tools. Adjusting the kerb points is essential to ensure that the simulated vehicles move freely between links without interruption.

PARAMICS includes a special function to code an on-ramp and another to code the deceleration lane that typically precedes an off-ramp. These two functions, however, could only be used in specific cases and do not represent the majority of ramp types found on Interstate-4. Therefore, most of the ramps in the network that could use the on-ramp or slip lane function were coded using these functions while the rest were coded as a regular merge or diverge area. Figure 3-5, from Gayah (2006), shows a brief summary of when the PARAMICS on-ramp or slip lane functions were used and when the merge / diverge areas were chosen.
There was one major problem that was noted concerning vehicular behavior on PARAMICS defined on-ramps. The problem was that vehicles were not merging onto the mainline freeway with regularity causing the ramp to back-up and not allowing any vehicles on
the network. This was alleviated by increasing the signpost distance for the ramp. The signpost distance is essentially a measure in PARAMICS given to drivers for their awareness of downstream geometry changes in the freeway. Therefore, increasing the signpost distance for the ramp increased the awareness of the mainline vehicles to the ramp, and thus caused the mainline vehicles to change lanes to give the on-ramp vehicles sufficient gaps to merge onto the mainline freeway. Once the signposting distance was adjusted, merging behavior using the ramp function behaved as would be expected in the field.

3.3.3 Addition of Zones, Vehicles and Loop Detectors

After completion of the network geometry, the origin and destination zones of the network were created. On the freeway, each origin and destination zone were simply represented by on-ramps and off-ramps, respectively. Once finished, vehicles were loaded artificially onto the network to test the vehicular behavior. It was noted in many places, particularly merge and diverge areas where the number of lanes changed at the end of one link and the beginning of another, that vehicles were changing lanes improperly (for example, a vehicle changing from the right-most lane directly to the left-most lane while crossing over other simulated vehicles in the process). This problem was addressed by adjusting the nextlanes function, which defines which lanes are available for vehicles on a specific lane traveling from one link to the next.

One of the primary reasons that this section of Interstate-4 was chosen as the study section is because this specific section of freeway contains induction loop detectors embedded in the asphalt. These loop detectors are spaced approximately every 0.5 miles apart on the roadway and give values of the average speed, volume, and lane occupancy at 30-second intervals for the mainline freeway lanes. Therefore, in order to make sure the data obtained from the
PARAMICS network matched the real-life data obtained in the field, the detectors had to be placed in the same locations. The detector mileposts were obtained from the FDOT Roadway Characteristics Information (RCI) database and placed in the appropriate positions along the network. After every simulation run, during the post-processing stage, the loop data was extracted from the PARAMICS data files and stored in the same format as the loop data obtained from the real I-4 roadway. The data from the loop detectors is used to calculate the crash risk measure every 5 minutes throughout the network. This process is detailed in Section 3.6.

3.4 Network Calibration and Validation

Typically, the most tedious process in creating any micro-simulation network is the Calibration and Validation procedures. The calibration procedure involves changing pre-specified model parameters that affect driving behavior of the simulated vehicles to match the driving behavior of drivers in the real network. Although the default values have been shown to approximate decent driving behavior, tweaking the parameters would more accurately represent driving behavior for the particular study area. A review of the literature shows that the values of these parameters are found by comparing the flow and travel time along the network (Bertini et al, 2002; Abdullhai et al, 2002; Trapp, 2002, and Stewart, 2001). The calibration parameters that are typically changed are mean headway and driver reaction time (Gardes et al, 2002; Abdulhai et al, 2002; Lee et al, 2004).

3.4.1 Dhindsa’s Calibration Procedure

In a previous study, Dhindsa (2006) created a 20-mile segment of Interstate-4, which is contained within the 36-mile section used in this study. Dhindsa went through an exhaustive
calibration procedure to properly calibrate the network for Interstate-4 drivers in Orlando, FL for four parameters: driver reaction time, mean drive headway, queuing distance, and queue speed. The mean driver reaction time refers to the time vehicles take to react to events around them such as a merging vehicle coming into its lane or the deceleration of a vehicle that it was following. The mean driver headway refers to the average time gap that vehicles try to maintain while moving through the freeway. The queue speed is the maximum speed with which queuing behavior occurs. Vehicles traveling below this speed will behave using the built-in PARAMICS queue behavior. Queuing distance is the minimum distance between two vehicles that causes queuing behavior to end. Once this distance is achieved between successive vehicles in a queue, the queuing behavior of that vehicle will end. Because of the comprehensiveness of Dhindsa’s work and the fact that his network is embedded in the center of this one, Gayah (2006) opted to use Dhindsa’s calibration parameters in this present network.

Dhindsa’s procedure, which is outlined in detail in Gayah (2006) and in Dhindsa (2006), basically consisted of the calibration of the origin-destination (OD) matrix and the calibration of flows and speeds. The OD matrix was calibrated by first specifying trips that could not occur with zeros in the matrix. These include trips originating from off-ramps, trips ending at on-ramps, trips beginning and ending on different directions of the freeway, and trips originating downstream of the destination point. None of these are possible, so they were assigned zeros in the OD matrix. A simple gravity model was then used to fill in the remaining OD matrix cells, and the cells with observed errors were adjusted iteratively until a minimum error was reached. The final error of the OD matrix, comparing the simulation on-ramp and off-ramp volumes with those from the field, was 4.10%.
After calibrating the OD matrix, Dhindsa looked to the flows and speeds. The volumes from the simulation runs were compared to the data obtained from the loop detectors embedded in I-4. The flows in the simulation were checked against the average flows taken on Tuesdays, Wednesdays and Thursdays between September 20, 2003 and November 20, 2003. The flows, which were initially lower than expected, were adjusted to get a more adequate value. Additionally, plots were made that compared the average simulated speed and the average speed from the loop detectors in 5-minute intervals. These plots contained 10% error bars for the simulation data to show the acceptable range of speeds that the simulation data represented. An example of such a plot is shown below in Figure 3-6.

![Figure 3-6 Sample Plot Showing Real Average Speeds with 10% Error Bars and Simulated Speed from Dhindsa (2006)](image-url)
Various values of the mean driver headway and mean driver reaction time were selected to minimize the error in flows and speeds, while holding the queue speed and queue distance constant at 8 mph and 9 ft, respectively. These values for queue speed and queue distance were selected because increasing them tended to increase the persistence of queues at any location once formed.

Based on the errors in the flows and speed, the final mean headway and mean driver reaction time were chosen to be 1.00 seconds and 0.45 seconds, respectively. These values were close to previous values used on a similar project performed by Dilmore (2005) and, therefore, found to be acceptable in the calibration of driver behavior along Interstate-4 in Orlando, Florida.

3.4.2 Updated Origin-Destination Matrix

As noted in the previous section, the same calibration parameters found by Dhindsa (2006) were used in this study. However, with the network increasing in size and with the addition of several on-ramps and off-ramps in both directions, the origin-destination matrix had to be reconfigured for this network. Gayah (2006) used a similar procedure to calibrate this network as Dhindsa used. Zero counts were applied to impossible trips, as described in Section 3.4.1. Gayah used traffic counts from the Florida Traffic Information (FTI) 2003 CD, hourly data obtained from the Center for Advanced Transportation Systems Simulation (CATSS), and Dhindsa’s initial origin-destination matrix as a starting point for his updated origin-destination matrix. Gayah then applied a simple gravity model to complete the OD matrix.

Gayah found, however, that the PARAMICS simulation did not produce the expected queues in the downtown areas when his OD matrix was applied. The OD matrix was then modified to increase vehicles around the Interstate-4 / SR-408 interchange and at the terminus
zones of the network. These modifications served to induce congestion and form queues in their places as observed in the field.

3.4.3 Network Validation

With the updated OD matrix compete for the network, a brief validation procedure was carried out to ensure the network’s integrity. Data extracted from Interstate-4 loop detectors during the period of September 23, 2003 to November 20, 2003 was checked against the output from PARAMICS. Notice that this was the same time period used by Dhindsa (2006).

Twenty stations were selected to check the simulated data against the field data. These twenty stations were chosen based on loop data availability and location on the network, so as to get the best representation on the entire network while focusing somewhat on the downtown area. The GEH statistic was used to compare the two sets of data. The GEH statistic is essentially a modified chi-square measure that accounts for the relative and absolute difference in the observed (field) and simulated (PARAMICS) traffic data. According to Oketch and Carrick (2005), the GEH statistic is widely employed by PARAMICS users. This statistic was applied at each station as shown below:

\[
GEH = \sqrt{\frac{(M - O)^2}{1/2 \times (M + O)}} \tag{3.1}
\]

In the equation for GEH, M is the hourly flow rate obtained from the simulation and O is the hourly flow rate obtained from the field. In general, a good fit of data could give a GEH of
less than 10, with a GEH of less than 5 being very good. A GEH greater than 10 implies that the simulated data is not a good fit to the observed data.

Ten simulations were run, and the flows were averaged to get an hourly volume at each of the 20 stations. This data was then compared to the data extracted from the loop detectors in the field. Table 3-1 shows the stations, observed volumes, simulated volumes and the GEH statistic that Gayah (2006) used to validate the model.

Table 3-1  Comparison of Observed vs. Simulated Flow Rates from Gayah (2006)

<table>
<thead>
<tr>
<th>Station</th>
<th>Flow From Loop Data (veh/hr/lane)</th>
<th>Simulated Flow (veh/hr/lane)</th>
<th>GEH Statistic</th>
</tr>
</thead>
<tbody>
<tr>
<td>7</td>
<td>932</td>
<td>700</td>
<td>8.14</td>
</tr>
<tr>
<td>18</td>
<td>1102</td>
<td>1031</td>
<td>2.17</td>
</tr>
<tr>
<td>19</td>
<td>1104</td>
<td>1031</td>
<td>2.25</td>
</tr>
<tr>
<td>20</td>
<td>1129</td>
<td>959</td>
<td>5.26</td>
</tr>
<tr>
<td>21</td>
<td>1073</td>
<td>954</td>
<td>3.76</td>
</tr>
<tr>
<td>22</td>
<td>1183</td>
<td>1006</td>
<td>5.35</td>
</tr>
<tr>
<td>23</td>
<td>934</td>
<td>933</td>
<td>0.01</td>
</tr>
<tr>
<td>24</td>
<td>1011</td>
<td>920</td>
<td>2.91</td>
</tr>
<tr>
<td>26</td>
<td>914</td>
<td>994</td>
<td>2.59</td>
</tr>
<tr>
<td>30</td>
<td>1380</td>
<td>1038</td>
<td>9.82</td>
</tr>
<tr>
<td>34</td>
<td>1317</td>
<td>1316</td>
<td>0.01</td>
</tr>
<tr>
<td>35</td>
<td>1304</td>
<td>1333</td>
<td>0.80</td>
</tr>
<tr>
<td>36</td>
<td>1331</td>
<td>1251</td>
<td>2.22</td>
</tr>
<tr>
<td>37</td>
<td>1342</td>
<td>1494</td>
<td>4.03</td>
</tr>
<tr>
<td>41</td>
<td>1550</td>
<td>1748</td>
<td>4.87</td>
</tr>
<tr>
<td>42</td>
<td>1535</td>
<td>1706</td>
<td>4.24</td>
</tr>
<tr>
<td>43</td>
<td>1870</td>
<td>1884</td>
<td>0.33</td>
</tr>
<tr>
<td>44</td>
<td>1520</td>
<td>1917</td>
<td>9.57</td>
</tr>
<tr>
<td>49</td>
<td>1840</td>
<td>1791</td>
<td>1.15</td>
</tr>
<tr>
<td>50</td>
<td>1849</td>
<td>1987</td>
<td>3.16</td>
</tr>
<tr>
<td>52</td>
<td>1532</td>
<td>1581</td>
<td>1.24</td>
</tr>
<tr>
<td>53</td>
<td>1549</td>
<td>1483</td>
<td>1.69</td>
</tr>
<tr>
<td>56</td>
<td>1495</td>
<td>1514</td>
<td>0.50</td>
</tr>
<tr>
<td>57</td>
<td>1411</td>
<td>1257</td>
<td>4.22</td>
</tr>
<tr>
<td>61</td>
<td>1389</td>
<td>1250</td>
<td>3.83</td>
</tr>
</tbody>
</table>
As shown in the table, 80% of the loop detectors have a GEH value under 5. Two other observations border above 5, and the other two are less than 10. The average GEH is well under 5 at 3.36, and no station has a GEH greater than 10. Therefore, Gayah’s updated origin-destination matrix proves to also be a good fit of the hourly flows observed in the field, as well as expected queue formation. This process validates the network and therefore allows conclusions to be drawn regarding the implementation of various ITS strategies, including variable speed limits.

3.5 Implementation of Variable Speed Limits

There is much to consider for the implementation of variable speed limits in a corridor-wide scheme. First of all, the important factors that comprise the VSL strategies must be considered. These factors must be fully capable of accomplishing the primary objectives of VSL implementation. Second, it must be determined when is the best time to implement variable speed limits. As previously mentioned, this is to be a dynamic implementation strategy, so an appropriate threshold must be determined. Moreover, this threshold must have the capability of being calculated easily and often, in real-time, and in the field. Lastly, the strategy, along with its threshold, must be made able to be implemented into the simulation model. This is done through the use of PARAMICS Application Programmer Interface (API).

3.5.1 Important Factors

The first thing to consider is the important factors that will comprise the VSL implementation strategies for this study. These factors must be fully capable of accomplishing the primary objectives of VSL implementation. In this case, as in the case of most experiments,
the strategy’s level of complexity should be minimized while simultaneously maximizing its level of effectiveness. As mentioned in the Literature Review, many factors have been considered important in producing effective results through the use of variable speed limits.

3.5.1.1 Determining the Target Speed Limit

The most important factor is the actual change in speed limits that is considered. There are multiple approaches to determining the target speed limit. One approach could include setting the speed limit equal to some all-encompassing “safe speed”. In this approach, speed limits over the entire corridor would be reduced to some safe speed when the situation becomes dangerous enough. Different safe speeds could be tested to determine the most effective choice. Another approach for determining the target speed limit could depend on the average speed at some other location. For instance, the target speed limit could be set equal to the average speed downstream, or the target speed limit could be set to the average of the upstream and downstream average speeds. In both instances, the target speed limit is determined by the average speed at another location or other locations.

A third approach for determining the target speed limit at the station of interest is to depend on the speed limit at that respective location. In this approach, the speed limit being changed would be changed based on its current speed limit. Also, the potential to raise speeds or lower speeds at will is present, and the target speed is a function of the speed limit at that location. This approach is actually the best of the three mentioned, for a couple of reasons. It is better than the first approach because it is dynamically adaptable to the entire network, as opposed to a “one-size-fits-all” strategy. It is better than the second approach, due to a flaw in the second approach’s strategy. The second approach is dynamic based on the speeds in the
surrounding locations. However, wholly basing the target speed limit on the average speeds of the surrounding locations cannot be used in the field, due to regulatory policies and congestion limitations. In terms of regulatory policies, the second approach (1) could not likely produce a target speed limit that is desirable by 5 mph without rounding and (2) could potentially have the “snowball” effect of lowering speeds or raising speeds beyond proper design standards for the freeway. In terms of congestion limitations, setting speed limits below certain thresholds is prohibited on most freeways (i.e. 40 mph on Interstate-4), so this approach could not be used on most freeways in highly congested situations. This third approach not only is dynamically applied to each location, but also conforms to regulatory policies and, since it is based on the speed limit, is not susceptible to the “snowball” effect.

3.5.1.2 Spatial Extent of VSL Implementation: Homogeneous Speed Zones

Another important factor to consider is the length over the network in which variable speeds limits should be applied. All studies mentioned in the Literature Review used a set of fixed distances over which VSL was applied (e.g. Lee, 2004, Lee et al., 2006; Dilmore, 2005; Dhindsa, 2006). A fixed distance is hard to justify as the best option, considering the dynamic characteristics and vibrancy of traffic flows on freeways. For instance, a backward-forming queue may necessitate decreased speeds upstream and increased speeds downstream. It may do no good to increase speed limits 2 miles downstream if the queue is, say, 5 miles long. For this study, then, a dynamic distance was considered for variable speed limit application. As far as the author knows, this is uniquely the first time a dynamic distance has been considered for VSL applications.
In order to include a dynamic application distance in the VSL strategy, the concept of homogeneous speed zones was introduced. A homogeneous speed zone is the collection of similar, contiguous segments of highway into homogeneous groups, based on average speed, and distinguished from other homogenous groups. The similarity of these groups is determined by the difference in average speeds, from one station to another. The entire network, then, is not made up of 70 stations with 70 average speeds, but of a much smaller number of homogeneous speed zones bordered by average speeds at each extent. This concept is illustrated in Figures 3-7 and 3-8. In these figures, the highway is split into half-mile stations. The station number and the distance covered between each station is shown, as well as the 5-minute average speeds for each station.

<table>
<thead>
<tr>
<th>Distance</th>
<th>0.5</th>
<th>0.5</th>
<th>0.5</th>
<th>0.5</th>
<th>0.5</th>
<th>0.5</th>
<th>0.5</th>
<th>0.5</th>
<th>0.5</th>
<th>0.5</th>
<th>0.5</th>
<th>0.5</th>
</tr>
</thead>
<tbody>
<tr>
<td>Station</td>
<td>12</td>
<td>13</td>
<td>14</td>
<td>15</td>
<td>16</td>
<td>17</td>
<td>18</td>
<td>19</td>
<td>20</td>
<td>21</td>
<td>22</td>
<td>23</td>
</tr>
<tr>
<td>Average Speed</td>
<td>70</td>
<td>72</td>
<td>71</td>
<td>69</td>
<td>64</td>
<td>63</td>
<td>65</td>
<td>66</td>
<td>72</td>
<td>70</td>
<td>69</td>
<td>71</td>
</tr>
</tbody>
</table>

Figure 3-7 Typical Freeway Layout According to Stations

<table>
<thead>
<tr>
<th>Distance</th>
<th>2.0</th>
<th>2.0</th>
<th>2.5</th>
</tr>
</thead>
<tbody>
<tr>
<td>Station</td>
<td>12</td>
<td>13</td>
<td>20</td>
</tr>
<tr>
<td>Average Speed</td>
<td>(70, 69)</td>
<td>(64, 66)</td>
<td>(72, 73)</td>
</tr>
</tbody>
</table>

Figure 3-8 Typical Freeway Layout According to Homogenous Speed Zones

The process of defining homogeneous speed zones involves taking the difference of the 5-minute average speeds at each station and the station upstream. This is measure, known as the speed difference, will be discussed more in section 3.5.2. If the speed difference at a station is less than the speed zone threshold, then the current station is considered part of the same homogeneous speed zone as the station upstream. If the speed difference at the stations of interest is greater than the threshold, then the station of interest is the first station in a new speed
zone. The speed zones threshold in this example was an average speed difference of 5 mph or more. As shown in Figure 3-8, the freeway, which was once arbitrarily defined by 13 separate half-mile segments, now consists of 3 homogeneous speed zones with defined lengths, as well as beginning and ending average speeds. The defining speed zone difference, or the speed difference threshold for the homogeneous speed zones, is considered as a factor in this study in order to properly analyze the sensitivity of this measure.

The spatial extent over which the variable speeds limits are applied is directly related to the length of the homogeneous speed zone in which that speed limit falls. This multiplier, whether it is a fraction of the speed zone or the entire speed zone, was another factor considered in this study. The spatial extent is considered dynamic because the homogeneous speed zones are redefined every 5 minutes in the simulation. Therefore, the spatial extent is dynamic both in its location and in its point in time. This causes the spatial extent of VSL implementation to depend directly on the current speed profile of the network.

3.5.1.3 Temporal Extent of VSL Implementation

The final important factor that was considered was the temporal extent of the VSL implementation. Many previous studies included fixed time periods of application (i.e. 2 min, 5 min, 10 min, 15 min, 30 min), after which normal speed limits would be reinstated. In this study, a couple levels of time will be used to observe whether it is more effective to apply variable speed limits in shorter spurts or prolonged periods. Lee et al. (2006) found that the use of 5-min and 10-min time periods were the most effective. When the network demands the change of speed limits, speed limits will be changed to the target speed limit for the given time period and then returned to normal speed limits thereafter. However, if the network still demands variable
speed limits at the end of the time period, variable speed limits can be immediately reapplied without a second missed. Therefore, in this research, the temporal extent is by no means a maximum time period for application, but rather a minimum time before which the speeds are re-evaluated and have the ability to be changed back to normal.

3.5.2 Threshold Variable

One of the most important factors in the crash risk models is the variance of speed among the station of interest and the nearest stations upstream. Most notable of importance is the difference in the speed between the station of interest and the first station upstream, separated by one-half mile. This measure is referred to as the speed difference. The speed difference is defined as the difference between the 5-minute average speed at the station upstream and the 5-minute average speed at the station of interest, and is described by the equation below:

\[
\text{SpeedDiff} = \text{AvgSpeed}_E - \text{AvgSpeed}_F
\]  

(3.2)

Figure 3-9 is a schematic of the freeway layout, showing the eastbound direction of I-4. The station numbers and detector numbers are shown as they relate in the network. The station of interest, referred to as Station F, is station 20 in the figure. The surrounding stations then take on relative symbol names as shown (i.e. D, E, G and H).

![Figure 3-9 Schematic of Freeway Layout](image-url)
The measure of speed difference is very important for use in the implementation of variable speed limits in this micro-simulation experiment. PARAMICS features an Application Programmer Interface (API) that allows for the utilization of built-in functions within the PARAMICS code that can be used to control, or change, the behavior of vehicles in the network during the simulation. An API was created that could effectively change the speed limit on any desired links in the network, based on specific stimulating circumstances. This API will be used for to test the effectiveness of several factors of variable speed limit implementation and their inter-relatedness to one another (refer to Section 3.5.3).

The primary stimulation needed to implement VSL using the API is the speed difference. However, there is question as too how great a speed difference demands the implementation of variable speed limits in order to reduce the crash risk and homogenize average speeds. Exploratory analysis must be performed in order to determine the critical speed difference at which VSL should be implemented, according to its effect on crash risk. In other words, the threshold value of speed difference must first be observed before formal testing of the other factors (Section 3.5.1) can be carried out.

3.5.3 Variable Speed Limit API

As mentioned previously, PARAMICS features an Application Programmer Interface (API) that allows users to create programming code, written in the C++ computer language, which will access and modify certain pre-existing PARAMICS functions during the simulation run. The product of an API can be as simple as reporting average speeds at different locations in
regular time intervals, or as complex as changing the speed limits at particular locations based on specified conditions. Of course, this study is particularly interested in both.

To implement variable speed limits, at least three basic steps are needed: (1) Collection of pertinent data, (2) Decision to implement based on present conditions, and (3) the application of variable speed limits. An API was written for this project, using C++, that is able to do all of these steps and effectively apply variables speed limits based on changing traffic conditions. The API is complex, but it has been commented for easier understanding and is included in the Appendix of this Thesis.

The first thing the API does when the network is loaded is take stock of the network and assign beacons and pointers to all of the links and detectors on the network. This enables the collection of average speeds and the implementation of new speed limits to every link in the network. Once the simulation is initiated, the API allows for a 30 minutes warm-up period. After 30 minute, the API begins to collect the speeds every second on every specified loop in the network. The loops were specified in this study to include only the eastbound direction of travel of Interstate-4. Every second, a new batch of speeds is reported for each detector. Similar speeds are removed from the collection of speeds, since they are simply a second count of the same car.

Every 30 seconds, the collection speeds are averaged into a 30-second average speed for every station. This repeats until the end of 5-minutes, when the 30-second average speeds are combined into 5-minute average speeds. This too is done for each loop detector station. These processes are executed every second, 30 seconds, and 5 minutes for the duration of the simulation period.
At the end of each 5-minute period, the 5-minute average speeds for each station are compared with the 5-minute average speed one station upstream. In other words, the speed difference is calculated for each station specified on the network. This process occurs at the end of every 5-minute period. At this time, the speed difference is used to split the network into homogeneous speed zones according to the speed zone threshold. A new speed zone is begun at every station whose speed difference exceeds the speed zone threshold. After the homogeneous speed zones have been defined, the speed difference is then used to determine whether the implementation of variable speed limits, at the given location, is warranted. This is done by comparing the speed difference at every station to the threshold speed difference. If the speed difference on the network is greater than the threshold speed difference, then variable speed limits are warranted in that area.

Having now determined whether VSL is warranted, the API moves into the implementation stage of the code. This section is executed at the end of time period specified by the length of VSL implementation. This factor was previously discussed in Section 3.5.1.3. Once entering the implementation stage, normal speed limits are first restored to every station on the network. If variable speed limits were found to be warranted in a given speed zone, then the API will enforce new speed limits upstream and, if used, downstream as well. If variable speed limits were not found to be warranted, then the API will pass over that speed zone and leave it with its previously restored speed limits.

3.6 PARAMICS Output

PARAMICS has two types of output, global and point-specific, that are used in the study. Global output is data that is taken from the entire network and does not rely on specific locations
of interest. Such output includes travel time (total vehicle hours traveled - VHT), total distance traveled (total vehicle miles traveled – VMT) and total number of vehicles on the network at any time. Point-specific data, or just point data, is collected from specific locations on the network, such as loop detectors. Point data can include such measures as flow, speed, headway, occupancy, acceleration, gap and density.

In the network, loop detectors were placed in the same locations as on Interstate-4. This is important for two reasons. First, it is convenient that the network and Interstate-4 have the same locations for loop detectors, so that PARAMICS output for variable speed limits can be compared to and effectively implemented in the field, without any need for re-configuration. Secondly, the crash risk models were created from data extracted from the Interstate-4 loop detectors, so the determination of crash risk in the simulation should be by the same means. PARAMICS output from the loop detectors, however, is inconvenient for crash risk analysis. The loop data that is collected along I-4 is a 30-second average of the speed, flow, and occupancy on every lane. PARAMICS outputs the speed and occupancy data at every detector at every instant that a vehicle completely crosses the detector. In order to compare the two, the PARAMICS output must be post-processed to conform the instantaneous loop data into 30-second averages.

This is accomplished by using a Visual Basic macro written in Microsoft Excel. The macro goes through the data line by line and groups the data into 30-second time intervals. Within those groups, it then computes the average speed, volume, and occupancy. Once completed, the macro outputs the data in exactly the same format as that collected in the field. The disadvantage to the macro is the extensive processing time. While a typical simulation run takes anywhere between 20 to 80 minutes to run in the PARAMICS Processor module, running
the macro for a single set of output data takes anywhere from 1 to 6 hours, depending on the processing power of the computer.

Once the PARAMICS output has been successfully converted into loop data format, it must still be converted into 5-minute averages in order to be applied to the crash risk models. Therefore, another macro was written to group the 30-second averages into 5-minute intervals and compute the average, standard deviation and coefficient of variance for speed, volume and occupancy. However, this macro has a much shorter runtime than the first, because it only converts the specific variables of interest needed to calculate the crash risk measure. It performs at about 30 to 90 minutes for each run, so a typical set of 10 runs takes the second macro about 10 hours to complete. Therefore, the average time to perform a group of 10 runs in PARAMICS processor batch mode, convert the data to loop data format, and pull the variables needed to compute the crash risk to about 50 hours. Notice, however, that this value changes based on the processing power of the computer used to run the macro and the volume loading of the scenario. Due to the large number of scenarios and multiple runs performed per scenario, multiple computers were used to run the macros. The runtimes for the various computers and loading scenarios ranged from 20 hours per scenario at lower loading cases (60 percent loading) with the fastest machine to about three days per scenario for higher loading cases (90 percent loading) with the slowest machine.
CHAPTER 4. EXPERIMENTAL DESIGN

The experimental design of this study was developed in order to test a wide range of variable speed limit implementation strategies and observe their respective effects on the safety of the network. As mentioned in the Literature Review, transportation engineers have extensively utilized micro-simulation to test many different scenarios for alternative design analysis, operational analysis and safety analysis. In terms of operation, micro-simulation models output many useful measures of effectiveness, such as travel times, average speeds, flows, etc. These can be very useful measures when trying to quantify the effects that different ITS strategy have on a network.

However, there are no universal measures of effectiveness that can quantify the improvement of safety that ITS measures may cause. In the field, ITS strategies have been implemented, and before/after studies of crash frequencies were conducted to observe any safety effects the strategies may have had. This is much less than ideal for many reasons. First of all, it is unlikely that any change in crash frequency on a network can be solely attributed to the implementation of an ITS strategy. There is so much variation in the field that cannot be accounted for. Other reasons include the time and cost involved with conducting a pilot study such as this, as well as the possibility of negative safety effects, which would be detrimental to transportation in that area and public opinion of the responsible authority.

Since field studies are not desirable for this kind of research and micro-simulation software are not able to simulate crash occurrences, researches have attempted to create measures of safety that can be used to quantify the safety effects of various ITS strategies. As previously mentioned, Park and Yadlapati (2003) used a safe following distance measure to
quantify the safety within work zones. Abdel-Aty et al. (2004) created a measure of crash risk through logistic regression using within stratum matched sampling and real-time traffic variables obtained from loop detectors embedded within Interstate-4. Dilmore (2005) and Dhindsa (2006) used this crash risk measure to assess the safety effects of variable speed limits and localized ramp metering, respectively.

However, this measure of crash risk had some shortcomings. This measure described the crash risk locally to each loop detector station. The measure, therefore, was not comparable across different stations. In other words, the quantitative measures at stations 30 and 32 could not be used to make any comparison of their respective crash risks, nor could the most dangerous locations in the network be identified. To overcome this, Pande and Abdel-Aty (2006) created neural network crash models through the use of random sampling which are able to determine the real-time rear-end and lane-change crash risks. A major difference in the newer models is the inclusion of explicit variables that account for location, so that crash risks can be compared across locations. This study will use these updated models to assess the safety effects of corridor-wide variable speed limit implementation strategies on the simulated Interstate-4 network.

4.1 Rear-End Crash Risk

Rear-end crashes are the most prominent type of crash on Interstate-4 through the study corridor. Rear-end crashes typically occur when a vehicle following another vehicle is traveling faster than and collides with the rear-end of the leading vehicle.
4.1.1 Regime Conditions and Posterior Probability Models

Pande and Abdel-Aty (2006) found that rear-end crashes typically occur in two distinct traffic regimes, which is similar to their previous model of rear-end crashes in high-speed and low-speed conditions (Abdel-Aty et al., 2004). A classification tree method was used to separate crashes into two traffic regimes using data 5 to 10 minutes before the crash occurrence. In this way, the models could be used in a predictive approach, and ITS implementation could be used to prevent the probable crash from occurring. Figure 4-1 summarizes the classification tree rules as well as the associated percentages of regime 1 and regime 2 conditions for each leaf. Please refer to Pande and Abdel-Aty (2006) for more information on the classification tree method.

![Figure 4-1 Classification Tree to Determine Regime Conditions for Traffic Data (Pande, 2005)](image)

The first letter(s) in each variable name in the figure describes the measure that is calculated from the loop data. “A” represents average, “S” represents standard deviation and “CV” represents the coefficient of variation. The coefficient of variation is defined to be the
standard deviation divided by the average value. The following letter refers to what measure is being computed. “O” refers to occupancy, “S” refers to speed and “V” refers to volume. The last letter refers to which station is used to calculate this value. Figure 4-2 shows how the stations are referenced by letter. Station F refers to the station of interest. Stations A through E are upstream of the station of interest while Stations G and H are downstream. Since each station is approximately 0.5 mile apart, Station D represents the station one mile upstream of the station of interest, as shown in the figure.

The final number in the variable name refers to the time slice and is valued at levels 1 through 4. The time period used for all variables in this measure of rear-end crash risk is time period 2, which refers to the time 5 to 10 minutes before the time of interest. For reference, time period 1 refers to 0 to 5 minutes before the time of interest. Based on this coding system, the meaning of any of the loop variables above can be determined. For example, ASF2 refers to the average speed at the station of interest, 5 to 10 minutes before the time of interest. Figure 4-3 is
provided to summarize the variable nomenclature. As shown in the figure, the first letter represents the statistical function applied to the measure of interest; the second letter represents the measure of interest; the final letter represents the station; and the final number represents the time slice.

![Figure 4-3 Nomenclature for Describing Loop Data Variables from Gayah (2006)](image)

Once the classification tree model was created, the crash data was separated into regime 1 and regime 2 categories. It was determined that 46% of the rear-end crashes occurred in regime 1 conditions, and 54% occurred in regime 2. When random non-crash data was added to the classification tree, it was found that only 6% of the observed data fell into regime 1 conditions while the remaining 94% fell into regime 2. This goes to show that there is a much higher risk of crash occurrence in regime 1, since 46% of the total crashes occurred in this regime that only existed 6% of the time.

Separate crash risk models were created for each regime. Classification trees were used to determine the significant factors associated with the likelihood of a crash for each regime. These factors included off-line data (location and geometry of the freeway) and on-line factors...
(loop detector data describing current traffic conditions). Neural networks were then used to create the crash risk models. Please refer to Pande and Abdel-Aty (2006) for a more in-depth description of how the neural networks were used.

The final outcome from the neural networks were posterior probability models that could determine the crash risk within regime 1 and regime 2, 5 to 10 minutes before a crash would occur. A posterior probability of a random event is the conditional probability of the event occurring taking into account the relevant evidence of the dataset used to create the model. Since the models are able to describe the probability of crash occurrence 5 to 10 minutes before it takes place, there is time enough to prevent the occurrence of crashes through the use of various ITS strategies.

For regime 1, the best posterior probability model of crash risk included the following variables:

- AOF2
- CVSF2
- SOF2
- SVF2
- base_milepost
- downstreamon
- upstreamoff
- downstreamoff

As shown, the first four variables are derived directly from the loop detector data. The most significant variable in the model is AOF2 (Pande, 2005), which is the average occupancy at the station of interest, 5 to 10 minutes before the time of interest. Increasing the occupancy at
the station of interest significantly increases the risk of crash occurrence during regime 1 conditions. The final four variables are categorical variables derived from off-line information describing the location of the point of interest. The values for these variables are summarized in Table 4-1.

Table 4-1  List of Categorical Variables Used to Determine Rear-End Crash Risk from Gayah (2006)

<table>
<thead>
<tr>
<th>Variable</th>
<th>Definition</th>
</tr>
</thead>
<tbody>
<tr>
<td>CRASHTIME</td>
<td>0 if Time of crash between midnight to 12:26 AM</td>
</tr>
<tr>
<td></td>
<td>1 if Time of crash between 12:26 AM to 6:46 AM</td>
</tr>
<tr>
<td></td>
<td>2 if Time of crash between 6:46 AM to 7:24 PM</td>
</tr>
<tr>
<td></td>
<td>3 if Time of crash between 7:24 PM to midnight</td>
</tr>
<tr>
<td>BASE_MILPOST</td>
<td>0 if 0&lt;base_milepost&lt;=13.75</td>
</tr>
<tr>
<td></td>
<td>1 if 13.75&lt;base_milepost&lt;=15.96</td>
</tr>
<tr>
<td></td>
<td>2 if 15.96&lt;base_milepost&lt;=25.74</td>
</tr>
<tr>
<td></td>
<td>3 if 25.74&lt;base_milepost&lt;=36.25</td>
</tr>
<tr>
<td>DOWNSTREAMON</td>
<td>0 if nearest downstream on-ramp is located further than 0.7743 miles</td>
</tr>
<tr>
<td></td>
<td>1 if nearest downstream on-ramp is located within 0.7743 miles</td>
</tr>
<tr>
<td>DOWNSTREAMOFF</td>
<td>0 if nearest downstream off-ramp is located further than 0.6323 miles</td>
</tr>
<tr>
<td></td>
<td>1 if nearest downstream off-ramp is located within 0.6323 miles</td>
</tr>
<tr>
<td>UPSTREAMOFF</td>
<td>0 if nearest upstream off-ramp is located further than 0.3196 miles</td>
</tr>
<tr>
<td></td>
<td>1 if nearest upstream off-ramp is located within 0.3196 miles</td>
</tr>
<tr>
<td>STATIONF</td>
<td>0 if Loop detector station nearest to crash location is located upstream</td>
</tr>
<tr>
<td></td>
<td>1 if Loop detector station nearest to crash location is located downstream</td>
</tr>
</tbody>
</table>

In the table above, the variables are defined as follows: time (crashtime), the milepost of the location being analyzed (base_milepost), the distance from this location to the nearest
upstream off-ramp (upstreamoff), nearest downstream on-ramp (downstreamon), nearest
downstream off-ramp (downstreamoff), and whether or not the location being analyzed is
location upstream or downstream of the nearest loop detector station (stationf). The purpose of
these variables is to take into account location and geometry in the model which will allow the
-crash risk to be compared across locations.

The best posterior probability model for regime 2 conditions was much more complex.
This model used data from the station of interest (Station F) as well as the stations included up
to one mile upstream and one mile downstream of the station of interest. The reason for this is
explained by the difference in regimes 1 and 2. Regime 1 is typically a slow-moving, congested
situation, where only the station of interest needs to be examined. There is a lack of speed
difference among neighboring stations in this condition due to congestion. Regime 2 considers
un-congested situations where the speeds are much higher and the factors can change from
station to station. Important factors in these situations include speed difference between
upstream and downstream stations and speed variance at adjacent stations. The regime 2 model
included the following variables:

- ASD2
- AVD2
- ASE2
- ASE2
- AVE2
- SSE2
- ASF2
- AVF2
- ASG2
In this model, ASG2 and ASF2 were the most influential variables. This is sensible since the occurrence of rear-end accidents is caused primarily by faster moving vehicles approaching slower moving vehicles. The off-line variables included in the model were described previously in Table 4-1.

4.1.2 Combining Regime Probabilities to Assess Risk

The presence of two separate crash risk models, based on regime 1 or 2, presents some difficulties. The biggest problem with having two models is that they’re not comparable to each other. Since they are not on the same scale, values outputted from one model can only be compared to values outputted from the same model. This research, however, attempts to observe the crash risk continually along the entire network as variable speed limits are being applied and
restored periodically. Therefore, if the traffic conditions change from regime 2 to regime 1, or vice versa, the crash risks cannot be compared across that time period, even for the same location. This obviously presents a serious problem in assessing the safety benefits of the implementation of VSL if regime changes are present in the network, as they are expected to be.

For this reason, Gayah (2006) approached Pande’s models (Pande, 2005) with three different methods to combining the rear-end crash risk from regime 1 and regime 2 into a single, comparable probability. The various methods are discussed in detail in Gayah (2006), but the focus here will be on the best method found for combining regime 1 and regime 2 crash risks. A simple standardization procedure that forces the mean of a population to be 0 and the standard deviation to equal 1 was first applied to the crash risk values. The procedure involves normalizing the risk by subtracting the mean from it and dividing by the standard deviation, as shown below.

\[
\text{Norm}_{\text{Risk}} = \frac{\text{Risk} - \bar{\text{Risk}}}{\sigma_{\text{Risk}}} \tag{4.1}
\]

Gayah suggested two methods for evaluating the mean and standard deviation used in the normalized risk calculation for each model. The first method was to use the mean and standard deviation of the traffic data from the respective regime only (regime 1 mean and standard deviation for regime 1 conditions, and regime 2 statistics for regime 2 conditions). The second method provided that the mean and standard deviation be calculated from all random traffic situations, whether they are regime 1 or regime 2. The normalized risk values for each model
were denoted as follows: [normalization method]_[risk model]. For example, N2_R1 represents
the output of the regime 1 model normalized using the second (and best) method.

Gayah (2006) further argued that, since the output of the regime 1 and regime 2 models
are posterior probabilities (that probability of having a regime 1 or regime 2 crash given those
terms), it makes sense to compute a weighted average of the normalized regime 1 risk
(N1_R1 or N2_R1) and normalized regime 2 risk (N1_R2 or N2_R2) based on the probability of
the traffic conditions being regime 1 or regime 2. The probability of the traffic conditions being
regime 1 is represented by “a”, and the probability of the traffic conditions being regime 2 is
represented by “1 – a”. Gayah (2006) tested the accuracy of the weighted, normalized crash risk
based on the two normalization methods. He described each risk as follows:

\[
Risk_{3B} = a \times (N1_{_ R1}) + (1 - a) \times (N1_{_ R2}) \\
Risk_{3C} = a \times (N2_{_ R1}) + (1 - a) \times (N2_{_ R2})
\]

As shown, Risk_{3B} uses the first normalization method to determine the normalized rear-
end crash risks, and Risk_{3C} uses the second normalization method to determine the normalized
rear-end crash risks. When compared, Gayah found that Risk_{3C} provided not only the best
measure from a practical perspective, but Risk_{3C} also described the higher risk during regime 1
conditions better from a statistical perspective as well. Based on his findings, he used Risk_{3C}
as his overall, combined rear-end crash risk measure, which could be used to compare rear-end
backwash risks over any regime at any location and at multiple locations. Likewise, in this study, the
Risk_{3C} measure will be used as the combined rear-end crash risk measure.
4.2 Lane-Change Crash Risk

Pande and Abdel-Aty (2006) also modeled the lane-change crash risk using neural networks. Lane-change crashes on the freeway include all sideswipe crashes as well as angle crashes occurring in the leftmost or center lanes, since these crashes typically occur due to lane-changing maneuvers. Pande and Abdel-Aty (2006) did not find the lane-change crashes to occur in different regimes (as did the rear-end crashes), so only one model was needed. This, of course, simplifies the analysis of lane-change crash risk when compared to the joining of regime models for rear-end crash risk. The lane-change crash risk was, however, normalized so that both crash risks would have equivalent scales. The normalization procedure consisted of subtracting the mean and dividing by the standard deviation. The normalized lane-change crash risk has a mean of 0 and a standard deviation equal to 1.

Since the lane-change crash risk is sensitive to occupancies in different lanes across the freeway, variables that take individual lane occupancies into account were also considered in addition to aggregate variables that describe the traffic across all three lanes. As before, classification trees were used to describe the significant variables that influenced lane-change crashes on the freeway. The following variables were found to be significant:

- ASW2
- ASU2
- AOW2
- ADALOU2
- SVW2
- SSW2
The nomenclature used to describe the lane-change variables is nearly the same as the rear-end nomenclature. One difference is the location reference. For lane-change variables, the location is referenced by U or W, which refers to the area upstream or downstream the location of interest, respectively. The other difference is the inclusion of the variable, ADALOU2. This is a unique variable that describes the difference in occupancy across the different lanes. It is defined below. In the equation below, LO, CO and RO represent the occupancies in the left, center and right lanes, respectively.

\[
ADALOU2 = \frac{1}{10} \sum_{i=1}^{10} \left| LO - CO \right| + \left| CO - RO \right|
\]

(4.4)

### 4.3 Travel Time

Travel time is a typical value used to assess the overall operational effectiveness of the network. When trying new scenarios, the travel time measure can be very useful in seeing if the efficiency of the system has been improved or reduced. PARAMICS reports the overall travel time of the network, in total vehicle-hours traveled (VHT), throughout the simulation period. This value is calculated by summing the travel time of each vehicle over the entire network and the whole simulation time.

It is expected the implementation of variable speed limits would have some kind of effect on network travel time. On the one hand, one would think that lowering speed limits would increase travel times. However, it has also been observed that variable speed limits, though they may be lower, can produce a smoother flow of traffic on the freeway and therefore decrease
travel time. Of course, in this study, strategies that lower speed limits as well as strategies that lower and raise speed limits will be tested.

The main objective of this study is to reduce the risk of crash occurrences on Interstate-4 through the use of variable speed limits. Even though the rear-end and lane-change crash risk measures will be the primary measures of effectiveness, the treatment’s effect on travel time will also be considered for each treatment that shows positive safety improvements. In this way, some treatments that increase travel time considerably can be disregarded, and other treatments that may improve safety equally can be distinguished from one another by their travel time effects.

4.4 Network Loading Scenarios

During the peak period, the study segment of Interstate-4 through Orlando typically operates at a level of service F. This is reflected in the simulation by heavy congestion throughout downtown and the surrounding areas during the peak period. This study is not only interested in congested situations, though. Variable speed limits should be tested at uncongested and congested situations, alike, to observe their effects at each level of congestion.

To simulate this, three volume loading scenarios were created in the network to test the effects of variable speed limit implementation at different congestion levels. The loading scenarios are described as follows: 90% loading (typical congestion), 80% loading (approaching congestion) and 60% loading (free-flow conditions). The 90% loading scenario is typical and describes the average peak period on a weekday. Figure 4-4 shows the 5-minute average speeds for each time period in the simulation for each loading condition at station 43, which is located downtown. The average speeds over the simulation for the 60%, 80% and 90% loading
scenarios are 52.9 mph, 46.3 mph and 38.8 mph, respectively. In the experimental design, these three loading scenarios are considered blocking factors, and everything done under one loading scenario is done under every other loading scenario.

![5-Min Average Speed v. Time Period](image)

**Figure 4-4** Plot of Average Speeds for Different Loading Scenarios at Station 43

### 4.5 Variable Speed Limit Treatments

The experimental design for this study will be composed of different treatments, or combinations of variables, that fully define the implementation strategy of variable speed limits. These treatments are defined by several factors, as described in the following sections.
4.5.1 Determination of Threshold Value

One of the most important factors in the rear end crash risk model is the variance of speed among the station of interest and the nearest stations upstream. Most notably of importance is the difference in the speed between the station of interest and the first station upstream, separated by a half mile. This measure is referred to as the speed difference.

4.5.1.1 Speed Difference and Crash Risk

The speed difference is defined as the difference between the 5-minute average speeds at the station upstream and the 5-minute average speed at the station of interest.

\[
(SpeedDiff = \text{AvgSpeed}_e - \text{AvgSpeed}_f)
\]

Figure 4-5 is a schematic of the freeway layout, showing the eastbound direction of I-4. The station numbers and detector numbers are shown as they relate in the network. The station of interest, referred to as Station F, is station 20 in the schematic. The surrounding stations then take on relative symbols as shown (i.e. D, E, G, H).

![Figure 4-5 Schematic of Freeway Layout](image)

The measure of speed difference is very important for use in the implementation of variable speed limits in this micro-simulation experiment. PARAMICS features an Application
Programmer Interface (API) that allows for the utilization of built-in functions within the PARAMICS code that can be used to control, or change, the behavior of vehicles in the network during the simulation. An API was created that could effectively change the speed limit on any desired links in the network, based on specific stimulating circumstances, as discussed in Section 3.5.3. This API was used to test the effectiveness of several factors of variable speed limit implementation and their inter-relatedness to one another.

The primary stimulation needed to implement VSL with the API is the speed difference, that is, the measure of difference in average speeds between consecutive stations as defined in Equation 4.5. The API is programmed to collect speeds from each loop detector every second and conduct a 5-minute average speed at every station on the network. It then analyzes the speed difference measure at every station and determines whether VSL should be implemented or not. The purpose of this exploratory study is to determine the threshold speed difference for variable speed limits to be implemented.

4.5.1.2 Exploratory Analysis: 1st Location

It was resolved that the best way to determine the critical speed difference was to test a range of speed differences at a certain location and then observe the changes in crash risk over that range. Two stations were chosen that were found to have, under free flow conditions (60% volume loading), a mean speed difference of zero with 95% confidence. Over an hour of 5 minute average speed data was collected for stations 14 through 32. These stations represent the western 55-mile per hour speed limit section. Of this section, stations 19 and 20 had the lowest average speed difference of 0.008 mph.
The individual speeds from each station were then taken and compared using an F-test for variances and a T-test for means. The results from the F-test ($p = 0.302$) lead to the conclusion that the variances of speeds at stations 19 and 20 were not statistically different. A two sample T-test assuming equal variance was then performed to analyze the speed difference at station 20. The results from the T-test ($p = 0.976$) lead to the conclusion that there is no significant difference in the speeds between stations 19 and 20. Therefore, the speed difference between stations 19 and 20 is statistically zero.

In order to capture the full range of the effects of speed difference on crash risk, scenarios were run for increments of 5 mph over the speed difference range of -20 to +20 mph, and the crash risk was assessed for each scenario. The runs were set up to change the speed limit at all stations upstream of station 20 to promote the desired speed difference per scenario at station 20. This was done using the API written for PARAMICS by the author that is able to control the speed limits on desired links in the network. Ten runs were performed for each scenario to insure an adequate sample size. Table 4-2 shows the make up of scenarios for this experiment.
<table>
<thead>
<tr>
<th>Scenario</th>
<th>Speed Difference</th>
<th>Speed Limit</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Stations 4-19</td>
<td>Station 20</td>
</tr>
<tr>
<td>1</td>
<td>-20</td>
<td>35</td>
</tr>
<tr>
<td>2</td>
<td>-15</td>
<td>40</td>
</tr>
<tr>
<td>3</td>
<td>-10</td>
<td>45</td>
</tr>
<tr>
<td>4</td>
<td>-5</td>
<td>50</td>
</tr>
<tr>
<td>5</td>
<td>0</td>
<td>55</td>
</tr>
<tr>
<td>6</td>
<td>5</td>
<td>60</td>
</tr>
<tr>
<td>7</td>
<td>10</td>
<td>65</td>
</tr>
<tr>
<td>8</td>
<td>15</td>
<td>70</td>
</tr>
<tr>
<td>9</td>
<td>20</td>
<td>75</td>
</tr>
</tbody>
</table>

4.5.1.3 Exploratory Results: 1st Location

The output from PARAMICS was converted into 30-second averages using a Visual Basic macro written in Microsoft Excel. The output from this macro presents the data in the same form that it is received from the actual loop detectors in the field on I-4. The risk models, however, require that the data first be converted into 5-minute averages. A second Visual Basic macro, also written in Microsoft Excel, was used to convert the 30-second data into 5-minute averages, standard deviations and coefficients of variance for speed, volume and occupancy. The output from the second macro is used to compute the crash risk at every station for each 5-minute interval.

Once the risk is calculated for every station for each 5-minute interval, graphical analysis can be used to observe the relationship. Figure 4-6 shows the average crash risk over time at the station of interest and surrounding stations for each speed difference scenario.
The point of stratification on the graph is shown to be at 20 E1, which is the crash risk at station 20 looking downstream. This point can be further analyzed by plotting the average risk at that point versus the average speed at that point, for each speed difference scenario. The plot takes on better interpretation if we consider the change in rear-end crash risk (from the base) versus the speed difference (from the base). This graph is shown in Figure 4-7.
As seen in Figure 4-7, the crash risk steadily increases above a speed difference of zero. At a speed difference of 7 mph, however, the crash risk begins to significantly increase with every increase in speed difference. This 7 mph speed difference is shown to be the critical speed difference at this location. This critical speed difference is the best point at which VSL should be implemented to offset the increasing risk of a crash occurring.

4.5.1.4 Exploratory Analysis: 2nd Location

After analysis of the effect of speed difference on crash risk from station 20, the results must be verified for the network by repeating the procedure at a different location. Once again, a
location needed to be found where the difference in speeds under free flow conditions between two stations is nearly zero. Another analysis of 5-minute average speeds was performed over stations 35 through 48. This section represents the 50 mph speed limit section. Let it be noted that station 39 does not exist on this corridor. There is only a distance of one half mile between stations 38 and 40. Over the period of two and a half hours, station 47 had a very low speed difference of 0.032 mph. The individual 5-minute average speeds were then taken for stations 46 and 47, and compared using an F-test and a T-test. The results from the F-test ($p = 0.197$) led to the conclusion that the variance of speeds at stations 46 and 47 are not statistically different. A two sample T-test assuming equal variance was then performed to analyze the speed difference at station 47. The results from the T-test ($p = 0.122$) led to the conclusion that there is no significant difference in the speeds between stations 47 and 48. Therefore, the speed difference between stations 46 and 47 is statistically zero.

Nine scenarios were again run for increments of 5 mph over the speed difference range of -20 to +20 mph, and the crash risk was assessed for each scenario. The runs were set up to change the speed limit at stations 35 through 46 to promote the desired speed difference per scenario at station 47. Ten runs were performed for each scenario to insure an adequate sample size, and Table 4-3 shows the make up of scenarios for this experiment.
### Table 4-3  2\textsuperscript{nd} Experimental Design for Speed Difference Analysis

<table>
<thead>
<tr>
<th>Scenario</th>
<th>Speed Difference</th>
<th>Speed Limit</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Stations 35-46</td>
</tr>
<tr>
<td>10</td>
<td>-20</td>
<td>30</td>
</tr>
<tr>
<td>11</td>
<td>-15</td>
<td>35</td>
</tr>
<tr>
<td>12</td>
<td>-10</td>
<td>40</td>
</tr>
<tr>
<td>13</td>
<td>-5</td>
<td>45</td>
</tr>
<tr>
<td>14</td>
<td>0</td>
<td>50</td>
</tr>
<tr>
<td>15</td>
<td>5</td>
<td>55</td>
</tr>
<tr>
<td>16</td>
<td>10</td>
<td>60</td>
</tr>
<tr>
<td>17</td>
<td>15</td>
<td>65</td>
</tr>
<tr>
<td>18</td>
<td>20</td>
<td>70</td>
</tr>
</tbody>
</table>

#### 4.5.1.5 Exploratory Results: 2\textsuperscript{nd} Location

Once again, the output from PARAMICS was converted to 5-minute averages, and the average, standard deviation and coefficient of variance for speed, volume and occupancy were computed using the two Visual Basic macros in Excel. The output from the second macro was used to compute the crash risk at every station for each 5-minute interval.

The crash risks were averaged over the course of the simulation to find average risks for every station, considering the given scenario. Figure 4-8 shows the average crash risk v. location for the station of interest and its surrounding stations. As seen in the figure, the point of stratification is at 47 E1, which is the crash risk at station 47 looking downstream.
Figure 4-8  Average Rear-End Crash Risk v. Location for Location 2

The crash risks at 47 E1 can then be taken and compared with the average speed difference between stations 46 and 47. The change in crash risk versus the speed difference is shown graphically in Figure 4-9.
The results from the figure are comparable to the results from Figure 4-7. After the speed difference climbs above zero, a distinct point is seen where the crash risk begins to climb steeply with increasing speed difference. Again this point is approximately at 7 mph speed difference. This confirms that 7 mph is a trustworthy critical speed difference, when considering the implementation of VSL on the network.

4.5.1.6 Conclusions

The exploratory, speed difference analysis has shown, first of all, that variation in speed is an important factor when considering the trend of crash risk in the traffic flow. Moreover, the
measure of speed difference is a simple way of capturing the variation in speed in real time. From two locations representing two different speed limit sections on the network, this study has found that crash risk begins to significantly increase when speed difference goes above 7 mph. Therefore, it will be at this critical speed difference that VSL will be implemented on the network throughout this study. If the speed difference between two stations is less than 7 mph, then there will be no considerable need for varying the speed limits. However, when the speed difference between two stations exceeds the threshold value of 7 mph, VSL will be used to control the variation of speed in the traffic flow and to limit the increasing risk of an imminent crash.

4.5.2 Experimental Factors

In Section 3.5.1, the factors that are necessary for the implementation of variable speed limits were discussed in detail. These factors included the implementation strategy, the defining speed difference of the speed zones, the speed zone multiplier describing the spatial extent over which speed limits would be changed, and the minimum time period for which the change in speed limits would be sustained. For clarity, the speed zone multiplier is the fraction by which the length of a speed zone is multiplied in order to give a value to the dynamic distance over which variable speed limits are implemented.

It was decided that the implementation strategy would consist of changing the existing speed limits by a predetermined amount. This provides for smooth speed limits that are divisible by 5 mph and an implementation strategy that is dynamic to the location on the freeway. What must be considered here is the minimum and maximum speed limits that can be posted on the freeway. Interstate-4 has a minimum possible speed limit of 40 mph, although this speed limit is
nowhere posted on the freeway. The minimum speed limit is the study section is 50 mph. Therefore, the maximum that speed limits can be reduced is 10 mph. A 10-mph reduction in speeds ought to be set as a maximum anyway. It is not reasonable to change the speed limits in an area by 15 mph, especially when the area is traveled heavily by familiar commuters. Two strategies of upstream speed limit lowering will be tested, then. A decrease of 10 mph and a decrease of 5 mph in the upstream speed limits will be investigated in the experimental design.

Another strategy should also be included, that of lowering upstream speed limits and simultaneously raising downstream speed limits. The maximum allowable speed limit on most roadways is the design speed. The design speed ensures adequate stopping sight distance, adequate radii on horizontal curves and reasonable super-elevations, among many other things. Therefore, the posted speed limit should never exceed the design speed on the freeway. The minimum design speed occurs throughout the downtown segment of Interstate-4 at 60 mph. The speed limit in that area is set at 50 mph. Therefore, there should be no increase in speed limits that exceed 10 mph. Increasing speed limits by 10 mph, however, may also seem a bit unreasonable and extreme. For this reason, the third strategy will implement a decreasing of upstream speed limits by 5 mph coupled with an increasing of downstream speed limits by 5 mph.

The next variable defines the threshold speed difference for homogeneous speed zones. Since the threshold speed difference for variable speed limit implementation is 7 mph, as discussed in the previous section, the threshold for speed zones must be less. A natural choice is 5 mph. This would separate the segments with different speed limits into speed zones, and any disruptions greater than 5 mph within a speed limit section would also be separated. However, it may be more desirable to have more defined speed zones, based on a lower threshold. A
threshold of 2.5 mph could also be used, so that the speed zones that are defined have average speeds within 2.5 mph of each other. When this threshold is exceeded, a new speed zone is defined. Based on this, then, the speed zone threshold variable will have two levels: 5 mph and 2.5 mph.

Based on the speed zones, the spatial extent of the VSL implementation can be defined by a multiplier. The idea behind homogeneous speed zones was to capture the stations with similar traffic conditions and change them all together. Therefore, a necessary multiplier for the speed zone is 1.0. If a given speed zone is 5 miles in length and it is determined that VSL should be implemented in that area, then a multiplier of 1.0 would allow the entire 5 mph to be effected. It may also be of interest to see the effects that only implementing over a fraction of the speed zone has. In this case, 0.5 is an ideal multiplier. For the scenario above, a multiplier of 0.5 would allow speed limits to be changed over the 2.5 miles closest to the location of interest, while the remainder of the speed zone is unchanged. The speed zone multiplier, then, will consist of two levels: 1.0 and 0.5.

Finally, the minimum time period for VSL to be extended will be discussed. Previous studies found time periods of 5 min and 10 min to work best (Lee, 2004, 2006). Longer time periods should be avoided, since there effects may be over-stated or exaggerated. Also, to create a treatment that is as dynamic as possible, the most-adaptable factors should be used. In this case, the minimum feasible time is 5 min, since the speed differences are only analyzed every 5 minutes. To maintain highly adaptable time periods, then, the minimum time periods for VSL implementation will investigated at 5 minutes and 10 minutes.

Considering these four variables, one at 3 levels and three at 2 levels, there are 24 possible treatment combinations. A treatment is here referred to as a unique combination of
factors that is expected to produce a unique result. Each combination of factors is given a unique treatment ID, as shown in Table 4-4.

Table 4-4  Layout of Experimental Design

<table>
<thead>
<tr>
<th>Treatment ID</th>
<th>Speed Change Implementation</th>
<th>Speed Zone Threshold</th>
<th>Speed Change Distance</th>
<th>Speed Change Time Period</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>-10</td>
<td>5 mph</td>
<td>Speed Zone</td>
<td>5 min</td>
</tr>
<tr>
<td>2</td>
<td>-10</td>
<td>5 mph</td>
<td>Speed Zone</td>
<td>10 min</td>
</tr>
<tr>
<td>3</td>
<td>-10</td>
<td>5 mph</td>
<td>1/2 Speed Zone</td>
<td>5 min</td>
</tr>
<tr>
<td>4</td>
<td>-10</td>
<td>5 mph</td>
<td>1/2 Speed Zone</td>
<td>10 min</td>
</tr>
<tr>
<td>5</td>
<td>-10</td>
<td>2.5 mph</td>
<td>Speed Zone</td>
<td>5 min</td>
</tr>
<tr>
<td>6</td>
<td>-10</td>
<td>2.5 mph</td>
<td>Speed Zone</td>
<td>10 min</td>
</tr>
<tr>
<td>7</td>
<td>-10</td>
<td>2.5 mph</td>
<td>1/2 Speed Zone</td>
<td>5 min</td>
</tr>
<tr>
<td>8</td>
<td>-10</td>
<td>2.5 mph</td>
<td>1/2 Speed Zone</td>
<td>10 min</td>
</tr>
<tr>
<td>9</td>
<td>-5</td>
<td>5 mph</td>
<td>Speed Zone</td>
<td>5 min</td>
</tr>
<tr>
<td>10</td>
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<td>5 mph</td>
<td>Speed Zone</td>
<td>10 min</td>
</tr>
<tr>
<td>11</td>
<td>-5</td>
<td>5 mph</td>
<td>1/2 Speed Zone</td>
<td>5 min</td>
</tr>
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<td>5 mph</td>
<td>1/2 Speed Zone</td>
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<tr>
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</tr>
<tr>
<td>14</td>
<td>-5</td>
<td>2.5 mph</td>
<td>Speed Zone</td>
<td>10 min</td>
</tr>
<tr>
<td>15</td>
<td>-5</td>
<td>2.5 mph</td>
<td>1/2 Speed Zone</td>
<td>5 min</td>
</tr>
<tr>
<td>16</td>
<td>-5</td>
<td>2.5 mph</td>
<td>1/2 Speed Zone</td>
<td>10 min</td>
</tr>
<tr>
<td>17</td>
<td>-5/+5</td>
<td>5 mph</td>
<td>Speed Zone</td>
<td>5 min</td>
</tr>
<tr>
<td>18</td>
<td>-5/+5</td>
<td>5 mph</td>
<td>Speed Zone</td>
<td>10 min</td>
</tr>
<tr>
<td>19</td>
<td>-5/+5</td>
<td>5 mph</td>
<td>1/2 Speed Zone</td>
<td>5 min</td>
</tr>
<tr>
<td>20</td>
<td>-5/+5</td>
<td>5 mph</td>
<td>1/2 Speed Zone</td>
<td>10 min</td>
</tr>
<tr>
<td>21</td>
<td>-5/+5</td>
<td>2.5 mph</td>
<td>Speed Zone</td>
<td>5 min</td>
</tr>
<tr>
<td>22</td>
<td>-5/+5</td>
<td>2.5 mph</td>
<td>Speed Zone</td>
<td>10 min</td>
</tr>
<tr>
<td>23</td>
<td>-5/+5</td>
<td>2.5 mph</td>
<td>1/2 Speed Zone</td>
<td>5 min</td>
</tr>
<tr>
<td>24</td>
<td>-5/+5</td>
<td>2.5 mph</td>
<td>1/2 Speed Zone</td>
<td>10 min</td>
</tr>
</tbody>
</table>

Figure 4-10 shows a schematic of how Treatment 17 might work, for instance. In the example, speeds are fairly homogeneous from stations 12 through 15. At station 16, however, the 5-min average speed jumps from 56 mph up to about 63 mph. This negative speed difference of 7 mph constitutes the creation of a new speed zone, Speed Zone 2, which begins at station 16.
High average speeds are maintained until station 24, where the speeds go back down to around 55 mph. This speed difference also constitutes a new speed zone, Speed Zone 3, which begins at station 24. Note that the distance between every station is 0.5 miles. When evaluating the speed differences at every station for VSL implementation, it is observed that station 24 has a high positive speed difference (10 mph), which warrants VSL application. Based on Treatment 17, the speed limit for the whole speed zone upstream (Speed Zone 2) is lowered by 5 mph, and the speed limit for the whole speed zone downstream (Speed Zone 3) is raised by 5 mph. These speed limits would be maintained for 5 minutes and then re-evaluated.

Figure 4-10  Example of VSL Implementation on Interstate-4
This experimental design of 24 treatments was applied to each of the volume loading blocks described earlier, 60%, 80% and 90%. Recall that these three loading scenarios represent free-flow conditions, conditions approaching congestion before the peak period, and congested conditions during the peak period, respectively. The analysis of these treatments will be done according to rear-end and lane-change crash risk, followed by travel time analysis if needed. This study attempts to reveal the combination of factors which produces the best safety benefit for use on Interstate-4 in Orlando.

4.6 Replications of Simulation Runs

In order to establish statistical significance in the effects that different VSL treatments may have on the crash risks, replications must be carried out. Replications are needed in every experimental design in order to establish an estimate of the error due to environmental factors. Treatments cannot be compared to the BASE case if there are no replications. If there are no replications for a given treatment, then the variance of data inherent to that treatment cannot be estimated, and the variation of data between two scenarios cannot be compared.

Therefore, replications of each treatment must be carried out in order to establish an estimate of the error variance within each treatment. At that point, then, the variance between two scenarios can be compared to the variance of error within each treatment, and significance of the difference can be considered. PARAMICS accounts for this error through the use of random numbers assigned to different vehicle and driver characteristics in the simulation. A random number generator is used to assign individual headways, trip departure times, minimum gap acceptance values, etc. for individual vehicles in the network in stochastic fashion. For each run a different random number seed is used which defines the rest of the random numbers used by
PARAMICS. Enough replications must be performed in order to be confident in the estimate of variance for each treatment. The following equation was used to determine the number of replications needed to fulfill this requirement:

\[ N \approx \left( \frac{KS}{E} \right)^2 \]  

(4.6)

In this equation, \( N \) is the number of replications needed, \( K \) is the statistical factor related to the level of confidence desired, \( S \) is a measure of the standard deviation of the data for a treatment, and \( E \) is the allowable error for the runs. For determining the number of replications, a \( K \)-value of 1.645 was used for 90% confidence. The standard deviation was estimated from the replications initially performed. The allowable error used for the rear-end crash risk, \( E \), was 0.100, which is about 2% of the range of crash risks (-1.0, 3.5). Notice the rear-end crash risk data was used in this equation, since it usually varies more than the lane-change crash risk.

Typically, 5 runs were performed for each treatment to get an estimate of the variation of data. Then the equation was applied to the average rear-end crash risk measure at every location over all the runs. The maximum \( N \)-value was taken to be the minimum number of replications required, and that many more replications were run. Afterward, the equation was checked again, taking into account the additional replications. Most of the treatments in the 60% block did not require additional runs. Treatments in the 80% and 90% blocks typically required 12-15 runs, with the maximum number of replications being 20.
CHAPTER 5. RESULTS

5.1 Analyzing Simulation Runs

The simulation and crash risk models allow for the rear-end and lane-change crash risks to be calculated for every location at every time period in each of the replication runs. Although enough replications were performed statistically, there is still a considerable amount of variation in the different replications, which is modeled to reflect the randomness of traffic fluctuations over different days in the field. An example of this variation is shown in Figure 5-1, which is a plot of the upstream rear-end crash risk for 5 replications of Treatment 05 at 60% volume loading. As Gayah (2006) noted, these probabilities are obtained from the neural network and are normalized by subtracting the mean and dividing by the standard deviation. Therefore, these probabilities can be used to compare the crash risk at different times and locations. Each of the crash risk values occur between -1.0 and 3.5. A crash risk of 0.0 represents a crash risk that is equal to the mean crash risk, and a crash risk greater than 0.0 represents more dangerous conditions.

It would be hard to use a figure like Figure 5-1 when comparing the effectiveness of a given treatment against the BASE case. Multiple curves are used in this figure to describe the crash risk at a single station during different periods in the simulation. Of course, it would be much easier to visually compare the difference between strategies if the multiple replications were joined into an average crash risk.
Average crash risks are used in this study to compare between the effect of different treatments and the BASE case. A single crash risk profile was created for each treatment for both the rear-end and lane-change crash risks. This single crash risk profile has a value of the crash risk at every station and time that is equal to the average of the crash risk values at the respective station and time over the number of simulation runs performed at that particular scenario.

\[
(Risk\ _{profile})_{tl} = \frac{1}{N} \sum_{r=1}^{N} (Risk)_{trl}
\]  

(5.1)
Where \((\text{Risk})_{t\,r\,l}\) is the crash risk for time \(t\), run \(r\), and location \(l\) for particular test case scenario; \(N\) is the number of runs required for the particular test case scenario; \((\text{Risk\_profile})_{t\,l}\) is the crash risk averaged over the number of runs for each location \(l\) and time \(t\).

The benefit of using a single crash risk profile is shown in Figure 5-2. As shown, the single average profile curve could be much more easily compared to the average profile of the BASE case. Taking the average over different simulation seed values also makes sense, because the variation of data in the field among different weekdays is not kept separate but is averaged in order to determine the true conditions in that area.

![Upstream Rear-End Crash Risk at Station 50](image)

**Figure 5-2** Average Crash Risk Profile at Station 50 E 0 for Treatment 05 at 60% Loading
Another, and more comprehensive, way to view the effect that a treatment has on the crash risk is to average the crash risk profiles over time and display them by location. In this way, the entire network’s response to the given treatment can be observed and compared to the BASE case at once. The equation below shows how the average risk is calculated from the risk profile at every location.

\[
(Average\ _Risk)_l = \frac{1}{T} \sum_{t=1}^{T} (Risk\ _profile)_{tl}
\]  

(5.2)

Where \((Risk\ _profile)_{tl}\) is the crash risk averaged over the number of runs for each location \(l\) and time \(t\); \(T\) is the total number of time periods that the risk is calculated during the simulation (\(T = 36\) as there are 36 5-minute periods in the 3 hour simulation time); and \((Average\ _Risk)_l\) is the crash risk averaged by both time and number of simulation runs at location \(l\).

The average rear-end crash risk and the average lane-change crash risk was calculated for the BASE case and every treatment for each percent loading scenario. Figure 5-3 shows the average rear-end crash risk for the BASE case at 60% loading. All treatments at 60% loading are compared to this plot to determine the treatment’s effect on rear-end crash risk.
Notice that for each loop detector station two crash risk values are calculated. The first value is calculated for the area immediately upstream of the loop detector station while the second value describes the crash risk immediately downstream of the detector station. In order to denote crash risk both upstream and downstream of each detector, the following naming convention was adopted. The first number identifies the loop detector station; the following letter represents the direction of the freeway being considered (which is E since this study focuses solely on the eastbound direction of travel); and the last number represents whether the area is upstream or downstream of the loop detector station. The number 0 represents the upstream area while 1 represents the downstream area. The location 44 E 0, therefore, refers to
the area directly upstream of the eastbound detector station 42. The various plots of the crash risk given do not include the label for the downstream crash risk value due to the limited space on the plots but it should be noted that this value has been calculated and is included on all plots. Please note that Station 39 does not exist. All plots, therefore, show Station 38 immediately followed by Station 40.

In the analysis of results, then, the more comprehensive plot (average crash risk v. location) will be used first to identify which treatments are effective in reducing crash risk on the network, and specifically which areas receive the most benefit. These areas will be looked at more closely, and a paired t-test will be employed to determine if the change in crash risk due to the implementation of a given VSL strategy is significant. The benefit will also be quantified using a measure of effectiveness, the Overall Risk Change Index (ORCI). The ORCI will be calculated for the rear-end crash risk and the lane-change crash risk for each treatment. The ORCI is basically the sum of the difference in crash risk due to the treatment across the effected area. This sum includes both positive and negative differences. A positive difference is one in which the treatment was able to lower the crash risk from the BASE case. When several treatments are found to have significantly beneficial effects, the ORCI can be used to determine which treatment had the greatest effect.

The second type of plot (crash risk profile v. time period) will then be used to ensure that the crash risk is reduced in time. That is, that the majority of the time periods receive crash risk benefits during VSL implementation. On top of this, a travel time analysis will also be done for treatments that show positive potential. If a treatment is found to significantly reduce the crash risk throughout the simulation time, then the treatment’s effect on the network travel time will also be observed as a secondary measure of effectiveness. It is secondary, of course, because
this study is mainly concerned with crash risk and safety, and a small increase in travel time will not sway the recommendation of safe treatments. Previous studies have considered an increase in travel time as much as 5% to be an acceptable trade-off for the lowering of crash risk (Gayah, 2006). However, a treatment that can reduce both crash risk and network travel time will be most highly regarded.

5.2 60 Percent Loading Scenario

The 24 VSL treatments were run at the 60% loading scenario, which simulates non-congested, free-flow conditions. These treatments were run to observe whether variable speed limits could effectively reduce the crash risk during non-congested situations.

5.2.1 Treatments 01 – 08

For the 60% loading scenarios, the first eight treatments are analyzed in this section. Treatments 01-08 are unique in their VSL implementation strategy. In these treatments, though the speed zone definition, multiplier and time period are changed, the strategy of decreasing speed limits by 10 mph upstream is constant. Table 5-1 displays the different factors that make up Treatments 01-08, which are reviewed in this section.
Table 5-1  Description of Treatments 01-08

<table>
<thead>
<tr>
<th>Treatment ID</th>
<th>Speed Change Implementation</th>
<th>Speed Zone Threshold</th>
<th>Speed Change Distance</th>
<th>Speed Change Time Period</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>-10</td>
<td>5 mph</td>
<td>Speed Zone</td>
<td>5 min</td>
</tr>
<tr>
<td>2</td>
<td>-10</td>
<td>5 mph</td>
<td>Speed Zone</td>
<td>10 min</td>
</tr>
<tr>
<td>3</td>
<td>-10</td>
<td>5 mph</td>
<td>1/2 Speed Zone</td>
<td>5 min</td>
</tr>
<tr>
<td>4</td>
<td>-10</td>
<td>5 mph</td>
<td>1/2 Speed Zone</td>
<td>10 min</td>
</tr>
<tr>
<td>5</td>
<td>-10</td>
<td>2.5 mph</td>
<td>Speed Zone</td>
<td>5 min</td>
</tr>
<tr>
<td>6</td>
<td>-10</td>
<td>2.5 mph</td>
<td>Speed Zone</td>
<td>10 min</td>
</tr>
<tr>
<td>7</td>
<td>-10</td>
<td>2.5 mph</td>
<td>1/2 Speed Zone</td>
<td>5 min</td>
</tr>
<tr>
<td>8</td>
<td>-10</td>
<td>2.5 mph</td>
<td>1/2 Speed Zone</td>
<td>10 min</td>
</tr>
</tbody>
</table>

5.2.1.1 Rear-End Crash Risk

In the previous chapter, Experimental Design, it was explained that 24 treatments of variable speed limits that were applied to the study corridor. Each block of 24 treatments were applied to the network at 60% volume loading, 80% volume loading and 90% volume loading. This section will focus on the first eight treatments applied at 60% loading. The first eight treatments have a common variable speed limit implementation, in that the upstream speed limits are decreased by 10 mph when the threshold is reached.
Figure 5-4 compares the average rear-end crash risk, for Treatments 01 to 08, at every location, to the BASE case. As shown in the figure, there is hardly any noticeable difference due to the decreasing of upstream speed limits by 10 mph. The greatest difference in crash risks are seen over a 10-mile stretch between station 10 and station 30. Figure 5-5 shows the average crash risk for each treatment between the locations of stations 10 and 30. However, this section actually shows a general increase in crash risk due to the implementation of variable speed limits when compared with the BASE case.
The possibility of increasing the crash risk due to the implementation of variable speed limit strategies is a very dangerous scenario. Therefore, it was desirable to know if the increase in average crash risk was statistically significant. That is, do Treatments 01–08 simply have no effects on the crash risk, or do they have hazardous effects on the crash risk? In order to answer this question, a paired t-test was performed for each treatment and the BASE case to determine the significance of the resulting increase in crash risk at the most highly effected locations.

The average crash risks for each treatment, over the location of stations 10 to 30, were compared to the average crash risks over the same location for the BASE case. Table 5-2 summarizes the t-test, along with the Overall Risk Change Index (ORCI), for the given location.
It is seen from the table that the majority of the treatments have an ORCI around -2. The critical 
t-statistic, using alpha = 0.05, is 1.645. Since none of the treatments have a t-statistic greater 
than the critical t-statistic, the resulting increase is not considered statistically significant. 
Therefore, although the results seem to show that VSL Treatments 01-08 increase the average 
crash risk, the lowering of speed limits by 10 mph upstream do not statistically increase the 
average rear-end crash risk.

Table 5-2 Summary of Difference in Rear-End Crash Risk for Treatments 01-08

<table>
<thead>
<tr>
<th>Test Case ID</th>
<th>Base</th>
<th>T01</th>
<th>T02</th>
<th>T03</th>
<th>T04</th>
<th>T05</th>
<th>T06</th>
<th>T07</th>
<th>T08</th>
</tr>
</thead>
<tbody>
<tr>
<td>Average Crash Risk (Stations 10 to 30)</td>
<td>-0.510</td>
<td>-0.462</td>
<td>-0.462</td>
<td>-0.463</td>
<td>-0.465</td>
<td>-0.463</td>
<td>-0.456</td>
<td>-0.459</td>
<td></td>
</tr>
<tr>
<td>Crash Risk Benefit</td>
<td>---</td>
<td>-0.048</td>
<td>-0.047</td>
<td>-0.047</td>
<td>-0.027</td>
<td>-0.045</td>
<td>-0.046</td>
<td>-0.054</td>
<td>-0.051</td>
</tr>
<tr>
<td>T-Statistic (Significance of Change)</td>
<td>---</td>
<td>0.710</td>
<td>0.710</td>
<td>0.694</td>
<td>0.405</td>
<td>0.672</td>
<td>0.691</td>
<td>0.795</td>
<td>0.756</td>
</tr>
<tr>
<td>ORCI</td>
<td>---</td>
<td>-1.997</td>
<td>-1.989</td>
<td>-1.962</td>
<td>-1.130</td>
<td>-1.879</td>
<td>-1.938</td>
<td>-2.253</td>
<td>-2.139</td>
</tr>
</tbody>
</table>

5.2.1.2 Lane-Change Crash Risk

The difference in lane-change crash risk, due to the effects of Treatments 01-08, was also 
evaluated. Figure 5-6 shows the average lane-change crash risk v. location for the entire 
network. Once again there is little change, throughout the network as a whole, in crash risk due 
to the implementation of VSL Treatments 01-08.
Figure 5-6  Average Lane-Change Crash Risk v. Location for Treatments 01-08

The most effected area is located between stations 10 and 14. As shown in Figure 5-7, Treatments 01-08 again show an increase in crash risk in the most effected area. This increase is shown to be much more substantial than what was shown earlier in rear-end crash risk. Once again, a paired t-test will be used to determine the significance of the difference in lane-change crash risk, comparing the BASE case to each treatment.
Table 5-3 summarizes the t-test and the ORCI for the effects of Treatments 01-08 on average lane-change crash risk from stations 10 to 15. As shown, all eight treatments have a negative average crash risk difference with the BASE. That is, they all have higher crash risks than the BASE case. The t-statistic, for each treatment, is compared with the critical t-statistic, 1.645. Treatments 01 and 02 have the highest t-values with both surpassing 1.60. They are still not, however, higher than the critical t-value. Therefore, the lowering of speed limits by 10 mph upstream does not significantly increase the average lane-change crash risk in the 60% loading scenario.
Table 5-3  Summary of Difference in Lane-Change Crash Risk for Treatments 01–08

<table>
<thead>
<tr>
<th>Test Case ID</th>
<th>Base</th>
<th>T01</th>
<th>T02</th>
<th>T03</th>
<th>T04</th>
<th>T05</th>
<th>T06</th>
<th>T07</th>
<th>T08</th>
</tr>
</thead>
<tbody>
<tr>
<td>Average Crash Risk (Stations 10 to 15)</td>
<td>-1.104</td>
<td>-0.626</td>
<td>-0.622</td>
<td>-0.721</td>
<td>-0.746</td>
<td>-0.634</td>
<td>-0.639</td>
<td>-0.707</td>
<td>-0.735</td>
</tr>
<tr>
<td>Crash Risk Benefit</td>
<td>---</td>
<td>-0.478</td>
<td>-0.482</td>
<td>-0.383</td>
<td>-0.358</td>
<td>-0.470</td>
<td>-0.465</td>
<td>-0.397</td>
<td>-0.369</td>
</tr>
<tr>
<td>T-Statistic (Significant of Change)</td>
<td>---</td>
<td>1.615</td>
<td>1.631</td>
<td>1.247</td>
<td>1.159</td>
<td>1.581</td>
<td>1.552</td>
<td>1.296</td>
<td>1.193</td>
</tr>
<tr>
<td>ORCI</td>
<td>---</td>
<td>-5.740</td>
<td>-5.789</td>
<td>-4.596</td>
<td>-4.293</td>
<td>-5.636</td>
<td>-5.578</td>
<td>-4.767</td>
<td>-4.423</td>
</tr>
</tbody>
</table>

5.2.2 Treatments 09 – 16

For the 60% loading scenarios, Treatments 09-16 are analyzed in this section. Treatments 09-16 are unique in their VSL implementation strategy, in that the speed limits are decreased by 5 mph upstream for all treatments. Table 5-4 displays the different factors that make up Treatments 09-16, which are reviewed in this section.
Table 5-4  Description of Treatments 09-16

<table>
<thead>
<tr>
<th>Treatment ID</th>
<th>Speed Change Implementation</th>
<th>Speed Zone Threshold</th>
<th>Speed Change Distance</th>
<th>Speed Change Time Period</th>
</tr>
</thead>
<tbody>
<tr>
<td>9</td>
<td>-5</td>
<td>5 mph</td>
<td>Speed Zone</td>
<td>5 min</td>
</tr>
<tr>
<td>10</td>
<td>-5</td>
<td>5 mph</td>
<td>Speed Zone</td>
<td>10 min</td>
</tr>
<tr>
<td>11</td>
<td>-5</td>
<td>5 mph</td>
<td>1/2 Speed Zone</td>
<td>5 min</td>
</tr>
<tr>
<td>12</td>
<td>-5</td>
<td>5 mph</td>
<td>1/2 Speed Zone</td>
<td>10 min</td>
</tr>
<tr>
<td>13</td>
<td>-5</td>
<td>2.5 mph</td>
<td>Speed Zone</td>
<td>5 min</td>
</tr>
<tr>
<td>14</td>
<td>-5</td>
<td>2.5 mph</td>
<td>Speed Zone</td>
<td>10 min</td>
</tr>
<tr>
<td>15</td>
<td>-5</td>
<td>2.5 mph</td>
<td>1/2 Speed Zone</td>
<td>5 min</td>
</tr>
<tr>
<td>16</td>
<td>-5</td>
<td>2.5 mph</td>
<td>1/2 Speed Zone</td>
<td>10 min</td>
</tr>
</tbody>
</table>

5.2.2.1 Rear-End Crash Risk

This section will focus on the effects that Treatments 09-16 had on the rear-end crash risk. The only difference between Treatments 09-16 and 01-08 is the variable speed limit implementation. Other than the implementation, Treatment 01 is the same as Treatment 09, and Treatment 02 is the same as Treatment 10, etc. The difference in implementation is the lowering of upstream speed limits by 5 mph, as opposed to the lowering of 10 mph.

Since the direction of implementation is the same, with only the magnitude changing, it is expected that this set of treatments will produce similar results as the first set. Figure 5-8 shows a plot of the average rear-end crash risk v. location for the entire network, comparing the BASE case to Treatments 09-16.
The figure shows, as expected, very similar results to Treatments 01-08. There seems to be little, if any, change in the average rear-end crash risk due to the lowering of 5 mph upstream. The most effected segment is located between stations 22 to 31. This shows a generally negative effect on the average crash risk. This section is shown in Figure 5-9. Even in the magnified figure, it is hard to see a substantial difference in average crash risk.
These results are expected; since the lowering of upstream speed limits by 10 mph created a small increase in the crash risk, the lowering of upstream speed limits by only 5 mph created an even smaller increase in crash risk. The results are consistent. To be sure of their insignificance, the effected segment is tested against the BASE case, for each Treatment, using the paired t-test. The results from the paired t-test are shown in Table 5-5. As shown, all of the t-statistics are far less than the critical t-value, 1.645. Therefore, the lowering of upstream speed limits by 5 mph does not significantly increase the average rear-end crash risk.
Table 5-5  Summary of Difference in Rear-End Crash Risk for Treatments 09–16

<table>
<thead>
<tr>
<th></th>
<th>Test Case ID</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Base</td>
</tr>
<tr>
<td>Average Crash Risk</td>
<td></td>
</tr>
<tr>
<td>(Stations 22 to 31)</td>
<td>-0.379</td>
</tr>
<tr>
<td>Crash Risk Benefit</td>
<td>---</td>
</tr>
<tr>
<td>T-Statistic (Significance of Change)</td>
<td>---</td>
</tr>
<tr>
<td>ORCI</td>
<td>---</td>
</tr>
</tbody>
</table>

5.2.2.2 Lane-Change Crash Risk

The lane-change crash risk is also analyzed for Treatments 09-16, to see what effect the upstream lowering of speed limits by 5 mph has on crash risk. From the first set of treatments, it is expected that the upstream lowering of speed limits by 5 mph will actually increase the average crash risk, but not significantly. Figure 5-10 shows the average lane-change crash risk plotted versus location, for the whole network. As shown, the plot is very similar to the effects of Treatments 01-08 for lane-change crash risk, except to a lesser degree. It can be seen that, when the implementation does affect the crash risk, it typically increases it.
Figure 5-10 Average Lane-Change Crash Risk v. Location for Treatments 09–16

The segment most affected by these treatments is located between stations 11 and 14. Figure 5-11 shows the magnification of this segment, plotting average lane-change crash risk v. location. As seen from the figure, the increase in average lane change crash risk from Treatments 09-16 is substantially higher than the increase in average rear-end crash risk. The increase must be tested for significance in order to know if the proposed treatments are actually hazardous or just ineffective. Once again, a paired t-test is used to test the difference in the average lane-change crash risk, on the effected segment, due to the lowering of upstream speed limits by 5 mph.
Table 5-6 shows the summary of the paired t-test and the ORCI. The negative ORCI value indicates that the treatment actually increases the crash risk over the segment, which is counter-productive. The t-statistic gives an indication of whether the change is significant or could be attributed to random error and variation. Since none of the t-values exceed the critical t-statistic for alpha = 0.05, the increase in average lane-change crash risk is considered insignificant. Therefore, the decreasing of upstream speed limits by 5 mph has an ineffective, but not harmful, affect on the lane-change crash risk.
### Table 5-6  Summary of Difference in Lane-Change Crash Risk for Treatments 09–16

<table>
<thead>
<tr>
<th>Test Case ID</th>
<th>Base</th>
<th>T09</th>
<th>T10</th>
<th>T11</th>
<th>T12</th>
<th>T13</th>
<th>T14</th>
<th>T15</th>
<th>T16</th>
</tr>
</thead>
<tbody>
<tr>
<td>Average Crash Risk (Stations 22 to 43)</td>
<td>-1.289</td>
<td>-1.041</td>
<td>-1.061</td>
<td>-1.137</td>
<td>-1.130</td>
<td>-1.083</td>
<td>-1.051</td>
<td>-1.146</td>
<td>-1.135</td>
</tr>
<tr>
<td>Crash Risk Benefit</td>
<td>---</td>
<td>-0.248</td>
<td>-0.228</td>
<td>-0.151</td>
<td>-0.159</td>
<td>-0.206</td>
<td>-0.238</td>
<td>-0.143</td>
<td>-0.154</td>
</tr>
<tr>
<td>T-Statistic (Significance of Change)</td>
<td>---</td>
<td>0.820</td>
<td>0.751</td>
<td>0.483</td>
<td>0.510</td>
<td>0.682</td>
<td>0.791</td>
<td>0.454</td>
<td>0.492</td>
</tr>
<tr>
<td>ORCI</td>
<td>---</td>
<td>-1.986</td>
<td>-1.826</td>
<td>-1.212</td>
<td>-1.274</td>
<td>-1.645</td>
<td>-1.907</td>
<td>-1.142</td>
<td>-1.230</td>
</tr>
</tbody>
</table>

5.2.3 Treatments 17 – 24

For the 60% loading scenarios, Treatments 17-24 are analyzed in this section. Treatments 17-24 are unique in their VSL implementation strategy, in that the speed limits are decreased by 5 mph upstream and increased by 5 mph downstream for all treatments. Table 5-7 displays the different factors that make up Treatments 17-24, which are reviewed in this section.
Table 5-7  Description of Treatments 17-24

<table>
<thead>
<tr>
<th>Treatment ID</th>
<th>Speed Change Implementation</th>
<th>Speed Zone Threshold</th>
<th>Speed Change Distance</th>
<th>Speed Change Time Period</th>
</tr>
</thead>
<tbody>
<tr>
<td>17</td>
<td>-5/+5</td>
<td>5 mph</td>
<td>Speed Zone</td>
<td>5 min</td>
</tr>
<tr>
<td>18</td>
<td>-5/+5</td>
<td>5 mph</td>
<td>Speed Zone</td>
<td>10 min</td>
</tr>
<tr>
<td>19</td>
<td>-5/+5</td>
<td>5 mph</td>
<td>1/2 Speed Zone</td>
<td>5 min</td>
</tr>
<tr>
<td>20</td>
<td>-5/+5</td>
<td>5 mph</td>
<td>1/2 Speed Zone</td>
<td>10 min</td>
</tr>
<tr>
<td>21</td>
<td>-5/+5</td>
<td>2.5 mph</td>
<td>Speed Zone</td>
<td>5 min</td>
</tr>
<tr>
<td>22</td>
<td>-5/+5</td>
<td>2.5 mph</td>
<td>Speed Zone</td>
<td>10 min</td>
</tr>
<tr>
<td>23</td>
<td>-5/+5</td>
<td>2.5 mph</td>
<td>1/2 Speed Zone</td>
<td>5 min</td>
</tr>
<tr>
<td>24</td>
<td>-5/+5</td>
<td>2.5 mph</td>
<td>1/2 Speed Zone</td>
<td>10 min</td>
</tr>
</tbody>
</table>

5.2.3.1 Rear-End Crash Risk

This section focuses on the effects of the VSL Treatments 17 through 24 on the rear-end crash risk. In these treatments, the speed limits upstream were reduced by 5 mph, and the speed limits downstream were raised by 5 mph. The motivation behind the raising of downstream speed limits was discussed earlier, but essentially it is a strategy implemented to dissipate an existing queue more quickly by pulling out the front.

This set of treatments, unlike the lowering of upstream speed limits only, shows very good benefit for crash risk. Figures 5-12 and 5-13 show the average rear-end crash risk over the entire network for Treatments 17-20 and 21-24, respectively, with the BASE case. When compared with similar graphs from the previous implementation strategies (Treatments 01-16),
this plot shows considerable difference in the crash risk throughout the network due to the implementation of variable speed limits. When summed across the entire network, every one of these treatments has the positive effect of lowering the rear-end crash risk.

![Average Rear-End Crash Risk v. Location](image)

**Figure 5-12** Average Rear-End Crash Risk v. Location for Treatments 17–20
The segment of corridor most affected by the treatments is located between stations 33 and 49. Figures 5-14 and 5-15 show the average rear-end crash risk for each treatment and the BASE case over the most effected section. The BASE case, which is bolded in blue, is shown to have a higher crash risk than the each treatment for the majority of the segment. The best cases seem to be Treatments 17, 18, 21 and 22.
Figure 5-14  Average Rear-End Crash Risk v. Location for Treatments 17–20 – Magnified
Once again, the analysis of the significance of the difference in crash risk is important. It must be determined whether the lowering of crash risk in this segment can be attributed to variation in the data, or if it is actually a product of variable speed limit implementation. Eight paired t-tests were performed to compare each treatment to the BASE case. The outcome of the t-tests and the ORCI are summarized in Table 5-8.
As shown from the table, Treatments 17 and 18 are the best treatments. With t-values greater than 1.645, both treatments are significantly different than the BASE case, with 95% confidence. Also, Treatment 18 is shown to be better than Treatment 17, with a higher t-value and a better ORCI. Though not found to be significant, Treatments 21 and 22 were observed to be the next best cases.
Treatments 17 and 18 were found to be significant. Figure 5-16 shows the rear-end crash profiles for Treatments 17 and 18 at station 37 E 0, as an example. This plot shows that Treatments 17 and 18 effectively decrease the rear-end crash risk, in time, from the BASE case at station 37 E 0. Treatment 18 is also shown here to have an edge on Treatment 17, as was shown from the t-tests.

5.2.3.2 Lane-Change Crash Risk

Having seen significant results from the rear-end crash risk analysis, the lane-change crash risk must also be analyzed. Figures 5-17 and 5-18 show the average lane-change crash risk
for each treatment and the BASE case over the entire network. Once again, all eight treatments perform positively in lowering the crash risk in general over the entire network.

Figure 5-17  Average Lane-Change Crash Risk v. Location for Treatments 17–20
All treatments performed very well near the beginning of the network (stations 15–23), and Treatments 17, 18, 21 and 22 performed particularly well between stations 33 and 53. Taking both of these together, the entire section of stations 15-53 is considered to be the effective segment. This 19-mile effective segment is shown in Figures 5-19 and 5-20, with each of the treatments plotted against the BASE case. The BASE case, which is bolded in blue, is shown to be higher than every treatment for most of the effective segment.
Average Lane-Change Crash Risk v. Location

Figure 5-19  Average Lane-Change Crash Risk v. Location for Treatments 17–20 – Magnified
All treatments are considered for significance in crash risk difference from the BASE case across the 19-mile effective segment. Eight t-tests were conducted to determine the significance of each treatment against the BASE case. These t-tests, along with the ORCI, are summarized in Table 5-9.
Table 5-9  Summary of Difference in Lane-Change Crash Risk for Treatments 17–24

<table>
<thead>
<tr>
<th>Test Case ID</th>
<th>Base</th>
<th>T17</th>
<th>T18</th>
<th>T19</th>
<th>T20</th>
<th>T21</th>
<th>T22</th>
<th>T23</th>
<th>T24</th>
</tr>
</thead>
<tbody>
<tr>
<td>Average Crash Risk (Stations 15 to 53)</td>
<td>0.603</td>
<td>0.426</td>
<td>0.401</td>
<td>0.480</td>
<td>0.491</td>
<td>0.484</td>
<td>0.473</td>
<td>0.523</td>
<td>0.528</td>
</tr>
<tr>
<td>Crash Risk Benefit</td>
<td>---</td>
<td>0.177</td>
<td>0.203</td>
<td>0.123</td>
<td>0.113</td>
<td>0.119</td>
<td>0.131</td>
<td>0.080</td>
<td>0.075</td>
</tr>
<tr>
<td>T-Statistic (Significant of Change)</td>
<td>---</td>
<td>3.063</td>
<td>3.480</td>
<td>1.772</td>
<td>1.628</td>
<td>2.068</td>
<td>2.201</td>
<td>1.195</td>
<td>1.124</td>
</tr>
</tbody>
</table>

From the table, it is shown that Treatments 17, 18, 19, 21 and 22 are able to significantly lower the lane-change crash risk from the BASE over the 19-mile effective segment. Treatments 17 and 18 perform much better than any other treatment. The ORCI’s for Treatments 17 and 18 are 13.485 and 15.410, respectively. This means that Treatment 18 also performs slightly better than Treatment 17, as was the case in the rear-end crash risk. Overall, the analysis shows that Treatments 17 and 18 are able to effectively lower the rear-end and lane-change crash risks on the network.

The lane-change crash profiles at Station 37 E 0 for Treatments 17, 18, 19, 21 and 22 are plotted in Figures 5-21 and 5-22. From the figures, it is clear that these treatments can effectively lower the lane-change crash risk in time at Station 37 E 0. While all were found significant above, Treatments 17 and 18 clearly outperform the others, as indicated by the ORCI. And once again, Treatment 18 is consistently better than Treatment 17, as also indicated by the ORCI.
Figure 5-21  Lane-Change Crash Profile at Station 37 E 0 for Treatments 17, 18 and 19
5.2.3.3 Travel Time Analysis

From the rear-end and lane-change crash risk analyses, it was found that Treatments 17 and 18 are able to significantly reduce both types of crash risk. Since these two are recommendable for implementation, their effects on network travel time should also be noted. The travel times were taken for every replication of the BASE and the two treatments. By looking at the averages, it was found that both treatments improve the travel time. Treatment 17 reduces the BASE travel time by 43.6 vehicle-hours, and Treatment 18 reduces the BASE travel time by 91.4 vehicle-hours.
Table 5-10  Analysis of Travel Times for Treatments 17 and 18

<table>
<thead>
<tr>
<th>Test Case ID</th>
<th>Base</th>
<th>T17</th>
<th>T18</th>
</tr>
</thead>
<tbody>
<tr>
<td>Average Travel Time (vehicle-hours)</td>
<td>11633.538</td>
<td>11589.892</td>
<td>11542.094</td>
</tr>
<tr>
<td>Travel Time Benefit</td>
<td>---</td>
<td>43.646</td>
<td>91.444</td>
</tr>
<tr>
<td>Percent Decrease</td>
<td>---</td>
<td>0.375</td>
<td>0.786</td>
</tr>
<tr>
<td>T-Statistic (Significance of Change)</td>
<td>---</td>
<td>1.519</td>
<td>3.186</td>
</tr>
</tbody>
</table>

The possibility of improving safety without having a negative effect on the travel time is a great thing, but here is the even greater possibility of improving both the crash risk and the travel time simultaneously. A t-test was performed for each treatment to compare their travel times with BASE travel times. A summary of these tests are shown in Table 5-10. As shown in the table, travel times for Treatment 18 are found to be significantly less than the travel times in the BASE case. This means that Treatment 18 can not only reduce the rear-end and lane-change crash risks on Interstate-4, but this strategy can also reduce the network travel time. This reduction in travel time is calculated to be about 0.8%. Therefore, Treatment 18 is more highly regarded than any of Treatments 17-20.

5.3  80 Percent Loading Scenario

The 24 VSL treatments were also run at the 80% loading scenario, which simulates conditions approaching congestion. These treatments were run to observe whether variable speed limits could help reduce the crash risk during the period approaching congestion.
5.3.1 Treatments 01 – 08

The analyses of treatments 01-08 are reviewed in this section. Treatments 01-08 are unique in their implementation strategy of variable speed limits. Each of these treatments, when implemented, lowers the upstream speed limits by 10 mph. Table 5-11 displays the different factors that make up Treatments 01-08, which are reviewed in this section.

Table 5-11 Description of Treatments 01-08

<table>
<thead>
<tr>
<th>Treatment ID</th>
<th>Speed Change Implementation</th>
<th>Speed Change Zone Threshold</th>
<th>Speed Change Distance</th>
<th>Speed Change Time Period</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>-10</td>
<td>5 mph</td>
<td>Speed Zone</td>
<td>5 min</td>
</tr>
<tr>
<td>2</td>
<td>-10</td>
<td>5 mph</td>
<td>Speed Zone</td>
<td>10 min</td>
</tr>
<tr>
<td>3</td>
<td>-10</td>
<td>5 mph</td>
<td>1/2 Speed Zone</td>
<td>5 min</td>
</tr>
<tr>
<td>4</td>
<td>-10</td>
<td>5 mph</td>
<td>1/2 Speed Zone</td>
<td>10 min</td>
</tr>
<tr>
<td>5</td>
<td>-10</td>
<td>2.5 mph</td>
<td>Speed Zone</td>
<td>5 min</td>
</tr>
<tr>
<td>6</td>
<td>-10</td>
<td>2.5 mph</td>
<td>Speed Zone</td>
<td>10 min</td>
</tr>
<tr>
<td>7</td>
<td>-10</td>
<td>2.5 mph</td>
<td>1/2 Speed Zone</td>
<td>5 min</td>
</tr>
<tr>
<td>8</td>
<td>-10</td>
<td>2.5 mph</td>
<td>1/2 Speed Zone</td>
<td>10 min</td>
</tr>
</tbody>
</table>

5.3.1.1 Rear-End Crash Risk

The rear-end crash risk was analyzed for Treatments 01-08. Figure 5-23 shows the average rear-end crash risk v. location for the whole network. Generally, most locations are not affected substantially by Treatments 01-08. The most affected segment occurs between stations 10 and 32.
The most affected segment is magnified in Figure 5-24. Stations 10 through 32 seem to be negatively affected by the implementation of VSL Treatments 01 through 08. As shown in the figure, the average rear-end crash risk for each treatment is shown to be higher in magnitude than the BASE case. This signifies that these treatments may actually negatively affect the rear-end crash risk on the network.
Table 5-12 summarizes the two sample t-tests performed to compare each treatment to the BASE case. As shown by the t-statistics, Treatments 01 through 07 have significantly higher observations of rear-end crash risk than the BASE case, over the affected segment of Stations 10 to 32. From the OI’s, Treatments 01, 02, 03 and 05 appear to be the worst treatments, with Treatment 02 being the absolute worst.
Table 5-12  Summary of Difference in Rear-End Crash Risk for Treatments 01-08

<table>
<thead>
<tr>
<th>Test Case ID</th>
<th>Base</th>
<th>T01</th>
<th>T02</th>
<th>T03</th>
<th>T04</th>
<th>T05</th>
<th>T06</th>
<th>T07</th>
<th>T08</th>
</tr>
</thead>
<tbody>
<tr>
<td>Average Crash Risk</td>
<td></td>
<td>-0.482</td>
<td>-0.322</td>
<td>-0.316</td>
<td>-0.326</td>
<td>-0.346</td>
<td>-0.325</td>
<td>-0.336</td>
<td>-0.356</td>
</tr>
<tr>
<td>(Stations 10 to 32)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Crash Risk Benefit</td>
<td>---</td>
<td>-0.160</td>
<td>-0.166</td>
<td>-0.156</td>
<td>-0.136</td>
<td>-0.157</td>
<td>-0.146</td>
<td>-0.125</td>
<td>-0.060</td>
</tr>
<tr>
<td>T-Statistic</td>
<td></td>
<td>2.215</td>
<td>2.293</td>
<td>2.150</td>
<td>1.892</td>
<td>2.189</td>
<td>2.049</td>
<td>1.770</td>
<td>0.863</td>
</tr>
<tr>
<td>(Significance of Change)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

5.3.1.2  Lane-Change Crash Risk

The lane-change crash risk was also observed for Treatments 01-08. Figure 5-25 shows the average lane-change crash risk for each location over the entire network. The segment that was most affected was from station 4 to 32. Like the rear-end crash risk, the general effect of the treatments in the affected section seems to be negative.
Figure 5-25  Average Lane-Change Crash Risk v. Location for Treatments 01–08

Figure 5-26 magnifies the affected section of stations 4 through 32. As shown the BASE has a lower lane-change crash risk than every treatment for most of the section.
Figure 5-26  Average Lane-Change Crash Risk v. Location for Treatments 01–08 - Magnified

The significance of the difference in average lane-change crash risk for each treatment from the BASE is summarized in Table 5-13. The crash risk benefit shows that all eight treatments negatively affect the lane-change crash risk. The t-statistics assert that Treatments 01 and 02 have significantly higher lane-change crash risks than the BASE case and, therefore, are considered negative treatments.
Table 5-13  Summary of Difference in Lane-Change Crash Risk for Treatments 01-08

<table>
<thead>
<tr>
<th>Test Case ID</th>
<th>Base</th>
<th>T01</th>
<th>T02</th>
<th>T03</th>
<th>T04</th>
<th>T05</th>
<th>T06</th>
<th>T07</th>
<th>T08</th>
</tr>
</thead>
<tbody>
<tr>
<td>Average Crash Risk (Stations 4 to 32)</td>
<td>-0.498</td>
<td>-0.189</td>
<td>-0.177</td>
<td>-0.289</td>
<td>-0.296</td>
<td>-0.223</td>
<td>-0.230</td>
<td>-0.311</td>
<td>-0.377</td>
</tr>
<tr>
<td>Crash Risk Benefit</td>
<td>---</td>
<td>-0.309</td>
<td>-0.321</td>
<td>-0.209</td>
<td>-0.202</td>
<td>-0.275</td>
<td>-0.267</td>
<td>-0.186</td>
<td>---</td>
</tr>
<tr>
<td>T-Statistic (Significance of Change)</td>
<td>---</td>
<td>1.786</td>
<td>1.863</td>
<td>1.105</td>
<td>1.071</td>
<td>1.516</td>
<td>1.478</td>
<td>0.994</td>
<td>0.656</td>
</tr>
<tr>
<td>ORCI</td>
<td>---</td>
<td>17.913</td>
<td>18.612</td>
<td>12.098</td>
<td>11.700</td>
<td>15.938</td>
<td>15.508</td>
<td>10.800</td>
<td>7.014</td>
</tr>
</tbody>
</table>

Based on the analysis, then, Treatments 01 and 02 were found to be significantly harmful to the network for rear-end and lane-change crash risks. Treatments 03-07 were found to be significantly harmful to the network’s rear-end crash risk only, and Treatment 08 was not found to have any significant effect to rear-end or lane-change crash risk. Therefore, none of these first eight treatments are recommended for application of VSL in the field.

5.3.2 Treatments 09 – 16

The analyses of treatments 09-16 are reviewed in this section. Treatments 09-16 are unique in their implementation strategy of variable speed limits. Each of these treatments, when implemented, lowers the upstream speed limits by 5 mph. Table 5-14 displays the different factors that make up Treatments 09-16, which are reviewed in this section.
Table 5-14 Description of Treatments 09-16

<table>
<thead>
<tr>
<th>Treatment ID</th>
<th>Speed Change Implementation</th>
<th>Speed Zone Threshold</th>
<th>Speed Change Distance</th>
<th>Speed Change Time Period</th>
</tr>
</thead>
<tbody>
<tr>
<td>9</td>
<td>-5</td>
<td>5 mph</td>
<td>Speed Zone</td>
<td>5 min</td>
</tr>
<tr>
<td>10</td>
<td>-5</td>
<td>5 mph</td>
<td>Speed Zone</td>
<td>10 min</td>
</tr>
<tr>
<td>11</td>
<td>-5</td>
<td>5 mph</td>
<td>1/2 Speed Zone</td>
<td>5 min</td>
</tr>
<tr>
<td>12</td>
<td>-5</td>
<td>5 mph</td>
<td>1/2 Speed Zone</td>
<td>10 min</td>
</tr>
<tr>
<td>13</td>
<td>-5</td>
<td>2.5 mph</td>
<td>Speed Zone</td>
<td>5 min</td>
</tr>
<tr>
<td>14</td>
<td>-5</td>
<td>2.5 mph</td>
<td>Speed Zone</td>
<td>10 min</td>
</tr>
<tr>
<td>15</td>
<td>-5</td>
<td>2.5 mph</td>
<td>1/2 Speed Zone</td>
<td>5 min</td>
</tr>
<tr>
<td>16</td>
<td>-5</td>
<td>2.5 mph</td>
<td>1/2 Speed Zone</td>
<td>10 min</td>
</tr>
</tbody>
</table>

5.3.2.1 Rear-End Crash Risk

The rear-end crash risk was analyzed for Treatments 09-16. Figure 5-27 shows the average-rear end crash risk v. location for the whole network. Most locations are not substantially affected by Treatments 09-16. The most affected segment occurs between stations 14 and 43.
The most affected segment is magnified in Figure 5-28. Stations 14 through 43 seem to be negatively affected by the implementation of VSL Treatments in general. However, these differences appear to be very minute.
Table 5-15 summarizes the two sample t-tests performed to compare each treatment to the BASE case. As shown by the t-statistics, Treatments 09 through 16 do not have significantly higher observations of rear-end crash risk than the BASE case. Therefore, in terms of rear-end crash risk, Treatments 09-16 do not have any significant effect on the network.

**Table 5-15  Summary of Difference in Rear-End Crash Risk for Treatments 09-16**

<table>
<thead>
<tr>
<th>Test Case ID</th>
<th>Average Crash Risk (Stations 14 to 43)</th>
<th>Crash Risk Benefit</th>
<th>T-Statistic (Significance of Change)</th>
<th>ORCI</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Base 0.249</td>
<td>T09 0.321</td>
<td>T10 0.315</td>
<td>T11 0.270</td>
</tr>
<tr>
<td></td>
<td>--- -0.072</td>
<td>--- -0.066</td>
<td>--- -0.021</td>
<td>--- -0.029</td>
</tr>
<tr>
<td></td>
<td>--- 0.380</td>
<td>--- 0.349</td>
<td>--- 0.109</td>
<td>--- 0.150</td>
</tr>
<tr>
<td></td>
<td>--- -4.179</td>
<td>--- -3.812</td>
<td>--- -1.207</td>
<td>--- -1.675</td>
</tr>
</tbody>
</table>
5.3.2.2 Lane-Change Crash Risk

The lane-change crash risk was also observed for Treatments 09-16. Figure 5-29 shows the average lane-change crash risk for each location over the entire network. The segment that was most affected was from station 11 to 31. Like the rear-end crash risk, the general effects of the treatments in the affected section seem to be minimal.

![Average Lane-Change Crash Risk v. Location](image)

Figure 5-29  Average Lane-Change Crash Risk v. Location for Treatments 09-16

Figure 5-30 magnifies the affected section of stations 11 through 31. As shown the BASE generally has a lower lane-change crash risk than every treatment for most of the section.
The significance of the difference in average lane-change crash risk for each treatment from the BASE is summarized in Table 5-16. The crash risk benefits show mixed effects for the lane-change crash risk of the eight treatments. The t-statistics show that, like the rear-end crash risk analyses, none of the treatments appear to have a significant effect on the lane-change crash risk. Therefore, based on the rear-end and lane-change crash risk analyses, Treatments 09-16 are considered ineffective (positively or negatively) in the 80% loading scenario.
Table 5-16  Summary of Difference in Lane-Change Crash Risk for Treatments 09-16

<table>
<thead>
<tr>
<th>Test Case ID</th>
<th>Base</th>
<th>T09</th>
<th>T10</th>
<th>T11</th>
<th>T12</th>
<th>T13</th>
<th>T14</th>
<th>T15</th>
<th>T16</th>
</tr>
</thead>
<tbody>
<tr>
<td>Average Crash Risk (Stations 11 to 31)</td>
<td>-0.067</td>
<td>0.094</td>
<td>0.103</td>
<td>0.001</td>
<td>-0.022</td>
<td>0.041</td>
<td>0.044</td>
<td>-0.034</td>
<td>-0.018</td>
</tr>
<tr>
<td>Crash Risk Benefit</td>
<td>---</td>
<td>-0.161</td>
<td>-0.170</td>
<td>-0.068</td>
<td>-0.045</td>
<td>-0.108</td>
<td>-0.110</td>
<td>-0.033</td>
<td>-0.049</td>
</tr>
<tr>
<td>T-Statistic (Significance of Change)</td>
<td>---</td>
<td>1.289</td>
<td>1.360</td>
<td>0.530</td>
<td>0.351</td>
<td>0.861</td>
<td>0.874</td>
<td>0.258</td>
<td>0.378</td>
</tr>
</tbody>
</table>

5.3.3 Treatments 17 – 24

Treatments 17-24 are unique in their VSL implementation strategy, in that the speed limits are decreased by 5 mph upstream and increased by 5 mph downstream for all treatments. Table 5-17 summarizes the factors for Treatments 17-24, which are reviewed in this section.

Table 5-17 Description of Treatments 17-24

<table>
<thead>
<tr>
<th>Treatment ID</th>
<th>Speed Change Implementation</th>
<th>Speed Zone Threshold</th>
<th>Speed Change Distance</th>
<th>Speed Change Time Period</th>
</tr>
</thead>
<tbody>
<tr>
<td>17</td>
<td>-5/+5</td>
<td>5 mph</td>
<td>Speed Zone</td>
<td>5 min</td>
</tr>
<tr>
<td>18</td>
<td>-5/+5</td>
<td>5 mph</td>
<td>Speed Zone</td>
<td>10 min</td>
</tr>
<tr>
<td>19</td>
<td>-5/+5</td>
<td>5 mph</td>
<td>1/2 Speed Zone</td>
<td>5 min</td>
</tr>
<tr>
<td>20</td>
<td>-5/+5</td>
<td>5 mph</td>
<td>1/2 Speed Zone</td>
<td>10 min</td>
</tr>
<tr>
<td>21</td>
<td>-5/+5</td>
<td>2.5 mph</td>
<td>Speed Zone</td>
<td>5 min</td>
</tr>
<tr>
<td>22</td>
<td>-5/+5</td>
<td>2.5 mph</td>
<td>Speed Zone</td>
<td>10 min</td>
</tr>
<tr>
<td>23</td>
<td>-5/+5</td>
<td>2.5 mph</td>
<td>1/2 Speed Zone</td>
<td>5 min</td>
</tr>
<tr>
<td>24</td>
<td>-5/+5</td>
<td>2.5 mph</td>
<td>1/2 Speed Zone</td>
<td>10 min</td>
</tr>
</tbody>
</table>
5.3.3.1 Rear-End Crash Risk

The effects that Treatments 17-24 have on rear-end crash risk are reviewed in this section. Figures 5-31 and 5-32 show the average rear-end crash risk for every location for the BASE case and Treatments 17-20 and 21-24, respectively.

![Average Rear-End Crash Risk v. Location for Treatments 17-20](image)

Figure 5-31 Average Rear-End Crash Risk v. Location for Treatments 17-20
Figure 5-32  Average Rear-End Crash Risk v. Location for Treatments 21-24

Unlike the 60% loading scenario, the effects of these treatments in the 80% scenario seem to affect 2 segments separately. Segment 1 is from stations 14-31, and Segment 2 is from stations 33-44.

5.3.3.1.1  Segment 1: Stations 14 – 31

Across this 1st segment, the major differences in rear-end crash risk appear to be negative. That is, the implementation of these treatments may actually increase the crash risk at this segment. Figures 5-33 and 5-34 magnify stations 14-31 for Treatments 17-20 and 21-24,
respectively. The treatments are broken up for easier distinction on the graphs. It is shown that, in this section, there seems to be an increase in rear-end crash risk for Treatments 17, 18, 21 and 22. For Treatments 19, 20, 23 and 24, however, there is very little difference, if any.

Figure 5-33  Average Rear-End Crash Risk v. Location for Treatments 17-20 for 1st Section - Magnified
Table 5-18 summarizes the differences in rear-end crash risk and computes the t-statistic for each treatment against the BASE, for the first segment. Though none of the differences, negative or positive, are shown to be significant, it is interesting to note that Treatments 17, 18, 21 and 22 have a harmful effect while Treatments 19, 20, 23 and 24 have a beneficial effect. The defining factor that differentiates these treatments is the spatial distance over which the VSL is implemented. The harmful treatments implement over the entire speed zone (multiplier = 1.0), and the beneficial treatments implement over half the speed zone (multiplier = 0.5).
Table 5-18 1st Section: Summary of Difference in Rear-End Crash Risk for Treatments 17-24

<table>
<thead>
<tr>
<th>Test Case ID</th>
<th>Average Crash Risk (Stations 14 to 31)</th>
<th>Crash Risk Benefit</th>
<th>T-Statistic (Significance of Change)</th>
<th>ORCI</th>
</tr>
</thead>
<tbody>
<tr>
<td>Base</td>
<td>-0.459</td>
<td>---</td>
<td>---</td>
<td>---</td>
</tr>
<tr>
<td>T17</td>
<td>-0.377</td>
<td>-0.083</td>
<td>1.245</td>
<td>-2.981</td>
</tr>
<tr>
<td>T18</td>
<td>-0.370</td>
<td>-0.090</td>
<td>1.343</td>
<td>-3.228</td>
</tr>
<tr>
<td>T19</td>
<td>-0.479</td>
<td>0.019</td>
<td>0.294</td>
<td>0.700</td>
</tr>
<tr>
<td>T20</td>
<td>-0.474</td>
<td>0.015</td>
<td>0.222</td>
<td>0.533</td>
</tr>
<tr>
<td>T21</td>
<td>-0.383</td>
<td>-0.077</td>
<td>1.129</td>
<td>-2.765</td>
</tr>
<tr>
<td>T22</td>
<td>-0.391</td>
<td>-0.069</td>
<td>1.004</td>
<td>-2.476</td>
</tr>
<tr>
<td>T23</td>
<td>-0.474</td>
<td>0.015</td>
<td>0.227</td>
<td>0.542</td>
</tr>
<tr>
<td>T24</td>
<td>-0.473</td>
<td>0.014</td>
<td>0.205</td>
<td>0.486</td>
</tr>
</tbody>
</table>

5.3.3.1.2 Segment 2: Stations 33 – 44

The 2nd segment affected by these treatments occurs downstream from station 33 to station 44. In this segment, each treatment shows a potential benefit for rear-end crash risk. Figures 5-35 and 5-36 magnify stations 33-44 for Treatments 17-20 and 21-24, respectively. The greatest benefit appears to occur with Treatments 17, 18, 21 and 22.
Figure 5-35  Average Rear-End Crash Risk v. Location for Treatments 17-20 for 2nd Section - Magnified
Table 5-19 summarizes the differences in rear-end crash risk and computes the t-statistic for each treatment against the BASE, for the second segment. The table confirms that all eight treatments perform beneficially for the rear-end crash risk. None of these, however, are found to be significant. The ORCI’s show that Treatments 17, 18, 21 and 22 have the greatest benefit in the 2nd segment. It should be noted that, due to the complex nature of homogeneous speed zones and the corridor-wide implementation, it cannot be asserted that multiple treatments implemented simultaneously at different locations would work best. This would have to be subject to further testing.
Table 5-19  2nd Section: Summary of Difference in Rear-End Crash Risk for Treatments 17-24

<table>
<thead>
<tr>
<th>Test Case ID</th>
<th>Base</th>
<th>T17</th>
<th>T18</th>
<th>T19</th>
<th>T20</th>
<th>T21</th>
<th>T22</th>
<th>T23</th>
<th>T24</th>
</tr>
</thead>
<tbody>
<tr>
<td>Average Crash Risk (Stations 33 to 44)</td>
<td>1.440</td>
<td>1.259</td>
<td>1.275</td>
<td>1.418</td>
<td>1.426</td>
<td>1.300</td>
<td>1.288</td>
<td>1.414</td>
<td>1.427</td>
</tr>
<tr>
<td>Crash Risk Benefit</td>
<td>---</td>
<td>0.182</td>
<td>0.165</td>
<td>0.023</td>
<td>0.015</td>
<td>0.141</td>
<td>0.152</td>
<td>0.026</td>
<td>0.013</td>
</tr>
<tr>
<td>T-Statistic (Significance of Change)</td>
<td>---</td>
<td>0.987</td>
<td>0.913</td>
<td>0.118</td>
<td>0.078</td>
<td>0.758</td>
<td>0.815</td>
<td>0.136</td>
<td>0.067</td>
</tr>
<tr>
<td>ORCI</td>
<td>---</td>
<td>3.997</td>
<td>3.635</td>
<td>0.496</td>
<td>0.325</td>
<td>3.092</td>
<td>3.345</td>
<td>0.570</td>
<td>0.284</td>
</tr>
</tbody>
</table>

5.3.3.1.3 Summary of Affected Segments

Since there were two affected segments in this analysis, the ORCI’s must be summed to compute a cumulative ORCI, taking into account the negative and positive effects of each treatment. Table 5-20 shows the cumulative rear-end ORCI’s for Treatments 17-24 across the two affected segments. The best two treatments are shown to be Treatments 19 and 23, which commonly share the experimental values of 0.5 for the speed zone multiplier and 5 min for the minimum implementation time. From this rear-end analysis, it seems as though the treatments implementing over only half the speed zones may be more robust against the crash risk migration phenomenon.

Table 5-20  Cumulative Rear-End ORCI Values for Treatments 17-24

<table>
<thead>
<tr>
<th>Test Case ID</th>
<th>Base</th>
<th>T17</th>
<th>T18</th>
<th>T19</th>
<th>T20</th>
<th>T21</th>
<th>T22</th>
<th>T23</th>
<th>T24</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cumulative ORCI</td>
<td>---</td>
<td>1.017</td>
<td>0.406</td>
<td>1.196</td>
<td>0.858</td>
<td>0.327</td>
<td>0.869</td>
<td>1.112</td>
<td>0.770</td>
</tr>
</tbody>
</table>

5.3.3.2 Lane-Change Crash Risk

The effects that Treatments 17-24 have on lane-change crash risk are reviewed in this section. Figures 5-37 and 5-38 show the average lane-change crash risk for every location for
the BASE case and Treatments 17-20 and 21-24, respectively. Like the rear-end crash risk, Treatments 17-24 affect the lane-change crash risk over the network in segments. Three affected segments exist in this case: stations 6-14 (Segment 1), stations 15-21 (Segment 2), and stations 32-52 (Segment 3).

![Average Lane-Change Crash Risk v. Location](image)

**Figure 5-37 Average Lane-Change Crash Risk v. Location for Treatments 17-20**
5.3.3.2.1 Segment 1: Stations 6 – 14

Like the 1<sup>st</sup> segment in the rear-end crash risk analysis, there is a general increase in lane-change crash risk for the treatments over the 1<sup>st</sup> segment. Again, Treatments 17, 18, 21 and 22 have the greatest increase, while the changes due to the other treatments are minimal. Figures 5-39 and 5-40 magnify stations 6-14 for Treatments 17-20 and 21-24, respectively.
Figure 5-39  Average Lane-Change Crash Risk v. Location for Treatments 17-20 for 1st Section - Magnified
Figure 5-40  Average Lane-Change Crash Risk v. Location for Treatments 21-24 for 1st Section - Magnified

Table 5-21 summarizes the differences in lane-change crash risk and computes the t-statistic for each treatment against the BASE, for the first segment. As shown, none of the treatments actually creates a significant increase in the lane-change crash risk over the first segment.
### Table 5-21 1st Section: Summary of Difference in Lane-Change Crash Risk for Treatments 17-24

<table>
<thead>
<tr>
<th>Test Case ID</th>
<th>Base</th>
<th>T17</th>
<th>T18</th>
<th>T19</th>
<th>T20</th>
<th>T21</th>
<th>T22</th>
<th>T23</th>
<th>T24</th>
</tr>
</thead>
<tbody>
<tr>
<td>Average Crash Risk (Stations 6 to 14)</td>
<td>-1.562</td>
<td>-1.276</td>
<td>-1.280</td>
<td>-1.519</td>
<td>-1.509</td>
<td>-1.398</td>
<td>-1.401</td>
<td>-1.528</td>
<td>-1.520</td>
</tr>
<tr>
<td>Crash Risk Benefit</td>
<td>---</td>
<td>-0.287</td>
<td>-0.282</td>
<td>-0.043</td>
<td>-0.054</td>
<td>-0.164</td>
<td>-0.162</td>
<td>-0.035</td>
<td>-0.042</td>
</tr>
<tr>
<td>T-Statistic (Significance of Change)</td>
<td>---</td>
<td>1.371</td>
<td>1.354</td>
<td>0.224</td>
<td>0.281</td>
<td>0.794</td>
<td>0.791</td>
<td>0.180</td>
<td>0.219</td>
</tr>
<tr>
<td>ORCI</td>
<td>---</td>
<td>-5.161</td>
<td>-5.076</td>
<td>-0.773</td>
<td>-0.966</td>
<td>-2.954</td>
<td>-2.911</td>
<td>-0.623</td>
<td>-0.761</td>
</tr>
</tbody>
</table>

#### 5.3.3.2.2 Segment 2: Stations 15 – 21

The 2nd segment occurs over stations 15-21. Figures 5-41 and 5-42 magnify stations 15-21 for Treatments 17-20 and 21-24, respectively. These plots show that all of the treatments have a beneficial effect over this 2nd segment, and that Treatments 19, 20, 23 and 24 have the greatest benefits.
Figure 5-41  Average Lane-Change Crash Risk v. Location for Treatments 17-20 for 2nd Section - Magnified
Table 5-22 summarizes the differences in lane-change crash risk and computes the t-statistic for each treatment against the BASE, for the second segment. As shown from the t-values, all of the treatments significantly reduce the lane-change crash risk over the second segment. Of these beneficial treatments, Treatments 19, 20, 23 and 24 are the best.
Table 5-22 2nd Section: Summary of Difference in Lane-Change Crash Risk for Treatments 17-24

<table>
<thead>
<tr>
<th>Test Case ID</th>
<th>Base</th>
<th>T17</th>
<th>T18</th>
<th>T19</th>
<th>T20</th>
<th>T21</th>
<th>T22</th>
<th>T23</th>
<th>T24</th>
</tr>
</thead>
<tbody>
<tr>
<td>Average Crash Risk (Stations 15 to 21)</td>
<td>0.293</td>
<td>0.214</td>
<td>0.201</td>
<td>-0.018</td>
<td>-0.023</td>
<td>0.168</td>
<td>0.132</td>
<td>0.051</td>
<td>0.092</td>
</tr>
<tr>
<td>Crash Risk Benefit</td>
<td>---</td>
<td>0.079</td>
<td>0.093</td>
<td>0.311</td>
<td>0.316</td>
<td>0.125</td>
<td>0.162</td>
<td>0.242</td>
<td>0.202</td>
</tr>
<tr>
<td>T-Statistic (Significance of Change)</td>
<td>---</td>
<td>2.502</td>
<td>3.033</td>
<td>7.589</td>
<td>8.159</td>
<td>3.419</td>
<td>4.359</td>
<td>6.075</td>
<td>5.088</td>
</tr>
<tr>
<td>ORCI</td>
<td>---</td>
<td>1.109</td>
<td>1.297</td>
<td>4.354</td>
<td>4.427</td>
<td>1.753</td>
<td>2.262</td>
<td>3.393</td>
<td>2.824</td>
</tr>
</tbody>
</table>

5.3.3.2.3 Segment 3: Stations 32 - 52

The 3rd segment occurs over stations 32-52. Figures 5-43 and 5-44 magnify stations 32-52 for Treatments 17-20 and 21-24, respectively.

Figure 5-43 Average Lane-Change Crash Risk v. Location for Treatments 17-20 for 3rd Section - Magnified
Table 5-23 summarizes the differences in lane-change crash risk and computes the t-statistic for each treatment against the BASE, for the third segment. As shown from the t-values, Treatments 17 and 18 create a significant decrease in the lane-change crash risk over the third segment. No other treatment is shown to have a significant effect on the lane-change crash risk in this area.
Table 5-23 3rd Section: Summary of Difference in Lane-Change Crash Risk for Treatments 17-24

<table>
<thead>
<tr>
<th>Test Case ID</th>
<th>Base</th>
<th>T17</th>
<th>T18</th>
<th>T19</th>
<th>T20</th>
<th>T21</th>
<th>T22</th>
<th>T23</th>
<th>T24</th>
</tr>
</thead>
<tbody>
<tr>
<td>Average Crash Risk (Stations 32 to 52)</td>
<td>0.901</td>
<td>0.782</td>
<td>0.804</td>
<td>0.886</td>
<td>0.910</td>
<td>0.839</td>
<td>0.842</td>
<td>0.896</td>
<td>0.889</td>
</tr>
<tr>
<td>Crash Risk Benefit</td>
<td>---</td>
<td>0.119</td>
<td>0.097</td>
<td>0.015</td>
<td>-0.008</td>
<td>0.063</td>
<td>0.059</td>
<td>0.005</td>
<td>0.012</td>
</tr>
<tr>
<td>T-Statistic (Significance of Change)</td>
<td>---</td>
<td>2.757</td>
<td>2.133</td>
<td>0.346</td>
<td>0.181</td>
<td>1.453</td>
<td>1.348</td>
<td>0.109</td>
<td>0.270</td>
</tr>
<tr>
<td>ORCI</td>
<td>---</td>
<td>4.779</td>
<td>3.889</td>
<td>0.611</td>
<td>-0.339</td>
<td>2.505</td>
<td>2.349</td>
<td>0.196</td>
<td>0.474</td>
</tr>
</tbody>
</table>

5.3.3.2.4 Summary of Affected Sections

Like the rear-end analysis for Treatments 17-24, the cumulative ORCI for each treatment must be calculated by taking into account the positive and negative effects across the three affected segments. As shown, Treatments 19 and 20 clearly outperform every other treatment. The next best strategies are Treatments 23 and 24.

Table 5-24 Cumulative Lane-Change ORCI Values for Treatments 17-24

<table>
<thead>
<tr>
<th>cumulative ORCI</th>
<th>Base</th>
<th>T17</th>
<th>T18</th>
<th>T19</th>
<th>T20</th>
<th>T21</th>
<th>T22</th>
<th>T23</th>
<th>T24</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cumulative ORCI</td>
<td>---</td>
<td>0.727</td>
<td>0.109</td>
<td>4.191</td>
<td>3.122</td>
<td>1.304</td>
<td>1.699</td>
<td>2.966</td>
<td>2.538</td>
</tr>
</tbody>
</table>

The lane-change crash profile over the 5-min time steps for Treatment 19 at Station 18 E 1 is shown in Figure 5-45. This station is located in the 2nd affected section, in which Treatment 19 significantly reduced the lane-change crash risk. Although there is an apparent variation in the crash risk, Treatment 19 is very effective in keeping it well below the BASE case crash risk for the majority of the simulation. This segment is located downstream from Station 14, where the speed limit is reduced from 65 mph to 55 mph. Therefore, there is an inherent 10 mph speed difference built into the network at this point, which often triggers the application of variable speeds limits in this area. Every time VSL is implemented, the crash risk is brought down.
However, as soon as the speed limit is restored, the crash rises back to its original location. Suggestions for this special case include a gradual speed limit change upstream from 65 mph to 60 mph, before changing to 55 mph at Station 18. Another suggestion would be to implement VSL for longer in this area, such as 30 minutes or for the entire peak period.

![Average Lane-Change Crash Risk v. Time Period](image)

**Figure 5-45**  Lane-Change Crash Profile at Station 18 E 1 for Treatment 19

Treatments 19, 20, 23 and 24 perform very well, not only due to their superior abilities to significantly reduce the lane-change crash risk in segment 2, but also because of their resistance to increasing the lane-change crash risk in segment 1. As mentioned in the rear-end analysis of these treatments, this implies that the implementation of variable speed limits over only half the length of speed zones makes the strategy robust against crash migration. Furthermore, the crash
risk reduction is more significant when the speed zones are more liberally defined (5 mph thresholds rather than 2.5 mph). Therefore, due to its superior capabilities in the rear-end and lane-change crash risk analyses, Treatment 19 is declared to be the best variable speed limit strategy for the 80% loading scenario.

5.3.3.3 Travel Time Analysis

Treatment 19 proved to be the best strategy for variable speed limit implementation in the 80% loading scenario. It was observed to be superior in both the rear-end and lane-change crash risk analyses. The effect that this treatment has on travel must also be observed. Twenty replications of this treatment were performed, and the total network travel time was taken from each run. The travel times for each replication were compared with the travel times collected from the BASE case replications, as shown in Table 5-25.

<table>
<thead>
<tr>
<th>Test Case ID</th>
<th>Base</th>
<th>T19</th>
</tr>
</thead>
<tbody>
<tr>
<td>Average Travel Time (vehicle-hours)</td>
<td>16391.447</td>
<td>16451.873</td>
</tr>
<tr>
<td>Travel Time Increase</td>
<td>---</td>
<td>60.425</td>
</tr>
<tr>
<td>Percent Increase</td>
<td>---</td>
<td>0.369</td>
</tr>
<tr>
<td>T-Statistic (Significance of Change)</td>
<td>---</td>
<td>0.875</td>
</tr>
</tbody>
</table>

The table shows that an increase in travel time of 60 VHT is experienced by the network with the application of Treatment 19. This increase is less than 0.4 percent and was not found to be significant with a t-test. Therefore, the observed effect of Treatment 19 on the network travel time is acceptable, and Treatment 19 is shown to be the best 80% loading VSL Treatment.
5.4 90 Percent Loading Scenario

The 24 VSL treatments were also run at the 90% loading scenario, which simulates typical peak period congestion. These treatments were run to observe whether variable speed limits could help lower crash risk during the typical peak period on Interstate-4.

5.4.1 Treatments 01 – 08

The analyses of treatments 01-08 are reviewed in this section. Treatments 01-08 are unique in their implementation strategy of variable speed limits. Each of these treatments, when implemented, lowers the upstream speed limits by 10 mph. Table 5-26 displays the different factors that make up Treatments 01-08, which are reviewed in this section.

Table 5-26 Description of Treatments 01-08

<table>
<thead>
<tr>
<th>Treatment ID</th>
<th>Speed Change Implementation</th>
<th>Speed Zone Threshold</th>
<th>Speed Change Distance</th>
<th>Speed Change Time Period</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>-10</td>
<td>5 mph</td>
<td>Speed Zone</td>
<td>5 min</td>
</tr>
<tr>
<td>2</td>
<td>-10</td>
<td>5 mph</td>
<td>Speed Zone</td>
<td>10 min</td>
</tr>
<tr>
<td>3</td>
<td>-10</td>
<td>5 mph</td>
<td>1/2 Speed Zone</td>
<td>5 min</td>
</tr>
<tr>
<td>4</td>
<td>-10</td>
<td>5 mph</td>
<td>1/2 Speed Zone</td>
<td>10 min</td>
</tr>
<tr>
<td>5</td>
<td>-10</td>
<td>2.5 mph</td>
<td>Speed Zone</td>
<td>5 min</td>
</tr>
<tr>
<td>6</td>
<td>-10</td>
<td>2.5 mph</td>
<td>Speed Zone</td>
<td>10 min</td>
</tr>
<tr>
<td>7</td>
<td>-10</td>
<td>2.5 mph</td>
<td>1/2 Speed Zone</td>
<td>5 min</td>
</tr>
<tr>
<td>8</td>
<td>-10</td>
<td>2.5 mph</td>
<td>1/2 Speed Zone</td>
<td>10 min</td>
</tr>
</tbody>
</table>

5.4.1.1 Rear-End Crash Risk

The rear-end crash risk was analyzed first to see if any of these treatments can effectively reduce the crash risk on Interstate-4. In previous loading scenarios (60%, 80%), these treatments have not been shown to be effective at reducing crash risk. In fact, in the 80% loading scenario,
all but Treatment 08 significantly increased the rear-end crash risk on the freeway. Figure 5-46 shows the average rear-end crash risk plotted over each location for Treatments 01-08.

Two main sections are affected by these treatments on the network. Segment 1 is negatively affected and stretches from stations 10-34. Segment 2 is positively affected and is located between stations 38 and 43.

5.4.1.1.1 Segment 1: Stations 10 – 34

Figure 5-47 shows the average rear-end crash risk for each location in Segment 1. This is typical of the rear-end crash risk increase observed by these treatments in other loading scenarios. Every treatment is shown to increase the rear-end crash risk in this area.
Table 5-27 summarizes the t-tests and the ORCI for this segment. The average crash risk benefit is negative for each treatment, which means that the effect of the treatments in harmful to the network safety. These values range between -0.20 and -0.45. The t-statistics show that the first 7 treatments significantly increase the average rear-end crash risk over segment 1. These results are consistent with the 80% loading results as well.
Table 5-27 1st Segment: Summary of Difference in Rear-End Crash Risk for Treatments 01-08

<table>
<thead>
<tr>
<th>Test Case ID</th>
<th>Test Case ID</th>
</tr>
</thead>
<tbody>
<tr>
<td>Average Crash Risk</td>
<td></td>
</tr>
<tr>
<td>(Stations 10 to 34)</td>
<td>Base</td>
</tr>
<tr>
<td></td>
<td>T01</td>
</tr>
<tr>
<td></td>
<td>T02</td>
</tr>
<tr>
<td></td>
<td>T03</td>
</tr>
<tr>
<td></td>
<td>T04</td>
</tr>
<tr>
<td></td>
<td>T05</td>
</tr>
<tr>
<td></td>
<td>T06</td>
</tr>
<tr>
<td></td>
<td>T07</td>
</tr>
<tr>
<td></td>
<td>T08</td>
</tr>
<tr>
<td>-0.347</td>
<td>0.101</td>
</tr>
<tr>
<td>-0.100</td>
<td>-0.106</td>
</tr>
<tr>
<td>0.009</td>
<td>-0.021</td>
</tr>
<tr>
<td>-0.102</td>
<td>-0.147</td>
</tr>
<tr>
<td>Crash Risk Benefit</td>
<td></td>
</tr>
<tr>
<td></td>
<td>---</td>
</tr>
<tr>
<td></td>
<td>-0.448</td>
</tr>
<tr>
<td></td>
<td>-0.407</td>
</tr>
<tr>
<td></td>
<td>-0.248</td>
</tr>
<tr>
<td></td>
<td>-0.241</td>
</tr>
<tr>
<td></td>
<td>-0.357</td>
</tr>
<tr>
<td></td>
<td>-0.326</td>
</tr>
<tr>
<td></td>
<td>-0.246</td>
</tr>
<tr>
<td></td>
<td>-0.201</td>
</tr>
<tr>
<td>T-Statistic</td>
<td></td>
</tr>
<tr>
<td>(Significance of Change)</td>
<td>---</td>
</tr>
<tr>
<td></td>
<td>3.723</td>
</tr>
<tr>
<td></td>
<td>3.418</td>
</tr>
<tr>
<td></td>
<td>2.029</td>
</tr>
<tr>
<td></td>
<td>1.962</td>
</tr>
<tr>
<td></td>
<td>2.929</td>
</tr>
<tr>
<td></td>
<td>2.704</td>
</tr>
<tr>
<td></td>
<td>1.961</td>
</tr>
<tr>
<td></td>
<td>1.621</td>
</tr>
<tr>
<td>ORCI</td>
<td></td>
</tr>
<tr>
<td></td>
<td>---</td>
</tr>
<tr>
<td></td>
<td>-22.41</td>
</tr>
<tr>
<td></td>
<td>-20.35</td>
</tr>
<tr>
<td></td>
<td>-12.39</td>
</tr>
<tr>
<td></td>
<td>-12.06</td>
</tr>
<tr>
<td></td>
<td>-17.84</td>
</tr>
<tr>
<td></td>
<td>-16.30</td>
</tr>
<tr>
<td></td>
<td>-12.28</td>
</tr>
<tr>
<td></td>
<td>-10.03</td>
</tr>
</tbody>
</table>

5.4.1.1.2 Segment 2: Stations 38 – 43

Figure 5-48 shows the average rear-end crash risk of each treatment for each location in Segment 2. In this segment, Treatments 01-08 are able to reduce the rear-end crash risk. The typical reduction in crash risk ranges from 0.47 to 0.53, as shown in Table 5-28. Though this is a greater change in average rear-end crash risk than was shown in segment 1, it is for a much shorter length of 2.5 miles, compared to segment 1’s length of 12.5 miles. With 18 degrees of freedom, the critical t-value is 1.734. The table shows that some treatments are close, and Treatment 06 is very close, to being significant. However, none of them actually are, and the ORCI values for segment 2 do not begin to compensate for the negative ORCI values in segment 1.
5.4.1.1.3 Summary of Affected Sections

In the 90% loading scenario, as in the other loading scenarios, Treatments 01-08 do not positively reduce the rear-end crash risk on the network. These treatments actually serve to
significantly increase the risk over a large area, as previously shown in Figure 5-47. The cumulative ORCI values for each treatment is shown in Table 5-29, taking into account both the negative and positive affects of Treatment 01-08 on the average rear-end crash risk. As seen from the table, the treatments have highly negative ORCI values, which is unacceptable for VSL treatments intended to reduce the risk of crashes occurring.

**Table 5-29  Cumulative Rear-End ORCI Values for Treatments 01-08**

<table>
<thead>
<tr>
<th>Cumulative ORCI</th>
<th>Base</th>
<th>T01</th>
<th>T02</th>
<th>T03</th>
<th>T04</th>
<th>T05</th>
<th>T06</th>
<th>T07</th>
<th>T08</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>---</td>
<td>-17.4</td>
<td>-15.2</td>
<td>-7.5</td>
<td>-7.3</td>
<td>-13.0</td>
<td>-11.0</td>
<td>-7.3</td>
<td>-5.3</td>
</tr>
</tbody>
</table>

5.4.1.2 Lane-Change Crash Risk

The lane-change crash risk was analyzed next. Like the rear-end crash risk, these treatments have not been shown to be effective at reducing lane-change crash risk in previous loading scenarios. Figure 5-49 shows the average lane-change crash risk plotted over each location for Treatments 01-08.
Two main sections are affected by these treatments on the network. Segment 1 is negatively affected and stretches from stations 4-40. Segment 2 is positively affected and is located between stations 42 and 50.

5.4.1.1.1 Segment 1: Stations 4 - 40

Figure 5-50 shows the average lane-change crash risk for each location in Segment 1. Every treatment is shown to increase the rear-end crash risk in this area, which is typical of these treatments in the previous loading scenarios as well.
Figure 5-50  Average Lane-Change Crash Risk v. Location for Treatments 01-08 for Segment 1

Table 5-30 summarizes the t-tests and the ORCI for this segment. The average crash risk benefit is negative for each treatment, which means that the effect of the treatments is harmful to the network safety. The t-statistics show that the Treatments 01, 02, 05, 06 and 07 significantly increase the average lane-change crash risk over segment 1. Once again, it is interesting to note that the treatments using the entire speed zone distance for implementation (Treatments 01, 02, 05 and 06) consistently have the greatest negative effect in this segment, as seen from the ORCI values.
<table>
<thead>
<tr>
<th>Test Case ID</th>
<th>Base</th>
<th>T01</th>
<th>T02</th>
<th>T03</th>
<th>T04</th>
<th>T05</th>
<th>T06</th>
<th>T07</th>
<th>T08</th>
</tr>
</thead>
<tbody>
<tr>
<td>Average Crash Risk</td>
<td>-0.208</td>
<td>0.343</td>
<td>0.300</td>
<td>0.068</td>
<td>0.058</td>
<td>0.166</td>
<td>0.133</td>
<td>0.081</td>
<td>0.037</td>
</tr>
<tr>
<td>(Stations 4 to 40)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Crash Risk Benefit</td>
<td>---</td>
<td>-0.551</td>
<td>-0.508</td>
<td>-0.276</td>
<td>-0.266</td>
<td>-0.374</td>
<td>-0.341</td>
<td>-0.288</td>
<td>-0.245</td>
</tr>
<tr>
<td>T-Statistic</td>
<td>---</td>
<td>3.590</td>
<td>3.281</td>
<td>1.587</td>
<td>1.530</td>
<td>2.229</td>
<td>2.031</td>
<td>1.653</td>
<td>1.413</td>
</tr>
<tr>
<td>(Significance of Change)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

5.4.1.1.2 Segment 2: Stations 42 - 50

Figure 5-51 shows the average lane-change crash risk of each treatment for each location in Segment 2. In this segment, Treatments 01-08 are able to significantly reduce the rear-end crash risk. Table 5-31 shows the t-statistics and ORCI value for each treatment in segment 2. All of the t-values are significant at alpha = 0.05, which is very good.
Figure 5-51  Average Lane-Change Crash Risk v. Location for Treatments 01-08 for Segment 2

Table 5-31  2nd Segment: Summary of Difference in Lane-Change Crash Risk for Treatments 01-08

<table>
<thead>
<tr>
<th>Test Case ID</th>
<th>Base</th>
<th>T01</th>
<th>T02</th>
<th>T03</th>
<th>T04</th>
<th>T05</th>
<th>T06</th>
<th>T07</th>
<th>T08</th>
</tr>
</thead>
<tbody>
<tr>
<td>Average Crash Risk (Stations 42 to 50)</td>
<td>0.897</td>
<td>0.637</td>
<td>0.640</td>
<td>0.642</td>
<td>0.645</td>
<td>0.637</td>
<td>0.633</td>
<td>0.646</td>
<td>0.637</td>
</tr>
<tr>
<td>Crash Risk Benefit</td>
<td>---</td>
<td>0.260</td>
<td>0.257</td>
<td>0.255</td>
<td>0.252</td>
<td>0.260</td>
<td>0.264</td>
<td>0.251</td>
<td>0.260</td>
</tr>
<tr>
<td>T-Statistic (Significance of Change)</td>
<td>---</td>
<td>4.934</td>
<td>5.020</td>
<td>4.806</td>
<td>4.776</td>
<td>4.885</td>
<td>4.901</td>
<td>4.818</td>
<td>5.050</td>
</tr>
<tr>
<td>ORCI</td>
<td>---</td>
<td>4.68</td>
<td>4.63</td>
<td>4.58</td>
<td>4.54</td>
<td>4.68</td>
<td>4.75</td>
<td>4.53</td>
<td>4.68</td>
</tr>
</tbody>
</table>
5.4.1.1.3 Summary of Affected Sections

The effects of Treatments 01-08 on lane-change crash risk were summarized in the previous two sections. In the first affected segment, Treatments 01, 02, 05, 06 and 07 were found to significantly increase the lane-change crash risk, making the network a more dangerous place. The remaining treatments had similar effects and highly negative ORCI values.

The second segment, however, showed promising results. All of the treatments were able to significantly reduce the lane-change crash risk from stations 42-50. The length of segment 2, however, was not long enough to compensate for the high-risk segment 1. Table 5-32 shows the cumulative ORCI values for Treatments 01-08, which takes into account the negatively and positively affected portions of the network. As seen in the table, these values are highly negative and cannot be recommended for the improvement of safety on the network.

<table>
<thead>
<tr>
<th>Table 5-32 Cumulative Lane-Change ORCI Values for Treatments 01-08</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Cumulative ORCI</strong></td>
</tr>
<tr>
<td>Base</td>
</tr>
<tr>
<td>---</td>
</tr>
<tr>
<td>---</td>
</tr>
</tbody>
</table>

Therefore, based on the significantly negative results in the rear-end and lane-change crash risk analyses, Treatments 01-08 cannot be recommended for use in the 90% loading scenario.

5.4.2 Treatments 09 – 16

The analyses of treatments 09-16 are reviewed in this section. Treatments 09-16 are unique in their implementation strategy of variable speed limits. Each of these treatments, when implemented, lowers the upstream speed limits by 5 mph. Table 5-33 displays the different factors that make up Treatments 01-08, which are reviewed in this section.
<table>
<thead>
<tr>
<th>Treatment ID</th>
<th>Speed Change Implementation</th>
<th>Speed Zone Threshold</th>
<th>Speed Change Distance</th>
<th>Speed Change Time Period</th>
</tr>
</thead>
<tbody>
<tr>
<td>9</td>
<td>-5</td>
<td>5 mph</td>
<td>Speed Zone</td>
<td>5 min</td>
</tr>
<tr>
<td>10</td>
<td>-5</td>
<td>5 mph</td>
<td>Speed Zone</td>
<td>10 min</td>
</tr>
<tr>
<td>11</td>
<td>-5</td>
<td>5 mph</td>
<td>1/2 Speed Zone</td>
<td>5 min</td>
</tr>
<tr>
<td>12</td>
<td>-5</td>
<td>5 mph</td>
<td>1/2 Speed Zone</td>
<td>10 min</td>
</tr>
<tr>
<td>13</td>
<td>-5</td>
<td>2.5 mph</td>
<td>Speed Zone</td>
<td>5 min</td>
</tr>
<tr>
<td>14</td>
<td>-5</td>
<td>2.5 mph</td>
<td>Speed Zone</td>
<td>10 min</td>
</tr>
<tr>
<td>15</td>
<td>-5</td>
<td>2.5 mph</td>
<td>1/2 Speed Zone</td>
<td>5 min</td>
</tr>
<tr>
<td>16</td>
<td>-5</td>
<td>2.5 mph</td>
<td>1/2 Speed Zone</td>
<td>10 min</td>
</tr>
</tbody>
</table>

5.4.2.1 Rear-End Crash Risk

The rear-end crash risk was analyzed for Treatments 09-16. Figure 5-52 shows the average rear-end crash risk v. location for the whole network. Most locations are not substantially affected by Treatments 09-16. The most affected segments occur between stations 13 and 34, and between 36 and 50.
5.4.2.1.1 Segment 1: Stations 13 – 34

The first segment occurs between stations 13 and 34 and is negatively affected by the application of Treatments 09-16. Figure 5-53 shows the average rear-end crash risk for each treatment over the length of segment 1. As shown, each treatment actually increases the rear-end crash risk over this segment. Table 5-34 summarizes the t-tests performed for each treatment against the BASE case. As shown, none of these changes are significant.
Figure 5-53  Average Rear-End Crash Risk v. Location for Treatments 09-16 for Segment 1

Table 5-34  1st Segment: Summary of Difference in Rear-End Crash Risk for Treatments 09-16

<table>
<thead>
<tr>
<th>Test Case ID</th>
<th>Average Crash Risk (Stations 13 to 34)</th>
<th>Crash Risk Benefit</th>
<th>T-Statistic (Significance of Change)</th>
<th>ORCI</th>
</tr>
</thead>
<tbody>
<tr>
<td>Test Case ID</td>
<td>Base</td>
<td>T09</td>
<td>T10</td>
<td>T11</td>
</tr>
<tr>
<td>--------------</td>
<td>-------------</td>
<td>--------</td>
<td>--------</td>
<td>--------</td>
</tr>
<tr>
<td>Average Crash Risk (Stations 13 to 34)</td>
<td>-0.201</td>
<td>0.034</td>
<td>0.020</td>
<td>-0.101</td>
</tr>
<tr>
<td>Crash Risk Benefit</td>
<td>---</td>
<td>-0.236</td>
<td>-0.221</td>
<td>-0.100</td>
</tr>
<tr>
<td>T-Statistic (Significance of Change)</td>
<td>---</td>
<td>1.632</td>
<td>1.523</td>
<td>0.673</td>
</tr>
</tbody>
</table>
5.4.2.1.2 Segment 2: Stations 36 – 50

The second segment occurs between stations 36 and 50 and is positively affected by the application of Treatments 09-16. Figure 5-54 shows the average rear-end crash risk for each treatment over the length of segment 2. Table 5-35 summarizes the t-tests performed for each treatment against the BASE case. As shown, each treatment actually decreases the rear-end crash risk over this segment; however, none of these changes were found to be significant.

![Average Rear-End Crash Risk v. Location](image)

Figure 5-54 Average Rear-End Crash Risk v. Location for Treatments 09-16 for Segment 2
Table 5-35  2nd Segment: Summary of Difference in Rear-End Crash Risk for Treatments 09-16

<table>
<thead>
<tr>
<th>Test Case ID</th>
<th>Base</th>
<th>T09</th>
<th>T10</th>
<th>T11</th>
<th>T12</th>
<th>T13</th>
<th>T14</th>
<th>T15</th>
<th>T16</th>
</tr>
</thead>
<tbody>
<tr>
<td>Average Crash Risk (Stations 36 to 50)</td>
<td>1.131</td>
<td>1.112</td>
<td>1.104</td>
<td>1.104</td>
<td>1.109</td>
<td>1.124</td>
<td>1.124</td>
<td>1.113</td>
<td></td>
</tr>
<tr>
<td>Crash Risk Benefit</td>
<td>---</td>
<td>0.019</td>
<td>0.027</td>
<td>0.027</td>
<td>0.021</td>
<td>0.022</td>
<td>0.007</td>
<td>0.008</td>
<td>0.018</td>
</tr>
<tr>
<td>T-Statistic (Significance of Change)</td>
<td>---</td>
<td>0.089</td>
<td>0.129</td>
<td>0.130</td>
<td>0.099</td>
<td>0.104</td>
<td>0.035</td>
<td>0.036</td>
<td>0.085</td>
</tr>
<tr>
<td>ORCI</td>
<td>---</td>
<td>0.525</td>
<td>0.750</td>
<td>0.759</td>
<td>0.586</td>
<td>0.605</td>
<td>0.205</td>
<td>0.212</td>
<td>0.506</td>
</tr>
</tbody>
</table>

5.4.2.1.3 Summary of Affected Sections

Based on the analyses of the two affected sections, no treatment was found to significantly reduce or increase the rear-end crash risk. Table 5-36 shows the cumulative rear-end ORCI values for each treatment, taking into account the negative effects in segment 1 and the positive effects in segment 2. As shown, all of the ORCI values are negative, which represents a harmful effect on the safety.

Table 5-36 Cumulative Rear-End ORCI Values for Treatments 09-16

<table>
<thead>
<tr>
<th>Test Case ID</th>
<th>Base</th>
<th>T09</th>
<th>T10</th>
<th>T11</th>
<th>T12</th>
<th>T13</th>
<th>T14</th>
<th>T15</th>
<th>T16</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cumulative ORCI</td>
<td>---</td>
<td>-9.84</td>
<td>-8.96</td>
<td>-3.63</td>
<td>-2.96</td>
<td>-5.58</td>
<td>-5.66</td>
<td>-2.30</td>
<td>-1.41</td>
</tr>
</tbody>
</table>

5.4.2.2 Lane-Change Crash Risk

The lane-change crash risk was analyzed for Treatments 09-16. Figure 5-55 shows the average lane-change crash risk v. location for the whole network. There are two affected segments in the network, occurring between stations 11 and 32, and between 33 and 42.
5.4.2.2.1 Segment 1: Stations 11 – 32

The first segment occurs between stations 11 and 32 and is negatively affected by the application of Treatments 09-16. Figure 5-56 shows the average lane-change crash risk for each treatment over the length of segment 1. As shown, each treatment actually increases the lane-change crash risk over this segment. Table 5-37 summarizes the t-tests performed for each treatment against the BASE case. From the table, Treatments 09, 10, 13 and 14 were found to significantly increase the crash risk in this first segment. The other treatments, however, which involve the application of speed limits over only half the speed zones, do not significantly increase the lane-change crash risk.
Figure 5-56  Average Lane-Change Crash Risk v. Location for Treatments 09-16 for Segment 1

Table 5-37  1st Segment: Summary of Difference in Lane-Change Crash Risk for Treatments 09-16

<table>
<thead>
<tr>
<th>Test Case ID</th>
<th>Base</th>
<th>T09</th>
<th>T10</th>
<th>T11</th>
<th>T12</th>
<th>T13</th>
<th>T14</th>
<th>T15</th>
<th>T16</th>
</tr>
</thead>
<tbody>
<tr>
<td>Average Crash Risk (Stations 11 to 32)</td>
<td>-0.024</td>
<td>0.326</td>
<td>0.306</td>
<td>0.118</td>
<td>0.109</td>
<td>0.189</td>
<td>0.180</td>
<td>0.079</td>
<td>0.063</td>
</tr>
<tr>
<td>Crash Risk Benefit</td>
<td>---</td>
<td>-0.351</td>
<td>-0.331</td>
<td>-0.142</td>
<td>-0.133</td>
<td>-0.214</td>
<td>-0.204</td>
<td>-0.103</td>
<td>-0.088</td>
</tr>
<tr>
<td>T-Statistic (Significance of Change)</td>
<td>---</td>
<td>2.999</td>
<td>2.805</td>
<td>1.151</td>
<td>1.086</td>
<td>1.782</td>
<td>1.684</td>
<td>0.831</td>
<td>0.708</td>
</tr>
</tbody>
</table>
5.4.2.2.2 Segment 2: Stations 33 – 42

The second segment occurs between stations 33 and 42 and is positively affected by the application of Treatments 09-16. Figure 5-57 shows the average lane-change crash risk for each treatment over the length of segment 2. As shown, each treatment actually increases the lane-change crash risk over this segment. Table 5-38 summarizes the t-tests performed for each treatment against the BASE case. From the table, none of the treatments were found to reduce the crash risk significantly.

![Average Lane-Change Crash Risk v. Location](image)

Figure 5-57 Average Lane-Change Crash Risk v. Location for Treatments 09-16 for Segment 2
Table 5-38 2nd Segment: Summary of Difference in Lane-Change Crash Risk for Treatments 09-16

<table>
<thead>
<tr>
<th>Test Case ID</th>
<th>Base</th>
<th>T09</th>
<th>T10</th>
<th>T11</th>
<th>T12</th>
<th>T13</th>
<th>T14</th>
<th>T15</th>
<th>T16</th>
</tr>
</thead>
<tbody>
<tr>
<td>Average Crash Risk (Stations 33 to 42)</td>
<td>1.182</td>
<td>1.146</td>
<td>1.146</td>
<td>1.159</td>
<td>1.147</td>
<td>1.138</td>
<td>1.162</td>
<td>1.164</td>
<td>1.161</td>
</tr>
<tr>
<td>Crash Risk Benefit</td>
<td>---</td>
<td>0.036</td>
<td>0.037</td>
<td>0.023</td>
<td>0.035</td>
<td>0.044</td>
<td>0.02</td>
<td>0.018</td>
<td>0.021</td>
</tr>
<tr>
<td>T-Statistic (Significance of Change)</td>
<td>---</td>
<td>0.602</td>
<td>0.608</td>
<td>0.39</td>
<td>0.562</td>
<td>0.75</td>
<td>0.326</td>
<td>0.299</td>
<td>0.345</td>
</tr>
<tr>
<td>ORCI</td>
<td>---</td>
<td>0.653</td>
<td>0.658</td>
<td>0.422</td>
<td>0.627</td>
<td>0.793</td>
<td>0.361</td>
<td>0.329</td>
<td>0.374</td>
</tr>
</tbody>
</table>

5.4.2.2.3 Summary of Affected Sections

Based on the analyses of the two affected sections, no treatment was found to significantly reduce the lane-change crash risk. Table 5-39 shows the cumulative lane-change ORCI values for each treatment, taking into account the negative effects in segment 1 and the positive effects in segment 2. As shown, all of the ORCI values are negative, which represents a harmful effect on the safety.

Table 5-39 Cumulative Lane-Change ORCI Values for Treatments 09-16

<table>
<thead>
<tr>
<th>Test Case ID</th>
<th>Base</th>
<th>T09</th>
<th>T10</th>
<th>T11</th>
<th>T12</th>
<th>T13</th>
<th>T14</th>
<th>T15</th>
<th>T16</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cumulative ORCI</td>
<td>---</td>
<td>-14.78</td>
<td>-13.89</td>
<td>-5.84</td>
<td>-5.24</td>
<td>-8.61</td>
<td>-8.62</td>
<td>-4.22</td>
<td>-3.49</td>
</tr>
</tbody>
</table>

Treatments 09, 10, 13 and 14 were found to significantly increase the lane-change crash risk over the section of stations 11-32. No treatments were found to have a significant reduction in the rear-end or lane-change crash risk. Therefore, Treatments 09-16 cannot be recommended for application in the 90% loading scenario.
5.4.3 Treatments 17 – 24

Treatments 17-24 are analyzed in this section. Treatments 17-24 are unique in their VSL implementation strategy, in that the speed limits are decreased by 5 mph upstream and increased by 5 mph downstream for all treatments. Table 5-40 displays the different factors that make up Treatments 17-24, which are reviewed in this section.

Table 5-40 Description of Treatments 17-24

<table>
<thead>
<tr>
<th>Treatment ID</th>
<th>Speed Change Implementation</th>
<th>Speed Zone Threshold</th>
<th>Speed Change Distance</th>
<th>Speed Change Time Period</th>
</tr>
</thead>
<tbody>
<tr>
<td>17</td>
<td>-5/+5</td>
<td>5 mph</td>
<td>Speed Zone</td>
<td>5 min</td>
</tr>
<tr>
<td>18</td>
<td>-5/+5</td>
<td>5 mph</td>
<td>Speed Zone</td>
<td>10 min</td>
</tr>
<tr>
<td>19</td>
<td>-5/+5</td>
<td>5 mph</td>
<td>1/2 Speed Zone</td>
<td>5 min</td>
</tr>
<tr>
<td>20</td>
<td>-5/+5</td>
<td>5 mph</td>
<td>1/2 Speed Zone</td>
<td>10 min</td>
</tr>
<tr>
<td>21</td>
<td>-5/+5</td>
<td>2.5 mph</td>
<td>Speed Zone</td>
<td>5 min</td>
</tr>
<tr>
<td>22</td>
<td>-5/+5</td>
<td>2.5 mph</td>
<td>Speed Zone</td>
<td>10 min</td>
</tr>
<tr>
<td>23</td>
<td>-5/+5</td>
<td>2.5 mph</td>
<td>1/2 Speed Zone</td>
<td>5 min</td>
</tr>
<tr>
<td>24</td>
<td>-5/+5</td>
<td>2.5 mph</td>
<td>1/2 Speed Zone</td>
<td>10 min</td>
</tr>
</tbody>
</table>

5.4.3.1 Rear-End Crash Risk

The effects that Treatments 17-24 have on rear-end crash risk are reviewed in this section. Figures 5-58 and 5-59 show the average rear-end crash risk for every location for the BASE case and Treatments 17-20 and 21-24, respectively. Like the 80% loading scenario, the effects of these treatments influence 2 segments separately. Segment 1 is from stations 13-32, and Segment 2 is from stations 37-52.
Figure 5-58  Average Rear-End Crash Risk v. Location for Treatments 17-20
5.4.3.1.1 Segment 1: Stations 13 – 32

Across this 1st segment, the major differences in rear-end crash risk are negative. That is, the implementation of these treatments increases the crash risk at this segment. Figures 5-60 and 5-61 magnify stations 13-32 for Treatments 17-20 and 21-24, respectively. The treatments are broken up for easier distinction on the graphs. In this section, there is a more substantial increase in rear-end crash risk from Treatments 17, 18, 21 and 22 than for Treatments 19, 20, 23 and 24.
Table 5-41 summarizes the t-tests performed for each treatment against the BASE case. As shown from the table, Treatments 17 and 18 were found to have significantly negative effects on the rear-end crash risk in this segment. The remaining treatments were not found to be significant, but all ORCI values are negative. The treatments implementing over only half the speed zones (Treatments 19, 20, 23 and 24) are negatively affected the least, as shown by the ORCI values.
Figure 5-61  Average Rear-End Crash Risk v. Location for Treatments 21-24 for Segment 1

Table 5-41  1st Segment: Summary of Difference in Rear-End Crash Risk for Treatments 17-24

<table>
<thead>
<tr>
<th>Test Case ID</th>
<th>T17</th>
<th>T18</th>
<th>T19</th>
<th>T20</th>
<th>T21</th>
<th>T22</th>
<th>T23</th>
<th>T24</th>
</tr>
</thead>
<tbody>
<tr>
<td>Average Crash Risk (Stations 13 to 32)</td>
<td>-0.372</td>
<td>-0.158</td>
<td>-0.194</td>
<td>-0.323</td>
<td>-0.318</td>
<td>-0.256</td>
<td>-0.254</td>
<td>-0.336</td>
</tr>
<tr>
<td>Crash Risk Benefit</td>
<td>---</td>
<td>-0.214</td>
<td>-0.178</td>
<td>-0.050</td>
<td>-0.054</td>
<td>-0.116</td>
<td>-0.119</td>
<td>-0.036</td>
</tr>
<tr>
<td>T-Statistic (Significance of Change)</td>
<td>---</td>
<td>2.284</td>
<td>1.912</td>
<td>0.516</td>
<td>0.560</td>
<td>1.237</td>
<td>1.255</td>
<td>0.378</td>
</tr>
<tr>
<td>ORCI</td>
<td>---</td>
<td>-8.559</td>
<td>-7.131</td>
<td>-1.993</td>
<td>-2.157</td>
<td>-4.654</td>
<td>-4.752</td>
<td>-1.449</td>
</tr>
</tbody>
</table>
5.4.3.1.2 Segment 2: Stations 37 – 52

Across the 2nd segment, the differences in rear-end crash risk are positive. Figures 5-62 and 5-63 magnify stations 37-52 for Treatments 17-20 and 21-24, respectively. The treatments are broken up for easier distinction on the graphs. The changes in rear-end crash risk due to these treatments in this section are minimal.

![Average Rear-End Crash Risk v Location](image)

Figure 5-62 Average Rear-End Crash Risk v. Location for Treatments 17-20 for Segment 2

Table 5-42 summarizes the t-tests performed for each treatment against the BASE case. As shown from the table, all of the ORCI values are positive, but none of the changes were found to be significant.
Figure 5-63  Average Rear-End Crash Risk v. Location for Treatments 21-24 for Segment 2

Table 5-42  2nd Segment: Summary of Difference in Rear-End Crash Risk for Treatments 17-24

<table>
<thead>
<tr>
<th>Test Case ID</th>
<th>Base</th>
<th>T17</th>
<th>T18</th>
<th>T19</th>
<th>T20</th>
<th>T21</th>
<th>T22</th>
<th>T23</th>
<th>T24</th>
</tr>
</thead>
<tbody>
<tr>
<td>Average Crash Risk (Stations 37 to 52)</td>
<td>0.993</td>
<td>0.978</td>
<td>0.970</td>
<td>0.964</td>
<td>0.981</td>
<td>0.973</td>
<td>0.975</td>
<td>0.978</td>
<td>0.987</td>
</tr>
<tr>
<td>Crash Risk Benefit</td>
<td>---</td>
<td>0.015</td>
<td>0.022</td>
<td>0.029</td>
<td>0.011</td>
<td>0.020</td>
<td>0.018</td>
<td>0.015</td>
<td>0.006</td>
</tr>
<tr>
<td>T-Statistic (Significance of Change)</td>
<td>---</td>
<td>0.068</td>
<td>0.104</td>
<td>0.132</td>
<td>0.053</td>
<td>0.093</td>
<td>0.081</td>
<td>0.067</td>
<td>0.029</td>
</tr>
<tr>
<td>ORCI</td>
<td>---</td>
<td>0.445</td>
<td>0.673</td>
<td>0.859</td>
<td>0.344</td>
<td>0.603</td>
<td>0.529</td>
<td>0.439</td>
<td>0.186</td>
</tr>
</tbody>
</table>
5.4.3.1.3 Summary of Affected Sections

Based on the analyses of the two affected sections, no treatment was found to significantly reduce the rear-end crash risk. Treatments 17 and 18, however, were found to significantly increase the rear-end crash risk between stations 13 and 32. Table 5-43 shows the cumulative rear-end ORCI values for each treatment, taking into account the negative effects in segment 1 and the positive effects in segment 2. As shown, all of the ORCI values are negative, which represents a harmful effect on the safety.

Table 5-43 Cumulative Rear-End ORCI Values for Treatments 17-24

<table>
<thead>
<tr>
<th>Cumulative ORCI</th>
<th>Base</th>
<th>T17</th>
<th>T18</th>
<th>T19</th>
<th>T20</th>
<th>T21</th>
<th>T22</th>
<th>T23</th>
<th>T24</th>
</tr>
</thead>
<tbody>
<tr>
<td>---</td>
<td>---</td>
<td>-8.11</td>
<td>-6.46</td>
<td>-1.13</td>
<td>-1.81</td>
<td>-4.05</td>
<td>-4.22</td>
<td>-1.01</td>
<td>-0.61</td>
</tr>
</tbody>
</table>

5.4.3.2 Lane-Change Crash Risk

The lane-change crash risk is next to be analyzed for the implementation of VSL Treatments 17-24. Figures 5-64 and 5-65 show the comparison of lane-change crash risk for the BASE case with Treatments 17-20 and 21-24, respectively. A large section is affected between stations 10 and 30, with the remainder of the network showing little change.
Figure 5-64  Average Lane-Change Crash Risk v. Location for Treatments 17-20
Figure 5-65  Average Lane-Change Crash Risk v. Location for Treatments 21-24
Figure 5-66  Average Lane-Change Crash Risk v. Location for Treatments 17-20 - Magnified
Figures 5-66 and 5-67 magnify the affected section of stations 10 to 30 for Treatments 17-20 and 21-24, respectively. As shown from the figures and from the ORCI values from Table 5-44, every treatment has an overall, negative effect on the lane-change crash risk. Treatments 17 and 18 have significantly negative effects on the lane-change crash risk, as in the rear-end crash risk analysis. Interestingly, however, Treatments 19, 20, 23 and 24 showed positive results between stations 14 and 20, and t-tests confirmed their significance. However, since the ORCI for stations 10 to 30 is negative for these treatments, there is more harm done than good.
Based on the rear-end and lane-change crash risk analyses, it was found that Treatments 17 and 18 have significantly negative effects in both cases. The remaining treatments have negative ORCI values for rear-end and lane-change crash risks. Therefore, since there were not found any VSL treatments that can significantly reduce the rear-end or lane-change crash risk on the network, no treatment can be recommended for use in the 90% loading scenario. This confirms the conclusions of Dilmore (2005), that VSL is not effective in congested situations. Other ITS strategies must be used to reduce crash risk in congested situations, such as ramp metering. Dhindsa (2006) found that VSL in conjunction with ramp metering could help to reduce crash risk in congested situations.

5.5 Discussion of Results

The analysis has been carried out for every treatment at every loading scenario, both for rear-end and lane-change crash risks. This section will serve for the review of significant output and the interpretation of the results.
5.5.1 Effect of Various Strategies on Crash Risk Factors

The different strategies of implementing variable speed limits give much different results in the crash risks, as seen especially in the 60% and 80% loading scenarios. The first 16 treatments deal exclusively with lowering the upstream speed limit, by 10 or 5 mph. The last 8 treatments involve the lowering of upstream speed limits and the raising of downstream speed limits, by 5 mph. In the 60 and 80% loading scenarios, the first 16 treatments have almost exclusively negative results, while the last 8 treatments have minimal negative segments coupled with more positive segments. It has been mentioned earlier that the latter treatments are more resistant against crash migration, and this is discussed in depth in Section 5.5.2.

It is of interest, however, to analyze exactly how the characteristics of traffic flow are being affected in order to make the asserted change in crash risks. A pair of treatments from the 80% loading scenario were chosen for closer examination. Treatments 02 and 19 were compared in the 80% loading scenario, due to their different effects on crash risk. Treatment 02 has the most detrimental effect on crash risk, and Treatment 19 was found to be the most beneficial in the 80% loading condition.

5.5.1.1 Rear-End Crash Risk Factors

It was mentioned earlier that the factors AOF2 (average occupancy at station F for time period 2), ASF2 (average speed at station F for time period 2) and ASG2 (average speed at station G at time period 2) are the most important factors influencing rear-end crash risk. The segments over which these treatments were most effective were compared with the BASE, and the changes in these three variables were noted. The factors affecting rear-end crash risk, as well as their percentage difference from the BASE case, are noted in Table 5-45.
Table 5-45  Percentage Change in Rear-End Crash Risk Factors from the BASE Case

<table>
<thead>
<tr>
<th>Treatment</th>
<th>AOF2</th>
<th>ASF2</th>
<th>ASG2</th>
</tr>
</thead>
<tbody>
<tr>
<td>02</td>
<td>6.66</td>
<td>-3.71</td>
<td>-3.71</td>
</tr>
<tr>
<td>19</td>
<td>-0.98</td>
<td>2.23</td>
<td>2.03</td>
</tr>
</tbody>
</table>

Treatment 02 was compared with Treatment 19. In this situation, Treatment 19 actually shows improvement over the same area in which Treatment 02 increases the rear-end crash risk. As shown, Treatment 02 increases the occupancy (AOF2) and decreases the average speeds (ASF2, ASG2). Treatment 19, however, actually reduces the occupancy by about 1% while increasing the speeds by more than 2%. Here we see a stark contrast between two treatments and their effects on the traffic flow conditions.

The regime probabilities were also analyzed for Treatments 02 and 19 and compared with the regime probabilities for the BASE case. It was found that, over the whole network, Treatment 19 carried the same regime 1 and regime 2 probabilities as the BASE case, which has an average regime 1 probability of 5.4%. On the other hand, for Treatment 02, the regime 1 probability increased from 5.4% to 10.4%. This is nearly a 100% increase in the regime 1 probability, and, as mentioned earlier, regime 1 is the most dangerous situation for rear-end accidents. Therefore, strategies like Treatment 02 can be said to increase the probability of regime 1 traffic flow, as well as increase the occupancy at the station of interest and decrease the average speeds. All of this serves to increase the rear-end crash risk on the network. Strategies like Treatment 19, however, do not show any affect on the regime 1 probability and are shown to decrease the occupancy at the station of interest while increasing the average speeds. These effects serve to decrease the rear-end crash risk on the network.
5.5.1.2 Lane-Change Crash Risk Factors

The lane-change crash risk depends on 6 factors that change with the traffic flow conditions. They include ASW2 (average speed downstream at time period 2), ASU2 (average speed upstream at time period 2), AOW2 (average occupancy downstream at time period 2), ADALOU2 (measure of the difference in occupancy of adjacent lanes), SVW2 (standard deviation of the volume downstream at time period 2), and SSW2 (standard deviation of the speed downstream at time period 2). These factors were considered for the areas over which Treatments 02 and 19 had their negative and positive effects on lane-change crash risk, respectively. The percentage differences in the important factors, compared with the BASE case, are shown for each treatment in Table 5-46.

Table 5-46 Percentage Change in Lane-Change Crash Risk Factors from the BASE Case

<table>
<thead>
<tr>
<th>Treatment</th>
<th>ASW2</th>
<th>ASU2</th>
<th>AOW2</th>
<th>ADALOU2</th>
</tr>
</thead>
<tbody>
<tr>
<td>02</td>
<td>-3.64</td>
<td>-3.59</td>
<td>5.10</td>
<td>2.44</td>
</tr>
<tr>
<td>19</td>
<td>1.95</td>
<td>2.31</td>
<td>-1.11</td>
<td>-5.25</td>
</tr>
</tbody>
</table>

The factors ASW2, ASU2, AOW2 and ADALOU2 were found to have the greatest differences. These factors represent the average speeds downstream and upstream, the average occupancy downstream and the measure representing the relative lane occupancies, respectively, for the time period 2. In Treatment 02, which negatively affected the lane-change crash risk, the average speeds were reduced, the average occupancy downstream was increased, and the difference in occupancy across the different lanes was also increased. In Treatment 19, however, where a significant reduction of lane-change crash risk was observed, the average speeds were increased, the average occupancy was decreased, and the difference in lane occupancies were decreased. It can be said, then, that strategies like Treatment 02 serve to decrease the average
speeds and increase the occupancies, as well as the difference in occupancies across lanes, which effectively increases the lane-change crash risk. Strategies like Treatment 19, however, serve to increase average speeds and decrease average occupancies, as well as the difference in lane occupancies. This serves to decrease the lane-change crash risk. This extra analysis has provided a logical explanation of why the VSL strategies tested had their respective effects on rear-end and lane-change crash risks.

5.5.2 Effect of Various Strategies on Crash Migration

In the 60% loading scenario, it was observed that Treatments 17 and 18 alone could produce a significant reduction in the rear-end crash risk. These same treatments, along with Treatments 19, 21 and 22, also produced a significant reduction in the lane-change crash risk. Treatments 17 and 18 lowered the crash risk in real time throughout the simulation, and they were found to reduce the total travel time by 0.4 and 0.8 percent, respectively. This was found to be a significant travel time reduction in the case of Treatment 18.

In an explanation of why these treatments work so much better than the other treatments, it is important to look at the trends of how each treatment affected the crash risk on the network. Figures 5-68 and 5-69 show the trends that each treatment had on the rear-end and lane-change crash risk, respectively, over the network. These figures represent the average change in rear-end and lane-change crash risk, respectively, compared to the BASE case, over the entire course of the simulation. The medium (grey) shade represents no change between the given treatment and the BASE case. The light shade represents a positive change, and the dark shade represents a negative change, or a detrimental effect on crash risk. Any sections that are bordered by a bold box were found to be significantly different from the BASE. From these figures, it is apparent
that crash risk deterioration is present in Treatments 01-16. Treatments 17-24, however, are shown to resist the effects of crash migration. These treatments lower the speed limit upstream and raise the speed limit downstream of the stations found to have risk. It is these treatments that show the greatest benefit and the least detriment to the rear-end and lane-change crash risk. These could be attributed to the capability that this specific VSL strategy has to clearing a congested area and allowing for the dissipation of a queue. It seems to work well in the 60% loading scenario, where the average speed at a location is mostly dependent upon the speed limit.
Figure 5-68 Trend of Affected Locations for Rear-End Crash Risk at 60% Loading
| T01 | T02 | T03 | T04 | T05 | T06 | T07 | T08 | T09 | T10 | T11 | T12 | T13 | T14 | T15 | T16 | T17 | T18 | T19 | T20 | T21 | T22 | T23 | T24 |
|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|
| 10  | E 0 |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |
| 10  | E 1 |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |
| 11  | E 0 |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |
| 11  | E 1 |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |
| 12  | E 0 |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |
| 12  | E 1 |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |
| 13  | E 0 |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |
| 13  | E 1 |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |
| 14  | E 0 |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |
| 14  | E 1 |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |

Figure 5-69 Trend of Affected Locations for Lane-Change Crash Risk at 60% Loading
In the 80% loading scenario, Treatment 19 was found to have the greatest benefit in the rear-end crash risk analysis and was also found to significantly reduce the lane-change crash risk. Treatment 19 was found to increase the total travel time by less than 0.4 percent, as well, which is acceptable. Figures 5-70 and 5-71 show the trends that each treatment had on the rear-end and lane-change crash risk, respectively. These figures show that crash risk migration and crash risk deterioration is more present in the 80% loading scenario than the 60% loading scenario. When comparing Figures 5-70 and 5-71 to Figures 5-68 and 5-69, respectively, the negative effect of crash migration and crash risk deterioration is more prevalent in the upstream areas of the network. This eludes to the supposition that crash migration and crash risk deterioration increases with increasing congestion.

As in the 60% loading scenario, Treatments 17-24 are shown to resist crash migration to some degree. Treatments 19, 20, 23 and 24, which only implement over half the distance of speed zones, appear to resist crash migration extremely well. This is actually the primary reason that Treatment 19 is able to outperform Treatments 17 and 18, which were the best cases from the 60% scenario. As seen from the figures, Treatment 19 experiences much less harm to the crash risk upstream of beneficial areas than Treatments 17 and 18. In the lane-change crash risk scenario, shown in Figure 5-71, Treatment 19’s resistance to crash migration allows it to outperform Treatments 17 and 18, even though they experience two segments of significant benefit. It is believed that this resistance to crash risk migration is due primarily to the stepwise change in speeds (and thus, speed differences) encouraged by implementing over half the speed zone. This is a very interesting trend, and it is also shown to carry through into the 90% loading scenario.
Figure 5-70  Trend of Affected Locations for Rear-End Crash Risk at 80% Loading
Figure 5-71 Trend of Affected Locations for Lane-Change Crash Risk at 80% Loading
In the 90% loading scenario, no treatments were found to have a positive ORCI in the rear-end or lane-change crash risk analysis. Figures 5-72 and 5-73 show the trends that each treatment had on the rear-end and lane-change crash risk, respectively. These figures show that crash risk migration is more present in every treatment in the 90% loading scenario than the 80% loading scenario, confirming the supposition that crash migration increases as congestion increases. This is especially seen in the lane-change crash risk, comparing Figures 5-73 with 5-71.

From Figure 5-72, no treatment was found to have a significant benefit to the rear-end crash risk. As reviewed in Section 5.4, Treatments 01-07 had very high t-values approaching the critical t-statistic, 1.734, for stations 38-43. However, this segment of benefit is overwhelmingly overcome by the large, significantly detrimental segment from stations 10-34. Since the ORCI is negative, the trade-off is not worth it, so the treatments should not be implemented. Likewise, in the lane-change crash risk, shown in Figure 5-73, Treatments 01-08 show significant benefit from stations 42-50, but the significantly detrimental segment from stations 10-34 overcomes the benefit in the ORCI values. Similarly, Treatments 19, 20, 23 and 24 produce a significantly beneficial segment in the midst of increasing lane-change crash risk. This beneficial segment, however, is not enough to overcome the harm done to the crash risk in the surrounding areas. Nevertheless, it is interesting to note the evidence in Figures 5-72 and 5-73 of Treatments 19, 20, 23 and 24 being more resistant to crash migration than any other treatments, as noted in the 80% loading scenario.
Figure 5-72  Trend of Affected Locations for Rear-End Crash Risk at 90% Loading
Figure 5-73  Trend of Affected Locations for Lane-Change Crash Risk at 90% Loading
5.5.3 Summary

Therefore, the general conclusions from the implementation of variable speed limits on Interstate-4 are as follows:

• In the 60% loading condition, Treatments 17 and 18 are the best strategies for reducing rear-end and lane-change crash risk, with Treatment 18 being slightly better than Treatment 17. These treatments involve the strategy of decreasing speed limits upstream by 5 mph and increasing speed limits downstream by 5 mph. The variable speed limits are extended over the entire length of the necessary homogeneous speed zones, which are defined by a difference of 5 mph in average speed. Treatment 18 prescribes a minimum time interval of 10 minutes as opposed to Treatment 17’s time interval of 5 minutes.

• Treatments 17 and 18 are able to reduce the total network travel time by 0.4 and 0.8 percent, respectively. The reduction by Treatment 18 was found to be significant.

• Therefore, Treatment 18 is regarded as the best combination of variables for VSL implementation in the 60% loading condition.

• In the 80% loading condition, Treatment 19 was observed to be the best strategy for reducing rear-end and lane-change crash risk. It was found to significantly reduce the lane-change crash risk and was able to effectively resist the effects of crash migration to obtain the highest rear-end and lane-change ORCI values. Treatment 19 involves the strategy of decreasing speed limits upstream by 5 mph and increasing speed limits downstream by 5 mph. The variable speed limits are
extended over half the length of the necessary homogeneous speed zones, which are defined by a difference of 5 mph in average speed, for a minimum period of 5 min.

- Treatment 19 was found to increase the network travel time by less than 0.4%, which is an acceptable loss for the benefits in safety.

- It was also noticed that, in the 80% loading scenario, crash migration and crash risk deterioration intensities increased, and the implementation of variable speed limits over half the length of speed zones, when decreasing upstream speeds and increasing downstream speeds, were found to be more resistant to the effects of crash migration than other treatments.

- Contrasting the best treatments for the 60% and 80% loading, Treatment 18 actually benefits its positively-effected areas better than Treatment 19 in both scenarios. Since Treatment 18 is applied over the full length of homogeneous speed zones, the areas that are positively affected by variable speed limits are positively affected by Treatment 18 to a greater extent than Treatment 19. It is the best, therefore, in the 60% loading scenario, where neither treatment is subject to crash migration. In the 80% loading scenario, however, though Treatment 18 has greater benefits in its positive segments, it has substantially more negative effects in the crash migration zones. Since Treatment 19 is robust against crash migration in the 80% scenario, its positive benefits coupled with its resistance to crash migration make it better than Treatment 18 overall.

- The 90% results confirmed that the increasing effects of crash migration as congestion increases and that the implementation of variable speed limits over
half the length of speed zones, when decreasing upstream speeds and increasing downstream speeds, are more robust against the effects of crash migration than other treatments.

- In the 90% loading condition, no proposed VSL treatment was found to produce positive crash risk results on the network. This conclusion concurs with Dilmore’s (2005) results.
CHAPTER 6. CONCLUSIONS

This study has examined the potential of variable speed limits to be used as a real-time crash prevention technique. The objectives of this study were to examine multiple strategies for the implementation of variable speed limits and to recommend the best treatments for use on urban freeways in different situations. To that end, a 36.25-mile section of Interstate-4 running though Orlando, FL was simulated using the PARAMICS micro-simulation program. Twenty-four separate, corridor-wide VSL strategies were tested in three different loading conditions (60%, 80% and 90%). These strategies depended on the magnitude of speed limit change, the distance over which they were changed upstream and downstream, and the minimum time period for which they were changed. The primary effect of each treatment was measured by the change in rear-end and lane-change crash risk from the BASE case. A secondary measure of effectiveness for potentially useful treatments included a travel time impact analysis. This was to ensure that the operational capabilities of the highway were not sacrificed in an attempt to improve the measure of safety.

This study found that the implementation of variable speed limits successfully reduces the rear-end and lane-change crash risks at low-volume traffic conditions (60% and 80% loading conditions). In every case, the most successful treatments involved the lowering of upstream speed limits by 5 mph and the raising of downstream speed limits by 5 mph. In the free-flow condition (60% loading), the best two treatments (18, 17) involved the more liberal threshold for homogeneous speed zones (5 mph) and the more liberal implementation distance (the entire speed zone). The best treatment (18) demanded a minimum time period of 10 minutes, while the other (17) only required 5 minutes. Both of these treatments were actually shown to reduce the
network travel time by a fraction of a percent. It was also shown that this particular implementation strategy (lowering upstream, raising downstream) is wholly resistant to the effects of crash migration in the 60% loading scenario.

In the condition approaching congestion (80% loading), the best treatment (19) again involved the more liberal threshold for homogeneous speed zones (5 mph), yet the more conservative implementation distance (half the speed zone), along with a minimum time period of 5 minutes. This particular treatment arises as the best due its unique capability to resist the effects of crash migration in the 80% loading scenario. It was shown that the treatments implementing over half the speed zone (19, 20, 21, 22) were more robust against crash migration than the other treatments. Treatment 19 exemplifies the greatest benefit in reduced sections and the greatest resistance to crash migration in other sections. This treatment was found to increase the network travel time by less than 0.4%, which is deemed acceptable.

Finally, no treatment was found to successfully reduce the rear-end and lane-change crash risks in the congested traffic condition (90% loading). This is attributed to the fact that, in the congested state, the speed of vehicles is subject to the surrounding traffic conditions and not to the posted speed limit. Therefore, changing the posted speed limit does not affect the speed of vehicles in a desirable way. These conclusions agree with Dilmore (2005). It was shown, however, that the effects of crash migration are even more prevalent in the congested situation than in the previous conditions, confirming that the effects of crash migration increase as traffic volume increases. It was also confirmed that the treatments implementing speed limit changes upstream and downstream over half the length of the speed zones, though they were unable to effectively reduce the rear-end and lane-change crash risks, were more resistant against the effects of crash migration than other treatments.
6.1 Recommendations for Further Research

This research has shown multiple strategies in which variable speed limits could benefit the safety of urban freeways. Further research could be done to expand on this study. First of all, the transferability of these recommendations could be tested by repeating the study on another corridor. The rear-end and lane-change crash risk models would have to be recalibrated using loop detector and crash data from the other corridor, and then the new corridor would have to be built using micro-simulation software. These strategies could then be repeated at different volume loading conditions and compared to the conclusions of this study.

Secondly, further analysis could be done on this network by trying different strategies. A strategy involving only the raising of downstream speed limits by 5 mph could be tried. Also, other values for the speed zone threshold and the speed change distance multiplier could be tried. This study used 2.5 mph and 5 mph for the speed zone thresholds and 1.0 and 0.5 for the speed change distance multipliers. Any other values less than the speed difference threshold for variable speed limits (7 mph in this study) could be tried for the homogeneous speed zone threshold. Any other values between 0 and 1.0 could be tried for the speed change distance multiplier.

Further research could also include the logistics of implementing these strategies in the field. This study enabled the network to change the speed limits at will every half mile on the highway (where every detector station exists). This may not be feasible in the field, and further research could be done to analyze where those speeds are changed most often and what locations could be best utilized to obtain similar results.
Another way to expand on this study is to observe the effects of combining the strategies used here with other previously tested ITS strategies. Previous work by Gayah (2006) found that route diversion and ramp metering are able to successfully reduce the crash risk, using the same crash risk models, on the same simulated network. Combining Gayah’s recommendations for route diversion or ramp metering with variable speed limits could prove to be very successful in the reduction of crash risk on Interstate-4 through Orlando.
APPENDIX: API CODE
VSL Homogeneous Speed Zones

#include <stdlib.h>
#include <stdio.h>
#include <string.h>
#include <math.h>
#include "programmer.h"

/* include our function definitions explicit to this example */
#include "plugin_p.h"

#define SPEED_A 1
#define SPEED_B 2
#define SPEED_C 3
#define SPEED_D 4
#define SPEED_E 5
#define SPEED_F 6
#define SPEED_G 7
#define SPEED_H 8
#define SPEED_I 9
#define SPEED_NORM 10

static char **beacons = NULL;
static char **links = NULL;
static char **detectors = NULL;
static int startcount1 = 0;
static int startcount2 = 0;
static int startcount3 = 0;
static int simstarttime = 56700;

int numbeacons;
int numdetectors;
int SpeedZone[70];
int start_SeedZone[70];

static float **g_Speeds1 = NULL;
static float **g_Speeds2 = NULL;
static float **g_Speeds3 = NULL;

static float numspeed1;
static float numspeed2;
static float numspeed3;
static float numspeed;
static float speedtotal1;
static float speedtotal2;
static float speedtotal3;
static float speedtotal;
static float volsum;

float linkFspeed1[90][300];
float linkFspeed2[90][300];
float linkFspeed3[90][300];
float linkF30secspeed1[90][10];
float linkF30secspeed2[90][10];
float linkF30secspeed3[90][10];
float linkF30seccount1[90][10];
float linkF30seccount2[90][10];
float linkF30seccount3[90][10];
float linkF5minavgspeed[90];
float upstreamspeed[115];
float downstreamspeed[115];
float SpeedDiff[115];
float Speedchange_distance[70];

// speedchange_decrease is the number that the speed limit will decrease, in mph, upstream [5 or 10]
// speedchange_increase is the number that the speed limit will increase, in mph, downstream [0 or 5]
// SpeedZone_definition is the difference in speeds, in mph, that will define separate speed zones [5.0 or 2.5]
// distance_multiplier is the amount of speed zone distance that will be used for VSL implementation [1.0 or 0.5]
// speedchange_time is the amount of time, in minutes, that VSL will be implemented before re-evaluation [5 or 10]

const int speedchange_decrease = 5;
const int speedchange_increase = 0;
const float SpeedZone_definition = 2.5;
const float distance_multiplier = 0.5;
const int speedchange_time = 10;

// 80 Percent Loading - Treatment 16
void pp_allocate_memory();
void speedchange(int detector, float speed);

/* 更像是如何将网络中每个节点的信息存储在数组中，以及如何根据不同的速度变化来调整限速。
   另外，还有一个函数用于在模型器中读取完整的网络。*/

/* call qpx_NET_postOpen once when the full network has been read into modeller */
void qpx_NET_postOpen(void)
{
  int i;
  int j;
  int k;
  int x;
  int y;
  int station;

  // gets the number of links and detectors on the network
  numbeacons = qpg_NET_beacons();
  numdetectors = qpg_NET_detectors();

  // 80 Percent Loading - Treatment 16
  void pp_allocate_memory();
  void speedchange(int detector, float speed);
  }
qps_GUI_printf("The number of beacons on the network is %i\n", numbeacons);
qps_GUI_printf("The number of detectors on the network is %i\n", numdetectors);

// define the length of the arrays representing the links and detectors on the network
pp_allocate_memory();

// assign the individual beacon and link pointers to an array
for (i = 0; i < numbeacons; i++)
{
    beacons[i] = qpg_NET_beaconByIndex(i+1);
    links[i] = qpg_BCN_link(beacons[i]);
    // qps_GUI_printf("Link %s has speed limit of %f\n", qpg_LNK_name(links[i]),speedlimit[i]);
}

// assign detectors to an array
for (j = 1; j < numdetectors; j = j + 2)
{
    detectors[j] = qpg_NET_detectorByIndex(j+1);
    // qps_GUI_printf("Link Name %s\n", qpg_LNK_name(qpg_DTC_link(detectors[j])));
}

/* We want to know the name of each link that each eastbound detector is on. Eastbound detector names are even numbered from 4 to 136 */

/* for (x = 2; x < 69; x++)
{
    y = (2 * x) - 3;
    qps_GUI_printf("Eastbound Detector's Link Name %s\n", qpg_LNK_name(links[y]));
} */

}
int t;
int n;
int r;

int timeincrement;
int station;
int index;
int loop_count;
int loop_index;
int loop_lanes;
int speedchange_station;

LINK* linkF;
LOOP* looplane1;
LOOP* looplane2;
LOOP* looplane3;
DETECTOR* detec;

timeincrement = ((int)qpg_CFG_simulationTime() % 300);
station = 0;

if ((int)qpg_CFG_simulationTime() >= 58500)
{
  n = 1;

  for (i = 1; i < 109; i = i + 2)
  {
    station = i + 3;

    /* 'station' gives the ACTUAL detector NUMBER */
    // speedchange(i, 70);

    detec = detectors[i];

    linkF = qpg_DTC_link(detectors[i]);

    looplane1 = qpg_DTC_multipleLoop(detec, 1);
    looplane2 = qpg_DTC_multipleLoop(detec, 2);
    looplane3 = qpg_DTC_multipleLoop(detec, 3);
    loop_count = qpg_LNK_detectors(linkF);

    qps_GUI_printf("Inside Loop\n");
    loop_index = qpg_LNK_detectorIndexByIndex(linkF, 1);
    loop_lanes += qpg_DTI_lanes(loop_index);

    /* Collect the instantaneous speed at the given detector for each lane */
    g_Speeds1[i] = qpg_DTI_speed(loop_index, 1, APILOOP_COMPLETE);
    g_Speeds2[i] = qpg_DTI_speed(loop_index, 2, APILOOP_COMPLETE);
    g_Speeds3[i] = qpg_DTI_speed(loop_index, 3, APILOOP_COMPLETE);
/* Convert the instantaneous speed at the given detector from m/s to mph */

linkFs speed1[station][timeincrement] = g_Speeds1[i]*2.2369;
linkFs speed2[station][timeincrement] = g_Speeds2[i]*2.2369;
linkFs speed3[station][timeincrement] = g_Speeds3[i]*2.2369;

/* qps_GUI_printf("%f ", linkFs speed1[station][timeincrement]);
 qps_GUI_printf("%f ", linkFs speed2[station][timeincrement]);
 qps_GUI_printf("%f ", linkFs speed3[station][timeincrement]); */

/* This operation will run at the end of every 5 minutes */

if (((int)qpg_CFG_simulationTime() % (60*speedchange_time)) == 299)
{
    simstarttime = (int)qpg_CFG_simulationTime();

    volsum = 0;

    /* There are ten (10) 30-sec intervals every 5 minutes
    This operation will go through each 30-sec time interval for the 5-minute period */

    for (x = 0; x <= 9; x++)
    {
        numspeed1 = 0.0;
        numspeed2 = 0.0;
        numspeed3 = 0.0;
        speedtotal1 = 0;
        speedtotal2 = 0;
        speedtotal3 = 0;

        for (t = 30*x; t <= 30*x + 29; t++)
        {
            /* This will add up all UNIQUE speeds from each 30-sec interval to get a 30-sec speed sum */

            if (t != 299)
            {
                if (linkFs speed1[station][t] != linkFs speed1[station][t+1])
                {
                    numspeed1++;
                    speedtotal1 += linkFs speed1[station][t+1];
                }

                if (linkFs speed2[station][t] != linkFs speed2[station][t+1])
                {
                    numspeed2++;
                    speedtotal2 += linkFs speed2[station][t+1];
                }

                if (linkFs speed3[station][t] != linkFs speed3[station][t+1])
                {
                    numspeed3++;
                }
            }
        }
    }
}
speedtotal3 += linkFspeed3[station][t+1];
}
}
}

/* This will give us the total sum of unique speeds divided by the number of observations for every 30-sec. That is, the 30-sec average speeds for each lane */

if (numspeed1 == 0)
{
    linkF30secspeed1[station][x] = 0;
}
else
{
    linkF30secspeed1[station][x] = speedtotal1 / numspeed1;
}

if (numspeed2 == 0)
{
    linkF30secspeed2[station][x] = 0;
}
else
{
    linkF30secspeed2[station][x] = speedtotal2 / numspeed2;
}

if (numspeed3 == 0)
{
    linkF30secspeed3[station][x] = 0;
}
else
{
    linkF30secspeed3[station][x] = speedtotal3 / numspeed3;
}

/*/  qps_GUI_printf("n");
qps_GUI_printf("%f %f \n", linkF30secspeed1[station][x], numspeed1);
qps_GUI_printf("%f %f \n", linkF30secspeed2[station][x], numspeed2);
qps_GUI_printf("%f %f \n", linkF30secspeed3[station][x], numspeed3); */
}

numspeed = 0.0;
speedtotal = 0;

for (t = 0; t <= 9; t++)
{
    /* We sum up the non-zero 30-sec average speeds for each lane, for 5 minutes, and name it 'speedtotal'. */

    if (linkF30secspeed1[station][t] != 0)
{ speedtotal += linkF30secspeed1[station][t];
  numspeed += 1;
}

if (linkF30secspeed2[station][t] != 0)
{
  speedtotal += linkF30secspeed2[station][t];
  numspeed += 1;
}

if (linkF30secspeed3[station][t] != 0)
{
  speedtotal += linkF30secspeed3[station][t];
  numspeed += 1;
}

/* Finally, we divide the 'speedtotal' by the number of non-zero 30-sec average speeds to get the 5-minute average speed */
linkF5minavgspeed[station] = speedtotal / numspeed;

upstreamspeed[station] = linkF5minavgspeed[station - 2];
downstreamspeed[station] = linkF5minavgspeed[station];

/* qps_GUI_printf("\n%f Avg. Speed at Detector %i\n", linkF5minavgspeed[station], station);
qps_GUI_printf("\n%f Upstream speed at Detector %i\n", upstreamspeed[station], station);
qps_GUI_printf("\n%f Downstream speed at Detector %i\n", downstreamspeed[station], station); */

// The Speed Difference is the upstream speed minus the downstream speed
SpeedDiff[i + 3] = upstreamspeed[station] - downstreamspeed[station];

// qps_GUI_printf("\n%f Speed Difference at Detector %i\n", SpeedDiff[i + 3], station);

// SpeedZone[n] defines the length of the nth speed zone.
if (i > 0 && i < 109)
{
  if (abs(SpeedDiff[i + 3]) < SpeedZone_definition)
  {
    SpeedZone[n] = SpeedZone[n] + 2;
    qps_GUI_printf("SpeedZone %i has a distance of %i\n", n, SpeedZone[n]);
  }
  else
  {

n = n + 4;
SpeedZone[n] = 2;
start_SpeedZone[n] = station;
SpeedDiff[n] = SpeedDiff[i + 3];
qps_GUI_printf("SpeedZone %i has a distance of %i and SpeedDiff of %f\n", n, SpeedZone[n], SpeedDiff[n]);
}
}
}
}

if (((int)qpg_CFG_simulationTime() % (60*speedchange_time)) == 299)
{
for (r = 1; r < n + 1; r = r + 4)
{
// RESTORE all speed limits before VSL Implementation

for (speedchange_station = start_SpeedZone[r]; speedchange_station < start_SpeedZone[r] + SpeedZone[r]; speedchange_station = speedchange_station + 2)
{
// qps_GUI_printf("speedchange_station %i restored\n", speedchange_station);

if (speedchange_station < 28)
{
speedchange(speedchange_station - 3, 65);
}
else if (speedchange_station < 66)
{
speedchange(speedchange_station - 3, 55);
}
else if (speedchange_station < 100)
{
speedchange(speedchange_station - 3, 50);
}
else if (speedchange_station < 109)
{
speedchange(speedchange_station - 3, 55);
}
}

for (r = 1; r <= n; r = r + 4)
{
// qps_GUI_printf("mSpeedZone %i begins at Detector %i and has a distance of %i\n", r, start_SpeedZone[r], SpeedZone[r]);

// qps_GUI_printf("Speed Zone %i has SpeedDiff of %f\n", r, SpeedDiff[r]);
Get rid of any outliers

```c
if (SpeedDiff[r] > 45)
{
    SpeedDiff[r] = 0;
}
```

```c
Speedchange_distance[r] = distance_multiplier * SpeedZone[r];
Speedchange_distance[r] = (int)(0.5 * Speedchange_distance[r]) * 2;
if (Speedchange_distance[r] < distance_multiplier * SpeedZone[r])
{
    Speedchange_distance[r] = Speedchange_distance[r] + 2;
}
```

Increase Speeds Downstream

// VSL Implementation

```c
if (SpeedDiff[r] > 7.0)
{
    // Decrease Speeds Upstream
    for (speedchange_station = start_SpeedZone[r] - (int)Speedchange_distance[r - 4]; speedchange_station < start_SpeedZone[r]; speedchange_station = speedchange_station + 2)
    {
        qps_GUI_printf("speedchange_station %i decrease\n", speedchange_station);
    }
    if (speedchange_station < 28)
    {
        speedchange(speedchange_station - 3, 65 - speedchange_decrease);
    }
    else if (speedchange_station < 66)
    {
        speedchange(speedchange_station - 3, 55 - speedchange_decrease);
    }
    else if (speedchange_station < 100)
    {
        speedchange(speedchange_station - 3, 50 - speedchange_decrease);
    }
    else if (speedchange_station < 109)
    {
        speedchange(speedchange_station - 3, 55 - speedchange_decrease);
    }
    // Increase Speeds Downstream
```
for (speedchange_station = start_SpeedZone[r]; speedchange_station < start_SpeedZone[r] + (int)Speedchange_distance[r]; speedchange_station = speedchange_station + 2)
{
    if (r > 5);
    // qps_GUI_printf("speedchange_station %i increase\n", speedchange_station);

    if (speedchange_station < 28)
    {
        speedchange(speedchange_station - 3, 65 + speedchange_increase);
    }

    else if (speedchange_station < 66)
    {
        speedchange(speedchange_station - 3, 55 + speedchange_increase);
    }

    else if (speedchange_station < 100)
    {
        speedchange(speedchange_station - 3, 50 + speedchange_increase);
    }

    else if (speedchange_station < 109)
    {
        speedchange(speedchange_station - 3, 55 + speedchange_increase);
    }

    }

for (r = 1; r <= n; r = r + 4)
{
    // Reset the SpeedZone distances to zero for the next 5-minute analysis
    SpeedZone[r] = 0;
}

void pp_allocate_memory()
{
    int i;
/* for each object clear old memory, and allocate new */

if (beacons != NULL) free(beacons);
beacons = calloc(sizeof(int), numbeacons);

if (links != NULL) free(links);
links = calloc(sizeof(int), numbeacons);

if (detectors != NULL) free(detectors);
detectors = calloc(sizeof(int), numdetectors);

if (g_Speeds1 != NULL) free(g_Speeds1);
g_Speeds1 = calloc(sizeof(int), 70);

if (g_Speeds2 != NULL) free(g_Speeds2);
g_Speeds2 = calloc(sizeof(int), 70);

if (g_Speeds3 != NULL) free(g_Speeds3);
g_Speeds3 = calloc(sizeof(int), 70);

}

void speedchange(int detector, float speed)
{
    switch (detector)
    {
    case 1:
        qps_LNK_speedlimit(links[0], speed);
        qps_LNK_speedlimit(links[1], speed);
        break;
    case 3:
        qps_LNK_speedlimit(links[2], speed);
        qps_LNK_speedlimit(links[3], speed);
        break;
    case 5:
        qps_LNK_speedlimit(links[4], speed);
        qps_LNK_speedlimit(links[5], speed);
        break;
    case 7:
        qps_LNK_speedlimit(links[6], speed);
        qps_LNK_speedlimit(links[7], speed);
        break;
    case 9:
        qps_LNK_speedlimit(links[8], speed);
        break;
    case 11:
        qps_LNK_speedlimit(links[9], speed);
        qps_LNK_speedlimit(links[10], speed);
        break;
    case 13:
        qps_LNK_speedlimit(links[11], speed);
        qps_LNK_speedlimit(links[12], speed);
        qps_LNK_speedlimit(links[13], speed);
        break;
    }
case 15:
    qps_LNK_speedlimit(links[14], speed);
    qps_LNK_speedlimit(links[15], speed);
    qps_LNK_speedlimit(links[16], speed);
    break;
    case 17:
    qps_LNK_speedlimit(links[17], speed);
    qps_LNK_speedlimit(links[18], speed);
    break;
    case 19:
    qps_LNK_speedlimit(links[19], speed);
    qps_LNK_speedlimit(links[20], speed);
    break;
    case 21:
    qps_LNK_speedlimit(links[21], speed);
    break;
    case 23:
    qps_LNK_speedlimit(links[22], speed);
    break;
    case 25:
    qps_LNK_speedlimit(links[23], speed);
    break;
    case 27:
    qps_LNK_speedlimit(links[24], speed);
    qps_LNK_speedlimit(links[25], speed);
    qps_LNK_speedlimit(links[26], speed);
    break;
    case 29:
    qps_LNK_speedlimit(links[27], speed);
    qps_LNK_speedlimit(links[28], speed);
    qps_LNK_speedlimit(links[29], speed);
    break;
    case 31:
    qps_LNK_speedlimit(links[30], speed);
    break;
    case 33:
    qps_LNK_speedlimit(links[31], speed);
    break;
    case 35:
    qps_LNK_speedlimit(links[32], speed);
    break;
    case 37:
    qps_LNK_speedlimit(links[33], speed);
    qps_LNK_speedlimit(links[34], speed);
    break;
    case 39:
    qps_LNK_speedlimit(links[35], speed);
    qps_LNK_speedlimit(links[36], speed);
    qps_LNK_speedlimit(links[37], speed);
    break;
    case 41:
    qps_LNK_speedlimit(links[38], speed);
    qps_LNK_speedlimit(links[39], speed);
    qps_LNK_speedlimit(links[40], speed);
break;
case 43:
qps_LNK_speedlimit(links[41], speed);
qps_LNK_speedlimit(links[42], speed);
qps_LNK_speedlimit(links[43], speed);
break;
case 45:
qps_LNK_speedlimit(links[44], speed);
qps_LNK_speedlimit(links[45], speed);
qps_LNK_speedlimit(links[46], speed);
qps_LNK_speedlimit(links[47], speed);
break;
case 47:
qps_LNK_speedlimit(links[48], speed);
qps_LNK_speedlimit(links[49], speed);
qps_LNK_speedlimit(links[50], speed);
break;
case 49:
qps_LNK_speedlimit(links[51], speed);
qps_LNK_speedlimit(links[52], speed);
break;
case 51:
qps_LNK_speedlimit(links[53], speed);
qps_LNK_speedlimit(links[54], speed);
break;
case 53:
qps_LNK_speedlimit(links[55], speed);
qps_LNK_speedlimit(links[56], speed);
break;
case 55:
qps_LNK_speedlimit(links[57], speed);
break;
case 57:
qps_LNK_speedlimit(links[58], speed);
qps_LNK_speedlimit(links[59], speed);
qps_LNK_speedlimit(links[60], speed);
break;
case 59:
qps_LNK_speedlimit(links[61], speed);
break;
case 61:
qps_LNK_speedlimit(links[62], speed);
qps_LNK_speedlimit(links[63], speed);
qps_LNK_speedlimit(links[64], speed);
break;
case 63:
qps_LNK_speedlimit(links[65], speed);
qps_LNK_speedlimit(links[66], speed);
qps_LNK_speedlimit(links[67], speed);
qps_LNK_speedlimit(links[68], speed);
break;
case 65:
qps_LNK_speedlimit(links[69], speed);
qps_LNK_speedlimit(links[70], speed);
case 67:
qps_LNK_speedlimit(links[71], speed);
qps_LNK_speedlimit(links[72], speed);
buzz

case 69:
qps_LNK_speedlimit(links[73], speed);
qps_LNK_speedlimit(links[74], speed);
buzz

case 71:
qps_LNK_speedlimit(links[75], speed);
qps_LNK_speedlimit(links[76], speed);
qps_LNK_speedlimit(links[77], speed);
qps_LNK_speedlimit(links[78], speed);
buzz

case 73:
qps_LNK_speedlimit(links[79], speed);
qps_LNK_speedlimit(links[80], speed);
qps_LNK_speedlimit(links[81], speed);
qps_LNK_speedlimit(links[82], speed);
qps_LNK_speedlimit(links[83], speed);
buzz

case 75:
qps_LNK_speedlimit(links[84], speed);
qps_LNK_speedlimit(links[85], speed);
qps_LNK_speedlimit(links[86], speed);
buzz

case 77:
qps_LNK_speedlimit(links[87], speed);
qps_LNK_speedlimit(links[88], speed);
qps_LNK_speedlimit(links[89], speed);
qps_LNK_speedlimit(links[90], speed);
buzz

case 79:
qps_LNK_speedlimit(links[91], speed);
qps_LNK_speedlimit(links[92], speed);
qps_LNK_speedlimit(links[93], speed);
buzz

case 81:
qps_LNK_speedlimit(links[94], speed);
qps_LNK_speedlimit(links[95], speed);
qps_LNK_speedlimit(links[96], speed);
buzz

case 83:
qps_LNK_speedlimit(links[97], speed);
qps_LNK_speedlimit(links[98], speed);
qps_LNK_speedlimit(links[99], speed);
buzz

case 85:
qps_LNK_speedlimit(links[100], speed);
qps_LNK_speedlimit(links[101], speed);
buzz

case 87:
qps_LNK_speedlimit(links[102], speed);
qps_LNK_speedlimit(links[103], speed);
buzz
case 87:
    qps_LNK_speedlimit(links[104], speed);
    qps_LNK_speedlimit(links[105], speed);
    break;
    case 89:
    qps_LNK_speedlimit(links[106], speed);
    qps_LNK_speedlimit(links[107], speed);
    break;
    case 91:
    qps_LNK_speedlimit(links[108], speed);
    qps_LNK_speedlimit(links[109], speed);
    break;
    case 93:
    qps_LNK_speedlimit(links[110], speed);
    qps_LNK_speedlimit(links[111], speed);
    break;
    case 95:
    qps_LNK_speedlimit(links[112], speed);
    break;
    case 97:
    qps_LNK_speedlimit(links[113], speed);
    qps_LNK_speedlimit(links[114], speed);
    qps_LNK_speedlimit(links[115], speed);
    break;
    case 99:
    qps_LNK_speedlimit(links[116], speed);
    qps_LNK_speedlimit(links[117], speed);
    qps_LNK_speedlimit(links[118], speed);
    break;
    case 101:
    qps_LNK_speedlimit(links[119], speed);
    qps_LNK_speedlimit(links[120], speed);
    break;
    case 103:
    qps_LNK_speedlimit(links[121], speed);
    break;
    case 105:
    qps_LNK_speedlimit(links[122], speed);
    qps_LNK_speedlimit(links[123], speed);
    qps_LNK_speedlimit(links[124], speed);
    break;
    case 107:
    qps_LNK_speedlimit(links[125], speed);
    break;
    case 109:
    qps_LNK_speedlimit(links[126], speed);
    qps_LNK_speedlimit(links[127], speed);
    break;
    case 111:
    qps_LNK_speedlimit(links[128], speed);
    break;
    case 113:
    qps_LNK_speedlimit(links[129], speed);
    qps_LNK_speedlimit(links[130], speed);
break;
case 115:
qups_LNK_speedlimit(links[131], speed);
qups_LNK_speedlimit(links[132], speed);
qups_LNK_speedlimit(links[133], speed);
qups_LNK_speedlimit(links[134], speed);
break;
case 117:
qups_LNK_speedlimit(links[135], speed);
qups_LNK_speedlimit(links[136], speed);
break;
case 119:
qups_LNK_speedlimit(links[137], speed);
qups_LNK_speedlimit(links[138], speed);
break;
case 121:
qups_LNK_speedlimit(links[139], speed);
break;
case 123:
qups_LNK_speedlimit(links[140], speed);
break;
case 125:
qups_LNK_speedlimit(links[141], speed);
break;
case 127:
qups_LNK_speedlimit(links[142], speed);
qups_LNK_speedlimit(links[143], speed);
break;
case 129:
qups_LNK_speedlimit(links[144], speed);
break;
case 131:
qups_LNK_speedlimit(links[145], speed);
break;
case 133:
qups_LNK_speedlimit(links[146], speed);
qups_LNK_speedlimit(links[147], speed);
qups_LNK_speedlimit(links[148], speed);
qups_LNK_speedlimit(links[149], speed);
qups_LNK_speedlimit(links[150], speed);
break;

default:
qps_GUI_printf("\nNo detector with that call number\n");
break;
}
LIST OF REFERENCES


Research and Innovative Technology Administration, Bureau of Transportation Statistics.


