

Final Report

**Linking Crash Patterns to ITS-Related Archived Data: Phase II
Volume II: Evaluations of ITS Strategies**

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16. Abstract In Volume I of the report models for assessing real-time crash risk were developed by analyzing freeway loop detector data corresponding to historical crash and non-crash cases. This volume (Volume II) of the report provides application of a subset of those models for evaluating ITS strategies in a simulation environment (PARAMICS) for their potential benefits in improving real-time safety on the I-4 corridor. It was found that under congested conditions Ramp Metering and/or Route Diversion can yield significant reduction in real-time crash risk while under free flow and conditions approaching congestion Variable Speed Limits may be more beneficial.			
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EXECUTIVE SUMMARY

In this volume of the report the models developed in previous volume are utilized to evaluate three ITS (Intelligent Transportation System) strategies, namely, route diversion, ramp metering, and variable speed limits (independently as well as in combination) for reducing crash risk along the freeway. These strategies or combination of them are tested as follows using a microscopic traffic simulation model. First of all, a smaller subsection of the study corridor was simulated in the PARAMICS micro-simulation package. A base-case scenario, i.e., the one without application of any of the ITS strategies is ran first. The resulting traffic parameters are subjected to the models developed for crash risk estimation. The models proposed for real-time application and used in the simulation are slightly different. The reason being a continuous measure of crash risk is required for the simulation runs. The rationale and measures of crash risk are explained in Chapter 6 (Also see Chapter 5—Volume I) of this volume. A series of ITS strategies would be applied to the simulated section of the freeway and resultant measures of traffic parameters would be collected. The new traffic data would be once again input to the crash risk estimation measure. The measures of crash risk would be compared with the base case to see which ITS strategy yields the best improvement.

The ITS strategies would be selected based on experimental designs and the report (See Chapters 6 and 8 for the experimental designs) addresses those in detail. The selection of ITS strategies depends on the traffic conditions being observed and were tested to determine not only the effects of each strategy but also how to best apply them to an urban freeway.

In the analysis presented in Chapters 6 and 7, Route diversion was found to decrease the overall rear-end and lane-change crash risk along the network at free-flow conditions to low levels of congestion. On average, both crash risk measures were found to be reduced between the location where vehicles were diverted and the location where they were reinserted back into the network. However, a crash migration phenomenon was observed at higher levels of congestion as the crash risk would be greatly increased at the location where vehicles were reinserted back onto the network. Ramp metering in the downtown area was found to be beneficial during heavy congestion. Both coordinated and uncoordinated metering algorithms showed the potential to significantly decrease the crash risk at a network wide level. When the network is loaded with 100 percent of the vehicles the uncoordinated strategy performed the best at reducing the rear-end and lane-change crash risk values. The coordinated strategy was found to perform the best from a safety and operational perspective at moderate levels of congestion. Ramp metering also showed the potential for crash migration so care must be taken when implementing this strategy to ensure that drivers at certain locations are not put at unnecessary risk. When ramp metering is applied to the entire freeway network both the rear-end and lane-change crash risk is decreased further. ALINEA is found to be the best network-wide strategy at the 100 percent loading case while a combination of Zone and ALINEA provides the best safety results at the 90 percent loading case.

It should also be noted that both route diversion and ramp metering were found to increase the overall network travel time. However, the best route diversion and ramp

metering strategies were selected to ensure that the operational capabilities of the network were not sacrificed in order to increase the safety along the freeway. This was done by setting the maximum allowable travel time increase at 5% for any of the ITS strategies considered.

Variable speed limits is an ITS strategy that is typically used upstream of a queue in order to reduce the effects of congestion. In Chapters 8 and 9 we have evaluated these strategies. The essential principal behind the investigations was that 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 analysis uses variable speed limit strategies with a corridor-wide approach to reduce the rear-end and lane-change crash risk 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 the first time a dynamic distance has been considered for variable speed limit implementation, to the research group's knowledge.

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%. This increase was found to be statistically insignificant and is therefore 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 our preliminary explorations with microscopic simulations described in Chapters 2 and 3.

Development of ITS strategies for mitigating real-time crash risk is a relatively unexplored concept and has only been attempted by few researchers. The work from other researchers has been provided as Background in Chapter 1. It is clear to the research group that our work is the easily the most comprehensive in terms of crash risk assessment as well as the evaluations of ITS strategies for reduction of crash risk in real-time.

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LIST OF ACRONYMS

AADT: Annual Average Daily Traffic

API: Application Programmer Interface

CATSS: Center for Advanced Transportation Systems Simulation

DDHV: Directional Design Hourly Volume

FDOT: Florida Department of Transportation

FHWA: Federal Highway Administration

FTI: Florida Traffic Information

GUI: Graphic User Interface

HOV: High Occupancy Vehicle

ITS: Intelligent Transportation Systems

OD: Origin-Destination

ORCI: Overall Risk Change Index

RCI: Roadway Characteristics Inventory

VHT: Vehicles Hours Traveled

VSL: Variable Speed Limits

CHAPTER 1

EVALUATING ITS STRATEGIES FOR CRASH RISK ALLEVIATION: BACKGROUND

The work described in Volume I of this report so far has focused on crash data analysis to make reliable assessment of traffic conditions on the freeways for their crash potential. The work in this volume (Volume II) describes the evaluation of ITS strategies for reduction in crash risk. These strategies would be evaluated using a model of the Interstate-4 corridor in microscopic traffic simulation environment.

1.1 Traffic Simulation

Traffic simulation is defined as the process of using a computer and various numerical techniques to model the behavior of vehicles on a transportation network. The simulation model uses numerous mathematical models that are created by transportation engineers who have studied vehicular behavior and verified that the models give an accurate representation of how drivers and vehicles behave. Individual models are created to govern specific behavioral properties of either a traffic stream or individual vehicles. For example, a simple model that is used to govern the behavior of a traffic stream is the fundamental flow-density equation which states that the flow rate is equal to the average speed multiplied by the traffic density (Roess et al., 2003). A typical model that governs the behavior of individual vehicles is a car following model. This type of model relates the distance between two successive vehicles (gap) to parameters such as the average speed of each vehicle and the current distance between them. These mathematical models use no more information than is necessary (information proven significant in the field) to output information about the transportation system. This

allows the user to input the minimum amount of data in order to extract realistic results about the traffic network.

The advantages of traffic simulation are numerous. One of the main advantages to using a traffic simulation package is the ability to test multiple scenarios and alternatives and determine which one performs the best. Although no simulation will be able to determine the exact effects of any traffic management strategy, the simulation will give researchers and engineers a better idea of what to expect with the implementation of a certain alternative in the field. By using simulation software, researchers will be able to determine the alternative that has the highest chance of providing the best results when multiple alternatives are considered. This should allow them to implement the “best” alternative with confidence. Without the simulation, engineers would be forced to use field tests to determine the outcome of a potential strategy. Field tests are extremely expensive compared to traffic simulations and require much more time and effort to implement. Although traffic simulation packages are not cheap (they usually cost in the range of a couple thousand dollars) and require many hours to build and calibrate, this pales in comparison to the cost and time required to outfit an existing roadway. Additionally, continuous field testing can instill a lack of confidence in the driving public and create confusion caused by a constantly changing environment. This can lead to the drivers shying away from the area in question or, worse, a decrease in the safety at the particular area.

Another advantage of traffic simulation is the ability to implement scenarios that simply cannot be field tested. Such an example would be the effect of widening a lane of a freeway on the traffic flow or installing a traffic signal at an intersection that is

currently stop-controlled. Another example would be to test the effect of a technology that is still in the developmental stages. By using a traffic simulations researchers can gain some insight into the potential benefits and drawbacks of the new technology. Additionally, traffic simulations can be used to predict the effects of future demand scenarios on the existing transportation network and can help engineers to determine the best course of action to improve the transportation infrastructure.

1.1.1 Types of Traffic Simulation Models

There are many ways with which to classify traffic simulation models. One method is to classify the models with respect to the level of detail that the simulation software represents the traffic network. Using this criteria, there are three basic simulation types – macroscopic, mesoscopic, and microscopic. Macroscopic models are also known as low fidelity models since they describe the traffic behavior with a low level of detail. These models do not consider the movements of individual vehicles but rather focus on specific sections of the network and aggregate the traffic flow over each section. The model would not consider events such as individual lane changes but would rather assume that the vehicles are spread out across all the lanes of a roadway in a predefined distribution. Microscopic models, on the other hand, are high fidelity models in that they provide the highest level of detail. These models describe the behavior of each vehicle as it moves through the network as well as the interactions between any vehicle and the other vehicles that it encounters. Vehicles will adjust their speed, change lanes, and sometimes even change routes based on the speed and distance of nearby vehicles. Mesoscopic models are mixed fidelity models that describe the vehicles in the network at a higher level of detail than the macroscopic model but a lower level of detail

than the microscopic model. For example, while microscopic models would base lane changing maneuvers on the interaction of vehicles in the traffic stream, mesoscopic models would include lane changes but instead base them on lane densities or lane speeds (Lieberman and Rathi, 1997).

In general, low fidelity models are easier to develop, execute, and maintain. However, the primary drawback to using these models for real world situations is that they are often less accurate than other model types. Higher fidelity models are by far the most accurate but they are extremely difficult to develop since they involve many complex mathematical models that require a tremendous amount of information to accurately describe vehicular behavior. Additionally, they require a longer time investment to simulation a real life network and require a tremendous amount of computational power to run. Lastly, these models require much more time to run than macroscopic models. Whereas a macroscopic model can be run and the results reported inside of a few minutes, microscopic models could take hours or even days to get similar results.

Selecting which level of complexity is needed in a traffic simulation model then becomes very important to the situation that is being described. If a model is being built to determine flow rates on a freeway that does not have many weaving sections then a macroscopic model would probably be the best choice. However, if this same freeway has multiple weaving sections and merging areas then it is possible that the vehicular interactions at these areas would be of high importance. Additionally, if the number of lane changes that is performed in these sections is required as an output then a

microscopic or mesoscopic model would be better suited to the task since the macroscopic model cannot give these results.

Another classification of traffic simulation models addresses the processes that represent variation within the models. The types of these models are either deterministic or stochastic. In deterministic models all interactions are represented by exact relationships. Stochastic models, on the other hand, include probability functions rather than exact functions to describe the vehicular relationships. For example, in deterministic models if a mean headway is specified then each vehicle would maintain that specific headway as it moves through the network. However, in stochastic models the mean headway would be defined by the user but the individual vehicular headways would follow a predefined distribution with the specified headway as the mean. In this type of model, each vehicle would have a unique headway but the average of all vehicular headways throughout the network would be close to the specified mean headway value.

In almost all traffic simulation models time is the basic independent variable. How the simulation models time, however, is another method with which traffic simulation models can be categorized. Continuous models describe how elements in the network change continuously over time. Discrete models describe how elements change abruptly at specific instances in time. There are two different types of discrete models, discrete time and discrete event. Discrete time models split the simulation period into a number of segments or time intervals. The model updates the position, speed, and other information about each vehicle at the end of each time interval. Between these updates the speed and position are interpolated by the model. Discrete event models operate similarly but instead of segmenting the simulation period into equal time intervals, the

model is segmented based on known events. An example of such an event could be when a pre-timed traffic signal changes from the green phase to the amber phase, the amber phase to the red phase, etc. While these models are more economical with respect to time they should only be used when a continuous change in the traffic flow is not expected. For continuous changes, the discrete time model gives far more accurate results.

1.1.2 Various Applications of Traffic Simulation Models

Micro-simulation, which is used in this study, has been used by numerous other researchers to predict the behavior of traffic flow. Using traffic simulation, Mahmassani and Jayakrishnan (1991) modeled route choice dynamics in the case of lane closures. Their studied proved that providing in-vehicle information to drivers increased the ability of the network to reach a steady state condition (as opposed to a congested condition). It was also found that providing in-vehicle information to users reduced the time required for the network to achieve these steady state conditions.

An example of traffic simulation used in the industry of transportation engineering and the design process is a study performed by Korve Engineers (1996). As a part of this study the WATSIM program was used to model part of a 20 mile freeway corridor in California including SR 242, SR 4, and I-680. Future traffic demands were predicted for the years 2000, 2010, and 2020 and various alternatives were tested to determine their effect on the network. These alternatives included geometric changes, widening of roadways, HOV lanes, and ramp metering. This example showed the superiority of micro-simulation in analyzing candidate designs of large scale systems over simpler methods such as a straight-forward Highway Capacity Manual analysis.

Gardes et al (2002) used the PARAMICS micro-simulator to create a 19 mile network which simulated the I-680 freeway in San Francisco, California. The purpose of this study was to model several corridor improvement strategies that were being considered by CALTRANS (the governing body over the California freeway system). These improvements included ramp metering, auxiliary lanes, and HOV lanes. The results of this study provided CALTRANS with valuable input as to which strategies provided the best results compared to the cost of implementation.

Bertini et al (2002) used PARAMICS to model a diamond interchange at the intersection of I-5 and Wilsonville Road in Wilsonville, Oregon. Once calibrated, the model was run at different loading conditions (50%, 75%, 100%, and 125%) to examine the traffic flow through the interchange. Traffic flows, queue lengths, delays and travel times were collected and compared with data observed in the field. The study also compared the average vehicle delay obtained by PARAMICS with the average vehicular delays that were predicted using the HCM 2000 methodology and found that they were relatively consistent.

Chu et al (2004) used PARAMICS to evaluate the potential benefit of several Intelligent Transportation Systems strategies such as local and coordinated ramp metering. A portion of the freeway network through Irvine, California was modeled and ITS techniques such as ramp metering, traveler information systems and their combinations were programmed into the simulation. This study used various measures of effectiveness such as vehicle hours traveled (VHT), the standard deviation of travel time for a particular origin-destination pair, average mainline travel speed, total on-ramp delay, and average travel time through the length of an arterial. The study found that

while providing traffic information to motorists helps to relieve traffic congestion due to an incident the quickest, ramp metering fails to provide significant benefits unless paired with a traveler information strategy.

1.1.3 Applications of Traffic Simulation for Safety

Traffic simulation models have also been used to some extent in the field of traffic safety. While the safety of a freeway is not directly measurable by the simulator, researchers use surrogate measures of safety and examine how they change under different scenarios in order to indirectly measure how different traffic management strategies affect the safety of a roadway.

One such instance is the work of Drummond et al (2002) who performed a study to assess the feasibility of comparing the output of traffic simulation software to actual crash profiles to show whether there was a link between the number of crashes in a corridor and another parameter which is measurable in the simulation. Using historical traffic data from two sites in Virginia, the study found that there was an increasing relationship between delay per mainline vehicle and stops per mainline vehicle with traffic crashes along the corridors. The researchers then used the Synchro Plus SimTraffic Software to model and make various changes to the traffic flow along the two traffic corridors. By adding signals to the modeled corridors, the researchers were able to determine the effect of the increasing signal density on the two surrogate measures of safety and, indirectly, the crash rates. This is one example of using historical data for a specific location to determine a relationship between traffic crashes and a measurable traffic parameter and then simulating the same location to test alternatives to alleviate the crashes.

Park and Yadlapati (2003) used the VISSIM program to examine the potential of variable speed limits to increase the safety of work zone areas. To capture the safety of the freeway, the researchers created an equation called the Minimum Safe Distance Equation which yields a value that shows whether or not vehicles are following at safe speeds through the work zone. The idea behind this equation is that if vehicles are following at safe speeds there will be a large enough headway between consecutive vehicles to reduce the risk of a crash (particularly rear-end crashes) within the work zone area. The research found that implementing variable speed limit control strategies served to reduce the average speed variation through the work zone area which increased the safety considerably.

Another study performed by Lee et al (2004, 2005) used PARAMICS to determine the safety benefits of variable speed limits and ramp metering. Using historical loop data collected from a 10 km stretch of the Gardiner Expressway in Toronto, Canada, Lee et al (2003) developed 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 portion of this freeway and applying the model, they discovered that the individual application of variable speed limits and ramp metering successfully reduced the crash risk along the simulated freeway. However, this simulation was rather simplistic as it included only a small portion of the freeway (2.5 km) and did not use real traffic data to calibrate the simulation.

1.2 Selecting Traffic Simulation Software for this Study

Since the objective of this project is to determine the effects of ITS strategies on the safety of an urban freeway 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 purpose of the ramp metering and route diversion would be to reduce the negative effect of vehicles merging from the on-ramps to disrupt the traffic stream. Since merging behavior is directly related to vehicular interaction, it is decided that the best method to capture this phenomena and the effect on the traffic safety would be through the use of a microscopic simulation software. Additionally, a stochastic software would be best used since this more accurately models the behavior of vehicles on the freeway and, additionally, 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.

A review of the literature shows that there are a myriad of transportation simulation packages that are available. Shaw and Nam (2002) worked extensively with the CORSIM 4.2 software but 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. The purpose of their simulation was to model the metropolitan Milwaukee freeway system. However, creating just a single, complex interchange required the use of 430 nodes which severely limited the rest of their model. Additional problems with CORSIM included cumbersome network editing, older traffic algorithms (reflecting the fact that the original CORSIM code dated back to the 1970's), and unrealistic simulation results. They looked instead to two newer simulation packages, PARAMICS (PARAllel MICroscopic Simulation) and VISSIM. A comparison of the three microscopic simulation software packages was performed and the results are summarized below in

Table 1-1. As shown, both PARAMICS and VISSIM were found to be far superior to the CORSIM package.

Table 1-1: Comparison of CORSIM, PARAMICS, and VISSIM Packages (Shaw and Nam, 2002)

Evaluation Criteria		CORSIM	PARAMICS	VISSIM
A. Model Capability				
1	Network Size Limit	★	★★★	★★★
2	Network Representation	★	★★★	★★★
3	Traffic Flow Representation	★★★	★★★	★★★
4	Detail of Output	★★	★★★	★★
5	Network Merge	★	★★★	★
6	3-D Modeling	★	★★★	★★
7	Traffic Composition	★	★★★	★★★
8	Animation	★★	★★★	★★
B. Ease of Use				
9	Input Data Requirements	★★★	★	★★
10	Network Coding / Editing	★	★★★	★★
11	Input / Output Review	★	★★★	★★
C. FSOA Application Requirements				
12	VISTA (GIS) Interface	★★★	★★★	★★
13	Economic Analysis Interface	★★	★★★	★
14	Incident Management Analysis	★	★★★	★★
15	Actuated Signal Control Devices	★	★★	★★★
16	User-Defined Traffic Control & API	★	★★★	★★★
17	Public Transportation	★	★★	★★★
D. Other				
18	Calibration Results	★★	★★★	★★
19	Program Integrity	★	★★★	★★★
20	Technical Support	★	★★	★★★
21	Documentation	★	★★	★
22	Record of Large-Scale Freeway Applications	★	★★★	★★
23	Software Cost per Copy	★★★	★	★
FINAL RATING				
★★★		4	17	9
★★		4	4	10
★		15	2	4

Boxill and Yu (2000) evaluated over 70 software packages to determine which were more suited to study Intelligent Transportation Systems (ITS) issues. Of the 76 software packages examined, the top nine were determined which included CORSIM, VISSIM, and PARAMICS. Table 1-2 (below) shows a summary of the nine top software packages and evaluates their ability to model certain ITS features as well as other helpful properties.

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

	AIMSUN 2	CONTRAM	CORFLO	CORSIM	FLEXYT II	HUTSIM	INTEGRATION	PARAMICS	VISSIM
ITS Features Modeled									
Traffic devices	X						X	X	
Traffic device functions	X						X	X	
Traffic calming					X	X	X	X	X
Driver behavior	X			X	X		X	X	
Vehicle interaction	X			X	X		X	X	
Congestion pricing						X		X	
Incident	X		X	X	X	X	X	X	X
Queue spillback	X			X	X	X	X	X	X
Ramp metering	X			X	X	X	X	X	X
Coordinated traffic signals	X	X	X	X	X	X	X	X	X
Adaptive traffic signals	X	X	X	X	X	X	X	X	X
Interface w/other ITS algorithms	X								
Network conditions		X				X		X	
Network flow pattern predictions					X	X	X	X	X
Route guidance									
Integrated simulation	X	X		X	X	X	X	X	X
Other Properties									
Runs on a PC	X	X		X	X	X	X	X	X
Graphical Network Builder	X	X			X	X			X
Graphical Presentation of Results	X	X		X	X	X	X	X	X
Well Documented	X	X	X	X	X	X	X	X	X

As shown above, PARAMICS and AIMSUN 2 appear to be the best software packages with respect to ITS features modeled as well as other properties. Although none of the software packages appear to be able to implement route guidance, PARAMICS features an Application Programmer Interface (API) that allows to 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 route of a vehicle along the network while the simulation is running which effectively mimics the implementation of route guidance. Therefore, based on the studies performed by Shaw and Nam (2002) and Boxill and Yu (2000), as well as the numerous studies that have used PARAMICS in the past and have shown its reliability on freeways and urban roads, the PARAMICS software package was selected for use in this study.

1.3 Route Diversion

Most regional planning processes include a network assignment stage where vehicles are placed on a traffic network. Usually the most important factor concerning the network assignment is the travel time. Using the travel times of various links, planners are able to determine the optimal distribution of vehicles through a traffic network. However, this distribution is made under the assumption that the travel times are known to the users of the traffic network. In fact, this is rarely the case as the travel time is continually changing due to congestion, incidents, and events that cannot be fully predicted. Therefore, the actual assignment of vehicles onto a real traffic network is usually less than ideal. One method that has been proposed to alleviate this problem is to provide drivers with real-time information regarding the status of the network. Providing

this information through Advanced Traveler Information Systems (ATIS) will allow users to make more informed decisions about mode and route choice which should help the traffic network achieve a state of equilibrium.

As summarized by Hall (1983), “For any given quantity of passive information there exists an ideal travel time and an actual travel time. The ideal travel time is the travel time for a traveler who chooses the best possible route using the given information, the actual travel time is the travel time for actual travelers using the given information.” Therefore, it can be seen that the purpose of ATIS is to reduce the gap between ideal travel times and actual travel times.

Abdel-Aty et al (1993) performed a literature review noting several important studies that were performed to assess the effect of ATIS on route choice. These studies showed, in general, that the impact of ATIS on driving behavior and route choice was significant. However, it was not clear what factors prompted drivers to change their route when not given travel information. This needs to be fully understood if researchers hope to gauge the true benefits of providing users with advanced travel information. Abdel-Aty noted that the effect of ATIS on an actual traffic network was not significant and this was mainly due to the lack of market penetration of ATIS at the time of his investigation. A later study performed by Abdel-Aty and Abdalla (2004) showed that as the level of information provided to the driver increases, the average travel time decreases. This clearly shows the effectiveness of ATIS and route diversion on travel time.

In the past few years, however, drivers have been receiving increasingly more information about the state of the traffic network than ever before. News stations provide traffic updates throughout the day that inform viewers of increasing congestion,

incidents, and gives suggested alternative routes based on these situations. Radio programs offer the same advice but this is mostly heard in the car by users who have already embarked on their trip. The installation of navigation systems in vehicles, which were first used to give general directions, help to show drivers potential problems and can offer alternate routes as well. In Florida, a telephone service has been implemented that drivers can call and receive updated information about travel time and traffic incidents (511 Traveler Information Telephone Services). This information is also available on the internet (www.fl511.com). Systems such as these allow drivers to make more informed decisions about their departure time and route choice in order to reach their destination as quickly as possible.

1.3.1 Studies on Benefits of Route Diversion

Recent advances in traffic simulation have also allowed researchers the opportunity to model some of the effects of ATIS on a traffic network. Yang and Koutsopoulos (1996) used the MITSIM program to model a segment of the A10 beltway in Amsterdam. Non-recurrent congestion was simulated by a 20 minute incident and the effect of ATIS on drivers was modeled. When 30% of the vehicles were given updated real-time traffic information, the average travel time in the network fell 2-4%. For drivers who had alternate routes available, this resulted in an 18% savings in travel time.

Chu et al (2004) performed a study in which various ITS strategies were employed to reduce incident related congestion in Irvine, California. The results showed that the most effective way to reduce incident based congestion was to provide real-time information to the users. When drivers were given information about the incident, they diverted their routes so that they would experience shorter travel times. This helped the

network reach a state of equilibrium much faster than if the drivers traveled under the assumption that there was no incident.

Shah et al (2003) performed a study in the Washington, D. C. area comparing simulated travel times of network users. Paired travelers were created: one using ATIS technologies and one using a habitual route and departure time. The results showed that using ATIS significantly reduced the number of early and late arrivals of a trip. In some cases this involved users altering their departure time but it also involved drivers using an alternate route if their habitual route would cause significant delay that would reduce the chances of arriving on time. The study found that the addition of ATIS also reduced trip disutility by 15%; however, no significant change in the overall travel time was found.

Work done by Oh and Jayakrishnan (2002) examined a system in which private ATIS companies could give real-time information to users currently in route. In this system, the companies would collect information from users with in-vehicle navigation systems equipped (IVNS) while also providing them with real-time traffic information. As a part of the study, the researchers also created two simulation networks to examine the ability of the IVNS to accurately describe the real-time traffic situation of the network. The two networks created were a simple traffic corridor and a model of the highway infrastructure in Anaheim, California. The simulation showed that users who were guided by ATIS information had a reduced travel time compared to non-ATIS users. However, the gap in travel time between guided and unguided vehicles decreased with the increase in market share of the IVNS. This is probably due to the fact that more vehicles having IVNS would mean fewer vehicles without. Additionally, the average travel time for all vehicles decreased significantly as the market penetration of the IVNS

increased. This, once again, showed that route diversion based on information given by ATIS has a positive effect on the overall travel time of the network. However, while there have been many studies performed to describe the effectiveness of route diversion on the operational characteristics of a traffic network no studies have been found relating route diversion to improved safety conditions.

1.4 Ramp Metering

Ramp metering is becoming an increasingly popular method of relieving congestion on freeways. The idea behind ramp metering is to limit the number of vehicles that enter the freeway to reduce the turbulence caused at on-ramps when slow moving vehicles try to enter the faster moving traffic stream. Ramp meters are basically traffic signals placed on the ramp that control when and how many vehicles enter the freeway. The earliest recorded use of ramp metering was on I-290 in Chicago, Illinois in 1963. This was rather simplistic, however, in that a police officer directed traffic onto the freeway one vehicle at a time. However, the current use of ramp metering has expanded throughout the United States and the rest of the world and now involves complex algorithms that use traffic data taken from the freeway to determine how many vehicles are allowed to enter the freeway. Extensive use of ramp metering can currently be seen in the United States in Minnesota, California, New York and Washington State as well as across the world in Amsterdam, Paris, and Glasgow.

Originally, the signals that controlled ramp metering were pre-timed signals which allowed vehicles into the freeway at a controlled, but constant, rate. Now, however, actuated signals are used that take into account the conditions on the mainline when determining how much green time to allot to the meter. These strategies have two

distinct types: local and coordinated. Local ramp metering takes into account the traffic conditions only near the ramp that is being metered. When using this type of strategy the metering rate of a particular ramp is independent of the rate at another ramp. Coordinated ramp metering, on the other hand, requires that the metering rate of a particular ramp to be based on traffic data from various locations within the corridor. This effectively allows the metering rate of each ramp in the corridor to be related to each other so that traffic conditions at one location could affect the metering rate of a ramp located miles away.

1.4.1 Ramp metering algorithms

There are many different methods that are used to determine the metering rate of the on-ramps. The demand capacity algorithm, proposed by Masher et al (1975) works by measuring the occupancy downstream from the location where the on-ramp merges onto the freeway. If this occupancy value exceeds a predetermined critical occupancy value related to congested conditions then the metering rate is set to a minimum value. In this algorithm it is undesirable to have occupancies greater than the critical occupancy since this will lead to congested conditions. If the measured occupancy is equal to less than the critical occupancy then the metering rate is set to a value that is equal to the difference between the downstream capacity and upstream volume. This is summarized below in Equation 1-1.

$$R = \begin{cases} \max(q_{cap} - q_{in}), & O_o \leq O_c \\ R_{min}, & O_o > O_c \end{cases}$$

...Equation 1-1

Please note that in this equation R represents the metering rate, $R_{\min}$ the minimum rate, O_o the measured occupancy, O_c the critical occupancy, q_{cap} the flow rate at capacity, and q_{in} the upstream flow rate.

The percent occupancy algorithm suggested by Koble et al (1980) uses occupancy measurements upstream of the on-ramp entrance area to measure levels of congestion. In this strategy the critical occupancy is determined from historical data (Hadj-Salem et al, 1988). Equation 1-2 below describes the logic of this algorithm.

$$R[k] = K_1 - K_2(O_{in})[k-1]$$

Equation 1-2

Where $R[k]$ is the metering rate for time interval k , K_1 is the flow at capacity, K_2 is the slope of a straight line approximation of the un-congested flow portion of fundamental traffic flow diagram, and O_{in} is occupancy of the upstream detector.

These two algorithms are known as feed-forward algorithms since the metering rate at a particular time interval is independent of the rate at any previous interval. This is the major flaw in these methods as there is too much variation in successive metering rates during the application of these algorithms. Papageorgiou et al (1991) solved this by developed a feed-back algorithm that considers previous metering rates when determining the current metering rate. This method, known as ALINEA, is one of the most common ramp metering algorithms used and is based on the Proportional Integral

(PI) feed-back control law. The metering rate, $R[t]$, is also a function of the difference between the measured occupancy $O[t]$ downstream of the ramp at time t and a target set critical occupancy, O_c . K_R is a regulator parameter and δ_t is the length of interval at which each re-evaluation of the algorithm is done. The equation for ALINEA algorithm is given in Equation 1-3.

$$R[t] = R[t - \delta_t] + K_R (O_c - O[t])$$

Equation 1-3

The ALINEA algorithm has been modified to improve its accuracy in different situations. MALINEA proposed by Oh and Sisiopiku (2001) is modified to take into account the occupancy both upstream and downstream of the on-ramp. FL-ALINEA proposed by Smaragdis and Papageorgiou (2003) expands the application of ALINEA to use flow measurements from downstream detectors rather than occupancy measurements. UP-ALINEA (Smaragdis and Papageorgiou, 2003) uses occupancy measurement upstream of the on-ramp and estimates the downstream occupancy. This method shows good results in areas that previously used the demand capacity algorithm or the percent occupancy method. Lastly, X-ALINEA/Q (Smaragdis and Papageorgiou, 2003) modifies ALINEA to add queue control. This method accounts for queues building into the on-ramp and increases the release rate to reduce the amount of congestion on the on-ramp that spills over into surrounding surface streets. Excluding X-ALINEA/Q, these variations of ALINEA are all less efficient than the traditional ALINEA method but

allow for use of the ALINEA method when certain detector information is missing. The ALINEA method has also been modified for coordinated ramp metering use. The algorithm, termed METALINE, was assessed by Papageorgiou et al (1997) and it was determined that ALINEA was not inferior to METALINE. Since ALINEA is much simpler to implement than the complex METALINE algorithm, ALINEA is preferred for use.

Another popular ramp metering algorithm is the ZONE ramp metering algorithm. In this method, which is a coordinated algorithm, the freeway is broken up into small sections 3 to 6 miles long (called zones) and all ramps within each zone are examined together (Stephanedes, 1993). Typically, the upstream area of each zone is a free flow area and the downstream area is a bottleneck section but this does not have to be the case. The objective of this method is to balance the volume of traffic entering the zone with the traffic leaving the zone. Stephanedes (1994) tested this strategy on a ring road in Minneapolis, Minnesota and found that it yielded a 31% reduction in travel time.

Note that there are two ways to implement any ramp metering method in the field (Papageorgiou and Kotsialos, 2002). The first, termed one-car-per-green realization, allows only one car to enter the freeway per metered traffic cycle. Therefore, the green time is fixed at a small value (between 1.3 and 2.0 seconds) to allow for a single vehicle to enter. The metering rate that is determined using the algorithms mentioned above is then enacted by altering the cycle length. A longer cycle length would be more restrictive while a shorter cycle length would allow more vehicles to enter the freeway. The second method is called the traffic-cycle realization. In this method the cycle length

is held constant and the green time is altered. When this method is implemented more than one vehicle will be allowed to enter the traffic stream at a time, permitting that there is enough green time on the cycle to allow this.

1.4.2 Studies on the Benefits of Ramp Metering

There have been several field studies performed to assess the benefits of ramp metering. A study performed by Cambridge Systematics (2001) on the Zone ramp metering strategy in Minnesota determined that ramp metering saves the city over \$40 million annually and increases average mainline freeway speeds from 46 mph to 53 mph. This is also noted with a reduction in the number of traffic crashes.

Papageorgiou et al (1997) also tested the benefits of ALINEA on the Boulevard Peripherique in Paris, France, as well as the A10 Motorway in Amsterdam, Netherlands. The study showed that ALINEA provided superior results and helped to reduce travel and congestion along the roadway. The study also proclaimed that ALINEA distinguished itself from other local ramp metering strategies due to its simplicity, transferability, low implementation costs, efficiency, and flexibility.

Due to public opposition to ramp metering, the Minnesota legislature passed a law in 2000 requiring ramp metering (which used the Zone algorithm) to be turned off for 8 weeks. In those eight weeks, congestion and the number of traffic crashes were shown to increase significantly. Additionally, the travel patterns of drivers changed based on the increased congestion. However, the study also showed that ramp metering increases travel time for shorter trips while decreasing travel time for longer trips (Levinson et al., 2002).

Zhang et al (2001) as part of the PATH program at the University of California used PARAMICS to compare four typical metering algorithms: ALINEA, Bottleneck, Zone, and SWARM. The results showed that all of the algorithms serve to improve the traffic flow. Additionally, none of the algorithms significantly distinguished itself in terms of operational benefits. Hasan et al (2002) used MITSIMLab to study ramp metering on the Big Dig network in Boston, Massachusetts. The study compared the local strategy ALINEA with the coordinated strategy FLOW. The results showed that although ramp metering almost always improved the mainline flow, the overall system performance was reduced when ramp metering was implemented at lower demand levels. Coordination was found to be very effective at higher demand levels and at locations with bottlenecks while the addition of queue control always served to improve the system performance.

Therefore, ramp metering has been shown to have tremendous operational benefits. Additionally, in Minnesota it was proved that removing ramp metering decreased traffic crashes by 26 percent (Cambridge Systematics, 2001). However this is not a real-time benefit of ramp metering but rather a benefit aggregated after some time. Lee et al (2006) used PARAMICS to show the real-time benefit of ramp metering on the crash potential on a freeway. However, this study considered a very small network with only a single ramp.

1.5 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 used 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) (described later in this report in more detail) 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.

1.5.1 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.

1.5.2 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 maximize 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 loglinear 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.

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.

1.6 Conclusions

A thorough review of literature provided in this chapter helped us select the optimal microscopic traffic simulation package for our project. As discussed in section 1.2 PARAMICS would be used in this study. The chapter also provided a summary of recent

advances in field application and simulation evaluations of the ITS strategies tested in this research project for real-time alleviation of crash risk. The next chapter provides the first of the two step exploration. It investigates preliminary variable speed limit strategies over a small subsection of the 36.25-mile Interstate-4 corridor. Following chapters provide detailed investigation of the benefit of VSL in the reduction of real-time crash risk.

CHAPTER 2

PRELIMINARY VSL EVALUATIONS: NETWORK PREPARATION, SIMULATION, AND EXPERIMENTAL DESIGN

2.1 Scope of the Preliminary Analysis

This chapter summarizes the preliminary evaluation of the ITS strategies that can potentially reduce the real-time crash risk on the freeways. The strategies evaluated in the preliminary analysis include VSL, Ramp metering, and some combinations of the two. Note that this analysis, exploratory in nature, was conducted on subsection(s) of the 36.25-mile corridor of the study area. It extends from Station 33 to 49, about 9 miles of Interstate-4. Moreover, the models used for crash risk assessment for evaluation at this preliminary stage are logistic regression models based on within stratum matched sampling of non-crash cases (Models discussed in Chapter 4 — Volume I of this report).

The preliminary analysis is conducted in two stages; first variable speed limit strategies are evaluated. Based on the results of the preliminary exploration the analysis is extended (with better validation of the network) in the second stage to ramp metering as well as co-ordination of ramp metering with the VSL strategies. The map in **Figure 2-1** shows the area that is studied in this preliminary exploration. A few major roads are labeled to help familiarize the reader with the study area.

This stage of preliminary analysis pursues the goal of a best case implementation of VSL. Two loading scenarios, a fully loaded case (90% of ramp maximums) and an off-peak loading case (60% of ramp maximums), at multiple stations with multiple implementation methods are strategically attempted until a best case implementation is found. No final recommendation is made for the use of VSL in the fully loaded case (low-speed case). However, ramp metering indicated a promising potential for low-speed case; which was explored in the second stage of the preliminary analysis.

2.2 Overlay Generation

The overlay that is used to draw the I-4 corridor is generated by a combination of aerial photography, obtained from the Orange County Property Appraiser's Office and the Seminole County Property Appraiser's Office, and AutoCAD drawings, obtained from the Orange County Transportation GIS department.

2.2.1 Autocad

Knowing that the I-4 corridor will be drawn in PARAMICS and that the program accepts .dxf files, an overlay is drawn in AutoCAD and then saved as a .dxf file. Preventing any unnecessary work, drawings of I-4 that could be found were obtained. An AutoCAD drawing of the entire Orange County Network was obtained from the Greater Orlando Aviation Authority (GOAA) and a drawing of I-4 from Orange Blossom Trail to the Seminole County-Orange County border was obtained from the Center for Advanced Transportation System Simulation (CATSS) office at the University of Central Florida (UCF).

The drawings, after being overlaid, show good agreement. The Orange County Network shows only centerlines and does not give a clear indication as to the location of

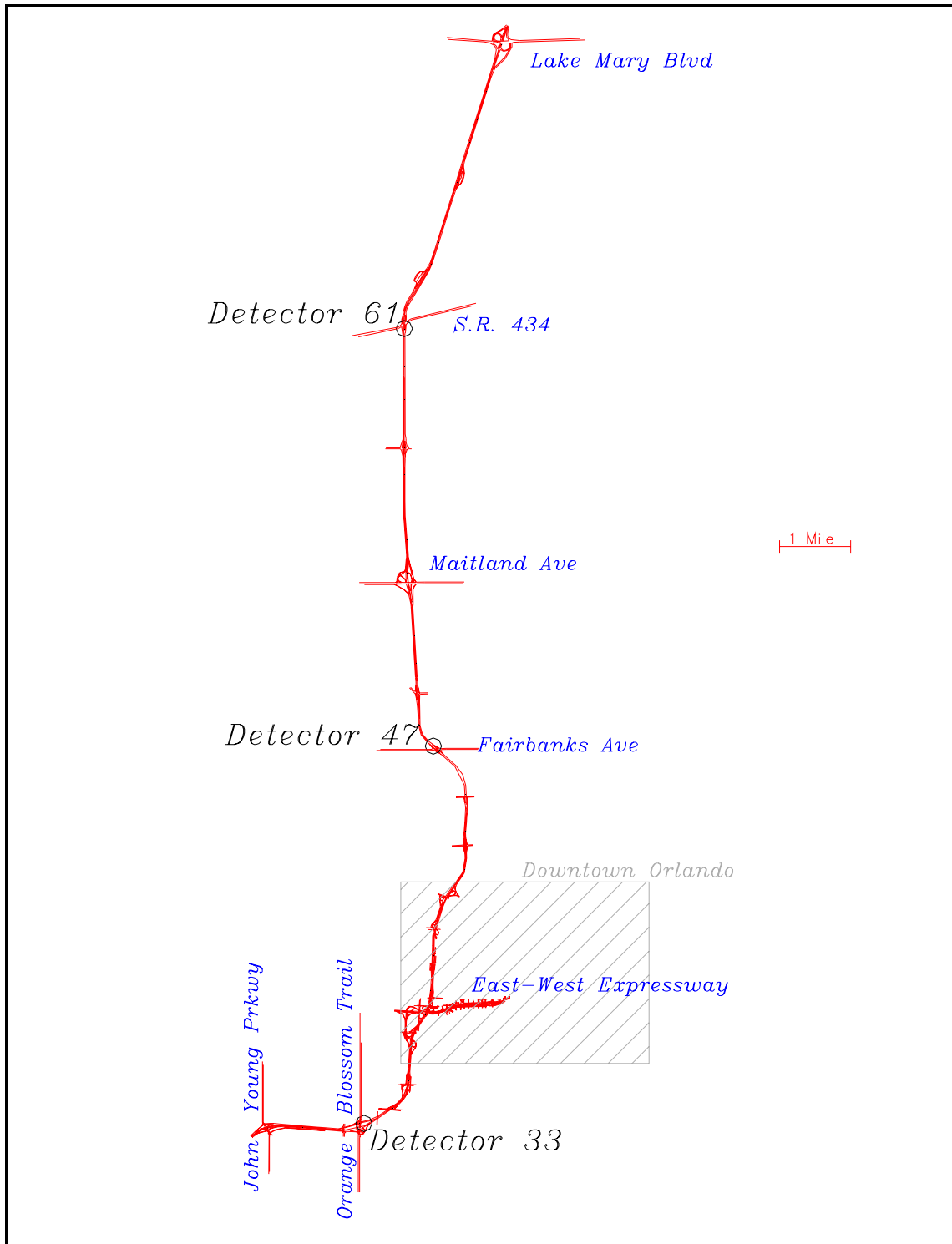


Figure 2-1: I-4 section of preliminary simulation analysis with major road and station locations

on and off ramps, accelerations and deceleration lane lengths, or exact geometries (turn radius, median locations). CATSS's drawing does an adequate job of displaying geometries and on and off ramp locations, but does not show acceleration and deceleration lane lengths. Unfortunately, the drawing is smaller than the length of I-4 to be simulated. To regain the missing links of I-4 aerial photography is used.

2.2.2 Aerial photographs

Orange County aerials must be purchased by section from the county office. Due to the cost only the aerials that added to the I-4 drawing are selected (from Orange Blossom Trail to the Orange County-Osceola County border). Seminole County GIS provides aerials free of charge. The aerials are downloaded from Seminole County's GIS website (<http://www.seminolecountyfl.gov/it/programming/gis/pcl.asp>) for the length of I-4. After obtaining all the different sections, all of the aerials are resized and combined in Photoshop. To see a screenshot of the aerial photography in AutoCAD see Figure 2-2 and Figure 2-3.

2.2.3 As-built drawings

After using the aerials, merge and diverge lengths remain the only geometric information missing. At first it may seem as though the lengths could be determined using aerials, but the accuracy of the aerials is too low. In order to determine these lengths as-built drawings are consulted. The drawings, available at the CATSS office at UCF, provide information for I-4 from Orange Blossom Trail to the Orange County-Seminole County border. Other as-builts could not be acquired, so the scope of the

project has to be downgraded to Orange Blossom Trail in the south. The northern border remained Lake Mary Boulevard, because all of the interchanges up to Lake Mary have long acceleration and deceleration lanes and their lengths do not affect the performance of the vehicles moving through them.

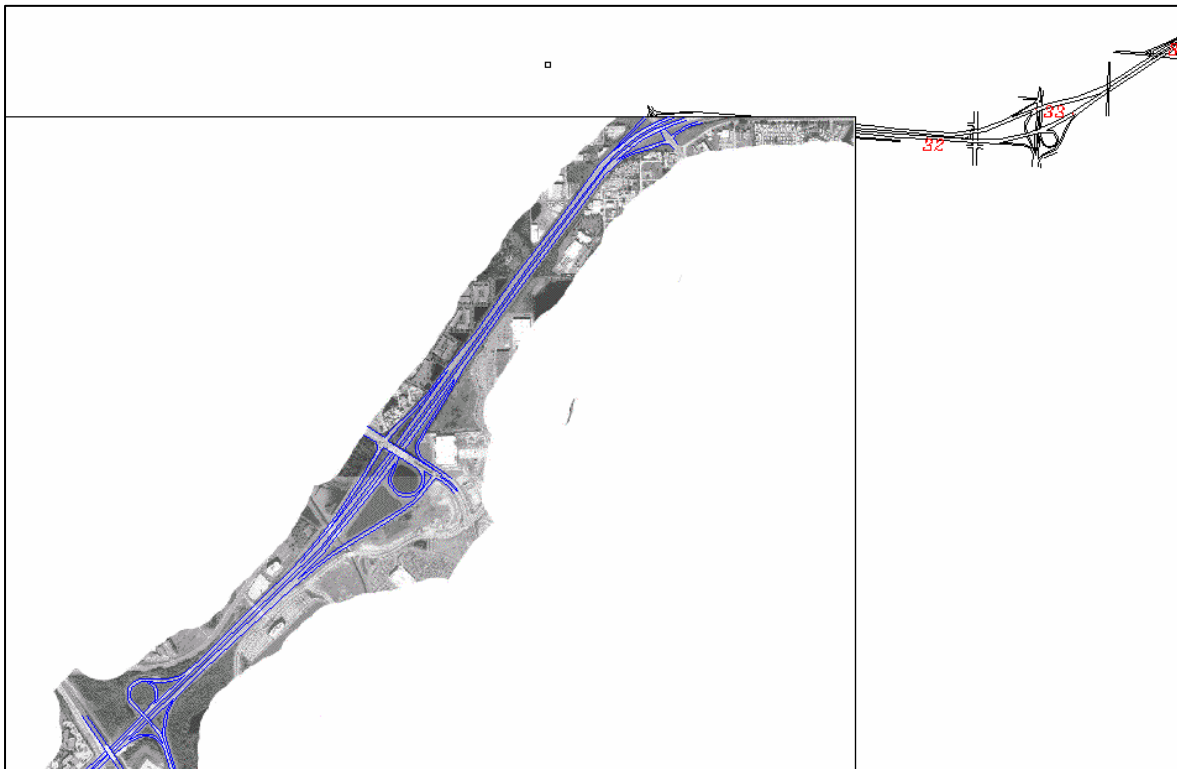


Figure 2-2: View of original AutoCAD file with xref aerial Orange County

All long length ramps, greater than 750 ft, can be assumed to have a length of 750 ft, which is enough length in PARAMICS to allow for smooth behavior.

2.3 Overlay Importation

The size of the new combined aerial file proved to be too much for AutoCAD to handle. Therefore, the combined file needed to be split into thirds. Three AutoCAD files

are generated to handle each of the overlays. The road edges are then redrawn within AutoCAD. The aerial photos are then removed, and the resulting drawing files combined.

2.3.1 Zero lay required

The combined drawing files, unfortunately, could not be imported into PARAMICS. One of PARAMICS quirks is that it does not accept X-Ref layers into its overlays. As a result, each of the aerial photos has to be imported separately, generating five overlays (one for each aerial photos, making three in Orange County and one in Seminole County, and one for the AutoCAD file described above in the AutoCAD subsection).

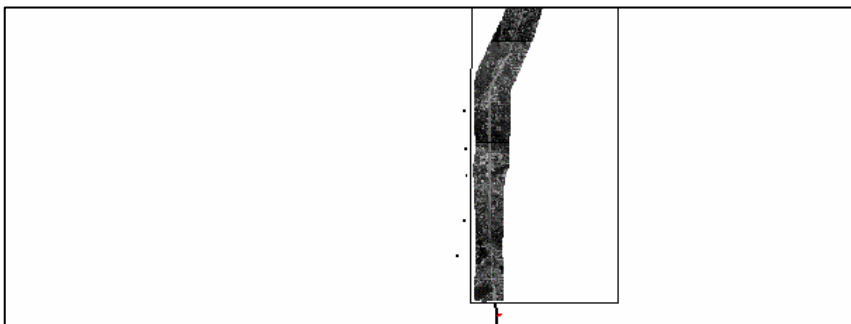


Figure 2-3: View of original AutoCAD file with xref aerial of Orange and Seminole County

2.4 Definitions

Signposting distance, ramp awareness distance, kerb points, and speed memory are all terms associated with the PARAMICS software. Signposting distance is a link attribute that shows how far in advance of a particular geometry, such as an off ramp, that a driver becomes aware of that geometry. Ramp awareness distance is the distance before a ramp that vehicles begin to increase their headway to allow for merging behavior. Kerb points are similar to hot grips in AutoCAD. They are located at the four vertices of a link and control the location of the outer edge of the lanes. While lane width is specified in PARAMICS and almost always aligns links, correctly link modifiers such as curves or slip lanes (deceleration lanes) can alter otherwise smooth fit. Speed memory refers to how often the vehicles' information (location, speed, headway, acceleration) is updated.

PARAMICS is a continuous simulator, not a discrete event simulator and thus requires this parameter.

Some other key definitions are the factors, called core variables, PARAMICS uses to affect driver behavior. Mean reaction time, target headway, queue speed, queue distance make up the core variables for PARAMICS. Driver characteristics aggressiveness and awareness can also be used to a lesser extent to change driver behavior. Mean reaction time is the average amount of time it takes a driver to react to being too close or too far away from the vehicle in front of them. The target headway is the headway, in seconds, which a driver attempts to maintain in non-queuing conditions. Queue speed is the speed under which a queue is formed. In other words queuing behavior is exhibited once a vehicle goes beneath this speed. Queue distance is the distance between two vehicles that once obtained causes queue behavior to end. Each of the specified numbers is an average of a distribution, not a value given to every vehicle. In other words specifying a mean reaction time of 1.3 seconds means that some drivers will have a reaction time above 1.3 seconds and some below 1.3 seconds, but the average works out close to 1.3 seconds. Driver aggressiveness helps to determine the allocation of reaction time and mean reaction time. Drivers are given an aggressiveness level between 1 and 10, with 10 being a high aggressiveness. High aggressiveness is more likely to correspond to lower headways and reaction times. Driver awareness has to do with the likeliness that a driver will miss an off ramp and have to correct there route. In this network there is only one path from one zone to another, so awareness does not come into play.

2.5 Drawing Network

Once the overlay is generated the next step is drawing the roads in PARAMICS. In order to draw, lane categories need to be identified, nextlane behaviors need to be specified, and priorities need to be assigned.

2.5.1 Road categories

Finding the differences in speed limits and number of lanes proved to be a challenge, even though there is little variability along I-4. The Florida Department of Transportation (FDOT) was contacted first to obtain the information. They provided screen shots from their 2002 Florida Traffic Information (FTI) CD with speed limits written on them. The speed limit is shown to be 55 for the length of I-4. Field inspection shows this to be inaccurate. Eventually, field data is used to determine the number of lanes and speed limits north of the East-West Expressway. Speed limits and number of lanes south of the East-West Expressway was determined through conversation with the FDOT's Construction Project Manager, Seema Jagtap. Figure 2-4 shows a screenshot of PARAMICS's Configuration Manager.

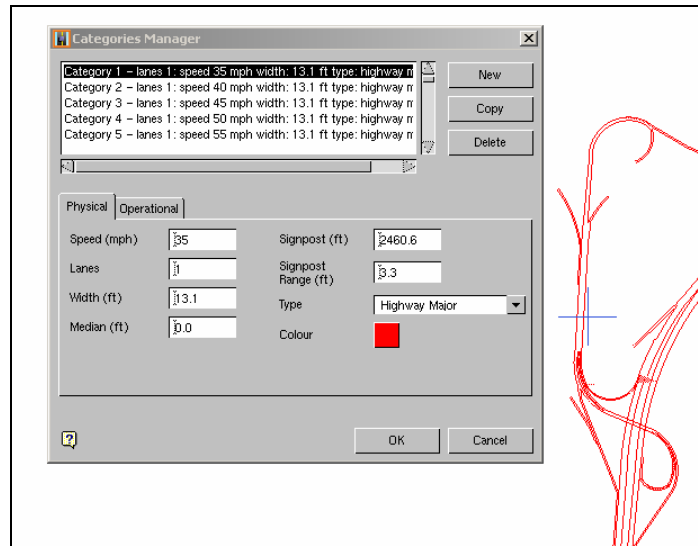


Figure 2-4:: Category window

2.5.2 Zones

In other PARAMICS drawings of I-4, combined zones are used to minimize the complexity of OD Matrixes, and are preferred, if accuracy is not compromised, in the process. However, the chance of misrepresenting the traffic behavior, by oversimplifying the OD, is too high to risk. Instead, every on ramp starts with a zone and every off ramp ends with a zone. No zones are combined in order to simplify the problem.

2.5.3 Two Roads versus One Road

After the first round of drawing is complete, there are a few touchup issues. The first concerns the ramps along I-4. The ramps were originally drawn without regard to

awareness distance and deceleration lane lengths and modeled with extra lanes on the freeway. Both decisions proved to be errors. Using extra lanes on the interstate instead of slip ramps leads to confusing driver behavior that does reflect real world scenarios. The awareness distance needs to be positive in order for vehicles traveling on the interstate to allow gaps to merging traffic from onramps. Without positive awareness distances large backups form on the mainline and the onramp. The deceleration lanes are referred to as slip lanes within PARAMICS and have their own designation within the links file.

Getting properly functioning on and off ramps involves more than specifying awareness distances and slip lane lengths, the two functions are related through node positioning and often come into conflict. Slip lanes stop at the end of a link and start a distance specified in the links' characteristics, back from the end of the link. On ramps join in the middle of links with the awareness distance extending backward and acceleration lane extending forward. Neither the awareness distance nor the merge distance can extend beyond the link to which the on ramp connects. Conflict results when on and off ramps occur at, or near, the same location on opposite sides of the road. The off ramp needs a node at its location, while the on ramp requires that the node be placed at least a distance equal to the awareness distance upstream or the acceleration lane distance downstream. Figure 2-5 shows screenshots of links that have been specified correctly.

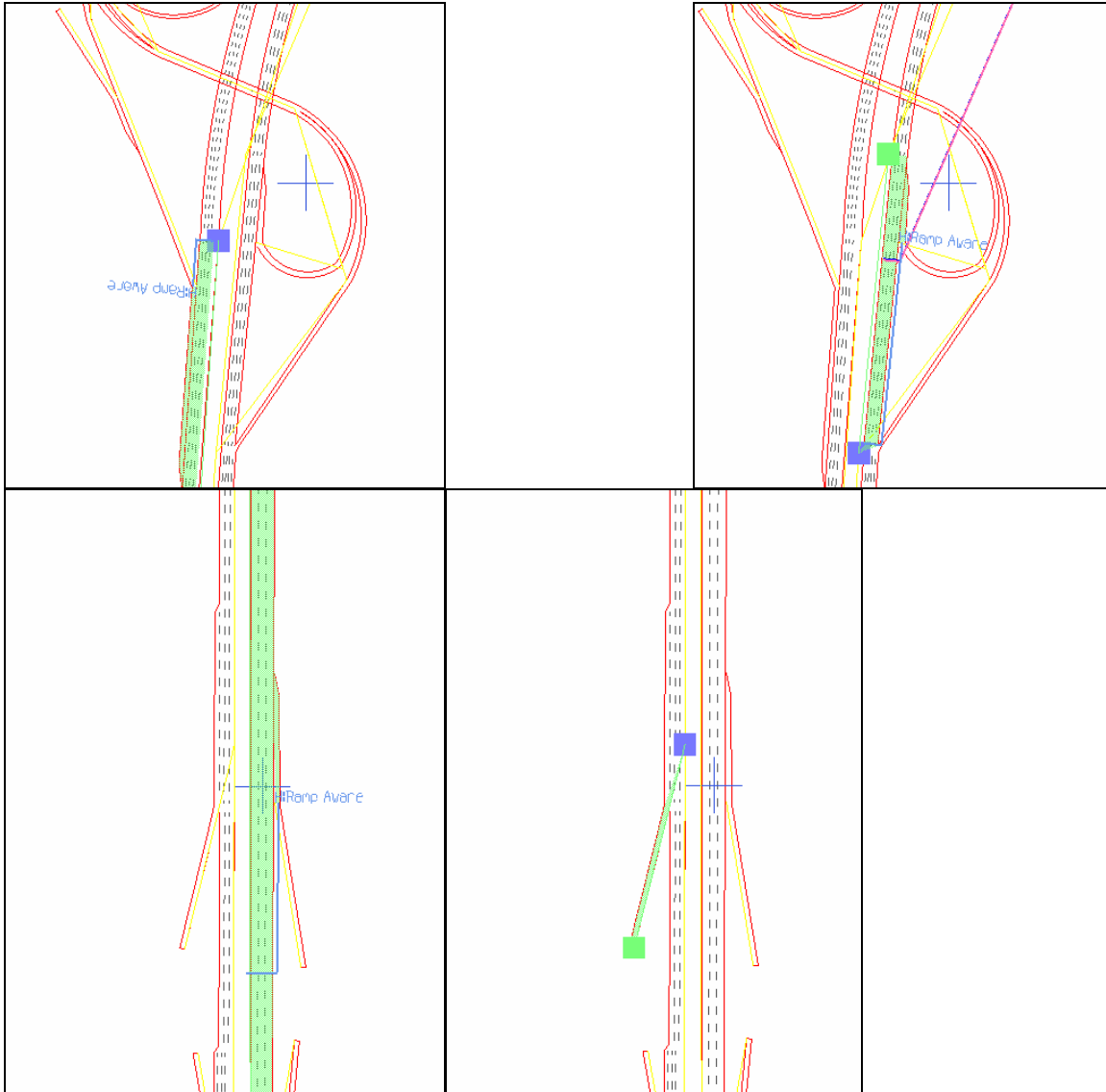


Figure 2-5: View of node location for the ramps

When the ramps are redrawn nextlane information has to be updated, because the default setting allows for left turns on the interstate. In other words, westbound traffic is allowed to use eastbound ramps, and vice versa, a huge modeling problem.

The node location problem and the need to control nextlane information can be avoided all together by using two separate one-way roads. In essence, that is how an

interstate operates, so conceptually it is not a problem. However, setting up separate roads calls for extra nodes and links which all have to be tracked during runs. If the network is sensitive enough to be affected by changing ramp locations, awareness distances, or acceleration or deceleration lane lengths then the extra memory and therefore runtime is justified. Because the network is an interstate, the model is redrawn with two roads: one running north (east) and one running south (west).

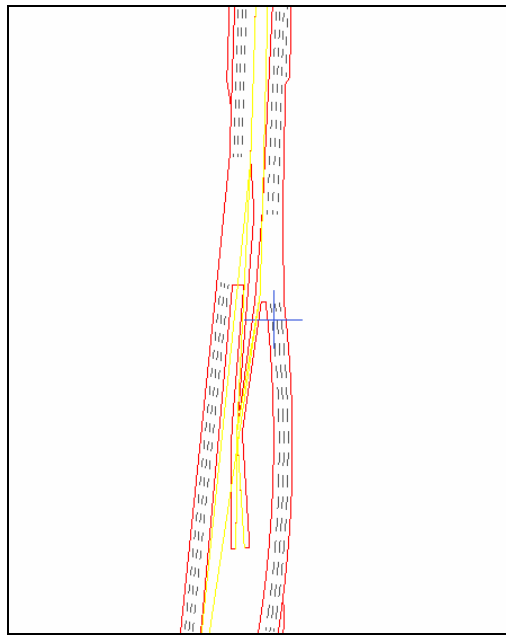


Figure 2-6: Kerb properties of a northern South Street Ramps

2.5.4 Left ramps

South Street in downtown Orlando has three left ramps leading to and from it. PARAMICS does not allow for left hand ramps to be modeled. Contacting PARAMICS User Support, they suggest checking the wide entrance and wide exit characteristics on

the link that has a left ramp. Checking the characteristics did not lead to changes in the drivers' behavior, so the suggestion of PARAMICS Support is not used. Instead, the ramp is modeled as a continuation of the interstate with a lower speed limit. Luckily, the roads have low enough counts that the lane changing behavior (which is different for ramps than lanes) does not have a dramatic impact on the overall system. If the Kirkman Road Exit had been modeled this assumption may not have been able to be made. Kirkman Road connects I-4 to Universal Studios, Islands of Adventure, Wet and Wild, and other tourist locations; therefore, it is a heavily traveled ramp. A screenshot of the South Street Interchange is shown in Figure 2-6.

2.5.5 Loop detectors

2.5.5.1 Location determination

A second drawing based on data obtained from the FDOT RCI database indicates loop locations that needed to be incorporated into the simulation. The drawing, which spans the entire length of the simulation project, is overlaid onto the network to find loop location. The location of loop detector is critical, because they are the eyes for the crash risk estimation algorithm.

2.5.5.2 Number of Lanes

The loop detectors on I-4 do not span all of the lanes. Instead, they only reach the three left most lanes, in some sections there are 4 loops, but only 3 are consistently archived. Therefore the crash risk assessment algorithm uses data from only 3 lanes (Abdel-Aty et al., 2005). Instead of making sure each loop detector in the model only spans three lanes, the post processing only calls for counts, occupancies, and densities from the inner most three lanes even though the model records all lanes. The loops are drawn to encompass all the lanes, whether three or four, on the cross-section of I-4 where they are located. The process simplifies the drawing.

2.6 Calibration of the OD Matrix

2.6.1 Data sources

To construct the Origin Destination Matrix of the I-4 corridor six sources are looked into: the 2002 Florida Traffic Information (FTI) CD, 2003 FTI CD, the 2002 AADT provided on the FDOT website, the 2003 AADT provided on the FDOT website, a 2002 study constructed by URS, and the 2003 Florida Standard Urban Transportation Model Structure (FSUTMS) model output. When comparing the sources it becomes apparent that the data on the FDOT website is the same data as the FTI CD, dropping the number of sources to four. The FTI CD data is not used because the FDOT website has a better format.

Because the number of lanes on I-4 has been increased to 4 on the most heavily traffic portions and only three lanes have loop detectors, the counts would have to be adjusted to avoid underestimation. It could be assumed that the forth lane could be interpreted using the first three lanes data. The estimated values of the forth lane detectors could be higher or lower than the real value and correcting it could introduce more error. In any case, the workload would be too large and error prone, making the method undesirable.

2.6.2 Data sources compared

A non-parametric analysis is used to compare the data sources to see if they are different and if a decision as to which one should be used needs to be made. The Friedman Test shows a significant difference between the four data sources. Using the ranks it shows that the URS data is significantly higher than the FDOT data, which is unexpected. As expected, the 2002 FDOT data is significantly lower than the 2003 FDOT data. The 2003 FDOT data is chosen. It is the most recent data and from a reliable source.

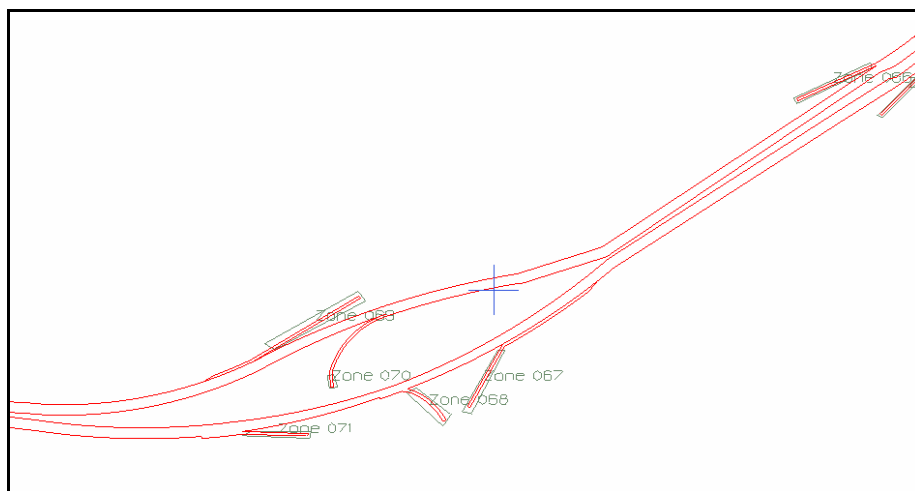


Figure 2-7: Zones view at the Orange Blossom Trail Interchange.

2.6.3 Finding zeros

With all but two of the zones (the ones on the I-4 terminuses) in the modeling being one way, the OD matrix is mostly populated with zeros. In determining the location of all the zeros, all rows that are associated with off ramps are given zeros. Next, all columns associated with on ramps are given zeros. Then, all indices associated with on ramps and off ramps on different roads (different directions on the interstate) are given zeros. Finally, all off ramps behind a given on ramp are given zeros.

2.6.4 Gravity model

Research Conducted at UCF used the gravity model in order to sort out an OD, but did not have the accuracy that the heuristically corrected OD showed (Kanike, 2003). As a result, for this corridor the gravity model will start the process and then the resulting OD will be corrected. Equation 2-1 below shows the gravity model.

$$\text{Count} = P_i * (A_j * T_{ij}) / (\sum A_i * T_{ij})$$

...Equation 2-1

P_i : Number of trips produced by the zone

A_i : Number of arrivals received by the zone i

T_{ij} : Time of the trip from zone i to zone j

Notice that the equation does not include a correction for socioeconomic factors. Instead the equation relies on the on and off ramp peak hour counts and the travel time

between ramps. The peak hour is determined using the AADT and the K factor for each ramp. The CATSS office at UCF provides the travel time data.

2.6.5 Heuristically corrected

The output of the gravity model shows several problems. First, there are large difference between the sum of the columns and the counts at that off ramp. To correct this all the counts are adjusted until the error is minimized. The adjustments are done in order to give values that minimize error. The steps are as follows:

1. When rows and columns both are either too low or too high then the count is adjusted appropriately.
2. When a column is too high then the count is adjusted downward.
3. If both a column and row for a cell is too low the count is adjusted upward.
4. Steps 2 and 3 are repeated.

It should be noted that the correction steps are preformed at all other ramps besides the East-West Expressway first and then the East-West Expressway counts are adjusted. It is assumed that in the calibration and validation step the values will be changed in order to get accurate value.

2.6.6 Remaining error

The remaining error, the difference between the peak hour ramp counts and the OD matrix, after adjustment, is 3.90% and has been concentrated in the zones leading to the East-West Expressway. The error is concentrated here because it has the largest counts

and thus will have the smallest effect on the outcome. The final OD for the 73 zones, in terms of vehicles per hour, may be **seen in Appendix II-A.**

2.6.7 Measurements

PARAMICS requires that the measurements, or outputs of each network, be specified. In each case only the minimum data is collected in order to maximize run speed and minimize runtime. Figure 2-8 shows the measurements that maybe collected using loop data.

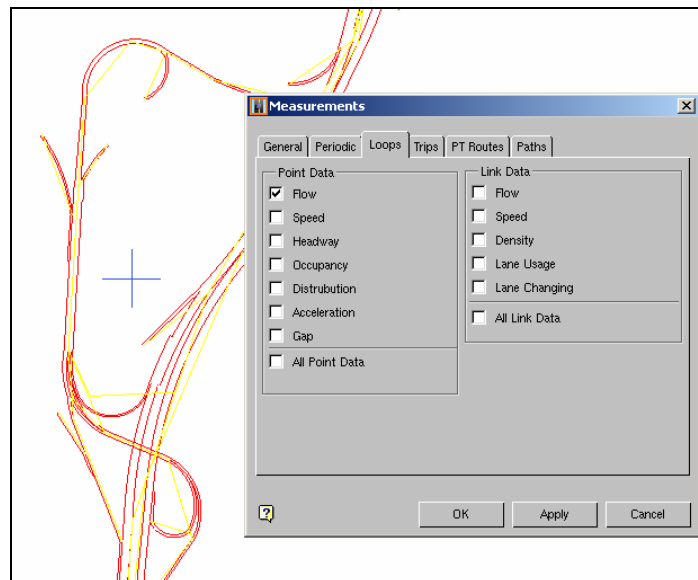


Figure 2-8: Measurements window

2.6.7.1 Occupancy

The occupancy measurement in PARAMICS is part of the instantaneous measures generated by loops. The output is not grouped and averaged into neat thirty second intervals as are needed to predict crashes, instead every time a vehicle passes over a loop the time and the amount of time spend over the detector is recorded. This outputted information is organized into thirty-second packets afterwards, using a macro in Excel.

2.6.7.2 Flow

Flow can be measured as a thirty second average. The measurement does not take place at the detector, even though the document that the information is reported is labeled according to the detector. Instead, the measurement takes place at the end of the link that the detector is on. It would seem that the best way to account for the discrepancy would be to place the detector at the very end of the link, or to create a node just past the end of the loop detector. However, the PARAMICS modeler user guide warns that double counting may occur if this approach is used. It advises that a distance equal to twice the length of the largest vehicle in the network should be used in order to prevent this double counting. Twice the length of the largest truck in the network works out to be about 80ft. Also, when nodes are adjusted the loops must be redrawn, making the technique tedious and time consuming.

The instantaneous flow is calculated by PARAMICS by inverting the headways of two consecutive vehicles. The flows categorized by the vehicle type are extreme and difficult to work with therefore the instantaneous flow is not used.

By using the data generated by the occupancy, the number of vehicles that pass over a detector within a thirty-second interval are known. By dividing that count by the thirty seconds the hourly flow is obtained and can be compared to existing data. Using this method the amount of information collected by PARAMICS is minimized, speeding up the runtime.

2.6.7.3 Average speed

The average speed is collected in the same manner as the occupancy. The average speed measurement in PARAMICS is part of the instantaneous measures generated by loops. The output, similar to occupancy, is not grouped and averaged into neat thirty second intervals as needed to predict crashes. Instead, every time a vehicle passes over a loop the time the speed of the vehicle is recorded. This information is organized into thirty-second packets afterwards, using a macro in Excel.

2.7 Verification

Generally, verification takes place before calibration, but vehicles needed to be added to the network in order for the verification step to take place. As a result the steps are out of their traditional order. Often times the verification step is confused with the validation and calibration steps. Verification is meant simply to observe that the network's behavior insure it looks similar to the behavior seen in the real world. Numbers do not need to be consulted. This level of exacting is reserved for the validation step.

2.7.1 Merging behavior

Awareness distance must be long enough in order to allow merging behavior to be smooth. Otherwise, large backups can clog the entire length of the simulation within a few minutes. Instead of checking each distance by hand, a simulation run is completed and problems with awareness distance become immediately obvious, due to the large backup that is formed, starting at the ramp with an error. Figure 2-9 shows typical ramp properties.

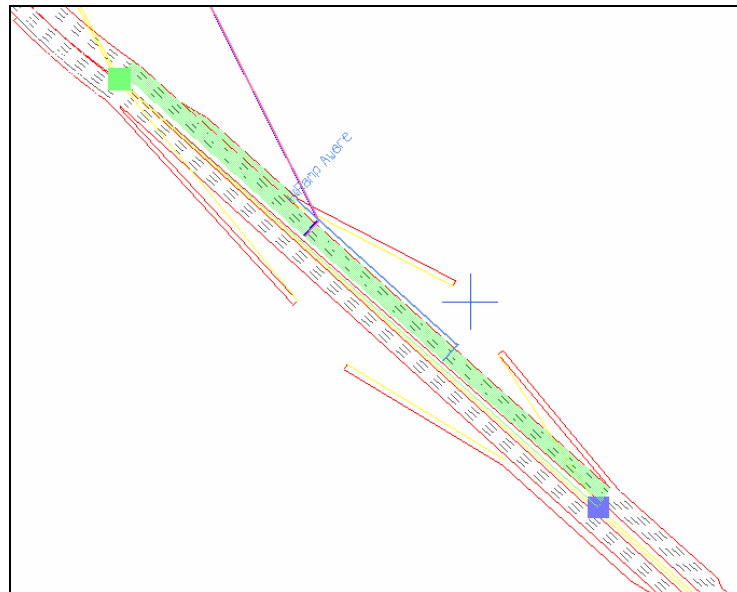


Figure 2-9: Ramp properties at the Fairbanks Interchange.

2.7.2 Absence of Prohibited Turns

Because the network was drawn using two roads (one for each direction) instead of one road, there are no prohibited turns. All turns also are required to be major priority, because there are no conflicting movements. Figure 2-10 shows how to specify allowable turns.

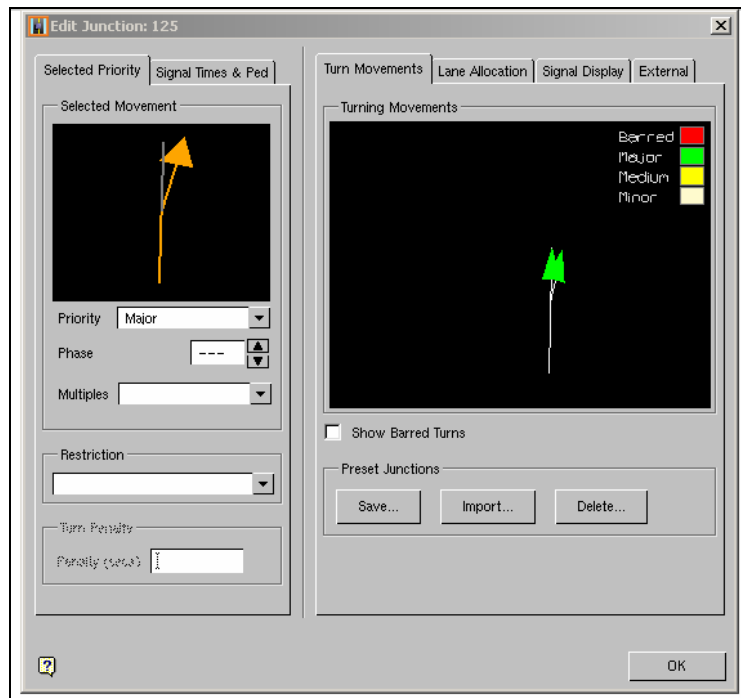


Figure 2-10: Allowed turns as shown in the Junction window

2.7.3 Smooth Behavior between Links

If lanes do not match up, or if a minor priority is given to through movements, vehicles do not travel through the roads smoothly. To correct this all priority settings are checked. Also, all kerb points are checked to make sure they are smooth. Specific

attention is paid to off ramps. Because of the way they are drawn, it is common for kerb settings to be off.

2.7.4 Appropriate nextlanes

PARAMICS tends to allow all lanes to turn off at an exit ramp. To correct this all nextlanes settings are changed so that only the outside lane, along with the sliplane, can use the off ramp. Figure 2-11 shows the nextlanes function in PARAMICS.

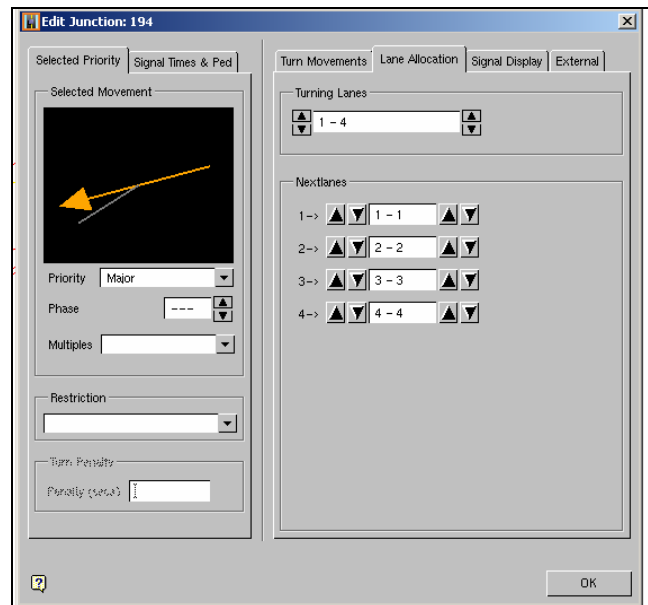


Figure 2-11: Nextlanes as shown in the Junction window

2.7.5 Od adjustment

Having completed the heuristically corrected origin destination matrix further revisions are made when geometries cannot change improperly exhibited behavior. Making sure the changes are in areas that the heuristically corrected values were made,

the variance from the gravity model can be minimized while a corridor that behaves correctly is obtained.

2.7.6 Determining running time

As part of the scope of the simulation the run length of the simulation was decided to be three peak hours in the afternoon. The time is chosen to be approximately 4:00pm to 7:00pm. However, because the network begins without any vehicles on the roadway there needs to be a warm-up period, unless it is assumed that the warm-up period is short enough that the values generated will be overwhelmed by the other data. To test this, the number of vehicle within the network is tracked and plotted. Figure 2-12 shows that there is a significant warm-up period. Also that steady state range of approximately 1400 to 1600 vehicles per hour is reached. The time period that first reaches this range is approximately 15 minutes into the simulation. Therefore the start of the simulation is pushed back to 3:45pm, and the 3:45pm to 4:00pm data is disregarded.

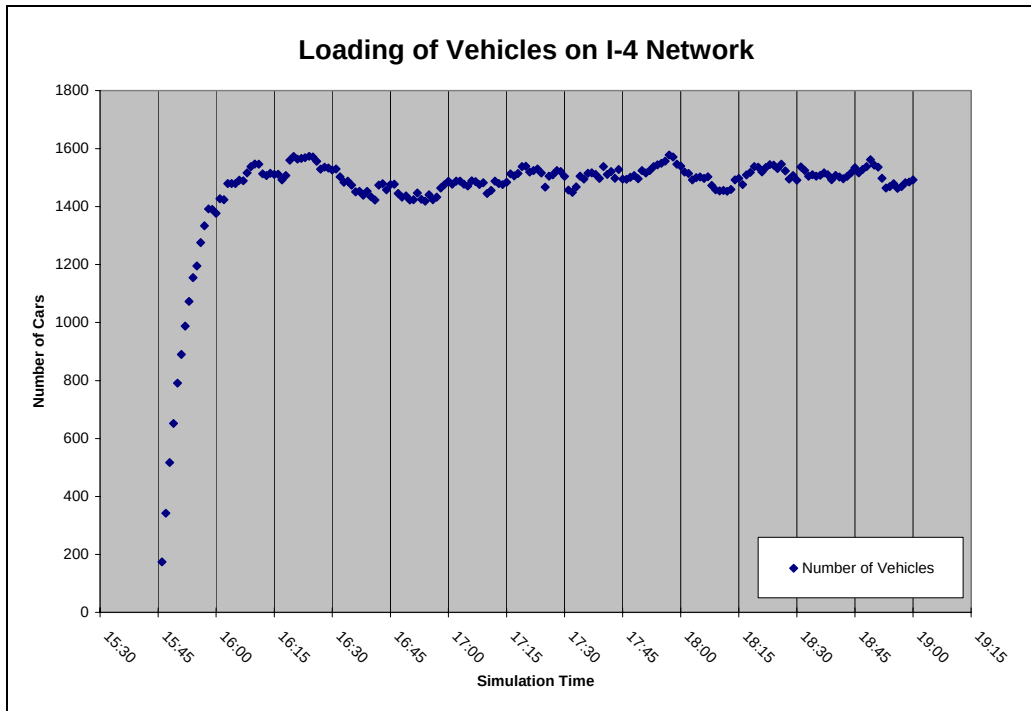


Figure 2-12: Loading of vehicles versus simulated time (The simulation begins at 3:45pm)

2.7.7 Ramp modeling

The lights at the end of ramps and the number of lanes of ramps are not modeled. The reason they are not modeled is because the ramps are assumed to have the capacity to handle all the loads present. As a result each ramp is one lane wide and ends with a zone that absorbs the vehicles when they reach the ramps. This is a simplification that should be addressed in future work.

2.8 Validation

The mainline counts provided by the 2003 FTI CD data allow for the network to be validated. The OD is not adjusted to account for any difference in the counts. Instead the core parameters within PARAMICS are changed.

2.8.1 Mean target headway & mean driver's reaction time

The mean target headway and mean driver's reaction time are changed to values found in the literature. The values along with the corresponding results are shown in Table 2-1. For a recap, Gardes et al. (2002), Addulhai (2002), and Lee et al. (1999) stated their numbers as 1 second and .6 seconds, 1.65 seconds and .42 seconds, and .615 seconds and .415 seconds, respectively.

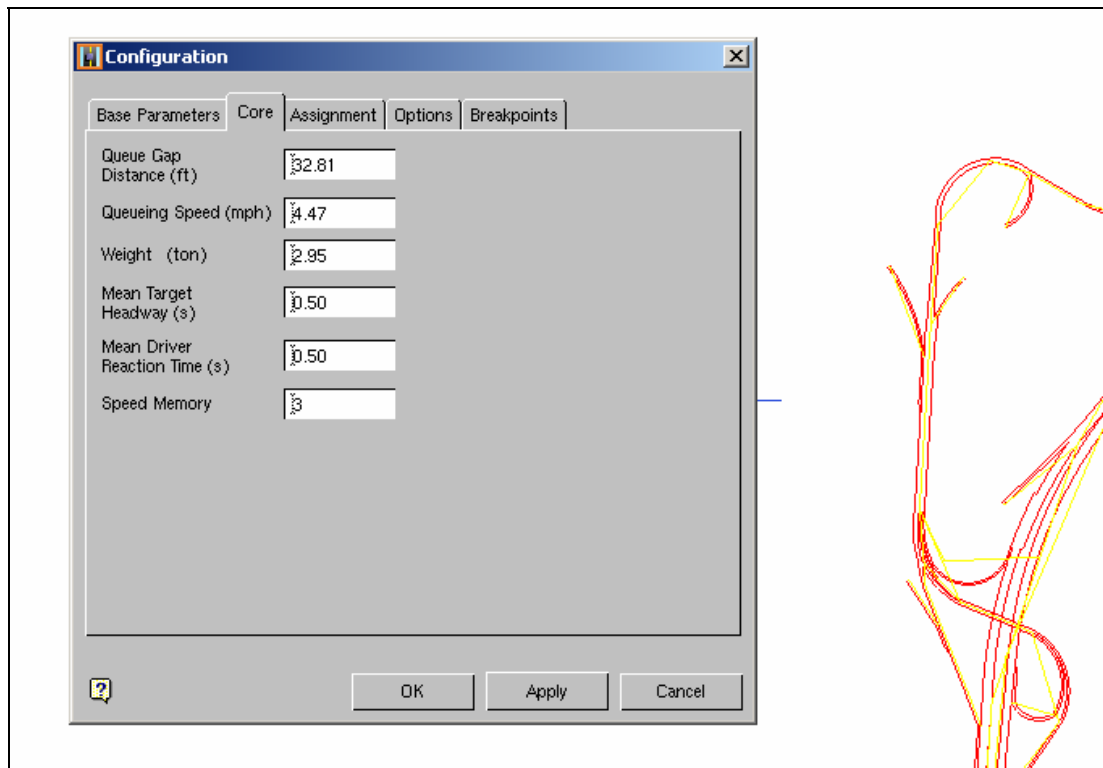


Figure 2-13: The core behavior variables as shown in the Configuration window

2.8.2 Queuing distance

The default queuing distance is 32.81 ft. To see the effect of the queuing distance an alternative distance is chosen. The value, 20 ft, does not have any significance, other than to test the effect of queuing distance. The results of the different queuing distances are shown in Table 2-1. The results show that queuing distance does not have a major effect on flows except for the cases with low mean target headways. It appears that when headways are low enough there is an interplay with the two factors.

2.8.3 Queuing speed

The default queuing speed is 4.47 miles per hour. To see the effect of the queuing speed an alternative speed is chosen. The value, 24.47 miles per hour, does not have any significance, other than to test the effect of queuing speed. The results of the different queuing speeds are shown in Table 2-1. The results show that increasing the queue speed decreases the percent error. Most of the error in the model is due to the low flow numbers, so, taking into consideration what queue speed controls, it is plausible to assume that the overall effect of increasing the queue speed is to increase flows when congestion takes place. Queue speed is a significant variable and produces a large change in flow.

Table 2-1: Core variable specification and their resulting error percentages

Run Number	Headway	Reaction Time	Queue distance	Queue speed	Error Rate
1	1.00 sec	0.60 sec	31.80 ft	4.470 mph	17.14
2	1.65 sec	0.42 sec	31.80 ft	4.470 mph	20.5
3	0.61 sec	0.42 sec	31.80 ft	4.470 mph	18.91
4	0.50 sec	0.50 sec	31.80 ft	4.470 mph	14.19
5	1.00 sec	0.60 sec	31.80 ft	24.470 mph	14.77
6	1.65 sec	0.42 sec	31.80 ft	24.470 mph	19.77
7	0.61 sec	0.42 sec	31.80 ft	24.470 mph	13.52
8	0.50 sec	0.50 sec	31.80 ft	24.470 mph	13.19
9	1.00 sec	0.60 sec	20 ft	4.470 mph	17.14
10	1.65 sec	0.42 sec	20 ft	4.470 mph	20.5
11	0.61 sec	0.42 sec	20 ft	4.470 mph	14.54
12	0.50 sec	0.50 sec	20 ft	4.470 mph	14.36
13	1.00 sec	0.60 sec	20 ft	24.470 mph	14.77
14	1.65 sec	0.42 sec	20 ft	24.470 mph	19.77
15	0.61 sec	0.42 sec	20 ft	24.470 mph	13.52
16	0.50 sec	0.50 sec	20 ft	24.470 mph	13.19

2.8.4 Inspecting speeds

While the flows were verifying rather well, the speeds, the crucial output of the simulation, looked uncharacteristic. Generally, a high traffic roadway runs in one of two states, uncongested or congested. In the uncongested states the speeds are similar to speed seen on low traffic roadways of similar geometry; with limited access roads this means around 55 mph. In the congested state speed take a nosedive, dropping down to around half the uncongested speed if not less. Looking at the transition of a roadway from uncongested to congested over time. The change happens at a specific instance. At one time the average speed at a given point is 57 mph and then at the same point 30 seconds later it is 15 mph. Once the congestion takes hold and forms a queue. The queue grows until the number of vehicles entering the queue from the backside is below the

number of vehicle leaving the queue at the front side. After the demand on the roadway drops below this threshold the queue begins to dissipate.

As a result of this queuing behavior, a graph similar to Figure 2-14 is expected. When running PARAMICS using its default values the results looked like Figure 2-14. Notice the sharp peaks in Figure 2-14 in the congested area. The peaks are due vehicles leaving the front side of the queue. Using PARAMICS's default values vehicles leaving the front side of the queue will not begin to accelerate until the vehicle in front of them is 35.41 feet (the default queue distance) away from their front bumper. The space causes vehicles to spread out into a free flow condition, preventing any queues from forming just upstream. Once the queue is clear the flow and speed increases, causing a peak, until congestion conditions reappear, causing a sudden drop in speed.

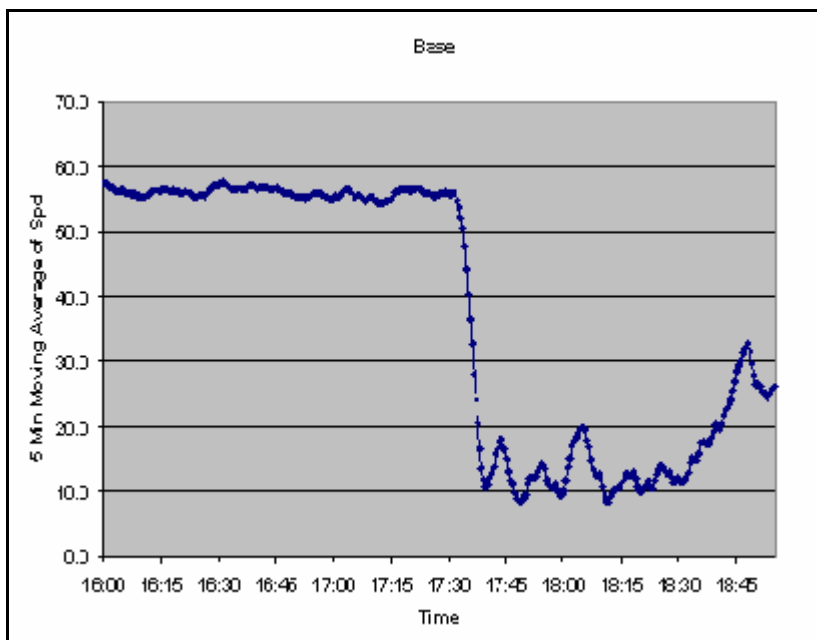


Figure 2-14: The expected shape of a speed curve when congestion occurs

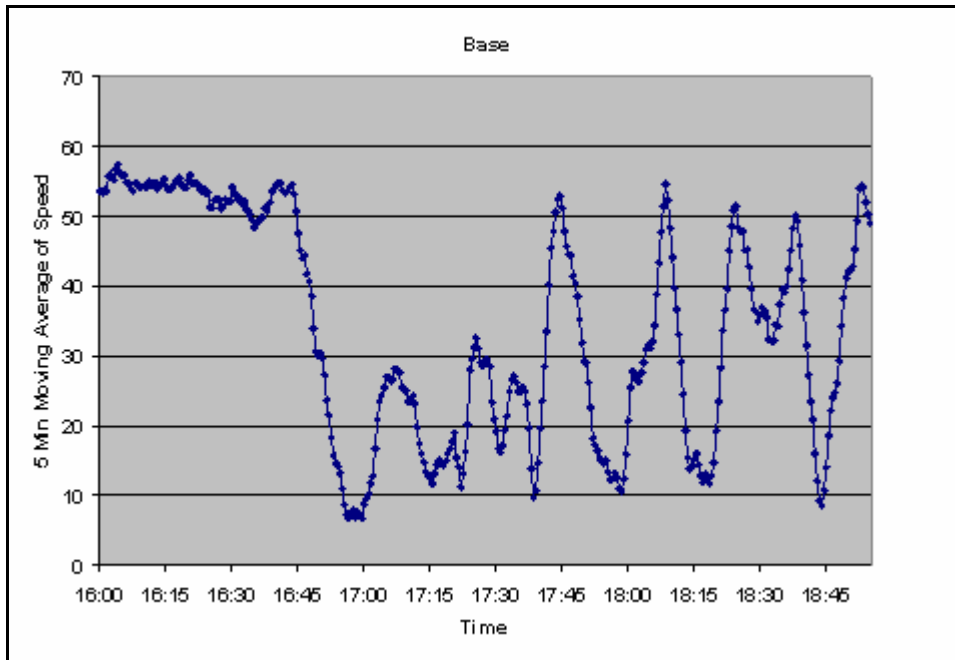


Figure 2-15: The shape of a speed curve with PARAMICS' default queue speed and distance values

A strong backward shockwave at the head of the queue is expected when a crash is cleared, or some other type of bottleneck is removed from a system, but when the bottleneck does not move and persists over the entire simulation there is no reason to expect this type of shockwave. To correct the simulation, by removing the shockwave the driver characteristics are changed and their effect determined.

Twenty-five runs of PARAMICS are inspected with different values of core parameters: target headway, mean reaction time, queuing speed, and queuing distance. Behavior characteristics aggressiveness and awareness are also tested, but no effect is seen from changing their values. The values for each run tested are shown in Table 2-2. Each runs effect on vehicle behavior is qualitatively compared in the modeler to determine a ballpark values for each parameter.

Table 2-2: Validation values inspected

Curve Speed Factor	Mean Headway	Mean Reaction Time	Queue Speed	Queue Distance	Weight Heavy
1	0.5	0.5	4.470 mph	32.810 ft	2.950 ton
0	0.5	0.5	4.470 mph	32.810 ft	2.950 ton
100	0.5	0.5	4.470 mph	32.810 ft	2.950 ton
100	0.5	0.5	4.470 mph	6.000 ft	2.950 ton
100	0.5	0.5	0.100 mph	1.000 ft	2.950 ton
100	1.65	0.5	0.100 mph	1.000 ft	2.950 ton
100	1.65	0.4	0.100 mph	1.000 ft	2.950 ton
100	1.65	0.42	0.100 mph	0.100 ft	1.000 ton
100	1.65	0.5	0.100 mph	1.000 ft	2.950 ton
100	1.2	0.5	1.000 mph	1.000 ft	2.950 ton
100	1	0.25	1.000 mph	1.000 ft	2.000 ton
100	1	0.42	4.000 mph	1.000 ft	2.000 ton

Once an idea about what the values is determined 6 cases with different values are run and their speed versus time graphs for the 36 detector locations are compared qualitatively. Table 2-3 shows a summary of the 6 cases tested. The result of the inspection of the graphs is that Case 1 and Case 2 show the best results. To determine which of these two cases will be used a final test comparing congestion location is run.

Table 2-3: Six cases inspected with their factors

	Headway	Reaction Time	Queuing Speed	Queuing Distance	Evaluation
Case 1	1.0 s	0.42 s	8 mph	9 ft	Accurate regions for West, less for East
Case 2	1.0 s	0.42 s	8 mph	10 ft	Accurate regions for West, less for East
Case 3	1.0 s	0.42 s	8 mph	7 ft	Too much middle speeds (Lack of Congested uncongested regions)
Case 4	1.2 s	0.42 s	8 mph	6 ft	Too much middle speeds (Lack of Congested uncongested regions), low speeds not low enough
Case 5	1.2 s	0.42 s	10 mph	8 ft	Too much volatility in Speed

Case 6	1.2 s	0.42 s	12 mph	10 ft	Too much middle speeds (Lack of Congested uncongested regions)
Conclusion: Cases 1 and 2 need to be inspected further to determine which should be used.					

By inspecting each of the detectors 5 minute average speeds over the three hour simulation and determining if the detector is experiencing no congestion, mild congestion, or heavy congestion and comparing the results to where congestion is expected the best core values is determined. Speeds of under 30 mph, but over 20 mph are considered to indicate mild congestion, speeds under 20 mph are considered heavy congestion and speeds over 30 mph are considered no congestion. Table 2-4 shows the qualitative analysis of the two scenarios. Case 1 slightly overestimates congestion, but is more realistic than Case 2, which underestimates congestion. Congestion locations are shown in Figure 2-16. The locations are determined by listening to morning radio reports.

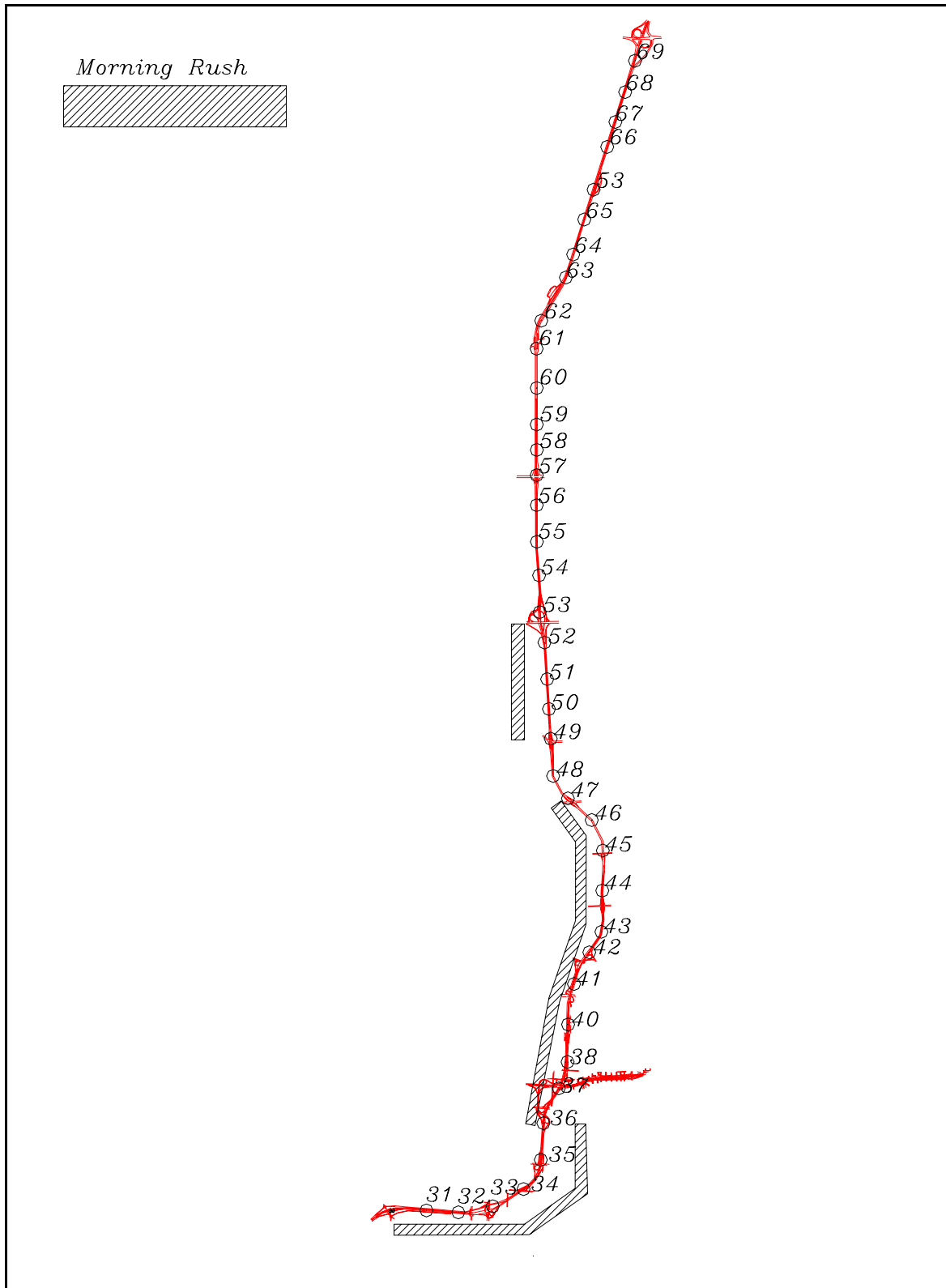


Figure 2-16: Morning congestion locations

Table 2-4: Case 1 and Case 2 inspection

Station	1		2	
	EB	WB	EB	WB
28	Mild			
29				
30				
31				
32				
33			Mild	
34				
35				
36				
37				
38				
39				
40				
41				
42			Mild	
43		Mild		Mild
44				
45			Mild	
46	Mild			
47				
48	Congested	Mild		Mild
49	Mild		Mild	
50				
51				
52		Congested	Mild	
53				
54				Congested
55				
56				
57				
58	Mild			
59				Mild
60				
61				
62				
63				
64				
65				
66				
67				
68				

2.8.5 Concluding: Which Variables are Appropriate

While the exact effect of any of the variable is not known for certain, as the author did not find any documentation of the effect in the PARAMICS User Guides, to evaluate which of the variables is plausible it is assumed that the author definitions given in the preceding paragraphs are correct. Disregarding the 24mph speeds, the lowest error rate is produced by a mean headway of .5 seconds, a mean reaction time of .5 seconds, and a queuing distance of 32.81ft. However, these values do not provide the appropriate queuing behavior. Because the end use of the simulation will be to evaluate intervehicle behavior, the values that produce the most appropriate behavior are used in the network. Those values are a mean headway of 1.0 second, a mean reaction time of .42 seconds, a queuing speed of 8 mph and a queuing distance of 9 ft. The values for mean reaction time and mean headway most closely resemble Lee's et al. values (1999) of .42 and .62. The mean headway is shorter for Lee et al. (1999) because they used the default queue distance and queue speed. As pointed out in section 2.8.2 the queue distance and the mean headway appear to interact when there the headway is small enough. A headway of 1.0 is the default offered by PARAMICS and is supported by Gardes' et al. work (2002). The literature did not comment on queue speed and queue distance values, as a result, it is assumed that they used the default values.

2.9 Implementing the Crash Risk Estimation Algorithm

PARAMICS allows a user to write code in the incidence file to induce a crash. While this method could have been adopted in order to detect crashes the runs would have had to be long enough for crashes to take place. Instead the occupancy, speed, and flow data

collected are exported into an Excel spreadsheet, which is processed using the crash detection algorithm developed by Abdel-Aty et al. (2005). After the analysis the probability of a crash is determined.

2.10 Changing Speed Limits

In a similar attempt, Lee et al. (2004) chose VMS to change speed limits. The API program for PARAMICS allowed them to write code for changing the VMS display speed in real-simulated-time. The method has strong merit, but the PARAMICS modeler user guide describes the criteria that effect drivers' adherence to VMS as being different that the criterion affects their adherence to speed limits.

In an attempt to keep the reaction of drivers' to VSL the same as to a static speed limit, a different tactic for changing speed limits is employed. PARAMICS was not designed with VSL in mind, but it was designed to look at bus routes and other forms of mass transit. As a result link properties are allowed to change over time. This feature can be used to change speed limits on roadways by changing link characteristics.

The advantage of changing the link characteristics is that drivers will respond to the new link speeds in the same way they responded to the old link speeds, keeping a uniform adherence to speed. The disadvantage is that PARAMICS allows the changes to be made only at 5 minute intervals, meaning a speed limit could not be changed 18 minutes into the simulation and then back 22 minutes into the simulation. Instead, both of the times have to be changed so that their values are multiples of 5 minutes.

2.10.1 Adjusted detector length

In modeling loop detectors in PARAMICS double loop detectors must be shown as two separate detectors. Each has its own function and speed is not calculated more accurately than using a single loop detector. Twice the output is generated, both slowing down the runtime and further complicating the data processing work after the runs are completed. To avoid these pitfalls the double loop detectors on I-4 are modeled using a single loop detector. The only problem with this approach is trying to figure out what size loops are appropriate for the job. The network is run with three possible different loop sizes in an attempt to determine the loop size that does the best job of simulating the double loop detectors. Occupancy information is selected to compare the three situations. The three sizes chosen to compare are six ft, 10 ft, and 22 ft. Six feet corresponds to an individual loop detector size. Thirty-two feet corresponds to the total length of the double loops and the distance between the loops ($6\text{ ft} + 6\text{ ft} + 10\text{ ft} = 22\text{ ft}$). Ten feet represents a value somewhere between the six feet and 22 ft and happens to be the same value as the distance between the two loops. The analysis was inconclusive. Since detectors use only one six foot loop in order to determine occupancy a six foot loop length is used.

2.10.2 Adjusted link length

Following the example set by previous work the VSL values are changed at regular intervals upstream of the loop detector that indicates a high probability of a crash. In order to allow this change to take place links within the network must end at the regular

intervals. In order to accommodate this, node locations are added just upstream of loop detectors. The detector file is then checked to insure that the detectors are not too close to the nodes. If the nodes are too close double counting could occur. In three cases the loops were too close and the detector file was changed in order to move the detector away for the node. In each case the distance moved will not affect the performance to the detector.

2.10.3 How accurate is the network

As mentioned before the error rate for the ramps in PARAMICS is approximately 3.9%. This error represents discrepancies with the FDOT's 2003 AADTs peak hour counts. The error is partly because peak numbers are used in all cases. If one ramp peaks in the morning and another in the afternoon then the simulation is comparing two different times. In any case, the error does not represent the overall error of the corridor, because the system was calibrated using ramp numbers not mainline counts. The error is actually higher. After validation the percent error of the mainline is found to be 14.19%. This error represents the overall accuracy of the network.

2.10.4 Network accuracy

An error rate of over fourteen percent is high, especially for a corridor. The error could be lowered by adjusting the OD, but this would effectively be using the mainline counts as calibration tools. Validation could not be accomplished, because there would not be any sources of information that had not already been used to calibrate the corridor. Furthermore, the interest in the corridor is with the occupancy, flows, and speeds found at

the loop detectors within the thirty-second intervals, not with the AADT produced. In order to look at the thirty second intervals it is important that the reaction of the drivers in the simulation is similar to that of drivers in the real world situation and that the overall behavior of the flow should be similar to what is seen in the real world. The verification step insures that real world behaviors are seen, and the way PARAMICS is programmed the drivers react reasonable to each others presence. Particular emphasis is placed on modeling congested conditions. Queue speed and queue distance, which have not been dealt with in the literature, are adjust fit these behaviors. Because vehicles has been taken to accurately reflect intervehicle behaviors the 14% error is assumed to be low enough to get an accurate model.

2.11 Problems with PARAMICS

While a literature review revealed that PARAMICS as the microscopic simulation package is superior in many ways, it is not without faults. In the process of working with PARAMICS some of its problems had surfaced. Most are minor, but are worth to better understand the role the software played and the possible limitations caused by the software.

2.11.1 Merging Behavior on Onramps

In PARAMICS vehicles do not merge before the end of acceleration lanes. As a result, long acceleration lanes provide little additional benefit over short acceleration lanes. Congestion occurs even with long acceleration lanes. The lengthening of the lanes only provides additional storage space for vehicles, but does not relieve congestion.

2.11.2 Overestimation of Jams

PARAMICS tends to overestimate traffic jams in networks. As an example, in another network that the author developed certain seed values would cause a vehicle to pause just north of the 528 on I-4, reducing its speed to 30mph in a 65mph zone. The result was cataclysmic. A three mile backup then follows within 15 minutes of simulated time. In the next 45 minutes of simulated time, the full length of the simulation, I-4 does not recover a quarter of a mile of uncongested highway. With other seeds, no backup occurs because no vehicle pauses. While I-4 routinely hinges on congestions one, vehicle causing so much congestion is an overestimation.

2.11.3 Overreaction of Drivers

Part of the problem with the congestion is due to overreaction by drivers. When the vehicle mentioned before paused, the vehicle behind it slow down immediately as did the vehicle behind it and so on, quickly propagating a shockwave. The problem here is how quickly the shockwave propagates. Without measuring the wave, just by viewing the simulation, it is obvious it is moving far too quickly. Drivers in the adjacent lanes react by slowing down, looking as though they are rubber necking, causing a backup in their lanes. Drivers with low aggressiveness show themselves by slowing down more and creating more backup. Soon all lanes on I-4 are backed up.

2.11.4 Difficult to Import Overlays

PARAMICS has a grid that is unlimited in size and can be used to scale of distances, but attempting to use it alone to draw by requires knowing coordinates of nodes, which

can be difficult to determine. To overcome the conflict, overlays are imported to allow for drawing references. PARAMICS allows for bmp or dxf files to be imported, which seems to include all AutoCAD files, but experience shows differently. AutoCAD files must be saved as AutoCAD R12/LT2 dxf version. Even then only information on the zero layer is imported, xref files are not imported. Small links are often lost in the process. The best solution that the author has found is to try multiple imports, with data missing on the first import comprising the second file. This multiple overlays then have to be matched up, which can be time consuming. After using the overlays they are removed from the network to improve save and refresh times.

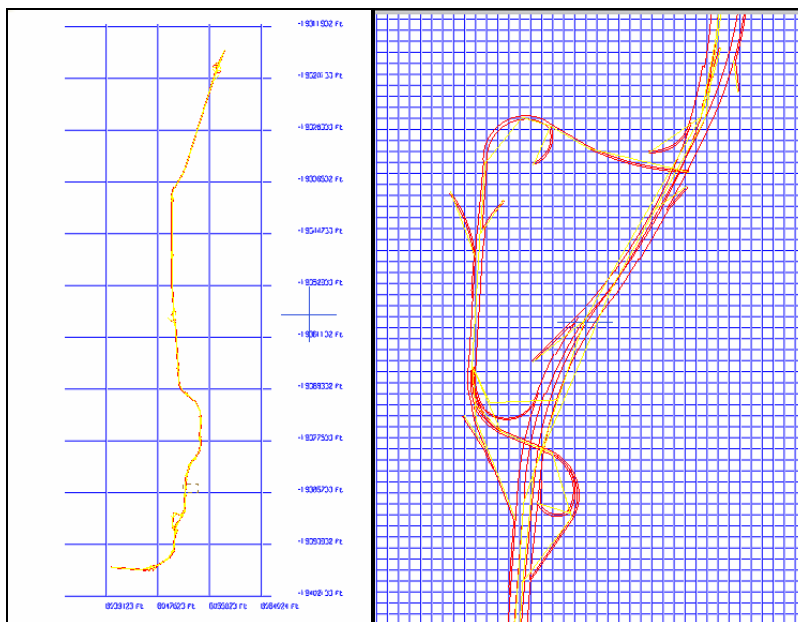


Figure 2-17: Grids at different spacing values over the I-4 network

2.11.5 Inaccurate Examples of Code

The modeler user guide and modeler reference guides are valuable resources, yet they are not without their problems. The format of files reference by PARAMICS differs slightly but significantly from the documented code. Only through multiple try and error attempts can the correct code be determined. As a result, Appendix II-B of this report contains some example files with correct coding.

2.11.6 Difficulty collecting specific loop information

The information collected by loops is either instantaneous data that needs refining or average data that is taken from a point other than the loop location, making it impossible to coordinate the to sets of data. Lee et al. (2004) overcomes some of these shortcomings by accessing loop data that PARAMICS displays in the simulation using the API interface. The data is a better grouping of the flow and speed measurements, but is not offered in the measurements toolbar. If the data is already being process by PARAMICS, it seems reasonable that this data is available to be exported as a measurement. The way the author has post-processed information using a macro requires an addition hour of computer time for each simulation run.

2.11.7 Five minute interval required on all link changes

PARAMICS is an exceedingly flexible software with measurements that can be taken every minute, OD matrixes that can be changed every five minutes, and network that can be, theoretically, infinitely big. However, more flexibility is desirable. For the purpose

of the authors work, it would be desirable to change link speeds down to the thirty-second interval. Others working with the software might desire the ability to change OD down to ever thirty seconds. Currently PARAMICS has a number of features that are updateable only in 5 minute intervals. These features are the ones that reference the profile file.

2.11.8 Unable to Specify Distributions

PARAMICS uses distributions in order to create randomness. One distribution is used to create randomness in the release rate of vehicles from origins. Data collection could possibly yield information on the type of distribution that would be desirable from a specific origin, but PARAMICS does not allow a user to specify distribution types, in its modeler program. This applies to all distributions, not just the release rate.

2.11.9 Too many u-turns, lost drivers

When modeling I-4 as part of a network and not as its own corridor, many drivers get “lost” or “miss an exit.” The behavior is meant to model a behavior that happens everyday. The problem is that certain exits are missed frequently, the I-4 and East-West Expressway interchange while heading west on I-4, and cause back up on the next exit as drivers attempt to U-turn and work their way back to the exit. Changing the awareness level of these drivers can improve the situation, but having to specify a separate OD for one exit is frustrating and time consuming.

2.11.10 No Approaches; Too Much Lane Changing Behavior at Intersections

PARAMICS does not have a function that allows for approach to intersections or toll booths to be simulated. Instead it is up to the modeler to add separate links with the appropriate number of lanes. Unfortunately, more lanes, or possibly the presence of an intersection, induce lane changes by drivers as they approach intersections. As a result backups can occur, though they are relatively infrequent. It makes for poor presentation quality. When displaying the network to clients or co-workers the behaviors sticks out and may reduce their confidence in the model.

2.11.11 No left ramps

As mentioned before left ramps are not modeled in PARAMICS. PARAMICS Support suggests using a wide entrance and wide exit option on the preceding link to approximate a ramp, but the authors attempt to use these options produced no recognizable changes to vehicle behavior.

2.12 Assumptions for preliminary evaluations of VSL strategies

In order to determine how to best reduce the probability of a crash, the variables that affect traffic flow are established and possible values of each variable are considered. The variables that are shown in the literature to affect VSL are where the signs are located, what speed limits are displayed on the signs, when the speed limits are displayed, and where among the sign locations the speed limits is displayed. To make the problem easier to simulate a few assumptions are made about the variables. To see a list of the assumptions see Table 2-5. Due to the assumptions, the total number of variables is

reduced to three: temporal variation (when the speed limit changes are displayed), spatial variation (where the speed limits are displayed), and speed limit variation (what is magnitude of speed limit is being displayed).

Table 2-5: VSL variable related assumptions

Assumptions
1. A suitable solution maybe found using only VSL.
2. The VSL attributes of PARAMICS are correct.
3. The low speed and high speed models are accurate (Abdel-Aty et al. 2005).
4. VSL are regulatory and will be followed by drivers in the same manner that speed limits are followed.
5. VSL values will be in intervals of 5mph.
6. VSL values are determined in five minute intervals.
7. VSL are distributed along I-4 every half a mile at the loop detector locations.

2.13 Statistical Model

The statistical model used (Abdel-Aty et al., 2005) is divided into two regimes: a high-speed and a low-speed. The cut off point for the two models occurs at 37.5 mph. Above this speed, a high-speed model that takes into account occupancy and flow data are used. Below this speed, a low-speed model that takes into account volume, occupancy, and speed variation data are used. The high-speed model is shown in Equation 2-2. The equation variables are made up of three parts: a function, a location, and a time. The function tells what operation to perform and the location and time are taken relative to the location and time of the location being tested for crash potential.

FUNCTIONS

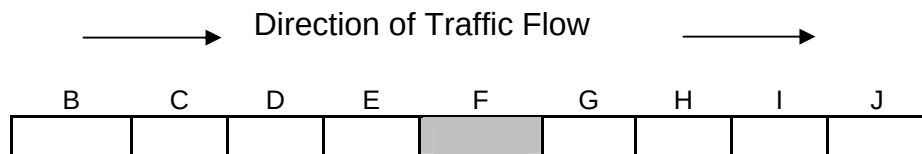
LogAO: Log of the average occupancy

SV: Standard Deviation of Volume

AV: Average Volume

LogCVS: Log of (the Standard Deviation of Speed Divided by the Average Speed)

LOCATIONS



TIMES

1: Zero to five minutes before time being tested

2: Five to ten minutes before time being tested

3: Ten to fifteen minutes before time being tested

Figure 2-18: Crash Model Definitions

The relative measures, times and location, are difficult to grasp. Station F is the station being tested. For instance imagine testing detector 33 in the westbound direction. D is the detector two stations upstream from detector 33, or detector 35. Had the direction been eastbound D would stand for detector 31. Time works similarly. If the time of interest is 6:15 then time 1 corresponds to the time 6:10. Because the model is predictive

all times are before the crash. By combining the three effects, in order, the variable of interest is determined.

$$Risk = -0.93423 \text{ LogAOF2} + 1.14584 \text{ LogAOH3} - 0.22878 \text{ SVH2} - 0.10055 \text{ AVG2} + 0.5932 \text{ AVE3}$$

...Equation 2-2

For the low-speed model the same notation is used. However an earlier model than that specified by Abdel-Aty et al. (2005) was used to first analyze the data. The reason for the change is that the low speed models were carried out before the publication of the more recent model. The model is not specific for the low speed case it incorporates the variables that show up only in the low speed case in the more recent paper. The generic model from Abdel-Aty et al. (2004) is shown in Equation 2-3. For consistency all the low speed model results that are shown in this preliminary analysis have been recalculated using the low speed model by Abdel-Aty et al. (2005), shown in Equation 2-4.

$$Risk = 1.21405 \text{ LogCVSF2} + .02466 \text{ AOG2} - .19124 \text{ SVG2}$$

...Equation 2-3

$$Risk = 2.64827 \text{ LogCVSF2} + 0.88842 \text{ LogCVSF3} + 1.33966 \text{ LogAOE2} + 0.97766 \text{ Log AOH3} - 0.43603 \text{SVF}^2$$

...Equation 2-4

The left side of Equations 2-2, 2-3, and 2-4 are the same measure, risk. Risk is the log of the odds-ratio ignoring the population adjustment factor. An odds-ratio is the probability of a crash divided by the probability of a non-crash. Abdel-Aty et al. (2004 and 2005) developed the equations to measure the relative odds-ratio. By making the odds-ratio relative, which is the difference of two corresponding odds ratios, the population parameter, which is the intercept of the regression equation, drops out. In this way Abdel-Aty does not need to solve for the population parameter. Because this population parameter is unknown the value of risk has no meaning, but the difference between risks represents a measure of crash likelihood. A positive difference means the likelihood of a crash is increasing, while negative difference means the likelihood of a crash is decreasing. Keeping in mind that the difference in two risks is the only meaningful result the graphs in the next chapter (i.e., results) may be understood.

$$\begin{aligned} \text{odds} - \text{ratio} &= \frac{P_c}{P_{nc}} = e^{(\beta_0 + \beta_1 X_1 + \beta_2 X_2 + \beta_3 X_3)} \\ \text{relative} - \text{odds} - \text{ratio} &= \frac{\frac{P_c'}{P_{nc}'}}{\frac{P_c''}{P_{nc}''}} = \frac{e^{(\beta_0' + \beta_1' X_1' + \beta_2' X_2' + \beta_3' X_3')}}{e^{(\beta_0'' + \beta_1'' X_1'' + \beta_2'' X_2'' + \beta_3'' X_3'')}} = e^{((\beta_1' - \beta_1'') X_1' + (\beta_2' - \beta_2'') X_2' + (\beta_3' - \beta_3'') X_3')} \\ \log_{\text{relative}} - \text{odds} - \text{ratio} &= \beta_1' X_1' + \beta_2' X_2' + \beta_3' X_3' - (\beta_1'' X_1'' + \beta_2'' X_2'' + \beta_3'' X_3'') \\ \log_{\text{relative}} - \text{odds} - \text{ratio} &= \text{risk}' - \text{risk}'' \end{aligned}$$

Figure 2-19: Relative log odds-ratio derivation

2.14 Network Loading

With the models being dependent on the speed of the vehicles traveling the corridor and the speed of the vehicles being dependent on the loading condition it is crucial that the loading be linked to the model type. Interstate-4 normally operates at an F Level of Service. It is common for average speeds to be below 30 mph for certain stations throughout the rush hour. With the way the OD was specified, using ramp peak hour counts, the loading errors are on the high side, even though the mainline counts are below the mainline peaks for the network. The lower mainline counts can be attributed to congestion lowering the flow. Because the counts error on the high side the peak hour scenario is assumed to have a loading of 90% of the OD matrix. This loading causes the low-speed crash risk model to be applicable since the average speed is below 37.5 mph. The assumption that the low-speed model holds is tested in the next chapter. The 90% case is henceforth called the low speed case and will have its crash potential evaluated with the low-speed algorithm (Equation 2-4).

The off-peak loading is approximated by a 60% loading of the OD matrix. The loading should cause speeds to be above the 37.5 mph mark. Therefore the 60% loading causes the high speed model to be applicable. The assumption that the high-speed model holds is tested in the next chapter. The 60% case is henceforth called the high-speed case and will have its crash potential evaluated with the high-speed algorithm (Equation 2-2).

2.15 Definitions

In order to make it easier to understand the experimental design as well as the results presented in the next chapter; a few definitions are laid out using Figure 2-20.

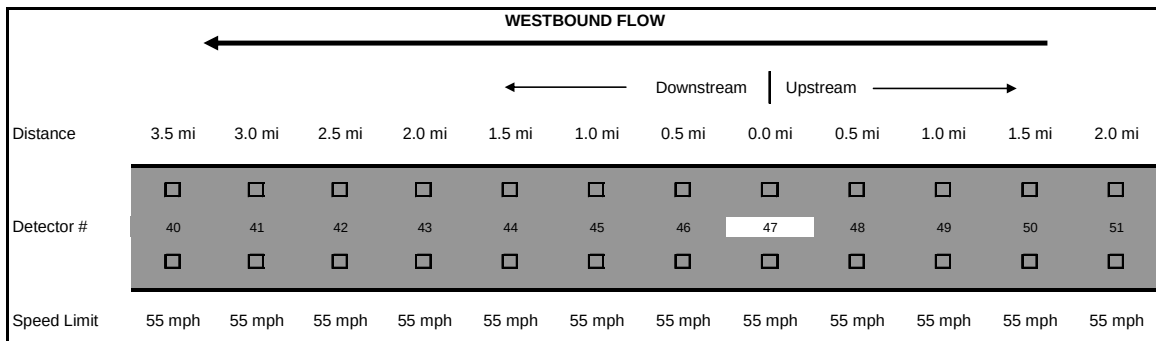


Figure 2-20: Sample westbound roadway with detector 47 as the detector of interest.

The first terms to consider are upstream and downstream. Downstream refers to detectors that a vehicle has yet to pass when at the detector of interest. Upstream refers to detectors that a vehicle has already passed when it arrives at the detector of interest. Notice how distances are measured from the detector of interest to the other detectors. Distances can be listed as either number of miles or number of detectors plus the direction. For example 1 mile upstream of detector 47 or 2 detectors upstream of detector 47 would be specifying the same location.

Notice that the default speed limit is 55 mph at all locations. This is true for the majority of the length of Interstate-4 except for the Fairbanks curve shown in Figure 2-21. The Fairbanks curve is a curve on I-4 just west of the Fairbanks exit where a restrictive

geometry brings the speed limit down to 50 mph, although the geometry is a 45 mph design.

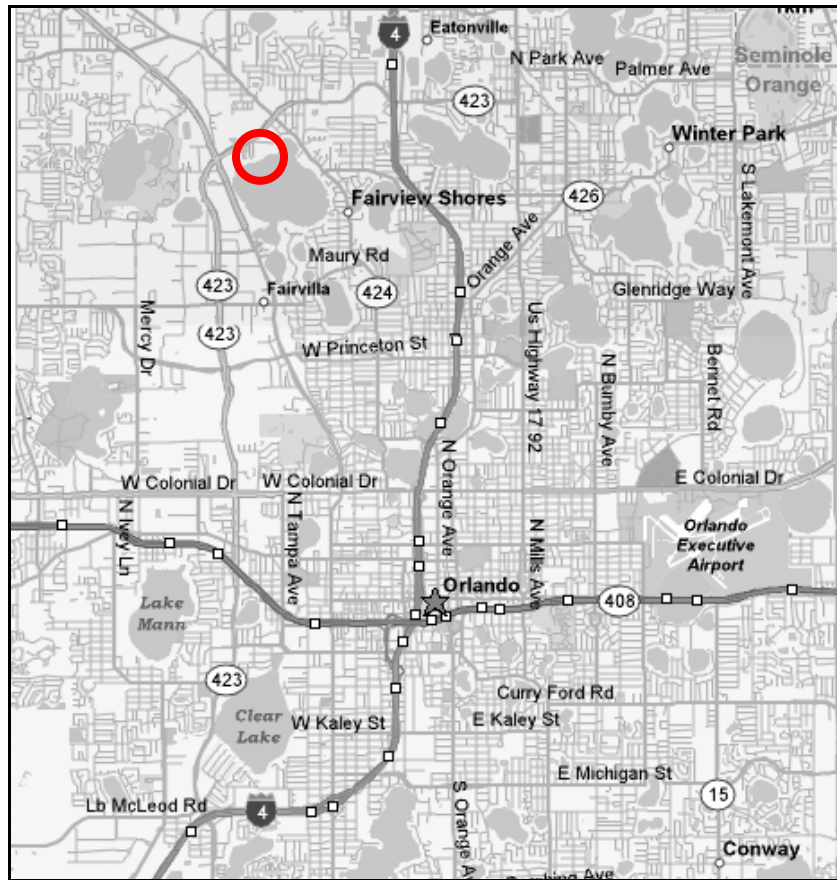


Figure 2-21: Map of Interstate-4 through Orlando.

When looking at the scenarios that are to be compared a few terms, such as the downstream raising, the upstream lowering, and the gap, will be used. Figure 2-22 shows these terms as they will be used. Upstream lowering refers to the section of roadway upstream of the detector of interest where the speed limit will be lowered in order to keep vehicles from rushing onto slower moving traffic. Notice the distance for the upstream lowering in the diagram is 1.5 miles. Downstream raising refers to the section of

roadway downstream of the detector of interest where the speed limit will be raised in order to speed up high density traffic thereby raising the capacity of the roadway. Notice the distance for the downstream raising in the diagram is 1.5 miles. Gap refers to the distance between the detector downstream of the detector of interest and the detector where the downstream raising begins. The half of a mile after the detector of interest is maintained at 55 mph in all scenarios therefore its distance is not considered. Gap is suppose allow the raised speed limits to effect the front of any queue or high density traffic instead of bumping up the speed limit where traffic is already contained by the traffic downstream of their location.

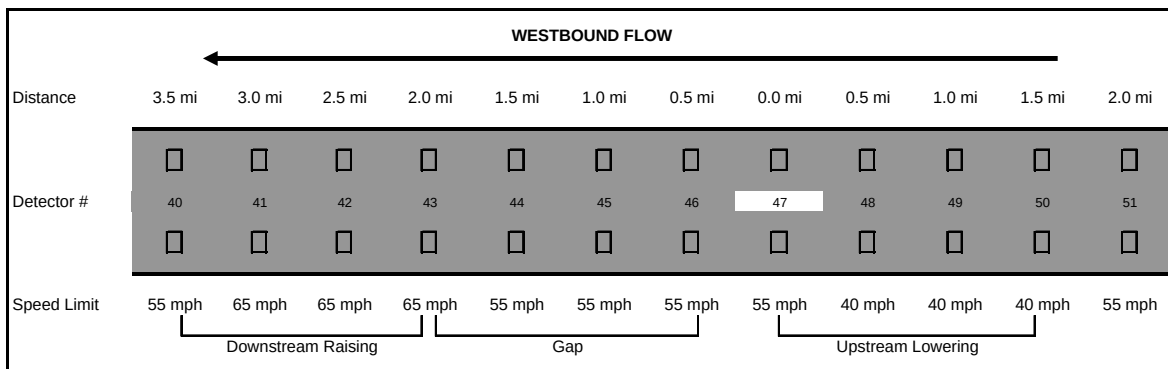


Figure 2-22: Sample scenario for a westbound roadway with detector 47 as the detector of interest.

2.16 Variables in the Scenarios

The variables that are used in the different scenarios are divided into three groups: temporal, spatial, and speed limit variation. The three groups look into possible values for each type of variation.

2.16.1 Temporal variation

The temporal variation, or the rate that the speed limit is changed, is confined to 5 mph interval changes in speed limit every five-minute intervals because the algorithm makes use of the 30-second data in five-minute intervals and PARAMICS is only capable of handling changes every five minutes. The changing speed limits before and after a potential crash will be experimented with to determine how to best prevent a crash and then reintegrate the speed limits without inducing a crash. After changing the speed limits, their values will need to return back to their default values.

2.16.2 Spatial variation

Although the spatial variation is greatly reduced by confining the location of the VSL signs, the assumption is needed to limit the number of cases. Half mile spacing serves to line up the signs with the detectors, so that the VSL line up with the loop detectors. Because PARAMICS has a factor known as sign distance (the distance before a link that a driver is aware of the links characteristics, including speed limit) drivers react before the detector and allow that behavior to be captured in the measurements. Certainly, the literature has shown that a half-mile distribution is dense, but the solution is not meant to be cost effective. Assuming that this simulation produces positive results, future research could experiment with less dense distributions of signs.

As with temporal variation, the spatial variation is comprised of two parts, preventing a crash and reintegrating the speed limit. In this case, reintegrating the speed limit consists of determining how far after the potential crash location does the roadway return

to its normal speed limit. According to Abdel-Aty et al. (2005), crashes are caused by vehicles at higher speeds (low occupancy) approaching lower speed vehicles (higher occupancy). In order to change this situation the higher speed vehicles can be given a lower speed limit, the low speed vehicles can be given a higher speed limit, or both. The allocation of these speed limits over the VSL is their spatial variation.

2.16.3 Speed limit variation

The vast majority of I-4, in the study area, operates with static speed limits of 55 mph. Reducing these speeds more than five mph maybe unrealistic, but two scenarios with greater reductions, reducing by 10 mph and by 15 mph, are tested. When raising speed limits, two scenarios are chosen: increasing the speed limit by 10 mph and increasing the speed limit by 15 mph. A 15 mph increase would result in a 70 mph speed limit in all areas except the Fairbanks curve where the speed limit would be 65 mph. As a result of safety and design considerations, any speeds higher than this cannot be supported. The speed limit change is also capped at 15mph (70mph top speed limit) due to current safety standards. On the low side a decrease of 15 mph results in a speed limit of 40mph. While this speed is low it will be considered in the model, but will be the lower bound. PARAMICS will not allow for the safe speed of a curve to be exceeded, therefore any geometric constrains will be override the specified speed limits in the links file. For example, when a speed of 65 mph is specified for the Fairbanks curve that speed limit will be dropped to 50 mph as soon as the network is opened in the program.

2.17 Scenarios for the Low Speed Model

To test the effect of VSL on the low speed model three cases are considered for temporal variation: five minutes before a crash, 10 minutes before a crash, and 15 minutes before a potential crash. For reintegrating the speed limits three time periods are considered at the potential crash time, five minutes after the potential crash, and 10 minutes after the potential crash. For spatial variation, distances of half a mile upstream, one mile upstream, and one and a half miles upstream are chosen. For reintegrating, the speed limit distances of zero miles, half a mile, and one mile are chosen. Spatial variation is limited to lowering the speed limit upstream, because of the extensive queuing downstream. It is assumed that raising the speed limit downstream would not affect flow because the vehicle will already be traveling below the speed limit. Both of the possible speed limits changes, 10 mph and 15 mph, are considered. Instead of testing all the possible combinations eight sample scenarios are picked to represent different effects. See Figures 2-23 and 2-24 for a list of the eight scenarios.

To explain the notation used in Figures 2-23 and 2-24, Case 4 is described. Looking at time t , the time where the scenario is based, the gradual implementation of speed limit changes across space can be seen. From 2 miles upstream to 1.5 miles upstream the speed is reduced by 5 mph. If the base speed were 55 mph then the speed would be 50 mph here. From 1.5 miles to 1.0 mile upstream the speed would be 45 mph, from 1.0 miles upstream to the half a mile downstream of the detector of interest the speed would be 40, and from half a mile downstream to 1.0 mile downstream the speed limit would be 50 mph. As for the implementation across time

By varying time, and holding the location constant, the gradual implementation of speed limit changes across time can be seen. From 15 minutes before to 10 minutes before the time of interest the speed limit is reduced by 5 mph at the location of interest. Following a similar logic the other values across time can be determined. The cases in Appendix II-C are expressed in the same manner.

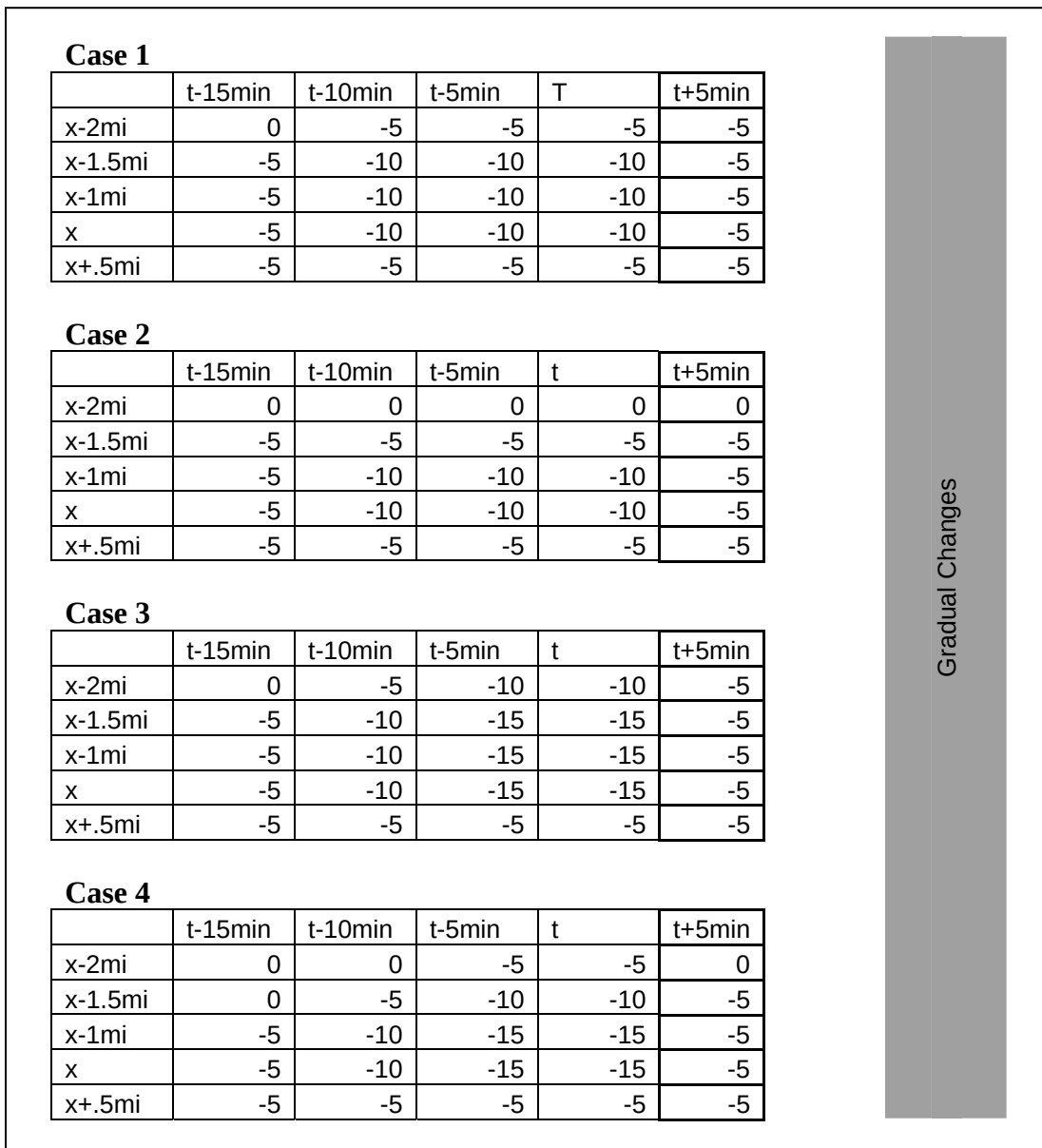


Figure 2-23: Cases 1 through 4 for the low-speed model

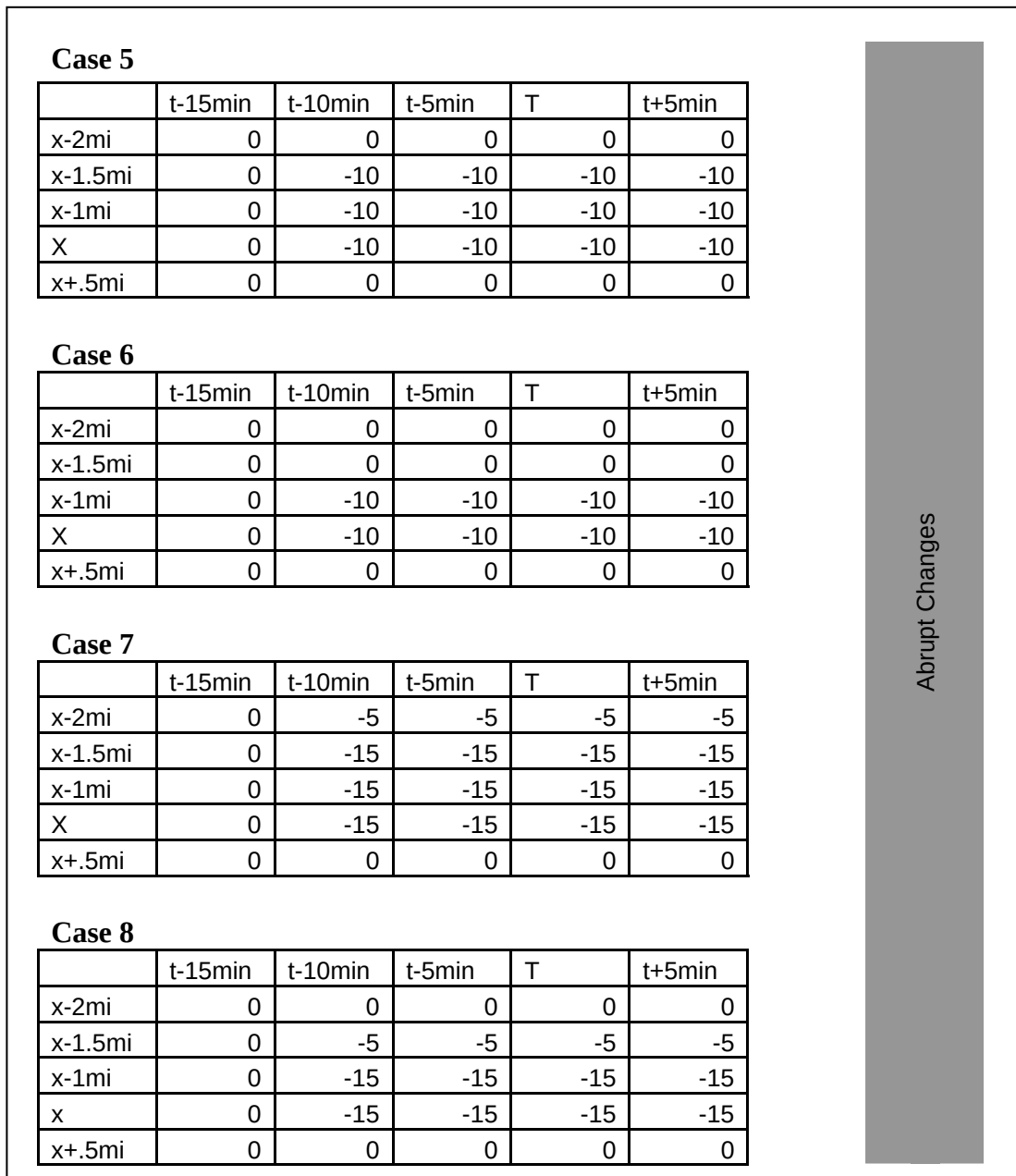


Figure 2-24: Cases 5 through 8 for the low-speed model

By comparing the cases an idea about general trends in the use of VSL can be determined. For example comparing the effect of Cases 1-4 to Cases 5-8 the effect of changing speed limits gradually over time and space versus changing abruptly over time

and space can be determined. Two other comparisons can be made using the eight cases. One, the effect of a 10 mph change in speed limit (Cases 1, 2, 5, and 6) versus a 15 mph change in speed limit (Cases 3, 4, 7, and 8). Two, the effect of a longer upstream lowering (Cases 1, 3, 5, and 7) versus a shorter upstream lowering (Cases 2, 4, 6, and 8) on crash potential. The eight cases represent a full factorial design, so the interaction of the effects may also be determined.

2.18 Scenarios for the High Speed Model

To test the high-speed model the different variables that need to be changed are assumed to act independently. As a result the effect of each may be tested while holding the others constant. In order to save time multiple effects will be considered at once when possible.

In order to start testing the effectiveness of the high speed model a base case with a problem area must first be identified. The base case for this model is I-4 from Lake Mary to Orange Blossom Trail with a 60% of full loading situation. A three-hour simulation with a 15-minute warm-up period is being used. The westbound traffic at detector 47 is showing a high crash potential and is the detector whose crash potential should be improved. In order to see the effect of crash potential changing in time, thirty minutes into the simulation speed limits will start to change and will be maintained for only 30 minutes. After that time the speed limits will come back to their base value. In this way if the VSL cause a negative effect after their implementation then it will be observed in the model.

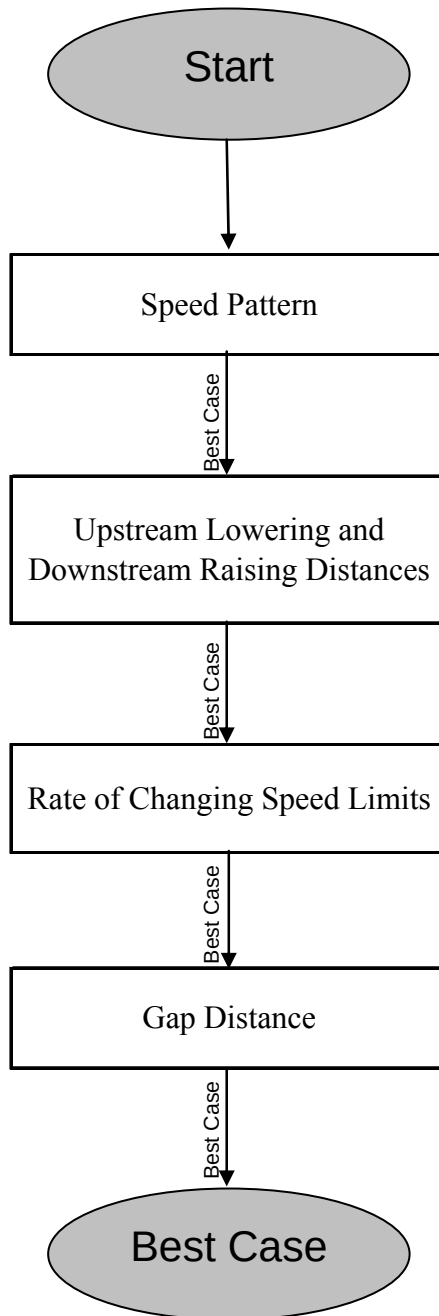


Figure 2-25: Tests that are to follow for the high-speed model.

In order to test the various scenarios for the high-speed model four separate tests are conducted with the best scenario from the previous scenario being used as the input for the next test. To see the layout of the test see Figure 2-25. The tests that are referenced in the figure are shown in Tables 2-6, 2-7, 2-8, and 2-9, respectively.

The first variables that are considered are the spatial variation of various speed limits (whether upstream lowering the speed limits and downstream raising speed limits is better than just downstream raising speed limits), speed limit variation (whether a 10 mph maximum change is better than a 15 mph speed limit change), and the combination of the two variables (whether gradually changing the speed limit over space or abruptly changing the speed limit over space is better). A summary chart of the different scenarios is shown in Table 2-6.

Table 2-6: Different scenarios based on changing speed limit patterns

	Distance of Change	Amount of Spd. Change	Where Changed
Case 1	Abrupt	10 mph	Up and Downstream
Case 2	Abrupt	15 mph	Up and Downstream
Case 3	Gradual	10 mph	Up and Downstream
Case 4	Gradual	15 mph	Up and Downstream
Case 5	Gradual	10 mph	Downstream
Case 6	Gradual	15 mph	Downstream

The second variable tested is the spatial location of changing speed limit signs. In this test the best case from the first test is used and the location of the change in speed limit is altered to see its effect. If the upstream case is found not to have an effect from the first test then the number of cases will be reduced to three by not have an upstream variable to change. See Table 2-7 for the scenarios.

Table 2-7: Different scenarios based on changing distance for speed limit change

	Upstream Lowering Distance	Downstream Raising Distance
Case 1	4 mi	5.5 mi
Case 2	6 mi	5.5 mi
Case 3	2 mi	5.5 mi
Case 4	4 mi	4 mi
Case 5	6 mi	4 mi
Case 6	2 mi	4 mi
Case 7	4 mi	2 mi
Case 8	6 mi	2 mi
Case 9	2 mi	2 mi

The third variable test is the temporal implementation of the VSL on the best case scenario from test 2. The first case that is considered is abruptly changing from the standard speed limits to the new speed limits. The other cases involve changing the speed limit stepwise from the normal speed limit to the crash prevention speed limit. The step size varies by the time period between changes and the value of the speed. The speed limits can be stepped either every five minutes or every 10 minutes and the speed can change by 5 mph or 10 mph. As with the second test, if the scenario from the earlier tests makes one or more of the cases irrelevant then the case will be disregarded. See Table 2-8 for the scenarios.

Table 2-8: Different scenarios based on changing speed limits over time

	Time Between Changes	Amount of Change
Case 1	Abrupt	Abrupt
Case 2	5 min	5 & 10 mph
Case 3	10 min	5 & 10 mph
Case 4	5 min	10 mph
Case 5	10 min	10 mph
Case 6	5 min	5 mph

The forth variable test is the gap distance, after the detector of high incidence, before raising the speed limit. In this test it is assumed that in order to affect the high density traffic the front of the high density needs run faster than the back, thereby increase the distance that the vehicles occupy, lowering their density. To test this 4 values of the gap are looked into. See Table 2-9 for the scenarios.

Table 2-9: Different scenarios based on the changing gap after the detector of interest before raising the speed limit.

	Gap Distance
Case 1	0 mi
Case 2	1 mi
Case 3	2 mi
Case 4	3 mi

To get an idea about what a scenarios might look like Figure 2-26 shows the zero gap case in a graphical manner different manner.

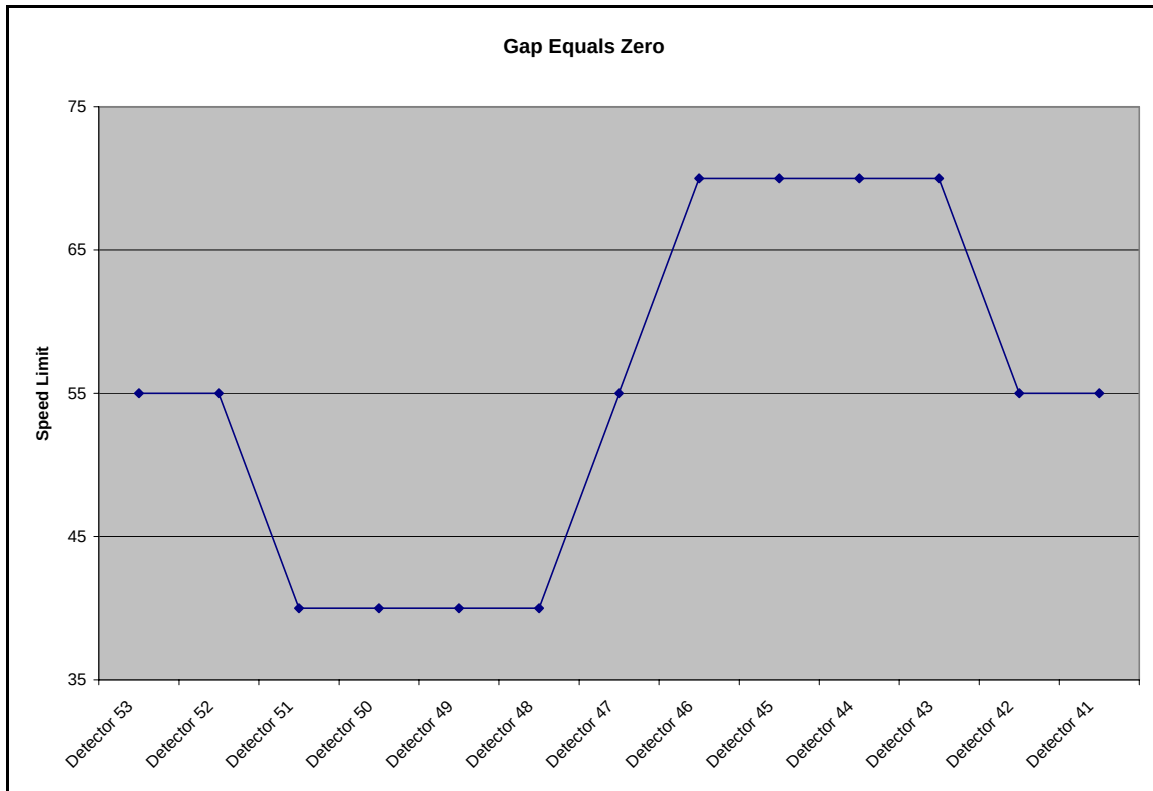


Figure 2-26: Scenario with zero gap and 15 mph abrupt change in speed limit (time held constant)

2.19 Ramp Metering Application

If after the use of VSL on the high speed and low speed models no case shows positive results then ramp metering with VSL implementation is tested. Ramp metering is not to be applied using the API included with PARAMICS' programmer, but is coded by a more brute force method. Because the effect of ramp metering is to clam traffic peaks the effect can be over exaggerated by reducing the ramp counts at specific locations. The high flow periods probability of occurring will be greatly reduced, causing a ramp-metering-like effect. Because this method overestimates, the effect of ramp metering, the API should be implemented upon a successful result.

2.20 Statistical Significance Testing

Once best scenarios for the high-speed and low-speed models are determined from the statistically significant effect of the implementation is determined. Twenty replications of the base case and the VSL implemented case are run. The average crash potential from 16:20 to 17:15 is then calculated for each of the forty runs. The 95% confidence interval for the two cases at each detector is then determined. Finally, the effect is concluded by determining if the confidence intervals overlap.

2.21 Travel Time Analysis

The last step before a scenario can be successfully recommended is a travel time analysis. Twenty replications of the base case and the VSL implemented case are run. The average travel time from 16:30 to 17:30 is then calculated for each of the forty runs. The 95% confidence interval for the two cases. Finally, the effect is concluded by determining if the confidence intervals overlap.

CHAPTER 3

PRELIMINARY EVALUATIONS OF VSL STRATEGIES: SIMULATION RUNS, RESULTS, AND CONCLUSIONS

The main goal of this analysis is to find VSL implementations that successfully lower crash risk on I-4 in Orlando, FL. I-4 is the major interstate in Orlando and runs North-

South (East and West on the roadway) through the center of the city. It is equipped with dual loop detectors every half of a mile. These detectors output information for the simulated model are collected in the same way that the real-world data are gathered.

3.1 Model Applicability

Abdel-Aty et al. (2005) (also see last section of Chapter 4 — Volume I in this report) showed two different models that used traffic measurements to assess real-time crash risks. Their models output a factor called crash potential which when high, indicates that a crash is likely to occur and when low means that a crash is unlikely to occur. The low-speed model is based on speed below 37.5 mph and the high-speed model is based on speeds above 37.5 mph. To verify that the models are being properly utilized, the speed profiles for the detectors of interest are shown.

3.1.1 Low-speed model

Detector 33 in the eastbound direction, with 90% loading, shows that when congestion occurs, the low-speed model is the correct model to use. Keep in mind that the base 100% case incorporates peak ramp counts, so both directions are peaking at the same time. By using a 90% case the loading is more similar to true peak hour conditions. Congestion sets in at 17:00. To see the outputted speeds, see Figure 3-1. Detector 61 in the westbound direction, with 90% loading, shows a speed below the 37.5 mph threshold through the entire simulation. To see the outputted speeds for detector 61, see Figure 3-2. These speeds validate the application of the low-speed model, Equation 3-1, because their five-minute moving average of speeds are below the 37.5 mph threshold. Note that Detector 61 is east of downtown, while Detector 33 is just west of downtown. The five-

minute moving average of speeds is an average of ten consecutive 30-second values of average speed. Every thirty second a new average is computed, effectively sliding the 5-minute average along with time, or moving the average.

$$\text{Risk} = 2.64827 \text{ LogCVSF2} + 0.88842 \text{ LogCVSF3} + 1.33966 \text{ LogAOE2} + 0.97766 \text{ Log AOH3} - 0.43603 \text{ SVF}^2$$

...Equation 3-1

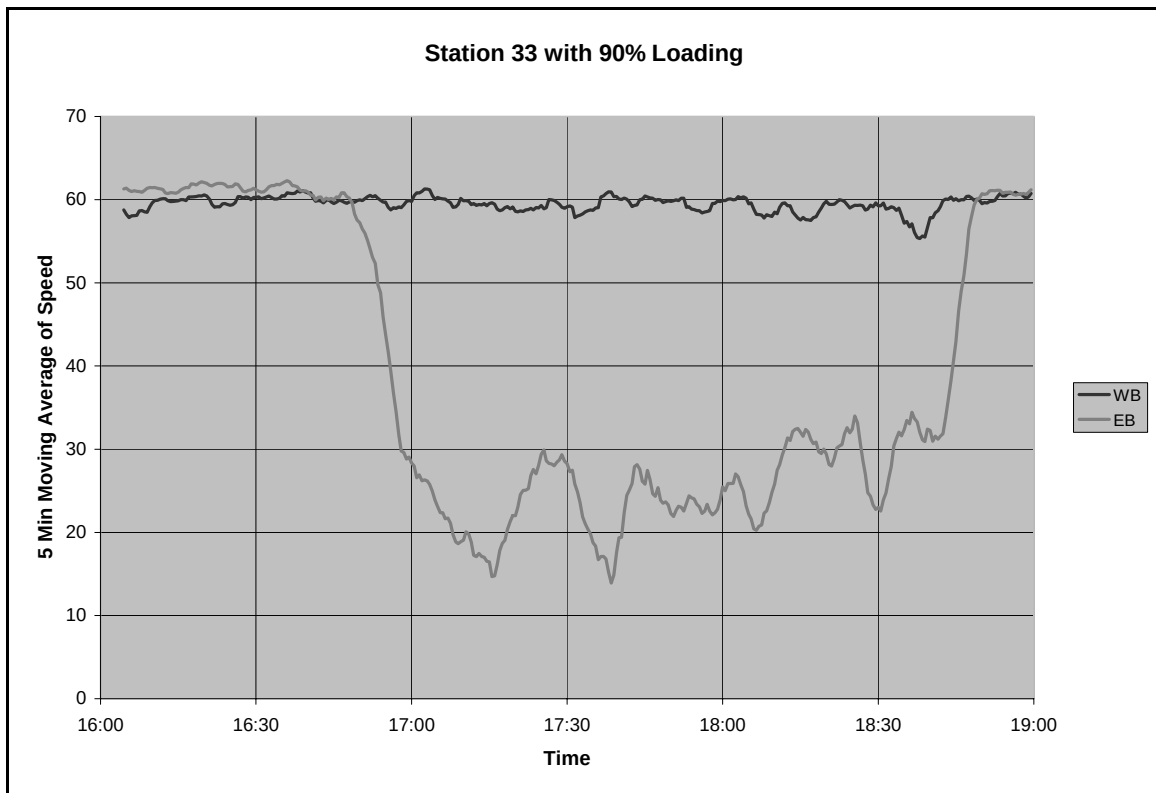


Figure 3-1: Station 33 speed profile through out the simulation period.

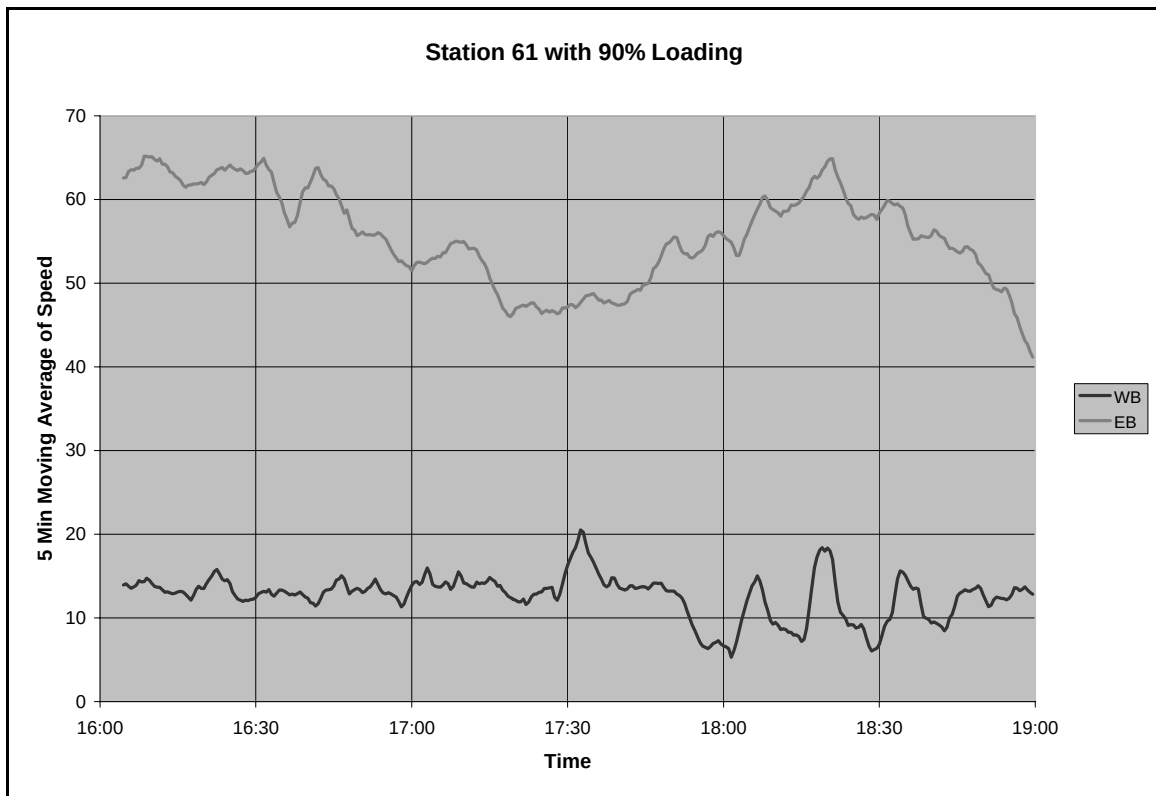


Figure 3-2: Station 61 speed profile through out the simulation period.

3.1.2 High-speed model

Detectors 61 and 47 are used in order to perform the high-speed implementation of variable speed limits. With a 60% loading, both stations show a speed of well over 37.5 mph for the entire simulation period. See Figure 3-3 and Figure 3-4 for the speed profiles.

$$Risk = -0.93423 \text{ LogAOF2} + 1.14584 \text{ LogAOH3} - 0.22878 \text{ SVH2} - 0.10055 \text{ AVG2} + 0.5932 \text{ AVE3} \quad (5-2)$$

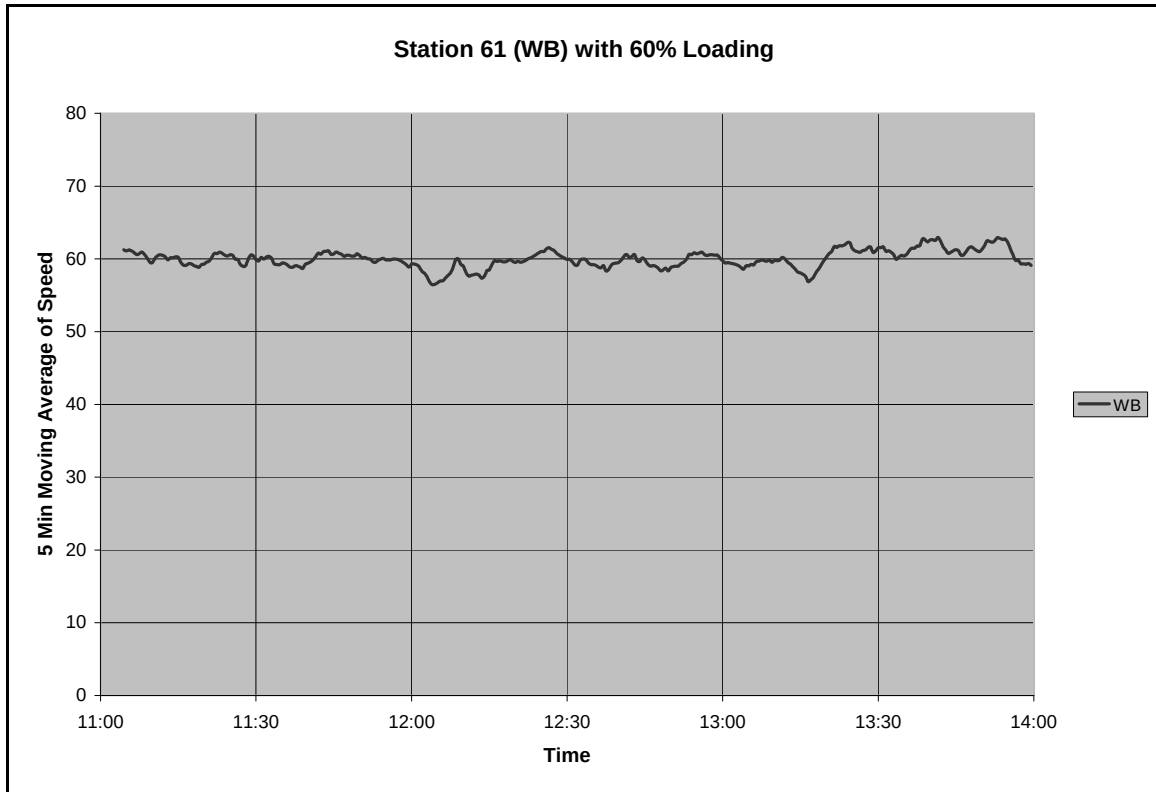


Figure 3-3: Station 61 speed profile through out the simulation period.

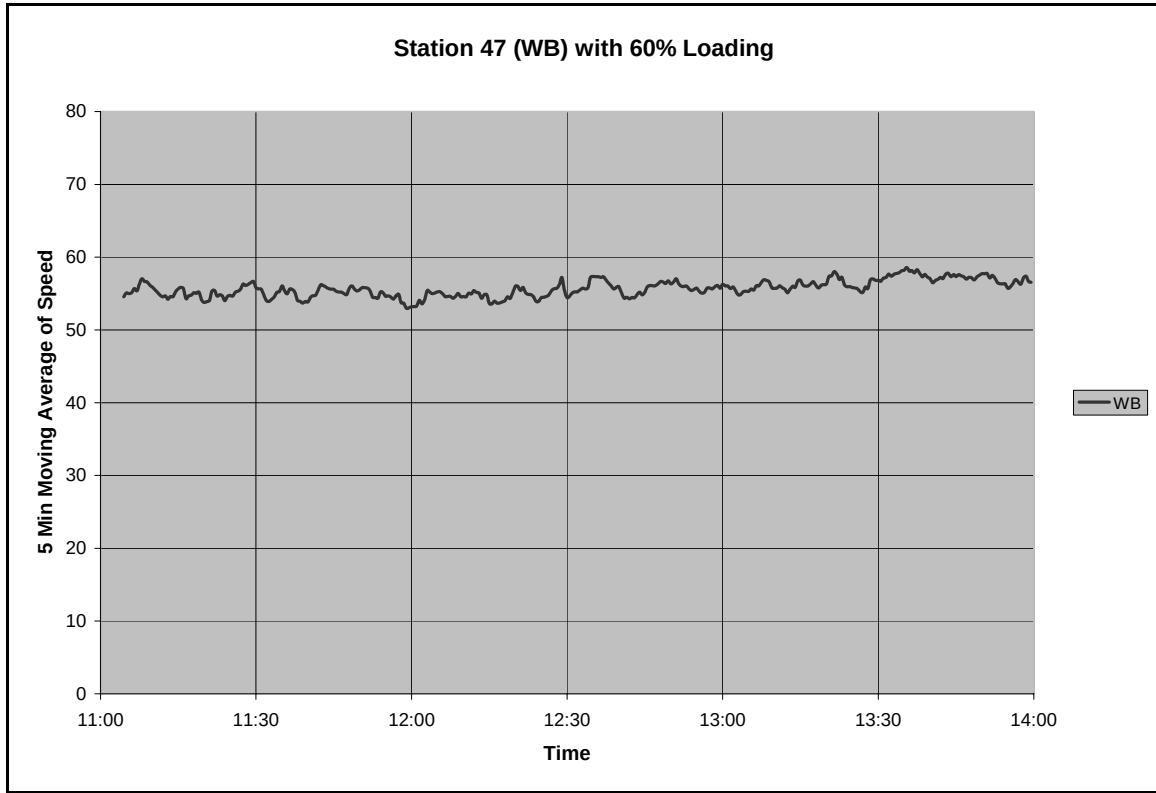


Figure 3-4: Station 61 speed profile through out the simulation period.

3.1.3 Speed limits changing

In implementing variable speed limits, in both the high speed and low speed models, a different technique than the one suggested by PARAMICS (Programmer) is used. Instead of implementing the change in speed limits using a C++ program, different links files are used. PARAMICS writes to multiple ascii files instead of one central file when saving. One of these files is the links file. The links file keeps all information about the links on the network, including speed limits. The model's profile file, another ascii file,

is then changed to specify which links files are referenced based on the simulated time. To insure that the speed limits are changing properly, vehicle speed, at a location where the speed limit is supposed to drop, is checked. Figure 3-5 shows the effect of changing the speed limit and proves that a change is taking place. Cases 1 and 2 are two different versions of the networks where the speed is changed at 16:30 using VSL in the 2nd and third links file, respectively. The two cases are shown to be effective in Figure 3-5.

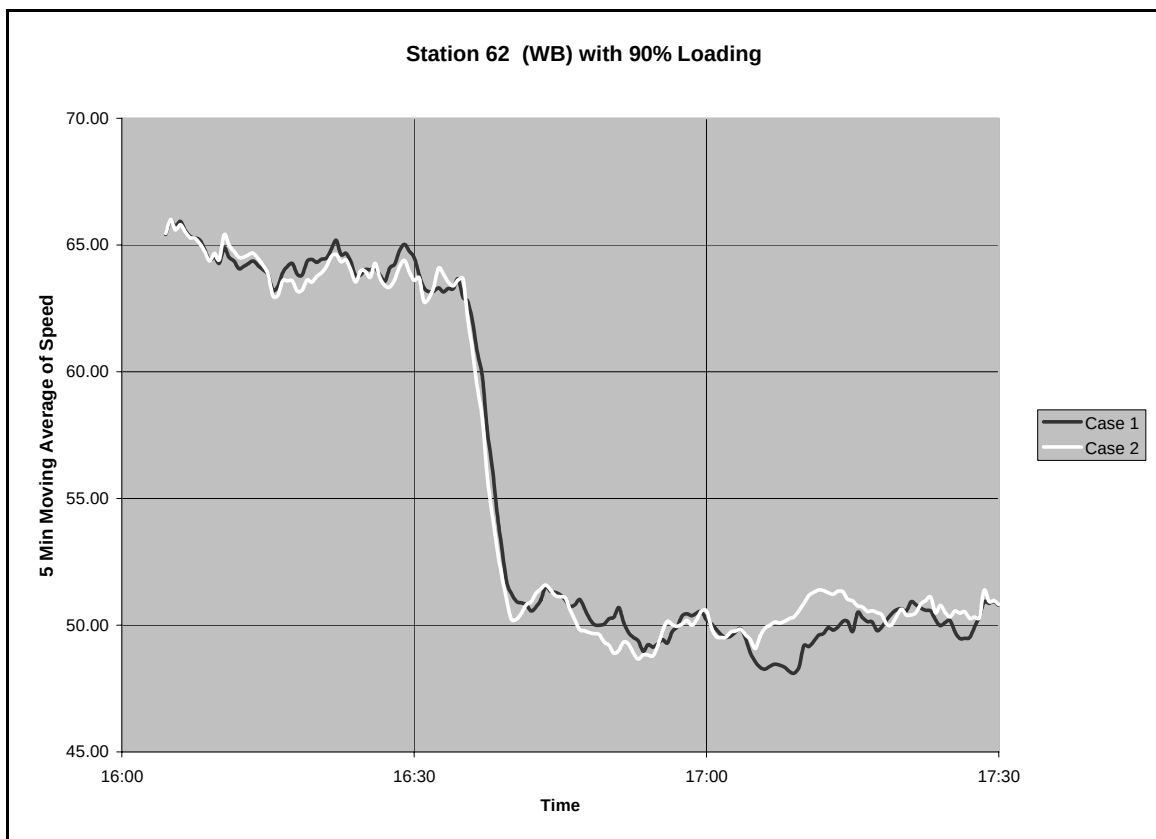


Figure 3-5: Station 62 speed profile through out the simulation period.

3.2 Model Application

3.2.1 Low-speed model

After showing that the low-speed model is applicable to detectors 33 and 61, when 90% loading is used, the next step is to apply the low-speed algorithm to see what kind of crash potentials are present. Figure 3-6 shows that as congestion begins to take hold of detector 33 (as shown in the speed profile, Figure 3-1) there is no impact on crash potential. Variable speed limits (VSL) can be applied before and during congestion and their effect on crash potential can be assessed. VSL will be implemented at 16:30 and 17:00 to see the effect of both. Figure 3-7 shows a different scenario, at detector 61, in which congestion is persistent and the crash potential (calculated by Equation 2-4) is relatively high, compared to station 33, throughout the simulation. By applying VSL here, their behavior with an already congested section can be determined.

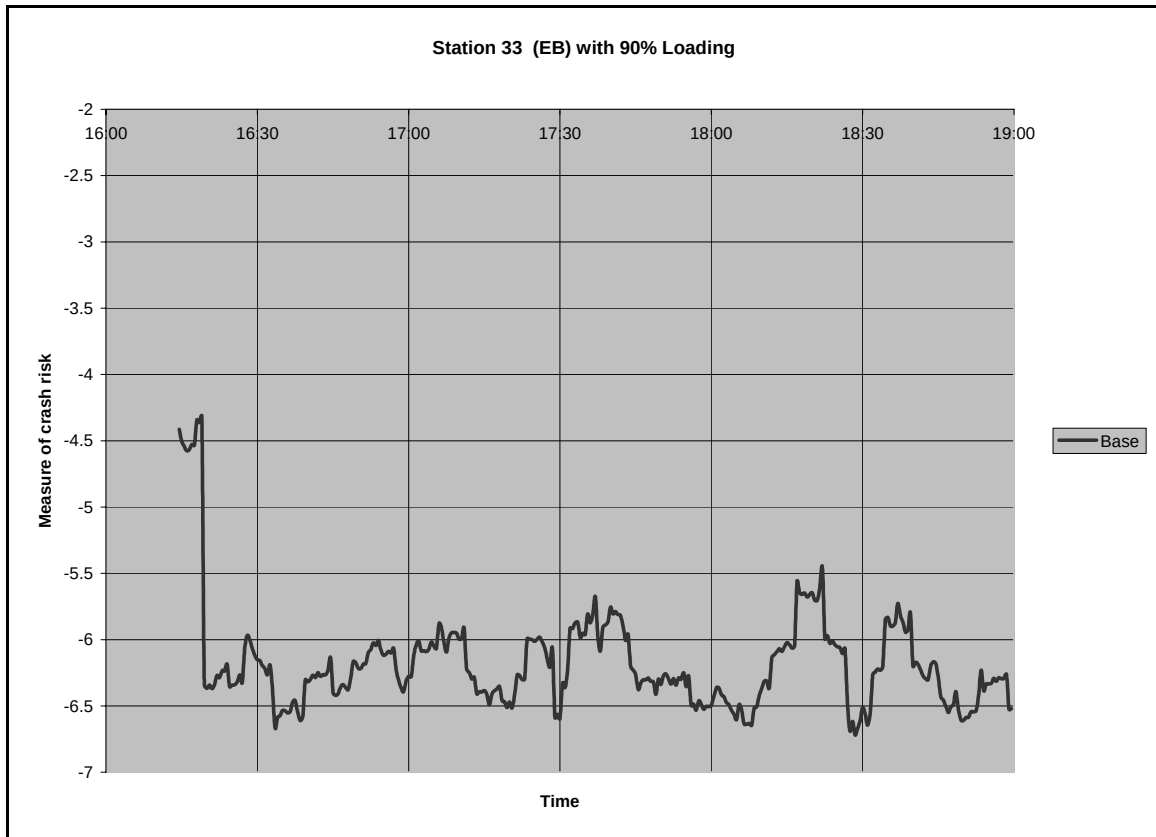


Figure 3-6: Station 33 crash potential through out the simulation period.

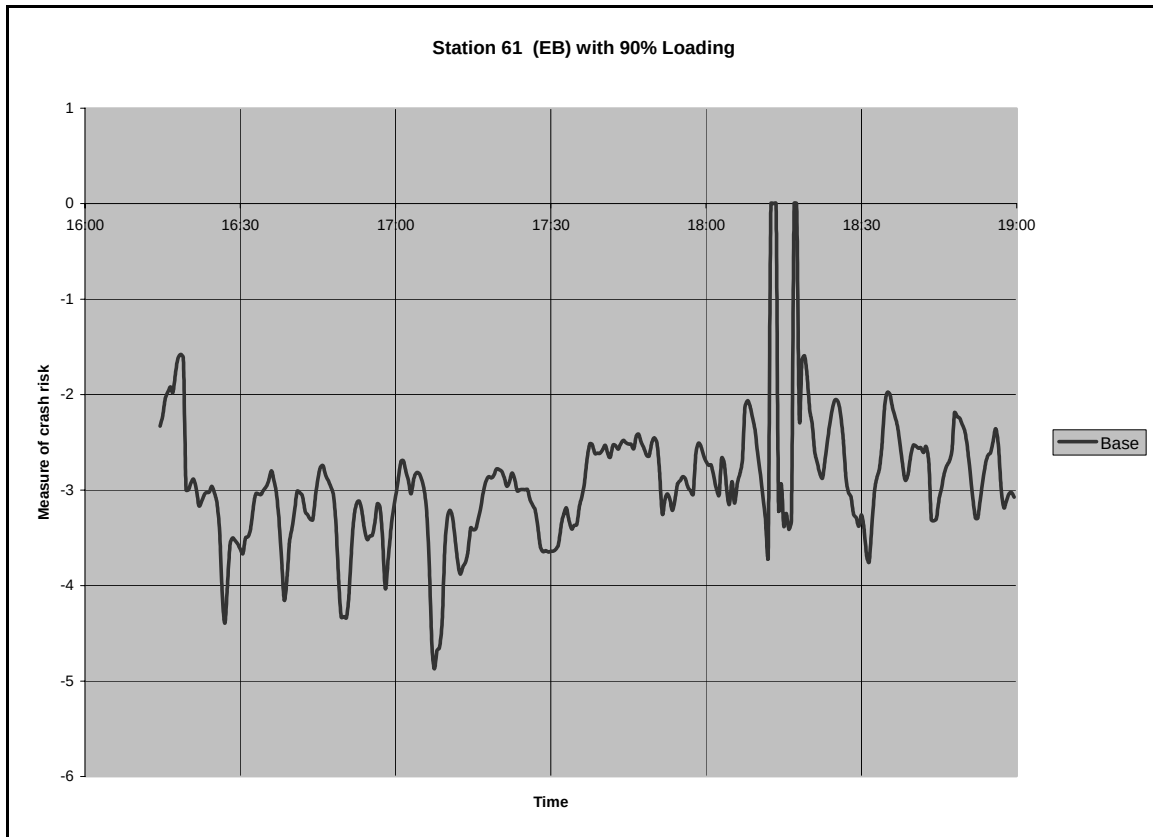


Figure 3-7: Station 61 crash potential through out the simulation period.

3.2.2 High-speed model

The high-speed model detector 61, with a 60% loading, displays the effects of a low crash potential section. Detector 47, with a 60% loading, displays the effects of a high crash potential area. Detector 61 originally was chosen because of a sharp spike in crash potential at 16:45, but after working with the simulation it is determined that the spike is caused by improperly drawn geometry. Figure 3-8 shows the corrected crash potential diagram for detector 61. Figure 3-9 shows the crash potential calculated for detector 47 based on Equation 2-2.

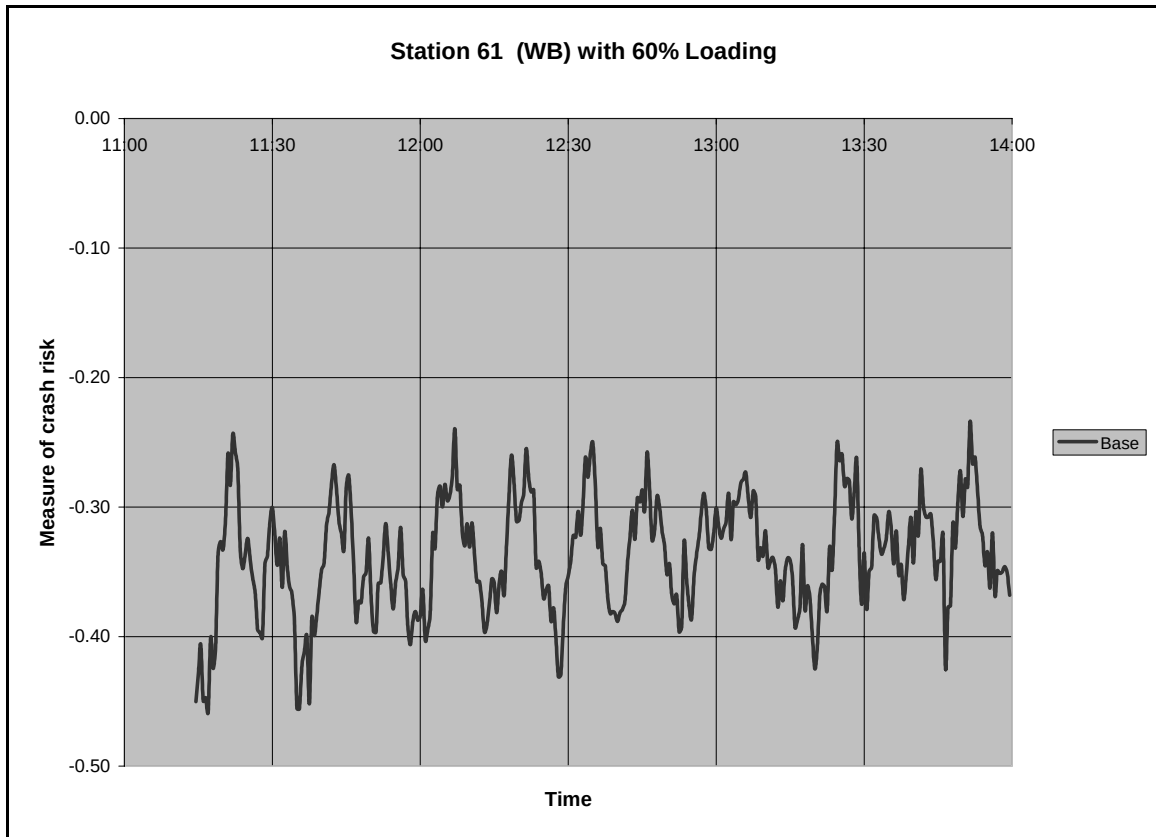


Figure 3-8: Station 61 crash potential through out the simulation period.

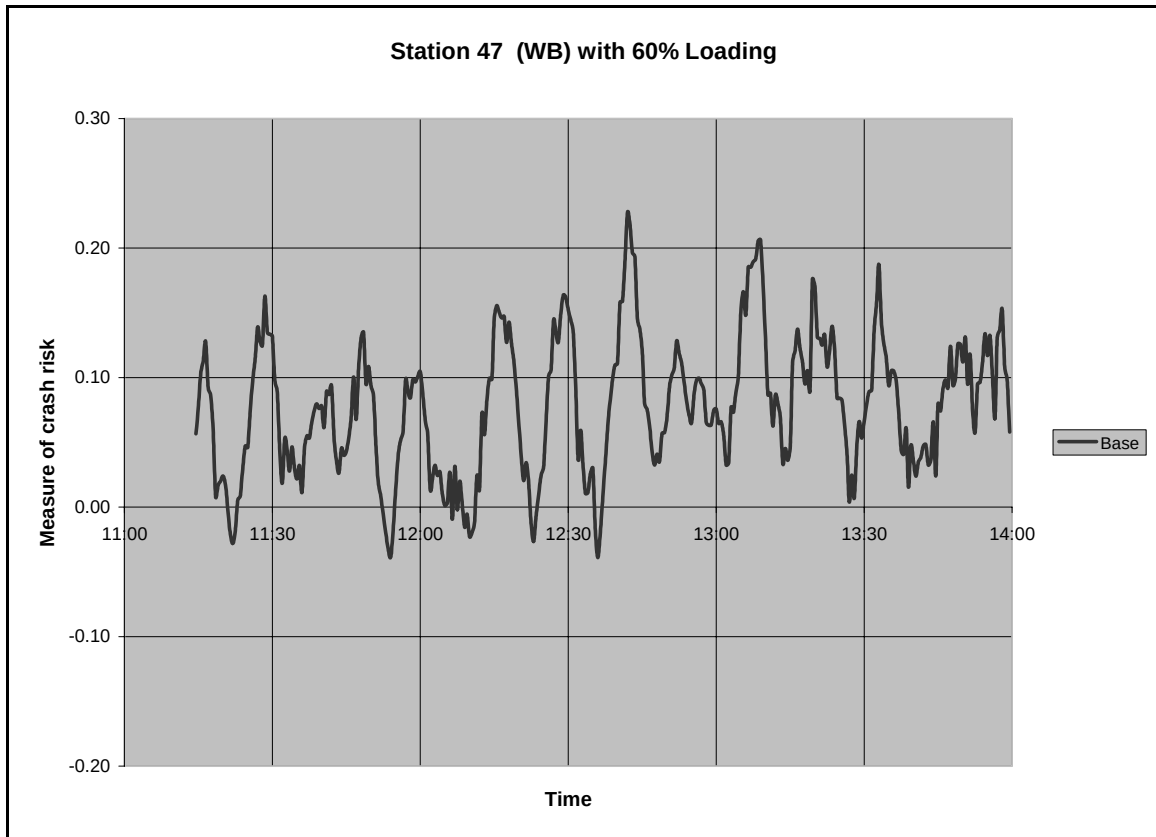


Figure 3-9: Station 47 crash potential through out the simulation period.

3.3 Low-Speed Model after VSL Implementation

3.3.1 Station 33

VSL are implemented using an upstream lowering technique. One of the scenarios is tested on each detector to see if the system is affected. If no effect is seen then the other cases are not attempted. The scenario chosen is shown below in Table 3-1. The scenario shows a gradual implementation in both time and space.

Table 3-1: The test scenario for the low speed model.

	t-15min	t-10min	t-5min	T	t+5min
x-2mi	0	0	-5	-5	0
x-1.5mi	0	-5	-10	-10	-5
x-1mi	-5	-10	-15	-15	-5
x	-5	-10	-15	-15	-5
x+.5mi	-5	-5	-5	-5	-5

Detector 33's crash potential curve shows no significant difference after the implementation of VSL, as shown in Figure 3-10, where Case 1 is the scenario in Table 3-1. The difference between the two curves is attributed to randomness in the simulator.

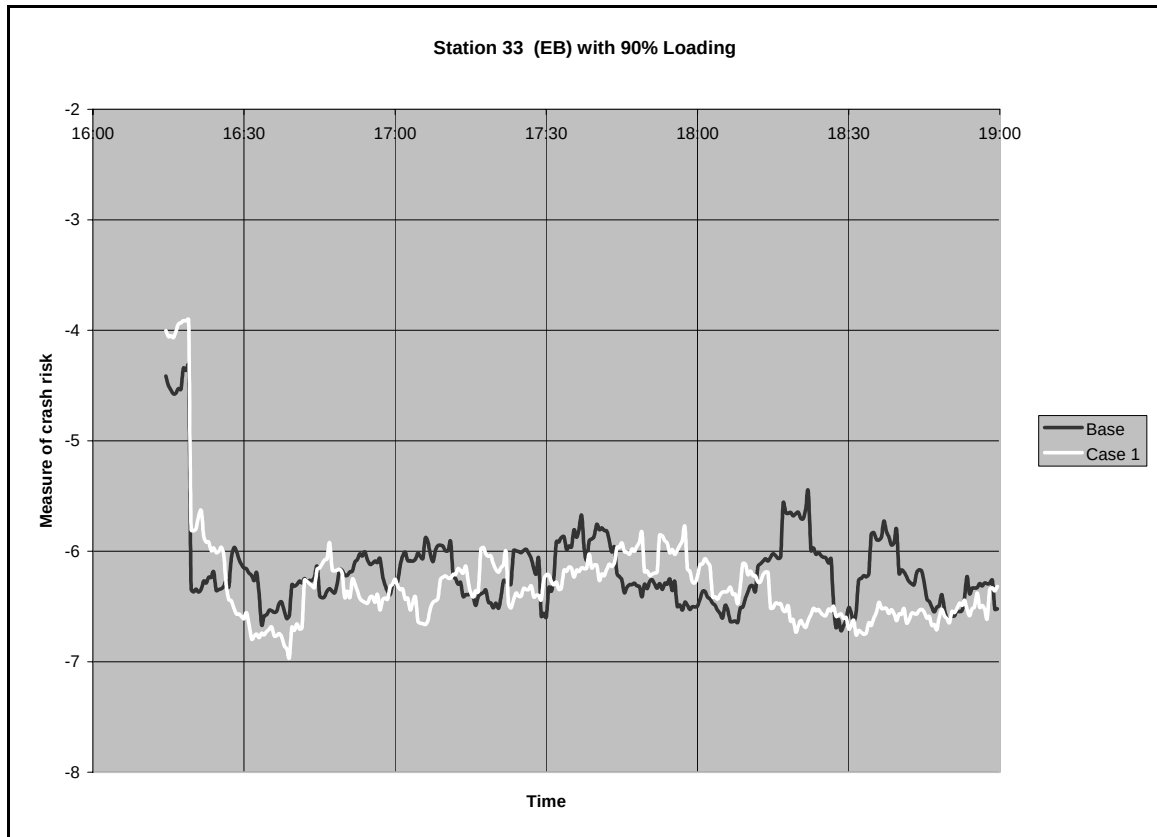


Figure 3-10: Station 33 crash potential for the base case and VSL implementation cases.

3.3.2 Station 61

VSL are implemented using an upstream lowering technique. Again only one of the scenarios (from Appendix II-C) is tested on each detector to see if the system is affected. The same low speed scenario, shown in Table 3-1, is used.

Detector 61's crash potential curve shows no significant difference after the implementation of VSL, as shown in Figure 3-11. As a result, the other cases are not implemented.

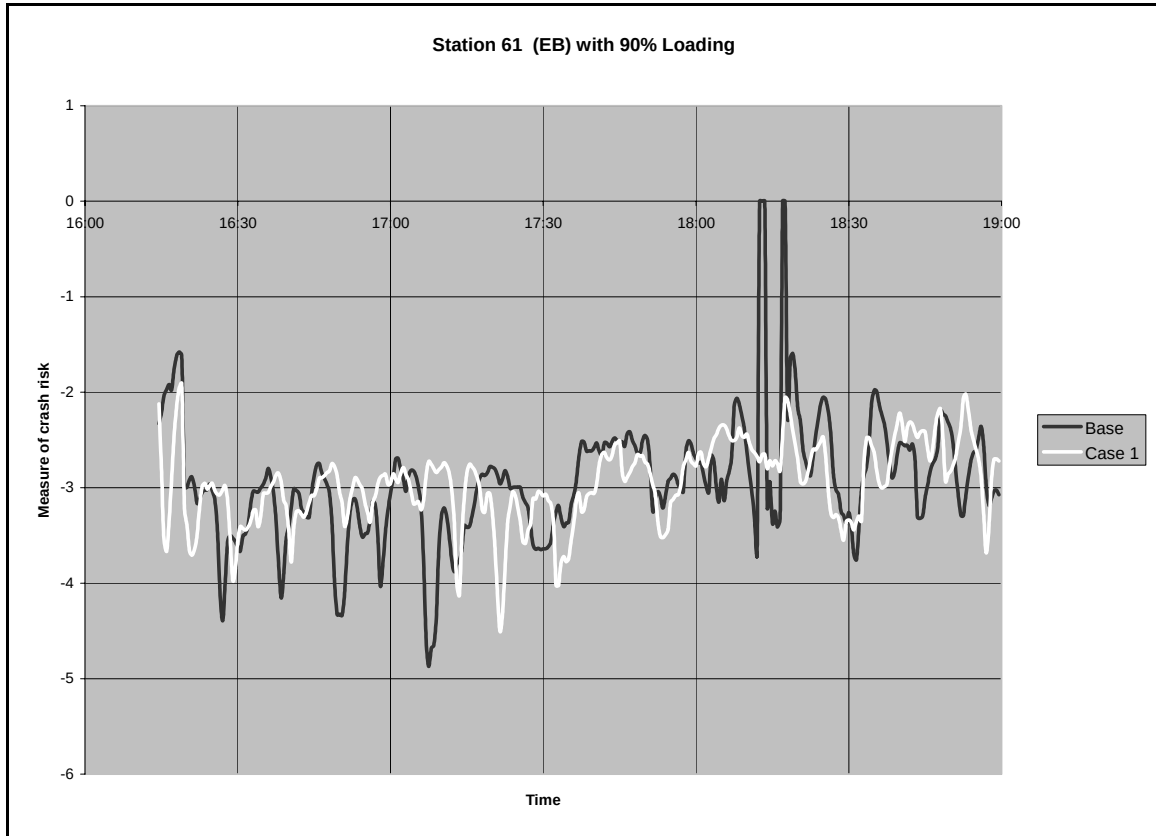


Figure 3-11: Station 61 crash potential for the base case and VSL implementation cases.

3.4 High-Speed Model after VSL Implementation

3.4.1 Station 61

Station 61 is rerun with the 60% loading using all 8 cases, making it the high-speed model. As shown in Figure 3-12, the application of VSL when using Case 4 shows a strong reduction in crash potential at detector 61, half a mile upstream from the detector where the changes were centered. The other cases do not show a significant difference from the base case, which is not expected. For example, Case 2 and Case 4 are similar

cases, so if Case 4 dramatically improves the situation, Case 2 is at least expected to have an effect. For a look at the case refer to Figures 3-13 and 3-14.

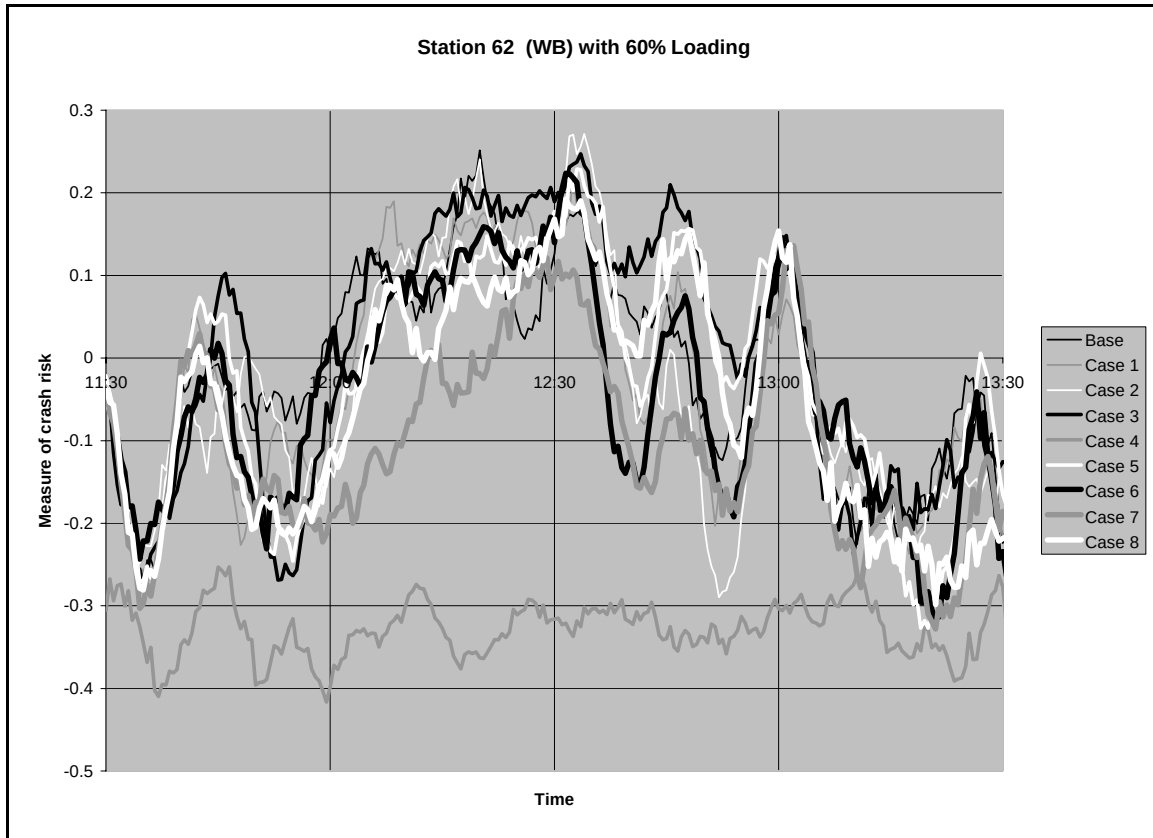


Figure 3-12: Station 62 crash potentials for the base case and upstream VSL implementation cases.

To understand why Case 4 is the only case to improve safety, a speed plot is taken. Figure 3-15 shows the speed diagram at detector 64. Detector 64, which is 1.5 miles upstream of Detector 61, is used because it has the most dramatic difference in speeds and best illustrates what the problem is. When Case 4 is run, there is a different release rate. The release rate is such that congestion occurs before the detector of interest. The

congestion limits the speeds and intervehicle interaction downstream, causing a complete different vehicle behavior. The case has been rerun several times and each time the exact same result is obtained in Case 4, but is not seen in any other case. The cause of the problem is not determined.

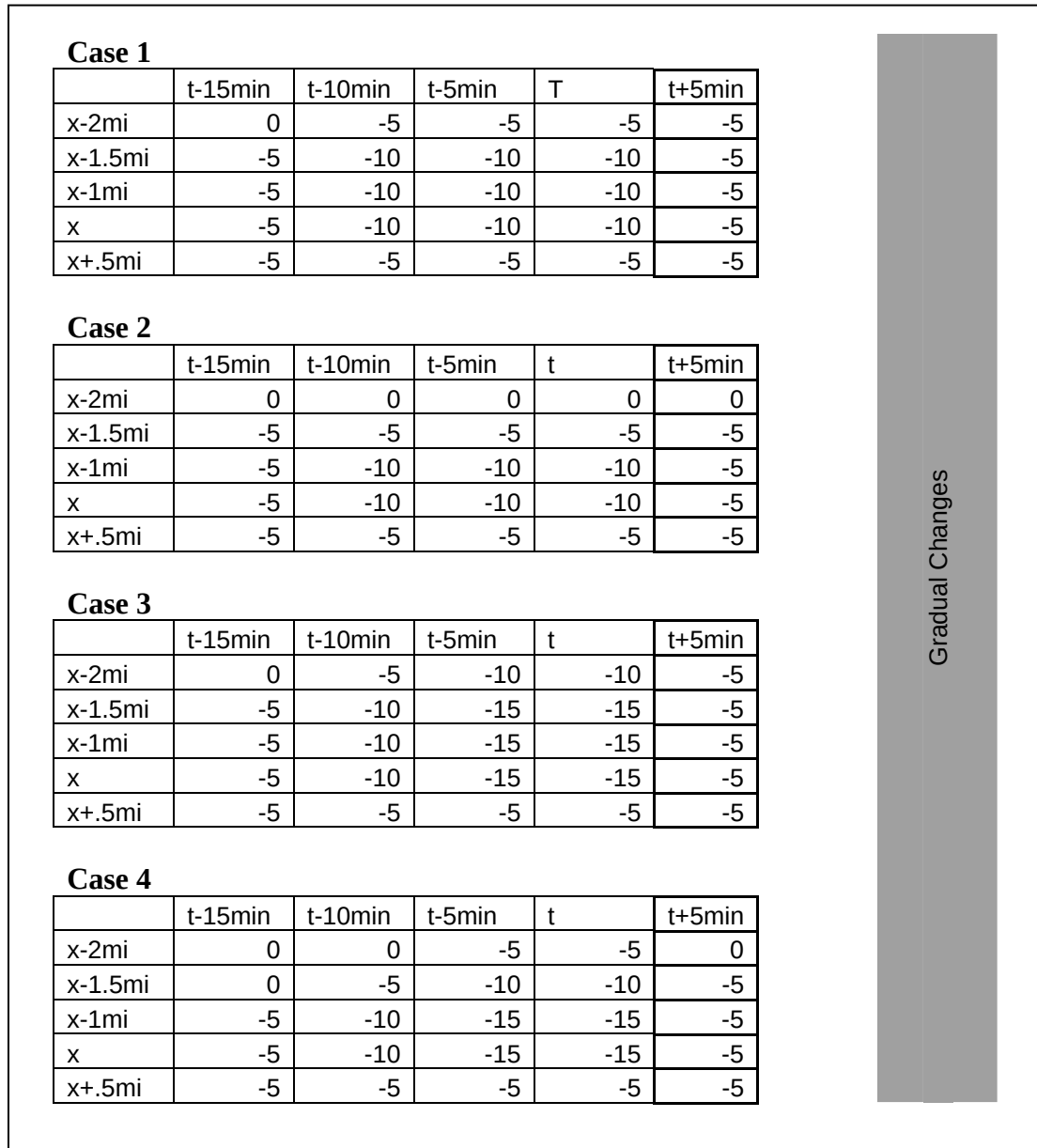


Figure 3-13: Cases 1 through 4 for the low-speed model

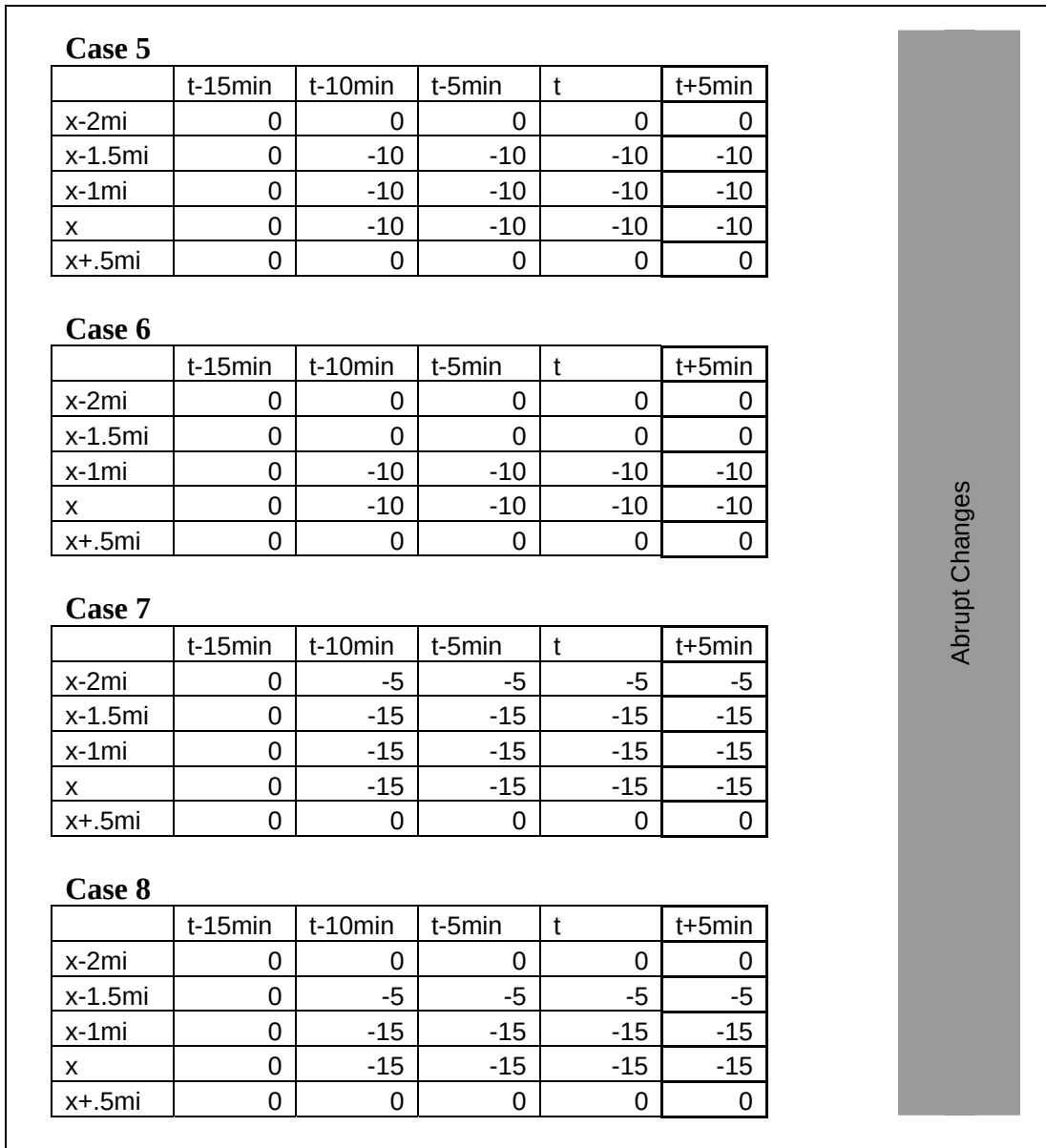


Figure 3-14: Cases 5 through 8 for the low-speed model

Case 4 therefore must be disregarded. After throwing out the case, none of the other cases produce a result that may say, convincingly, that they improve safety.

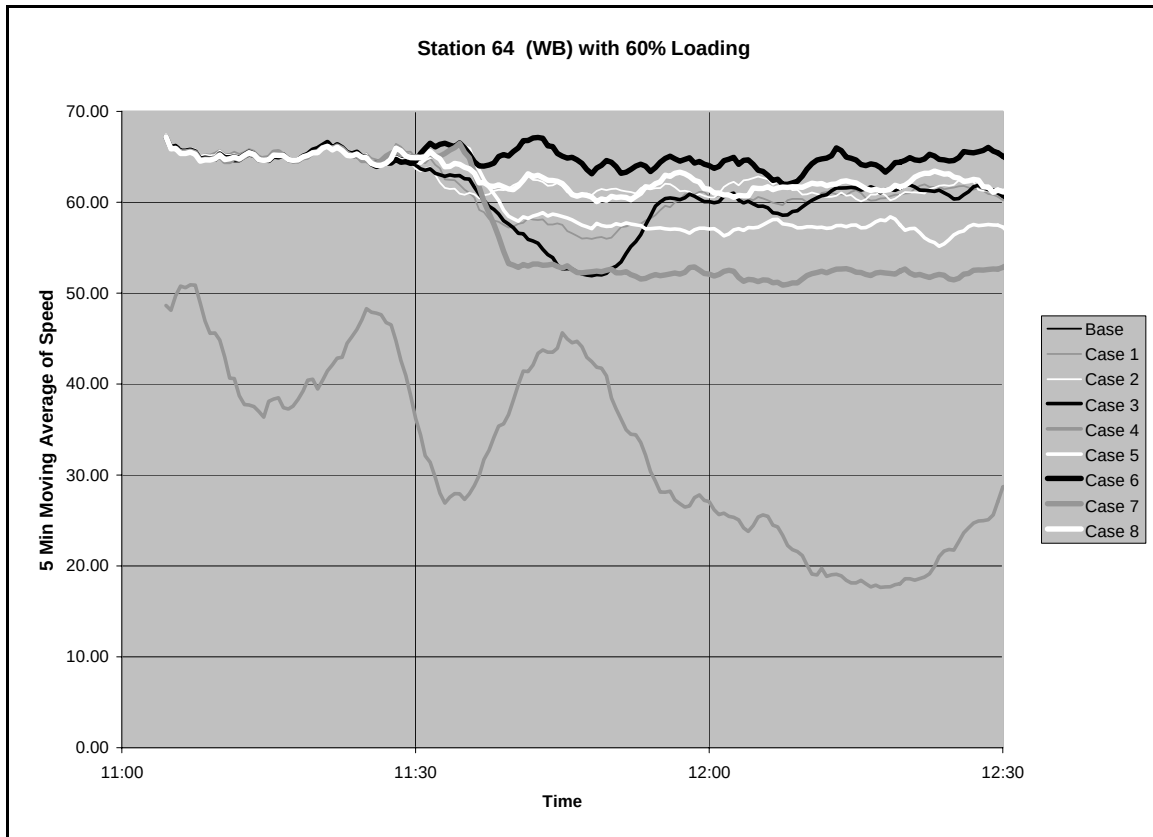


Figure 3-15: Station 64 average speeds for the base case and upstream VSL implementation cases.

Figure 3-16 and Figure 3-17 show a different story. By using both upstream and downstream VSL, a trend seems to have developed. While not significantly different, the VSL case looks to be lower in crash potential than the base case. It should be remembered that station 61 did not have a high crash potential to begin with. To see if this trend plays out, a higher crash potential location, station 47, is considered.

Figure 3-16 and Figure 3-17 differ, because Figure 3-16 uses the same speed limit function throughout the simulation, while Figure 3-17 uses speed control in order to lower the speed limit. Speed control is the technique PARAMICS uses in its programmer

guide. Comparing these two techniques further validates the method of multiple links files.

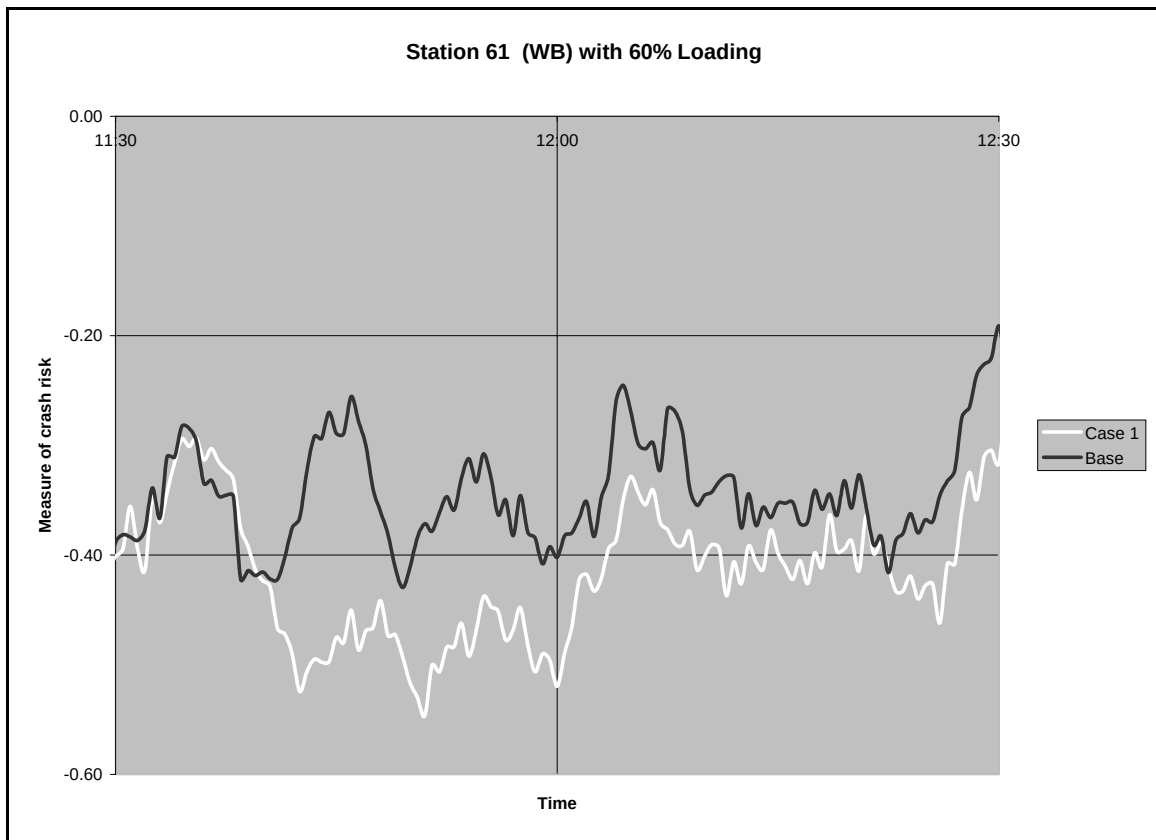


Figure 3-16: Station 61 crash potentials for the base case and downstream VSL implementation case.

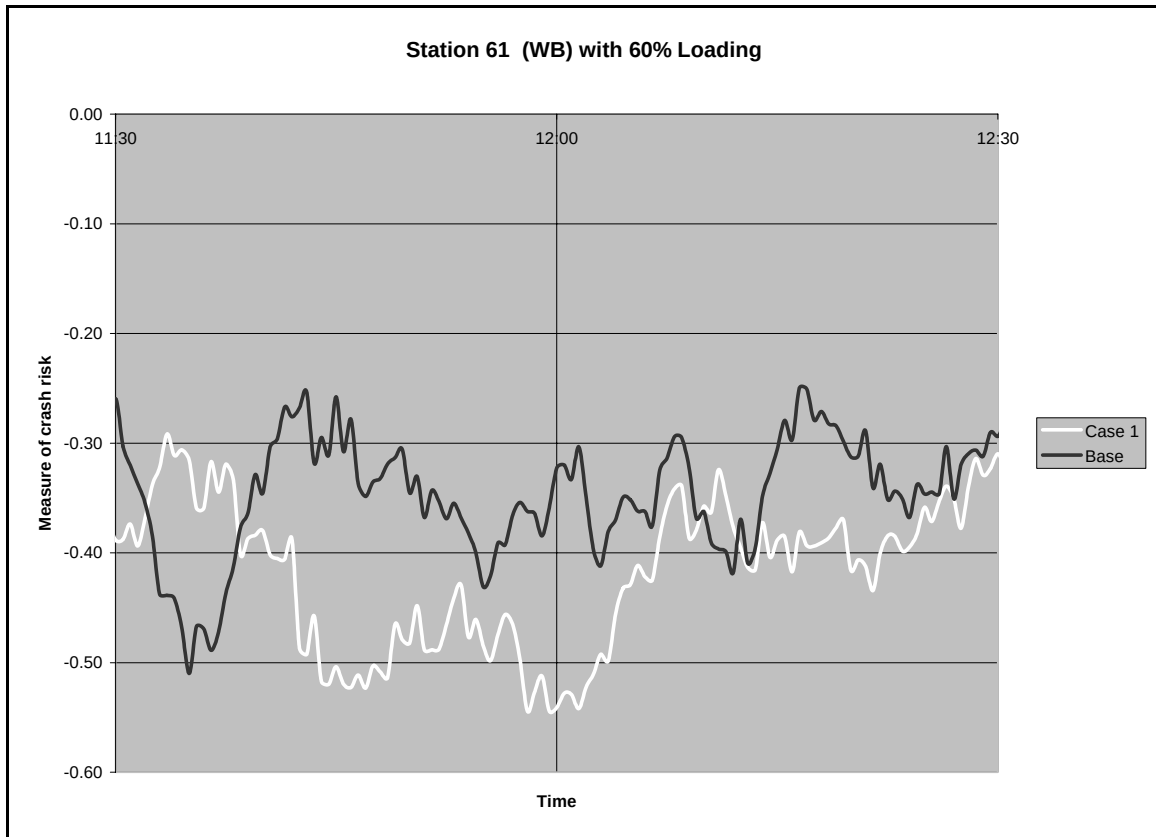


Figure 3-17: Station 61 crash potentials for the base case and downstream VSL implementation case via speed control.

3.4.2 Station 47

3.4.2.1 Speed Limit Patterns Scenarios

The first set of scenarios tested compare the effect of changing speed limits by different values across one time period. All speed limits change to their variable speed at 4:30pm (16:30) and change back to their base speed at 5:00pm (17:00). The difference between the scenarios is summarized in Table 3-2. The change in crash potential due to

these different scenarios is shown in Figure 3-18. In Table 3-2 the variables in the different scenarios are spatial implementation rate, change amount, and location of the changes. Spatial implementation can either be gradual or abrupt. Gradual implementation involve making the first half a mile either a 5 mph change or a ten mph change based on whether the total change is 10 mph or 15 mph. In other words for a 15 mph change 3.5 miles upstream the speed limit would be lowered by 15 mph, but from 4.0 miles to 3.5 miles the speed limit would only be lowered 10 mph. In the abrupt case all 4 miles would be lowered 15 mph. The amount of the change is limited to a 10 mph or 15 mph change. In either case both upstream and downstream changes will be by the same amount. The location of the change refers to whether the change is going to take place in the 5.5 miles downstream of the detector, or for the same stretch downstream plus an additional 4 miles upstream.

Table 3-2: Different scenarios based on changing speed limit patterns

Case 1	Abrupt	10 mph	Up and Downstream
Case 2	Abrupt	15 mph	Up and Downstream
Case 3	Gradual	10 mph	Up and Downstream
Case 4	Gradual	15 mph	Up and Downstream
Case 5	Gradual	10 mph	Downstream
Case 6	Gradual	15 mph	Downstream

*All upstream speeds changes are negative and all downstream changes are positive.

Figure 3-18 shows that the abrupt cases outperform the gradual cases and that 15 mph differences outperform 10 mph differences. Also, upstream decreases in speed increase the effect of the variable speed limits, especially at the beginning and ending periods of the change.

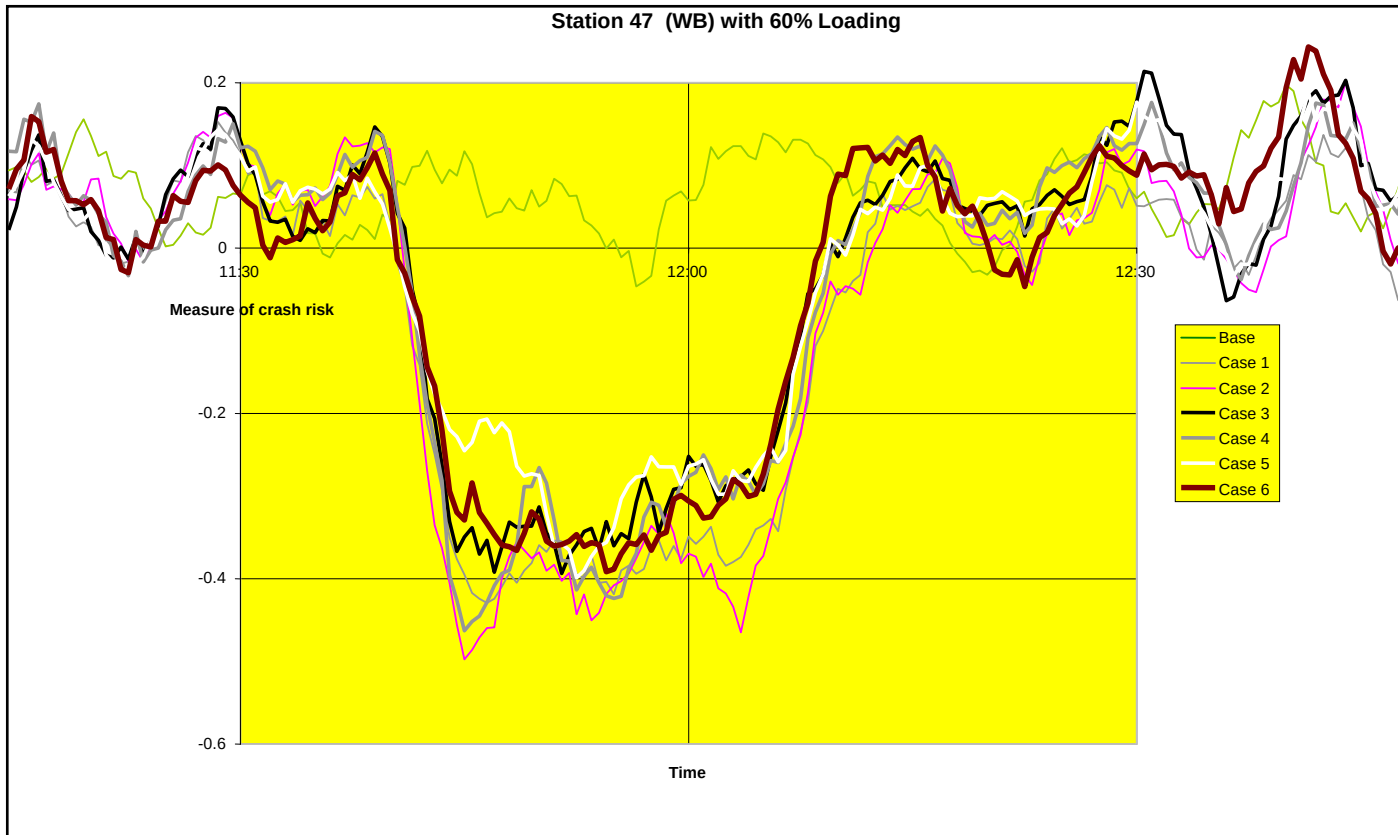


Figure 3-18: Crash potentials at Station 47 for the 6 scenarios in Table 3-2.

3.4.2.2 Crash Potential Relocation

Upstream of the location of interest, there are minor changes in crash potential with some stations showing a higher crash potential and some showing a lower crash potential. However, one mile downstream of the location of interest there is a spike that occurs at 16:30 as will be seen later this problem has been resolved. The reason for the increase at this station has not been determined. The speed limit changes here and half a mile upstream and downstream are exactly the same. As a result, the change in crash potential is expected to be the same.

The upstream and downstream edges where the change in speed limits takes place are where crash potentials are expected to relocate. Figures 3-19 and 3-20 clearly show that this is not the case. While there is a slight increase at the upstream edge, there is no difference at the downstream edge.

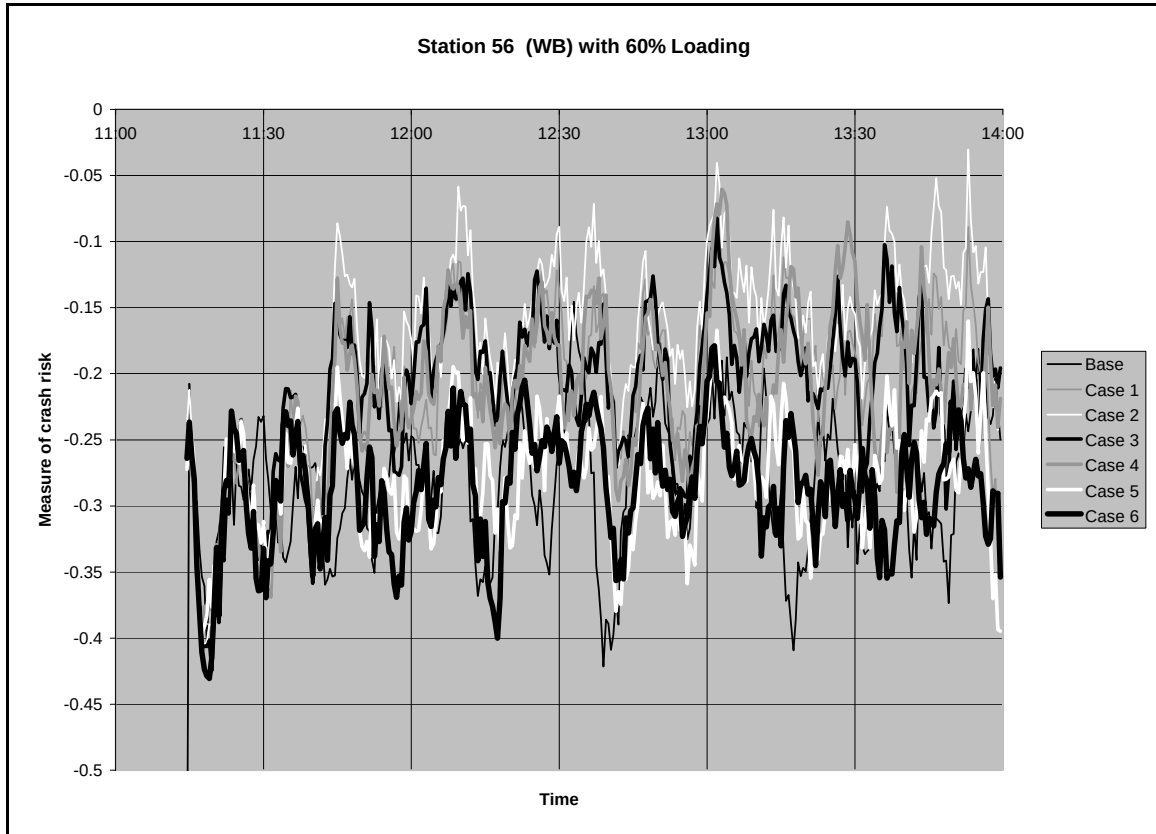


Figure 3-19: Crash potentials at the upstream edge, Station 56, for the 6 scenarios in Table 3-2.

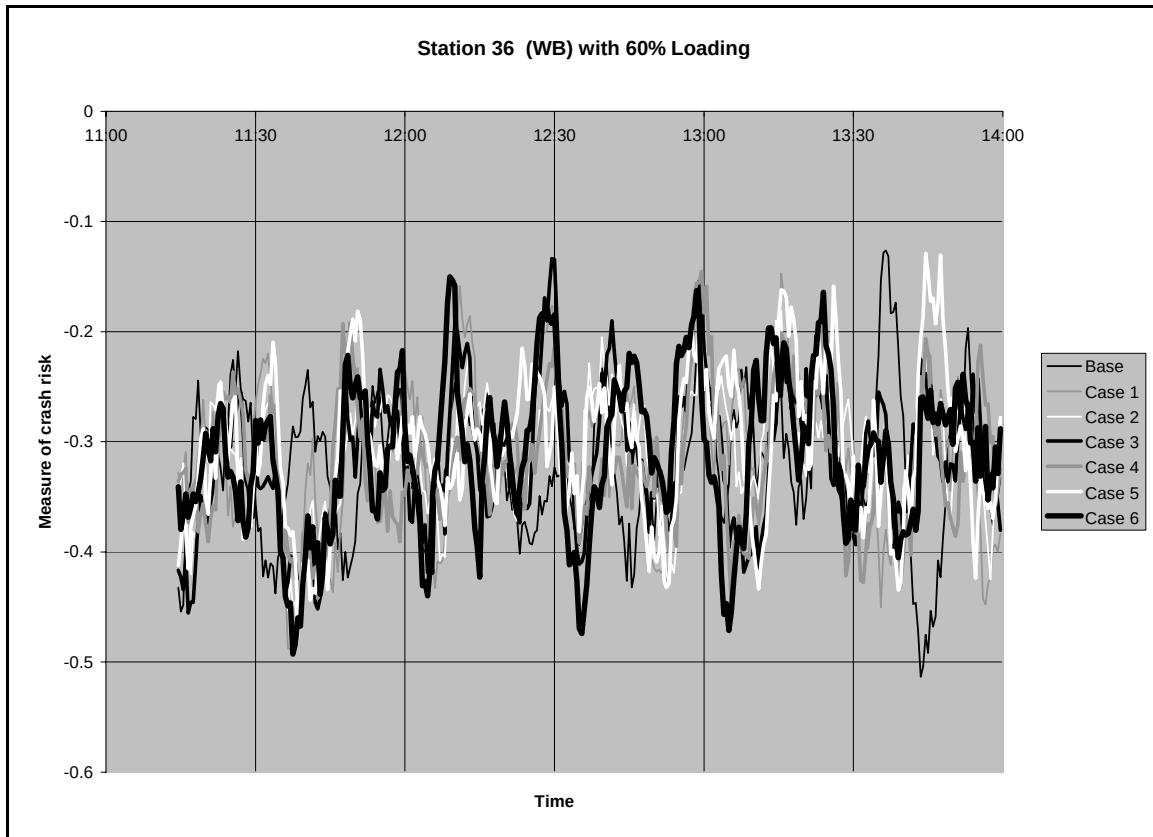


Figure 3-20: Crash potentials at the downstream edge, Station 36, for the 6 scenarios in Table 3-2.

3.4.2.3 Distance Scenarios

While the scenarios from Table 3-2 show that an abrupt 15 mph change both upstream and downstream have the greatest effect, there are still lingering questions. It is important to ask how large of an area should be changed, how long of a time period will changing the speed limits be effective for, and if the abrupt change in distance should change slowly over time. Also of interest is the effect that changing the speed limits will have over time. In order to look into these questions another set of scenarios is devised.

To look at how much of an area should be changed, the scenarios in Table 3-3 will be run. Case 1 is the Case 2 from the earlier scenario trials. Upstream values both greater than and less than those in the base case are attempted because the upstream edge showed an increase in crash potential (Figure 3-19). Only downstream values less than those in the base case are attempted because the downstream edge showed no change in crash potential, indicating that the distance was long enough (Figure 3-20).

In Table 3-3 there are only two variables, the upstream length and the downstream length, because gap distance is equal to zero miles. Figure 3-21 below shows what distance these distances translate into when comparing to the real world. Keep in mind that in the figure that detector 47 is the detector of interest.

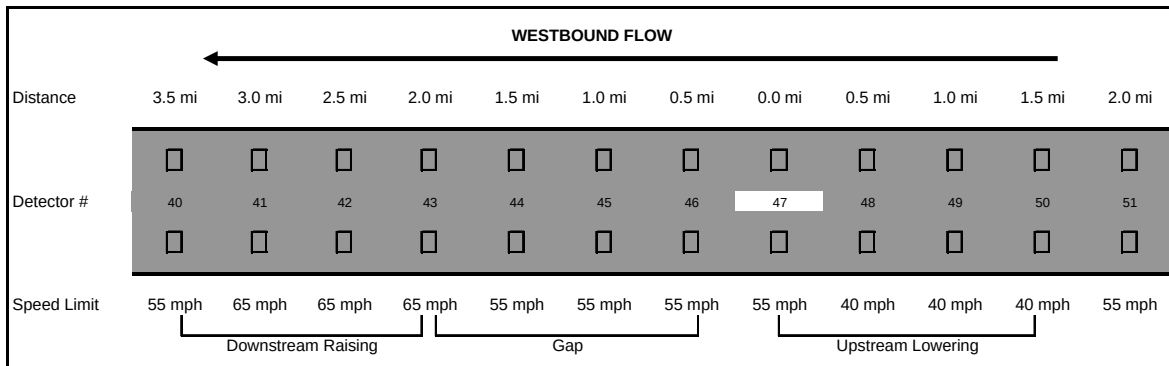


Figure 3-21: Sample scenario for a westbound roadway.

Table 3-3: Different scenarios based on changing distance

	Upstream	Downstream
Case 1	4 mi	5.5 mi
Case 2	6 mi	5.5 mi
Case 3	2 mi	5.5 mi

Case 4	4 mi	4 mi
Case 5	6 mi	4 mi
Case 6	2 mi	4 mi
Case 7	4 mi	2 mi
Case 8	6 mi	2 mi
Case 9	2 mi	2 mi

Figures 3-22 and 3-23 show that all of the cases are effective in reducing the crash potential. No one case outperforms the others in reducing the crash potential at detector 47. Although Case 8 shows an increased crash potential at the onset of the change in speed limits (Figure 3-23). In order to determine which is best, the downstream and upstream effects are inspected.

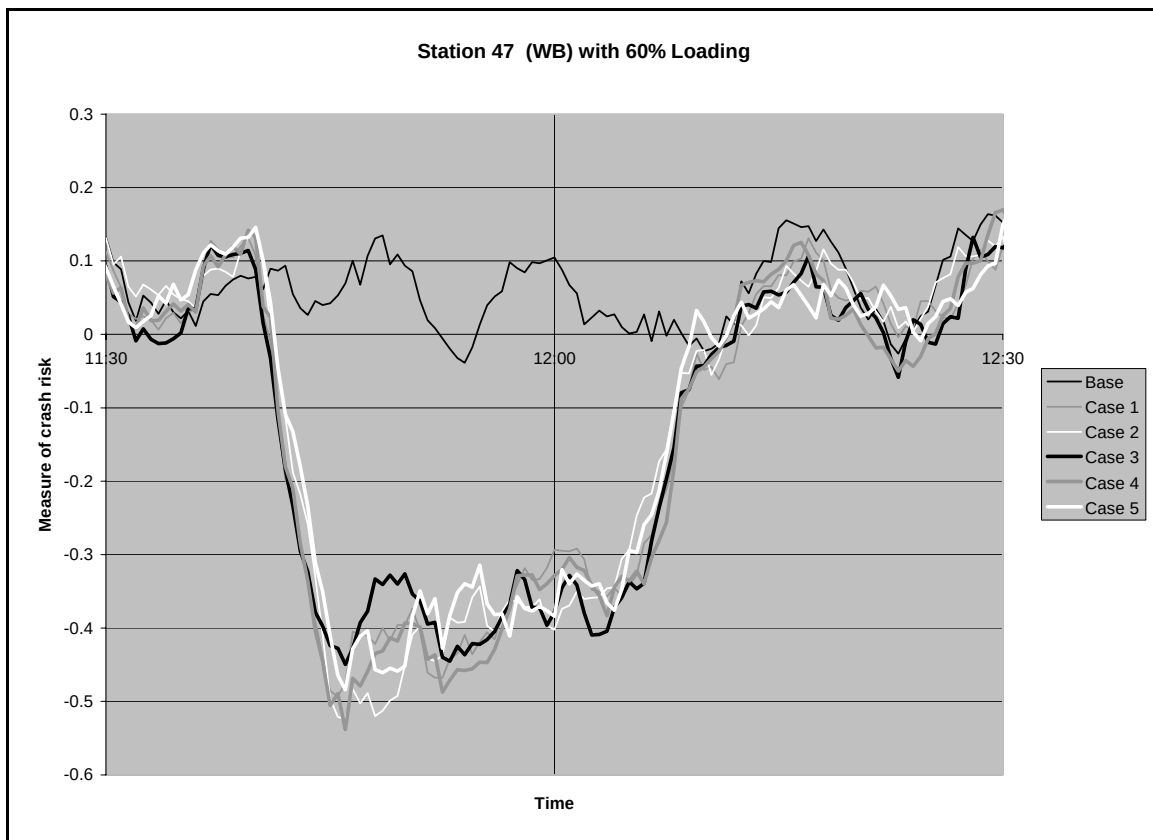


Figure 3-22: Crash potentials at the detector of interest, Station 47, for the first 5 scenarios in Table 3-3.

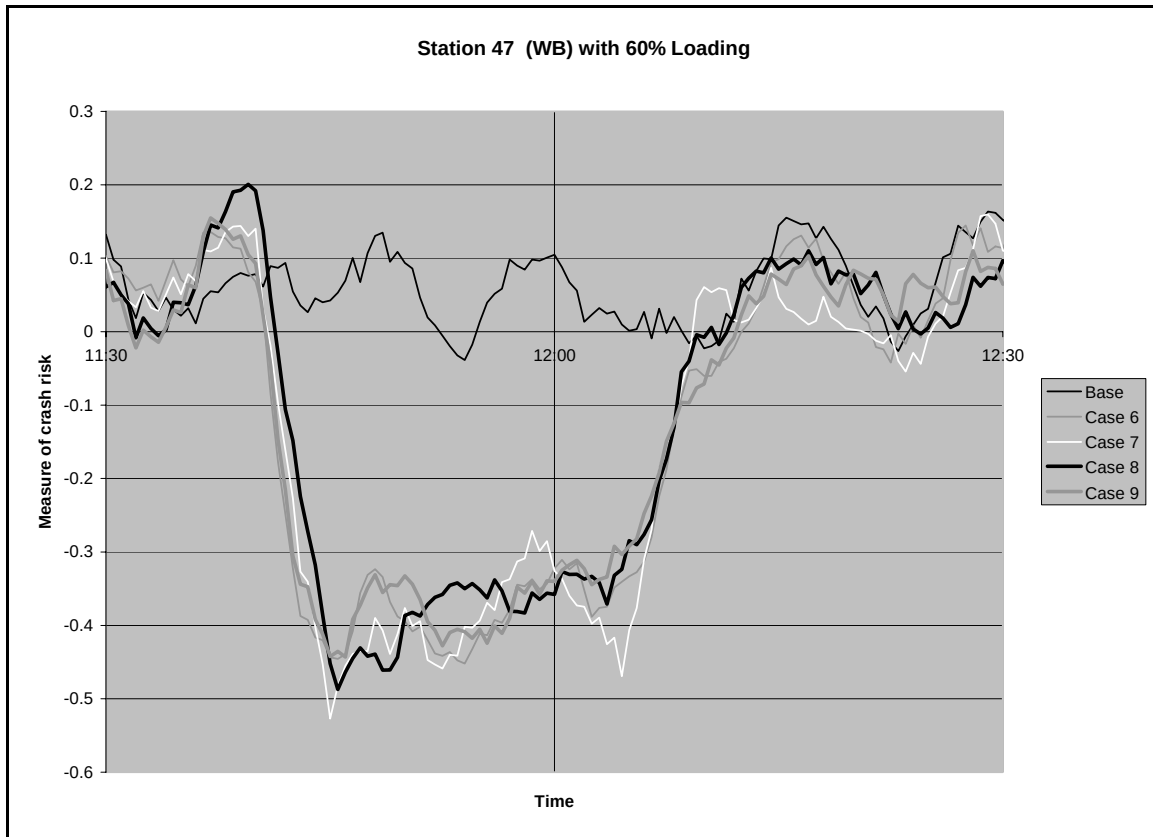


Figure 3-23: Crash potentials at the detector of interest, Station 47, for the last 4 scenarios in Table 3-3.

Figures 3-24 through 3-29 characterize the upstream effects of implementing the VSL. Because the upstream distance is varied, the location of the change in crash potential also varies. Cases with the same upstream distance show similar effects on crash potential, which is an encouraging result. Unfortunately, all of the cases worsen the safety of the roadway approximately equally with the implementation of VSL. In order to determine which is best, the effect on travel time is considered. By reducing the speed limit for a shorter period of time, travel time will be increased the least; therefore, the

shortest upstream distance is chosen (Cases 3, 6, and 9). To determine the downstream length the downstream effect is investigated.

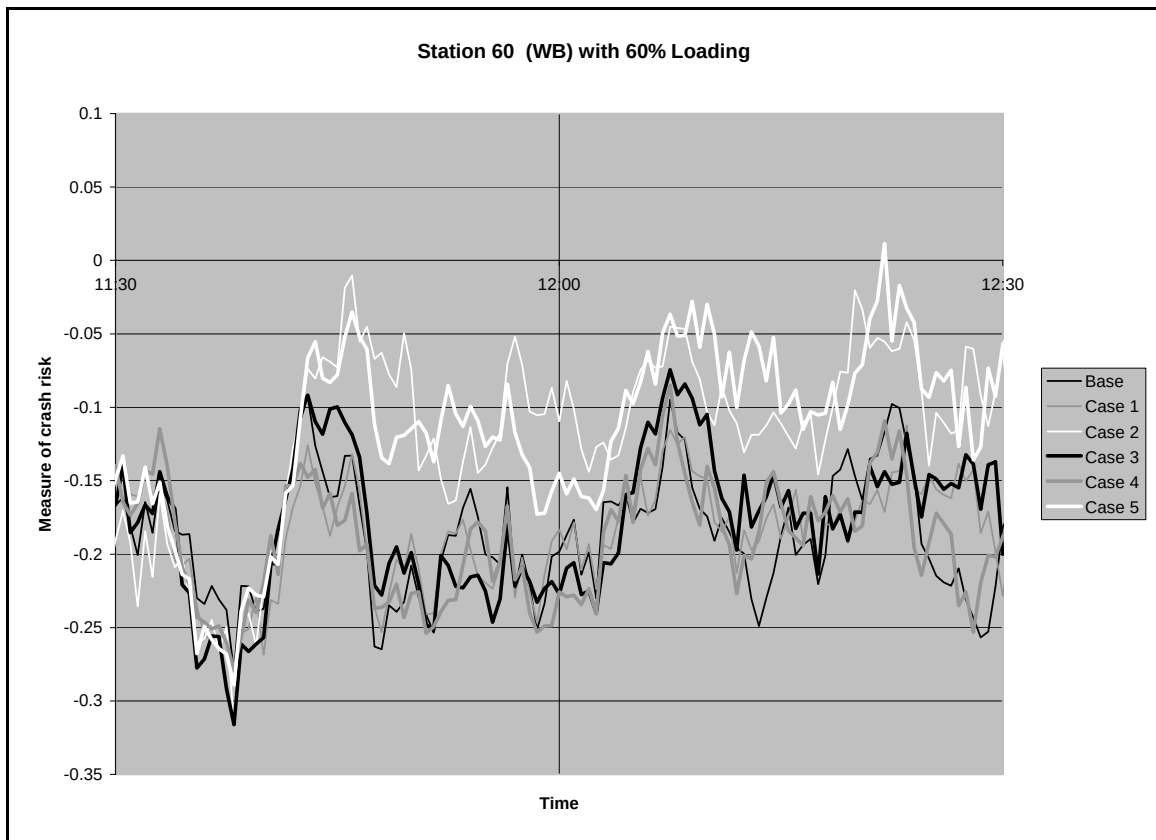


Figure 3-24: Crash potentials upstream of the detector of interest, Station 60, for the first 5 scenarios in Table 3-3.

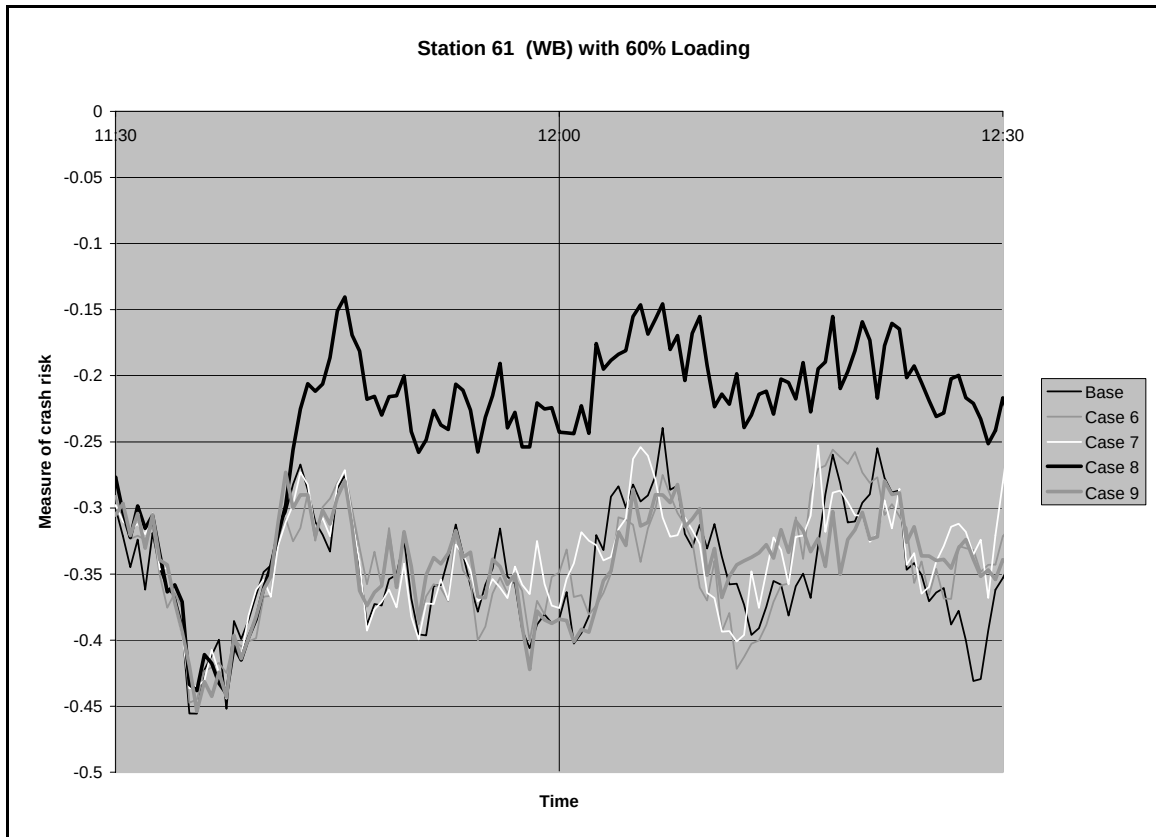


Figure 3-25: Crash potentials upstream of the detector of interest, Station 61, for the last 4 scenarios in Table 3-3.

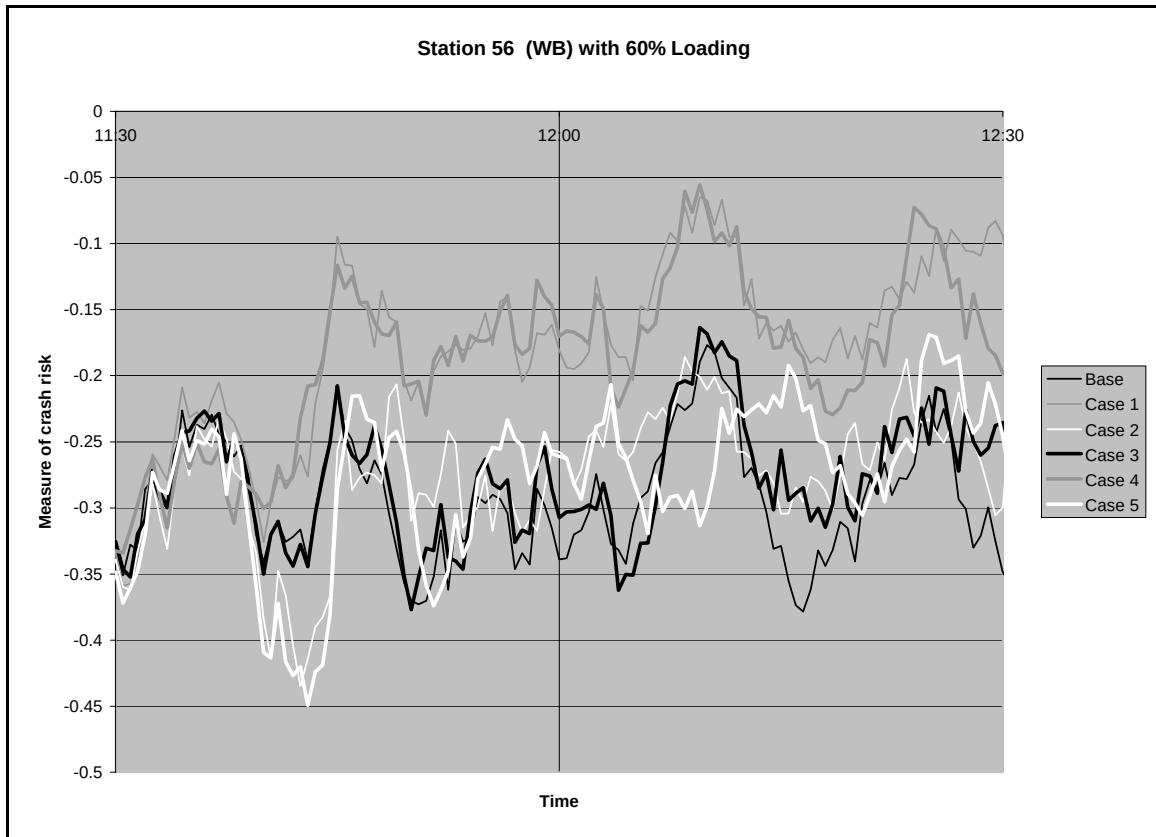


Figure 3-26: Crash potentials upstream of the detector of interest, Station 56, for the first 5 scenarios in Table 3-3.

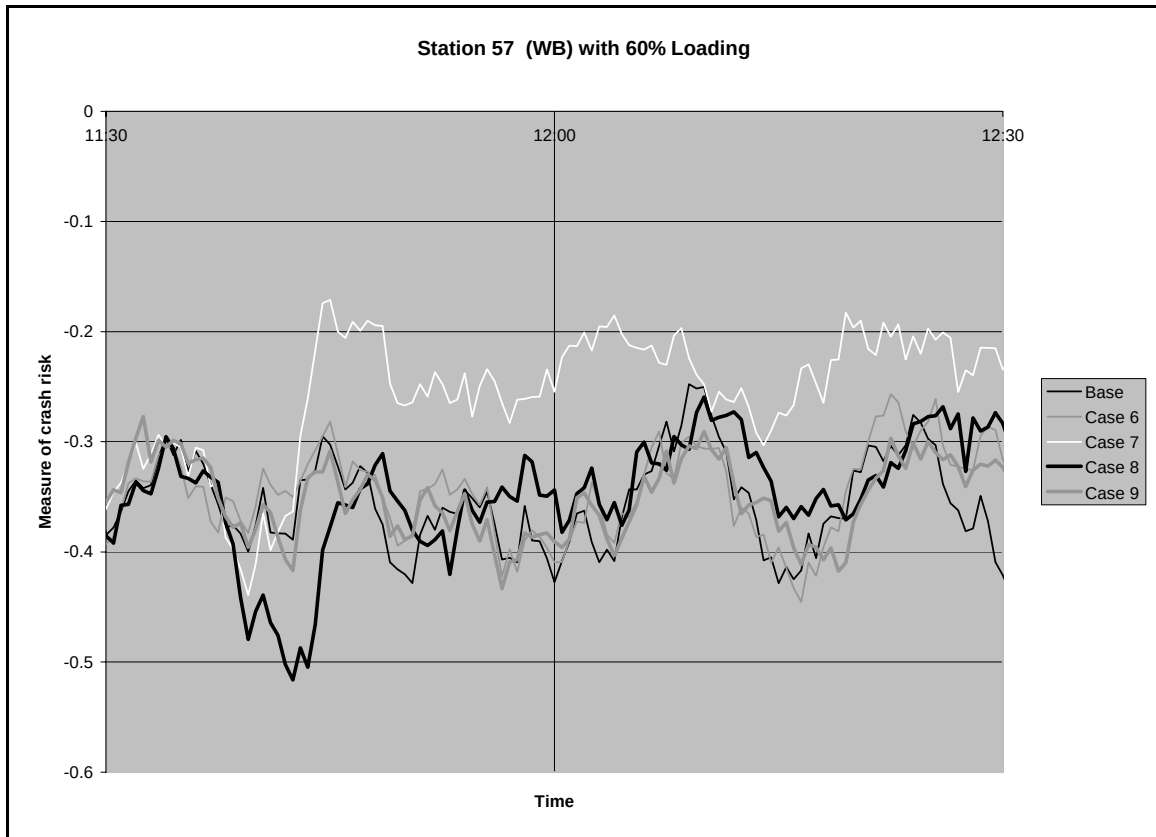


Figure 3-27: Crash potentials upstream of the detector of interest, Station 57, for the last 4 scenarios in Table 3-3.

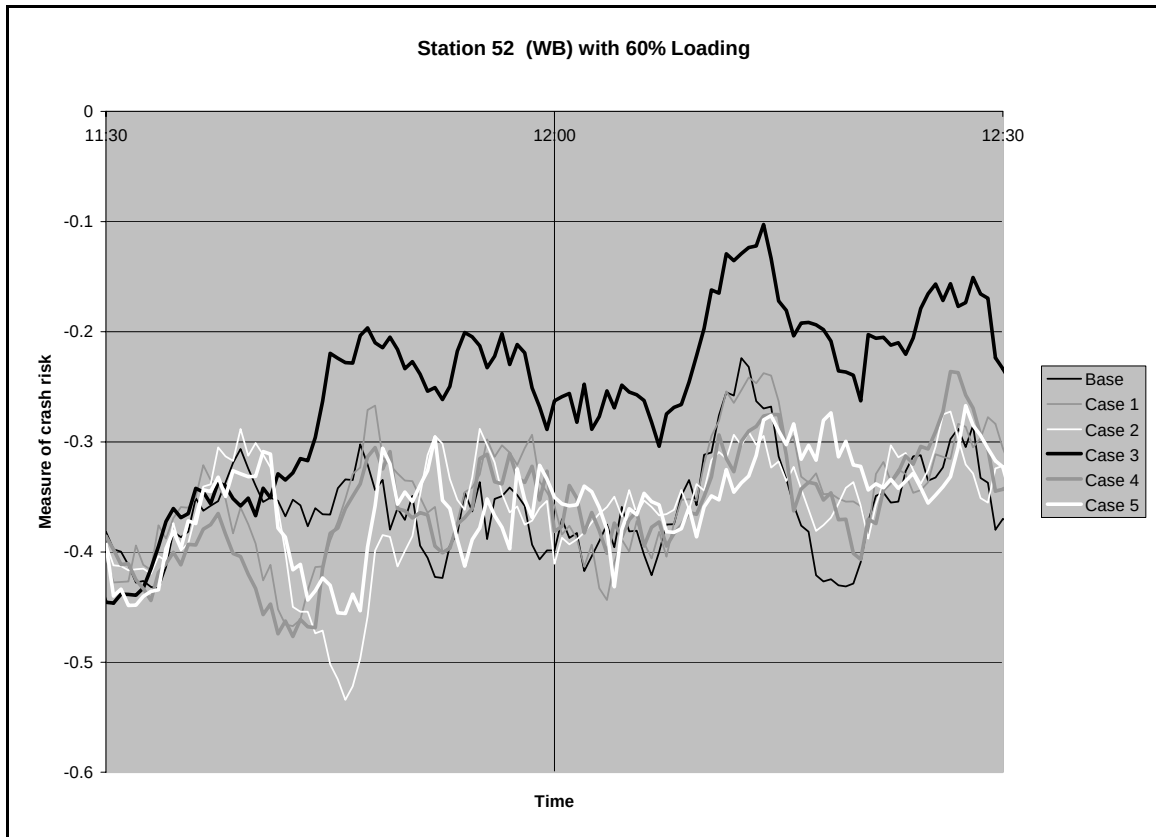


Figure 3-28: Crash potentials upstream of the detector of interest, Station 52, for the first 5 scenarios in Table 3-3.

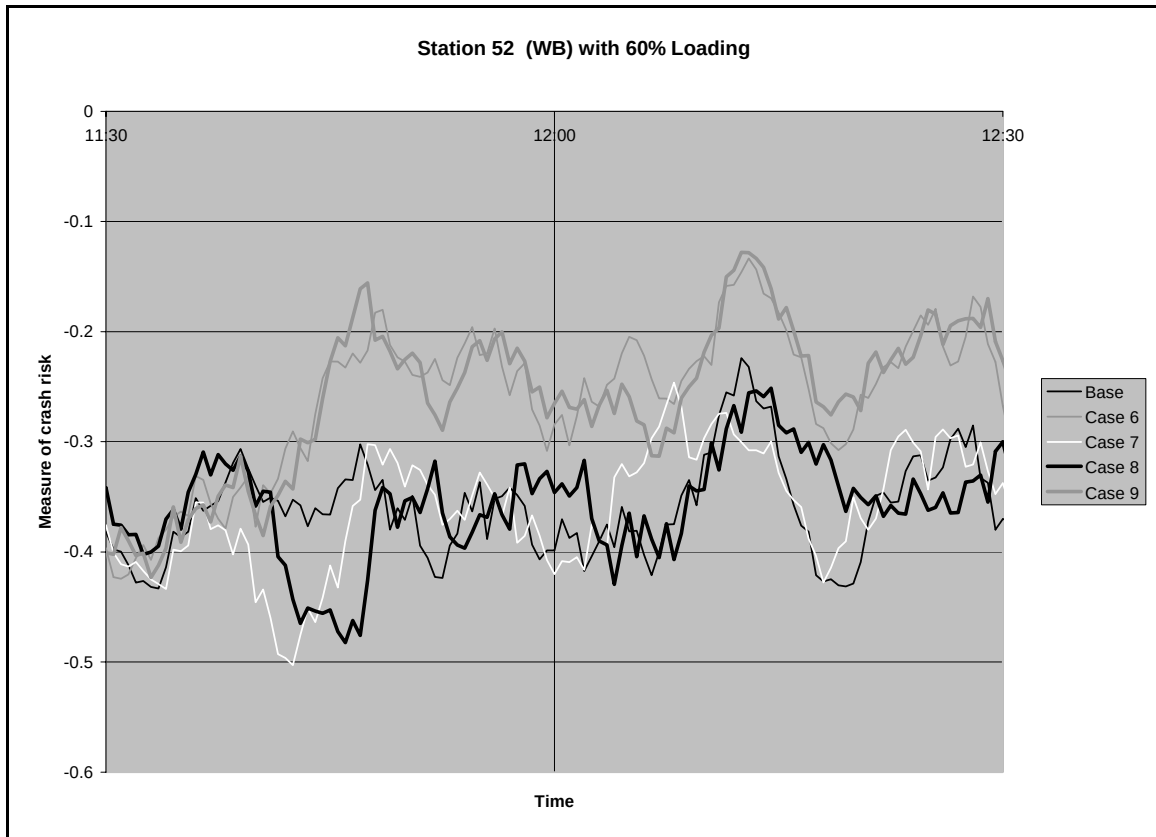


Figure 3-29: Crash potentials upstream of the detector of interest, Station 52, for the last 4 scenarios in Table 3-3.

Figures 3-30 through 3-35 characterize the downstream effects of implementing the VSL. Because the downstream distance is varied, the location of the change in crash potential varies. Cases with the same downstream distance show similar effects on crash potential. Case 9 shows the best results, particularly in Figure 3-31. Its lower crash potential and agreement with the earlier result make it the best case.

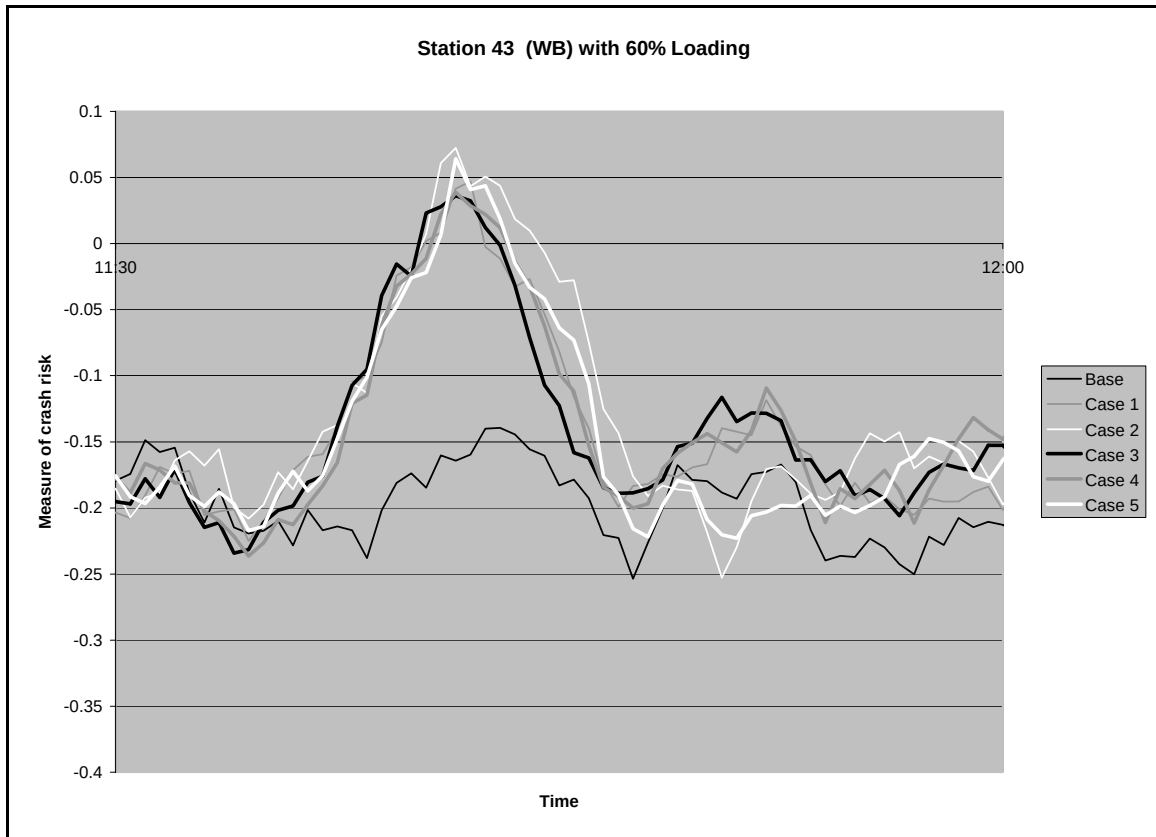


Figure 3-30: Crash potentials downstream of the detector of interest, Station 43, for the first 5 scenarios in Table 3-3.

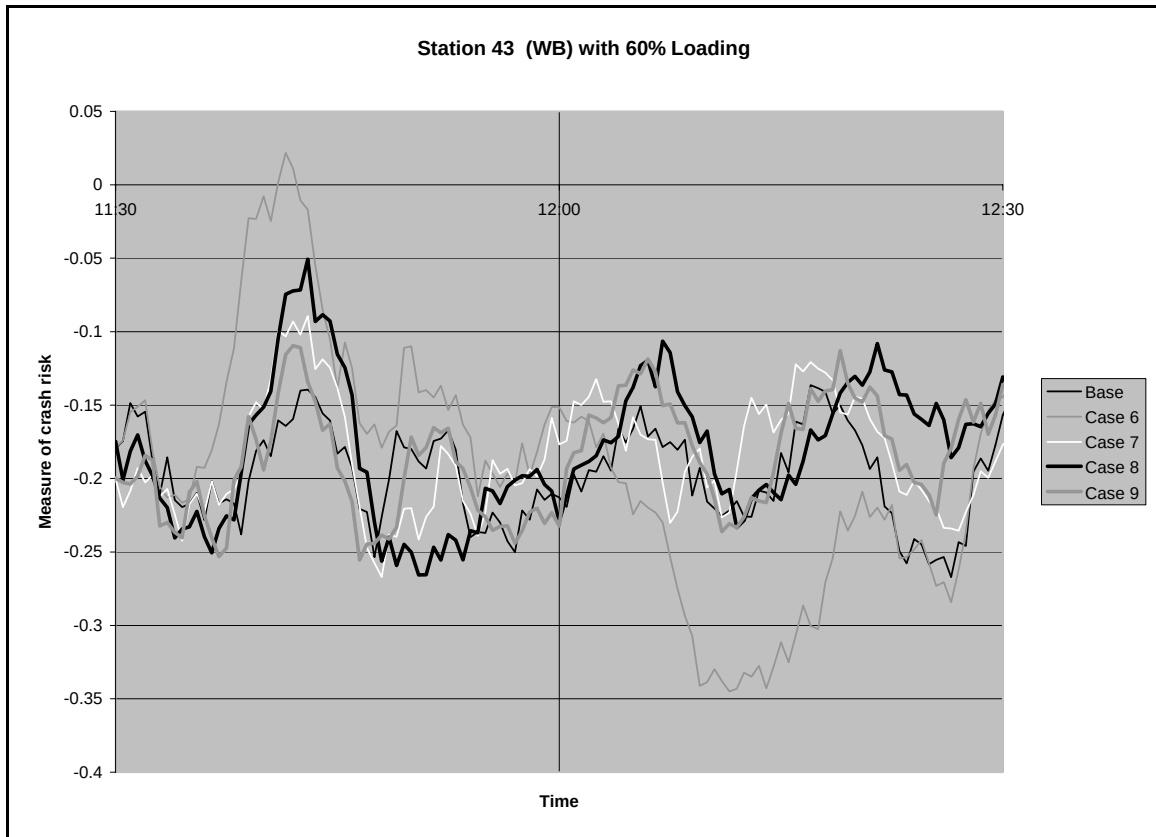


Figure 3-31: Crash potentials downstream of the detector of interest, Station 43, for the last 4 scenarios in Table 3-3.

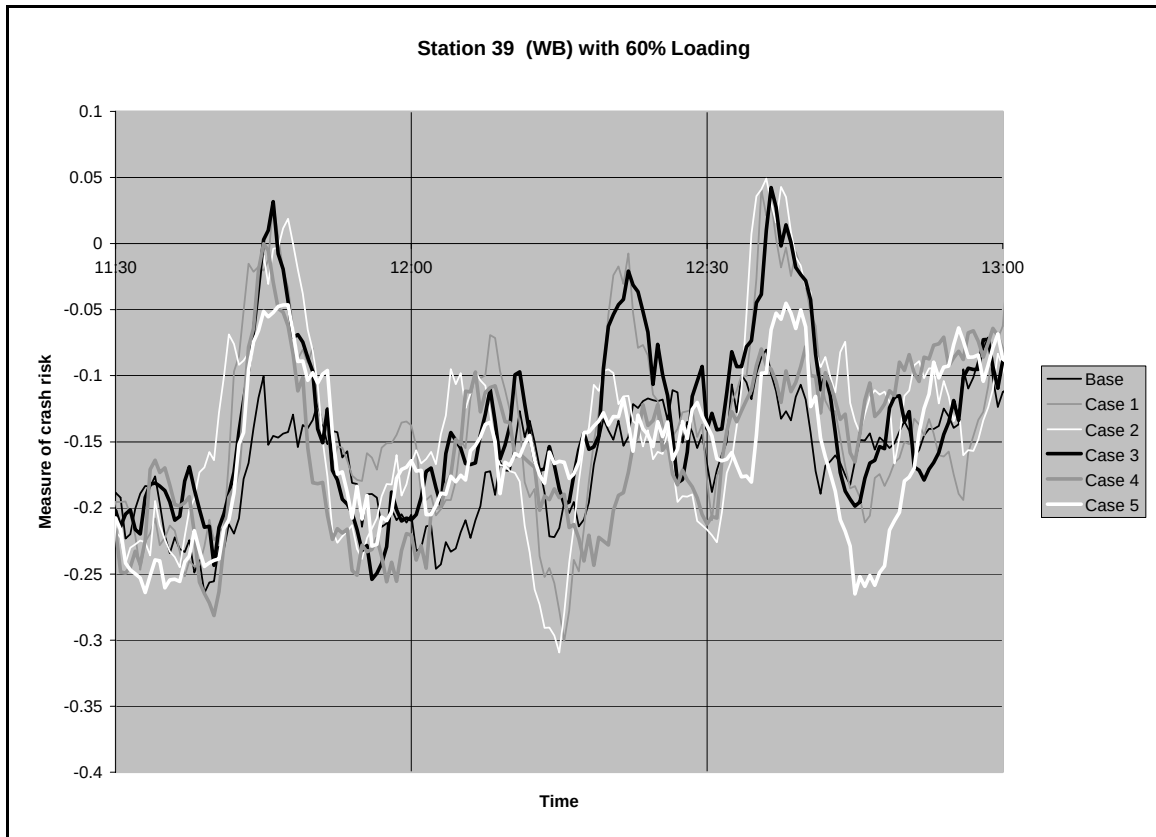


Figure 3-32: Crash potentials downstream of the detector of interest, Station 39, for the first 5 scenarios in Table 3-3.

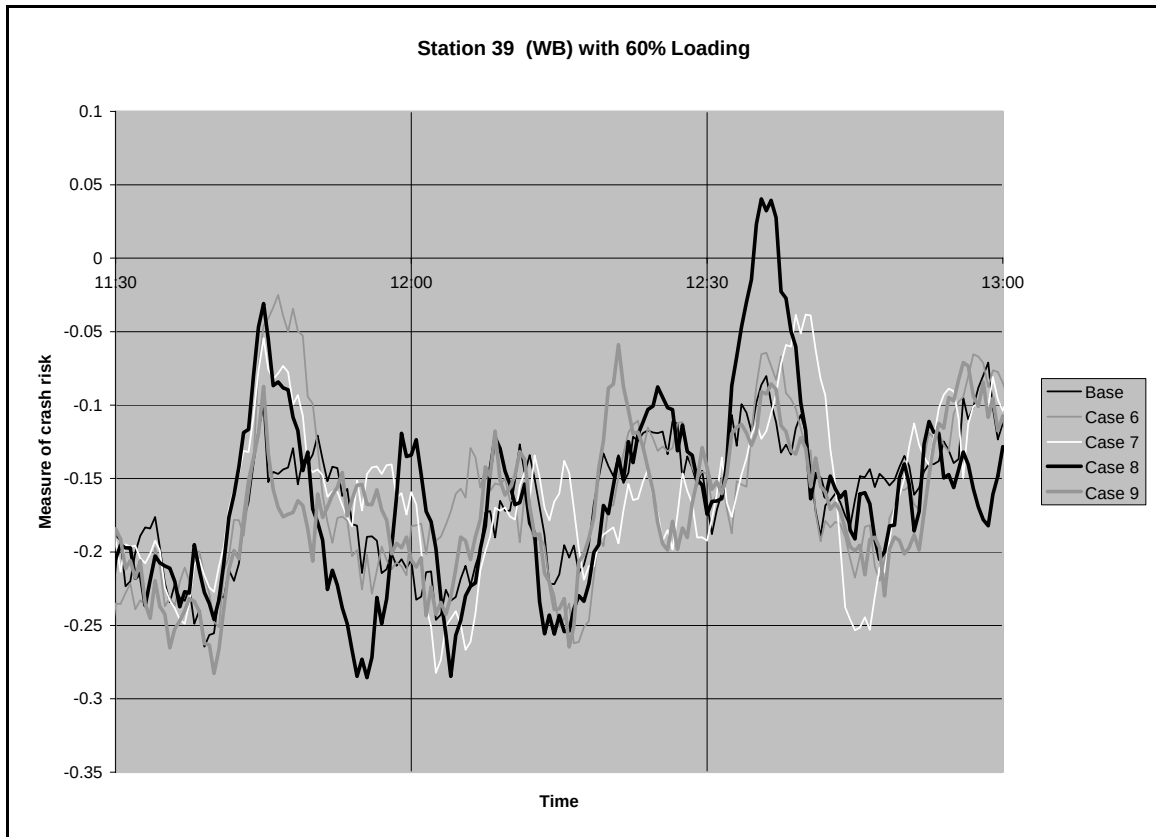


Figure 3-33: Crash potentials downstream of the detector of interest, Station 39, for the last 4 scenarios in Table 3-3.

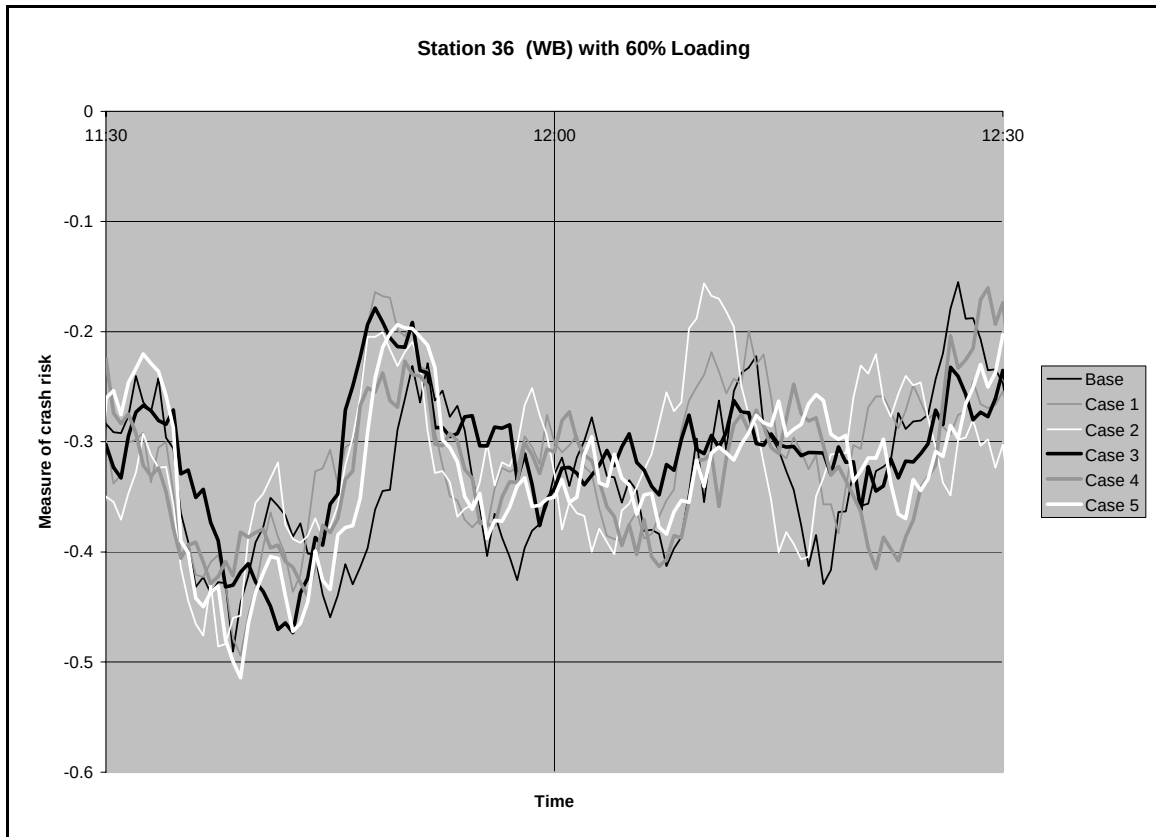


Figure 3-34: Crash potentials downstream of the detector of interest, Station 36, for the first 5 scenarios in Table 3-3.

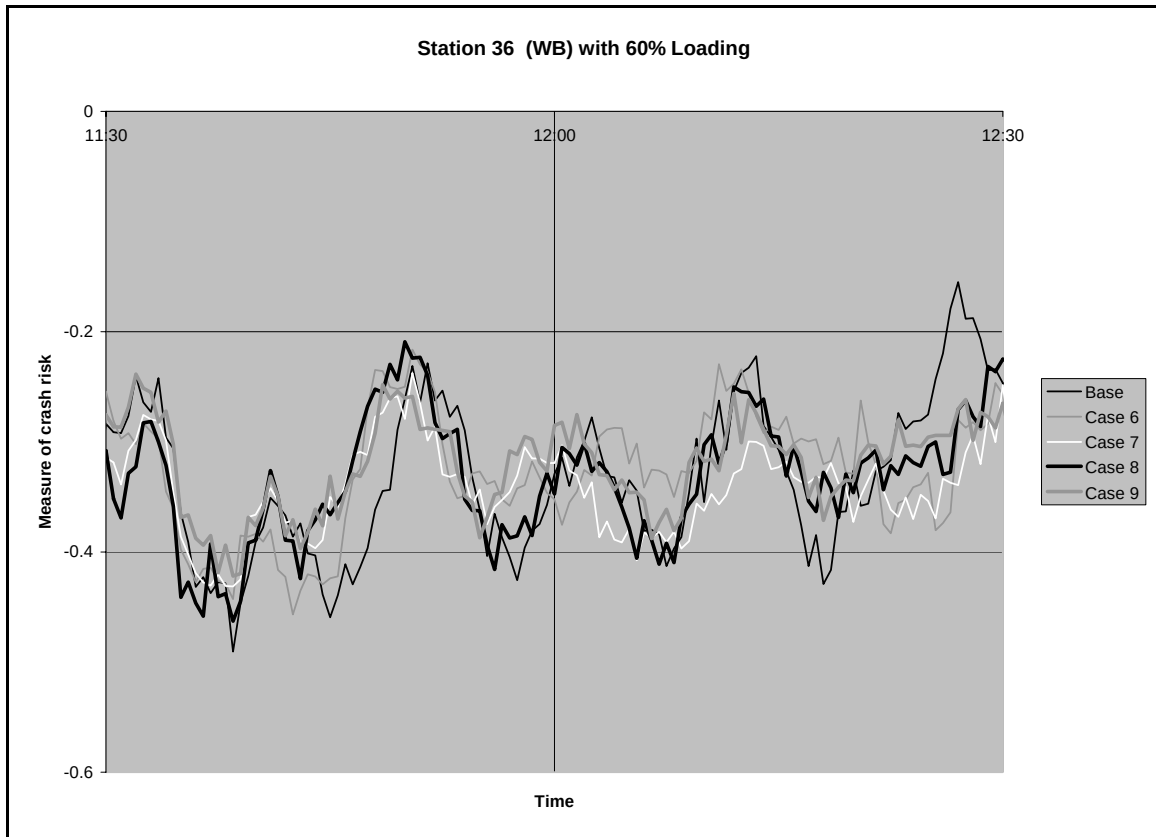


Figure 3-35: Crash potentials downstream of the detector of interest, Station 36, for the last 4 scenarios in Table 3-3.

3.4.2.4 Time Scenarios

The next step is to consider the effect of the implementation of the speed limit patterns and distances over time. By fixing the pattern and the distance that will be affected, the only factor is how quickly to raise and lower the speed limits. Because the difference in speeds is so large, it is expected that a slow rate of implementing the change in speed will work out best. Table 3-4 shows the scenarios that are tested.

In Table 3-4 there are two variables the time between changes in speed limit and the amount of the change in speed limit. The time between changes can be either 5 minutes or 10 minutes and the amount of change can be either 5 mph or 10 mph. For example if the change is 5 mph every 5 minutes, then 10 minutes before the change for the abrupt case the speed limit steps up 5 mph for the detectors downstream, five minutes later the speed downstream is 10 mph higher than the original speed limit, and then when the abrupt case would just be changing, the speed limit would be 15 mph higher than the original speed limit.

Table 3-4: Different scenarios based on changing the temporal implementation.

Case 1	Abrupt	
Case 2	5 min	5 & 10 mph
Case 3	10 min	5 & 10 mph
Case 4	5 min	10 mph
Case 5	10 min	10 mph
Case 6	5 min	5 mph

*Changes were made to both the upstream and downstream stations.

As shown in Figure 3-36, all of the cases reduce crash potential quickly and efficiently at the detector of interest. However, Case 3 has the most dramatic effect just downstream, as shown by Figure 3-37. Further downstream, as shown in Figure 3-37, Case 3 has an elevated crash potential for a longer time, but does not have a large value at anytime relative to the other cases. As a result Case 3 is the best scenario.

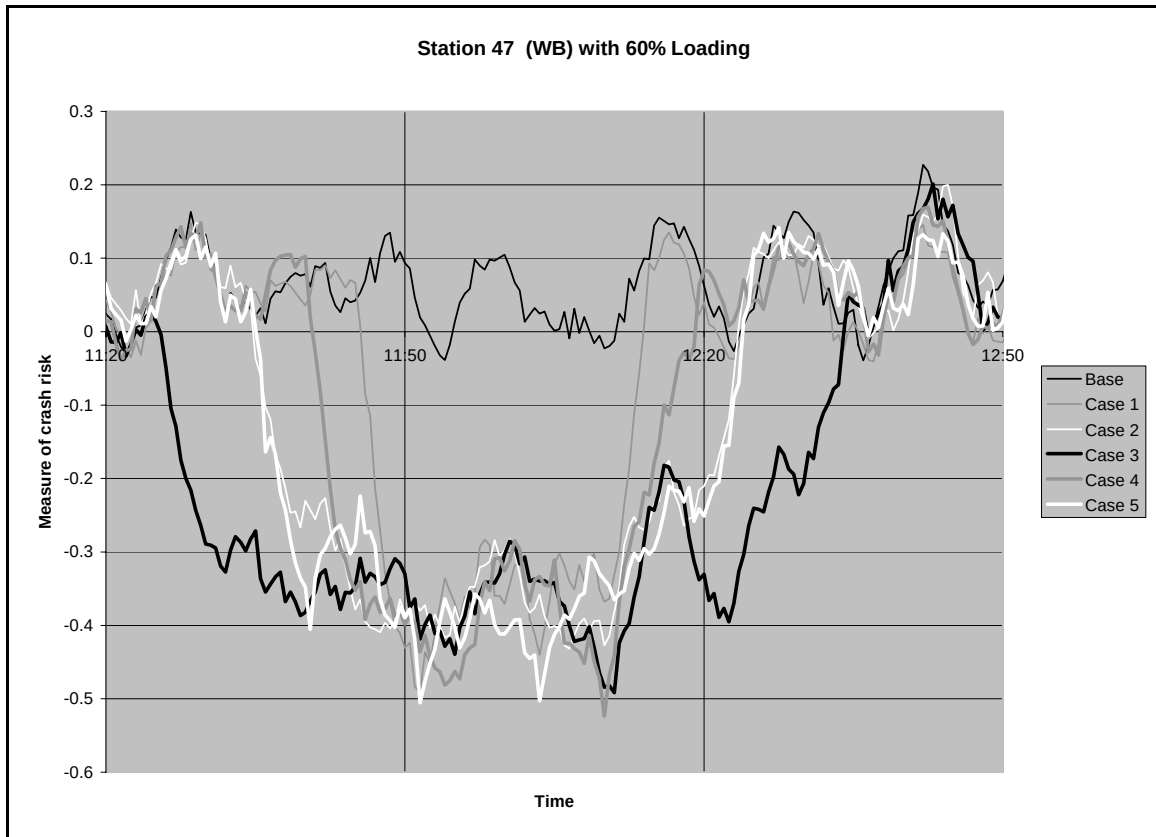


Figure 3-36: Crash potentials at the detector of interest, Station 47, for the scenarios in Table 3-4.

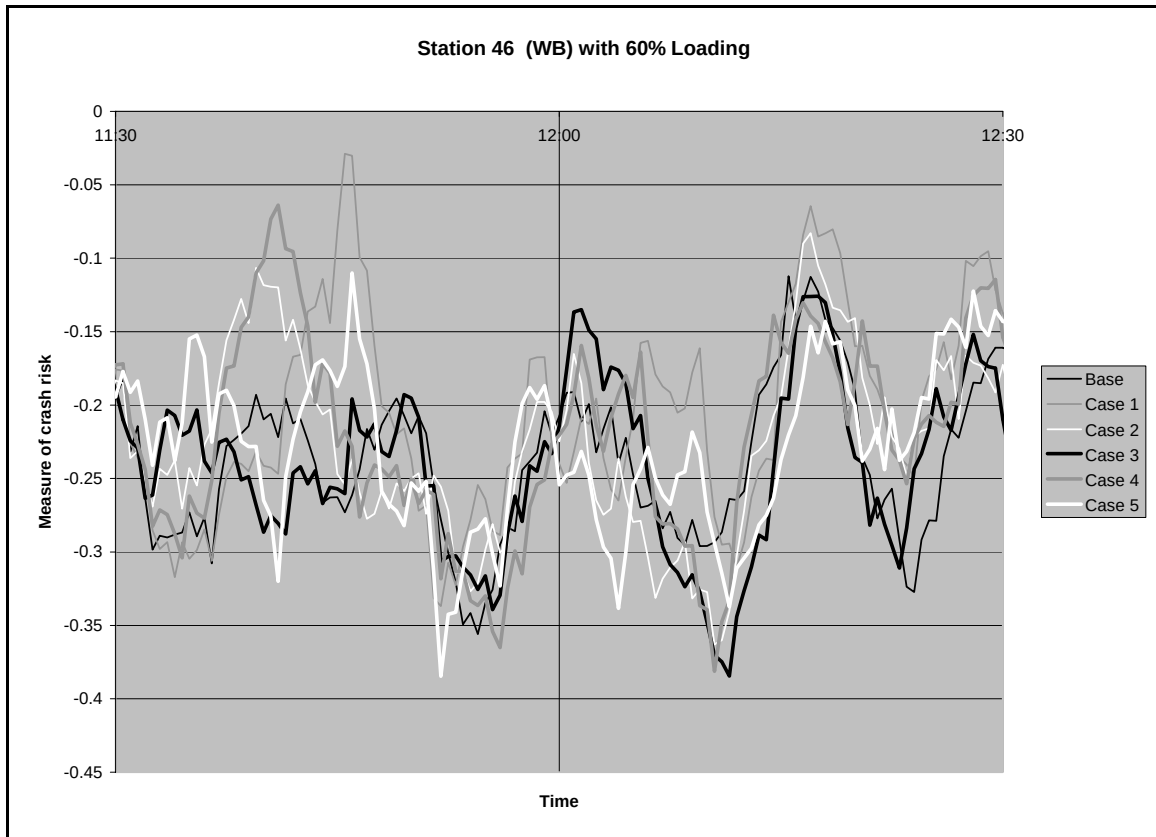


Figure 3-37: Crash potentials downstream of the detector of interest, Station 46, for the scenarios in Table 3-4.

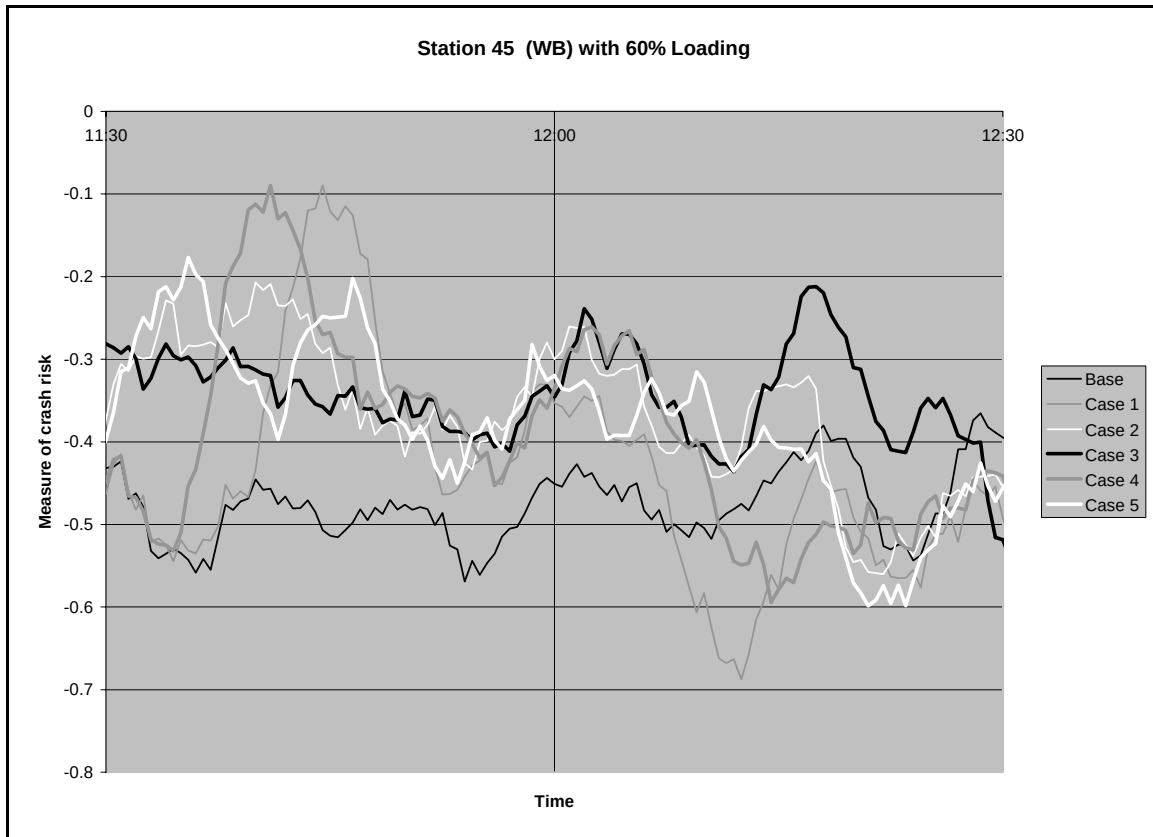


Figure 3-38: Crash potentials downstream of the detector of interest, Station 45, for the scenarios in Table 3-4.

3.4.2.5 Gap Scenarios

The last step is to consider the effect of the gap distance on crash potential. Because raising the speed limits should help only in locations where there is enough freedom for the driver to increase their speed safely and the crash algorithm work by identifying location with high occupancy (low freedom) just upstream as dangerous, it is expected that a gap of 1 to 2 miles will be ideal. Table 3-5 shows the scenarios that are tested. In Table 3-5 there is only one variable, the gap length. Figure 3-39 below shows what

location these distances translate into when comparing to the real world. Keep in mind that in the figure detector 47 is the detector of interest.

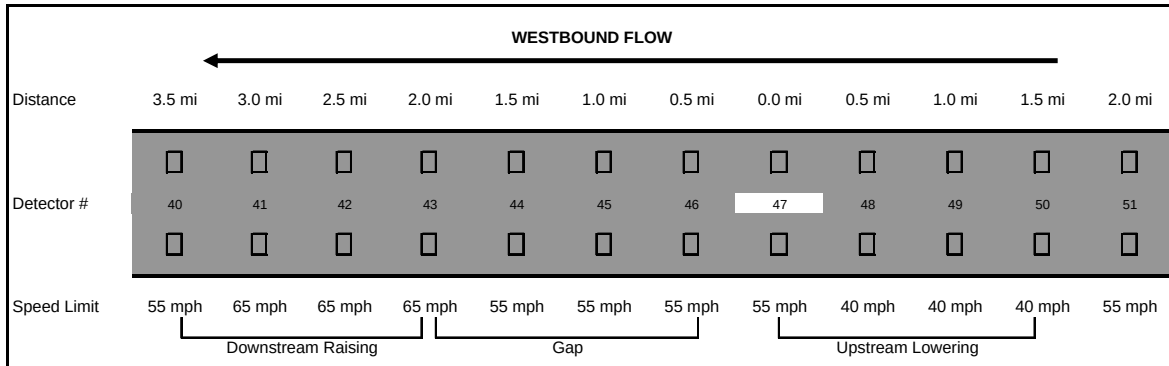


Figure 3-39: Sample scenario for a westbound roadway.

Table 3-5: Different scenarios based on changing the gap distance.

Case 1	0 mi
Case 2	1 mi
Case 3	2 mi
Case 4	3 mi

Figure 3-40 clearly shows that Case 1 is the best case. In fact Case 1 is the only case where the VSL have an effect. While this solution is not intuitive it is consistent with other findings (Lee et al., 2004).

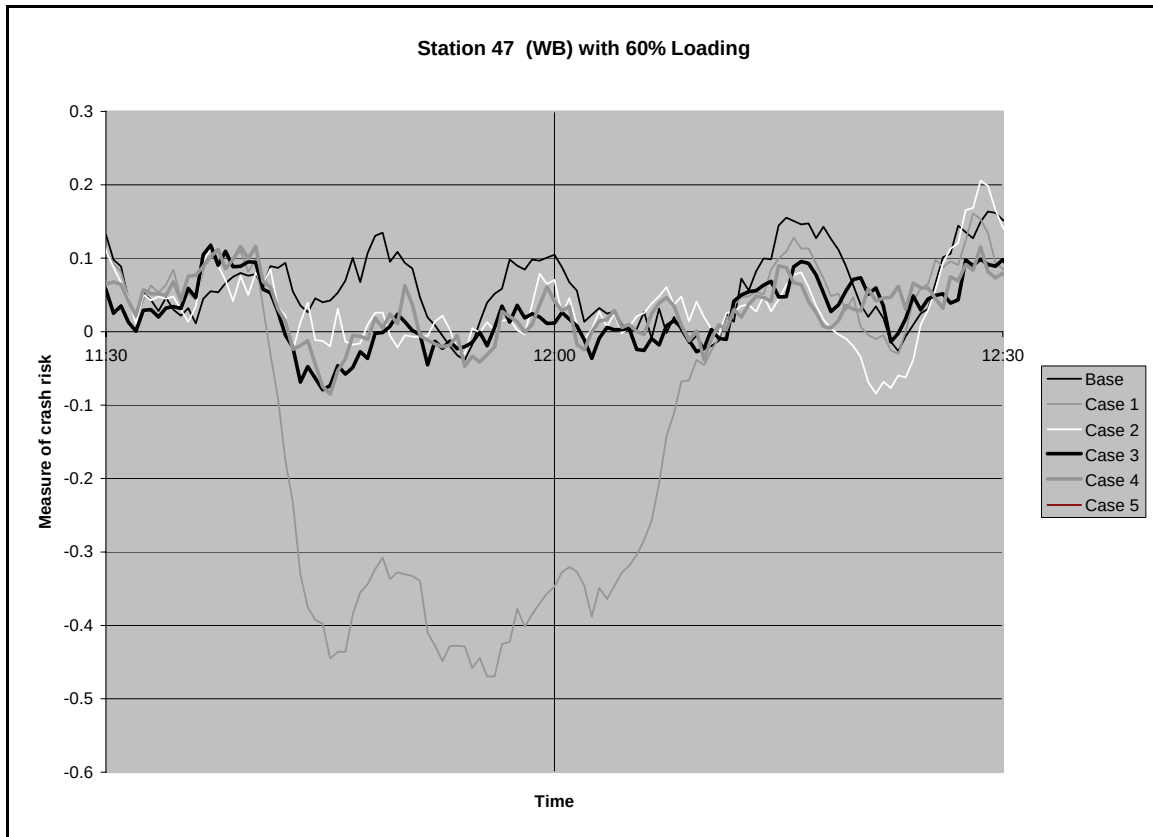


Figure 3-40: Crash potentials at the detector of interest, Station 47, for the scenarios in Table 3-5.

3.5 Ramp Metering

The multiple tests conducted left a strong recommendation for the high speed model, but the low-speed model did not have a successful implementation of VSL. In order to try and find a satisfactory result ramp metering attempted. Table 3-6 shows the scenarios that are tested. Ramp metering in this case means simply reducing the ramp counts by 40%. This will reduce the strength of the peak flows on the ramps and give a metering like effect. While the method is not the most elegant it is quick to implement. In the second case the scenario chosen for the high-speed model is used in conjunction with ramp metering to see it effect on the low-speed model.

Table 3-6: Different scenarios for the low speed model.

Case 1	Ramp metering
Case 2	Ramp Metering and High-Speed Result

The results in Figure 3-41 show that neither ramp metering nor the best high-speed scenario impacts the low-speed model.

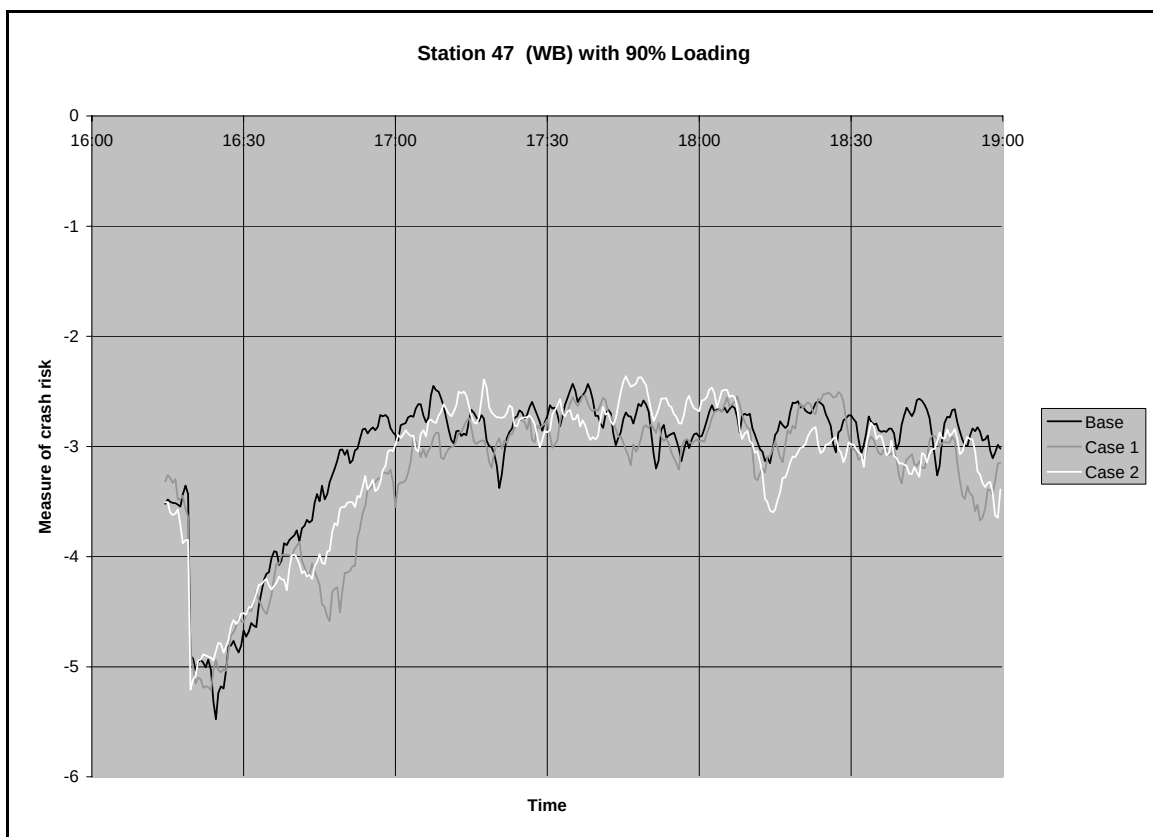


Figure 3-41: Crash potentials at the detector of interest, Station 47, for the scenarios in Table 3-5.

To test the use of ramp metering in the low speed situation further a different form of representing ramp metering is used and it is used in conjunction with VSL

implementation. Instead of metering all of the ramps equally the ramp that has the largest effect is blocked. This extreme case is used not to illustrate a step that should be taken in the field, but rather to see if ramp meter may hold a potential solution. The VSL implementation is equally extreme. The detector that shows the largest crash potential will have all speed limits upstream lowered by 15 mph and all speed limits downstream raised by 15 mph. The implementation is abrupt in time and space with zero gap distance. The implementation start and finish time varies to see its effect.

Figure 3-42 shows the station with the worst crash potential, station 56. Station 56 is just after an on ramp, so that ramp (SR 436) is closed for some of the test cases. The figure also shows that the crash potential changes over time. By implementing the VSL at different times the use of VSL during high crash potentials, before high crash potentials, and during the transition in potential. Table 3-7 shows the cases that are implemented. Cases 0 and 4 do not involve the use of VSL. Cases 1 and 5 take place after crash potential has risen. Cases 2 and 6 take place before crash potential has risen. Cases 3 and 7 take place while crash potential is increasing.

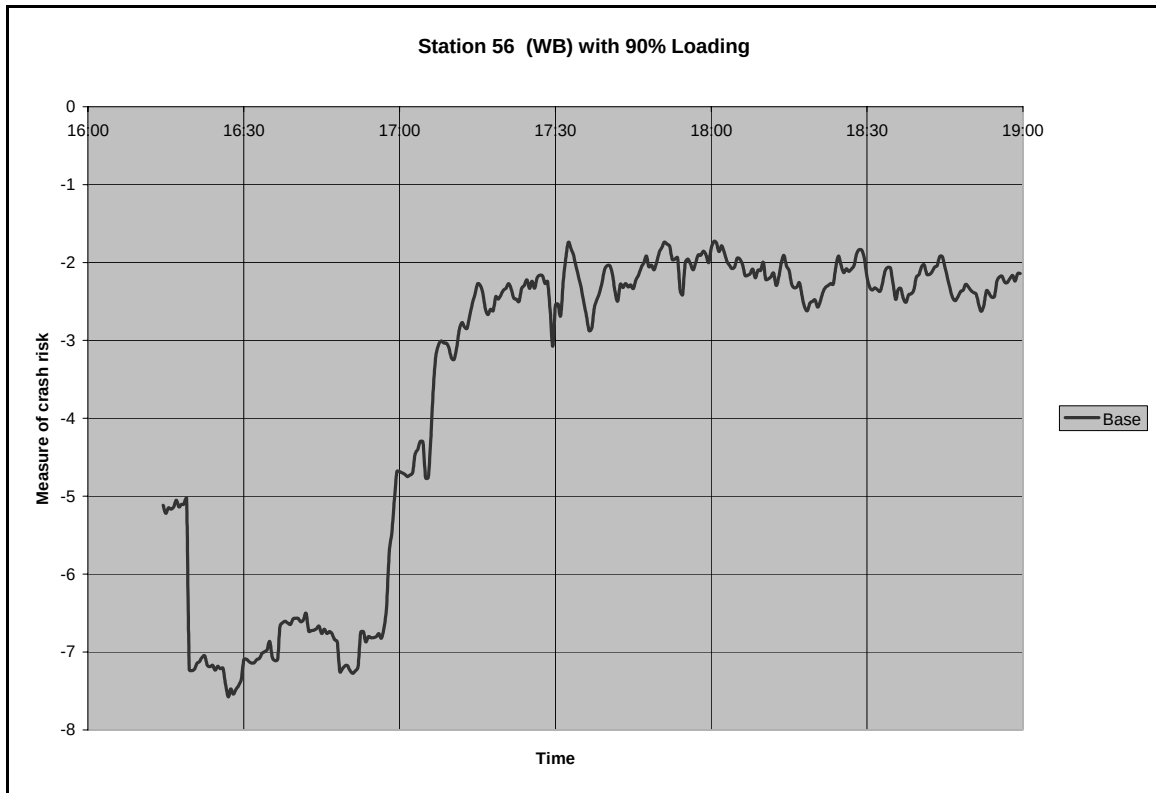


Figure 3-42: Base scenario for the worst case (Detector 56, upstream of the station of interest)

Table 3-7: Cases for ramp metering and VSL implementation.

Case	Implementation	Amount of Change	VSL	Ramp Metering
0	Abrupt	15 mph	None	NO
1	Abrupt	15 mph	17:30-18:00	NO
2	Abrupt	15 mph	16:30-18:00	NO
3	Abrupt	15 mph	17:00-18:30	NO
4	Abrupt	15 mph	None	YES
5	Abrupt	15 mph	17:30-18:00	YES
6	Abrupt	15 mph	16:30-18:00	YES
7	Abrupt	15 mph	17:00-18:30	YES

Figure 3-43 shows the implementation of the low-speed algorithm at Detector 56. From this figure it appears that ramp metering has an effect, but the VSL do not. To make sure that the algorithms are being applied correctly the average speed for each

scenario is measured in Figure 3-44. The figure shows that the ramp metering moves the average speed into the high-speed regime. As a result, the cases are rerun with the high-speed algorithm. Figure 3-45 shows the output. Station 56 does not show a difference in crash potential for the different scenarios. However, Station 58 (in Figure 3-46) does show a difference in crash potential.

The application of just the high-speed algorithm to all case is no more valid than the application of the low-speed algorithm so another method is explored. Figure 3-47 shows the high-speed cases evaluated by the high-speed algorithm and the low-speed cases evaluated by the low-speed algorithm. The algorithms do not appear to be on the same scale, because of the degree of difference in the scenarios and Figure 3-47 shows the opposite result as Figure 3-43 and Figure 3-46. The only option that appears to be open to compare high-speed and low-speed cases is Abdel-Aty's et al. (2004) algorithm, which will be referred to as the generic algorithm (Equation 2-3).

Figure 3-47 shows the implementation of the generic algorithm. Ramp metering appears to have a strong positive effect, but just as important the VSL show a difference in response when no ramp metering is present. Cases 2 and 3 appear to outperform the no VSL case, while Case 4 seems to perform worse than the base. Cases 2 and 3 correspond to early implementation of the VSL, with the earliest implementation, Case 2, slightly outperforming all other cases. This behavior is also reflected in Figure 3-43, the low speed modeling. However, the effectiveness of VSL is dwarfed by that of ramp metering. In the ramp metering cases VSL have no significant impact. Therefore the conclusion from these tests is that ramp metering should be explored, without VSL, in depth in order to improve safety during high congestion periods.

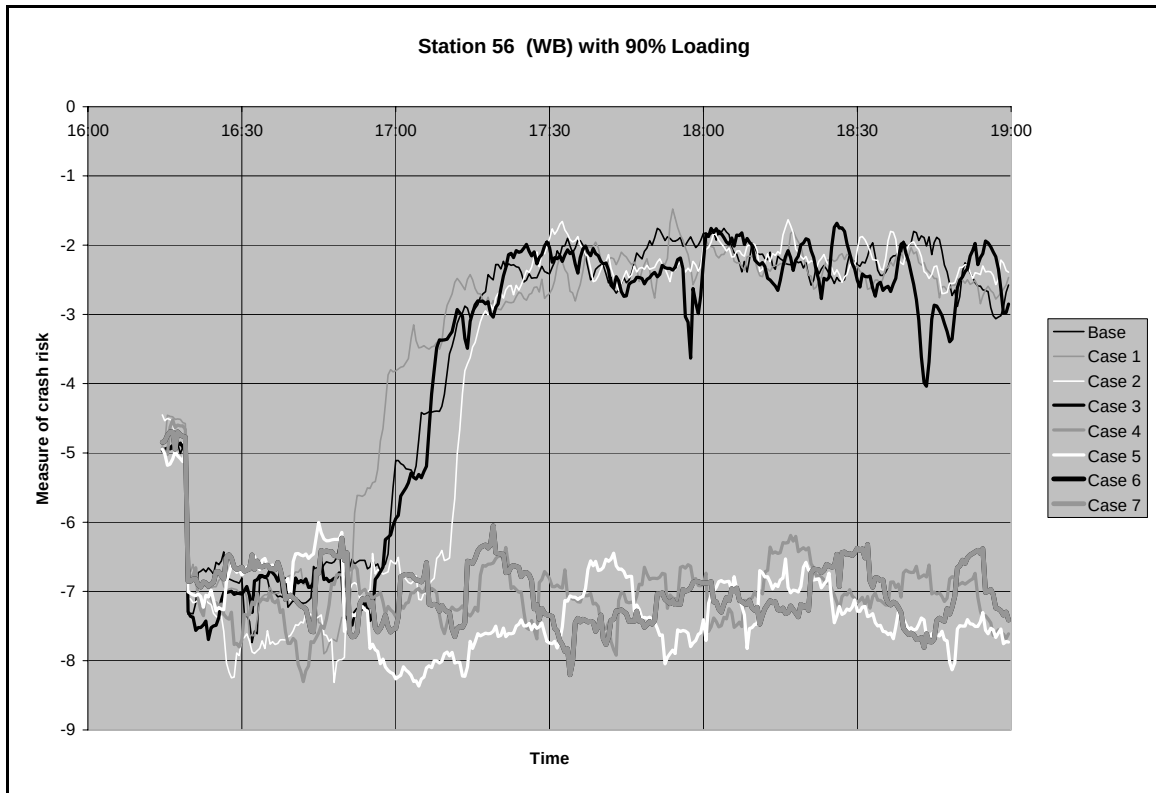


Figure 3-43: Scenarios from Table 3-7 evaluated using the low-speed algorithm at Detector 56.

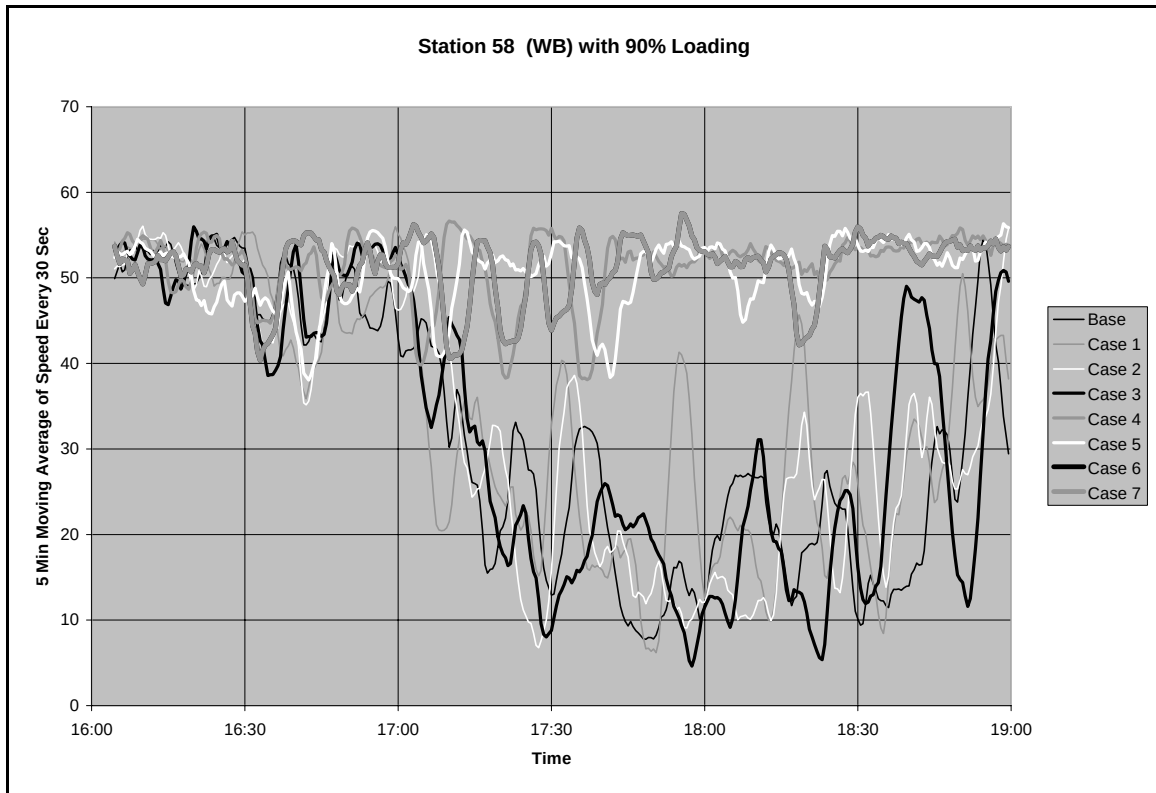


Figure 3-44: Scenarios from Table 3-7 average speeds at Detector 58 (Upstream).

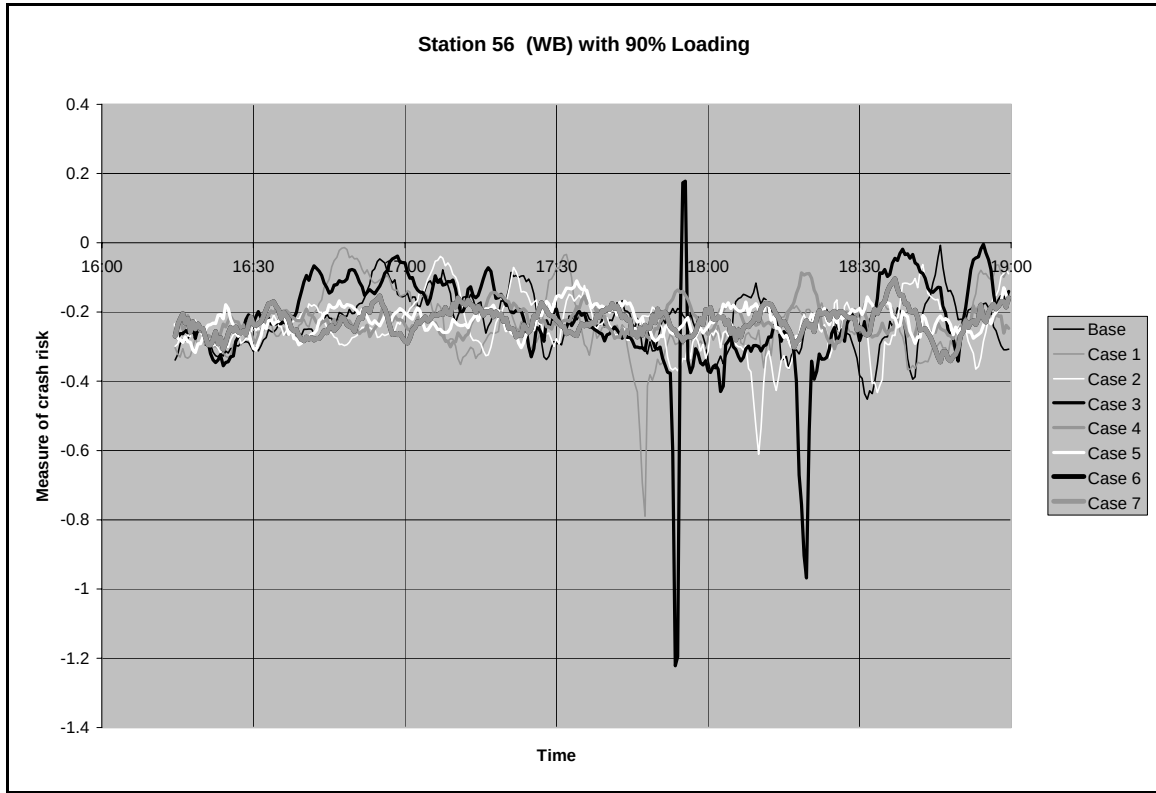


Figure 3-45: Scenarios from Table 3-7 evaluated using the high-speed algorithm at Detector 56.

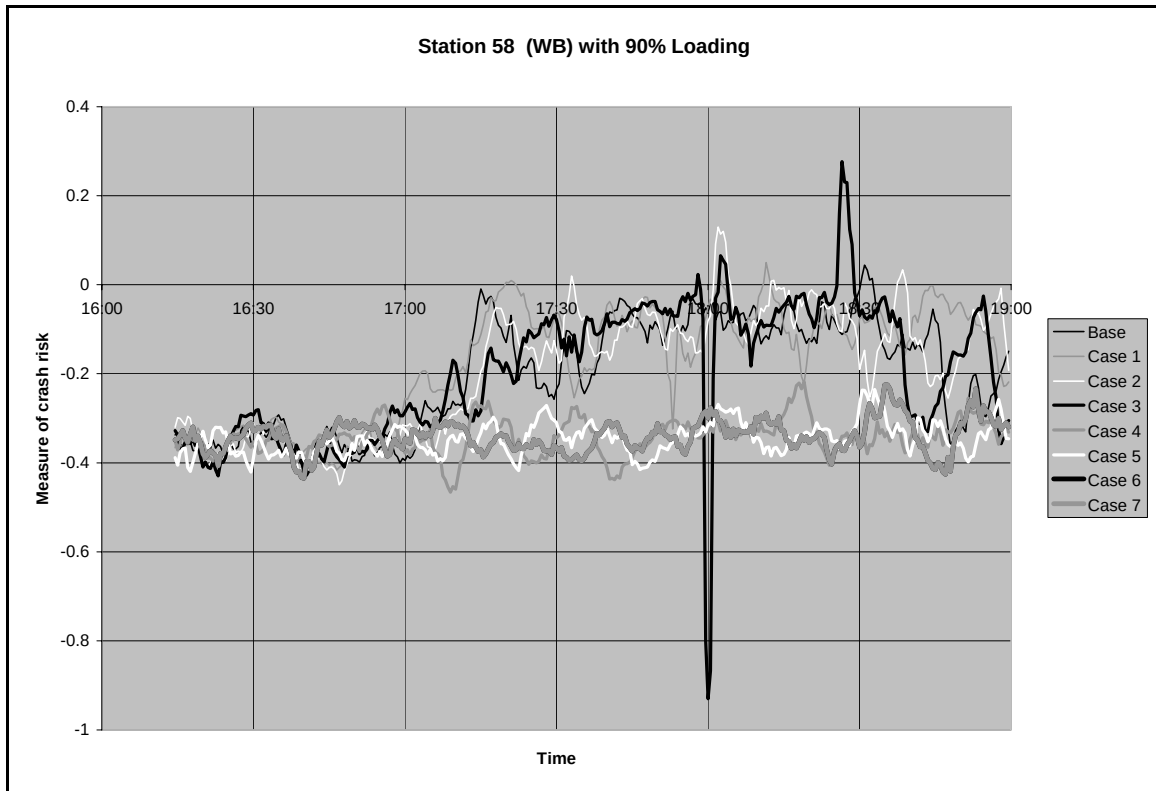


Figure 3-46: Scenarios from Table 3-7 evaluated using the high-speed and low-speed algorithm at Detector 58.

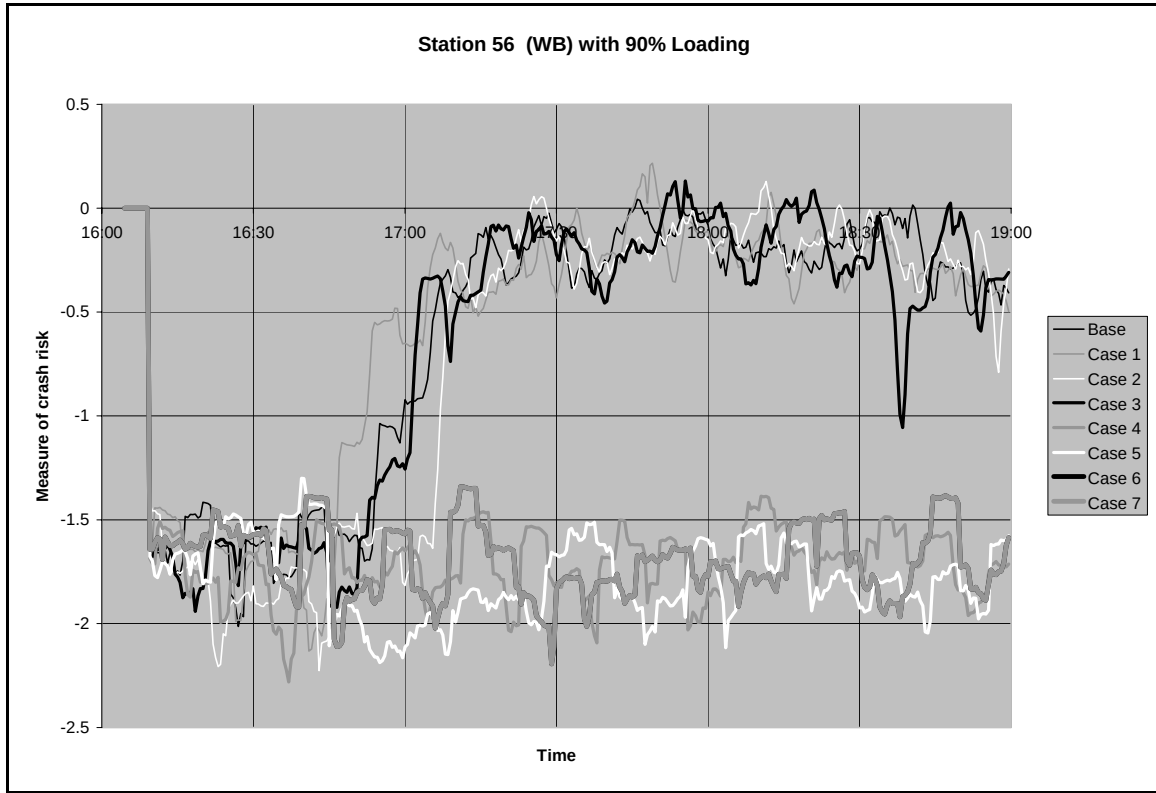


Figure 3-47: Scenarios from Table 3-7 evaluated using the generic algorithm at Detector 56.

3.6 High Speed Best Case Scenario

In the previous section various test cases have determined the best scenario for implementing VSL. While no conclusive result about the 90% loaded case maybe determined it does provide evidence that ramp metering and not VSL could lead to improved safety. The 60% case did show a clear positive result.

In the first round of tests the pattern of variable speed limit was tested. The patterns consisted of scenarios with a rate of change in speed limit over space that is either abrupt

or gradual and a maximum speed change of either 15 mph or 10 mph. The results showed that an abrupt change of 15 mph produced the best result.

In the second round of scenarios the length upstream and downstream of the detector of interest was tested. The different lengths for both the upstream and downstream changes attempted were 2 miles, 4 miles, and 5.5 miles. The results showed that a distance of 2 miles upstream and 2 miles downstream produced the best results.

In the third round of scenarios the rate of implementation of speed limits over time was tested. Changes in speed limit of 5 mph and 10 mph were tested along with time between changes of 5 minutes and 10 minutes. The results showed that a change of 5 mph every 5 minutes produced the best results.

In the forth and final test the effect of gap distance on crash potential was tested. Gap distance of 0 miles, 1 mile, 2 miles, and 3 miles were tested. The results showed that a gap of 0 miles produced the best results.

The best case is therefore an abrupt 15 mph change in speed limits both 2 miles upstream decrease and downstream increase, implemented in 5 mph increments every 10 minutes. Taking a closer look at the recommended case shows that the VSL have both positive and negative effect. Figures 3-48 and 3-49 show that the condition at station 51 and station 45 are worse with the implementation of VSL; while Figures 3-50 and 3-51 demonstrate that conditions at station 48 and station 47 have improved. Other stations shown in Figures 3-52 through 3-53 show minor differences when using VSL. While the trend appears to be that the leading and the station one mile downstream of the station of interest edge have problems, while the stations where the speed limit is being raised are improved. To make sure that these effects are coming as a result of the VSL and are not

due to the randomness in the system twenty matched runs of the base case and the recommended case are undertaken.

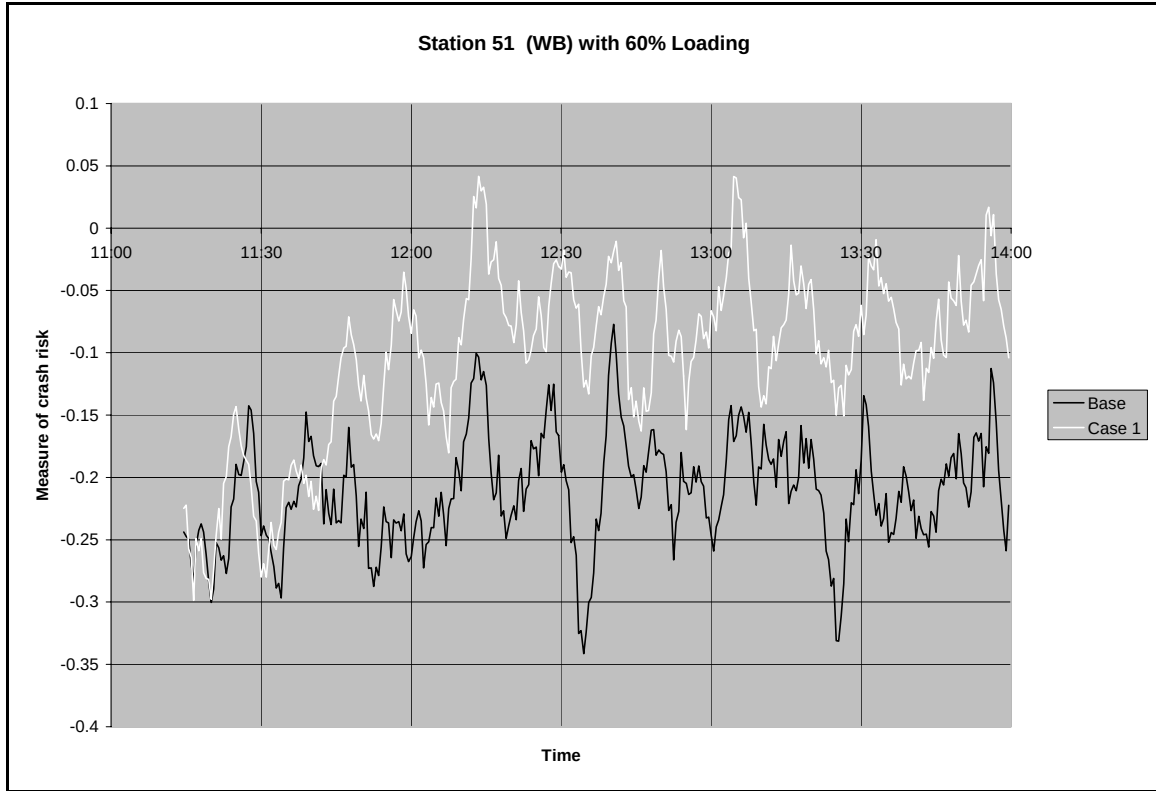


Figure 3-48: The crash potential at Station 51 (Upstream) for the best scenario under 60% loading.

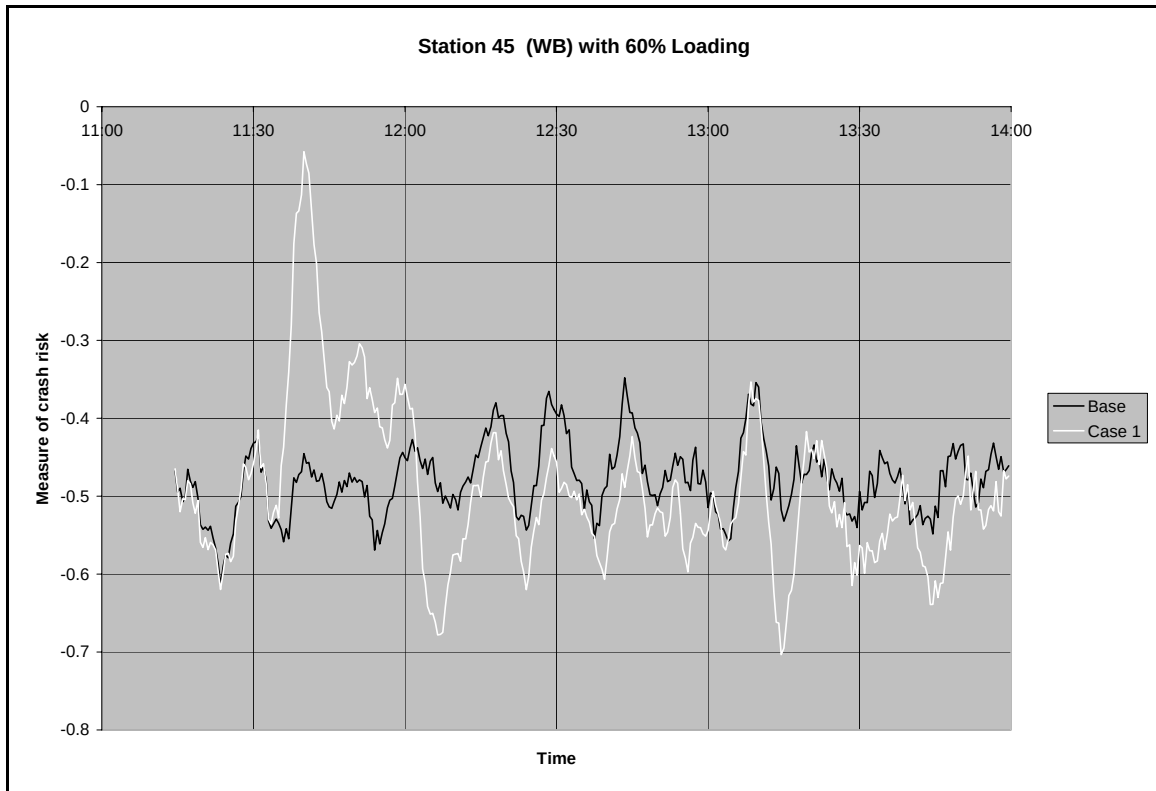


Figure 3-49: The crash potential at Station 45 (Downstream) for the best scenario under 60% loading.

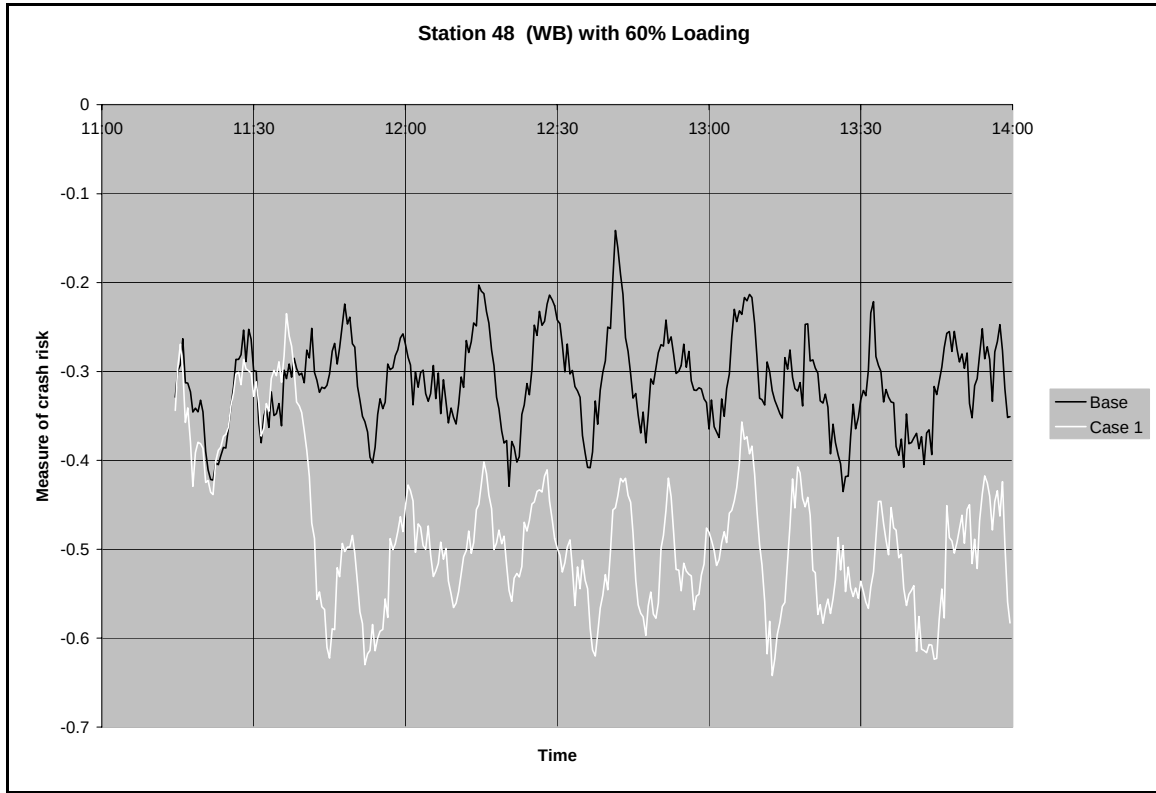


Figure 3-50: The crash potential at Station 48 (Upstream) for the best scenario under 60% loading.

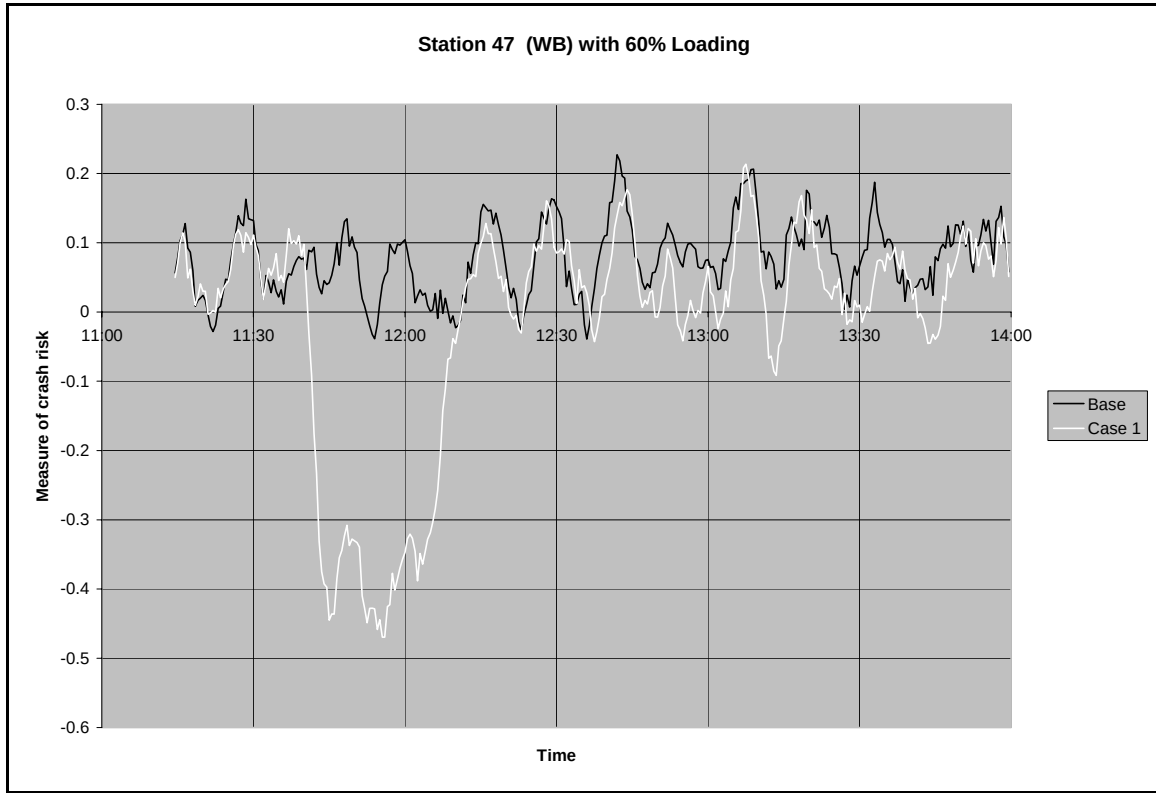


Figure 3-51: The crash potential at Station 47, (Location of Interest) the detector of interest, for the best scenario under 60% loading.

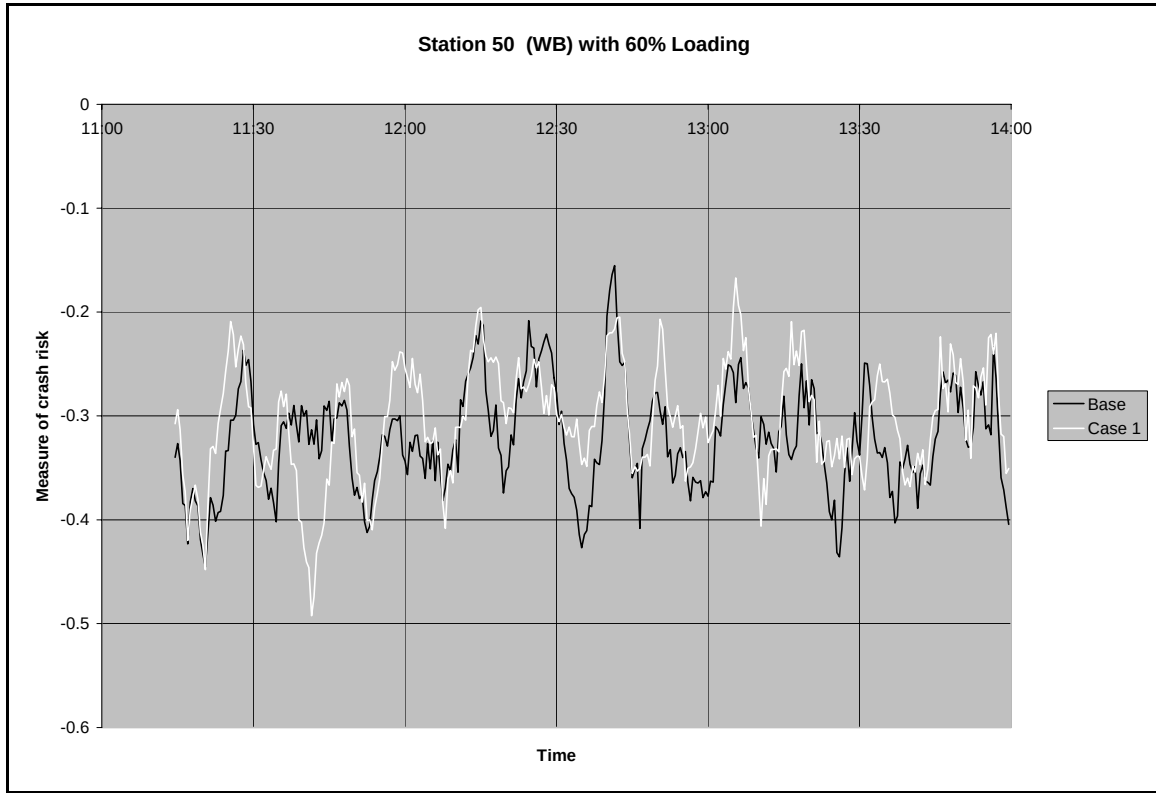


Figure 3-52: The crash potential at Station 50 (Upstream) for the best scenario under 60% loading.

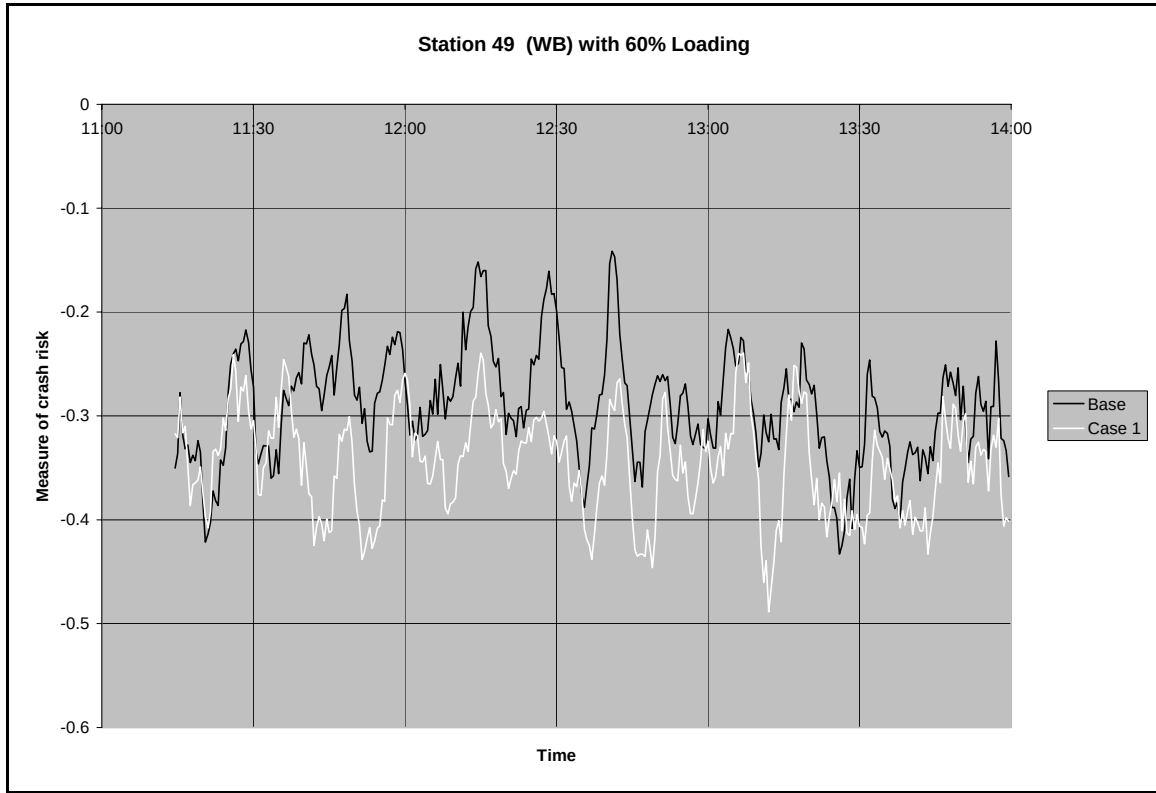


Figure 3-53: The crash potential at Station 49 (Upstream) for the best scenario under 60% loading.

3.6.1 Statistical analysis

After running nineteen matched cases of a base scenario and a best-case scenario the results are analyzed using a t-test. The half width at 95% for each station is shown in Table 3-8. The resulting ranges are plotted in Figure 3-54. The half width is determined by multiplying the t-value by the standard deviation divided by the square root of the number of observations, 19. For the differences that are significant the percent change of each are calculated. The percent change is the difference between the best case and the base case (less their half width) normalized by the base case

value. The result is shown in Table 3-8. Note that the dark gray boxes are used to show an improvement, while the lighter shows a worsened condition. All values are risk values calculated using Dr. Abdel-Aty et al. (2005) high-speed model.

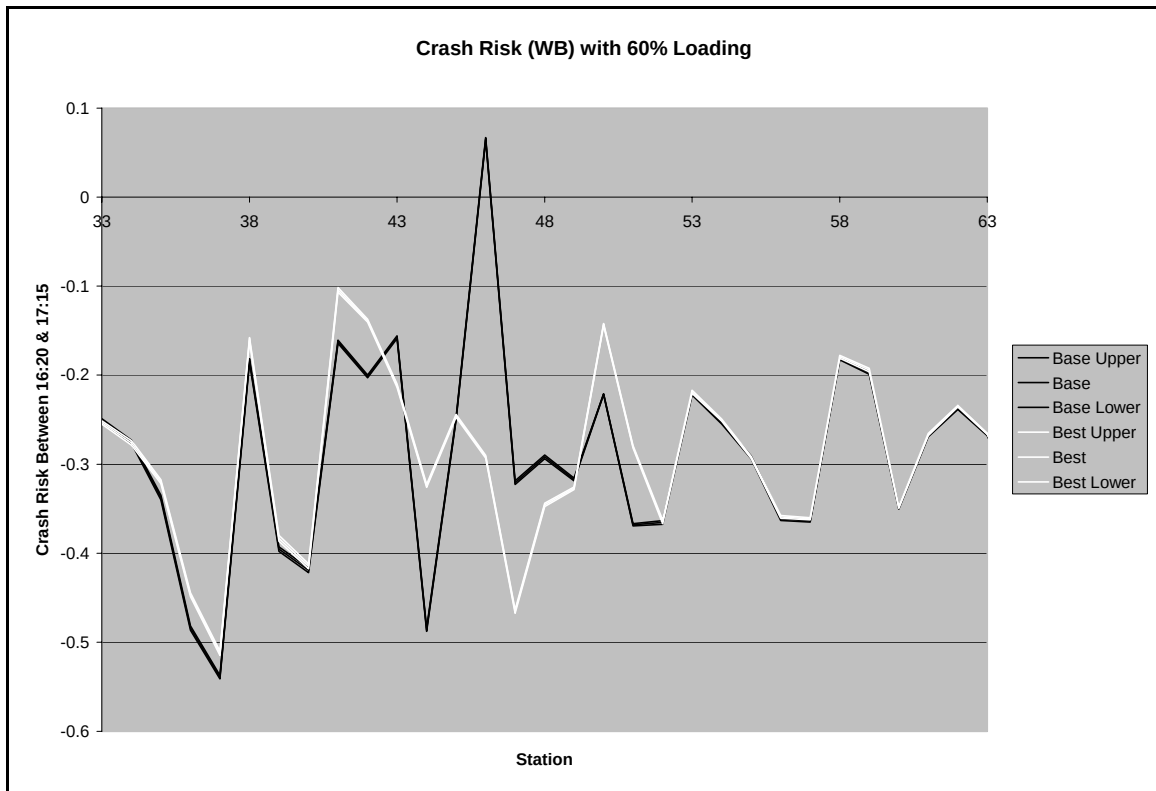


Figure 3-54: Crash risk with confidence intervals for all stations.

Table 3-8: Crash risk with half-width and percent change for all stations.

	Station	Best Case		Base Case		Percent Change
		Average	Half Width	Average	Half Width	
Upstream	64	-0.267	0.001	-0.266	0.001	
	63	-0.238	0.001	-0.235	0.001	1%
	62	-0.268	0.001	-0.267	0.001	
	61	-0.349	0.001	-0.348	0.001	
	60	-0.197	0.002	-0.194	0.002	
	59	-0.181	0.001	-0.179	0.001	
	58	-0.364	0.001	-0.361	0.001	1%
	57	-0.361	0.002	-0.359	0.001	
	56	-0.292	0.002	-0.292	0.001	
	55	-0.254	0.001	-0.250	0.001	1%
	54	-0.221	0.002	-0.219	0.002	
	53	-0.365	0.002	-0.365	0.002	
	52	-0.368	0.001	-0.280	0.001	31%
	51	-0.221	0.001	-0.143	0.001	55%
	50	-0.317	0.002	-0.327	0.001	-3%
	49	-0.292	0.002	-0.346	0.002	-16%
	48	-0.321	0.002	-0.465	0.002	-31%
	47	0.065	0.002	-0.292	0.002	-122%
Downstream	46	-0.249	0.002	-0.246	0.002	1%
	45	-0.486	0.002	-0.324	0.002	50%
	44	-0.157	0.002	-0.212	0.002	-26%
	43	-0.201	0.002	-0.139	0.002	45%
	42	-0.162	0.002	-0.104	0.003	57%
	41	-0.419	0.003	-0.415	0.003	
	40	-0.395	0.003	-0.383	0.003	3%
	39	-0.184	0.002	-0.160	0.002	15%
	38	-0.539	0.002	-0.512	0.003	5%
	37	-0.484	0.002	-0.447	0.002	8%
	36	-0.337	0.003	-0.319	0.002	5%
	35	-0.275	0.002	-0.276	0.002	
	34	-0.251	0.002	-0.253	0.002	

3.6.1.1 Station of Interest

The station of interest, Station 47, shows the largest change. With a positive change of 122% conditions at Station 47 move from being the most at risk station to being the

least at risk station. Figure 3-51 also shows that the situation just upstream of the station of interest shows great improvement.

3.6.1.2 Upstream of the Station of Interest

As mentioned before the possibility of crash relocation is the biggest concern. Intuitively, crash potential would be more likely to relocate upstream of the detector of interest in the high-speed case. Abdel-Aty's et al. (2005) algorithm can be interpreted to say that packs of faster moving vehicle coming into contact with slower traffic cause high crash potential. If this is the case after the speed limits have been change there is a region of high speed that suddenly comes on a lower speed limit. The lower speed limit increases the density of the vehicles raising the loop detector's occupancy, effectively creating the situation that the VSL are attempting to solve.

However, looking at the simulation results shows that the upstream the situation does not change much, except at the detectors where the sudden change in speed takes place. Here at station 51 and station 52 the crash potential increases, but never near the level of what station 47 was prior to the implementation of VSL.

3.6.1.3 Downstream of the Station of Interest

Making a case for crash potential relocating to a downstream station is a little more difficult. Increasing the speed limit could clear large packs, but geometries downstream (a lane drop or curve) could induce a bottleneck that causes packs to reform. However,

VSL themselves would not be thought of as the direct cause of these increases in crash potential.

The results here are surprising. The crash potential appears to relocate to the detectors downstream of the detector of interest. The relocation occurs both one and two miles downstream of detector 47. However, the station 1.5 miles downstream sees an improvement in crash potential. Geometry could be the culprit as there are curves at these locations as there are curves present near these detectors.

3.6.2 Travel time analysis

With a best case scenario for safety in hand the effect of the scenario on travel time is evaluated. Because base scenario is an off-peak case no improvement in travel time is expected to be seen, instead a success implementation is expect to have minimal effect on travel time.

To analyze the effect on travel time the same twenty runs of the base case and best case from section 3.6 are used and the travel time on each link across the entire three-hour simulation is summed. Because the scenarios have a common random number seed a paired t-test can be used. The paired t-test and a 95% confidence interval for both scenarios are shown in Table 3-9.

Table 3-9 shows there is a significant reduction in travel time due to the implementation of VSL. This result means that the best case can be concluded to effectively reduce crash potential and decrease travel times, meaning that crash migration is the only foreseeable drawback. In this case the overall maximum reduction in crash

potential from 0.065 to -.104 even when considering crash migration, which is a significant improvement in safety for the roadway.

Table 3-9: T-test for the network travel time (minutes)

	Base Scenario	Best Scenario	Difference
Run 1	36868.9	36754.5	-114.4
Run 2	36510.6	36526.3	15.7
Run 3	36632.9	36529.3	-103.6
Run 4	36775.7	36763.6	-12.1
Run 5	36718.4	36563	-155.4
Run 6	36823.4	36644.9	-178.5
Run 7	36683.6	36456.4	-227.2
Run 8	36789.9	36612.5	-177.4
Run 9	36838.6	36628.1	-210.5
Run 10	36954.3	36618.8	-335.5
Run 11	36943	36487.8	-455.2
Run 12	36733.1	36598.8	-134.3
Run 13	36844.5	36645.4	-199.1
Run 14	36810.1	36743	-67.1
Run 15	36799.5	36720	-79.5
Run 16	36626.4	36444.7	-181.7
Run 17	36818.2	36795.3	-22.9
Run 18	36733.7	36551.4	-182.3
Run 19	36886.8	36419.8	-467
Average	36569.27	36608.655	-173.053
95% C.I.	421.2758197	49.87324454	60.38447

3.6.3 Model application

For completeness the average speed of the base and best case scenarios are determined and shown to be within the regime of the applied model, as shown in Figure 3-55.

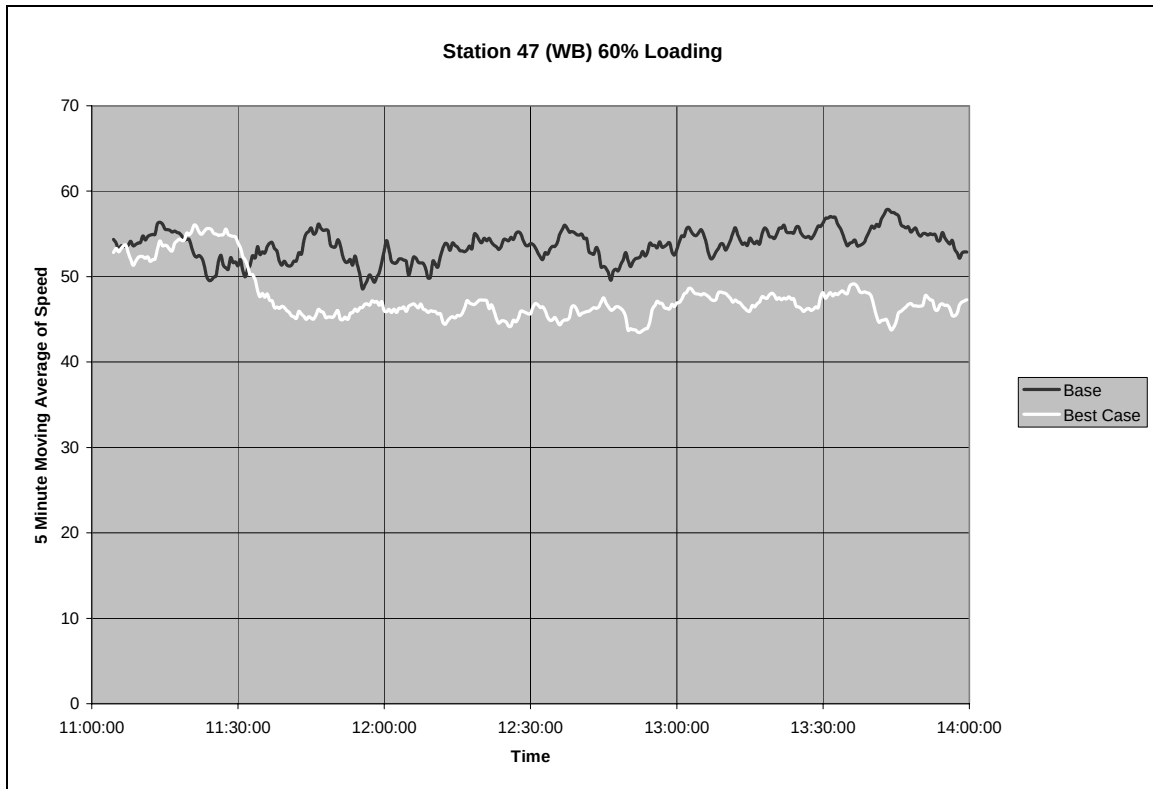


Figure 3-55: Base and Best Case Scenarios' average speed at Detector 47 (Detector of Interest).

3.7 Best Low Speed Scenario

The best low-speed case involves the implementation of drastic ramp metering just upstream of the detector of interest without VSL. VSL has effect when ramp metering is not used, but fails to have effect when ramp metering is used. Figure 3-55 shows a plot of the best case versus the base case for detector 56, the detector of interest.

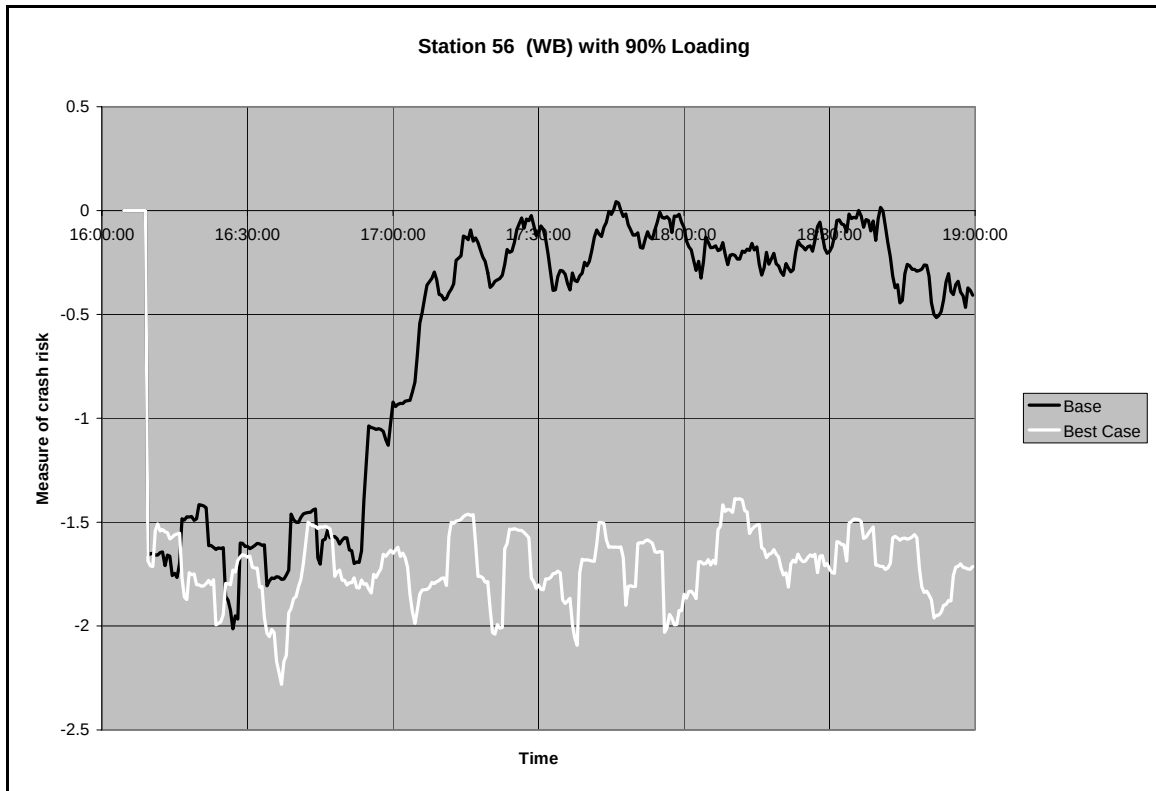


Figure 3-56 Best scenario from Table 3-7 (Case 4) evaluated using the generic algorithm at Detector 56

3.8 Conclusions from Preliminary analysis: Stage I

The objective of this part of the research was to explore different ITS strategies to improve safety on Interstate 4 (I-4), specifically variable speed limits (VSL) and ramp metering applications. Through the implementation of variable speed limits on I-4, via simulation, a best case for improving safety was to be determined. Multiple scenarios with characteristic trends in them were to be used to generate general “rules” or best practices that result in an optimal safe condition. The work was to focus on a single site for improvement. However, the relocation of crash risk was to be considered when making recommendations and the single site location was limited only by the length of

roadway simulated. Two loading conditions, a high speed (low loading, 60%) and a low speed (high loading 90%), are considered.

The objective is successfully achieved for the high speed (greater than 37.5 MPH) case. Through 4 rounds of evaluations a final best model was obtained. The first round considered is the spatial variation of various speed limits (whether upstream lowering the speed limits and downstream raising speed limits is better than just downstream raising speed limits), speed limit variation (whether a 10 mph maximum change is better than a 15 mph speed limit change), and the combination of the two variables (whether gradually changing the speed limit over space or abruptly changing the speed limit over space is better). The abrupt 15 mph change both upstream and downstream are concluded to produce the best case. The second round tested the spatial location of changing speed limit signs. In this test the upstream and downstream distances are set as 2 miles, 4 miles, or 5.5 miles and the impact on crash potential evaluated. The shortest downstream distance, 2 miles, improved safety, while the shortest upstream distance, 2 miles, was used to have the least impact on travel time. The third round tested temporal implementation of the VSL on the best case scenario from test 2. Step size for VSL implementation was either 5 mph or 10 mph and step either every five minutes or every 10 minutes. The most gradual implementation, 5 mph steps every 10 minutes, had the best result. The forth round tested the gap distance used, after the detector of high incidence, before raising the speed limit. A gap of 0 miles greatly outperformed all other cases.

As a result of this best practice determination existing VSL can be thought of in a new light. Instead of just using them before or during periods of high congestion, VSL

can be thought of as applicable during off-peak periods as well. Instead of aiming to reduce congestion time the VSL can be used to effectively reduce the hazard of certain locations. This study also concluded that by using VSL in off-peak conditions travel time is not effected, so there are no negative effects of having implemented their use with regard to delay. When implementing VSL practitioners should follow the recommendations of this study:

- Gradually introduce speed changes in time (5 mph every 10 minutes)
- Abruptly introduce speed changes in space (No gap distance)
- Use upstream reductions in speed and downstream increases in speed
- Changes speed limit by large values (up to 15 mph)
- Short upstream and downstream distance are sufficient (2 miles each)

While the low-speed case analysis did not show clear results, it did indicate future areas that should be studied. VSL were introduced via the scenario shown in Table 3-10 and through the best case determined in the high-speed case. The scenario has a large change in speed limit and is introduced gradually in space and time. Downstream raising is not performed, because the driver's speed is being limited by factors outside of the speed limit and therefore any change is assumed to not have an effect. VSL implementation showed no significant change in crash potential during the simulation. As a result more drastic measures were taken and ramp metering was approximated using 60% loading from all ramps in one case and through shutting off a problem ramp in another case. While the first implementation of ramp metering did not have an effect the ramp specific approach appears to effectively reduce crash potential.

Table 3-10: The test scenario for the low speed model.

	t-15min	t-10min	t-5min	t	t+5min
x+2mi	0	0	-5	-5	0
x+1.5mi	0	-5	-10	-10	-5
x+1mi	-5	-10	-15	-15	-5
x	-5	-10	-15	-15	-5
x-.5mi	-5	-5	-5	-5	-5

The reduction in crash potential was not analyzed and the method of ramp metering is crude, but is suggestive that if ramp metering via a ramp just downstream of the detector of interest may be used to effectively reduce congestion that there might be a safety benefit as well. Future research with a more realistic implementation of ramp metering should be looked into because of this result. Also Borough's (1997) work suggests that lane changing behavior is an important factor in high congestion situation, therefore the impact of VSL implementation on lane change behavior could be an important factor that is missing for PARAMICS. It was research team's belief that this factor may be the most significant factor in observing reductions in crash due to VSL implementation, and should be studied in detail. The findings from this phase of preliminary analysis would be referred and utilized in the next phase of preliminary explorations.

CHAPTER 4

PRELIMINARY EVALUATIONS OF ITS STRATEGIES: RAMP METERING AND RAMP METERING COMBINED WITH VSL

The last three chapters described the process of selection of the simulation package and simulation of a 9-mile segment of the Interstate-4 corridor under consideration for this research. The previous chapter also showed the benefits of the Variable Speed Limits separately in improving safety on freeway sections, especially in moderate to high speed scenarios. The work presented in this and the next chapter attempts to try to enhance the work in two directions. First, it considers the application of ramp metering along with the VSL over a whole simulated road network of Interstate-4. The work presented in the last chapter ITS (mainly VSL) strategies were implemented over multiple locations and their effect observed over the whole network. The study also explores the possibility of coordination of VSL with ramp metering strategies in an attempt to maximize their utility in mitigating the risk of crashes. In conducting this effort our research premise was that a strategy that combines both these ITS technologies in a system would result in greater system wide benefits with lesser amount of resource usage. All the strategies are tested from an implementation perspective to ensure their assimilation into everyday roadway operations.

The work presented in this chapter had three major objectives:

- Test Feedback Ramp Metering Strategies and evaluate their network wide safety benefits
- Test Variable Speed Limits effectiveness in congested situations for improving safety over the network

- Test coordinated strategies for Ramp metering and Variable speed limits and determine their safety benefits.

The proposed work also attempts to improve the calibration and validation work presented in the earlier chapter(s) (Chapters 3 and 4). The advances documented in this chapter from the last chapters are the result of research team's increased familiarity with the details of the microscopic traffic simulation package and experience in crash risk estimation.

4.1 Background

Ramp Metering as studied by Lee et al. (2005) used ALINEA as the algorithm of choice and was tested on simulation of a small section (2.9 mile) of the freeway with a single on-ramp. Although this proved that Ramp Metering effectively improves safety, no recommendations were made to the implementation of the measure on freeway networks. Chu et al. (2003) in their work show that ALINEA as an algorithm can be implemented after proper calibration. Lee et al. (2005) used values based on literature and did not calibrate their ramp metering model for implementation. Besides one of the key factors when looking at the implementation of any application is the fact that implementation at any location is bound to have effects on the network as a whole and not just at upstream and downstream of the ramp in question. This means that while we might be improving the safety at a particular location there might be another location where the conditions become crash prone because of our implementation effort. This stands for ramp metering and Variable speed limits (Abdel- Aty et al., 2006).

As such the objective of this study was to test the effectiveness of the ITS strategies like Ramp Metering and Variable Speed Limits, and also to test the different

configuration of these strategies to come up with a recommendation towards their implementation. An exhaustive calibration process has been carried out on the simulation network to make sure that the simulated conditions are as close as possible to the actual scenario on the freeway. This process would be described in detail later in this chapter. This calibrated scenario of the actual roadway recreated in simulation has been labeled as the Base Case. It serves as the control case through out this study and will be referred to time and again for comparison with different strategies.

The work described in the last chapter shows that VSL is effective only for stations which operate at moderate to high speeds and not for stations which operate at lower speeds (defined as consistent speeds of less than 37.5 mph). Thus one of the major factors in this design had to be the choice of stations. The stations chosen for testing the scenarios for all the low speed stations were Detector Station 33-49. For the East Bound Direction, stations 33- 49 are tabulated along with their classification as low speed or high speed in Table 4-1. All the stations in this range belonged to the low speed regimes i.e. for the length of the simulation operated at speeds less than 37.5 mph. This holds true for these stations in the actual conditions and the speeds prevalent at these stations for the congested hour from 5 P.M to 6 P.M in evening peak are shown as Mean Hourly Speeds in Table 4-1. Also, from a study of the loop data it can be easily seen that these stations operate in the low speed regime for most of the time during evening peak. This classification for stations would hold for the cases where we test any of our strategies and evaluate them also. This could be done because preliminary analysis shows that these measures implemented under conditions of peak hour loading do not affect the mean

speeds of the stations enough to change the speed regime (convert a low speed station to a high speed station). The primary effect of these measures on the safety index is by influencing factors like Variation of speeds, etc. It should be mentioned here for understanding the stations on Interstate-4 are numbered in increasing order from West to East in both directions using numbering 2,3.....69. A detailed explanation of this terminology follows in Section 5.1.2.

Table 4-1 Station Classification for I-4 East Bound based on Speed Regime

Station	Hourly Mean Speed	Classification
33	35.432	Low Speed
34	27.324	Low Speed
35	24.7180	Low Speed
36	22.763	Low Speed
37	34.3059	Low Speed
38	30.1890	Low Speed
39	23.8280	Low Speed
40	16.2494	Low Speed
41	18.890	Low Speed
42	25.67226	Low Speed
43	28.5642	Low Speed
44	35.663	Low Speed
45	36.240	Low Speed
46	35.080	Low Speed
47	36.802	Low Speed
48	35.432	Low Speed
49	34.675	Low Speed

4.2 Risk Index

One of the key areas of interest in the studies related to safety has been the development of numerical models which might prove to be good surrogate indicators of the risk of crash. Abdel-Aty et al. (2005) developed a model based on matched case control logistic regression. The purpose of the matched case-control analysis is to explore the effects of independent variables of interest on the binary outcome (crash or no crash) while controlling for other confounding variable (e.g. location, time, etc.). This extension of logistic regression has been described below in the context of the present research problem. Even though the discussion on the derivation of this index was provided in Chapter 3; it is worth repeating for the sake of continuity.

4.2.1 Derivation of Risk Index

Let's assume that there are N strata with l case and m controls in each stratum. The conditional likelihood for the j^{th} stratum is the probability of the observed data given the total number of observations and the number of crashes observed in the stratum. The probability of any observation in a stratum being a crash may be modeled using the following linear logistic regression model:

$$\text{logit}(p_j(x_{ij})) = \alpha_j + \beta_1 x_{1ij} + \beta_2 x_{2ij} + \dots + \beta_k x_{kij}$$

...Equation 4-1

where $p_j(x_{ij})$ is the probability that the i^{th} observation in the j^{th} stratum is a crash; $x_{ij} = (x_{1ij}, x_{2ij}, \dots, x_{kij})$ is the vector of k traffic flow variables $x_1, x_2, \dots, x_k$; $i = 0, 1, 2, \dots, m$; and $j = 1, 2, \dots, N$.

Note that the intercept term α_j summarizes the effect of control variables (used to form the strata) on the crash probability and would be different for different strata. In

order to account for the stratification in the analysis, a conditional likelihood is constructed. The complex mathematical derivation of the relevant likelihood function is omitted here and the reader is referred to Collett (1991) for more details. This conditional likelihood function is independent of the intercept terms $\alpha_1, \alpha_2, \dots, \alpha_N$ (Collett, 1991). So the effects of matching variables cannot be estimated and Equation 4-1 cannot be used to estimate crash probabilities. However, the values of the β parameters that maximize the conditional likelihood function would also be estimates of β coefficients in Equation 4-1. These estimates are log odds ratios and can be used to approximate the relative risk of a crash.

The log odds ratios can also be used for crash risk estimation purposes under this matched crash-non-crash analysis. Consider two observation vectors $x_{1j} = (x_{11j}, x_{21j}, \dots, x_{k1j})$ and $x_{2j} = (x_{12j}, x_{22j}, \dots, x_{k2j})$ from the j^{th} strata on the k traffic flow variables. The log odds ratio of crash occurrence due to traffic flow vector x_{1j} relative to vector x_{2j} may be derived from Equation 4-1 and its form is shown in Equation 4-2.

$$\log \left\{ \frac{P(x_{1j})/[1-P(x_{1j})]}{P(x_{2j})/[1-P(x_{2j})]} \right\} = \beta_1(x_{11j} - x_{21j}) + \beta_2(x_{21j} - x_{22j}) + \dots + \beta_k(x_{k1j} - x_{k2j})$$

...Equation 4-2

The right hand side of Equation 4-2 depends only on β_j , therefore the estimate for log odds ratio may be obtained using estimated β coefficients. One may utilize the above relative log odds ratio for predicting crashes by replacing x_{2j} by the vector of values of the traffic flow variables in the j^{th} stratum under normal traffic conditions. Simple average of all non-crash observations within the stratum for each variable may conveniently be used. If $\bar{x}_{2j} = (\bar{x}_{12j}, \bar{x}_{22j}, \dots, \bar{x}_{k2j})$ denotes the vector of mean values of the

k variables over non-crash cases within the j^{th} stratum, then the log odds of crash relative to non-crash may be approximated by Equation 4-3.

$$\log \left\{ \frac{P(x_{1j})/[1-P(x_{1j})]}{P(x_{2j})/[1-P(x_{2j})]} \right\} = \beta_1(x_{11j} - \bar{x}_{12j}) + \beta_2(x_{21j} - \bar{x}_{22j}) + \dots + \beta_p(x_{p1j} - \bar{x}_{p2j})$$

... Equation 4-3

The above log odds ratio can then be used to predict crashes by establishing a threshold value that yields desirable classification accuracy. Alternatively, it also serves as the measure of the risk of the crash in a comparison scenario. A decreased value for this index is therefore signifying a decrease in the likelihood of a crash and vice-versa. Based upon these Abdel-Aty et al. (2005) developed two models for moderate to high speed (average speeds > 37.5mph) and low speed regime (average speeds < 37.5mph).

4.2.2 Terminology

The inductive loops are placed roughly half a mile each on a 36 mile stretch of interstate 4 starting from South of Disney to north of Maitland Blvd. The stations are numbered from 2 through 69 from South to North (West to East) in both directions. Figure 4-1 demonstrates the numbering of these loop detectors. This study does not use the whole network of I-4 with loop detectors on it.

Each of these stations consists of dual loops in each direction and measures average speed, volume and occupancy over 30 seconds period on each of the through travel lane. The models used have a number of variables like 5 minute average of speeds 5- 10 minutes before our time of interest at the detector station upstream from our station of interest. These variables are named using a specific terminology whereas terms like

CVSF2, AOE3, etc. are used. To understand the variables it is necessary to look at the naming convention for the stations and the meaning of each of the letters in the variable. The station of interest in all the models is assigned the letter F. The station upstream of it is assigned the letter E and the station downstream is assigned the letter G and so on. This naming convention is also shown in Figure 4-1. As can be seen in Figure 4-1, the letters in the alphabet increase with increasing station number in the East Bound Direction and decrease with increasing numbers in the West Bound Direction. The lettering changes whenever we look at a new station of interest and that station is always assigned the letter F.

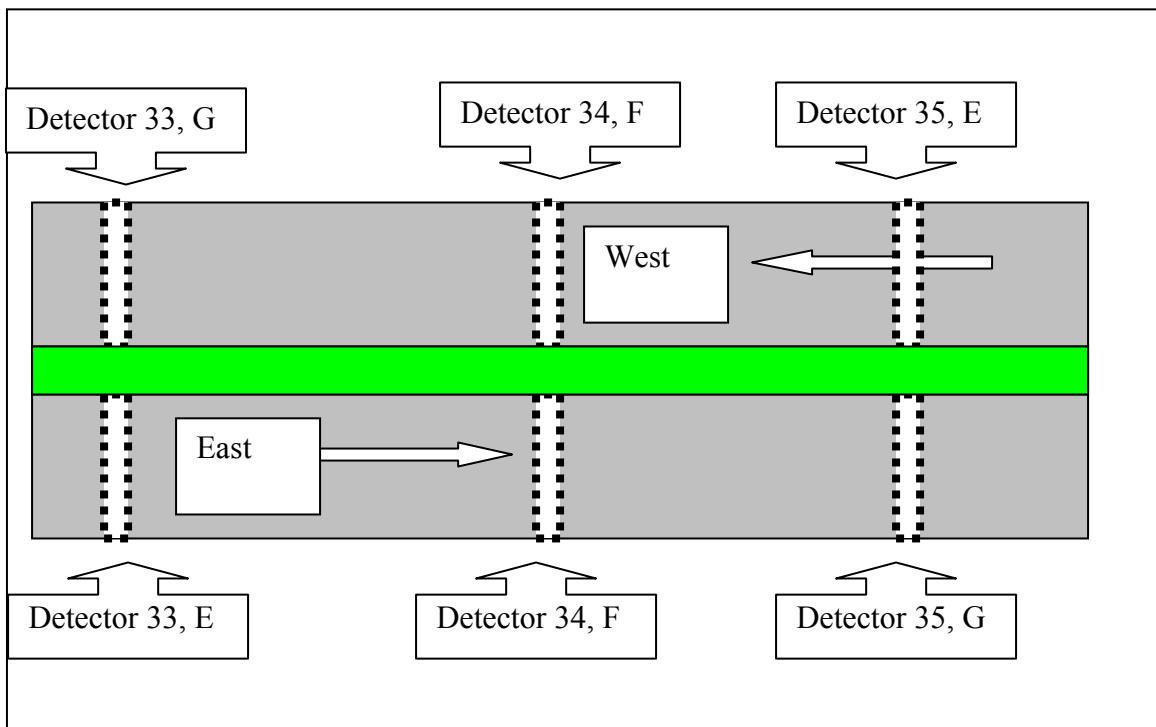


Figure 4-1 Detector Numbering on I-4

The first letter in the variable name can be A, S or C. The A stands for average over five minutes, the S stands for standard deviation over 5 minutes and CV stands for Coefficient of variation. The next letter represents the specific parameter we are looking

at and could be S, V or O, each representing Speed, Volume and Occupancy, respectively. The last letter represents the station where we are measuring that parameter e.g. D, for two stations upstream of station of interest. The number at the end depicts the time slice number before our time of interest. The time slices are five minutes in length so slice 1 would be 0- 5 minutes before the time of interest, slice 2 would be 5- 10 minutes before time of interest and so on. The only exception to this single letter naming convention is variables starting with CV which represent coefficient of variation (standard deviation divided by mean) for the parameter.

4.2.3 Model Interpretation: Low-speed Regime

Both coefficients of variation in speed (measured during time *slice 2* as well as 3) from the station closest to the crash location (*Station F*) remain in the model, with one during time slice 2 (5-10 minutes) being much more significant. Both *LogCVSF2* and *LogCVSF3* have positive model coefficients implying highly varying speeds around the crash location. It indicates frequent formation and dissipation of queues. Also, with *LogCVSF2* having larger and far more significant hazard ratio than *LogCVSF3*, it is apparent that persisting variation in speed is hazardous, however, the shorter term effect (5-10 minutes) having stronger influence on crash occurrence (note that *logCVSF2* and *logCVSF3* are expected to be correlated, as is the case here, however retaining both in the model shows the persistence of speed variation starting at 15 minutes before the crash). This high *LogCVSF2* is also coupled with low standard deviation in volume (indicated by negative coefficient of *SVF2*) implying the number of cars on three lanes remains fairly equal over time. High variation in speeds with little or no difference in volume across lanes might cause drivers in the slow lane to make lane changes; resulting in increased

odds of experiencing a crash. The other factors in the model are *AOH3* and *AOE2*, both with positive coefficients. High occupancy at one mile downstream (during 10-15 minutes slice) and half a mile upstream (during 5-10 minutes slice) indicates the backward propagation of congested flow regime. Note that in 5-15 minutes period high occupancy conditions “travel” about 1.5 mile on the freeway causing speed variation at the crash location to rise. It is interesting to note that the *Station F* and *Station G* occupancy does not enter in the model simply because high occupancy conditions pass through these stations within the time frame used to aggregate the data. In other words, 5 minutes is a large enough interval for a shock-wave to travel from *Station G* to *Station E*. The interpretations (indicating congested conditions with frequent formation and dissipation of queues) largely fit into the mechanism of rear-end crashes, which are the most common type of collision on the freeway at least under low-speed regime. The final model is presented in Equation 4-4.

$$Risk\_Index = 2.64827 \text{ LogCVSF2} + 0.88842 \text{ LogCVSF3} + 1.33966 \text{ LogAOE2} + 0.97766 \text{ LogAOH3} - 0.43603 \text{ SVF2}$$

...Equation 4-4

4.2.4 Model interpretation: high speed regime

The most interesting aspect of the model is that it does not include the coefficient of variation in speed as one of the factors. One possible reason could be that since the model is for the high speed regime, the coefficient of variation in speed may not be able to capture the variation in speed due to the large denominator. This led us to estimate the effect of standard deviation of speed, which also turned out to be insignificant. Another

important feature of this model is that the coefficient of *LogAOF2* is negative indicating smooth operating conditions at the station of the crash during 5-10 minutes before the crash (this is confirmed with the high average volume at slice 3 – *AVE3*). The only other occupancy variable entering in the model is *LogAOH3*. With a positive coefficient it indicates some cause of congestion about one-mile downstream of the crash site (again confirmed with low *AVG2* and *SVH2*). It is well known, that under high speed regimes on freeways such disruptions do not travel backward with high speed (Daganzo, 1997), which might be the reason that no other upstream station occupancy variable appeared in the model with a positive coefficient. The disruptions characterized by high occupancy at *Station H* during *Slice 3* probably reach at the crash location causing turbulence right before the time of the crash. Of course, such short term disruptions at the crash location are not captured in this model because *Slice 1* has been discarded due to practical application considerations. It also indicates that the high speed model without data from within 5-minutes before the crash might have less crash prediction accuracy because the actual disruptions resulting in crashes “hit” the crash location just before their occurrence. More significant contribution of errors on drivers’ part towards crash occurrence under high-speed regime might also cause the model classification to be less accurate compared to the low speed model. The final model is presented in Equation 4-5.

$$Risk\_Index = -0.93423 \text{ LogAOF2} + 1.14584 \text{ LogAOH3} - 0.22878 \text{ SVH2} - 0.10055 \text{ AVG2} + 0.5932 \text{ AVE3}$$

...Equation 4-5

4.3 Measure of Effectiveness

This study aims towards not just identifying successful risk mitigation strategies but also tries to optimize them for application. As such a measure had to be evaluated for

the quantification of risks that could be compared for the control case and our testing cases. The Measure of effectiveness (MOE) used was the cumulative Risk Index defined in the previous section averaged over 20 runs for each case. Equation 4-6 shows the measure of Effectiveness for the East Bound Direction. A detailed explanation of the study corridor was presented in Chapter 2.

$$(MOE)_i = 1 / n \left(\sum_{j=1}^n \sum_{t=1}^{360} (RiskIndex) \right)$$

Equation 4-6

Where i= Station Number (33-49 for East Bound Direction)

j=Seed Number (20)

t= time slice (360 for 3 hours evaluated every 30 seconds)

This would serve as the primary MOE. A secondary MOE in the form of Total Vehicle Travel Time would be calculated amongst the cases to make sure that our measures are not adversely affecting traffic conditions on the freeway.

4.4 Experimental Design

The basic methodology for evaluation of any scenario would involve comparing the base case and a test case. The test case would be the base case modified to include our ITS applications like Ramp Metering. When introducing these ITS applications there are a number of variable measures which might significantly affect the output of the simulation as well as result in varying conditions in the real world scenario also. For instance, the metering locations, metering algorithms and variables within the algorithms are all terms that would affect the final result we are looking for in the analysis. To be able to test all these effects for all the possible scenarios a series of experimental designs

had to be set up. The study was therefore divided into three modules which are described in Sections 4.4.1, 4.4.2, and 4.4.3.

4.4.1 Module 1: ramp metering algorithms

In the early part of the study a comprehensive literature review was carried out to determine the best possible local ramp metering algorithms. ALINEA was considered for the algorithm because of its proven application in the literature as well as in real life (Papageorgiou et al, 1991, 1994). However, simpler metering techniques were also explored to determine the basic effects of metering. This was done keeping in mind locations where there might be a lack of loop detector data resulting in inability to actually install feedback algorithms like ALINEA. Two techniques tested were Manually Metering ramps and Using Fixed Pre-timed signals for metering the freeways.

4.4.1.1 Manual Metering

This part of the module was mainly exploratory to determine if metering does actually help us in achieving our goal of crash mitigation. The implementation phase of Manual Metering involved using the Application Programming Interface of PARAMICS. The implementation was mainly experimental and was intended for visual verification only. By controlling the meters and observing the relationship between metering rates and traffic behavior upstream and downstream of the ramps, significant understanding was gained about the metering effects on traffic stream. This proved very valuable during the construction of designs for the other algorithms.

A number of runs were used up in gaining this insight and subsequently a final run was made based upon the patterns observed during the observation runs. The results of this run are reported in the next chapter (Section 5.1.3).

4.4.1.2 Pre-Timed Metering

Constant Time metering or fixed time metering is perhaps the simplest possible application of the ramp metering control. It involves the inclusion of a fixed signal cycle with double or multiple phases in order to control the inflow of vehicles. The length of the green phase can be determined from the desired hourly flow onto the mainline. Chu et al. (2003, 2004) showed that Feedback Ramp metering works better as compared to the Fixed Time metering in terms of improving the capacity of the freeway. Since in this metering the only variables are the phase and cycle length, it is relatively easier to evaluate the different levels of this metering. The cases investigated in this application are tabulated in Table 4-2. The cases here considered are only for 1 ramp and 7 ramps metered to determine the effectiveness of this technique. As the results would indicate in the next chapter, feedback ramp metering works much better than pre-timed, so a detailed exploration of parameters was of little value. Figure 4-2 represents the levels of each factor that were considered for analysis of pre-timed metering.

Table 4-2 Cases for Pre- Timed Metering Evaluation

Case	Cycle Length	Green time/ Phase	Number of Ramps Metered
1	50	25	1
2	50	15	1
3	25	10	1
4	25	15	1
5	50	25	7
6	50	15	7
7	25	10	7
8	25	15	7

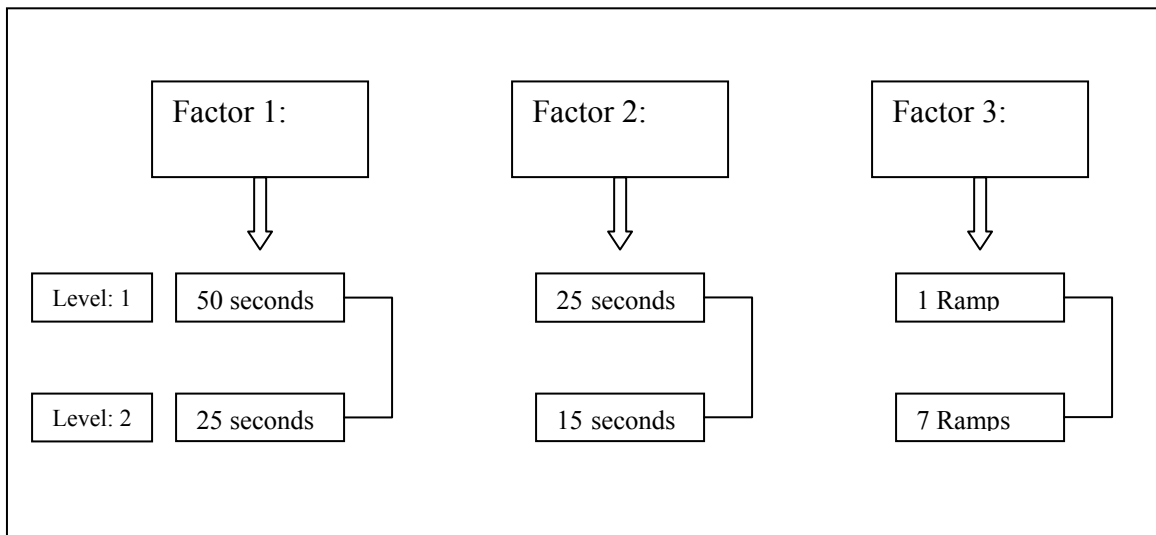


Figure 4-2 Experimental Variables and their levels for Module 1

The main case that needs to be considered and compared with other scenarios is the one which caters to the flow from the ramp within the time period. From a look at the ADT values for the ramps under consideration, it was decided that during the three hour period most of the ramps cater to 1500-2000 vehicles. This leads to an hourly flow of 500- 667vph. As such a flow of 600 vph was chosen as a targeted flow rate for each of the metered ramps (7 in total).

The pre-timed metering was implemented through the Plans and Phases interface for Actuated Signal interface of PARAMICS too. Although this could also be done

through putting the junction interface of PARAMICS too, however an additional modification was made to improve the effectiveness of metering. A continuous green was provided whenever there was less than 5 % occupancy on the freeway. On monitoring the results, however it was discovered that this never happened during the interval of simulation under observation (after the warm up period). Hence the final result was same as that which would have been obtained with the signalized junction interface of PARAMICS. The benefit of continuing to use the Vehicle Actuated signals interface was that the metering could be started after the initial warm-up period was over to ensure that there is no aberration in the results.

4.4.2 Module 2: feedback ramp metering using alinea

The ALINEA algorithm has four parameters to be calibrated (Chu et al.(2003)): the location of the downstream detector station, the desired occupancy of the downstream detector station O^* , the update cycle of each metering rate t , and a constant regulator parameter, K_R . The following is a summary of parameter settings used in previous research and implementations (Papageorgiou et al. 1990, 1991, 1997; Chu et al., 2003)

1. The desired occupancy is set equal to or slightly less than the critical occupancy, or the occupancy value at capacity (Hegyi et al., 2002), which can be found in the volume-occupancy diagram (Daganzo, 1997). Various values ranging from 18% to 31% have been found in previous applications (Chu and Xang, 2003; Lee et al., 2005).
2. Control results have been found to be insensitive for a wide range of values of the regulator K_R (Chu and Xang, 2003). The regulator parameter, as can be seen in Equation 1-2, regulates the conversion of occupancy values into green time for

- metering. In real-world experiments (Papageorgiou, 1997), the algorithm has been determined to perform well for $K_R = 70$. The range of parameter could vary from 70 to 120 without significant effects on the metering rate (Chu and Xang, 2003). In order to validate this premise from literature 70 and 120 were used as the values as can be seen in Table 4-3 and Figure 4-4.
3. The downstream detector should be placed at a location where the congestion caused by the excessive traffic flow originated from the ramp entrance can be detected. In reported implementations, this site was located between 40 m and 500 m downstream of the on-ramp entrance to the freeway.
 4. A wide range of values for the update cycle of metering control have been used: from 40 seconds to 5 minutes. The update cycle is the time after which an evaluation is performed for the length of the next phase of metering. In theory, if the value is small, the location of the downstream detector station should be close to the entrance ramp. Otherwise, there is a risk of congestion build-up in the interior of the stretch (viz. acceleration lane) from the ramp entrance to the detector. This is because, in the time taken by vehicle to reach the acceleration lane from the meter, the conditions on the mainline may change to one of relatively higher occupancy lane- forcing vehicles on acceleration lane to wait for gaps in traffic. The combination of these and other factors is shown in Figure 4-4 and Table 4-3.

This information helped us form the experimental design. The effect of changing the cycle length was found to be significant even from the results of the fixed time metering. This was because, once the metering rate algorithm (Equation 1-2) determined the

effective green time on the meter, rest of the phase for that cycle was red. As such having a two phase metering helped us in biasing the phase towards more red time. Also this would show us if vehicles waiting for longer times is the best option or vehicle released at intermittent intervals is a better option. The maximum green time was a factor that interacted with the cycle length and in this case serves as the cutoff for maximum green time in a phase. This meant the metering would continue at a certain maximum rate even when the occupancy is well below the critical occupancy. The minimum green time was kept uniformly at two seconds to ensure at least one vehicle passing in a heavily congested situation and the maximum value for green time was kept at 15 seconds in each phase, both for cycle lengths of 25 and 50 seconds. The cycle lengths were used to determine the update cycle of the meter, so in our case we tested two possible update cycles, 25 seconds and 50 seconds. Although a number of cycle lengths could be used, they would mean longer waiting times per phase for vehicles which might have an adverse effect on network travel times.

In ramp metering studies (Hegyi et al., 2002) it has been seen that ramp metering is effective till the flow values are below a critical point. As congestion progresses beyond that point- metering becomes redundant. As such we would be looking to meter so as to avoid or delay the attainment of such a high level of congestion. This is taken into account in ALINEA by setting the value of critical occupancy, O^* . This value is determined so that it lies inside our region of effective metering, i.e. on the left side of the flow density curve as shown in Figure 4-3.

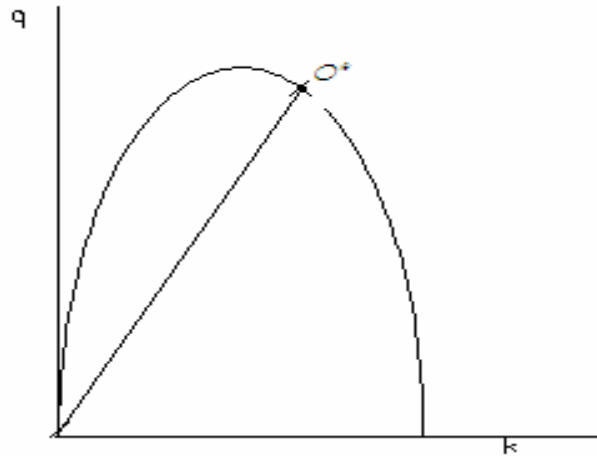


Figure 4-3 Q-K relationship with critical density marked

By using loop data, it was determined that the value of critical occupancy corresponding to a determined critical density as shown in Figure 4-3 and it was found to lie in the region of 15%-25% occupancy. As such, three values were chosen to test and find out which one works best with our primary MOE. The values of 17%, 20% and 23% were chosen. A complete experimental design with all the factors is listed below in Table 4-3 and is graphically depicted in Figure 4-4. Table 4-3 shows a total of 48 cases of parameter combinations used for optimization.

Table 4-3 Complete Blocked Design for Optimizing ALINEA for application

Module2	Number of Ramps Metered	Cycle Length	Critical Occupancy	Critical Occupancy	Critical Occupancy
	1	25s	0.17	0.2	0.23
	1	50s	0.17	0.2	0.23
$K_R=70$	3	25s	0.17	0.2	0.23
	3	50s	0.17	0.2	0.23
	5	25s	0.17	0.2	0.23
	5	50s	0.17	0.2	0.23
	7	25s	0.17	0.2	0.23
	7	50s	0.17	0.2	0.23

	1	25s	0.17	0.2	0.23
	1	50s	0.17	0.2	0.23
	3	25s	0.17	0.2	0.23
	3	50s	0.17	0.2	0.23
$K_R=120$					
	5	25s	0.17	0.2	0.23
	5	50s	0.17	0.2	0.23
	7	25s	0.17	0.2	0.23
	7	50s	0.17	0.2	0.23

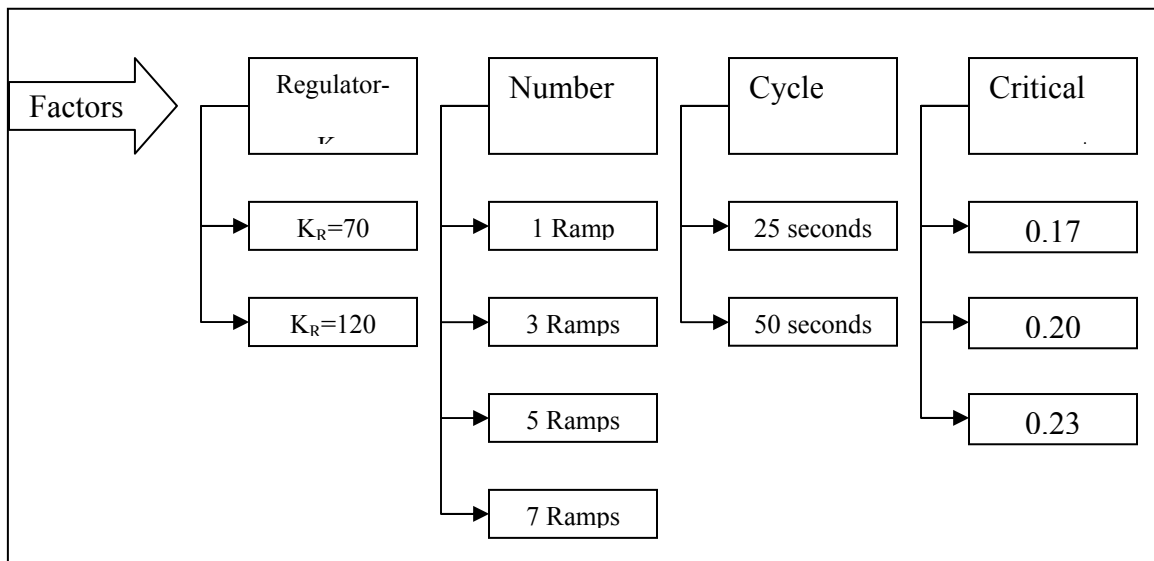


Figure 4-4 Experimental Variables and their levels for Module 2

4.4.3 Module 3: Variable Speed Limits and Ramp Metering

This module tests for Variable Speed Limits and its interactions with Ramp Metering. To assess the effects of the multiple parameters by this stage, the best scenarios from the previous modules has to be used. As discussed in Chapter 2 and 3, the important factors for us to consider while analyzing VSL strategies are:

- Plan for Speed Limit changes in terms of strategies for lowering and/or increasing speed limits at certain locations around the station of interest

- Gradually lowering the speed limits over a longer distance or suddenly introducing a speed limit drop
- Implement Variable Speed Limits at Single and Multiple locations and determining which works better on a case by case basis.

The strategies can be categorized into four main classes (U/S –upstream, D/S - downstream):

- Lowering Speed Limits U/S and Lowering Speed Limits D/S(S1)
- Lowering Speed Limits U/S and Increasing Speed Limits D/S(S2)
- Lowering U/S only(S3)
- Increasing D/S only(S4)

Another case (Increasing U/S and Lowering D/S) was found to be an extreme outlier and hence discarded from experimental design. These cases include the scenarios where there is no speed limit change at one location, e.g. lowering speeds U/S only or increasing speeds D/S only.

Although the basic premise can be that ramp metering can have only a positive effect on these scenarios, nevertheless, it is included as blocking factor in the experimental setup. As such this module not only explores Variable Speed Limits, it also explores the interaction of VSL and Ramp Metering.

A split plot experimental design (Dean and Voss, 2003) was chosen to evaluate the effects of each of these parameters. The split plot design considers two different levels of variables. The split plot factors are the first variables that are to be tested for detailed effects within each whole plot factors. As shown in Table 4-4, the speed plans were chosen as split plot factors and Gradual and Sudden Changes as Whole plot factors. Thus

by comparing each of the speed plans within each whole plot factor we would be able to determine the best plan (S1, S2, S3 or S4) for each whole plot factor (gradual or sudden change). Two blocks were created to check for Single and Sequential Implementation. The design was further blocked for both Metering and No Metering Scenarios where the metering was done for all the seven ramps. The final design is illustrated in Table 4-4. All the different variables and their levels are indicated in Figure 4-5.

Table 4-4 Split Plot Experimental Design

	Block Single	Gradual	Gradual	Sudden	Sudden	Gradual	Gradual	Sudden	Sudden
	Metering	S1	S2	S1	S2	S3	S4	S3	S4
	Block Network	Gradual	Gradual	Sudden	Sudden	Gradual	Gradual	Sudden	Sudden
		S1	S2	S1	S2	S3	S4	S3	S4
	Block Single	Gradual	Gradual	Sudden	Sudden	Gradual	Gradual	Sudden	Sudden
	No-metering	S1	S2	S1	S2	S3	S4	S3	S4
	Block Network	Gradual	Gradual	Sudden	Sudden	Gradual	Gradual	Sudden	Sudden
		S1	S2	S1	S2	S3	S4	S3	S4

Experimental Setup:

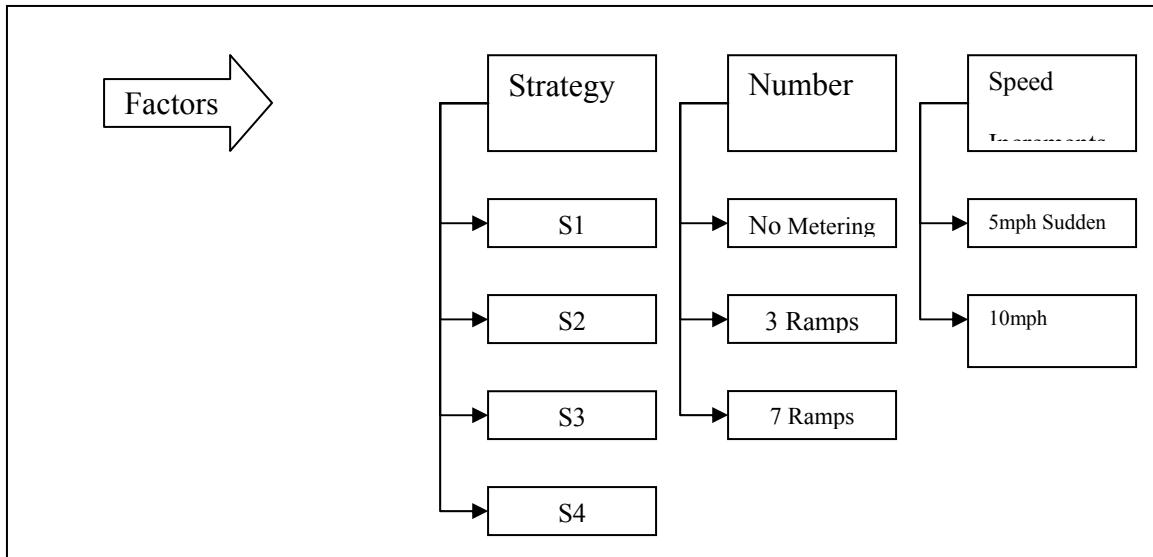


Figure 4-5 Experimental Variables and their levels for Module 3

The gradual change would mean changes of speeds in consecutive stations of no more than 5mph and sudden changes would mean drops or increments of 10mph. Sudden changes of 15mph are being excluded because of the fact, that implementation of change of this magnitude could not be of practical significance. For instance, raising the speed limit of a 60mph road to 75mph might end up having an adverse effect. However, in the instance of the current setup not working, that could be considered on a case by case basis.

Case numbers and the variable definitions for all Modules and their experimental scenarios are provided for reference in a tabular format at the end of this chapter in Corollary 1.

4.5 Quantification of Safety Benefits

The models developed for calculation of the risk index were developed using a matched case-control scenario. As such, when evaluating the effects of each of the models it has to be kept in mind, the instances where the models can be applied. The

methodology means that the model would give a unique index for each and every station. As such no formal conclusion can be made based on the individual safety indices between stations in general. For example, using the index for two stations at the same time, we cannot compare and tell which one is corresponding to a high risk potential. The comparison is possible for say, another time interval at both these locations and the two time intervals risk indices can be compared individually. Thus, the index can be said to be spatially independent. However, the effectiveness of our strategies can be compared across stations using a measure of quantification of time-risk benefits.

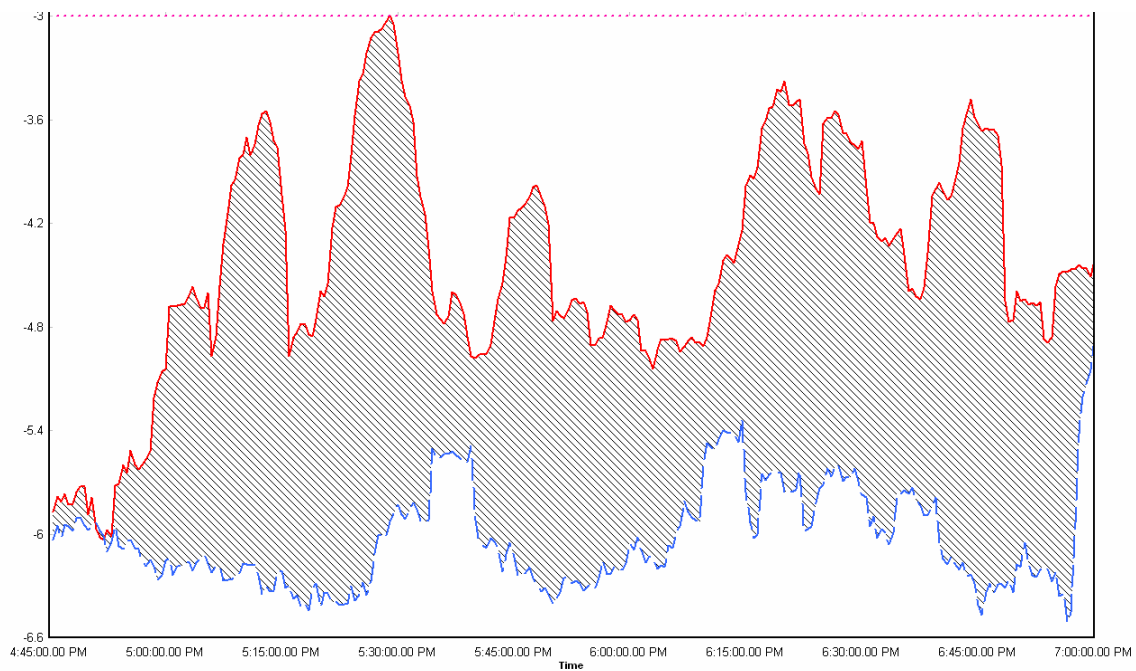


Figure 4-6 Quantification of benefits (*x-axis* – time, *y-axis* – Risk Index)

Figure 4-6 above shows two typical curves for the same location- under different conditions. The shaded region represents the safety benefits over the time interval. To standardize these benefits, the area is divided by the average area under the base curve from 20 different replications. 3-7 shows the area under the base curve.

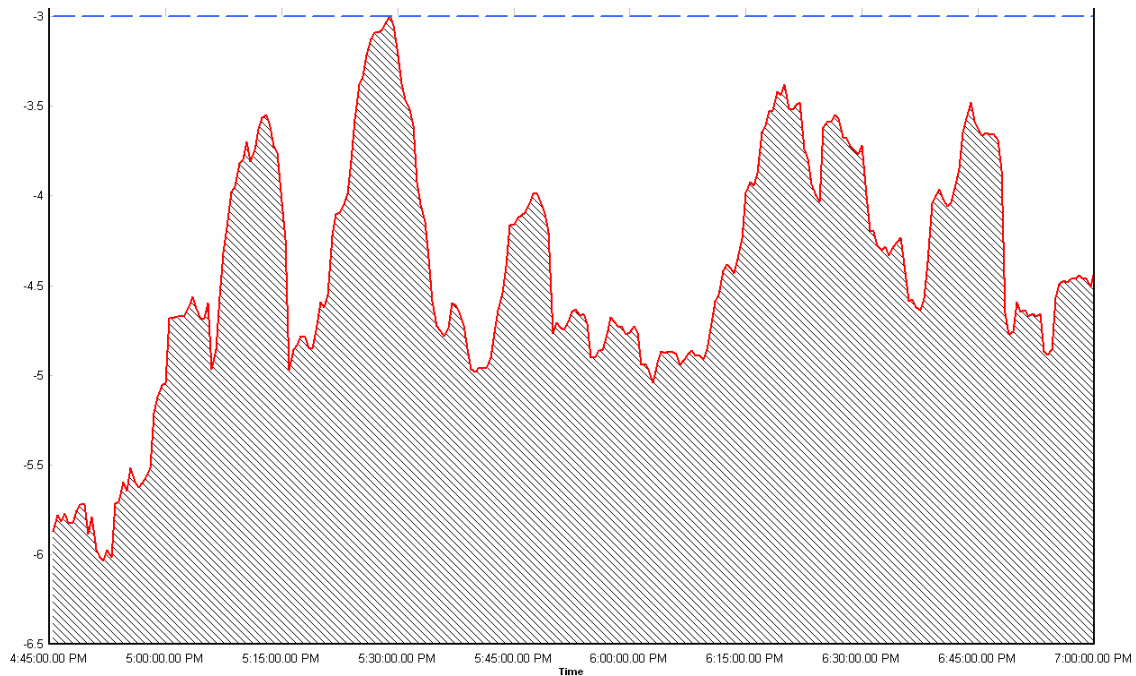


Figure 4-7 Quantification of benefits-denominator area (Base) (*x-axis* – time, *y-axis* – Risk Index).

The area over the total time interval was then divided by the total time interval under consideration to change the ratio to an hourly value that could be compared over different lengths of simulations runs.

So if Base Case is the area under the risk index when there is no implementation of ramp metering or VSL and Test Case be the area under risk index whenever there is a case of implementation of any ITS measure, the Quantification of benefits equation can be written as shown:

$$\text{Safety Benefits} = \frac{|\text{Area under Test Case} - \text{Area under Base Case}|}{|\text{Area under Base Case}|} / t$$

...Equation 4-7

where, t is the time interval of the risk index under consideration. It can be in any time units as long as the same unit is used in both cases. For calculation of Area, it can even be the number of time intervals (e.g. for a three hour period t could 36 for 5 minute intervals to calculate 5 minute benefits).

4.6 Ramp Metering

Ramp Metering in PARAMICS can be implemented by making use of the Vehicle Actuated (VA) Signals. The VA signals can be made conditional using two main methods. One is the use of the Plans and Phases framework in the network. This requires coding of the signal's plan in the Plans and Phases format specified in the VA manual supplied by Quadstone (2002). The other method is to access the signals using the Application Programming Interface. The phases for each signal can be externally controlled. The programmer files in PARAMICS come with examples which provide useful guide to this process. API examples 10 and 11 in specific refer to this procedure. For purposes of this both these processes were used at different stages. The former was used for stand alone analysis of ramp metering. When using coordinated strategies, it became imperative to use the latter method. The following sections provide the methodology for using both these methods for Ramp Metering.

4.6.1 Phases

Once we have a ramp in the network, it is not possible to use the nodes of the ends of the ramp for use as a signalized junction. Thus, a dummy node has to be coded into the network which can be used as a normal junction and hence be signalized. This can be done in two ways.

The ramp length can be shortened and a node created behind the entrance node to the ramp and this node be used. Also, a node can be put on the ramp itself and coded as a signalized junction. The latter is preferable to using the former. Since it is desired to retain the character of the vehicles on the ramp for their whole distance on the ramp, shortening the length would mean not being able to simulate the ramp behavior of the vehicles for some distance. Also the second method allows us the flexibility of adjusting the distance of the meter stop line from the main line. A ramp meter far away from the mainline makes it difficult to adjust the release of vehicles based on mainline conditions. However, practical reasons also demand that the meter be not too close to the entrance to the mainline. As such a distance of 50 feet was chosen in most cases to install the meter away from the mainline. The exceptions were curved ramps and multiple legged ramps. The multi-legged ramps had to be metered at the junction of the two legs of the incoming roads in order to prevent queue extension into the intersection itself by metering downstream of it. The curved ramps had to be metered closer to the freeway mainline as an extra node could not be placed in the middle of the curve (Quadstone 2002).

Once a signal had been placed on the node on the ramp, its phases were controlled using the Phases files. Using this file we can specify the parameters and the loops and mainline lanes to be used to calculate the signal plans for this particular node. The standard coding procedure for the phases file is demonstrated in the VA signals manual as well as in the phases file itself and in the various examples for VA signals provided by Quadstone (2002). A brief example from Quadstone (2002) follows below:

use plan 1 on node 8 phase 1

plan 1 to apply to intersection node 8 - phase 1

with loop

Loop8N1 lane 1 ## loop detector name for upstream loop lane 1

Loop8N1 lane 2 ## loop detector name for upstream loop lane 2

with parameters

1200 ## upper flow threshold

500 ## lower flow threshold

35 ## high flow green time

15 ## low flow green time

The first line specifies the plan to be used corresponding to the node and phase. We can then specify which loops and lane are to be referenced in the plans files. Also some of the parameters to be used as constants in the plan files are listed.

4.6.2 Plans

The plans file is used to code in the specific signal plan for the node in question. The plans file for the example one in Quadstone (2002) is:

plan count 1

plan 1 definition

loops 2

parameters 4

if (init)

On initialisation make ensures the traffic signals are Variable Cycle

{

variable;

}

if ((flow [1] + flow [2]) > Parameter [1])

**## if the flow in lanes 1 and 2 on the upstream loop combined is greater than the threshold
(parameter [1])**

then set the next green time to {parameter [3]} seconds

```

{
    green2 = parameter [3];
}

if ((flow [1] + flow [2]) < Parameter [2])
## if the flow in lanes 1 and 2 on the upstream loop combined is less than the threshold
(parameter [2])
## then set the next green time to {parameter [4]} seconds
{
    green2 = parameter [4];
}

```

All the references to flow etc are from the loops mentioned in the Phases files. The conditional statements are written in the main body and the statements within the braces {} are executable commands for the specified signal.

Using these guidelines phases and plan file was constructed for a single ramp at first. The plan was designed so as to implement the ALINEA algorithm. Node 206 was used for this purpose.

Once this was successfully implemented, six more on ramps in the east bound direction were metered using the same algorithm ALINEA. The ramps metered were those that were within the areas of our interest.

The plans and phases method for signalized intersections- although very efficient and flexible still had some limitations. Although the speeds at the detectors connected with every node could also be used in the plans file, it was difficult to coordinate a number of meters with each other using this method. Also as the number of detectors we were concerned with became larger, it was more cumbersome to code each of them and change easily.

The PARAMICS API tool provides a very good platform for coding signalized junctions for ramp metering purposes. For exploratory purposes, the signalized node was first controlled manually using the API and the GUI parameter interface. With this method the phases were predefined and could be chosen based on manual judgment. This method- although efficient as it allows us to change the phase plan based on judgment and takes out the element of surrogate measures to predict congestion onset- is not really practical. Hence, the feedback algorithm was coded the API for ramp metering was integrated into the API for VSL. A variety of scenarios could then be tested and the phases for the signals be controlled easily and dynamically.

4.7 Variable Speed Limits

Variable speed limits in PARAMICS can be implemented in two main ways. Dillmore (2005) in his work refers to the work by Lee et al. (2004) in pointing out that they used VMS as measures for speed controls by warning drivers of change in speed limits. The arguments provided by him for using Link files for changing the speed limits hold true and we also know that both methods are useful and applicable. However, the method of changing the link file is pre-meditated. This means we have to specify a certain number of scenarios and change speed limits at certain times. For doing this a prior observation of the network is required. This method can be very useful in situations where we are simulating recurring scenario and are testing only specific interventions in the traffic scenario at fixed time intervals only. However, a more encompassing strategy can be formulated using the PARAMICS API. Using API the implementation can be made conditional upon the current traffic conditions on the mainline. Lee et al. (2004) used VMS beacons in API to specify the speed limits in the network. However, the speed

limits on any links can also be reset using the API. This also gives us the leeway of formulating conditions where a certain strategy of VSL should be implemented. This is the more beneficial strategy for our use. VMS can be used in PARAMICS to specify the level of compliance whereas the method of resetting the link speed limits uses the internal PARAMICS logic of following a normal distribution for driver behavior. This logic makes sure that the most aggressive drivers drive at a maximum of 110% over the specified speed limit. The compliance that can be adjusted through VMS, has only a graphical interface for driver behavior and there is no certain way to determine the exact nature of driver behavior effect on compliance.

The logical plan for using the VSL and enforcing changed speed limits hinges on the information collected through out the simulation. This was done by collecting speed data at all the detectors every time step and aggregating it over 30 seconds and 5 minutes. The 30 second data is the loop data available to us. This was aggregated over five minutes to give us the average over time for our parameters. The decision logic for implementation of speed limits was then run every 5 minutes. Thus the strategy as a whole was changed every five minutes over all the detectors we were interested in. Pointers were created to five consecutive links with detectors in every direction on the freeway. These were referenced in the same way as the stations in crash risk assessment model, with station F for the station of the crash and subsequently station G, H for downstream stations and station D, E for upstream stations. This nomenclature is illustrated in Figure 4-8.

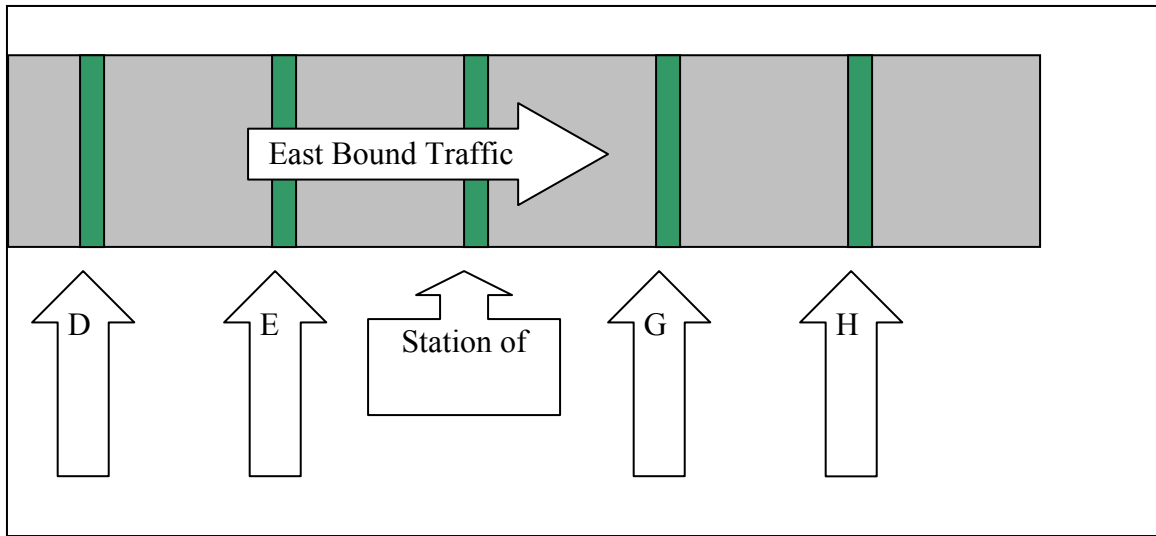


Figure 4-8 Nomenclature for stations of interest

The restriction distance for the Variable speeds was changed dynamically but was kept constant at 1200 ft in the beginning. This is the maximum distance the speed limits are enforced on a particular link. This distance was set based on the link lengths as multiple links would have to be reset every time there is a change in speed limits. If larger values are chosen for restriction distance, there is a possibility of the same link being reset to two different speed limits based on upstream and downstream detector locations. In such a case, the PARAMICS programmer shuts down the simulation due to error.

4.8 Application Programming Interface

The Advanced Programming Interface (API) in PARAMICS allows us to override and extend functions as well as set and extract values for traffic parameters during simulation runs. The PARAMICS manual provides the listing and functions of all the objects inbuilt into the API interface. The API interface allows us also to construct data structures but the limitation still remains that we can use only the functions that are specified in the programmer header file. PARAMICS deprecated a few functions from v4

to v5 but these were added later on because the functions from v4 were already being used and it wasn't thought prudent to change logic in order to use the new functions. PARAMICS programmer comes with a base set which has to be used as the foundation for building any new API.

The editing of the API is done by opening a workspace in Visual Studio editor for VC++. Most of the guidelines for constructing an API are available in the Programmer User Manual. A complete listing of the functions is available in Programmer Reference Manual. (The author came across at least one function which was listed in the header file but not listed in the html documentation of reference manual). However, the documentation is not detailed enough to be used for starting from scratch. The examples provided by Quadstone (2002) provide the required help in that area. All these resources when used together are sufficient to start building API. Some of the issues that are not comprehensible to the programmer can be addressed by contacting the support staff at Quadstone (2002) as well as visiting the discussion group on PARAMICS Online. Both these resources proved invaluable from time to time.

Once the API is ready in the VC workspace, the program is compiled and a dll file is built. This location of this dll file is then referenced in the network. A programming/ programming.modeller/ programming.processor file is created and the location to the dll file referenced on the first line. An additional file is also created in which the parameters corresponding to the particular API are listed with their default values and precision. This file is referenced to in the configurations file of PARAMICS. The coefficient listed in the parameters file can then be changed dynamically in the

modeler. The reporter window in PARAMICS can provide us with information about the successful loading of the particular API.

Since this study required API that referred to different things like VSL and Ramp Metering, at first an attempt was made to load two different API and control them separately. However, the modeler had problems with two API being loaded simultaneously. Also from a logical point of view it made more sense to combine the two API to be able to test scenarios for any coordinated strategy with the minimum effort. However two different parameter files were maintained so that the two sets of parameters could be controlled in different windows.

4.9 Corollary

Table 4-5 Pre- timed metering case numbers and variable values

Case	Cycle Length	Green time/ Phase	Number of Ramps Metered
1	50	25	1
2	50	15	1
3	25	10	1
4	25	15	1
5	50	25	7
6	50	15	7
7	25	10	7
8	25	15	7

Table 4-6 Feed-back Metering case numbers and variable values

Case	No. of Ramps	Cycle Length	O*	K
1	1	25 seconds	0.17	70
2	1	25 seconds	0.20	70
3	1	25 seconds	0.23	70
4	1	50 seconds	0.17	70
5	1	50 seconds	0.20	70
6	1	50 seconds	0.23	70
7	3	25 seconds	0.17	70
8	3	25 seconds	0.20	70
9	3	25 seconds	0.23	70
10	3	50 seconds	0.17	70
11	3	50 seconds	0.20	70
12	3	50 seconds	0.23	70
13	5	25 seconds	0.17	70
14	5	25 seconds	0.20	70
15	5	25 seconds	0.23	70
16	5	50 seconds	0.17	70
17	5	50 seconds	0.20	70
18	5	50 seconds	0.23	70
19	7	25 seconds	0.17	70
20	7	25 seconds	0.20	70
21	7	25 seconds	0.23	70
22	7	50 seconds	0.17	70
23	7	50 seconds	0.20	70
24	7	50 seconds	0.23	70
25	1	25 seconds	0.17	120
26	1	25 seconds	0.20	120
27	1	25 seconds	0.23	120
28	1	50 seconds	0.17	120
29	1	50 seconds	0.20	120
30	1	50 seconds	0.23	120
31	3	25 seconds	0.17	120
32	3	25 seconds	0.20	120
33	3	25 seconds	0.23	120
34	3	50 seconds	0.17	120
35	3	50 seconds	0.20	120
36	3	50 seconds	0.23	120
37	5	25 seconds	0.17	120
38	5	25 seconds	0.20	120
39	5	25 seconds	0.23	120
40	5	50 seconds	0.17	120
41	5	50 seconds	0.20	120
42	5	50 seconds	0.23	120
43	7	25 seconds	0.17	120
44	7	25 seconds	0.20	120
45	7	25 seconds	0.23	120

46	7	50 seconds	0.17	120
47	7	50 seconds	0.20	120
48	7	50 seconds	0.23	120

Table 4-7 VSL and FB Metering case numbers and variable values

Case	No. of Metering Locations	Speed Increments	Strategy
1	0	5mph	S1
2	0	5mph	S2
3	0	5mph	S3
4	0	5mph	S4
5	0	10mph	S1
6	0	10mph	S2
7	0	10mph	S3
8	0	10mph	S4
9	3	5mph	S1
10	3	5mph	S2
11	3	5mph	S3
12	3	5mph	S4
13	3	10mph	S1
14	3	10mph	S2
15	3	10mph	S3
16	3	10mph	S4
17	7	5mph	S1
18	7	5mph	S2
19	7	5mph	S3
20	7	5mph	S4
21	7	10mph	S1
22	7	10mph	S2
23	7	10mph	S3
24	7	10mph	S4

4.10 Micro-simulation Calibration

4.10.1 Network description

The overall simulation was done for an approximately 20-mile section of Interstate 4 in metropolitan Orlando area. The Interstate runs between Daytona Beach in the East to Tampa Bay in the West. The section in this study runs from South of Orange Blossom Trail (SR 441) ramp to north of Lake Mary Blvd. (CR 427) in roughly the North South

Direction. The section consists of 17 interchanges and 59 ramps, curves with radii varying from 1910 to 85,944 ft, and speed limits from 50 mph to 65 mph. The section consists of 40 detector stations in the each of the East Bound and West Bound Corridors with detector numbers from 28 to 68 increasing in nomenclature from West to East on both corridors. The simulation treats both these directions as different corridors in network building and will henceforth be referred to as EB Corridor and WB Corridor.

4.10.2 Calibration

Calibration is probably the most important part of the simulation experiment. A well calibrated network provides a certainty and validity to the model results. The Calibration process has some objectives according to which the whole process is defined. In the traditional process of model calibration, model parameters were adjusted until reasonable (qualitative and quantitative) correspondence between the model and field observed data is achieved. Hourdakis et al. (2002) proposed the gradient approach and Cheu et al. (2004) proposed using genetic algorithm. These approaches regard the model calibration procedure as an optimization problem in which a combination of parameter values that best satisfies an objective function is searched. Ben-akiva et al. (2004) and Chu et al. (2003) provided a methodology for calibration using aggregated data and use dynamic OD and route choice as factors in the calibration process.

Most studies to date have focused on Calibration of their simulation using aggregated measures of flow (Abdulhai et al., 1999). Research in the field of Calibration has been focused towards calibrating driver behavior parameters and validating using aggregated data requirements (Hourdakis et al., 2002; Cheu et al., 1998). The basic procedure involved is adjusting OD till a reasonable flow on links is obtained and then

using the links to adjust driver behavior parameter to adjust flows. However, most of the studies stop at 15 minute resolution of data for validation and almost never even consider such resolution for speeds (Chu et al. 2003). Wu et al. (2003) provided an exhaustive methodology for the validation of car following model where speeds at high resolution of time could be validated. However, the research does not propose a method for fine tuning such speed requirements for microscopic models.

As a primary objective of this study, we were concerned with improving safety of this section. The crash risk estimation models we described in the Section 4.2 (and developed first in Chapter 4—Volume I of this report) were concerned with the speeds of the vehicles at different locations as well as other measures like average 5 minute occupancy, etc. Thus, a simulation of the network has to closely model the real life conditions in order for the results to be wholly justifiable. Also it is necessary that the ITS strategies we test should also be applicable to conditions that were mimicking the actual conditions on which the models has been developed. This serves a two pronged purpose; it can give us an idea of the effectiveness of the models under different traffic regimes as well as validate the effectiveness of the strategies that we apply to different locations and traffic stream characteristics. In order for us to be able to achieve both the goals, the network is validated and verified for flows, however, all the initial scenario testing is done on the East Bound Corridor of the network to make the problem more scaleable. This is the section which is also calibrated thoroughly for speeds. Figure 4-9 depicts a flow chart showing the operational sequence for the Calibration and Validation process and the process of testing different scenarios.

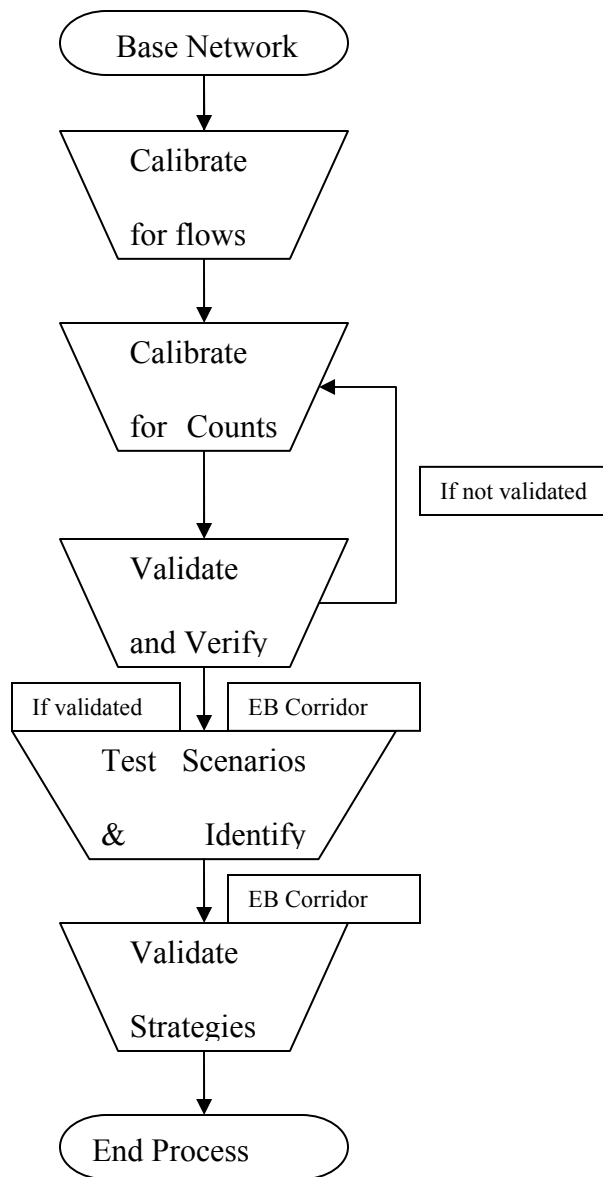


Figure 4-9 Flow Chart for Calibration Process and Scenario testing

For the problem at hand, it was very necessary to come up with a model that not only mimicked real world in terms of flow and speed values but did so even at the micro-level. However, this meant a longer calibration process with an intertwined Validation process. In this process the calibration parameters were identified and revised at the end of every

step of the calibration process. Also, the calibration process ended with a validation step in order to make sure that the errors of the simulation were not increasing with any change in the calibration parameters. Some simplifications in the calibration process enabled us to shorten the time required to calibrate the network which simulates a 15 mile section of the actual roadway.

- An introduction of a large number of zeroes in the model (All the on-ramps were used as origins and all the off ramps, used as destinations; hence all off-ramps became zeroes). This simplified OD provided us with the flexibility needed to compare the results for all the other parameters.
- A section of the freeways was chosen which best suited our needs for testing the strategies. A greater emphasis was placed on the Calibration of flows and speeds at these locations. This section extended from Station 33 to Station 49 in the East Bound Corridor of the network. This section covered all the six ramps which were eventually used in the testing phase for ramp metering.

The section ultimately chosen for Micro-calibration consisted of 17 Loop Detector stations, 11 On-ramps and 10 off-ramps.

4.10.3 Data Preparation and calibration of the OD Matrix

The overlay that is used to draw the I-4 corridor is generated by a combination of aerial photography, obtained from the Orange County Property Appraiser's Office and the Seminole County Property Appraiser's Office, and AutoCAD drawings, obtained from the Orange County Public Works Department. The process of putting this overlay into PARAMICS as well as the calibration of the OD matrix for this corridor was described in Sections 2.2 through 2.6 of this report (Also see Dilmore, 2005).

4.10.4 Loop detector data

Due to the sensitivity of the tests involved in the study, the Calibration had to take care of not only the broader, more general parameters like Driver behavior and queuing behavior but was also supposed to take into account the traditionally aggregated data like flows and speeds measurements. The best source of these data available was that from the Loop Detectors on the section of Interstate-4 under consideration. The inductive loops are placed roughly half a mile each on a 36 mile stretch of interstate 4 starting from South of Disney to north of Maitland Blvd. The stations are numbered from 2 through 69 from South to North (West to East) in both directions. Figure 4-10 demonstrates the numbering of these loop detectors.

Each of these stations consists of dual loops in each direction and measures average speed, volume and occupancy over 30 seconds period on each of the through travel lane. The loop detector data were continuously transmitted to the Regional Traffic Management Center (*RTMC*) and UCF data warehouse. The source of crash and geometric characteristics data for the freeway is *FDOT* (Florida Department of Transportation) intranet server.

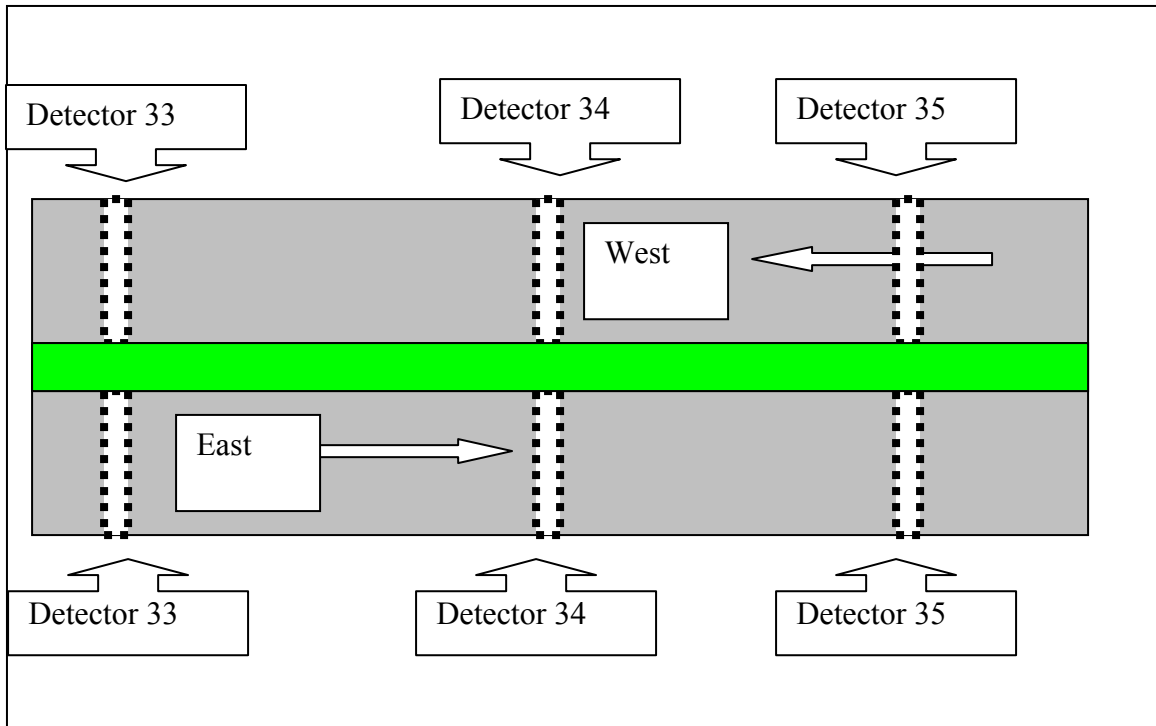


Figure 4-10 Detector Numbering on I-4

The loop data is archived at UCF. As we were looking for the most common data set available to be used for Calibration and this data needed to be from the latest possible time matching the roadway characteristics that were simulated. In accordance with using the 2003 FTI CD for OD matrix generation, data was extracted for all the Tuesdays, Wednesdays and Thursdays of the year 2003. Since the simulation was being carried out from 4:00 PM to 7:00 PM, this was the time for which data was extracted for stations 28 through 65. Although the final calibration procedure is carried out for only a section of this, the whole data is extracted for future use in an extended network.

4.10.4.1 *Missing Data*

The loop data extracted from the UCF data warehouse had a considerable number of missing observations. There was no one set of data in which 10 consecutive days of

complete loop data could be found for all the stations in all the lanes. Also this failure wasn't random and there were certain stations which had consistent loop failure. For instance, stations 37 through 40 had single and double lane data consistently missing for the time period under consideration.

As such the loop data was treated using the logic from another study, also conducted using the same data at UCF (Chandra and Al-Deek, 2004). The study had come up with regression models for imputing missing data points using available loop data at 5 minute intervals, both historical and current. Regression indices were developed for stations 33 through 55. These indices were then used to fill in all the missing values. The regression models used available data for speed, occupancy and volumes to estimate the missing data. The best performing Regression Model from their study, The Pair wise Quadratic Model is used to impute the missing data values. In the pair-wise quadratic form a variable, (e.g., flow) is expressed as a second degree model of all the traffic variables (flow, speed, and occupancy) from each of the neighboring loop detectors.

In the case when no lane from the station reports good data, only then the upstream and downstream stations' data could be used to estimate the missing value at the lane of interest. However, this part was not relevant as days with completely missing values were excluded from the data set. This model was used as it performed best with imputing data from that available from the same station in other lanes. This had to be done to avoid undue over or under- estimation of traffic parameters in different lanes. This would typically happen if simple averages are taken from the data. To demonstrate this process missing speed values in center lane for station 34 East Bound are imputed using Equation 4-8.

$$CS = \text{Intercept} + 0.0135 * LS + 1.98401 * LV - 0.59656 * LO + 0.012772 * LS\_LS - 0.037441 * LV\_LS + 0.031231 * LV\_LV + 0.006897 * LO\_LO$$

...Equation 4-8

A sample for calculation of Center Lane speed data for station in East Bound direction is provided in Table 4-8. The calculation of the table is based on Chandra and Al-deek (2004) and the coefficients have been calculated for the particular dates and times that this study deals with.

LS → 5 minute Speed in Left Lane

LS_LS → Square of 5minute speeds in left lane etc.

Detailed Models for all stations and parameters are listed in Appendix II-D.

Table 4-8 Regression Coefficients for Stations 34 through 48 for Center Lane Speed Imputation

Station	_RMSE_	Intercept	LS	LV	LO	LS_LS	LV_LS	LV_LV	LO_LS	LO_LV	LO_LO
34	6.4682	9.5559	0.0135	1.98401	-0.59656	0.012772	-0.037441	0.031231	0.006897	-0.025402	0.008259
35	8.5776	22.276	0.71235	0.07119	-0.84815	-0.003378	0.015291	0.015245	-0.012776	-0.002136	0.012004
36	5.8745	-1.1069	1.03346	-0.59828	0.12001	-0.000425	-0.000902	0.02494	-0.008342	0.014123	-0.000592
37	7.8069	0.8885	0.38509	1.69633	-0.51676	0.010188	-0.017719	-0.009736	-0.004815	0.005322	0.00669
38	12.0179	16.3157	1.13505	-0.38067	-0.16028	-0.007426	0.02129	-0.036441	-0.014599	0.024457	0.001625
40	9.3866	-3.1172	0.87824	0.65216	-0.14445	0.00093	0.004276	-0.011923	-0.022965	0.019416	0.004441
41	6.5219	-1.2012	1.3248	-0.92748	0.22652	-0.003118	0.007069	0.021107	-0.016405	0.020411	-0.00204
42	7.525	-21.3096	2.69798	-1.65025	0.92191	-0.022052	0.041008	0.007314	-0.048086	0.031075	-0.006247
43	8.8967	9.9595	1.34689	0.15663	-0.54603	-0.012695	0.029138	-0.019066	-0.051786	0.049441	0.009304
44	4.7489	-16.9913	1.27354	0.80499	0.13681	-0.001525	0.005898	-0.038152	-0.008527	0.014394	0.000524
45	9.8202	-47.2398	3.32845	1.44276	1.01223	-0.029991	0.015669	-0.016647	-0.062981	0.003984	-0.002838
46	7.0798	-21.9014	1.92251	2.86147	-0.34803	-0.009065	-0.02979	-0.002173	-0.015913	-0.028681	0.010984
47	7.2548	10.0684	1.46145	-0.43827	0.23353	-0.009388	0.032291	-0.022451	-0.063033	0.068477	-0.004716
48	5.8375	-24.6009	2.02121	-0.17482	0.30031	-0.010395	0.016676	-0.004156	-0.032148	0.026667	0.002903

Chu et al. (2003) in their study chose a single day for calibrating the model volumes with real world scenario and used the Travel Time data for the same day. Their methodology was constricted by the fact that there was only one day for which they could match the travel time and loop data. Since, this study aims at calibrating speeds and flows along a section of freeway with no alternative routes, loop data could be chosen in a more appropriate manner. In order to choose a typical data, GEH statistic is calculated for a number of days to determine a median day. A different methodology was used in this study. Instead of relying on median data values for both the network and real world, a more robust approach was used. Based on preliminary data analysis, 3 consecutive 20 day periods were chosen. (This was based on estimation of number of runs for the simulation, 20 being much more than the required minimum, 15). The reason for choosing these 20 day periods was to determine the most recurring traffic scenario. Each of the days was individually compared to the mean of the overall 20 day period in terms of Flow for each station.

A GEH statistic was calculated for all the values of the stations under consideration (till this part of analysis we were looking at 21 stations (33-55 except 39), however, the network study section was reduced to stations 33-49 except 39 when the calibration part was started in PARAMICS). The GEH statistic for Flow was calculated over every hour for each station (i.e. 3 times for every station from the available data)

$$GEH(S) = \sqrt{\frac{(F_{ij} - F_{i.})^2}{(F_{ij} + F_{i.})/2}}$$

...Equation 4-9

S—station under consideration

F – Flow at station for the hour

i —Time interval(1,2,3)

j —Day(1...20)

The periods were chosen from Feb 11 to April 10, June 4 to August 5 and September 03, 2005 –November 1, 2003. The GEH statistic for most of the stations was found to be greater than 5 for a number of stations. A GEH statistic of less than 5 is desirable for making sure that the data is indeed part of the same set (Chu et al., 2003; Wisconsin DOT, 2002). A study of the problematic values showed in the scatter plot that for the period chosen there were significant correlation between date and average flows. A scatter plot (Figure 4-11) for the observations showed that values in the initial part of September were abnormally higher than values at all other periods. The *x axis* in the figure represents the number of 5 minute intervals starting at 4:30 PM (interval from 0-5 means 5 x 5 =25 minutes interval) and the *y axis* represents the counts in the time interval from loop data (this terminology is followed for the graphs throughout this calibration process). As such the choice of date was moved so as to exclude the ten day period from September 3 to September 18, 2003. The new period was chosen from September 23, 2003 to November 20, 2003. This change improved the GEH statistic significantly and brought it to acceptable levels. Only station 48 presented consistent loop failures that could not be accounted, hence the data for station 48 had to be excluded from the calibration process. This period was then used for all further calculations as it presented the best prospect of data availability and consistency.

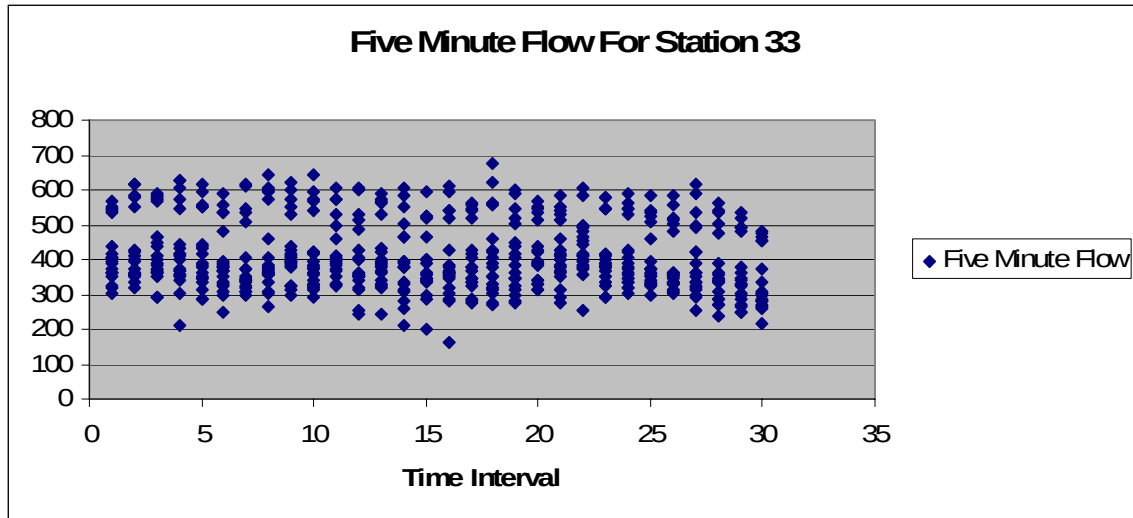


Figure 4-11 Scatter of Five Minute Flows over Time for Station 33 for Data set Sept 03-
Nov 03, 2003

The new data was found to have a GEH stat value of less than 5 for 980 (78%) out of the 1260 observations (20 days x 21 stations (33-55 except 39) x 3 hour). The GEH statistic for a single peak hour would have provided better results perhaps but there was a lot of variation from day to day in the first and last half hour intervals which negatively affected the GEH value. However, considering the scope of this network and time length of simulation, this was a very reasonable value and could be easily accepted.

A similar test for speeds was considered impractical due to the prevalent noise in the data as well as the fact that one hour speed averages didn't make much sense. Note that the same network was used in first phase of the preliminary evaluations (in last two chapters) for testing the effects of VSL on this corridor. However, significant emphasis wasn't paid on calibration and validation of the network and mainly verification and calibration of the queues were kept as the major objective. This resulted in having a network that was well calibrated for volumes and which mirrored pretty accurately the existence of queues in the real world scenario.

Theoretically, the results from the study would still hold. The comparisons that we were looking at could very well be taken from a generic or imaginary network where the vehicles were showing reasonable behavior. As such a network with a straight section and a few on ramps and off ramps could be used for testing of this study. However, to examine the exact effect of the measures we introduce on the risk models (which were developed using the data from these sections of I-4 itself) it is necessary to model existing conditions on the road network.

The network that we started with had an error of less than 4% on the ramps. This was a good number for the ramps. The Origin Destination Matrix was however tinkered with to calibrate some of the other measures and this process successively resulted in different numbers for the matrix. The overall error for both EB and WB ramps increased to 4.10 % as was reported earlier. Since East Bound is the main area of focus for exact calibration and therefore study, the error on the on-ramps finally for East Bound section only came down to less than 3%. A large amount of error was concentrated around the SR 408 interchange. A complete description of the calibration process follows.

4.10.5 Data Collection from PARAMICS

Although, PARAMICS carries the capability to provide the loop data, the format is different from the one that we actually come across in the field. As such the data extracted from PARAMICS had to be changed in format too. To decrease the overall processing time, the data was extracted for speeds only. This data was extracted as point data. Although link data could also be extracted, point data was preferred because, this would require extracting only speed data and the flow and occupancy data could be easily imputed. This data gives the speed every time a vehicle crosses over a loop so we could

estimate the speeds over 30 seconds as also the flow values. The post processing was done using macros in excel. The first step in the post processing brought the data into the loop data format that we desired. This data was collected over all the stations and over 20 random seeds. All the different runs from this process were then accumulated separately and the data converted into five minute packets in the form it was going to be used in the calibration process. All the runs were then accumulated according to stations and means were taken for all the measures, including 5 minute flows and speeds. This whole process took about 36 hours to finish for one set of output of 20 runs using multiple computers. Visual Basic codes for each part of the process are provided in Appendix II-E.

4.10.6 Calibration of Flows

Once the Ramp Volumes were close to the values expected from the values given in the FTI CD, the next part was the calibration of the flows on the mainline. A preliminary analysis of flows from 20 runs showed that the mainline flows were much lower than the expected values at some stations and higher at some other stations. This could be attributed to the lower number of vehicles released from the end zone for the East Bound traffic. Also, the speed controls required for generation of the queues demanded some minimum demands at any location in the traffic stream. As such a profile was prepared for all the stations indicating their 5 min difference from expected values of flows at each of the locations.

An example of the profile created for 6 stations is given in Table 4-9. The dark regions were flow values from simulation exceeding the actual flows and light regions marked the lower data points. Another similar profile was prepared marking the regions

according to the percentage error from the actual counts. Regions were marked according to the category they fell in terms of errors, $<10\%$, $10\% \leq \text{error} < 20\%$, $\geq 20\%$.

Based upon the overall difference in counts in flows, successive changes were made in 2 major factors, release from the end zones and profile of vehicle release. This way the control of vehicles over the 3 hour interval could be carried out more efficiently. At the end of every 20 simulation runs, the same process was carried out to determine the effect on the flow values. However, the process was not carried through out looking at the flows. Once a decent error was obtained over a number of stations, the simulation run was verified visually over a number of runs. It was decided to carry out the rest of the simulation processes by simultaneously calibrating speeds and flows.

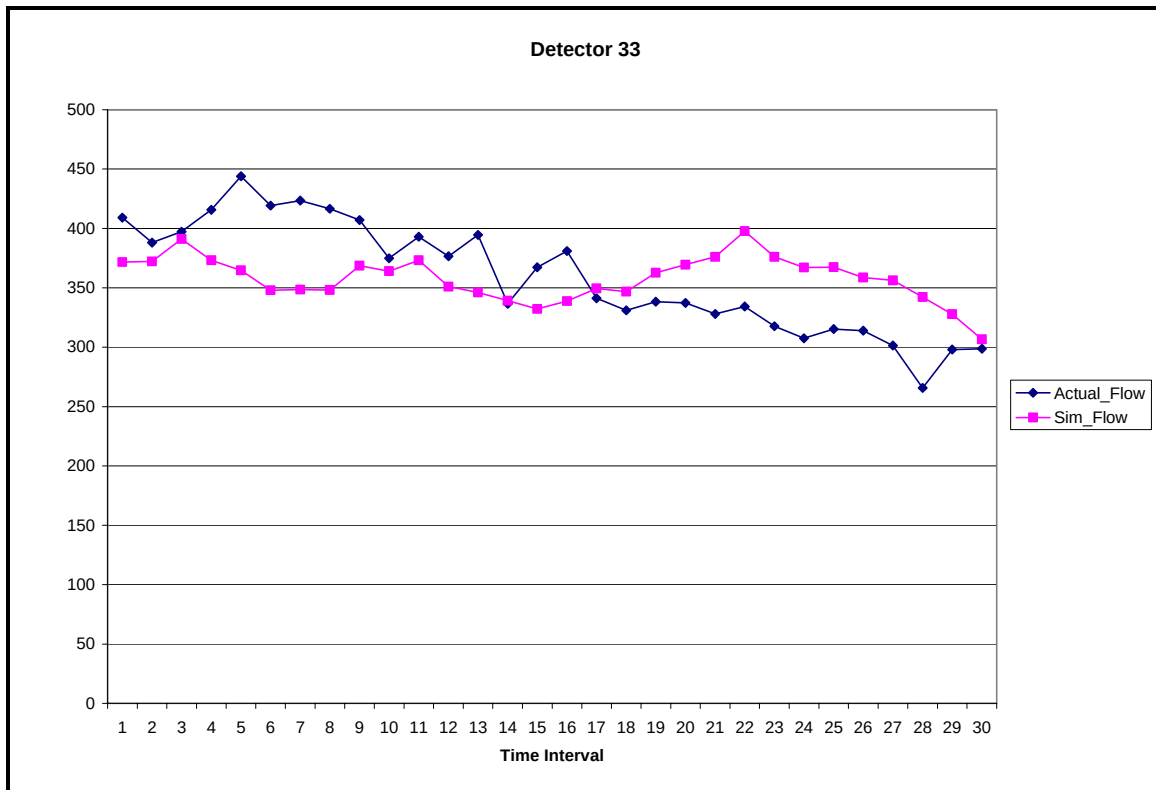
Table 4-9 Sample Profile for Calibration

5 min. Time Interval	Detector 33	Detector 34	Detector 35	Detector 36	Detector 37	Detector 38
1	69.559375	72.533838	51.863793	51.558779	129.51719	164.17302
2	46.72305	59.15042	24.954762	57.257103	128.39211	150.62191
3	20.343805	59.248712	41.972168	61.921264	120.50598	138.7657
4	20.960125	48.351163	23.536957	52.832558	102.17303	120.85668
5	-20.506291	5.006061	-18.988889	3.674081	47.386508	71.49776
6	7.134429	0.126744	-30.644385	-6.235185	48.409259	77.183736
7	-31.27769	-25.332443	-34.850922	1.847475	51.675889	73.670356
8	-27.381818	-26.556075	-49.192157	-32.141471	39.66626	78.234146
9	-75.958924	-68.602363	-85.178414	-11.09697	45.691897	48.743347
10	-84.176549	-55.179707	-43.985176	-24.458333	28.491304	62.654348
11	-82.743647	-65.775732	-61.265928	10.441667	35.980435	58.023144
12	-85.721599	-29.162552	-36.471795	17.5	71.547826	94.797101
13	-50.323689	-20.768519	13.516667	51.77963	60.041546	61.100813
14	-6.820243	25.917918	-4.553704	12.61693	6.212102	26.190041
15	-20.501675	-15.542718	-59.300233	-6.565862	39.674411	83.048816
16	-80.546296	-49.853061	-15.62037	33.086054	44.526753	51.88547
17	-42.275655	6.35	-2.598945	-9.793197	11.117384	35.853226
18	-36.687892	-36.570523	-48.399828	-39.792424	13.945652	47.010145
19	-105.93318	-77.943647	-44.980203	-11.457549	36.570126	81.76406
20	-103.86521	-61.036957	-41.527508	-6.069723	50.949467	102.65708
21	-57.753812	-14.925221	-8.555769	-5.079762	58.938272	93.217901
22	-62.604878	-30.108696	-18.383054	-15.012783	44.445347	100.82022
23	-12.505777	-33.54809	-33.056535	-10.717325	47.992329	87.554821
24	-40.156911	-47.879952	-28.678682	-32.104854	53.903892	118.15185

5 min. Time Interval	Detector 33	Detector 34	Detector 35	Detector 36	Detector 37	Detector 38
25	-19.265591	-56.014286	-27.589968	16.825397	67.988935	121.83099
26	11.096801	-5.117472	14.225281	-26.216505	41.827273	120.78418
27	-8.344332	-30.931673	-37.086606	-20.21	52.805923	109.06915
28	-31.042593	-52.697162	-34.95	-31.869565	76.545455	124.33706
29	14.266082	-33.002475	-24.125214	-24.139956	51.460606	96.069697
30	41.708408	-12.217391	-33.37549	-49.85786	37.583333	94.836364

The objective of changing the flows from end zone was to obtain the best possible counts from the mainline in the controlled scenario. A continuous appraisal of the OD matrix had to be carried out in order to make sure that no large errors were being introduced. The calibration parameters like driver reaction time, mean headway and queue speed were changed in the next part of the calibration process. The final speed and flow profiles at the end of this process are shown in Figures 4-12 through 4-19. The *x-axis* is numbered as the number of 5 minute time intervals since the start of simulation. The *y-axis* represents the counts corresponding to the five minute interval. Table 4-10 shows the difference between actual counts and simulated counts for every five minute interval. Figure 4-12 and 4-13 depict that stations 33-35 show errors which are not large in magnitude. This can also be seen by the regions marked in lighter shade in Table 4-10. Also notable is the larger errors towards the end of the simulation between time intervals 22 to 30. Figure 4-13, 4-15 and 4-15 show that the errors at these stations are much larger. This can also be seen in Table 4-10 as completely dark regions in columns for Stations 36-40. Stations 41-47 however show decent agreement with actual values of counts as is seen by the closeness of the data points in the Figures 4-15, 4-16, 4-17 and 4-

18. Station 49 shows larger errors for the all the first 22 time intervals as is seen in Table 4-10. The next step in the process was to look at speeds corresponding to these flows and make adjustments in the profiles and if necessary in the OD matrix to decrease the errors in flows. This part is dealt with in the next section of the report.



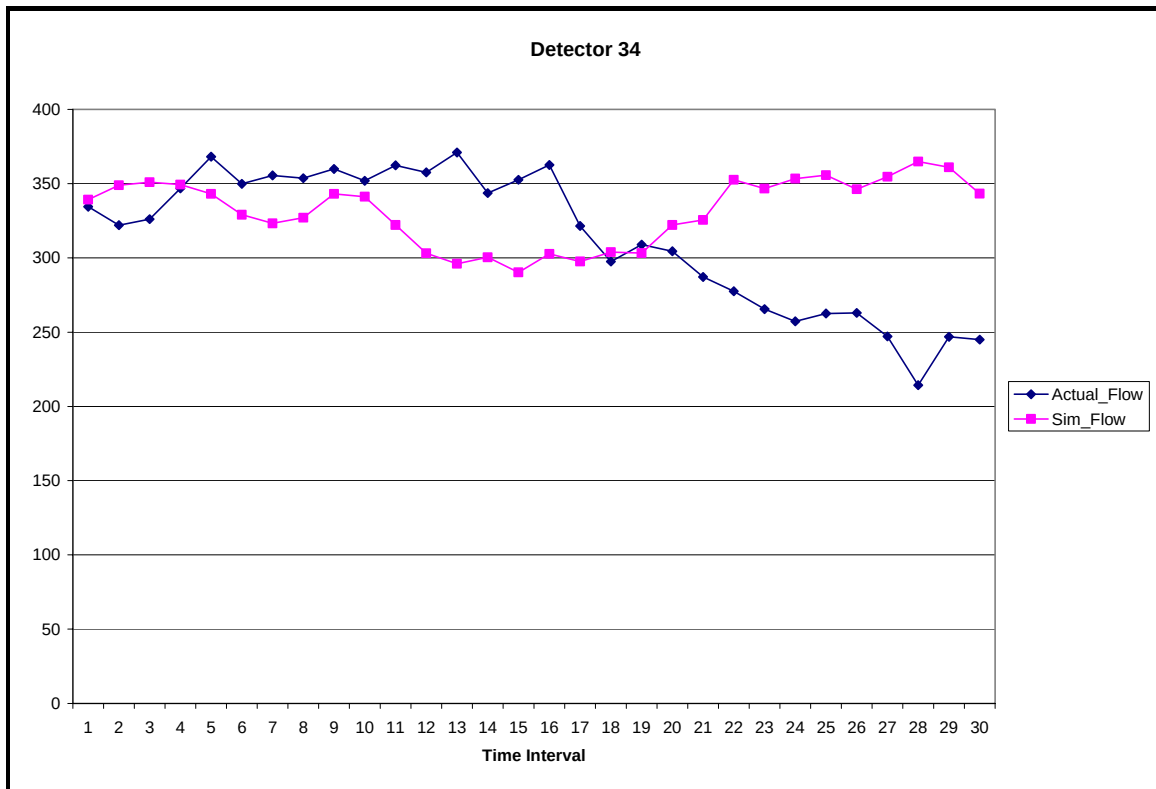
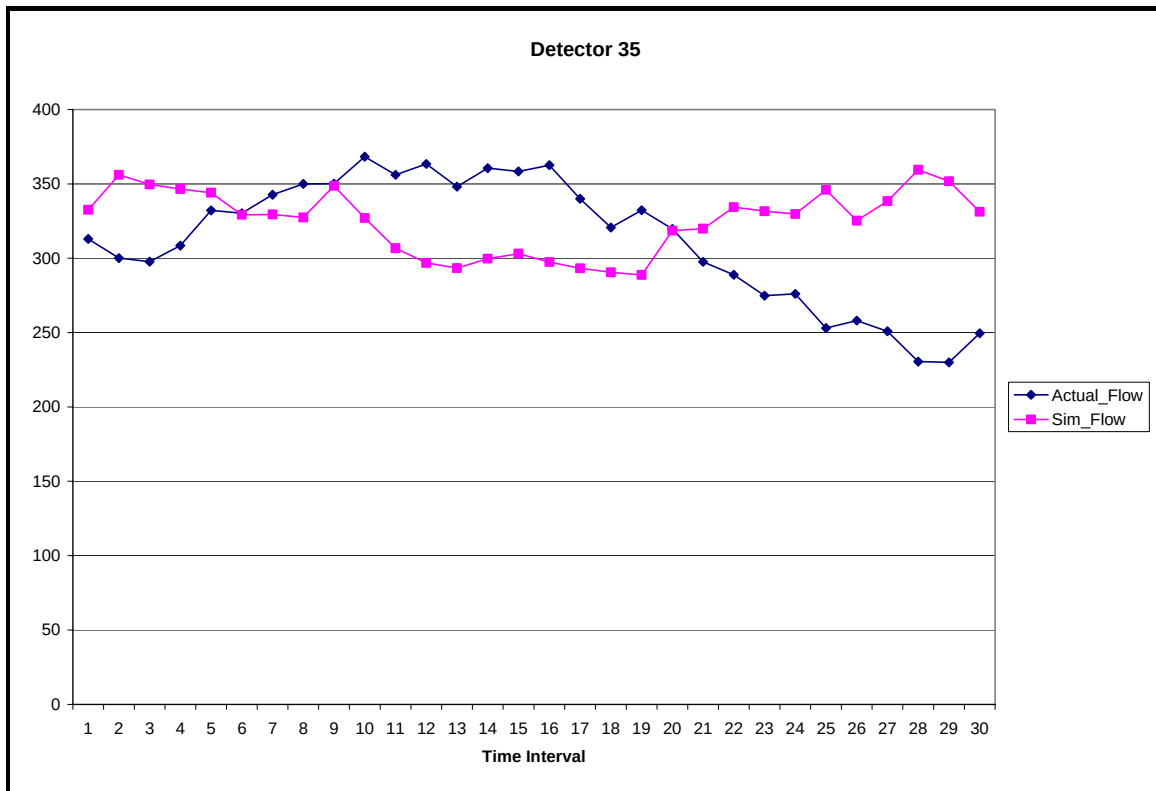


Figure 4-12 Comparison of mainline counts Station 33 and 34 EB (x axis- number of 5 min. intervals)



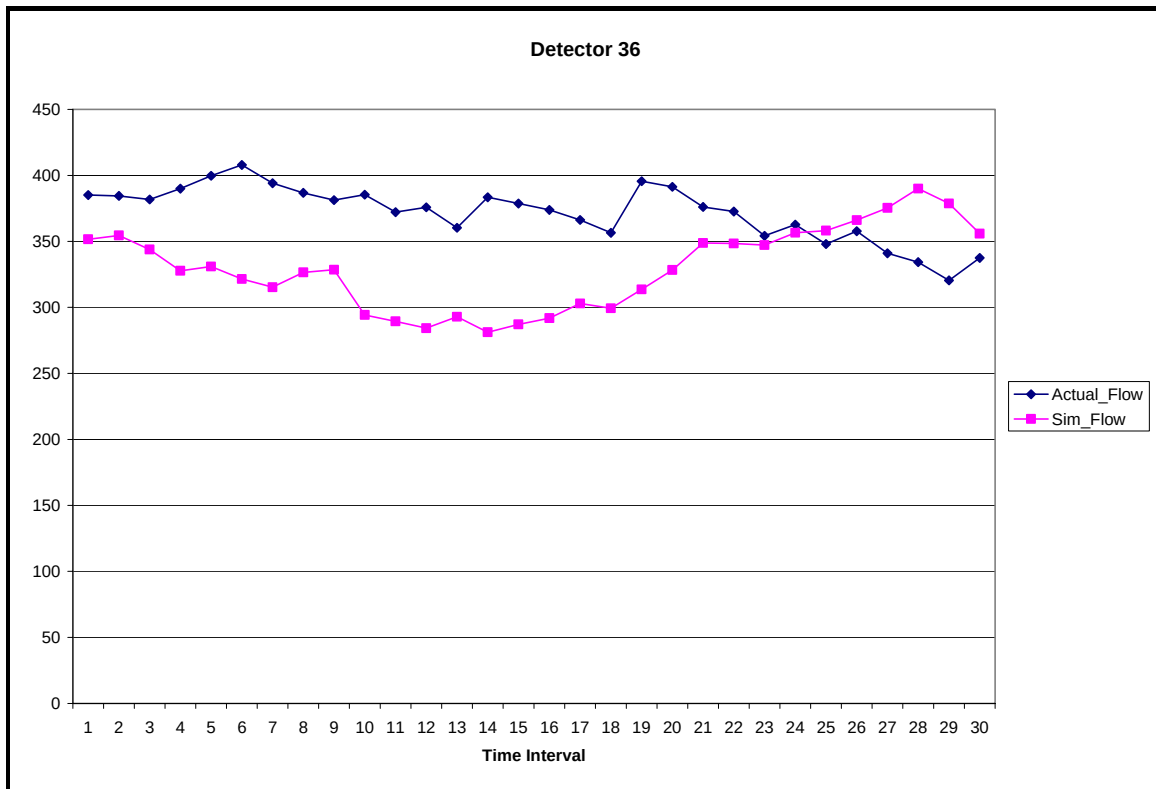
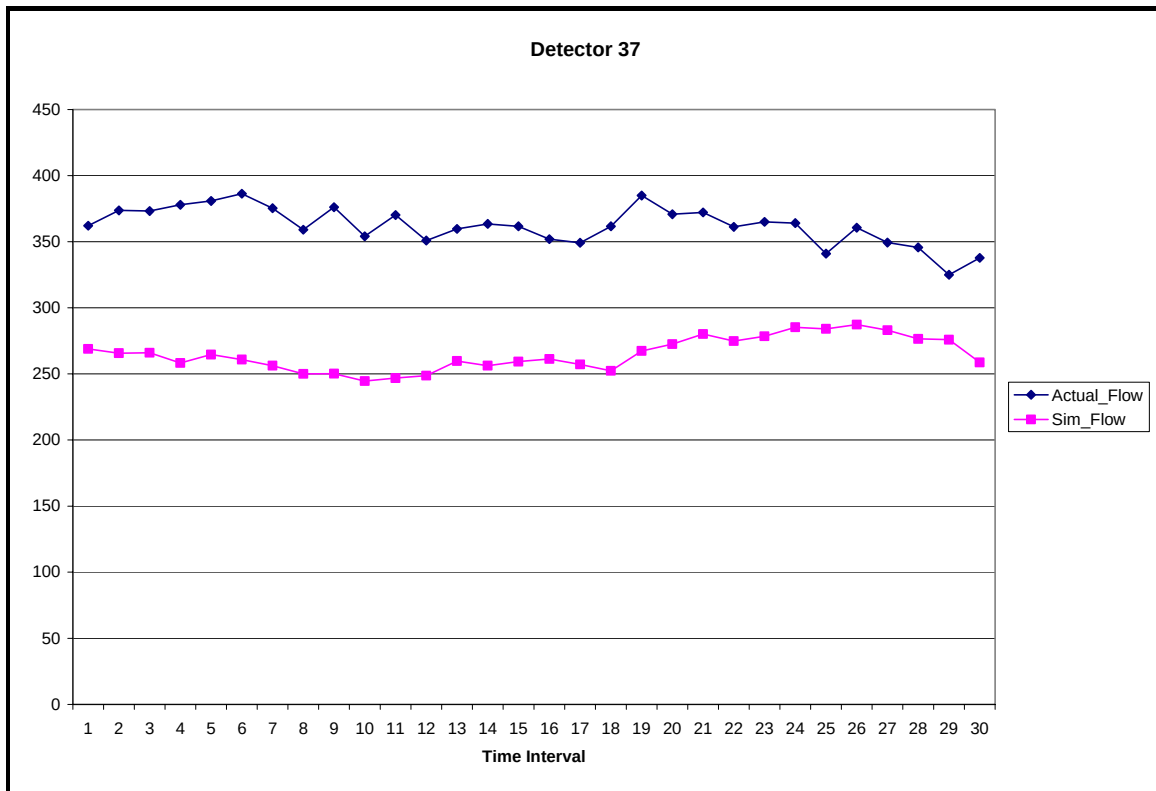


Figure 4-13 Comparison of mainline counts Station 35 and 36 EB (x axis- number of 5 min. intervals)



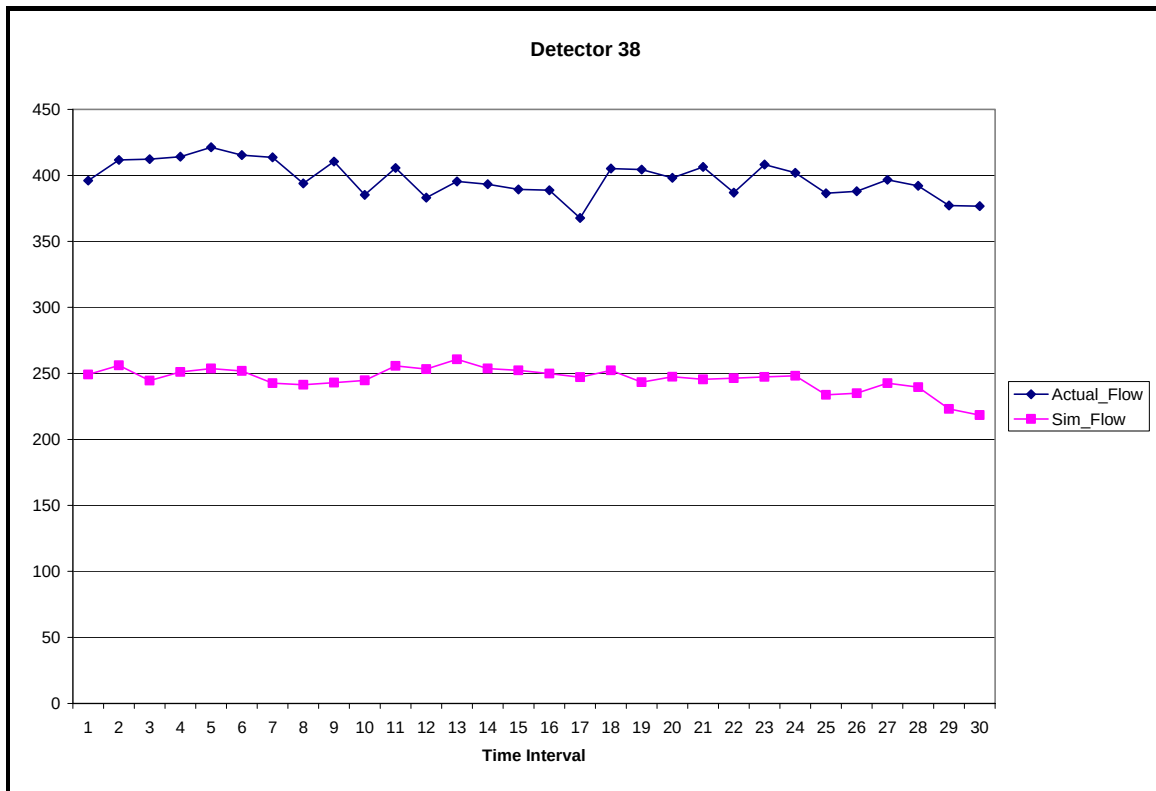
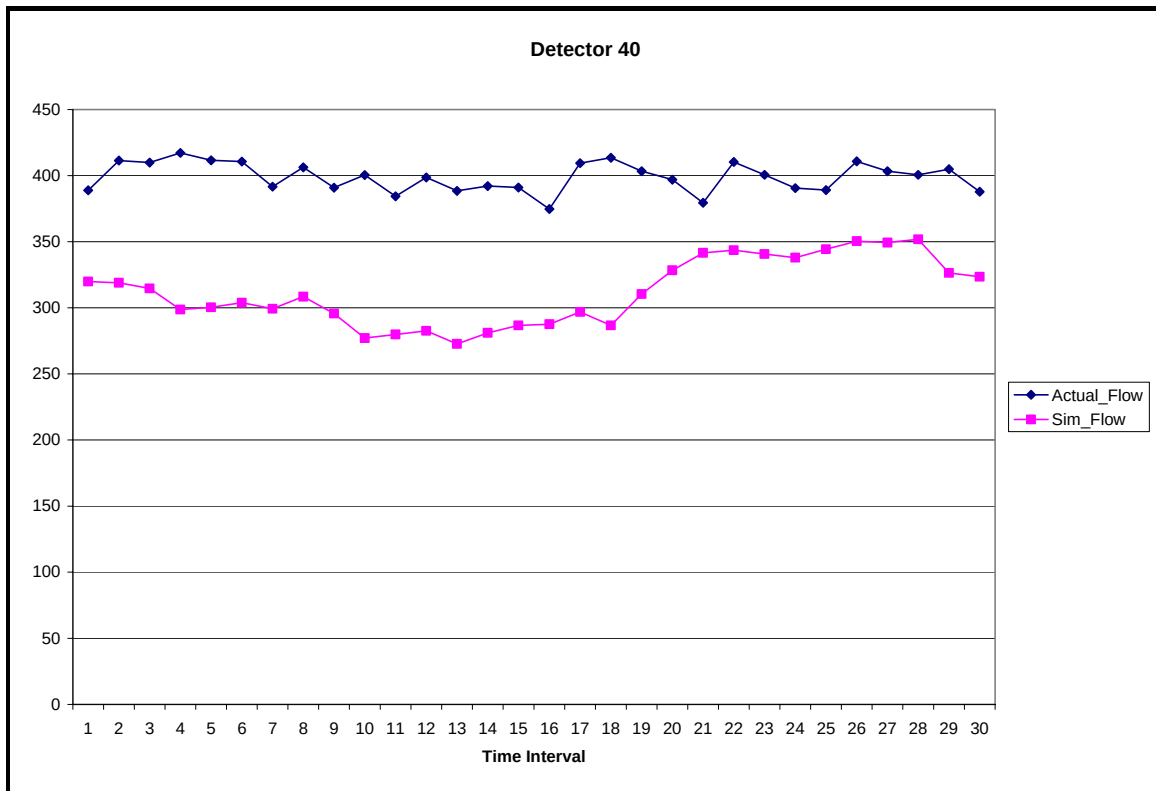


Figure 4-14 Comparison of mainline counts Station 37 and 38 EB (x axis- number of 5 min. intervals)



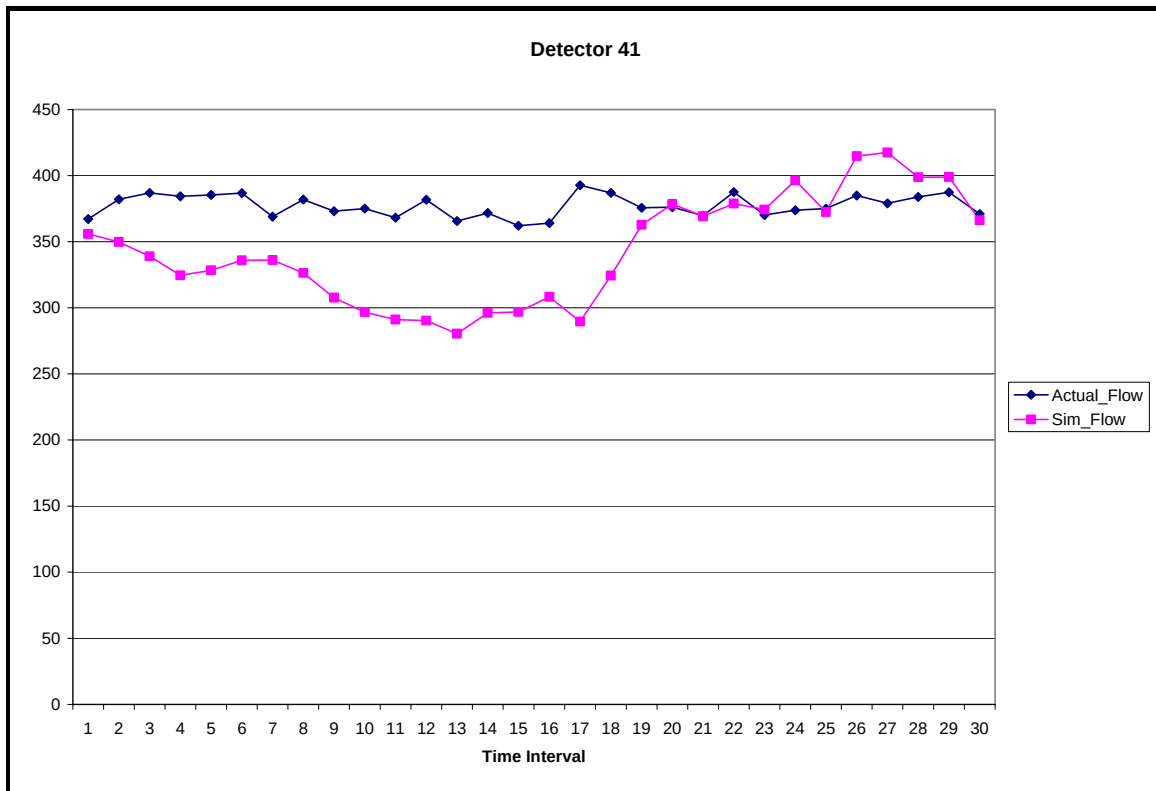
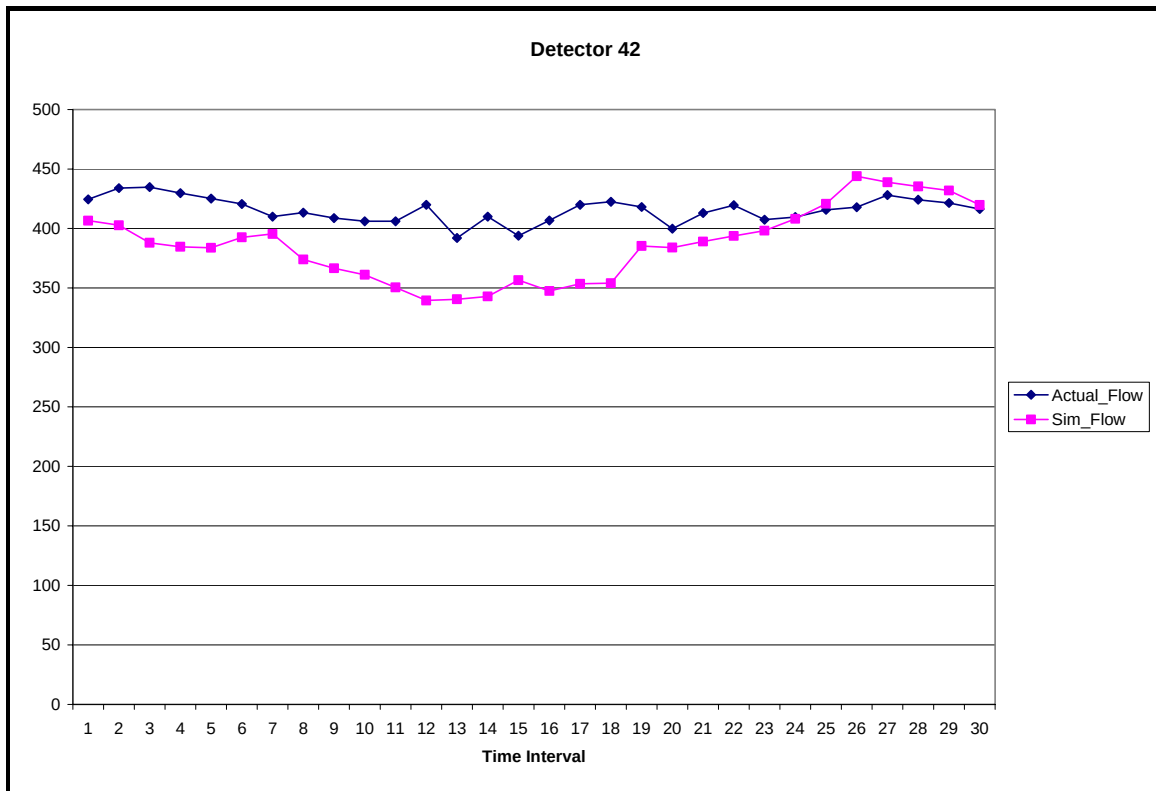


Figure 4-15 Comparison of mainline counts Station 40 and 41 EB (x axis- number of 5 min. intervals)



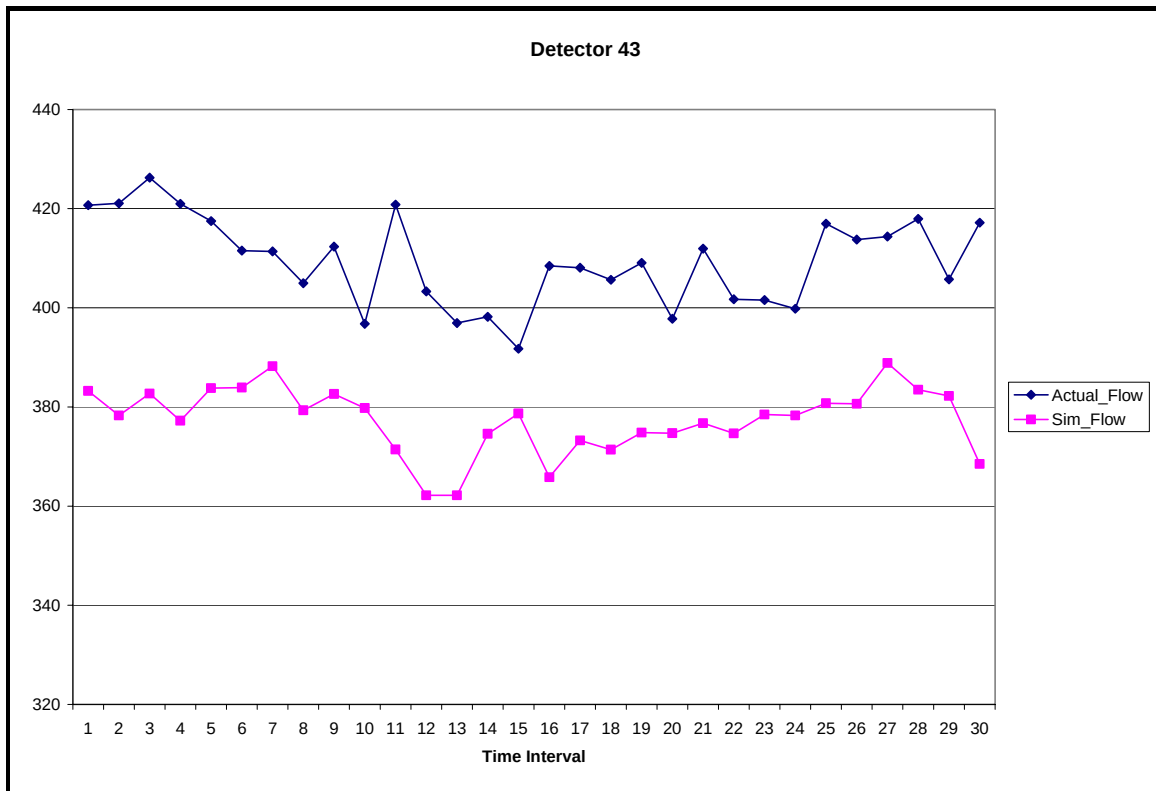
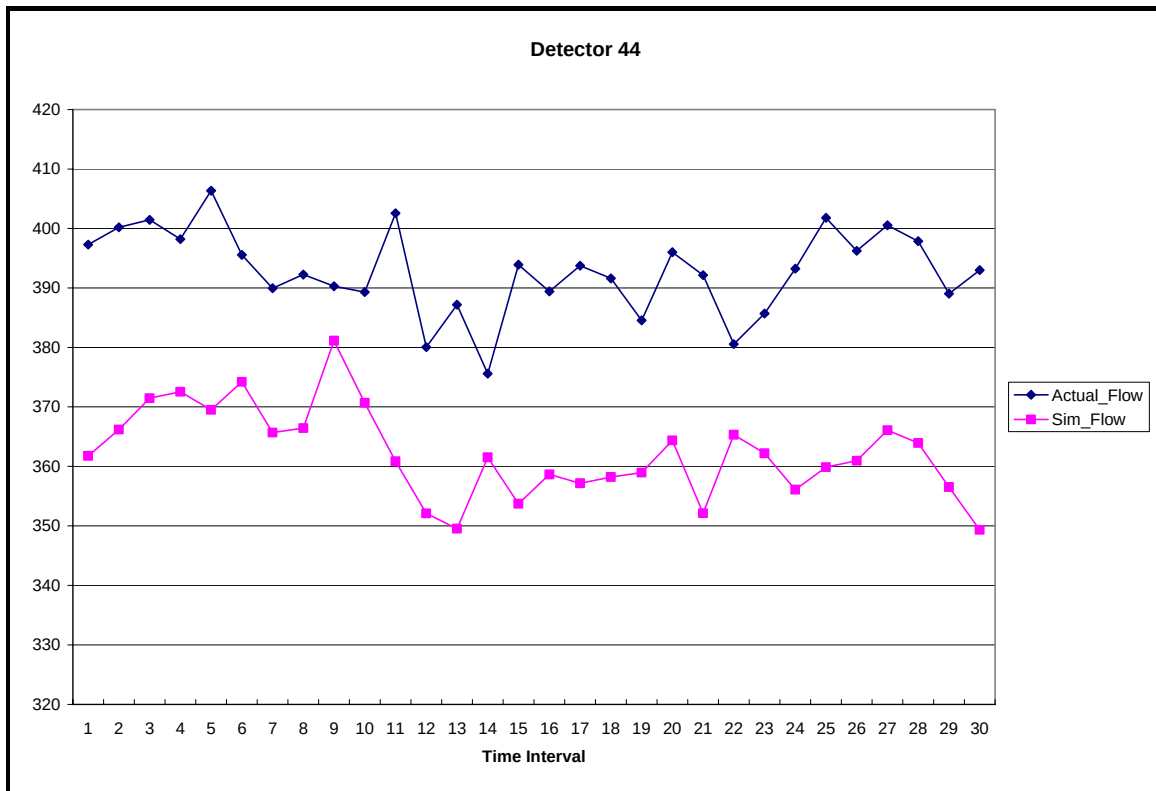


Figure 4-16 Comparison of mainline counts Station 42 and 43 EB (x axis- number of 5 min. intervals)



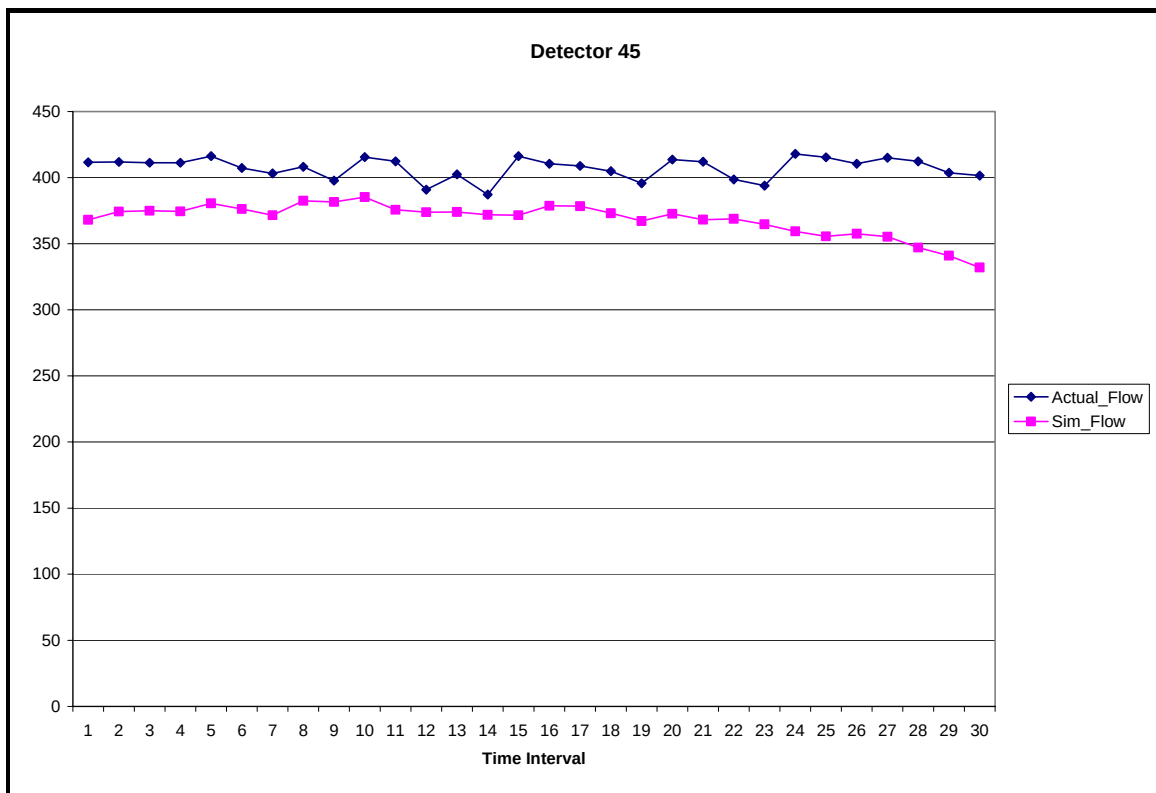
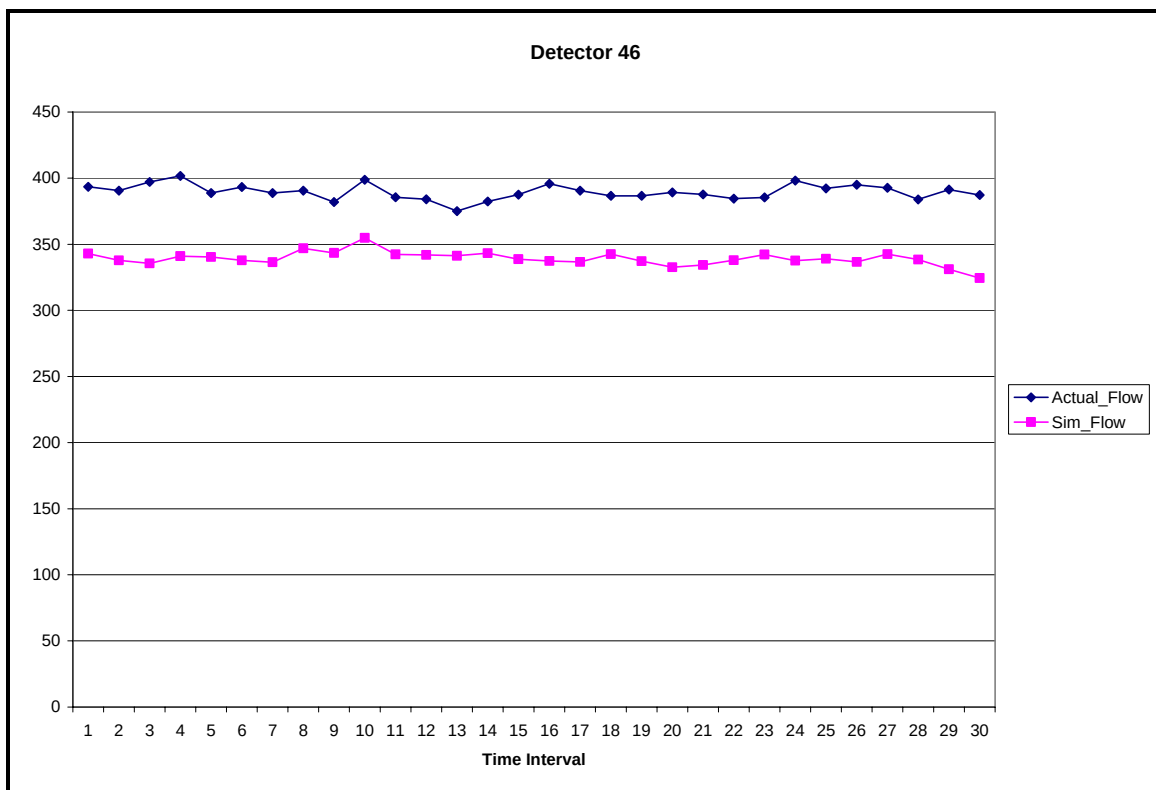


Figure 4-17 Comparison of mainline counts Station 44 and 45 EB (x axis- number of 5 min. intervals)



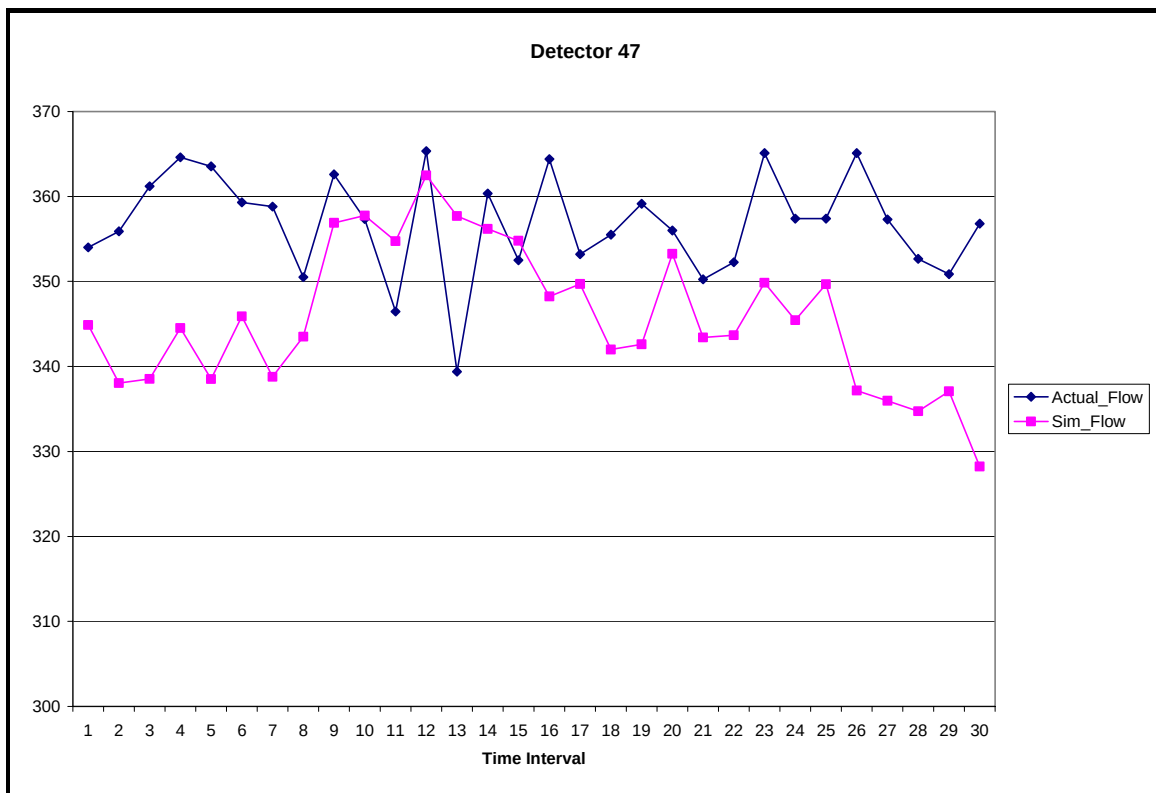
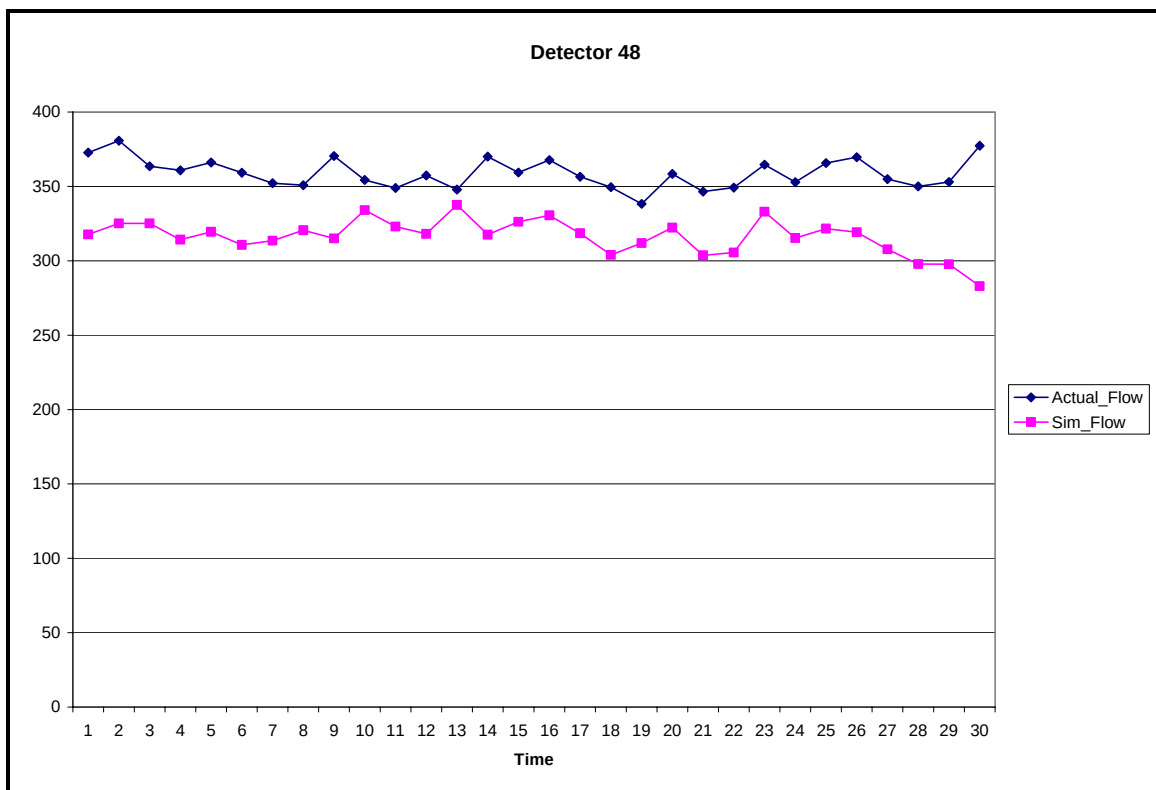


Figure 4-18 Comparison of mainline counts Station 46 and 47 EB (x axis- number of 5 min. intervals)



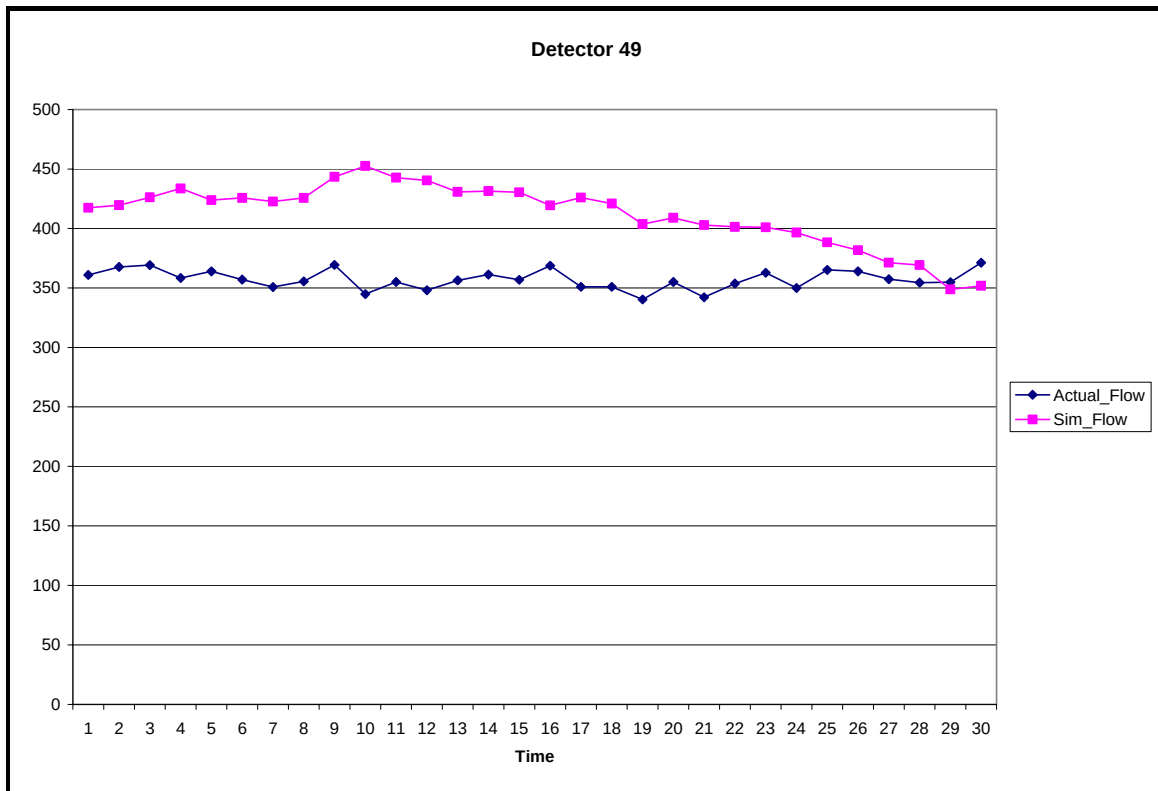


Figure 4-19 Comparison of mainline counts Station 48 and 49 EB (x axis- number of 5 min. intervals)

Table 4-10: Flow errors for five minutes Stations 33- 49. (T- 5 min Interval)

T	33	34	35	36	37	38	40	41	42	43	44	45	46	47	48	49
1	37	-5	-20	33	93	147	69	11	18	37	36	44	51	9	55	-56
2	16	-27	-56	30	108	156	92	32	31	43	34	37	53	18	56	-52
3	6	-25	-52	38	107	168	95	48	47	44	30	36	62	23	38	-57
4	42	-3	-38	62	120	163	119	60	45	44	26	37	61	20	47	-75
5	79	25	-12	69	116	167	111	57	41	34	37	36	48	25	47	-60
6	71	21	1	86	126	163	107	51	28	28	21	31	55	13	49	-69
7	75	32	13	79	119	171	92	33	14	23	24	32	52	20	39	-72
8	68	27	23	60	109	153	98	55	39	26	26	26	44	7	30	-70
9	38	17	1	53	126	167	95	65	42	30	9	16	38	6	55	-74
10	11	11	41	91	109	140	123	78	45	17	19	30	44	0	20	-108
11	20	40	49	83	123	150	105	77	56	49	42	37	43	-8	26	-88
12	26	55	66	92	102	130	116	91	80	41	28	17	42	3	39	-92
13	48	75	55	67	100	135	116	85	51	35	38	29	34	-18	10	-74
14	-3	43	61	102	107	140	111	76	67	24	14	15	39	4	52	-70
15	35	62	55	92	102	137	104	65	37	13	40	45	49	-2	33	-74
16	42	60	65	82	91	139	87	56	59	43	31	32	58	16	37	-51
17	-8	24	47	63	92	121	113	103	66	35	37	30	54	3	38	-75
18	-16	-6	30	57	109	153	127	62	68	34	33	32	44	14	46	-70
19	-24	6	44	82	118	161	93	13	33	34	26	29	50	17	26	-63
20	-32	-18	1	63	98	151	69	-2	16	23	32	41	57	3	36	-54
21	-48	-38	-22	27	92	161	38	0	24	35	40	44	53	7	43	-61
22	-64	-75	-45	24	86	140	67	9	26	27	15	30	47	9	44	-48
23	-58	-81	-57	7	87	161	60	-4	9	23	23	29	43	15	32	-38
24	-60	-96	-54	6	79	154	53	-23	2	22	37	58	61	12	38	-47
25	-52	-93	-93	-10	57	153	45	2	-5	36	42	60	53	8	44	-23
26	-45	-83	-67	-8	73	153	60	-30	-26	33	35	53	58	28	50	-18
27	-55	-107	-87	-34	66	154	54	-38	-11	25	34	60	50	21	47	-14
28	-77	-151	-129	-56	69	153	49	-15	-11	35	34	65	46	18	52	-15
29	-30	-114	-122	-58	49	154	79	-12	-10	24	33	63	60	14	55	6
30	-8	-98	-82	-18	79	158	64	5	-3	49	44	69	63	29	94	19

4.10.7 Calibration of Mainline Counts and Speeds from Simulations

The calibration of speeds requires us to come up with a sample of speeds at the locations in the network where we were identifying our speed scenarios. Under these circumstances it is usually recommended to use either the loop data (if it is available) or to conduct field studies. Fortunately, for our case, the presence of loop data meant that there were a large number of observations from which we could derive the maximum probable speed regimes at our locations. For the purpose of this study the data is extracted for the time interval of 4PM to 7PM, over the same time period mentioned above.

Since for the final testing of the scenarios it was important to have speed regimes that mirrored the actual conditions at all the five minute intervals. This meant that the speeds had to be calibrated at every five minute intervals too. The analysis for the flows at the end of last section showed (Table 4-10) that there were larger errors related to stations 36, 37, 38 and 40. Since the simulated counts at these stations were below the actual counts, logically it would follow that the speeds at these stations will be considerably higher than the actual speeds too. Figures 4-20 and 4-21 show the speed comparisons for these stations. The errors bars on the actual speed curves indicate the range of 10 % errors. A data point lying within these error bars would indicate an average speed for 5 minutes from simulation that is within 10% of the actual speed. Figure 4-20 and 4-21 show that the speeds at stations 36, 37 and 40 are almost always much higher than the speeds in the actual conditions.

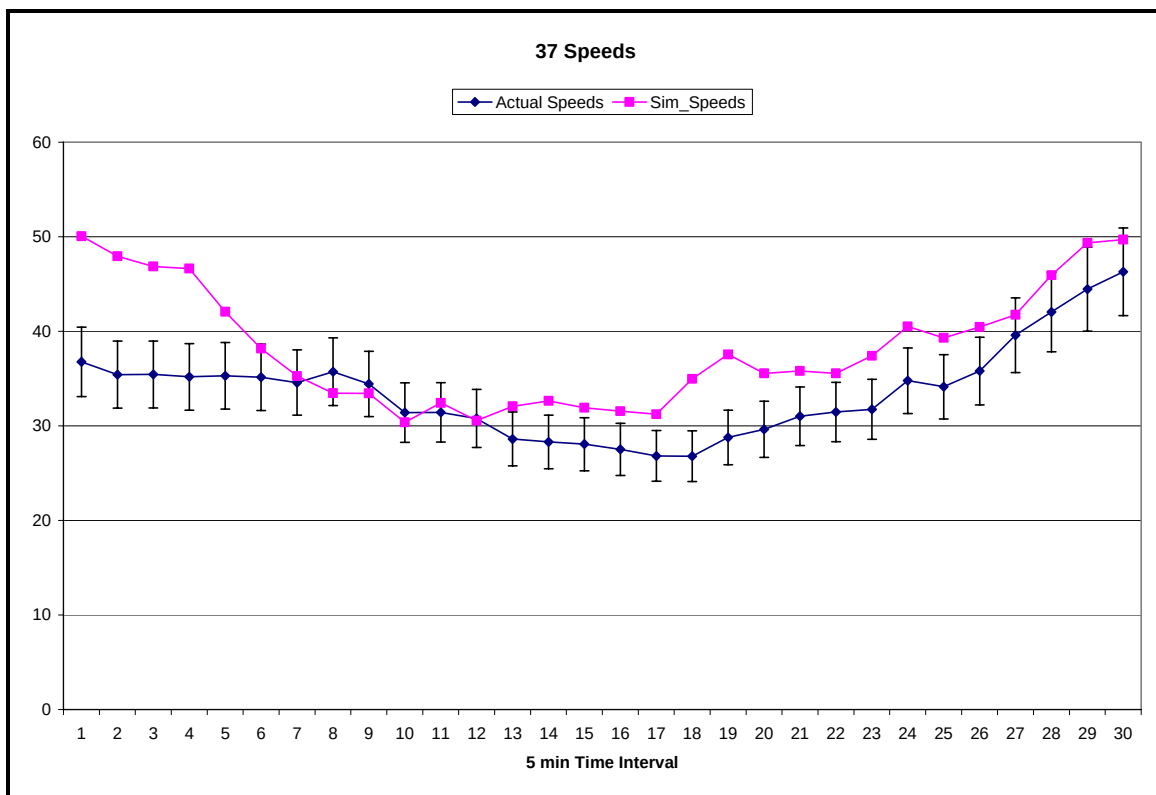
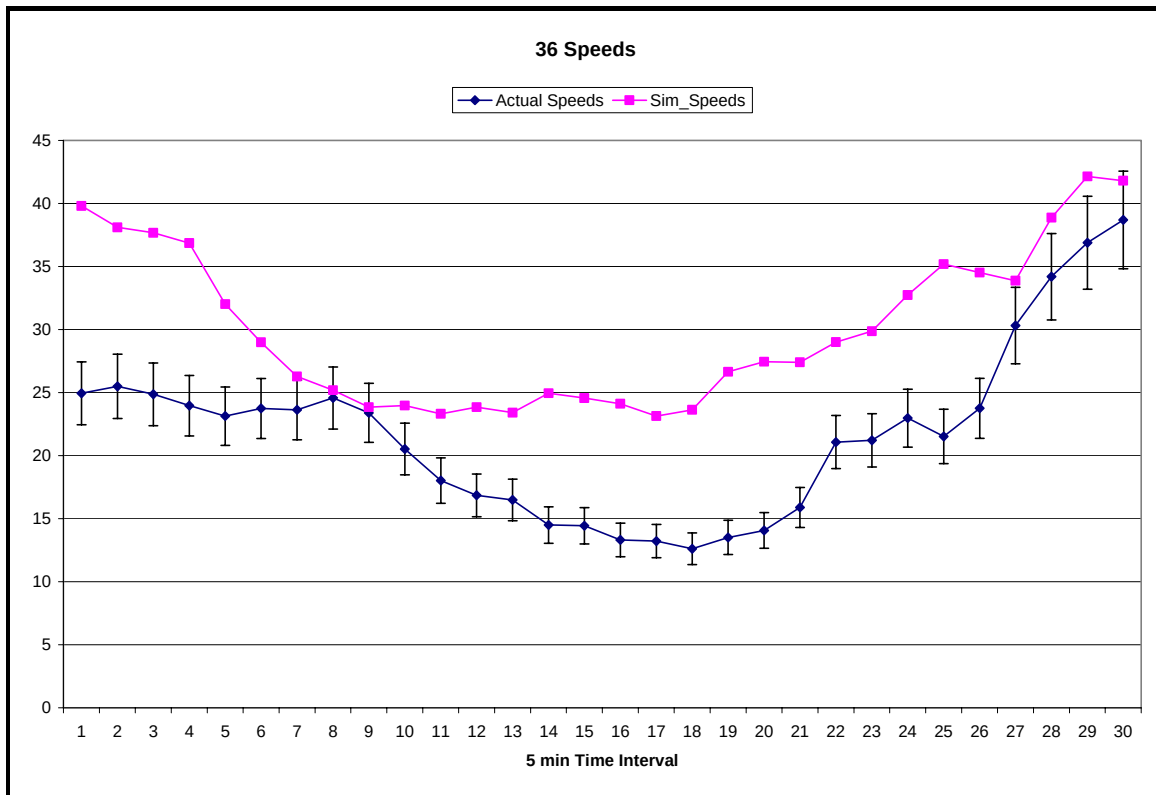


Figure 4-20 Speed comparisons for Stations 36 and 37 with 10% error bars

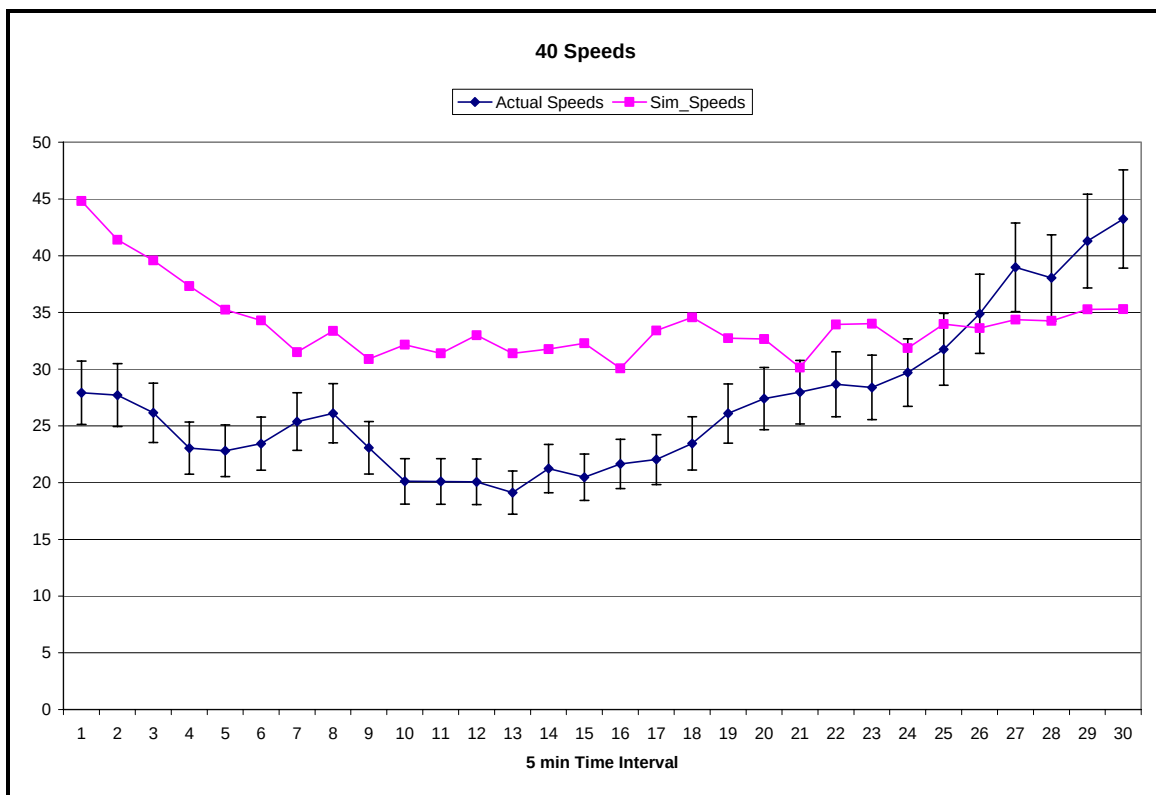
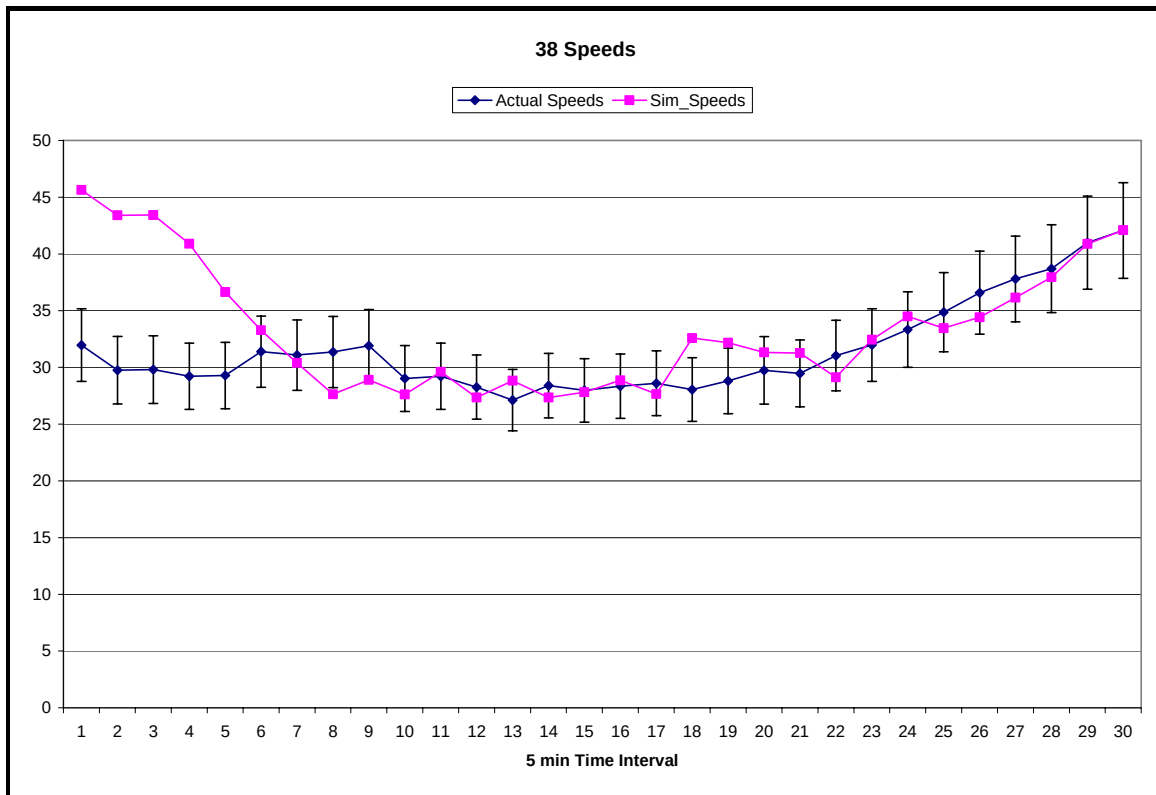


Figure 4-21 Speed comparisons for Stations 38 and 40 with 10% error bars

This analysis of speeds showed that there are significant differences in speeds at most of the stations which needed to be adjusted. The really low speeds generally occurred at the same time when queues would form at a station within the time interval. As such one of the ways to locate queues was to look for significant drops in speeds in actual conditions and then try to induce congestion within those time intervals in the simulation also. This comparison and adjustment for speeds is detailed in section 4.10.8. There were some anomalies in the behavior of simulation after completing the process of matching the speeds. Each of these problems had to be dealt with on a station by station basis and adjustments made accordingly.

4.10.8 Comparison of speeds

Unlike the flow, where a GEH statistic can be used to make sure that the network is properly calibrated, literature does not seem to have such a measure of the speeds. Wisconsin DOT guidelines (2002) for calibration criteria leave the calibration of speeds to the satisfaction of the analyst. The nature of this study however meant that speed had to be an intrinsic parameter verifiable numerically. A 95 % CI for a $(CI / \text{Standard dev.})$ Ratio of 1, requires somewhere between 18 to 23 repetitive runs in order to account for random variances in simulation data. As such we choose to do a total of 20 runs for calibration in either case. Another notable objective is to make sure that the network follows the criteria at least 85 % of the locations. Figure 4-22 shows the speed profile for station 43 along with error bars that depict the variance of data for the actual speeds. Each error bar denotes the 95% confidence interval based on the t-statistic. The figure

depicts that points that lie within the error bar for any time interval are likely to be from the same set of points as the actual data.

The speeds were compared for the 20 runs of the network calibrated for flows. Since we were looking at the speeds already in the calibration process and there is intrinsic relationship with flows, the speeds were found to be within reasonable error of the actual data at a few stations. The selection of data from 20 different days showed that for station 43 for example- there was a compliance with the general speed pattern over most of the time period except in the last half hour of simulation. These errors in the speed patterns were then addressed on a station by station basis.

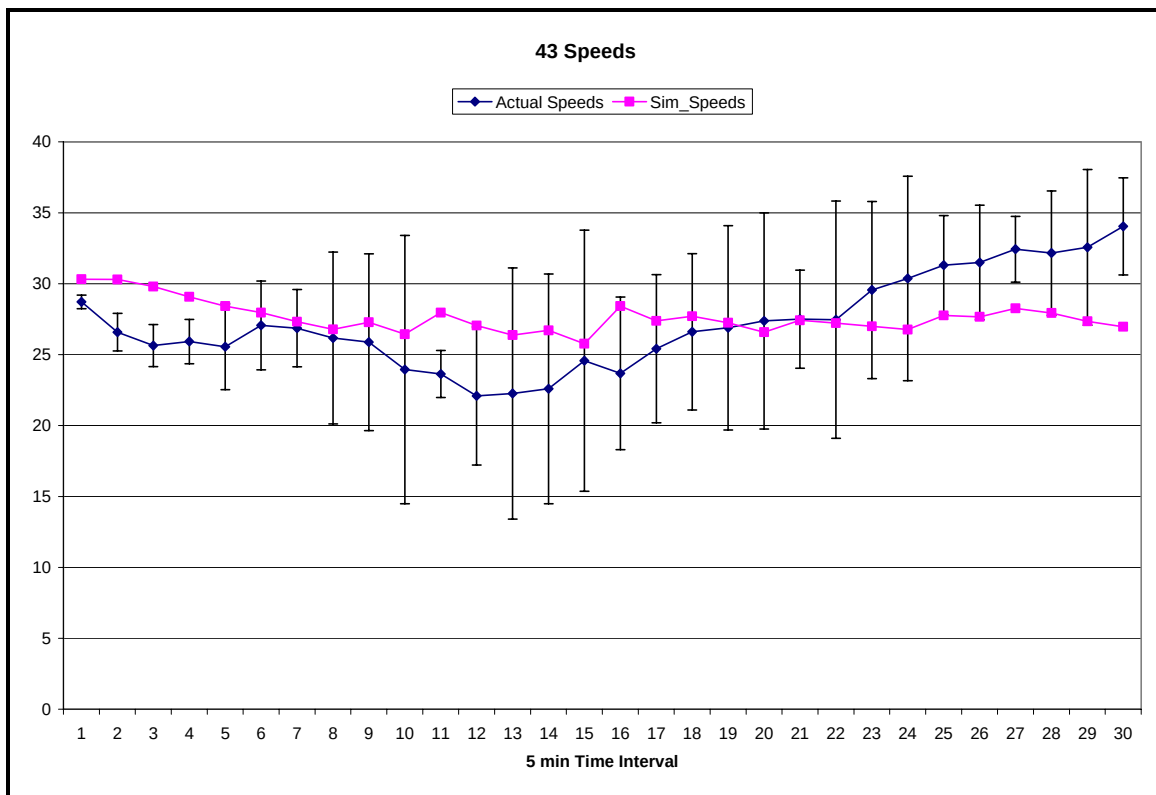


Figure 4-22 Speed comparisons for Stations 38 and 40 with 95% CI bars

For the East Bound corridor of the network, a behavior continuously observed at the stations 38 to 45 was that the speeds in the end tended to be lower than the speeds under the actual conditions. This can also be seen in Figure 4-22 where the speeds in the last 5 intervals fall significantly below the actual speeds.

A t-test was done for every five minute intervals to check if the simulated mean speed lies within the sample data collected from the actual scenarios.

The test was randomized and it was checked if the means from speeds within the different runs of the simulation corresponded to the actual means over five minutes. An important observation here is that the variance of the data from the simulation was much more as compared to the data obtained from the field. Hence comparing distributions is not the reasonable way to go. This despite the fact, the variance in the field data was far too much which meant that there was a larger confidence interval for the speeds at any interval at any station.

After all the post processing for the speed data was finished the error rates needed to be calculated. A visual inspection of comparison of mean five minute speeds from 20 simulation runs and mean 5 minute speeds over 20 weekdays was done from the graphs at every step of the calibration. A consistent phenomenon observed was that apart from stations where heavy congestion was taking place (these could be flagged by the formation of queues), the comparison of speeds showed that the less congested stations showed higher speeds than in the actual scenario. This could be attributed in PARAMICS to the fact that in the absence of congestions the vehicles tend to go at the maximum

possible speeds. This meant that every time the vehicles emerged from congestion, they tended to speed up leading to higher over all average speeds at some locations. However, there were a few stations which were not following the trend. From an analysis of different days it was found that stations 45 through 49 tended to have greater congestion. However, the average speeds at these stations were still significantly higher than at the more congested locations. A visual inspection of the simulation showed that there was recurring queues forming downstream of station 49- something not happening at the end of the calibration process for flows.

A change in mean headway which was already at a low value wasn't thought wise in this case. The queuing behavior parameters that were used were queueing speed of 8mph and queueing distance of 9 ft. These values provided better results for dissipation of queues. Increasing either of these values tended to increase the persistence of queues at any location. There was also a strong relationship between these figures and the headway value used. However, the OD matrix could be adjusted so as to decrease the flow of vehicles at three ramps close to this location by 50 vehicles each and this change was effected half an hour into the simulation, when the first congestion was hitting. As was expected this prevented the queue from forming instantly into the simulation. This meant that there was some increase in the speeds at some downstream stations too. However, this brought the values for the simulation closer to actual value. The possible increase of the error in OD was compensated by distributing this loss over later profiles within the hour. However, the decrease was necessary to carry out in two 5 minute intervals before the time when the queues were forming previously.

At the end of each calibration process, a single run of seed was observed to make sure that there was no inconsistent queuing in the scenarios. This check for queues had to be done at every step of the simulation.

To calibrate the speeds and flows, the following cases shown in Table 4-11 were tested along with adjustments in end zones of OD.

Table 4-11 Cases Tested for Calibration Parameters

Case	Headway	Reaction Time
1	1.00 sec	0.60 sec
2	1.25 sec	0.45 sec
3	0.60 sec	0.45 sec
4	0.50 sec	0.50 sec
5	1.00 sec	0.60 sec
6	1.65 sec	0.45 sec
7	0.60 sec	0.45 sec
8	0.50 sec	0.50 sec

Based on the overall errors in Speeds and Flows, the best measure was chosen to be Headway of 1.00 second and Reaction time of 0.45 seconds. The values agreed almost completely with those chosen by Dilmore (2005) (1 second and 0.4 second respectively) and the values of queuing speed (8 mph) and queuing distance (9 ft) were used from those provided by Dilmore (2005) (Also see Chapters 2 and 3 of this volume). The only difference being the value of reaction time. The fact that these values provided the best possible queuing behavior confirmed the conclusions drawn in the verification process (in Chapter 2; also by Dilmore, 2005) about the effect of these parameters.

Since, there is no alternative route available to the drivers as only the freeway has been simulated, it wasn't necessary to calibrate the dynamic route choice parameters. The simulation parameters were found to be consistent with those found in literature although the reaction time was lower than most observed values in literature. This however had to be done to improve the queue dissipation at locations just downstream of any ramp to prevent the occurrence of shockwaves.

4.10.8.1 Speed Increases

One of the problems in the calibration process was the fact that some stations had very high speed values in the midst of a congested period. The reason was the dissipation time of the upstream queues especially at locations upstream of stations 38 and 49. Thus the counts at these stations were unusually lower than the actual scenario (steady congestion and no dissipation upstream) and the speeds were therefore higher for longer periods. Thus a way had to be found to induce queues at the downstream locations and create breaks in the traffic stream and in a way induce virtual incidents along with breaking the queues upstream long enough to allow regular flows. This was done by introducing slight peaks in the volumes from stations about 45 minutes into the simulation. A similar strategy as that used for queues at stations 45 to 49. This meant that there was an initial spike in the counts but due to the simultaneous decrease at upstream locations in the profile only, the behavior could be improved drastically.

4.10.8.2 Accepted Scenario

The final accepted scenarios are presented in Figures 4-23 to 4-31. These figures show that there are significant errors that have persisted at some locations.

The overall speeds had an average error of 18.9% calculated every 5 minutes and flows had an average error of 8.26 % calculated every 5 minutes. It did involve some give and take in terms of flows and matching speeds accurately however, the speeds for a number of stations were found to be within acceptable error. Some of the stations e.g. Station 40 and 41 present significantly higher speeds than in normal scenarios, however, the corresponding errors in flows are minimal and it was necessary to make sure that the speeds at station 43 are comparable to field data because station 43 was the key station used in the testing of ramp metering scenarios. A large part of the error for speeds and flows was contributed by station 38, 40 and 49. This is illustrated in Table 4-12. This table follows directly from Table 4-10 and as can be seen, the dark colored regions (marking higher errors in flow) have been reduced for most of the network except stations 38, 40 and 49.

Table 4-12 Station wise errors in flows at end of calibration (T- 5 min Interval)

T	33	34	35	36	37	38	40	41	42	43	44	45	46	47	48	49
1	70	73	52	52	130	164	78	15	5	33	36	35	33	-13	56	-50
2	47	59	25	57	128	151	67	16	4	12	4	-2	26	-4	10	-86
3	20	59	42	62	121	139	64	-8	-30	-14	-11	-4	4	-20	26	-79
4	21	48	24	53	102	121	46	5	-19	-2	-31	-24	24	-13	12	-111
5	-21	5	-19	4	47	71	41	-5	-31	-34	-11	-22	-16	-34	16	-97
6	7	0	-31	-6	48	77	26	-24	-38	-30	-55	-38	18	-36	11	-85
7	-31	-25	-35	2	52	74	33	-34	-72	-46	-28	-39	-4	-14	51	-62
8	-27	-27	-49	-32	40	78	-6	-22	-37	-53	-37	-22	8	-17	19	-94
9	-76	-69	-85	-11	46	49	23	-14	-33	-10	-22	-16	7	-53	-4	-127
10	-84	-55	-44	-24	28	63	59	33	1	-12	-36	-57	-36	-44	27	-100
11	-83	-66	-61	10	36	58	62	23	-9	-46	-34	-19	16	-40	-12	-134
12	-86	-29	-36	18	72	95	45	-3	-21	-9	8	-13	-25	-73	2	-120
13	-50	-21	14	52	60	61	27	32	12	-4	-33	-61	-11	-59	-22	-105
14	-7	26	-5	13	6	26	47	0	-12	-55	-41	-47	-32	-45	50	-77
15	-21	-16	-59	-7	40	83	25	-20	-50	-61	-51	-24	27	-33	-8	-108
16	-81	-50	-16	33	45	52	3	-28	-43	-24	-3	-28	4	-28	24	-64
17	-42	6	-3	-10	11	36	2	17	3	-23	-18	-23	18	-16	33	-81
18	-37	-37	-48	-40	14	47	57	-3	-2	-8	-4	-15	-12	-39	38	-86
19	-106	-78	-45	-11	37	82	43	-30	-19	-21	-22	-14	15	-23	18	-62
20	-104	-61	-42	-6	51	103	8	-69	-40	-17	-11	-3	17	-36	32	-56
21	-58	-15	-9	-5	59	93	-9	-50	-30	-14	2	-1	15	-11	66	-47
22	-63	-30	-18	-15	44	101	-7	-62	-31	4	-8	7	30	-24	17	-68
23	-13	-34	-33	-11	48	88	27	-26	-27	-13	1	-5	-11	-28	16	-59
24	-40	-48	-29	-32	54	118	-6	-77	-30	-16	-6	11	11	-41	3	-77
25	-19	-56	-28	17	68	122	12	-54	-56	-9	-6	5	-7	-46	16	-55
26	11	-5	14	-26	42	121	16	-76	-76	-19	-26	-13	-1	-25	18	-29
27	-8	-31	-37	-20	53	109	-16	-104	-96	-43	-34	-7	4	-4	66	-10
28	-31	-53	-35	-32	77	124	-26	-95	-91	-42	2	46	26	-2	62	-15
29	14	-33	-24	-24	51	96	-14	-90	-71	19	18	43	43	-17	18	-21
30	42	-12	-33	-50	38	95	36	-13	-30	-1	14	32	6	-10	87	10

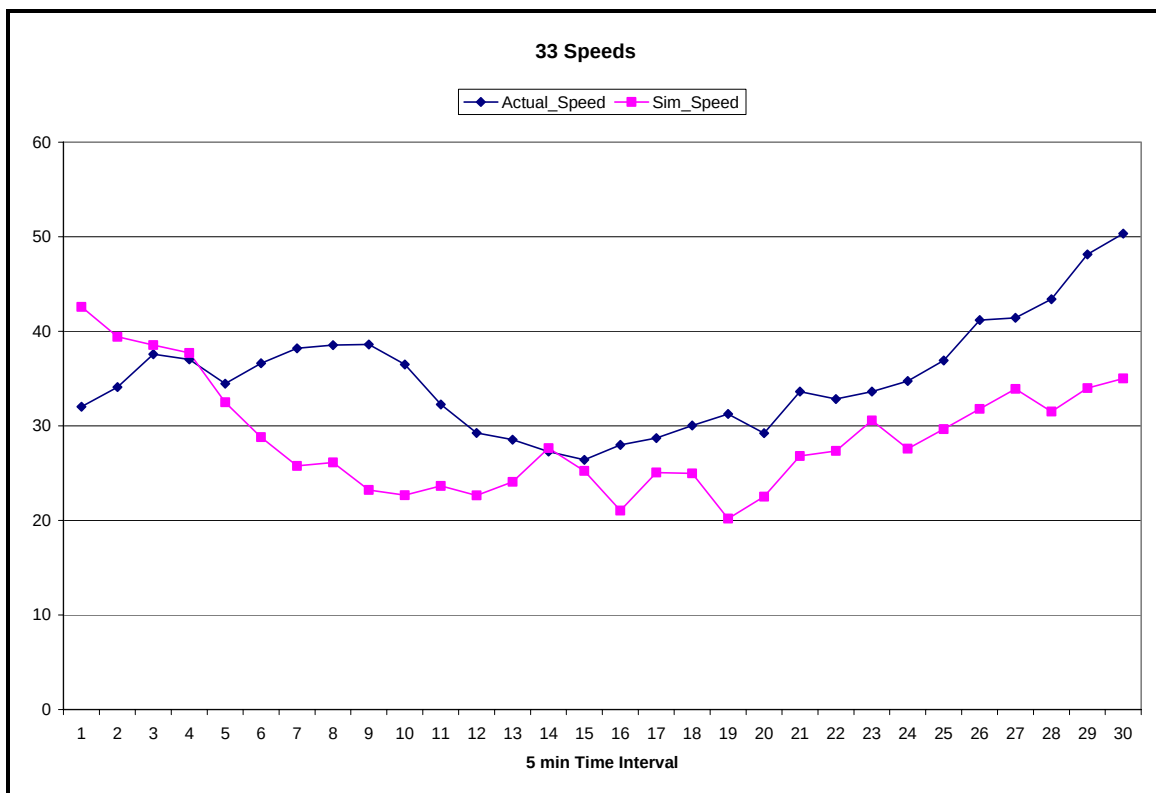
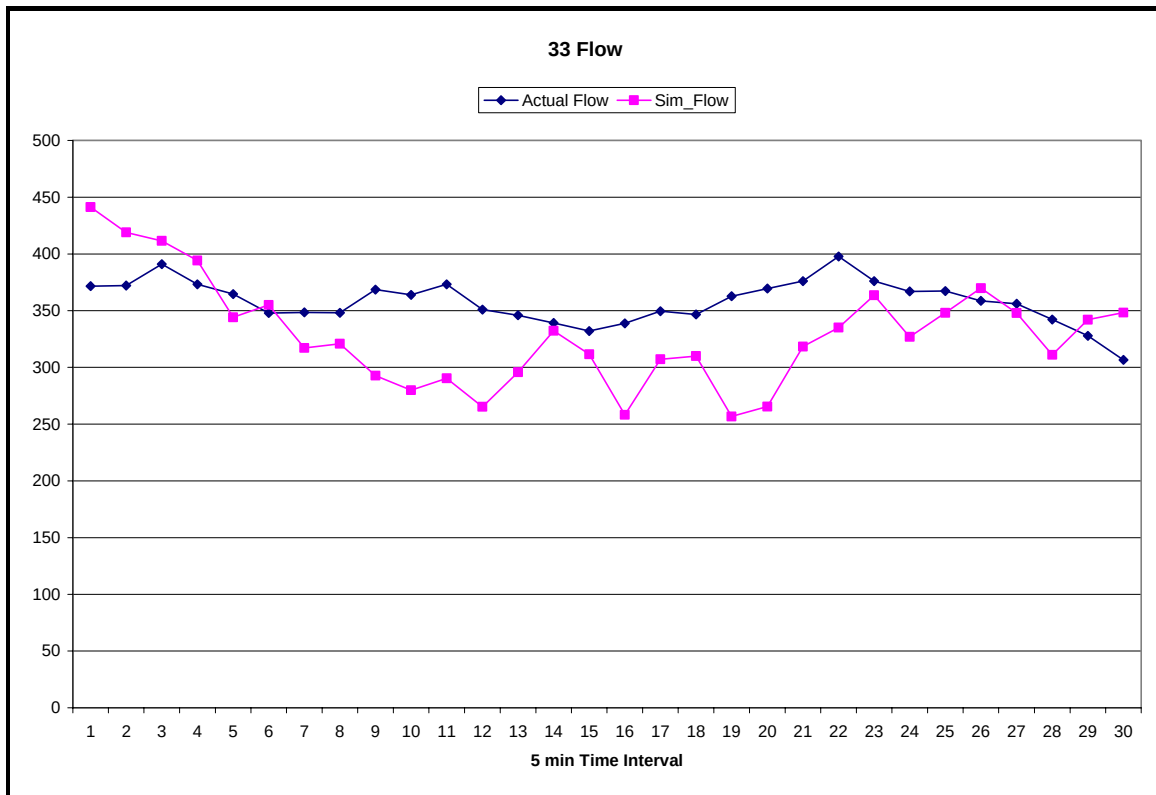


Figure 4-23 Flow and Speed profile for Station 33 at end of Calibration

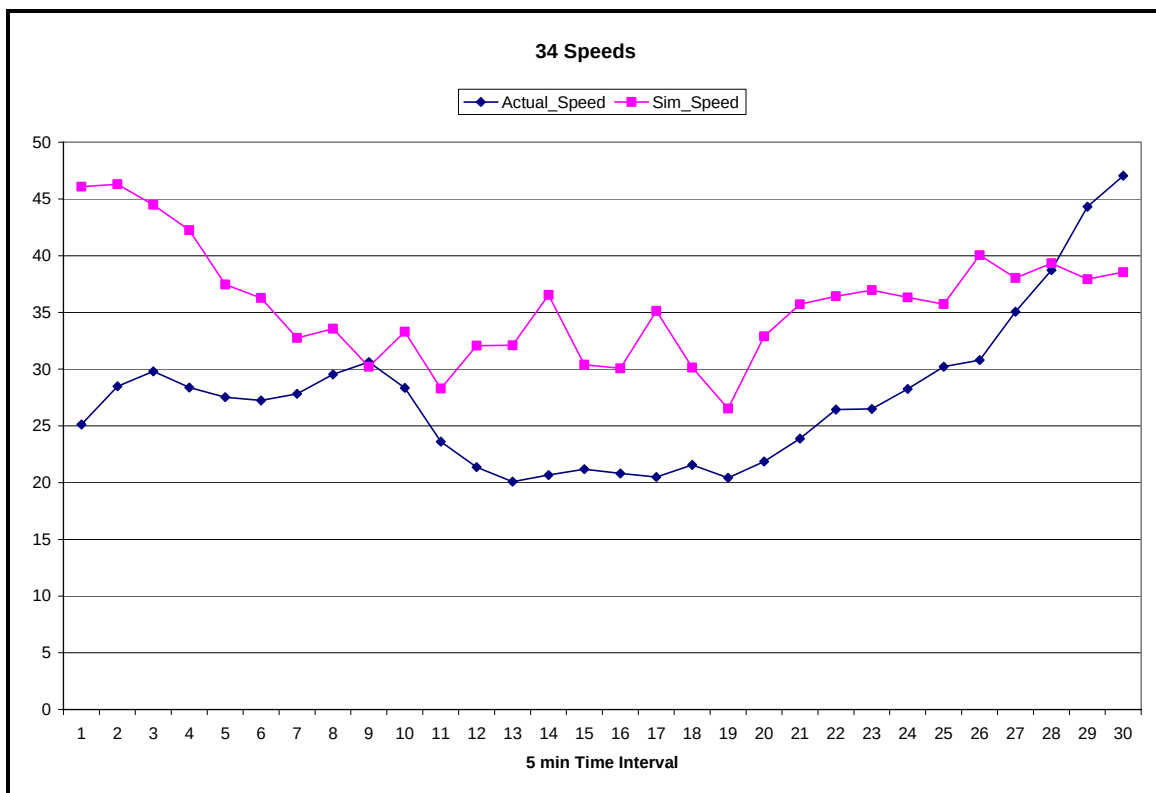
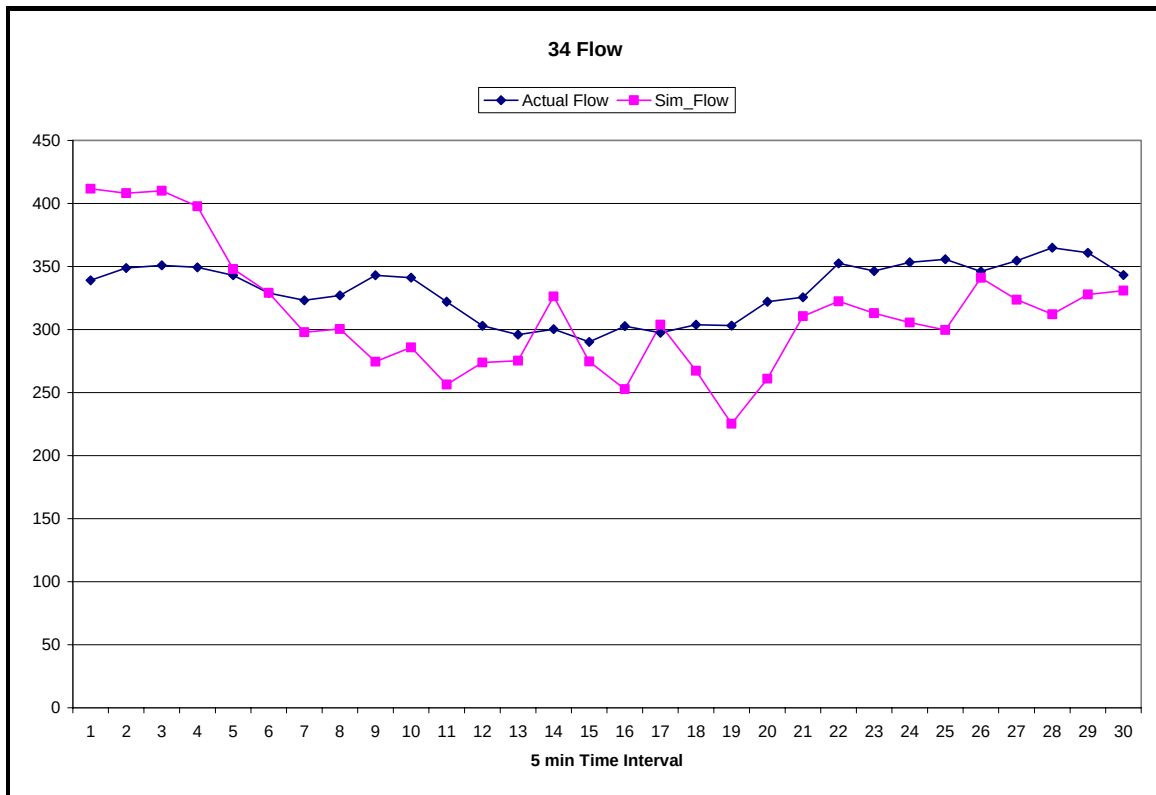


Figure 4-24 Flow and Speed profile for Station 34 at end of Calibration

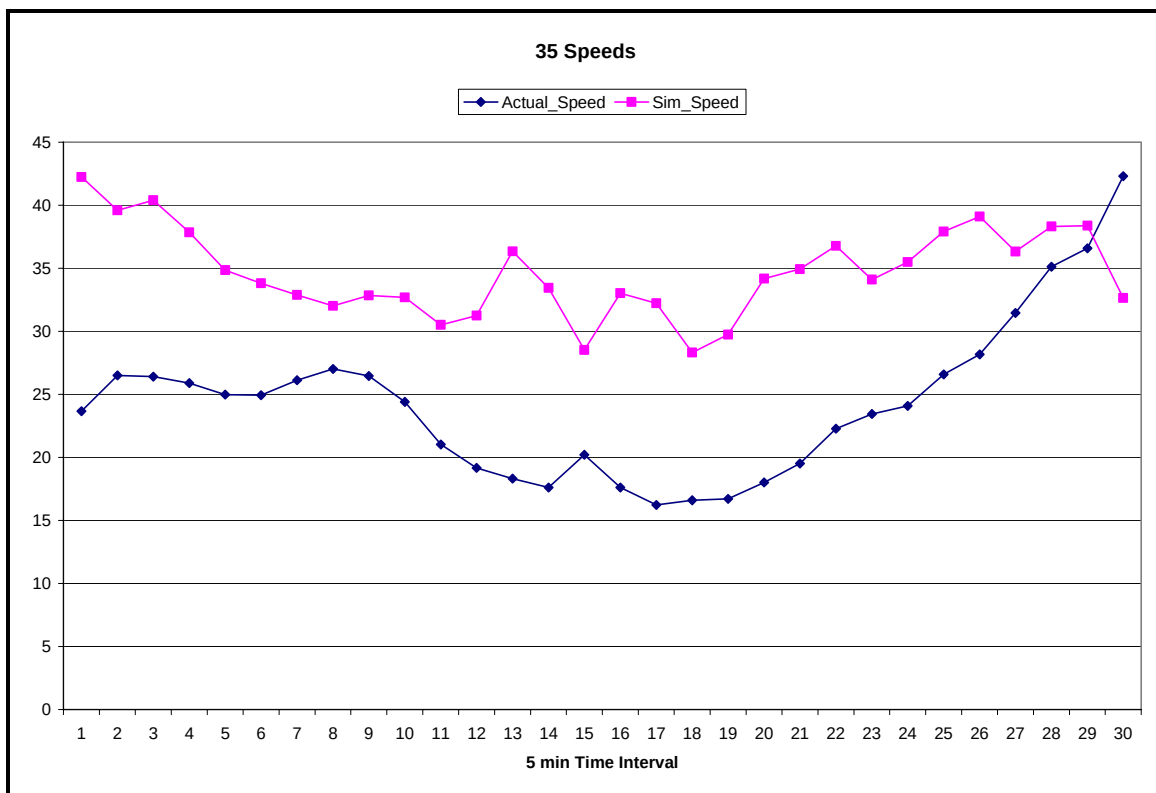
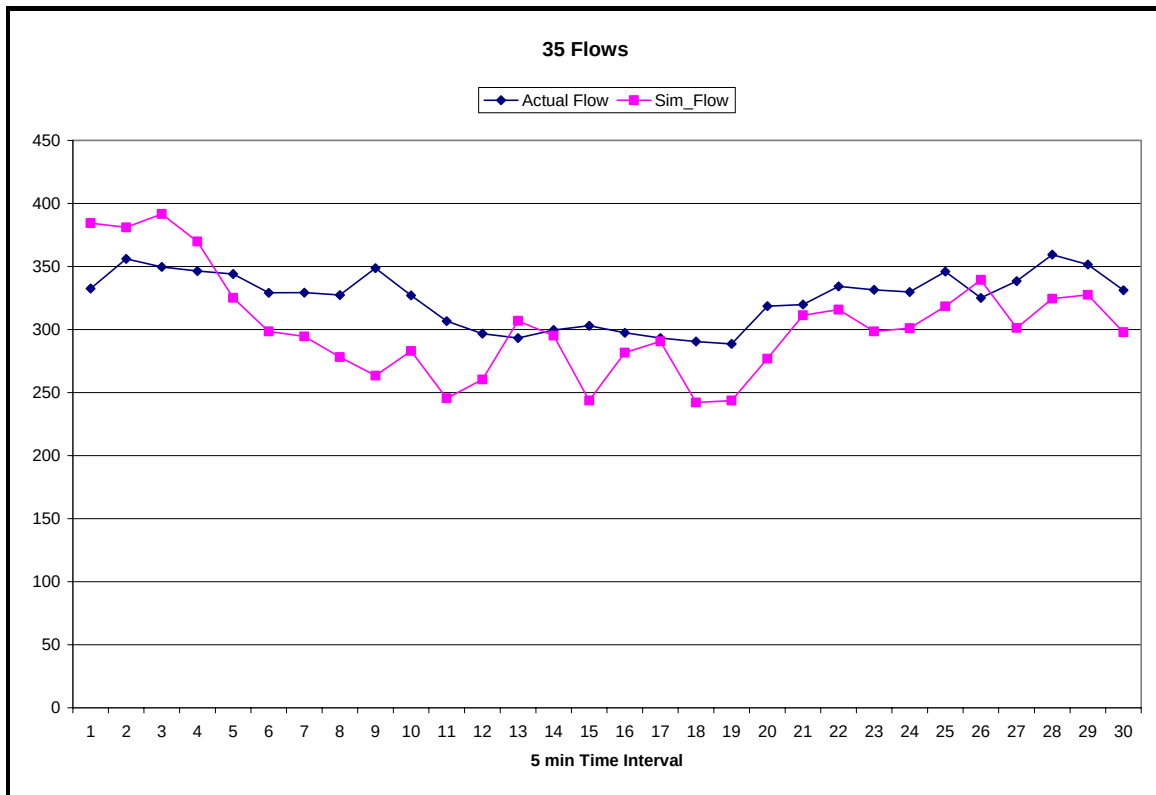


Figure 4-25 Flow and Speed profile for Station 35 at end of Calibration

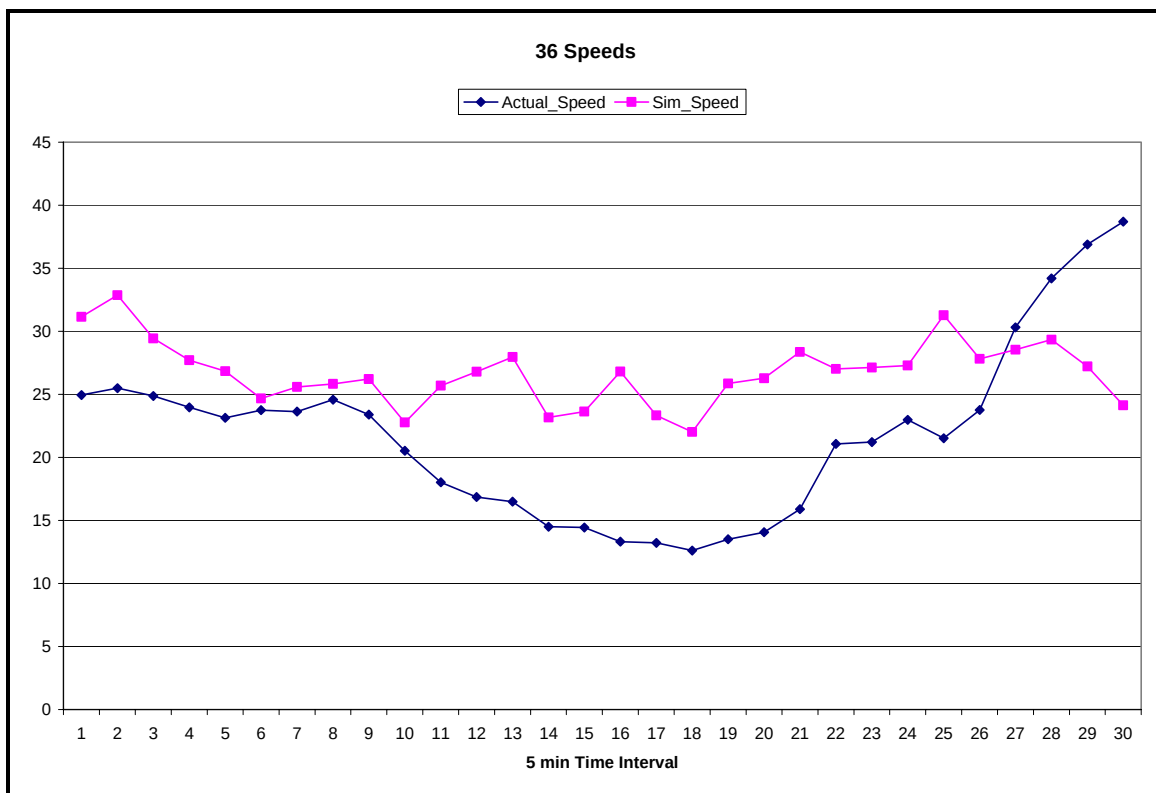
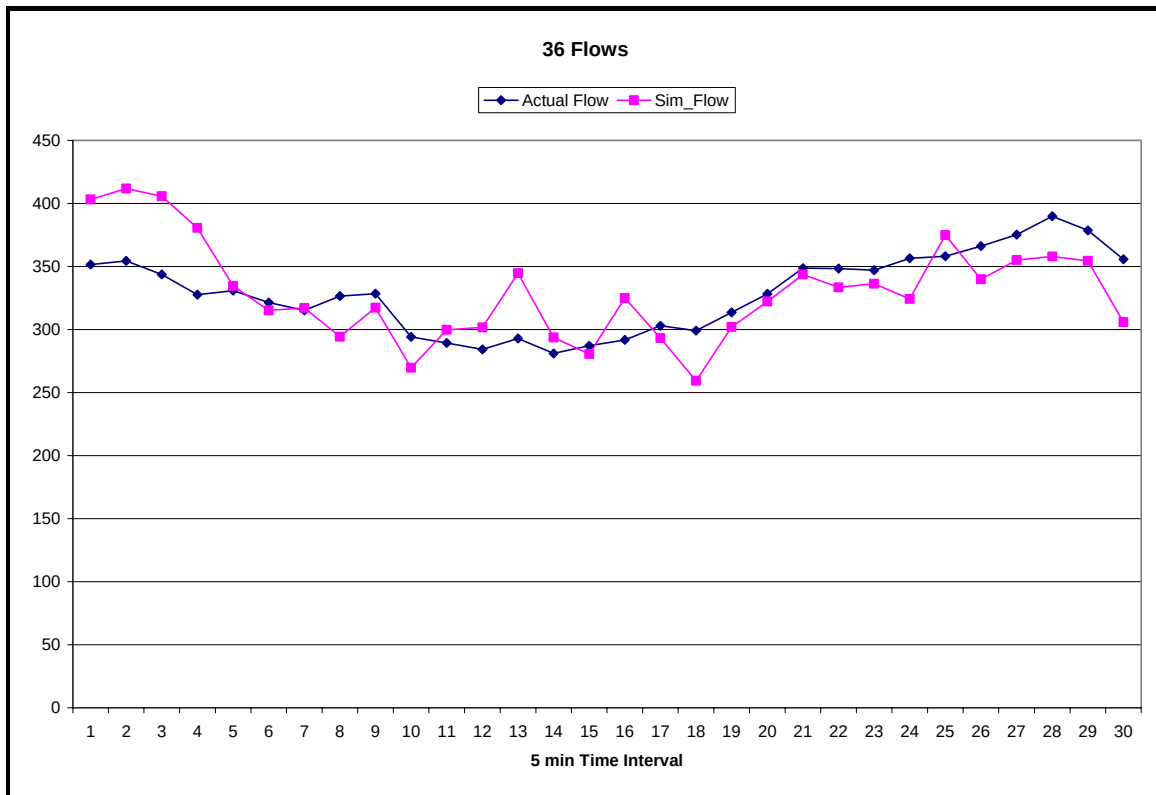


Figure 4-26 Flow and Speed profile for Station 36 at end of Calibration

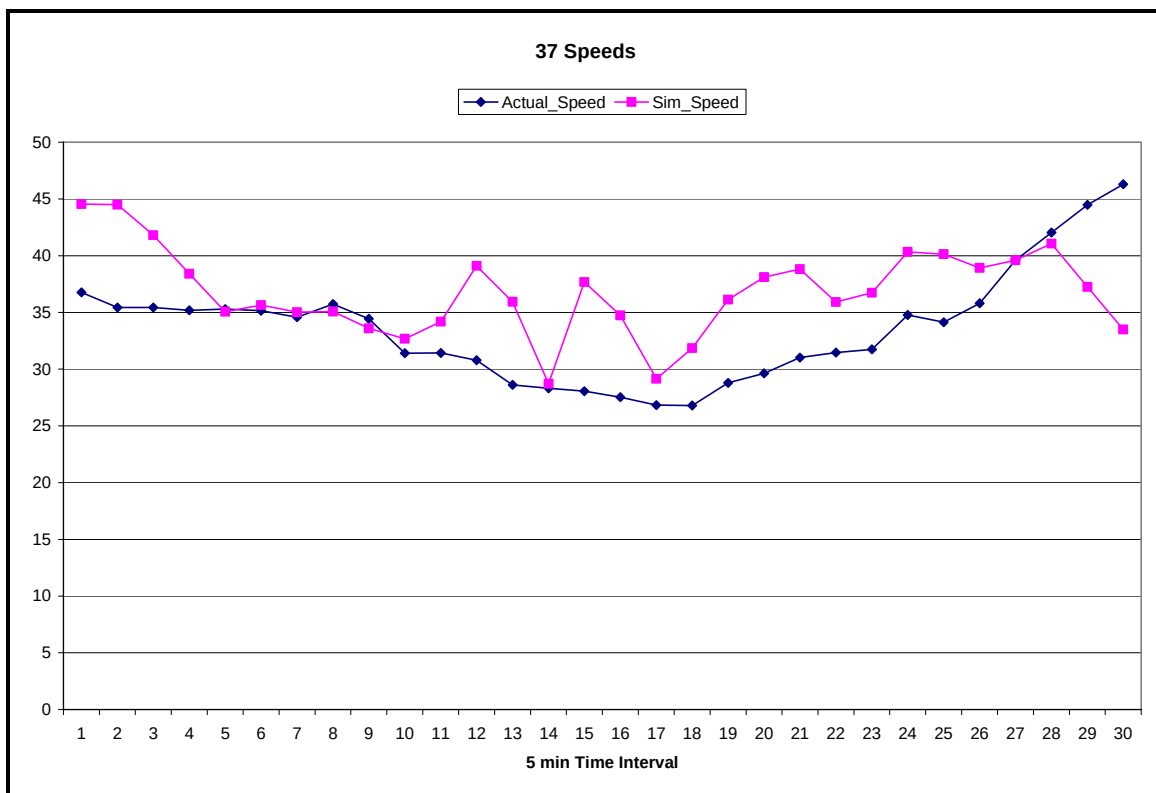
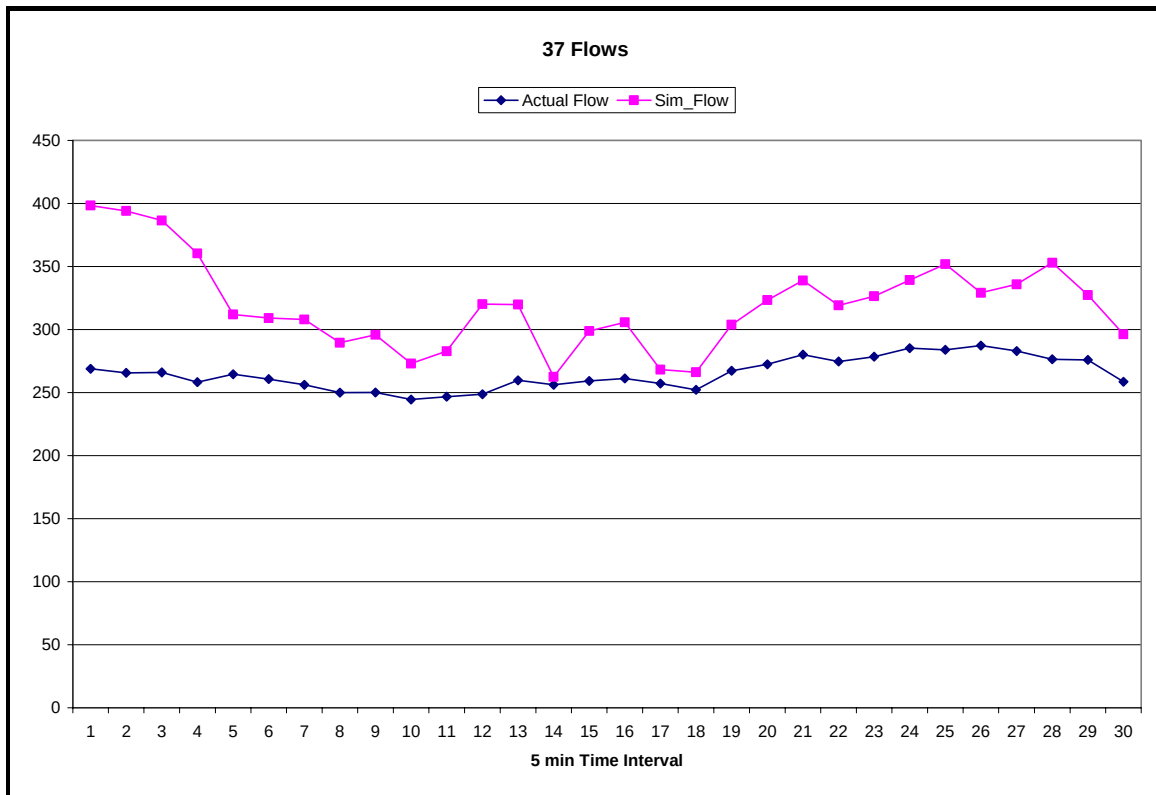


Figure 4-27 Flow and Speed profile for Station 37 at end of Calibration

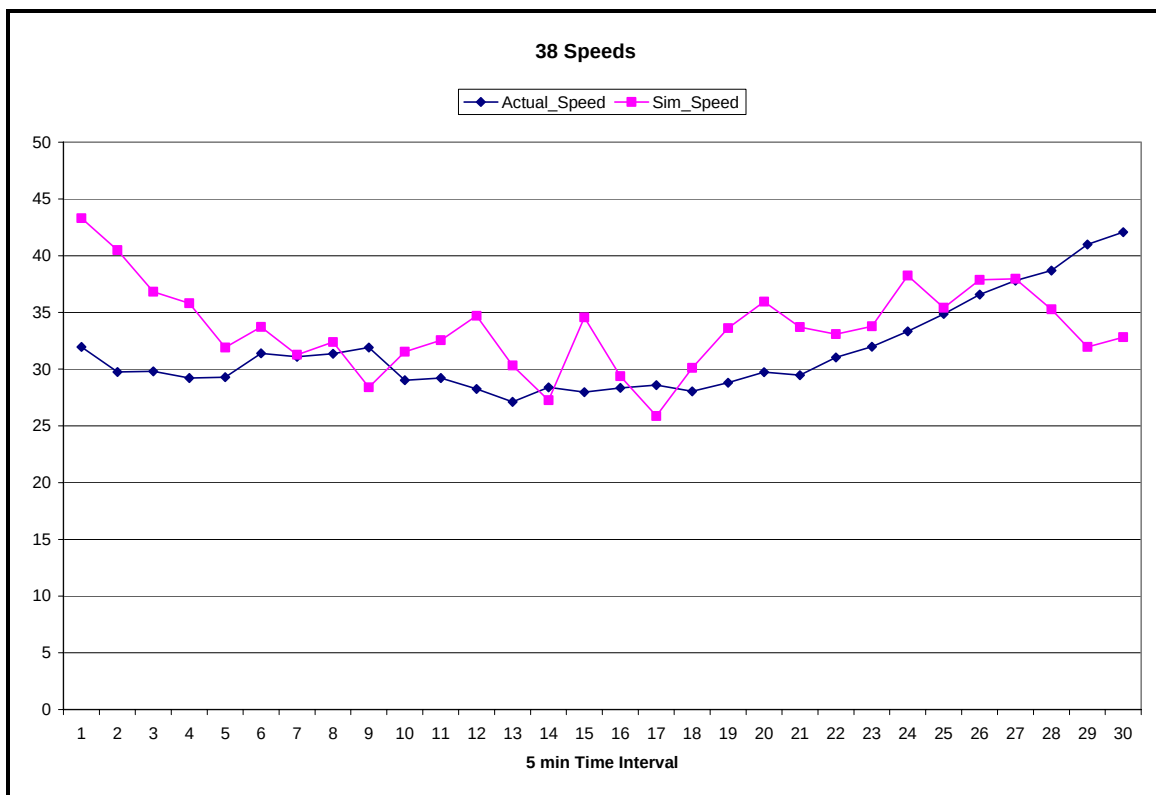
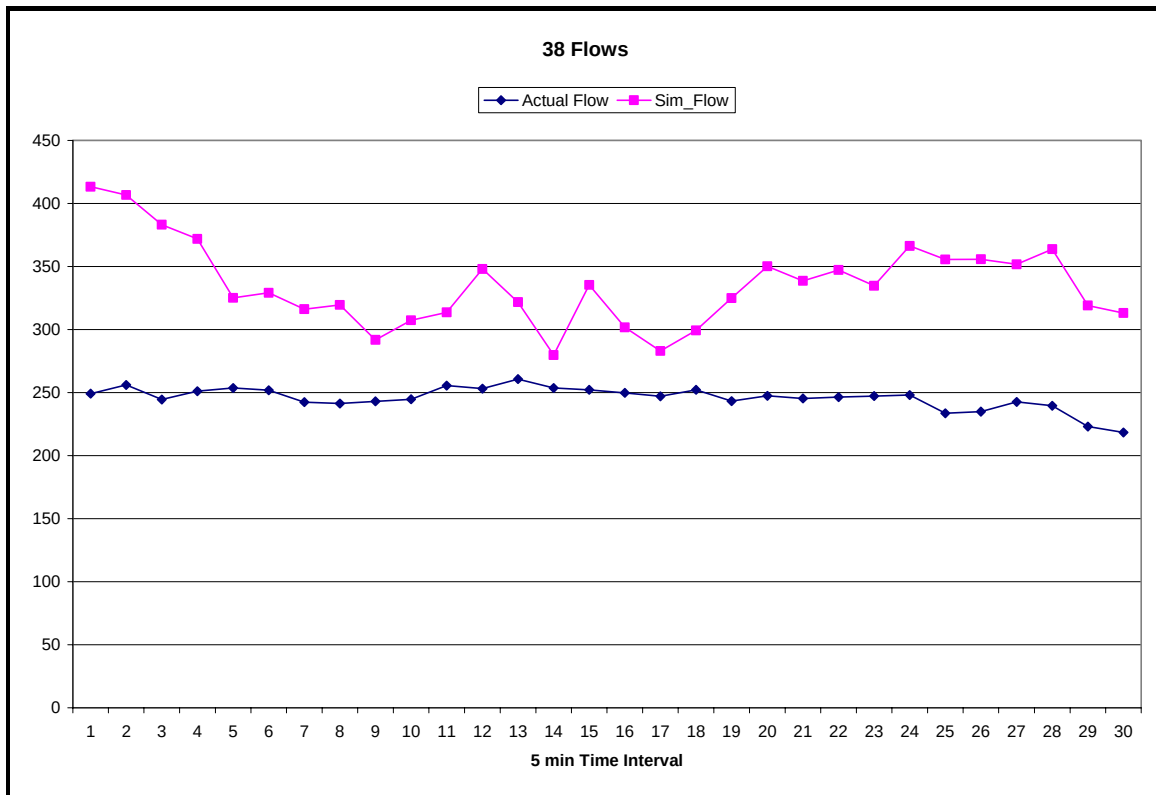


Figure 4-28 Flow and Speed profile for Station 38 at end of Calibration

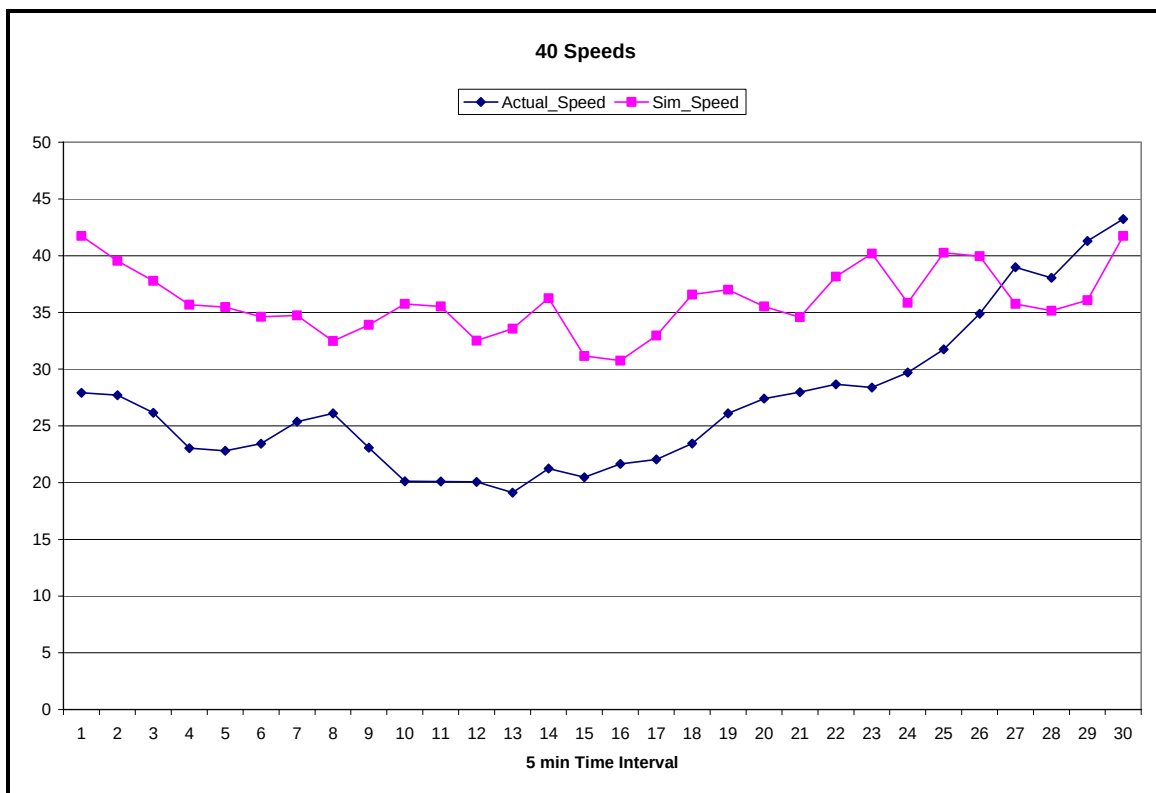
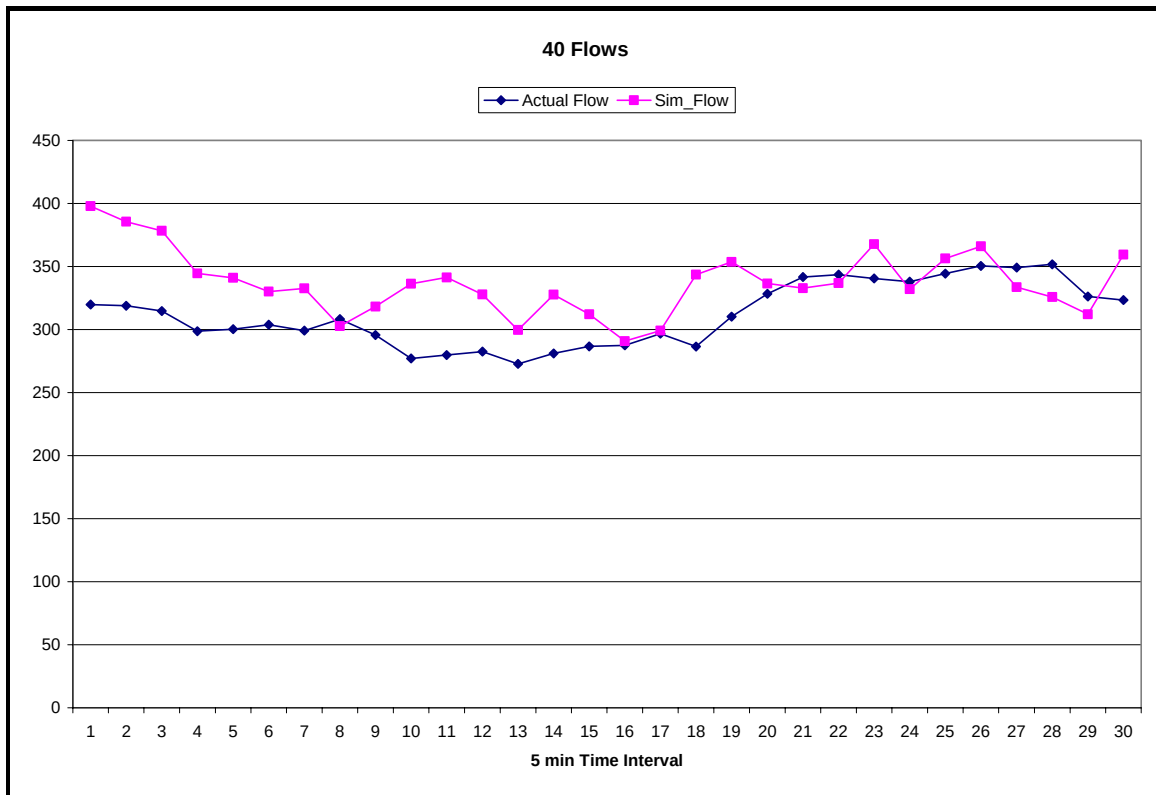


Figure 4-29 Flow and Speed profile for Station 40 at end of Calibration

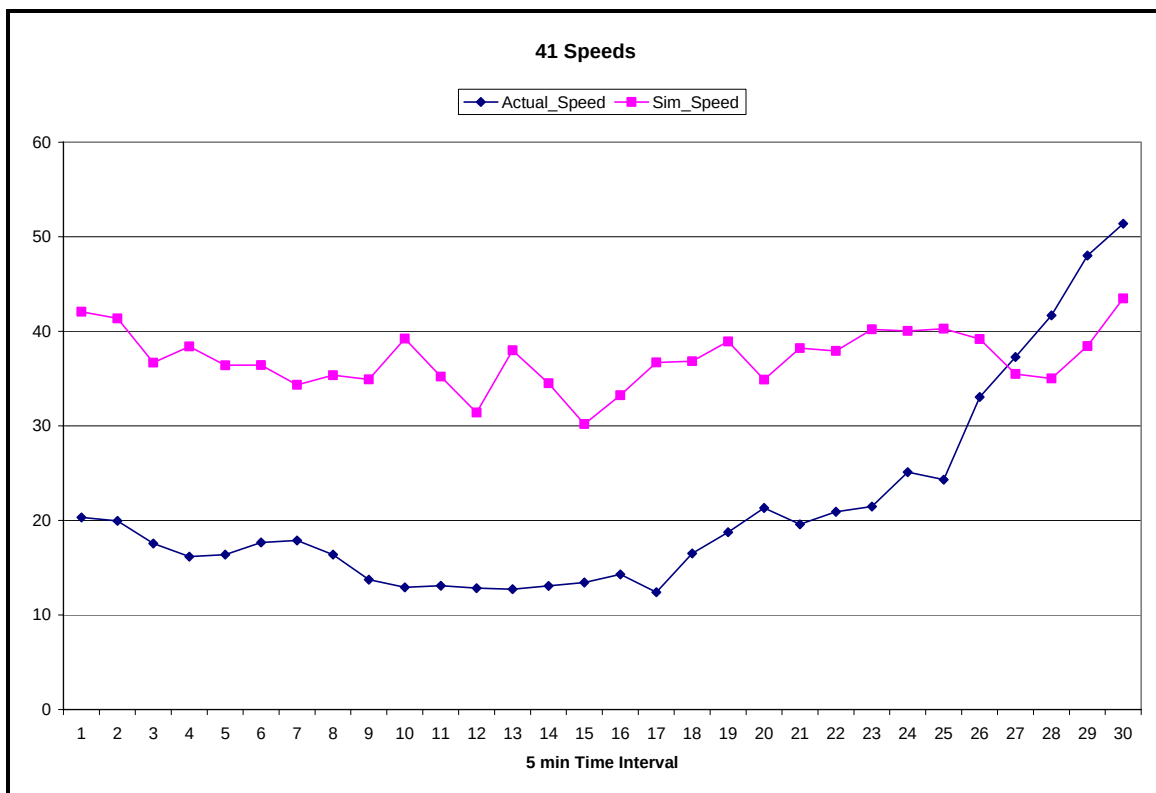
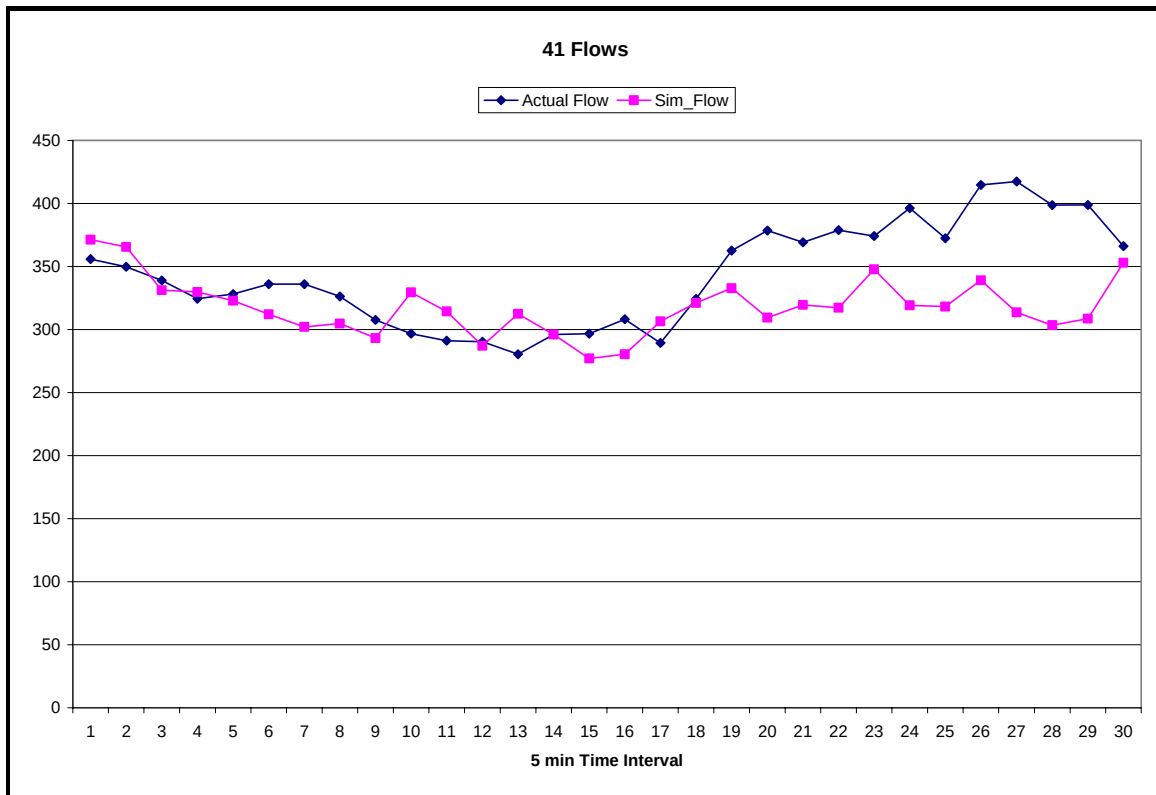


Figure 4-30 Flow and Speed profile for Station 41 at end of Calibration

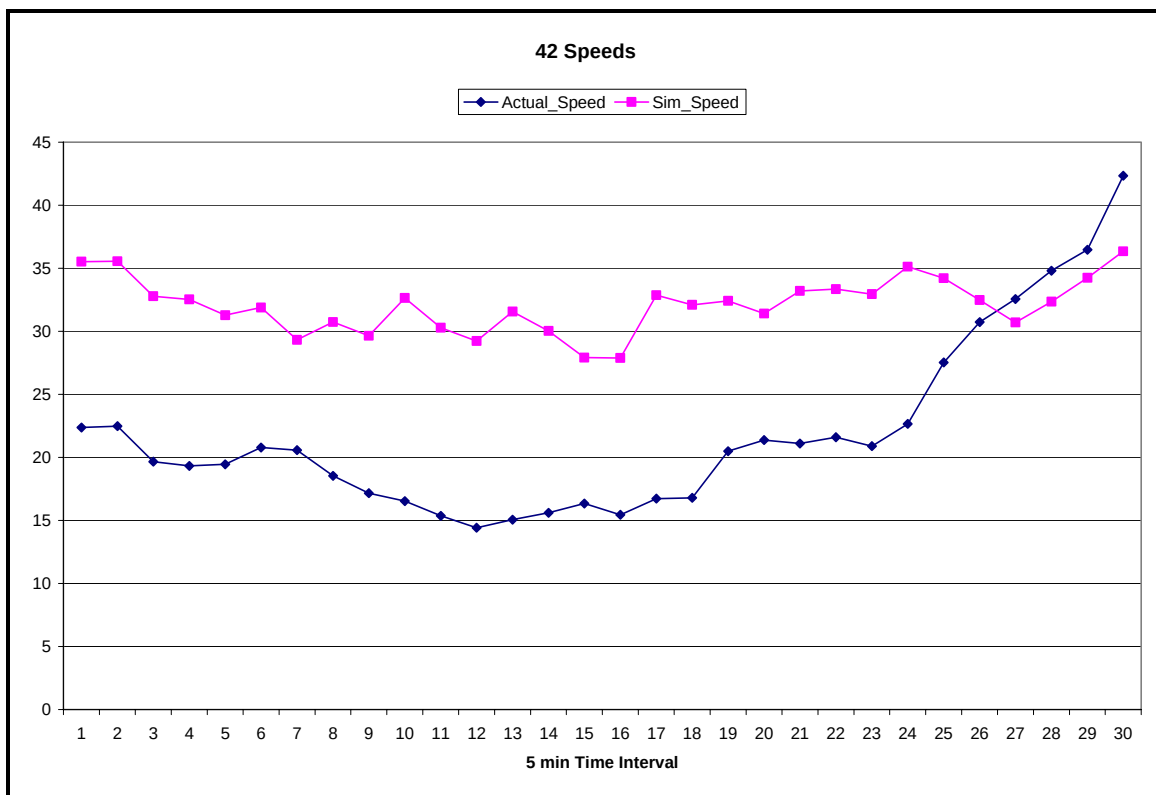
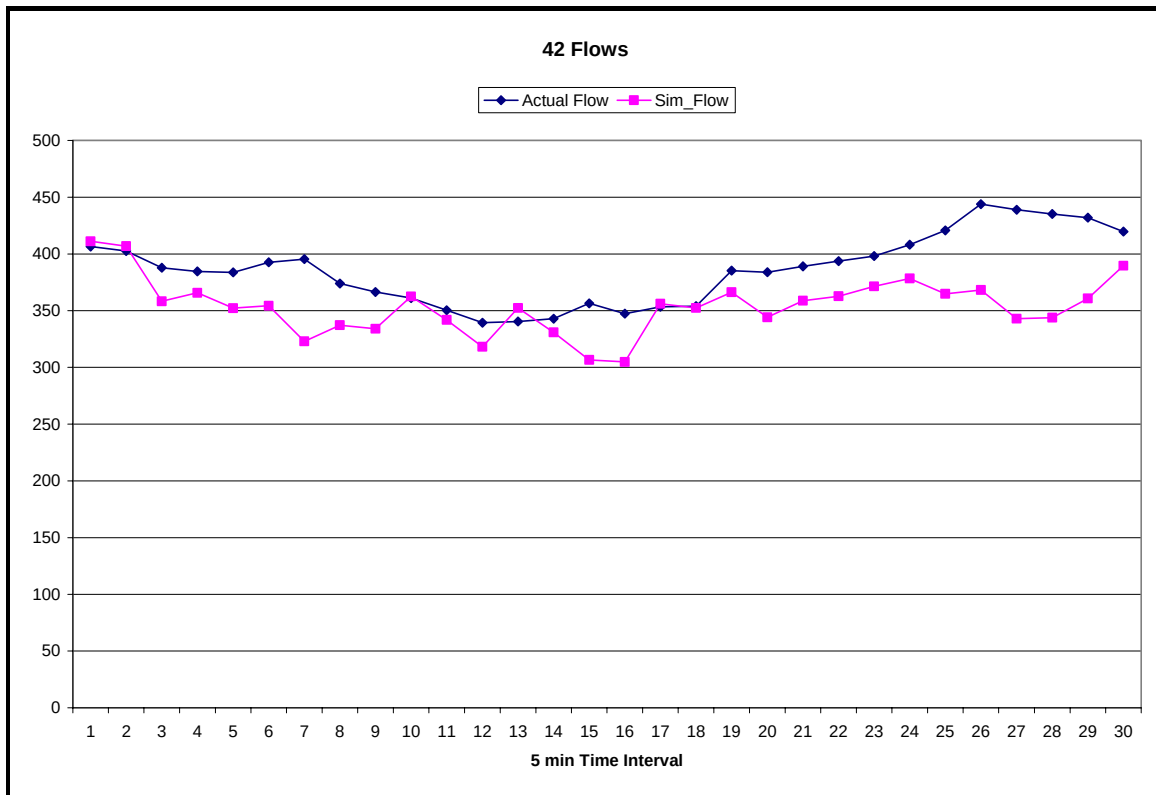


Figure 4-31 Flow and Speed profile for Station 42 at end of Calibration

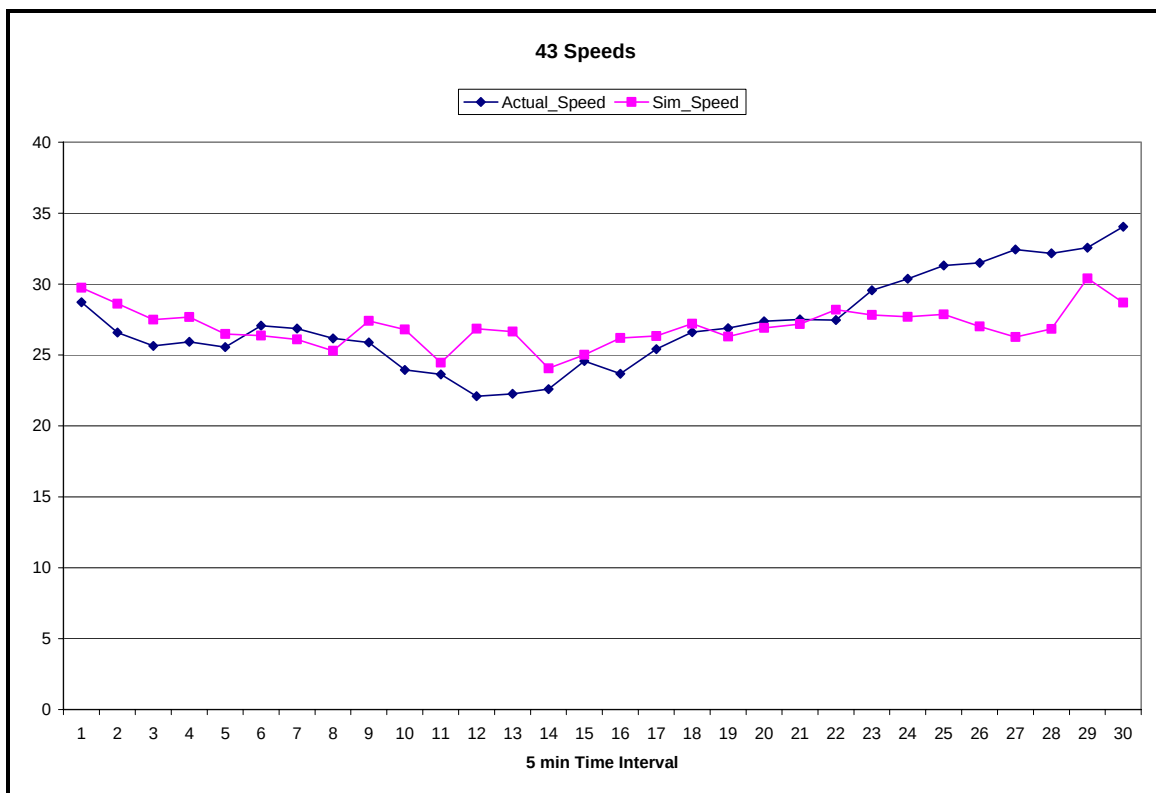
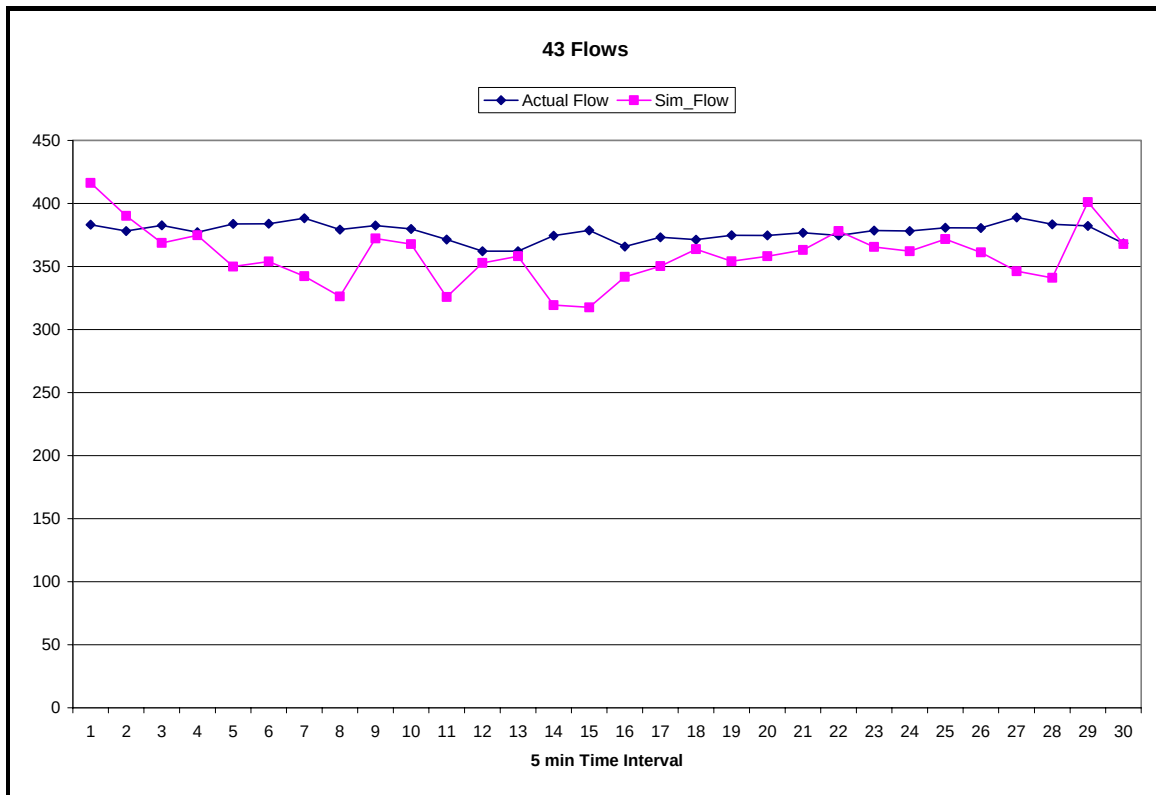


Figure 4-32 Flow and Speed profile for Station 43 at end of Calibration

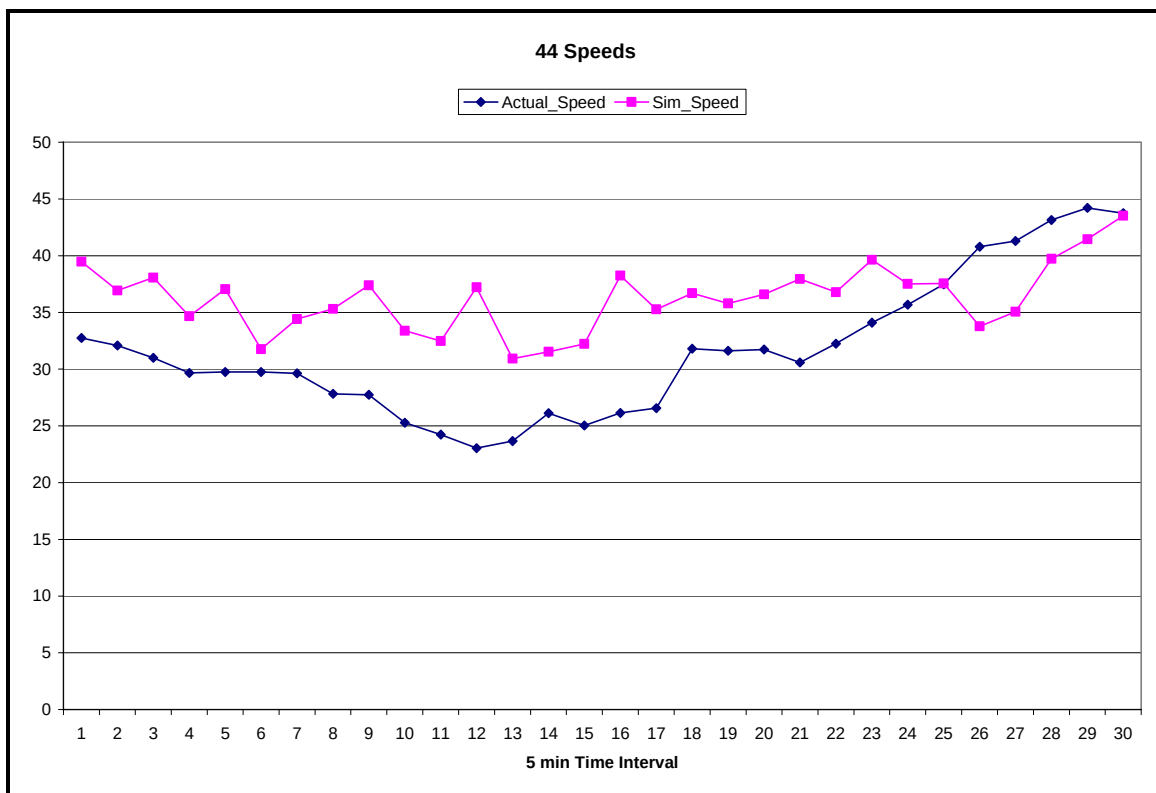
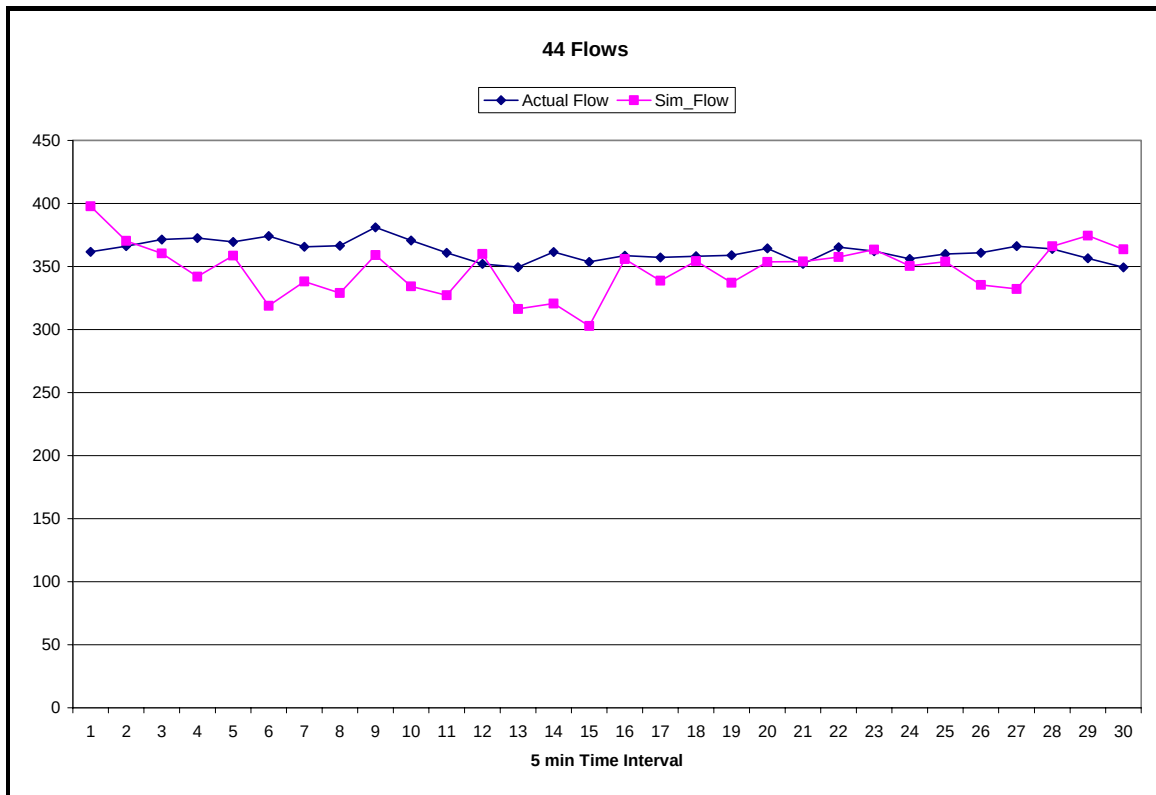


Figure 4-33 Flow and Speed profile for Station 44 at end of Calibration

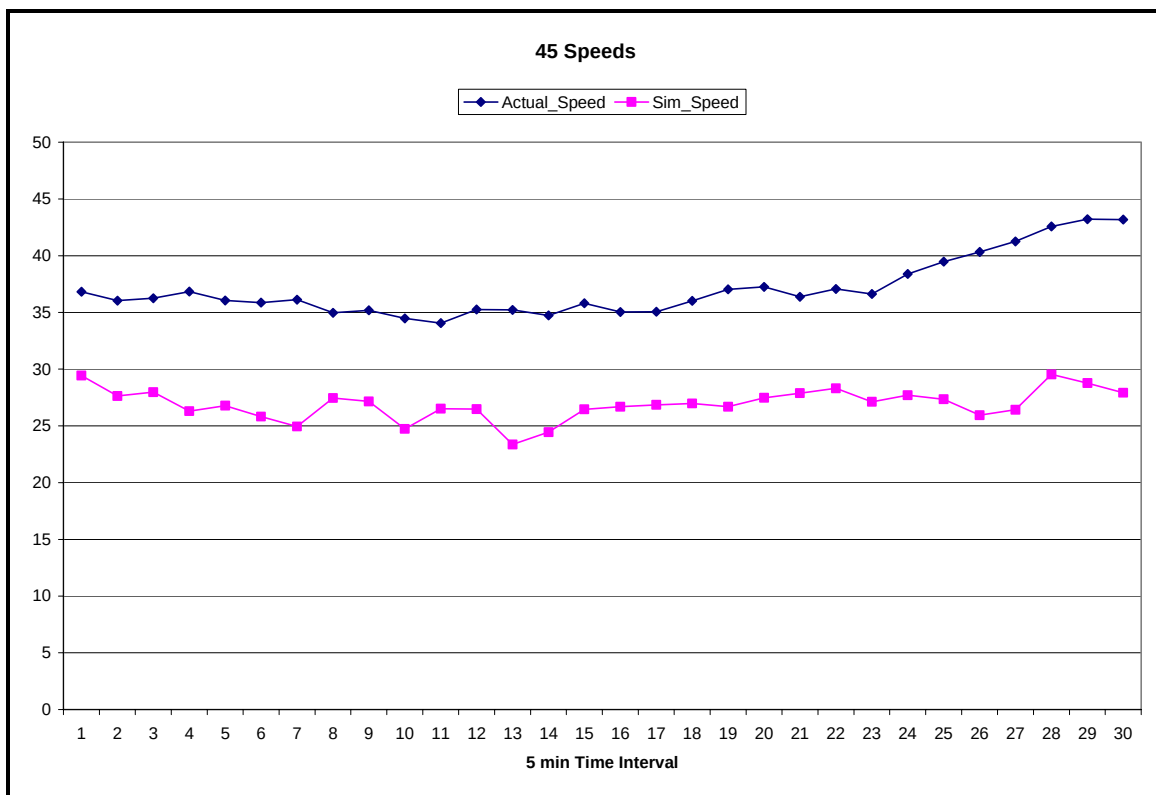
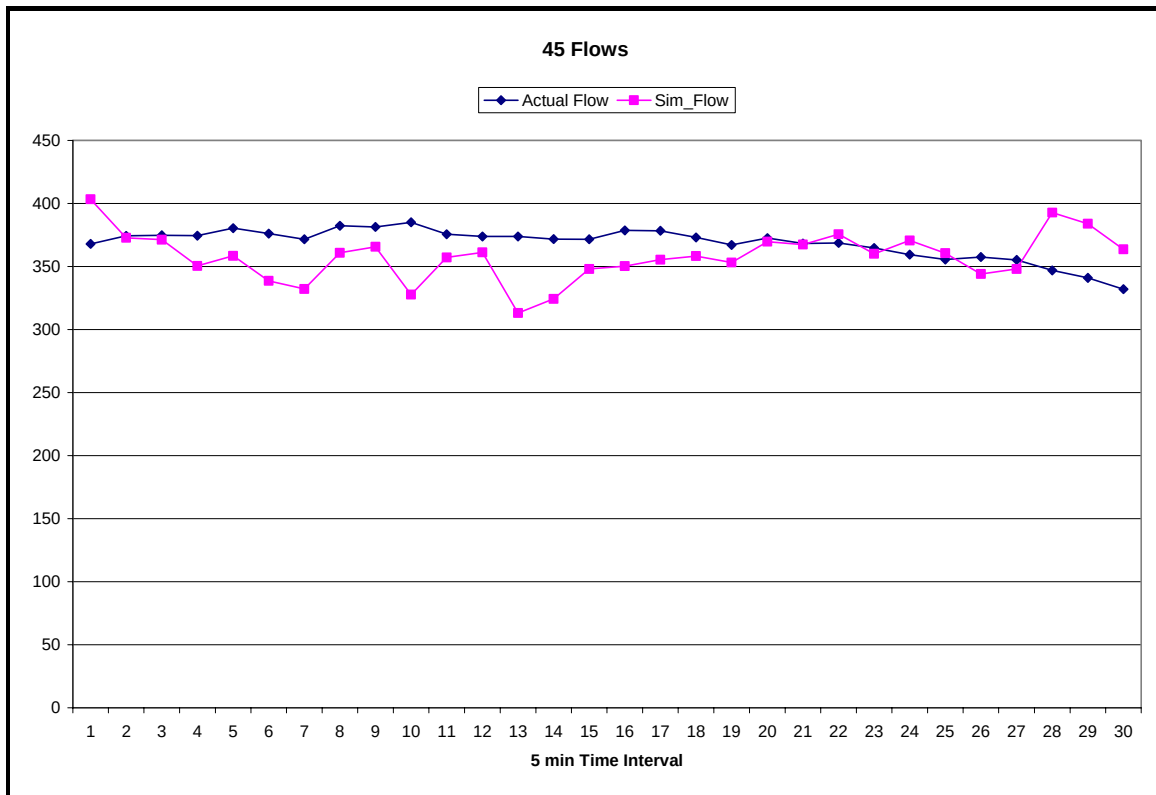


Figure 4-34 Flow and Speed profile for Station 45 at end of Calibration

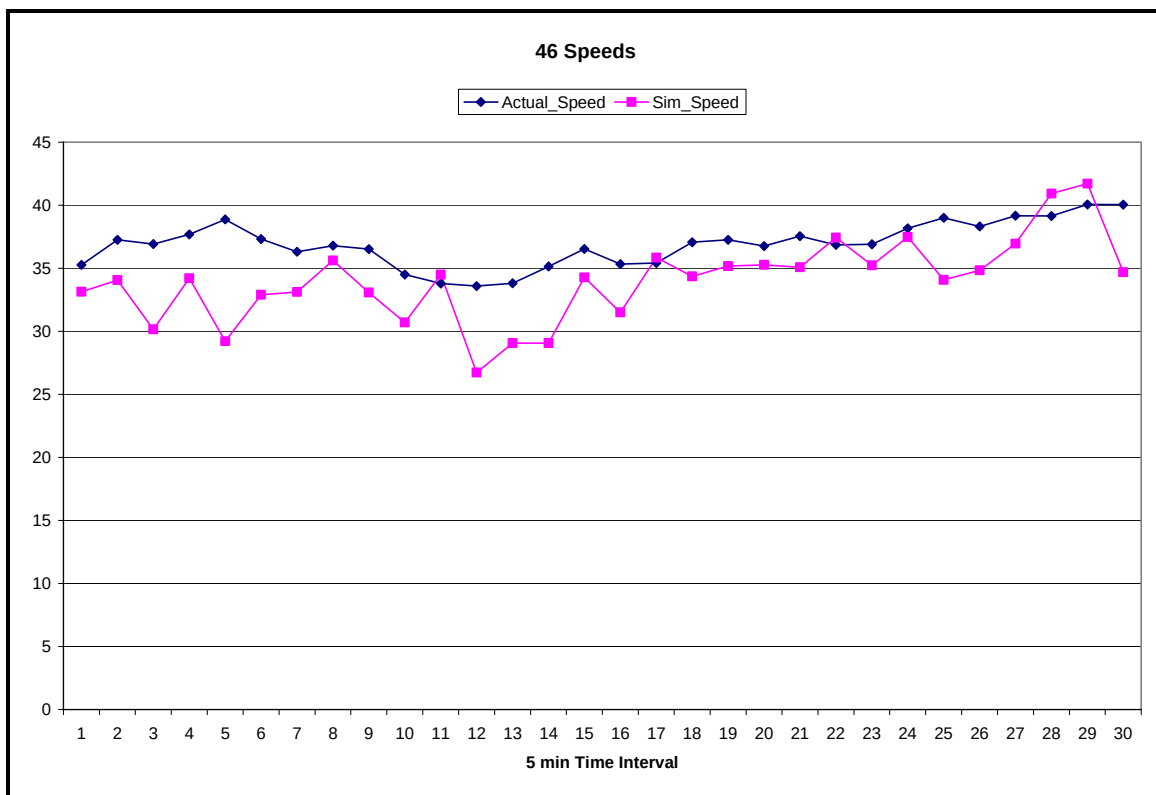
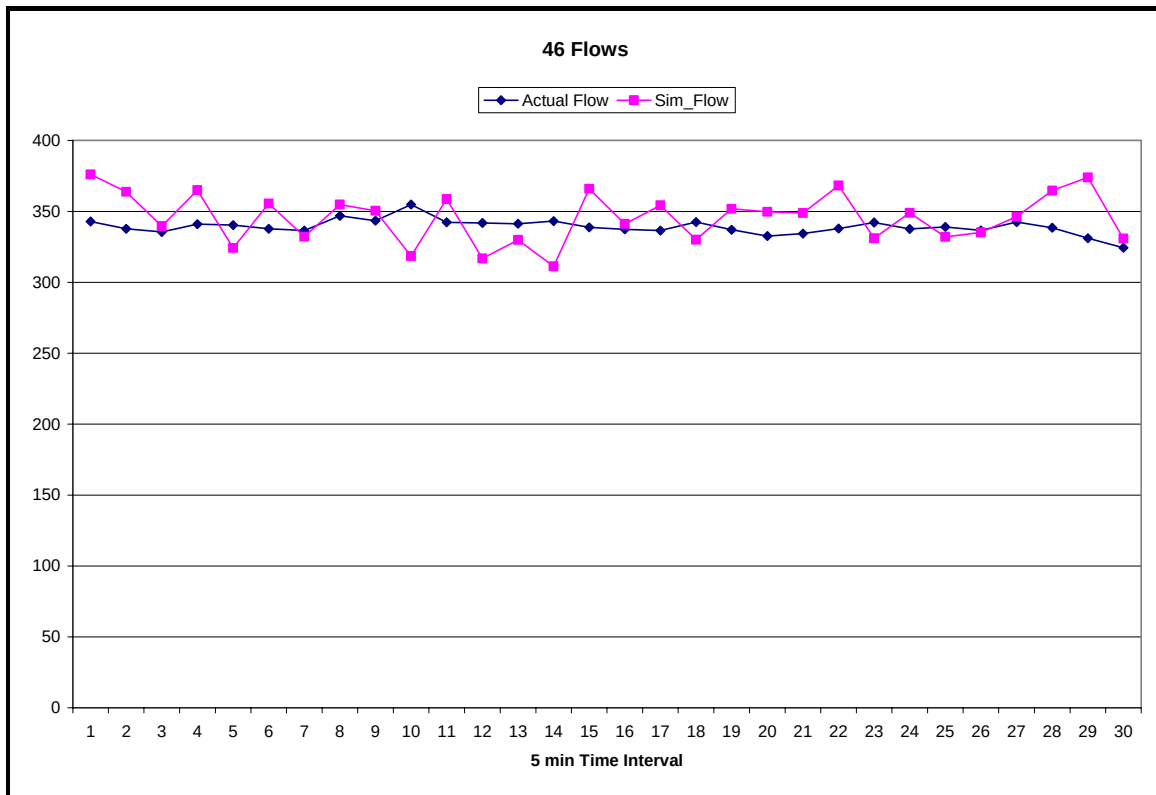


Figure 4-35 Flow and Speed profile for Station 46 at end of Calibration

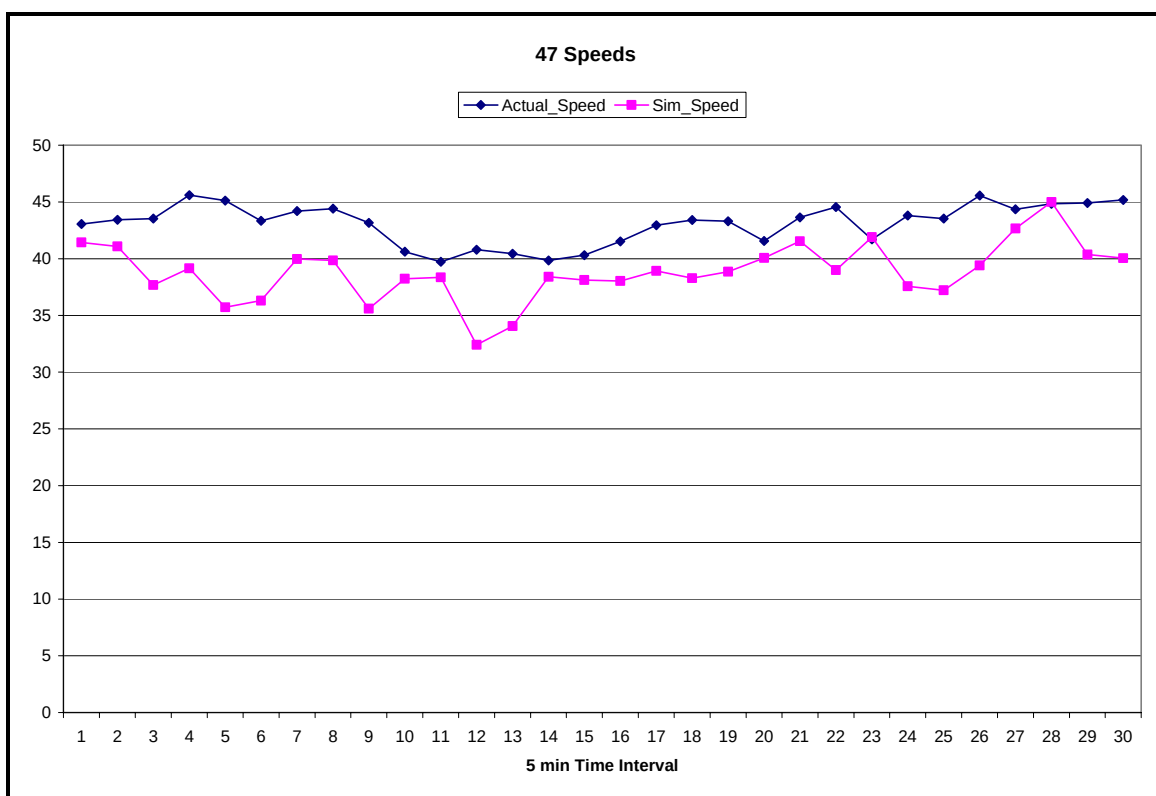
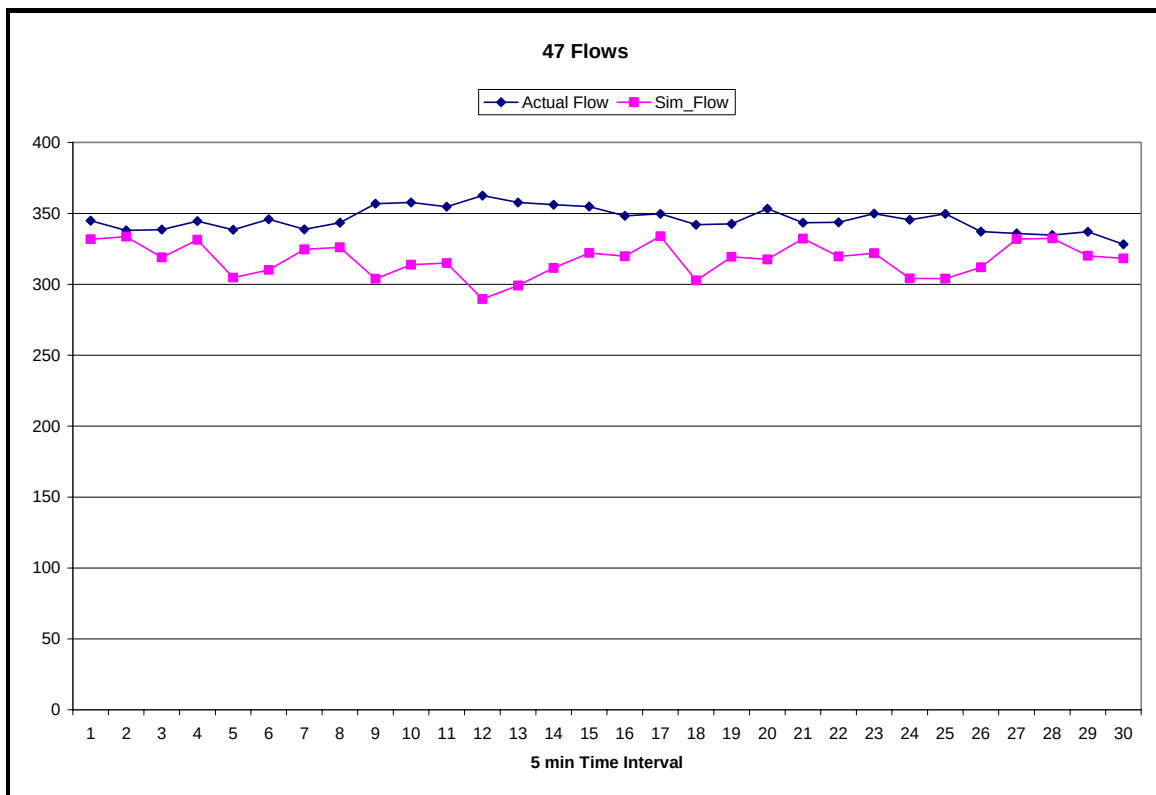


Figure 4-36 Flow and Speed profile for Station 47 at end of Calibration

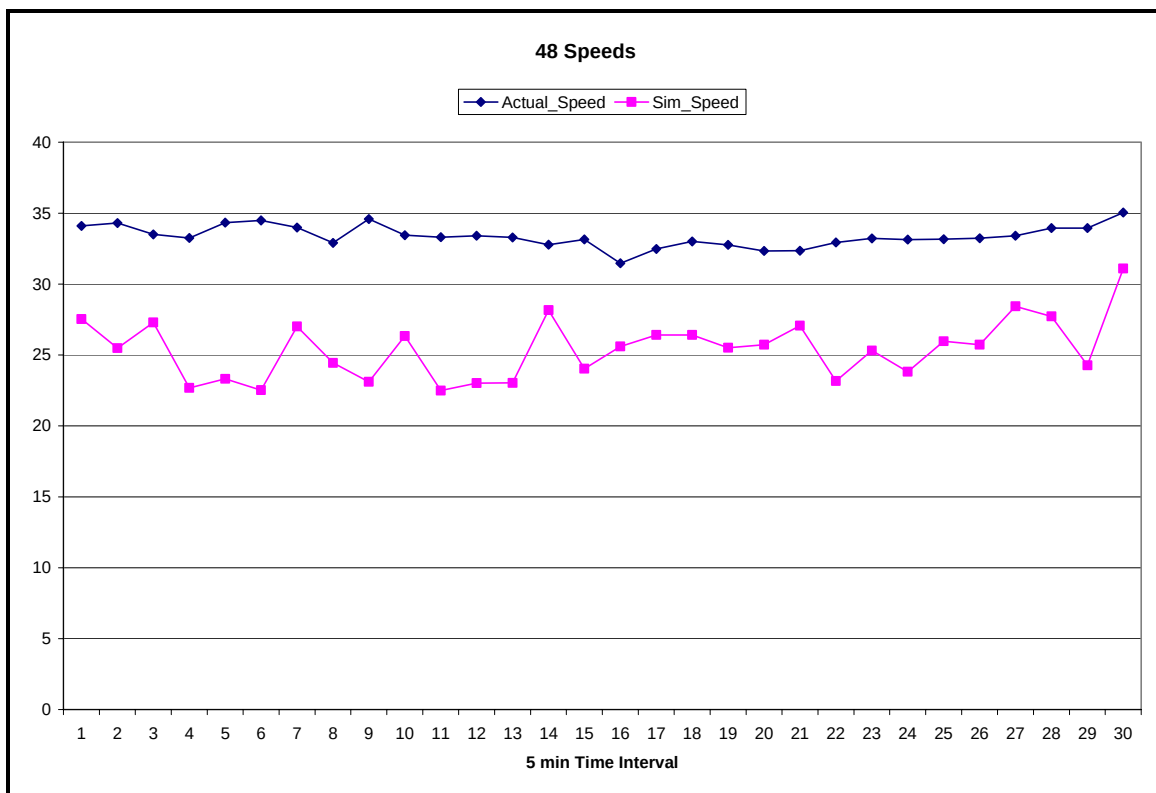
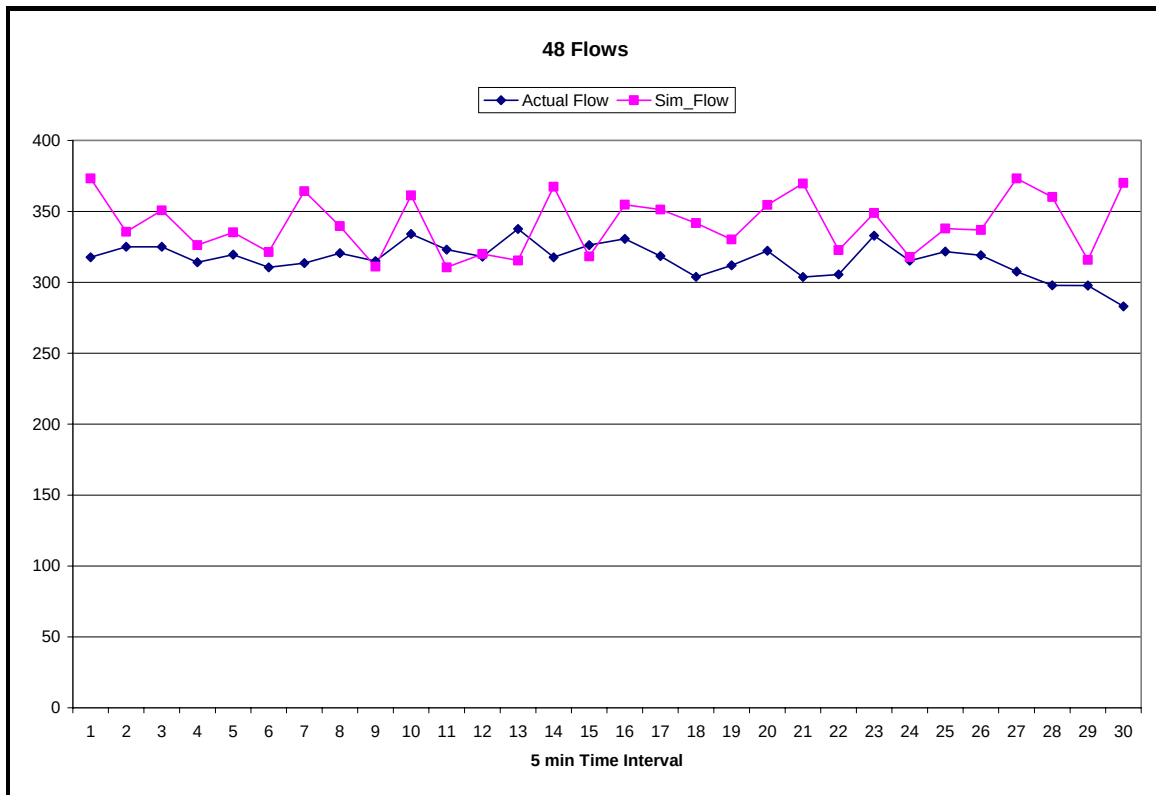


Figure 4-37 Flow and Speed profile for Station 48 at end of Calibration

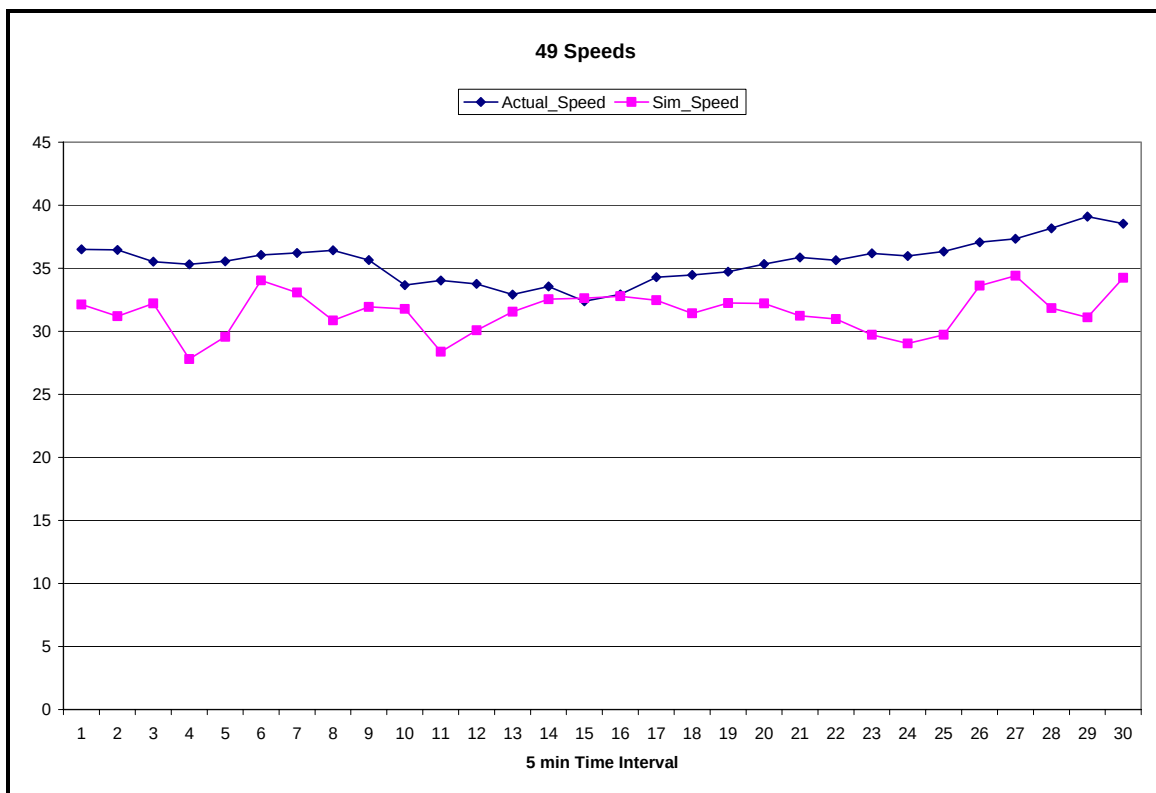
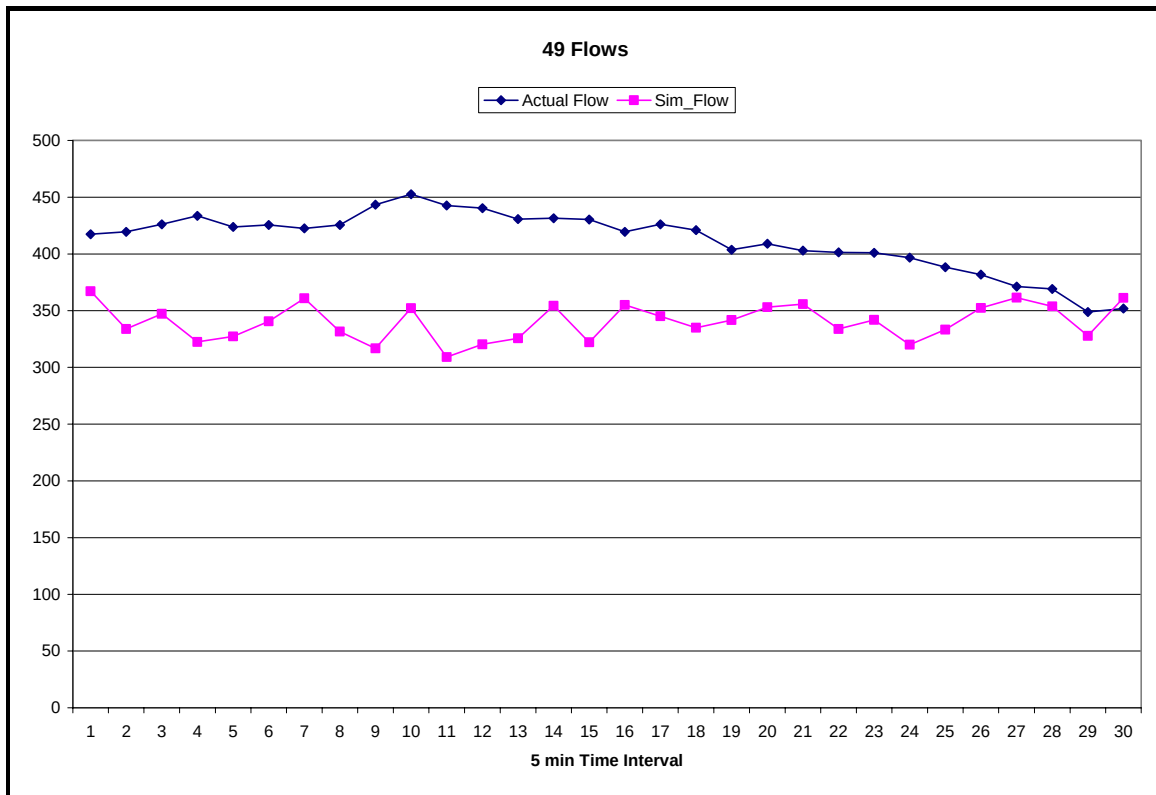


Figure 4-38 Flow and Speed profile for Station 49 at end of Calibration

CHAPTER 5

RESULTS OF PRELIMINARY EVALUATIONS OF RAMP METERING COMBINED WITH VSL

5.1 Ramp Metering

Section 4.6 in the previous chapter provided the methodology for the implementation of the different possible scenarios with Ramp Metering. Before the scenarios for the implementation of different applications like Ramp Metering, VSL, etc. can be discussed; there is a need to provide the quantitative estimates of our measures for defining our benefits and scenarios. For this purpose three measures were defined in Sections 4.2 (Risk Index), 4.3 (Measure of Effectiveness), and 4.5 (Quantification of Safety Benefits). These measures were named, Risk Index, Mean Cumulative Index (MCI) and Safety Benefits, respectively. These numbers, derived for the base case as well as every test case would then be used when we are comparing the different alternative scenarios with each other and the base case (Base Case here as well as in the rest of the document refers to the simulation of the roadway network without any ITS applications).

5.1.1 Crash risk assessment model

Abdel-Aty et al. (2005) presented two different models that used traffic measurements to predict crashes. The usage of each of these models is dependent on the traffic regime (i.e. moderate to high speeds or low speeds). Their models output a factor called crash potential which is the log of the odds of a crash to non crash. The magnitude of this index when high indicates that a crash is likely to occur and when low means that a crash is unlikely to occur. The low-speed model is based on average speeds being below 37.5 mph and the high-speed model is based on average speeds being above 37.5 mph. The next Equations 5-1 and 5-2 present the indices for each of these models.

Low Speed Model

$$\text{Risk\_Index} = 2.64827 \text{ LogCVSF2} + 0.88842 \text{ LogCVSF3} + 1.33966 \text{ LogAOE2} + 0.97766 \text{ LogAOH3} - 0.43603 \text{ SVF2}$$

...Equation 5-1

Moderate to High Speed Model

$$\text{Risk\_Index} = -0.93423 \text{ LogAOF2} + 1.14584 \text{ LogAOH3} - 0.22878 \text{ SVH2} - 0.10055 \text{ AVG2} + 0.5932 \text{ AVE3}$$

...Equation 5-2

The terms in the notation follow a specific notation that is followed through out this study. The generic description of the variables is as follows:

AO: Average occupancy

SV: Standard Deviation of Volume

AV: Average Volume

CVS: Coefficient of Variation of Speed (Standard Deviation of Speed Divided by the Average Speed over five minutes)

The last letter represents the station with respect to the station of crash. The nomenclature was defined in Figure 4-1.

The number at the end represents the number of the 5 minute time slice, namely

2 → 5 - 10 minute interval before the time of interest

3 → 10 - 15 minute interval before the time of interest

Consider a Station that usually operates at very low speeds, e.g. Station 43. The output of speeds from PARAMICS is used to evaluate the value of this Risk Index at every thirty seconds. This gives 360 Data points for a 3 hour simulation (excluding 15 minute warm-up period), and these points are measures of this Risk Index at every thirty seconds. A graphical representation of these measures for 20 different runs with different seeds in PARAMICS would look like Figure 5-1. The figure depicts the risk index for 20 different cases without any ITS applications (i.e. Base Case). The sharp drop at the beginning marks the end of the measures extracted for the initialization period of the simulation.

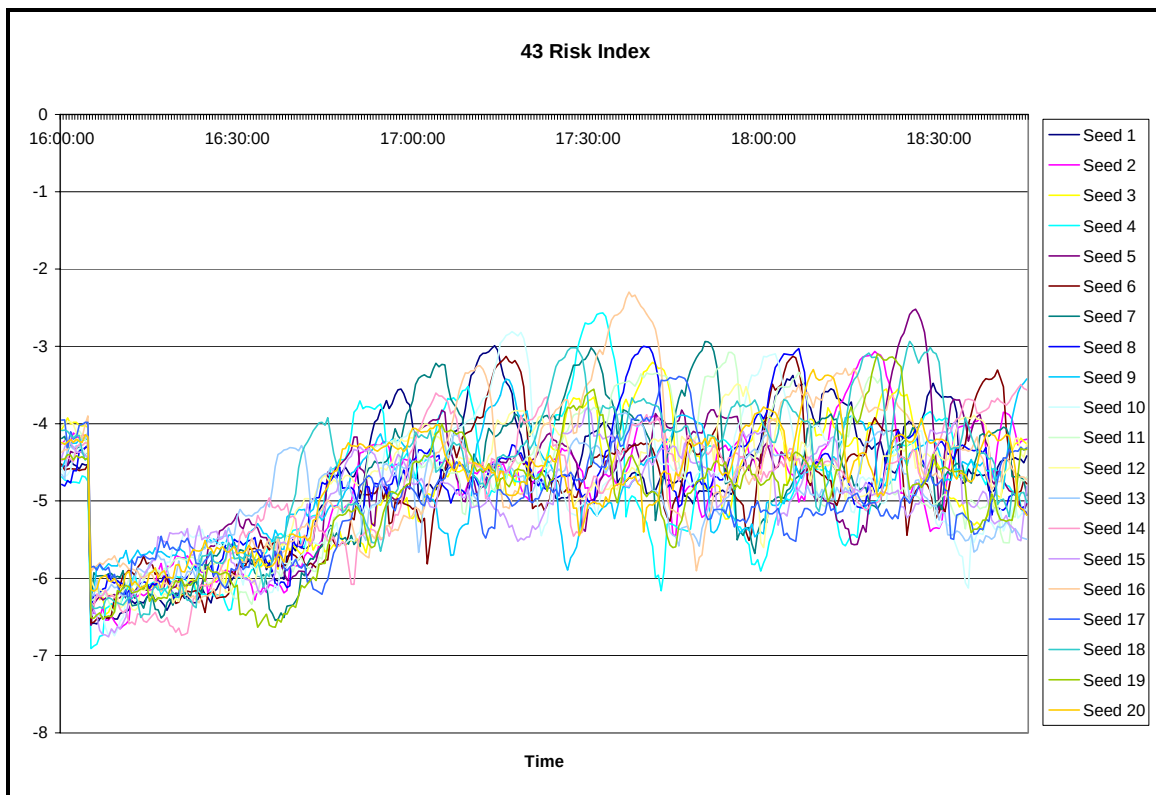


Figure 5-1 Risk Evolution at Station 43 for 20 different seed values

The best way to ascertain the changes in crash potential is to look at the propagation with matched seeds on a case by case basis. However quantification is

necessary in order to compare cases even when matching graphs isn't possible for multiple seeds. To achieve this, the Index that we defined as Measure of Effectiveness in Section 4.3 is used. Equation 5-3 shows the method of calculation of this index. It is the sum of the risk indices at every 30 second.

$$(MOE)_i = 1/n \left(\sum_{j=1}^n \sum_{t=1}^{360} (RiskIndex) \right)$$

...Equation 5-3

Where i= Station Number (33-49 for East Bound Direction)

j=Seed Number (20)

t= time slice (360 for 3 hours evaluated every 30 seconds)

For the twenty cases in Base Case shown in Figure 5-1, the MOE (henceforth called Mean Cumulative Index (MCI)) has a value equal to -1591.3. It should be made clear at this stage that to achieve a reduction in the index the MCI for a test case at this station should be less than the MCI mentioned above. The safety benefits and the index for comparison of benefits are calculated by the procedure used in Section 4.5. The safety benefits from any case are measured by the area between the risk index curves for the base case and the test case. When summed over the entire network and divided by the area under the curve for base case, an index is obtained which illustrates the safety benefit for that particular test case over the base case. This calculation of the Safety Benefit Index (SBI) is illustrated below.

Safety Benefits= $\frac{|(\text{Area under Test Case} - \text{Area under Base Case})|}{|\text{Area under Base Case}|} \times t$

Where,

t is the time interval of the risk index under consideration.

These 3 indices (Risk Index, MCI, SBI) when used together for comparison would provide a well rounded up comparison for determining the risk of crash in base case and the test case over the whole network.

5.1.2 Ramp metering strategies

5.1.2.1 Manual Metering

Manual Metering was used as an experimental idea to test the initial effects of metering on the basic freeway safety parameters like variance of speeds and average occupancy. The idea behind implementing manual metering is use visual information in order to manipulate the rate of release of vehicles onto the ramps. Although this is not a practically implement-able idea, it does provide insights into the effects of metering and more importantly the simulation constraints. Even the most elaborate algorithm would lack the ability to use random cycle lengths and random actuation of meters which can be brought on whenever there is a possibility of extraneous conditions forming.

However there had to be certain rules established in order for this metering to take place. To achieve this, a comprehensive visual analysis of the simulation along with the successive development of the risk index is undertaken. This was done with different seed values and the simulation was first run to obtain the risk index at every 30 second interval. Then the same seeds were matched and the simulation runs done in visualization mode. At each time step, at the location of interest the traffic characteristics were visually

identified. Based on this certain rules for switching meters on and off were obtained for each location to be metered.

For the ramps upstream of stations it was noticed that the increase in the safety indices coincided with the occurrence of queues and high congestion upstream of every location. In fact, whenever this condition occurred, there was congestion over longer lengths of a section, spanning 4 detector stations. Figure 5-2 shows the occurrence of these queues at Detector location 43. The formation of these queues could be visually identified by multiple stopped vehicles at any location in the simulation. As can be seen in the figure the vehicles close to the upstream ramp are stalled (indicated by vehicles moving out of the queue). This queue formation could be identified early and subsequently entrance from ramps be stopped to allow for queue dissipation.

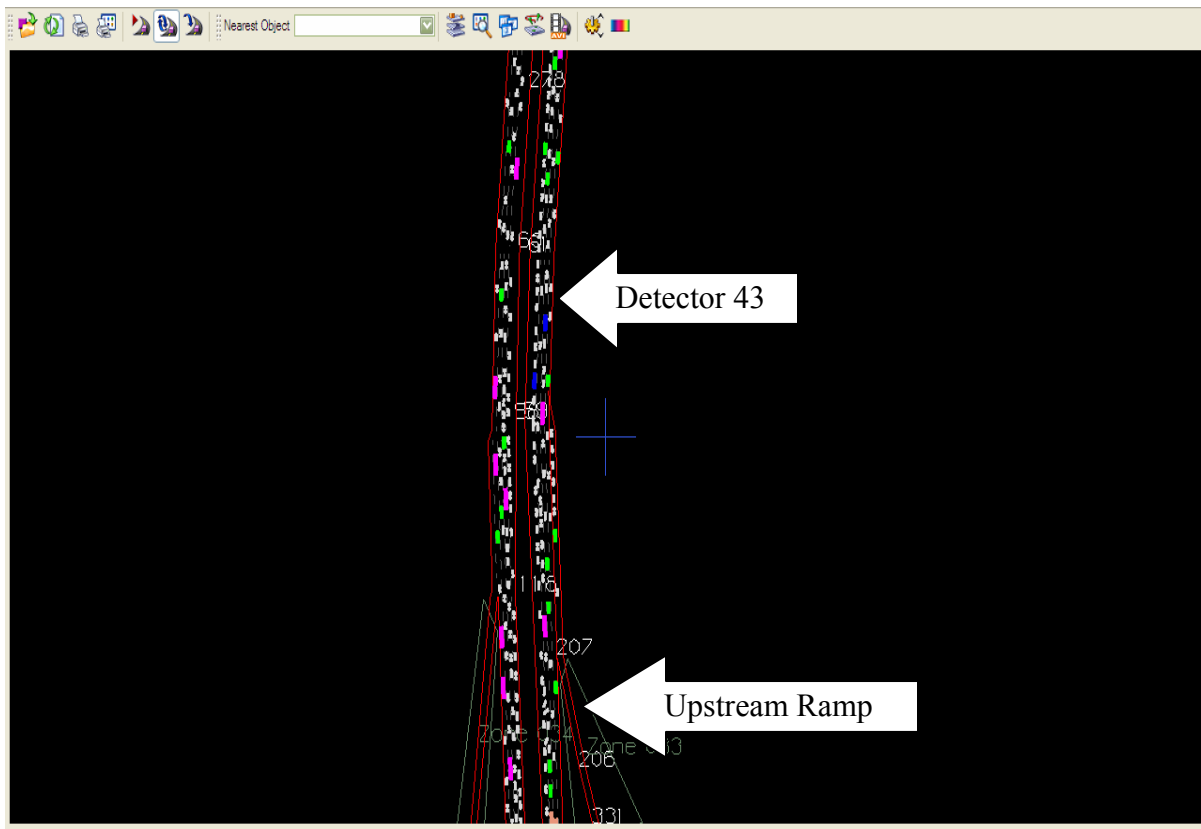


Figure 5-2 Snapshot of Queues at Detector 43 at 16:59:00

The conditions like those shown in Figure 5-2 could be identified with the starting of formation of a queue. A rule could be specified where every time more than 5 vehicles were seen to be going at stop and roll speed, metering could be manually actuated at the ramp upstream of that station. This could be done by specifying this rule in PARAMICS Modeler and every location with a queue was flagged.

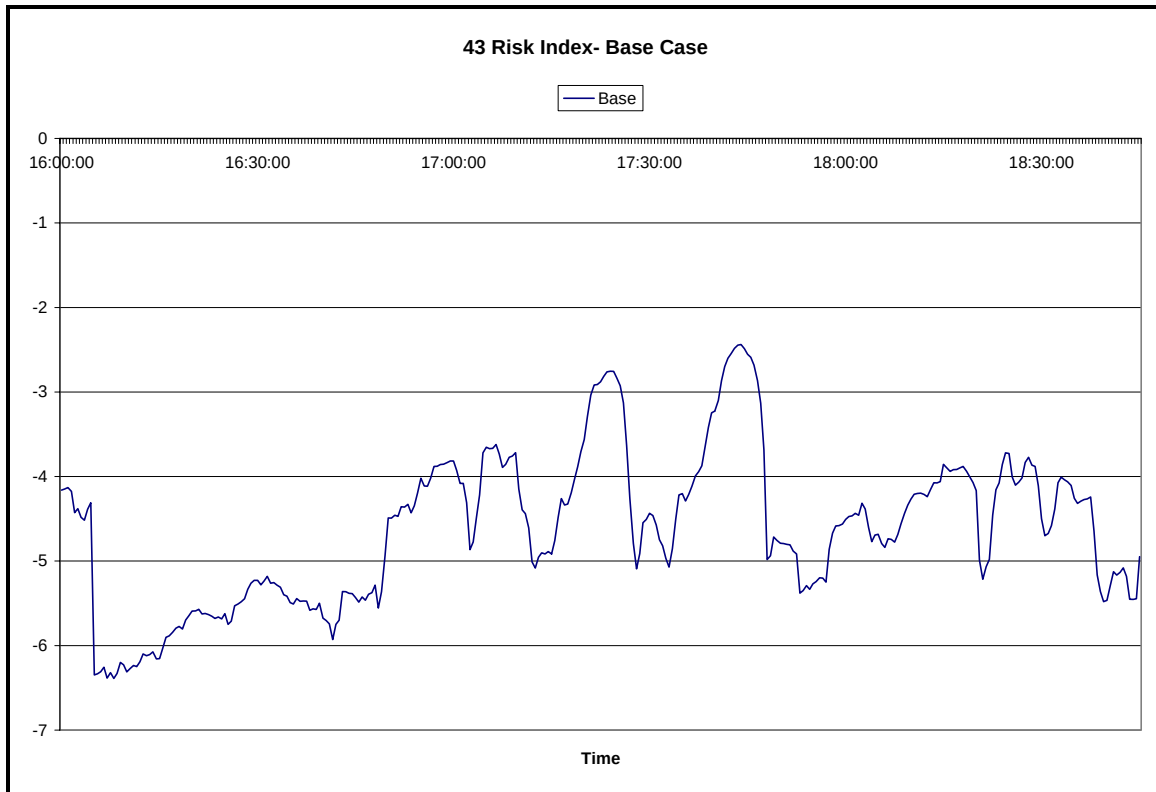


Figure 5-3 Risk Index Evolution in Base Case for Station 43 EB

Using this methodology and running multiple runs over the same seed values, the case with the best possible effect was identified. This process was manual though, based on judgment only. Figure 5-3 shows the risk evolution of Base Case over the period of simulation with a single. This curve would be used time and again in referring to evaluation of alternative scenarios with the same matched seed. A data series below the above curve would indicate a decrease in crash potential and vice-versa.

A data series was also calculated to show the evolution of risk in our case of testing manual metering. The comparison of matched seeds is illustrated in Figure 5-4. The test case in this figure shows the effect on risk index when manual metering was done during the simulation. In this case, meters were used at two ramps upstream of station 43. The meters were turned on to restrict entrance to mainline according to existing conditions. Every time a queue was occurring, the meters would be turned on. However, frequently the vehicles were allowed on the mainline to decrease queuing on the ramps.

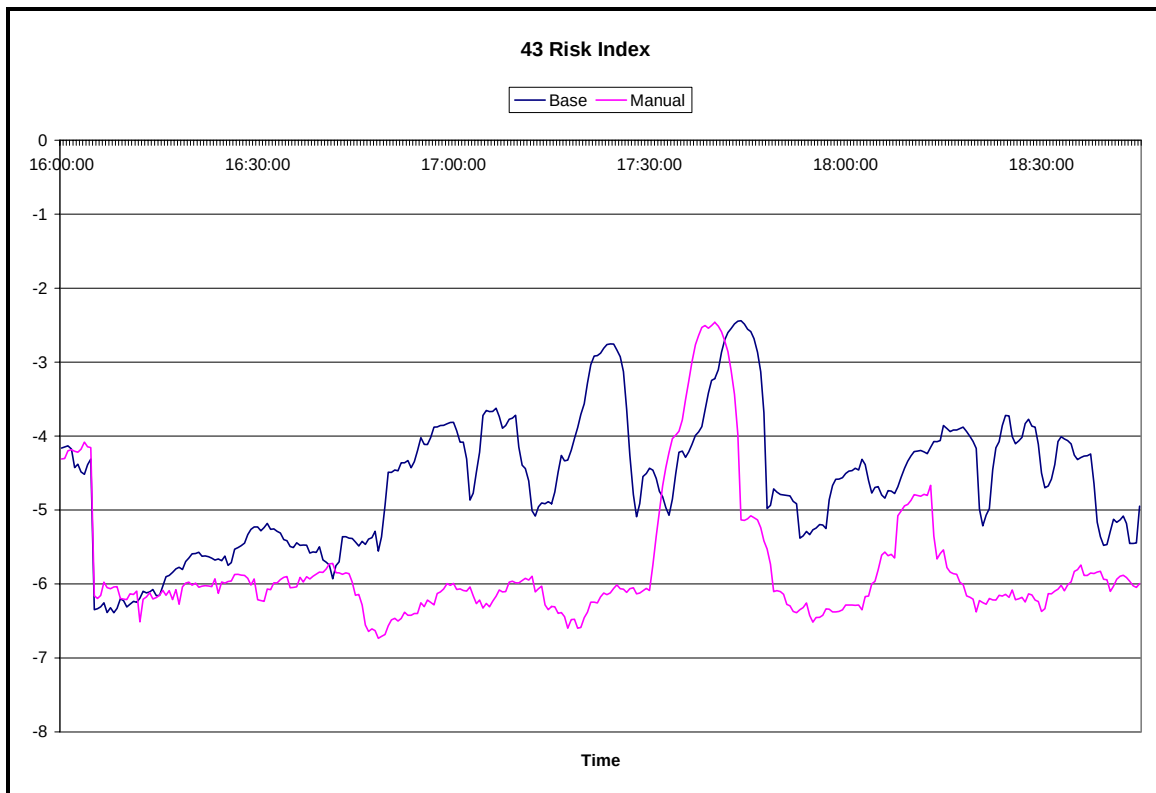


Figure 5-4 Comparison of Manual Metering and Base Case for Station 43 EB

If significant queuing was allowed to take place on the ramps, i.e. delays on the ramps could be significantly increased, better results could be obtained (spike in risk around 5:30 PM could be eliminated). Figure 5-5 shows the effect of metering manually and

allowing queues to form on the ramps. This meant that the meters were almost always signaling red and not allowing vehicles on the mainline about 10 minutes after the end of initialization period. In effect, this amounted to shutting down the ramps almost completely. The Figure 5-5, however, shows an improvement in the risk index as comparison to the case in Figure 5-4.

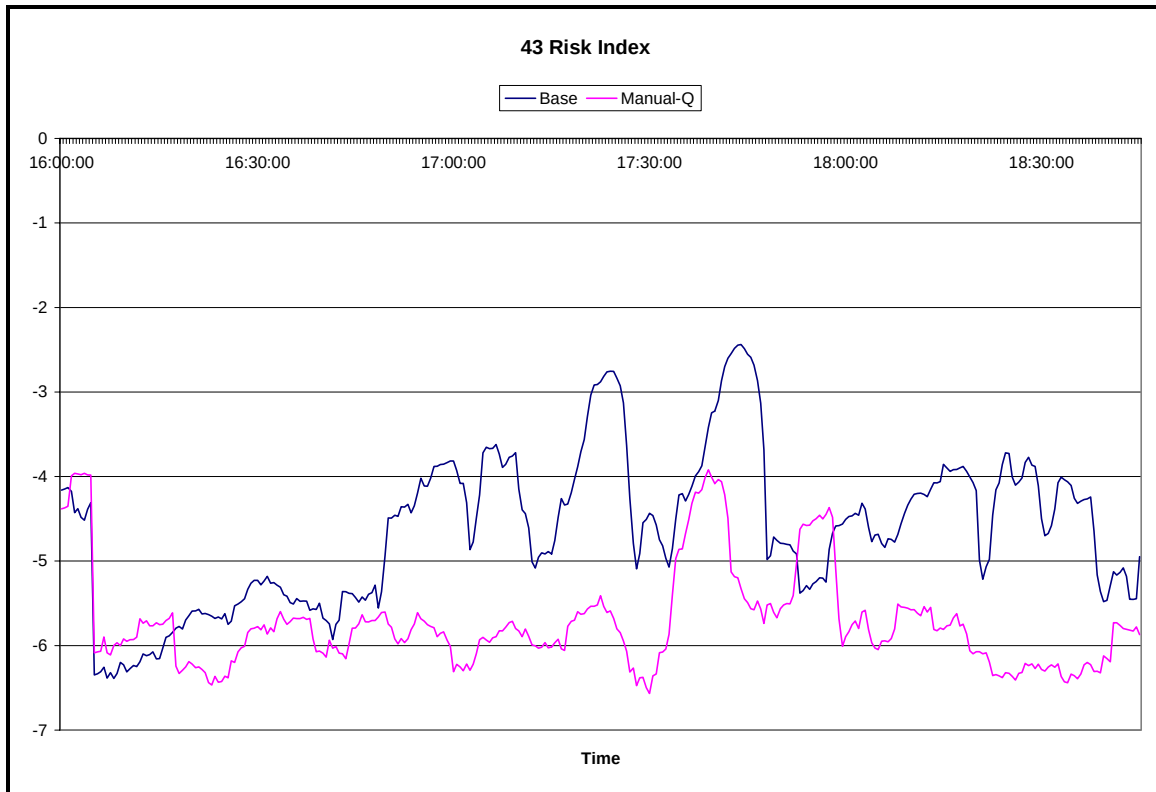


Figure 5-5 Comparison of Manual Metering with Queuing allowed and Base Case for Station 43 EB

Figures 5-4 and 5-5 show that decreasing flows onto the mainline improves the index for risk i.e. decreases the probability of a crash taking place. In the beginning of the study, this signaled the potential of ramp metering in alleviating risk of crash in congested situations. However, there remained the need to describe a better approach

than almost completely shutting down entry from on-ramps. A number of possible strategies are discussed in the following sections that try to attain this objective.

Although the advantage of metering based on observation is that there is a lot of flexibility in actuation and queues on the ramps can be controlled effectively, the disadvantages are apparent. Firstly, as a practical idea it is not really implement-able as it would require many people to observe the conditions every day at different locations. Secondly, it is open to interpretation on the discretion of each individual and the fact that sometimes observation by each individual might perceive different traffic conditions in separate ways. This part of the study was purely for exploration of possible benefit of ramp metering and provided the direction for rest of the process of evaluation of scenarios.

5.1.2.2 Constant Time Metering

Constant Time metering or fixed time metering is perhaps the simplest possible application of the ramp metering control. It involves the inclusion of a fixed signal cycle with double or multiple phases in order to control the inflow of vehicles. The length of the green phase can be determined from the desired hourly flow onto the mainline. Chu et al. (2003, 2004) showed that Feedback Ramp Metering works better as compared to the fixed time metering in terms of improving the capacity of the freeway. Since in this metering the only variables are the phase and cycle length, it is relatively easier to evaluate the different levels of this metering. The cases investigated in this application are tabulated in Table 4-2. The main case that needs to be considered and compared with other scenarios is the one which caters to the flow from the ramp within the time period. From a look at the ADT values for the ramps under consideration in 2003, it was decided

that during the three hour period most of the ramps cater to 1500-2000 vehicles although some ramps cater to larger numbers. This leads to an hourly flow of 500- 667vph. As such a flow of 600 vph was chosen as a targeted flow rate for each of the metered ramps. The ramps from SR-408 could not be considered in the process because of the heavy flows during peak hour, where metering even for short times might lead to queues, extending to the mainline of SR-408. A study of the single ramp upstream of station 43 showed that indeed the flow rate of 600vph caters to the demand from this source.

Also, any reduction in green time affects the total inflow into the mainline, which in simple terms translates into fewer vehicles onto the freeway. In order to study the effects of this metering without any interaction with total demands from a zone, station 43 was chosen as the only station to be metered with fixed time metering and its results compared to the base case as well as the cases of other strategies where only single ramps were metered.

5.1.3 Results for Module 1

Module 1 deals mainly with the cases in Pre- timed metering. The cases for this section are listed in Table 4-6. The cases performed almost equally well under all the scenarios within single and multiple metering. The differences seemed to be insignificant in most of the cases. Figure 5-6 shows the MCI for stations 33-49 for the base case as well as pre-timed metering implemented at ramp upstream of station 43(Princeton St. On-ramp).

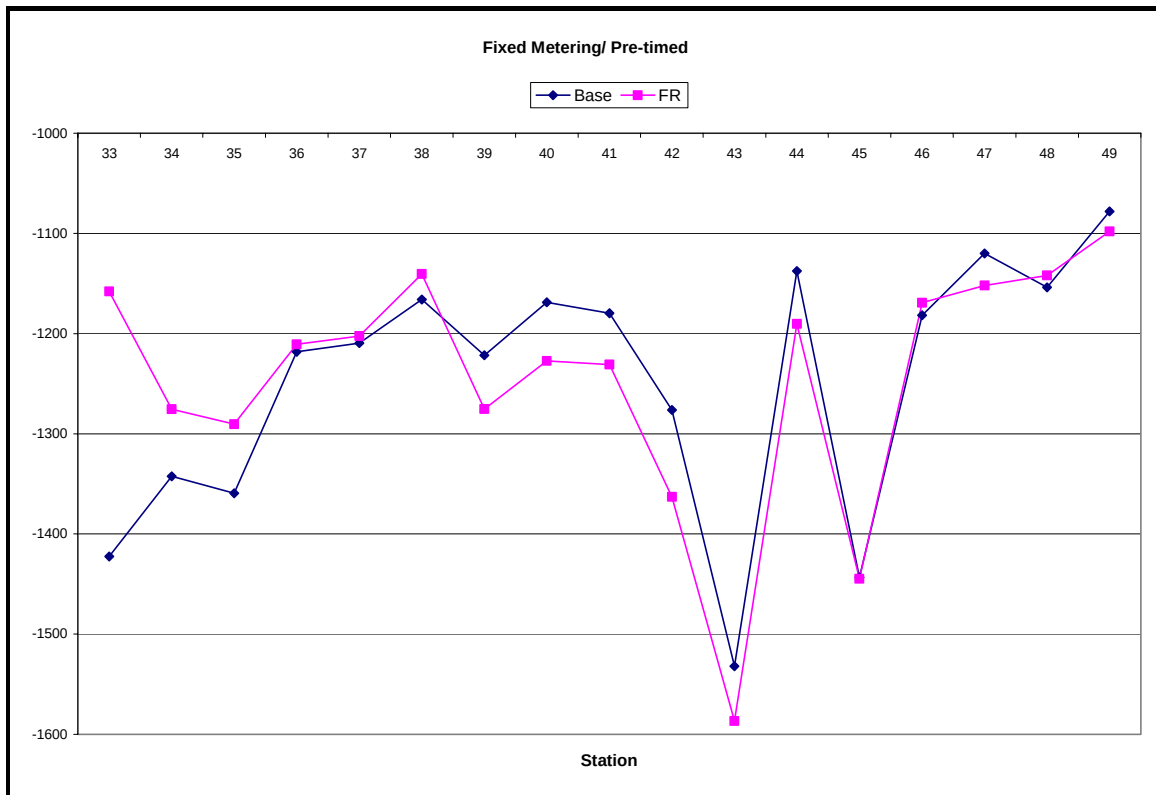


Figure 5-6 Base Case vs. Fixed Ramp Metering (One location) for all stations in range (y-axis MCI)

Figure 5-6 results show that there is very less improvement with the pre-timed metering. The station 43 shows an improvement in the cumulative risk index of 3.6 %. However, there is also an effect upstream of station 43. The improvement at station 42 is 6.6% over the base value and there are some significant improvements at some of the stations further upstream. The effect however, reduces 10 stations upstream and there is a significant increase in the risk at station 33. The profile for station 43 is depicted in Figure 5-7. It shows that the risk index shows several peaks which means that this strategy in congested situations does not show any significant improvement.

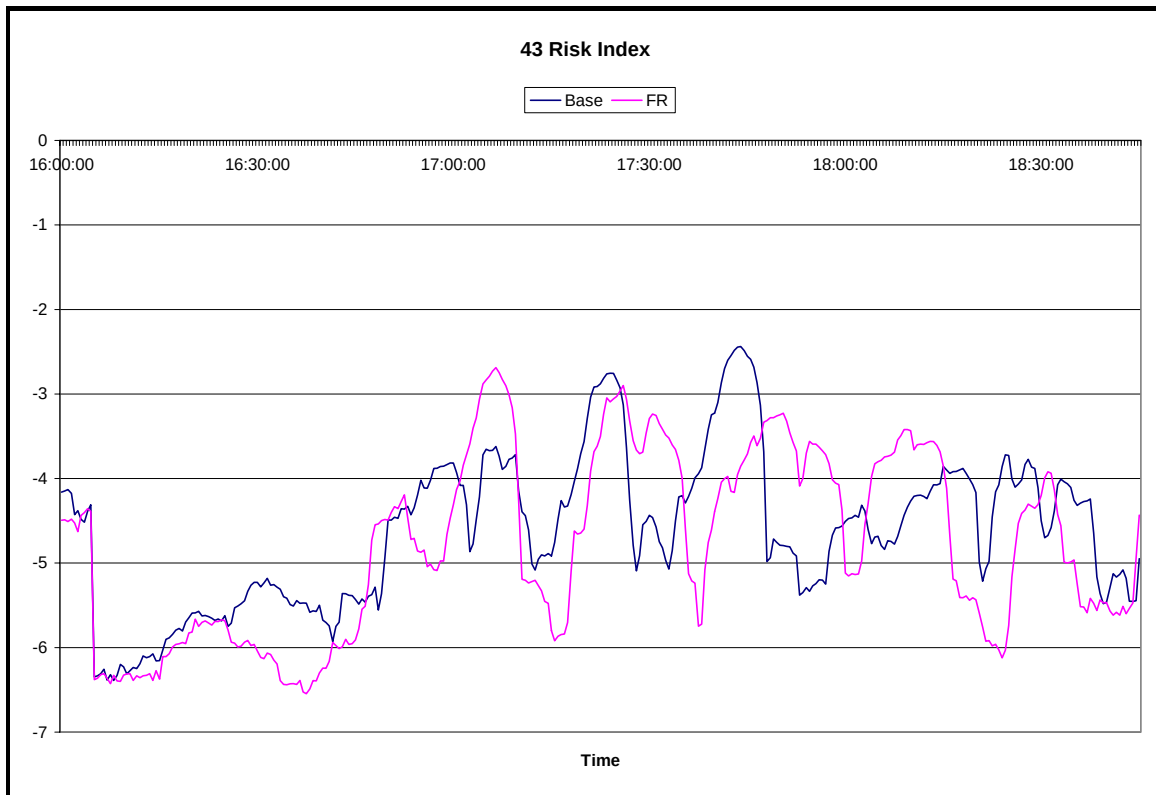


Figure 5-7 Risk Evolution Station 43, Base Case and Fixed Ramp Metering at single location

Figures 5-6 and 5-7 show that Pre-timed metering when applied to a single station did not present any major benefits. The implementation was then expanded to include multiple ramps in an effort to improve the benefits. Ramps upstream of stations 35, 38, 40, 42, 43, 47 and 49 in East Bound direction of I-4 were metered. These ramps lead to I-4 from Kaley Street, Church Street, Robinson Street, Ivanhoe Road, Princeton Street, SR-426 and Lee Road, respectively. These ramps will be used for metering throughout the analysis in this study. The comparison of MCI from the best case of Pre-timed metering at these 7 locations and the base case is shown in Figure 5-8.

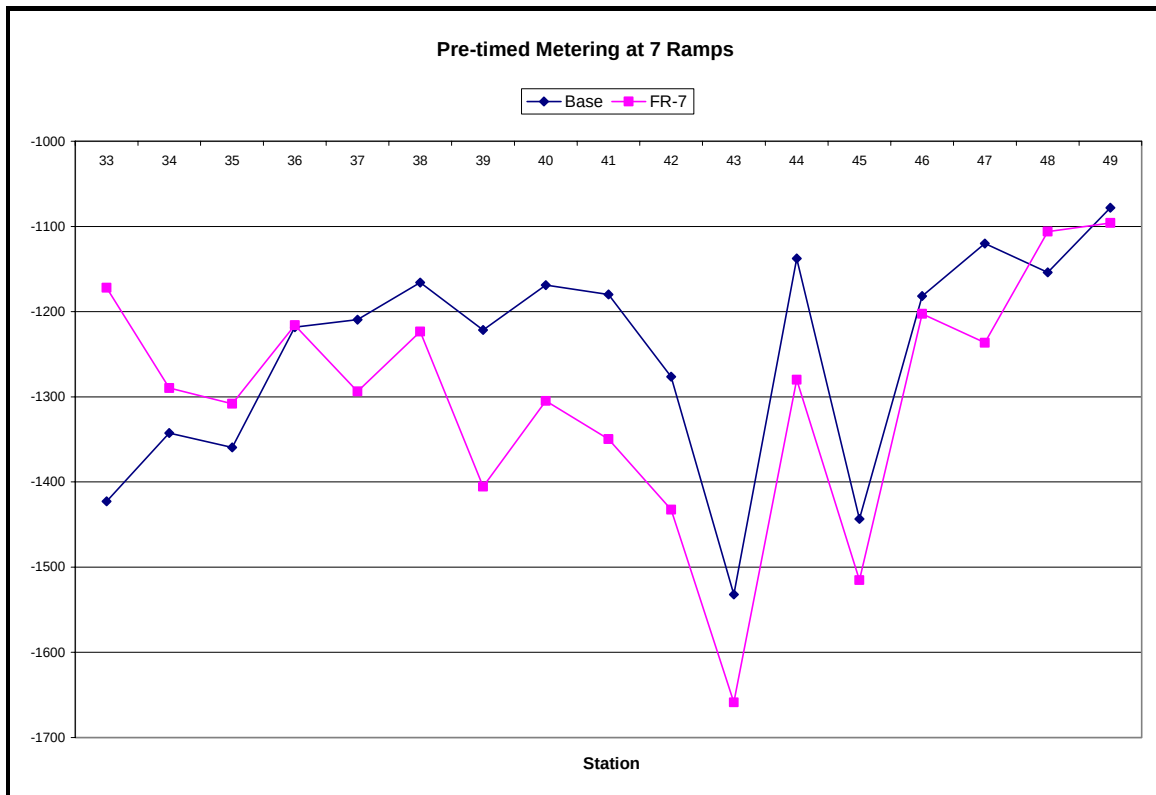


Figure 5-8 Case vs. Fixed Ramp Metering (Seven locations) for all stations in range (y-axis MCI)

Figure 5-8 shows that the MCI levels at 37- 45 have gone down substantially indicating an improvement in the risk at these locations. Location 36 shows no significant improvement but there is an increase in MCI for stations upstream of station 36 and also those downstream of station 45. To analyze the results of MCI further, the risk index evolution of station 43 and 42 was plotted. Figures 5-9 and 5-10 depict the risk index throughout the simulation at both these locations, respectively. Figures 5-9 and 5-10 clearly show that although there are times when the risk index for these two stations in metering scenario is far below the corresponding index in the base case, there are still significant periods of times when the risk index in metering scenario is above the base case indicating a potential increase in crash risk. The overall decrease in MCI from

Figure 5-8 signals an improvement at most times but it is offset to some extent by the relative increase of risk at other times as seen in the Figures 5-9 and 5-10 for individual stations.

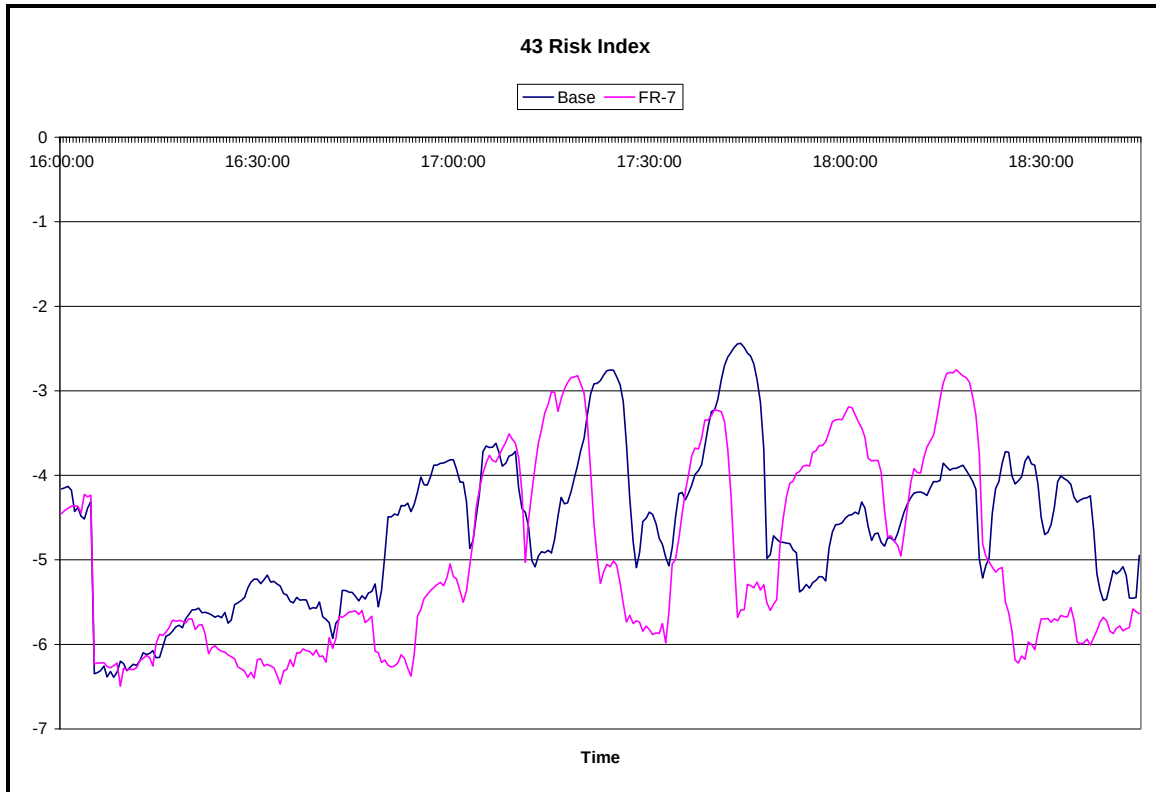


Figure 5-9 Risk Evolution Station 43, Base Case and Fixed Ramp Metering at 7 locations

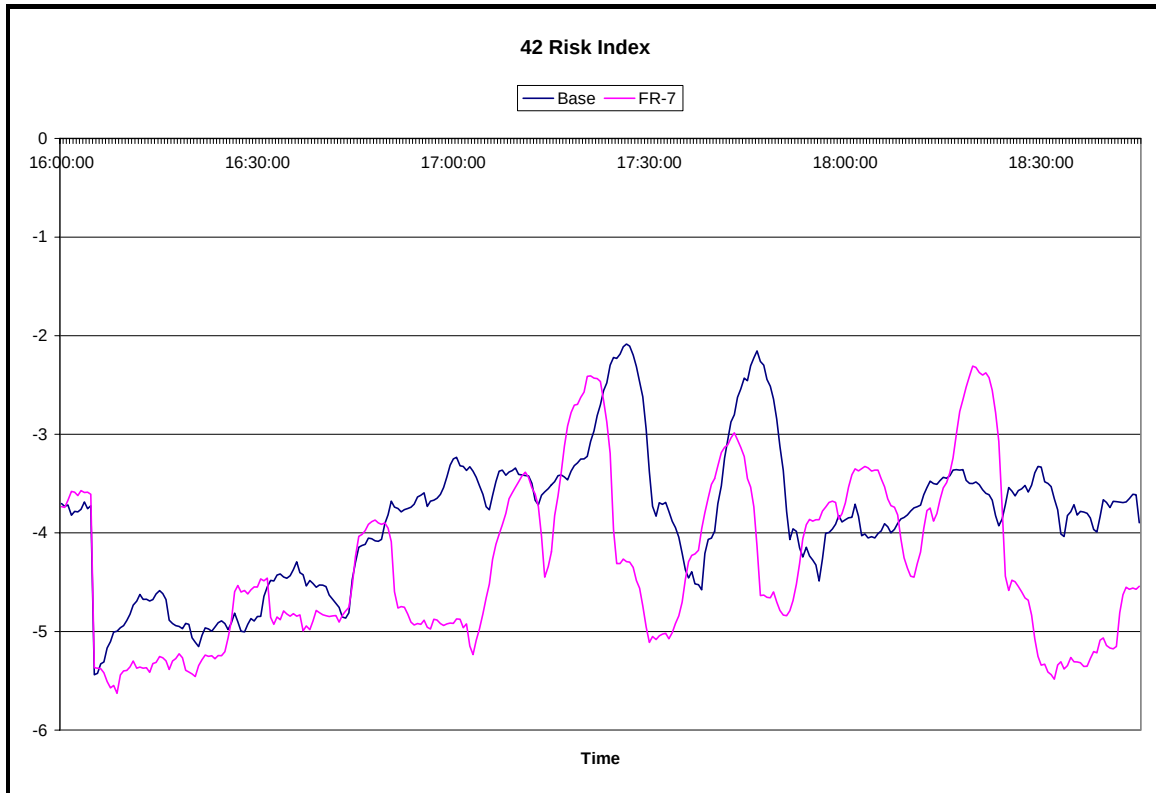


Figure 5-10 Risk Evolution Station 42, Base Case and Fixed Ramp Metering at 7 locations

Although the MCI for fixed time metering at multiple locations indicated some benefits, there still seemed to be scope for much more improvement to the risk index. As such, it was decided to move onto the next step of Ramp Metering which was the use of Algorithms which metered the ramps based on real time freeway conditions.

5.2 Feedback Control Ramp Metering

In the literature review (Section 1.4) it was indicated that there is significant improvement in Traffic Flow by using Feedback control over the feed-forward control algorithms where real time information was used at each stage of calculation of metering rate without considering the metering rate at a previous time interval. The fact that this works better for the traditional traffic measures (Papageorgiou et al., 1991) makes it the

better alternative to test rather than the feed-forward algorithms for application in crash risk mitigation strategies.

The ramp metering can be done using various different measures. The overall objective is to come up with a coordinated ramp metering algorithm which not only coordinates the ramps but also coordinates with the variable message signs to make use of the available ITS options in our bid to reduce crashes on freeways.

ALINEA ramp-metering control strategy, proposed by Papageorgiou (1991) in 1990s, has been shown to be a remarkably simple, highly efficient and easily implemented ramp metering application based on the results of several field implementations in European countries (Papageorgiou et al., 1991; 1997). Zhang et al. (2002) compare the effectiveness of ALINEA as a ramp metering algorithm against other algorithms and found it to be highly efficient. Because of the high performance of this algorithm, it is an excellent candidate for cost effective ramp control as well as for being embedded into a coordinated ramp control or integrated control system. Chu et al. (2003c) suggest that ALINEA as an adaptive ramp access control works better than BOTTLENECK in that the delays for the vehicles on the ramps are less.

The ALINEA algorithm is a local feedback ramp metering control policy. The algorithm attempts to maximize the mainline throughput by maintaining a desired occupancy on the downstream mainline freeway. The metering rate($R(t)$) during the time interval $(t, t+\delta t)$ is calculated based on the formula in Equation 5-4.

$$R(t) = R(t - \delta t) + K_R * (\hat{O} - O(t))$$

...Equation 5-4

δt is the update cycle of ramp metering implementation;

$\hat{O}(t)$ is the desired occupancy of the downstream detector station;

$O(t)$ is the measured occupancy of time interval $(t - \delta t, t)$ at the downstream detector station;

$R(t - \delta t)$ is the measured metering rate of the time interval of $(t - \delta t, t)$;

K_R is a regulator parameter, used for adjusting the constant disturbances of the feedback control.

Chu and Yang (2003) identified four parameters that needed to be calibrated for ALINEA implementation. These are

- Desired Downstream occupancy $\hat{O}$
- Distance of the downstream detector station from the On-Ramp merge
- Update cycle time for the ramp metering algorithm δt
- Regulator parameter K_R

Previous literature provides values that give an idea of the possible values for these parameters. Papageorgiou et al. (1991; 1997; 2001) suggests values for these parameters which are listed below

- The values of critical occupancy range from 18% to 31%
- Optimum values of Regulator parameter is set at 70

- Distance of downstream detector stations varies between 150 ft to 2000ft or even more
- The length of the update cycle too varies from 30 seconds to 5 minutes depending also on the length of the downstream detector station.

Chu et al. (2003) used genetic algorithms to find the optimal value for these parameters. A summary of their findings follows:

- Regulator K_R 70~200
- Desired occupancy 19~21%, 30~31%
- Update cycle of metering rate 30~60 sec
- Location of downstream detector 120~140 m

Lee et al. (2004) in a similar study to identify the safety impacts of ramp metering implemented on a simulated section of I-880 used K_R value as 59 and critical occupancy values as 0.18.

Although effective as a ramp metering algorithm for single ramps ALINEA has its disadvantages too. Primary among them is the fact that it is unable to meter in anticipation of congestion but rather kicks in when there is existing congestion at the downstream detector station. Also Papageorgiou et al. (2001) found that ALINEA didn't perform as well in a coordinated metering set up under conditions of non-recurrent congestion. In that case it makes sense to use a coordinated ramp metering algorithm like METALINE or Rule based Fuzzy Coordinated ramp metering. Despite these disadvantages, ALINEA is still an effective control strategy and worthy of being tested for network wide implementation.

5.3 Results for Module 2

Ramp Metering using Feedback Control Algorithm involves many factors that need to be considered. All these factors are listed in Sections 4.4.2 and 5.2. The experimental design for module consisted of making a total of 48 runs to test for the entire variable set mentioned, namely

- Critical Occupancy, O^*
- Signal Cycle Length
- Number of Ramps to be controlled
- Regulator Parameter K_R

The analysis was done according to the case numbers provided in Table 4-7. The primary effects to be tested using the experimental design were:

- Effect of Signal Cycles at metered ramps
- Effect of Number of Metered Ramps
- Effect of ALINEA Parameters- Critical Occupancy and Regulator Parameter

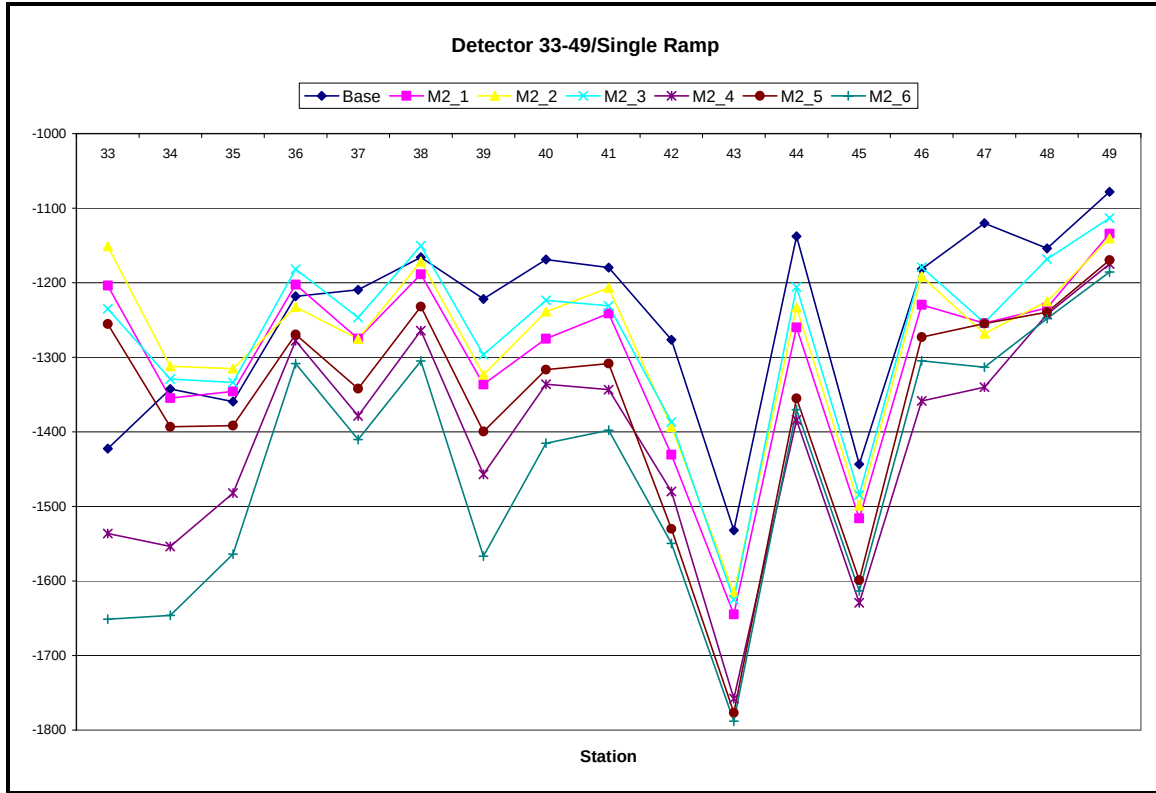


Figure 5-11 Cases 1-6 Module 2. MCI comparison for Station 33 -49

The first six cases were meant for analysis of the case when only one ramp in the network is metered. Princeton St. On-ramp, just upstream of Station43 in East Bound direction of I-4 was metered for this part of the analysis. Figure 5-11 shows the results from an analysis of MCI for the first 6 cases of Module 2. All these cases considered single metering location. The effect of metering at a single location using ALINEA on the whole network is depicted in this figure.

The points on the graph depict level of our Measure of Effectiveness (MOE), the Mean Cumulative Index (MCI) according to the station of loop detectors. Figure 5-11 clearly shows that the case that works best in case is Case 6 which involves metering a single station with a signal cycle of 50 seconds, taking critical occupancy value of 23%.

The corresponding cases for the K_R value of 120, i.e. Cases 24 to 30, mirrored the exactly same results and there were very minimal variations in the results for that case also. This trend was observed at every stage of the analysis. The MCI across stations of cases 18 to 24 is presented in Figure 5-12.

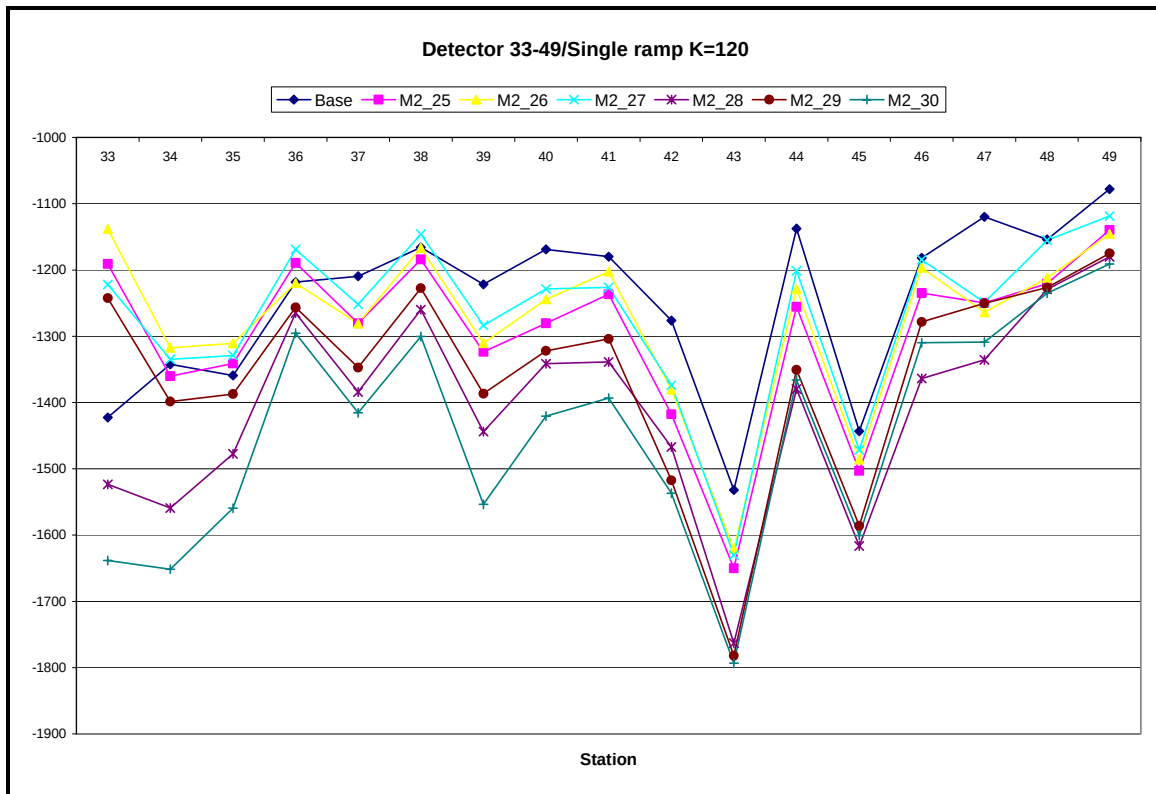


Figure 5-12 Cases 25-30 Module 2. MCI comparison for Station 33 -49

Figure 5-12 shows that the parameter combination from Case 6 provides the best results here too and the difference between the indices for Case 6 and Case 30 is very less. This suggests a lack of effect of the regulator parameter on our results. The insignificance of this effect is illustrated in Figure 5-13. It shows the MCI for the two cases where all parameters are matched but the value of K_R is 70 and 120 for cases 6 and 30 respectively.

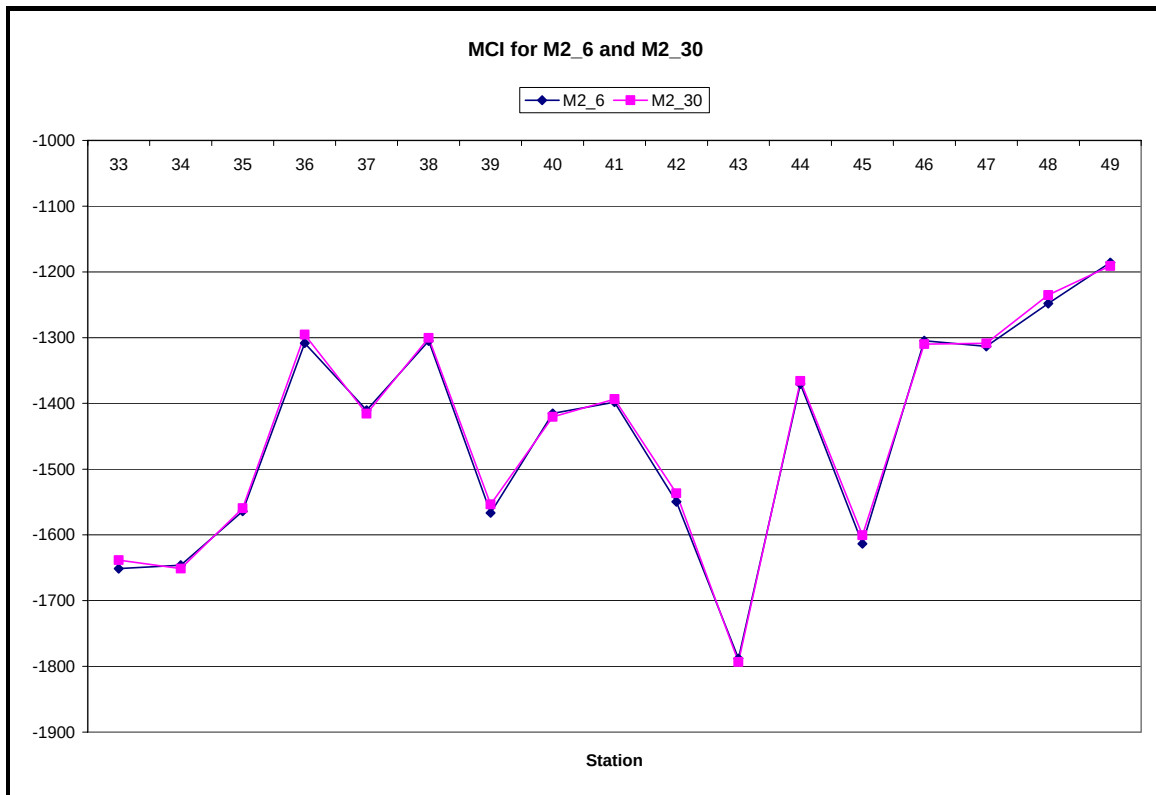


Figure 5-13 Cases 6 and 30 Module 2. MCI comparison for Station 33 -49

For further analysis of the crash risk indices, evolution of risk index with time for the base case and the best case from the scenario Module 2 Case 6 is provided in Figure 5-14 for station 43 (also the location of metering). The notable feature of the figure is that there are time intervals in which the risk matches the measures of risk in the base case; however, the net decrease in MCI (16.15 %), which is a cumulative measure, can be attributed to the time intervals when the risk index is significantly less than the base case index.

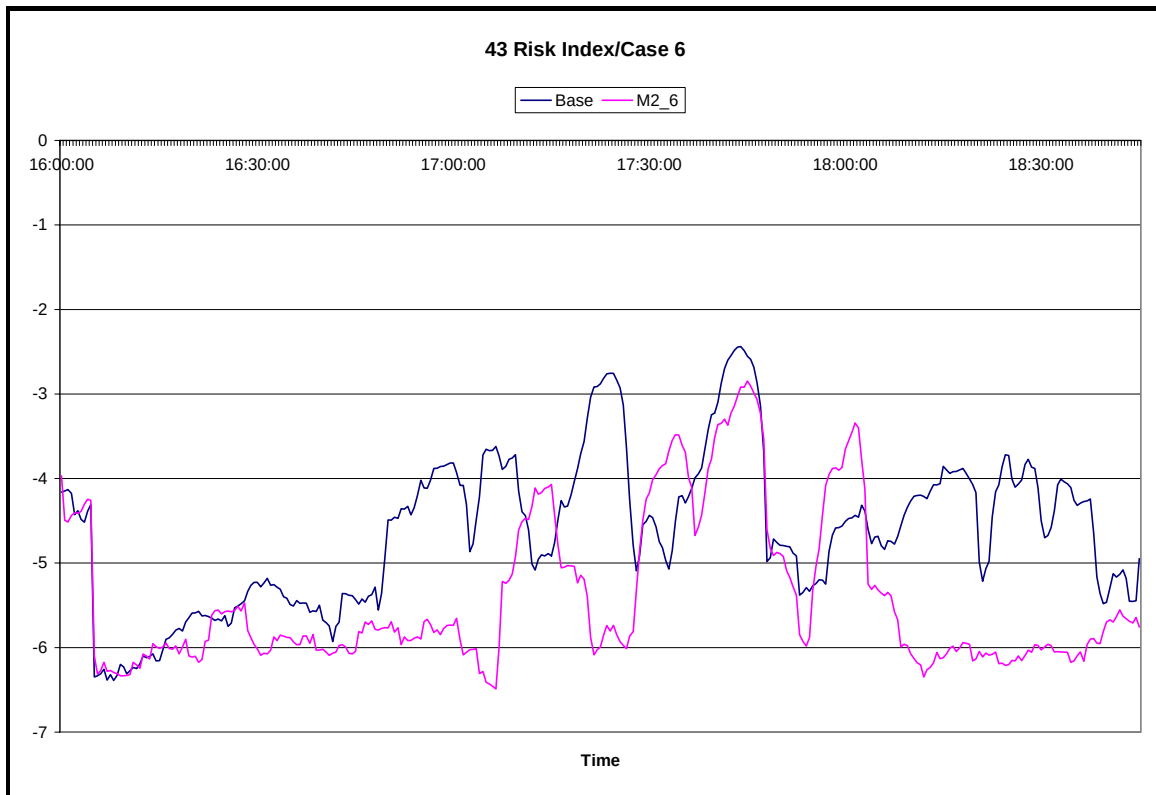


Figure 5-14 Module 2, Evolution of risk index with time for the base case and the best case

However, despite the time intervals where risk is still high, the result still shows an overall improvement which is reflected in our MOE.

The first six cases present three significant results from the analysis

- Higher value of critical occupancy provides best results in terms of MOE
- Longer signal cycles provide better results than shorter signal cycles for a single ramp metered in a network
- K_R is an insignificant factor in the comparison cases.

The first two points provide very interesting cases for follow-up. If we look at things from a logical perspective, the best case would be to completely shut down ramps, i.e., do not allow any more vehicles onto the mainline. Dillmore (2005) in his work concluded

that since shutting down the ramps works, hence ramp metering should be effective in reducing the risk of crash too. This might lead us to conclude that making fewer cars get onto the ramp would improve safety. As such the critical occupancy for the meters should be a small value, so that it can be readily exceeded and there is less green time for vehicles coming in from the ramps. However, as the first point concludes that higher critical occupancy values work better, which means that there is no correlation between numbers of vehicles entering but rather the time of entry of vehicles for assimilation into the traffic stream is the key factor that affects safety. Another crucial point is the fact that longer cycle lengths work better when a single ramp is metered. This observation will be of significance when it is analyzed in reference to the results from rest of the experimental design.

There is a transformation in results when the results for multiple ramp metering are analyzed. The metering cases were expanded adding successively two meters on each side of Princeton Street Ramp (location upstream of station 43).

Figure 5-15 shows the results for MCI for 3 metered ramps, viz. Cases 7 – 12 in the experimental design. The additional ramps metered were upstream of stations 42 and 47.

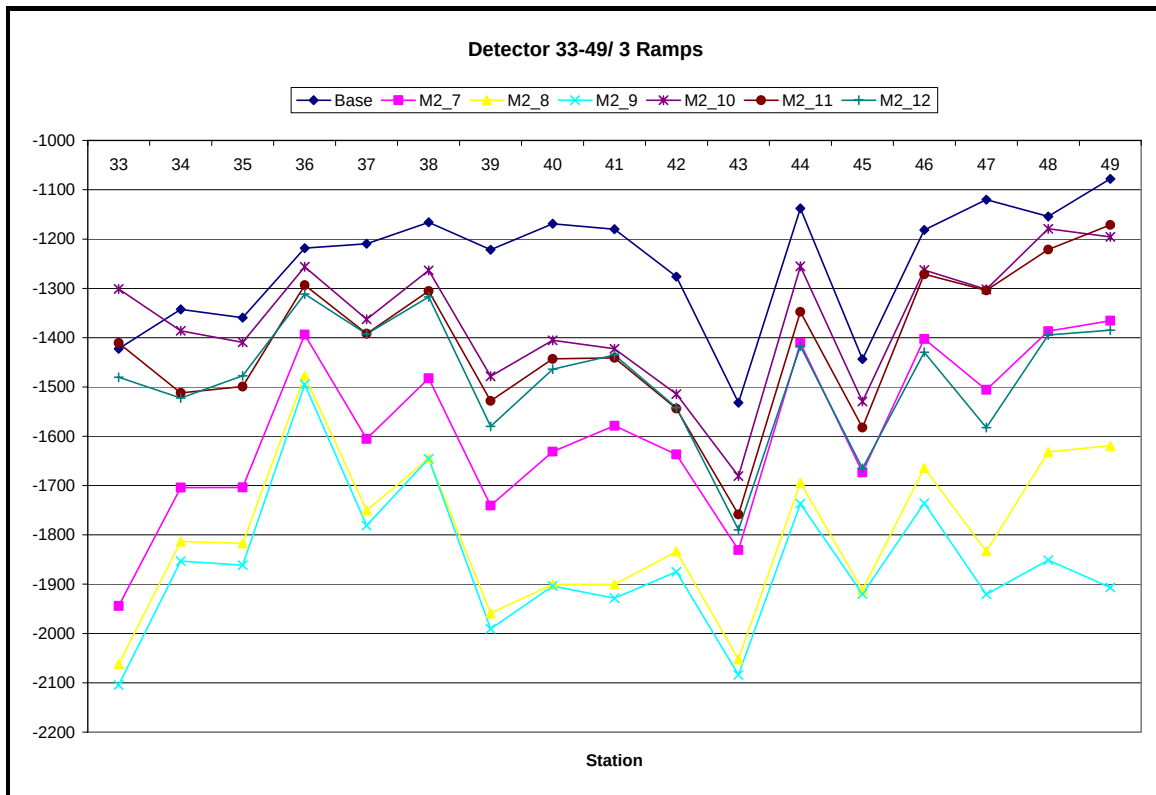


Figure 5-15 Module 2 Case 7- 12 MCI Station 33-49

The MCI for the cases in Figure 5-15 shows that cases 8 and 9 outperform the others by a large amount. Case 8 has a critical occupancy value of 20% and signal cycle length of 25 seconds. Case 9 which slightly outperforms Case 8 represents a critical occupancy of 23% for a signal cycle length of 25 seconds. The corresponding cases for the different regulator parameter values were again found to be insignificantly different from these cases and hence do not effect the final observations. As such only the first 24 cases will be presented for interpretation of results.

The results from this part of the analysis presents one significant reversal from the previous results in that shorter signal cycles work much better and significantly outperform the longer signal cycles (Cases 7, 8, 9 for shorter cycles).

This trend continued into the next part of the analysis where 5 ramps were metered. These ramps were upstream of stations 40, 42, 43, 47 and 49. In this section too, cases 13, 14 and 15 again outperformed the other cases. These factors represent exactly the same configuration as cases 8 and 9 respectively, except that the number of metered ramps was increased from 3 to 5. Figure 5-16 shows the results for MCI from this part of the analysis.

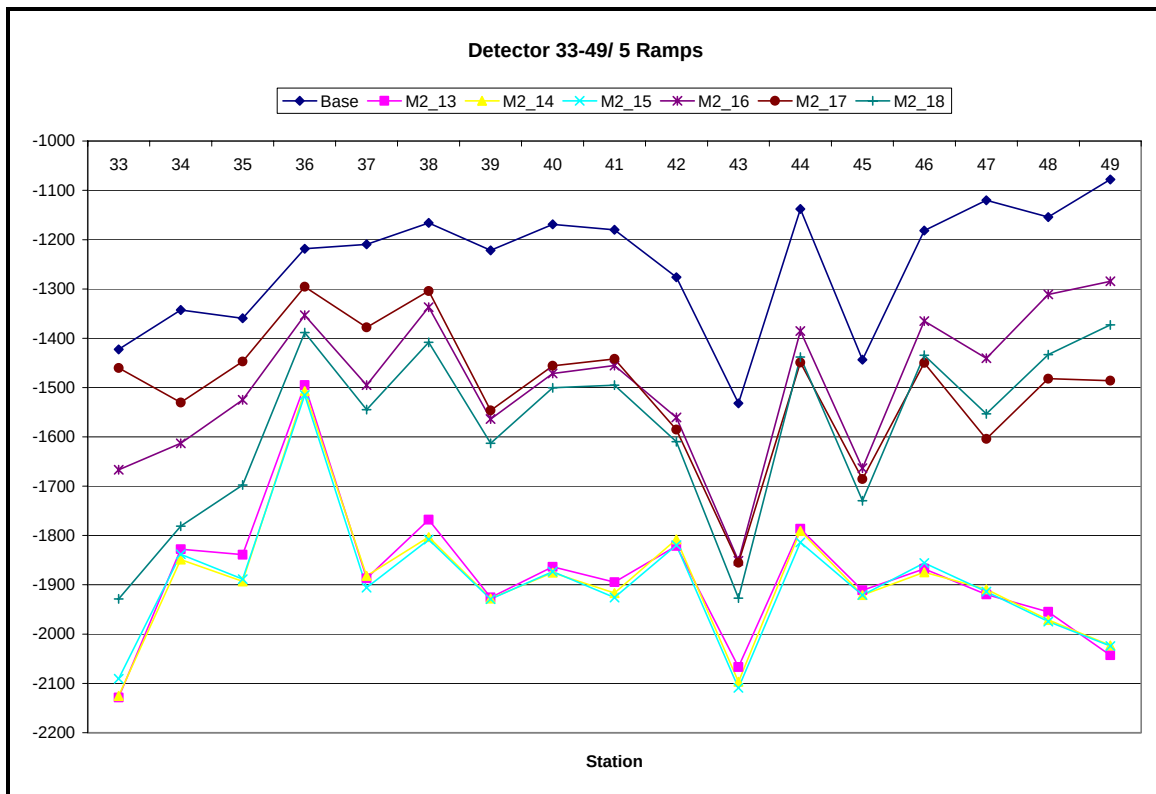


Figure 5-16 Module 2 Case 13-18 MCI Station 33-49

The last part of the analysis was testing the parameter combinations for 7 metered ramps. The ramps upstream of stations 35, 38, 40, 42, 43, 47 and 49 were metered using ALINEA. An analysis of Figures 5-16 and 5-17 showed that this followed the trend of the previous two parts and performed better for cases with higher critical occupancy and

shorter signal cycles. Figure 5-17 shows the MCI for the stations when 7 ramps are metered using different parameter combinations.

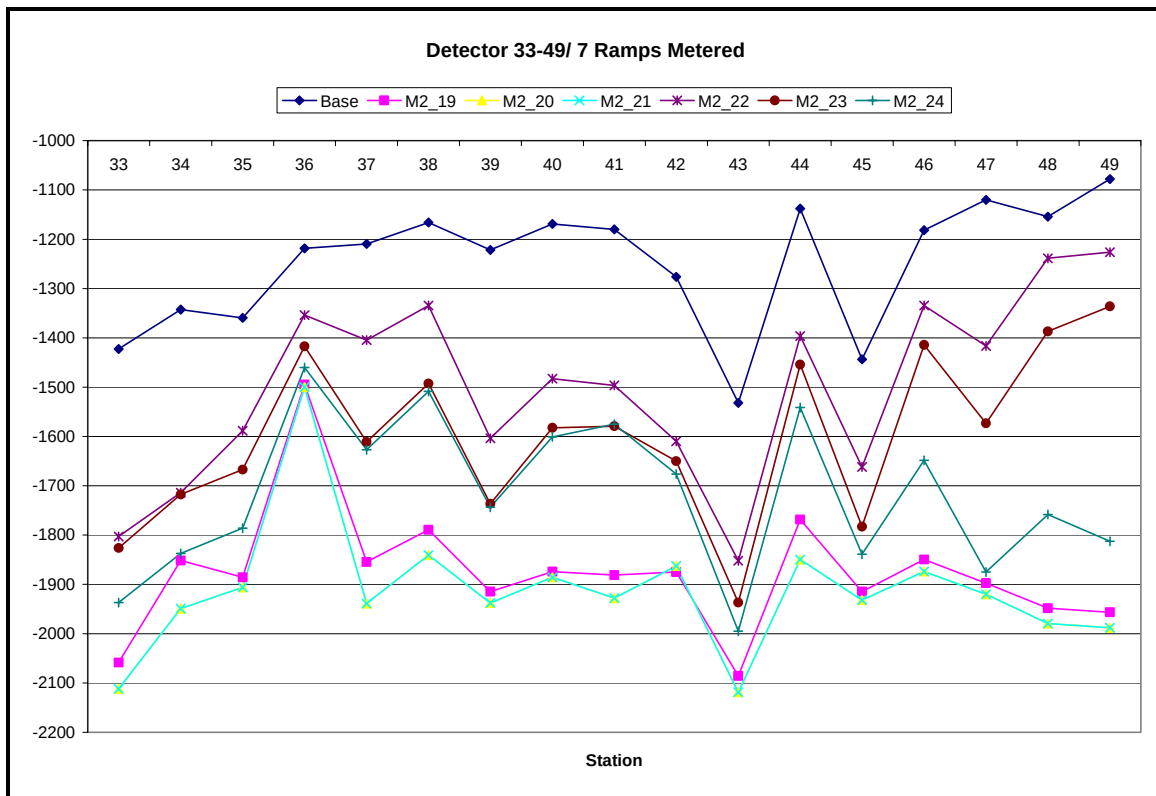


Figure 5-17 Module 2 Case 19- 24 MCI Station 33-49

Figure 5-17 shows that amongst all the scenarios tested for multiple ramps, scenarios 19, 20 and 21 perform a lot better than the others. The best scenario was found to be Case 21, with critical occupancy at 23%, cycle lengths of 25 seconds. To further analyze this, the effect of multiple metered ramps on the individual risk index at the stations was observed under the scenarios found to be most effective for all stations. This is shown in Figure 5-18 for station 43.

Figure 5-18 provides the comparison for station 43 between all the cases for 7 metered ramps a single matched seed. As can be seen case 21 performs the best even here for the 3 hour period.

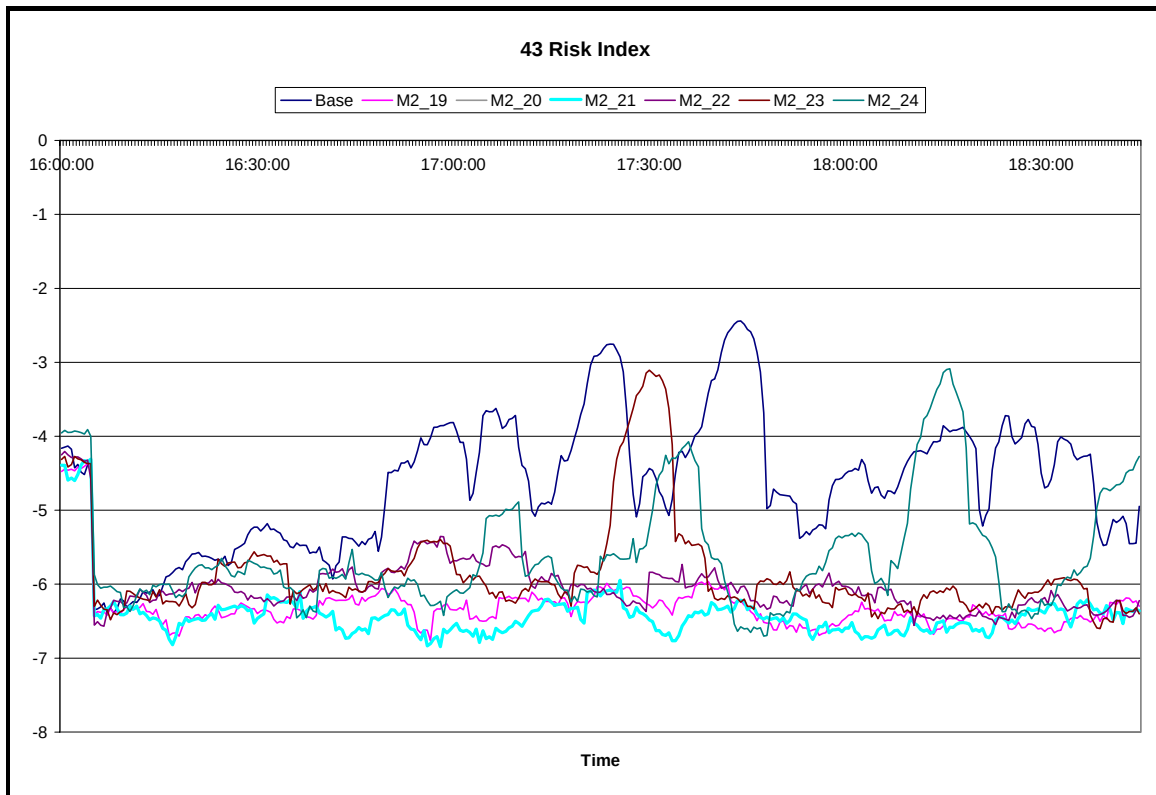


Figure 5-18 Risk Evolution Module Cases 19-24 Station 43

The reason for the crucial change in risk index can also be seen by looking at the simple measures for the stations that we consider. Figure 5-19 shows the 30 second speed data from station 43 before and after our analysis (Case 21). The network-wide metering has the effect of decreasing the variation in the speeds at this location.

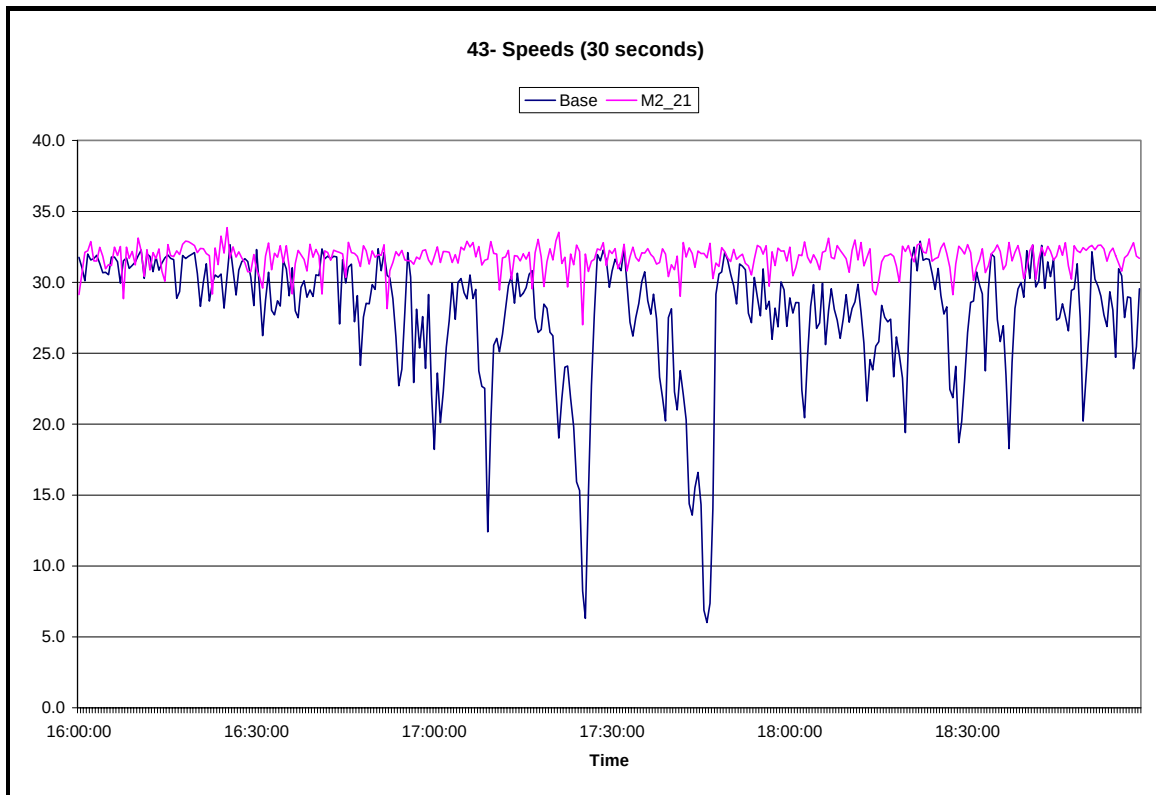


Figure 5-19 30 sec. speed profiles Base Case and Module 2 Case 21

To determine if case 21 performs best for a number of stations, risk index evolution for the period of simulation was compared for stations 40 – 45. The cases are depicted in Figures 5-20 through 5-22.

As can be seen from Figures 5-20, 5-21 and 5-22, scenario 21 outperforms the others for all the stations. Also, the cases with lower cycle length outperform all the cases with higher cycle lengths. The observation that critical occupancy value of 23% works best is true here too.

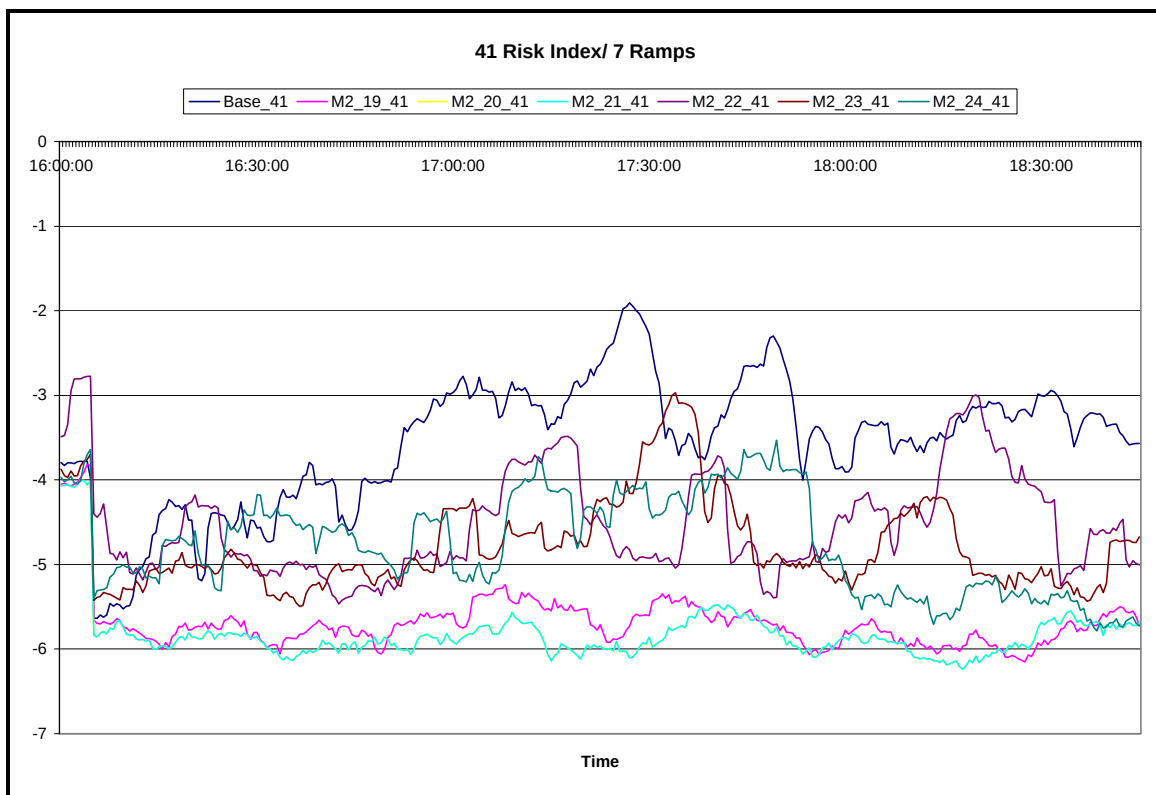
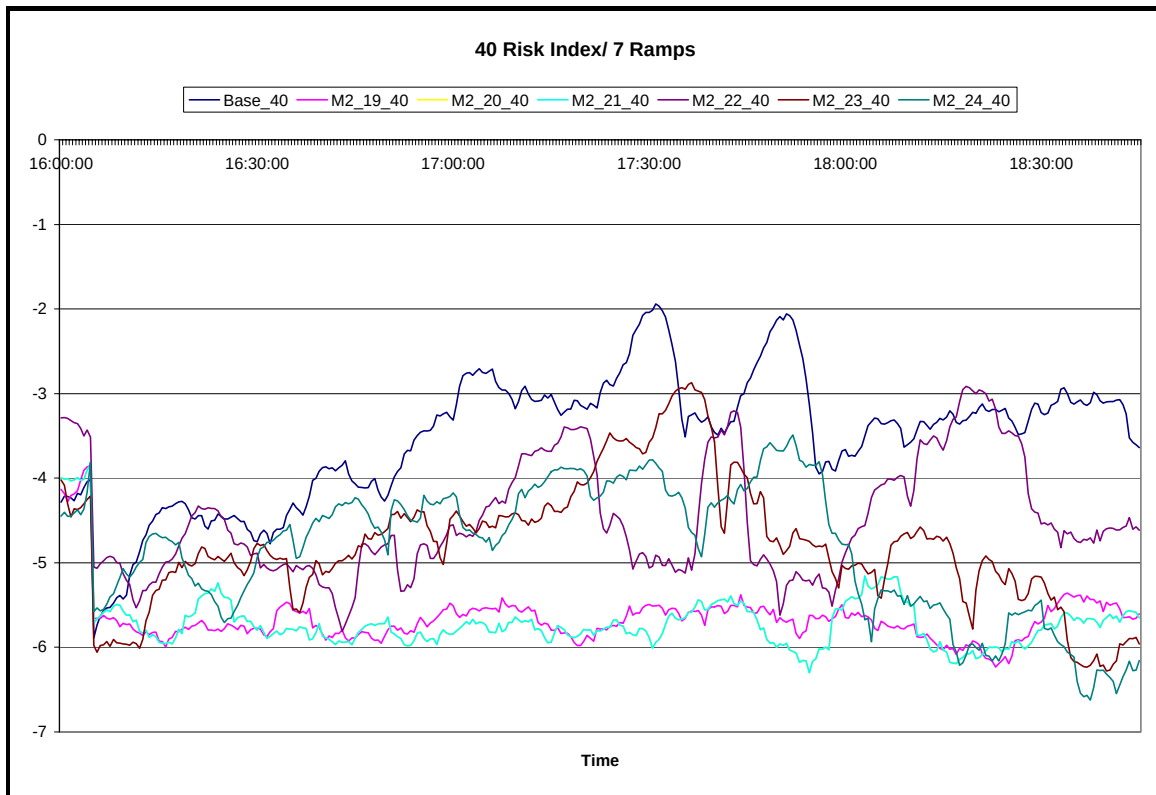
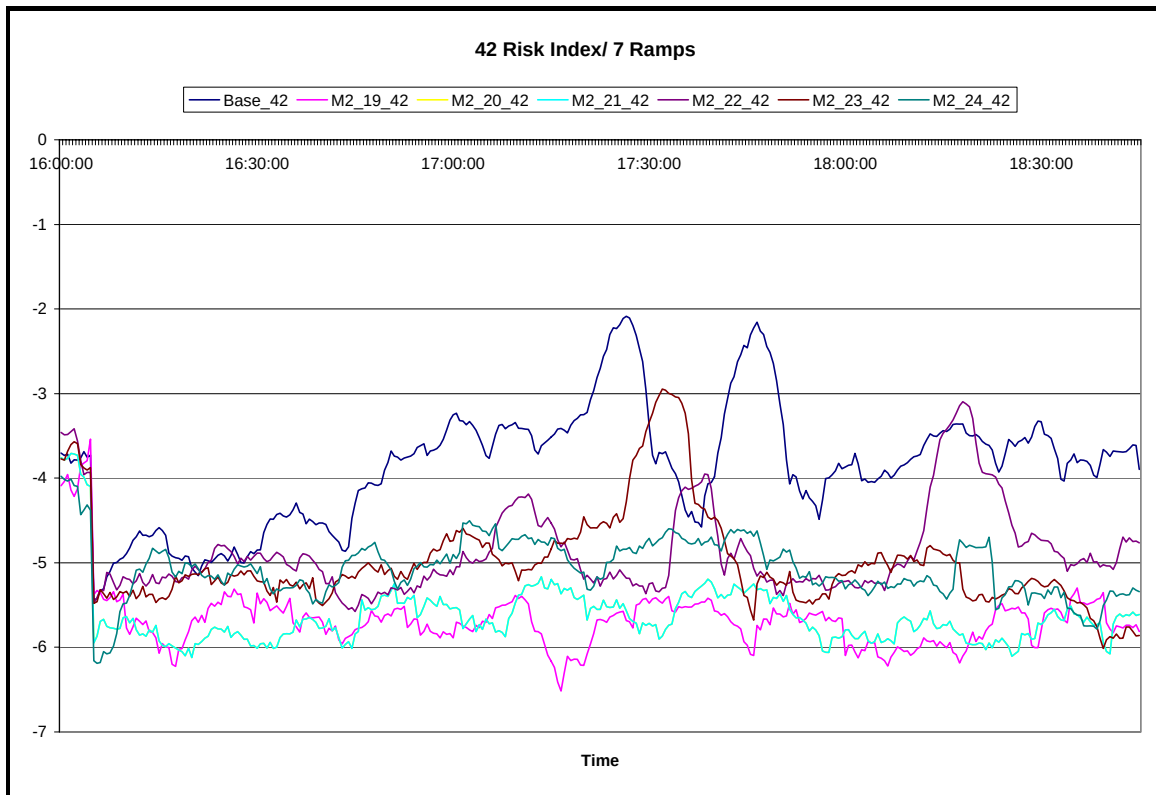


Figure 5-20 Module 2 Cases 1-6, Risk Index Evolution Station 40-41



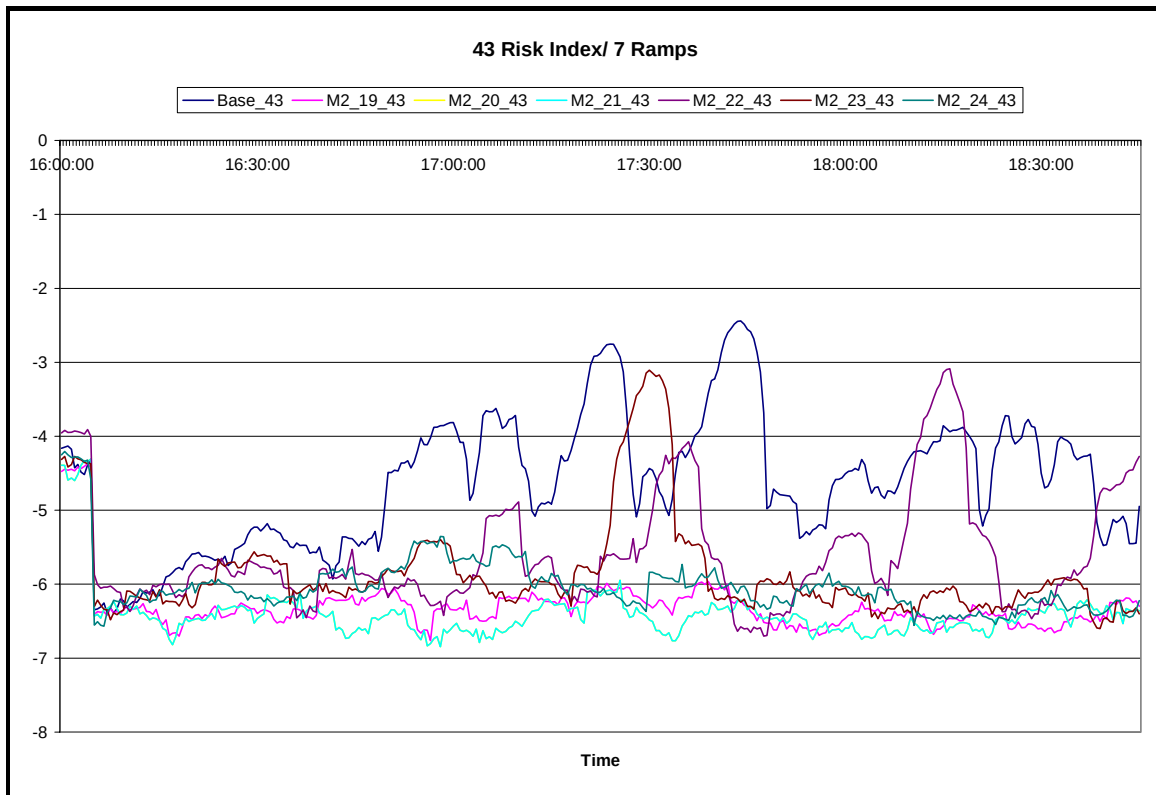
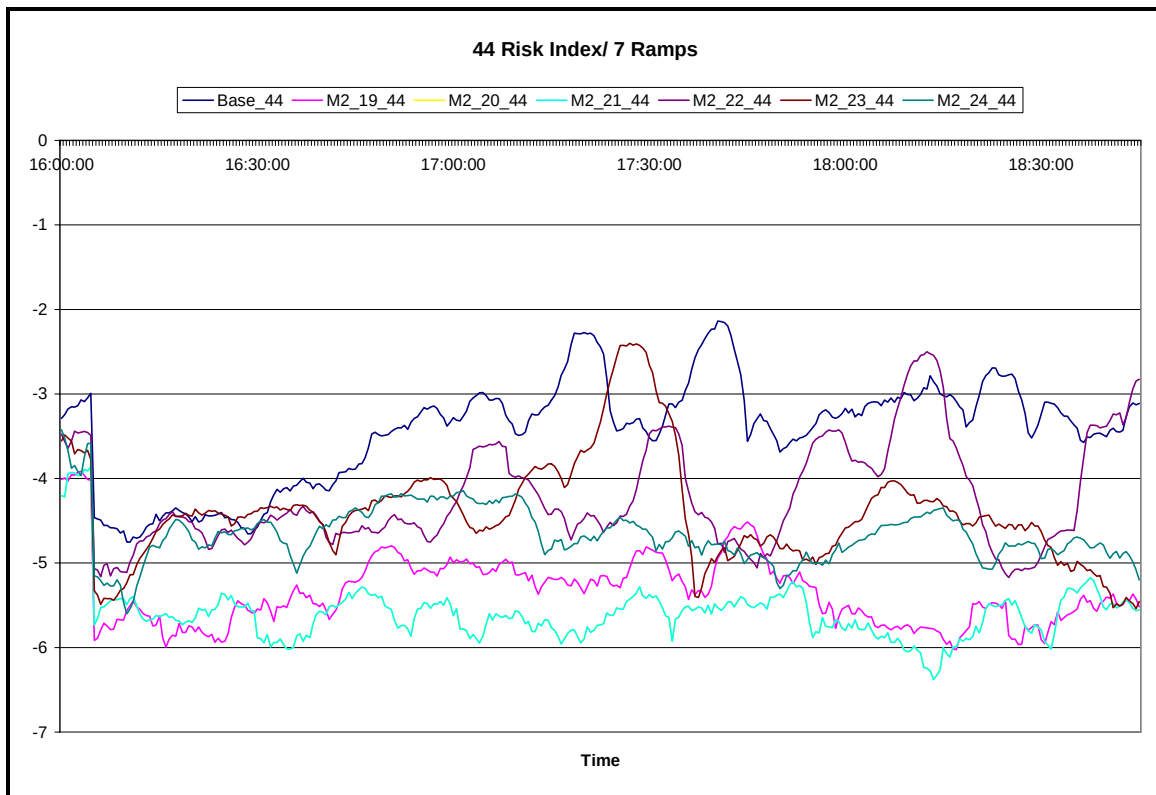


Figure 5-21 Module 2 Cases 1-6, Risk Index Evolution Station 42-43



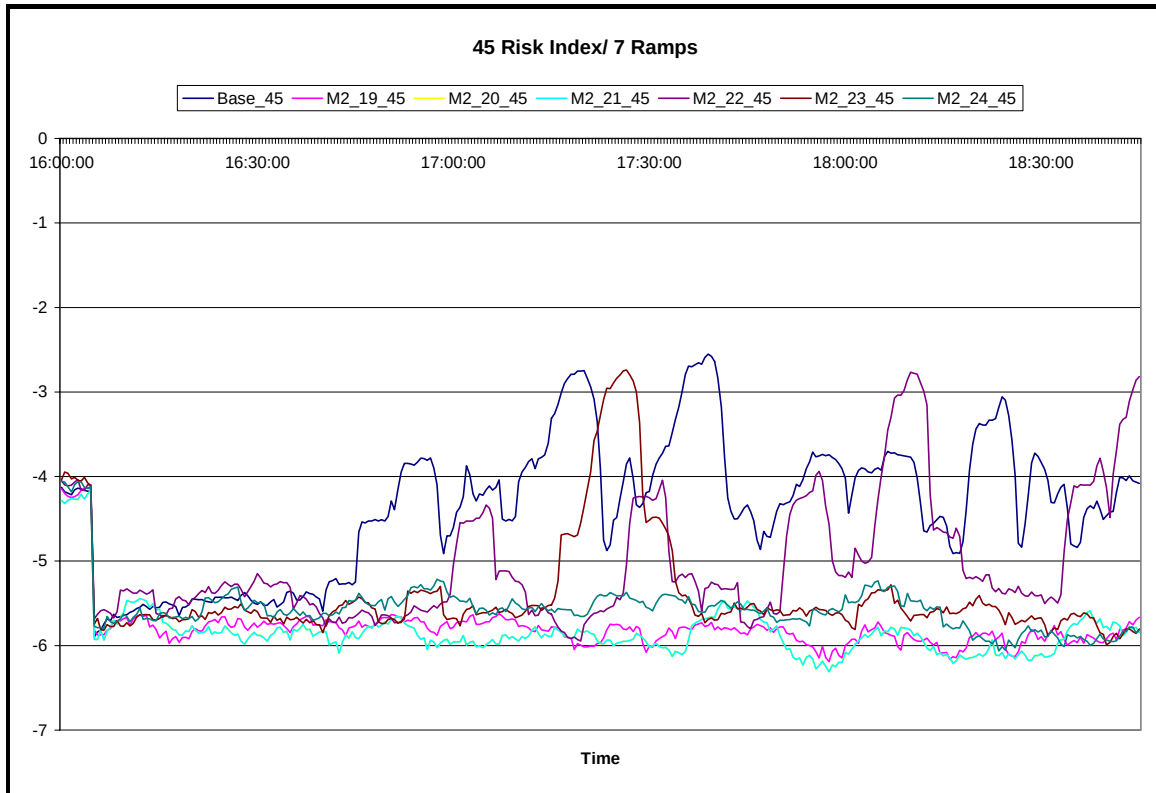


Figure 5-22 Module 2 Cases 1-6, Risk Index Evolution Station 44-45

To comprehend the effect of the observations made in this section, we need to revisit the low speed model for crash risk assessment provided in Equation 5-1. From Equation 5-1, it is apparent that the most significant parameters are the Coefficient of Variation of Speeds (CVS) at the Station of Interest between 5 and 15 minutes before the time of crash. The coefficient of variation of speed for every 5 minute interval is the standard deviation of speed for that 5 minute interval divided by the average of speeds for the same interval. As can be seen from the profile of speeds in Figure 5-19 and also in Figures 5-23, 5-24, 5-25 and 5-26, there is a significant decrease in the deviation of speeds which leads to a decrease in the numerator of CVS. Also the average speeds go up which causes the denominator to increase thus further suggesting a lowered value of

CVS. An increasingly small value for CVS leads to an increasingly negative value for log of CVS. In addition, the coefficient for this term is large which makes it a very significant factor in the risk index. Thus, a decrease in the variance of speeds leads to the lowering of the risk index. Figures 5-23, 5-24, 5-25 and 5-26 depict the 30 seconds speeds for base case and case 21 for stations 40-47. The figures show that there is considerable decrease in the variance of speeds throughout the simulation period.

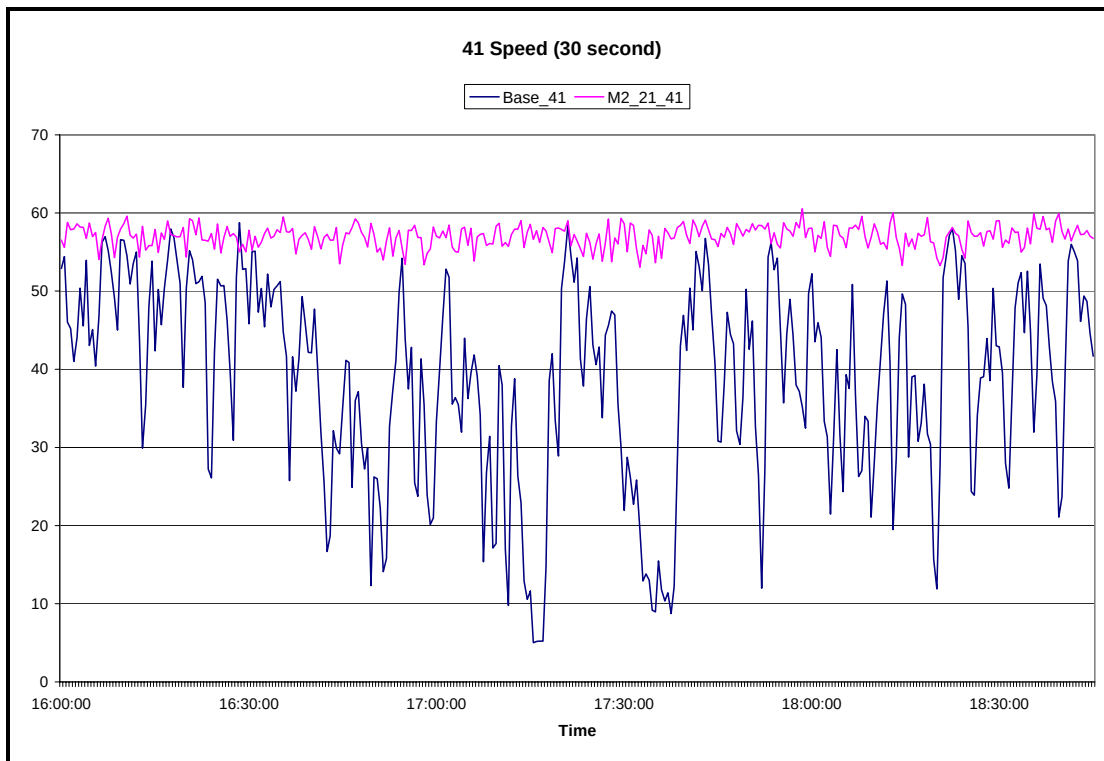
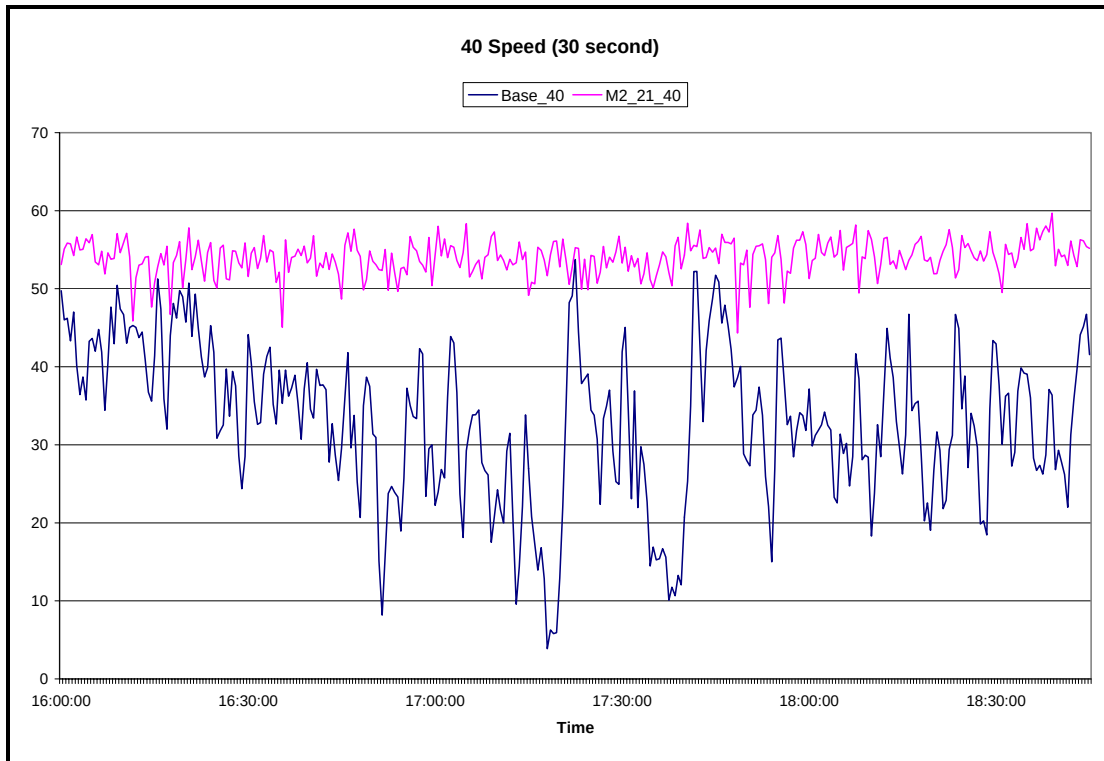


Figure 5-23 30 sec. Speeds Comparison Station 40 and 41

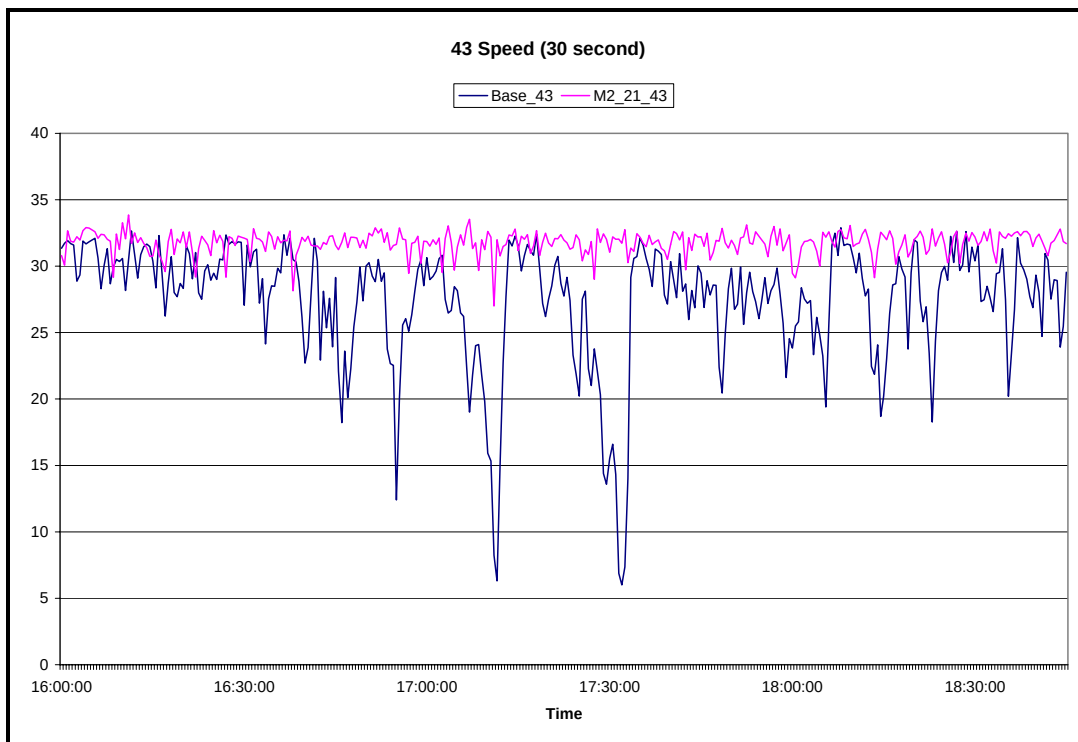
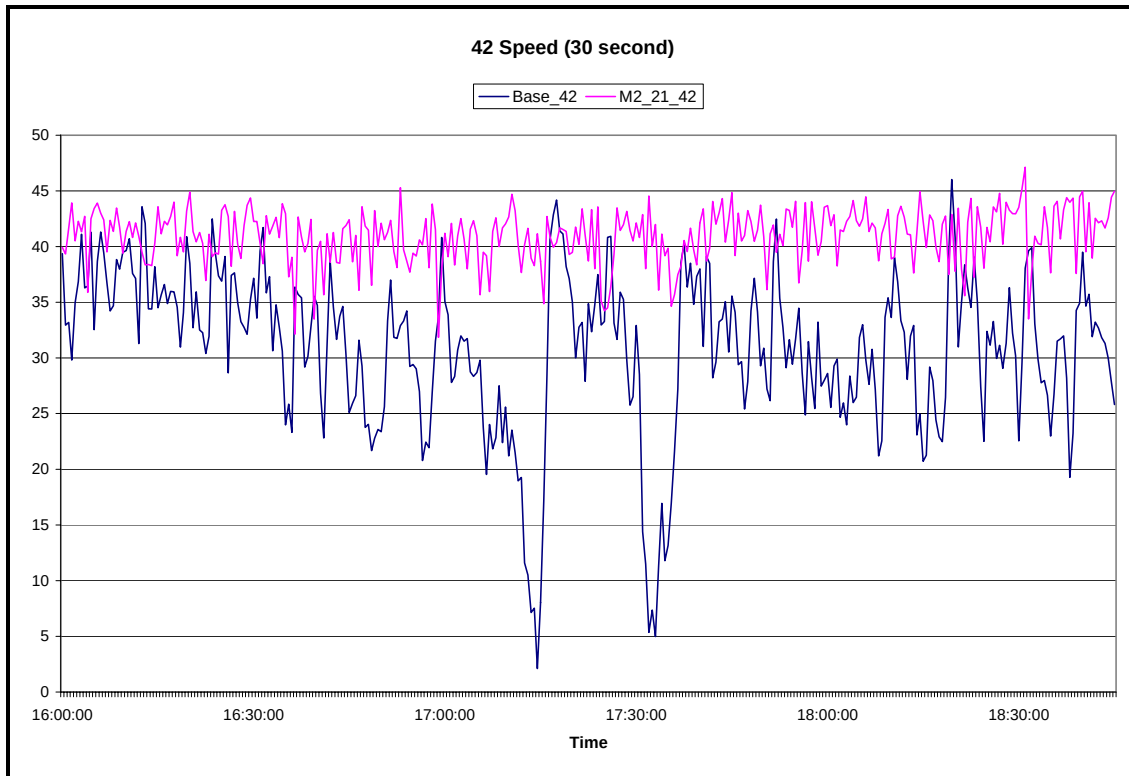


Figure 5-24 30 sec. Speeds Comparison Station 42 and 43

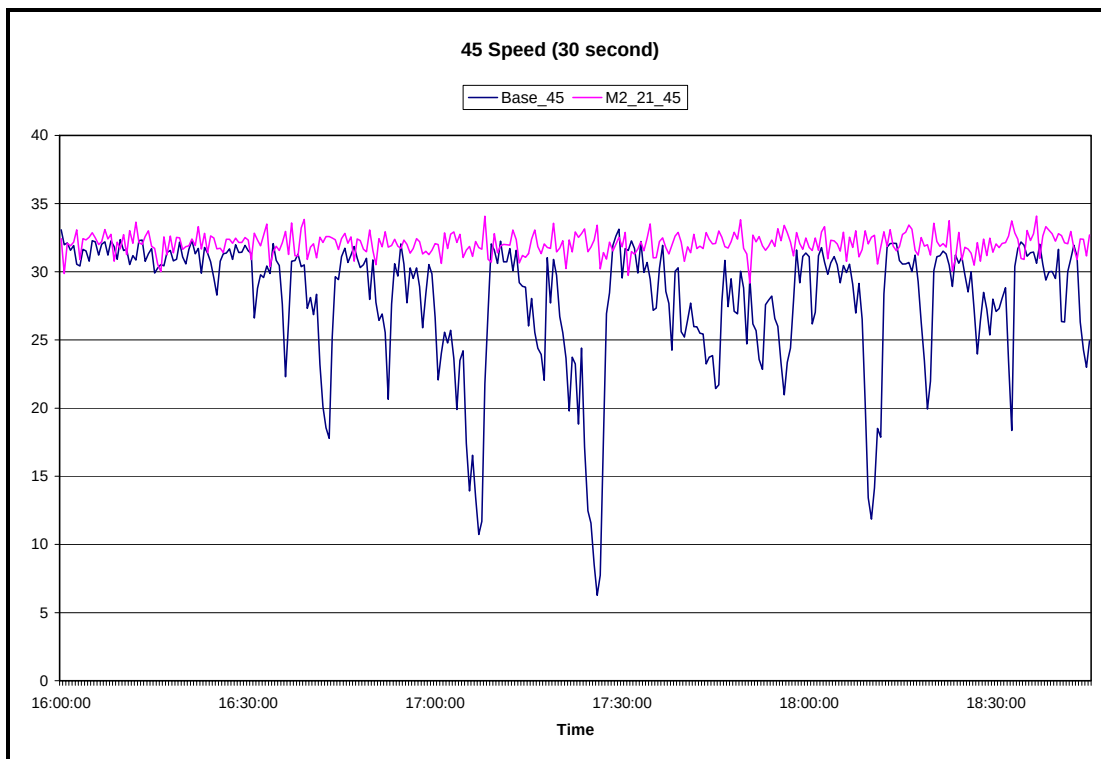
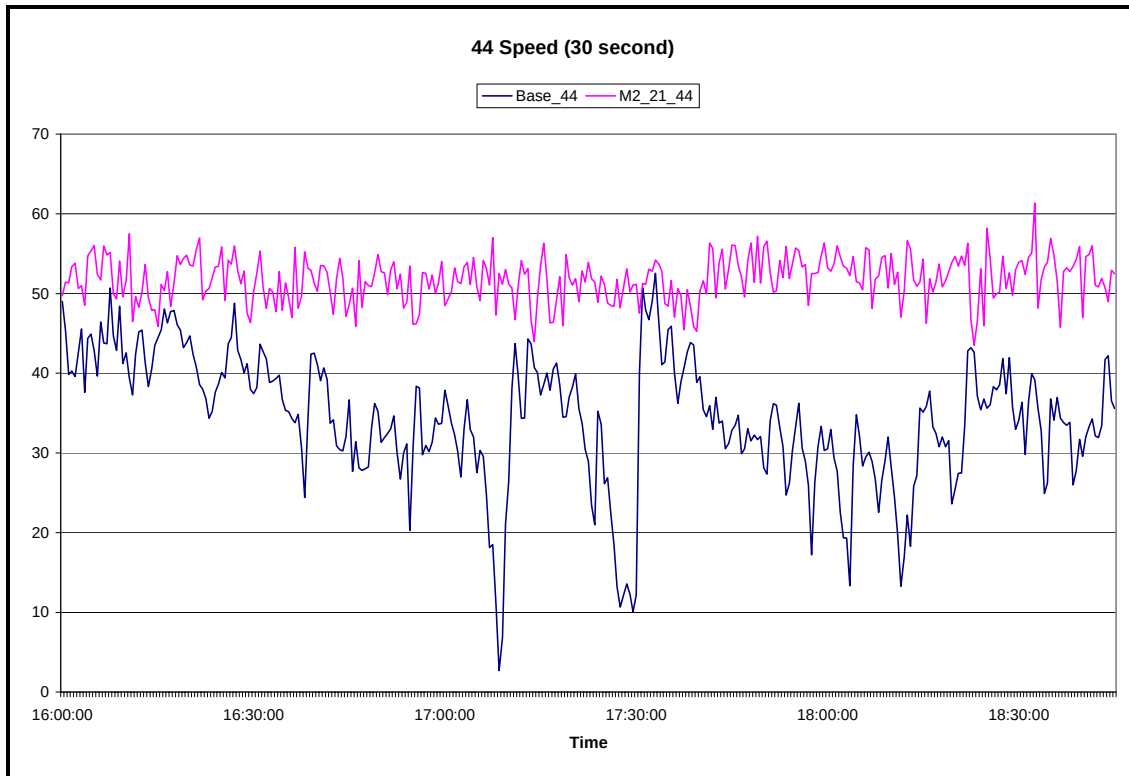


Figure 5-25 30 sec. Speeds Comparison Station 44 and 45

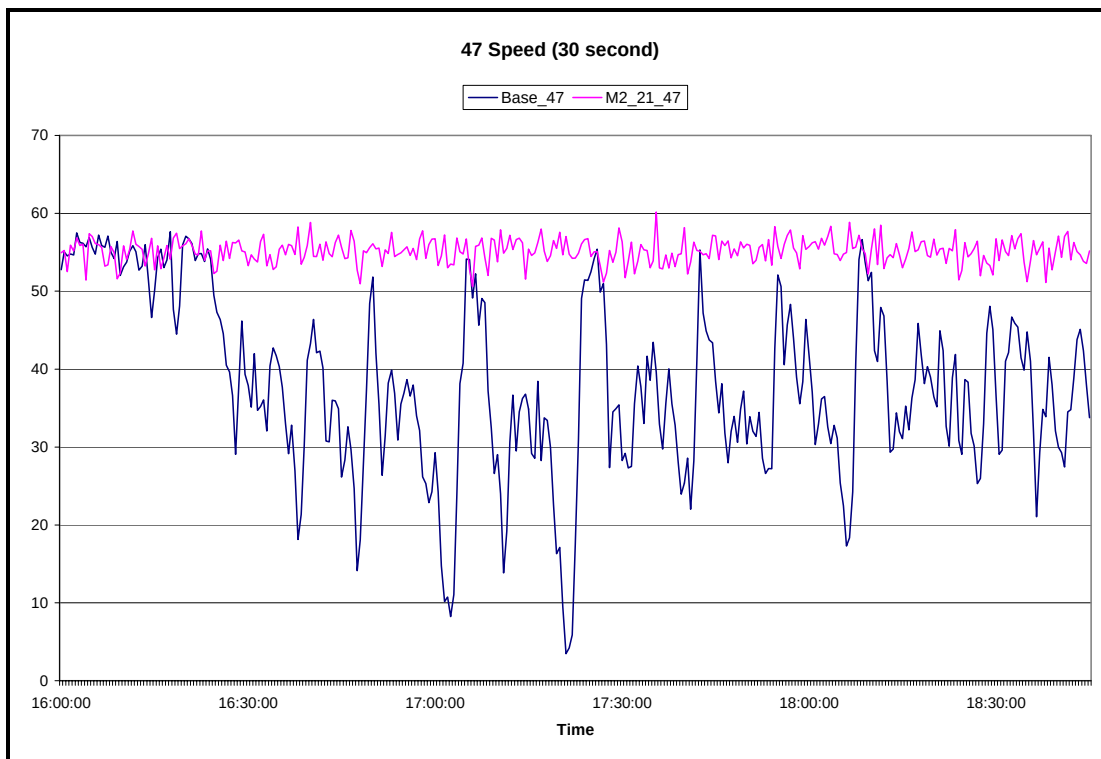
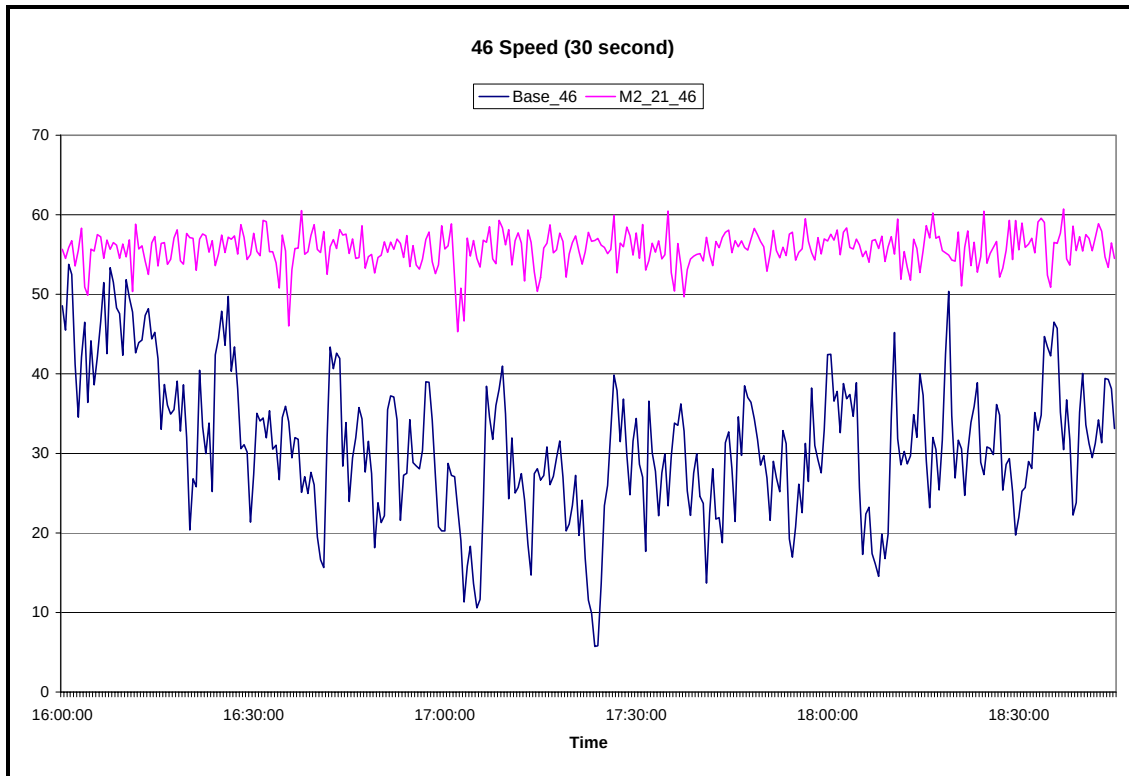


Figure 5-26 30 sec. Speeds Comparison Station 46 and 47

The other important factor in the index is the average occupancy upstream and downstream of station of interest in the 5-10 and 10-15 minute interval respectively before the time of interest. The log of average occupancy for the stations 42, 43 and 44 for the base case and best case (7 metered ramps, Case 21) are presented in Figures 5-27, 5-28 and 5-29.

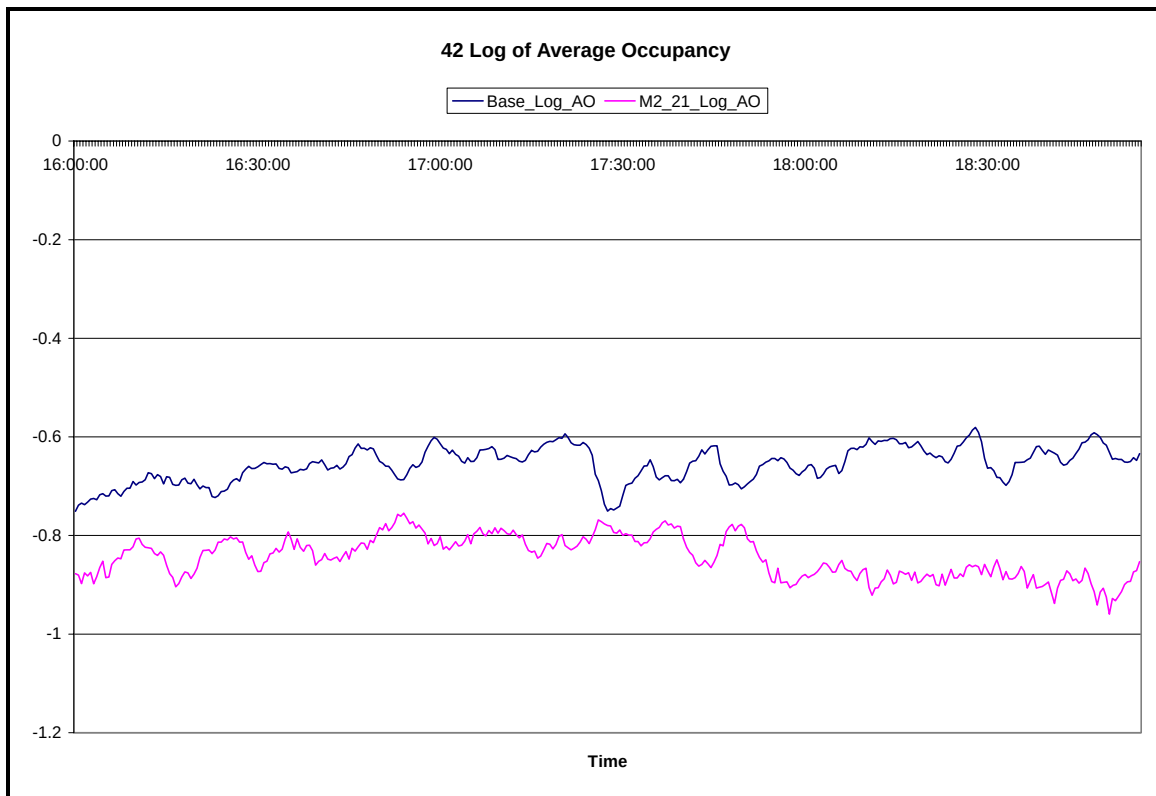


Figure 5-27 Average Occupancy- Base Case and Module 2 Case 21 Station 42

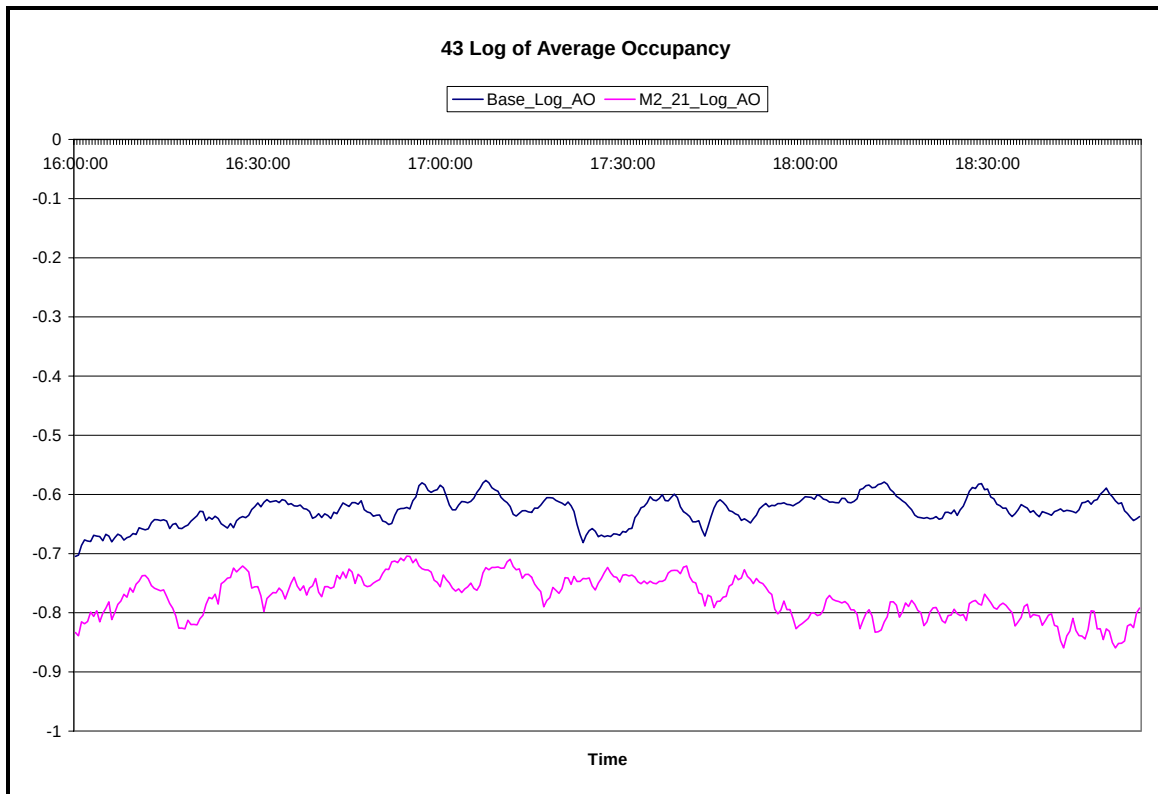


Figure 5-28 Average Occupancy- Base Case and Module 2 Case 21 Station 43

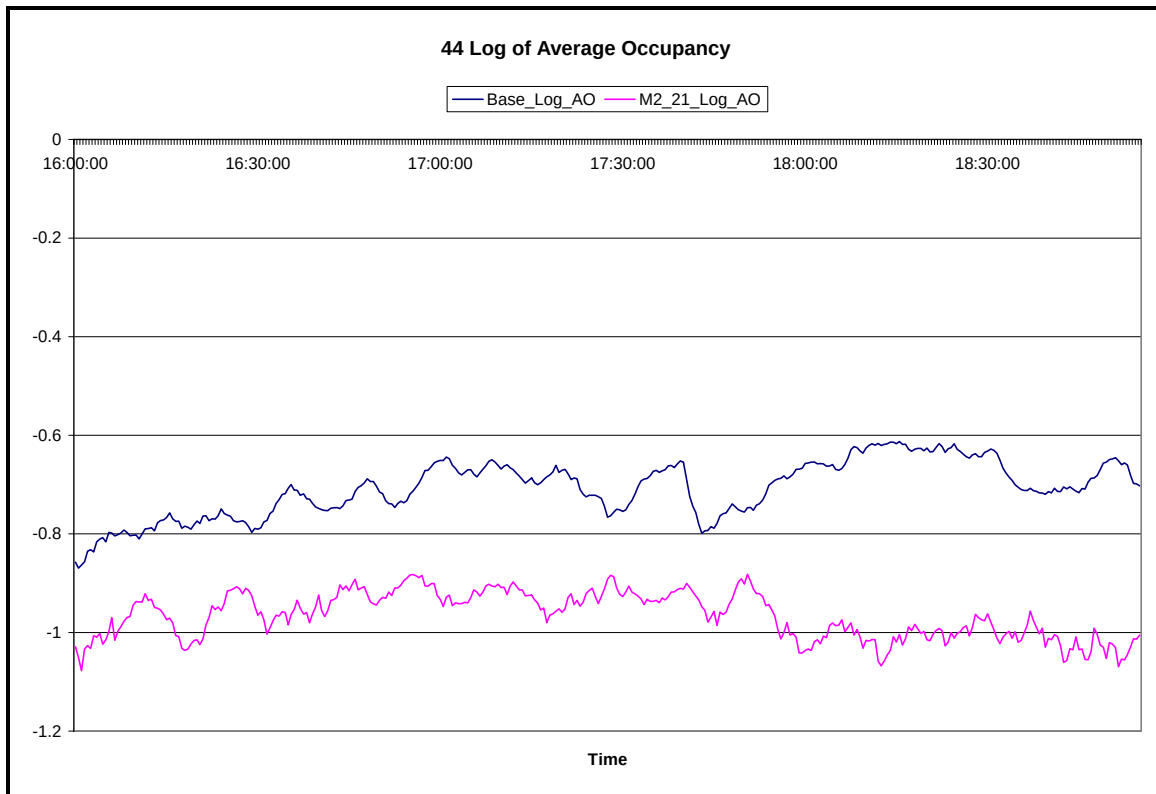


Figure 5-29 Average Occupancy- Base Case and Module 2 Case 21 Station 44

The Figures 5-27, 5-28 and 5-29 show that there is a consistent improvement in the average occupancy. It is clear that there is a decrease in the log of average occupancy which signals a decrease in the average occupancy. This drop further contributes to the decrease in the overall risk index as there are two terms with significant indices which are contributing in addition to a decrease in variance of speeds. Thus, a decrease in variance of speeds and decrease in average occupancy for us signals a reduced risk of crash.

These measures depict the crash risk in indicators that can be more intuitively understood as having a relationship with the occurrence of a crash. It is easy to see that even if these measures are not analyzed in relation to crash risk assessment only, the

changes in these measures show significant gains in terms of traffic parameters. As will be seen later, the positive affect on the speeds and variance of speeds shows up as a gain in the travel time of the network. The Risk Index captures the overall effect of the interplay of these factors and proves to be a very good surrogate measure for both safety and throughput perspective.

5.3.1 Effect of Number of Metered Ramps

It was crucial as part of our analysis to determine the best strategy that led to the greatest benefits at the network level and not just at local stations. As seen in Figure 5-12 above, even a single metered ramp can cause improvements in the cumulative index over a number of stations. However, this is not enough, because the spikes that persist as shown in Figure 5-13 would mean that there is still a significant risk of crash taking place although for a shorter period of time. Implementing multiple ramp metering locations was one way tested to introduce significant changes in Risk Index.

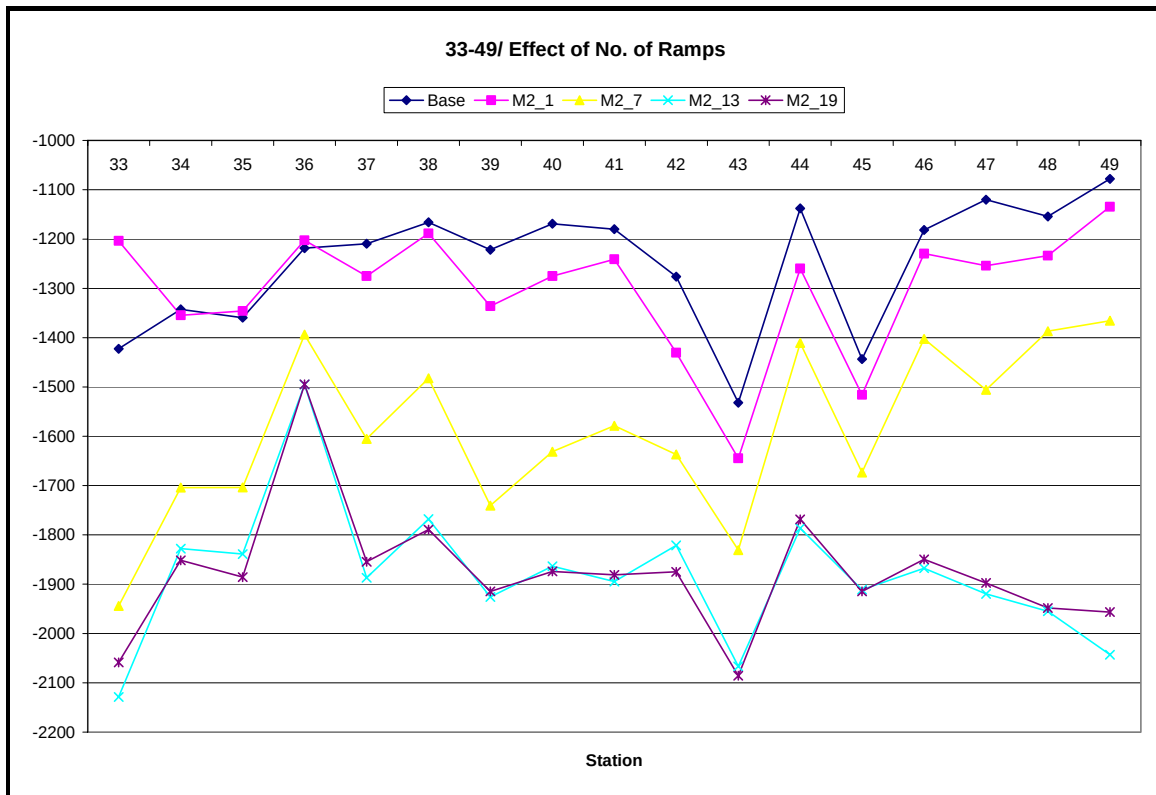


Figure 5-30 Module 2 Comparison for 1,3,5,7 metered Ramps

Figure 5-30 above shows that there is a significant improvement in MCI, the moment the number of metered ramps is increased from 1 to 3. The 2 additional ramps (upstream of stations 42 and 47) are chosen such that they are upstream and downstream of the single ramp (upstream of station 43) we were analyzing before. As the number of metered ramps was increased there was a further improvement in the risk index. The cases were compared in a controlled manner, such that all other factors were kept constant. The results were same from any combination of controlled factors leading us to the conclusion that in fact multiple ramps are better at mitigating risk as compared to single ramps. The best cases were deliberately not used to show that the effect of number of ramps is independent of other parameters. The matched cases for best cases would however, also lead to the same results. Importantly, amongst the cases compared, 5 ramps

and 7 ramps cases performed almost equally well with shorter signal cycles. This was observed mainly because, the last two ramps are added very close to the ends of the network and thus do not seem to have a very significant effect on the network as a whole. It can therefore be deemed important to choose locations which are in the middle of our network of interest to test for the strategies related to ramp metering. In an expanded network, the results of metering 7 ramps would outperform the result of metering 5 ramps in terms of MCI. The effect of multiple locations being metered was also found to be statistically significant and will be analyzed in Section 5.3.4.

5.3.2 Effect of Signal Cycles on Metered Ramps

For the single ramp metered case, we had found that although higher critical occupancy provided best results, so did longer signal cycle. However, as we move from single to multiple ramps, at each step the observation is that the shorter signal cycle provides the best results. The cases performing best for multiple locations being metered are Cases 9, 15, 21. All these cases perform best within their sub-modules if they compared with other cases which had the same number of metered locations. Thus, the main conclusion we can draw here is that shorter signal cycles perform much better in a network wide set-up although longer cycle lengths work better when a single ramp is being metered.

5.3.3 Effect of ALINEA Parameters

The two crucial parameters from ALINEA which were tested were:

- Critical Occupancy O^*
- Regulator Parameter K_R

It was consistently found that higher critical occupancy values performed better. All the best cases were found to be for Critical Occupancy Values of 23%. The best combination of parameters was shorter signal cycles with high values of critical occupancy. When cases with low signal cycle lengths viz. 25 seconds were compared within each other, it is obvious from Figures 5-14, 5-15, 5-16 that critical occupancy of 0.23 outperforms the others.

Regulator parameter was found to have no significant effect on the scenarios.

5.3.4 Confidence Interval for Random Variation

It was crucial to determine that all the differences in the alternative scenarios were not random variations due to the seed values and the benefits were statistically significant. As such a confidence interval was created using the data from 20 different seeds for the base case. A t-statistic with 95 % confidence was used to calculate a confidence interval which for each station and is provided in Table 5-1. Each of the successful strategies was compared with this confidence interval to determine that the changes were indeed not random variations.

Table 5-1 t-test for confidence intervals from 20 Base Case Runs

Station	Base(Mean)	Minimum	Maximum
33	-1422.62	-1560.73	-1284.51
34	-1342.46	-1521.24	-1163.68
35	-1359.3	-1497.17	-1221.43
36	-1218.24	-1391.71	-1044.78
37	-1209.53	-1423.84	-995.215
38	-1165.92	-1319.68	-1012.17
39	-1221.66	-1320.51	-1122.81
40	-1168.79	-1261.96	-1075.62
41	-1179.78	-1280.54	-1079.01
42	-1276.3	-1369.36	-1183.25
43	-1531.95	-1685.5	-1378.41
44	-1137.54	-1214.82	-1060.27
45	-1443.38	-1568.54	-1318.23
46	-1181.73	-1266.58	-1096.88
47	-1119.97	-1198.34	-1041.6
48	-1153.91	-1264.42	-1043.4
49	-1078.07	-1171.26	-984.872

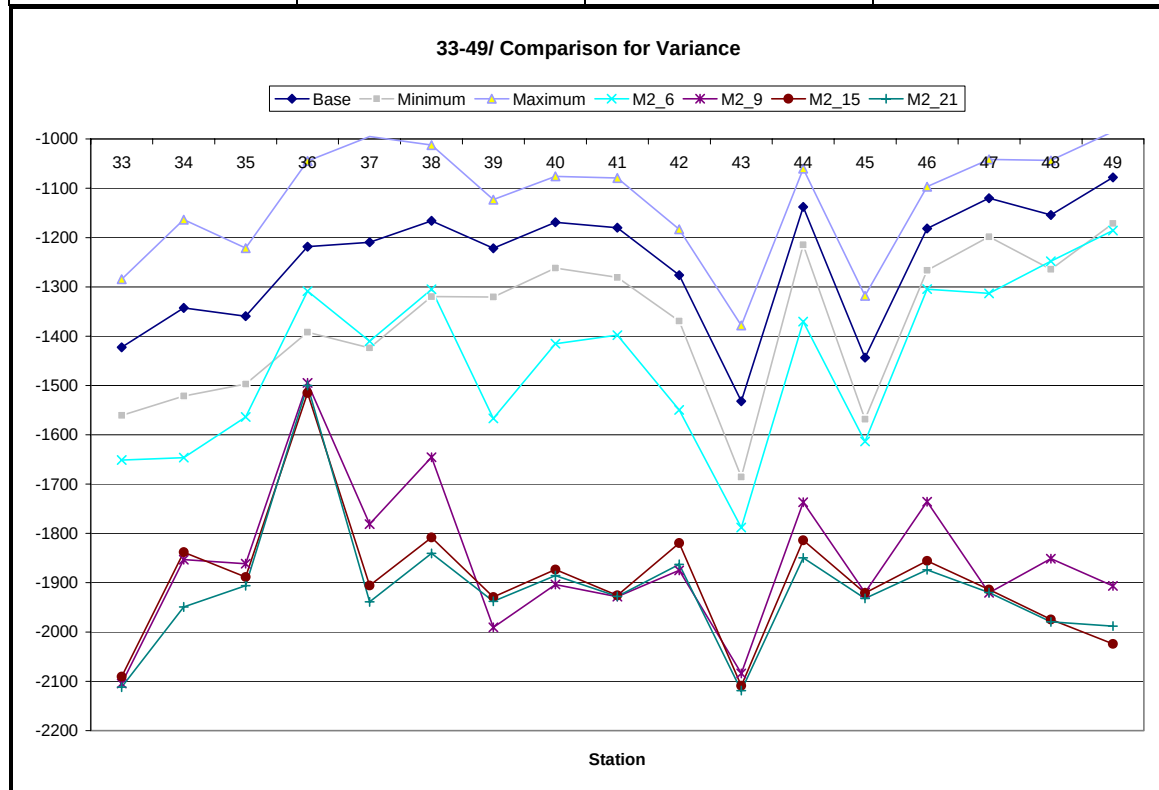


Figure 5-31 Test for Statistical Significance of Best Strategies

Figure 5-31 shows that all the successful implementations lie well outside the range of variation of the base case. As such we can safely conclude that all the strategies are effective as they are significantly different from the base case values.

5.3.5 Quantification of Benefits

A methodology for quantification of benefits from any strategy was defined in Section 4.5. According to that methodology, the areas were calculated between two time series curves and the total area for all the stations summed provided a measure of benefits provided by the strategy. Figure 5-32 depicts the area that would mark the safety benefits for station 43 for Module 2 Case 21. The shaded area represents the safety benefits for Case 21 at station 43. The value of these benefits was found to be 585.83. A similar procedure was used to calculate the benefits at all the other stations. A sum of these benefits (areas) for all the stations for a test case and base case pair would mark the safety benefits for that particular scenario.

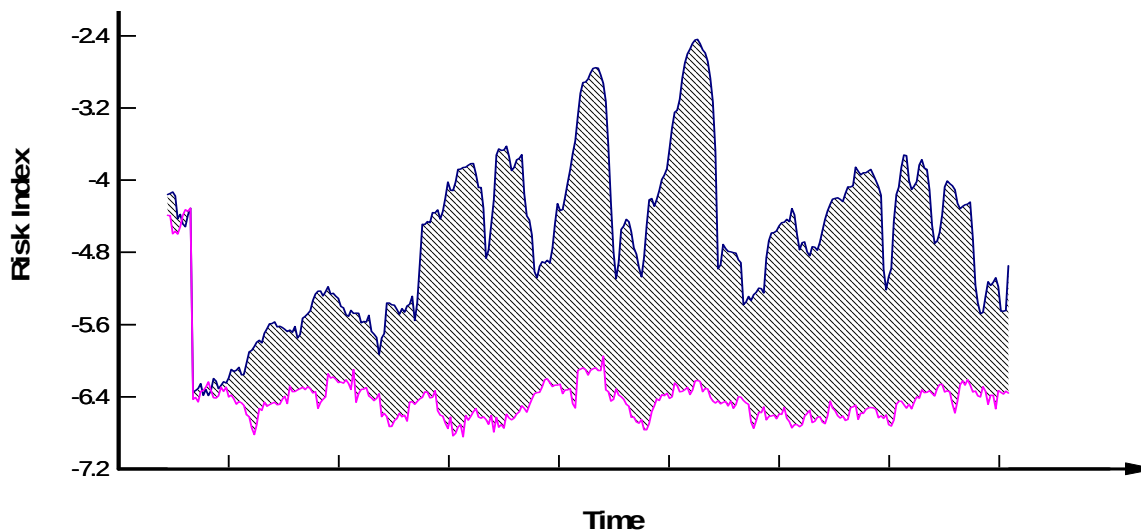


Figure 5-32 Risk Evolution for Base Case and Module 2 Case 21 Station 43 EB

Table 5-2 provides the station wise break up of the areas for curves corresponding to Case 21, identified as the best case in our analysis. This measure would serve as crucial indicator in identifying the best over all strategy. As can be seen from Table 5-2, each station shows significant safety benefits in the case where sevens ramps are metered.

Table 5-2 Area between x-axis and Base Case/ Case 21

Detector	Area Base-X	Area Case21-X	Station Benefit
33	-1321.47	-2104.88	783.4104
34	-1338.26	-1943.92	605.654
35	-1355.35	-1900.55	545.1948
36	-1214.82	-1496.05	281.2225
37	-1206.17	-1933.62	727.4552
38	-1162.59	-1834.99	672.3991
39	-1217.86	-1932.65	714.7853
40	-1164.55	-1880.83	716.2819
41	-1176.3	-1923.16	746.866
42	-1272.1	-1859.18	587.0803
43	-1527.09	-2112.92	585.8356
44	-1134.17	-1844.58	710.4139
45	-1438.9	-1926.66	487.7566
46	-1178.07	-1868.86	690.7929
47	-1115.92	-1914.41	798.4819
48	-1149.82	-1974.32	824.4998
49	-1074.12	-1982.66	908.548
Total	-21047.6	-32434.2	11386.68

The total benefit over base case for Case 21 came out to 11386.68 as can be seen in Table 5-2. The Safety Benefit index comes out to be 0.53 and is provided in Table 5-3.

Table 5-3 provides the benefits for the better performing strategy in Module 2.

Table 5-3 Case Wise Areas and Benefits for Module 2

Case	Area under Curve	Area b/n Curves	Benefit Index
M2_1	-22061.82892	912.9289237	0.043166733
M2_2	-21724.70458	575.8045791	0.027226219
M2_3	-21584.41023	435.5102281	0.020592571
M2_4	-23925.89139	2776.991385	0.131306658
M2_5	-23041.09894	1892.19894	0.089470324
M2_6	-24562.39571	3413.49571	0.161402991
M2_7	-26920.37061	5771.470609	0.272896964
M2_8	-30483.16441	9334.264406	0.441359333
M2_9	-25111.53805	3962.638048	0.187368518
M2_10	-23133.12718	1984.227176	0.093821767
M2_11	-23955.51606	2806.616064	0.132707425
M2_12	-25111.53805	3962.638048	0.187368518
M2_13	-31916.28168	10767.38168	0.50912254
M2_14	-32087.49327	10938.59327	0.517218071
M2_15	-32119.97422	10971.07422	0.518753893
M2_16	-25271.70075	4122.800745	0.194941616
M2_17	-25383.52037	4234.620372	0.200228871
M2_18	-26777.33874	5628.438742	0.266133877
M2_19	-31816.26006	10667.36006	0.504393139
M2_20	-32400.38473	11251.48473	0.532012763
M2_21	-32438.17133	11386.68133	0.533799457
M2_22	-25446.14301	4297.243007	0.203189906
M2_23	-27085.76812	5936.868122	0.280717584
M2_24	-29140.06992	7991.16992	0.377852745

The Safety Benefit Index (SBI) provides a summation of results in comprehensible terms. It is a linear index greater than zero. It is the ratio of the safety benefit for a case and the total area under the base case scenario for the network (Refer Section 4.5). The interpretation of the index is that, any benefits would be signified with a positive number. A value of 0 specified no improvement. A negative value signifies a worsening in conditions. A value of 1 signifies an improvement in cumulative MOE of a 100%. In simple terms, a positive number means benefit and larger the number, the greater is the benefit of the strategy over all the stations. Table 5-3 shows, the Safety Benefit index is always greater than zero for the cases of feedback ramp metering. Cases

14, 15, 20 and 21 provide the best four values for the Safety Benefit Index. These indices are close and this follows the trend discussed before about the small marginal benefit of metering 7 instead of 5 ramps in this study section. Figure 5-33 depicts the Safety Benefit Indices for all the cases in Module 2. This is a graphical representation of the results from Table 5-3.

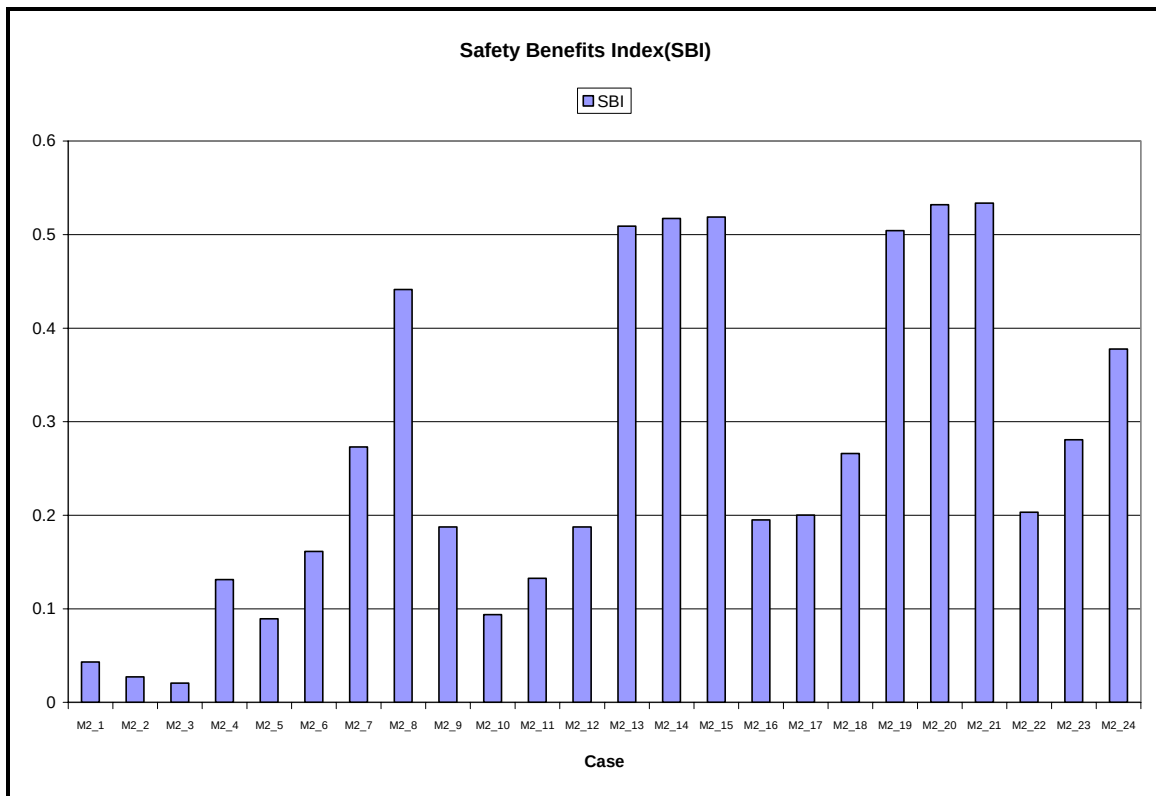


Figure 5-33 Case Wise Benefits for Module 2

Station wise benefits could also be compared using just the safety benefits for each station. From an analysis of Figure 5-33, it is clear that the maximum benefits are provided by cases 13, 14, 15, 19, 20 and 21, with case 21 providing the best results. The quantification provides an index which could be used for comparing even the benefits across the stations. This is because the unit difference in the risk index at every station

can be compared to the unit difference in risk index at any other station. Figure 5-34 gives the benefits- station wise- for the best and worst cases viz. Case 21 and Case 3. To provide a scale for comparison, the station wise benefits of Case 21 is compared with the benefit from the case with only one metered ramp and occupancy of 23 % and signal cycle of 25 seconds, i.e. Case 3 in Figure 5-34.

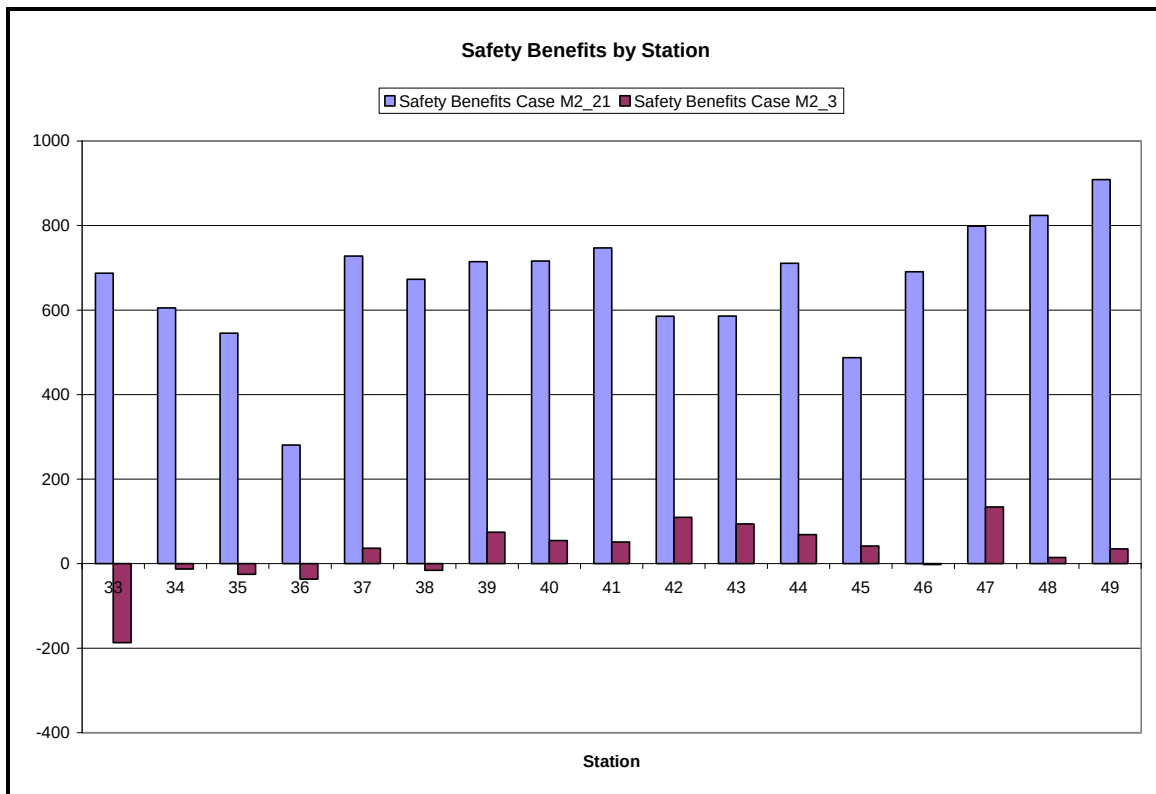


Figure 5-34 Station Wise Safety Benefits for Case 21 and Case 3

Figure 5-34 show that network wide metering provides benefits through out the corridor whereas local metering provides benefits only at some stations and the risk seems to migrate to other locations. In Figure 5-34 above, it appears that metering only at Station 43 leads to an increase in risk (negative benefits) at stations 33, 34, 35, 36 and 38 upstream of the location of metering. Another significant observation here is that there

are benefits downstream of the local metering location (Station 43) but upstream stations do not see much benefit (there is an increase in risk) as we move away from the metering location.

Figure 5-35 illustrates the station wise benefits for Case 22, which has the seven metered ramps but with non optimal values, and it shows that there is consistency in observation of benefits at all stations compared to Case 21 but the only difference is the magnitude of these benefits.

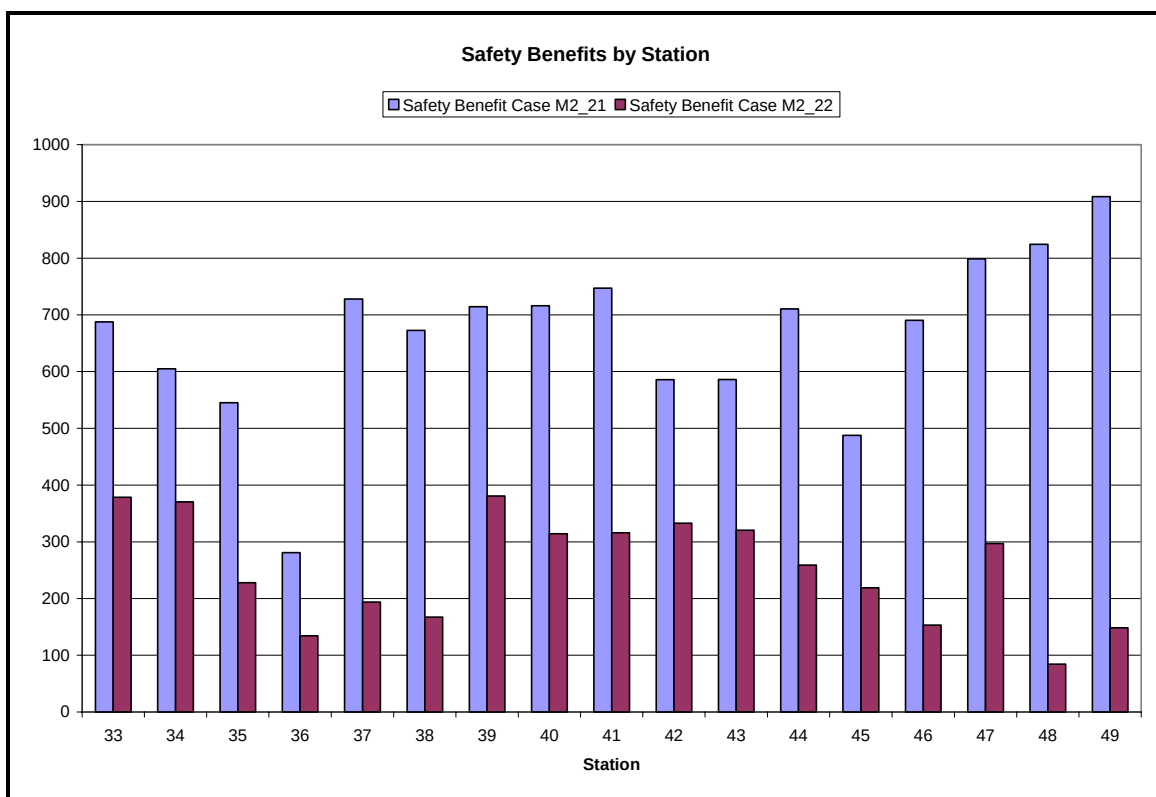


Figure 5-35 Station Wise Safety Benefits from Case 21 and 22 Module 2

5.3.6 Travel time analysis

It is crucial for analysis that our measures of intervention do not have an adverse effect on any normal traffic movement. As was seen in the analysis of speeds for station

43, there is a significant difference in the variation of speeds. This also leads to an increase in speeds through out and lesser queue formation when the network is observed. The effect of this change is also seen when an analysis is done for the travel time. A cumulative travel time measure is calculated for the mainline. The travel time measure is calculated using 5 minute speed averages at all the links on the mainline and the total sums are measures for the period of the simulation, as shown in Equation 5-5.

$$TT = \sum_{j=1}^{17} \sum_{i=1}^{30} (l_i) / s_{ij}$$

...Equation 5-5

Where TT – Travel Time Measure

i – Detector number (33-49 EB)

j – Number of 5 minute observation (1-30 for 4:30 PM-7:00 PM)

l_i – Length of link corresponding to detector i

s_{ij} – Average speeds over five minute interval j at detector i.

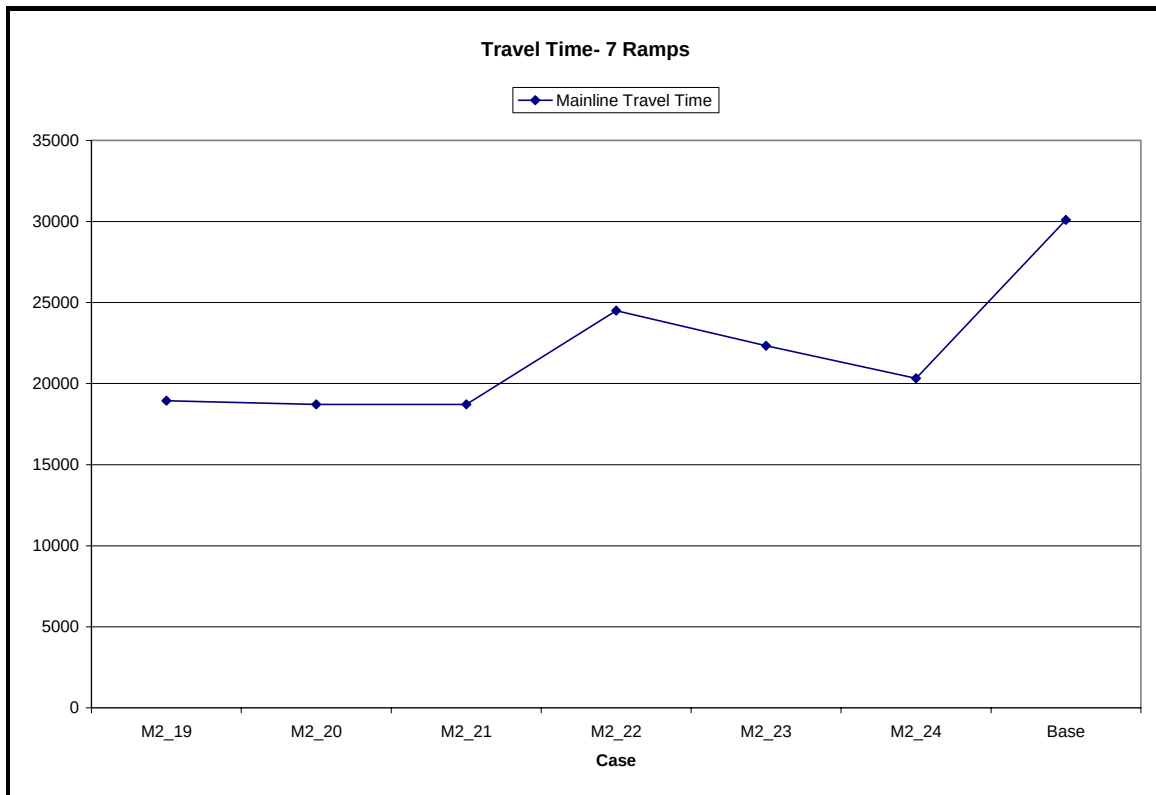


Figure 5-36 Case 19-24 – Base Case Mainline Travel Times

Figure 5-36 shows these travel time measure for the six cases when 7 ramps in the network were metered and the Base Case. The travel time measure for cases 19, 20 and 21 are much lower than the travel time measure for Base Case indicated by the higher points in Figure 5-36. The results have the same pattern as the results for Mean Cumulative Index (MCI) and case 21 again presents the least travel time.

Although it would seem that due to ramp metering the overall travel time on the network would go up, due to delays at the ramps, it was not found to be the case in our best scenario of ramp metering. PARAMICS provides a measure of average travel time for all the vehicles in the network every minute of the simulated run. All the average travel times at all times in the simulation were cumulated for the network to come up with a Total Average Travel Time. This measure for base case was 154925.6 seconds and

went down to 146285.6 seconds for our Case 21 which included 7 metered ramps. This measure included the westbound network where no ITS measures were being implemented. A look at the average network wide speeds also showed that the speeds in our test case were slightly better (a 5 % decrease). However, the fact that this also includes the westbound corridor of the network does affect the difference to a certain extent. The reason for using the network wide measure that included the west bound corridor also was to bring out a significant contrast. The network wide travel time measure includes the delays, the vehicles would encounter on the ramps which are metered also. Hence a decrease in these network wide travel time shows that despite the increased delays on the ramps encountered by vehicles, the overall mainline conditions improve significantly enough to counter these delays and improve over all travel times.

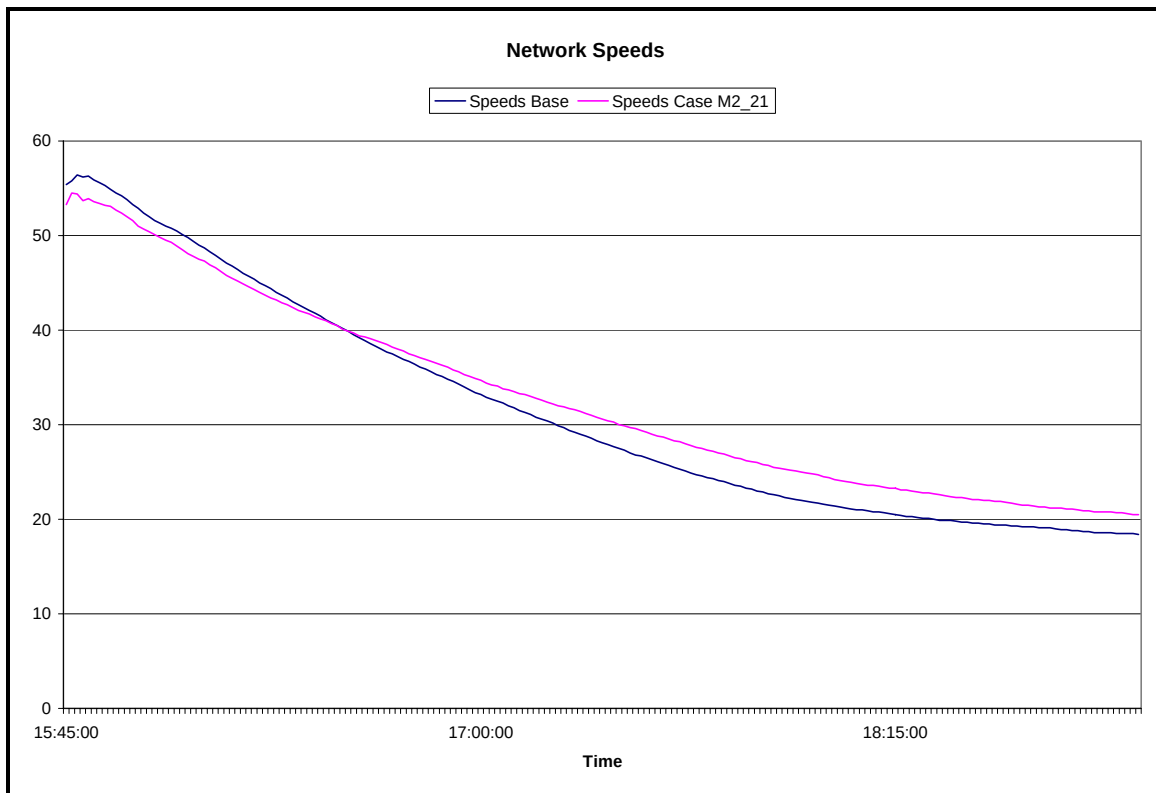


Figure 5-37 Average Network Speeds

5.3.7 Conclusions for Module 2

The main conclusions for this module are as follows:

- Safety benefits of metering increase with increase in number of meters
- Shorter signal cycles work better in multiple ramp implementations
- Longer signal cycles work better in single ramp implementations
- Higher values of critical occupancy ensure better benefits.

5.4 Results for Module 3

The module 3 includes the setup of cases meant to test the effectiveness of Variable Speed Limits in Low Speed Scenarios and their effect on Ramp metering. It is also of importance to work out the effectiveness of Variable Speed Limits, when used in conjunction with ramp metering. A coordinated strategy between both might provide greater benefits than each of them would be able to provide separately.

5.4.1 Methodology for Variable Speed Limits

In order to test each of the possible strategies, an experimental design was formulated as shown in Section 4.4. In Chapter 3 we reached the conclusion that Variable Speed limits when implemented in the case of low speeds do not show a significant change in the risk of crash. The strategy for implementation in the Variable Speeds had been mainly towards arriving at a general implementation strategy in a single station. The scope of this research in terms of network and implementation allows us to test the strategy at different locations and this issue will be addressed in this section. The

Application Programming Interface in PARAMICS was used to implement variable speeds. However, Variable Speed Limits in PARAMICS were implemented in a significantly different way compared to Chapters 2 and 3.

Firstly, the biggest difference is the scope of implementation. The said research used only a single location and different cases were tested upstream and downstream of a particular location. This chapter, however, extends the implementation to a 9 mile section which included 17 detector stations. Secondly, there is no preset time for changing the speed limits on the network. In previous research a pre-specified change in speed limits was made at particular times and its effect observed. The changes were also made to pre set speed limits e.g. if a particular section has a posted speed limit of 50 mph, then its speed limit would be changed from that value. The timing of the speed limit change is dependent on the conditions on the freeway. All the stations within this section operate at low speeds in the period of simulation. The algorithm that has been implemented operates at every 5 minutes at all the detector stations within our area of implementation (33 – 49, East Bound).

The objective is not just to use a strategy that affects a single location but also smoothes speeds over the whole network. This also presents another challenge in implementation. All strategies have to be such that they can be applied logically at all the locations, e.g. there can't be large increases in speeds at any location as it would mean problems downstream of that location. The stations within this range have lower speeds, so basic implementation was tested for two increments, i.e. 5 mph speed changes and 10 mph speed changes. The algorithm accumulates speed data from the loop detectors for every 5 minute period in the simulation. At the end of this 5 minute period, each loop

detector is analyzed for speed differences at the stations upstream and downstream of it. Thus for loops 34 to 48, this process is run every 5 minutes. VSL is considered if there is a negative difference in speeds for the upstream and downstream detector, i.e. vehicles are faced with lower speeds at the downstream location. If a speed differential of greater than 5mph exists at two consecutive detectors, then the five minute average speeds are classified into one of the four categories:

$$\text{Average 5 min speeds=} \left\{ \begin{array}{l} \leq 35\text{mph} \\ 35\text{mph} < \text{speed} \leq 45\text{mph} \\ 45\text{mph} < \text{speed} \leq 55\text{mph} \end{array} \right.$$

Thereafter depending upon the speed conditions at the detector, four different speed limit changes were tested. However, in any of the cases the drop or increase in speed was never greater than 10mph. In case a particular location had two different requirements e.g. a downstream condition requires a 5 mph speed drop and an upstream condition requires a 10 mph speed increase at the same location, an average is taken, i.e. a 5 mph increase is implemented. This is very important because this research is not looking at a single location to implement speed limit changes, but the variable speeds would be activated any time there is a speed differential at any location. If the speed differential disappears, the speeds continue in the same configuration.

In first phase of the preliminary exploration of VSL (Chapters 2 and 3), we used the concept of sudden and gradual speed changes both in terms of speed change and the distance over which they are implemented. The main conclusion from that study being

that sudden speed changes of 15 mph implemented over a very small distance. Such a strategy presents some practical limitations when seen from a field implementation point of view. A sudden 15 mph decrease in speeds might cause the drivers to break hard. As such, keeping in mind an integrated system over the whole network, the speeds were changes such that a 5mph drop is carried out every half a mile except for downstream increasing. A speed of 10mph would be therefore implemented over 1 mile distance, except when the speed limits are being increased. In that case a 10 mph change is allowed over half a mile. This ensures that there are no sudden changes with which the drivers have to deal with over short distances. The strategy of multiple implementations depending upon real time conditions thus has applicability in moderate to high speed conditions too.

This research deals with four different strategies for changing speed limits which are:

- S1: Decrease Speeds Limits Upstream of a location and Decrease them downstream
- S2: Decrease Speeds Limits Upstream of a location and Increase them downstream
- S3: Lower Speed Limits Upstream only
- S4: Increase Speed Limits Downstream only

These strategies are also tested in conjunction with Ramp Metering. This goes farther than the work described in Chapters 2 and 3; where either one of them (VSL or Ramp metering) was used one at a time.

When talking of decreasing speed limits upstream by 5 mph, it means decreasing speeds at the link corresponding to the loop detector upstream of the one being evaluated.

If there is an upstream decrease of 10 mph, it is implemented as a 5 mph drop at two stations upstream of the location of interest and another 5 mph drop at one station upstream of the location of interest. Increasing speeds downstream was also done in the same manner.

Lee et al. (2004) were closer to this study in that they used real time speeds at downstream locations of a single detector, as the target value to reduce speed limits to a transition value. Their basic objective was to minimize the speed differential between upstream and downstream locations. However, their study also looked at a single station and a single strategy (that of lowering speed limits). In Chapters 3 and 3, we looked at different strategies on an extended network but again did not consider implementing a strategy over the whole network, i.e., at multiple detector locations. This meant that only a single location became the location of interest and the rest of the network was not considered. This also led to crash risk migration to a different location (Abdel-Aty et al., 2006). Besides, in Chapter 2 and 3 we primarily found solutions for freeway locations with moderate to high speeds and not for the low-speed scenario.

The 24 cases that resulted from this experimental design could be split into three main categories. The first 8 cases, only variable speed limits were implemented over the whole section without using any ramp metering. The cases 9-16 ramp metering was implemented at 3 ramps and variable speed limits were implemented over the whole section. In the last 8 cases, from 17-24, Ramp Metering was implemented over 7 ramps and variable speed limits were used in conjunction with the ramp metering. Each of the 8

cases tested the different strategies for variable speed limit implementation making use of 5 mph and 10 mph speed increments and decrements.

Figure 5-38, shows the results from the first 8 cases over the whole section. The implementation seems to cause a significant difference in the MCI over the whole network which marks a significant change over the conclusion made in Chapter 3, in that Variable Speed limits do not affect low speed stations. A network wide implementation does improve the risk over the whole section, although this change is much smaller when compared with the network wide implementation of Ramp Metering and the effect of that on Risk Index. Individual Stations show changes in their risk propagation with time although there are significant highs and lows associated with the test scenarios also.

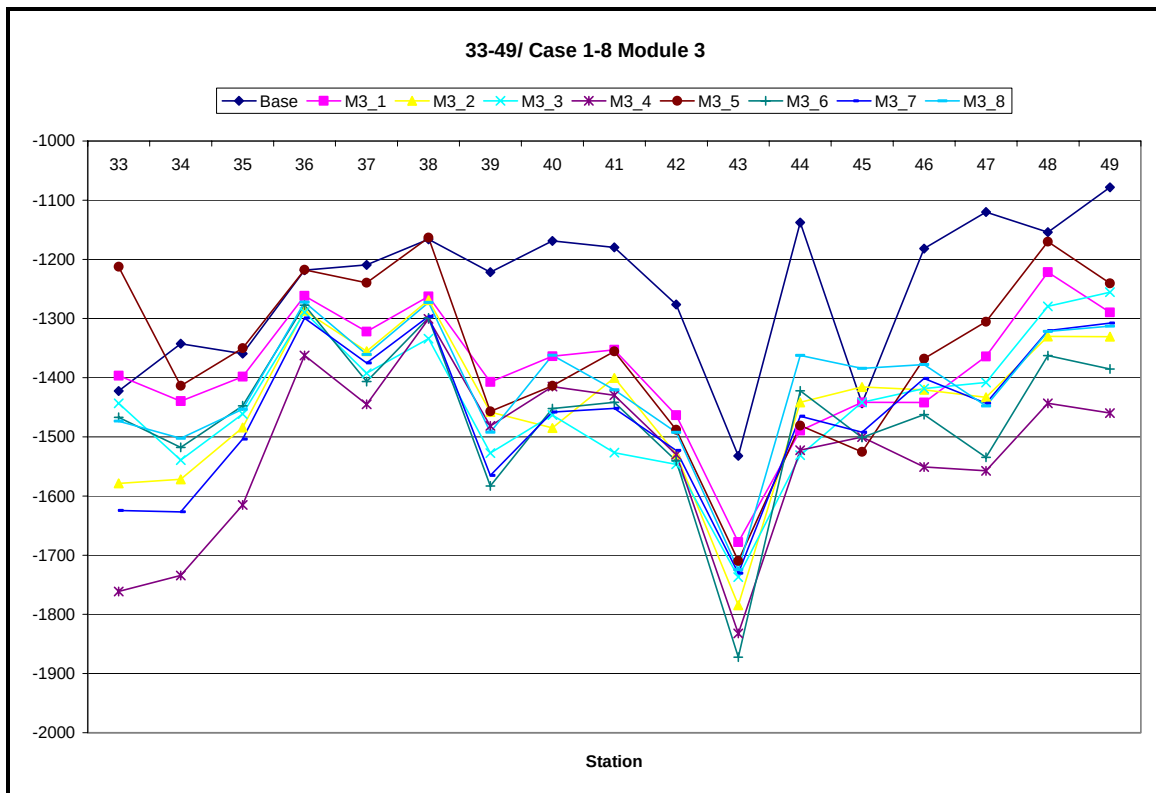


Figure 5-38 Module 3 MCI Cases 1-8 Station 33-49

The key here is to look at the speed patterns over a number of stations to determine if the variable speed limits are decreasing the variance in speed like coordinated ramp metering did. The best case from Figure 5-38 above is Case 4 (no metering, 5 mph speed increments downstream of location of interest only) which outperforms the others at all the stations. Figures 5-39, 5-40 and 5-41 show over six stations the risk index evolution over time for matched seeds of base case and the best case from low speed implementation of VSL. These results conclusively show that there is a definite improvement in the risk index but the improvement is not without time intervals when there are spikes (increases in risk index over short time intervals) in the risk indices. These spikes can be seen in Figures 5-41 for station 45 where there are period of times when the risk index for Case 4 is above the risk index for base case. However, the risk indices from stations 43, 44 and 46 in Figures 5-40 and 5-41 show that for most of the times, the risk index for case 4 is well below the base case.

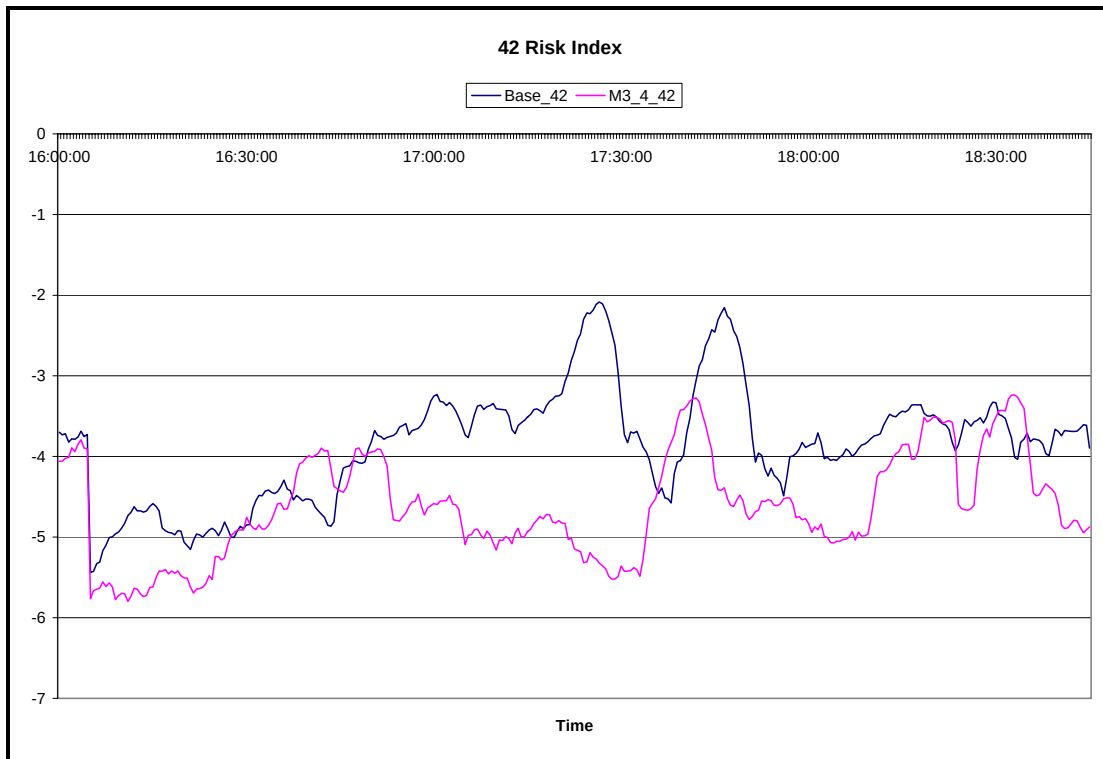
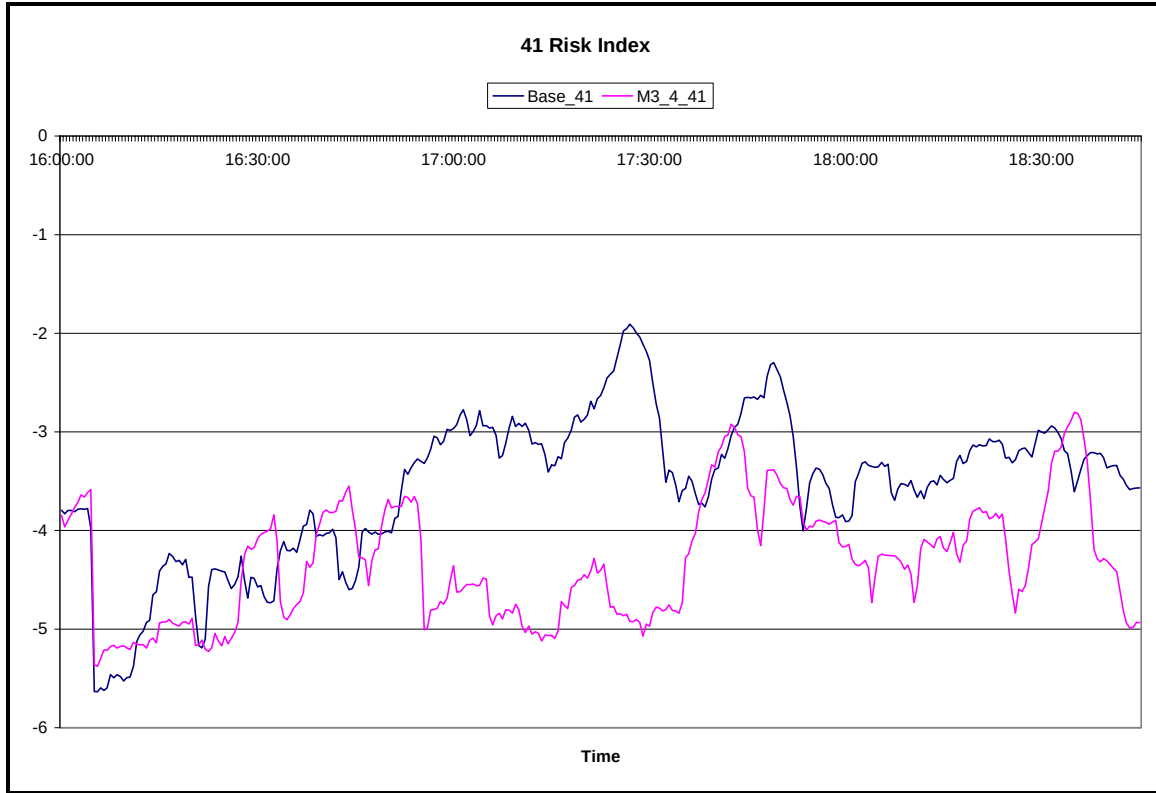


Figure 5-39 Risk Index Evolution Comparison Station 41-42

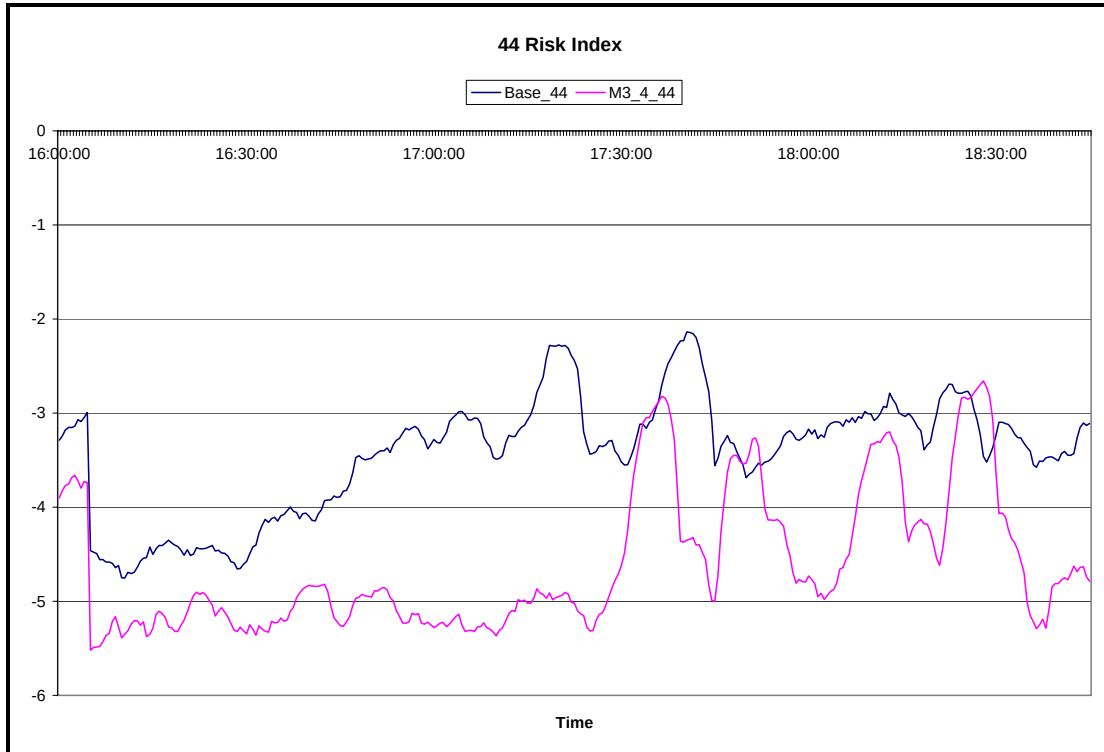
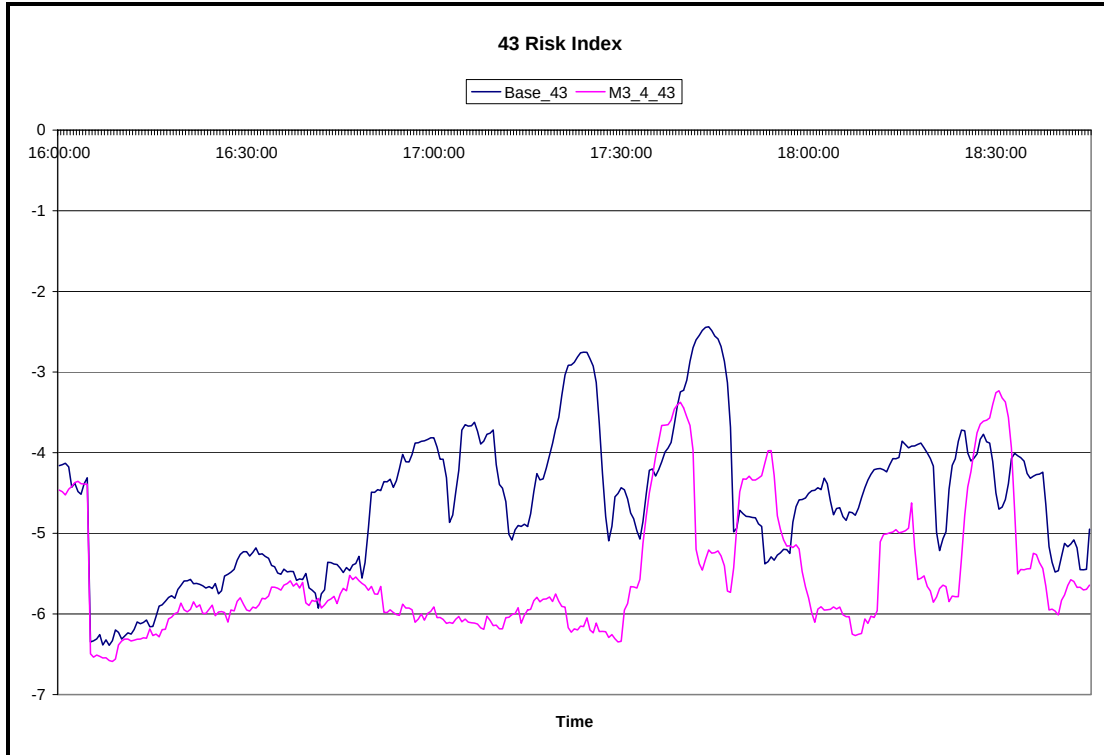


Figure 5-40 Risk Index Evolution Comparison Station 43-44

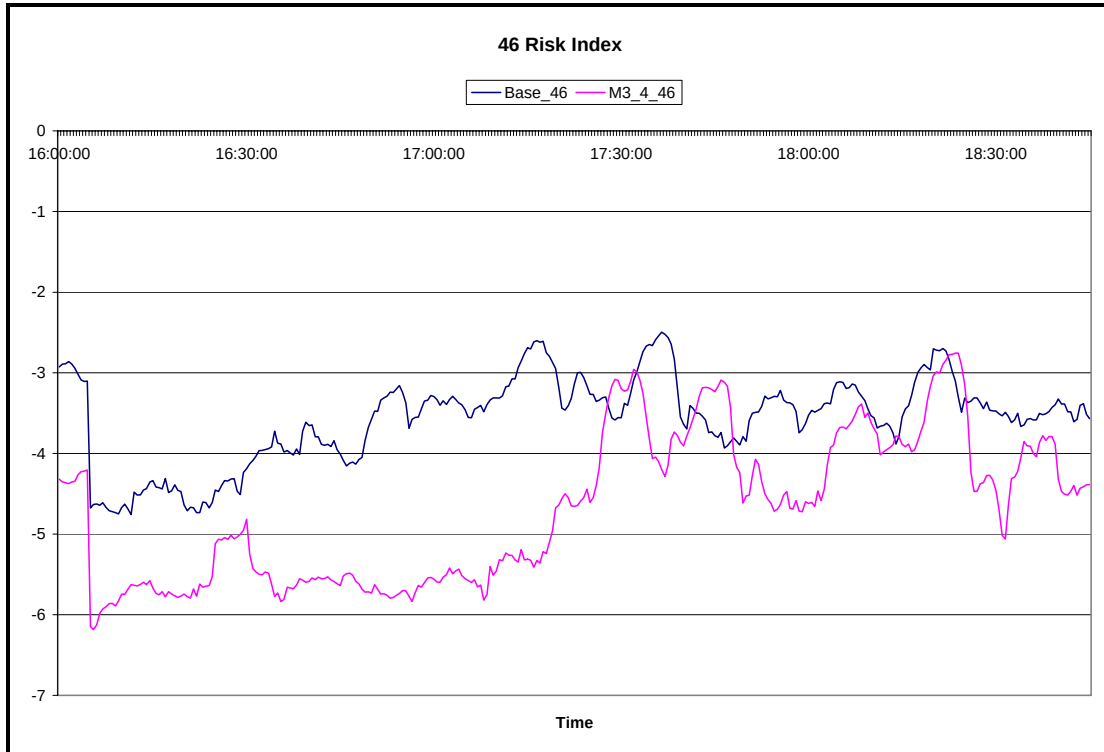
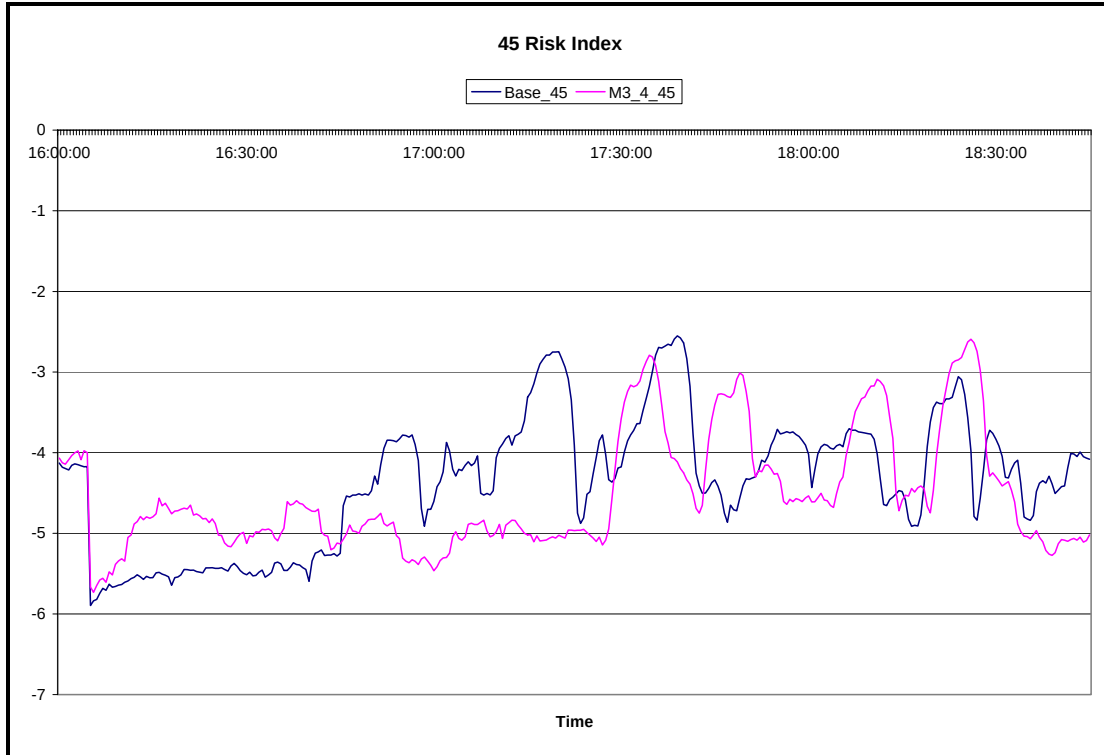


Figure 5-41 Risk Index Evolution Comparison Station 45-46

Although there is significance in the difference of risk indices as can be seen from Figures 5-39 to 5-41, there still remains the question of determining the effect of VSL that brought about this difference in risk indices and the probable reason for the continuous presence of the spikes in the risk indices. Figures 5-42, 5-43, 5-44 and 5-45 show the 30 second speeds for three cases, the Base Case, Module 3 Case 4 (VSL only) and Module 2 Case 9 (3 ramps metered). The comparisons from Figure 5-42 and 5-43 show that the decrease in variance of speeds for Module 3 Case 4 and Module 2 Case 9 is comparable and there are no standout benefits for either of the case over the other. However, as Figures 5-44 and 5-45 show, Module 3 Case 4 significantly increases over all speeds and decreases variances for stations 45, 46 and 47. These are the stations at the downstream end of the network and variable speed limits over the whole section cause much more improvement in these stations. The possible reason for this change is the fact that stations further downstream of these locations (Station 50 onwards) are usually operating at higher speeds with less congestion. As such the effect of relieving congestion for improving safety at these locations is much acute than the stations which are in the middle of congestion.

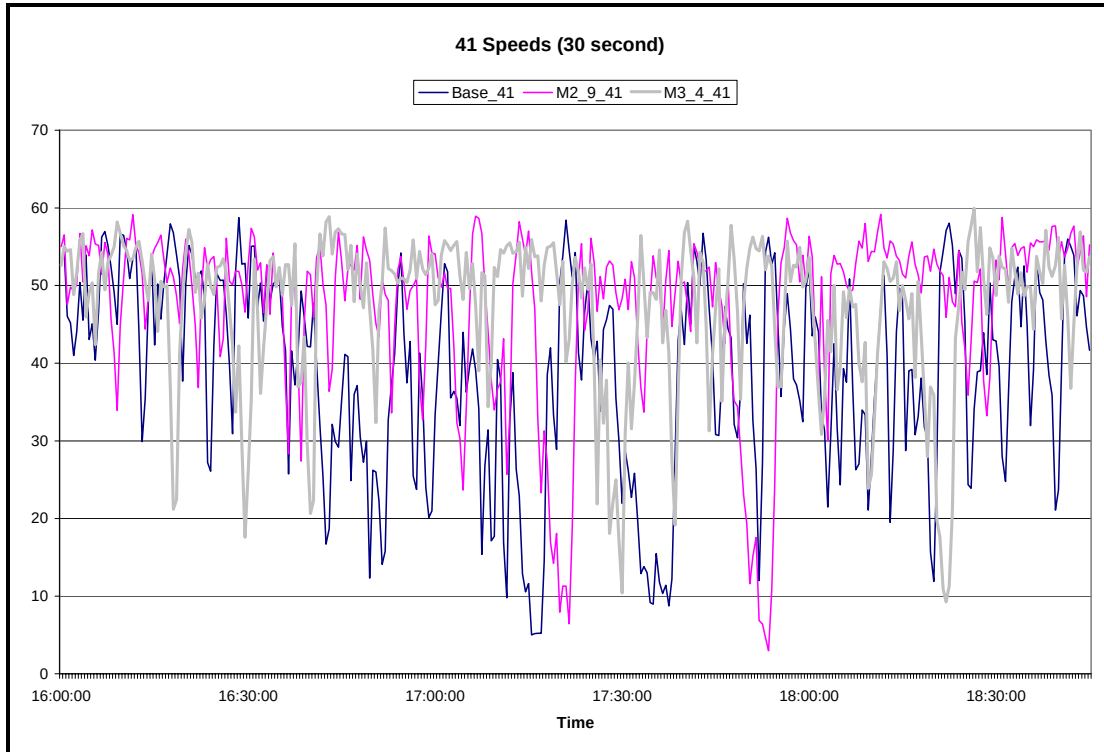
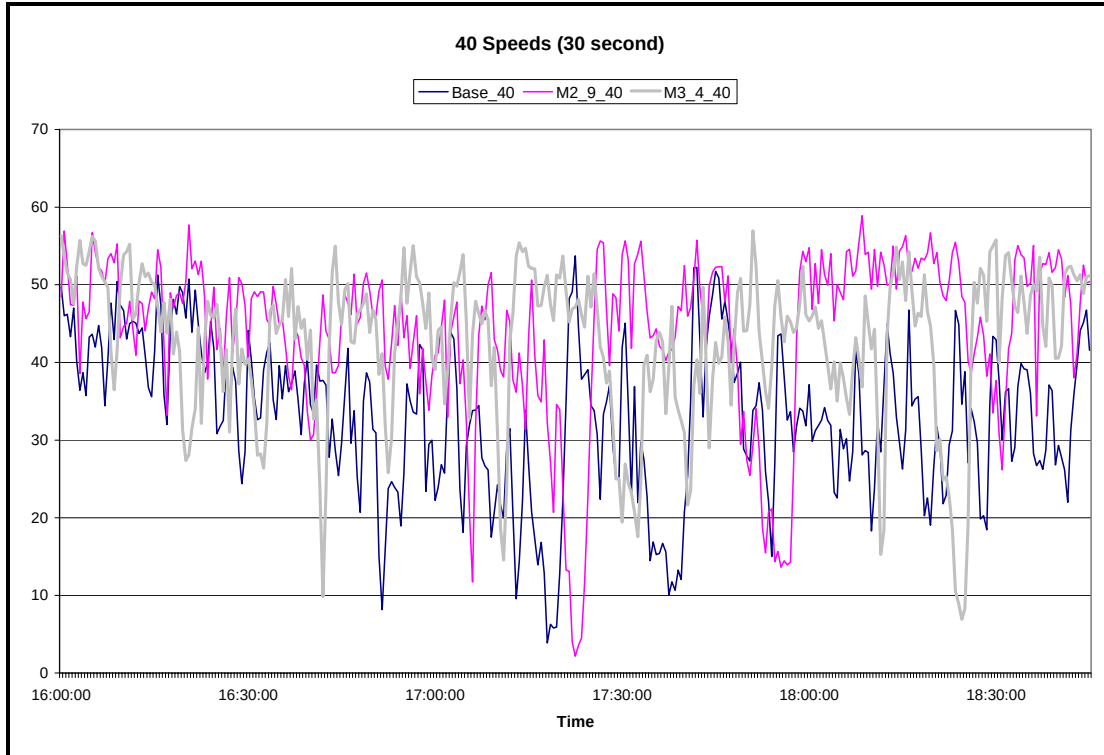


Figure 5-42 Comparison 30 sec. Speed Profiles Station 40-41

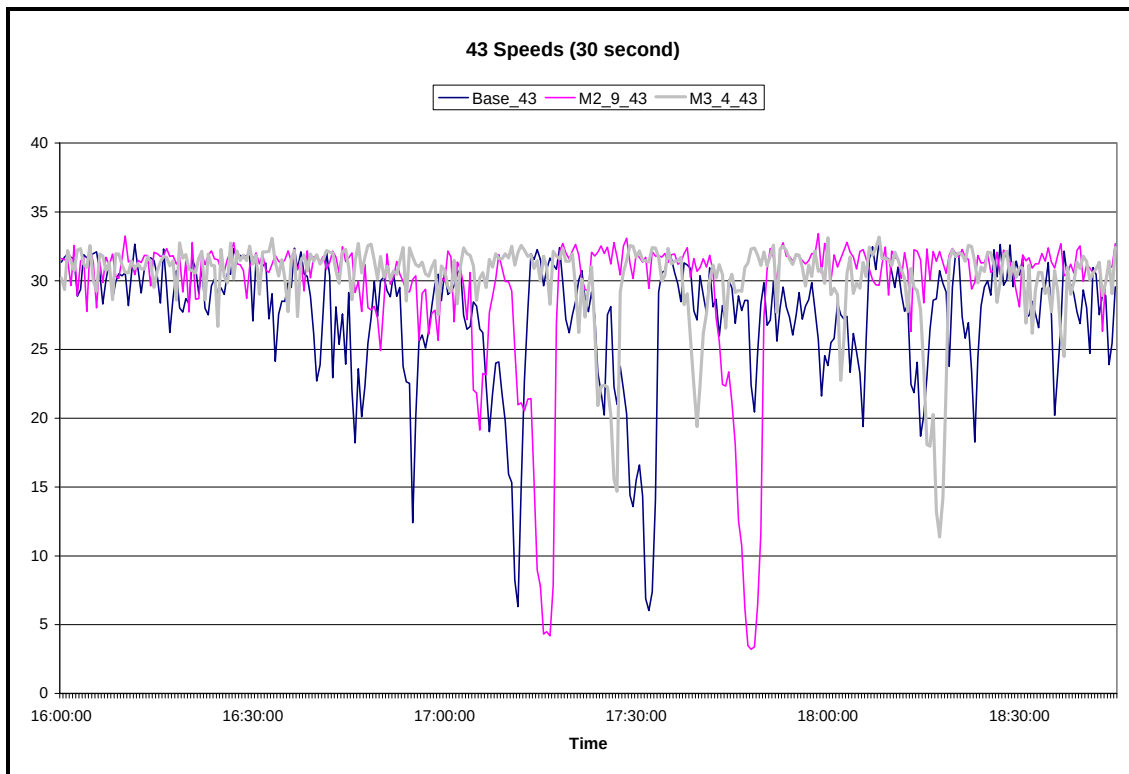
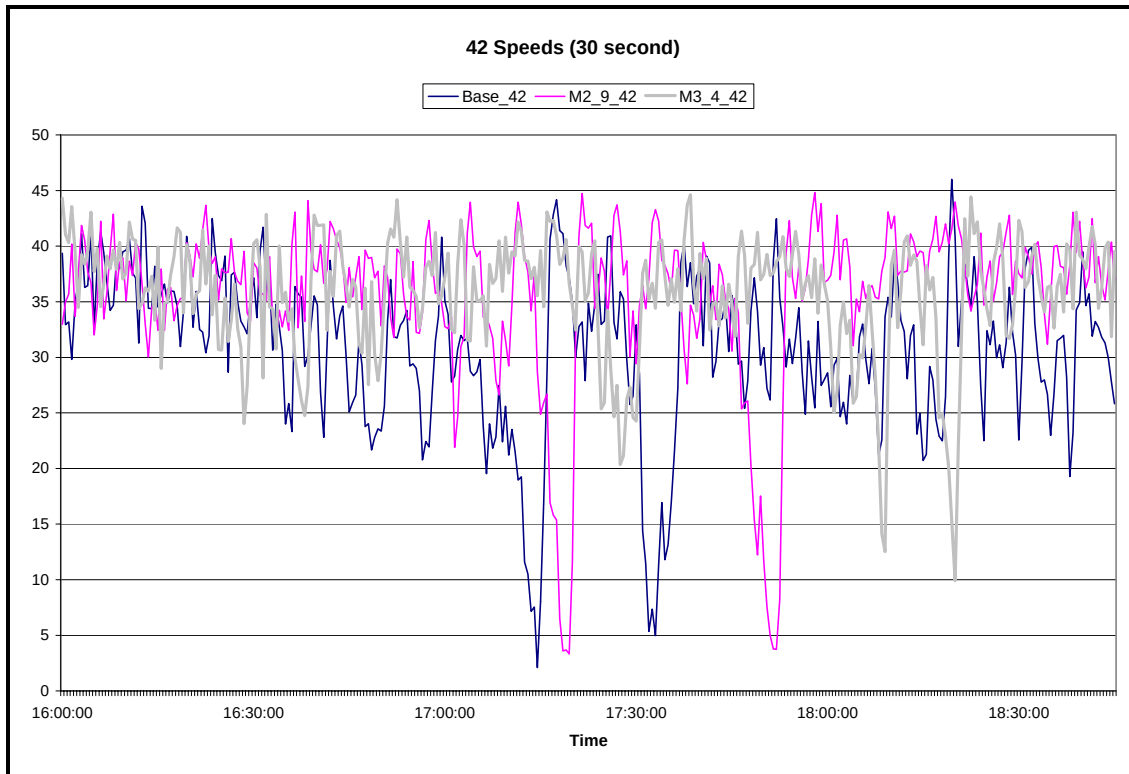


Figure 5-43 Comparison 30 sec. Speed Profiles Station 42-43

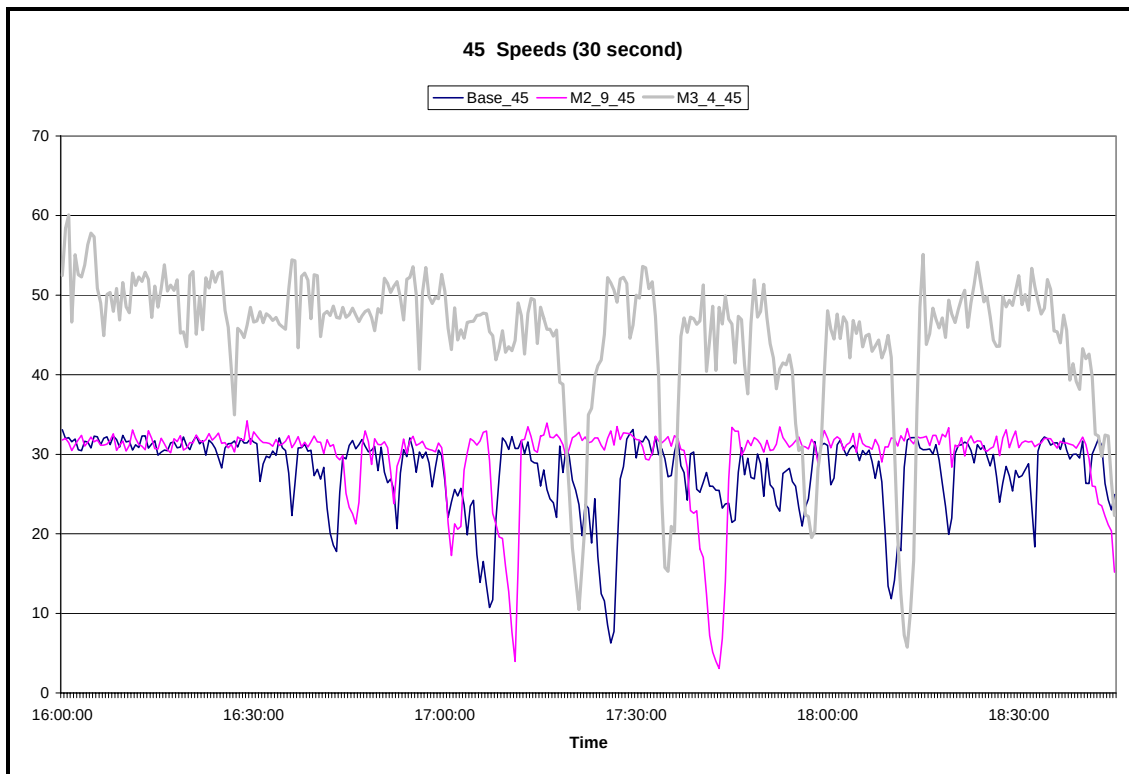
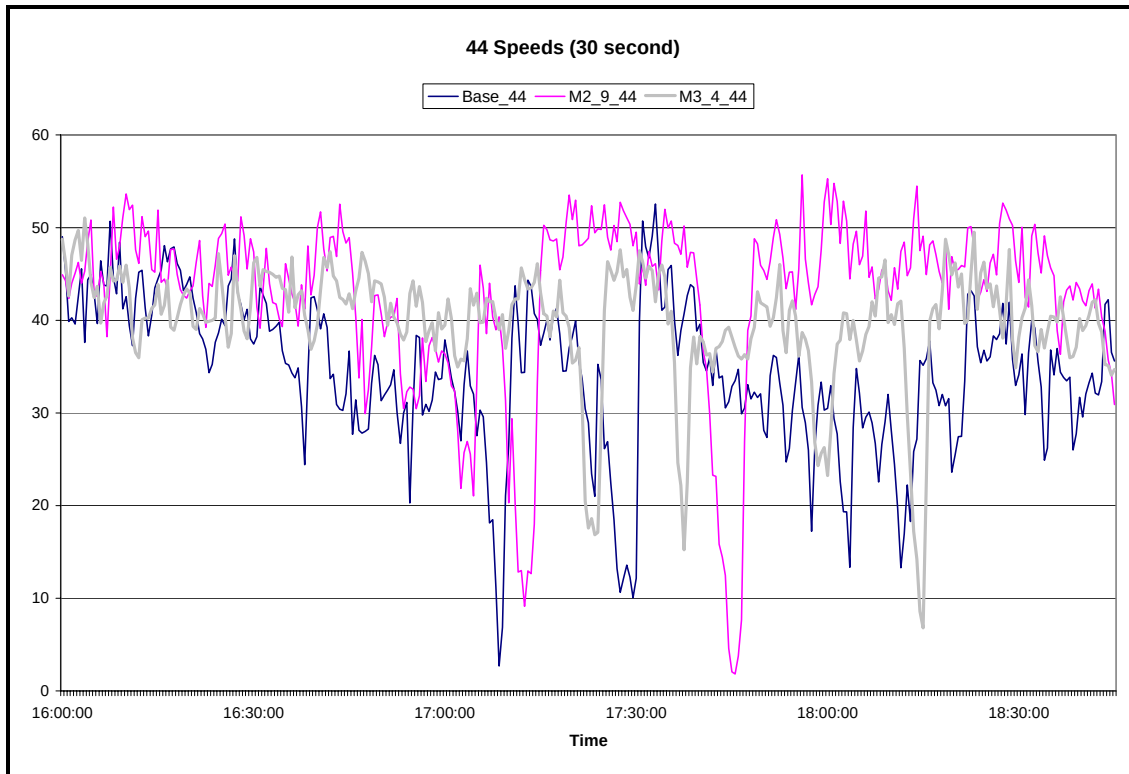


Figure 5-44 Comparison 30 sec. Speed Profiles Station 44-45

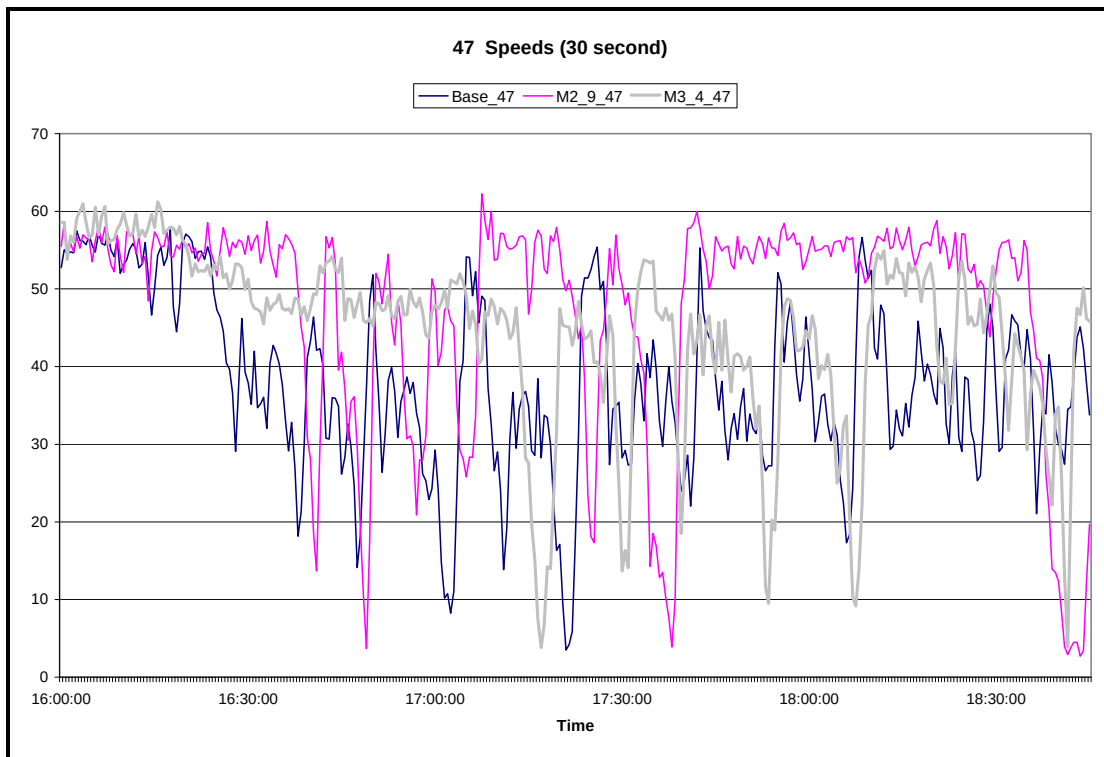
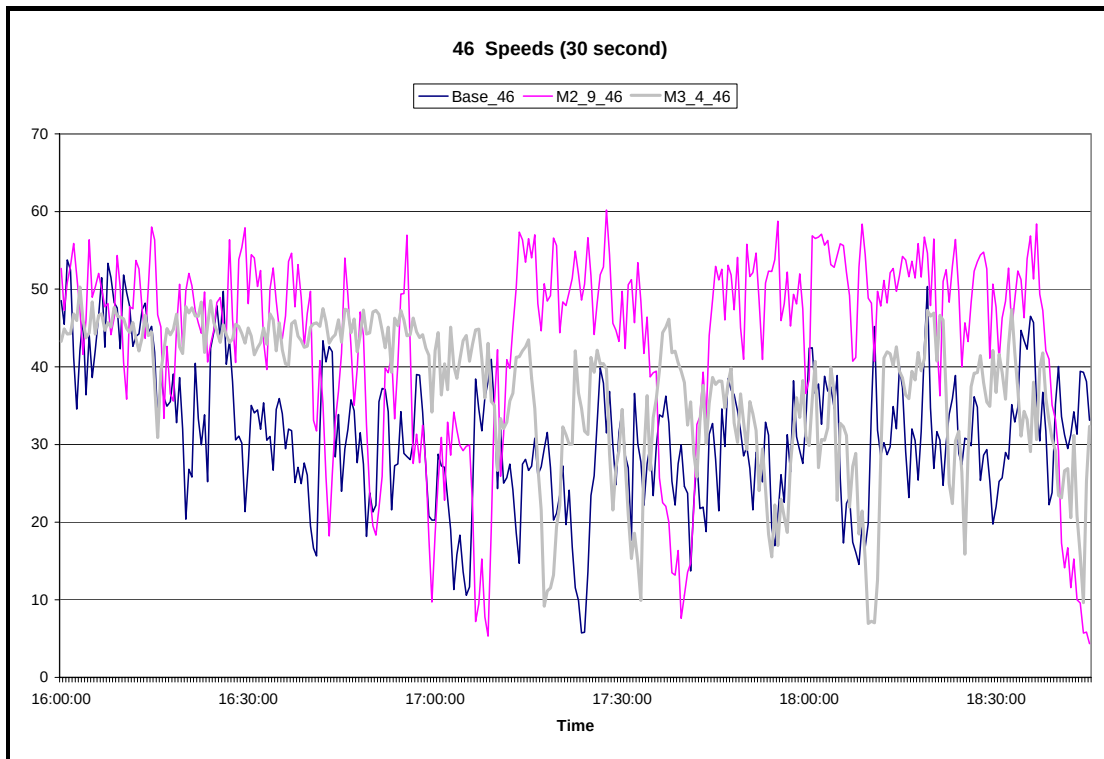


Figure 5-45 Comparison 30 sec. Speed Profiles Station 46-47

The Figures 5-42 through 5-45 show that VSL is comparable here is reducing the variance of speeds to the case where 3 ramps were being metered and performs better at improving the speeds. Also significant of course, is that there is significant improvement of speed variation (i.e. a decrease in variance) over base case.

The next part of the analysis would be look at situations where VSL and Ramp Metering are implemented together and to compare these cases with the standalone implementation of VSL and standalone implementation of Ramp Metering..

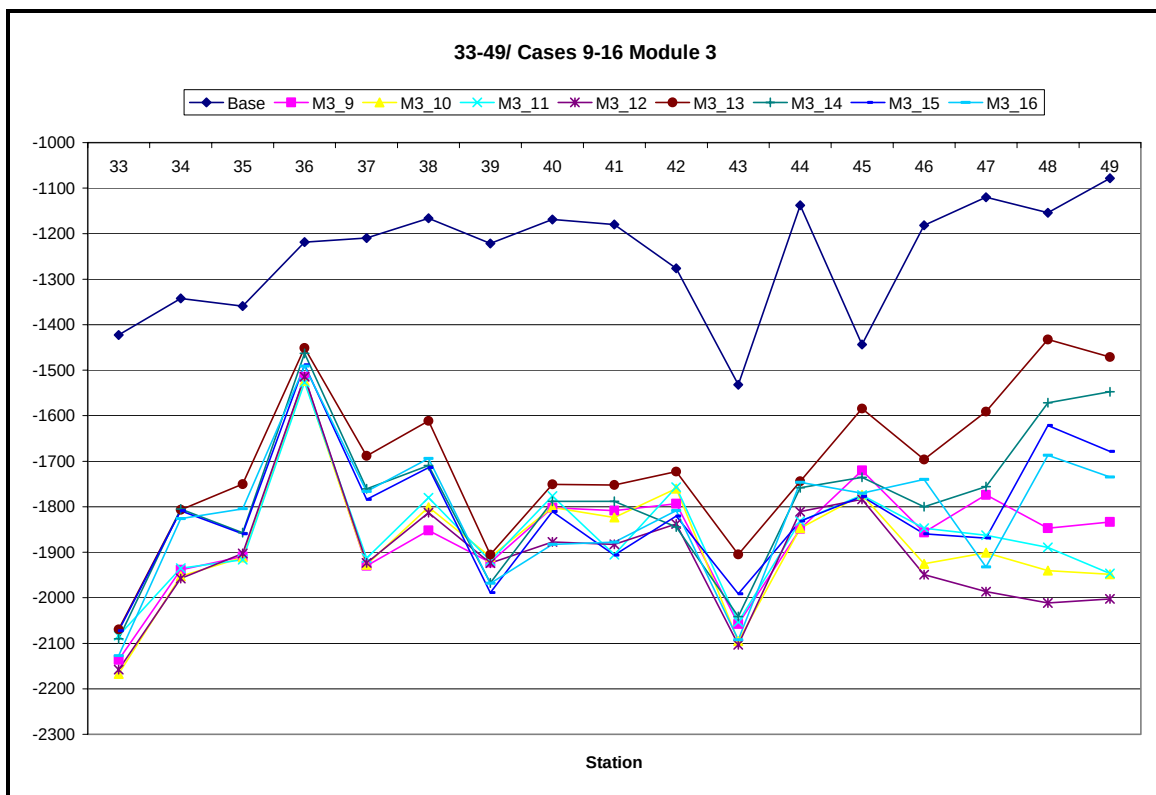


Figure 5-46 Module 3 MCI Cases 9-16 Station 33-49

Figure 5-46 shows the comparison of MCI for the base case and cases 9-16 of Module 3. These cases consist of 3 metered ramps (upstream of stations, 42, 43 and 47) and the VSL implementation implemented together. An analysis of this figure reveals that a number of scenarios perform almost equally well, however, case 12 performs better

than the others at most of the stations. Case 10 performs equally well. Case 12 is the strategy S4 with 5mph speed increments (speed change per location every 5 minute evaluation time step). Case 10 is the strategy of lowering upstream and raising downstream (S2), also using 5 mph increments in speed limits. The effect of the improvement in speed variation can also be seen in the risk index with time for each of these stations. The curves for risk propagation with time for each station show that a coordinated strategy (Case 12) far outperforms any of the individual strategies. However, the only case of individual strategy that performs equally well is the case with all 7 ramps metered.

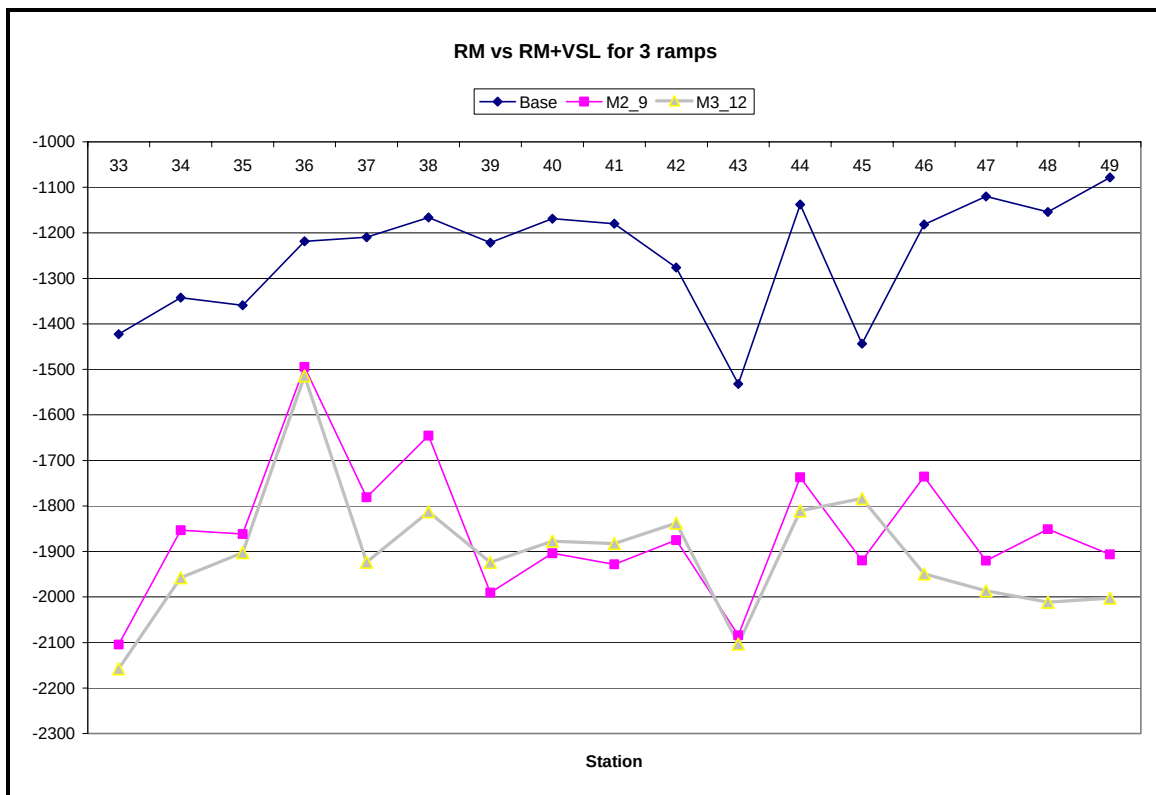


Figure 5-47 MCI Comparison Station 33-49

Figure 5-47 compares the MCI for the base case, Case 9 (3 Metered Ramps) of Module 2 and Case 12 of module 3 (3 Metered Ramps and VSL). The MCI values seem to indicate

mixed results for the two test scenarios. They seem to be comparable at all the locations but significant benefits are seen at the locations downstream of the metered ramps. As indicated in the discussion of Figure 5-42 through 5-45, the improvement for MCI seems much more acute for stations 46, 47, 48 and 49. This behavior can be interpreted from Figures 5-48 through 5-51. These figures present the 30 second speeds for stations 40-47 for Base Case, Case 9 of Module 2 and Case 12 of Module 3 (i.e. cases discussed in Figure 5-47). The speeds from all these figures show a decrease in variances of speeds for Case 12 of Module 3. Significantly, Figures 5-50 and 5-51 also show that for stations 44-47, there are overall increases in speeds at these locations.

Comparing the Figure 5-52 to 5-45 and Figure 5-48 to 5-51, it is clear that while VSL implemented by itself shows considerable improvement in the variance of speeds (i.e. decreases it), when implemented with metering, it produces even better results.

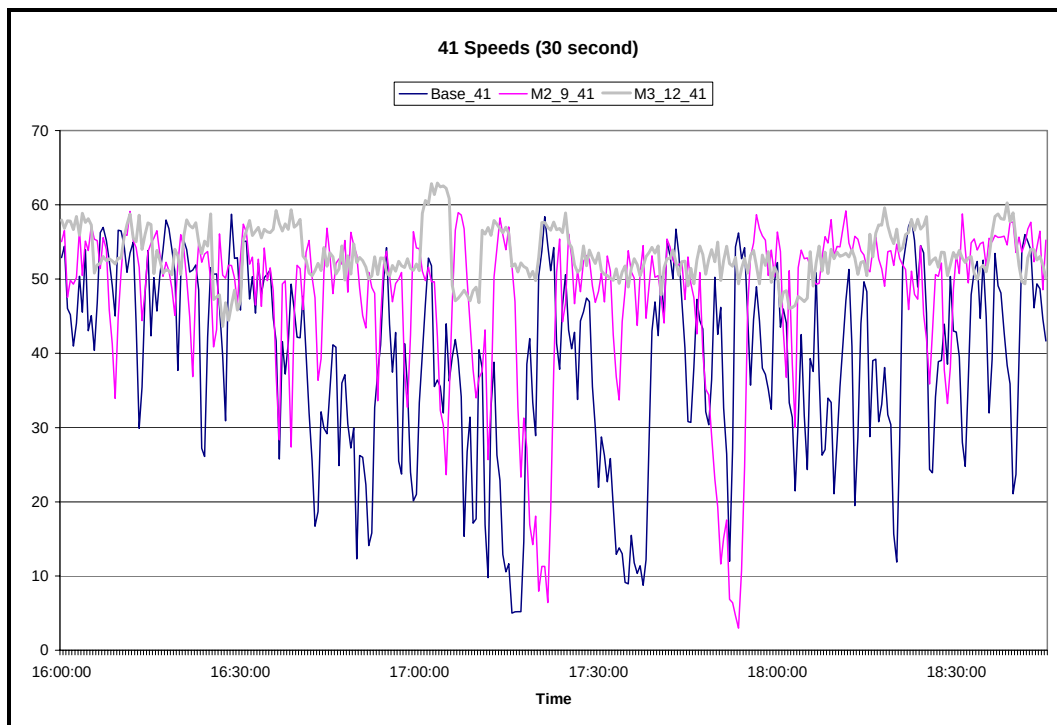
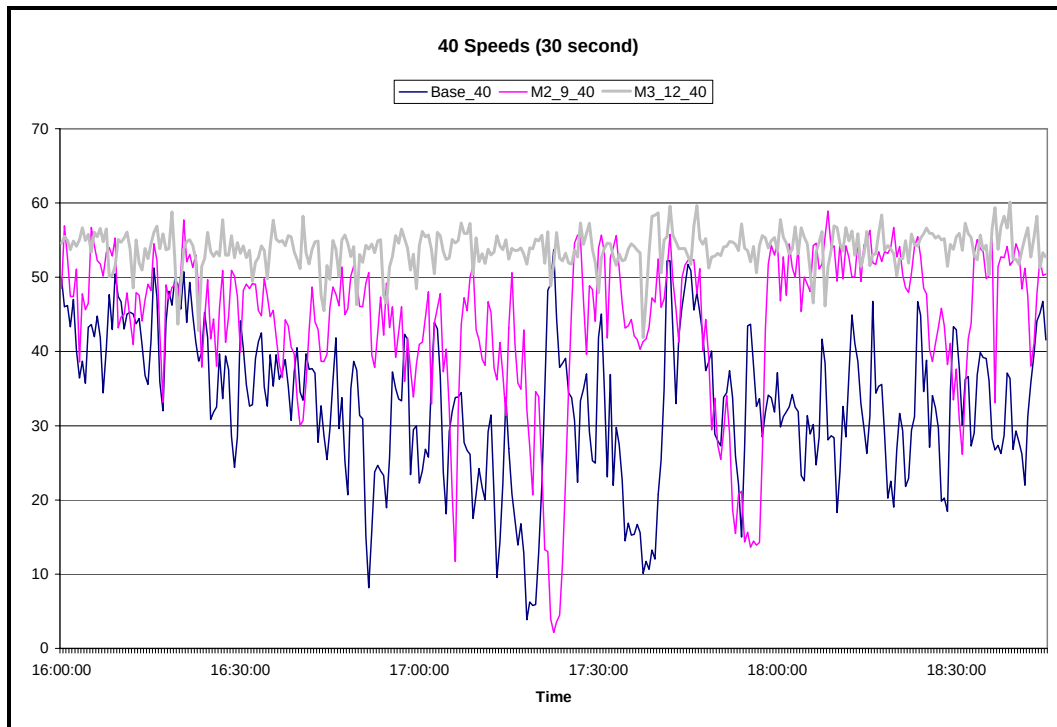


Figure 5-48 Comparison of 30 second speeds for Base case, Module 2 Case 9, Module 3 Case 12
– Stations 40-41

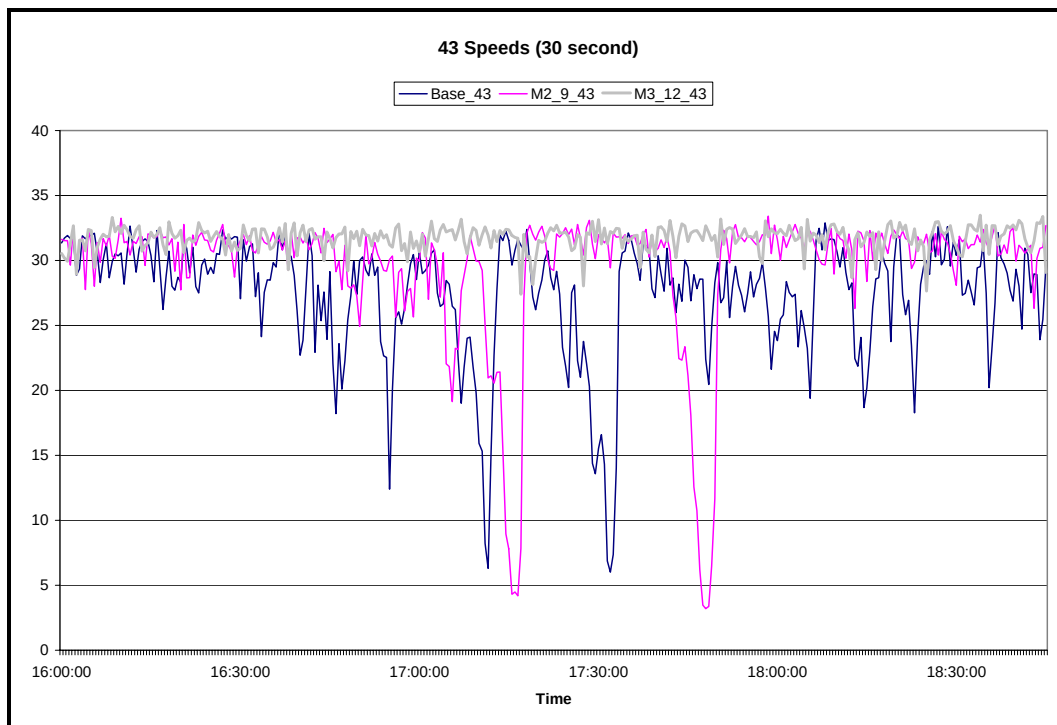
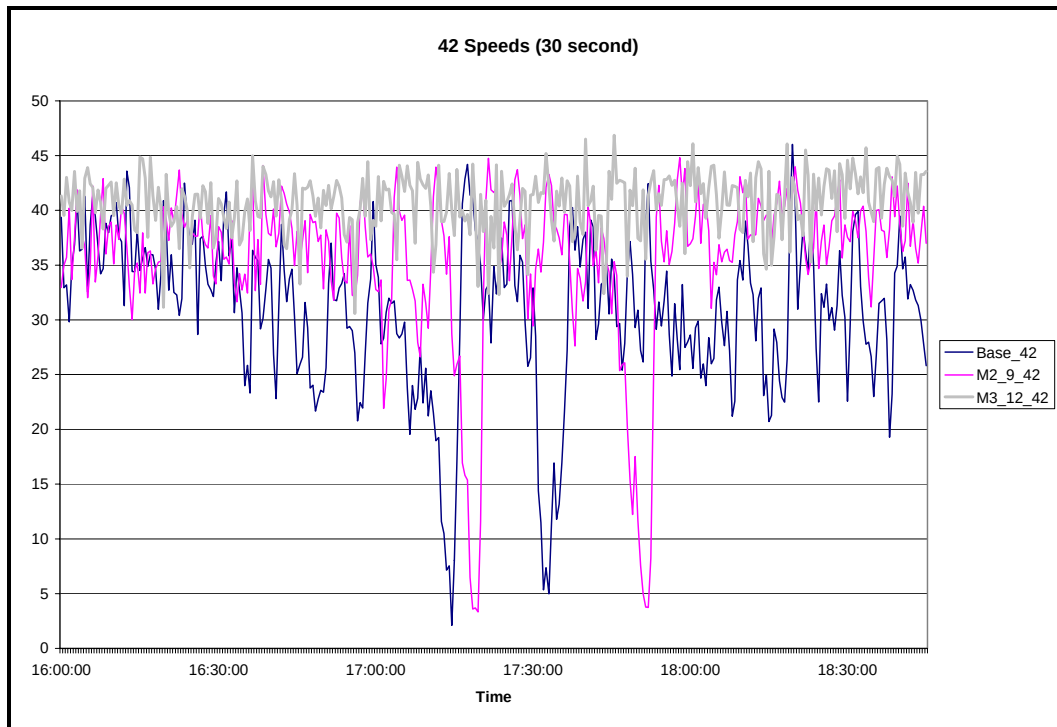


Figure 5-49 Comparison of 30 second speeds for Base case, Module 2 Case 9, Module 3 Case 12
– Stations 42-43

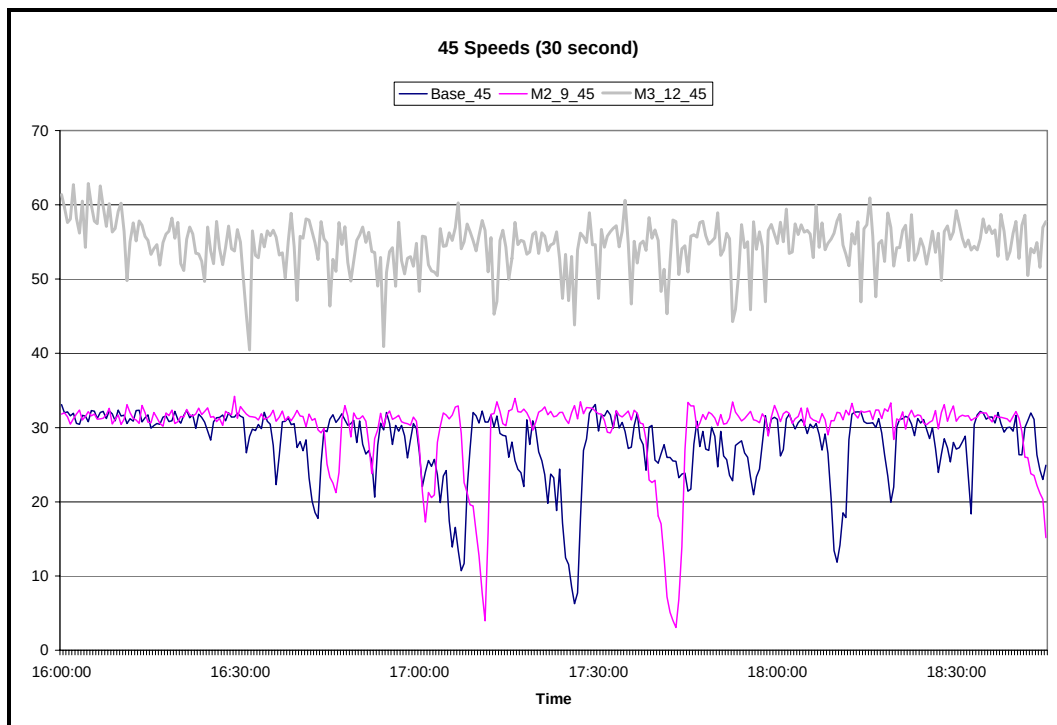
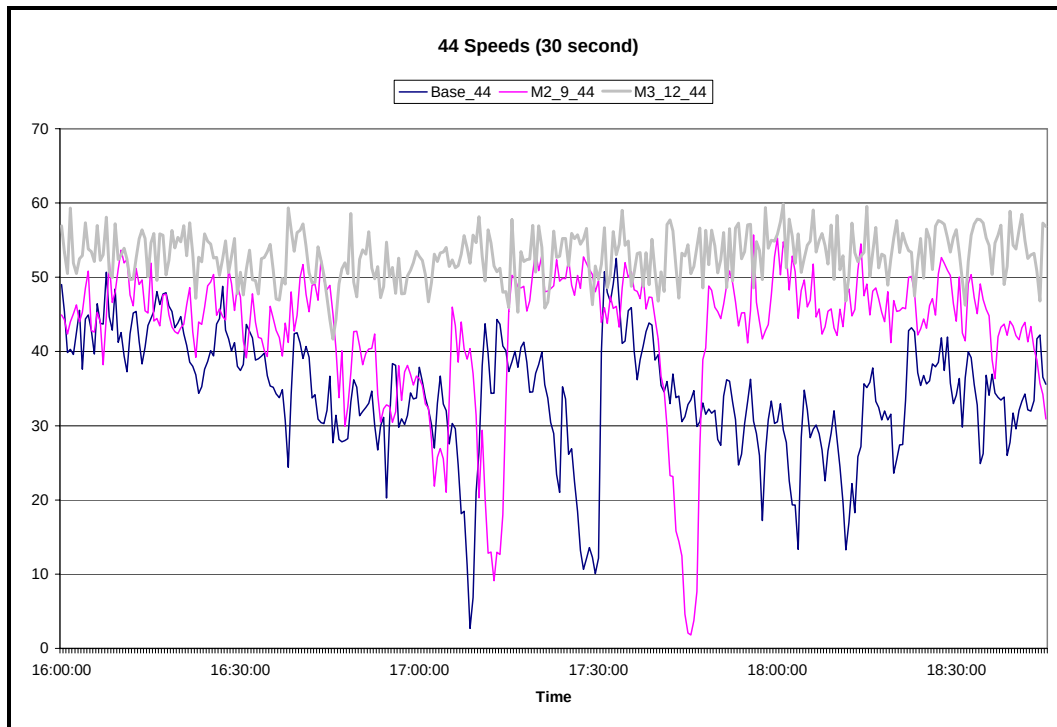


Figure 5-50 Comparison of 30 second speeds for Base case, Module 2 Case 9, Module 3 Case 12
– Stations 44-45

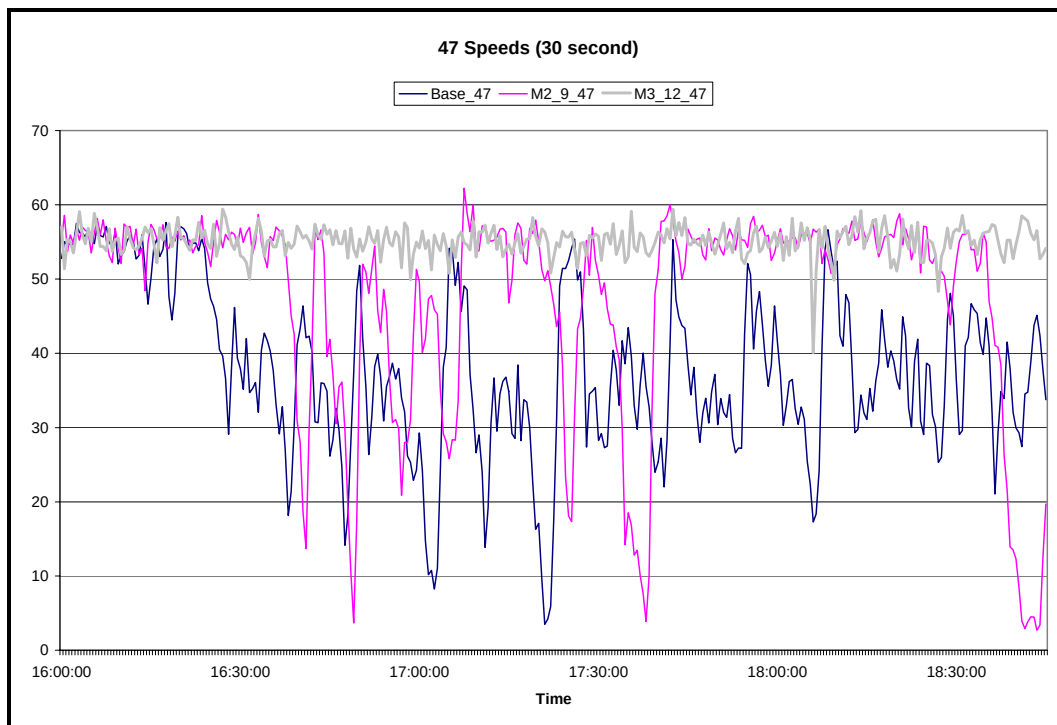
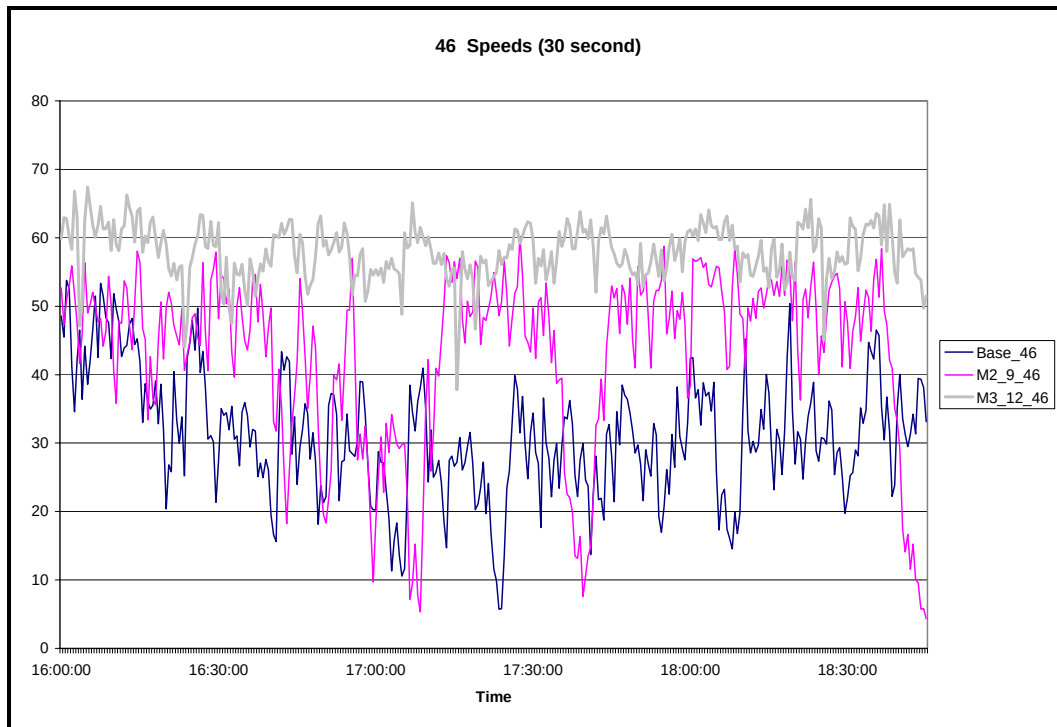


Figure 5-51 Comparison of 30 second speeds for Base case, Module 2 Case 9, Module 3 Case 12
– Stations 46-47

To sum up, the effect of metering over three ramps and coordinating with VSL is also shown in Figure 5-46 over all the stations. The figure shows the cases 9-16 which are implemented with three metered ramps and various VSL strategies. Figure 5-47 seems to suggest that there isn't a significant difference between the coordinated strategy (with VSL and 3 metered ramps) and the case where 3 ramps are metered only, however the difference in speed profiles is very crucial and conclusively proves that indeed using a coordinated strategy improves the speed variance and actually increases the speeds on the mainline thereby enhances the performance of ramp metering. The significant increase in speeds at stations means that the strategy would be also good from a traffic performance point of view. The importance of the increase in speeds arises because there is a shift in regime from low speed to high speed which by itself is related to less number of crashes (Abdel-Aty et al., 2005; Pande and Abdel-Aty, 2006).

Although it would be expected that the benefits of a coordinated strategy would keep on increasing as the number of metered ramps keeps increasing. However, this improvement loses its marginal benefit as the number of metered ramps increases. The coordinated strategy case with 7 metered ramps decreases the MCI, the results of which are presented in Figure 5-52.

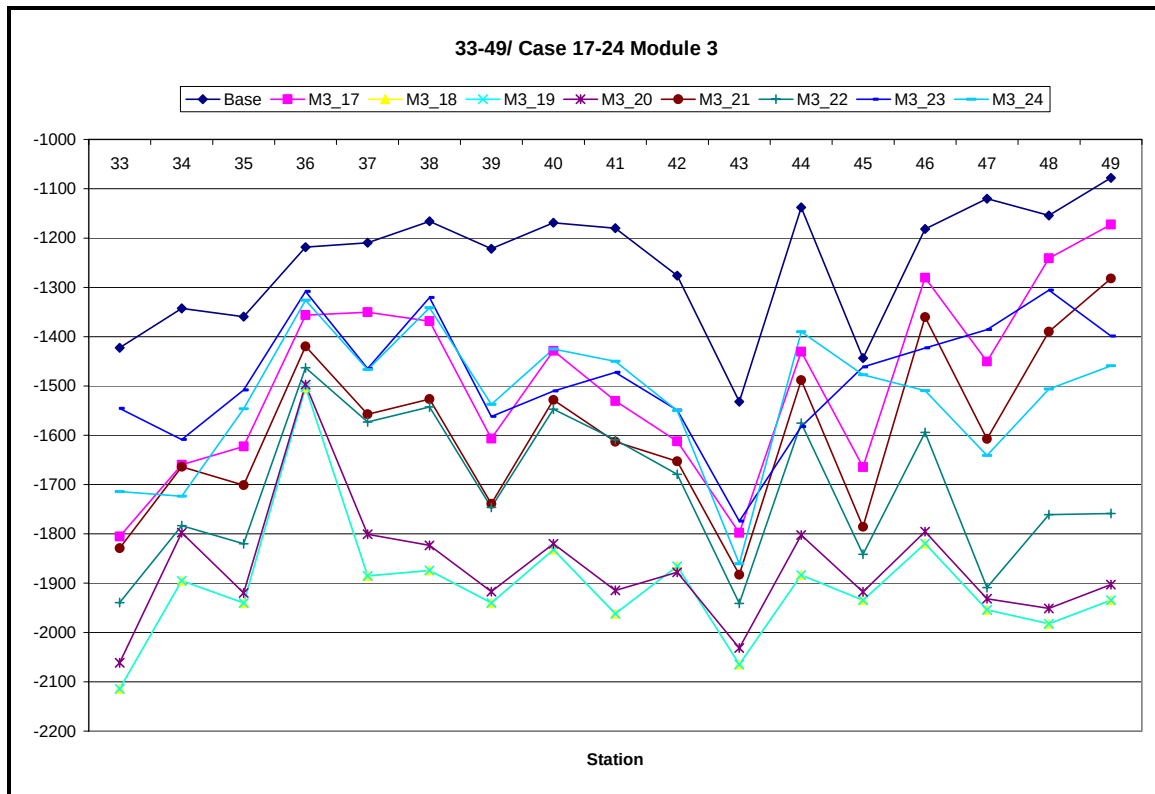


Figure 5-52 Module 3 MCI Cases 17-24 Station 33-49

Figure 5-52 compares the MCI for the network in the cases where VSL is implemented with 7 metered ramps (upstream of stations 35, 38, 40, 42, 43, 47, 49). The values suggest that Cases 18 (7 metered ramps, network wide VSL with 5 mph speed decreases upstream and 5 mph speed increases downstream at every location), 19 (7 metered ramps, network wide VSL with 5 mph speed decreases upstream only at every location) and 20 (7 metered ramps, network wide VSL with 5 mph speed increases downstream at every location) outperform the others in terms of MCI. Case 18 perform marginally better than case 19 at most of the locations.

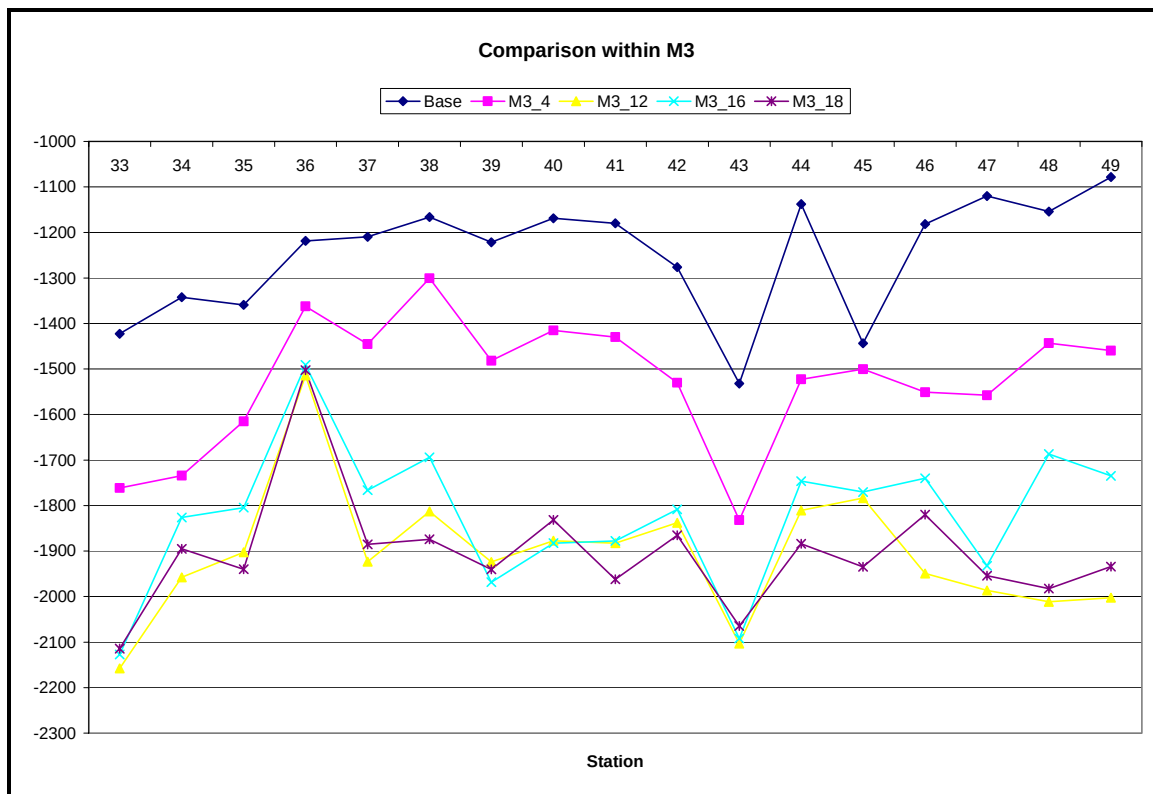


Figure 5-53 MCI Comparison Cases within Module 3

However, there is a lack of improvement in marginal benefits by increasing the number of metered ramps to 7 from 3. To reinforce the previous statement, Figure 5-53 presents a within module 3 comparison which shows that there is very little benefit from expanding the scope of the strategy from 3 Ramps to 7 ramps. The trend observed here is that VSL without ramp metering performed comparable to the case of only ramp metering. Furthermore, a coordinated strategy outperformed any of the single strategies. However, the difference between the coordinated strategy with 7 metered ramps and 7 metered ramps only, is very minimal. This lead to the conclusion that a larger ramp metering strategy overall would perform equally well as a coordinated strategy over the same interval. However, it is better to increment a limited ramp metering with Variable Speed Limits strategy in order to get the best possible speeds even though there might be equivalent safety benefits.

5.4.2 Effect of VSL in Low Speed Conditions

In Chapter 3, it was inferred that variable speed limits did not perform in the low speed regimes. However, in this phase of the preliminary evaluations we focused on testing at a single location with a strategy that varied speed limits once and was not dependent upon the conditions on the freeway. A pre-meditated strategy was used in that case and variable speed limits were found to be ineffective. Although this makes sense if the speeds of the vehicles are already very low, the kind that would exist in the stop and go conditions; there is an anomaly when we look at it from a safety perspective. The average 5-minute speeds in testing being around 25-30 mph, these conditions cannot be classified as stop and go. Also, the Risk Index for low speed regimes includes coefficient of variation of speed as an important factor. Hence any strategy that could effect the variance of speeds should have some kind of effect on the Risk Index. An effort was made to find out if at all a network wide implementation of Variable Speed Limits based on real time response could affect this index.

As can be seen from Figures 5-34, 5-35, 5-36 and 5-37, Variable Speed Limits can improve the Risk Index as well as improve speeds and reduce variances of speeds over the network with a well implemented strategy. The improvements in Risk and Speed are comparable to the improvements marked by metering 3 Ramps from the cases seen in Module 2. In fact VSL does better than the Case 9 of Module 2(3 Metered Ramps, Critical occupancy 23%, Signal Cycle of 25 seconds) in that it improves the speeds over most of the section. This improvement would show itself in improved travel times in Section 5.4.7.

5.4.3 Effect of Speed Increments and Decrements

Another crucial element that was tested within this module was the effect of the different strategies and the possible changes of speed limits that should be introduced in order to achieve the best possible improvement. As can be seen from Figures 5-32, 5-37, 5-43, the better strategies amongst the four scenarios tested were always S2 and S4 used with speed increments of 5mph. VSL Strategy S2 refers to increase of speed limit downstream and lowering the speed limits upstream. Strategy S4 refers to just increasing the speed limits downstream. Strategy S4 outperformed S2 in almost every case that we tested (Figures 5-32, 5-37, 5-38, 5-43). The fact that each of these strategies when compared with implementations of 5mph and 10mph increments or decrements performed better under 5mph case, makes the results even more useful and implementation worthy. This overcomes one of the biggest disadvantages of implementation recommendations in Chapter 3. 15 mph sudden changes in speeds would probably not have been the best strategy for drivers on freeways. A network wide traffic responsive speed limit control strategy which works by introducing 5 mph speed limit changes provides a very strong strategy that makes practical sense and improves the over all network safety.

5.4.4 Effect of VSL in conjunction with Ramp Metering

The best strategy mentioned in Section 5.4.2 could be further enhanced by implementing in coordination with Ramp Metering. The part of analysis dealing with Figures 5-38 and 5-44 shows that every time there is a coordinated strategy it manages to perform better than both the ramp metering and variable speed limits strategy implemented alone. Even the comparable strategy of 7 ramps metered ramps is comparable to the coordinated strategy with 3 ramps metered and network wide Variable Speed Limits. The travel time analysis would prove the

superiority of coordinated strategy when we are comparing strategies for traffic parameters. The fact that the coordinated strategies improve speeds on the freeway makes them a more desirable option from both the safety and operations perspective.

5.4.5 Confidence Interval for Random Variance

Section 5.3.4 showed the procedure for testing the significance of the improvements in MCI based on a t-statistic for base case and determining if the decreases were more than random variances. Using the same method, the MCI for the best cases from each part of Module 3 are compared and found to be significantly different as can be seen in Figure 5-54. The cases 12, 16, 18 and 20 lie outside the interval of random variance and this indicates an improvement in MCI for these cases. Although the cases perform differently at various locations, Cases 12 and 18 seem to perform consistently better than most of the other.

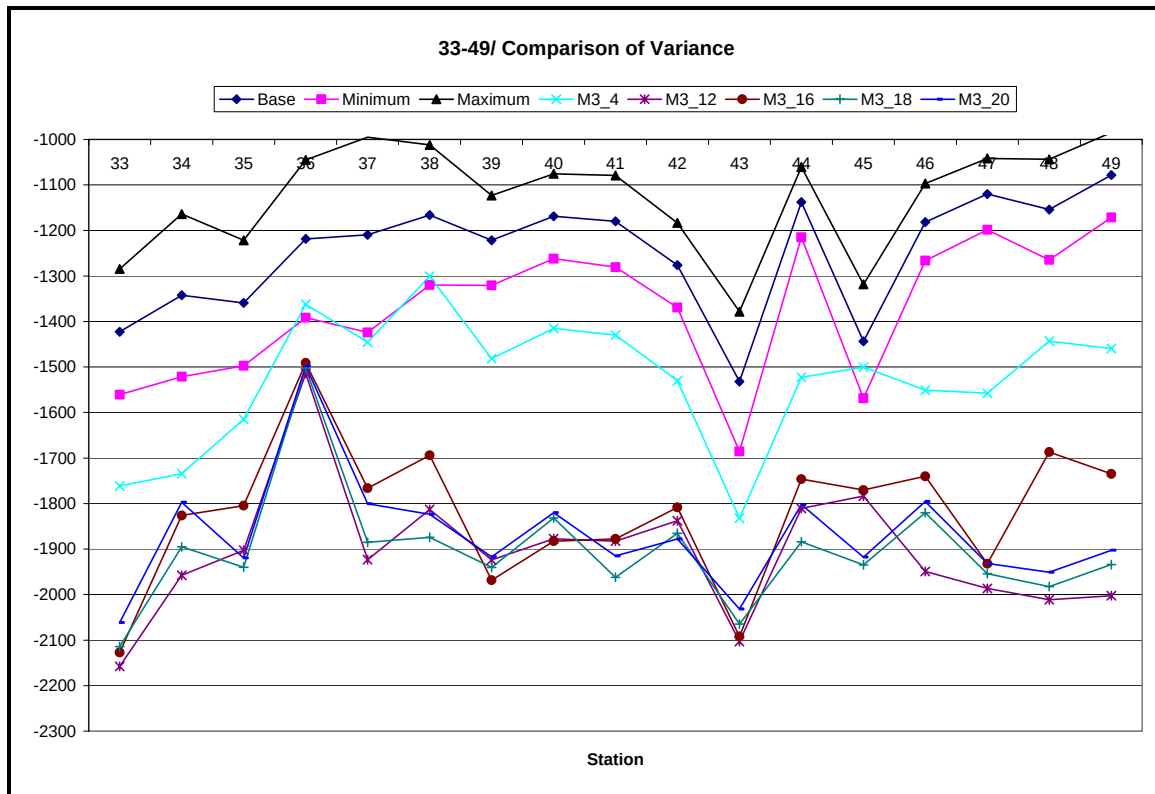


Figure 5-54 95% CI t-test for testing significance of Module 3 Cases

5.4.6 Quantification of Benefits

The section 5.3.5 showed the procedure for calculating the Safety Benefits and Safety Benefit Index from the respective test cases. Figure 5-55 shows the benefit index for the best cases among Module 3. As can be seen Case 12 and Case 18 dominate the benefits.

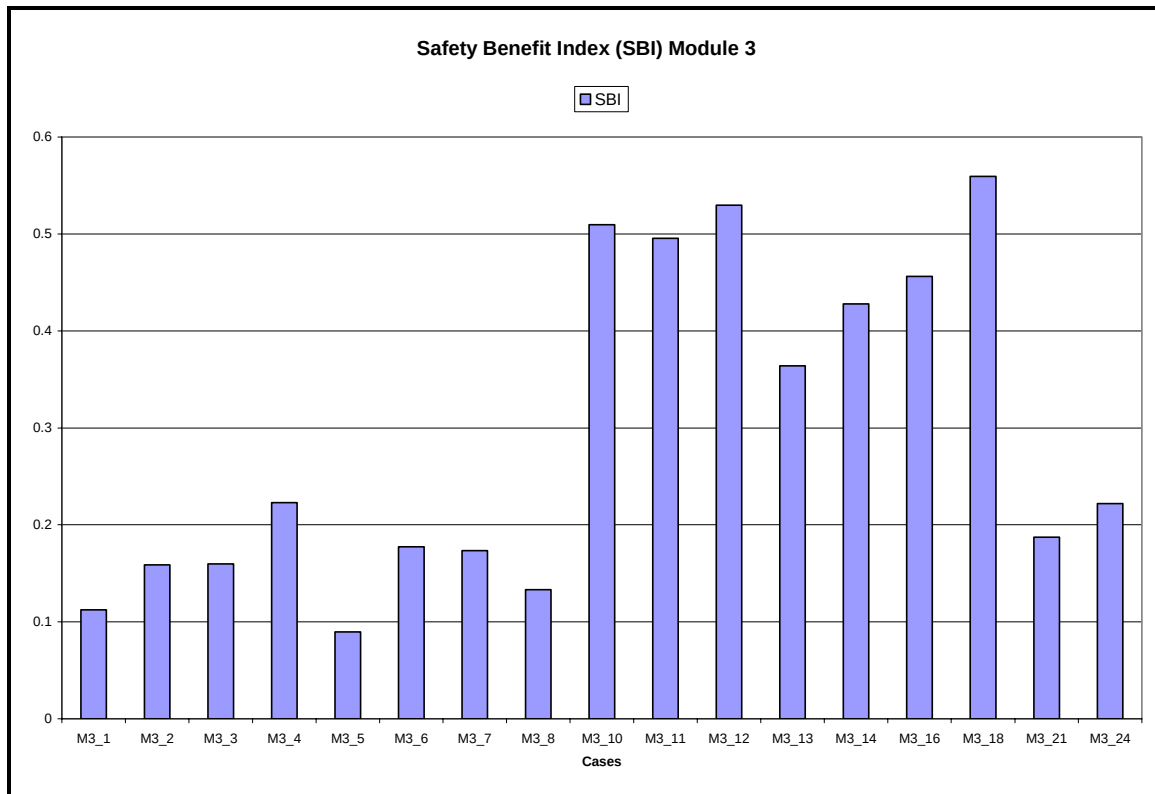


Figure 5-55 Case Wise Safety Benefit Index Module 3

The benefits are also compared with some of the cases from Module 2 in Figure 5-56. Figure 5-56 shows the comparison of safety benefits index of the best cases from Module 2 and Module 3. It clearly shows that the benefits from Module 3 are much more than the similar cases from the previous module. Especially the fact that the performance of the cases 4 and 12 from module 3 is better than the corresponding cases 6 and 9 from Module 2 conclusively proves the benefits from implementing a coordinated strategy.

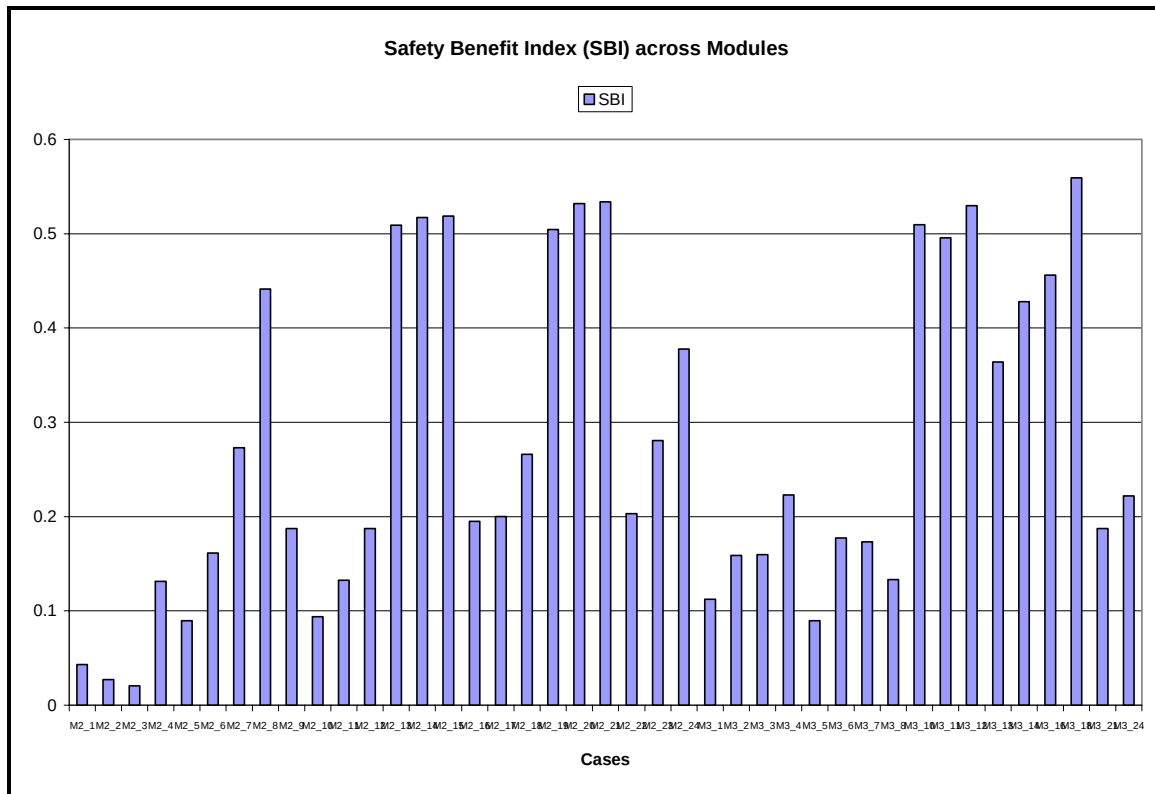


Figure 5-56 Case Wise Safety Benefit Index Comparison Module 2 and 3

In fact case 12 from Module 3 marks a large improvement over Case 9 from Module 2 which shows that implementing a network wide variable speed limit strategy can help us in limiting the number of ramp meters on the freeway and still get greater benefits without having to deal with lot of social issues generally associated with Ramp Metering.

Figure 5-57 shows the station wise safety benefits for Case 4 and Case 12 from Module 3. As is apparent, there are significant benefits comparable to those seen in section 5.3.5 for Case 21 for 7 metered ramps. However, the station wise benefits of implementing a coordinated strategy are much more than the safety benefits of implementing just VSL.

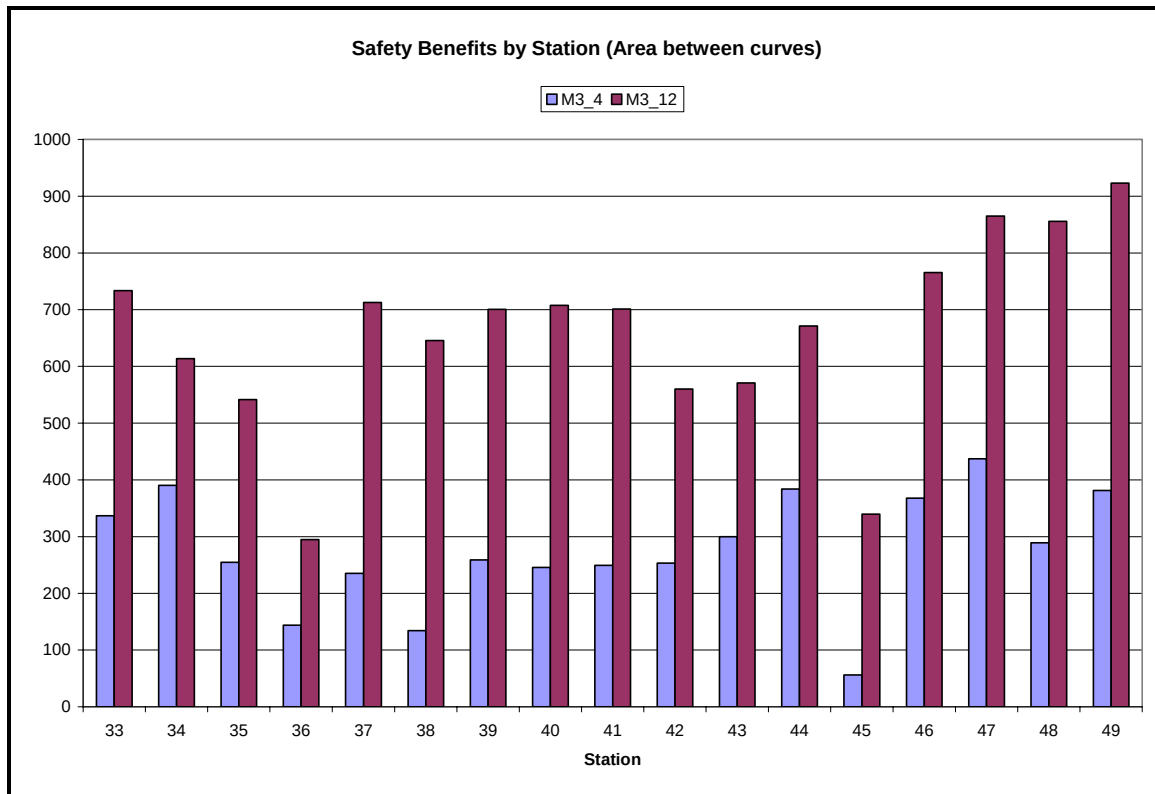


Figure 5-57 Station Wise Safety Benefits Case 12 Module 3

5.4.7 Travel time analysis

Travel Time Analysis was carried out in the same manner as was described in section 5.3.6. Table 5-4 shows the base case and the important cases analyzed in the previous scenarios. Confidence intervals using the t-statistic were constructed for each data point. The tests showed that the differences in travel times were indeed insignificant as had been predicted from the analysis of speed profiles for all the stations. The 95% confidence intervals for each scenario are provided in columns 4 and 5 of Table 5-4. It can be seen that none of the scenarios have overlapping CI's indicating a significant difference between any pair of the three cases. The difference for travel times for just the east bound traffic would be even more significant. The best case from Module 2 (Case 21) gave a mean travel time of 146285.6 seconds from 20 runs.

The difference of the travel times from Module 3 (viz. Case 18) was found to be significantly less than even the best case from Module 2 reinforcing the conclusion that even if there is equivalent difference in the improvement in risk index but there is a very significant difference in travel times and therefore VSL and Ramp metering together could provide over all greater benefits. Also the travel times from the Case 12 of Module 3 are comparable to and significantly less than the best travel times from Module 2.

Table 5-4 Mean Network Travel Times and CI

	TT	Max. variance	CI Minimum	CI Maximum
Base	154925.9	3525.1	151400.8	158451
Module 3 Case 12	139096.1	2664.33	136431.8	141760.4
Module 3 Case 18	121685.9	2235.4	119450.5	123921.3

5.4.8 Conclusions from Module 3

The main conclusions from this module are summed below:

- Variable Speed Limits implemented over multiple locations can be effective even in the low speed cases (<37.5 mph).
- Variable Speed Limits are effective when implemented in conjunction with Ramp Metering and enhance the performance of ramp metering both in terms of risk and also in terms of speeds.
- The best strategy for implementation involves increasing the speed limits downstream of any location in increments of 5mph over half a mile.

- Ramp Metering and VSL in conjunction provide better travel times and increased speeds than either of them separately
- The marginal benefits of adding VSL to ramp metering decreases as the number of metered ramps increases.

5.5 Validation with Generic Models

A small issue with the analysis using the split models proposed by Abdel-Aty et al. (2005) could be seen in the analysis. Although the models were developed for different speed regimes (i.e. low speed and moderate to high speed), sometimes the scenarios that were implemented improved the situation of the particular station to such an extent that the low speed stations changed regime. To ensure that there were no safety side effects of this speed increase, a common index was used that could be applied to any speed regime. This generic model was developed through the same research as the split model and presented in Abdel-Aty et al. (2004). The model is presented in Equation 5-6 and parameters bear the same nomenclature as defined for equations 5-1 and 5-2.

$$Risk = 1.21405 \text{LogCVSF2} + .02466 \text{AOG2} - .19124 \text{SVG2}$$

...Equation 5-6

The generic model in equation 5-6 uses the Coefficient of variation of speed at the station of interest 5-10 minutes before the time of interest (CVSF2), average occupancy at the station downstream of the station of interest 5-10 minutes before the time of interest (AOG2) and the Standard deviation of volume at the station downstream of the station of interest 5-10 minutes before the time of interest (SVG2). From an analysis of Figures 5-44 and 5-50, station 45 is seen to change from a low speed (average speeds of 37 mph) to a high speed (average speeds of 48

and 52 mph, respectively) station. As such a risk index was calculated for station 45 in the base case and Case 12 module 3 (3 Ramps metered and VSL implemented). Figure 5-58 shows this risk index. It shows that the decrease in risk index for the case 12 is still significant and our strategy helps in reducing the risk of crash.

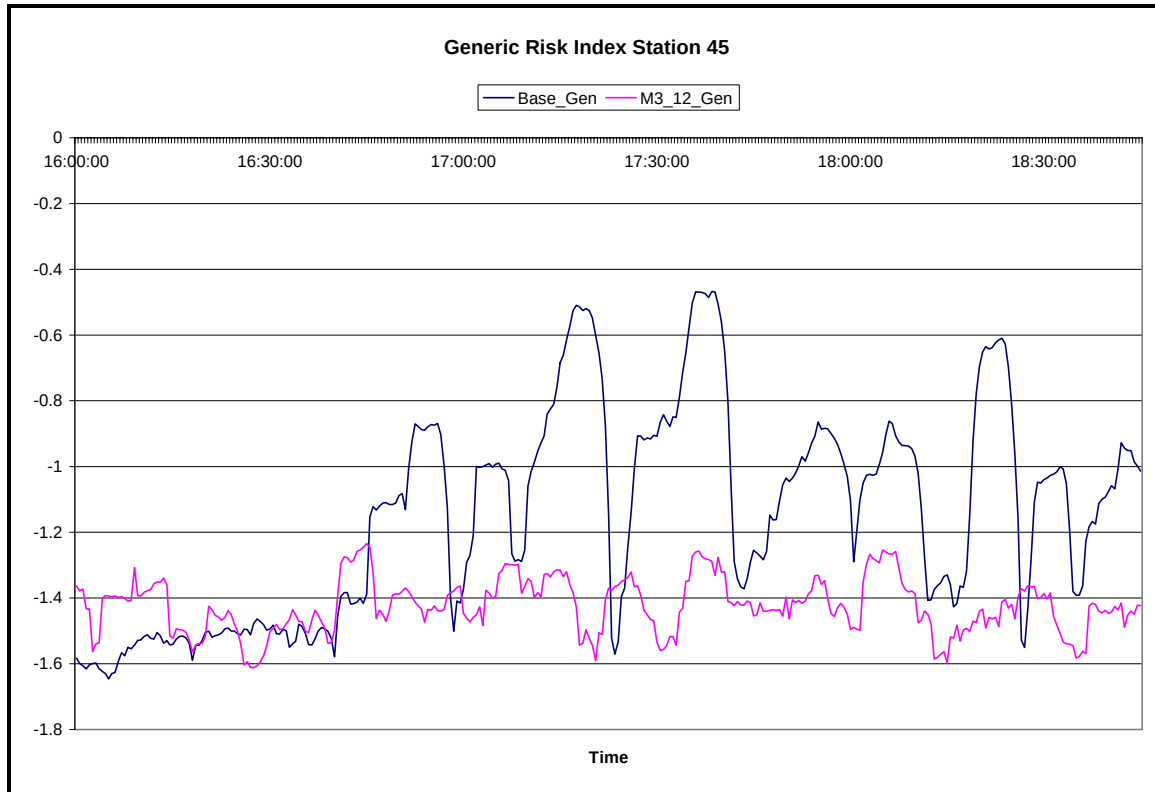


Figure 5-58 Generic Risk Index Station 45 EB

The results from Figure 5-58 are also important for the credibility of the low speed models we used. The models seem to be able to capture the important effects and a decrease in risk index for low speed model would imply a decrease in the risk of a crash too.

5.6 Conclusions from Preliminary Analysis: Stage II

This research in this preliminary analysis focused on the safety benefits of two crucial ITS technologies, that of Ramp Metering and Variable Speed Limits. The objective was to evaluate

the various configurations of these technologies, implemented separately and together, that would provide the best benefits in terms of reducing the risk of crash at a network wide level. Micro-simulation PARAMICS was used for testing a number of strategies that would be feasible for implementation in real world. As part of the study a 9 mile stretch of Interstate- 4 was simulated in PARAMICS. The network was calibrated and validated using loop detector data at every 5 minutes for flow and speeds. The calibration effort resulted in a network whose results provide high fidelity and accuracy for testing the scenarios at hand using statistical surrogate measures for risk of crash. Ramp Metering and VSL were tested separately and also evaluation scenarios were created where both of them could be used in a coordinated manner. The whole testing was done taking into account all the possible variables within each ITS application. Ramp Metering algorithm ALINEA was used to meter up to 7 on ramps within the test area.

Based on the results from simulation, useful conclusions could be drawn regarding implementation of Ramp Metering and Variable Speed Limits on congested freeways.

- The effectiveness of Ramp Metering in decreasing risk of crash increases significantly with the increase in the number of metered ramps.
- When implementing a feedback Ramp Metering algorithm like ALINEA, smaller signal cycles and re-evaluation of metering rates at smaller intervals of time works better than longer cycles. This study found that 25 second cycles always performed better than 50 second cycles whenever more than one ramp was metered.
- It is also more effective to use higher cut-offs for critical occupancy for ramp metering. This would mean more effective actuation of ramp meters and help in reducing the delays on ramps.

- An optimally implemented Ramp Metering algorithm, when implemented at multiple ramps results in an improvement of mainline travel times as well as over all network travel times.
- Variable speed limits when implemented in congested situations on a network wide level are effective in reducing the risk of crash. This study concluded that implementing a network wide VSL strategy performs as well as metering 3 ramps in the same network section.
- The best VSL strategies for implementing network wide included increasing the speed limits downstream of the locations of interest by 5 mph or decrease speed limits upstream by 5 mph and simultaneously increase speed limits downstream by 5 mph. This change could be re-evaluated every 5 minutes based on real time conditions.
- Although Ramp Metering was found to be more effective as a risk reducing strategy, the strategy of using Variable Speed Limits and Ramp Metering in conjunction provided significant benefits even when all the 7 ramps were not metered. This could prove to be useful risk mitigation strategy whenever implementing multiple ramp meters is not feasible.
- VSL and Ramp Metering when used in conjunction significantly increase the average speeds at a number of locations and also decrease the variances in speeds. This also translates into better over all travel times which were found to be better than even the case where all 7 ramps in our network were metered.

5.7 Future Scope for Research

This research led to number of useful conclusions for implementing ITS systems like Variable Speed Limits and Ramp Metering with a perspective on improving safety on congested freeways. These systems could be explored further for coordinated ramp metering strategies and an effort made to find the most compatible Ramp Metering and VSL strategy. Other strategies like Route Diversion could be explored individually as well as to complement each other. In the final analysis for this effort the evaluations would be conducted to address these issues. The corridor would be also be expanded to include the complete 36.25-mile corridor used for developing the crash risk assessment models.

CHAPTER 6

EVALUATION OF ITS STRATEGIES: ROUTE DIVERSION AND MULTIPLE RAMP METERING

In the research work described so far, analysis was conducted to examine crashes that occur on Interstate-4 (Chapters 1 through 9—Volume I) and some preliminary evaluation of ITS strategies (Chapters 1 through 5—Volume II) that may reduce the real-time crash risk. The research team first developed statistical/data mining models that determined the measures of crash risk on the Interstate-4 corridor in Orlando, FL. These models used logistic regression to identify the crash potential for collisions (Abdel-Aty et al, 2005; end of Chapter 4—Volume I in this report) along the freeway based real-time traffic conditions. Using micro-simulation, several Intelligent Transportation Systems (ITS) strategies were tested on a simulated version of the freeway to determine the effect of those strategies on the real-time crash risk (Chapters 2 through 5). The work described in Chapter 2 and 3 showed (also see Dilmore, 2005) that implementing variable speed limits along the freeway is able to successfully reduce the real-time crash risk in situations at which the freeway is operating at moderate to high speeds. The work demonstrated in Chapter 4 and 5 (also see Dhindsa, 2005; Abdel-Aty and Dhindsa, 2007) explored the benefits of ramp metering and found it to be beneficial when the freeway was operating in congested conditions. The coordinated use of variable speed limits and ramp metering was also examined and it was found that combining the two methods reduces the number of ramps required to be metered to reduce the real-time crash risk along the freeway.

The research conducted in this study expands on the previous research in many ways. First, this research uses newer crash risk assessment models developed by Pande and Abdel-Aty (2006) (The models shown in Chapters 5 and 7—Volume I). These

models use neural networks to calculate the risk of both rear-end and lane-change crashes individually. They are more sophisticated than the logistic regression models used previously as they consider both real-time traffic data as well as off-line information regarding the time of day and location of ramps and curves. The crash prevention strategies that are employed in this study are tested to see which of the crash risk types are reduced, if not both. Second, this research expands on the ITS measures that were implemented previously. In the next two chapters, both ramp metering and route diversion (a type of ITS strategy that involves using Advanced Traveler Information Systems – ATIS – to give drivers real-time information about the roadway conditions so they can make better route choices) are considered. This research expounds on the ramp metering strategies that were tested in last two chapters by comparing uncoordinated and coordinated ramp metering strategies. This research also compares the two different implementation methods of ramp metering – traffic-cycle realization and one-car-per-cycle realization. Additionally, this study uses the results from ramp metering in a localized area to make recommendations about metering ramps throughout the entire network to reduce the respective crash risk values for the entire network corridor. Last but not the least, the work presented in this research expands the simulated corridor to include the complete 36.25-mile corridor instead of a probe section simulated for the preliminary analysis in the last four chapters. Therefore, the objectives of the research described in the remainder of this report are as follows:

- Examine Route Diversion and Ramp Metering to determine their affect on both the rear-end crash and lane-change crash risk.

- Compare the uncoordinated ALINEA ramp metering strategy with the coordinated Zone ramp metering strategy to determine which reduces the crash risk measures with the greatest efficiency
- Compare the two implementation methods for ramp metering (traffic-cycle and one-car-per-cycle) for both the ALINEA and Zone metering algorithms to determine which combination most effectively reduces the two crash risk values across the network
- Examine the negative impacts of both crash prevention plans on the operational capabilities of the Interstate-4 system
- Make recommendations for both route diversion and ramp metering to be implemented along Interstate-4

6.1 Study Area

The study area for this work was a portion of Interstate-4 that runs through Orlando, Florida. Interstate-4 generally runs in an east-west direction from Daytona Beach, Florida to Tampa, Florida, bisecting Orlando, Florida along the way. The specific area that was modeled for this study was the 36.25 mile segment that runs through the downtown Orlando metropolitan area. Although the movements along I-4 are described as east-west, this segment actually runs north-south through the heart of the downtown area starting at S.R. 192 in the southwest and ending just north of Lake Mary Blvd in the northeast. Figure 6-1 (below) shows a map of Orlando with the Study Area highlighted. In Figure 6-1, the downtown area is outlined with a dotted line. This is the area that will be mostly affected by route diversion and ramp metering. Note that the corridor for this

part of the simulation project is the whole corridor used in crash data analysis instead of 9-mile section used so far for simulation.

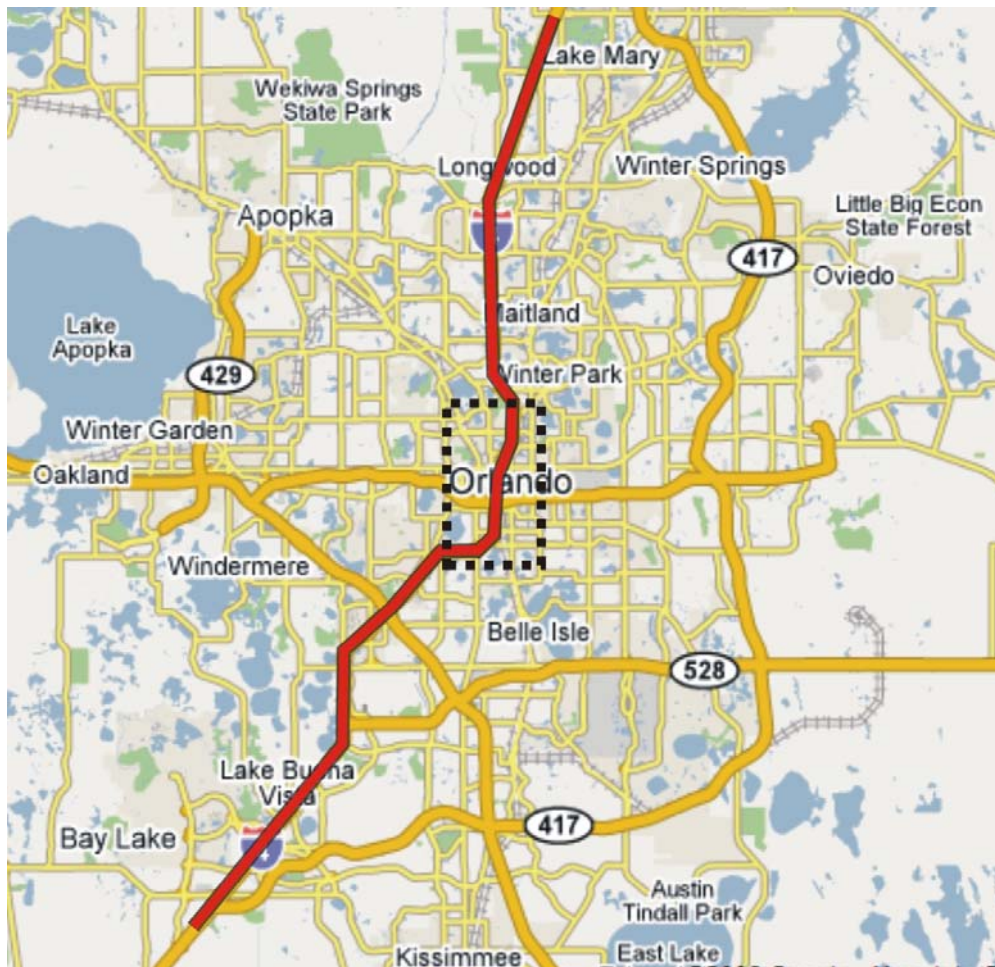


Figure 6-1: Map of Orlando Showing I-4 (Study Area)

For the majority of its length through downtown, I-4 is a 6-lane freeway (although in some areas the freeway is either 4 or 8-lanes wide) with 12-ft lanes and speed limits varying between 50 mph and 65 mph. The speed limit throughout the downtown area, the heart of the network, is the lowest (50 mph) while the speed limit generally increases with the distance from downtown. The composite AADT as defined by the Florida Department of Transportation to give an estimate of the amount of traffic that exists on I-

4 through the Orlando region is approximately 183,000 vehicles/day (FDOT, 2003). This composite AADT value is an average AADT from different locations along I-4 through Orlando.

6.2 Network Building

The following sections describe in detail the process that was completed to code the I-4 network into PARAMICS. The steps involved, in order, were:

- 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
- Implementation of Route Diversion
- Implementation of Ramp Metering

6.2.1 Overlay generation

In order to accurately code the freeway geometry an overlay of the roadway was needed. The previous network created by Dilmore (2005) (See Chapter 3), which studied a smaller section of the same freeway corridor, 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. This aerial photograph was also

combined with AutoCad drawings obtained from the Orange County Transportation GIS department (Dilmore, 2005).

While this process was sufficient for the study area used in Chapters 2 through 5; the study area for the present investigation was significantly larger. Therefore, the remaining geometry was obtained from aerial imagery obtained from the Google Earth Aerial Mapping program. This program allows users to have free access to aerial images of several heavily populated areas throughout the world. The aerial images in the Google Earth program available for Orlando, Florida were taken during April 2002.

Obtaining a single aerial for the entire study area was not practical as it did not have a sufficient resolution to see detailed portions of the roadway. Therefore, 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 when extracting the aerials from Google Earth to ensure that the graphical scales of all individual pieces were the same. Once the entire network was captured using individual aerials, they were overlaid against the low resolution image to ensure that there were no errors with the individual components that were used and that the roadway alignment was correct.

Since PARAMICS only accepts *.dxf 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 given below in Figure 6-2. Please note in Figure 6-2 the two

sample grids that were drawn next to the aerial. Each box in the grid represented a 500 ft x 500 ft square and was used in the scaling of the aerial in PARAMICS.

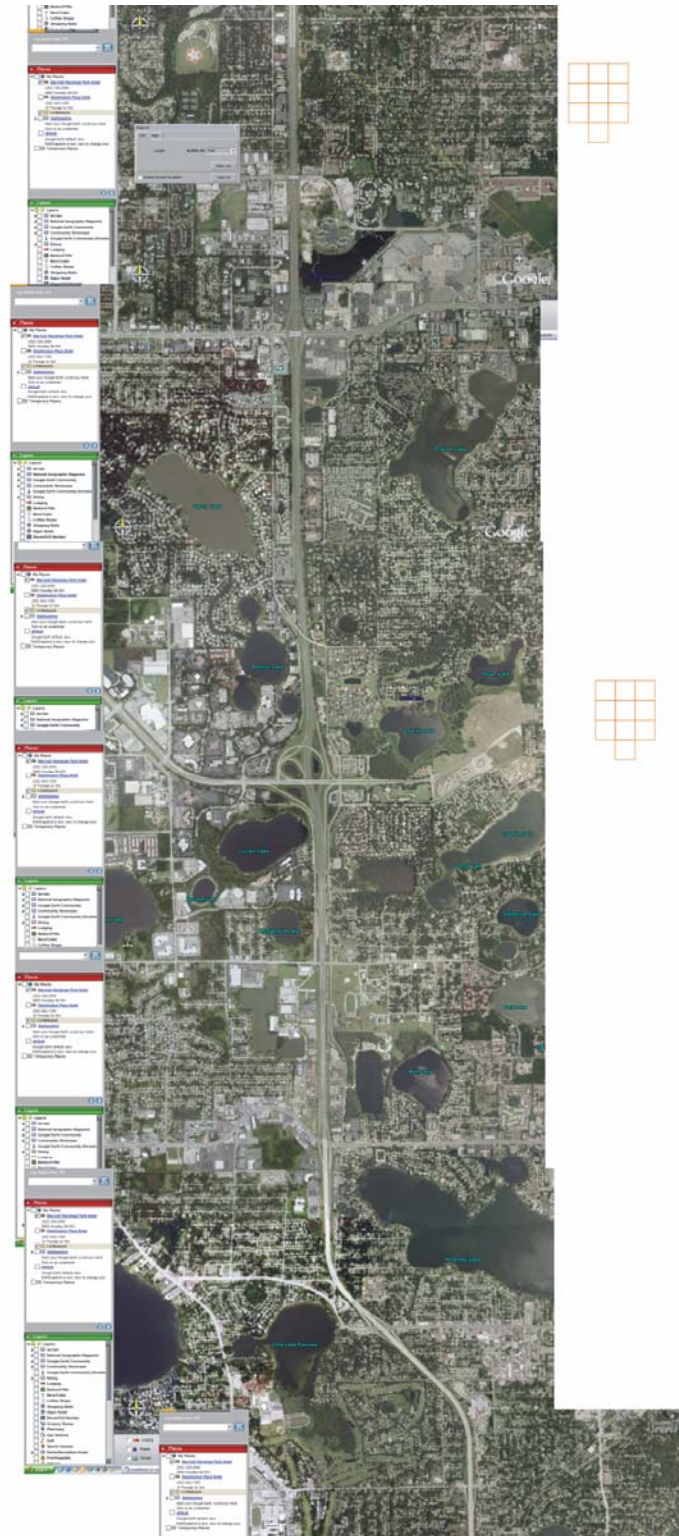


Figure 6-2: One of Seven Distinct Sections of Aerial Used in Network Construction

Once each segment was imported, it was scaled individually (using the grid boxes seen in Figure 6-2) and then reassembled in the PARAMICS program to form the master aerial. This was then 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.

6.2.2 Two One-Way Roads vs. One Two-Way Road

It was decided at this stage to code the eastbound and westbound directions separately as two different one-way roads rather than a single, two-way roadway. This choice was made for a few different reasons. First, and more simply, 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.

The second reason requires a more intimate knowledge of how a network is coded in PARAMICS and how vehicles behave in the network. A PARAMICS network is a series of “links” or segments of roadway that have homogenous properties (number of lanes, speed limits, etc). Each link is defined at the extremities by two “nodes.” These nodes designate the starting and ending point of a link. Therefore, changes in the roadway geometry, such as a lane addition or drop, is defined by a node which separates the two links with different properties. When a link merges with another link, such as a ramp entrance point, another node must be created to define this point. Vehicles in PARAMICS are only capable of making lane changing and routing decisions for the next two links it will encounter. If one two-way roadway is used to define the freeway, this will essentially double the number of links that will be used in either direction of freeway; the network will also have to include many extremely short links to properly

place the entrance ramps. Additionally, each on-ramp is given a value for the awareness distance. This awareness distance defines how far upstream of the ramp vehicles on the mainline recognize the existence of the ramp and merge left in order to allow vehicles on the ramp to merge into the traffic stream. 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 as there 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. Lastly, when a single roadway is used the default PARAMICS options would allow vehicles traveling in the eastbound direction to exit using off-ramps in the westbound direction and vice versa. Clearly this does not occur in the field as vehicles can only use off-ramps for the particular direction of the freeway that they are using. Although this could be turned off manually by editing the junction properties, this would have to be set for every single on-ramp and off-ramp which is a very tedious process. Using two one-way roadways will counter this problem without any extra coding effort.

The only drawback to using two one-way roads to code the freeway is that it neglects the presence of “rubber-necking” or the reduction in capacity on one direction of the freeway due to situations that occur on the other direction. The rubber-necking phenomenon is especially important when incidents occur on the network as it has been proven to reduce the capacity of the freeway by as much as 12.7% (Masinick and Teng, 2004). However, for this study this effect was assumed to be negligible since no incident occurred during the simulation.

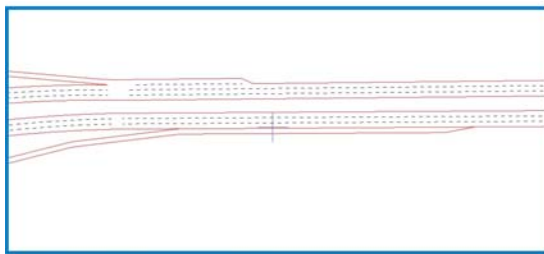
6.2.3 Building Network Nodes and Links

Once it was determined to code both directions of the freeway separately, the nodes that defined the links for each direction were 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. These different nodes were then connected with links which represented the roadway that the simulated vehicles would use. Each roadway link was assigned to a category that contained information about the number of lanes and speed limit on the link. 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.

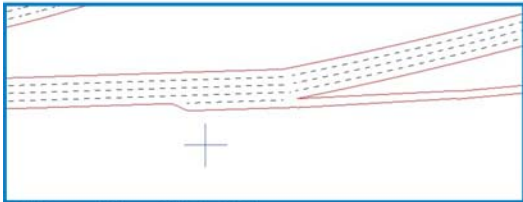
6.2.4 Creating ramps

Another decision that had to be made was the coding of the individual ramps. PARAMICS includes a special function to code an on-ramp and another to code the deceleration lane that typically precedes an off-ramp. However, these two functions can

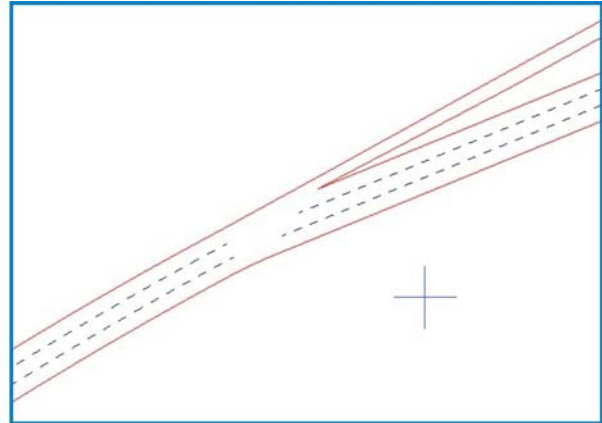
only be used in specific cases which do not represent the gamut of ramp types found on I-4. Therefore, most of the ramps in the network that could use the on-ramp or slip lane function were coded using these functions while the rest were coded as a regular merge or diverge area. Figure 6-3 shows a brief summary of when the PARAMICS on-ramp or slip lane functions were used and when the merge / diverge areas were chosen.



On-Ramp Using RAMP Function

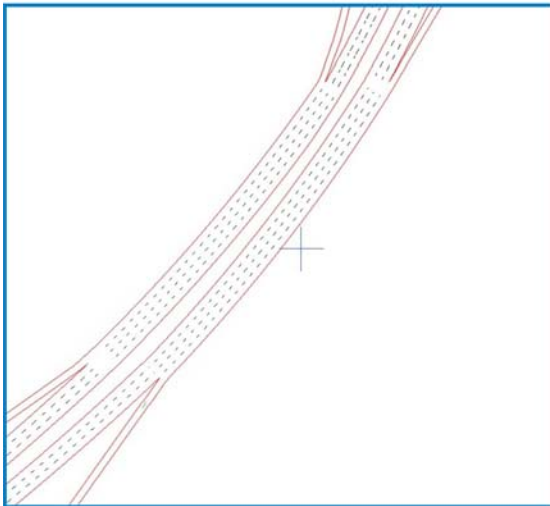


Off-Ramp Using SLIP LANE



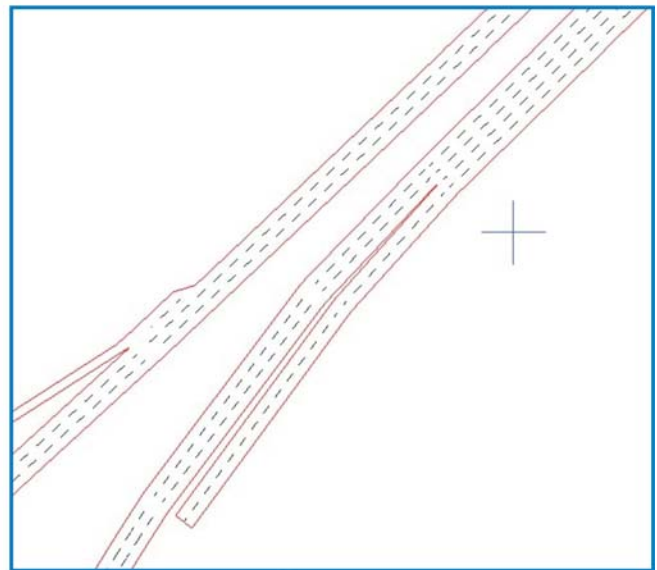
Off-Ramp Created Using DIVERGE Lane

(NOTE: SLIP LANE cannot be used since the off-ramp is on the left hand side of the freeway)



On-Ramp and Off-Ramp Pair Connected Using Traditional MERGE / DIVERGE Lanes

(NOTE: RAMP Function or SLIP LANE cannot be used since there is a weaving lane that connects the two ramps)



On-Ramp Created Using MERGE Lane

(NOTE: RAMP Function cannot be used since the on-ramp is a two-lane ramp)

Figure 6-3: Types of Ramps Encountered and How They Were Coded in PARAMICS

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 adjusting the signpost distance for the

ramp. Increasing the signpost distance for the ramp increased the awareness of the mainline vehicles to the ramp and caused the mainline vehicles to change lanes to give vehicles on the on-ramps 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.

6.2.5 Adding Zones and Vehicles

Once the geometry was completed, the zones that defined the origins and destinations of the network were created. For the freeway portion, defining the origins and destinations were rather simple as each on-ramp represented an origin and each off-ramp represented a destination. 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 fixed by adjusting the nextlanes function which defined which lanes were available for vehicles traveling from one link to the next on a specific lane.

6.2.6 Loop detectors

One of the reasons that this section of freeway was chosen was because the specific section of I-4 that is used in this study contained induction loop detectors embedded in the asphalt. These loop detectors were spaced approximately every 0.5 miles apart on the roadway and gave 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.

As previously mentioned, throughout the majority of the study section Interstate-4 is a 6-lane freeway. However, at some sections the roadway was widened to eight lanes. In these locations loop detectors exist in each lane but only information from three lanes is archived. These three lanes are considered to be the three “mainline” lanes. In most instances, these mainline lanes are the left-most lanes since vehicles typically enter and exit from ramps on the right-hand side of the freeway. However, on Interstate-4 there are a few locations where left-hand ramps are present. For these locations, the three mainline lanes are considered to be the right-hand most lanes. In order to simplify the network creation procedure, loop detectors in the network were built over all lanes and the mainline lane issue was resolved in the post-processing procedure. As will be explained, the post-processing procedure involved converting the PARAMICS output to the loop detector data format that is archived. In this process, only the mainline information will be converted to ensure that the data obtained by loop detectors in the network is equivalent to that found in the field.

6.3 Network Calibration and Validation

Typically, the most tedious process in creating any micro-simulation network is the Calibration and Validation. 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; Abdullhai et al, 2002; Lee et al, 2004).

6.3.1 Previous calibration procedure

In the work described in Chapter 5, approximately 20-mile subsection of I-4 (contained within the 36 mile corridor modeled in this study) was created in PARAMICS and used to test various ITS strategies (locally coordinated ramp metering and variable speed limits). An exhaustive calibration procedure was carried out to determine the values of four calibration parameters: mean driver reaction time, mean driver 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 the merging of a vehicle 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. This calibration procedure used 5-min vehicular flows as well as 5-min vehicular speeds as factors used to calibrate the network. Although the use of flows has been seen in previous studies, this was the first use of flows at such short time intervals (5-min) as well as the use of 5-min speeds.

Since the corridor used in last chapter is incorporated within the 36.25 mile corridor created for the final analysis, and also because the calibration procedure developed in the Chapter 4 was so comprehensive; the same procedure is used to represent traffic behavior in our final analysis. It is unlikely that the addition of just over 16 miles of the network would change the driver behavior significantly to alter the values of the calibration parameters that are needed to be used. The following sections will give a brief overview of some of the steps taken during the previous calibration procedure for the sake of continuity. More information, of course is available in Chapter 4.

6.3.1.1 Calibration of O-D Matrix

In order to estimate the OD Matrix that would be used to define vehicular demand on the network, the study by Dhindsa (2005) used traffic data obtained from the 2003 Florida Traffic Information (FTI) CD. Hourly ramp volumes were also available from the Center of Advanced Transportation Systems Simulation at the University of Central Florida.

Since the basic freeway segment contained two one-way roads, there were some simple rules that were used in the OD matrix estimation. First, all rows that held the origin data for zones on off-ramps were given a value of zero as no off-ramp could

produce vehicles onto the freeway. Second, all columns that held destination data for on-ramps were treated similarly as vehicles could not end their trip (exit the freeway) at an on-ramp. Additionally, ramps from one direction of the freeway could not have trips that originated from or were destined to ramps in the other direction of the freeway. Therefore, these pairs were given a value of zero in the OD matrix. Last, all cells that represented a trip from an on-ramp that was located downstream of an off-ramp was given a value of zero since a vehicle could not travel backwards on the freeway (Dhindsa, 2005). The rest of the values of the origin-destination matrix were calculated using a simple gravity model. Using the gravity model showed some differences between the sum of the columns and the known ramp volumes at that ramp. This was corrected using the following steps:

- If the sum of a row or column is either both too high or too low the count is adjusted appropriately to change this
- When the sum of a column is too high the count is lowered
- If both the sum of a column and row value for a cell is too low the count is increased
- Steps 2 and 3 are repeated until the error is minimized

It should be noted that the previous steps were performed at all ramps excluding the East-West Expressway (S. R. 408) ramps since the majority of error was expected to be found at this location. Once the errors at the other ramps were minimized, the S. R. 408 ramps were adjusted. The final error of the OD matrix, found by comparing the simulation on-ramps and off-ramp volumes with those found in the field was 4.10%.

6.3.1.2 Calibration of Flows and Speeds

The simulation was run and the volumes on the mainline were compared to data obtained from the loop detectors embedded in I-4. The flows in the simulation were checked against the average flows taken from a 20-day period between September 23, 2003 to November 20, 2003 (note: only data obtained from Tuesdays, Wednesdays, and Thursdays were used). Initially, the mainline flows were lower than what was expected by the loop data. The flows between the end zones were increased and re-run to get a more adequate value. A sample of the comparison between the actual flow and simulated flow on a 5-minute basis for a group of 6 stations is shown below in Table 6-1. Note that this table does not show the best case but is just a sample of the tables used to determine which set of calibration parameters yield the best results. Note that this issue was discussed in detail in Chapter 4 and here only a summary discussion is provided for the sake of continuity.

Table 6-1: Difference between 5-min flows for Simulated and Real Data from Dhindsa (2006) (Also see Chapter 4)

5 min. Time Interval	Detector 33	Detector 34	Detector 35	Detector 36	Detector 37	Detector 38
1	69.5594	72.5338	51.8638	51.5588	129.5172	164.1730
2	46.7231	59.1504	24.9548	57.2571	128.3921	150.6219
3	20.3438	59.2487	41.9722	61.9213	120.5060	138.7657
4	20.9601	48.3512	23.5370	52.8326	102.1730	120.8567
5	-20.5063	5.0061	-18.9889	3.6741	47.3865	71.4978
6	7.1344	0.1267	-30.6444	-6.2352	48.4093	77.1837
7	-31.2777	-25.3324	-34.8509	1.8475	51.6759	73.6704
8	-27.3818	-26.5561	-49.1922	-32.1415	39.6663	78.2341
9	-75.9589	-68.6024	-85.1784	-11.0970	45.6919	48.7433
10	-84.1765	-55.1797	-43.9852	-24.4583	28.4913	62.6543
11	-82.7436	-65.7757	-61.2659	10.4417	35.9804	58.0231
12	-85.7216	-29.1626	-36.4718	17.5000	71.5478	94.7971
13	-50.3237	-20.7685	13.5167	51.7796	60.0415	61.1008
14	-6.8202	25.9179	-4.5537	12.6169	6.2121	26.1900
15	-20.5017	-15.5427	-59.3002	-6.5659	39.6744	83.0488
16	-80.5463	-49.8531	-15.6204	33.0861	44.5268	51.8855
17	-42.2757	6.3500	-2.5989	-9.7932	11.1174	35.8532
18	-36.6879	-36.5705	-48.3998	-39.7924	13.9457	47.0101
19	-105.9332	-77.9436	-44.9802	-11.4575	36.5701	81.7641
20	-103.8652	-61.0370	-41.5275	-6.0697	50.9495	102.6571
21	-57.7538	-14.9252	-8.5558	-5.0798	58.9383	93.2179
22	-62.6049	-30.1087	-18.3831	-15.0128	44.4453	100.8202
23	-12.5058	-33.5481	-33.0565	-10.7173	47.9923	87.5548
24	-40.1569	-47.8800	-28.6787	-32.1049	53.9039	118.1519
25	-19.2656	-56.0143	-27.5900	16.8254	67.9889	121.8310
26	11.0968	-5.1175	14.2253	-26.2165	41.8273	120.7842
27	-8.3443	-30.9317	-37.0866	-20.2100	52.8059	109.0692
28	-31.0426	-52.6972	-34.9500	-31.8696	76.5455	124.3371
29	14.2661	-33.0025	-24.1252	-24.1400	51.4606	96.0697
30	41.7084	-12.2174	-33.3755	-49.8579	37.5833	94.8364

Additionally, plots were made that compared the average simulated speed and the average speed from the loop detectors at 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 6-4.

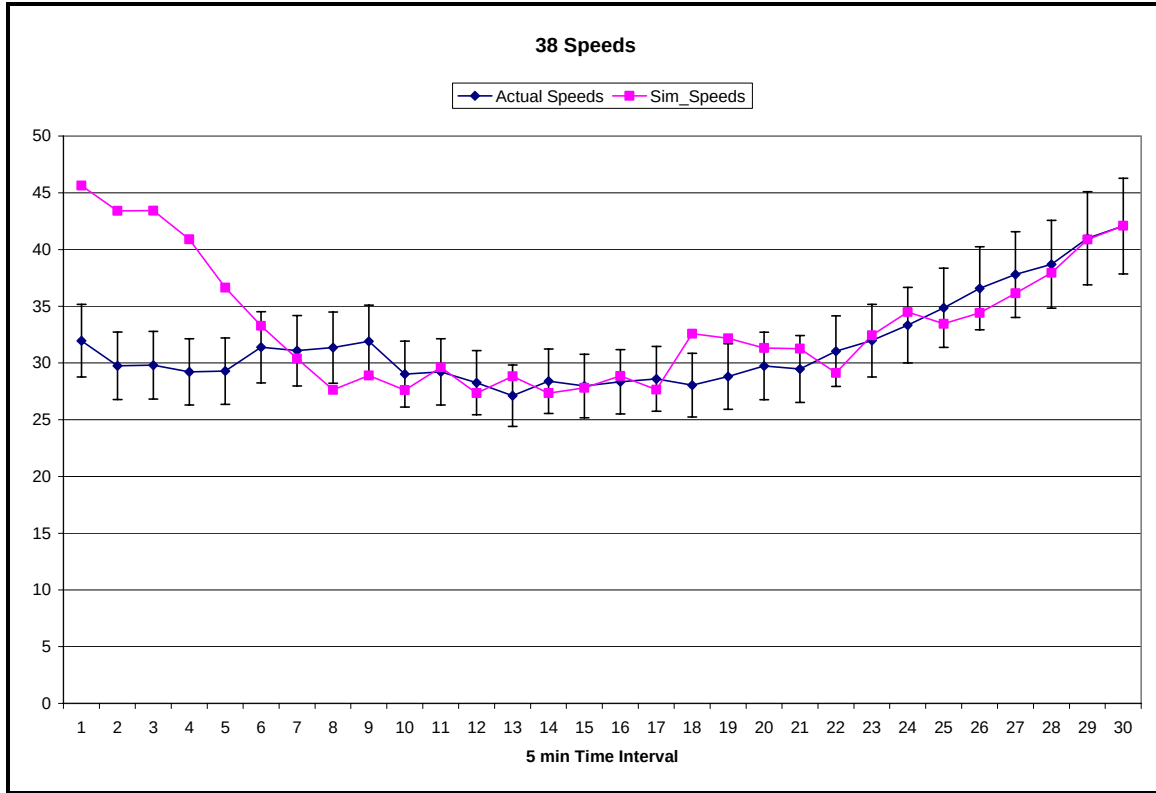


Figure 6-4: Sample Plot Showing Real Average Speeds with 10% Error Bars and Simulated Speed from Dhindsa (2006) (Also see Chapter 4)

At this point, various values of the mean driver headway and mean driver reaction time were selected and tried 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 chosen 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 analysis described in

Chapter 2 (Also, see Dilmore, 2005) and, therefore, found to be acceptable in the calibration of driver behavior along Interstate-4 in Orlando, Florida.

6.3.2 Origin-destination matrix

As previously mentioned, these same calibration parameters from Chapter 4 (Also see Dhindsa, 2006) were used in this study since the two studies have essentially the same study area. However, since the network used in this study is larger and contains several new on-ramps and off-ramps the origin-destination matrix for this study had to be recreated.

The procedure used to create the origin-destination matrix was similar to the procedure used in Chapter 4. Rows holding origin data for off-ramps were given a value of 0. Columns containing destination data for on-ramps were also given a value of 0. Last, values below the diagonal (which represented a vehicle traveling from an on-ramp downstream of a particular off-ramp) were given a value of 0. This minimized the number of cells that needed to be calibrated.

Using traffic data obtained from the Florida Traffic Information (FTI) 2003 CD and hourly data obtained from the Center for Advanced Transportation Systems Simulation (CATSS) as well as the origin-destination matrix that was used in Chapter 4 as a starting point, the O-D matrix for this study was computed using a simple gravity model. The resulting OD matrix was then input to PARAMICS and the network checked against expected queues. From basic field observations it was noticed that there were large queues that took occurred at the downtown areas during a majority of the simulation time period. However, using the OD matrix that was created from the gravity model did not reflect this observation. Instead, vehicles tended to move freely and the queues that

did form were not reoccurring queues formed by congestion but were instead short-term queues that involved 4 or 5 vehicles and dissipated very quickly. The OD matrix was then modified to include more vehicles at the Interstate-4 / S.R. 408 interchange and at the terminus zones of the network to increase the flow and induce congestion. This resulted in the formation of queues around the Interstate-4 / S.R. 408 interchange and a traffic flow that was more consistent with the field observations that were done.

6.3.3 Network validation

Once the OD matrix was determined, the next step was to perform a brief validation procedure to confirm that the traffic flow in the simulation closely resembled the traffic flow in the field. Field data for the validation procedure was taken from a database that contained loop detector data from Interstate-4. Loop data for a period of 20 weekdays, excluding Mondays and Fridays, were extracted from the loop detector database. The period during which data was extracted was from September 23, 2003 to November 20, 2003; this was the same time period that was used in the validation procedure in work done in Chapter 4. Please note that due to periodic loop failure data was not available for all loop stations during this time period.

Flow data from 20 selected stations were then compared against the simulated flows in the PARAMICS network. The comparison was performed by computing the GEH statistic. The GEH statistic is a measure that is essentially a modified chi-squared statistic that takes into account both the absolute and relative difference in the observed and simulated traffic data. This statistic is widely used by researchers working with PARAMICS (Oketch and Carrick, 2005). The GEH statistic is calculated at each location as follows in Equation 6-1.

$$GEH = \sqrt{\frac{(M - O)^2}{1/2 * (M + O)}}$$

...Equation 6-1

Where M = the hourly flow rate obtained from the simulated network and O = the hourly flow rate obtained from the field. Generally, a GEH statistic less than or equal to 5 represents a good fit while a GEH statistic between 5 and 10 could still be considered good but requires further investigation. A value of the GEH greater than 10 implies that the simulated flow rate is not a good fit of the observed data.

The 20 stations for which flows were compared were selected based on the availability of the loop data and based on the need to get a good representation of the entire traffic network with a focus on the downtown area (since this area will be the area mainly affected by the route diversion and ramp metering). Simulated data was averaged for 10 PARAMICS runs and the hourly volume was determined at each station selected and then compared to the loop data that was acquired from the loop database. Table 6-2 (below) gives a summary of the 20 stations selected, the actual and simulated volumes, and the GEH statistic at each location.

Table 6-2: Comparison of Observed vs. Simulated Flow Rates

Station	Flow From Loop Data (veh/hr/lane)	Simulated Flow (veh/hr/lane)	GEH Statistic
7	932	700	8.14
18	1102	1031	2.17
19	1104	1031	2.25
20	1129	959	5.26
21	1073	954	3.76
22	1183	1006	5.35
23	934	933	0.01
24	1011	920	2.91
26	914	994	2.59
30	1380	1038	9.82
34	1317	1316	0.01
35	1304	1333	0.80
36	1331	1251	2.22
37	1342	1494	4.03
41	1550	1748	4.87
42	1535	1706	4.24
43	1870	1884	0.33
44	1520	1917	9.57
49	1840	1791	1.15
50	1849	1987	3.16
52	1532	1581	1.24
53	1549	1483	1.69
56	1495	1514	0.50
57	1411	1257	4.22
61	1389	1250	3.83

Based on this comparison, 80% of the loop stations have a GEH statistic less than 5. Additionally, the average GEH value for all stations considered is well under 5 at 3.36 and no station has a GEH statistic greater than 10. Therefore, this shows that the origin-destination matrix, which was calibrated to induce queues at the downtown locations, actually provides a good fit of the observed field data in terms of hourly volumes as well. This validates the network as providing an accurate representation of the field data and should allow conclusions to be made as the result of the implementation of various strategies to the network.

6.4 Implementation of Route Diversion

The implementation of the route diversion into the PARAMICS network was a two-fold process. First, the alternate routes that were to be simulated had to be coded onto the network. Before this was completed, the only route available to vehicles was Interstate-4. The route diversion that is used here does not allow vehicles to bypass I-4 completely. Rather, vehicles forego entering I-4 at one location and travel to a later on-ramp for entry onto I-4. The second part of implementing route diversion involved getting vehicles to use the secondary route. Left to their own network assignment models, the simulated vehicles would travel on the shortest path. On this network, this is always on I-4. Therefore, a method had to be devised to force some simulated vehicles to use the alternative routes. These two steps are further explained in the following sections.

6.4.1 Alternate routes

When initially deciding to implement route diversion on the I-4 network the question of where to include the diversion routes arose. Since the goal of this research is to reduce the crash risk on I-4 the initial thought was to implement the route diversion in such a way that the changes in the crash risk would occur at a location of high crash risk along the freeway. However, this is all dependent upon the availability of an alternate route. Therefore, when selecting the area on the freeway to implement route diversion two factors were considered: the existence of natural diversion routes and an area known to have high risk. Previous knowledge of the area and driving experience in Orlando, Florida gives that the Interstate-4 / S. R. 408 Interchange is an area of high freeway turbulence. This turbulence is caused by the large amount of vehicles that travel from S.

R. 408 (the main east-west arterial road in Orlando) to Interstate-4 (the main north-south arterial through downtown Orlando) and vice versa. During the peak periods, this large merging volume onto Interstate-4 causes considerable congestion which is known to be related to traffic incidents. Therefore, when considering diversion routes, an emphasis was placed on routes that would have an effect on the crash risk at or around this interchange.

To determine the specific routes used in the route diversion, a map of Interstate-4 was visually inspected to determine what the best feasible alternate routes would be. Alternate routes were determined to be roads that formed a practical connector between one I-4 on-ramp and another downstream on-ramp. Three specific routes were found to be suitable through the downtown I-4 area which would directly affect traffic conditions around the Interstate-4 / S. R. 408 Interchange. These routes were defined as Route Diversion 1, 2, and 3. Route 1 is shown in Figure 6-5 as DP-1A and DP-1B. Route 2 is also shown in this figure. Route 3 is shown in Figure 6-6 as DP-3A and DP-3B and is a mirror image of Route 1 which affects the westbound direction of I-4 (Route 1 affects only the eastbound direction).

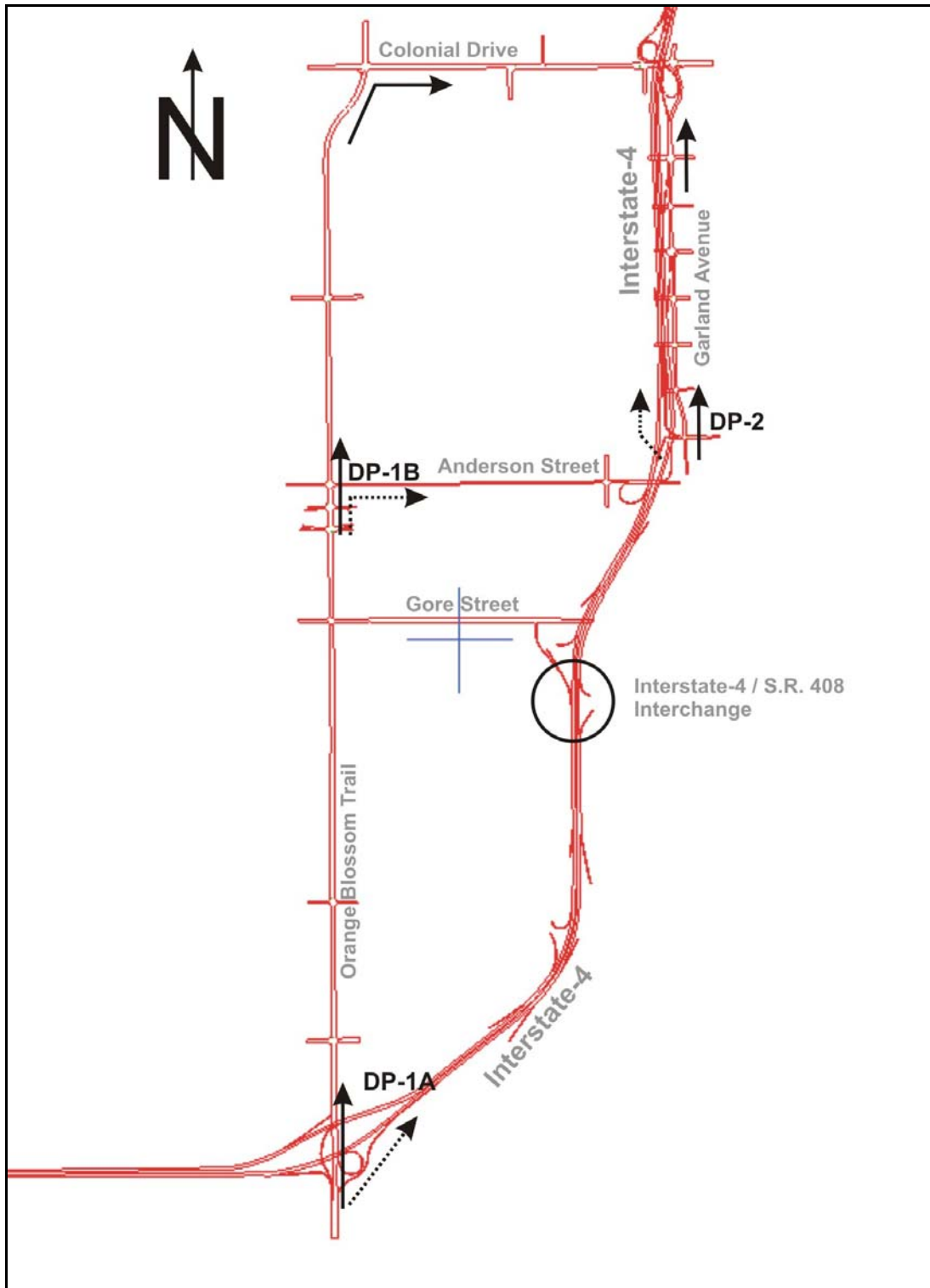


Figure 6-5: Decision Points for Eastbound Diversion Routes

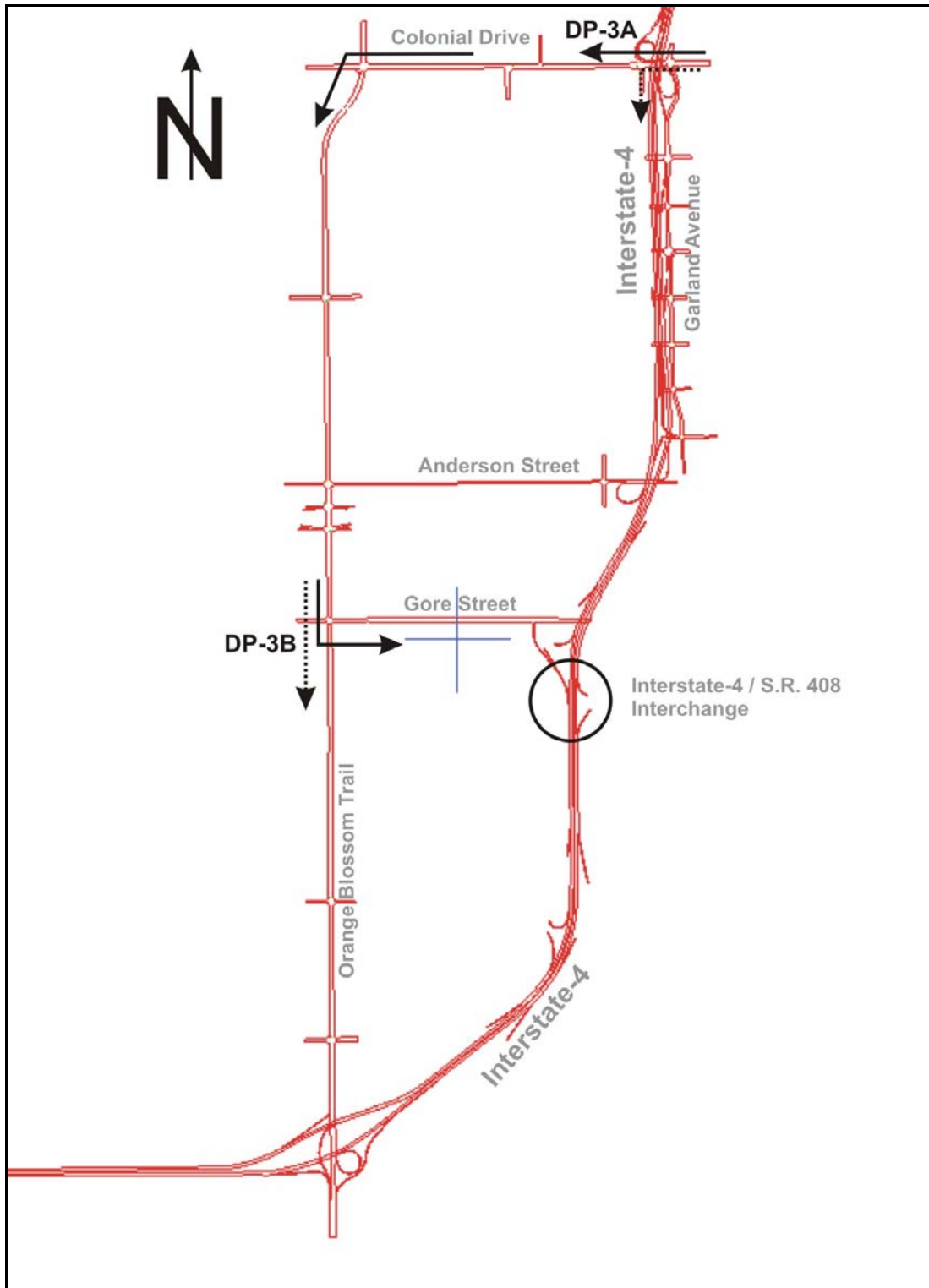


Figure 6-6: Decision Points for Westbound Diversion Routes

Each route has either one or two decision points, depending on the length of the diversion route. A decision point defines whether a vehicle enters I-4 at a particular location or continues traveling for the next convenient entrance point. The decision points are also shown in Figures 6-5 and 6-6 and are labeled as DP-1A through DP-3B. Route Diversion 1 is comprised of DP-1A and DP-1B. At DP-1A, the vehicle must choose between entering the EB direction of Interstate-4 or continue traveling north on Orange Blossom Trail to reach the next entrance point. While traveling north on Orange Blossom Trail, the vehicle will encounter another decision point, DP-1B. At DP-1B, the vehicle can decide to turn east onto Anderson Street and enter I-4 in the eastbound direction by using an on-ramp location further downstream. The alternative to this is traveling north to the intersection of OBT and Colonial Drive, traveling east on Colonial Drive and entering EB I-4 at an on-ramp located on Colonial Drive. This route would essentially divert vehicles from entering the freeway just before the Interstate-4 / S. R. 408 Interchange and have them re-enter downstream of this location.

Another choice is Route Diversion 2 which is rather short compared to the previous route. Route Diversion 2 consists of only one decision point, DP-2. At DP-2, vehicles traveling west on South Street can either enter eastbound I-4 immediately or travel north on Garland Avenue until they reach the I-4 on-ramp further downstream on Garland Avenue near Colonial Drive. This route affects the Interstate-4 / S. R. 408 by diverting vehicles that would enter the freeway just after the interchange (causing more congestion around this area) and require them to enter a little less than 1 mile downstream. By diverting vehicles in this manner the traffic volume is reduced immediate downstream of the I-4 / S. R. 408 Interchange and reintroduced onto the

freeway further downstream which should reduce the potential for a bottleneck at the Interchange.

The previous two diversion routes described affect the eastbound travel direction of Interstate-4. The only diversion route that affects westbound travel is denoted as Route Diversion 3 (shown in Figure 6-6). Route Diversion 3 is also comprised of two unique decision points, DP-3A and DP-3B. Please note that Route Diversion 3 is approximately a mirror image of Route Diversion 1 except it affects traveling in the westbound direction on I-4 as opposed to the eastbound direction (which is affected by Route Diversion 1). At DP-3A vehicles trying to enter I-4 in the westbound direction may do so or traveling west on Colonial Drive and then south on OBT until the next decision point. At DP-3B, the shorter path for vehicles is to continue traveling south on OBT until the WB I-4 on-ramp. However, vehicles may elect to travel east on Gore Street and enter I-4 on the WB on-ramp that is located on Gore Street. Since this is a mirror of Route Diversion 1, this route would halt vehicles traveling in the westbound direction from entering before the Interstate-4 / S. R. 408 Interchange and cause them to enter just downstream of this location.

Coding the Diversion Routes onto the network required the creation of traffic signals in the PARAMICS network. The three diversion routes actually included 29 traffic signals. However, in order to reduce the amount of coding required in the creation of the network as well as to reduce the amount of traffic data needed, only 19 traffic signals from major intersections were considered. The important signals were decided based on both the size of the intersection (all of the larger intersections were included) and field observations as to which signals delayed vehicles the most along the diversion

routes in order to successfully capture that added delay in this analysis. The traffic signal timing data was obtained from the Orange County Traffic Engineering Department.

The signal timing data indicated that most of the signals that were to be included in this network were actuated signals. However, in order to reduce the complexity of the PARAMICS simulation, the signals were treated as pre-timed signals. At the intersection of two major roadways (Orange Blossom Trail and Colonial Drive, for instance) the pre-timed signal timings that were used were set to maximum green time for each direction. Doing this assumed that all approaches of the intersection were saturated (or nearly so) during the peak period. Based on field observations, this was most always the case. At the intersection of a major roadway and minor roadway (Colonial Drive and Parramore Road, for example) preference in the signal timing was always given to the major roadway (Colonial Drive or Orange Blossom Trail in all cases).

Once the diversion routes were created, the next step required was to add vehicles to the diversion routes to approximate the traffic conditions that diverted vehicles would encounter on the surface streets. However, there was a question of how to approximate volumes and delays that are experienced by these vehicles. Unlike the freeway, where loop data was recorded and stored at regular intervals, there was no automatic counter in place along these roads. One idea was to use the AADT (annual average daily traffic) value and K value to approximate the DDHV (directional design hourly volume). However, there is question over the accuracy of these estimates. While they are traditionally used by traffic engineers and planners, AADT's are usually only an estimate from data taken over the course of a few days. Additionally, AADT values are not updated every year.

Therefore, the surface streets were calibrated based on previous knowledge of traffic flow on the diversion routes. The goal of this calibration became to match the long queues at intersections along Orange Blossom Trail and Colonial Drive through the downtown region that is typically expected during the afternoon peak hour on any particular day. Additionally, surface streets within the downtown area (diversion route 2) were always congested during this time period.

6.4.2 Route diversion api

As previously mentioned, there are two decision points that comprise Route Diversion 1. Each of these decision points represent a variable in the simulation in that a vehicle may either choose one of two paths to get onto I-4 from that location. However, after creating the network with the alternate routes and observing the simulation, it was seen that vehicles only took a single path to reach their final destination – the path through I-4. Therefore, a method had to be devised to force some vehicles to divert from their traditional route and to instead travel on the surface streets to re-enter I-4 downstream and, eventually, reach its destination zone.

PARAMICS allows for dealing with multiple paths between zones by the addition of dynamic traffic assignment. However, even when the dynamic traffic assignment is calibrated properly the dynamic traffic assignment method does not allow for specific user control over the amount of vehicles diverted from a particular point. Instead, each vehicle selects a path that will reduce its disutility of travel. Therefore, in order to control the amount of diversion, code had to be written to control vehicular behavior through the use of the PARAMICS Application Programmer Interface (API). The API allows a user to use C programming to write a code that will access and modify certain preexisting

PARAMICS functions during the simulation run. The result of an API can be as simple as reporting the average speed at a location at regular time intervals or as involved as changing the speed limit at a particular location based on specified conditions.

In order to get vehicles to divert from their natural path, an API was written to override the preexisting route choice algorithm in select situations. Since the route choice algorithm is accessed by every vehicle on the network, special conditions had to be met before the route choice algorithm was overridden to ensure that the right vehicles were diverted. The special conditions were the origin zone of the vehicle, destination zone, and current link. The origin zone was used to ensure that only vehicles that are entering I-4 at one of the diversion locations were diverted. The destination zone was used to ensure that vehicles that were diverted to a downstream on-ramp would not overshoot their intended off-ramp and have no logical route to reach their destination once they entered I-4 downstream. Note that if diverted vehicle did overshoot its destination, the vehicle would proceed to the downstream off-ramp, circle back to the original on-ramp and enter I-4 from the original location. Lastly, the current link was specified to ensure that vehicles were only diverted when they were located on the link immediately upstream of a decision point. Not specifying this link caused vehicles to travel in a circle continuously.

Therefore, if a vehicle was traveling from a specific zone near the on-ramp of interest and its destination was at a location further downstream on I-4 than the vehicle's re-entry ramp the vehicle would only divert its route at the link preceding the on-ramp of interest. Additional code was added to control how many vehicles diverted per simulation run. This was done by specifying the percentage of vehicles that would divert

at each decision point. Once each vehicle met the diversion conditions it was assigned a random number between 0 and 100. If that random number was less than the percentage of vehicles that were to be diverted, the vehicle's course was altered to use the route diversion. If not, the vehicle was left to its own route choice algorithm. Therefore, the percentage of vehicles diverting were not exact due to the random nature of the diversion assignment but generally averaged out to the number that was specified. The code for the API is given in the Appendix II-F.

6.5 Implementation of Ramp Metering

Because there were two types of ramp metering that were considered in this analysis, two unique methods of implementing ramp metering had to be used. In order for both methods to be implemented some changes first had to be made to the network. These changes included the lengthening of the on-ramps to include the full storage capacity since vehicles would be queued in the ramp while waiting for the meter. Additionally, traffic signals were added to the merge area on the ramp which would act as the main control (or meter) in the ramp metering process. For the ALINEA ramp metering method those were all the changes that were needed. However, for the ZONE ramp metering algorithm meters were also added to the beginning of each ramp to measure the inflow of vehicles onto the ramp. A basic description of how each method was implemented is in PARAMICS given in the following sections.

6.5.1 Alinea ramp metering

As mentioned in Section 1.4.1, the literature suggests that ALINEA ramp metering algorithm is rather simple to implement. The ALINEA algorithm calculates the metering rate for a particular freeway on-ramp by using only occupancy measurements

taken from the nearest loop detector downstream of the on-ramp in question. If the occupancy is higher than a pre-determined critical value, the metering rate is reduced to allow time for the congestion to decrease before the ramp is open to free-flow.

To implement the ALINEA ramp metering algorithm special controls that are available in PARAMICS to model vehicle actuated signal controls (VA signals) were used. The VA signals allows the program to access data obtained from loop detectors in the network and, based on an algorithm coded by the user, changes the signal timing with changes in the traffic flow. Using this methodology involves the creation of two new network files called the “phases” and “plans” files. The phases file specifies which signal is to be controlled (i.e. which ramp to be metered) and what detectors are to be accessed by the metering algorithm (which are the nearest downstream loop detectors). The plans file defines the algorithm that will be used to change the signal timing. In this file the occupancy values are calculated from the loop detectors in PARAMICS and the metering rate is changed as per Equation 1-3. Because the ALINEA controls are the same for each ramp, the code only had to be written once and was then copied for each subsequent meter. A sample of the phases and plans files for the ALINEA control is given in the Appendix II-F.

6.5.2 Zone ramp metering

6.5.2.1 Zone Ramp Metering Algorithm

As previously mentioned in the review of current literature, the main purpose of the Zone Ramp Metering Algorithm is to balance the number of vehicles entering a

freeway zone with the number of vehicles leaving. This is done by using Equation 6-2 given below (Bogenberger and May, 1999).

$$M + A + U \leq B + X + S$$

...Equation 6-2

where:

M = the on-ramp flow through the metered ramps

B = the downstream bottleneck capacity (1800 - 2100 vehicles per hour per lane)

X = the sum of the measured off-ramp volumes

S = the spare capacity available on the mainline (measure of density and speed)

A = the measured upstream mainline traffic volume

U = the measured sum of the un-metered on-ramps

Looking at Equation 6-2, the only variable that can be altered by ramp metering is the variable M. Therefore, M is a function of the measured in-flow volume, out-flow volume and the spare capacity within the zone. For the Zone algorithm, M will be updated every time period, in this study 5 minutes, to determine the allowable flow rate of vehicles from metering ramps onto the mainline. Once M has been calculated, the metering rate of each metered on-ramp, R_n in veh/hr, is calculated by using Equation 6-3.

$$R_n = \frac{M * D_n}{D}$$

...Equation 6-3

In this equation, R_n is equal to the proposed meter rate for ramp n, D_n is equal to the current flow rate of vehicles entering ramp n, and D is the total number of vehicles wishing to enter the freeway through any of the metered on-ramps in the zone. As can be seen in the equation, the equation basically assigns the total allowable inflow rate for the entire zone, M, to each of n metered ramps proportionally by the volume of vehicles that are using the ramp. Therefore, a ramp that has twice the volume of another ramp should receive twice the green time of the second ramp.

Once R_n is determined, this rate is checked against a predetermined minimum rate. If the proposed rate is less than the minimum metering rate, then the rate is set to the minimum rate. The minimum metering rate is determined based on the length of the ramp and the density of queued vehicles waiting on the on-ramp. The goal of the minimum metering rate is allow vehicles into the freeway stream after a maximum waiting time of four minutes. Therefore, calculating the minimum metering rate is done by determining the number of vehicles that can be queued in the ramp and dividing this value by four minutes. Once scaled to vehicles per hour this will yield the minimum metering rate for the ramp and would represent the rate that would allow vehicles queued on the ramp to enter the freeway after waiting for a maximum of four minutes.

6.5.2.2 Zone Algorithm in PARAMICS

Implementing the ZONE ramp metering algorithm was a considerably more difficult process than implementing the ALINEA algorithm due to the large amount of data required to calculate the respective metering rates. While the ALINEA algorithm

requires just the occupancy reading from a single detector, the Zone algorithm requires that speeds, densities, and flows be known from multiple locations on the freeway mainline as well as the various on-ramps and off-ramps within the metered zone. Even more importantly, however, is the fact that the respective metering rates within a zone are all dependant on one another. If it were not for this fact, the VA built-in signal controls in PARAMICS could still be used to model the Zone algorithm although the code would be much more complicated than the code for the ALINEA algorithm.

Instead, an API was created to control the signals. Using an API is advantageous for a few reasons. For one, using an API allows the code to be written in the C-language instead of the built-in VA control language. Although the VA control language resembles C, there are some functions that are not possible using the VA control language. Because the API is written in C-code, multiple functions can be created in the API, each representing the metering of a different zone. Doing this allows the user to easily turn zones on or off depending on the control strategy being tested. This allows multiple scenarios to be tested much easier than with the VA control method which would require the creation of individual phases and plans files for each combination of control strategies that is to be tested.

6.6 PARAMICS Output

There are two types of output from PARAMICS that will be useful to this analysis. Global output is data taken from the entire network and does not require a location to be specified in order for the data to be collected. This type of network output includes the 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 point

in time, among other measures. The other type of output is specific to a particular location and usually requires the inclusion of a loop detector on the network at that location. Data that can be collected from a particular detector includes flow, headway, speed, occupancy, acceleration, gap, and density (on a particular link – does not require a detector but the link must be specified).

The detectors included in the PARAMICS network were placed in the same locations as the detectors in the field so that the two sets of data could be compared. This is important because the models that will be used to define the crash risk in the simulation were created with the loop data extracted directly from Interstate-4. However, PARAMICS outputs the loop detector data in a manner that is inconvenient for analysis. The loop data that is collected along I-4 is an average of the speed, flow, and occupancy on every lane at 30 second intervals. PARAMICS outputs the speed and occupancy data at every detector at every instant that a vehicle completely crosses the detector. Therefore, the PARAMICS output provides a signal line of data every time a vehicle crosses over a detector. In order for the two to be compared, the output from PARAMICS must be post-processed to convert it from the given form into the form that is available in the field. This is accomplished by using a Visual Basic macro written in Microsoft Excel. This macro essentially reads the data line by line and groups the data into 30 second time intervals and, using those groups, 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 only downside to the macro is the 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. Therefore, the time needed to analyze a single set of 10 runs can be as little as 12 hours to 2 or more days.

Once the output has been converted into the loop data format, there is still one more step that must take place before the crash risk (the variable of interest) can be determined. All models defining crash risk use traffic parameters that have been aggregated into 5 minute intervals. However, the output from PARAMICS (and the real loop data) is given in 30 second intervals. Therefore, another macro was written to group the 30 second data into 5 minute intervals and compute the average, standard deviation, and coefficient of variance for speed, volume, and occupancy. In order to save time, however, this macro does not convert all of the data, only the specific variables that are needed in order to calculate the crash risk measure. These parameters are listed later in Sections 6.10 and 6.12. For this reason, this macro has a much shorter runtime (about 30 to 90 minutes per run) than the first macro. Therefore, a typical set of 10 runs takes about 10 hours to complete. This brings 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. Note, however, that this value changes based on the processing power of the computer used to run the macro and the loading case being performed. 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 at lower loading cases (60 percent loading) with the fastest computer to about three days for higher loading cases (100 percent loading) with the slowest.

6.7 Experimental Design

The purpose of the experimental design is to evaluate the effects of route diversion and compare the effects of different ramp metering strategies on the real-time safety of an urban freeway using the PARAMICS microscopic simulation program. Although micro-simulation packages have been used extensively in studying the effects of various ITS measures on traffic flow, the use of micro-simulation in the field of traffic safety presents the challenge of how to quantify the effects of the ITS strategy. For changes in the traffic flow, measures of effectiveness can include many simple metrics ranging from the traffic volume entering a particular roadway segment or intersection, the total travel time for all vehicles in the simulation, the density on a particular link, etc. These quantities are directly measurable by the simulation program and are often reported automatically upon the conclusion of the simulation run.

However, there is no standard metric which provides the level of safety on a simulated traffic network. Real networks can rely on case studies over a period of time in which the number of crashes that occur at a particular location or section of roadway is used as the variable of interest. Even though this is essentially a flawed approach due to the random nature and human error involved in most traffic crashes, it is still the best approach transportation engineers can use when a large dataset is present. This method allows researchers to determine the areas that are more prone to traffic collisions with respect to the rest of the traffic network. However, it is impossible to include the occurrence of crashes on a simulated network since it is impossible for a computer to determine when and where a crash will occur. Instead, when using simulation packages researchers look to other measurable variables that have a known relationship to traffic

crashes. These surrogate measures of traffic safety do not directly depend on the frequency of crashes and instead reflect the behavior of all the vehicles traveling in the traffic stream.

Some surrogate measures of safety include simple variables such as mean speed or speed variance (Gettman and Head, 2003). Other, slightly more complicated, safety measures described in this work were variables such as Time to Collision and Post Encroachment Time. However, some researchers have spent much time developing models which use these directly measurable values as inputs to describe the safety (or crash risk) on the roadway. These models would then be used to describe the level of safety on a network during the simulation. One such example, from researchers in Virginia, is an equation that compares the actual following distance of a vehicle to the recommended safe following distance (Park and Yadlapati, 2003). Another example comes from researchers at the University of Central Florida who created a measure of crash risk based on a logistic regression model developed using within stratum matched sampling and real-time traffic variables taken from loop detectors embedded in the freeway (Abdel-Aty et al, 2004). This measure was used in studies that assessed the change in the crash risk based on the implementation of variable speed limits (Chapter 3 and Dilmore, 2005) and localized ramp metering (Chapter 5 and Dhindsa, 2005).

However, one problem with using the metric described in Abdel-Aty et al. (2004) is the fact that it describes the crash risk locally for each loop detector station. Therefore, this measure of crash risk is not comparable across different stations. By using these measures one cannot determine which location on the freeway has the highest risk of a collision. To overcome this, more recent research done by Pande and Abdel-Aty (2006)

has created newer models to determine the real-time risk of rear-end and lane-change crashes using neural networks. These models include explicit variables to account for location (instead of implicitly accounting for location) and, therefore, produce rear-end and lane-change crash risk values that are able to be compared across location. This will provide a better picture of the rear-end and lane-change crash risk along the freeway.

This study will focus on these models presented by Pande and Abdel-Aty (2006) to describe the rear-end and lane change crash risk along the Interstate-4 freeway. The following sections will provide more information about the models that are used to calculate the rear-end and lane-change crash risks as well as how the models are implemented in this study.

6.8 Clustering of Rear-End Crashes

Similar to the research that calculated the crash risk based on logistic regression models (Abdel-Aty et al, 2005) (See Chapter 4—Volume I) which that found that crashes occur within two separate speed conditions (high-speed and low-speed), the more recent research by Pande and Abdel-Aty (2006) (See Chapter 5—Volume I) has found that rear-end crashes are more accurately described as occurring within one of two distinct traffic regimes. These traffic regimes cannot be defined as simply high-speed or low-speed, however, and require loop data from multiple locations in the freeway in order to be classified. To describe these different regimes, a classification tree model was created that used freeway speeds at different locations around the station of interest as the input. The two regimes were originally defined using speed data taken from 0 to 5 minutes before the rear-end crash. However, in an effort for the final rear-end crash risk model to be used in real-time in a predictive fashion, the classification tree used to separate crashes

into the two different regimes used speed data taken from 5 to 10 minutes before the crash. More information on the classification tree method can be found in Pande and Abdel-Aty (2006).

The result of the classification tree model was a set of simple if-then statements that used the speed variables to classify the data into seven distinct “leaves” on the tree. Each of these leaves had a different percentage of regime 1 crashes and regime 2 crashes and this probability was used to define the traffic conditions as either regime 1 or regime 2. If a particular leaf had a percentage of regime 1 crashes that was greater than 0.50, then that leaf was assumed to represent regime 1 conditions. Likewise, if the percentage of regime 1 crashes was less than 0.50, then the leaf was said to denote regime 2 conditions. Figure 6-7 below gives a summary of the classification tree rules as well as the associated percentages of regime 1 to regime 2 conditions for each leaf. Please note that the percentage of regime 1 crashes for a particular leaf is also known as the probability that traffic conditions belonging to regime 1. This value was given the variable name “a” and will be used as part of the surrogate measure of safety.

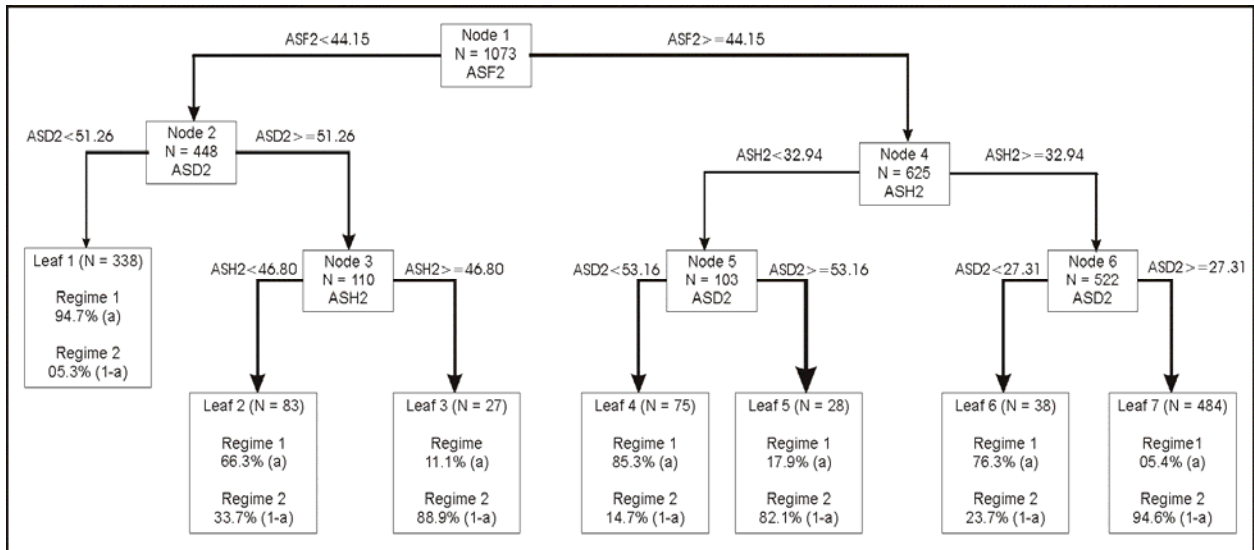


Figure 6-7: Classification Tree to Determine Regime Conditions for Traffic Data
(Chapter 5—Volume I; Also see Pande, 2005)

In order to understand the nomenclature for each variable in Figure 6-7 it is necessary to understand the format of the loop data that are used to calculate these variables. The details are available in Chapter 5 of this report but a summary is provided here for the sake of continuity. The loop data is given as the average speed, volume, and lane occupancy every 30 seconds for each of three lanes on the freeway. The variables listed above are all calculated for a time period of 5 minutes. Therefore, each variable takes into account 30 data points since there are 10 observations for each of the three lanes in the 5 minute period.

The first letter(s) in each variable name describes the measure that is calculated from the 30 data points. “A” represents average, “S” represents standard deviation and “CV” represents the coefficient of variation. The coefficient of variation is defined as the standard deviation divided by the average value. The next letter refers to what measure is being computed. “O” refers to occupancy, “S” refers to speed and “V” refers to the

volume. The final letter refers to which station is used to calculate this value. Figure 6-8 (below) 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 would represent the station 1 mile upstream of the station of interest. Although the classification tree in Figure 6-7 only uses variables of average speed, the other variable terms will be used in the crash risk models described in subsequent sections.

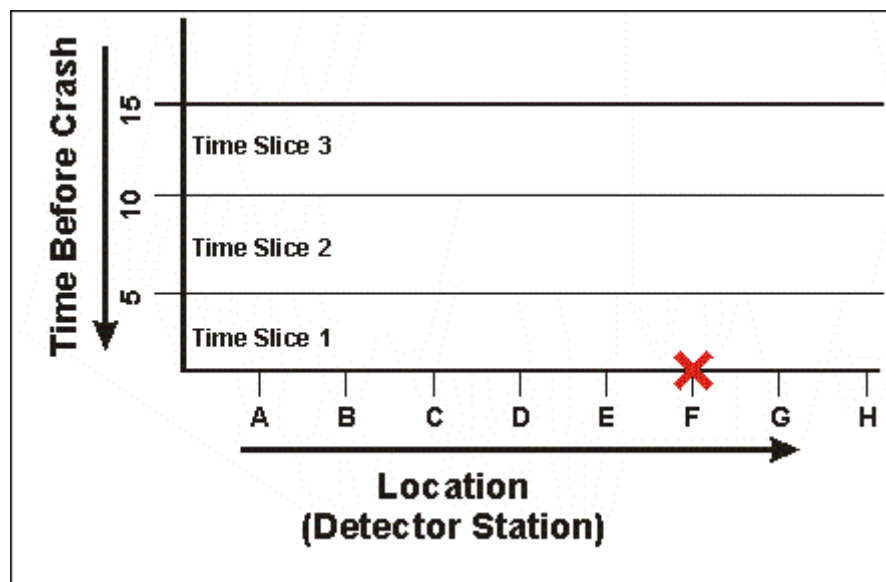


Figure 6-8: Time-Space Diagram Showing Time and Location of Interest

The final number at the end of the variable name refers to the time period over which this value is calculated. 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. Therefore, based on this coding system, the meaning of any of the loop variables above can be determined. For example, ASD2 refers to the average speed 1 mile

upstream of the station of interest, 5 to 10 minutes before the time of interest. Figure 6-9 is provided to summarize the variable nomenclature.

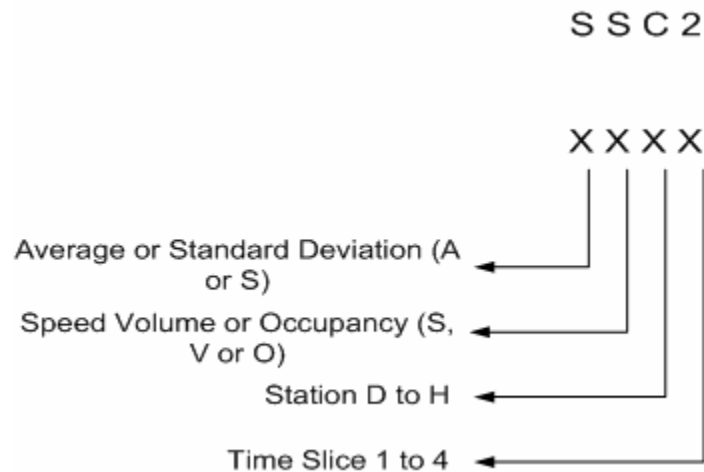


Figure 6-9: Nomenclature of Variables Describing Loop Data

As seen from Figure 6-7, regime 1 conditions are generally lower speed conditions that represent congestion on the freeway. Regime 2 conditions tend to occur at higher speeds and represent un-congested traffic flow. Also, please note that the typical value of “a” for each leaf is not close to 0.50. This shows that the classification tree does a good job of partitioning the data into one of the two traffic regime conditions.

6.9 Crash Frequencies in the Different Regime Conditions

Once the classification tree model was created, the rear-end crash data was sorted into two categories: regime 1 crashes and regime 2 crashes. It was found that 45.8% of all rear-end crashes occur during regime 1 conditions while the remaining 54.2% occurred during regime 2 conditions. Random non-crash data was then subjected to the classification tree and it was determined that regime 1 conditions were only prevalent in 6.3% of the random non-crash data while regime 2 conditions represented the remaining

93.7%. This led to the belief that regime 1 conditions were more risky since nearly 46% of the crashes occurred in situations that were only seen 6% of the time on the freeway. This shows that there is a definite difference between the regime 1 and regime 2 conditions. Initial measures of effectiveness tried to capture this increase in the crash risk during regime 1 conditions.

6.10 Posterior Probability Models

Once the rear-end crashes were grouped into their respective regimes the modeling procedure began. Separate models were created to describe the crash risk in regime 1 and regime 2 conditions since it was likely that these two types of crashes would have different factors associated with the likelihood of a crash. Classification trees were used to identify factors that were significantly associated with rear-end collisions within each respective regime. Factors considered included both on-line factors (loop detector variables describing the currently conditions on the roadway) and off-line data (variables describing location and geometry along the roadway). In the first stage, only loop data taken from the station of the crash was included. However, subsequent analyses included data from three and five loop detector stations around the crash in order to see if this would improve the accuracy of the model. As will be mentioned later, adding loop data from multiple stations was required to improve the accuracy of the regime 2 model.

Once significant factors were determined, models were created using neural networks. A neural network can be described as a parallel-distributed processor that is made up of several independent processing units, or nodes. These nodes are capable of storing data and making it available for use by other nodes. The basic structure for a

neural network involves an input layer of nodes, a hidden layer, and an output layer. Each node is connected to other nodes in the previous and following layer and each connection has an interconnection weight. Training the model is performed by applying the model to a set of data using arbitrary weight values and continually adjusting the interconnection weights in an effort to minimize the difference between the output provided by the model and the actual output. The most important factor affecting the performance of neural networks is the number of nodes that are used in the hidden layer. The modeling procedure performed by Pande and Abdel-Aty (2006) considered using between 1 and 10 nodes (in 1 node increments) in the hidden layer to determine the best case. For each trial, the output of the model was compared by using a measure to capture the accuracy of the model classification. More information on the model building procedure and the neural network architecture that was used can be found in Pande and Abdel-Aty (2006) and Pande (2005).

The final outcome of this was a model to determine the rear-end crash risk when traffic conditions are within regime 1 conditions and another to determine the crash risk in regime 2 conditions. The rear-end crash risk values that are outputted by each model are actually posterior probabilities that a crash will occur given the inputted traffic and offline factors. A posterior probability of a random event (crash) is the conditional probability of the event occurring taking into account the relevant evidence of the dataset used to create the model. Therefore, the output of the regime 1 model is analogous to the probability that a crash will occur for the inputted conditions given the data used to train the neural network model. Please note that the models describe the rear-end crash risk at a particular location and time use loop data taken 5 to 10 minutes before the particular

time of interest. Therefore, if the model is fed continuously with loop data taken in real-time it will essentially describe the crash risk that should be expected on the freeway for the next 5 to 10 minutes. This allows time for crash prevention measures to be implemented if this crash risk measure is used in real-time.

The best model created to determine the posterior probability of a crash during regime 1 conditions contained the following variables:

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

Note that the first four variables are variables derived from the loop detector data. The coding of these variables is given above in the previous section. The other four variables are categorical variables describing off-line information about the location in question. These variables describe the location and proximity of the location to ramps along the freeway. A full description of these variables is given in Table 6-3. Of the variables listed previously, the variable that most affects the Regime 1 crash risk is AOF2 (Chapter 5—Volume I of this report; also see Pande, 2005). Increasing the occupancy at the location of interest serves to increase the probability that a rear-end crash risk occur.

The best regime 2 model created 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 has to do with the differences between the two traffic regimes. Regime 1 generally represents congested traffic conditions. In these situations, only the station of interest needs to be examined since the traffic flow is moving very slowly and there is not much of a speed difference between adjacent stations. In such a case the occupancy at the station of interest becomes very important to describing the rear-end crash risk. However, if conditions are not congested (regime 2) then speeds are much higher and factors across multiple stations are needed to assess the risk of a rear-end crash. Such important factors are the speed difference between upstream and downstream locations as well as the speed variance at different stations. Therefore, more variables are needed from a larger area around the station of interest. The variables used in this model are as follows:

- ASD2
- AVD2
- ASE2
- AVE2
- SSE2
- ASF2
- AVF2
- ASG2
- SOG2
- SSG2

- SVG2
- AOH2
- ASH2
- AVH2
- crashtime
- downstreamon
- upstreamoff
- downstreamoff
- base_milepost
- stationf

The variables with the most influence on the Regime 2 crash risk are ASG2 and ASF2. These two variables are the most important because in un-congested conditions the speed differential (faster moving vehicles approaching slower moving vehicles) contributes to the chance of a rear-end crash occurring. In addition to the loop variables described previously, there are several offline variables needed to determine the crash risk. These variables are: the 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 coding of these variables is given below in Table 6-3. The categories that are created for each variable are not

random and were obtained by Pande and Abdel-Aty (2006) based on the relationship between each variable and the binary target variable (crash or non-crash).

Table 6-3: List of Categorical Variables Used to Determine Rear-End Crash Risk

CRASHTIME =0 if Time of crash between midnight to 12:26 AM =1 if Time of crash between 12:26 AM to 6:46 AM =2 if Time of crash between 6:46 AM to 7:24 PM =3 if Time of crash between 7:24 PM to midnight BASE_MILPOST =0 if $0 < \text{base\_milepost} \leq 13.75$ =1 if $13.75 < \text{base\_milepost} \leq 15.96$ =2 if $15.96 < \text{base\_milepost} \leq 25.74$ =3 if $25.74 < \text{base\_milepost} \leq 36.25$ DOWNSTREAMON =0 if nearest downstream on-ramp is located further than 0.7743 miles =1 if nearest downstream on-ramp is located within 0.7743 miles DOWNSTREAMOFF =0 if nearest downstream off-ramp is located further than 0.6323 miles =1 if nearest downstream off-ramp is located within 0.6323 miles <u>UPSTREAMOFF</u> =0 if nearest upstream off-ramp is located further than 0.3196 miles =1 if nearest upstream off-ramp is located within 0.3196 miles <u>STATIONF</u> =0 if Loop detector station nearest to crash location is located upstream =1 if Loop detector station nearest to crash location is located downstream

6.11 Problems with Assessing Risk Using Regime 1 and Regime 2 Models

As previously mentioned, two different models were created to assess the posterior probability of a rear-end crash occurring within regime 1 and regime 2 conditions. However, one of the major problems with these models is that they are not

comparable to each other. The values outputted by each model would only be comparable to values outputted by the same model since they are not on the same scale. In other words, the risk of a regime 2 crash occurring could be determined for two continuous time slices to compare whether or not the crash risk has increased, decreased, or remained the same. However, this assumes that the conditions for the two time slices are both regime 2 conditions. When the traffic conditions change from regime 1 to regime 2 conditions (or vice versa), the values outputted by the models cannot be used to say that the risk in regime 1 is greater or less than regime 2.

Since this shift between traffic conditions occurs mostly during specific times on the real network (only when congested conditions are formed and dissipated, i.e. the beginning and ending of the peak hour) this problem could possibly be neglected in the field since the change would occur for a more or less known period of time. However, the lack of comparability between the two models presents a problem to the simulation. The purpose of the simulation will be to test various ITS strategies and their effects on the crash risk. The strategies tested will change the rear-end crash risk by altering the traffic flow which can (and does) change the traffic conditions from regime 1 to regime 2 and vice versa. It is essential to know how changing the traffic conditions from one regime to another affects the crash risk. If the two models are not comparable, then there is no way of knowing if the implemented procedure is increasing or decreasing the risk of a rear-end crash on the freeway. Therefore, some method must be used to transform the output of the two models into a single measure that can be used to assess the crash risk. Details about the methods considered are given in the following sections.

6.11.1 Method 1

The first method that was considered to make the output of the two models comparable to each other was to apply a scale factor to the value outputted by one of the models and then compare this to the second model. To determine the value of the scale factor, the probability of observing a crash in each particular traffic condition as well as the probability of observing each particular traffic condition in the field was considered. In general, it was seen that 46% of the rear-end crashes occur during Regime 1 conditions while the remaining 54% occur during Regime 2 conditions. However, using randomly collected traffic data it was observed that Regime 1 conditions are only persistent 6% of the time while Regime 2 conditions are more prevalent and occur in 94% of all traffic cases. Using ratios, Regime 1 conditions are about 13.65 times more risky (on average) than Regime 2 conditions $([0.46 / 0.06] / [0.54 / 0.94])$. Therefore, the initial idea was to multiply the Regime 1 risk (R1) by 13.65 and compare it with the Regime 2 risk (R2). Using this method, Equation 6-4 was used to describe the crash risk:

$$Risk_{-1} = \begin{cases} 13.65 * R1 & \text{regime 1 Conditions} \\ R2 & \text{regime 2 Conditions} \end{cases}$$

...Equation 6-4

The initial thought was that using such a large number would greatly undervalue the importance of the Regime 2 risk. This could have been the case if the output from the models were actual probabilities of crash occurrence within each of the regime. However, the actual output is posterior probabilities which are relative measures of the crash risk and not actual probabilities of crash occurrence. In actuality, using a scale factor of 13.65

did not undervalue the importance of the Regime 2 risk at all due to the scale of the Regime 2 risk vs. the Regime 1 risk. When the values that were outputted by the two models for 10 simulations runs were compared, it was seen that the range of the values outputted by the regime 1 model was extremely different from the range of values outputted by the regime 2 model. Figures 6-10 and 6-11 (below) show the distribution of the outputs from the two models for random traffic conditions. As can be seen, the output of the regime 1 model lies between 0.0 and 0.015 while the output of the regime 2 model lies between 0.0 and 0.60 making them incomparable to each other.

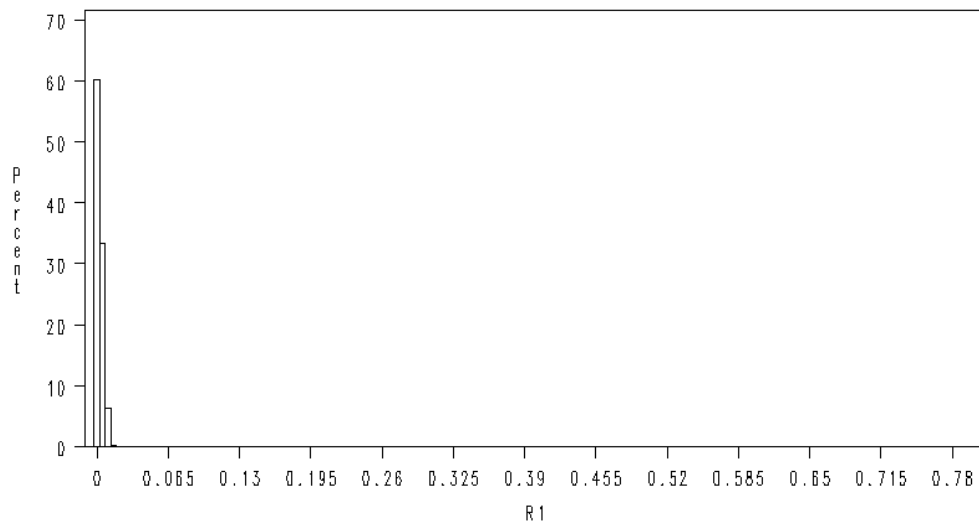


Figure 6-10: Distribution of Regime 1 Model Output

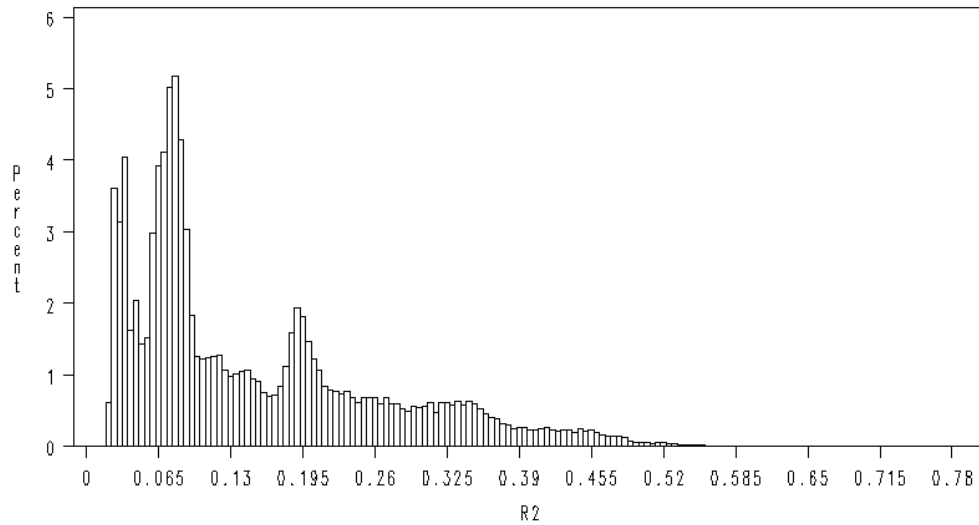


Figure 6-11: Distribution of Regime 2 Model Output

Therefore, applying a scaling factor of 13.65 would only increase the range of the regime 1 risk values to between 0.0 and 0.218. This range is still much smaller than the range of values outputted by the regime 2 model which would mean that regime 2 conditions are more risky than regime 1 conditions. However, since 46% of the crashes occur within regime 1 conditions (6% of the traffic flow) it is expected that the regime 1 risk value would be higher than the regime 2 risk value.

The range of the regime 1 conditions could be artificially increased by applying a larger scale factor. However, the problem still remains of which number to use as the scale factor. Applying too large of a scale factor would overemphasize the importance of the regime 1 risk while applying scale factor that is too small would emphasize the importance of the regime 2 risk. In either case, no other scaling factor is justifiable and since the value of the scale factor is essential to the calculation of the crash risk this method was abandoned.

6.11.2 Method 2

Since such a large number of crashes (46%) occurred during regime 1 conditions, which were typically rare (6% of all non-crash cases), the next method of assessing the rear-end crash risk considered treating all regime 1 conditions as equally risky. This method would assign a single value of the crash risk during regime 1 conditions to represent regime 1 conditions since it would be assumed that all regime 1 conditions are equally undesirable. One problem with this method lies with what value of risk should be assigned to regime 1 conditions. In general, the value of regime 1 risk that is to be selected needs to be higher than any regime 2 risk value (since regime 1 conditions have been shown to be more dangerous than regime 2 conditions). Since the range of regime 2 risk values is between 0.0 and 1.0, selecting a value of 1.0 (upper-bound on the posterior probability) seems the most appropriate. Therefore, the measure called Risk_2 was created (Equation 6-5).

$$Risk\_2 = \begin{cases} 1 & \text{regime 1 Conditions} \\ R2 & \text{regime 2 Conditions} \end{cases}$$

...Equation 6-5

Using this measure would ensure that the regime 1 risk is higher than the regime 2 risk at all times. Therefore, if a crash prevention plan is implemented and the conditions change from regime 1 to regime 2, the risk will always be reduced. However, if the resulting regime 2 risk turns out to be a high value, like 0.91, it would ensure that the improvement that was made is not overestimated.

Note that the value of regime 1 risk chosen should not be influenced by the value of the regime 2 crash-risk value. It was proposed that the equivalent regime 2 rear-end crash risk of regime 1 conditions be equal to the maximum of the rear-end risk during regime 2 conditions. However, this option was ruled out as the maximum would vary for different test cases, loading cases, and starting value seeds. Using 1.0 is independent of the regime 2 risk. It should also be noted that using the value 1.0 does not imply that regime 1 conditions will ALWAYS lead to a crash. Since the probabilities that are determined by the model are posterior probabilities, the value outputted by the crash risk assessment models give a measure of the crash likelihood – not the probability that it will occur. Therefore, using a value of 1.0 for regime 1 conditions merely implies that regime 1 conditions are always worse from a crash standpoint than regime 2 conditions.

The main problem with this method is that it inherently treats all regime 1 conditions as the same. This essentially ignores the output of the regime 1 crash risk model which has proved that there were indeed different levels of crash risk within the regime 1 conditions. Since the output of the regime 1 crash risk model shows that there is a definite variation in the crash risk during regime 1 conditions, treating all regime 1 conditions as having equal crash risk would be an incorrect assumption. Additionally, this method assumes that regime 1 conditions are ALWAYS more likely to have a crash than regime 2 conditions, regardless of the exact traffic conditions. While it is expected that regime 1 conditions would be more risky on average, it is an incorrect assumption to assume that the lowest regime 1 crash risk value is more risky than the highest regime 2 crash risk value. For these reasons, this method was abandoned.

6.11.3 Method 3

The next method that was tried considered standardizing the output of the two models in order to normalize the scales of the models. A simple standardization procedure that can be used to force the mean of a distribution to be equal to zero and the standard deviation to equal one is to subtract the actual mean from each value and divide this by the actual standard deviation. This is given in Equation 6-6.

$$Norm\_Risk = \frac{Risk - \overline{Risk}}{\sigma_{Risk}}$$

...Equation 6-6

Using this equation, the risk output from the regime 1 and the regime 2 models were standardized so that they were on the same scale. However, there is still the problem of how to determine the standard deviation and mean of each model's output. The mean and standard deviation of the regime 1 risk could be calculated from traffic situations that are in regime 1 only or all random traffic situations, regardless of if they are regime 1 or regime 2. A case can be made to use both methods to calculate the mean and standard deviation of the crash risk. Therefore, both methods were initially used in different forms to assess the crash risk. The standardized output from the models using only the respective traffic conditions (using only regime 1 conditions to determine the regime 1 risk) is denoted as the first normalized risk (N1) while the standardized output using all random traffic situations is denoted as the second normalized risk (N2). Therefore, the normalized risk values for each model will be denoted as follows:

[normalization method]_[risk model]. For example, N1_R1 would represent the output of the regime 1 model normalized using the first method.

The scales of both the R1 and R2 when standardized by the two normalization methods are presented below in Figures 6-12 through 6-15. Please note that the scales of the standardized risk values are now approximately equal.

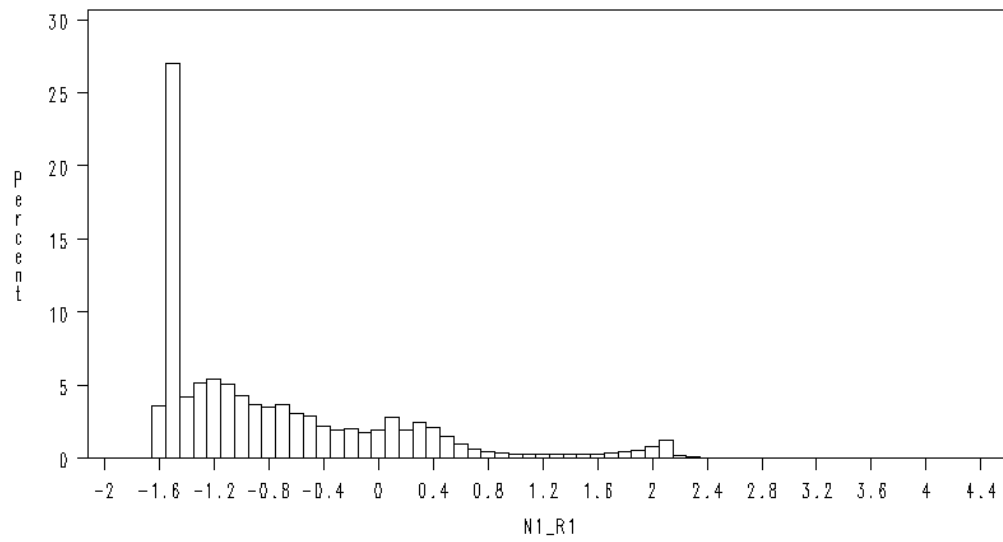


Figure 6-12: Regime 1 Model Normalized by First Method

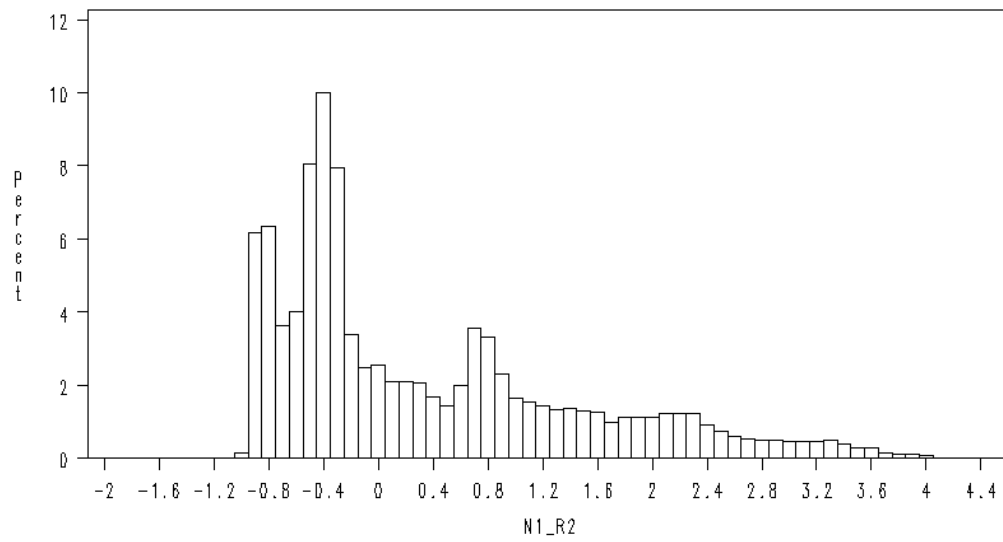


Figure 6-13: Regime 2 Model Normalized by First Method

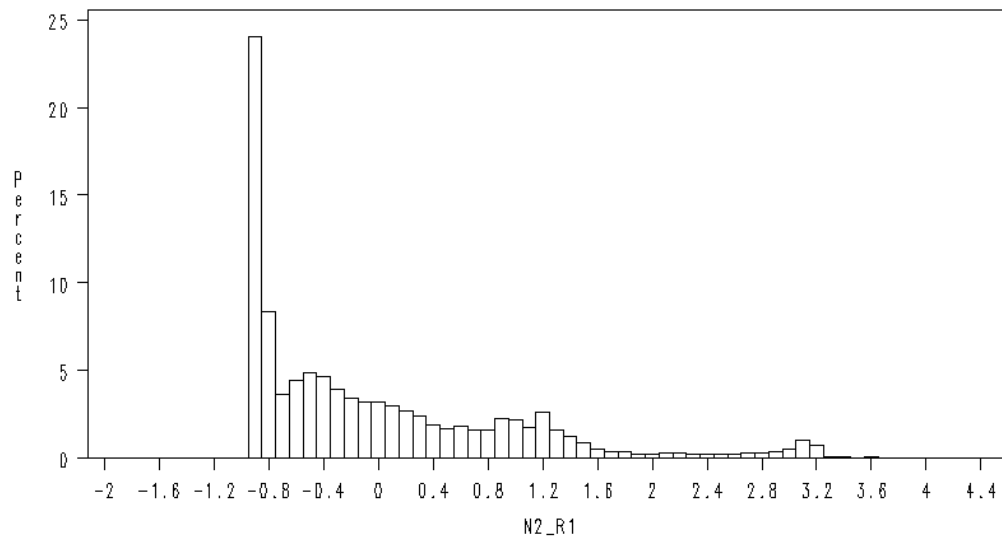


Figure 6-14: Regime 1 Model Normalized by Second Method

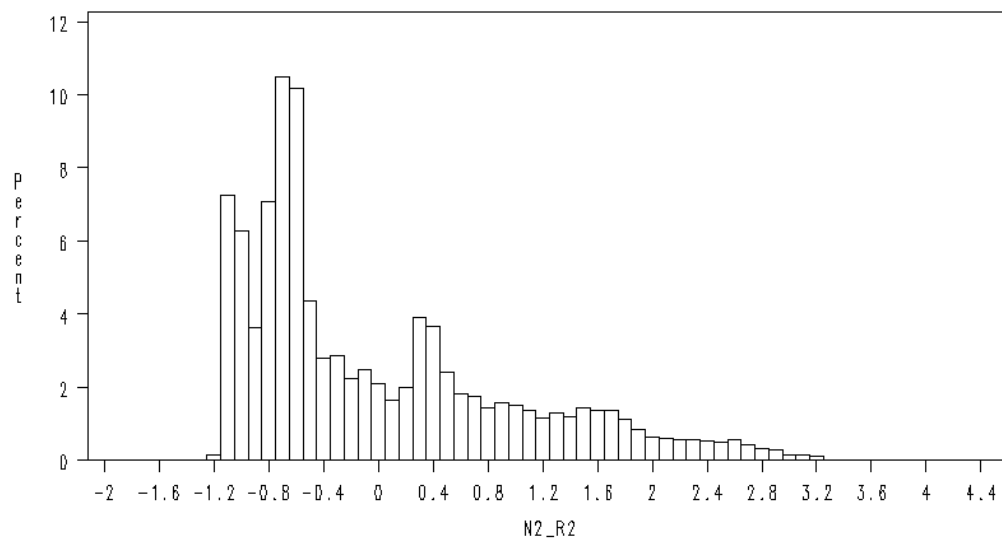


Figure 6-15: Regime 2 Model Normalized by Second Method

Now that there are two standardized risk variables for each model, the next step is to determine a single metric of the crash risk at a certain location and time. One method is to use the normalized risk of a respective regime condition if the traffic conditions fall into that regime. In this case, the first normalization method would be used since the

traffic conditions are already known to be of that regime. Therefore, the equation for Risk_{3A} is given in Equation 6-7.

$$Risk_{3A} = \begin{cases} N1\_R1 & \text{regime 1 Conditions} \\ N1\_R2 & \text{regime 2 Conditions} \end{cases}$$

...Equation 6-7

However, this assumes that regime 1 and regime 2 conditions (which are based on the classification tree model described in Figure 6-7) are absolute. In reality, after the classification tree is applied, the result is a probability that the traffic conditions are regime 1 conditions or regime 2 conditions; this probability is labeled as the value *a* or *1-a*, respectively, in Figure 6-7. Therefore, combining this probability into the risk equation would make the measure of crash risk more accurate. Since the output of the regime 1 and regime 2 models are posterior probabilities (the probability of having a regime 1 or regime 2 crash given those conditions), it would make sense to compute a weighted average of the normalized regime 1 risk and the normalized regime 2 risk based on the probability of the traffic conditions being regime 1 or regime 2. Therefore, risk metrics 3B and 3C are defined in Equations 6-8 and 6-9, respectively.

$$Risk_{3B} = a * (N1\_R1) + (1 - a) * (N1\_R2)$$

...Equation 6-8

$$Risk_{3C} = a * (N2\_R1) + (1 - a) * (N2\_R2)$$

...Equation 6-9

Where a = the probability that the traffic conditions are regime 1 conditions and $1 - a$ is the probability that the traffic conditions belong in regime 2. $N1\_R1$, $N1\_R2$, $N2\_R1$, and $N2\_R2$ have been previously defined as the risk models from regime 1 or 2 normalized by the first or second method. These risk measures ($Risk\_3B$ and $Risk\_3C$) are essentially weighted averages of the normalized output from the regime 1 and regime 2 models using the probability that the traffic conditions belong to that specific traffic regime as the weighting factor.

If the probability of a particular traffic condition belonging to regime 1 or regime 2 is used then the second method of crash risk normalization is more applicable since this recognizes that the regime conditions are not absolute. Therefore, $Risk\_3C$ is more justified than $Risk\_3B$ from a theoretical standpoint. This makes sense since using the weighted average risk measure inherently assumes that the regime of the traffic conditions is not an absolute value but a probability. The first method of normalization (used in $Risk\_3B$) assumes (by normalizing based on values only from within the corresponding regime) that the regime is absolute and, therefore, is not as appropriate as $Risk\_3C$. However, both measures will be compared to determine the final, best metric.

6.11.4 Comparison of Risk Methods

The first step taken to compare the usefulness of the three different risk measures is to compare them visually by graphing the measures for the same situation. Once graphed, the individual values can be compared against one another as well as against what is expected in that situation to determine which measure best represents the data.

This was done for different situations to see how the metrics change for different traffic conditions. Figures 6-16 to 6-21 show plots of the three measures in question (Risk_3A, Risk_3B, and Risk_3C) for various situations. In addition, a line is shown on the plot (regime) that shows whether or not the traffic conditions are of regime 1 or regime 2 to compare the crash risk values between regime 1 and regime 2 conditions.

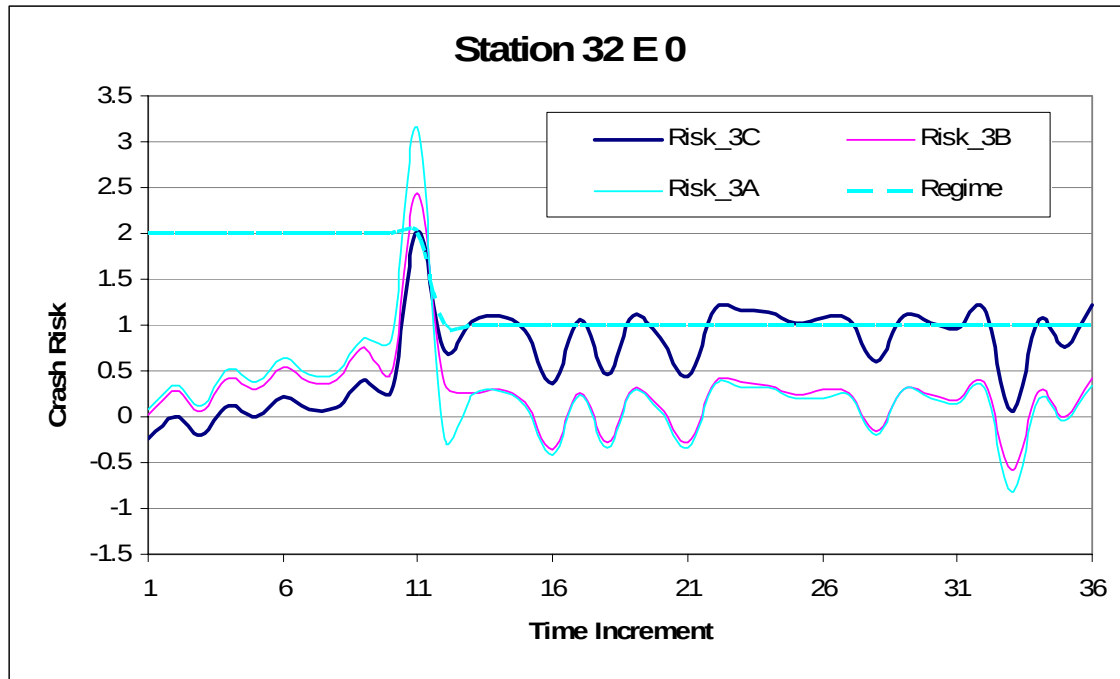


Figure 6-16: Graph of Crash Risk Values at Station 32 E 0

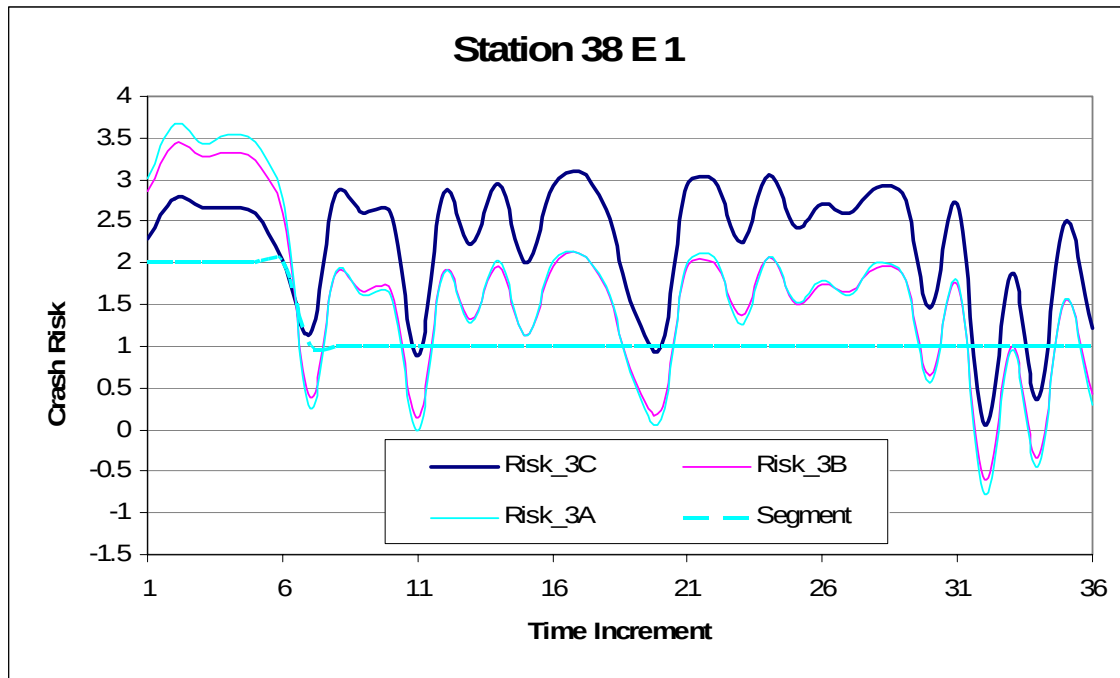


Figure 6-17: Graph of Crash Risk Values at Station 38 E 1

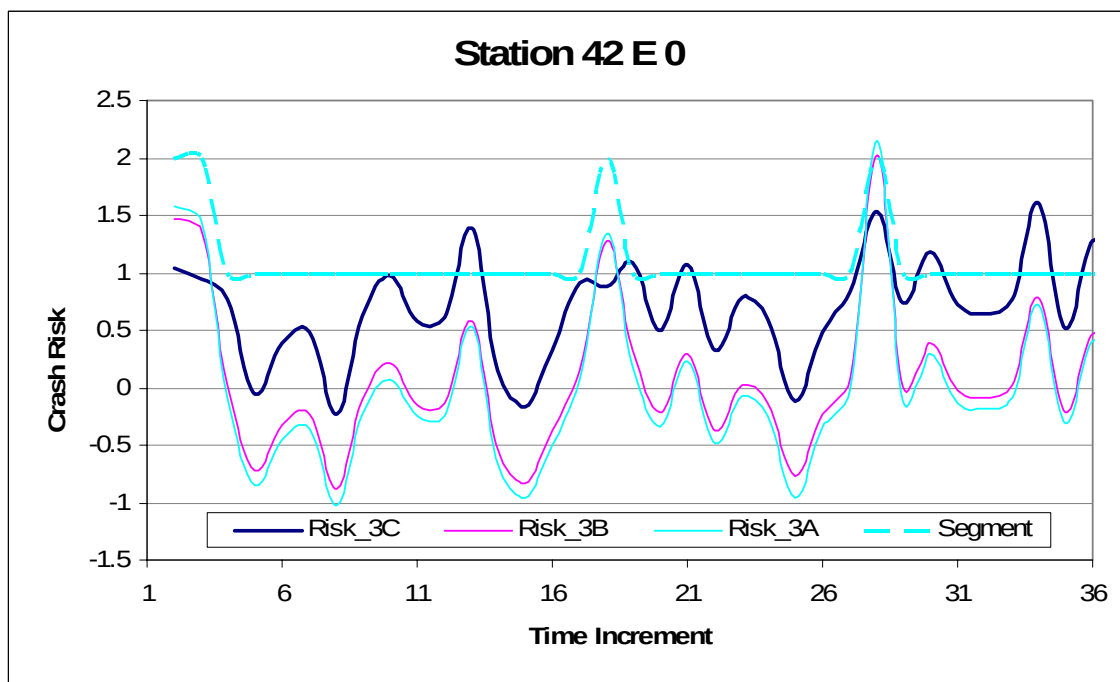


Figure 6-18: Graph of Crash Risk Values at Station 42 E 0

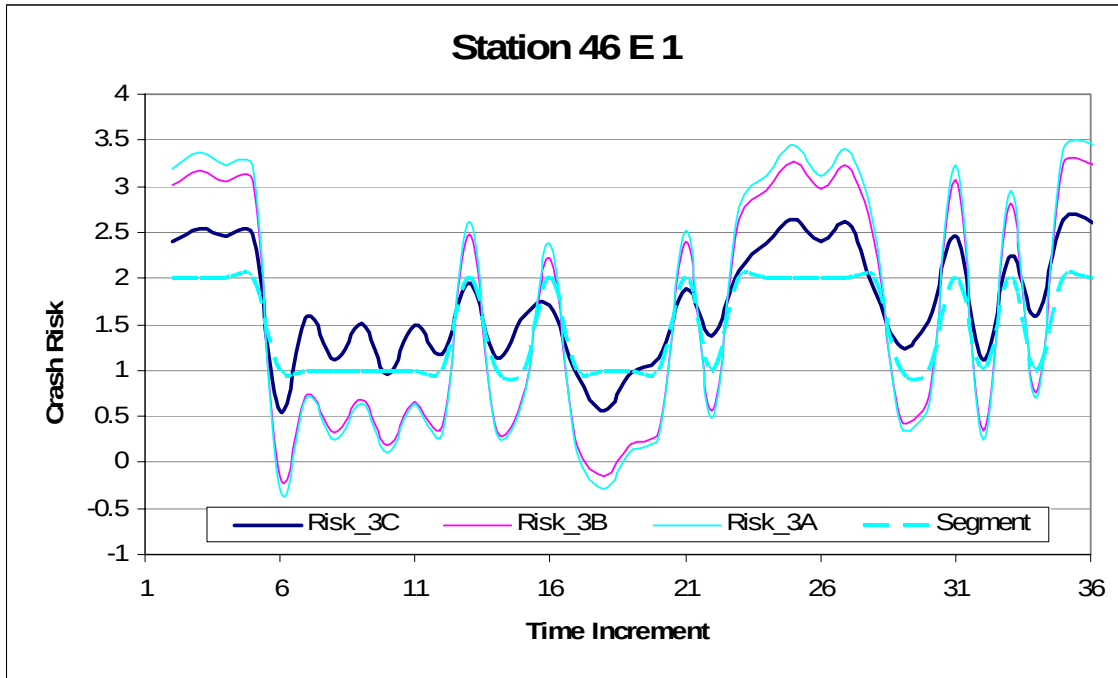


Figure 6-19: Graph of Crash Risk Values at Station 46 E 1

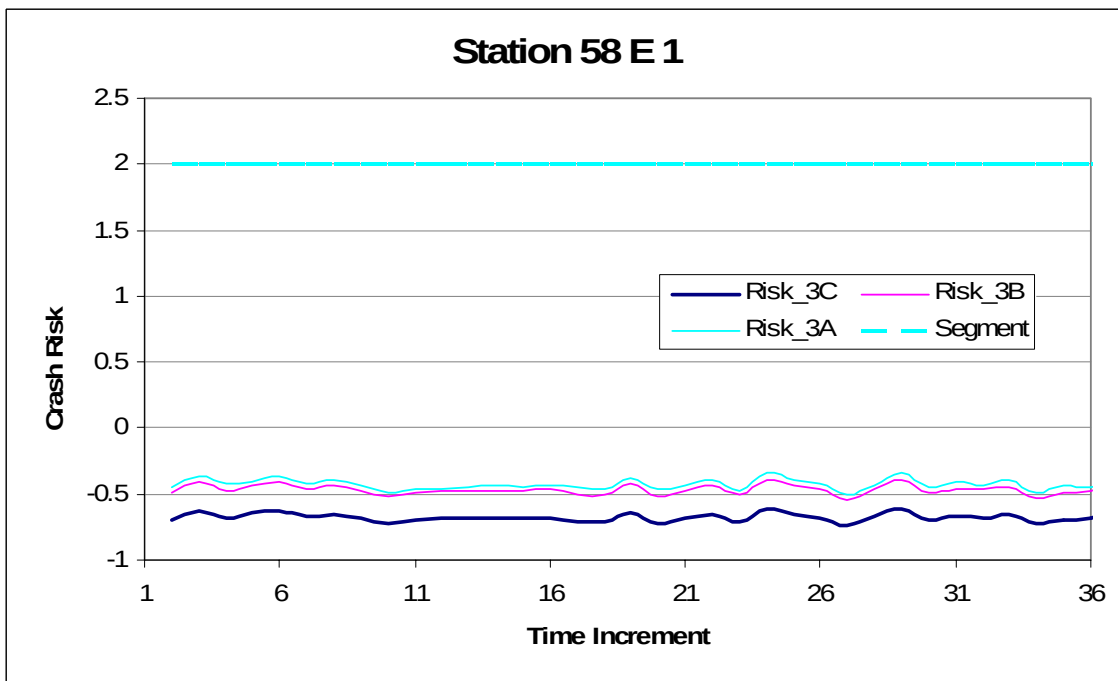


Figure 6-20: Graph of Crash Risk Values at Station 58 E 1

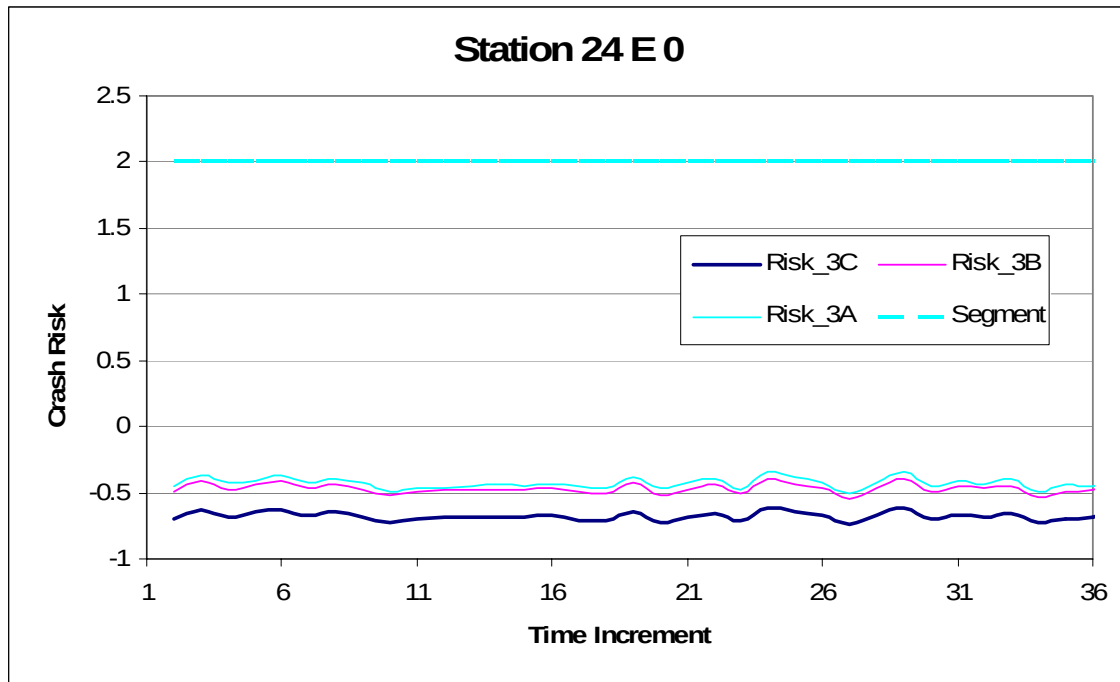


Figure 6-21: Graph of Crash Risk Values at Station 24 E 0

From Figures 6-16 to 6-21, it is seen that the values of Risk_3A and Risk_3B seem to be nearly equivalent. Judging by the equations that define Risk_3A and Risk_3B this only occurs when the value of “a” is close to 0 or 1 (which is most often the case as seen in Figure 6-7). Looking at the trend of values of each risk measure, the average risk values for Risk_3A and Risk_3B are lower when the traffic conditions are regime 1 compared to when traffic conditions are in regime 2. This is the opposite of what is expected to occur. This trend is also apparent at times for Risk_3C. However, there is an important difference in the measure Risk 3A, Risk_3B, and Risk_3C. As shown in the plot of the crash risks at nearly all stations provided, the value of Risk_3C is almost always lower than the value of Risk_3B / Risk_3A during regime 2 conditions and higher during regime 1 conditions. This means that Risk_3C gives higher values during regime

1 conditions and lower values during regime 2 conditions, which is what is expected. Therefore, Risk_3C shows potential to be used as the rear-end crash risk measure.

However, there is still the issue of whether or not Risk_3C provides higher risk values (on average) during regime 1 conditions compared to regime 2 conditions. To test whether or not this is true, the correlation of the risk values is computed with Risk_2 and the regime conditions. The correlation coefficient will give an indication of whether or not two separate measures have the same general trend. Comparing the correlation of Risk_3C with Risk_2 (which has a maximum risk value of 1 during ALL regime 1 conditions) will show whether or not the values of Risk_3C are higher during regime 1 conditions than regime 2 conditions. The correlation matrix between all 5 risk measures and the regime of the traffic conditions are shown below in Table 6-4.

Table 6-4: Correlation of Various Risk Metrics and Regime Conditions

	risk_1	risk_2	risk_3a	risk_3b	risk_3c	regime
risk_1	1.0000	-0.0102	0.9333	0.9071	0.6655	0.2206
risk_2	-0.0102	1.0000	0.2005	0.2903	0.6501	-0.9761
risk_3a	0.9333	0.2005	1.0000	0.9874	0.8599	-0.0142
risk_3b	0.9071	0.2903	0.9874	1.0000	0.9019	-0.1059
risk_3c	0.6655	0.6501	0.8599	0.9019	1.0000	-0.5114
regime	0.2206	-0.9761	-0.0142	-0.1059	-0.5114	1.0000

As shown in Table 6-4, the correlation between Risk_2 and Risk_3C is very high (0.65005) compared to the correlation of Risk_2 with Risk_3A (0.2005) and Risk_3B (0.2903). Additionally, the correlation between Risk_3C and the regime conditions is

also high (-0.51140) compared to the correlation of the regime conditions and Risk_3A (-0.0142) and Risk_3B (-0.1059). This implies that Risk_3C shows more clearly a higher value of crash risk for segment 1 conditions. Only Risk_2 has a higher correlation with the regime conditions (-0.9761) and that occurs because for this measure all regime 1 conditions are artificially given the highest value of risk (1.0). Therefore, Risk_3C provides not only the best measure from a practical perspective, but Risk_3C also describes the higher risk during regime 1 conditions better from a statistical perspective as well. Therefore, the measure of rear-end crash risk that will be used for this study will be computed using the method described above for Risk_3C.

6.12 Lane-Change Crash Risk

As previously mentioned, Pande and Abdel-Aty (2006) also created a model to assess the lane changing crash risk along Interstate-4. For this model a lane change crash was assumed to be all sideswipe crashes and also the angle crashes that occurred on the left most and center lanes along the freeway since these crashes are typically associated with lane changing maneuvers. Lane-changing crashes were not found to occur within different traffic regimes and, therefore, one model was sufficient to assess the lane-change risk for all situations encountered. This makes the lane-change crash risk measure extremely easy to use as no process is needed to relate different risk values (since there is only one model). However, the output of the lane-change crash risk model was normalized by subtracting the mean and dividing by the standard deviation in order to equate the scales of the rear-end and lane-change crash risk. The mean of this new lane-change crash risk measure is now equal to 0.0 and the standard deviation equal to 1.0.

Because lane-changing behavior is dependant upon the traffic characteristics in individual lanes of the freeway, some of the variables used considered occupancy in individual lanes in addition to variables aggregated over the three lanes like in the rear-end model case. Classification trees were used to determine variables that are significantly related to lane changing crashes and a neural network approach was used to create the model. The following variables were found significant to the assessment of lane changing crash risk:

- ASW2
- ASU2
- AOW2
- ADALOU2
- SVW2
- SSW2

The nomenclature used to describe the variables is the same as previously mentioned except for two important differences. First, the variable ADALOU2 is a unique variable that takes into account the difference in occupancy between adjacent lanes. This variable can be described as in Equation 6-10.

$$ADALOU2 = \frac{1}{10} \sum_{t=1}^{10} |LO - CO| + |CO - RO|$$

...Equation 6-10

In this equation LO, CO, and RO represent the lane occupancy in the left, center, and right lane, respectively. The second difference between the naming of the variables in the rear-end models and the lane changing models is the variable that represents the station name. In the lane changing models the station name is represented by U or W which represents the station upstream or downstream of the location of interest, respectively.

6.13 Travel Time

In addition to the measures of rear-end and lane-change crash risk, the travel time will be considered in examining the effects of the various test scenarios performed in this study. The travel time is an important measure to consider due to the potential negative impacts of route diversion and ramp metering on the operational capabilities on the network. Diverting vehicles from entering Interstate-4 at one location and moving them to another downstream location will increase the volume of vehicles using the surface streets. This would not only increase the travel time of the diverted vehicles but the vehicles currently on the surface streets would experience additional delays due to the added volume on the network. Ramp metering will also directly affect travel time by delaying vehicles on the on-ramps before they are allowed to enter the freeway. The travel time that will be used as a measure of effectiveness in this study is the overall network travel time. This value is given as a standard parameter in the PARAMICS program and is computed by summing the individual travel times for all vehicles on the network. A change in this value will indicate the effects of the ITS strategy on the overall network performance. For this reason, even a modest increase in the travel time of 10% is significant since this would mean that the average travel time for ALL vehicles

is increased by this amount. In selecting the best metering algorithm, while preference will be given to strategies that most effectively reduce the crash risk along the freeway, travel time will also be used to determine which strategies are feasible to implement. For this study the maximum allowable increase in the network-wide travel time for any of the scenarios was set at 5%. This is a small enough value that it will not significantly hinder the network users and can be sacrificed to improve the overall safety of the freeway.

6.14 Network Loading Scenarios

During the peak period, the downtown area of Interstate-4 usually operates at a Level of Service F. The simulation represents this as heavy congestion is present during a majority of the simulation runtime in the downtown area. However, this study wishes to test the ITS alternatives at various levels of congestion on the freeway that occurs throughout a typical day. This is desired as implementing route diversion and ramp metering may yield different results at the different loading cases. Therefore, four loading scenarios have been created for testing of the ITS alternatives. These loading scenarios are as follows: 100% loading (heavy congestion), 90% loading (typical congestion), 80% loading (approaching congestion), and 60% loading (free flow conditions). The 90% loading condition is what can be expected on a typical day on I-4 during the peak hour. The 100% loading condition is an extreme condition that occurs when the travel demand peaks at the same time. Since there are 4 different loading scenarios in this study, the network loading will be the first variable tested in the experimental design for both route diversion and ramp metering.

6.15 Route Diversion Scenarios

The purpose of the first half of the experimental design is to examine the feasibility of route diversion as a real-time crash risk prevention strategy. Additionally, the experimental design is meant to determine the best diversion strategy given the specific diversion routes that are being used. The two unique diversion routes presented in this study are Diversion Route 1 and Diversion Route 2.

6.15.1 Route diversion 1

The first diversion route has two decision points. The initial decision point (DP-1A) will determine the number of vehicles that divert from their initial ramp entrance. The second decision point will determine which entrance the diverted vehicles will use to re-enter the freeway. Vehicles that do not use the second diversion (DP-1B) will enter the freeway 2 miles downstream of their initial entrance location and enter the freeway on Anderson St. If DP-1B is used, however, these vehicles will enter the freeway 1 mile further downstream at Colonial Drive which increases the length that vehicles were diverted by 50% (see Figure 6-5). Please note that the secondary decision point is contingent upon the first. Only vehicles that have diverted at DP-1A will be available to divert at DP-1B. Therefore, diverting 50% of the vehicles at DP-1B when 60% of the vehicles were diverted initially (% 1A) will result in 30% of the vehicles using the nearer re-entry location and 30% of the vehicles traveling to the further re-entry location.

The variables that will be used in the experimental design to test the feasibility of Route Diversion 1 are: the percentage of vehicles diverted at DP-1A (% 1A), the percentage of vehicles diverted at DP-1B (% 1B), and the network loading. Figure 6-22 represents the levels of each variable that were used in the experimental design. For this

study, a factorial design was employed which examined every variable combination in order to determine the best case and trends within each variable. The individual test cases created by the factorial design are shown below in Table 6-5.

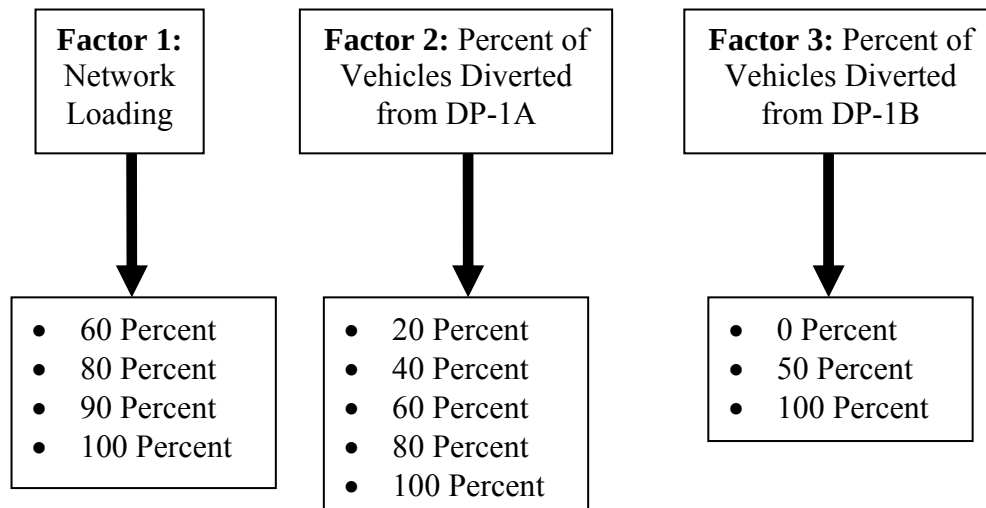


Figure 6-22: Factorial Design for Route Diversion 1

Table 6-5: Test Cases for Route Diversion 1

Case Number	Percent Loading	Percent Vehicles Diverted From DP-1A	Percent Vehicles Diverted From DP-1B
1	60	20	0
2	60	40	0
3	60	60	0
4	60	80	0
5	60	100	0
6	60	20	50
7	60	40	50
8	60	60	50
9	60	80	50
10	60	100	50
11	60	20	100
12	60	40	100
13	60	60	100
14	60	80	100
15	60	100	100
16	80	20	0
17	80	40	0
18	80	60	0

Case Number	Percent Loading	Percent Vehicles Diverted From DP-1A	Percent Vehicles Diverted From DP-1B
19	80	80	0
20	80	100	0
21	80	20	50
22	80	40	50
23	80	60	50
24	80	80	50
25	80	100	50
26	80	20	100
27	80	40	100
28	80	60	100
29	80	80	100
30	80	100	100
31	90	20	0
32	90	40	0
33	90	60	0
34	90	80	0
35	90	100	0
36	90	20	50
37	90	40	50
38	90	60	50
39	90	80	50
40	90	100	50
41	90	20	100
42	90	40	100
43	90	60	100
44	90	80	100
45	90	100	100
46	100	20	0
47	100	40	0
48	100	60	0
49	100	80	0
50	100	100	0
51	100	20	50
52	100	40	50
53	100	60	50
54	100	80	50
55	100	100	50
56	100	20	100
57	100	40	100
58	100	60	100
59	100	80	100
60	100	100	100

6.15.2 Route diversion 2

A similar factorial experimental design was used to examine the effects of Diversion Route 2. As shown in Figure 6-5, this diversion route is much smaller than the previous route. Diverted vehicles will travel less than 1 mile on the surface streets before re-entering the freeway at Colonial Drive. Because there is only one re-entry location for these diverted vehicles, route diversion 2 contains only a single decision point (DP-2) which controls the number of vehicles that are diverted. This is shown visually in Figure 6-5. Note from this figure that the area of the freeway affected by route diversion 2 is encompassed in the latter half of route diversion 1.

The two variables considered in this section of the experimental design were the network loading and the percentage of vehicles diverted at DP-2. The levels of the factorial design are presented below in Figure 6-23. The individual test cases are also given in Table 6-6.

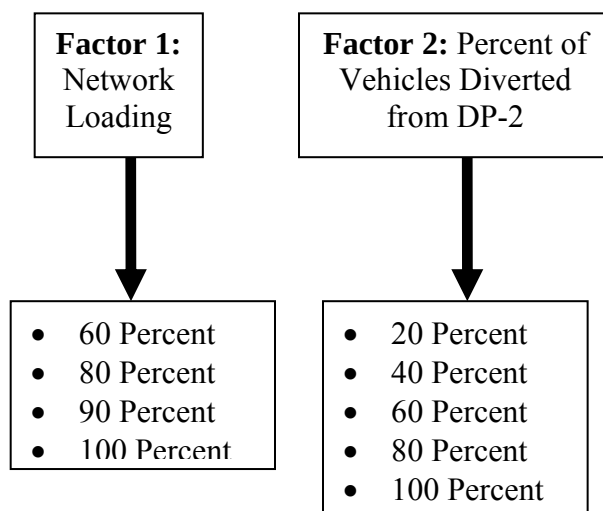


Figure 6-23: Factorial Design for Route Diversion 2

Table 6-6: Test Cases for Route Diversion 2

Case Number	Percent Loading	Percent Vehicles Diverted From DP-2
61	60	20
62	60	40
63	60	60
64	60	80
65	60	100
66	80	20
67	80	40
68	80	60
69	80	80
70	80	100
71	90	20
72	90	40
73	90	60
74	90	80
75	90	100
76	100	20
77	100	40
78	100	60
79	100	80
80	100	100

6.16 Ramp Metering Scenarios

The purpose of the second half of the experimental design is to find the best ramp metering strategy to reduce the crash risk through the downtown Orlando portion of the network. The experimental design will compare the feasibility of using the ALINEA ramp metering strategy and the Zone Algorithm against the no metering case and against each other (Section 6.16.2.1). Additionally, the experimental design will also compare the effectiveness of both the Zone and ALINEA algorithm using the traffic cycle realization and the one-car-per-cycle realization (Section 6.16.2.2). However, the first portion of the experimental design will be completed to confirm the best operational parameters for the ALINEA strategy.

6.16.1 Calibration of ALINEA Parameters

Previous work has been done to examine the effects of using the ALINEA ramp metering strategy to reduce the real-time crash risk on a freeway (Lee et al, 2006; Dhindsa, 2005). However, the calibration parameters for the ALINEA algorithm, such as the critical occupancy and cycle length, differ between the study performed by Dhindsa (2005) and what is expected in the field. In Chapter 5 we found that using a higher critical occupancy (which would make the algorithm less restrictive) increased the safety benefits which does not make sense as allowing more vehicles onto the network should increase the crash risk. Therefore, a portion of the experimental design must be dedicated to determining the calibration parameters for the ALINEA network. As seen in the ALINEA equation (Equation 1-3) the following parameters can be altered when using ALINEA: the regulator parameter (K_R) and the critical occupancy (o_c). Additionally, when the traffic-cycle realization is used the cycle length becomes another important factor. Previous work by Dhindsa (2005) (Also see Chapters 4 and 5) as well as studies performed to optimize the operational capabilities of ALINEA (Papageorgiou et al, 1997) have found that the optimal value of the regulator parameter is 70 veh/hr. However, various different cycle lengths and critical occupancy values have been used.

The first portion of the experimental design will implement ALINEA and the traffic-cycle realization using different cycle lengths and critical occupancies to determine the best combination to use to compare against the Zone algorithm. A factorial design was again chosen to test three values of cycle length (30, 45, and 60 seconds) as well as three values of the critical occupancy (0.17, 0.20, and 0.23). The last factor in the factorial design was the network loading level. Since the purpose of ramp metering is to

alleviate and eliminate congestion (Jin and Zhang, 2001), it is impractical to implement ramp metering in non-congested situations. Therefore, only two loading levels were deemed appropriate for this analysis – the 90 percent loading and 100 percent loading scenarios. The 60 percent loading and 80 percent loading scenarios were briefly examined outside the experimental design but were found to show no significant crash risk changes as a result of ramp metering. The individual cases in the experimental design are shown below in Table 6-7.

Table 6-7: Test Cases for ALINEA Parameters

Case Number	Percent Loading	Cycle Length	Critical Occupancy
81	100	30	0.17
82	100	30	0.20
83	100	30	0.23
84	100	45	0.17
85	100	45	0.20
86	100	45	0.23
87	100	60	0.17
88	100	60	0.20
89	100	60	0.23
90	90	30	0.17
91	90	30	0.20
92	90	30	0.23
93	90	45	0.17
94	90	45	0.20
95	90	45	0.23
96	90	60	0.17
97	90	60	0.20
98	90	60	0.23

The section of freeway that is metered using ALINEA for this portion of the experimental design is the area immediately upstream of the Interstate-4 / S. R. 408 Interchange. This section includes 3 ramps that are metered and denoted in Figure 6-24 (Section 6.16.3) as Zone 1. Note that even though the ramps are designated as Zone 1 this does not mean that the Zone algorithm is used or the metering algorithm is

coordinated. This just means that the ramps that make up Zone 1 will be metered individually in this portion of the experimental design.

6.16.2 Comparison of Ramp Metering Strategies

6.16.2.1 Differences in Zone and ALINEA Algorithms

The ALINEA and Zone ramp metering algorithms were described in detail in Sections 1.4.1 and 6.5.2.1, respectively. The major differences between the two algorithms are summarized below.

- The ALINEA algorithm considers only the occupancy on the freeway in order to determine the metering rate at a particular ramp. However, the Zone algorithm considers traffic volumes (at on-ramps, off-ramps, and entry and exit points to the zone in question), speeds, and densities. Therefore, the Zone algorithm is more complex and requires more information to be implemented but uses a more complete picture of the traffic flow to determine the metering rate.
- The Zone algorithm requires that the ramps in the freeway be grouped into zones or groups of ramps that will be metered together. This again increases the complexity of the algorithm as a single ramp cannot be metered by itself. However, the ALINEA algorithm can meter single ramps as well as consecutive ramps with ease.
- The ALINEA algorithm is not coordinated and only considers loop information from the nearest detector in order to meter a particular ramp. The algorithm cannot “look ahead” at traffic conditions upstream or downstream to see how to best change the metering rate. On the other hand, the Zone algorithm considers

traffic information for the entire zone in order meter any particular ramp. Therefore, this method is more suited to handle situations with large areas of congestion as the algorithm will recognize these areas and account for it by lowering the metering rate.

- The ALINEA algorithm does not take into account the traffic demand at any on-ramp. Therefore, when using ALINEA a ramp that has a high traffic demand could have a more restrictive metering rate than another ramp that few vehicles use. This would result in unnecessary delays and queues as vehicles that need to enter the freeway are not able to while excessive green time is allowed for ramps that have no traffic demand. This unnecessary restrictiveness actually causes the ALINEA algorithm to perform better from an operational standpoint since fewer vehicles are allowed onto the freeway. The Zone algorithm, on the other hand, accounts for on-ramp demand and proportionally allows for green time based on how many vehicles at each ramp need to enter the freeway.
 - Because ALINEA meters ramps individually this algorithm performs better when there is localized congestion. When localized congestion is present, the metering rate near the ramp in question is reduced to allow the congested conditions to dissipate. The Zone algorithm cannot handle these situations as well since the localized congestion at one detector would be just a small part of the larger zone.
- Looking at the various differences between the two metering algorithms there is no clear “best” strategy for reducing the rear-end and lane-change crash risk. Each algorithm has its own unique advantages and disadvantages. Therefore, in order to determine which algorithm provides the best safety results they must be

compared in the last part of the experimental design to determine the overall best ramp metering strategy.

6.16.2.2 *Differences in Traffic-Cycle and One-Car-Per-Cycle Realizations*

Both the ALINEA and Zone ramp metering algorithms yield the metering rate in veh/hr for each of the on-ramps that are being metered. Once the metering rate is known, however, the traffic signals that control the ramp must then reflect that particular metering rate. As mentioned by Papageorgiou et al (1997), there are two methods of implementing the ramp metering strategy – the traffic-cycle realization and the one-car-per-cycle realization.

In the traffic-cycle realization method, after the final metering rate values ($R[t]$ for ALINEA and R_n for Zone) are assigned, the rates must be converted to green time for the ramp meter. The meters used in this study assume a cycle length between 30 and 60 seconds. Assuming a saturation flow rate of the meter of 1900 vehicles per hour (Highway Capacity Manual, 2000), Equation 6-11 converts the flow rate into green time for the meter (assuming lost time is equal to extension of effective green time). Using this method allows vehicles to enter the freeway in platoons. The average size of the platoon that is allowed to enter the freeway is determined by the cycle length. For equal metering rates, a longer cycle length would mean that larger platoons are allowed to enter the freeway.

$$Green = \frac{rate * cycle\_length}{1900}$$

...Equation 6-11

For the one-car-per-cycle method, the implementation is slightly different. Once the metering rate is determined, the average cycle length is calculated assuming that only a single vehicle enters the freeway per cycle. From this cycle length, the red-time is determined by assuming a constant green-time for each cycle (2 seconds, enough for one vehicle to utilize the phase). The cycle lengths used range from 2.1 seconds (1714 veh/hr) to a maximum cycle length defined for each ramp that is metered. This maximum cycle length is calculated by determining the minimum metering rate for the respective ramp by using the method mentioned in Section 6.5.2.1 which sets the minimum metering rate to keep average vehicular delay at on-ramps to 4 minutes. When this method is used, platoons of vehicles are prohibited from entering the freeway at once. This reduces the amount of turbulence to the mainline traffic flow caused by vehicles entering the traffic stream from the on-ramps since only one vehicle would have to merge into the traffic stream at a time. However, the drawback to this strategy is reduced operational capabilities as vehicles are delayed at the ramp meters for a longer amount of time. Additionally, when this method is used at less restrictive metering rates vehicles (smaller cycle lengths) vehicles will have extremely small delays at the meters and will enter the freeway almost as quickly as they arrived at the ramp). In other words, the meter will essentially not do anything during these situations.

6.16.3 Ramp metering scenarios

The major variable in the implementation of the Zone ramp metering algorithm is the zone that is chosen to be metered. Once the zone is chosen, there are no major

algorithm variables that can be used to alter the effectiveness of the metering strategy. However, since the traffic-cycle realization is being applied the optimal cycle length is still a factor that needs to be determined. The experimental design will therefore include the varying cycle lengths and three different zones that have been created to explore the effectiveness of the Zone algorithm in downtown Orlando. The three chosen zones are shown in a schematic drawing presented in Figure 6-24.

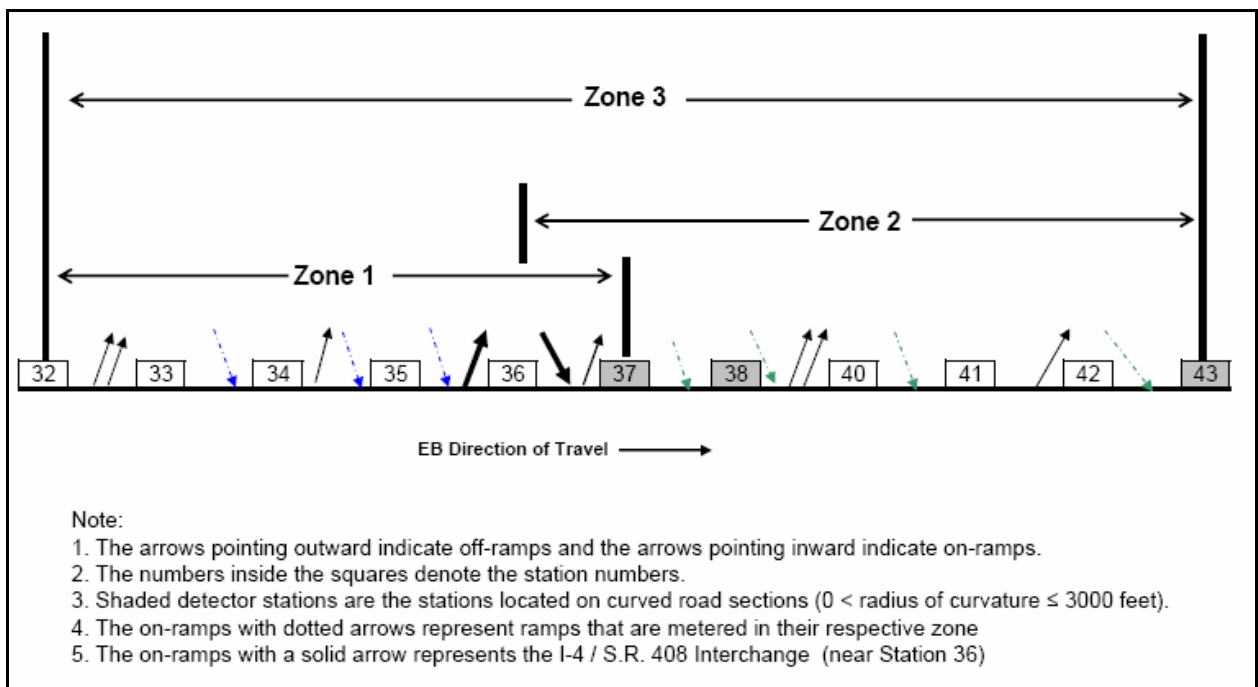


Figure 6-24: Zones Metered for Ramp Metering Strategies

The zones were chosen with the objective of reducing the crash risk in the downtown area around the Interstate-4 / S. R. 408 Interchange. The ramp leading from S. R. 408 to Interstate-4 (located between stations 36 and 37) is included as a non-metered ramp due to the large volumes that use this interchange. Each of the three zones includes this interchange so that the large in-flow volume from this interchange factors into the

metering algorithm. The first zone is located just upstream of the interchange and includes the three ramps upstream. The second zone extends downstream from the interchange to incorporate the ramps four downstream. The third zone is the largest zone that incorporates most of the downtown area and is a union of zones 1 and 2.

In the experimental design, each zone was metered using both the Zone and ALINEA algorithms. Although the ALINEA strategy is a localized strategy, the same zones could be metered using ALINEA by simply metering the ramps that encompass the zone individually. For the ALINEA algorithm the calibration parameters from Section 6.16.1 (Table 6-7) were used and the ramps involved in the zone were metered in an uncoordinated fashion. For the Zone algorithm with the traffic-cycle realization, each zone was metered three times with varying cycle lengths. This was done to determine if the cycle length was an important variable in the traffic-cycle realization of the Zone algorithm. Additionally, each zone was metered with the ALINEA and Zone algorithms using the one-car-per-cycle (OCPC) method of allowing vehicles onto the mainline. The full experimental design is listed below. Note this is basically another factorial design with 4 factors. The factors are: loading (90 or 100 percent), Zone (1, 2, or 3), Algorithm (Zone or ALINEA), and cycle properties (30 second, 45 second, 60 second, one-car-per-cycle). However, there is a partial factor in the experimental design as the ALINEA algorithm will only be run with the best cycle length (from Section 6.16.1) and the one-car-per-cycle length.

Table 6-8: Test Cases for Zone and ALINEA Ramp Metering

Case Number	Percent Loading	Metered Zone	Algorithm	Cycle Length
99	100	1	Zone	30
100	100	1	Zone	45
101	100	1	Zone	60
102	100	1	ALINEA	BEST
103	100	1	Zone	OCPC
104	100	1	ALINEA	OCPC
105	100	2	Zone	30
106	100	2	Zone	45
107	100	2	Zone	60
108	100	2	ALINEA	BEST
109	100	2	Zone	OCPC
110	100	2	ALINEA	OCPC
111	100	3	Zone	30
112	100	3	Zone	45
113	100	3	Zone	60
114	100	3	ALINEA	BEST
115	100	3	Zone	OCPC
116	100	3	ALINEA	OCPC
117	90	1	Zone	30
118	90	1	Zone	45
119	90	1	Zone	60
120	90	1	ALINEA	BEST
121	90	1	Zone	OCPC
122	90	1	ALINEA	OCPC
123	90	2	Zone	30
124	90	2	Zone	45
125	90	2	Zone	60
126	90	2	ALINEA	BEST
127	90	2	Zone	OCPC
128	90	2	ALINEA	OCPC
129	90	3	Zone	30
130	90	3	Zone	45
131	90	3	Zone	60
132	90	3	ALINEA	BEST
133	90	3	Zone	OCPC
134	90	3	ALINEA	OCPC

6.17 Number of Simulation Runs

Due to the stochastic nature of the PARAMICS micro-simulation there is a fair amount of variation in-between runs of a specific testing alternative. This variation occurs because each run that is performed is unique. 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. Therefore, if two runs are performed with the same seed value the results from PARAMICS will be identical for the two runs. In this study, multiple runs of different seed values were used to ensure that the results that were determined were based on the tested strategy and not an isolated event resulting from behavior due to a particular seed value. Because of this a large number of runs needed to be performed in order to reduce the variation in the crash risk to acceptable levels.

A good rule of thumb in simulation is that a minimum of 30 runs should be performed for each case in order to have a statistically sound experimental design. This requires no calculation of the amount of variation within the data as this number of runs is assumed to be large enough to account for the simulation variance. However, performing this many runs would require an enormous amount of processing time (especially for the macros that are used to summarize the data). Therefore, another method has been used to help reduce the number of runs to be performed. In this method, a group of runs (10 for this study) is performed for all cases. Once completed, the variation of within runs is determined. Using Equation 6-12 the number of required runs is calculated based on the variation observed in the initial group of runs.

$$N \approx \left(KS/E \right)^2$$

...Equation 6-12

In this equation, N represents the number of runs required, K is a statistical factor related to the confidence level that is required (K = 1.645 for the 90% confidence level which is used in this study), S is the standard deviation of the data in question, and E is the error that is allowable for the runs. For this study, S was calculated by examining the average crash risk at a particular station across the 10 initial runs. A separate N value was calculated for every station within a group of runs. The station that yielded the highest N value was used to determine the number of runs required for that particular group of test runs. The allowable error was assumed to be 0.100 for the rear-end crash risk; note the rear-end crash risk was used as it generally had more variation than the lane-change crash risk. The value of 0.100 is equivalent to about 2% of the range of the rear-end crash risk values that were observed (-1.0, 3.5).

In all cases, scenarios performed at the 60 percent loading scenario required no further runs to be performed. The majority of other scenarios also required no further runs to be performed. For those test cases that did require additional runs the maximum number that was needed to be done for any particular test case was a total of 20. Note that this is significantly less than 30 runs that are typically used to ensure a good experimental design.

CHAPTER 7

RESULTS FROM ROUTE DIVERSION AND MULTIPLE RAMP METERING EVALUATIONS

7.1 Analyzing Simulation Runs

Values of the rear-end and lane-change crash risk were calculated at every time period and location for each of the individual runs performed for each test case using the aforementioned crash risk models. Although a statistically sufficient number of runs were performed for each test case described in Section 6.17, there still is a fair amount of variation of the rear-end and lane-change for a particular time and location between the individual runs of a particular test case. This variation occurs due to the random, stochastic nature of the simulation program and is reflected equally in the field due to the random fluctuations in traffic flow over multiple days. An example of this variation is shown in Figure 7-1 which is a plot of the rear-end crash risk values vs. time for each run at a particular station of a particular scenario (case 25) in the experimental design. Note that the rear-end crash risk values plotted in all subsequent sections is the Risk_3C that is defined in Equation 6-9. The scale of the rear-end and lane-change crash risk is between -1.0 and 3.5. This occurs because the probabilities obtained from the neural network models are normalized by subtracting the mean and dividing by the standard deviation. Therefore, these crash risk values should be used to compare the risk at different times and locations. A crash risk value of 0.0 would represent crash risk conditions that are equal to the mean crash risk that is experienced while a higher value would show the crash risk is higher than the mean risk of the freeway corridor.

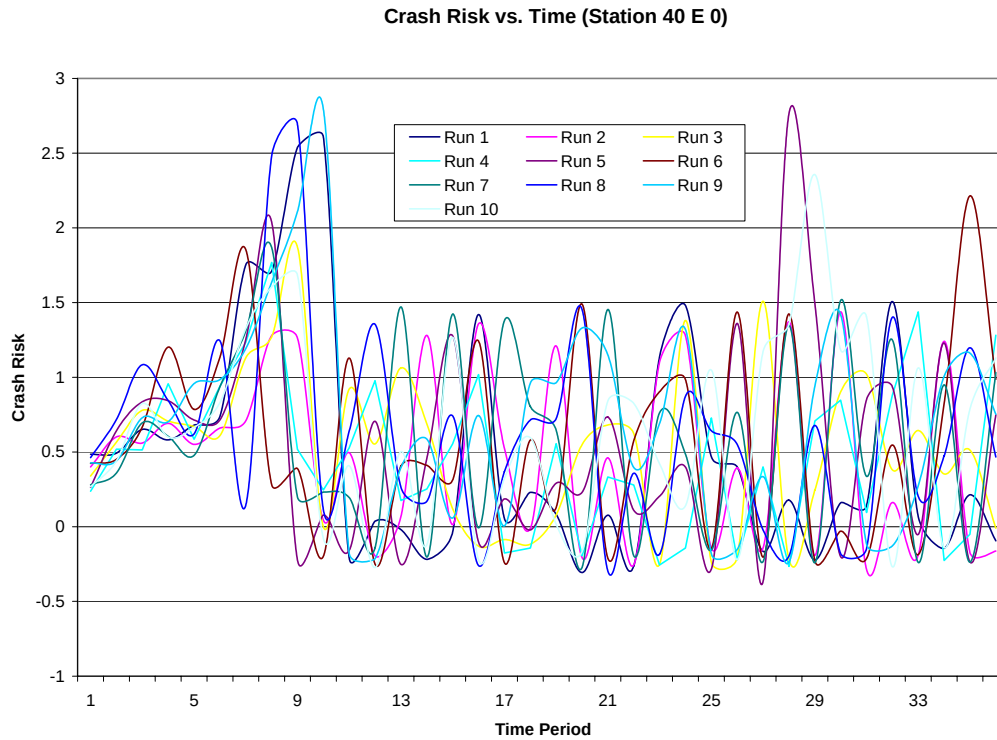


Figure 7-1: Plot of Crash Risk vs. Time at Station 40 E 0 for All Runs in Test Case 25

If the individual crash risk curves for each run were used to compare two different test cases even more crash risk vs. time curves would be required on a single plot than shown on Figure 7-1. If ten runs were performed for each of the two test cases this would mean that 20 curves would be required on a single plot to compare two alternatives. Visually comparing the change in crash risk in this manner would be difficult due to the large number of curves on the plot. Deciphering which of the test cases has a higher or lower crash risk would also be equally challenging. Additionally, these plots would be unreadable due to the large number of curves and would not provide any insights into the trends of the ITS strategies on the different crash risk values along the network corridor. In an effort to eliminate this problem, a single crash risk profile

was created for each test scenario for both the rear-end and lane-change crash risk. This single rear-end crash risk profile has a value of the rear-end (or lane-change) crash risk at every station and time that is equal to the average of the rear-end crash risk values at the respective station and time over the number of simulation runs performed at that particular scenario. Equation 7-1 (below) shows how this average crash risk profile is calculated.

$$(Risk\_profile)_{t,l} = \frac{1}{N} \sum_{r=1}^N (Risk)_{t,r,l}$$

...Equation 7-1

Where: $(Risk)_{t,r,l}$ = the crash risk for time t, run r, and location l for particular test case scenario

N = the number of runs required for the particular test case scenario

$(Risk\_profile)_{t,l}$ = the crash risk averaged over the number of runs for each location l and time t

Using a single crash risk curve for each scenario is ideal for two main reasons. First, comparing the crash risk between two scenarios would be much simpler since only two lines would have to be compared with each other. Second, averaging the rear-end crash risk over multiple runs should inherently account for the natural variation that occurs due to the different simulation seed values that were used. Comparing this to the field, this should account for that natural fluctuation in speed and flow that are observed over a particular location at the same time on multiple days. A plot of the average rear-

end crash risk profile for the scenario shown in Figure 7-1 is given below in Figure 7-2 to show the result of this process.

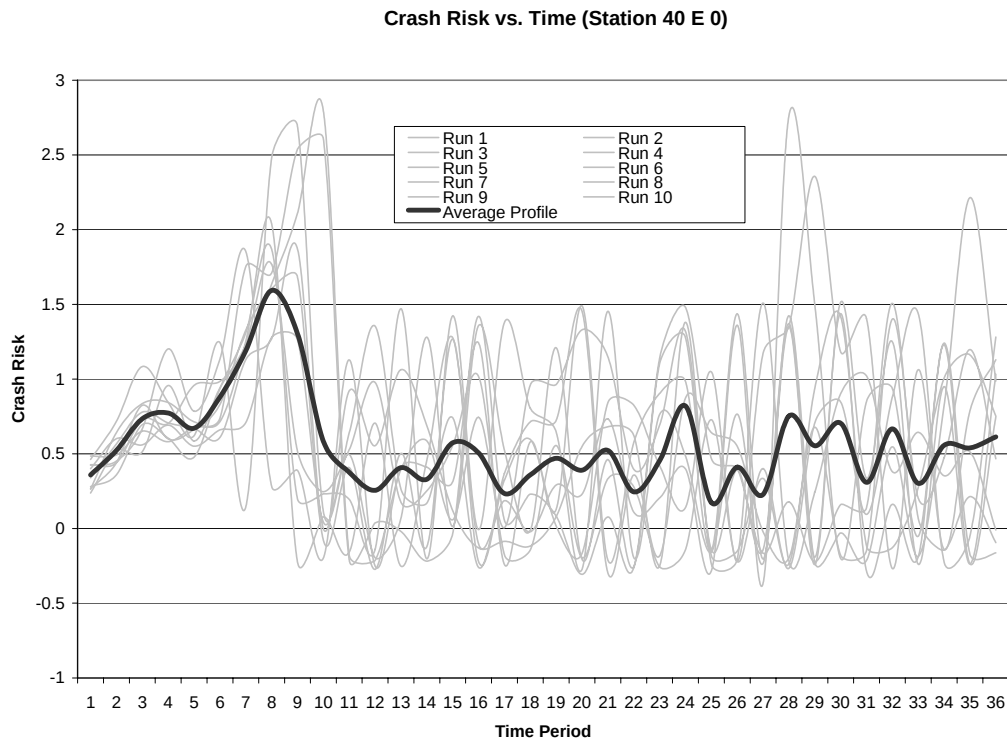


Figure 7-2: Average Crash Risk Profiles Station 40 E 0 for Test Case 25

To compare the change in the crash risk for the various route diversion and ramp metering strategies, two types of graphical comparisons were created. These two methods compare the crash risk across space (location of loop detector stations) and time, respectively. To compare the average crash risk versus location, the average risk at each station was found by computing the average of the crash risk values at individual time periods throughout the 3 hour simulation. Since the both the rear-end and lane-change crash risk are calculated every 5 minutes, this resulted in each station having 36 distinct

crash risk values per 3 hour period. The calculation of the average crash risk value for each location is shown more clearly in Equation 7-2.

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

...Equation 7-2

Where: $(Risk\_profile)_{tl}$ = the crash risk averaged over the number of runs for each location l and time t

T = 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)

$(Average\_Risk)_l$ = the crash risk averaged by both time and number of simulation runs at location l

This process was completed for the rear-end crash risk as well as the lane-change crash risk. A typical profile of the rear-end crash risk (for the base case of the 60 percent loading scenario) is shown in Figure 7-3.

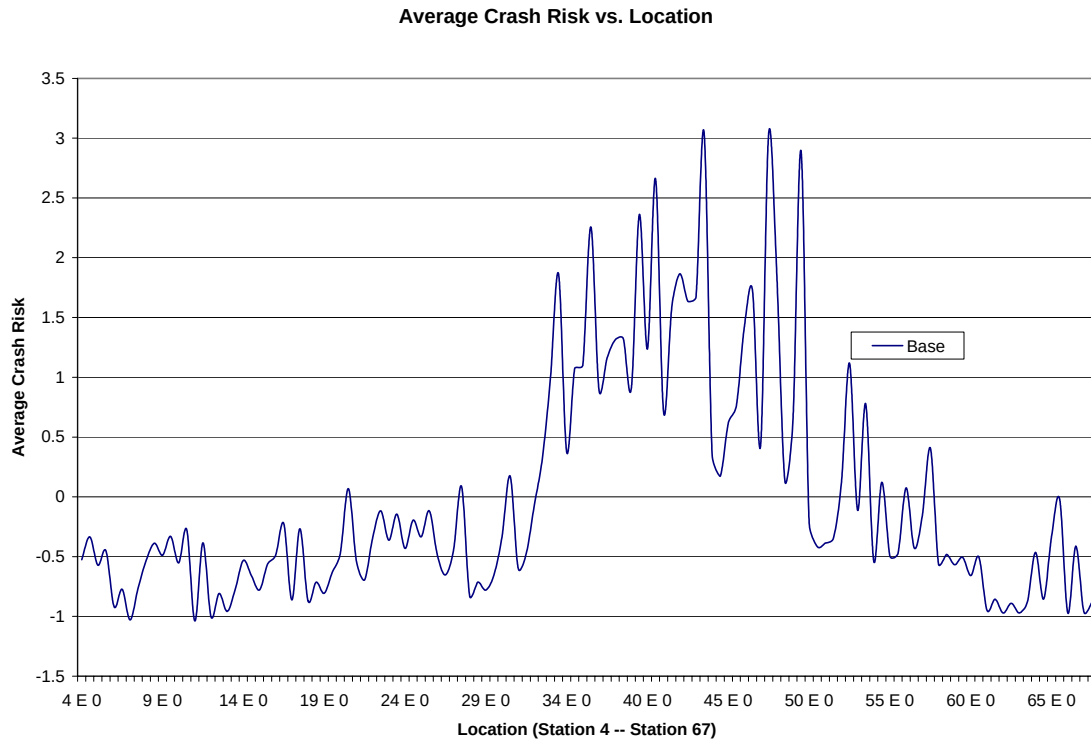


Figure 7-3: Plot of Average Crash Risk vs. Location for 60 Percent Base Scenario

Please note 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 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 (this study focuses solely on the eastbound direction of travel so this letter is always E) and the last number represents whether the area is upstream or downstream of the loop detector station. The number 0 represents an upstream area while 1 represents a downstream area. Therefore, the location named 42 E 0 represents the area upstream of loop detector station 42 in the eastbound direction. 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. Also, please note that Station 39 does not exist. Therefore, all plots show Station 38 immediately followed by Station 40.

As shown in Figure 7-3, the plot of average rear-end crash risk versus location shows numerous peaks and valleys where the rear-end crash risk increases or decreases suddenly. The sudden changes in the crash risk value over locations are caused by both the different traffic conditions at the various locations as well as the presence of on-ramps and off-ramps along the freeway. As previously mentioned, the rear-end and lane-change crash risk values are based on both on-line loop detector data and off-line information about the geometry of the freeway. The occurrence of these ramps greatly affects the value of the crash risk. Therefore, at a particular station the value of the crash risk for the upstream area and downstream area could differ greatly solely because a nearby ramp even though the loop detector data for that single location is identical.

Another type of plot that can be examined is the crash risk vs. time for a particular station. This graph will be used to ensure that the application of a particular crash prevention strategy reduces the crash risk over all (or most) time periods. A sample plot is given below in Figure 7-4 for the base case during the 90 percent loading scenario. As shown below in Figure 7-4, the rear-end crash risk is calculated every 5-minutes for the length of the 3-hour simulation run. The peaks and valleys in this plot are due solely to the interaction vehicles on the network since the location (in this example, Station 42 E 1) is constant.

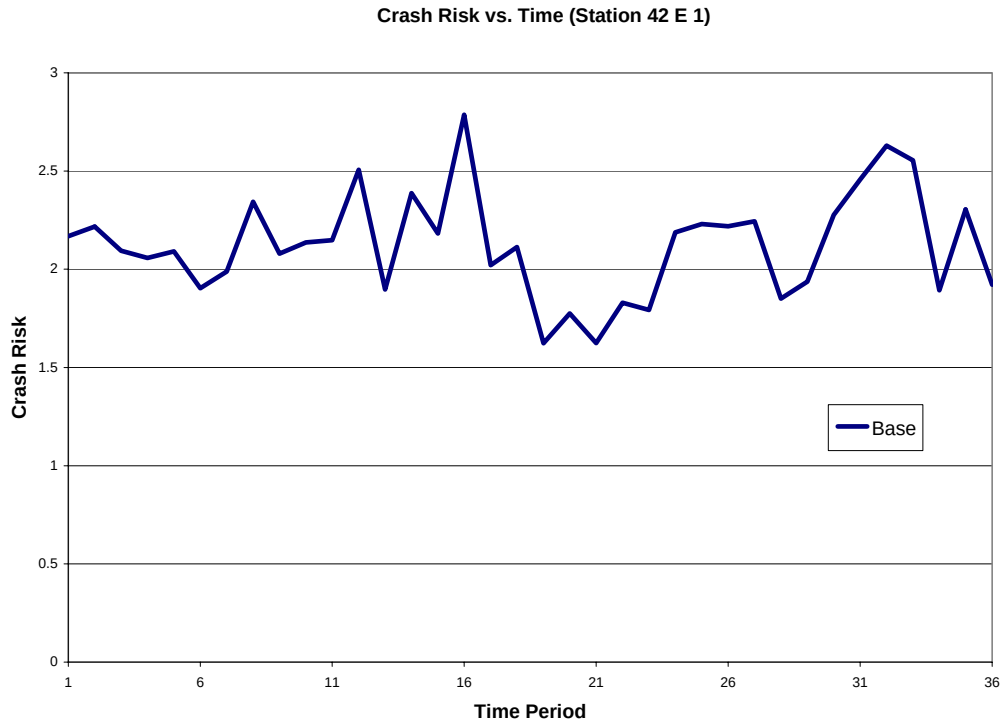


Figure 7-4: Plot of Crash Risk vs. Time for Station 42 E 1 at 90 Percent Base Case

The first type of plot (risk vs. location) will allow for the identification of areas that were affected by the various diversion and metering strategies that will be employed. Once discovered, the value of the change in risk over those individual areas can be determined by comparing the average risk value for that area for the base and test cases. Once this change in risk value is determined, the statistical significance of the change in risk will be calculated using a traditional two sample t-test. The t-test will show whether the change in rear-end or lane-change crash risk is based on the implementation of the crash prevention strategy (it is statistically significant) or random simulation error (not statistically significant).

In addition to quantifying the risk changes at particular locations the overall change in the both the rear-end and lane-change crash risk was calculated throughout the

length of the network. This measure, hereafter named the Overall Risk Change Index (ORCI) for the rear-end crash risk and the Lane-Change Risk Change Index (LCRCI) for the lane-change crash risk, is calculated by summing the differences in the average crash risk between the base case and test case for stations that exhibit a crash risk change. This value will include stations that show both a positive and negative change in the respective crash risk values. Therefore, the values of the ORCI or LCRCI can be either positive or negative which would show an overall improvement or deterioration in the crash risk, respectively. These values can then be used to compare various strategies across a single loading condition to determine which strategy more effectively reduces the respective crash risks. The result is that the strategy with the highest ORCI or LCRCI will yield the greatest overall improvement in rear-end crash risk or lane-change crash risk, respectively, over the study area.

Please note that the percent change in the risk is NOT calculated for the rear-end crash risk. The reason for this is because since the range of the risk values is (-1.5, 3.5) there is a chance that the average risk could be 0.0 for certain stations. This will lead to deceiving results as the percent change would be deceptively high for a station (or group of stations) whose average rear-end crash risk value is about equal to 0.0. When compared to another group of stations, the group with the average rear-end crash risk nearer to 0.0 will always have a much higher percent change in risk even if the change is miniscule. Therefore, in order to avoid this problem, the absolute difference in the rear-end crash risk will be determined and used to compare stations. Since the range of the lane-change crash risk is always greater than zero (0.02 to 0.06) the percent change in the crash risk is a meaningful value.

7.2 Analysis of Route Diversion

7.2.1 First diversion route

As previously mentioned in Section 6.15.1, using the 1st diversion route a vehicle can be diverted either 2 or 3 miles downstream of its initial entry ramp at Orange Blossom Trail. This on-ramp typically has a very high volume of about 1020 veh/hr in the peak hour (100 percent loading scenario). Therefore, it is expected that the effects of route diversion would be readily seen since such a large volume of vehicles can be diverted. Note that the nearer re-entry ramp typical has a volume of about 300 veh/hr while the further re-entry ramp has a much higher volume of about 970 veh/hr. Therefore, diverting too many vehicles to the further on-ramp would increase the volume beyond capacity and would lead to the deterioration of traffic operations on the ramp and surrounding surface streets.

7.2.1.1 60 Percent Loading Scenario

As previously mentioned in the Experimental Design (Section 6.15.1), there were 15 different route diversion cases that were run at the 60 percent loading case. The 15 cases were designed to test the effect of the total amount of vehicles diverted (controlled by DP-1A) as well as where the vehicles are diverted to (controlled by DP-1B). Figure 7-5 shows the average rear-end crash risk vs. location plot for the base case and cases 1 to 5 of the experimental design. Test cases 1 to 5 all have in common that 0% of the diverted vehicles are re-diverted at DP-1B which means that all the diverted vehicles re-enter I-4 at the nearer re-entry location. The cases differ in the percentage of vehicles initially diverted at DP-1A. For example, the line with reference 20% (1A) means that this line

shows the crash risk curve when 20% of the vehicles are diverted from decision point DP-1A.

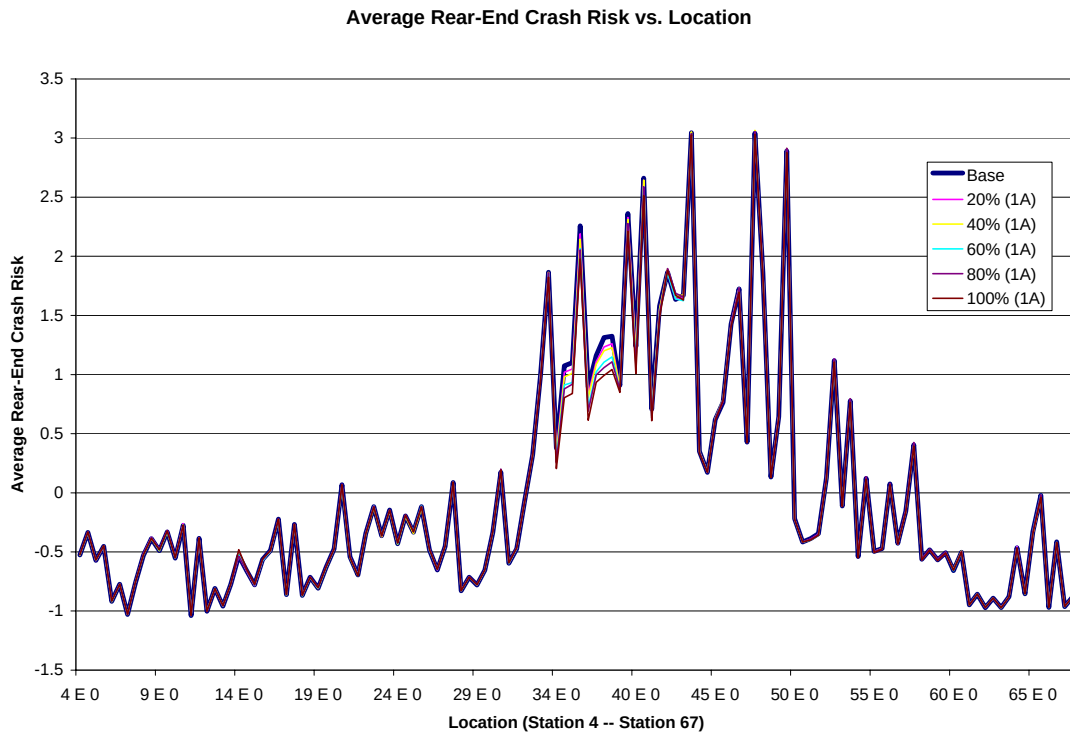


Figure 7-5: Average Rear-End Crash Risk vs. Location for Cases 1 to 5

As shown in Figure 7-5, the area over which the average rear-end crash risk is different for the test cases is rather small compared to the overall network. This makes sense since the route diversion occurs in a localized area and, therefore, should only affect the rear-end crash risk for a very short portion of the freeway. For Diversion Route 1, when vehicles forego the secondary diversion at DP-1B they are diverted from entering just before Station 34 and are instead enter the freeway at Station 38. In Figure 7-5, it appears that this area is encompassed within the area that shows a change in the rear-end crash risk along the freeway. In order to better interpret this plot it is advantageous to

zoom in on the portion that shows change in the rear-end crash risk due to the route diversion strategies. This plot is given below in Figure 7-6.

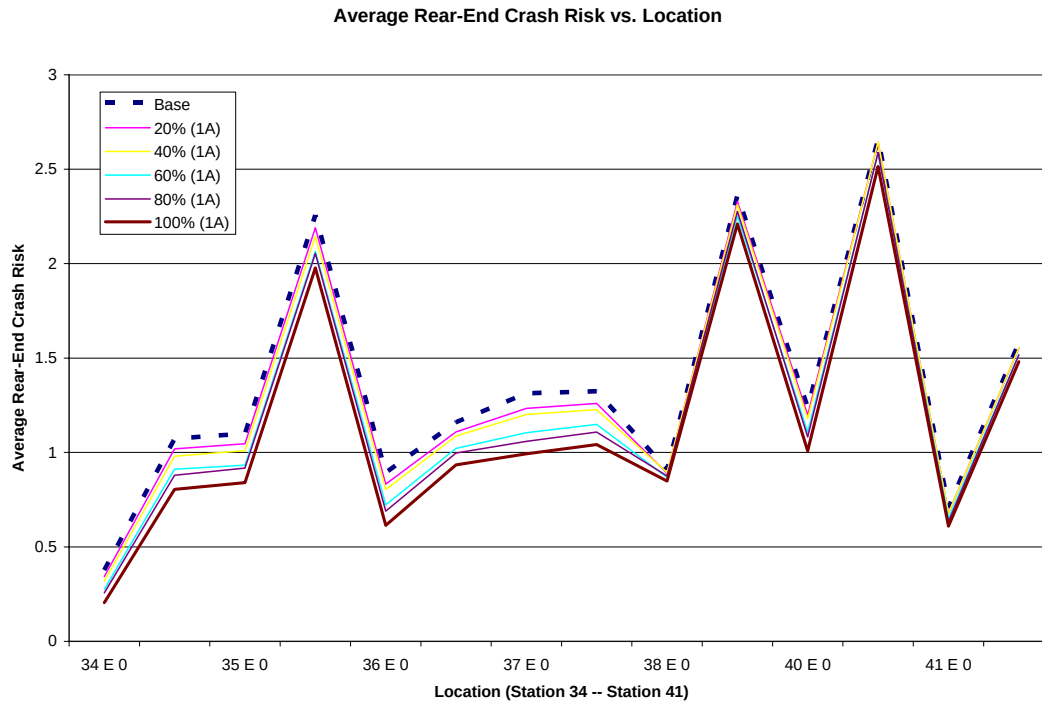


Figure 7-6: Average Rear-End Crash Risk vs. Location for Cases 1 to 5 - Magnified

As shown in Figure 7-6 (above), upon magnification of Figure 7-5 it can be seen that there is indeed a difference between the average rear-end crash risks for the different scenarios. This change in risk occurs between Stations 34 and 41, although the crash risk changes between Stations 39 and 41 are very small. Each line in the plot represents a different percentage of the vehicles diverted (% 1A) from the initial entry location on I-4. As expected, the value of the rear-end crash risk decreases proportionally with the number of vehicles that are diverted. When only 20% of the vehicles are diverted the change is minimal while there is considerable change when 100% of the vehicles are

diverted from the ramp. The average rear-end crash risk for each of the test cases along with the base case is given in Table 7-1. This table also includes the difference in the crash risk between the base case and each of the subsequent test cases. Each of the test cases is described by the case number as well as the type of diversion that was implemented. For example, test case 1 is also denoted as 20% 1A, 0% 1B. This means that for test case 1, 20% of the vehicles were diverted at the initial diversion location (DP-1A) and 0% of the vehicles were diverted at the secondary position (DP-1B).

Table 7-1: Summary of Average Rear-End Crash Risk Change for Cases 1 to 5

	Test Case ID					
	Base	Case 1	Case 2	Case 3	Case 4	Case 5
	---	20% 1A 0% 1B	40% 1A 0% 1B	60% 1A 0% 1B	80% 1A 0% 1B	100% 1A 0% 1B
Average Crash Risk (Stations 34 to 41)	1.3538	1.3088	1.2880	1.2261	1.2095	1.1486
Crash Risk Benefit	---	0.0450	0.0658	0.1277	0.1444	0.2053
T-Statistic (Benefit Significance)	---	3.7444	5.4070	9.3844	13.3827	18.7122
ORCI	---	0.6305	0.9213	1.7876	2.0213	2.8737

Please note that the amount of change has been calculated as the difference between the average risk values for the base case and scenario case for all stations that show a significant change in risk (Stations 34 to 41). What is also interesting to note is that there is a direct relationship between the amount of vehicles that are diverted and the change in the risk value. In Figure 7-6, this is shown by the fact that the amount of decrease in the rear-end crash risk at each station increases with the number of vehicles that are diverted (% 1A).

Also shown in Table 7-1 is the t-statistic comparing the change in risk between each test case and the base case versus the value zero. This value must be compared to the critical t-statistic value in order to determine whether or not the change in the crash

risk is statistically significant. The critical value for a t-test given 18 degrees of freedom and a 90% confidence level is found to be 1.7341. Since the calculated t-statistic for all levels of diversion are greater than this value, the average decrease in the rear-end crash risk for each level of diversion is shown to be statistically significant. This means that diverting any amount of vehicles at the 60 percent loading case will provide significant reductions in the rear-end crash risk between Stations 34 and 41. Additionally, Table 7-1 shows that the average overall reduction in the rear-end crash risk increases with the amount of vehicles that are diverted from DP-1A. This is also reflected in the value of the ORCI which increases with the number of vehicles that have been diverted. As previously mentioned, the ORCI value is calculated by summing the change in the crash risk for all the locations that exhibit a rear-end crash risk difference between the base case and test case.

A similar process has been carried out for cases 6 to 10 (in which 50% of the vehicles are diverted at DP-1B and, therefore, use the further re-entry location while the remaining 50% of the vehicles use the nearer re-entry location) and cases 11 to 15 (in which 100% of the vehicles are diverted at DP-1B and use the further re-entry location). The results of these analyses are presented below in Tables 7-2 and 7-3, respectively.

Table 7-2: Summary of Average Rear-End Crash Risk Change for Cases 6 to 10

	Test Case ID					
	Base	Case 6	Case 7	Case 8	Case 9	Case 10
	---	20% 1A 50% 1B	40% 1A 50% 1B	60% 1A 50% 1B	80% 1A 50% 1B	100% 1A 50% 1B
Average Crash Risk (Stations 34 to 41)	1.354	1.325	1.266	1.217	1.172	1.124
Crash Risk Benefit	---	0.028	0.088	0.136	0.182	0.230
T-Statistic (Benefit Significance)	---	2.567	6.596	13.024	11.852	19.130
ORCI	---	0.397	1.232	1.910	2.550	3.224

Table 7-3: Summary of Average Rear-End Crash Risk Change for Cases 11 to 15

	Test Case ID					
	Base	Case 11	Case 12	Case 13	Case 14	Case 15
	---	20% 1A 100% 1B	40% 1A 10% 1B	60% 1A 100% 1B	80% 1A 100% 1B	100% 1A 100% 1B
Average Crash Risk (Stations 34 to 41)	1.354	1.324	1.261	1.205	1.155	1.092
Crash Risk Benefit	---	0.030	0.093	0.149	0.198	0.262
T-Statistic (Significant of Change)	---	2.165	6.561	9.725	13.506	20.068
ORCI	---	0.418	1.299	2.085	2.777	3.666

Tables 7-2 and 7-3 show trends in the results that are similar to those presented in Table 7-1. The average change in rear-end crash risk for the cases where 50% of the vehicles are diverted at DP-1B (Table 7-2) and the cases where 100% of the vehicles are diverted at DP-1B (Table 7-3) are statistically significant for all levels of diversion. Additionally, the trend that diverting more vehicles leads to a greater reduction in the rear-end crash risk holds for these two groups of cases as well. This means that regardless of where the vehicles are diverted there will always be a safety benefit if vehicles are diverted during the 60 percent loading scenario. However, this still leaves the question of which diversion route is more preferable. When considering Tables 7-1, 7-2, and 7-3 simultaneously, it can be seen that 1) the safety benefit increases with an increase in the number of vehicles that are diverted and 2) the safety benefit increases with an increase in the number of vehicles diverted to the further re-entry area. This trend can be proved by fitting a simple linear regression equation to determine the value of the ORCI based on the values of % 1A and % 1B. The results of the linear regression analysis, presented in Table 7-4 below, show that both the percentage of vehicles diverted initially (% 1A) and the percentage of vehicles using the secondary diversion (% 1B) are

significant to the measurement of safety (ORCI). Additionally, both have a positive coefficient which states the more vehicles diverted and the more vehicles diverted to the farther re-entry location, the higher the value of ORCI which means an improved rear-end crash risk situation.

Table 7-4: Linear Regression Analysis for ORCI in Test Cases 1 to 15 (60 Percent Loading)

Parameter	Estimate	Standard Error	t Value	Pr > t
Intercept	-0.4016	0.1271	-3.16	0.0082
% 1A	0.0342	0.0017	20.16	<.0001
% 1B	0.0040	0.0012	3.42	0.0051

Please note from Tables 7-1, 7-2, and 7-3 that the area that is affected by the route diversion during this specific loading condition is the same regardless of the level of % 1B (the percentage of vehicles using the secondary diversion). This shows that, at the 60 percent loading scenario, diverting vehicles further downstream has no significant impact on the location of rear-end crash risk changes along the network for this specific diversion route. Figure 7-7 shows the location of the rear-end crash risk change along the network graphically for the various levels of % 1B tested. In this figure, the vertical axis represents location along the freeway with vehicles traveling downwards. The horizontal axis represents the different values of % 1B tested. Lighter shaded areas represent locations which realize a decrease in the rear-end crash risk with respect to the specific implementation of route diversion while darker shaded areas represent locations that realize an increase in the crash risk. Note that there is no increase in the crash risk during the 60 percent loading scenarios; therefore no dark areas are shown. Medium shades represent locations that exhibit no change in the crash risk due to route diversion. These shades can be seen upstream and downstream of Stations 34 and 41, respectively, in

Figure 7-7. The location at which vehicles are initially diverted from is denoted as a solid horizontal line. The locations where vehicles are diverted to are denoted as dashed horizontal lines. The nearer re-entry location is used only in the 0% 1B and 50% 1B case and the further re-entry location is only used in the 50% 1B and 100% 1B case. Note the 50% 1B case has two dotted lines showing the two re-entry locations that are used in this scenario. This figure will be used to compare with the results realized at higher loading levels.

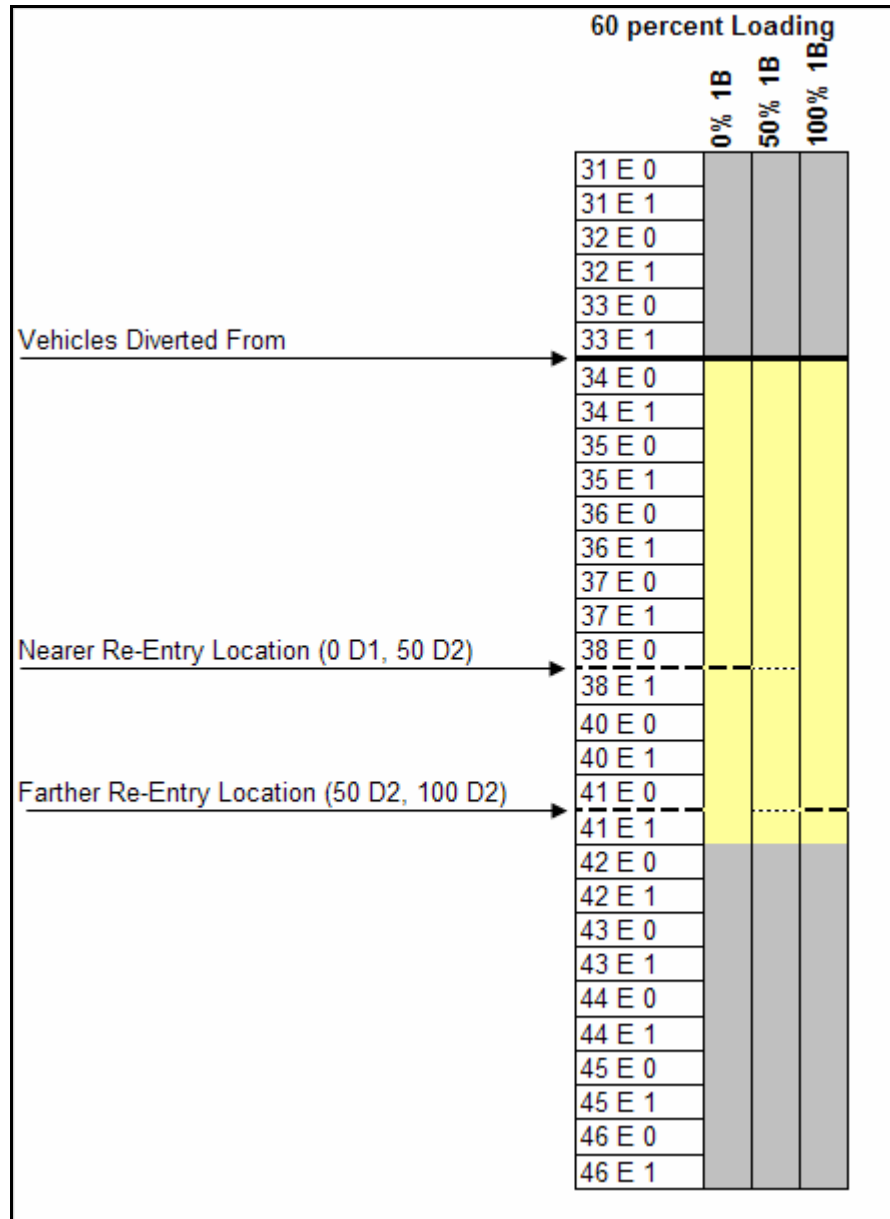


Figure 7-7: Locations Affected by Route Diversion at 60 Percent Loading Scenario

In addition to comparing the average crash risk between the base case and test cases, it is also important to look at the range of the average crash risk values for each run within a particular test case. The size of this range will show how reliable the change in crash risk is for any particular case. Figure 7-8 shows the best test case found for the 60

percent loading scenario (case 15) compared to the base case. In addition, the average crash risk range is given as a 95% confidence interval of the average rear-end crash value based on the individual simulation runs for test case 15. As shown in Figure 7-8, the range given by the 95% confidence interval is very small. Therefore, implementing route diversion in this manner should not induce any particularly high rear-end crash risk values compared to the average.

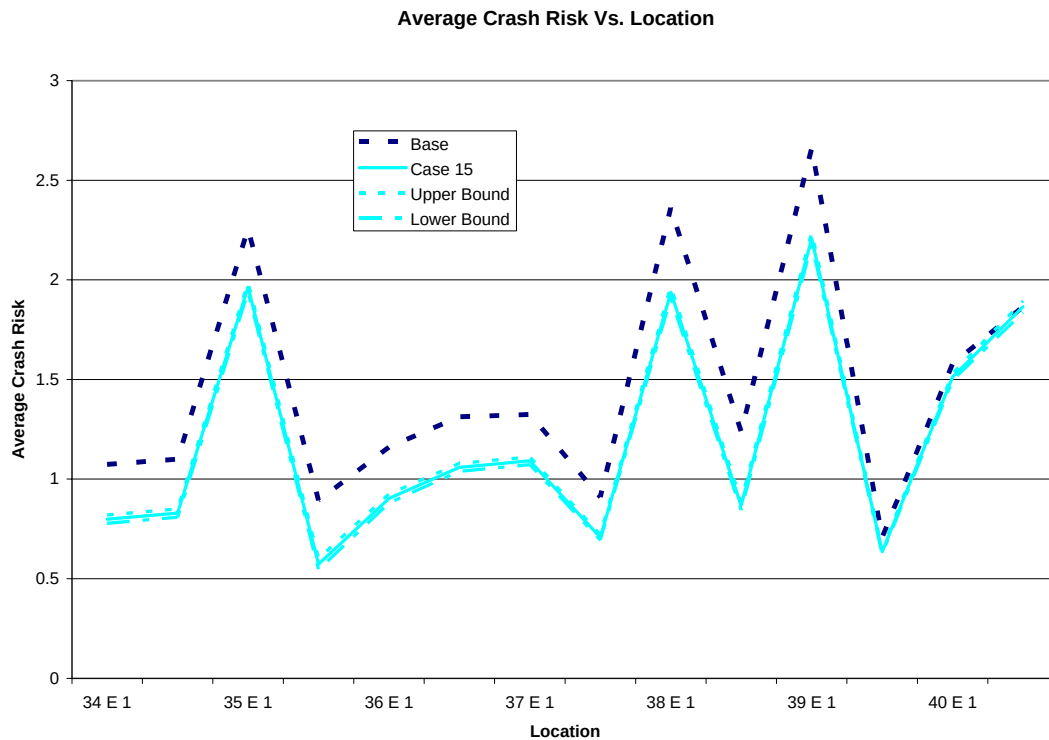


Figure 7-8: Range of Rear-End Crash Risk Values for Case 15

In addition to the average crash risk at every location, it is important to look at the crash risk over time at every location to ensure that the crash risk is reduced in real-time. A typical plot of the rear-end crash risk vs. time is provided below in Figure 7-9 for case 15. As shown, the rear-end crash risk is decreased for every time step during the

simulation. This shows the real-time benefit in addition to the overall benefit provided in the previous figures.

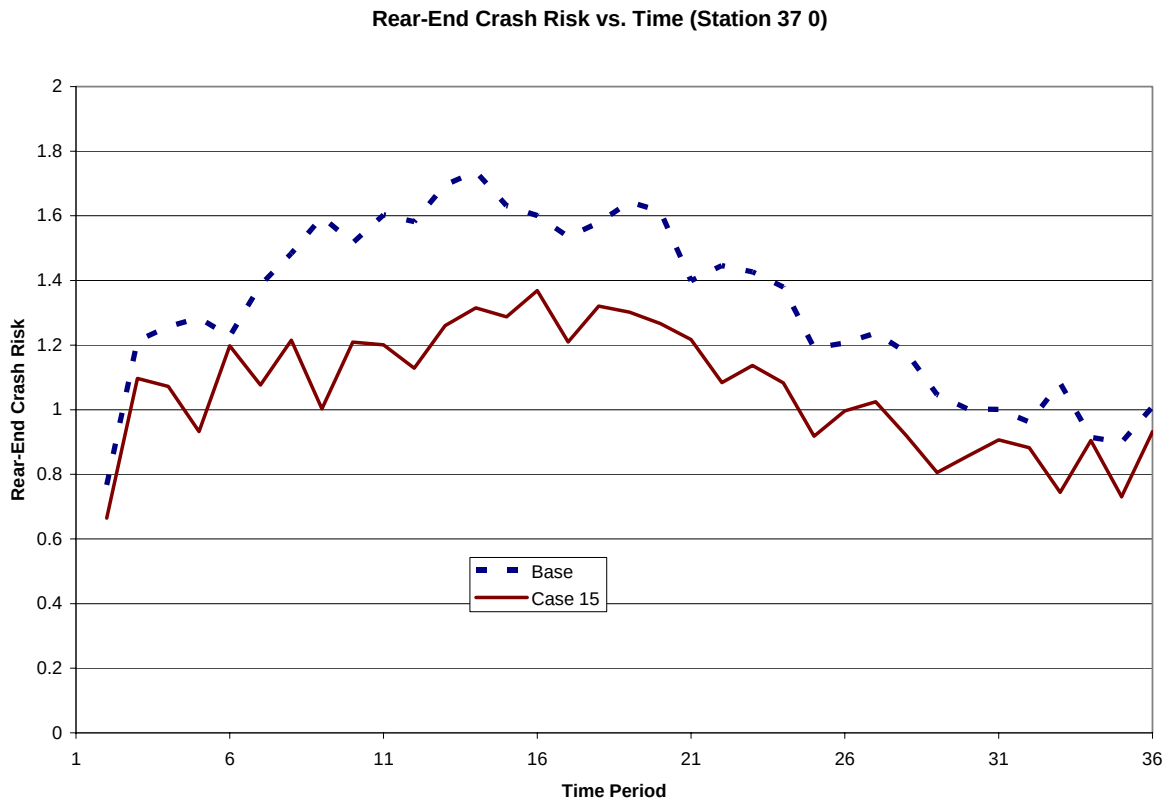


Figure 7-9: Rear-End Crash Risk vs. Time for Case 15 (Station 37 E 0)

The lane-change risk was also taken into account for these test cases. Figure 7-10 shows the lane change crash risk vs. location for the length of the simulation corridor for experimental cases 1 to 5.

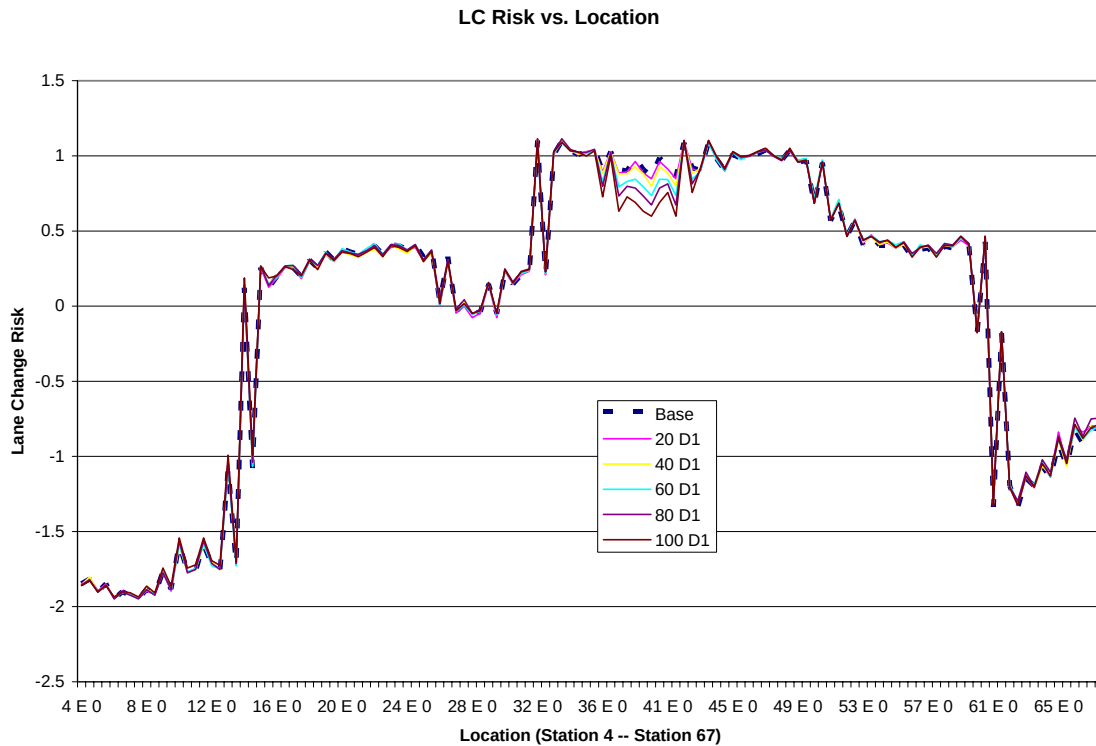


Figure 7-10: Average Lane-Change Crash Risk vs. Location for Cases 1 to 5 (20 % 1A to 100 % 1A, 0 % 1B)

This plot looks very similar to the corresponding rear-end crash plot for these test cases (Figure 7-5). As noticed, only a small area of the freeway is affected by the implementation of route diversion with respect to the lane-change crash risk. This area is magnified in Figure 7-11.

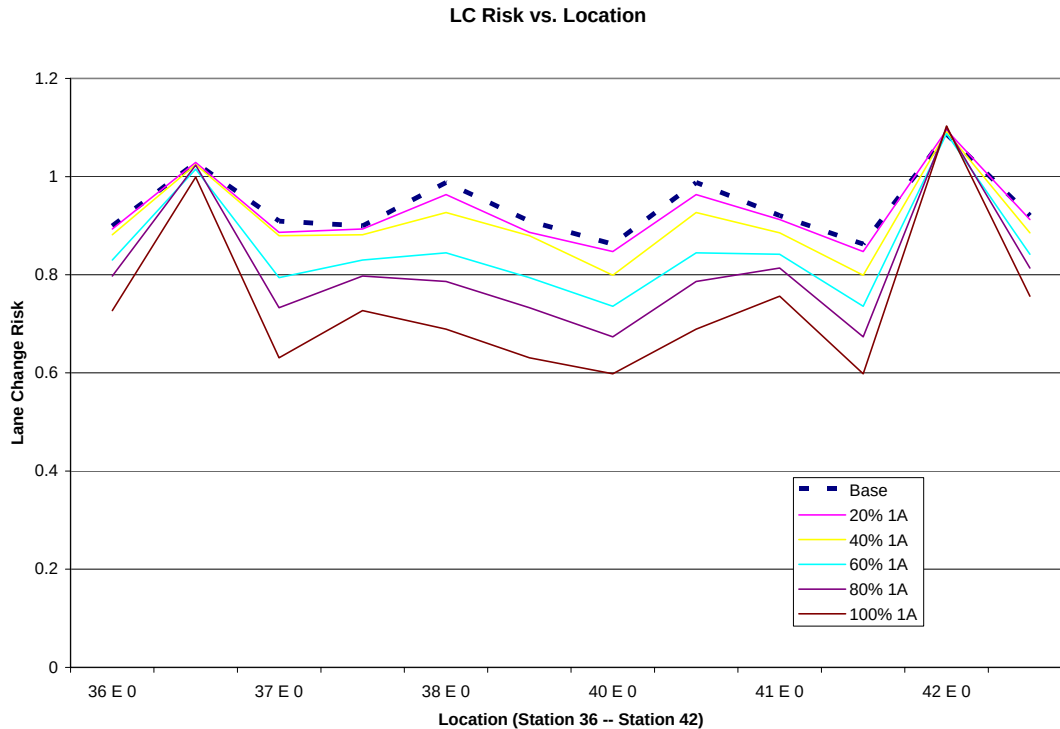


Figure 7-11: Average Lane-Change Crash Risk vs. Location for Cases 1 to 5 - Magnified

Figure 7-11 shows the same gradual trend in the reduction of the lane change crash risk as the number the diverted vehicles increase that was seen in the plot for the rear-end crash risk. Table 7-5 is a numeric summary of the change of the lane change crash risk with the number of vehicles that are diverted for test cases 1 to 5. The values of % 1A and % 1B shown in this table are defined the same as given in Tables 7-1, 7-2, and 7-3 regarding the rear-end crash risk. A measure similar to the ORCI is calculated to quantify the benefits of each test case using the lane-change risk value. This is called the Lane Change Risk Change Index (LCRCI). Note also that the crash risk benefit is calculated as a percentage for the lane change crash risk. This is allowed since the range of the lane change crash risk value does not include 0.0. Additionally, please note that

the values of the LCRCI are very small compared to the ORCI. This is due to the scale of the lane changing risk which is only 0.02 to 0.06 while the rear-end crash risk value has a scale of about -1 to 3.5.

Table 7-5: Summary of Average Lane-Change Crash Risk Change for Cases 1 to 5

	Test Case ID					
	Base	Case 1	Case 2	Case 3	Case 4	Case 5
	---	20% 1A 0% 1B	40% 1A 0% 1B	60% 1A 0% 1B	80% 1A 0% 1B	100% 1A 0% 1B
Average Crash Risk (Stations 36 to 42)	0.9395	0.9274	0.9050	0.8495	0.8108	0.7421
Crash Risk Benefit	---	0.0121	0.0345	0.0900	0.1287	0.1974
T-Statistic (Benefit Significance)	---	1.7950	4.8340	10.7082	14.6298	27.4801
LCRCI	---	0.1453	0.4137	1.0798	1.5440	2.3687

The numeric summary for the 50% 1B cases and 100% 1B cases are given in Tables 7-6 and 7-7, respectively.

Table 7-6: Summary of Average Lane-Change Crash Risk Change for Cases 6 to 10

	Test Case ID					
	Base	Case 6	Case 7	Case 8	Case 9	Case 10
	---	20% 1A 50% 1B	40% 1A 50% 1B	60% 1A 50% 1B	80% 1A 50% 1B	100% 1A 50% 1B
Average Crash Risk (Stations 36 to 42)	0.9395	0.9266	0.9075	0.8676	0.8289	0.7720
Crash Risk Benefit	---	0.0129	0.0320	0.0719	0.1106	0.1675
T-Statistic (Benefit Significance)	---	1.5796	3.5120	11.4692	13.7860	18.1482
LCRCI	---	0.1549	0.3844	0.8631	1.3271	2.0104

Table 7-7: Summary of Average Lane-Change Crash Risk Change for Cases 11 to 15

	Test Case ID					
	Base	Case 11	Case 12	Case 13	Case 14	Case 15
	---	20% 1A 100% 1B	40% 1A 100% 1B	60% 1A 100% 1B	80% 1A 100% 1B	100% 1A 100% 1B
Average Crash Risk (Stations 36 to 42)	0.9395	0.9252	0.9059	0.8686	0.8400	0.7928
Crash Risk Benefit	---	0.0143	0.0336	0.0709	0.0995	0.1467
T-Statistic (Benefit Significance)	---	2.1469	4.5328	8.2624	13.6808	18.2020
LCRCI	---	0.1713	0.4029	0.8503	1.1939	1.7599

For all cases (1 to 15) the general trend is that the crash risk benefits increase as the number of vehicles diverted increases. The change in the crash risk is also statistically significant in all cases which shows that the impact is statistically sound. What is interesting, however, is that the greatest benefit in the lane change crash risk is realized when vehicles are diverted to the nearer re-entry location (0% 1B). This opposes what is seen for the rear-end crash risk value which is minimized when vehicles are diverted further away. This is shown more clearly in the linear regression equation that is fit for this data which relates the LCRCI with % 1A and % 1B (percent of vehicles diverted at the respective diversion location). As shown in Table 7-8, the positive coefficient of the % 1A variable indicates that the LCRCI increases with the percentage of vehicles that are diverted while the negative coefficient of the % 1B variable means that the overall lane-change crash risk decreases as vehicles are diverted further downstream. One of the reasons for this could be the fact that the further re-entry location has a much higher original on-ramp volume than the nearer re-entry location. Therefore, diverting vehicles to the further re-entry location will increase the occupancy of the right most lane in the loop detectors near the on-ramp. Increasing this occupancy

increases the lane-change crash risk since the occupancy in each individual lane is an important factor in the model.

Table 7-8: Linear Regression Analysis for LCRCI in Test Cases 1 to 15 (60 Percent Loading)

Parameter	Estimate	Standard Error	t Value	Pr > t
Intercept	-0.3246	0.1089	-2.98	0.0115
% 1A	0.0237	0.0015	16.26	<.0001
% 1B	-0.0023	0.0010	-2.33	0.0383

7.2.1.2 Travel Time Analysis

Based on crash risk (rear-end and lane-change) results it is easy to say that if route diversion is being used then 100% of the vehicles should always be diverted (100% 1A) since this produces the maximum benefit in the crash risk index. However, there is an added negative effect to implementing route diversion. As vehicles are diverted from the freeway to the surface streets it is expected that the travel time would be greater as 1) vehicles would have to travel a further distance to re-enter Interstate-4, 2) the speed limit is much lower on the surface streets than the freeway, and 3) the additional traffic volume and presence of signals along the surface streets would increase congestion and cause more delays. Therefore, in addition to a plot of the change in crash risk for each test scenario, a plot of the average travel time versus the percentage of vehicles diverted has been created for cases 1 to 15 and is shown below in Figure 7-12. As can be seen, the data has been fit to three quadratic regression curves – one for each level of % 1B – to help explain the results. The R^2 values for the 0% 1B, 50% 1B, and 100% 1B curves are 0.9695, 0.8539, and 0.9189, respectively, which shows that the models provide a good fit of the data. The curves have been created to see if the change in the travel time can be

predicted with a reasonable accuracy at other diversion levels. If the curve has a good fit, this will help to determine the maximum amount of diversion that is allowed without an unreasonable increase in the overall network travel time.

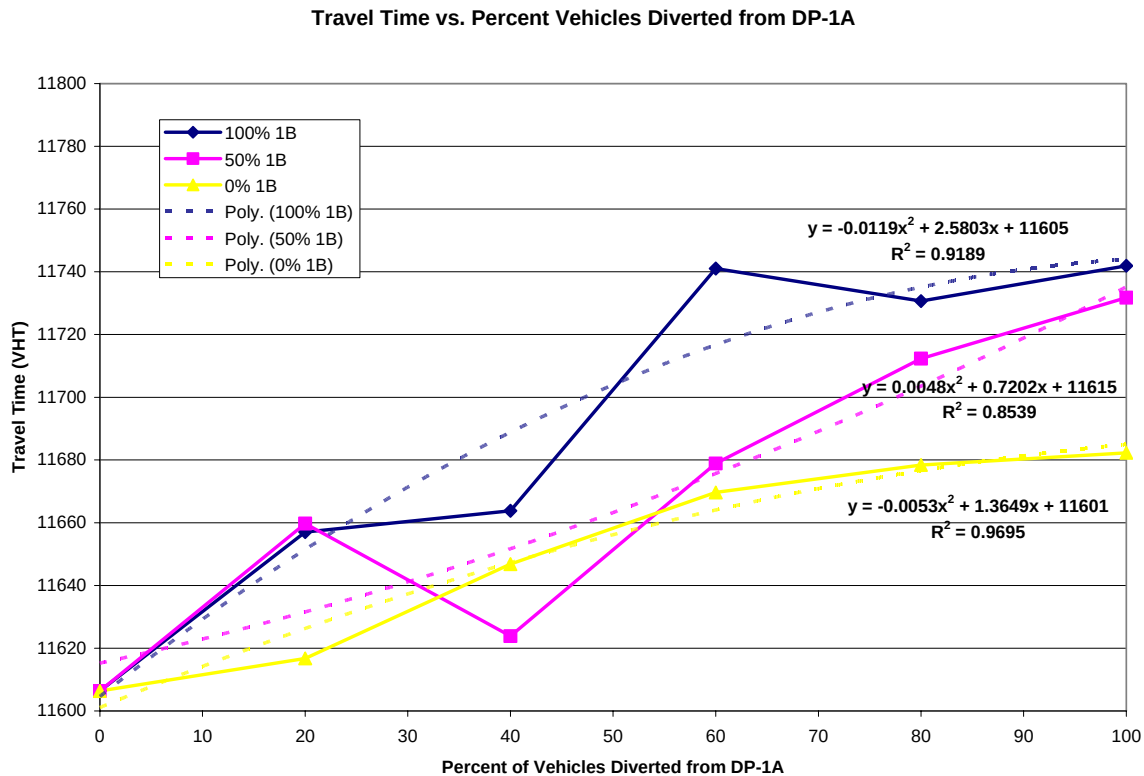


Figure 7-12: Travel Time Plot for Cases 1 to 15

As shown in the plot, the travel time and the number of vehicles that are diverted have a direct relationship. However, the maximum increase in the travel time – realized by diverting 100% of the vehicles from % 1A – is only 1.2%. Since this is a rather small increase it is safe to conclude that at the 60 percent loading scenario the maximum number of vehicles should be diverted to help reduce the crash risk the most. Please note

that this travel time increase includes the change in travel time for the freeway traffic as well as traffic on the surface streets which increases due to the route diversion.

7.2.1.3 80 Percent Loading Scenario

A similar analysis was performed for test cases 16 to 30 in the experiment design. These test cases contained the same diversion parameters (% 1A and % 1B) as cases 1 to 15 except considered the 80 percent loading scenario instead of the 60 percent loading scenario. However, there were two important differences between the scenarios performed at 60 percent loading and those performed at 80 percent loading. First, both a decrease and increase in the rear-end crash risk was noted due to the implementation of route diversion. This is significant as it shows that there is the potential for crash migration – the decreasing of rear-end crash risk at one location coupled with a simultaneous increase in rear-end crash risk at another location – due to the 1st diversion route. The second important difference is that the stations affected by the route diversion were different depending on the amount of vehicles using the secondary diversion. This is an important point as it shows that in addition to examining the magnitude of the risk change (whether positive or negative) the location of the risk change must also be assessed. The magnified plots of the average rear-end crash risk vs. location for the cases 16 to 20 (0% of vehicles using DP-1B) is presented in Figure 7-13. The plots for cases 21 to 25 (50% of the vehicles using DP-1B) and cases 26 to 30 (100% of the vehicles using DP-1B) were very similar to Figure 7-13 and are not shown.

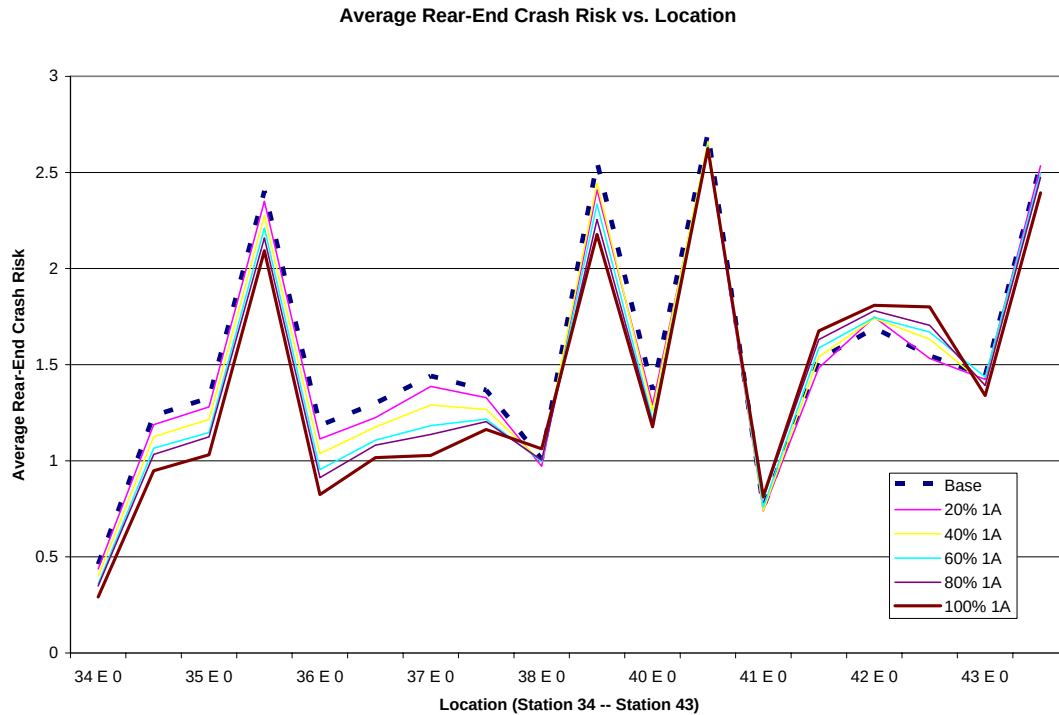


Figure 7-13: Average Rear-End Crash Risk vs. Location for Cases 16 to 20

As noted in Figure 7-13 during the 80 percent loading case, using diversion route 1 causes a decrease in the rear-end crash risk starting near the location that vehicles are diverted from. Near the area where vehicles are diverted to (Station 38, nearer re-entry point, and Station 41, farther re-entry location), however, an increase in the rear-end crash risk is noted. This is caused by the increase in the number of vehicles entering the freeway at this ramp location combined with the higher traffic volume that is currently on the freeway at that point. The added inflow on the ramp will increase the congestion in that area enough to increase the rear-end crash risk in this location. Directly downstream of this area exists a short length of freeway at which the rear-end crash risk was reduced. This is most likely a result of the added congestion upstream, at the re-entry location. The added congestion creates a small bottleneck and immediately downstream of this

bottleneck free-flow conditions persist which yields a slightly reduced rear-end crash risk versus the base case. However, this area is rather small and the effect is shown as not statistically significant in most cases. Also worth noting from Figure 7-13 is the fact that at most stations there is a direct relationship between the number of vehicles diverted and the change in the risk. Diverting 20% of the vehicles will reduce the risk less at a station of reduced risk than diverting 40% of the vehicles. This trend continues for all levels of diversion at the 80 percent loading scenario.

Figure 7-14 was created to show the locations affected by route diversion along the freeway graphically. This figure is similar to Figure 7-7 created for the 60 percent loading scenario. In Figure 7-14, the lighter colored areas represent stations that experienced a decrease in the rear-end crash risk due to route diversion while the darker areas represent areas that experienced an increase in the rear-end crash risk. Medium shades are used to represent areas that experienced no significant change in the rear-end crash risk due to the route diversion. The results of this figure are very interesting. First, the crash migration effect, described above, occurs directly downstream of the re-entry ramps in all cases. This is probably caused by the added inflow of vehicles at these ramps which increase the traffic flow and, therefore, induce short-term congestion. Second, the further vehicles are diverted from the original location, the further the effects of the route diversion on the rear-end crash risk. This result is expected since it makes sense that by diverting vehicles further away, the rear-end crash risk will be affected for a longer distance. What is also interesting is the fact that diverting vehicles further away appears to increase the overall safety benefit of the network. As shown in Figure 7-16, the further vehicles are diverted away from the original ramp location, the smaller the

area of negative safety benefit on the freeway. One reason for this could be the fact that if diverted vehicles have to travel farther to reach the re-entry location, there is a greater chance of platoons of diverted vehicles being separated by traffic signals and other vehicles on the surface streets. Therefore, they will enter the freeway in a more staggered arrival pattern which would affect the mainline flow in a less obtrusive manner. Vehicles diverted to the closer re-entry location have a greater probability of re-entering the freeway as a platoon which causes longer queues in the PARAMICS simulation.

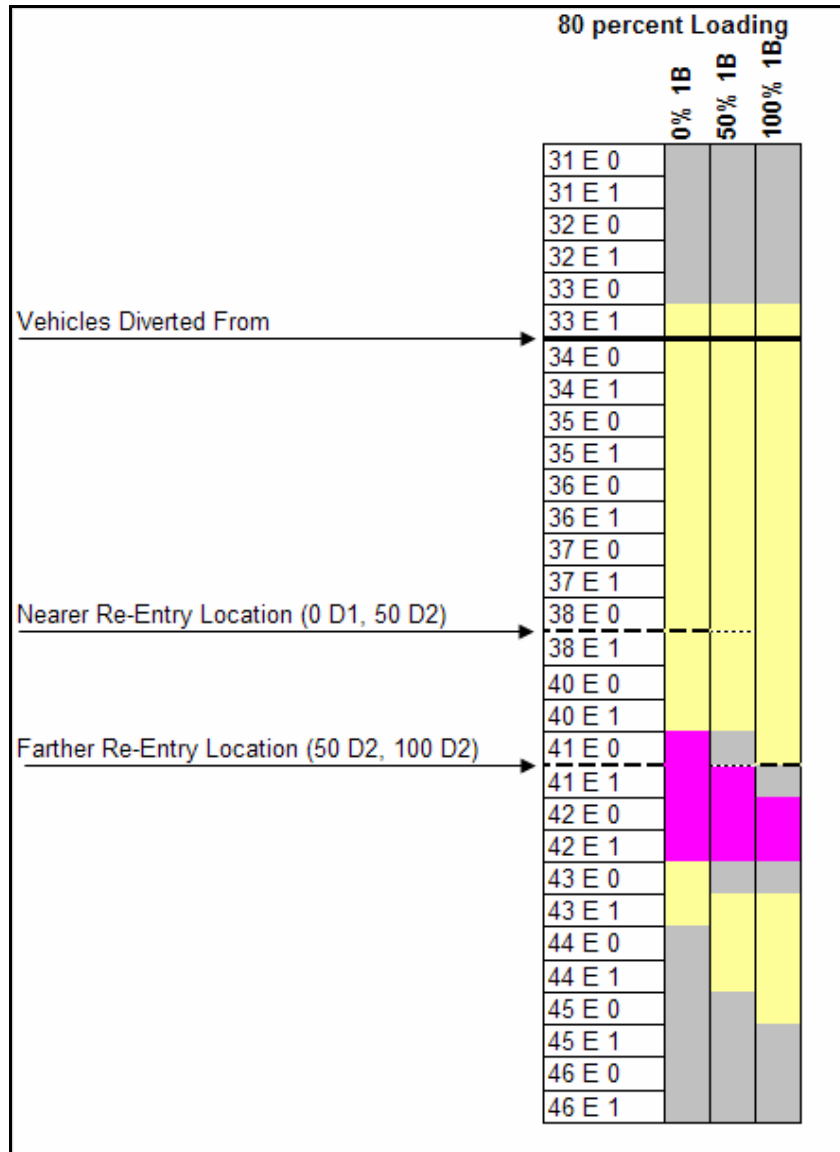


Figure 7-14: Locations Affected by Route Diversion at 80 Percent Loading Scenario

The average rear-end crash risk change over the different areas presented above in Figure 7-14 are given below in a Tables 7-9, 7-10, and 7-11 for the 0% 1B, 50% 1B, and 100% 1B cases, respectively. Instead of calculating the change in crash risk for a single area along the freeway, as was done for the cases performed at 60 percent loading, these tables show the crash risk for three distinct areas. This was done because, as mentioned previously, at 80 percent loading the route diversion caused three distinct areas of rear-end crash risk change – two areas with decreased risk and one area of increased risk.

Table 7-9: Summary of Average Rear-End Crash Risk Change for Cases 16 to 20

	Test Case ID					
	Base	Case 16	Case 17	Case 18	Case 19	Case 20
	---	20% 1A 0% 1B	40% 1A 0% 1B	60% 1A 0% 1B	80% 1A 0% 1B	100% 1A 0% 1B
Average Crash Risk (Stations 33 to 40)	1.561	1.504	1.466	1.410	1.383	1.332
Crash Risk Benefit	---	0.057	0.095	0.151	0.178	0.229
T-Statistic (Benefit Significance)	---	4.359	9.296	13.882	14.136	19.894
Average Crash Risk (Stations 41 to 42)	1.380	1.376	1.414	1.440	1.514	1.525
Crash Risk Benefit	---	0.004	-0.034	-0.060	-0.135	-0.145
T-Statistic (Benefit Significance)	---	0.106	1.099	2.096	3.548	4.113
Average Crash Risk (Station 43)	1.993	1.979	1.937	1.967	1.933	1.867
Crash Risk Benefit	---	0.015	0.056	0.027	0.061	0.127
T-Statistic (Benefit Significance)	---	0.264	1.302	0.569	1.266	2.690
ORCI	---	0.788	1.211	1.779	1.893	2.651

Table 7-10: Summary of Average Rear-End Crash Risk Change for Cases 21 to 25

	Test Case ID					
	Base	Case 21	Case 22	Case 23	Case 24	Case 25
	---	20% 1A 50% 1B	40% 1A 50% 1B	60% 1A 50% 1B	80% 1A 50% 1B	100% 1A 50% 1B
Average Crash Risk (Stations 33 to 40)	1.561	1.504	1.463	1.409	1.355	1.291
Crash Risk Benefit	---	0.057	0.098	0.152	0.206	0.270
T-Statistic (Benefit Significance)	---	4.431	8.304	12.956	15.193	23.837
Average Crash Risk (Stations 41 to 42)	1.588	1.616	1.646	1.650	1.666	1.709
Crash Risk Benefit	---	-0.028	-0.058	-0.062	-0.079	-0.121
T-Statistic (Benefit Significance)	---	0.627	1.268	1.446	2.006	3.786
Average Crash Risk (Stations 43 to 44)	1.193	1.191	1.169	1.155	1.136	1.111
Crash Risk Benefit	---	0.002	0.024	0.037	0.056	0.081
T-Statistic (Benefit Significance)	---	0.035	0.449	0.707	1.012	1.479
ORCI	---	0.666	1.173	1.901	2.613	3.391

Table 7-11: Summary of Average Rear-End Crash Risk Change for Cases 26 to 30

	Test Case ID					
	Base	Case 26	Case 27	Case 28	Case 29	Case 30
	---	20% 1A 100% 1B	40% 1A 100% 1B	60% 1A 100% 1B	80% 1A 100% 1B	100% 1A 100% 1B
Average Crash Risk (Stations 33 to 40)	1.503	1.445	1.402	1.353	1.295	1.229
Crash Risk Benefit	---	0.058	0.101	0.151	0.209	0.275
T-Statistic (Benefit Significance)	---	5.454	10.030	13.431	18.700	26.401
Average Crash Risk (Station 42)	1.618	1.622	1.675	1.700	1.693	1.775
Crash Risk Benefit	---	-0.003	-0.057	-0.082	-0.075	-0.157
T-Statistic (Benefit Significance)	---	0.060	1.088	1.593	1.258	3.324
Average Crash Risk (Stations 43 to 45)	1.112	1.091	1.091	1.093	1.029	0.977
Crash Risk Benefit	---	0.022	0.021	0.020	0.083	0.135
T-Statistic (Benefit Significance)	---	0.461	0.530	0.427	1.805	2.902
ORCI	---	0.899	1.390	2.026	3.105	4.069

The results show that increasing the percentage of vehicles that are diverted (increasing % 1A) increases the safety benefit in the upstream locations (near the initial diversion location) and increases the rear-end crash risk at the re-entry location. Additionally, diverting vehicles farther away from the original entry point increases the overall safety on the freeway. This is seen by the increasing ORCI values at the same level of diversion (% 1A) for increasing levels of % 1B. A simple linear regression analysis was performed again to confirm these results. The results (given below) show that the values of % 1A and % 1B are both statistically related to the value of ORCI.

Table 7-12: Linear Regression Analysis for ORCI in Test Cases 16 to 30 (80 Percent Loading)

Parameter	Estimate	Standard Error	t Value	Pr > t
Intercept	-0.2818	0.1918	-1.47	0.1676
% 1A	0.0323	0.0026	12.58	<.0001
% 1B	0.0063	0.0018	3.57	0.0039

Once again a plot was created to analyze the best case found above (case 30) with respect to the variation within the individual runs. This plot is given below in Figure 7-15. As shown in this figure, the range of the average rear-end crash risk values is relatively small for most of the locations that show a crash risk change. However, between Stations 41 to 44 there is fair amount of variation which shows that the average crash-risk value might be slightly higher or lower than the average. However, this change is not that great compared to the base case and should not be considered significant.

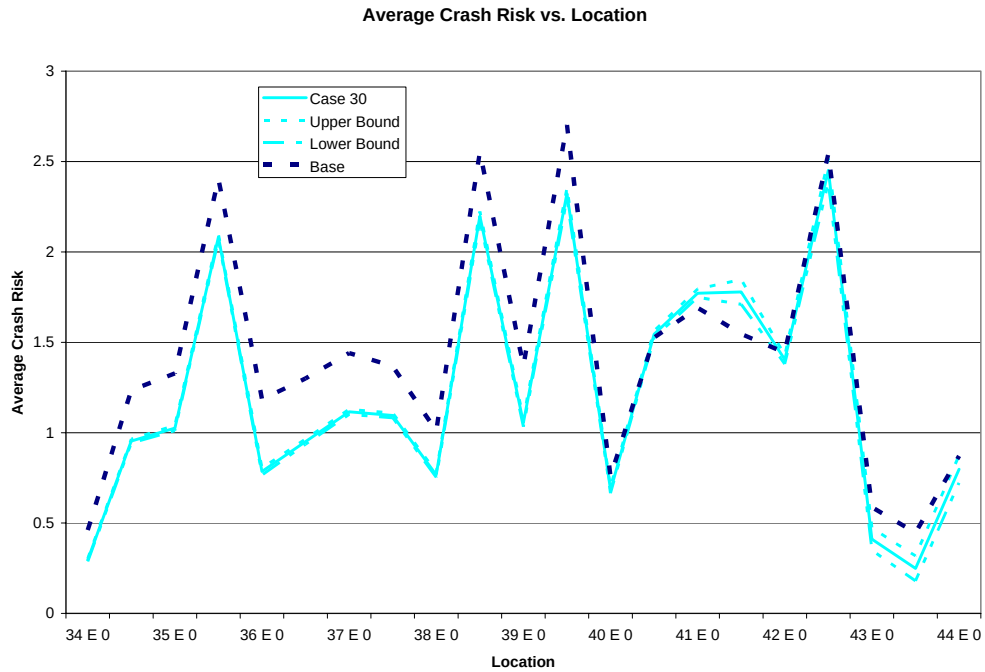


Figure 7-15: Range of Rear-End Crash Risk Values for Case 30

Plots were also created to analyze the effect of the route diversion on the lane-change crash risk at the 80 percent loading scenario. These plots are provided below in Figures 7-16, 7-17, and 7-18 for cases 16 to 20, 21 to 25, and 26 to 30, respectively. As shown in those Figures, the lane-change crash risk only differed between stations 36 to 42.

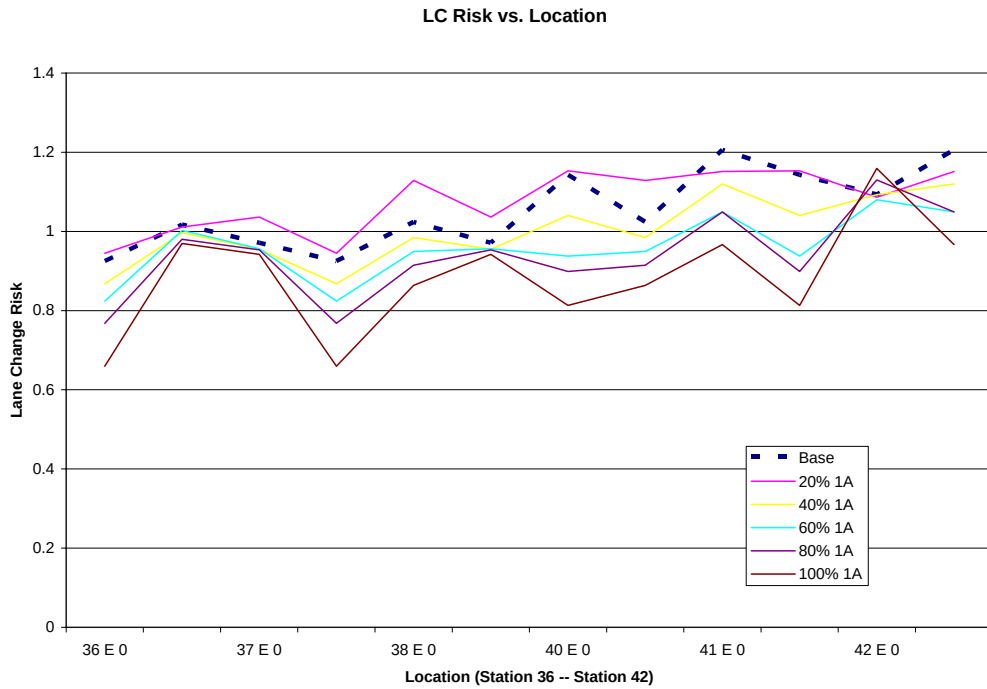


Figure 7-16: Average Lane-Change Crash Risk vs. Location for Cases 16 to 20

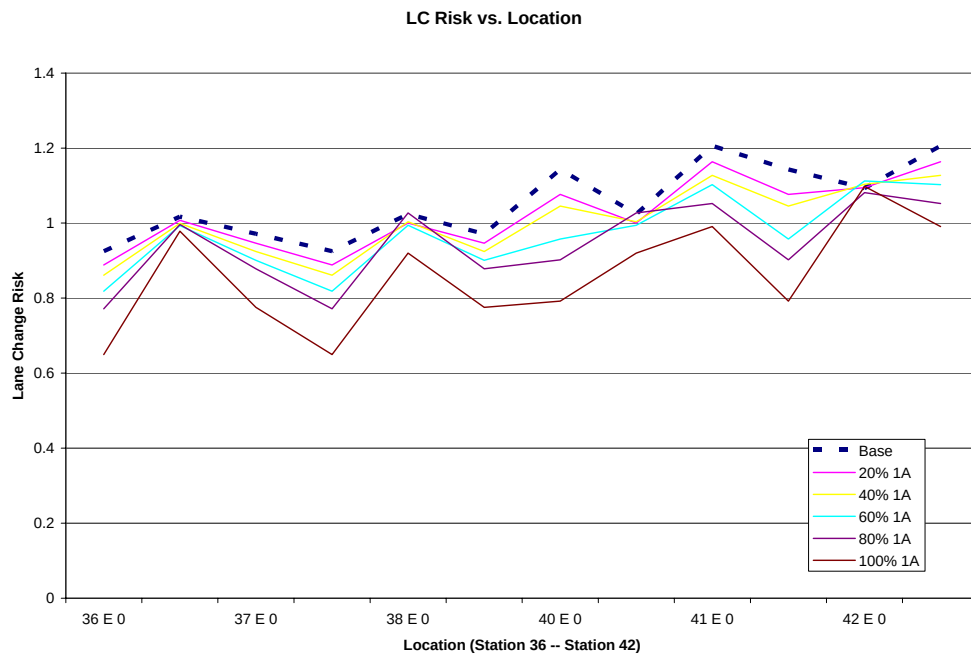


Figure 7-17: Average Lane-Change Crash Risk vs. Location for Cases 21 to 25

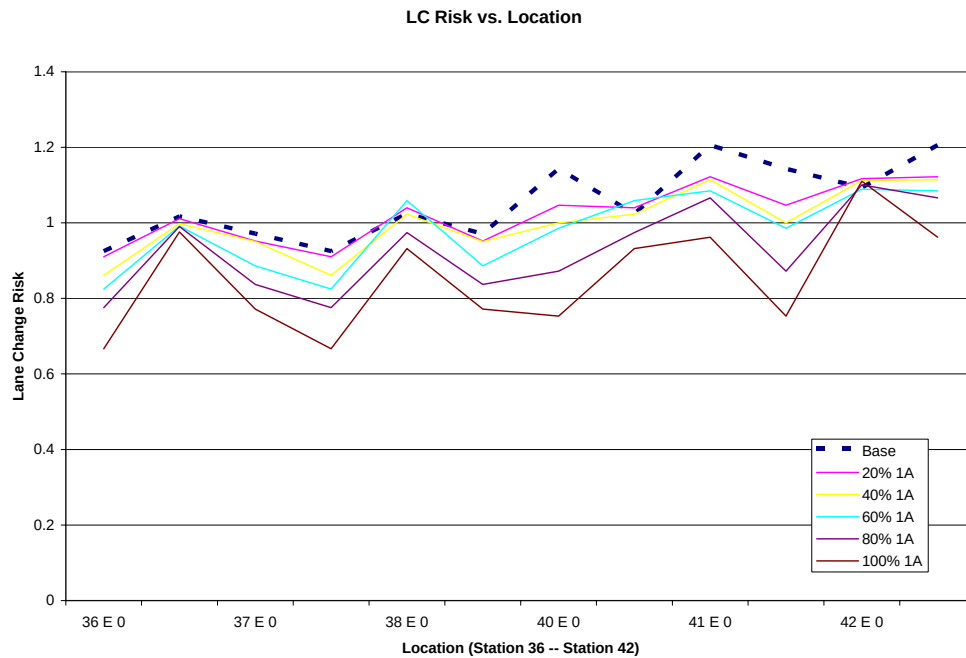


Figure 7-18: Average Lane-Change Crash Risk vs. Location for Cases 26 to 30

As shown in these plots, the lane-change crash risk decreases gradually with the number of vehicles that are diverted. Except for the few locations when there is no change between the base case and test cases (Station 36 E 1, for example) the change is logical in that the 20% 1A case has a higher risk than the 40% 1A case which, in turn, has a higher crash risk than the 60% 1A case and so on. The numerical summary of this risk value at the 80 percent loading case is given below in Tables 7-13, 7-14, and 7-15.

Table 7-13: Summary of Average Lane-Change Crash Risk Change for Cases 16 to 20

	Test Case ID					
	Base	Case 16	Case 17	Case 18	Case 19	Case 20
	---	20% 1A 0% 1B	40% 1A 0% 1B	60% 1A 0% 1B	80% 1A 0% 1B	100% 1A 0% 1B
Average Crash Risk (Stations 36 to 42)	1.0541	1.0771	1.0021	0.9596	0.9398	0.8848
Crash Risk Benefit	---	-0.0230	0.0520	0.0945	0.1143	0.1693
T-Statistic (Benefit Significance)	---	0.5500	2.1193	4.2403	4.5826	7.3334
LCRCI	---	-0.2765	0.6240	1.1340	1.3718	2.0311

Table 7-14: Summary of Average Lane-Change Crash Risk Change for Cases 21 to 25

	Test Case ID					
	Base	Case 21	Case 22	Case 23	Case 24	Case 25
	---	20% 1A 50% 1B	40% 1A 50% 1B	60% 1A 50% 1B	80% 1A 50% 1B	100% 1A 50% 1B
Average Crash Risk (Stations 36 to 42)	1.0541	1.0209	1.0023	0.9710	0.9450	0.8610
Crash Risk Benefit	---	0.0331	0.0518	0.0831	0.1091	0.1931
T-Statistic (Benefit Significance)	---	1.2178	1.8977	3.1574	2.8484	7.7045
LCRCI	---	0.3977	0.6216	0.9969	1.3087	2.3167

Table 7-15: Summary of Average Lane-Change Crash Risk Change for Cases 26 to 30

	Test Case ID					
	Base	Case 26	Case 27	Case 28	Case 29	Case 30
	---	20% 1A 100% 1B	40% 1A 100% 1B	60% 1A 100% 1B	80% 1A 100% 1B	100% 1A 100% 1B
Average Crash Risk (Stations 36 to 42)	1.0541	1.0224	1.0007	0.9802	0.9284	0.8547
Crash Risk Benefit	---	0.0317	0.0534	0.0739	0.1257	0.1994
T-Statistic (Benefit Significance)	---	1.2799	1.9297	2.0889	5.3954	9.6223
LCRCI	---	0.3809	0.6411	0.8872	1.5085	2.3923

As shown in the Tables and linear regression equation (Table 7-16), the general trend of the LCRCI at the 80 percent loading case is to increase with higher values of % 1A and % 1B. This is contrary to what was observed at the 60 percent loading case but since the parameter estimate for % 1B is insignificant ($p > 0.05$) it can be assumed that there is no real difference in the lane change risk change index for different levels of % 1B. A comparison of the tables shows that this makes sense since there is no definitive pattern of the LCRCI values compared to various levels of % 1B.

Table 7-16: Linear Regression Analysis for LCRCI in Test Cases 16 to 30 (80 Percent Loading)

Parameter	Estimate	Standard Error	t Value	Pr > t
Intercept	-0.4814	0.1479	-3.25	0.0069
% 1A	0.0246	0.0020	12.46	<.0001
% 1B	0.001851	0.001369	1.35	0.2013

7.2.1.3.1 Travel Time Analysis

Once again a travel time analysis was performed to ensure that the change in travel time due to route diversion is reasonable. Figure 7-19 shows the plot that was created to examine this. Quadratic equations were fit for the 0 % 1B, 50 % 1B, and 100 % 1B cases. The latter two equations have reasonable R^2 values. However, for the 0 % 1B cases a quadratic equation does not provide a good fit for the data as its R^2 value was extremely low at 0.1423. However, as shown in Figure 7-19, since the travel time does not change very much for this level of % 1B this should be okay since the expected travel time at the various diversion levels does not change much compared to the base case.

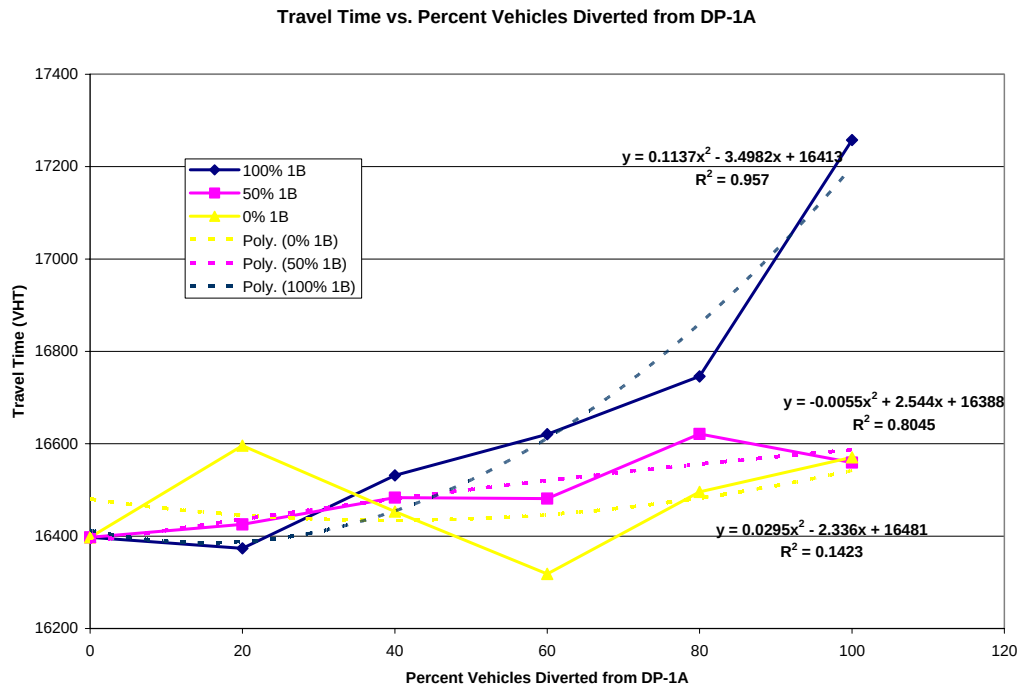


Figure 7-19: Travel Time Plot for Cases 16 to 30

The largest increase in travel time due to route diversion is 5.2%. This occurs when 100 percent of the vehicles are diverted to the further re-entry location. Since this is close to 5%, the target that is deemed reasonable, it can be concluded that diverting the maximum number of vehicles can be diverted at the 80 percent loading case would results in the maximum amount of risk without significantly increasing the travel time.

7.2.1.4 90 Percent Loading Scenario

Cases 31 to 45 all test the 1st diversion route during the 90 percent loading scenario. There are a few differences in the results between the cases run at the 80 percent loading scenario and the 90 percent loading scenario. The first major difference is that the area of the freeway that is affected by route diversion is significantly increased.

This is caused primarily because of the high traffic volume that is present at this loading scenario. Since the network is at typical levels of congestion during this scenario, diverting some of the vehicles serves to reduce the severity of some of the long queues that form during congestion. This essentially causes the traffic conditions to change from regime 1 to regime 2 which is the reason the rear-end crash risk models for regime 1 and 2 had to be normalized and combined into a single crash risk metric. The second difference is that the statistical significance of the crash risk increase near the re-entry areas is much greater than at the previous loading scenario. This means that diverted vehicles cause a much more significant increase in the crash risk than seen previously and the effects of crash migration are more apparent.

Figures 7-20 and 7-21 show the plot of the average rear-end crash risk vs. location for cases 31 to 35 in the experimental design. Note that two figures are needed to show the larger area that is affected by route diversion at this level of loading.

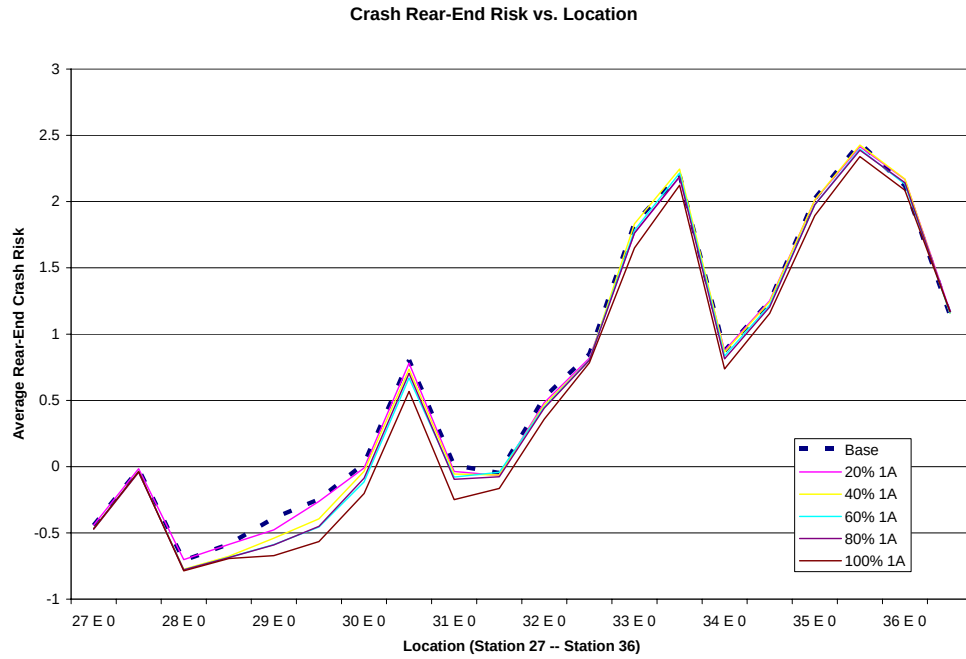


Figure 7-20: Average Rear-End Crash Risk vs. Location for Cases 31 to 35

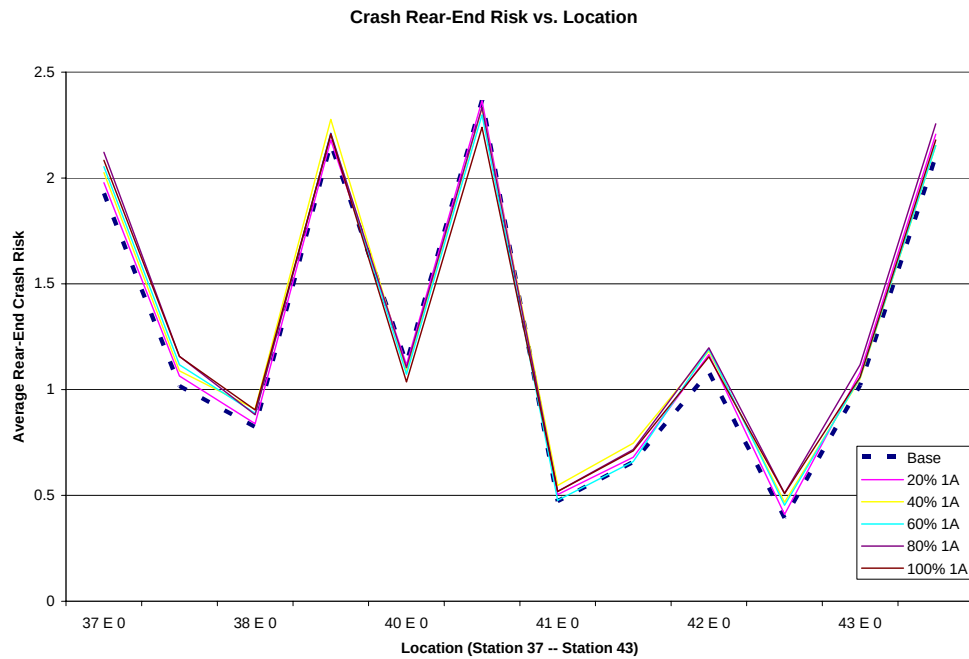


Figure 7-21: Average Rear-End Crash Risk vs. Location for Cases 31 to 35 - 2

The numerical summary of the change in rear-end risk realized in cases 31 to 35 is given below in Table 7-17. The changes in the rear-end crash risk for this case were contained within two distinct areas – one area with a crash risk decrease and another with a crash risk increase. The reduction in the crash risk only becomes statistically significant at the 60 percent diversion case while the increase in the crash risk is significant at diversion levels higher than 20 percent. As shown in Table 7-17, the value of ORCI does not always increase as the percentage of vehicles that are diverted from DP-1A increase. While all levels of diversion have a positive value of ORCI (which means an overall positive rear-end crash risk change) the fact that the crash risk increase at Stations 37 to 42 is statistically significant at lower levels of diversion shows that crash risk migration issue is more significant than the crash risk decrease. Therefore, there is a chance that the crash risk will be increased to unacceptably high levels at certain locations.

Table 7-17: Summary of Average Rear-End Crash Risk Change for Cases 31 to 35

	Test Case ID					
	Base	Case 31	Case 32	Case 33	Case 34	Case 35
	---	20% 1A 0% 1B	40% 1A 0% 1B	60% 1A 0% 1B	80% 1A 0% 1B	100% 1A 0% 1B
Average Crash Risk (Stations 28 to 35)	0.679	0.653	0.631	0.600	0.595	0.517
Crash Risk Benefit	---	0.026	0.048	0.079	0.084	0.162
T-Statistic (Benefit Significance)	---	0.649	1.444	2.169	2.506	4.157
Average Crash Risk (Stations 37 to 42)	1.203	1.230	1.266	1.242	1.275	1.252
Crash Risk Benefit	---	-0.027	-0.063	-0.039	-0.072	-0.049
T-Statistic (Benefit Significance)	---	1.188	2.559	1.491	2.996	1.908
ORCI	---	0.144	0.135	0.866	0.622	2.096

Similar results are seen for test cases 36 to 40 and 41 to 45. The summary of those cases are given below in Tables 7-18 and 7-19. Cases 37 and 38 show the first instances of negative ORCI values. This means that diverting vehicles in this manner decreases the overall safety along the network corridor. The cases involving diverting all vehicles to the further re-entry location (100% 1B) all show much higher decreases in the rear-end crash risk than using the nearer on-ramp exclusively or splitting the diverted vehicles between the two options.

Table 7-18: Summary of Average Rear-End Crash Risk Change for Cases 36 to 40

	Test Case ID					
	Base	Case 36	Case 37	Case 38	Case 39	Case 40
	---	20% 1A 50% 1B	40% 1A 50% 1B	60% 1A 50% 1B	80% 1A 50% 1B	100% 1A 50% 1B
Average Crash Risk (Stations 28 to 36)	0.763	0.702	0.744	0.699	0.652	0.605
Crash Risk Benefit	---	0.061	0.019	0.065	0.112	0.158
T-Statistic (Benefit Significance)	---	1.580	0.593	1.773	2.807	4.466
Average Crash Risk (Stations 37 to 43)	1.263	1.305	1.353	1.367	1.362	1.402
Crash Risk Benefit	---	-0.042	-0.090	-0.105	-0.099	-0.139
T-Statistic (Benefit Significance)	---	2.031	4.084	5.424	5.112	6.168
ORCI	---	0.535	-0.754	-0.155	0.704	1.021

Table 7-19: Summary of Average Rear-End Crash Risk Change for Cases 41 to 45

	Test Case ID					
	Base	Case 41	Case 42	Case 43	Case 44	Case 45
	---	20% 1A 100% 1B	40% 1A 100% 1B	60% 1A 100% 1B	80% 1A 100% 1B	100% 1A 100% 1B
Average Crash Risk (Stations 28 to 37)	0.845	0.810	0.818	0.713	0.600	0.456
Crash Risk Benefit	---	0.034	0.027	0.132	0.245	0.389
T-Statistic (Benefit Significance)	---	1.087	0.716	3.821	7.632	14.207
Average Crash Risk (Stations 41 to 43)	0.955	1.032	1.009	1.041	1.055	1.131
Crash Risk Benefit	---	-0.077	-0.054	-0.086	-0.100	-0.176
T-Statistic (Benefit Significance)	---	2.571	2.175	2.850	4.122	8.164
Average Crash Risk (Stations 44 to 45)	0.914	0.940	0.952	0.915	0.868	0.777
Crash Risk Benefit	---	-0.026	-0.038	0.000	0.047	0.137
T-Statistic (Benefit Significance)	---	0.961	1.616	0.020	1.652	5.577
ORCI	---	0.113	0.070	1.995	4.188	6.746

Table 7-20: Linear Regression Analysis for ORCI in Test Cases 31 to 45 (90 Percent Loading)

Parameter	Estimate	Standard Error	t Value	Pr > t
Intercept	-2.1239	0.9772	-2.17	0.0505
% 1A	0.0403	0.0131	3.09	0.0094
% 1B	0.0185	0.0090	2.04	0.0635

The linear regression analysis, presented in Table 7-20, show that the ORCI values increase with increasing values of % 1A and % 1B. However, even though the fit of the data to the regression line is statistically significant the results are in question based on the inconsistencies seen in the ORCI in Tables 7-17, 7-18, and 7-19. Especially in Table 7-18 which shows that diverting 20 percent of the vehicles (20% 1A) increases the safety of the freeway while diverting 40 or 60 percent of the vehicles (40% 1A, 60%

1A) decreases the safety. The regression parameters are also in question due to the large ORCI values of cases 44 and 45 on the model fit. These large values, 4.188 and 6.746, respectively, tend to influence the model more than the other values and therefore cause the linear regression model to be significant when values of the ORCI show otherwise. Therefore, this model should not be used to describe the data and it would appear that the application of route diversion at 90 percent loading is inconsistent at reducing the rear-end crash risk with respect to the variables % 1A and % 1B.

Figure 7-22 shows which locations the implementation of route diversion affects the rear-end crash risk at the 90 percent loading scenario. As expected when vehicles are diverted further away from the original ramp (50% 1B and 100% 1B cases) the location of rear-end crash risk change moves further downstream. The 50% 1B case affects both the nearer and further re-entry locations since vehicles are diverted to both locations.

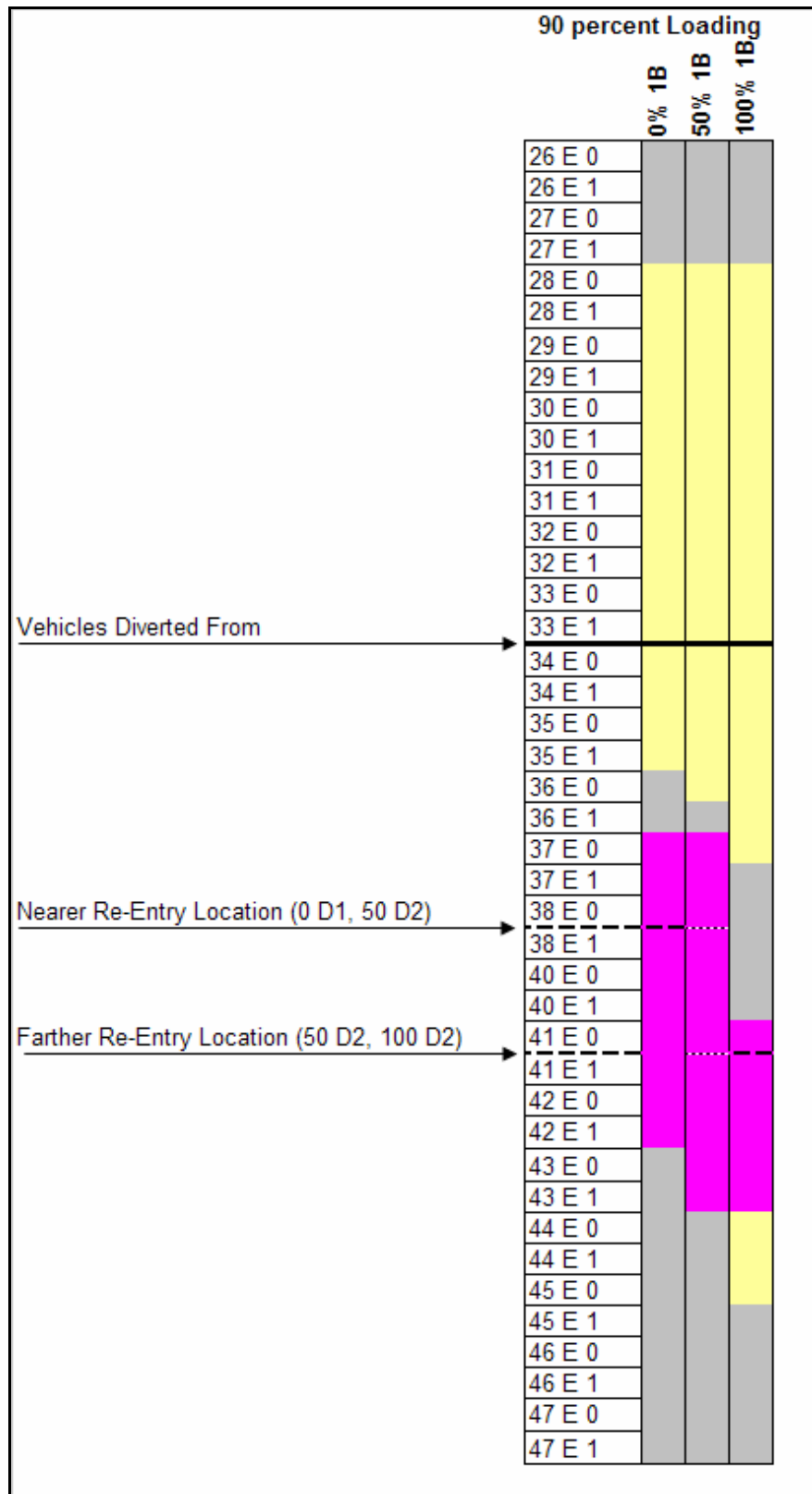


Figure 7-22: Locations Affected by Route Diversion at 90 Percent Loading Scenario

Figures 16-23 and 7-24 show the variation in the average rear-end crash risk vs. location. As shown, there is more variation for the best case (case 45) at the 90 percent loading scenario than the 60 or 80 percent loading scenarios. However, this variation is still rather modest and the upper bound does not deviate from the results found previously.

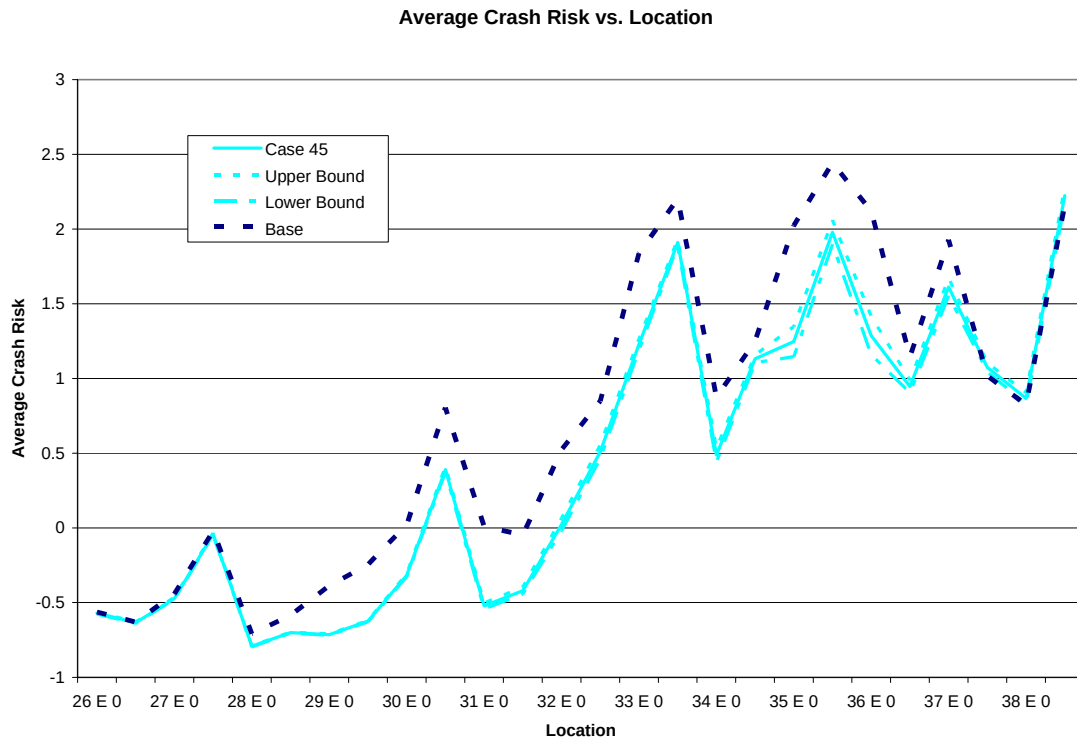


Figure 7-23: Range of Rear-End Crash Risk Values for Case 45

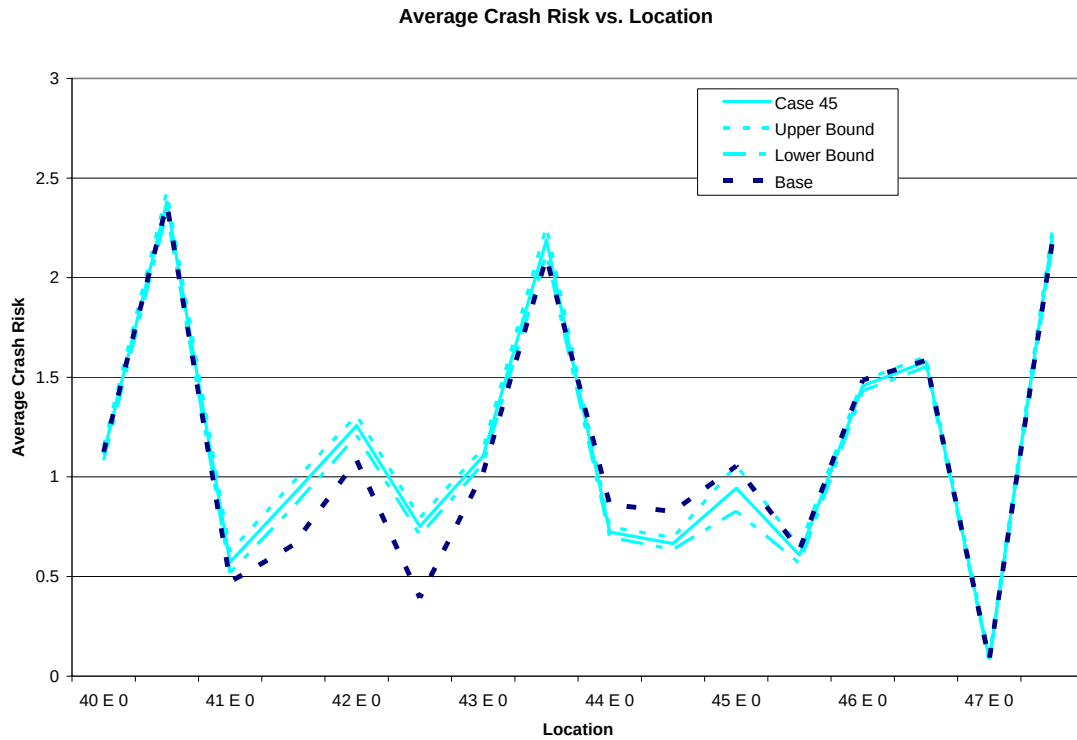


Figure 7-24: Range of Rear-End Crash Risk Values for Case 45 - 2

The effect of the route diversion on the lane-change crash risk is quite similar to the results found for the rear-end crash risk. Figure 7-25 shows a plot of the lane change crash risk for the areas affected by route diversion for cases 36 to 40. Please note in this figure the randomness of the order of LCRCI for different diversion values across different stations. There does not appear to be a pattern present for the crash risk vs. the percent of vehicles diverted like shown in the lower loading scenarios. Tables 7-21, 7-22, and 7-23 summarize the results for the lane-change crash risk for cases 36 to 40, 41 to 45, and 46 to 50, respectively. Note in Table 7-21 that diverting vehicles at the 90 percent loading scenarios almost always causes an increase in the overall lane-change crash risk along the network.

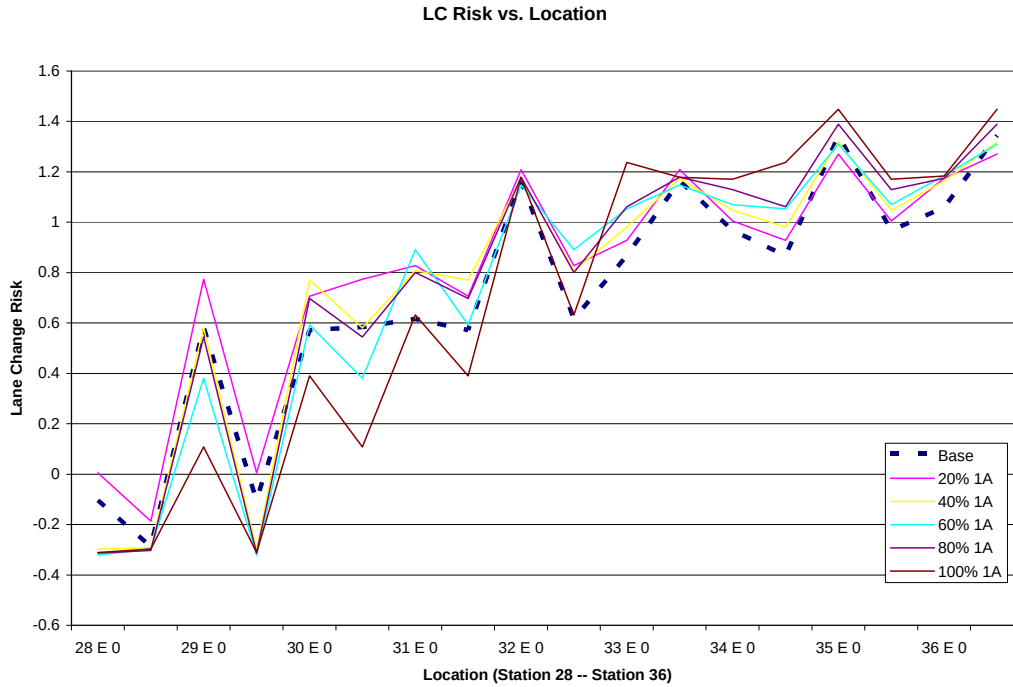


Figure 7-25: Average Lane-Change Crash Risk vs. Location for Cases 31 to 35

Table 7-21: Summary of Average Lane-Change Crash Risk Change for Cases 31 to 35

	Test Case ID					
	Base	Case 31	Case 32	Case 33	Case 34	Case 35
	---	20% 1A 0% 1B	40% 1A 0% 1B	60% 1A 0% 1B	80% 1A 0% 1B	100% 1A 0% 1B
Average Crash Risk (Stations 28 to 32)	0.4212	0.5648	0.4602	0.3936	0.4333	0.2520
Crash Risk Benefit	---	-0.1437	-0.0390	0.0276	-0.0122	0.1692
T-Statistic (Benefit Significance)	---	1.3870	0.4460	0.2980	0.1240	1.6610
Average Crash Risk (Stations 33 to 36)	1.0724	1.0986	1.1287	1.1496	1.1891	1.2592
Crash Risk Benefit	---	-0.0262	-0.0563	-0.0772	-0.1166	-0.1868
T-Statistic (Benefit Significance)	---	2.0350	1.9110	5.8960	7.3830	12.5740
LCRCI	---	-1.6460	-0.8402	-0.3415	-1.0547	0.1982

Table 7-22: Summary of Average Lane-Change Crash Risk Change for Cases 36 to 40

	Test Case ID					
	Base	Case 36	Case 37	Case 38	Case 39	Case 40
	---	20% 1A 50% 1B	40% 1A 50% 1B	60% 1A 50% 1B	80% 1A 50% 1B	100% 1A 50% 1B
Average Crash Risk (Stations 28 to 32)	0.5822	0.4651	0.5061	0.4307	0.3400	0.2530
Crash Risk Benefit	---	0.1170	0.0761	0.1514	0.2422	0.3292
T-Statistic (Benefit Significance)	---	1.9296	1.4832	2.5668	4.1311	6.1533
Average Crash Risk (Stations 33 to 36)	1.0900	1.1096	1.1256	1.1627	1.1716	1.2311
Crash Risk Benefit	---	-0.0196	-0.0356	-0.0726	-0.0816	-0.1410
T-Statistic (Benefit Significance)	---	1.8130	2.5044	6.1690	6.6057	10.9459
LCRCI	---	1.0137	0.4761	0.9333	1.7692	2.1639

Table 7-23: Summary of Average Lane-Change Crash Risk Change for Cases 41 to 45

	Test Case ID					
	Base	Case 41	Case 42	Case 43	Case 44	Case 45
	---	20% 1A 100% 1B	40% 1A 100% 1B	60% 1A 100% 1B	80% 1A 100% 1B	100% 1A 100% 1B
Average Crash Risk (Stations 28 to 42)	0.9567	0.9210	0.9095	0.8424	0.7761	0.6796
Crash Risk Benefit	---	0.0357	0.0471	0.1143	0.1805	0.2771
T-Statistic (Benefit Significance)	---	1.5987	2.1049	4.7948	9.1894	17.0662
LCRCI	---	0.9293	1.2258	2.9725	4.6942	7.2035

Table 7-24: Linear Regression Analysis for LCRCI in Test Cases 31 to 45 (90 Percent Loading)

Parameter	Estimate	Standard Error	t Value	Pr > t
Intercept	-3.0662	0.7174	-4.27	0.0011
% 1A	0.0385	0.0096	4.01	0.0017
% 1B	0.0414	0.0066	6.24	<.0001

A linear regression analysis of the change in LCRCI versus the various levels of route diversion is also presented in Table 7-24. As shown in this table, the linear regression is a good fit of the data due to the low p-values. This shows that there is a definite pattern in the LCRCI value compared to the amount of route diversion. In general, the LCRCI value increases as the number of vehicles are diverted and also increases as more vehicles re-enter the freeway further away from the initial diversion location. This conforms to the general trend that was realized at other loading cases.

7.2.1.4.1 Travel Time Analysis

The travel time plot for cases 31 to 45 are presented below in Figure 7-26. Both the 0% 1B and 100% 1B curves have well-fitting regression equations. The 50% 1B curve does not have a well-fitting equation but the travel time does not appear to change much in this case compared to the amount of diversion. The maximum change in the travel time is an increase of 7.3%. Since this is rather high, full diversion at the 90 percent loading scenario will have unreasonable effects on the network operations. The next level down (80% 1A, 100% 1B) only has a 5.2% increase in the travel time. This is much closer to 5% but still high. Diverting a lower number of vehicles than this would increase the travel time within reasonable limits. However, it should be noted that the benefits of route diversion on the crash risk are much lower at lower diversion levels.

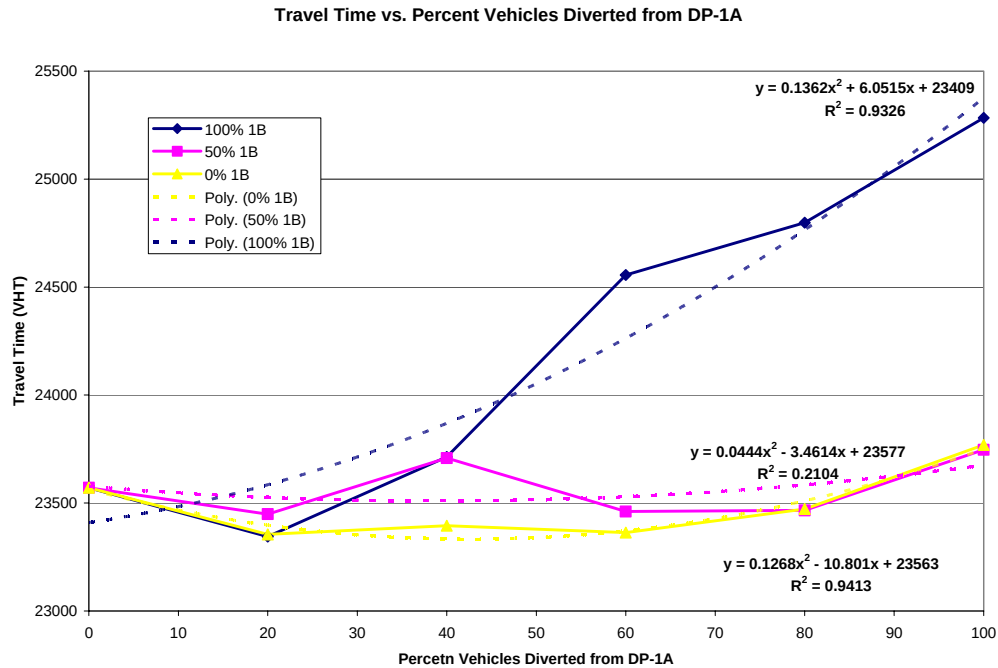


Figure 7-26: Travel Time Plot for Cases 31 to 45

7.2.1.4.2 Re-Entry Ramp Volume

In addition to the increase in the overall network travel time, there is another disadvantage that must be considered when diverting vehicles at such high loading scenarios. The ramp capacity at the on-ramp where vehicles are diverted to must be considered to make sure that the ramp is not oversaturated. At the 60 and 80 percent loading scenarios this is not much of an issue due to the very low levels of congestion. However, during the 90 percent loading scenario this can be quite problematic. For example, when 100 percent of the vehicles are sent to the further re-entry location the flow rate on that ramp increases by more than 900 vehicles / hour. Added to the 875 veh/hr that would normally use the ramp during the 90 percent loading scenario this means that the ramp volume would be increased to almost 1900 veh/hr. At the 100

percent loading scenario this ramp volume would be even higher at just under 2000 veh/hr. With such high volumes congestion builds up on the on-ramps and spills onto the surface street. Additionally the level of service of the on-ramp will be decreased severely. For the further re-entry location the surface street that is affected is S.R. 50 (Colonial Drive) which is a major arterial roadway throughout Orlando. Even though the overall travel time increase is acceptable, blocking such a major roadway would cause problems on portion of Colonial Drive that are not built on this network. Therefore, route diversion does not look to be a viable option at the 90 percent loading scenario.

7.2.1.5 100 Percent Loading Scenario

The 100 percent loading scenario is analyzed by test cases 46 to 60 as stated in the experimental design. The method of analysis mirrors the previous analysis of similar test runs performed at varying loading levels. As previously seen during the analysis of route diversion at the 90 percent loading scenario, the area of effect of route diversion on the crash risk along the freeway increases as the traffic volume increases. The 100 percent loading scenario is no different as the area of effect is quite large. As can be seen in Figure 7-27, diverting vehicles during such high traffic volumes changes the crash risk as far away from the diversion location (Station 33) as 7 miles upstream (Station 19). The effect can also extend downstream for 7 miles (Station 46) as evidenced in Figure 7-27.

Once again the most likely cause for this large area of effect is the heavy congestion that dominates the loading scenario. During this scenario there are several large queues that envelope the downtown area. When route diversion is applied, the queues still remain but they are either reduced in length or simply move downstream. Therefore, the tail end of the congested area would move downstream as well causing a

change in the traffic conditions, and therefore the rear-end crash risk, at only the upstream end of the queuing area. This effect can be seen in Figure 7-27 by noting the large decreases in crash risk from stations 18 to 22 at all diversion levels. When vehicles are diverted further downstream on the freeway – they are effectively kept off of the freeway for a longer period of time – the queue tends to move further downstream and causes an even greater safety benefit in the upstream area. This is evidenced by comparing the 0% 1B (short diversion) and the 100% 1B (long diversion) cases. Note this is also seen in Figures 7-14 and 7-22 for the 80 and 90 percent loading scenarios, respectively. However, in the latter two cases the effect is not as apparent as the 100 percent loading scenario since in this loading scenario the traffic volumes are much higher. Note that downstream of the crash risk reduction areas (stations 18 to 22 for 0% and 50% 1B and stations 18 to 27 for 100% 1B) there is an area of increased rear-end crash risk. This crash risk increase is caused by the speed differential that is created between the free flow areas upstream of the queue and the congested conditions that occur at the tail end of the queue. Upstream of this crash risk increase the traffic flow operates in regime 2 (free flow) conditions. At this location and downstream into the downtown area, the traffic conditions are congested (regime 1). The speed differential of faster moving vehicles approaching the slower moving vehicles in the queue increases the crash risk at the interface between the free flow and queuing conditions.

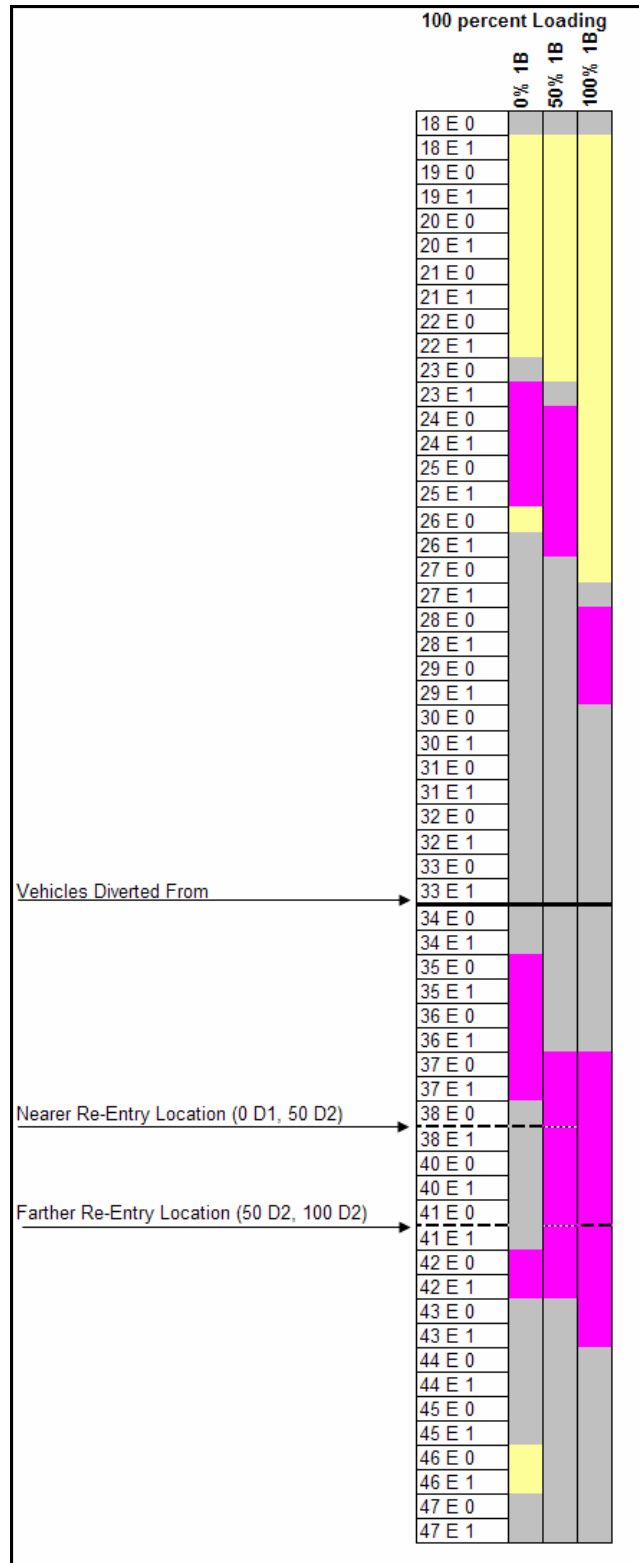


Figure 7-27: Locations Affected by Route Diversion at 100 Percent Loading Scenario

Tables 7-25, 7-26, and 7-27 summarize the change in the crash risk for the test cases 46 to 60. Please note that for the 0% 1B cases there are many areas where the risk changes as opposed to the 3 distinct areas for the other % 1B scenarios. Therefore, the overall average change in the risk over all of these stations is calculated.

Table 7-25: Summary of Average Rear-End Crash Risk Change for Cases 46 to 50

	Test Case ID					
	Base	Case 46	Case 47	Case 48	Case 49	Case 50
	---	20% 1A 0% 1B	40% 1A 0% 1B	60% 1A 0% 1B	80% 1A 0% 1B	100% 1A 0% 1B
Average Crash Risk (All Stations with Change)	0.466	0.479	0.474	0.482	0.487	0.473
Crash Risk Benefit	---	-0.013	-0.008	-0.015	-0.021	-0.007
T-Statistic (Benefit Significance)	---	1.481	0.965	2.084	2.896	0.563
ORCI	---	-0.318	-0.206	-0.385	-0.523	-0.179

Table 7-26: Summary of Average Rear-End Crash Risk Change for Cases 51 to 55

	Test Case ID					
	Base	Case 51	Case 52	Case 53	Case 54	Case 55
	---	20% 1A 50% 1B	40% 1A 50% 1B	60% 1A 50% 1B	80% 1A 50% 1B	100% 1A 50% 1B
Average Crash Risk (Stations 19 to 22)	-0.278	-0.331	-0.326	-0.377	-0.452	-0.482
Crash Risk Benefit	---	0.054	0.048	0.099	0.174	0.204
T-Statistic (Benefit Significance)	---	3.051	2.241	5.465	14.261	23.156
Average Crash Risk (Stations 24 to 26)	-0.131	-0.111	-0.105	-0.072	-0.009	0.020
Crash Risk Benefit	---	-0.020	-0.026	-0.059	-0.121	-0.151
T-Statistic (Benefit Significance)	---	1.544	2.016	4.372	10.882	6.405
Average Crash Risk (Stations 37 to 42)	1.102	1.152	1.167	1.177	1.221	1.242
Crash Risk Benefit	---	-0.051	-0.065	-0.075	-0.120	-0.140
T-Statistic (Benefit Significance)	---	3.466	3.094	4.115	7.310	8.276
ORCI	---	-0.198	-0.421	-0.312	-0.531	-0.676

Table 7-27: Summary of Average Rear-End Crash Risk Change for Cases 56 to 60

	Test Case ID					
	Base	Case 56	Case 57	Case 58	Case 59	Case 60
	---	20% 1A 0% 1B	40% 1A 0% 1B	60% 1A 0% 1B	80% 1A 0% 1B	100% 1A 0% 1B
Average Crash Risk (Stations 19 to 27)	-0.168	-0.167	-0.165	-0.204	-0.278	-0.367
Crash Risk Benefit	---	-0.001	-0.003	0.037	0.110	0.200
T-Statistic (Benefit Significance)	---	0.116	0.339	2.999	7.240	14.235
Average Crash Risk (Stations 28 to 29)	-0.225	-0.206	-0.219	-0.202	-0.151	-0.112
Crash Risk Benefit	---	-0.019	-0.006	-0.023	-0.074	-0.113
T-Statistic (Benefit Significance)	---	1.860	0.519	2.220	5.517	9.601
Average Crash Risk (Stations 37 to 43)	1.137	1.176	1.197	1.229	1.234	1.235
Crash Risk Benefit	---	-0.039	-0.060	-0.092	-0.097	-0.098
T-Statistic (Benefit Significance)	---	2.226	3.936	5.634	5.753	6.379
ORCI	---	-0.558	-0.795	-0.572	0.413	1.763

The first interesting trend that is noticed upon examining Tables 7-25, 7-26, and 7-27 is the fact that the ORCI values are almost all negative. For the previous loading scenarios there were very few negative values which showed that route diversion almost always provided a net safety benefit. However, for the 100 percent loading scenario it appears that route diversion primarily serves to decrease the overall safety of the corridor. Additionally, it does not appear that larger values of % 1A provide a larger value of ORCI as was seen in the previous loading scenarios. Lastly, there is no trend for the values of % 1B and the ORCI. In some cases larger % 1B values increases the crash risk (40% 1A), in other cases the crash risk is decreased (100% 1A). The linear regression

equation (Table 7-28) shows that both the % 1A and % 1B variables are statistically insignificant.

Table 7-28: Linear Regression Analysis for L in Test Cases 46 to 60 (100 Percent Loading)

Parameter	Estimate	Standard Error	t Value	Pr > t
Intercept	-0.89407	0.408276	-2.19	0.049
% 1A	0.007912	0.005456	1.45	0.1727
% 1B	0.003727	0.00378	0.99	0.3436

While the rear-end crash risk is increased due to route diversion at the 100 percent loading level, the lane-change crash risk tends to decrease. Tables 7-29, 7-30, and 7-31 summarize the change in the lane-change risk for cases 46 to 50, 51 to 55, and 56 to 60, respectively. Due to the large number of very small individual areas that are affected by route diversion, the average crash risk is calculated over the affected areas together rather than for specific areas that are affected. The areas that exhibit a change in the lane-change risk due to route diversion are outlined in Figure 7-28.

Table 7-29: Summary of Average Lane-Change Crash Risk Change for Cases 46 to 50

	Test Case ID					
	Base	Case 46	Case 47	Case 48	Case 49	Case 50
	---	20% 1A 0% 1B	40% 1A 0% 1B	60% 1A 0% 1B	80% 1A 0% 1B	100% 1A 0% 1B
Average Crash Risk (All Stations with Change)	0.5332	0.5109	0.4769	0.4521	0.3580	0.2695
Crash Risk Benefit	---	0.0224	0.0564	0.0812	0.1753	0.2637
T-Statistic (Benefit Significance)	---	0.8283	2.4508	3.5977	7.4541	10.9271
LCRCI	---	0.4027	1.0147	1.4612	3.1549	4.7468

Table 7-30: Summary of Average Lane-Change Crash Risk Change for Cases 51 to 55

	Test Case ID					
	Base	Case 51	Case 52	Case 53	Case 54	Case 55
	---	20% 1A 50% 1B	40% 1A 50% 1B	60% 1A 50% 1B	80% 1A 50% 1B	100% 1A 50% 1B
Average Crash Risk (All Stations with Change)	0.5643	0.5062	0.4961	0.4522	0.3923	0.3490
Crash Risk Benefit	---	0.0581	0.0682	0.1120	0.1720	0.2153
T-Statistic (Benefit Significance)	---	3.1448	3.4135	6.9400	10.8968	12.9103
LCRCI	---	1.5103	1.7742	2.9133	4.4731	5.5983

Table 7-31: Summary of Average Lane-Change Crash Risk Change for Cases 56 to 60

	Test Case ID					
	Base	Case 56	Case 57	Case 58	Case 59	Case 60
	---	20% 1A 100% 1B	40% 1A 100% 1B	60% 1A 100% 1B	80% 1A 100% 1B	100% 1A 100% 1B
Average Crash Risk (All Stations with Change)	0.5744	0.5496	0.4969	0.4049	0.3519	0.2975
Crash Risk Benefit	---	0.0248	0.0775	0.1694	0.2225	0.2769
T-Statistic (Benefit Significance)	---	1.4680	5.7177	10.8796	15.2773	19.1712
LCRCI	---	0.7432	2.3237	5.0834	6.6753	8.3067

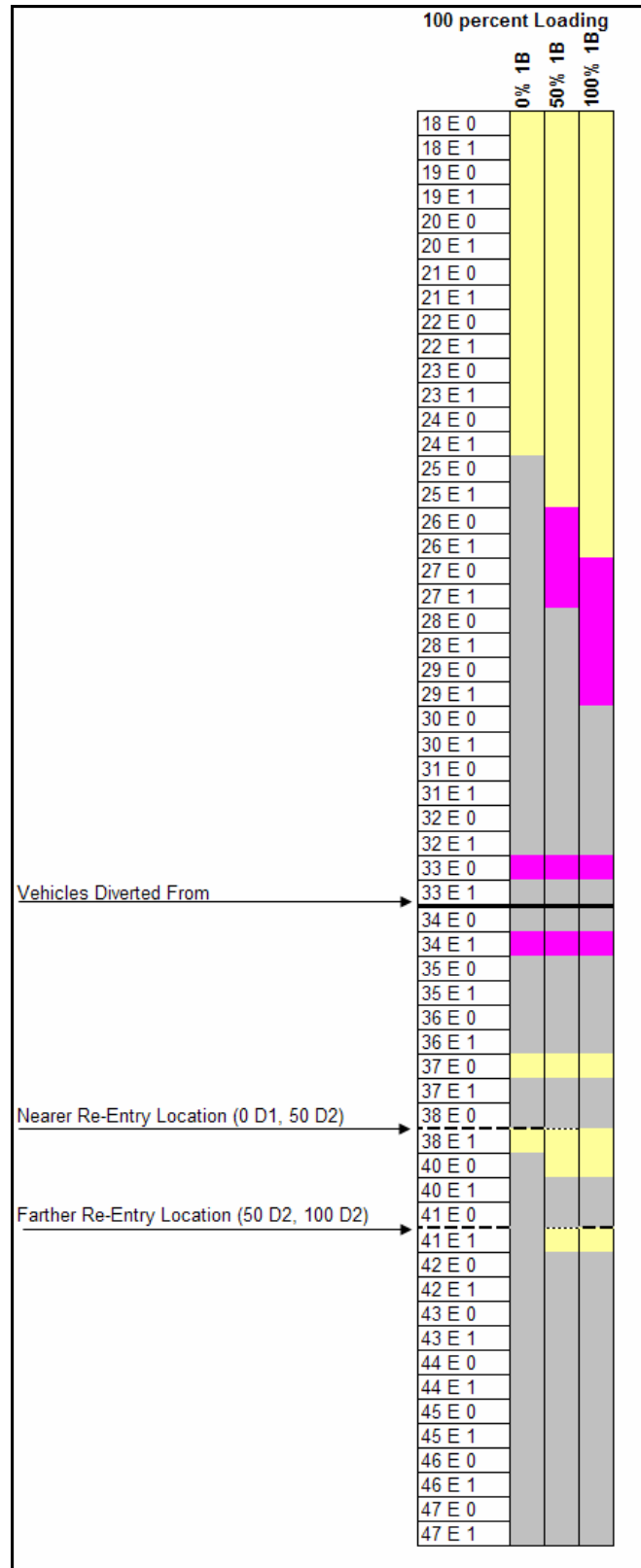


Figure 7-28: Locations Affected by Route Diversion at 100 Percent Loading Scenario (Lane-Change Risk)

It is interesting to note that the same trends appear in the lane-change crash risk values at the 100 percent loading scenario that were seen for the other measures for crash risk at lower loading levels. Also interesting is the fact that the LCRCI value shows an improvement in the lane-change crash risk due to route diversion even though the rear-end crash risk worsened. The general trend in the LCRCI versus the diversion is shown in the linear regression model given in Table 7-32.

Table 7-32: Linear Regression Analysis for LCRCI in Test Cases 46 to 60 (100 Percent Loading)

Parameter	Estimate	Standard Error	t Value	Pr > t
Intercept	-2.0080	0.5128	-3.92	0.0021
% 1A	0.0686	0.0069	10.02	<.0001
% 1B	0.024704	0.004748	5.20	0.0002

7.2.1.5.1 Travel Time Analysis

The travel time plot for cases 46 to 60 is presented below in Figure 7-29. The travel time during the 0% 1B scenarios do not change very much. Therefore, the low R^2 is not that unreasonable since it can be expected that the travel time would never change much when vehicles are diverted at that level. The other two curves show a much greater change in the travel time due to route diversion and are fit within reasonable limits by a quadratic curve. The maximum travel time seen by the route diversion is 7.2% for the 100 % 1A, 100% 1B case. To reach reasonable limits at the 100% 1B scenario the maximum amount of diversion at % 1A would only be about 40% which provides only a 4% increase in the travel time. For the 50% 1B curve, diverting any amount of vehicles

would provide a reasonable travel time for the network with a maximum increase of 4.9%.

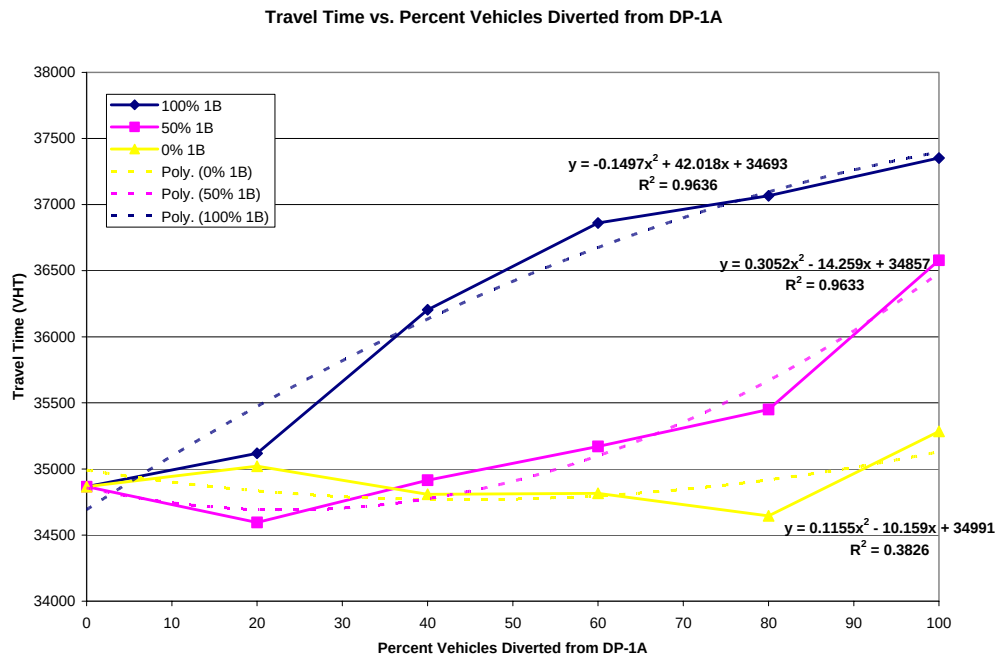


Figure 7-29: Travel Time Plot for Cases 46 to 60

7.2.1.5.2 Re-Entry Ramp Volume

Like the 90 percent loading scenario, diverting vehicles at the 100 percent loading scenario will increase the demand of the downstream on-ramps to levels greater than capacity. For example, when all vehicles are diverted to the furthest re-entry location, a scenario which provides the highest safety benefits – the ramp volume would be increased to 2000 veh/hr. This high ramp volume is unreasonable as it will reduce the level of service on the ramp to F and will also cause major delays and backups on Colonial Drive. Therefore, this cannot be accepted as a realistic strategy.

7.2.2 Analysis of Second Diversion Route

As previously mentioned in Section 6.15.2, the second diversion route is short compared to the first. There is only one diversion point for this route (DP 2) and this controls the percentage of vehicles that are diverted. The on-ramp that vehicles are diverted from typically has a low traffic flow rate as well (approximately 350 vehicles / hour) and the vehicles are diverted to the same ramp described as the furthest re-entry location in the previous section. This on-ramp has a typical volume of 970 veh/hr. The short diversion distance, in addition to the low diversion volumes, causes this route to not be as effective at reducing the crash risk along the freeway as the first route. The results of the analysis of the 2nd diversion route are given in the following sections.

7.2.2.1 60 Percent Loading Scenario

Cases 61 to 65 were created to examine the effects of diverting 20 to 100 percent of the vehicles from the South Street ramp using the 2nd diversion route. These test cases are labeled as 20% to 100%, respectively, which indicates the percentage of vehicles diverted for each case. As shown below in Figure 7-30, the rear-end crash risk does not change much during the application of the third diversion route at the 60 percent loading case. This lack of change is also presented in Table 7-33. Note the very small values of the ORCI given in this table.

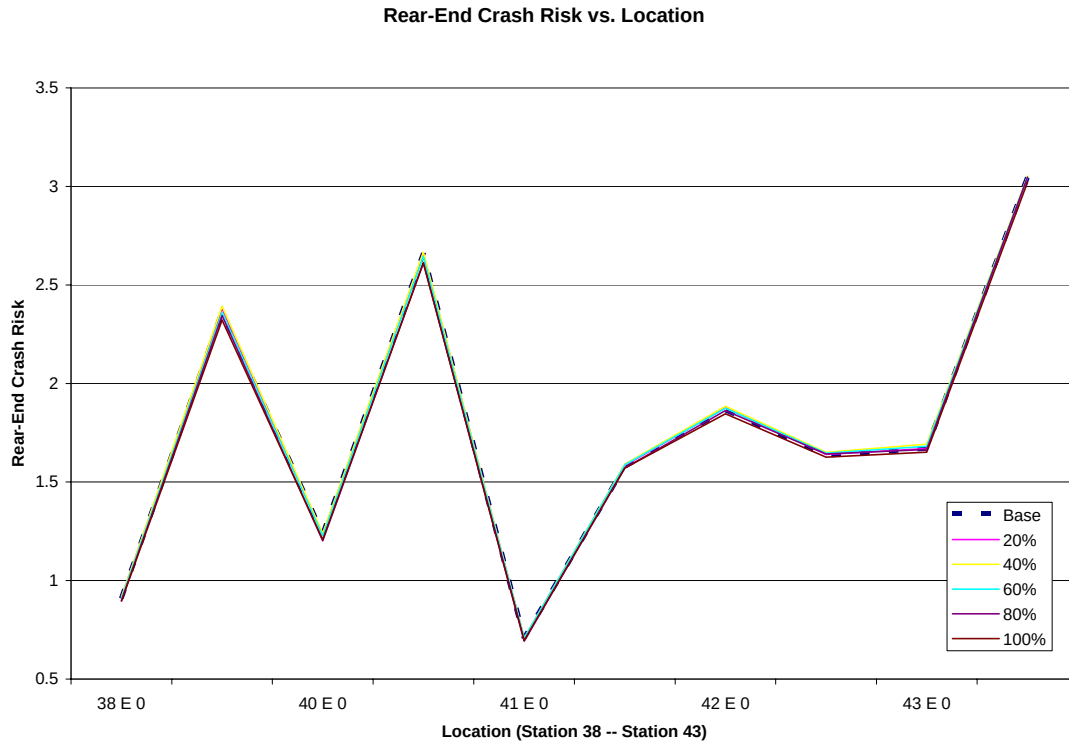


Figure 7-30: Average Rear-End Crash Risk vs. Location for Cases 61 to 65

Table 7-33: Summary of Average Rear-End Crash Risk Change for Cases 61 to 65

	Test Case ID					
	Base	Case 61	Case 62	Case 63	Case 64	Case 65
	---	20%	40%	60%	80%	100%
Average Crash Risk (Stations with Difference)	2.195	2.198	2.208	2.192	2.174	2.162
Crash Risk Benefit	---	-0.003	-0.013	0.003	0.021	0.033
T-Statistic (Benefit Significance)	---	0.148	0.902	0.190	1.356	2.422
ORCI	---	-0.014	-0.066	0.016	0.106	0.163

This lack of effect by the second diversion route compared to the first occurs for a few reasons. First, the ramp volumes affected are much different. The South Street ramp that vehicles are diverted from at DP-2 has a much lower volume (about 350 veh/hr) than the Orange Blossom Trail ramp that vehicles are diverted from using DP-1A (about 1020

veh/hr). Therefore, it is expected that the effects would be much lower since a much smaller volume of traffic flow is being moved. Secondly, in the second diversion route vehicles are only diverted a little less than 1.0 mile downstream. Because this distance is extremely short, the traffic stream may not develop a stable traffic pattern due to the reduced flow rate before the vehicles are reinserted onto the network. Lastly, since this is the 60 percent loading scenario the traffic flow volumes are not very high. Therefore, the effect of the route diversion would be minimized. Note that only at the highest level of loading (case 65 – 100%) was the change in the crash risk significant. Even still, this value is rather small at 0.033 for each of five stations that were affected.

The effect of cases 61 to 65 are almost the same on the lane-change risk as the rear-end risk. Basically, there is almost no change in the lane-change crash risk. This can be seen in the Figure 7-31. For this reason, the LCRCI values are not calculated since no station showed significant change in the crash risk in any of the test cases.

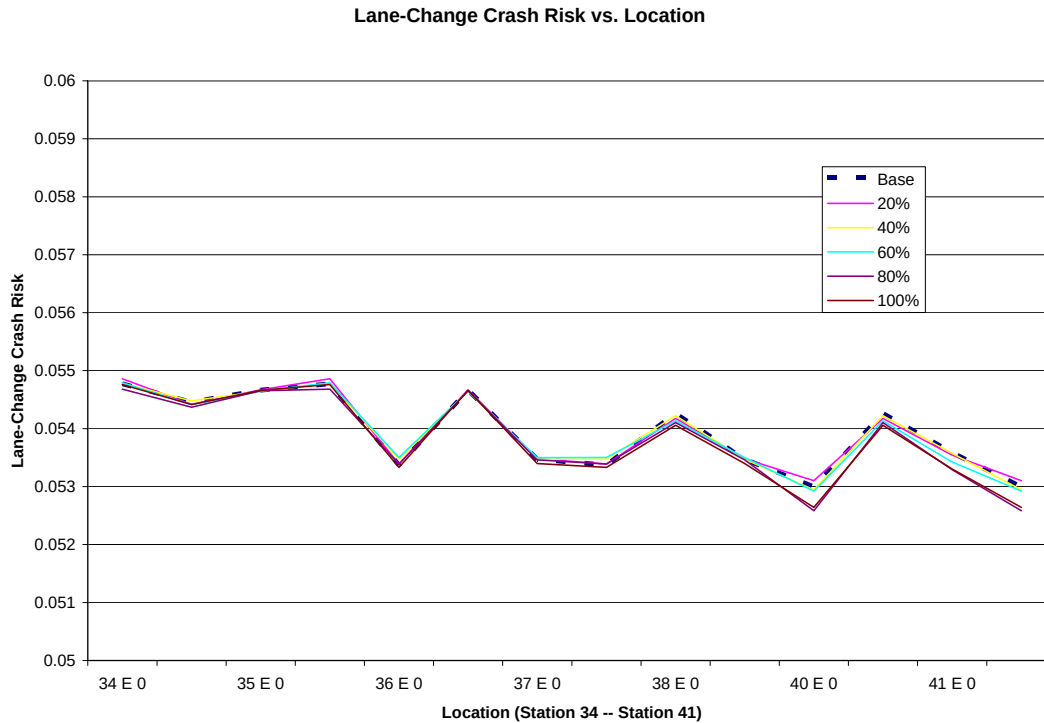


Figure 7-31: Average Lane-Change Crash Risk vs. Location for Cases 61 to 65

7.2.2.2 80 Percent Loading Scenario

The effects of using the shorter diversion route on the rear-end crash risk are not much more apparent during the 80 percent loading scenario than the 60 percent loading scenario. As can be seen in Figure 7-32, all of the curves representing the crash risk at different levels of diversion have about equal rear-end risk values. This is further shown in Table 7-34 where it can be seen that the change in the crash risk is insignificant at all cases and the values of ORCI are very low. Additionally, note that the values do not increase or decrease in sequence but rather they are random. This further shows the ineffectiveness of this shorter route diversion at this loading scenario.

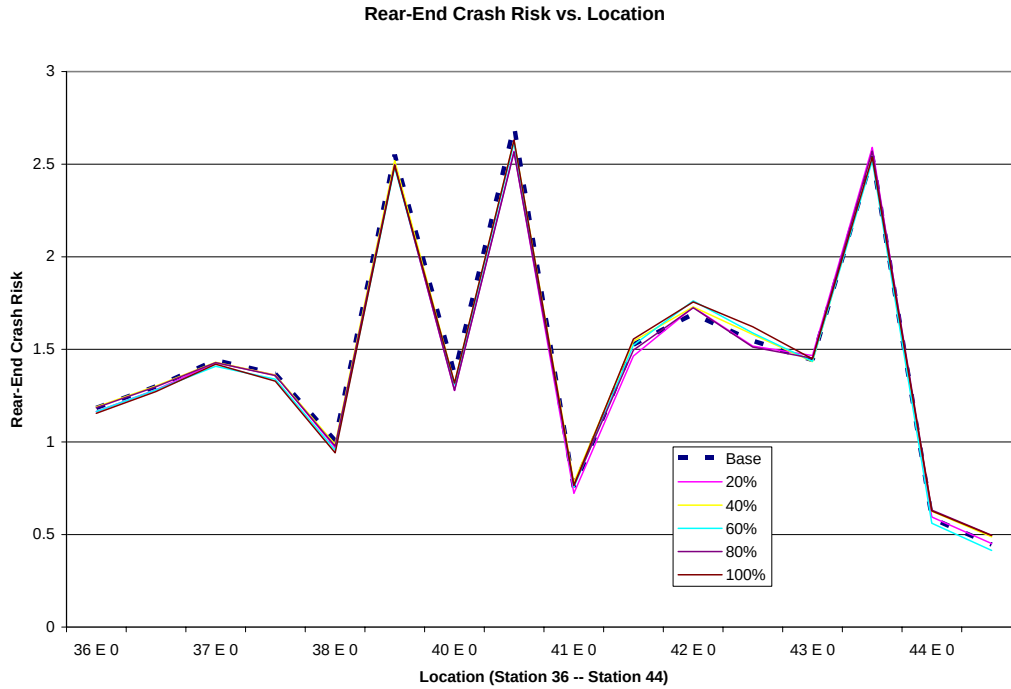


Figure 7-32: Average Rear-End Crash Risk vs. Location for Cases 66 to 70

Table 7-34: Summary of Average Rear-End Crash Risk Change for Cases 66 to 70

	Test Case ID					
	Base	Base 66	Case 67	Case 68	Case 69	Case 70
	---	20%	40%	60%	80%	100%
Average Crash Risk (Stations with Difference)	1.440	1.408	1.439	1.416	1.419	1.432
Crash Risk Benefit	---	0.032	0.002	0.024	0.021	0.008
T-Statistic (Benefit Significance)	---	1.730	0.095	1.310	1.090	0.691
ORCI	---	0.421	0.021	0.318	0.270	0.110

Once again the lane-change crash risk mirrors the rear-end crash risk for these test cases. There is almost no change between any of the five test cases (66 to 70) and the base case for the 80 percent loading case. The LCRCI values have been calculated for each case and are presented in 7-35. Please note that the changes in the risk are never statistically significant and all LCRCI values are extremely small.

Table 7-35: Summary of Average Lane-Change Crash Risk Change for Cases 66 to 70

	Test Case ID					
	Base	Case 61	Case 62	Case 63	Case 64	Case 65
	---	20%	40%	60%	80%	100%
Average Crash Risk (Stations with Difference)	1.1359	1.1487	1.1327	1.1047	1.1466	1.0778
Crash Risk Benefit	---	-0.0128	0.0033	0.0312	-0.0106	0.0581
T-Statistic (Benefit Significance)	---	0.1893	0.0602	0.5945	0.1594	1.0245
ORCI	---	-0.0767	0.0196	0.1875	-0.0639	0.3487

7.2.2.3 90 Percent Loading Scenario

At the 90 percent loading scenario the cases involving the 2nd diversion route (cases 71 to 75) show much more change in the crash risk than the previous loading scenarios. Multiple stations along the freeway show a change in the crash risk. Figures 7-33 and 7-34 (below) show a plot of the rear-end crash risk vs. location. It is important to note that unlike the cases for the 1st diversion route, which showed change in the crash risk that could be clearly defined into specific sections with lower and higher crash risk, the locations exhibiting change in the rear-end crash risk are not well defined. Most of the stations that show a change have a higher crash risk due to the route diversion. The stations that do show an improvement, Stations 28 and 37, are surrounded by stations which have a much higher crash risk when route diversion is implemented.

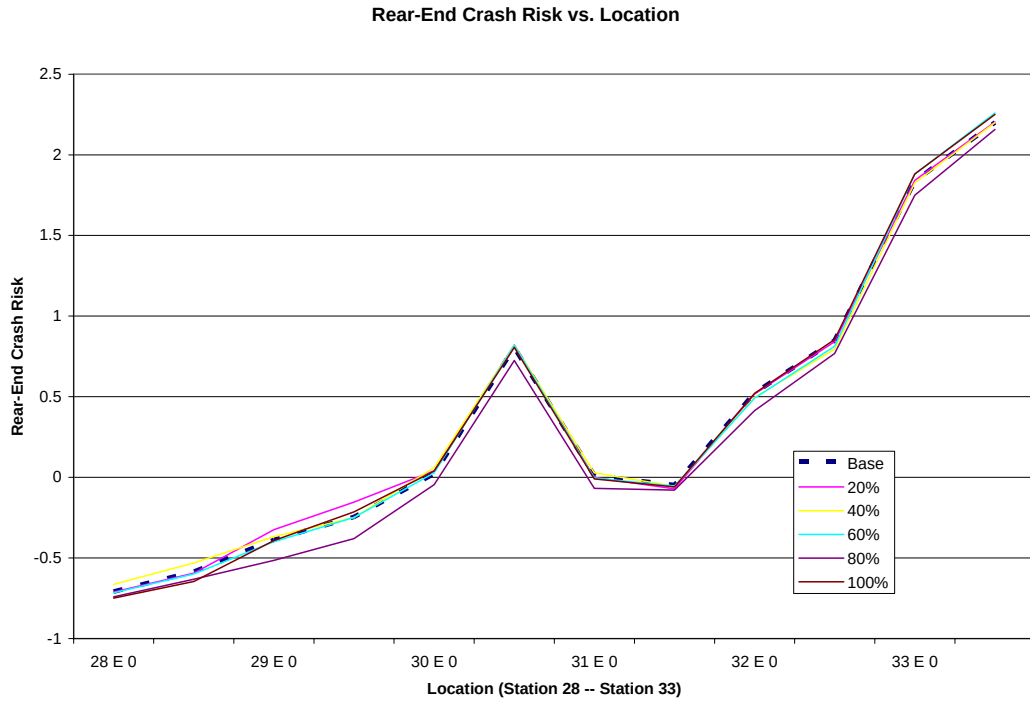


Figure 7-33: Average Rear-End Crash Risk vs. Location for Cases 71 to 75

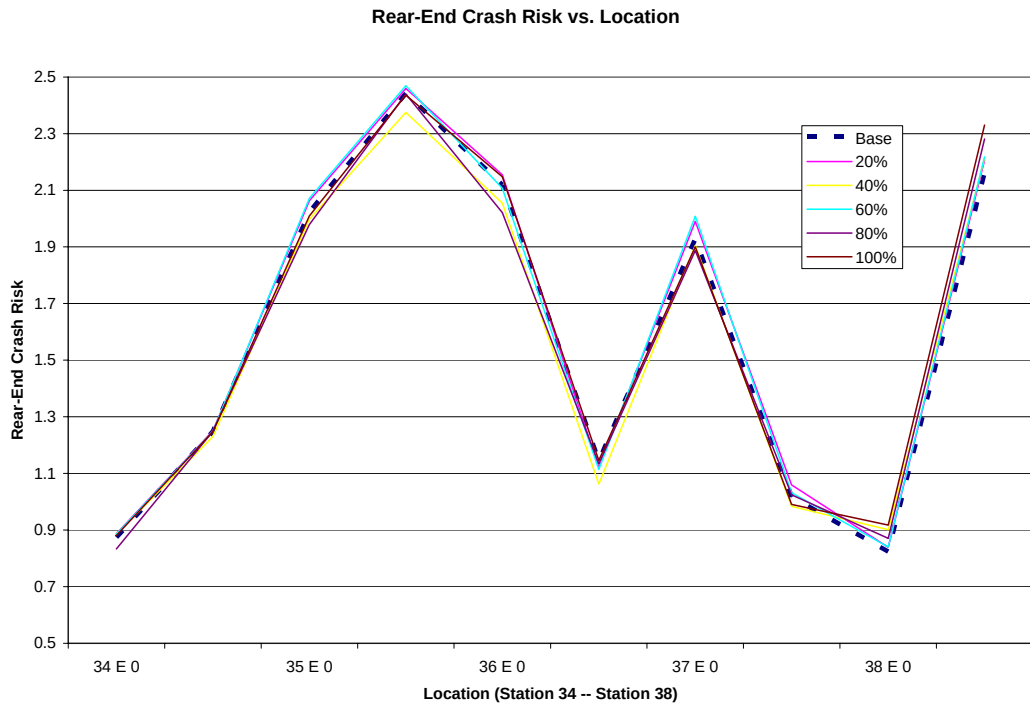


Figure 7-34: Average Rear-End Crash Risk vs. Location for Cases 71 to 75 – 2

The tabulated results of the rear-end crash risk for cases 71 to 75 are given in Table 7-36. Almost all values of the ORCI are negative implying that route diversion reduces the rear-end crash safety along the freeway in these cases. The general pattern is increasing values of ORCI until a peak at the 80% diversion case and then dipping back to a negative value.

Table 7-36: Summary of Average Rear-End Crash Risk Change for Cases 71 to 75

	Test Case ID					
	Base	Case 71	Case 72	Case 73	Case 74	Case 75
	---	20%	40%	60%	80%	100%
Average Crash Risk (Stations with Difference)	1.082	1.122	1.116	1.106	1.080	1.115
Crash Risk Benefit	---	-0.040	-0.034	-0.024	0.002	-0.033
T-Statistic (Benefit Significance)	---	2.084	0.797	0.833	0.070	1.427
ORCI	---	-0.556	-0.472	-0.340	0.028	-0.467

Although the lane-change crash risk shows a significant change during these scenarios, all values of the LCRCI are negative as well. This means that route diversion has a negative impact on the lane-change crash risk at all diversion levels. The results are given in Table 7-37 below. Additionally, no location showed a positive benefit in the lane-change risk during these test cases. The effects of route diversion 2 are to increase the lane-change risk significantly.

Table 7-37: Summary of Average Lane-Change Crash Risk Change for Cases 71 to 75

	Test Case ID					
	Base	Case 71	Case 72	Case 73	Case 74	Case 75
	---	20%	40%	60%	80%	100%
Average Crash Risk (Stations with Difference)	0.8391	0.9316	0.9315	0.9235	0.8654	0.9260
Crash Risk Benefit	---	-0.0925	-0.0924	-0.0844	-0.0263	-0.0869

T-Statistic (Benefit Significance)	---	1.9665	1.8845	1.7583	0.5316	1.8559
LCRCI	---	-2.2205	-2.2172	-2.0263	-0.6319	-2.0853

7.2.2.4 100 Percent Loading Scenario

At the 100 percent loading scenario the results are about the same. Implementation of the 2nd diversion route causes an increase in the overall rear-end crash risk throughout the network. This can be seen in Table 7-38. Additionally, much like the 90 percent loading case, the locations that experience a change in the rear-end crash risk are not well defined along the network. Because of this and the lack of a general pattern in the ORCI and the amount of vehicles the 2nd diversion route does not seem to be a reliable option.

Table 7-38: Summary of Average Rear-End Crash Risk Change for Cases 76 to 80

	Test Case ID					
	Base	Case 76	Case 77	Case 78	Case 79	Case 80
	---	20%	40%	60%	80%	100%
Average Crash Risk (Stations with Difference)	0.484	0.486	0.488	0.494	0.500	0.497
Crash Risk Benefit	---	-0.001	-0.003	-0.010	-0.016	-0.013
T-Statistic (Benefit Significance)	---	0.214	0.527	0.958	1.984	1.789
ORCI	---	-0.054	-0.153	-0.445	-0.714	-0.589

The lane-change crash also shows very little change due to the 2nd diversion route. The results are presented in Table 7-39 for inspection.

Table 7-39: Summary of Average Lane-Change Crash Risk Change for Cases 76 to 80

	Test Case ID					
	Base	Case 76	Case 77	Case 78	Case 79	Case 80

	---	20%	40%	60%	80%	100%
Average Crash Risk (Stations with Difference)	0.6310	0.6315	0.5999	0.6337	0.6222	0.5797
Crash Risk Benefit	---	-0.0005	0.0311	-0.0027	0.0087	0.0513
T-Statistic (Benefit Significance)	---	0.0230	1.2305	0.1118	0.3301	2.0912
ORCI	---	-0.0082	0.4976	-0.0435	0.1396	0.8203

7.2.3 Route diversion summary

It is important to note that the specific results found in this study are applicable only to the specific area of the network that is modeled and route diversion implemented. Therefore, the values of the risk change for a particular test case and the linear regression models that were created to compare the ORCI and LCRCI based on the amount of vehicles diverted are specific to Interstate-4 through downtown Orlando for the specific ramps used in this study. Because of this, the regression models created cannot be used to predict the change in the ORCI or LCRCI for other diversion locations. However, the general trends found from these should remain regardless of the location. For example, the trend that the risk decreases at the location where vehicles are diverted from and increases at the location where diverted vehicles are reinserted onto the freeway should hold for any situation. However, the amount of the decrease and increase in the crash risk will depend on the ramp volumes and traffic flow along the freeway location. The conclusions and trends that are found from this study are as follows:

- Route diversion serves to reduce the overall rear-end and lane-change crash risk on the freeway at lower loading levels (up to 90 percent loading) for the first diversion route.

- Route diversion serves to increase the overall rear-end and lane-change crash risk on the freeway at extremely high levels of congestion (100 percent loading) for the first diversion route.
- The length of the freeway that is affected by route diversion increases as the amount of traffic increases along the network.
- At lower levels of congestion, the primary area affected by route diversion is the area between the initial diversion point and the re-entry area.
- At higher levels of congestion, route diversion reduces the crash risk at the upstream end of queues that are formed during this loading case. Therefore, the main area affected by route diversion is located much upstream of the area where the route diversion actually occurs. This occurs because when vehicles are diverted the traffic demand is reduced at the middle of a very long queuing section. This reduced flow serves to shorten the length of the queuing section on the tail (upstream) end of the queue by providing more storage space in the middle of the queue that would have been filled by the diverted vehicles. Comparing the base case to the route diversion case, the only difference that is seen is upstream end of the queuing section – in the base case it remains congested while in the diversion case the congested is reduced. This area where the congestion is reduced shows lower values of both the rear-end and lane-change crash risk.
- At lower levels of congestion (60 and 80 percent loading), the reduction in rear-end crash risk increases with the increasing number of vehicles that are diverted and also increase the further away vehicles are diverted to. The reduction in the

lane-change crash risk only increases with the number of vehicles that were diverted although there is some evidence which shows that diverting vehicles further away helps to reduce the lane-change crash risk even further.

- At levels of moderate and high congestion (90 and 100 percent loading, respectively), the effects of route diversion are not predictable in nature. Diverting a higher percentage of vehicles may not necessarily reduce the crash risk more.
- At levels of moderate and high congestion (90 and 100 percent loading, respectively), high levels of route diversion causes congestion on the on-ramps that vehicles use to re-enter the freeway. This congestion spills onto the surface streets and blocks a major arterial roadway during the peak traffic periods.
- At lower levels of congestion (60 and 80 percent loading) the increase in travel time that is realized due to route diversion is minimal.
- The travel time increase due to route diversion increases as congestion increases.
- When congestion starts to build up on the network (80 – 100 percent loading), route diversion shows a crash migration effect at the area where the vehicles are diverted to on the network. Therefore, at higher levels of congestion route diversion decreases the crash risk in the upstream areas nearer the diversion location and increases the crash risk at the downstream areas where diverted vehicles re-enter the freeway.
- Diverting vehicles a very small distance along the freeway (less than 1 mile) from an on-ramp with a small traffic volume does not serve to change the crash risk significantly.

Therefore, route diversion should only be applied in off-peak situations (when the mainline traffic flow is un-congested) if the goal of diverting vehicles is to reduce the real-time crash risk along the freeway. Not only do the results show an overall improvement in the crash risk for all diversion levels but the increase in the travel time is relatively small during these low network loading scenarios. However, care must be taken during the 80 percent loading case to ensure that that small area of the freeway that experiences an increase in the crash risk does not already have a high crash risk value. If so, route diversion may serve to make an already high risk area even more risky and cause more crashes. Additionally, vehicles should be diverted from ramps with high inflow volumes and be diverted as far downstream as possible in order to maximize the safety benefits.

7.3 Analysis of Ramp Metering

As previously mentioned, the second half of the experimental design was performed to evaluate the effectiveness of ramp metering at reducing the two different crash risk measures along the freeway. The first step in this process was to determine the best ALINEA parameters which are used to compare the ALINEA metering strategy with the Zone metering strategy. Once completed, the different cases performed with the traffic-cycle realization of the Zone algorithm were examined to determine the best cycle length to use. Next the different one-car-per-cycle methods were compared and, finally, the most beneficial strategies from all categories were compared against each other to determine the “best” ramp metering strategy for reducing the crash risk along the network corridor.

The analysis of these test cases is performed in a similar manner as the route diversion test cases. However, there is one important difference that must be mentioned. In general, for route diversion, the area of the freeway that is affected by a particular diversion strategy is constant for varying percentages of vehicles that are diverted. In other words, the area of the freeway that is affected when 100 percent of the vehicles are diverted is about the same as when 40 percent of the vehicles are diverted – the 40 percent diversion case just shows a much smaller change in the crash risk. This is not true for the ramp metering cases, however. When different metering strategies are used, the locations that are affected by ramp metering change. Therefore, when analyzing the ramp metering cases, specific areas of crash risk increase and decrease are not specified. Instead, the network as a whole is examined and all locations that show a change in the crash risk are analyzed together.

7.3.1 Analysis of ALINEA Parameters

7.3.1.1 100 Percent Loading Scenario

The experimental design used to find the best ALINEA parameters is given in Table 6-7 in Section 6.16.1. The ramps metered as part of this subsection of the experimental design are the ramps that make up zone 1 (Figure 6-24). This zone encompasses the 3 ramps immediately upstream of the Interstate-4 / S. R. 408 Interchange. To analyze the changes in the rear-end and lane-change crash risks due to the different ALINEA parameter configurations, the ORCI and LCRCI are used, respectively. As noted previously, these values are calculated from all stations within the network that experience a change in the respective risk value due to the implementation

of the ramp metering strategy. The values of the ORCI and LCRCI are given below in Table 7-40 for portion of the experimental design performed at the 100 percent loading scenario. Note in Table 7-40 that each test case is denoted by its respective case number and the ALINEA parameters that the case represents. For example, Case 81 (C 30, O 0.17) represents the case with a 30 second cycle length and a critical occupancy of 0.17.

Table 7-40: Summary of ORCI and LCRCI for Cases 81 to 89

	Test Case ID								
	Case 81	Case 82	Case 83	Case 84	Case 85	Case 86	Case 87	Case 88	Case 89
	C 30 O 0.17	C 30 O 0.20	C 30 O 0.23	C 45 O 0.17	C 45 O 0.20	C 45 O 0.23	C 60 O 0.17	C 60 O 0.20	C 60 O 0.23
# of Stations with RE Risk Change	16	18	15	20	15	17	17	13	9
ORCI	0.0061	-0.4203	-0.2962	-0.2241	-0.1659	-0.5479	-0.3304	-0.3587	-0.3591
# of Stations with LC Risk Change	8	6	6	8	10	10	6	8	4
LCRCI	3.0360	2.6416	2.5904	3.3258	2.2548	2.0591	1.5371	1.1249	1.1040

It is important to note that the ORCI value is negative for all cases except the case with the smallest cycle length (30 seconds) and critical occupancy (0.17). Negative values of the ORCI and LCRCI mean that the rear-end and lane-change crash risk, respectively, are increased in the particular scenario; however, the magnitude of this increase is very small in every case. Therefore, the overall rear-end safety along the network is decreased slightly with the application of ALINEA ramp metering using most of the tested configurations. This slight increase in the crash risk occurs due to the particular ramps that are metered in this scenario – the ramps immediately upstream of the Interstate-4 / S. R. 408 Interchange. Metering these ramps alone does not significantly change the amount of congestion on the freeway since there is a large inflow

of vehicles at the Interstate-4 / S. R. 408 Interchange located immediately downstream of this area. In fact the occupancies increases (slightly) which causes a small increase in the rear-end and lane-change crash risk.

Looking at the values of ORCI and LCRCI in Table 7-40 it is also apparent that higher values of these two variables occur during shorter cycle lengths and lower values of the critical occupancy. In other words, crash risk is reduced more on the network with lower values of cycle length and critical occupancy. This is more clearly seen when the ORCI and LCRCI values are averaged across individual levels of a particular value (i.e. ORCI averaged for all cases with a 30 second cycle length compared to ORCI averaged for all cases with a 45 second cycle length). This is presented below in Table 7-41. As shown more clearly in Table 7-41, the values of ORCI and LCRCI increase with shorter cycle lengths and smaller critical occupancies. Therefore, shorter cycle lengths and smaller critical occupancy values provide the best safety results. The smaller critical occupancy values allow the metering algorithm to become more restrictive at lower levels of congestion while the smaller cycle lengths allow smaller platoons of vehicles onto the network which, in this particular traffic flow condition, results in smaller crash risk values.

Table 7-41: ORCI and LCRCI across different levels of ALINEA Parameters (100 Percent Loading)

	ORCI	LCRCI
O = 0.17	-0.1828	2.6330
O = 0.20	-0.3149	2.0071
O = 0.23	-0.4010	1.9178
C = 30	-0.2368	2.7560
C = 45	-0.3126	2.5466
C = 60	-0.3494	1.2553

A linear regression model was created to help explain the trends between the two ALINEA parameters and the ORCI and LCRCI. These two models are displayed in Tables 7-42 and 7-43, respectively. Since the coefficients of the cycle length and critical occupancy parameters are negative for both of the models, this implies that lower values of these two variables provide better safety benefits. Even though the parameter estimates in the ORCI model are not statistically significant (which probably signifies the lack of a linear relationship rather than the lack of a relationship altogether), the sign of the parameter estimate shows that if it were significant lower cycle lengths would provide greater safety benefits.

Table 7-42: Linear Regression Analysis for ORCI in Test Cases 81 to 89 (100 Percent Loading)

Parameter	Estimate	Standard Error	t Value	Pr > t
Intercept	0.5968	0.4080	1.46	0.1939
Cycle	-0.0038	0.0037	-1.01	0.3493
Crit_Occ	-3.6373	1.8491	-1.97	0.0967

Table 7-43: Linear Regression Analysis for LCRCI in Test Cases 81 to 89 (100 Percent Loading)

Parameter	Estimate	Standard Error	t Value	Pr > t
Intercept	6.8209	1.2175	5.60	0.0014
Cycle	-0.0500	0.0110	-4.53	0.0040
Crit_Occ	-11.9195	5.517058	-2.16	0.0740

7.3.1.2 90 Percent Loading Scenario

Like the ALINEA runs for the 100 percent loading cases, the values of the ORCI and LCRCI were calculated for cases 90 to 98 which were run at 90 percent loading. These values are represented below in Table 7-44.

Table 7-44: Summary of ORCI and LCRCI for Cases 90 to 98

	Test Case ID								
	Case 90	Case 91	Case 92	Case 93	Case 94	Case 95	Case 96	Case 97	Case 98
	C 30 O 0.17	C 30 O 0.20	C 30 O 0.23	C 45 O 0.17	C 45 O 0.20	C 45 O 0.23	C 60 O 0.17	C 60 O 0.20	C 60 O 0.23
# of Stations with RE Risk Change	16	12	11	16	17	8	14	5	9
ORCI	2.0057	0.8274	0.4980	1.3397	0.8875	0.0695	1.1504	0.2043	0.2055
# of Stations with LC Risk Change	4	2	4	6	0	2	4	6	6
LCRCI	1.3549	0.4174	0.0369	0.6966	0.0000	-0.5846	0.9367	-0.5275	-0.4792

Once again the values of the ORCI and LCRCI found in Table 7-45 have the general trend that they decrease with increasing values of the cycle length and critical occupancy. For the 90 percent loading cases (unlike the 100 percent loading cases), the ORCI and LCRCI values are mostly positive which implies that, in general, the ALINEA strategies serve to reduce both the rear-end and lane-change crash risk. The averages of the two crash risk summary variables across individual levels of a particular variable (presented in Table 7-45) confirm this trend.

Table 7-45: ORCI and LCRCI across different levels of ALINEA Parameters (90 Percent Loading)

	ORCI	LCRCI
O = 0.17	1.4986	0.9961
O = 0.20	0.6397	-0.0367
O = 0.23	0.2577	-0.3423
C = 30	1.1104	0.6031
C = 45	0.7655	0.0373
C = 60	0.5201	-0.0233

The linear regression models for the ORCI and LCRCI shown in Tables 7-46 and 7-47, respectively, confirm the same trends found for the scenarios performed at the 100 percent loading scenario. Please note that all of the parameter estimates are statistically significant in this case which shows that these trends are linear for the 90 percent loading scenarios. The coefficients of the cycle length and critical occupancy parameters are negative (like the 100 percent loading case) for both models. This shows that the overall risk benefit is maximized when lower values of the cycle length and critical occupancy are used for the ALINEA parameters.

Table 7-46: Linear Regression Analysis for ORCI in Test Cases 90 to 98 (90 Percent Loading)

Parameter	Estimate	Standard Error	t Value	Pr > t
Intercept	5.8205	0.7525	7.73	0.0002
Cycle	-0.0197	0.0068	-2.88	0.0279
Crit_Occ	-20.6821	3.4100	-6.07	0.0009

Table 7-47: Linear Regression Analysis for LCRCI in Test Cases 90 to 98 (90 Percent Loading)

Parameter	Estimate	Standard Error	t Value	Pr > t
Intercept	5.6066	0.9236	6.07	0.0009
Cycle	-0.0209	0.0084	-2.49	0.0469
Crit_Occ	-22.3063	4.185329	-5.33	0.0018

As a result of this portion of the experimental design, the following results are found:

- A lower value of the critical occupancy in the ALINEA algorithm provides the highest safety benefits

- A lower value of the cycle length provides the highest safety benefits when the traffic-cycle realization of ALINEA is applied

Dhindsa (2005) (Also see Chapter 5) performed a similar study to determine the best values of the cycle length and critical occupancy for the ALINEA algorithm when used to reduce the real-time crash risk on a freeway. In Chapter 5, much like the analysis performed here in this chapter, we found that a smaller cycle length provides greater safety results when the ALINEA algorithm is implemented. Using a smaller cycle length with this restrictive metering algorithm allows smaller platoons of vehicles to enter the freeway mainline at an increased frequency. Typically, this would serve to increase the risk of a rear-end crash as vehicles merge onto the freeway mainline more frequently. However, when the traffic pattern on the mainline has small gaps between successive vehicles that can accommodate only a few vehicles merging at a time, this situation provides greater safety benefits than when larger platoons of vehicles try to merge into the same small gaps in the mainline traffic. At the particular levels of traffic flows experienced on this network, these smaller platoons cause less turbulence in the traffic flow and effectively reduce the crash risk more than if a larger cycle length is used with the same metering algorithm.

However, in Chapter 5 we found that using a higher value of critical occupancy in the ALINEA algorithm would improve the safety conditions on the freeway; this is contradictory to the results found in this study. To understand which of these values is most appropriate then, it is important to examine the ALINEA algorithm in detail. From Equation 1-3, using lower value of the critical occupancy would cause the ALINEA

algorithm to restrict the metering rate at lower levels of congestion than if a higher value of the critical occupancy is used. This would essentially allow fewer vehicles onto the network per metering cycle during periods of light congestion which will induce less turbulence on the mainline traffic flow. However, if a higher critical occupancy value is used the meter would not be as restrictive and allow more vehicles onto the network for a given time period. This does not make sense from a safety perspective as this would increase congestion and speed variation. For this reason, the results of this study are used to describe the relationship between the critical occupancy and crash risk.

Therefore, the final outcome of this portion of the experimental design is that the optimal values for the ALINEA algorithm with the traffic-cycle realization are a cycle length of 30 seconds and a critical occupancy of 0.17. These values will be used to compare ALINEA traffic-cycle method with the other ramp metering methods being considered.

7.3.2 Analysis of Zone Strategy with Traffic-Cycle Realization

The experimental design given in Section 6.16.3 describes the other ramp metering test cases that are examined in this study. In order to determine the trends between the rear-end and lane-change crash risk and specific variables in the experimental design, these cases are split into different groups for analysis. The first group that will be looked at are the cases that use the Zone ramp metering algorithm with the traffic cycle realization. These cases are analyzed together in order to determine which cycle length and which metered zone most successfully reduces the two real-time crash risk measures and are reproduced below in Table 7-48. Zone 1 encompasses the area just upstream of the I-4 / S. R. 408 interchange, Zone 2 meters the four ramps just

downstream of this interchange, and Zone 3 meters an that is the union of Zones 1 and 2.

A schematic drawing of the Zones is shown in Figure 6-24.

Table 7-48: Test Cases Representing Zone Metering Algorithm and Traffic-Cycle Realization

Case Number	Percent Loading	Metered Zone	Algorithm	Cycle Length
99	100	1	Zone	30
100	100	1	Zone	45
101	100	1	Zone	60
105	100	2	Zone	30
106	100	2	Zone	45
107	100	2	Zone	60
111	100	3	Zone	30
112	100	3	Zone	45
113	100	3	Zone	60
117	90	1	Zone	30
118	90	1	Zone	45
119	90	1	Zone	60
123	90	2	Zone	30
124	90	2	Zone	45
125	90	2	Zone	60
129	90	3	Zone	30
130	90	3	Zone	45
131	90	3	Zone	60

The values of the ORCI and LCRCI have been calculated for the cases performed at the 100 percent loading scenario and are presented below in Table 7-49. The cases are denoted in Table 7-49 by the zone that is being metered and the cycle length that is used as well as the corresponding case number from Table 7-48. For example, Case 111 represents the case where zone 3 is metered using a 30 second cycle length. As shown in Table 7-49, the values of the ORCI and LCRI tend to increase for longer cycle lengths which show improved safety conditions as the cycle lengths increases. Metering zone 1 does not appear to yield significant safety benefits as evidenced by the low values of ORCI and LCRCI. In fact, the majority of the cases involving zone 1 have a negative

overall safety impact. Metering zone 2 provides better safety improvements but zone 3 provides the most substantial reduction in both the rear-end and lane-change crash risk. The average value of ORCI for zone 3 is about 6 times as high as the average value of the ORCI for zone 2.

Table 7-49: Summary of ORCI and LCRCI for Zone TC Cases at 100 Percent Loading

	Test Case ID								
	Case 99	Case 100	Case 101	Case 105	Case 106	Case 107	Case 111	Case 112	Case 113
	Z 1 C 30	Z 1 C 45	Z 1 C 60	Z 2 C 30	Z 2 C 45	Z 2 C 60	Z 3 C 30	Z 3 C 45	Z 3 C 60
# of Stations with RE Risk Change	1	2	1	10	21	27	42	45	46
ORCI	-0.0605	-0.0081	0.0657	0.7435	1.5832	1.7290	6.4865	8.9248	8.4413
# of Stations with LC Risk Change	0	6	14	14	14	12	32	36	38
LCRCI	0.0000	-0.6198	-0.9364	1.5090	1.7608	1.8896	7.3917	9.5295	8.7241

The test cases from Table 7-48 performed at the 90 percent loading scenario show similar results. The values of the ORCI and LCRCI are presented below in Table 7-50. The trends found in Table 7-49 are the same found above in Table 7-50. In general, the values of ORCI and LCRCI increase with the cycle length. Additionally, metering zone 1 tends to yield a negative safety impact while metering zones 2 and 3 have a positive safety impact.

Table 7-50: Summary of ORCI and LCRCI for Zone TC Cases at 90 Percent Loading

	Test Case ID								
	Case 117	Case 118	Case 119	Case 123	Case 124	Case 125	Case 129	Case 130	Case 131
	Z 1 C 30	Z 1 C 45	Z 1 C 60	Z 2 C 30	Z 2 C 45	Z 2 C 60	Z 3 C 30	Z 3 C 45	Z 3 C 60
# of Stations with RE Risk Change	8	3	1	17	20	28	25	30	34
ORCI	-0.2957	-0.3318	-0.0881	1.7783	2.3628	3.5568	5.2674	6.7534	7.0750
# of Stations with LC Risk Change	8	8	6	4	4	8	12	16	12
LCRCI	-1.5900	-1.7860	-1.5152	0.9005	0.9776	1.4372	3.7512	4.7700	3.8661

Note that for the runs performed at both the 90 percent and 100 percent loading scenarios the values of the ORCI and LCRCI are correlated with the number of stations that show a rear-end or lane-change crash risk, respectively. This means that greater safety benefits are found when a larger section of the freeway is affected by the ramp metering strategy. Therefore, it makes sense that metering zone 3 would provide the best results since this zone is the largest of the three zones considered in this study; this zone had 7 metered ramps and spanned a length of 4.5 miles. Although zone 1 is almost 2.5 miles long, the reason that large safety benefits are not realized when zone 1 is metered is that there is a bottleneck that occurs in the downstream end of this zone at the Interstate-4 / S. R. 408 Interchange.

The positive values of ORCI and LCRCI when zones 2 and 3 are metered show that net safety benefits are realized for these scenarios. However, looking at the individual loop detector stations shows that while most stations exhibit a decrease in the respective crash risk, at some locations the crash risk is increased significantly. This implies that ramp metering, like route diversion, has the potential to cause crash migration – the reduction of the crash risk at one area combined with the increase in the

crash risk at another. Because of this phenomenon the crash risk vs. location is examined to determine the strength of its effects. Figure 7-35 shows a plot of the rear-end crash risk for the test cases that are shown in Table 7-48 for the 100 percent loading case.

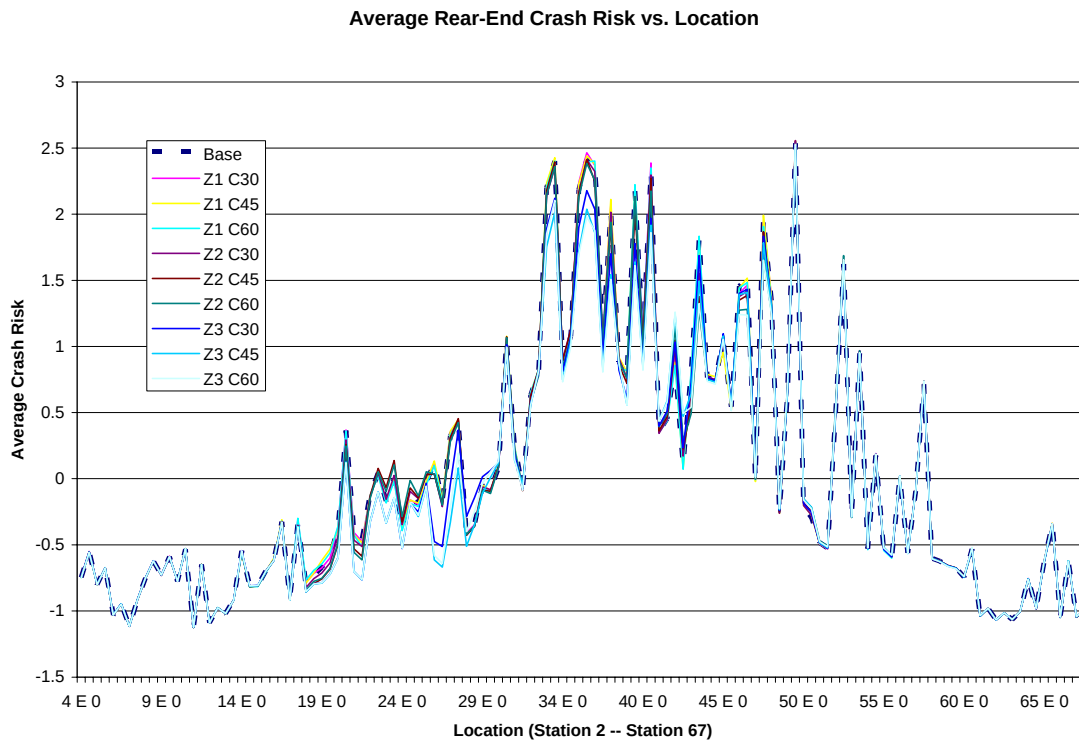


Figure 7-35: Average Rear-End Crash Risk vs. Location for Zone TC Cases at 100 Percent Loading

Figure 7-35 is rather confusing due to the large number of curves representing the different test cases. Additionally, the scale of the plot is relatively large compared to the differences in the crash risk. Zooming in on individual areas of the plot could be done (similar to the route diversion cases) however this would require numerous plots as the area of effect is spread out over multiple stations. Instead, a plot of the difference in the rear-end crash risk for each case compared to the base case vs. location is created for the stations that were affected by ramp metering. These crash difference values are found by

subtracting the crash risk at a particular location from the base case to determine the difference. Positive values imply a reduction in the crash risk while negative values show an increase in the crash risk. This plot is shown below in Figure 7-36.

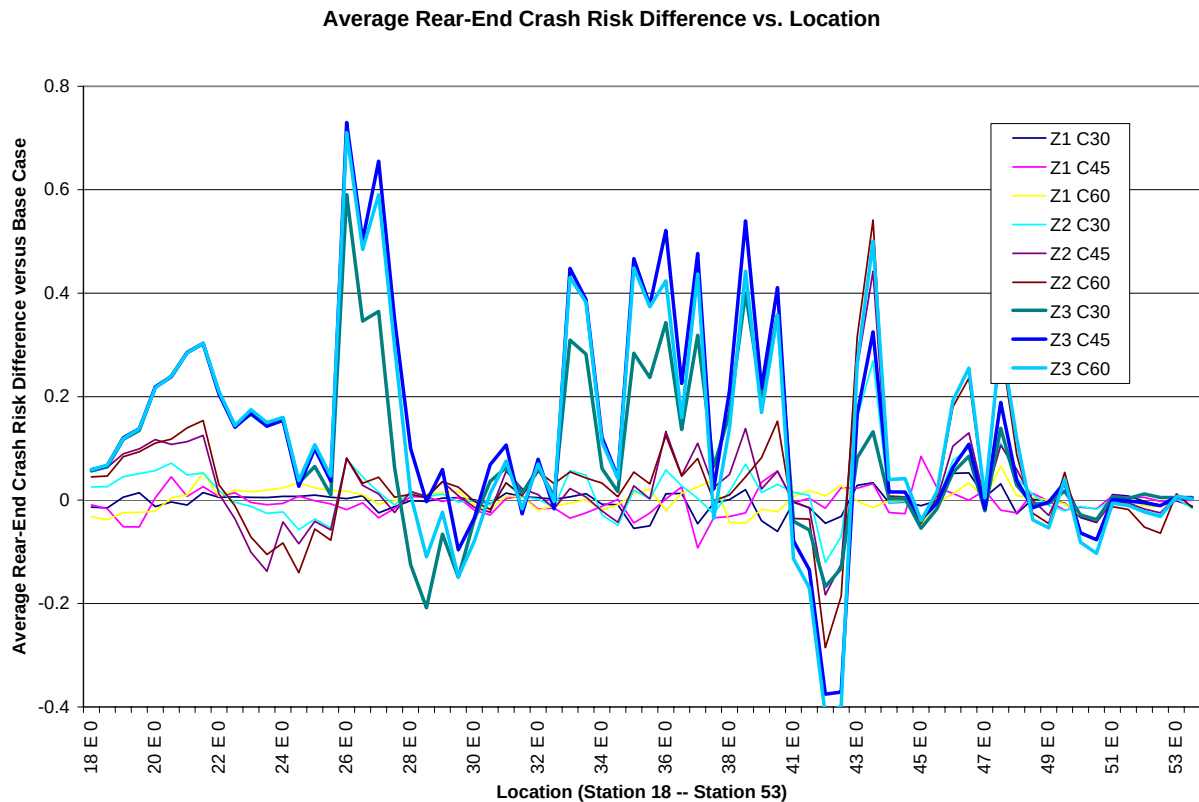


Figure 7-36: Average Rear-End Crash Risk Difference vs. Location for Zone TC Cases at 100 Percent Loading

From Tables 7-49 and 7-50, the best Zone algorithm, traffic-cycle realization cases are those that metered zone 3. In Figure 7-36, it is clear to see that for these cases the crash risk is increased between Stations 28-30 and 41-42 (denoted by a negative crash risk difference for these locations). These two locations have the largest increases in the crash risk compared to the other, trivial, increases elsewhere. The rear-end crash increase

at Stations 28-30 occurs due to the congested conditions that typically persist at these locations during the base case. When ramp metering is implemented the tail of the queue conditions, which typically occurs just upstream of this location, moves downstream near Stations 28-30. This causes slightly more congestion which increases the crash risk compared to the typical (base) conditions at this location. This movement of congestion throughout the network is the reason that the output of two rear-end crash risk regime models needed to be normalized. Ramp metering causes areas of congestion (regime 1 conditions) to become un-congested (regime 2 conditions) and vice versa. Only by combining the models were the crash risk effects able to be determined.

The crash risk increase at Stations 41-42 occurs for the same reason. Congested conditions from the downtown area move slightly downstream as the traffic volume in the downtown area decreases. This, in turn, increases the congestion around Stations 41-42 which causes increased values of the rear-end crash risk. Although the crash risk increases are rather large in these two areas they are considered allowable for two reasons. The first is that they occur at locations that typically have a low rear-end crash risk value compared to surrounding locations. This can be seen by examining Figure 7-37, below, which shows a plot of the base rear-end crash risk at the 100 percent loading scenario. At Stations 27-28 the average crash risk is 0.0 (which is already low) while at Stations 41-42 the average crash risk is 0.45. Just upstream and downstream of Stations 41-42 the rear-end crash risk value is 2.3 and 1.8, respectively. Therefore, since the crash risk is lower at Stations 41-42 than the surrounding stations, raising it slightly (to 0.65) in order to increase the overall safety of the network seems like a reasonable compromise. The second reason to allow this crash risk increase is that the overall rear-end safety

benefit is large and amounts to an average decrease of the rear-end crash risk of about 0.2 per location (34 stations). Increasing the rear-end risk by 0.4 for a small area (just 4 stations) in order to reduce the risk by 0.2 over a much larger area seems reasonable.

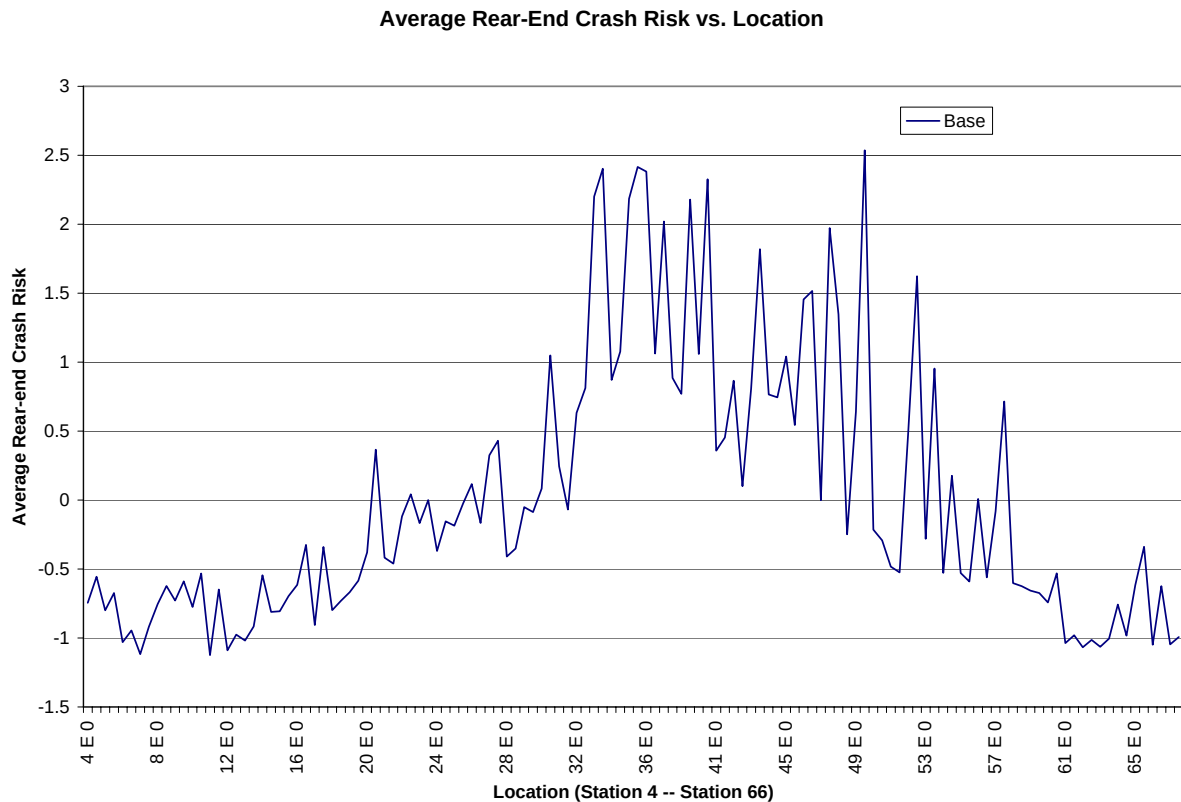


Figure 7-37: Average Rear-End Crash Risk for Base Case at 100 Percent Loading

The plot of the lane-change crash risk difference vs. location is given below in Figure 7-38 for the 100 percent loading scenarios. As shown in Figure 7-38, the best Zone traffic-cycle realization cases (metering zone 3) causes a decrease in the lane-change crash risk at the upstream locations while implementing this method increases the lane-change crash risk slightly between Stations 29 to 43 (which encompasses the downtown area). The decrease in the crash risk is much more substantial than (about 8

times the magnitude of) the increase in the crash risk. As shown in Figure 7-39, the location where the lane-change crash risk is increased due to the Zone ramp metering is an area where the lane-change crash risk is already high along the network corridor. Therefore, care would have to be taken when implementing the ramp metering due to the increase in the lane-change crash risk at a location where it is already high. However, the increase is only about 2% of the original value so it should not have that great of an effect. Metering zones 1 or 2 does not cause this increase in the crash risk at Stations 29 to 43 but the lane-change safety benefit provided by these scenarios is minimal.

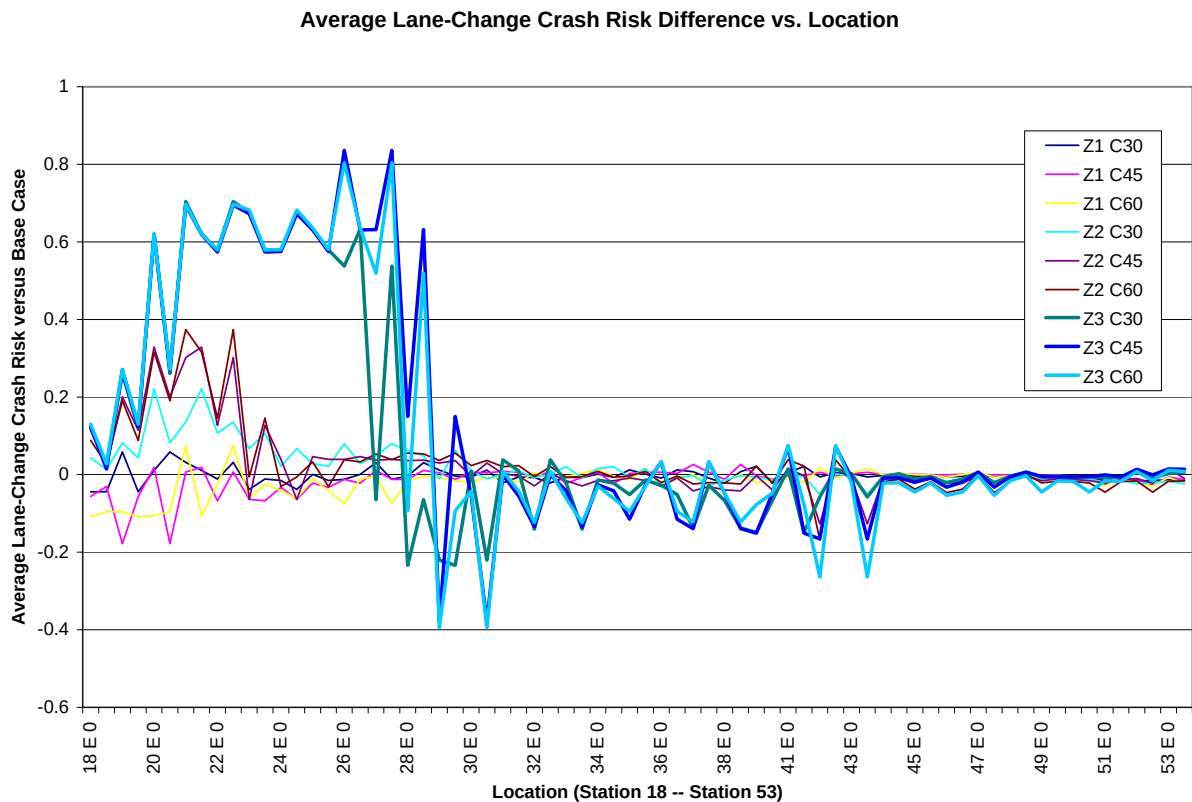


Figure 7-38: Average Lane-Change Crash Risk Difference vs. Location for Zone TC Cases at 100 Percent Loading

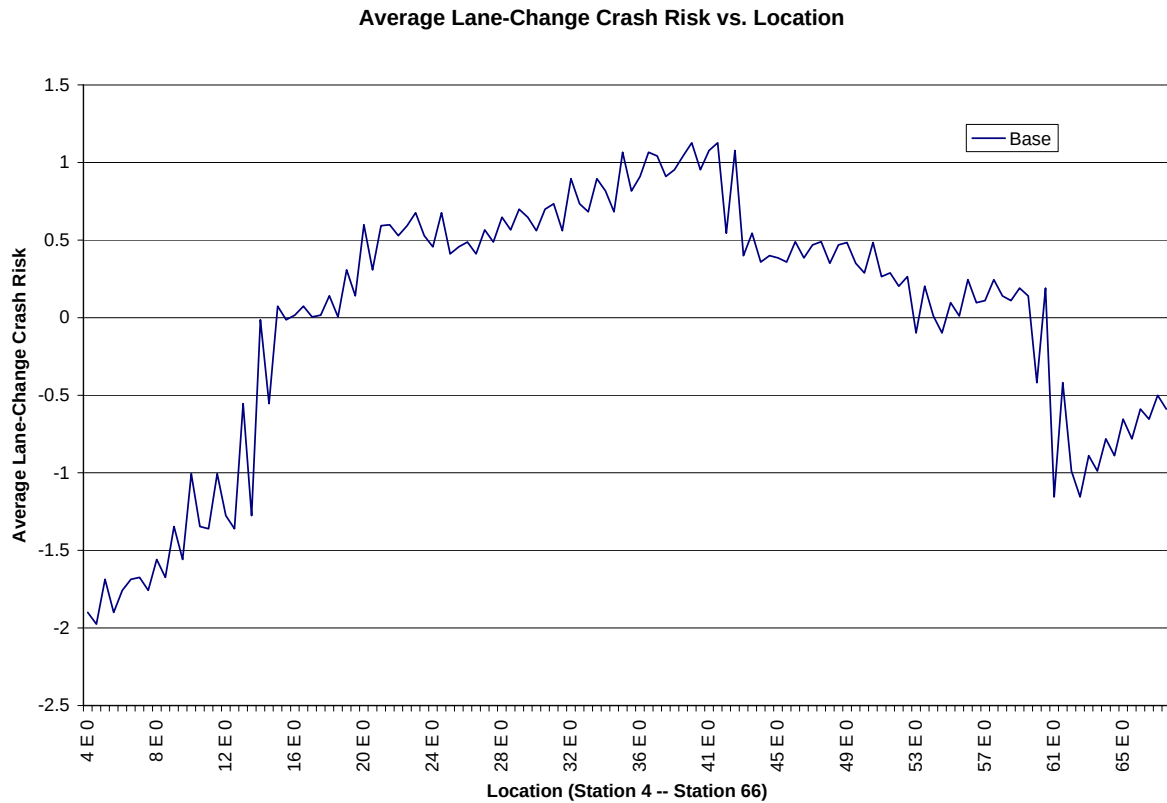


Figure 7-39: Average Lane-Change Crash Risk for Base Case at 100 Percent Loading

For the cases performed at the 90 percent loading scenario, the results are similar. Figure 7-40 shows the difference in the rear-end crash risk while Figure 7-41 shows the difference in the lane-change crash risk for the various test cases.

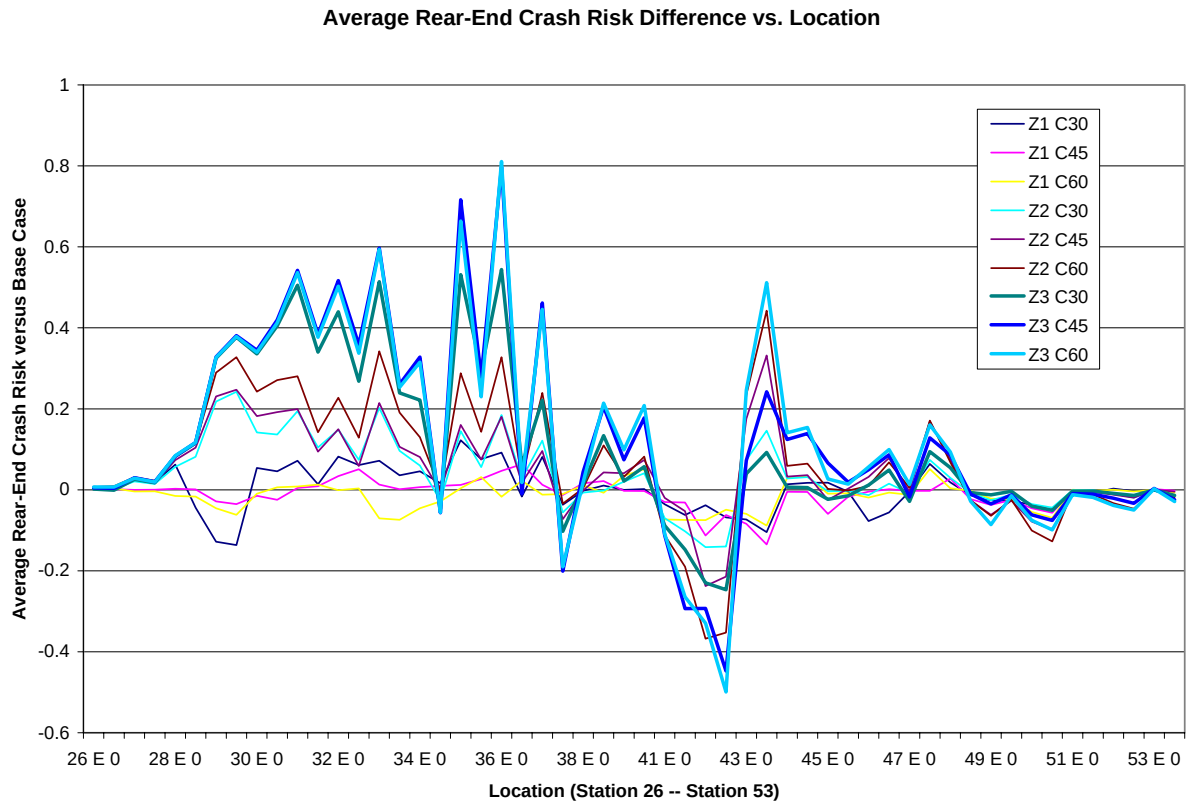


Figure 7-40: Average Rear-End Crash Risk Difference vs. Location for Zone TC Cases at 90 Percent Loading

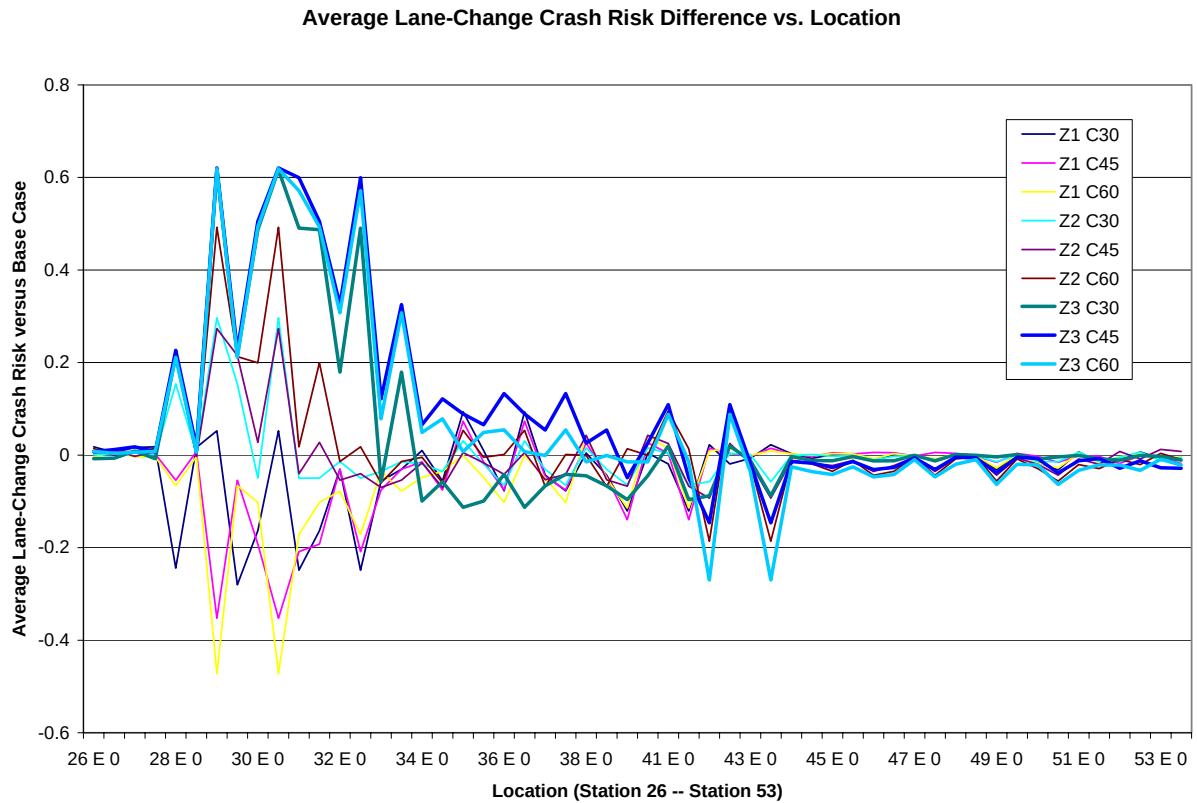


Figure 7-41: Average Lane-Change Crash Risk Difference vs. Location for Zone TC Cases at 90 Percent Loading

The plot of the rear-end crash risk difference shows that for the best zone ramp metering case the crash risk is only increased at Stations 41 to 42. Once again this is just an isolated instance and the base crash risk at this location is already lower than the surrounding stations so the small crash risk increase can be overlooked for the greater crash risk decrease. Figure 7-41 shows that at the 90 percent loading scenario the lane-change crash risk also increased for just a small area of the freeway as compared to the 100 percent loading scenarios. Therefore, this section shows that the best case involving the Zone algorithm with the traffic-cycle realization is to meter zone 3 using a 60 second cycle length. Although this method causes some crash migration effects, the locations

that experience high levels of rear-end crash risk migration are areas with traditionally low rear-end crash risk values. The effects of the lane-change crash risk migration is small compared to the overall reduction in the lane-change crash risk and can therefore be tolerated in order to reduce the overall lane-change crash risk across the network.

7.3.3 Analysis of One-Car-Per-Cycle Realization Strategies

The previous two sections examined both the ALINEA and Zone algorithm when used with the traffic-cycle realization. The next group of ramp metering cases that are examined are those cases that implement the one-car-per-cycle realization as opposed to the traffic-cycle realization. The specific cases from the experimental design that are analyzed in this group are displayed below in Table 7-51.

Table 7-51: Test Cases Representing One-Car-Per-Cycle Realization of Zone and ALINEA

Case Number	Percent Loading	Metered Zone	Algorithm	Cycle Length
103	100	1	Zone	OCPC
104	100	1	ALINEA	OCPC
109	100	2	Zone	OCPC
110	100	2	ALINEA	OCPC
115	100	3	Zone	OCPC
116	100	3	ALINEA	OCPC
121	90	1	Zone	OCPC
122	90	1	ALINEA	OCPC
127	90	2	Zone	OCPC
128	90	2	ALINEA	OCPC
133	90	3	Zone	OCPC
134	90	3	ALINEA	OCPC

The values of ORCI and LCRCI have been calculated for both the 100 percent loading scenarios and the 90 percent loading scenarios and are presented below in Tables 7-52 and 7-53, respectively. The tables reference the test case numbers presented above

in Table 7-51 as well as describes the cases by the metering type and traffic realization. For example, case 104 (Z 1 ALINEA OCPC) represents using the ALINEA algorithm to meter the ramps that comprise zone 1 while implementing the one-car-per-cycle (or OCPC) realization.

Table 7-52: Summary of ORCI and LCRCI for OCPC Cases at 100 Percent Loading

	Test Case ID					
	Case 103	Case 104	Case 109	Case 110	Case 115	Case 116
	Z 1 OCPC	Z 1 ALINEA OCPC	Z 2 OCPC	Z 2 ALINEA OCPC	Z 3 OCPC	Z 3 ALINEA OCPC
# of Stations with RE Risk Change	5	7	0	36	3	39
ORCI	0.1578	0.0883	0.0000	5.2296	0.1139	7.2222
# of Stations with LC Risk Change	0	8	4	26	0	34
LCRCI	0.0000	1.5736	-0.4676	8.1495	0.0000	12.0102

Table 7-53: Summary of ORCI and LCRCI for OCPC Cases at 90 Percent Loading

	Test Case ID					
	Case 121	Case 122	Case 127	Case 128	Case 133	Case 134
	Z 1 OCPC	Z 1 ALINEA OCPC	Z 2 OCPC	Z 2 ALINEA OCPC	Z 3 OCPC	Z 3 ALINEA OCPC
# of Stations with RE Risk Change	1	5	3	34	6	37
ORCI	-0.0719	0.0029	-0.4136	5.8677	-0.7912	5.7074
# of Stations with LC Risk Change	6	2	2	20	6	22
LCRCI	-1.5124	-0.5614	-0.7378	6.6130	-1.9127	8.3968

For each of the crash risk performance measures (ORCI and LCRCI) almost every test case using the ALINEA algorithm with the one-car-per-cycle realization outperforms using the Zone Algorithm with the one-car-per-cycle-realization. This is expected based on the previous findings. In the experimental design to determine the best ALINEA parameters, the safety along the freeway corridor increased as the cycle length decreased.

This shows that when using ALINEA, allowing smaller platoons of vehicles onto the freeway more frequently provides the best results. Therefore, it would make sense that the one-car-per-cycle realization would provide good safety results since this method allows only a single vehicle to enter the freeway at an increased frequency. Similarly, the previous section showed that using a longer cycle length improves the ability of the Zone algorithm to decrease the two crash risk values. Therefore, implementing the one-car-per-cycle realization, which uses the shortest cycle lengths, goes against this trend and, therefore, does not perform well.

Additionally, ALINEA is more restrictive than the Zone algorithm at higher levels of congestion. As seen in Equation 6-2, the Zone algorithm includes a term that accounts for the spare capacity of the zone. This spare capacity is found by examining all mainline loop detectors within the zone, not just those near entrance ramps. Therefore, if congestion is increased near the ramps that are being metered but the detectors still show a relatively low traffic density the Zone algorithm will allow more vehicles into the network to fill this spare capacity. The ALINEA algorithm does not consider the spare capacity and would therefore be more restrictive. This will lead to a reduction in the safety measures of effectiveness (ORCI and LCRCI) for the Zone algorithm as allowing more vehicles into the network would increase the crash risk. When this is combined with the one-car-per-cycle realization, at some ramps vehicles enter the freeway almost as soon as they arrive at the ramp. Therefore, the Zone algorithm with one-car-per-cycle methodology shows no great difference between the metering and non-metering case.

Figures 7-42 and 7-43 below show the differences in the rear-end and lane-change crash risk vs. location, respectively, for the cases run at 100 percent loading.

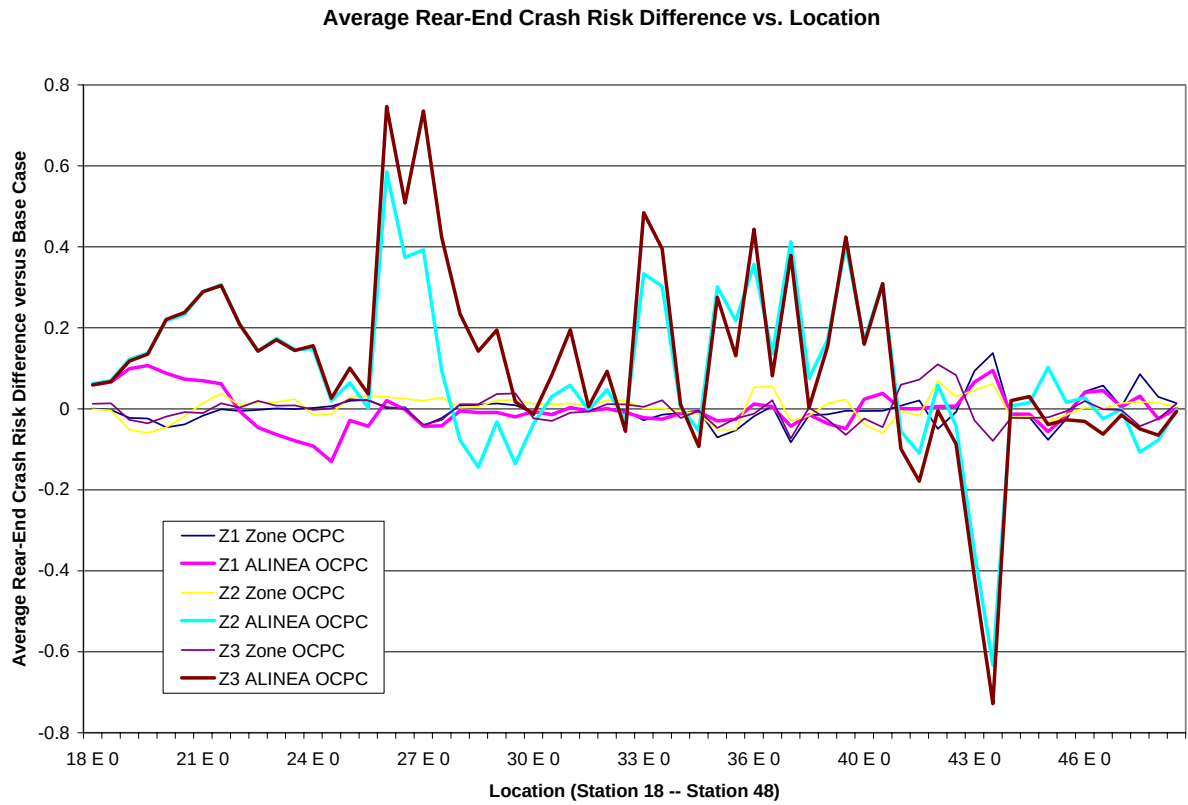


Figure 7-42: Average Rear-End Crash Risk Difference vs. Location for OCPC Cases at 100 Percent Loading

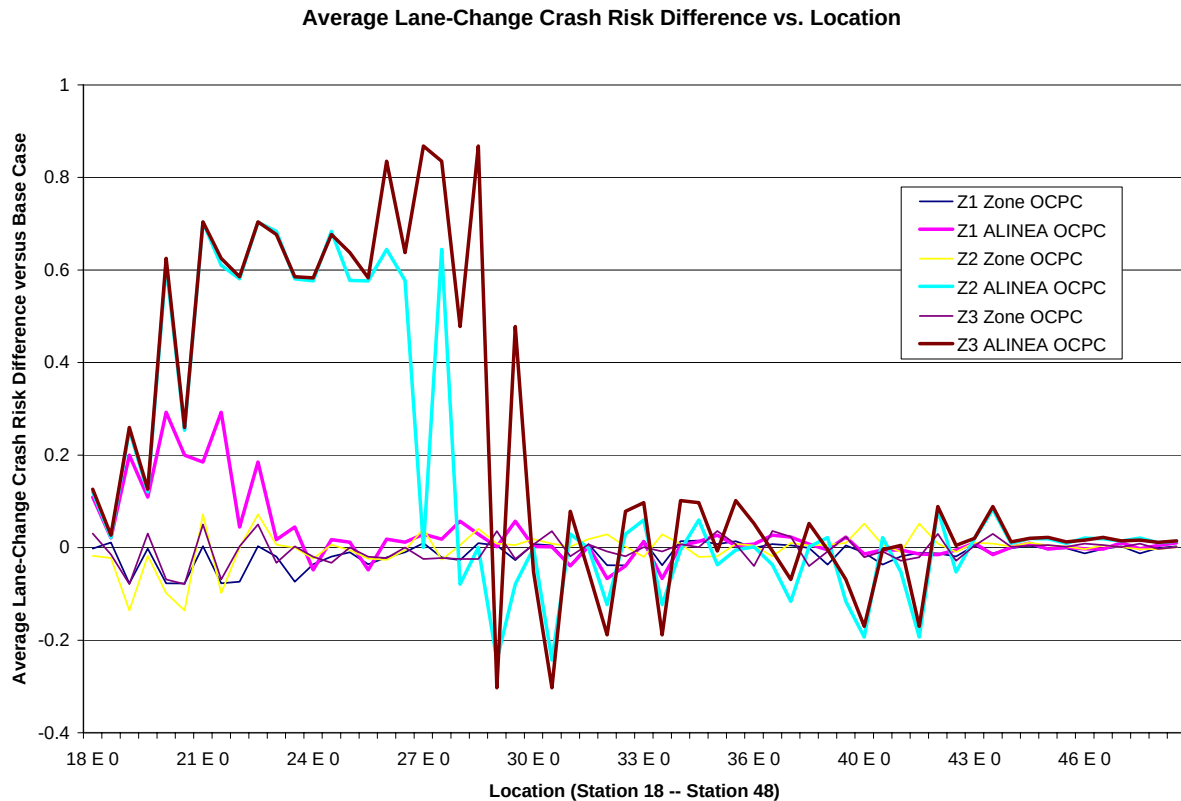


Figure 7-43: Average Lane-Change Crash Risk Difference vs. Location for OCPC Cases at 100 Percent Loading

Once again Figures 7-42 and 7-43 show that ramp metering causes a crash migration effect when the best ALINEA OCPC case (metering zone 3) is implemented. However, as evidenced by the positive values of ORCI and LCRCI for the one-car-per-cycle realization performed with the ALINEA algorithm, the overall effect of the ALINEA cases is a reduction in the crash risk through the network corridor. The rear-end crash risk is increased in a localized area and caused primarily by the creation of a new bottleneck area just downstream of the downtown area. The lane-change crash risk increases throughout the downtown area (Stations 32 to 40) but this increase is rather trivial compared to the crash risk decrease that is noted.

Similar plots are created for the 90 percent loading scenarios and are presented below in Figures 7-44 and 7-45. The same trends exist for metering at the 90 percent loading case. Any metering performed at zone 1 does little to reduce the crash risk significantly. Additionally, metering any zone using the Zone algorithm and one-car-per-cycle realization does not affect the crash risk substantially. The increase in the rear-end crash risk caused by the ALINEA strategies occurs near Stations 41 to 43 which is traditionally a low rear-end crash risk region. For the lane-change crash risk, implementing the ALINEA ramp metering strategy at the 90 percent loading scenario does not show any evidence of crash migration.

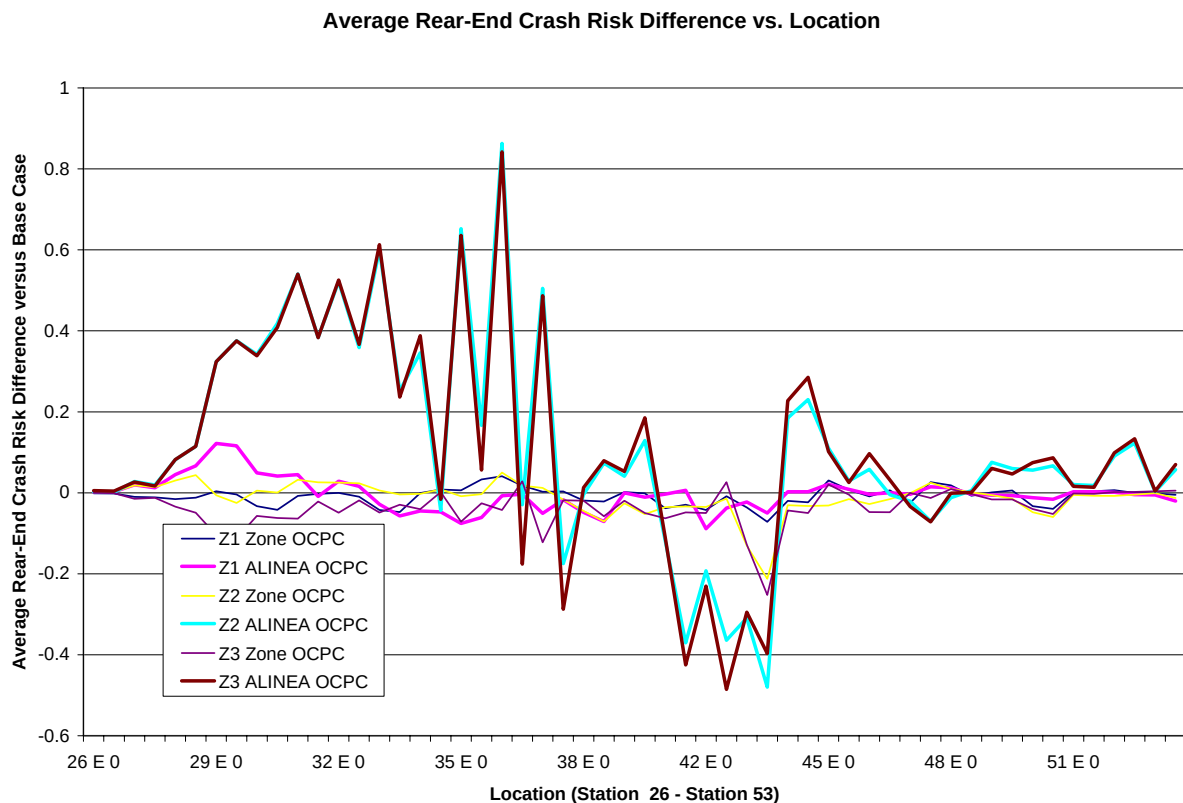


Figure 7-44: Average Rear-End Crash Risk Difference vs. Location for OCPC Cases at 90 Percent Loading

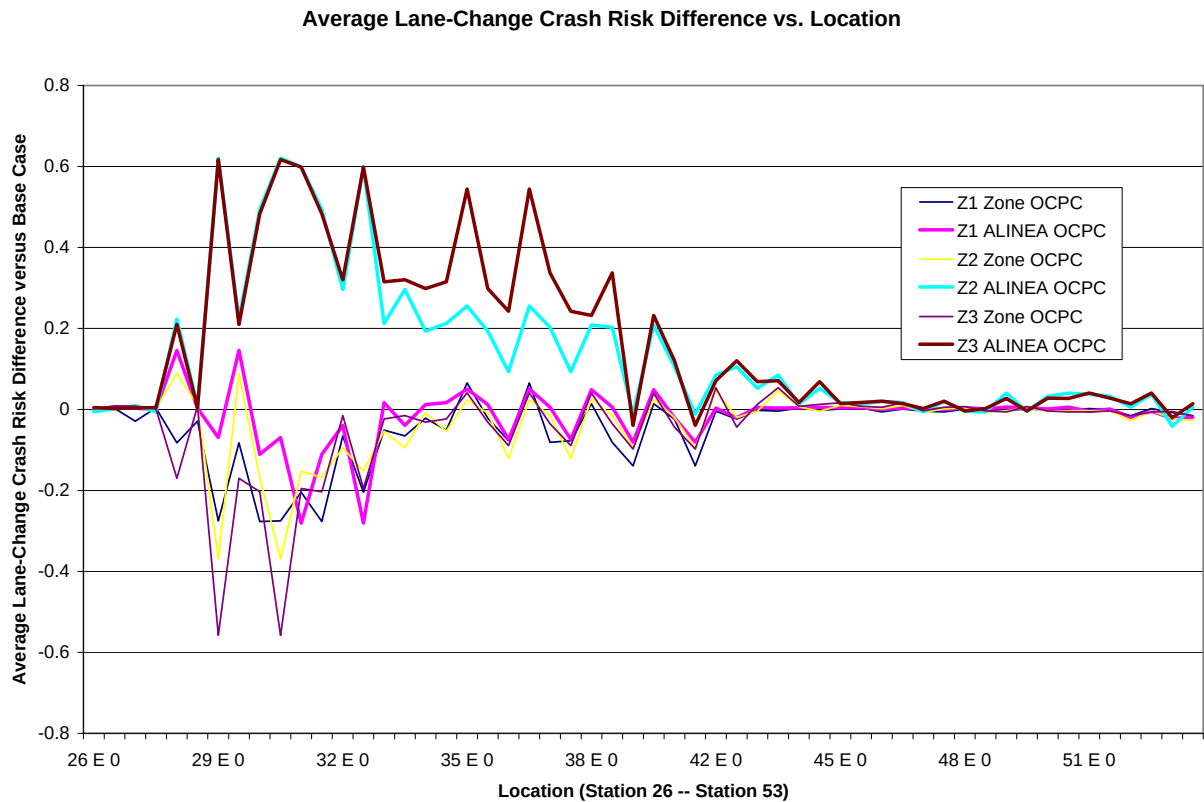


Figure 7-45: Average Lane-Change Crash Risk Difference vs. Location for OCPC Cases at 90 Percent Loading

Therefore, this section shows that when the one-car-per-cycle realization method is applied to the ramp metering algorithms that are being considered, ALINEA outperforms the Zone algorithm at reducing both the rear-end and lane-change crash risk. Also, the trends in this section show that metering zone 1 with either the ALINEA or Zone algorithm shows very minimal, though existent, crash risk benefits compared to metering zones 2 or 3.

7.3.4 Determining the Final “Best” Ramp Metering Strategy

The purpose of the previous sections was to compare similar ramp metering strategies to determine the best implementation methods for these strategies. For example, Section 7.3.1 found that the best implementation of the ALINEA algorithm with the traffic-cycle realization is to use a small cycle length of 30 seconds and a critical occupancy of 0.17 (found from the experimental design given in Table 6-7). Section 7.3.2 found that when the Zone algorithm is used with the traffic-cycle realization a longer cycle length is needed (60 seconds) to provide the best safety results. Section 7.3.3 showed that the one-car-per-cycle implementation method with the Zone algorithm causes negligible safety benefits while the one-car-per-cycle implementation of the ALINEA algorithm significantly decreases both crash risk measures. In all cases, metering zones 2 and 3 provided the best safety results. Previous results have shown that metering zone 1 (using any method) causes a negligible change in the crash risk on the network compared to metering the other two zones. This occurs because zone 1 is located immediately upstream of the bottleneck that occurs at the Interstate-4 / S. R. 408 Interchange. Therefore, zone 1 scenarios will be excluded from the comparison in this section in order to reduce the complexity of this analysis. However, these strategies now need to be compared against each other to determine which metering strategy provides the best overall crash risk change in the downtown portion of Interstate-4.

Table 7-54 shows the remaining test cases at the 100 percent loading level as well as the summarized ORCI and LCRCI values. The test cases are denoted with the same nomenclature that has been used in previous sections. A similar table is presented in Table 7-55 for the test cases run at the 90 percent loading scenario. Note that in these

tables the strategies that use the traffic-cycle realization are denoted with the abbreviation TC while the one-car-per-cycle abbreviation used is OCPC.

Table 7-54: Summary of ORCI and LCRCI for Best Ramp Metering Cases at 100 Percent Loading

	Test Case ID					
	Case 107	Case 108	Case 110	Case 113	Case 114	Case 116
	Z 2 Zone TC	Z 2 ALINEA TC	Z 2 ALINEA OCPC	Z 3 Zone TC	Z 3 ALINEA TC	Z 3 ALINEA OCPC
# of Stations with RE Risk Change	27	37	39	46	48	42
ORCI	1.7290	9.9240	5.4254	8.4413	12.7085	7.1622
# of Stations with LC Risk Change	20	40	34	48	40	44
LCRCI	1.9229	10.4808	8.1703	8.2556	15.5739	12.1375

Table 7-55: Summary of ORCI and LCRCI for Best Ramp Metering Cases at 90 Percent Loading

	Test Case ID					
	Case 125	Case 126	Case 128	Case 131	Case 132	Case 134
	Z 2 Zone TC	Z 2 ALINEA TC	Z 2 ALINEA OCPC	Z 3 Zone TC	Z 3 ALINEA TC	Z 3 ALINEA OCPC
# of Stations with RE Risk Change	28	37	32	35	41	34
ORCI	3.5568	5.0898	5.8645	6.9996	5.9780	5.6343
# of Stations with LC Risk Change	18	30	28	24	32	28
LCRCI	1.3670	10.3241	7.2852	4.1848	11.1706	8.9160

From Table 7-54, comparing the values of the ORCI and LCRCI shows that the best ramp metering combination at the 100 percent loading scenario is using the ALINEA strategy with the traffic-cycle realization. From Table 7-55, the best metering combination at the 90 percent loading scenario is not so clear. The ALINEA TC method provides the highest LCRCI values as well as the second highest ORCI for each metered zone. However, the other cases have a higher values of ORCI which shows they are

better at reducing the rear-end crash risk. Therefore, no judgment about the best case at the 90 percent loading scenario could be made until the effects of crash migration are considered.

To examine for possible crash migration issues of this best ramp metering strategy a plot of the rear-end crash risk difference and lane-change crash risk difference vs. location is produced. These plots are presented in Figures 7-46 and 7-47, respectively, for the 100 percent loading scenario.

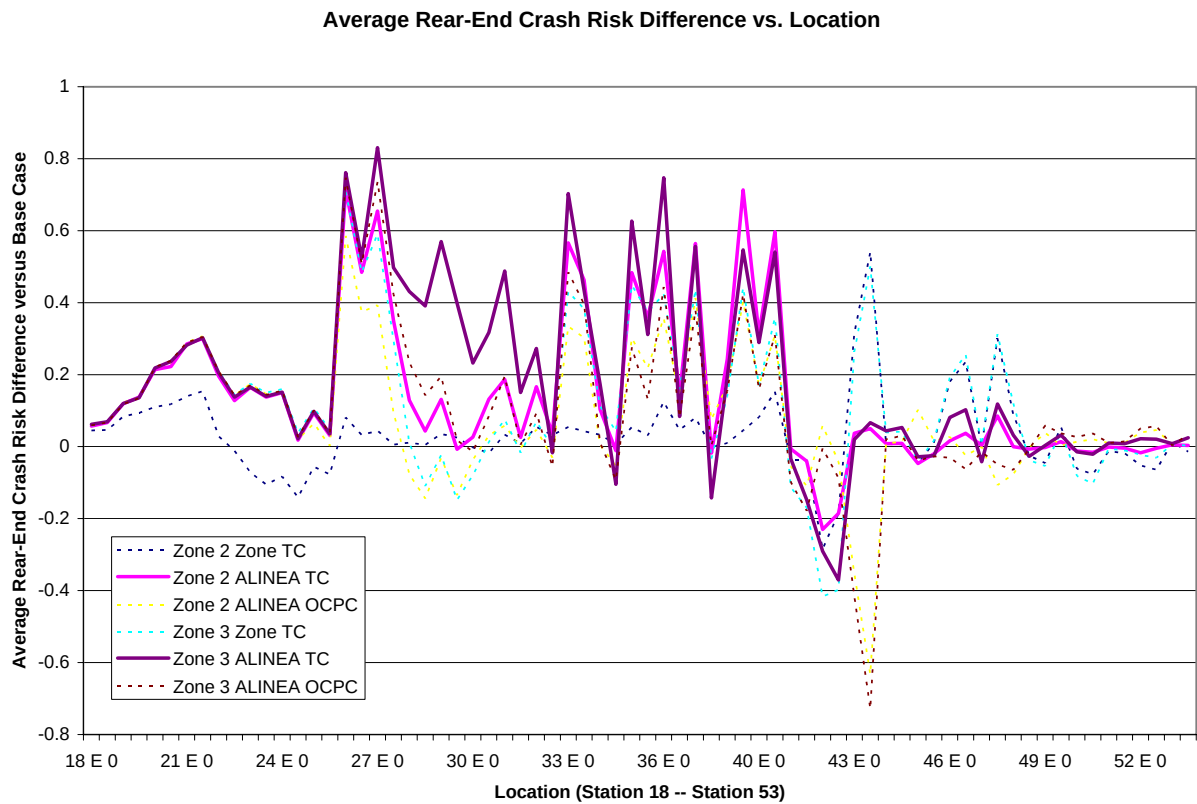


Figure 7-46: Average Rear-End Crash Risk Difference vs. Location for Best Cases at 100 Percent Loading

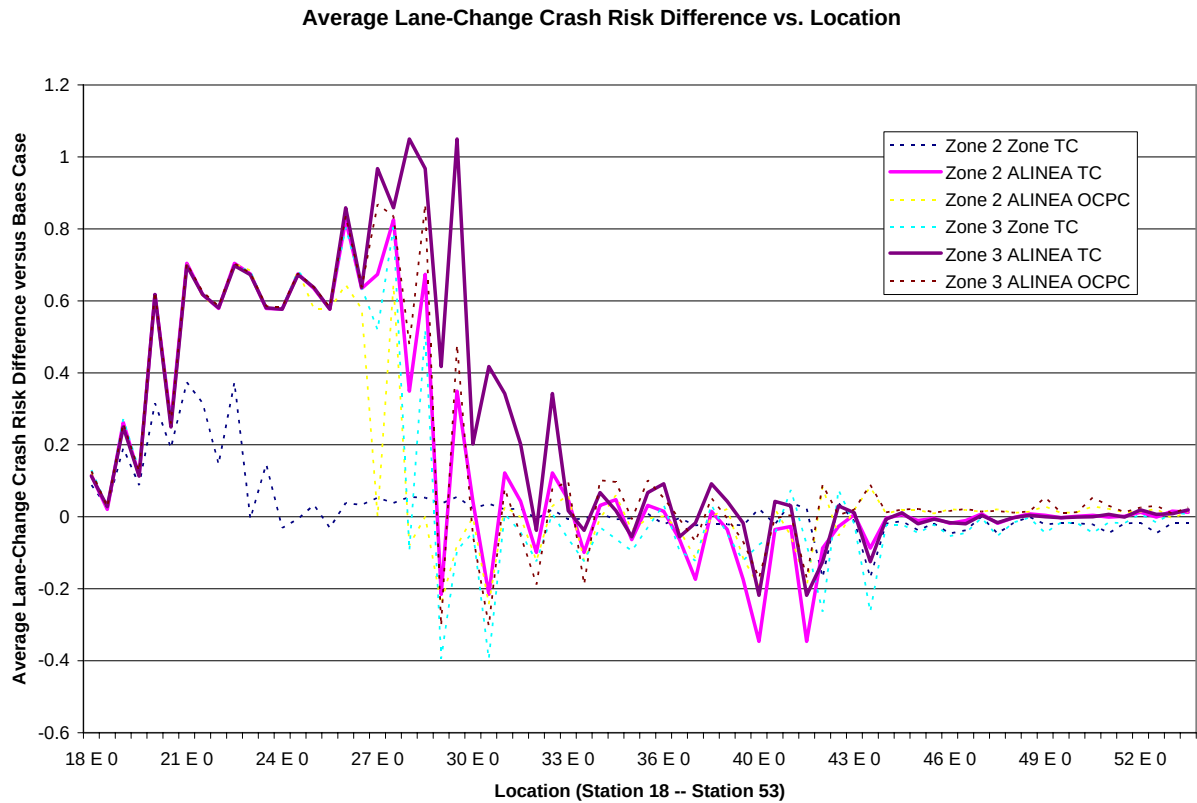


Figure 7-47: Average Lane-Change Crash Risk Difference vs. Location for Best Cases at 100 Percent Loading

As shown in Figure 7-46, the rear-end crash migration is minimal for the ALINEA traffic-cycle strategy compared with the other well-performing metering scenarios. The crash risk only increases significantly only for the area between Stations 41 and 42. This risk increase is much less than a similar increase experienced by other strategies. The lane-change crash risk increases slightly between stations 28 and 43 but this increase is very small in magnitude and does not compare to the crash risk decrease realized just upstream.

Similar plots are created for runs performed at the 90 percent loading scenario. These plots are produced below in Figures 7-48 and 7-49. From Figure 7-48 it is seen

that using the ALINEA strategy with the traffic-cycle realization causes a crash risk increase from stations 37 to 43. Although the magnitude of the increase is small from Stations 37 to 40, between Stations 41 to 43 this crash risk increase is substantial. The other metering strategies show a similar crash risk increase at Stations 41 to 43 but not to the magnitude that is realized with the ALINEA TC method. However, for the lane-change crash risk (presented in Figure 7-49) the ALINEA TC method causes greater crash risk decreases and less crash migration.

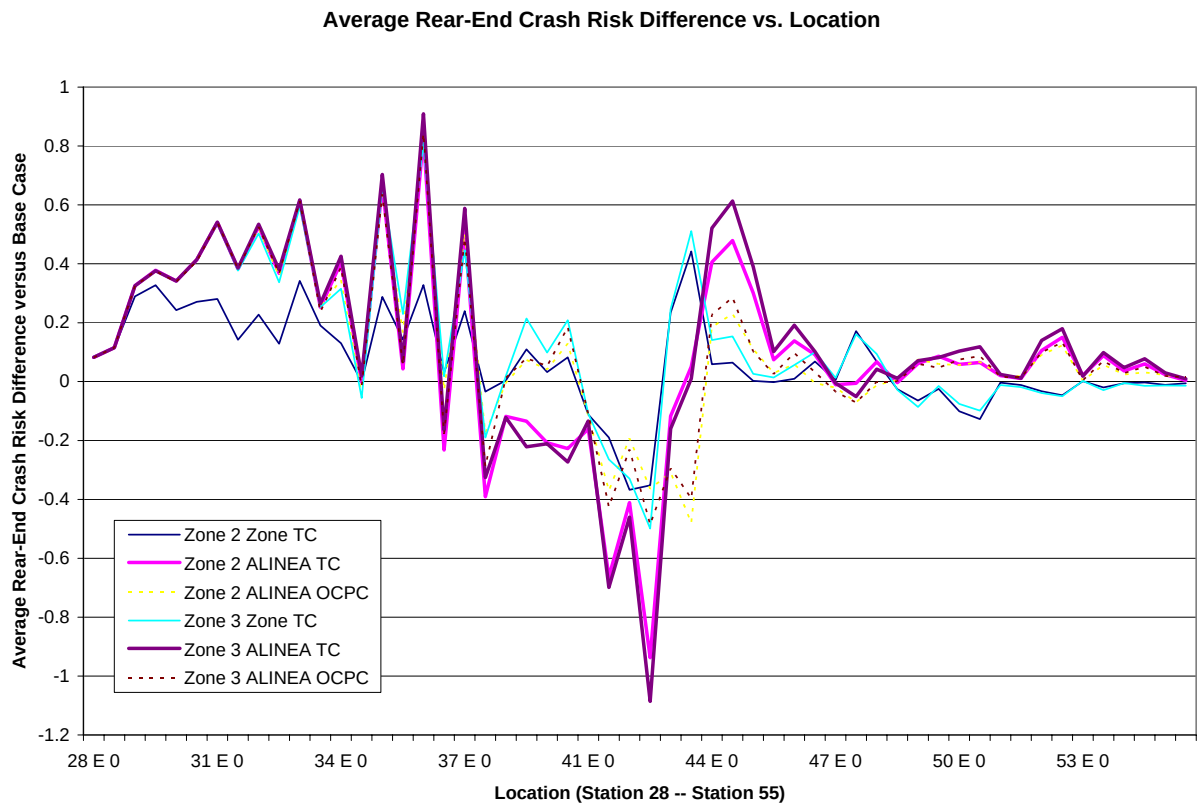


Figure 7-48: Average Rear-End Crash Risk Difference vs. Location for Best Cases at 90 Percent Loading

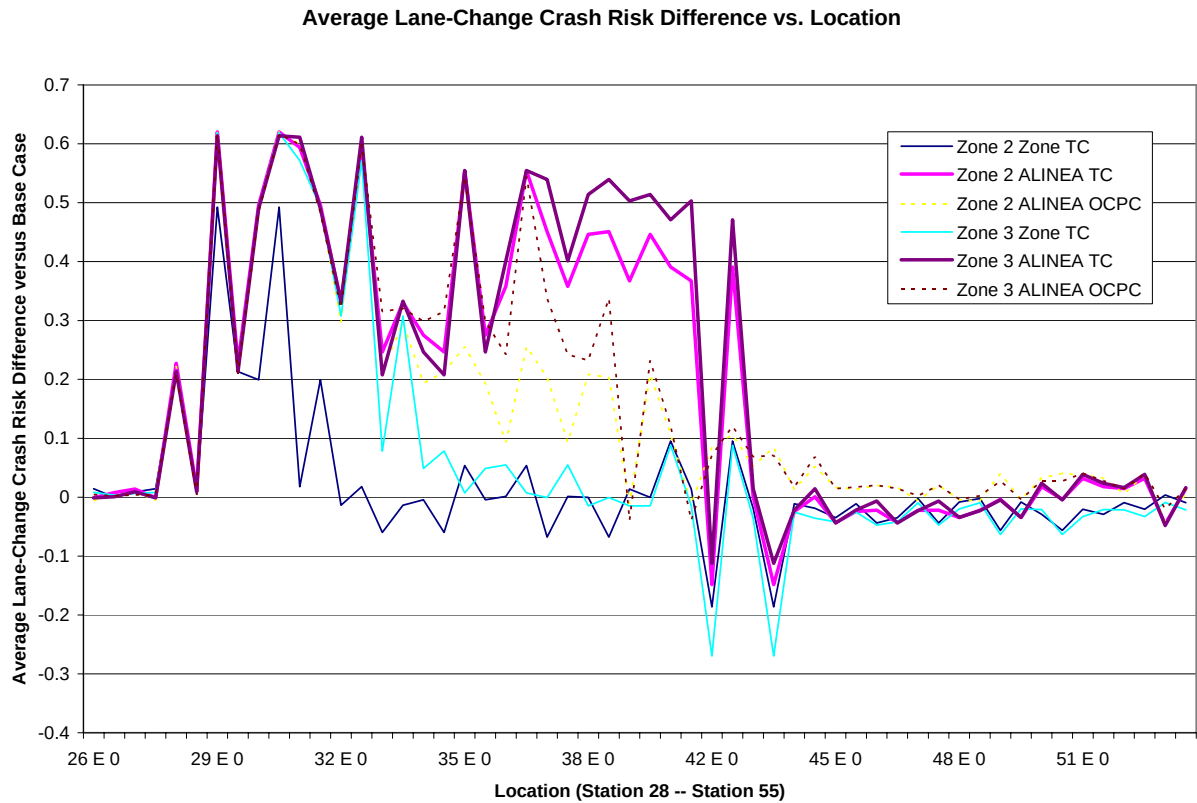


Figure 7-49: Average Lane-Change Crash Risk Difference vs. Location for Best Cases at 90 Percent Loading

Therefore, at the 90 percent loading scenario, there is significant rear-end crash risk migration caused by the best ALINEA traffic-cycle strategy. In fact, the largest increase in the rear-end crash risk (1.08 at Station 42 E 1) is greater than the largest decrease in the rear-end crash risk (0.91 at Station 36 E 0). This amount of crash migration has never occurred in any of the scenarios presented before and should not be considered as reasonable. The next best strategy appears to be using the ALINEA strategy with the one-car-per-cycle realization as evidenced by the high values of ORCI and LCRCI in Table 7-55. The crash migration that occurs for this scenario is much less than the ALINEA traffic-cycle strategy and it provides reasonable safety benefits.

7.3.5 Travel time analysis

While ramp metering has a positive effect on the rear-end and lane-change crash risk along the Interstate-4 corridor there is the potential that delaying vehicles on the on-ramps will have negative impacts on the operational capabilities of the network. In order to account for this fact the total network travel time was calculated for each of the scenarios performed and compared. The network travel time for all runs performed in this portion of the experimental design is given below in Table 7-56. The average travel time considered for this table was the overall network travel time which included both the decreased travel time on the mainline freeway and the increased travel times of vehicles delayed at the metered on-ramps.

Table 7-56: Travel Time Summary for Ramp Metering Scenarios

Case Number	Percent Loading	Metered Zone	Algorithm	Cycle Length	Average Travel Time Increase
99	100	1	Zone	30	0.77%
100	100	1	Zone	45	1.18%
101	100	1	Zone	60	1.57%
102	100	1	ALINEA	BEST	-0.84%
103	100	1	Zone	OCPC	-0.22%
104	100	1	ALINEA	OCPC	0.35%
105	100	2	Zone	30	1.42%
106	100	2	Zone	45	0.20%
107	100	2	Zone	60	1.49%
108	100	2	ALINEA	BEST	-0.37%
109	100	2	Zone	OCPC	0.84%
110	100	2	ALINEA	OCPC	3.35%
111	100	3	Zone	30	-1.42%
112	100	3	Zone	45	-1.92%
113	100	3	Zone	60	-1.69%
114	100	3	ALINEA	BEST	-1.87%
115	100	3	Zone	OCPC	0.63%
116	100	3	ALINEA	OCPC	3.17%
117	90	1	Zone	30	-1.44%
118	90	1	Zone	45	-1.02%
119	90	1	Zone	60	-0.35%
120	90	1	ALINEA	BEST	0.01%
121	90	1	Zone	OCPC	-1.14%
122	90	1	ALINEA	OCPC	0.04%
123	90	2	Zone	30	-0.60%
124	90	2	Zone	45	-0.35%
125	90	2	Zone	60	-1.71%
126	90	2	ALINEA	BEST	4.61%
127	90	2	Zone	OCPC	-1.04%
128	90	2	ALINEA	OCPC	10.23%
129	90	3	Zone	30	0.96%
130	90	3	Zone	45	1.77%
131	90	3	Zone	60	-0.28%
132	90	3	ALINEA	BEST	5.35%
133	90	3	Zone	OCPC	0.68%
134	90	3	ALINEA	OCPC	9.41%

From Table 7-56 it can be seen that the overall network travel times for the Zone algorithm are smaller than the network travel times that are experienced when the

ALINEA metering algorithm is used. The coordination of the Zone metering algorithm, which increases the metering rate at one ramp to compensate for reduced flows at other on-ramps, helps to decrease the average delay for all vehicles on the on-ramps. This, in turn, reduces the average travel time along the entire network. Similar travel time decreases have been realized in other studies that used the Zone metering algorithm to improve traffic operations (Stephanedes, 1994). When ALINEA is used the metering rate at one ramp is defined only by the conditions of the nearest downstream loop detector. Therefore, a large queue of vehicles can be formed at one ramp while another ramp operates at near free-flow due to the lack of demand. This causes the network travel time to be much higher than when the Zone algorithm is used. Additionally, as the cycle length increases for each metering algorithm the average network travel time decreases. This shows that using the traffic-cycle realization outperforms the one-car-per-cycle realization from a traffic operations perspective as well as a traffic safety perspective.

As noted in Table 7-56, the strategy that was found to be the most beneficial during the 100 percent loading scenario (ALINEA with traffic-cycle realization) also provided a decrease in the average travel time along the network. When zone 2 is metered, this travel time decrease was about 0.4% of the overall network travel time while there was a 1.9% decrease when zone 3 was metered. Therefore, this strategy provides both safety and operational benefits. The best metering strategy at the 90 percent loading strategy (ALINEA with one-car-per-cycle realization) did not have reasonable travel time changes. When this strategy is used the average network travel time is increased by 10%. Such a large travel time increase is unrealistic to impose on drivers in the field and, therefore, this scenario cannot be considered as a practical

metering alternative. Using the ALINEA algorithm with traffic-cycle realization also has increased travel times of about 5%. Combined with the high levels of crash migration shown in Section 7.3.4 and this strategy is not a viable alternative. Therefore, the next best strategy, using the Zone algorithm with the traffic-cycle realization, should be used. This method provides good safety benefits as seen in Section 7.3.4 and serves to help decrease the average network travel time (1.7% decrease for zone 2 and 0.3% decrease for zone 3). As shown in Figures 7-48 and 7-49, this method also does not induce high levels of crash migration. Therefore, this strategy should be used in the downtown area at moderate levels of congestion (90 percent loading).

7.3.6 Metering Ramps over the Entire Network Corridor

7.3.6.1 100 Percent Loading Scenario

As previously determined, the best ramp metering strategy at the 100 percent loading scenario is to employ the ALINEA metering algorithm with the traffic-cycle realization (30 second cycle length). In the previous sections, the safety effects of ramp metering were only examined when the ramps in the downtown portion of the Interstate-4 freeway were metered. However, ramp metering is not necessarily constrained to this area like route diversion is (since ramp metering does not depend on the availability of practical diversion routes). Therefore, in order to reduce the rear-end and lane-change crash risk over the entire freeway area it is possible to implement ramp metering at all on-ramps within the network corridor.

The ALINEA algorithm with the traffic-cycle realization and 30 second cycle length was applied to the on-ramps within the network on the eastbound direction of

Interstate-4. Summary statistics comparing the network wide implementation of ramp metering with the implementation in the downtown area only is given in Table 7-57. As shown in this table, both the rear-end and lane-change safety are clearly increased with the network wide metering implementation which is evidenced by the increased values of ORCI and LCRCI.

Table 7-57: Summary of Network-Wide Ramp Metering Strategies at 100 Percent Loading

	Test Case	
	Whole Network	Downtown Portion
# of Stations with RE Risk Change	68	48
ORCI	17.3678	12.7085
# of Stations with LC Risk Change	73	40
LCRCI	24.3552	15.5739

The increase in the ORCI and LCRCI is about 34% and 56%, respectively, when the all the ramps in the network corridor are metered compared to when just the ramps in the downtown region are metered. However, the number of ramps that are metered in the network wide implementation is about triple the number of ramps that are metered in the downtown implementation. This implies that the safety increase is not proportionally related to the number of ramps metered. A plot of the average rear-end and lane-change crash risk difference is shown below in Figures 7-50 and 7-51, respectively.

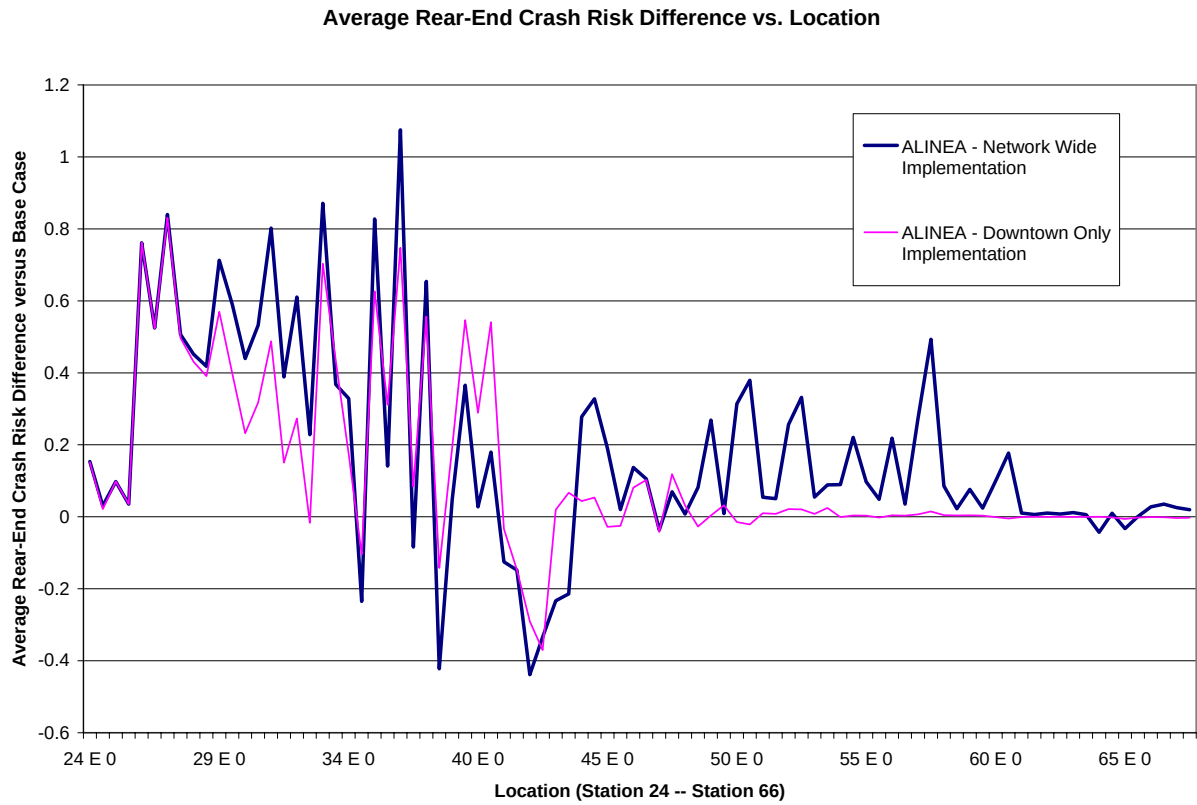


Figure 7-50: Average Rear-End Crash Risk Difference vs. Location for Network-Wide Cases at 100 Percent Loading

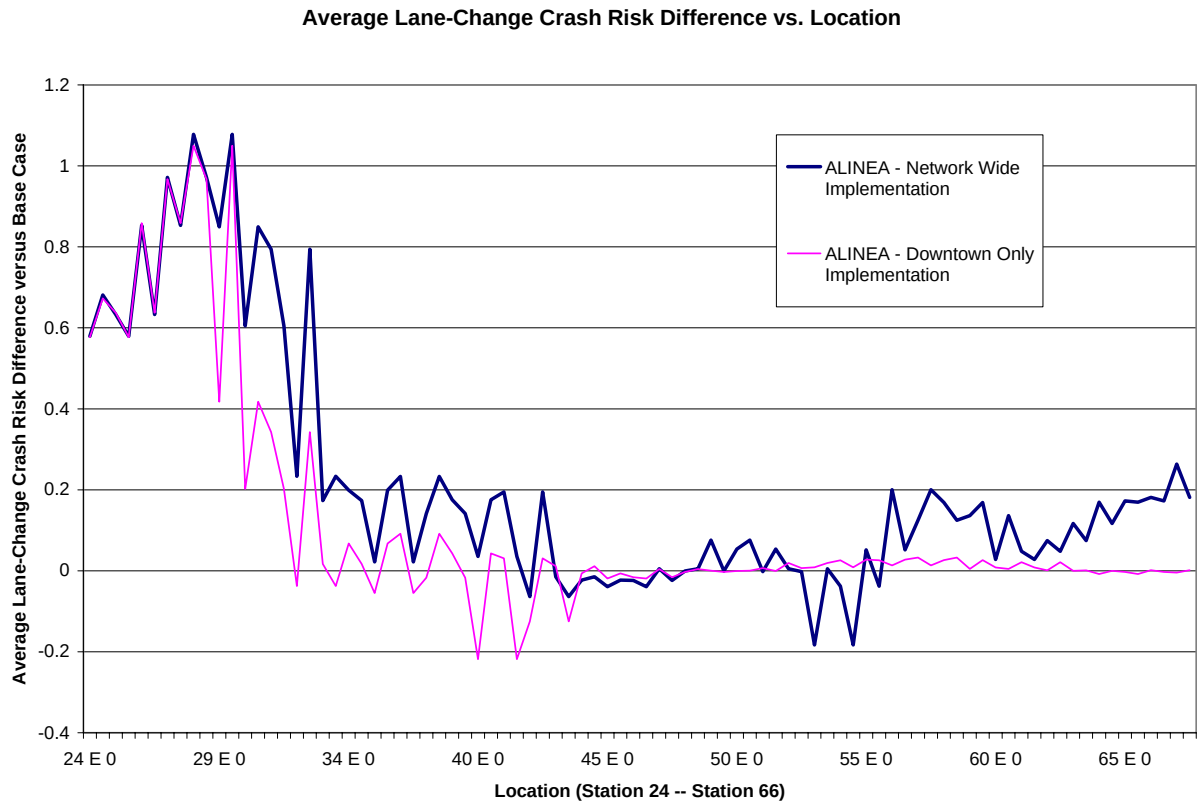


Figure 7-51: Average Lane-Change Crash Risk Difference vs. Location for Network-Wide Cases at 100 Percent Loading

As shown in Figures 7-50 and 7-51, at most locations the crash risk difference is greater for the network wide implementation than the downtown only implementation. This shows that the safety is improved when all of the ramps along Interstate-4 are metered as compared to when just the ramps in the downtown area are metered. What is interesting to note, however, is that the additional crash risk changes (both rear-end and lane-change) caused by the network wide implementation are only realized in the downtown area and downstream of this area (Stations 29 to 66). Stations 4 to 28 do not show a significant difference in the respective crash risk values between the network wide implementation and the downtown only implementation. One of the reasons for this

is that the crash risk values in this upstream area are already low. These low crash risk values are caused by the lower traffic volumes and higher speeds that occur in this area compared to other areas along the network. Additionally, the ramps in this area of the network are spaced further apart than the ramps in the downtown portion and just north of downtown. Therefore, when ramps are metered the effect isn't as magnified as when multiple, closer ramps are metered in succession in other areas of the network.

Looking at the real-time crash risk vs. time, it can be seen that this reduction in the crash risk occurs during the majority of the simulation run-time. This is shown in Figures 7-52 and 7-53 and confirms that the rear-end and lane-change crash risk, respectively, are reduced in real-time for most time periods when ramp metering is implemented. The time periods that show an increase in the crash risk have only a very small increase that can be deemed insignificant. Therefore, it is confirmed that at the 100 percent loading scenario implementing the ALINEA algorithm with the traffic-cycle realization throughout the entire network provides a better real-time safety benefit than by just metering the downtown ramps.

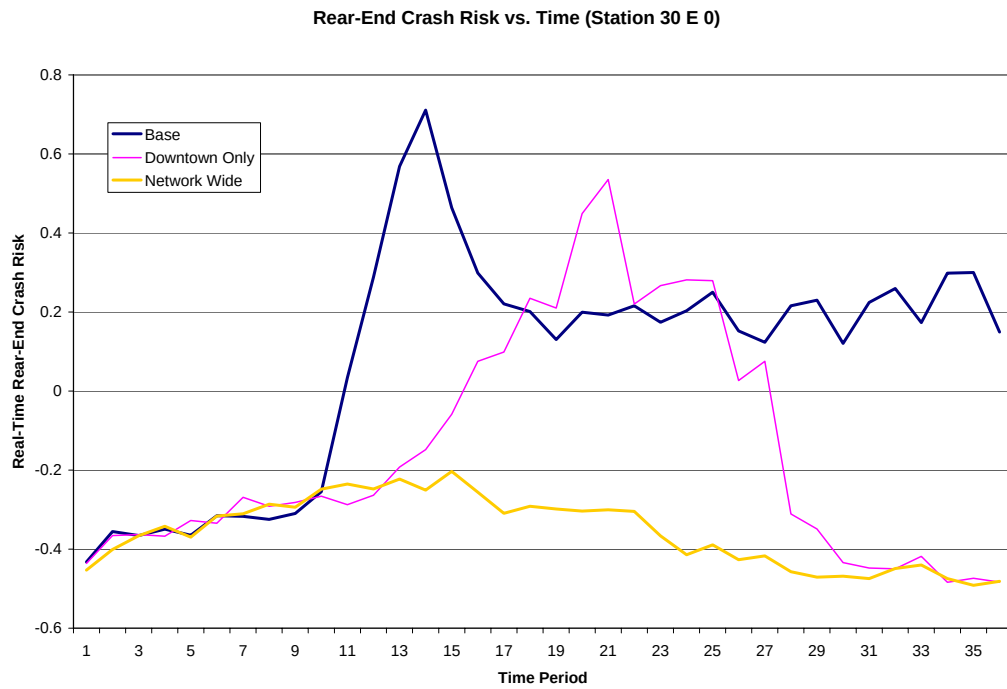


Figure 7-52: Plot of Rear-End Crash Risk vs. Time for Station 30 E 0 at 100 Percent Loading

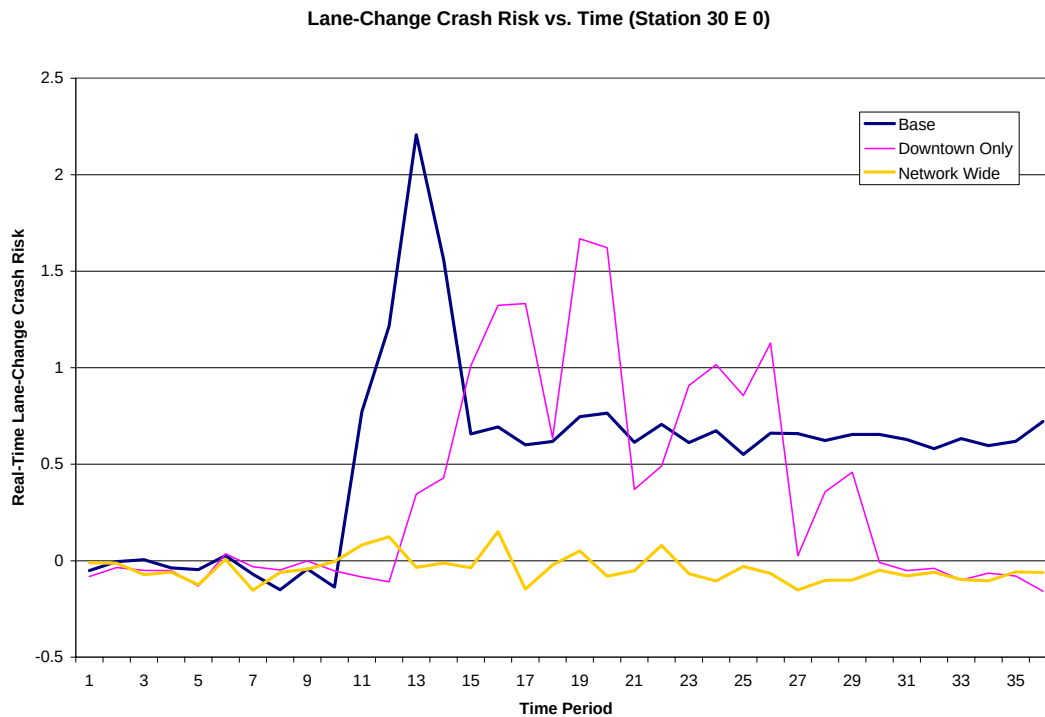


Figure 7-53: Plot of Lane-Change Crash Risk vs. Time for Station 30 E 0 at 100 Percent Loading

Additionally, implementing the ALINEA traffic-cycle strategy over the entire network serves to reduce the overall network travel time by 5%. When compared to the 2% reduction in travel time that is realized when ALINEA is implemented in the downtown area only, this means that implementing ALINEA network-wide provides both additional operational and safety benefits.

7.3.6.2 90 Percent Loading Scenario

In order to reduce the crash risk across the entire network at the 90 percent loading scenario, the best metering algorithm needs to be implemented in both the downtown and non-downtown regions throughout the network. However, unlike the 100

percent loading scenario, the 90 percent loading scenario did not have a clear-cut best algorithm. The ALINEA algorithm with a 30 second cycle length provided an excellent lane-change crash risk reduction and was the second best strategy at reducing the rear-end crash risk (as evidenced in Table 7-55). However, this strategy had very large crash migration effects that increased the crash risk greatly at some locations and even caused unreasonable travel time increases. The Zone algorithm with a 60 second cycle length had better rear-end crash risk reduction but also a less substantial lane-change crash risk benefit.

For this reason, Zone method was selected as the best and should be implemented throughout the network. However, because the ALINEA was ruled out in the downtown area mainly due to crash migration, there is the possibility that if implemented network-wide the effects of crash migration would be lessened and ALINEA would yield the best safety results. Therefore, both methods were tested throughout the entire network to determine the best network-wide metering case at the 90 percent loading scenario. Additionally, a combination of Zone in the downtown area and ALINEA in the non-downtown areas was tested to see if the safety could be improved while minimizing the effects of crash migration. A numerical summary of the results is presented below in Table 7-58.

Table 7-58: Summary of Network-Wide Ramp Metering Strategies at 100 Percent Loading

	Test Case			
	Zone Downtown Only	Zone Entire Network	Zone Downtown, ALINEA Elsewhere	ALINEA Entire Network
# of Stations with RE Risk Change	35	49	49	55
ORCI	6.9996	8.2178	9.0507	8.6979

# of Stations with LC Risk Change	24	77	63	63
LCRCI	4.1848	9.8322	11.5249	14.4103

From these results, it appears that applying the combination of Zone and ALINEA provides the best rear-end crash risk results while implementing the ALINEA algorithm alone throughout the network corridor improves the lane-change crash risk safety the most. In order to determine which strategy is the best overall strategy, the plot of the crash risk differences need to be examined to discover potential crash migration issues. The plots for the rear-end and lane-change crash risk differences are given below in Figures 7-54 and 7-55, respectively.

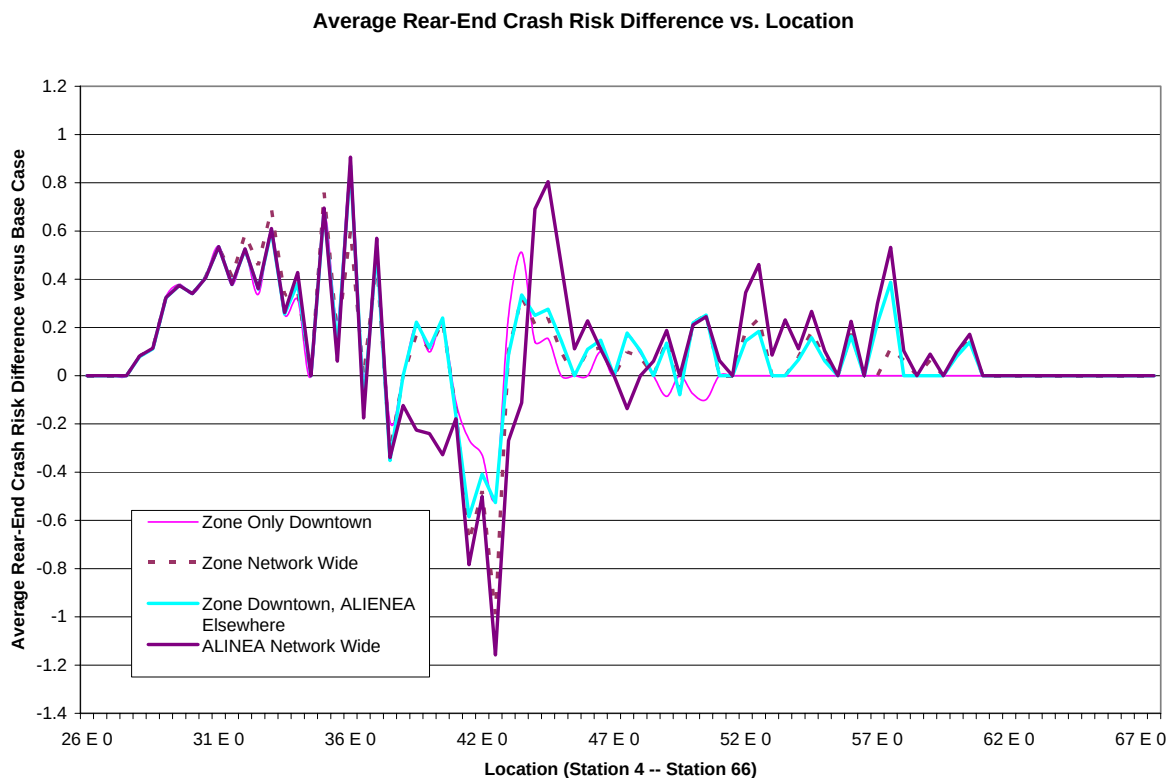


Figure 7-54: Average Rear-End Crash Risk Difference vs. Location for Network-Wide Cases at 90 Percent Loading

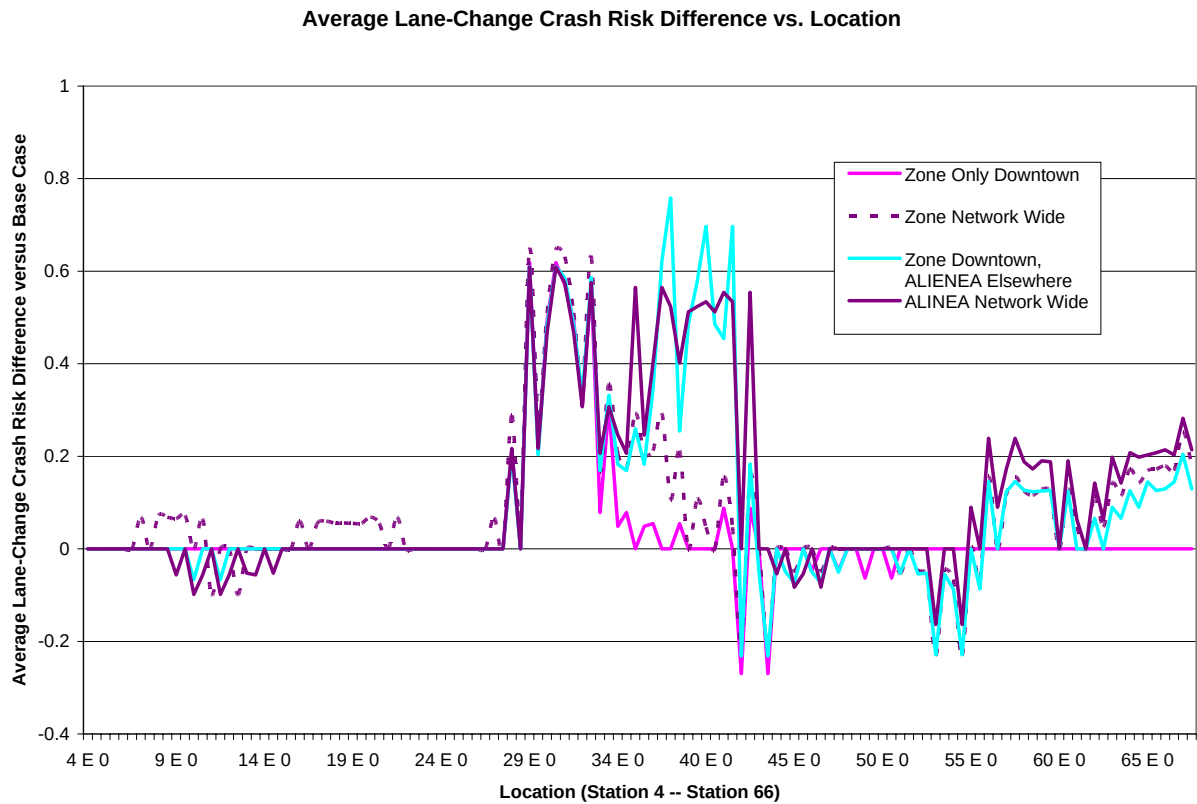


Figure 7-55: Average Lane-Change Crash Risk Difference vs. Location for Network-Wide Cases at 90 Percent Loading

Figure 7-54 shows that when the Zone and ALINEA algorithms are both implemented network-wide the effects of crash migration increase significantly at Stations 41 to 42. However, the combination of Zone and ALINEA, with the Zone algorithm implemented in the downtown areas and the ALINEA algorithm implemented in the non-downtown areas, decreases the effects of the crash migration to levels similar to the downtown only implementation of ramp metering. Figure 7-55, which shows the lane-change crash risk difference, shows that the effects of crash migration are about equal for each of the network-wide ramp metering implementations. Therefore, based on

the reduced crash migration effects as well as the high ORCI and LCRCI values, the best network-wide ramp metering strategy is determined to be the combination of Zone and ALINEA. Similar to the 100 percent loading scenarios, changes in the rear-end crash risk are mainly seen between Stations 28 through 62 even when all the ramps are metered. Although there are some lane-change crash risk changes upstream of this area (Stations 6 to 24) these differences are extremely small in magnitude and could be ignored.

Considering the change in the overall network travel time verifies the results of the best metering scenario. The overall network travel time is increased 3% when ALINEA is implemented network-wide. However, both the ALINEA and Zone combination as well as the Zone network-wide implementation yield a decrease in the overall network travel time of about 3%. Compared to the 0.5% reduction that is seen when just the Zone algorithm is implemented downtown, this shows that metering ramps throughout the entire corridor provides additional operational and safety benefits. Since the safety benefits are slightly higher for the Zone and ALINEA combination, this strategy is determined to be the best strategy at the 90 percent loading scenario.

7.4 Ramp Metering Summary

The general conclusions from the ramp metering portion of the experimental design are as follows:

- The implementation of ramp metering helps to reduce both the overall rear-end and lane-change crash risk across the network corridor.
- The implementation of ramp metering also causes crash risk migration. Although the extent of the crash migration is not large compared to the overall crash benefit (the increase in the crash risk is much smaller than the decrease in the crash risk)

it must still be noted. The area at which the crash risk is increased is usually an area that operated at mild levels of congestion in the base case and became congested due to the implementation of ramp metering. In most cases the migration effect was rather negligible due to already low crash risk values at that location. However, in some cases the increase was significant and, therefore, care must be taken before ramp metering can be implemented in that area.

- When using the ALINEA algorithm with the traffic-cycle realization, a lower critical occupancy (0.17) and shorter cycle length (30 seconds) performs the best at reducing the crash risk.
- When using the Zone ramp metering algorithm along with the traffic-cycle realization, a longer cycle length (60 seconds) provides the best results from a safety perspective.
- Using the one-car-per-cycle traffic realization method does not perform as well at reducing the rear-end and lane-change crash risk as using the traffic-cycle realization in the manner mentioned previously.
- The best ramp metering strategy at heavy congestion (100 percent loading) is to use the ALINEA algorithm with the traffic-cycle realization. The reason for this is that the ALINEA algorithm is more restrictive than the Zone algorithm during periods of heavy congestion. This occurs because the ALINEA algorithm does not account for spare capacity of the freeway to store vehicles like the Zone algorithm does (Equation 6-2). Therefore, as long as the detectors associated with on-ramps show congestion the meters will all operate at the minimum rates even if there is spare capacity within the metered section. Allowing less vehicles onto

the network will greatly reduce the overall crash risk on the network. When this method is applied to all the ramps in the network, not just the ones in the downtown area, the crash risk values and total network travel time are reduced even further.

- The best ramp metering strategy at lower levels of congestion (90 percent loading) is to use the Zone algorithm with the traffic-cycle realization. This occurs because the ALINEA algorithm causes unacceptable levels of crash migration during this loading scenario. However, neglecting to consider effects of crash migration, the ALINEA traffic-cycle algorithm also provides the good safety benefits at the 90 percent loading scenario. When ramp metering is applied to the entire network corridor at the 90 percent loading case, the best metering strategy is to employ the Zone algorithm in the downtown area and the ALINEA algorithm elsewhere. This provides a reduction in the two crash risk measures as well as the total network travel time.
- Of the three different zones that were metered (Zone 1, 2, and 3) the best results were found when Zone 3 was metered. This zone was the largest zone which encompassed the other two zones in their entirety and spanned the entire length of the downtown area of Interstate-4. Based on the results of this study, larger crash risk decreases are seen when a longer portion of the freeway is metered.
- When a large interchange (such as the I-4 / S. R. 408 Interchange) is present, metering the area just downstream of this interchange (Zone 2) provides much better safety benefits than metering the area just upstream (Zone 1).

7.5 Conclusions

This study has examined the potential of route diversion and ramp metering to be used as a real-time crash prevention technique. The objective of this study was to assess whether or not each of these ITS strategies would reduce the real-time rear-end and lane-change crash risk along an urban freeway and identify the most effective implementation methods and situations to employ them. To do this, Interstate-4 through Orlando, Florida was simulated in the PARAMICS micro-simulation program. Various implementations of route diversion and ramp metering were applied to the network and the crash risk values compared to determine the best cases. Multiple network loading scenarios were considered (60%, 80%, 90%, and 100%) to examine the effectiveness of the ITS measures at different levels of congestion. Additionally, the effect of the strategies on the network travel time was considered to ensure that the ITS strategies did not sacrifice operational capabilities of the traffic network in an attempt to improve the safety conditions.

The study found that route diversion successfully reduces both the rear-end and lane-change crash risk at periods of low traffic volumes (realized at the 60% and 80% network loading scenarios). In general, the further vehicles are diverted away from the initial diversion location, the more the crash risk values are reduced. Diverting more vehicles from a particular on-ramp increases the safety benefit. Therefore, vehicles should be diverted from on-ramps with higher demand volumes in order to maximize the safety benefits. At higher loading levels, although the crash risk is decreased at the diversion area, it is significantly increased at the location of the freeway that diverted vehicles re-enter the traffic stream. In order to minimize this crash migration effect,

vehicles should be diverted to locations with lower real-time crash risk values. In general, when vehicles are diverted during times of extreme congestion (100% loading) the overall safety benefit is an increase in the crash risk along the freeway.

Ramp metering is found to be beneficial at the higher loading scenarios (90% and 100% loading). Since the ramp metering algorithms are only activated when the freeway is congested, implementing ramp metering at the lower loading scenarios does not change the crash risk significantly along the network corridor. As found in this study, when the ramps in the downtown Orlando area are metered the best safety benefits during the 100% loading scenario is realized when the uncoordinated ALINEA metering algorithm is used. This occurs because ALINEA is generally more restrictive than the coordinated strategy tested – the Zone algorithm. At the 90% loading scenario, ALINEA causes large amount of crash migration downstream of the downtown area. Therefore, the coordinated Zone algorithm is preferred as it reduces the crash migration effects significantly. In both cases, the traffic-cycle realization of ramp metering yields significantly higher safety results than implementing the algorithms on a one-car-per-cycle basis.

When ramp metering is applied to the entire network, as opposed to just the extremely congested downtown areas, the crash risk values are shown to decrease for a longer section of the freeway. At the 100% loading scenario, applying ALINEA to all the ramps within the study area increases the overall safety benefits realized. At the 90% loading scenario, applying the Zone algorithm in the downtown areas and the ALINEA algorithm in the non-downtown areas serves to decrease the overall crash risk for the freeway corridor and reduces the crash risk for a longer section of the freeway. Also

found in this study is the fact that ALINEA yields the best safety results when a shorter cycle length of 30 seconds is used while the best benefits from the Zone algorithm are realized when a longer, 60 second, cycle length is applied.

7.6 Recommendations for Further Research

This research clearly shows the benefits of both route diversion and ramp metering on the real-time rear-end and lane-change crash risk on a typical urban freeway. However, there are several ways in which this research could be expanded. First, the transferability of this work needs to be examined. Route diversion and ramp metering should be tested on other freeways in different locations to ensure that the trends found in the results of this study hold for other regions as well. This will involve recalibrating the risk models that were used in this study using loop data taken from other freeways as the models used in this study are unique to Interstate-4. Once new models are created, the freeways used to calibrate the models should be simulated and the various route diversion and ramp metering strategies tested to confirm the trends found in this study. The transferability of the route diversion results could also be extended to this same freeway corridor by modeling the effects of other diversion routes through downtown Orlando. These routes would have to be feasible for drivers to use but would confirm that the general trends from this study are applicable to other diversions. Note that it is expected that the general trends from these results will hold on other freeways – mainly that more diverting vehicles further away and at lower loading scenarios will improve the safety conditions on the freeway. However, the exact results will not be transferable due to the uniqueness of geometry and ramp conditions on each freeway. Additionally, with the ramp metering strategies it is expected that the general trends of metering more ramps

and using the more restrictive algorithm (ALINEA) at higher loading conditions improves the safety conditions will hold true. The exact results, however, depend on different factors that define the uniqueness of each freeway.

Another way in which this research could be extended is to test the route diversion and ramp metering strategies in the field. While the micro-simulation gives researchers an idea of what should be expected when these strategies are implemented, it is possible that drivers will react to the strategies in a way that cannot be simulated. Therefore, the scenarios that show improved safety results could be implemented in the field to confirm that this is truly the case. Note that the strategies that show negative safety impacts should not be implemented in the field until there is evidence that they will not increase the crash risk. Only by field implementation will this be truly known.

Finally, combinations of route diversion and ramp metering could be tested. Previous work in Chapters 2 through 5 has shown that variable speed limits have potential for safety benefits. Therefore, combinations of these three ITS strategies could be examined to see if they will enhance the risk reducing potential of each strategy.

CHAPTER 8

EXPERIMENTAL DESIGN FOR CORRIDOR-WIDE VSL STRATEGIES

8.1 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).

8.1.1 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.

8.1.1.1 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 devisable 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.

8.1.1.2 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, 2006; Dilmore, 2005; Dhindsa, 2005). 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 road into homogeneous groups, based on average speed, and distinguishing them from other homogenous groups. The similarity

of these groups is determined by the difference in their 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 described by an average speed at each extent. This concept is illustrated in Figures 8-1 and 8-2. In these figures, the highway, traveled from left to right, are split into half-mile stations. So the station number and the distance covered between that station and the next are shown. The 5-minute average speed for each station is also shown below each station.

Direction of flow
→

Station	12	13	14	15	16	17	18	19	20	21	22	23	24
Average Speed	70	72	71	69	64	63	65	66	72	70	69	71	73

Figure 8-1: Typical Freeway Layout According to Stations

Direction of flow
→

Station	12	13	14	15	16	17	18	19	20	21	22	23	24
Average Speed	(70, 69)				(64, 66)				(72, 73)				

Figure 8-2: 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 8.1.2. 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 8-2, the freeway, which was once arbitrarily defined by 13 separate approximately 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 homogenous 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.

8.1.1.3 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 able to be changed back to normal.

8.1.2 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:

$$SpeedDiff = AvgSpeed_E - AvgSpeed_F$$

...Equation 8-1

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 8.1.3).

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 8.1.1) can be carried out.

8.1.3 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 II-G of this report.

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 8.1.1.3. 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.

8.2 PARAMICS Output and Network Loading Scenarios

The PARAMICS output and network loading scenarios used for final evaluations of variable speed limit strategies were the same as the ones used in Chapter 7. The details of the same may be found in sections 6.6 and 6.14 of this report, respectively.

8.3 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.

8.3.1 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*.

8.3.1.1 Speed Difference and Crash Risk

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.

$$(SpeedDiff = AvgSpeed_E - AvgSpeed_F)$$

Figure 8-3 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).

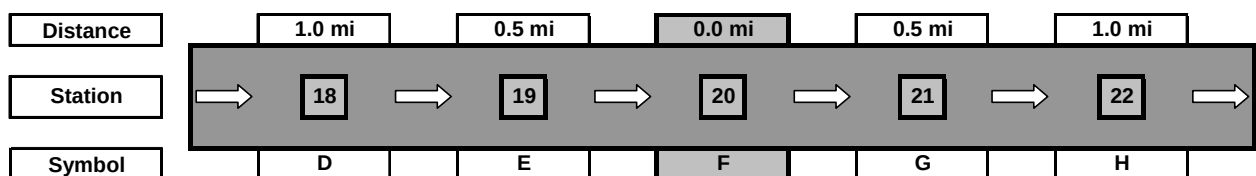


Figure 8-3: Schematic of Freeway Layout

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 Advanced 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 8.1.3. 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. 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.

8.3.1.2 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 8-1 shows the make up of scenarios for this experiment.

Table 8-1: 1st Experimental Design for Speed Difference Analysis

Scenario	Speed Difference	Speed Limit	
		Stations 4-19	Station 20
1	-20	35	55
2	-15	40	55
3	-10	45	55
4	-5	50	55
5	0	55	55
6	5	60	55
7	10	65	55
8	15	70	55

9	20	75	55

8.3.1.3 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 8-4 shows the average crash risk over time at the station of interest and surrounding stations for each speed difference scenario.

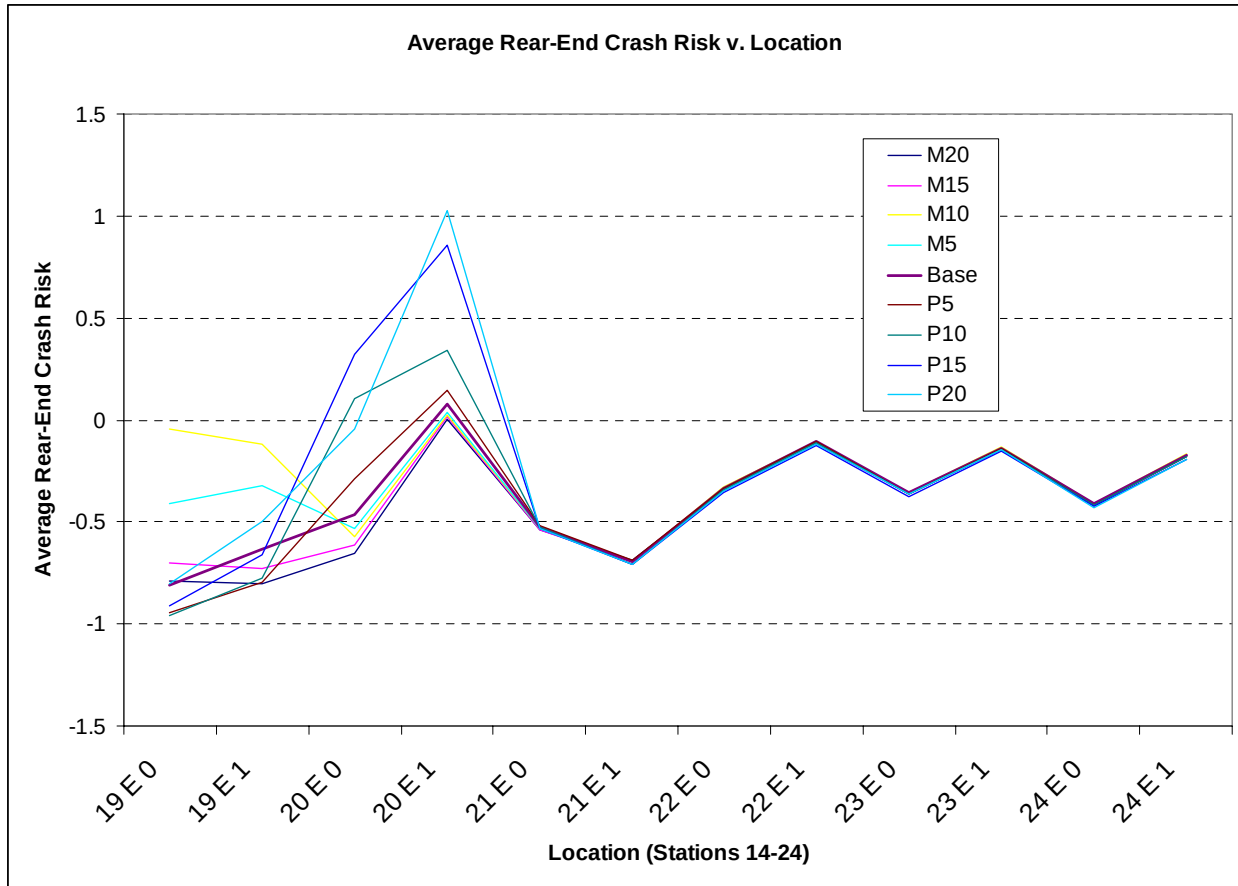


Figure 8-4: Average Rear-End Crash Risk v. Location for Location 1

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 8-5.

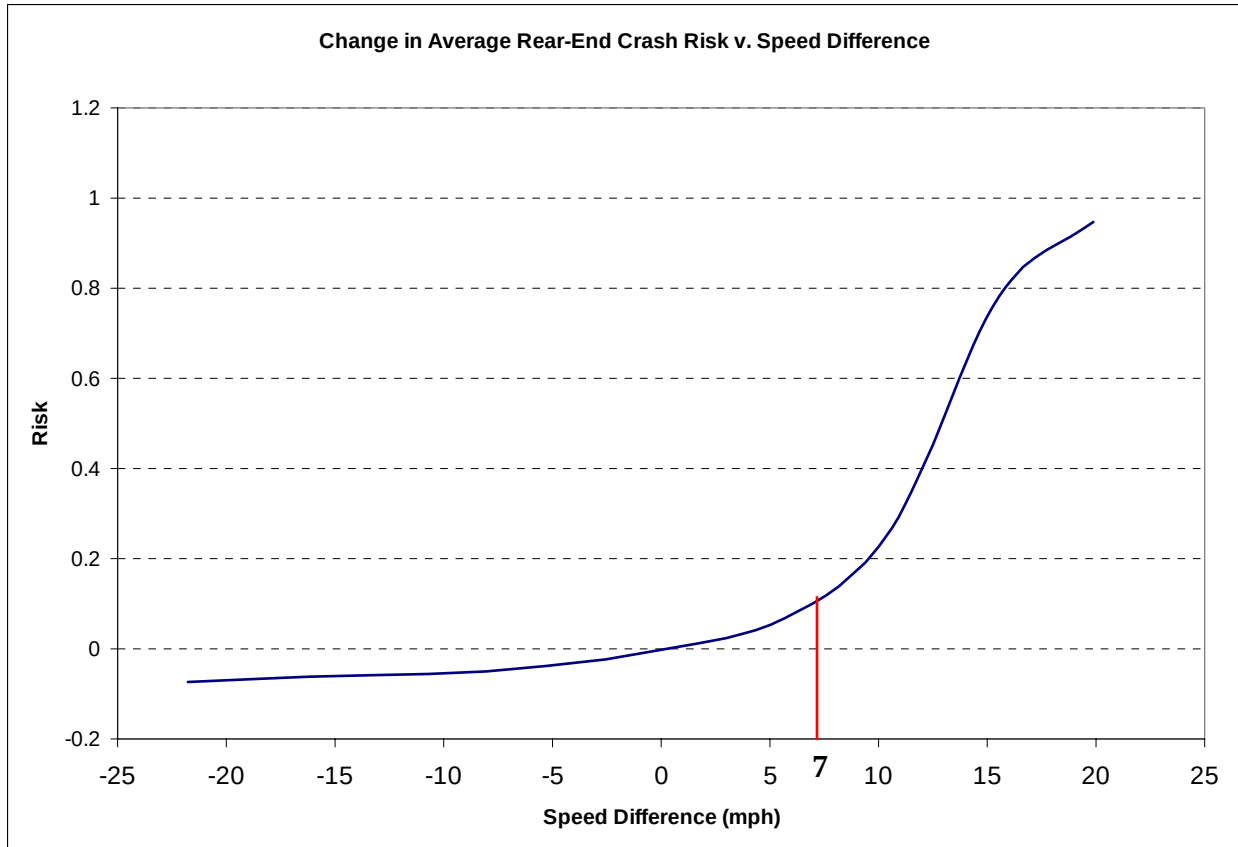


Figure 8-5: Change in Average Rear-End Crash Risk v. Speed Difference at Station 20 E 1

As seen in Figure 8-5, 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.

8.3.1.4 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 8-2 shows the make up of scenarios for this experiment.

Table 8-2: 2nd Experimental Design for Speed Difference Analysis

Scenario	Speed Difference	Speed Limit	
		Stations 35-46	Station 47
10	-20	30	50
11	-15	35	50
12	-10	40	50
13	-5	45	50
14	0	50	50
15	5	55	50
16	10	60	50
17	15	65	50
18	20	70	50

8.3.1.5 Exploratory Results: 2nd 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 8-6 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.

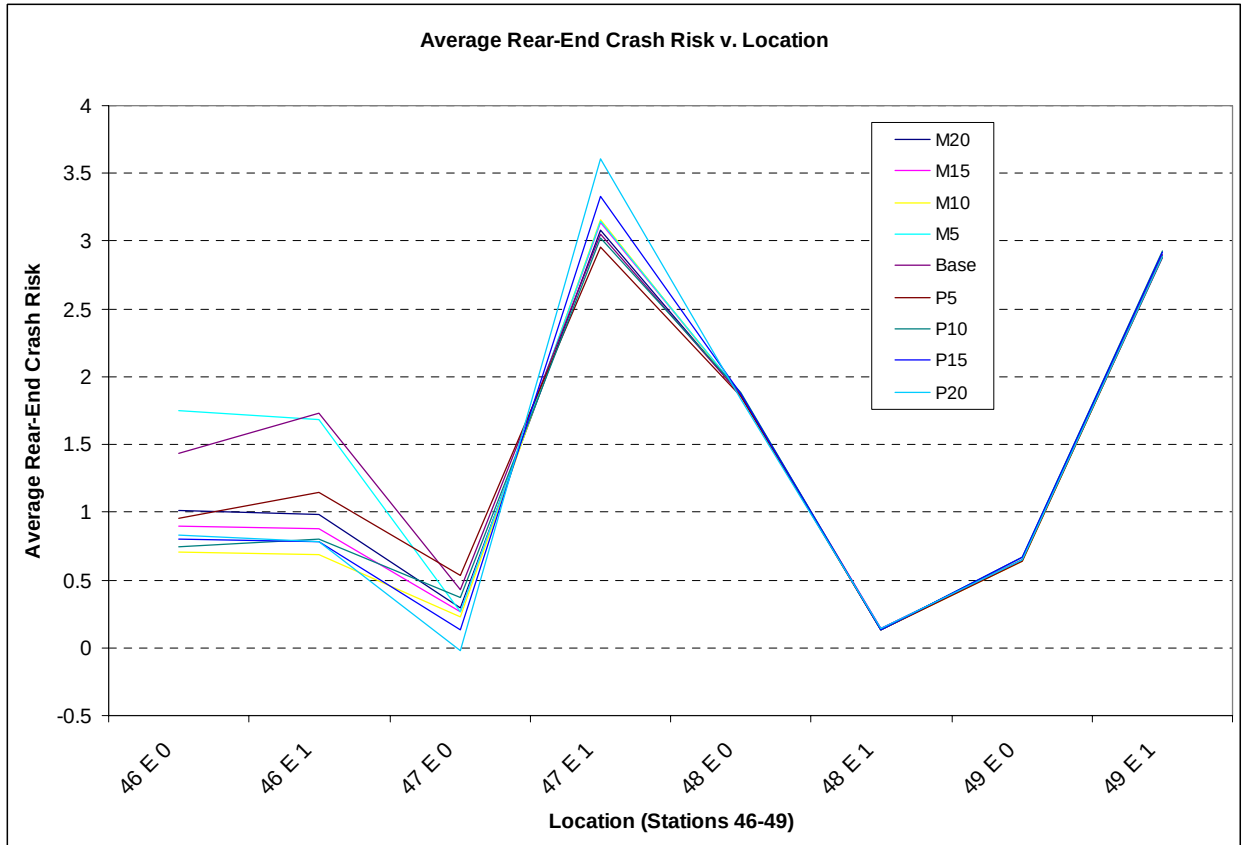


Figure 8-6: Average Rear-End Crash Risk v. Location for Location 2

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 8-7.

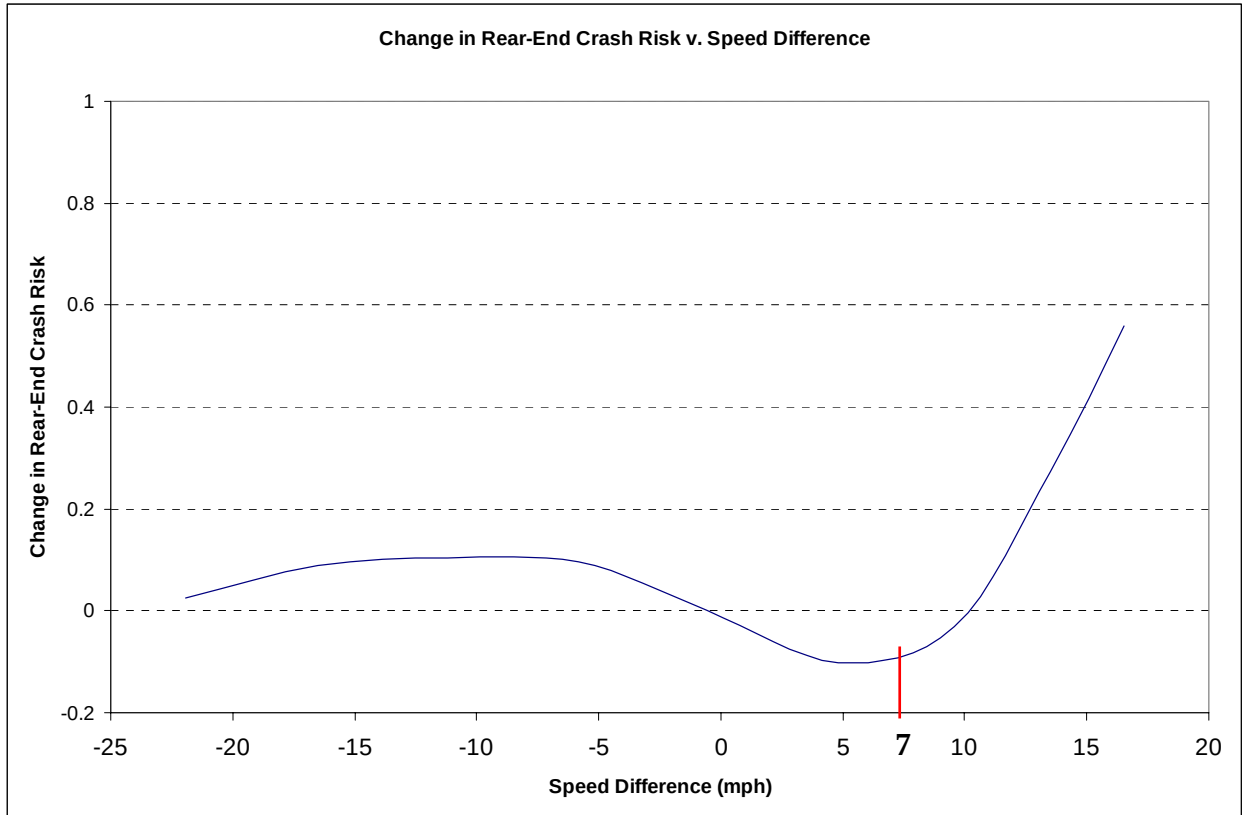


Figure 8-7: Average Rear-End Crash Risk v. Speed Difference at Station 47 E 1

The results from the figure are comparable to the results from Figure 8-5. 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.

8.3.1.6 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.

8.3.2 Experimental factors

In Section 8.1.1, 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 in 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 8-3.

Table 8-3: Layout of Experimental Design

Treatment ID	Speed Change Implementation	Speed Zone Threshold	Speed Change Distance	Speed Change Time Period
1	-10	5 mph	Speed Zone	5 min
2	-10	5 mph	Speed Zone	10 min
3	-10	5 mph	1/2 Speed Zone	5 min
4	-10	5 mph	1/2 Speed Zone	10 min
5	-10	2.5 mph	Speed Zone	5 min
6	-10	2.5 mph	Speed Zone	10 min
7	-10	2.5 mph	1/2 Speed Zone	5 min
8	-10	2.5 mph	1/2 Speed Zone	10 min
9	-5	5 mph	Speed Zone	5 min
10	-5	5 mph	Speed Zone	10 min
11	-5	5 mph	1/2 Speed Zone	5 min
12	-5	5 mph	1/2 Speed Zone	10 min
13	-5	2.5 mph	Speed Zone	5 min
14	-5	2.5 mph	Speed Zone	10 min
15	-5	2.5 mph	1/2 Speed Zone	5 min
16	-5	2.5 mph	1/2 Speed Zone	10 min
17	-5/+5	5 mph	Speed Zone	5 min
18	-5/+5	5 mph	Speed Zone	10 min
19	-5/+5	5 mph	1/2 Speed Zone	5 min
20	-5/+5	5 mph	1/2 Speed Zone	10 min
21	-5/+5	2.5 mph	Speed Zone	5 min
22	-5/+5	2.5 mph	Speed Zone	10 min
23	-5/+5	2.5 mph	1/2 Speed Zone	5 min
24	-5/+5	2.5 mph	1/2 Speed Zone	10 min

Figure 8-8 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. 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.

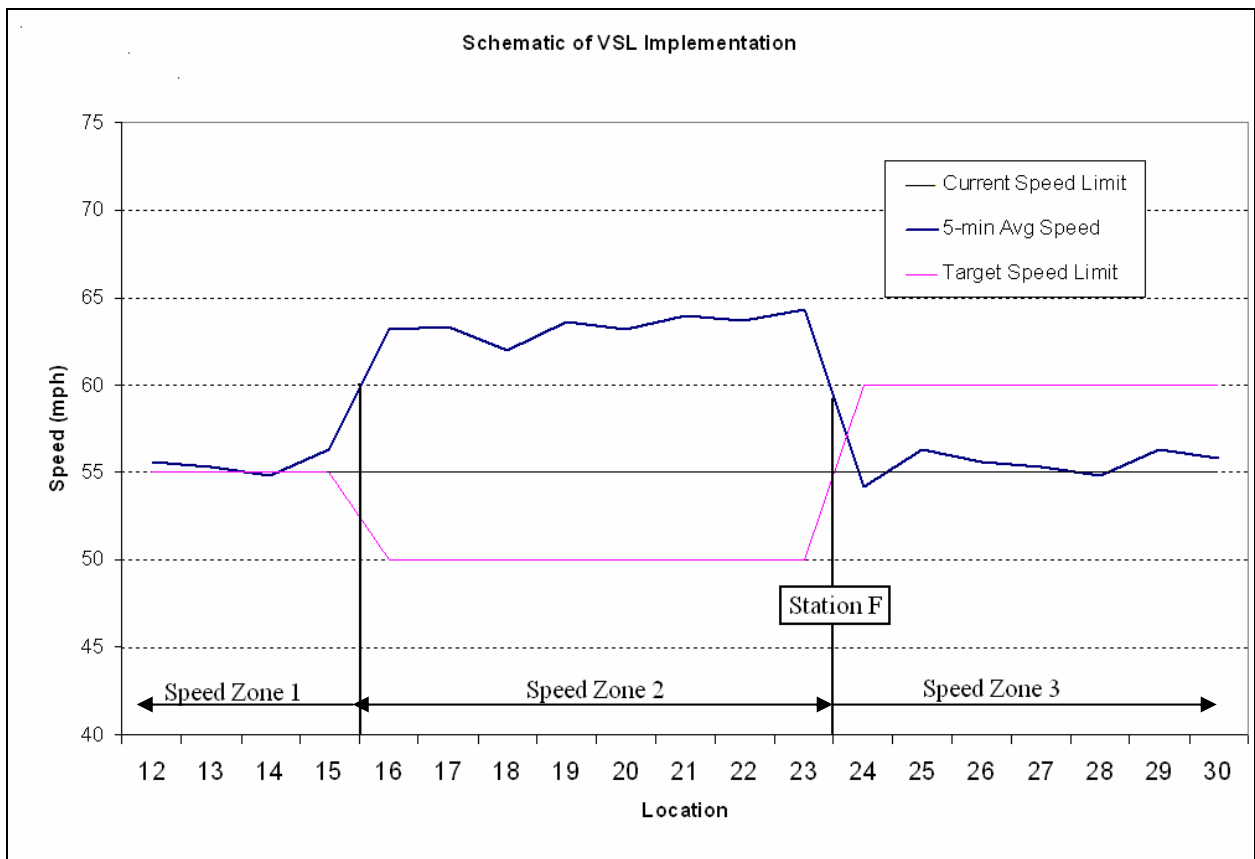


Figure 8-8: Example of VSL Implementation on Interstate-4

This experimental design of 24 treatments was applied to each of the volume loading blocks described earlier, 60%, 80% and 90%. 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.

8.4 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:

$$N \approx \left(KS/E \right)^2$$

...Equation 8-2

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 9

RESULTS FROM EVALUATIONS OF CORRIDOR-WIDE VSL STRATEGIES

9.1 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 9-1, which is a plot of the upstream rear-end crash risk for 5 replications of Treatment 05 at 60% volume loading. As noted in Chapter 6 (Also see Gayah, 2006), these posterior probabilities are obtained from the neural networks 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 9-1 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.

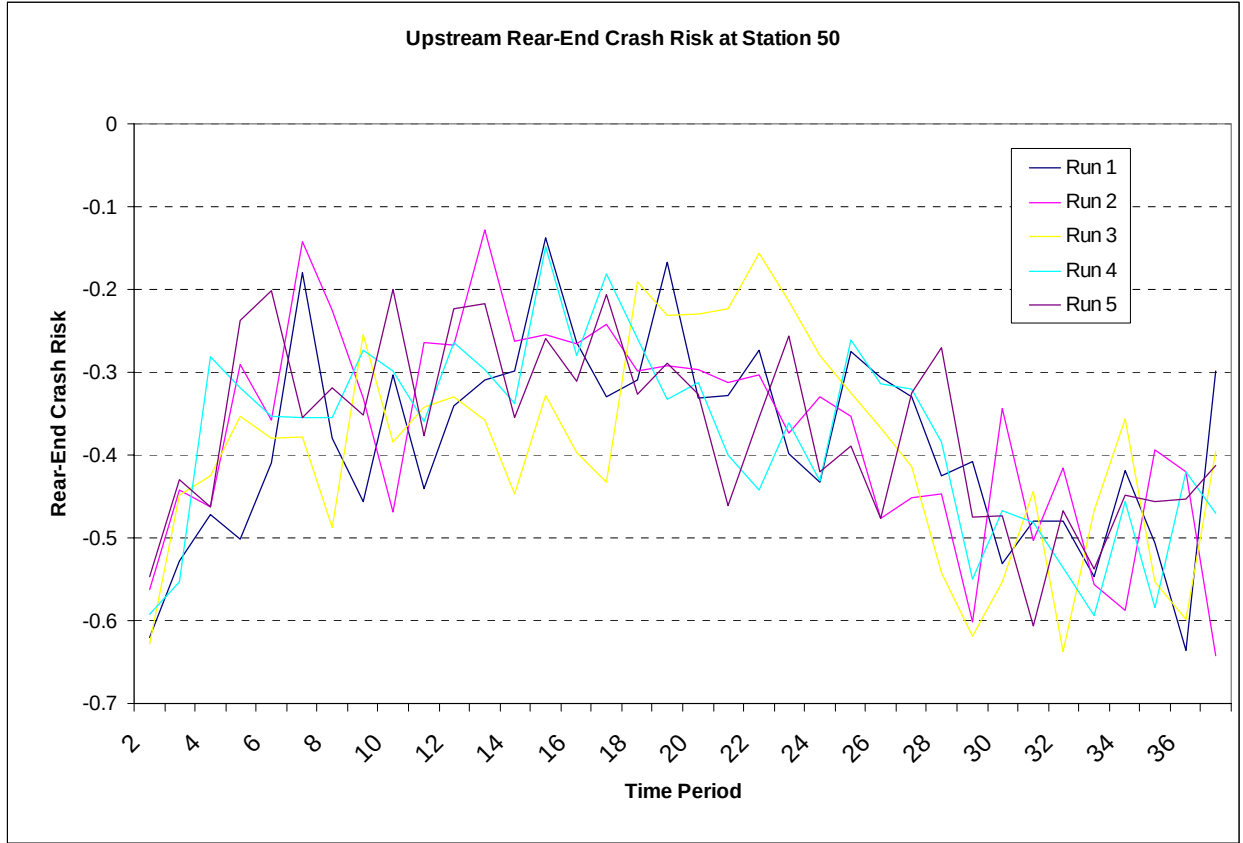


Figure 9-1: Crash Risk vs. Time at Station 50 E 0 for All Treatment 05 Runs at 60% Loading

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.

$$(Risk\_profile)_{tl} = \frac{1}{N} \sum_{r=1}^N (Risk)_{trl}$$

Where $(Risk)_{t|l}$ 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; $(Risk\_profile)_{t|l}$ 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 9-2. 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.

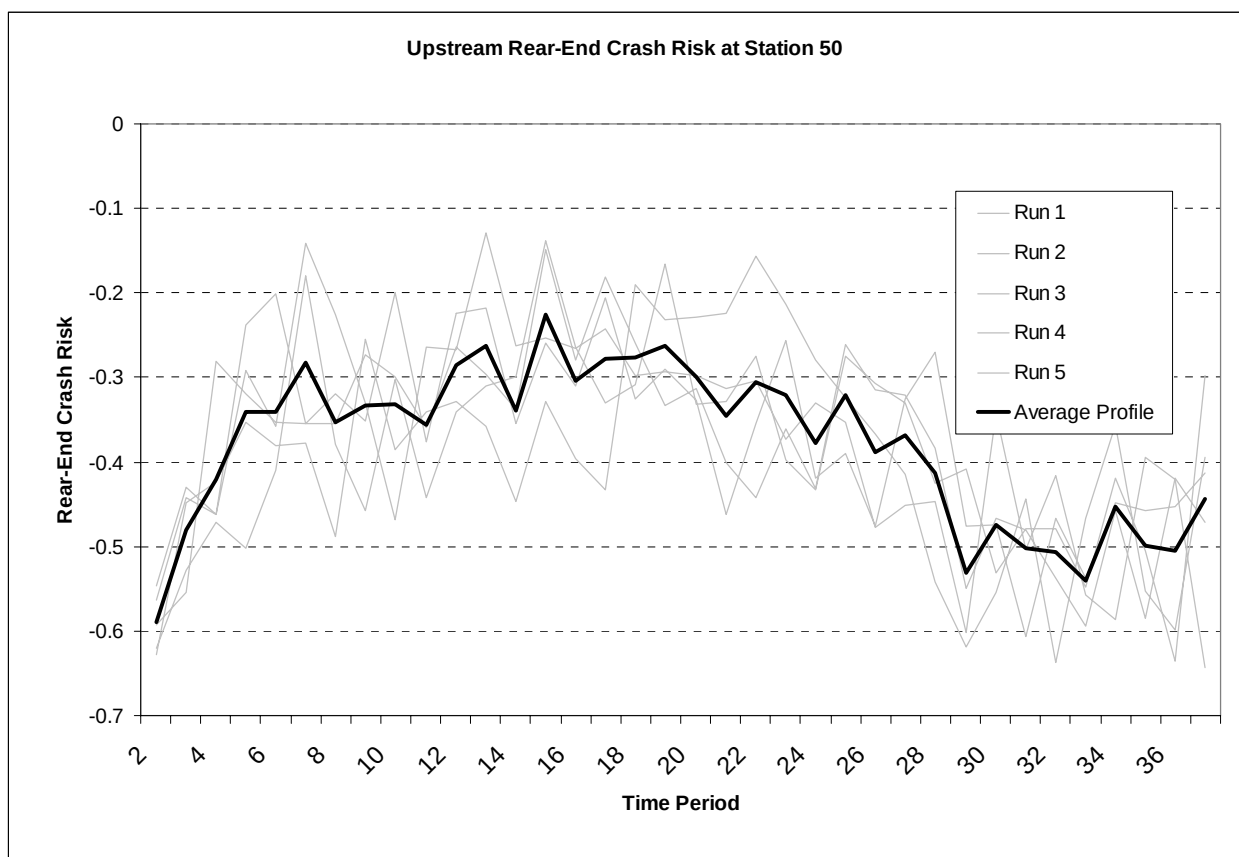


Figure 9-2: Average Crash Risk Profile at Station 50 E 0 for Treatment 05 at 60% Loading

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.

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

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 9-3 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.

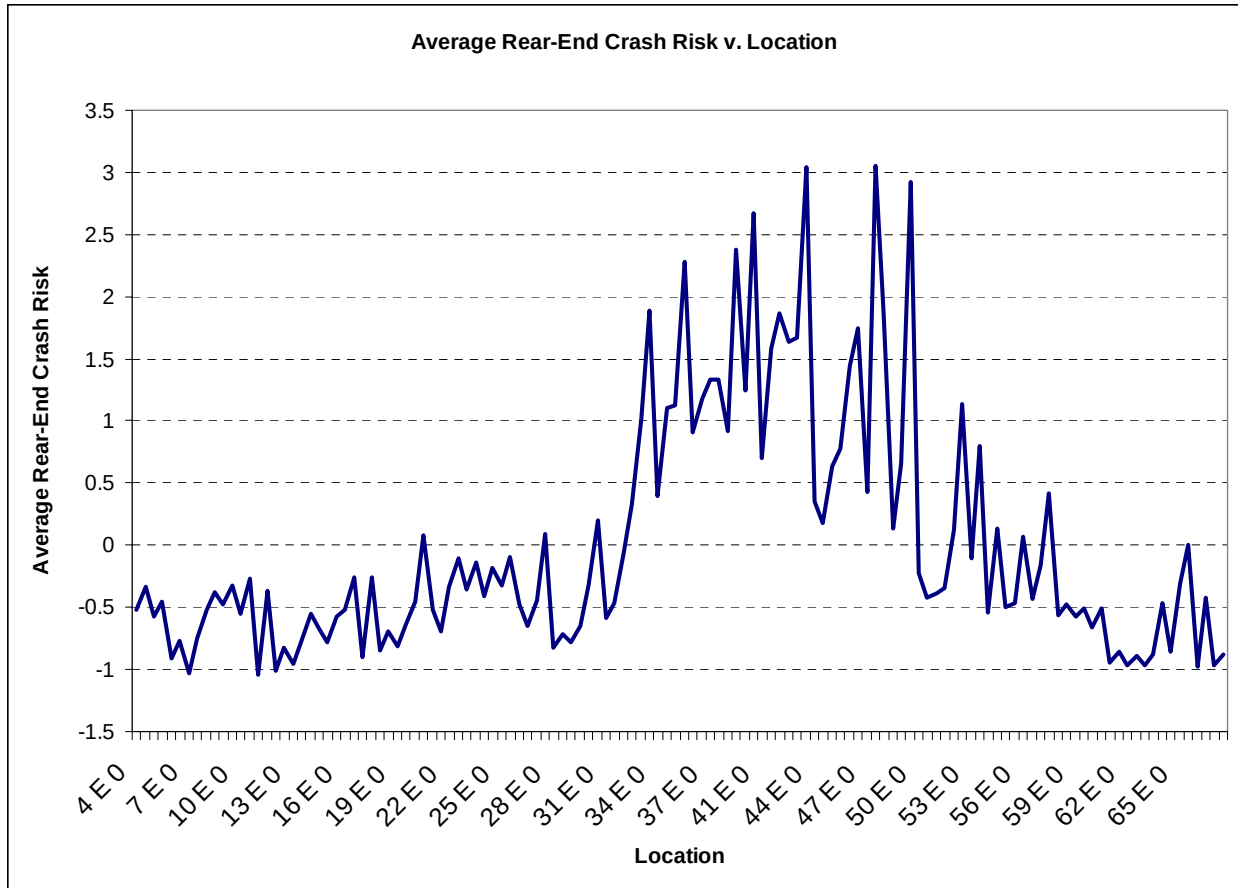


Figure 9-3: Average Rear-End Crash Risk v. Location for BASE Case at 60% Loading

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.

9.2 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.

9.2.1 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 9-1 displays the different factors that make up Treatments 01-08, which are reviewed in this section.

Table 9-1 Description of Treatments 01-08

Treatment ID	Speed Change Implementation	Speed Zone Threshold	Speed Change Distance	Speed Change Time Period
1	-10	5 mph	Speed Zone	5 min
2	-10	5 mph	Speed Zone	10 min
3	-10	5 mph	1/2 Speed Zone	5 min
4	-10	5 mph	1/2 Speed Zone	10 min
5	-10	2.5 mph	Speed Zone	5 min
6	-10	2.5 mph	Speed Zone	10 min
7	-10	2.5 mph	1/2 Speed Zone	5 min
8	-10	2.5 mph	1/2 Speed Zone	10 min

9.2.1.1 Rear-End Crash Risk

In the previous chapter, describing the 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.

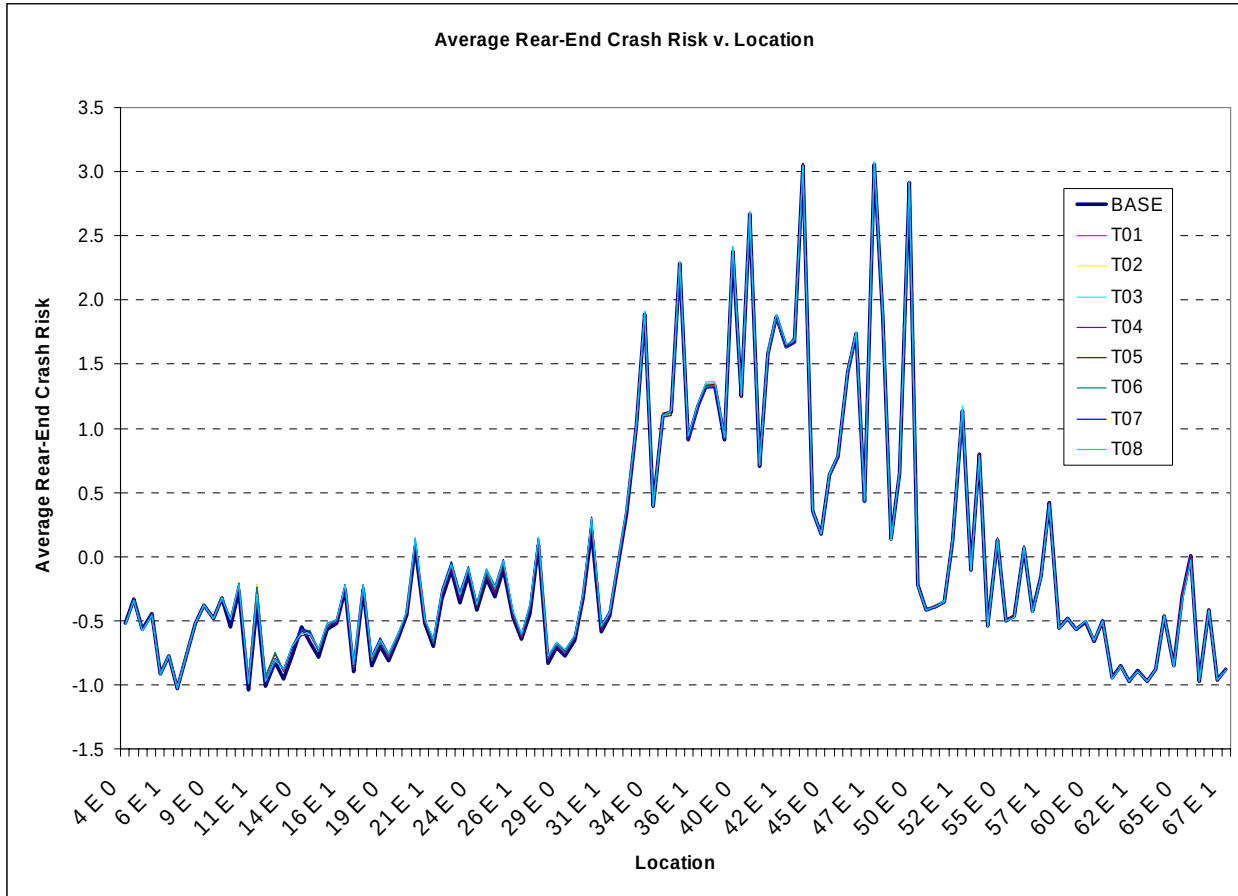


Figure 9-4: Average Rear-End Crash Risk v. Location for Treatments 01-08

Figure 9-4 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 9-5 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.

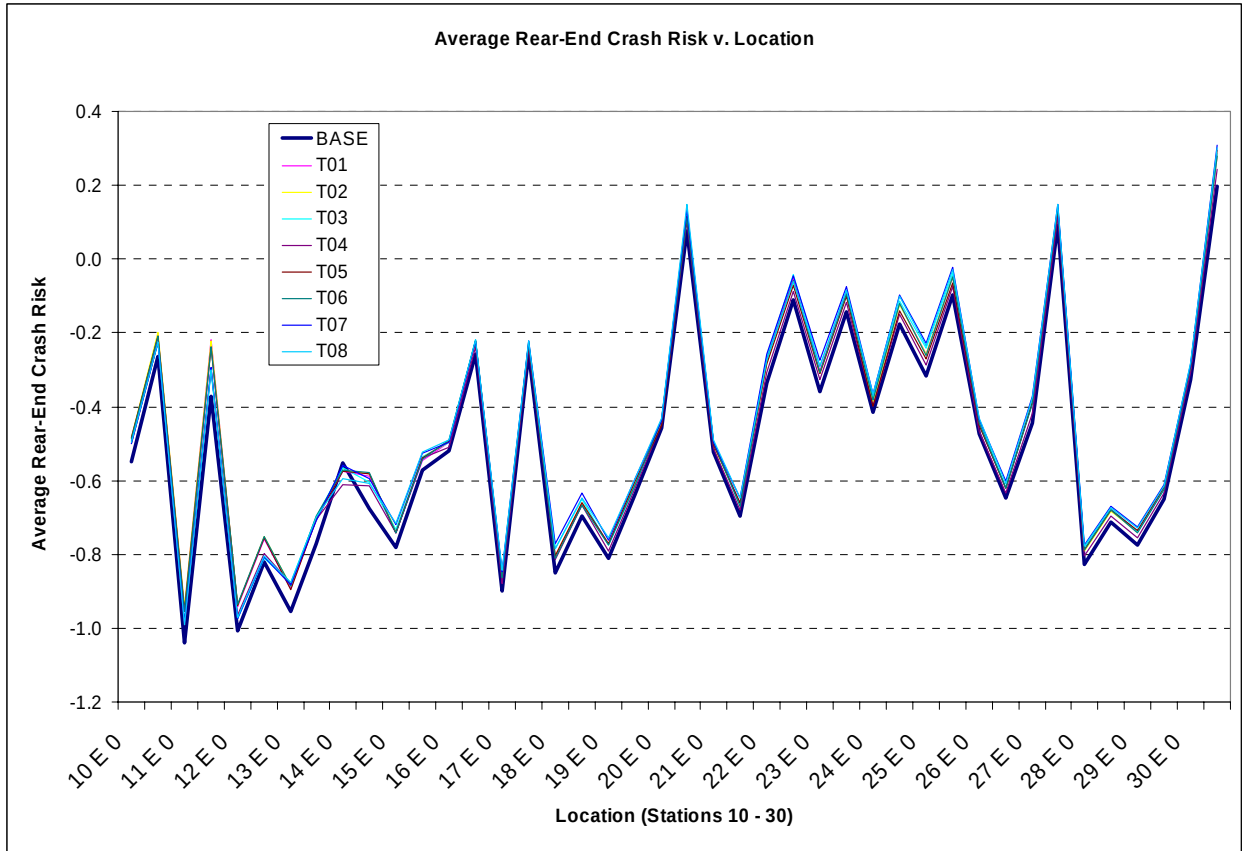


Figure 9-5: Average Rear-End Crash Risk v. Location for Treatments 01–08 - Magnified

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 9-2 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 9-2: Summary of Difference in Rear-End Crash Risk for Treatments 01-08

	Test Case ID								
	Base	T01	T02	T03	T04	T05	T06	T07	T08
Average Crash Risk (Stations 10 to 30)	-0.510	-0.462	-0.462	-0.463	-0.483	-0.465	-0.463	-0.456	-0.459
Crash Risk Benefit	---	-0.048	-0.047	-0.047	-0.027	-0.045	-0.046	-0.054	-0.051
T-Statistic (Significance of Change)	---	0.710	0.710	0.694	0.405	0.672	0.691	0.795	0.756
ORCI	---	-1.997	-1.989	-1.962	-1.130	-1.879	-1.938	-2.253	-2.139

9.2.1.2 Lane-Change Crash Risk

The difference in lane-change crash risk, due to the effects of Treatments 01-08, was also evaluated. Figure 9-6 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.

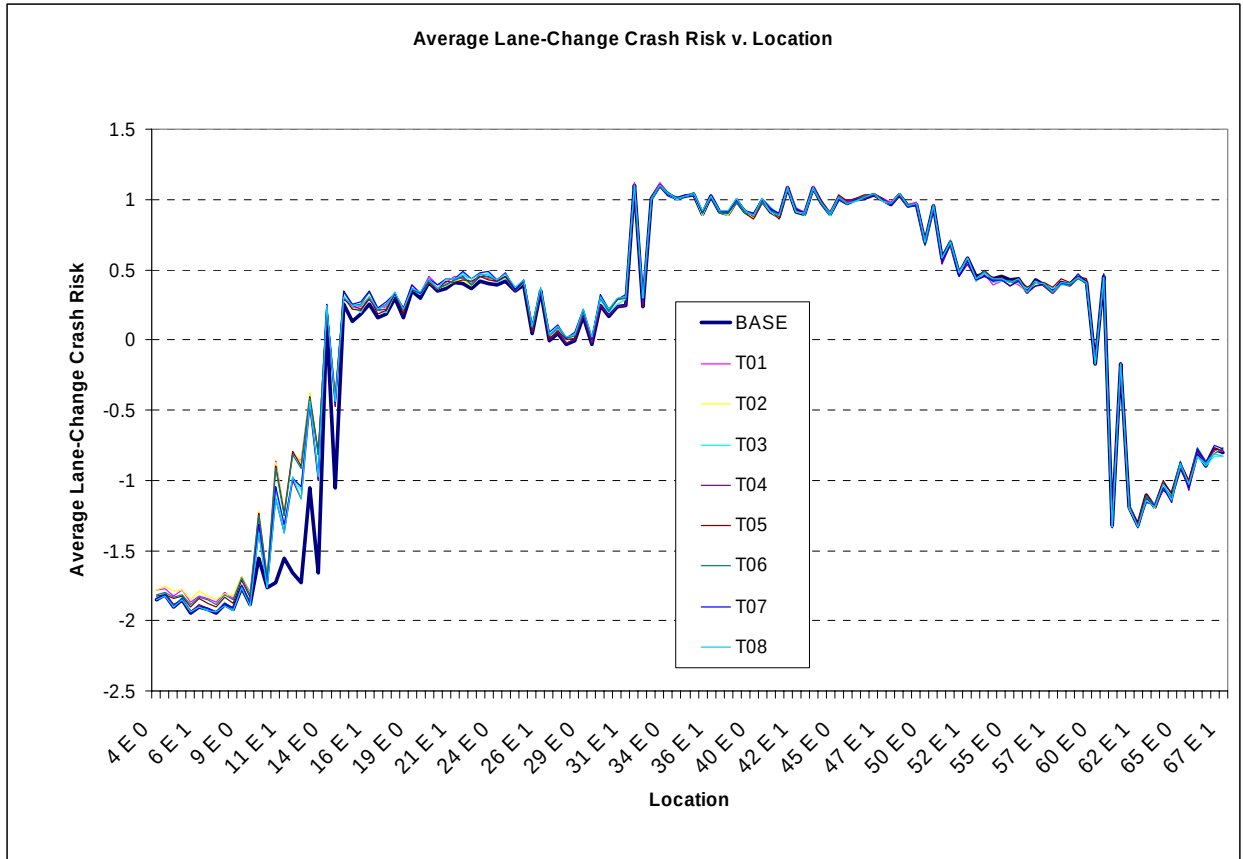


Figure 9-6: Average Lane-Change Crash Risk v. Location for Treatments 01-08

The most effected area is located between stations 10 and 14. As shown in Figure 9-7, 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.

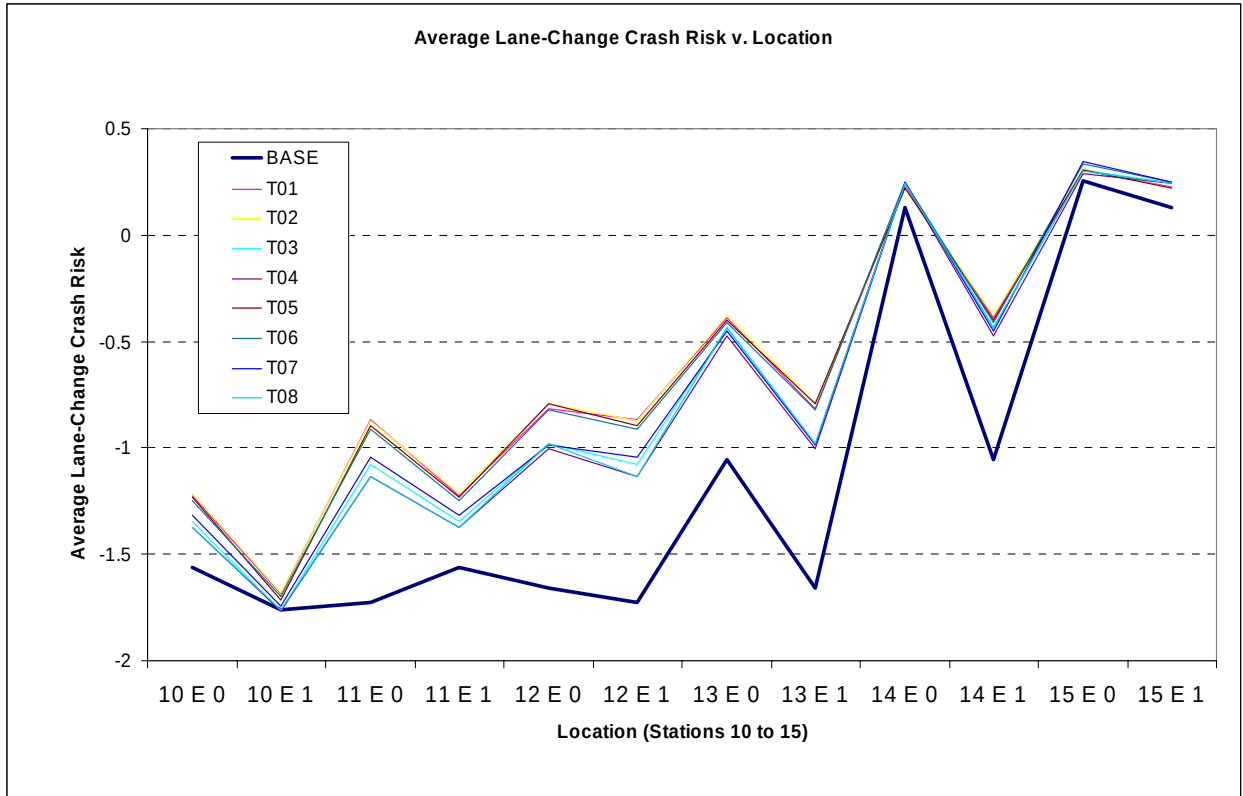


Figure 9-7: Average Lane-Change Crash Risk v. Location for Treatments 01–08 - Magnified

Table 9-3 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 9-3: Summary of Difference in Lane-Change Crash Risk for Treatments 01–08

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

9.2.2 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 9-4 displays the different factors that make up Treatments 09-16, which are reviewed in this section.

Table 9-4: Description of Treatments 09-16

Treatment ID	Speed Change Implementation	Speed Zone Threshold	Speed Change Distance	Speed Change Time Period
9	-5	5 mph	Speed Zone	5 min
10	-5	5 mph	Speed Zone	10 min
11	-5	5 mph	1/2 Speed Zone	5 min
12	-5	5 mph	1/2 Speed Zone	10 min
13	-5	2.5 mph	Speed Zone	5 min
14	-5	2.5 mph	Speed Zone	10 min
15	-5	2.5 mph	1/2 Speed Zone	5 min
16	-5	2.5 mph	1/2 Speed Zone	10 min

9.2.2.1 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 9-8 shows a plot of the average rear-end crash risk v. location for the entire network, comparing the BASE case to Treatments 09-16.

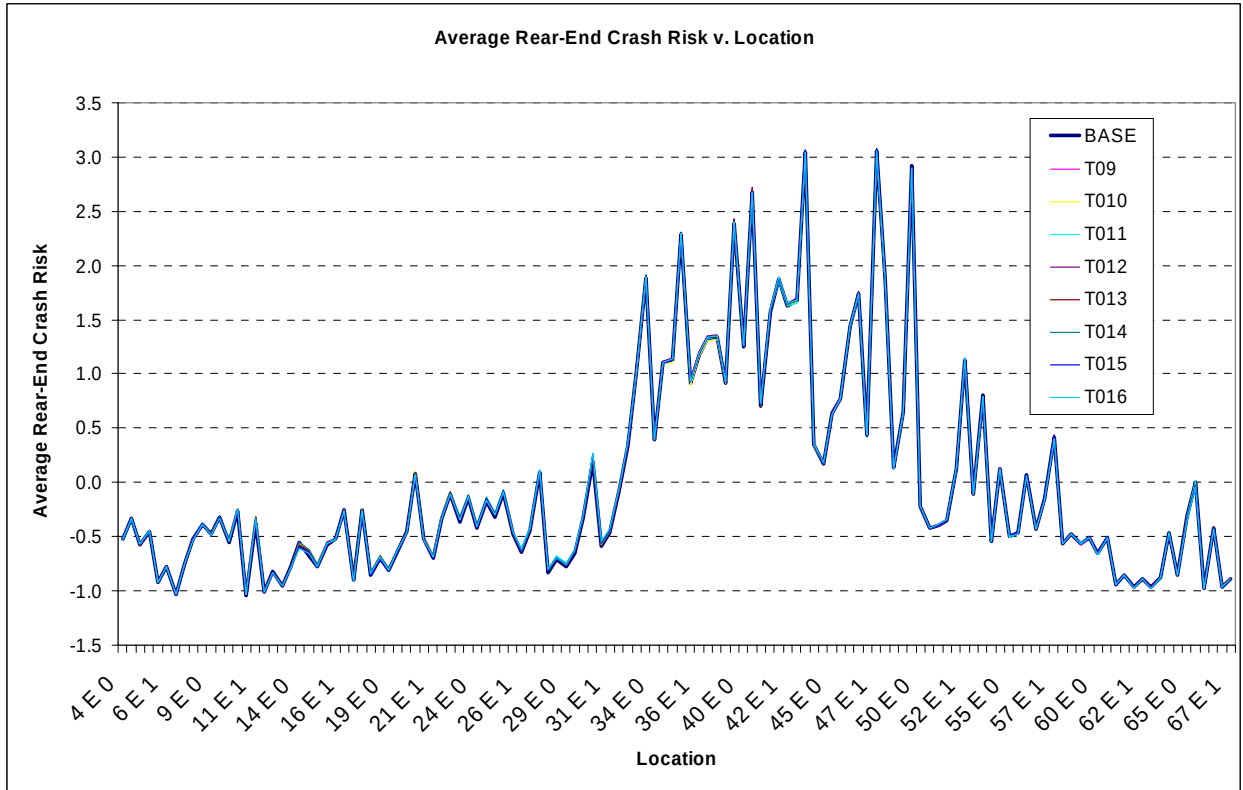


Figure 9-8: Average Rear-End Crash Risk v. Location for Treatments 09–16

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 9-9. Even in the magnified figure, it is hard to see a substantial difference in average crash risk.

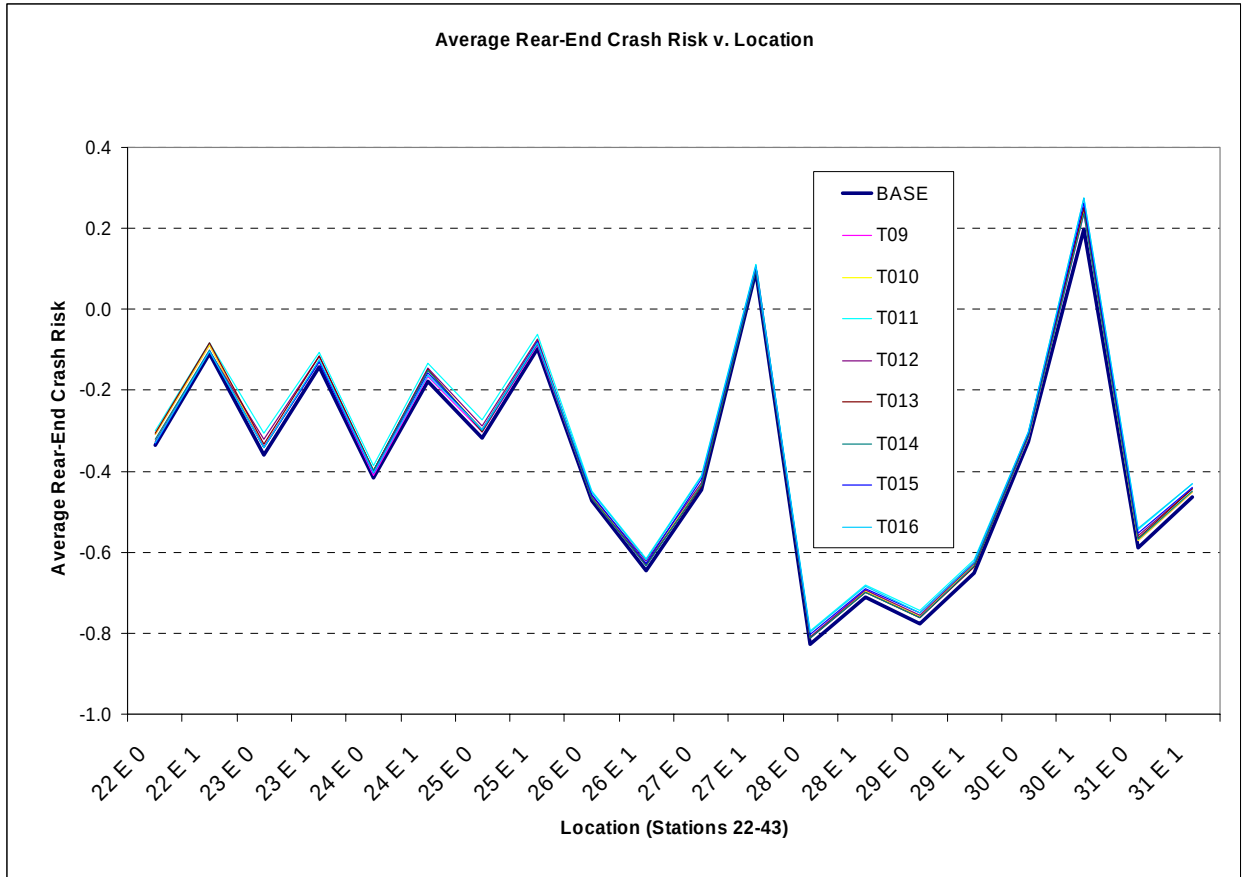


Figure 9-9: Average Rear-End Crash Risk v. Location for Treatments 09–16 - Magnified

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 9-5. 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 9-5: Summary of Difference in Rear-End Crash Risk for Treatments 09–16

	Test Case ID								
	Base	T09	T10	T11	T12	T13	T14	T15	T16
Average Crash Risk (Stations 22 to 31)	-0.379	-0.363	-0.360	-0.343	-0.356	-0.359	-0.363	-0.357	-0.354
Crash Risk Harm	---	-0.016	-0.019	-0.036	-0.022	-0.019	-0.016	-0.021	-0.024
T-Statistic (Significance of Change)	---	0.193	0.229	0.439	0.274	0.238	0.193	0.262	0.298
ORCI	---	-0.375	-0.448	-0.859	-0.534	-0.467	-0.376	-0.511	-0.580

9.2.2.2 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 9-10 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.

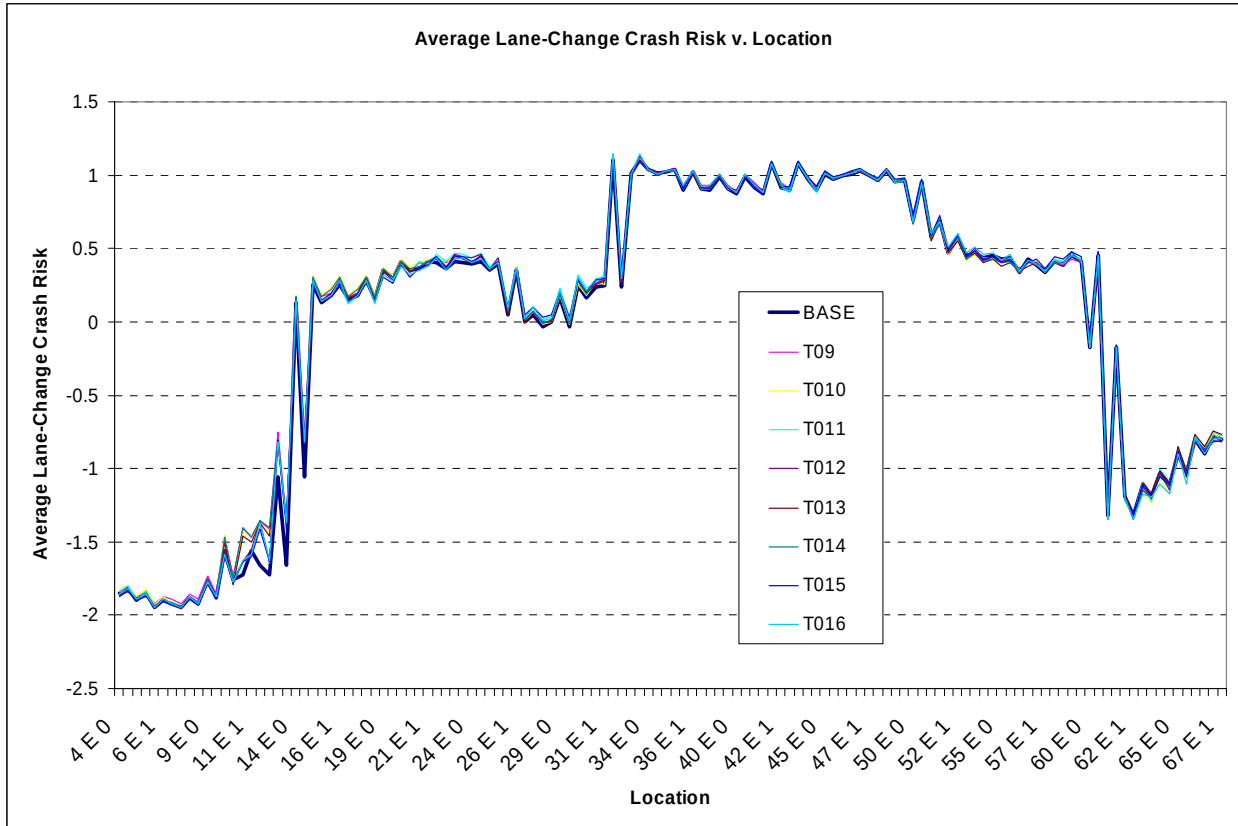


Figure 9-10: Average Lane-Change Crash Risk v. Location for Treatments 09–16

The segment most affected by these treatments is located between stations 11 and 14. Figure 9-11 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.

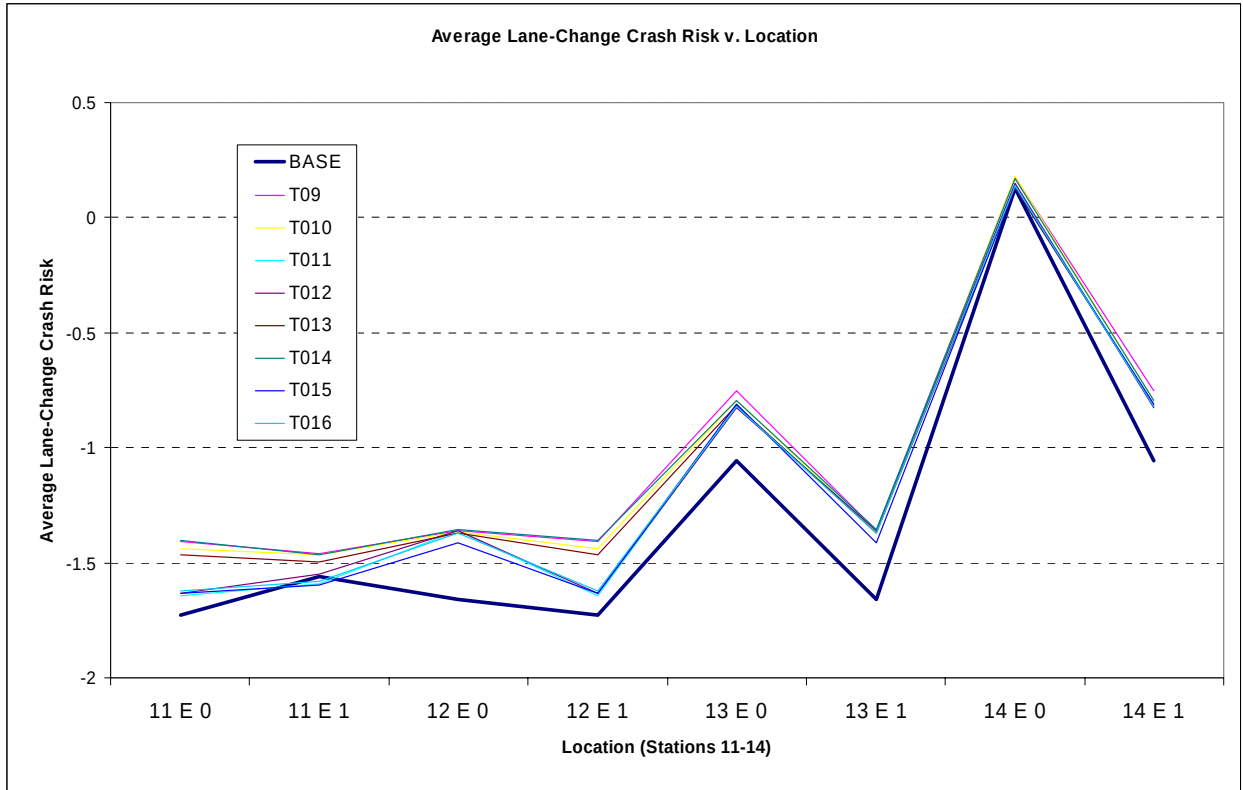


Figure 9-11: Average Lane-Change Crash Risk v. Location for Treatments 09–16 - Magnified

Table 9-6 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 9-6: Summary of Difference in Lane-Change Crash Risk for Treatments 09–
16**

	Test Case ID								
	Base	T09	T10	T11	T12	T13	T14	T15	T16
Average Crash Risk (Stations 22 to 43)	0.547	0.563	0.551	0.567	0.571	0.561	0.557	0.561	0.567
Crash Risk Harm	---	-0.015	-0.004	-0.019	-0.024	-0.014	-0.010	-0.014	-0.019
T-Statistic (Significance of Change)	---	0.066	0.018	0.084	0.103	0.061	0.042	0.060	0.084
ORCI	---	-0.645	-0.177	-0.819	-1.003	-0.592	-0.408	-0.587	-0.817

9.2.3 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 9-7 displays the different factors that make up Treatments 17-24, which are reviewed in this section.

Table 9-7: Description of Treatments 17-24

Treatment ID	Speed Change Implementation	Speed Zone Threshold	Speed Change Distance	Speed Change Time Period
17	-5/+5	5 mph	Speed Zone	5 min
18	-5/+5	5 mph	Speed Zone	10 min
19	-5/+5	5 mph	1/2 Speed Zone	5 min
20	-5/+5	5 mph	1/2 Speed Zone	10 min
21	-5/+5	2.5 mph	Speed Zone	5 min
22	-5/+5	2.5 mph	Speed Zone	10 min
23	-5/+5	2.5 mph	1/2 Speed Zone	5 min
24	-5/+5	2.5 mph	1/2 Speed Zone	10 min

9.2.3.1 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 9-12 and 9-13 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.

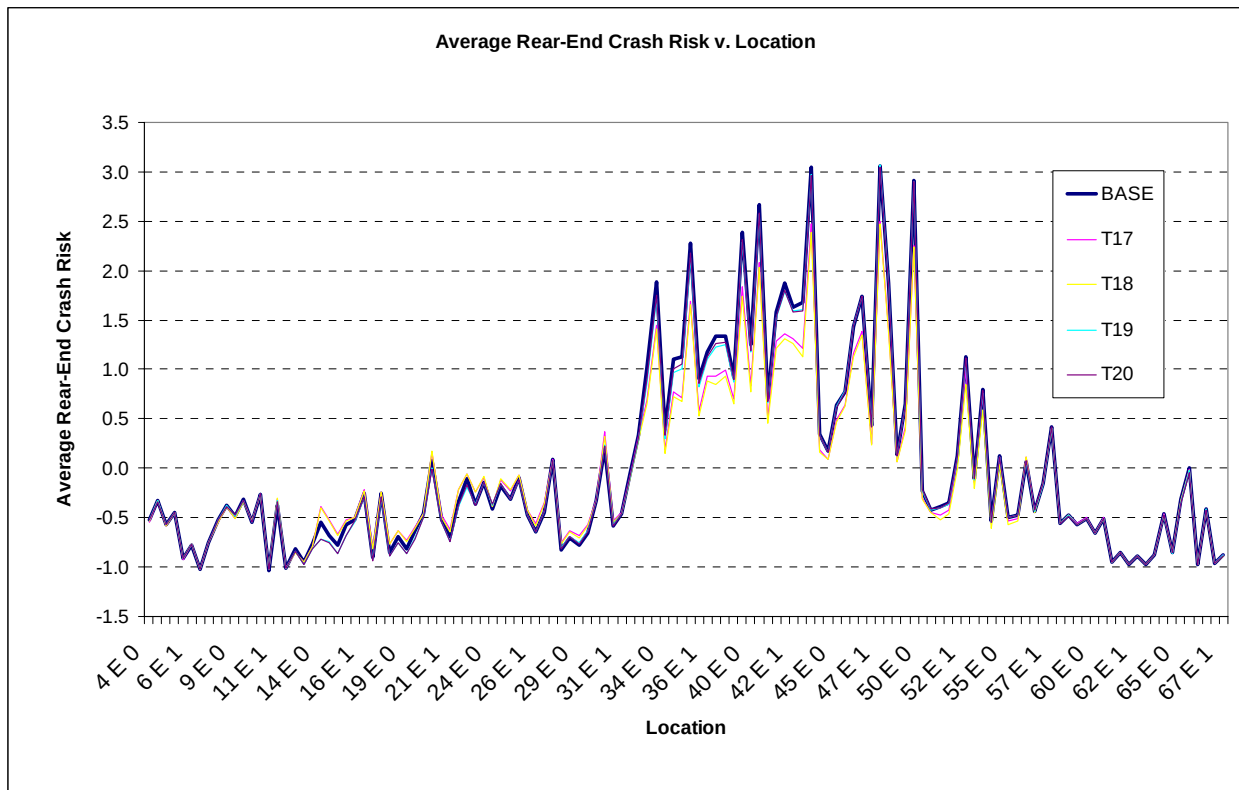


Figure 9-12: Average Rear-End Crash Risk v. Location for Treatments 17–20

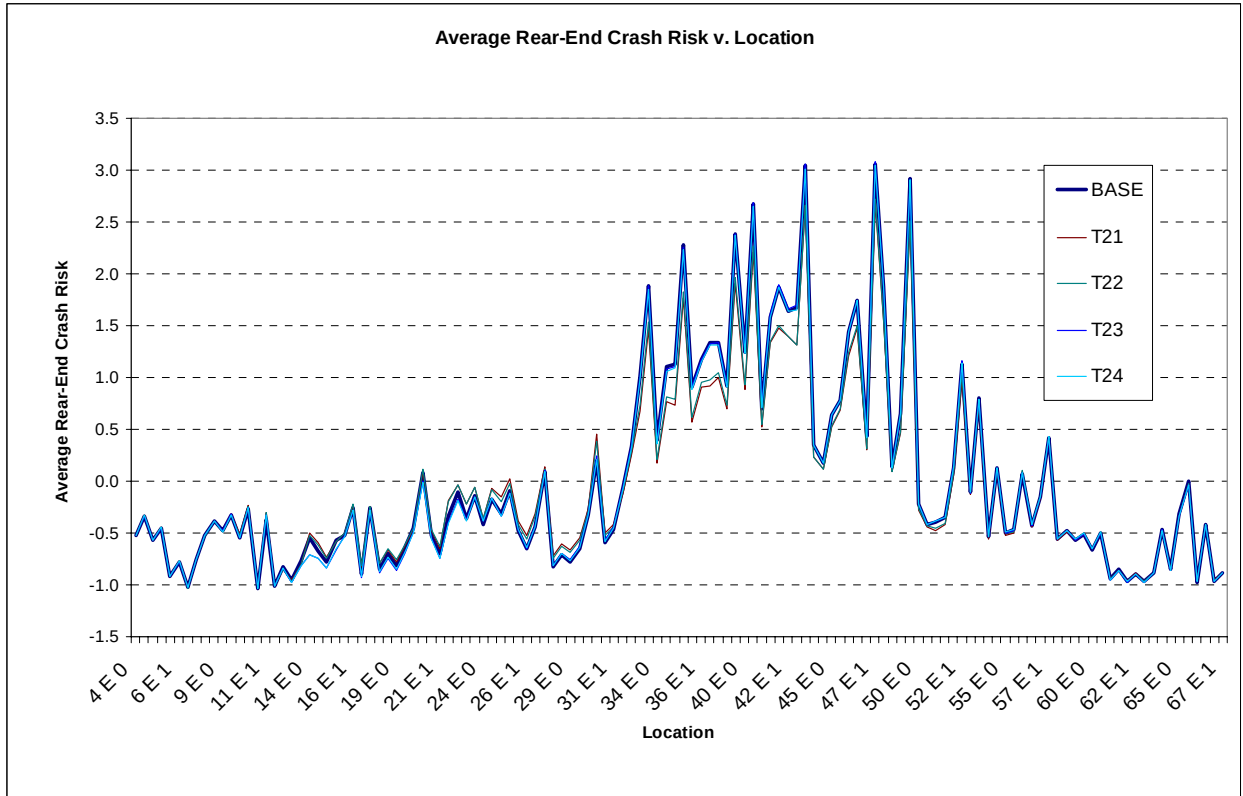


Figure 9-13: Average Rear-End Crash Risk v. Location for Treatments 21–24

The segment of corridor most affected by the treatments is located between stations 33 and 49. Figures 9-14 and 9-15 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.

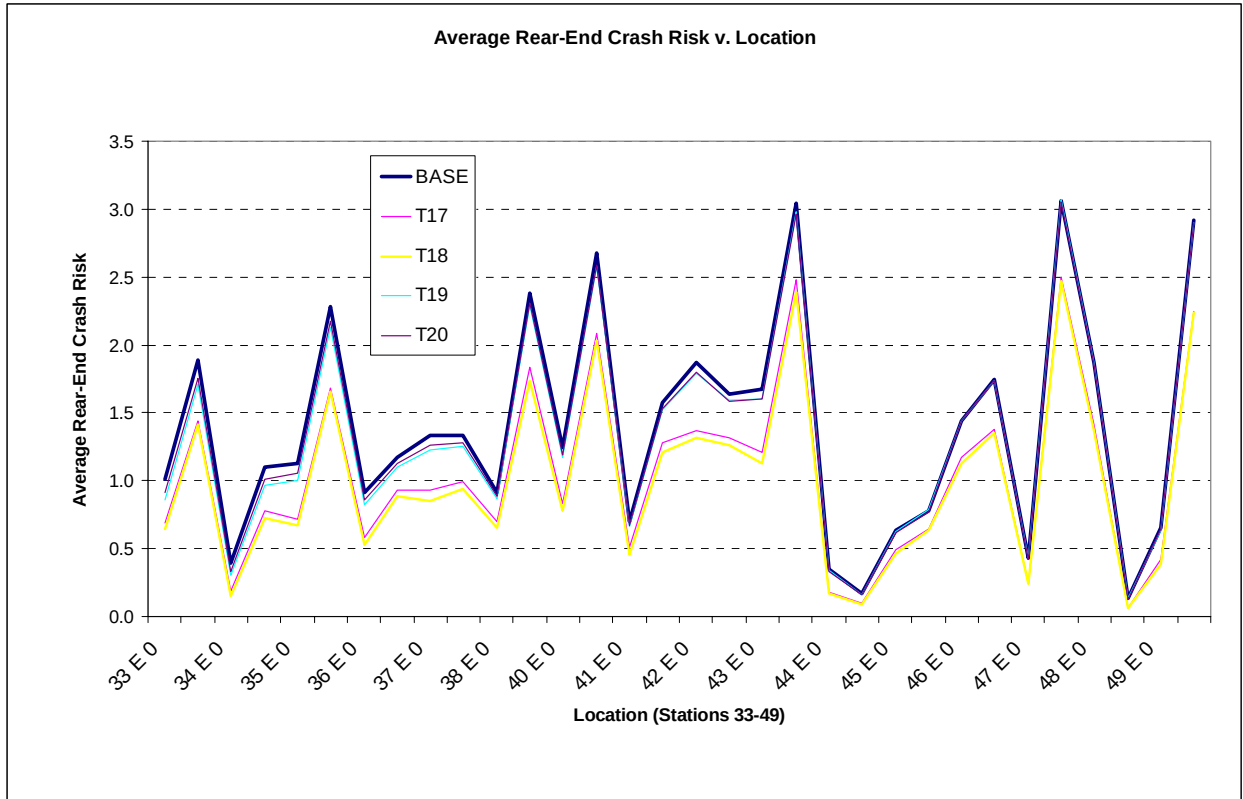


Figure 9-14: Average Rear-End Crash Risk v. Location for Treatments 17–20 – Magnified

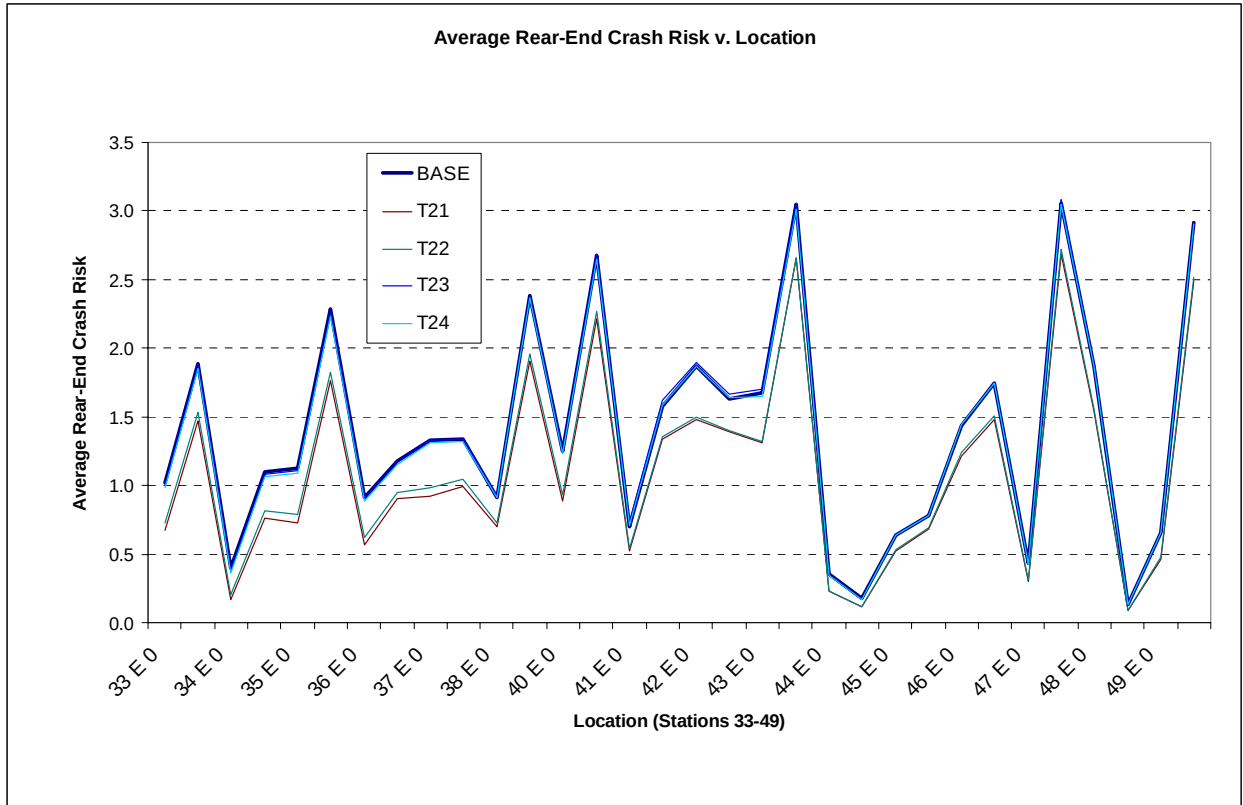


Figure 9-15: Average Rear-End Crash Risk v. Location for Treatments 21–24 - Magnified

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 9-8.

Table 9-8: Summary of Difference in Rear-End Crash Risk for Treatments 17–24

	Test Case ID								
	Base	T17	T18	T19	T20	T21	T22	T23	T24
Average Crash Risk (Stations 33 to 49)	1.390	1.044	1.001	1.330	1.342	1.100	1.129	1.393	1.376
Crash Risk Benefit	---	0.346	0.389	0.059	0.048	0.289	0.261	-0.004	0.014
T-Statistic (Significance of Change)	---	1.830	2.069	0.288	0.235	1.485	1.334	0.018	0.068
ORCI	---	11.060	12.440	1.894	1.540	9.256	8.336	-0.118	0.446

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.

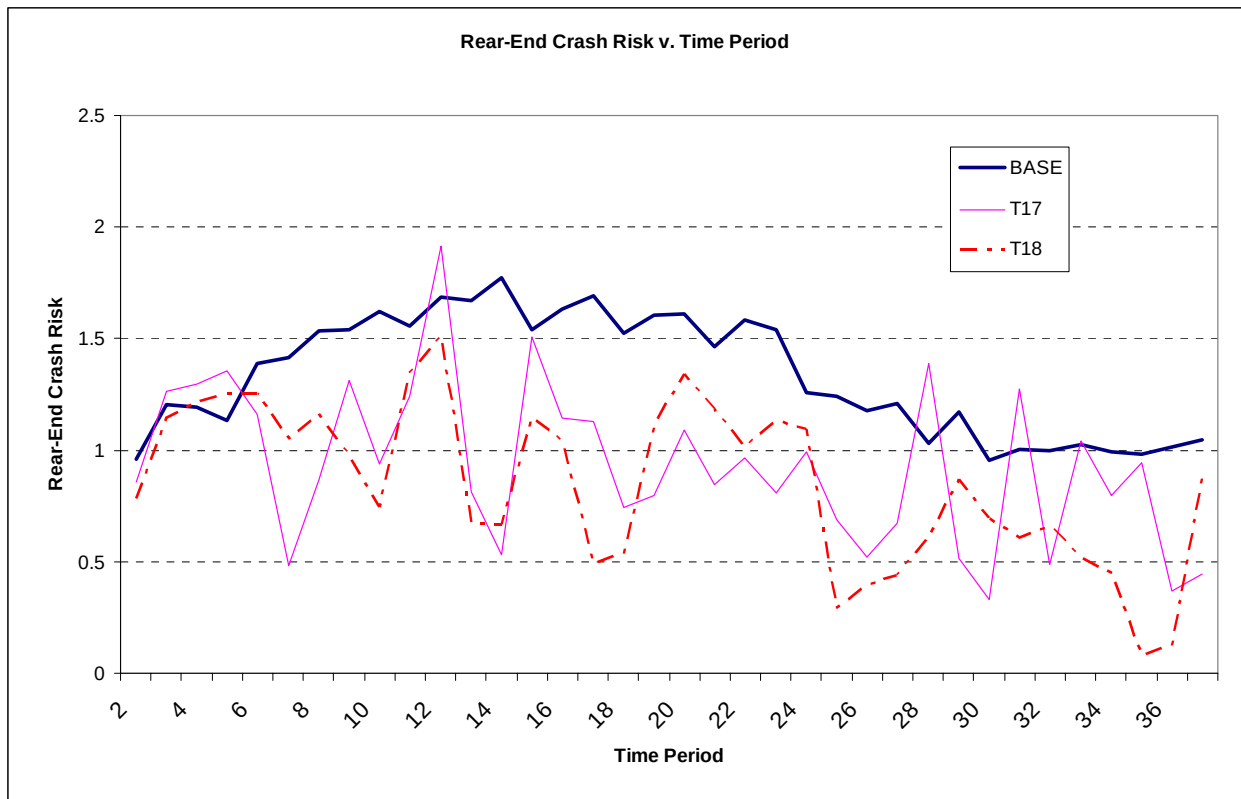


Figure 9-16: Rear-End Crash Profile at Station 37 E 0 for Treatments 17 and 18

Treatments 17 and 18 were found to be significant. Figure 9-16 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.

9.2.3.2 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 9-17 and 9-18 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.

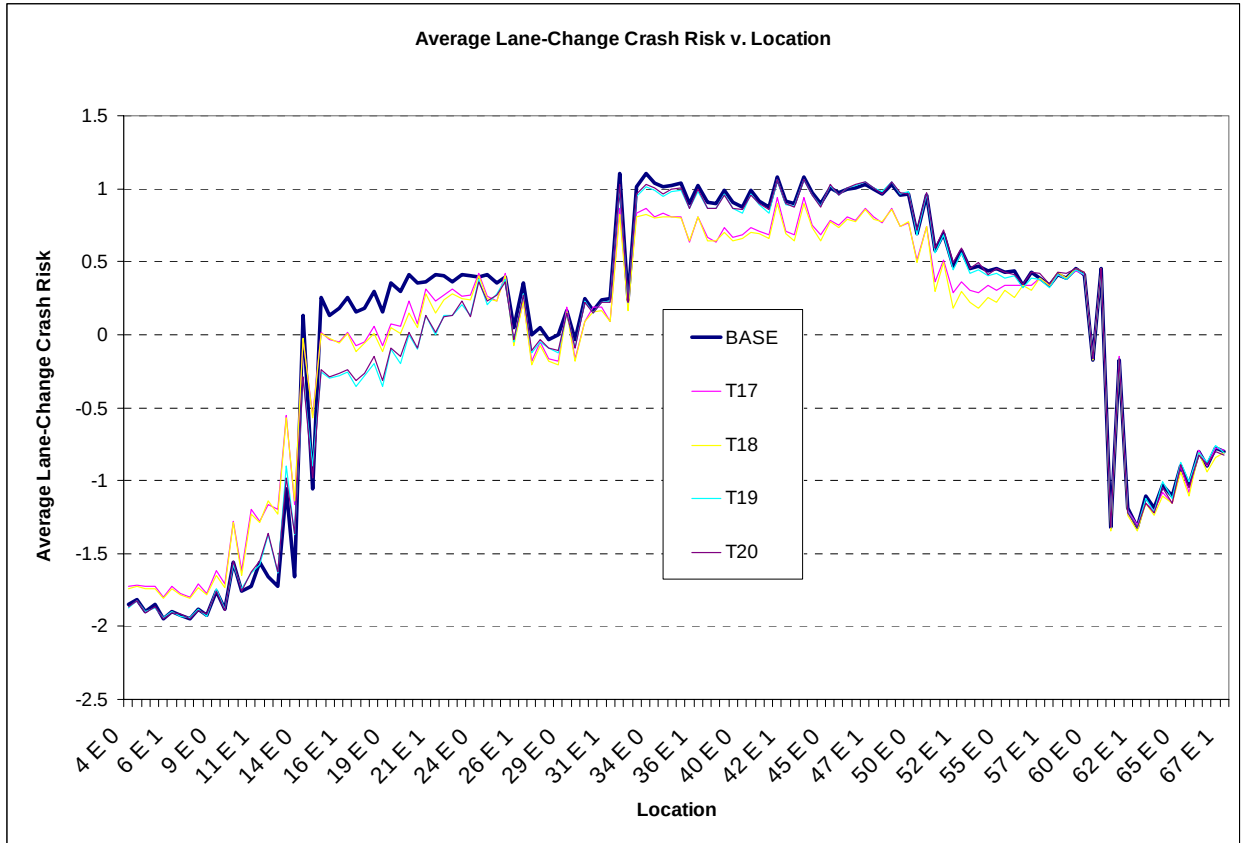


Figure 9-17: Average Lane-Change Crash Risk v. Location for Treatments 17–20

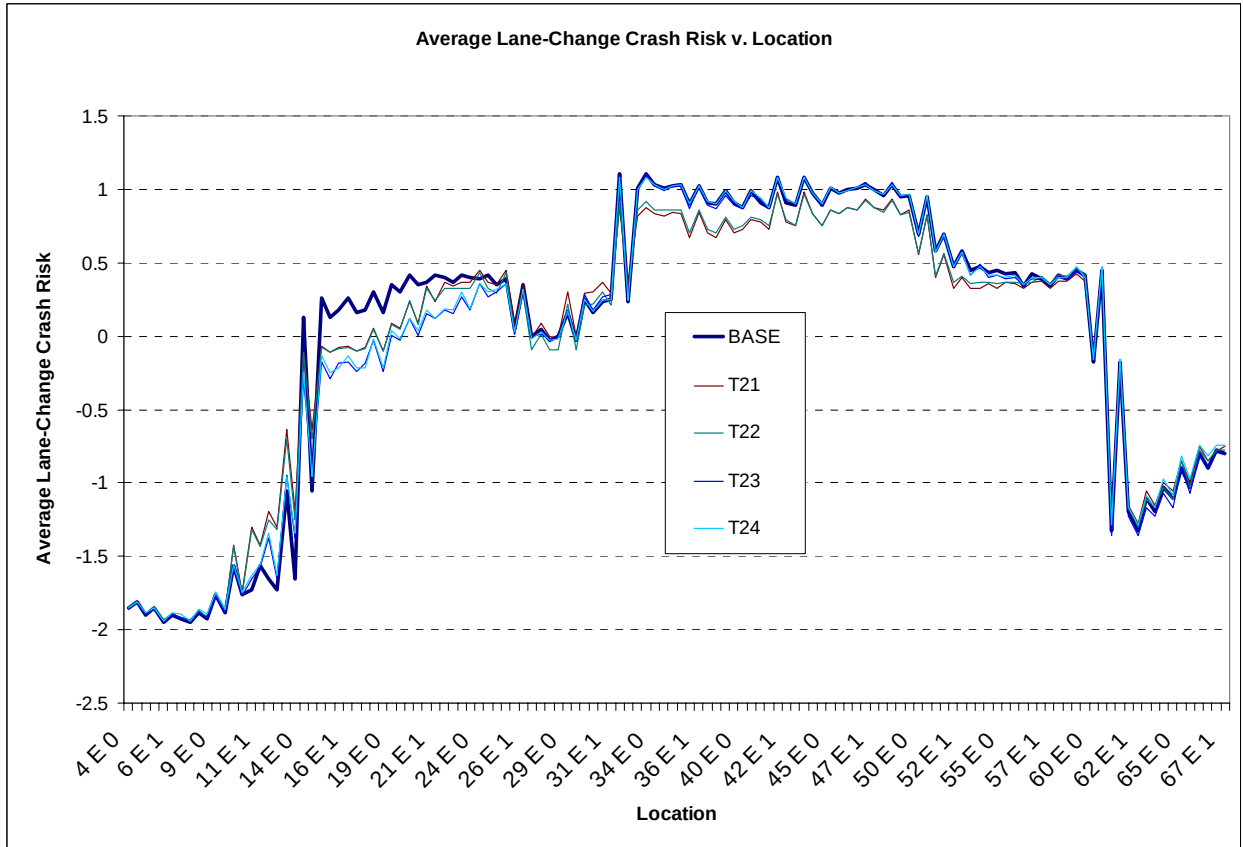


Figure 9-18: Average Lane-Change Crash Risk v. Location for Treatments 21–24

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 9-19 and 9-20, 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.

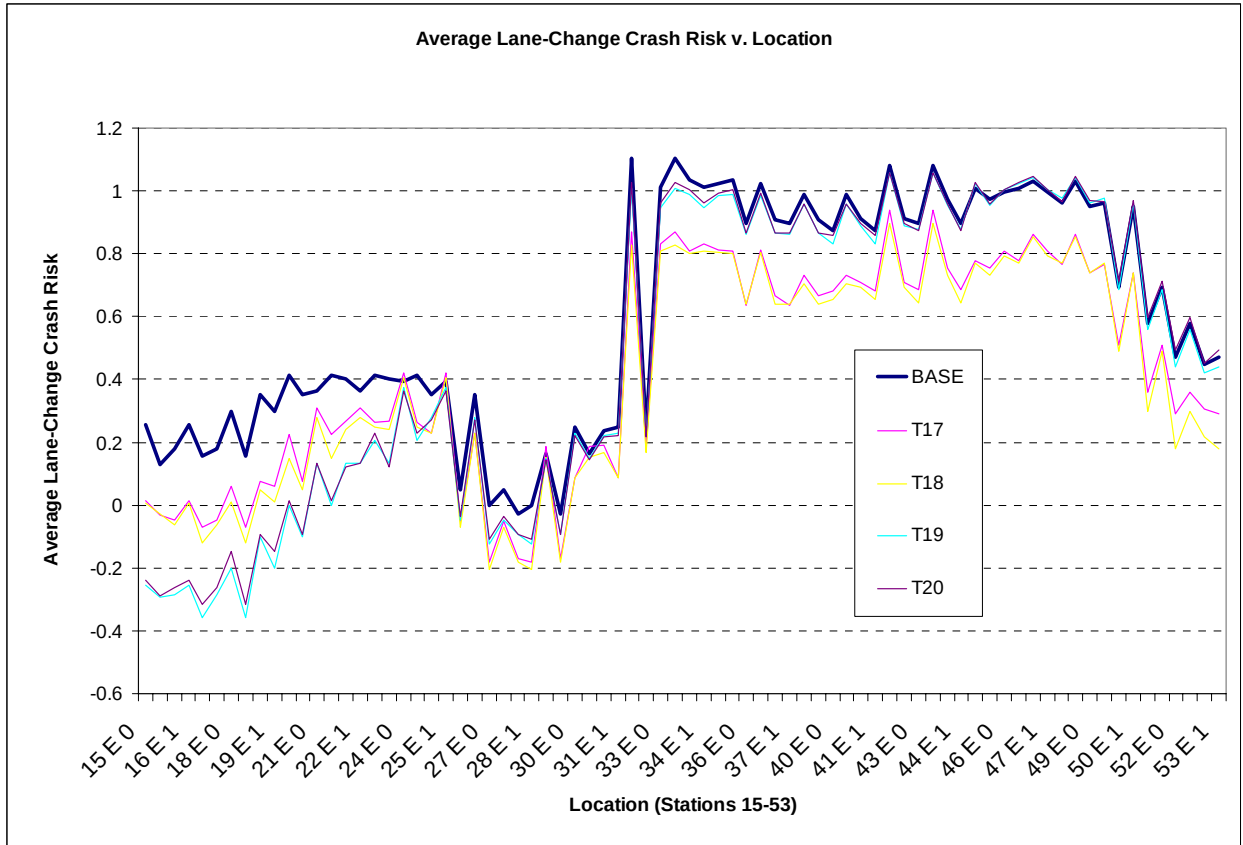


Figure 9-19: Average Lane-Change Crash Risk v. Location for Treatments 17–20 – Magnified

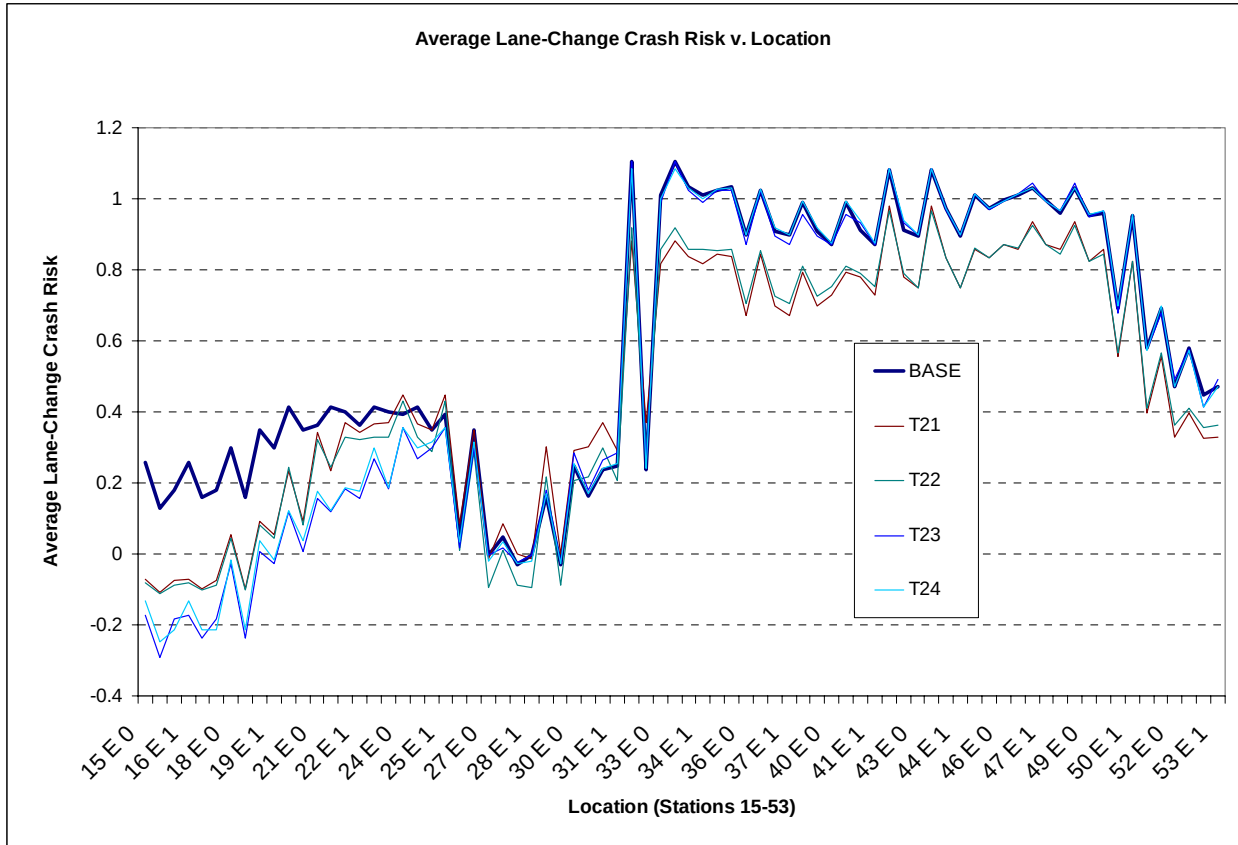


Figure 9-20: Average Lane-Change Crash Risk v. Location for Treatments 21–24 - Magnified

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 9-9.

Table 9-9: Summary of Difference in Lane-Change Crash Risk for Treatments 17–**24**

	Test Case ID								
	Base	T17	T18	T19	T20	T21	T22	T23	T24
Average Crash Risk (Stations 15 to 53)	0.603	0.426	0.401	0.480	0.491	0.484	0.473	0.523	0.528
Crash Risk Benefit	---	0.177	0.203	0.123	0.113	0.119	0.131	0.080	0.075
T-Statistic (Significant of Change)	---	3.063	3.480	1.772	1.628	2.068	2.201	1.195	1.124
ORCI	---	13.485	15.410	9.370	8.564	9.077	9.922	6.082	5.710

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 9-21 and 9-22. 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.

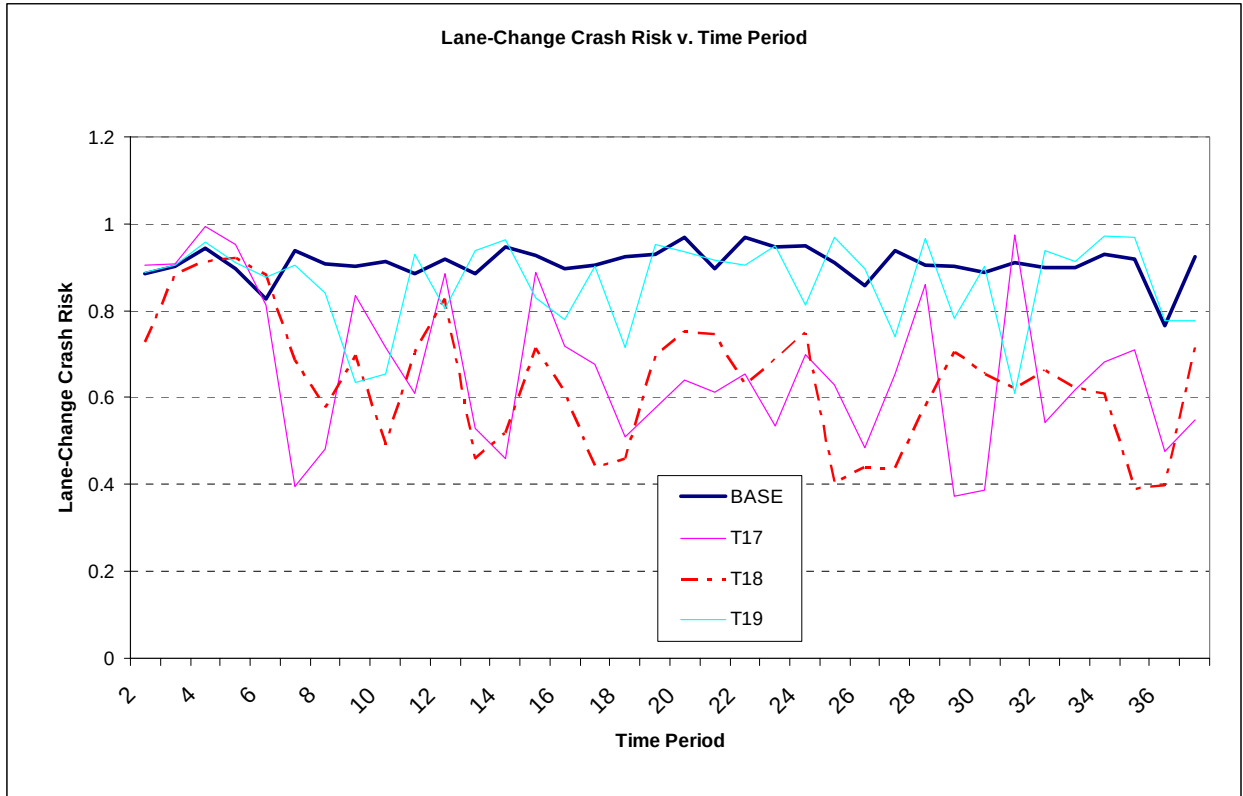


Figure 9-21: Lane-Change Crash Profile at Station 37 E 0 for Treatments 17, 18 and 19

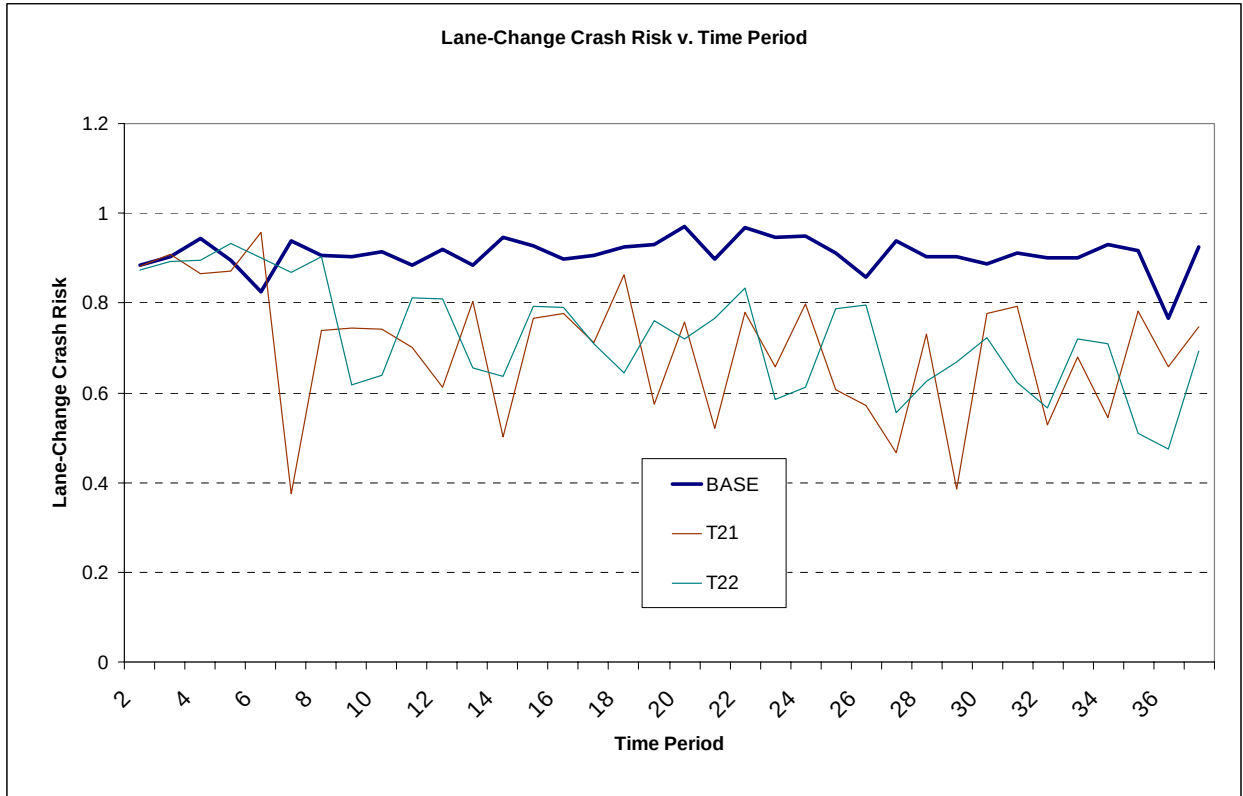


Figure 9-22: Lane-Change Crash Profile at Station 37 E 0 for Treatments 21 and 22

9.2.3.3 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 9-10: Analysis of Travel Times for Treatments 17 and 18

	Test Case ID		
	Base	T17	T18
Average Travel Time (vehicle-hours)	11633.538	11589.892	11542.094
Travel Time Benefit	---	43.646	91.444
Percent Decrease	---	0.375	0.786
T-Statistic (Significance of Change)	---	1.519	3.186

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 9-10. 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.

9.3 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.

9.3.1 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 9-11 displays the different factors that make up Treatments 01-08, which are reviewed in this section.

Table 9-11: Description of Treatments 01-08

Treatment ID	Speed Change Implementation	Speed Zone Threshold	Speed Change Distance	Speed Change Time Period
1	-10	5 mph	Speed Zone	5 min
2	-10	5 mph	Speed Zone	10 min
3	-10	5 mph	1/2 Speed Zone	5 min
4	-10	5 mph	1/2 Speed Zone	10 min
5	-10	2.5 mph	Speed Zone	5 min
6	-10	2.5 mph	Speed Zone	10 min
7	-10	2.5 mph	1/2 Speed Zone	5 min
8	-10	2.5 mph	1/2 Speed Zone	10 min

9.3.1.1 Rear-End Crash Risk

The rear-end crash risk was analyzed for Treatments 01-08. Figure 9-23 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.

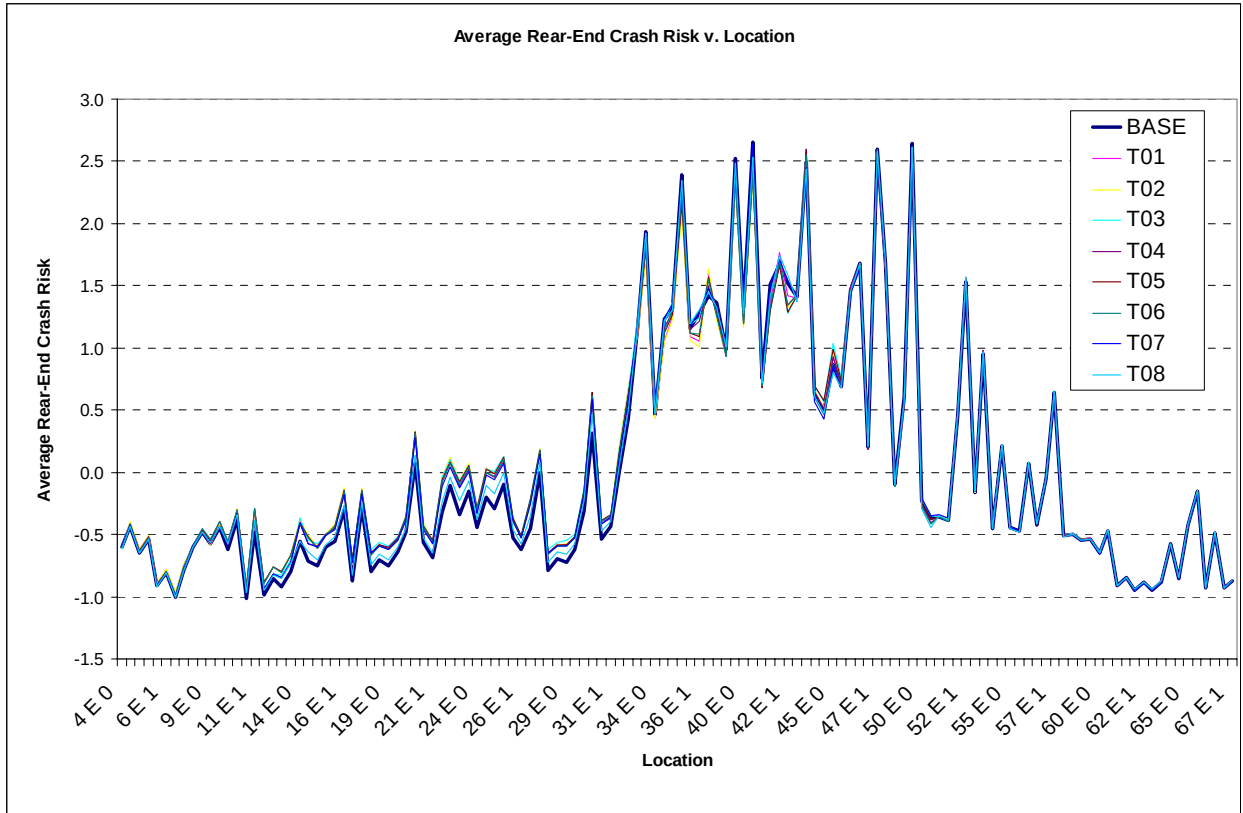


Figure 9-23: Average Rear-End Crash Risk v. Location for Treatments 01–08

The most affected segment is magnified in Figure 9-24. 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.

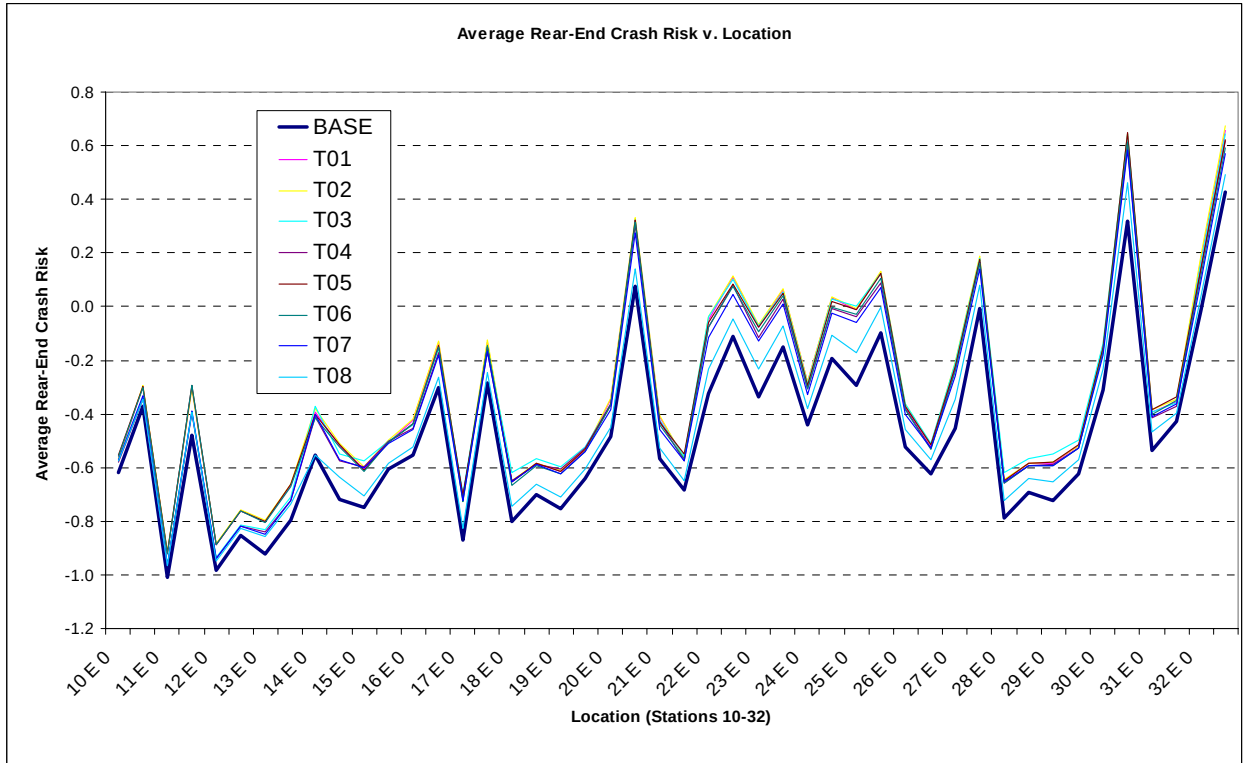


Figure 9-24: Average Rear-End Crash Risk v. Location for Treatments 01–08 - Magnified

Table 9-12 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 9-12: Summary of Difference in Rear-End Crash Risk for Treatments 01-08

	Test Case ID								
	Base	T01	T02	T03	T04	T05	T06	T07	T08
Average Crash Risk (Stations 10 to 32)	-0.482	-0.322	-0.316	-0.326	-0.346	-0.325	-0.336	-0.356	-0.422
Crash Risk Benefit	---	-0.160	-0.166	-0.156	-0.136	-0.157	-0.146	-0.125	-0.060
T-Statistic (Significance of Change)	---	2.215	2.293	2.150	1.892	2.189	2.049	1.770	0.863
ORCI	---	-7.369	-7.648	-7.174	-6.257	-7.221	-6.714	-5.768	-2.742

9.3.1.2 Lane-Change Crash Risk

The lane-change crash risk was also observed for Treatments 01-08. Figure 9-25 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.

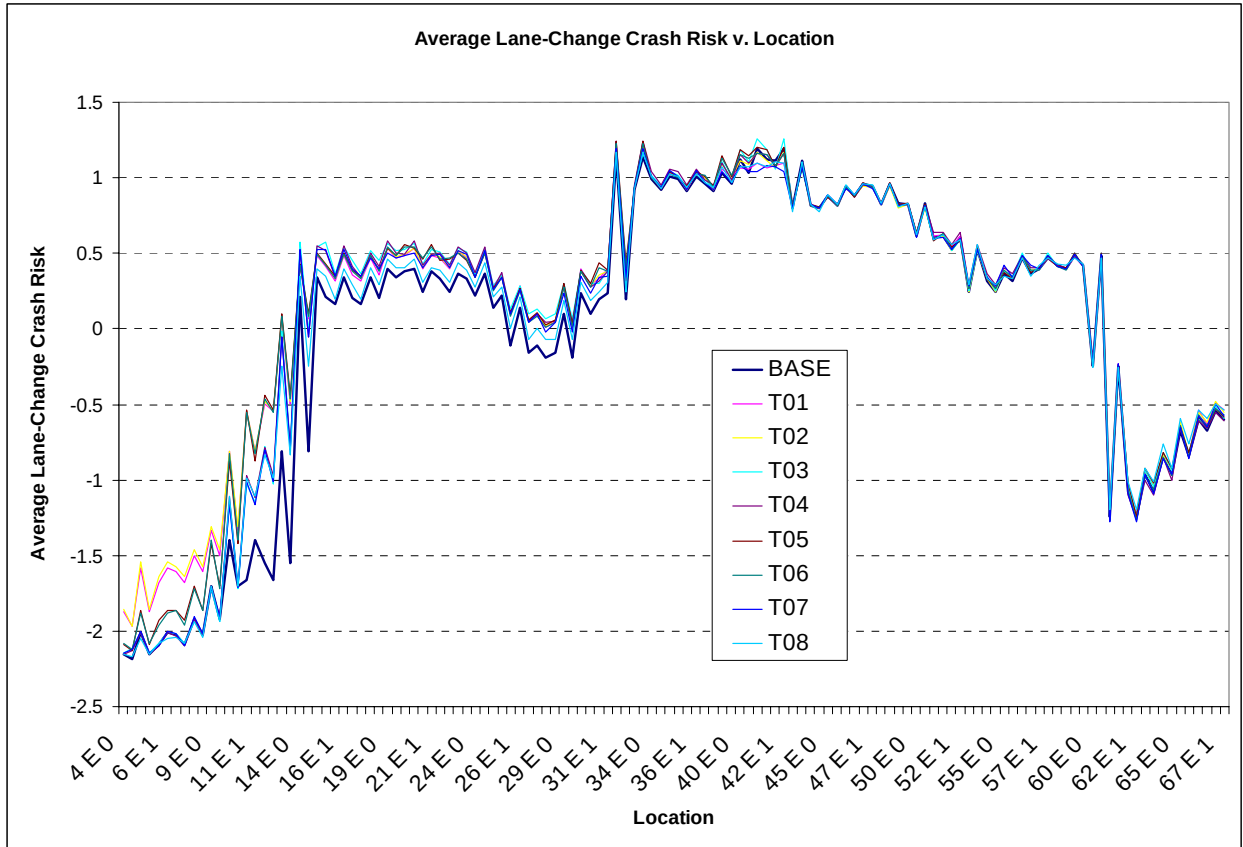


Figure 9-25: Average Lane-Change Crash Risk v. Location for Treatments 01–08

Figure 9-26 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.

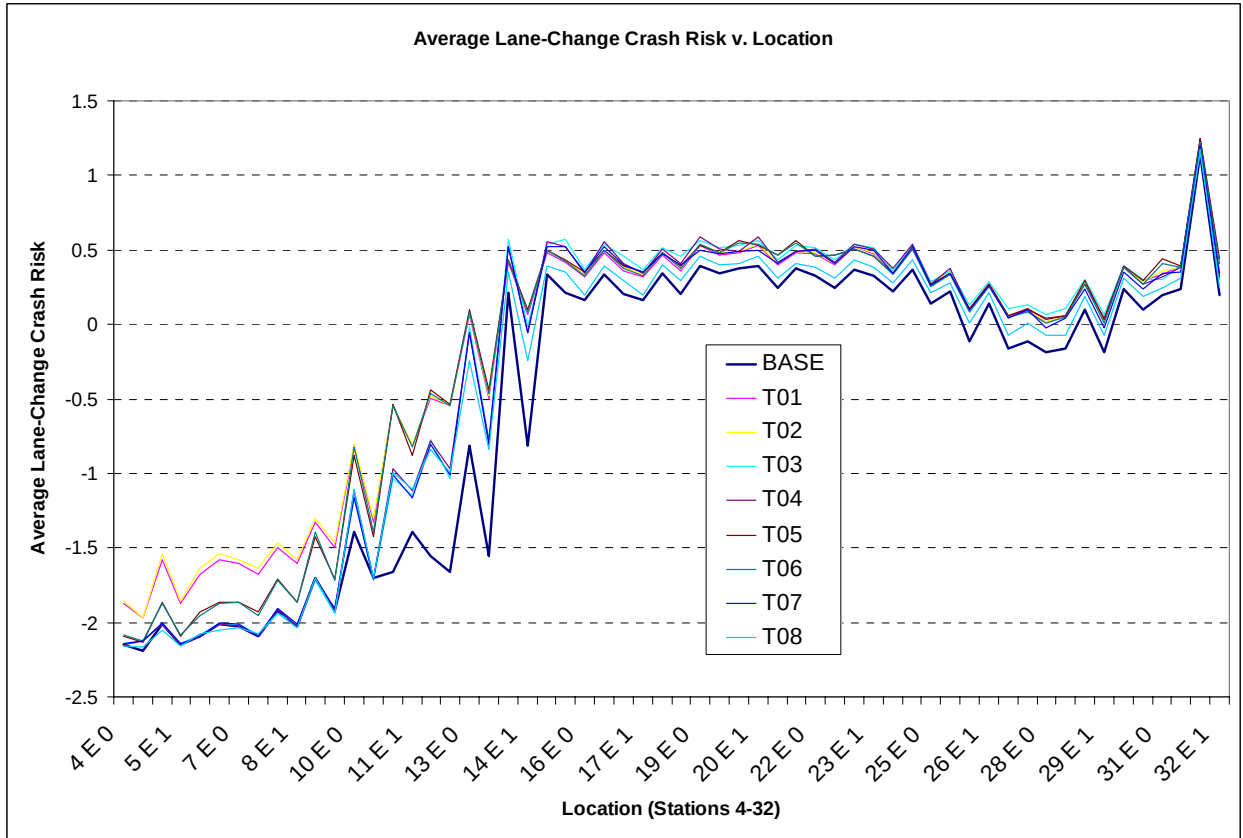


Figure 9-26: Average Lane-Change Crash Risk v. Location for Treatments 01–08 - Magnified

The significance of the difference in average lane-change crash risk for each treatment from the BASE is summarized in Table 9-13. 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 9-13: Summary of Difference in Lane-Change Crash Risk for Treatments 01-08

	Test Case ID								
	Base	T01	T02	T03	T04	T05	T06	T07	T08
Average Crash Risk (Stations 4 to 32)	-0.498	-0.189	-0.177	-0.289	-0.296	-0.223	-0.230	-0.311	-0.377
Crash Risk Benefit	---	-0.309	-0.321	-0.209	-0.202	-0.275	-0.267	-0.186	-0.121
T-Statistic (Significance of Change)	---	1.786	1.863	1.105	1.071	1.516	1.478	0.994	0.656
ORCI	---	-17.913	-18.612	-12.098	-11.700	-15.938	-15.508	-10.800	-7.014

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.

9.3.2 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 9-14 displays the different factors that make up Treatments 09-16, which are reviewed in this section.

Table 9-14: Description of Treatments 09-16

Treatment ID	Speed Change Implementation	Speed Zone Threshold	Speed Change Distance	Speed Change Time Period
9	-5	5 mph	Speed Zone	5 min
10	-5	5 mph	Speed Zone	10 min
11	-5	5 mph	1/2 Speed Zone	5 min
12	-5	5 mph	1/2 Speed Zone	10 min
13	-5	2.5 mph	Speed Zone	5 min
14	-5	2.5 mph	Speed Zone	10 min
15	-5	2.5 mph	1/2 Speed Zone	5 min
16	-5	2.5 mph	1/2 Speed Zone	10 min

9.3.2.1 Rear-End Crash Risk

The rear-end crash risk was analyzed for Treatments 09-16. Figure 9-27 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.

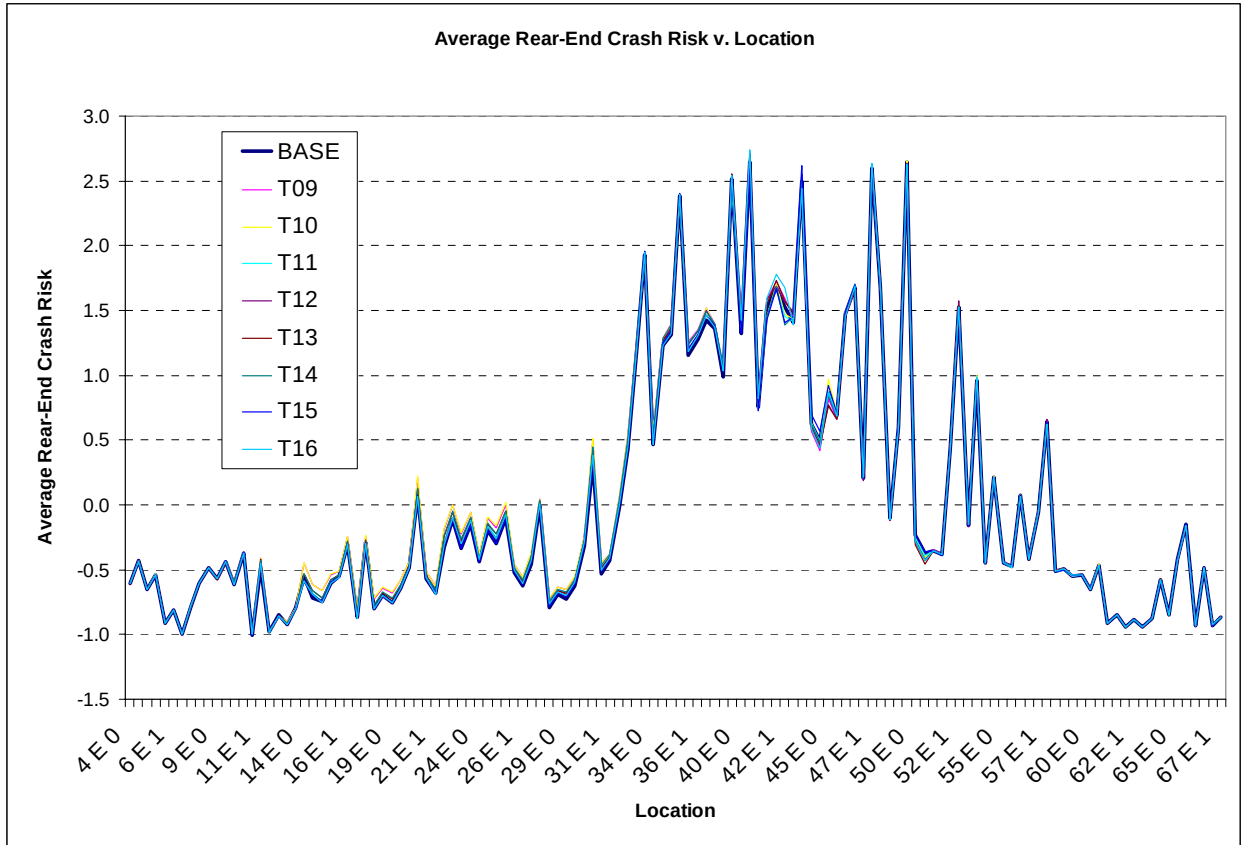


Figure 9-27: Average Rear-End Crash Risk v. Location for Treatments 09–16

The most affected segment is magnified in Figure 9-28. 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.

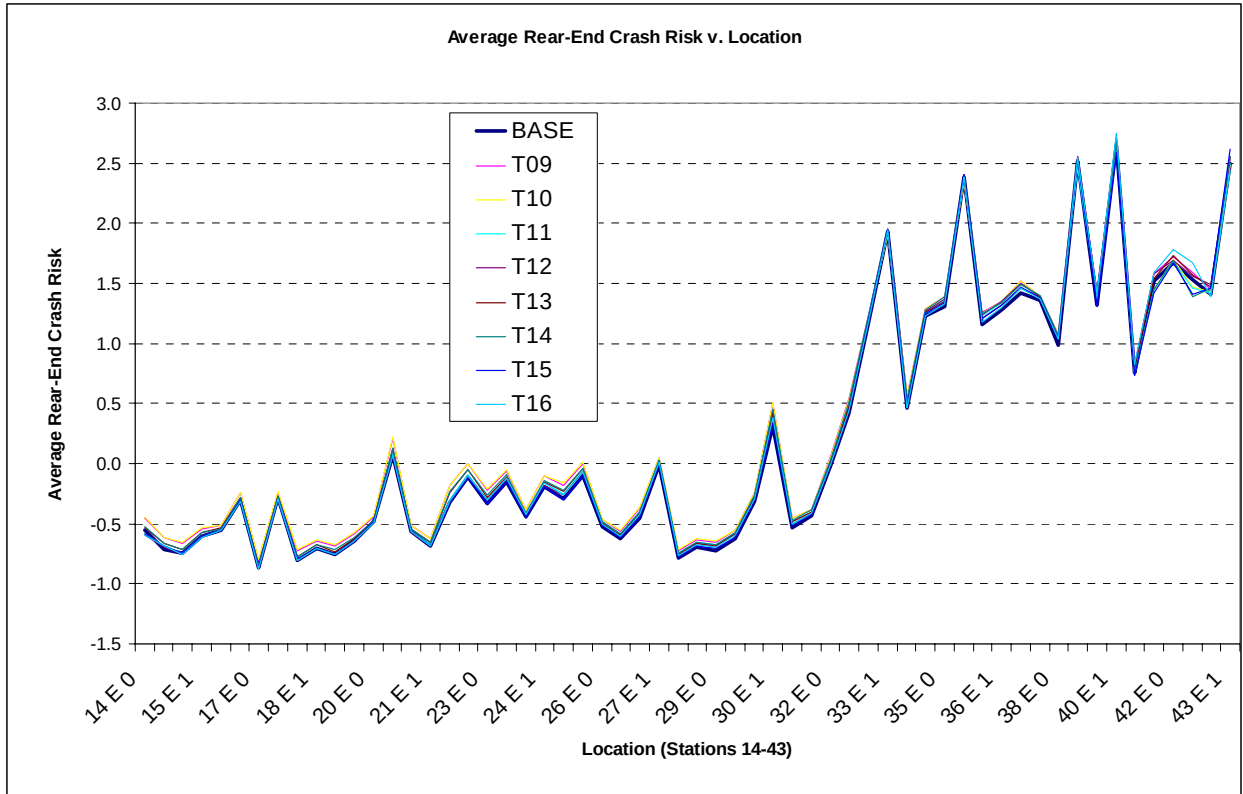


Figure 9-28: Average Rear-End Crash Risk v. Location for Treatments 09–16 - Magnified

Table 9-15 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 9-15: Summary of Difference in Rear-End Crash Risk for Treatments 09-16

	Test Case ID								
	Base	T09	T10	T11	T12	T13	T14	T15	T16
Average Crash Risk (Stations 14 to 43)	0.249	0.321	0.315	0.270	0.278	0.287	0.291	0.256	0.270
Crash Risk Benefit	---	-0.072	-0.066	-0.021	-0.029	-0.039	-0.043	-0.007	-0.021
T-Statistic (Significance of Change)	---	0.380	0.349	0.109	0.150	0.201	0.223	0.036	0.110

ORCI	---	-4.179	-3.812	-1.207	-1.675	-2.233	-2.466	-0.400	-1.228

9.3.2.2 Lane-Change Crash Risk

The lane-change crash risk was also observed for Treatments 09-16. Figure 9-29 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.

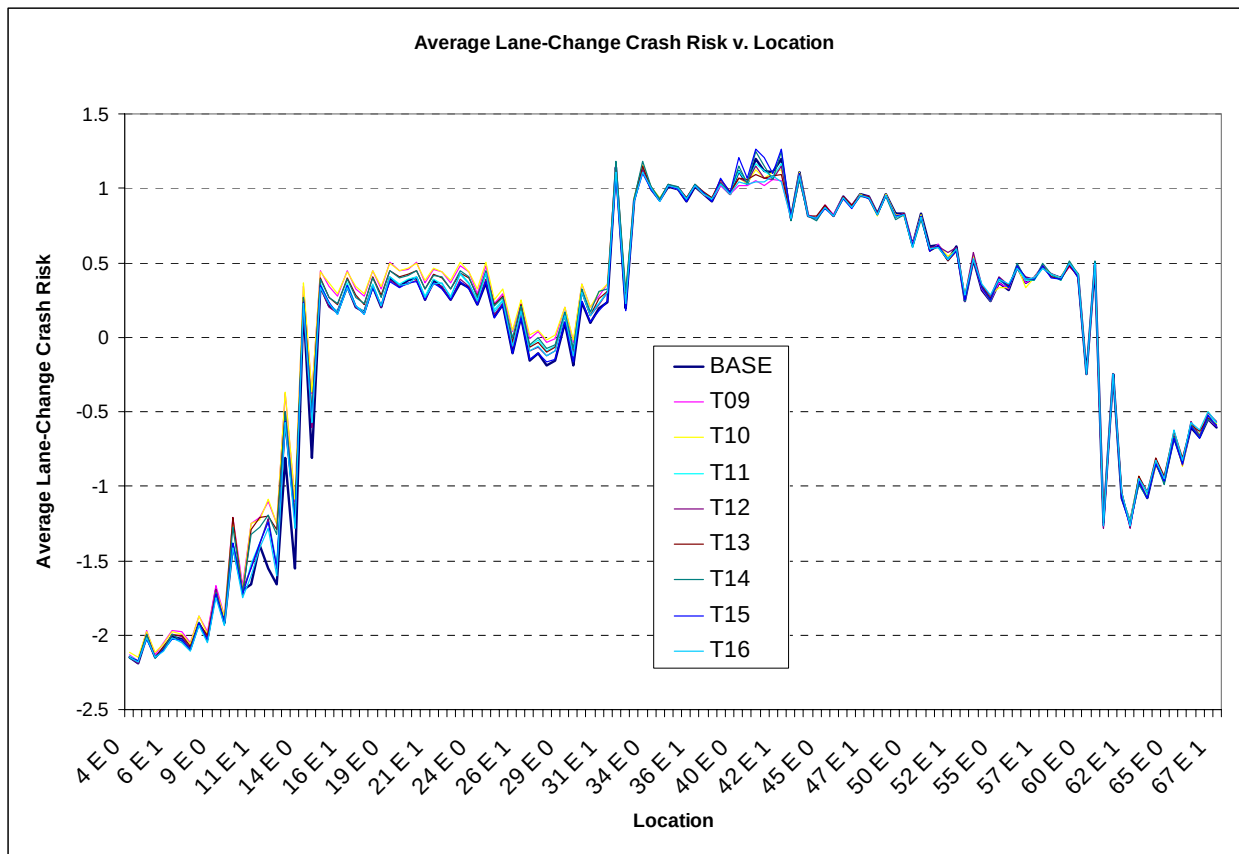


Figure 9-29: Average Lane-Change Crash Risk v. Location for Treatments 09-16

Figure 9-30 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.

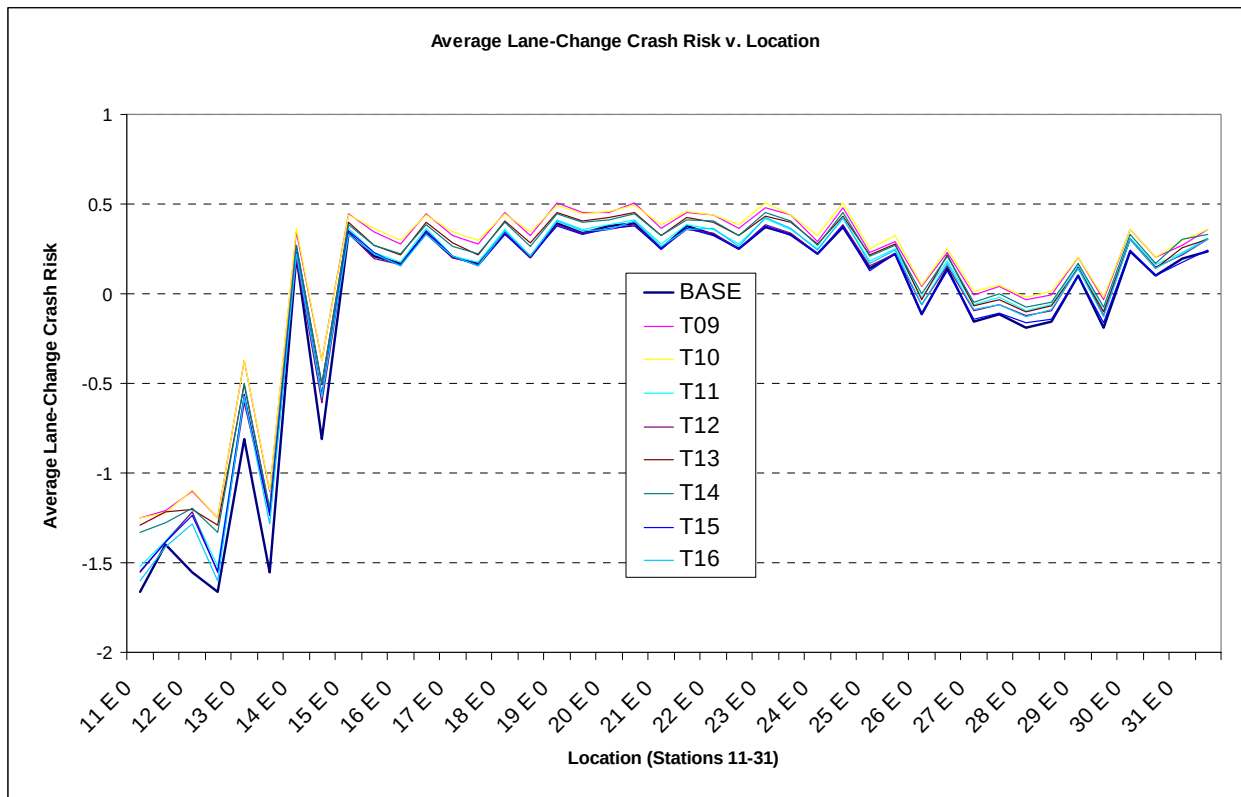


Figure 9-30: Average Lane-Change Crash Risk v. Location for Treatments 09-16 - Magnified

The significance of the difference in average lane-change crash risk for each treatment from the BASE is summarized in Table 9-16. The crash risk benefits shows 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 9-16: Summary of Difference in Lane-Change Crash Risk for Treatments 09-16

	Test Case ID								
	Base	T09	T10	T11	T12	T13	T14	T15	T16
Average Crash Risk (Stations 11 to 31)	-0.067	0.094	0.103	0.001	-0.022	0.041	0.044	-0.034	-0.018
Crash Risk Benefit	---	-0.161	-0.170	-0.068	-0.045	-0.108	-0.110	-0.033	-0.049
T-Statistic (Significance of Change)	---	1.289	1.360	0.530	0.351	0.861	0.874	0.258	0.378
ORCI	---	-6.759	-7.146	-2.868	-1.896	-4.539	-4.637	-1.392	-2.071

9.3.3 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 9-17 summarizes the factors for Treatments 17-24, which are reviewed in this section.

Table 9-17: Description of Treatments 17-24

Treatment ID	Speed Change Implementation	Speed Zone Threshold	Speed Change Distance	Speed Change Time Period
17	-5/+5	5 mph	Speed Zone	5 min
18	-5/+5	5 mph	Speed Zone	10 min
19	-5/+5	5 mph	1/2 Speed Zone	5 min
20	-5/+5	5 mph	1/2 Speed Zone	10 min
21	-5/+5	2.5 mph	Speed Zone	5 min
22	-5/+5	2.5 mph	Speed Zone	10 min
23	-5/+5	2.5 mph	1/2 Speed Zone	5 min

24	-5/+5	2.5 mph	1/2 Speed Zone	10 min

9.3.3.1 Rear-End Crash Risk

The effects that Treatments 17-24 have on rear-end crash risk are reviewed in this section. Figures 9-31 and 9-32 show the average rear-end crash risk for every location for the BASE case and Treatments 17-20 and 21-24, respectively.

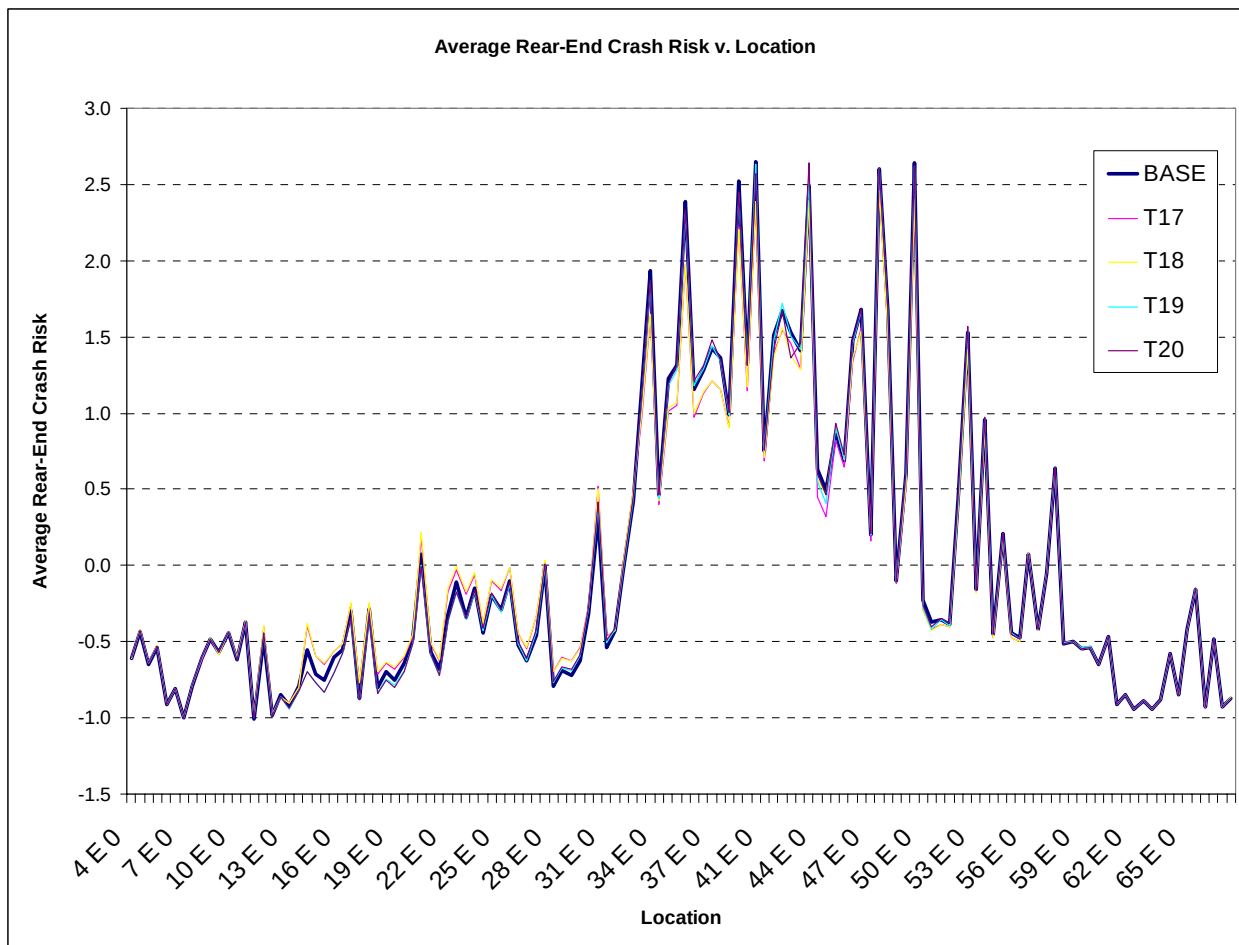


Figure 9-31: Average Rear-End Crash Risk v. Location for Treatments 17-20

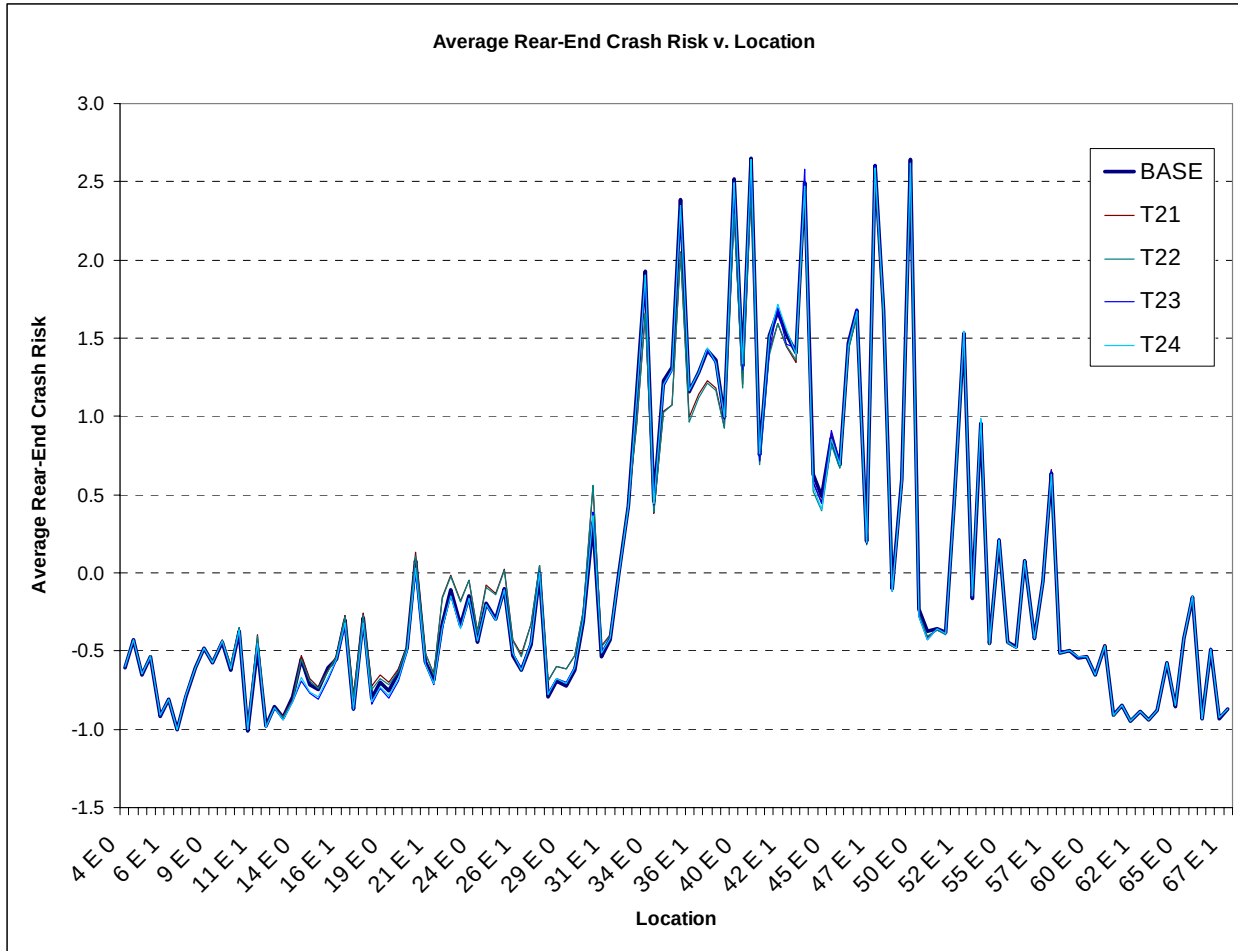


Figure 9-32: Average Rear-End Crash Risk v. Location for Treatments 21-24

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.

9.3.3.1.1 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 9-33 and 9-34 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.

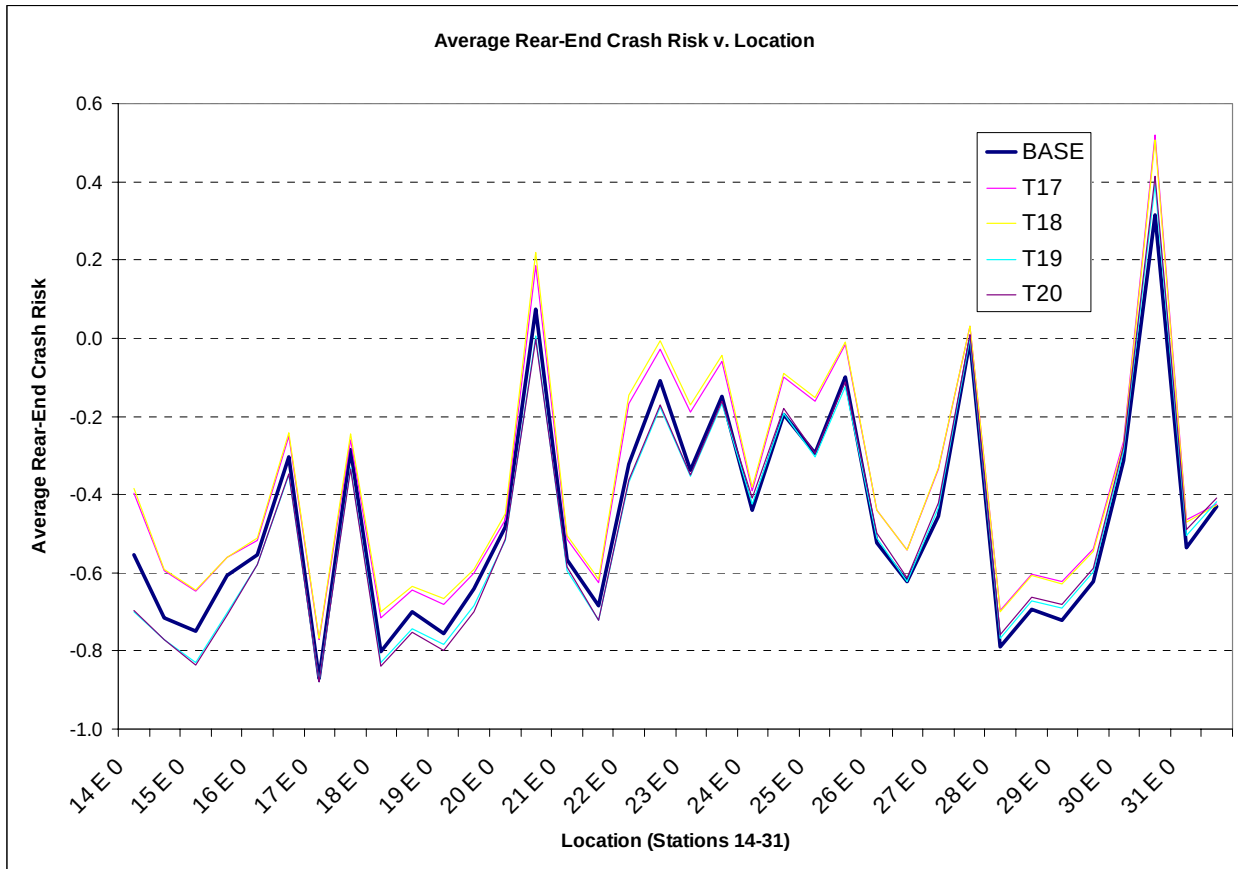


Figure 9-33: Average Rear-End Crash Risk v. Location for Treatments 17-20 for 1st Section - Magnified

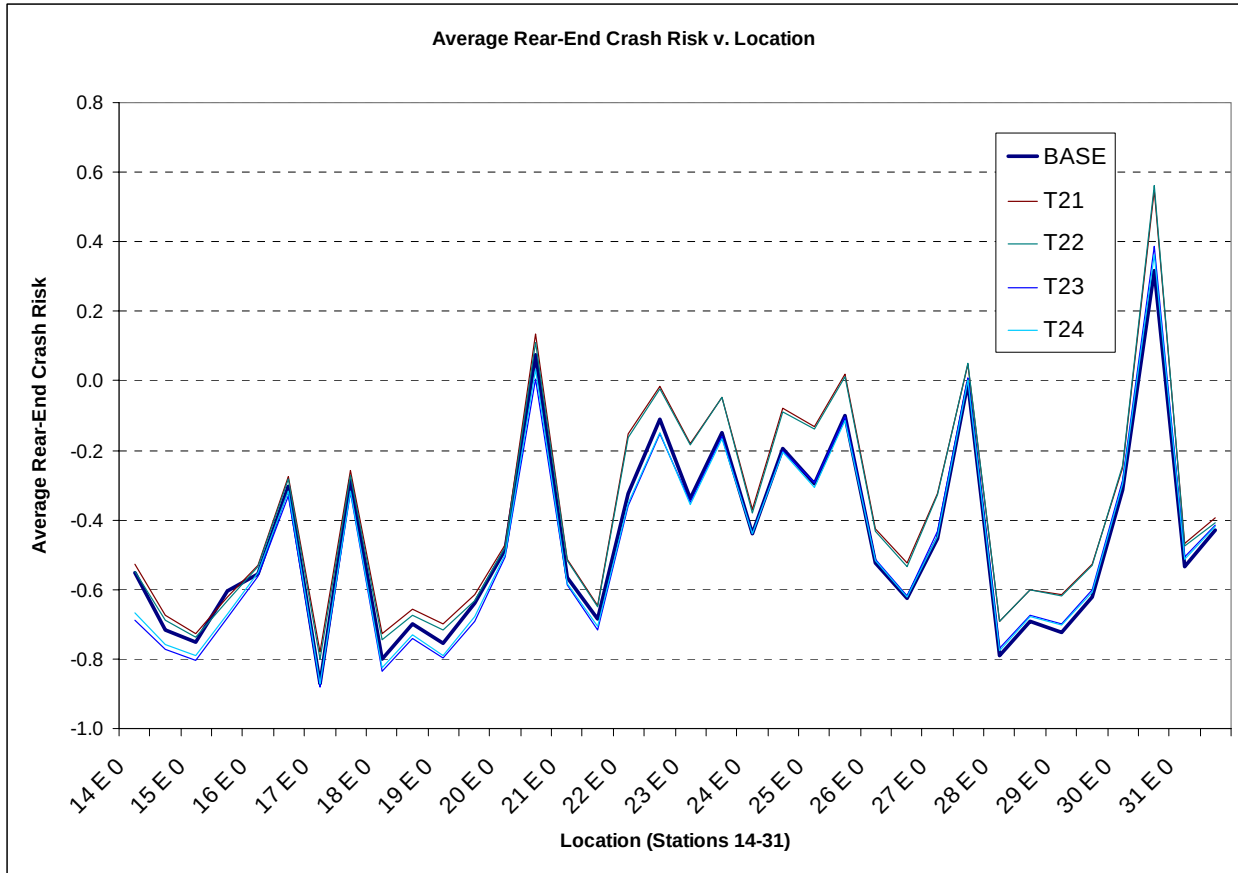


Figure 9-34: Average Rear-End Crash Risk v. Location for Treatments 21-24 for 1st Section - Magnified

Table 9-18 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 9-18: 1st Section: Summary of Difference in Rear-End Crash Risk for
Treatments 17-24**

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

9.3.3.1.2 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 9-35 and 9-36 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.

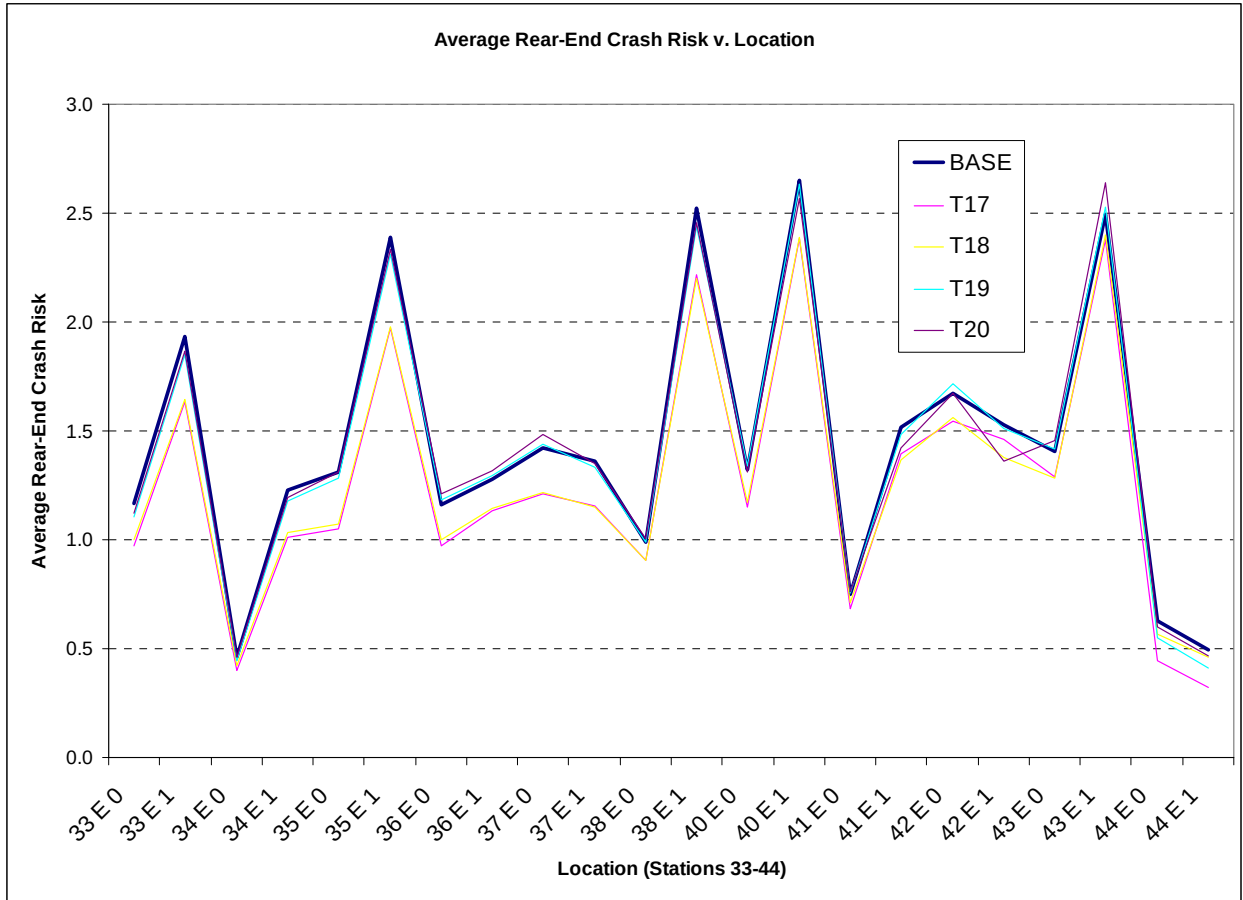


**Figure 9-35: Average Rear-End Crash Risk v. Location for Treatments 17-20 for
2nd Section - Magnified**

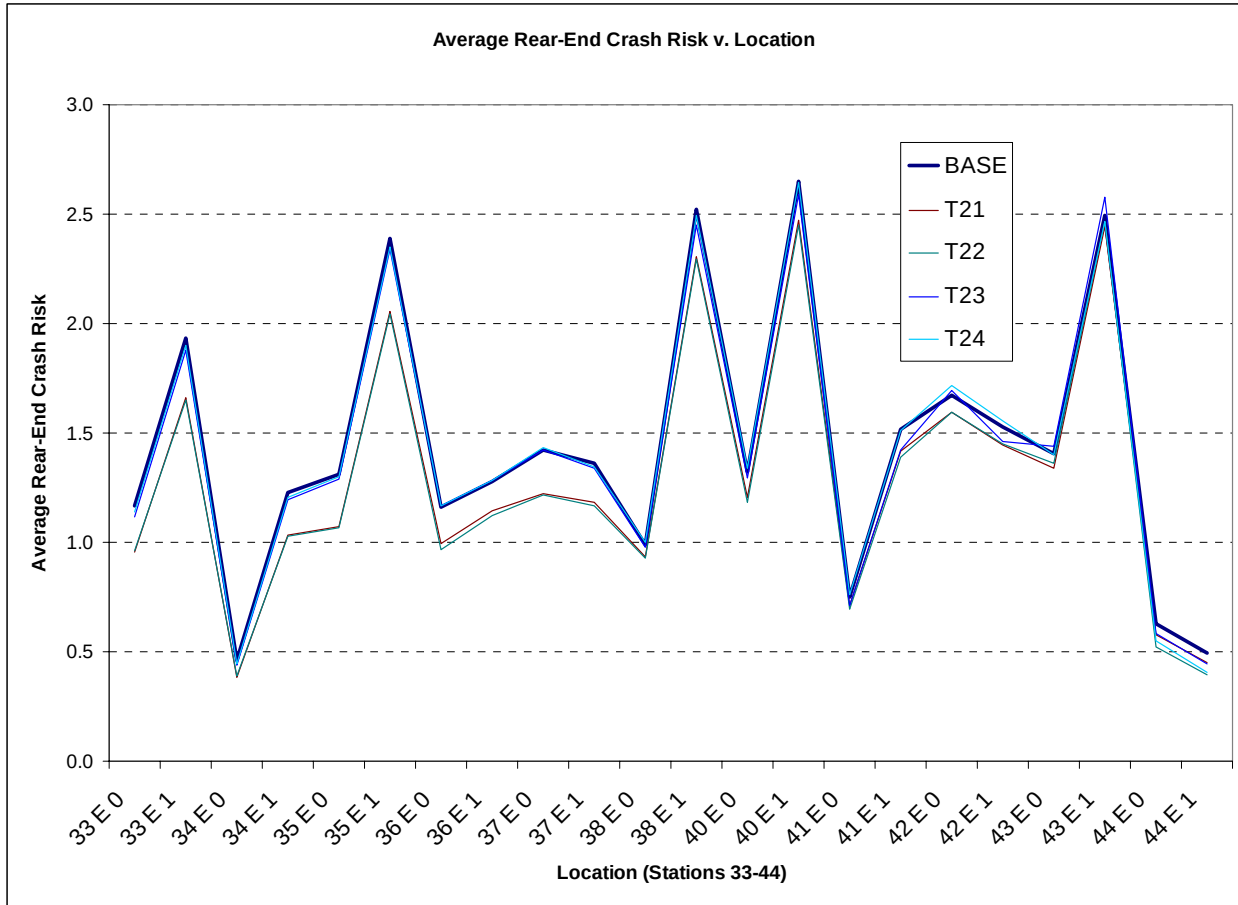


Figure 9-36: Average Rear-End Crash Risk v. Location for Treatments 21-24 for 2nd Section - Magnified

Table 9-19 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.

Table 9-19: 2nd Section: Summary of Difference in Rear-End Crash Risk for Treatments 17-24

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

9.3.3.1.3 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 9-20 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 9-20: Cumulative Rear-End ORCI Values for Treatments 17-24

	Base	T17	T18	T19	T20	T21	T22	T23	T24
Cumulative ORCI	---	1.017	0.406	1.196	0.858	0.327	0.869	1.112	0.770

9.3.3.2 Lane-Change Crash Risk

The effects that Treatments 17-24 have on lane-change crash risk are reviewed in this section. Figures 9-37 and 9-38 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 (Segment1), stations 15-21 (Segment 2), and stations 32-52 (Segment 3).

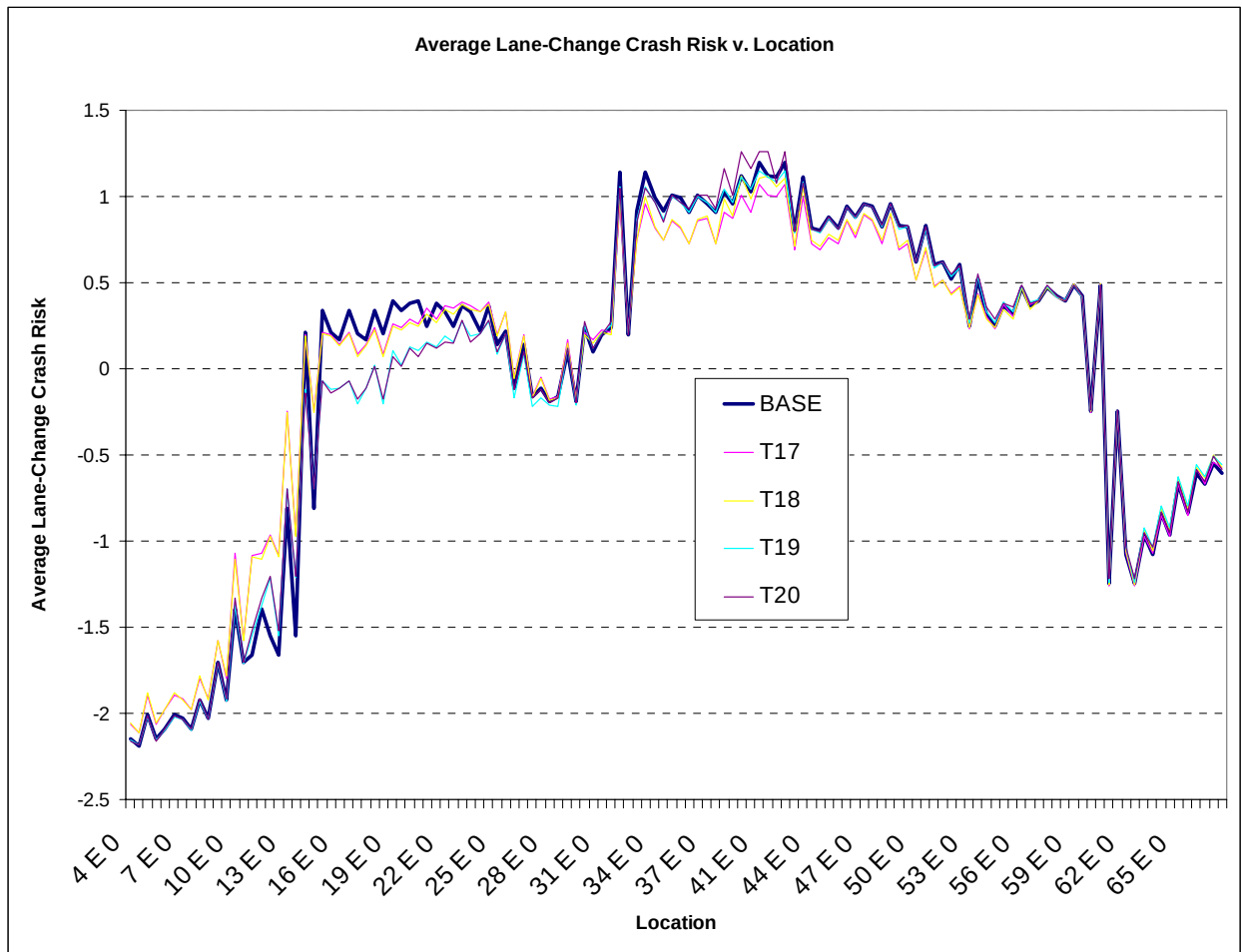


Figure 9-37: Average Lane-Change Crash Risk v. Location for Treatments 17-20

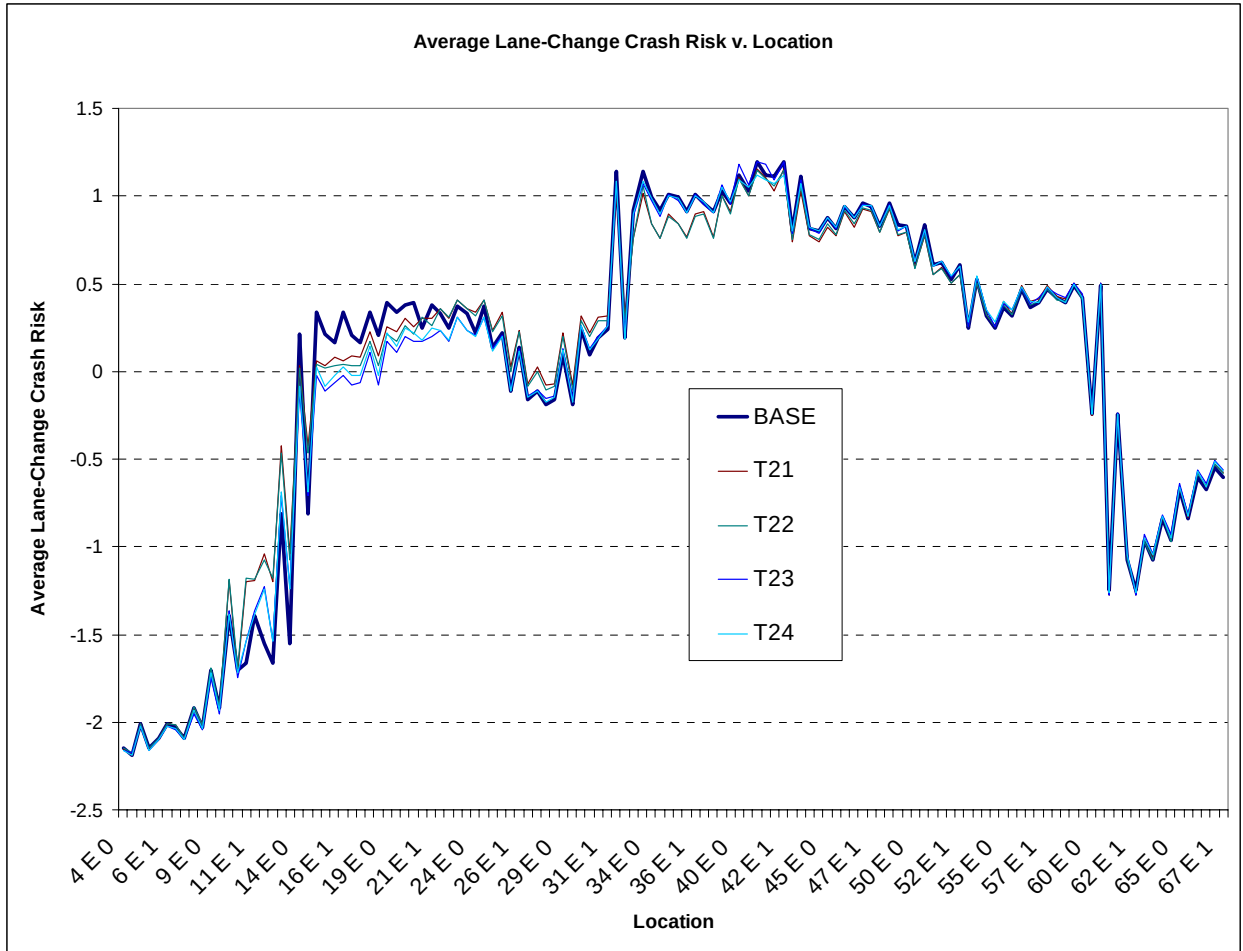


Figure 9-38: Average Lane-Change Crash Risk v. Location for Treatments 21-24

9.3.3.2.1 Segment 1: Stations 6 – 14

Like the 1st segment in the rear-end crash risk analysis, there is a general increase in lane-change crash risk for the treatments over the 1st segment. Again, Treatments 17, 18, 21 and 22 have the greatest increase, while the changes due to the other treatments are minimal. Figures 9-39 and 9-40 magnify stations 6-14 for Treatments 17-20 and 21-24, respectively.

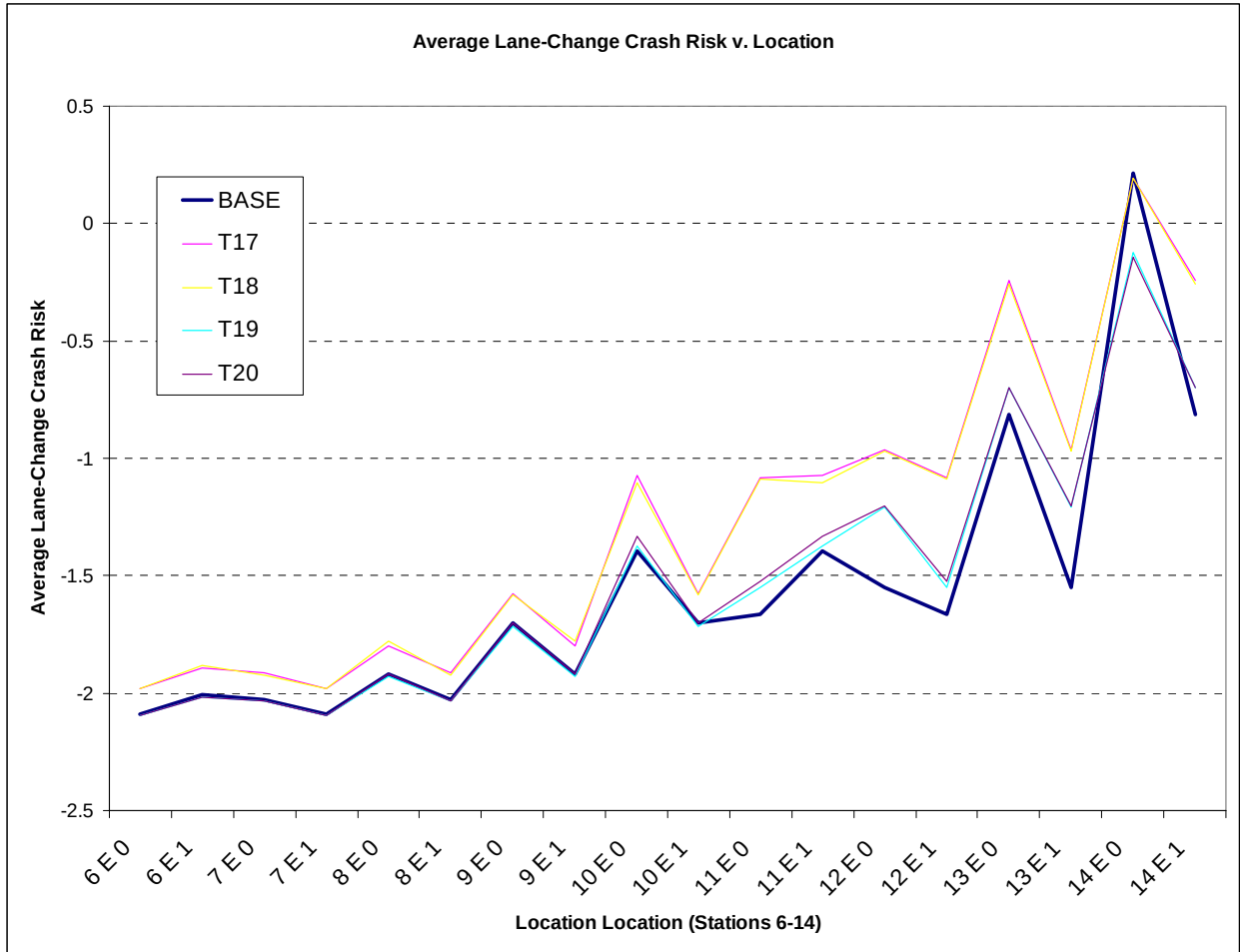


**Figure 9-39: Average Lane-Change Crash Risk v. Location for Treatments 17-20
for 1st Section - Magnified**

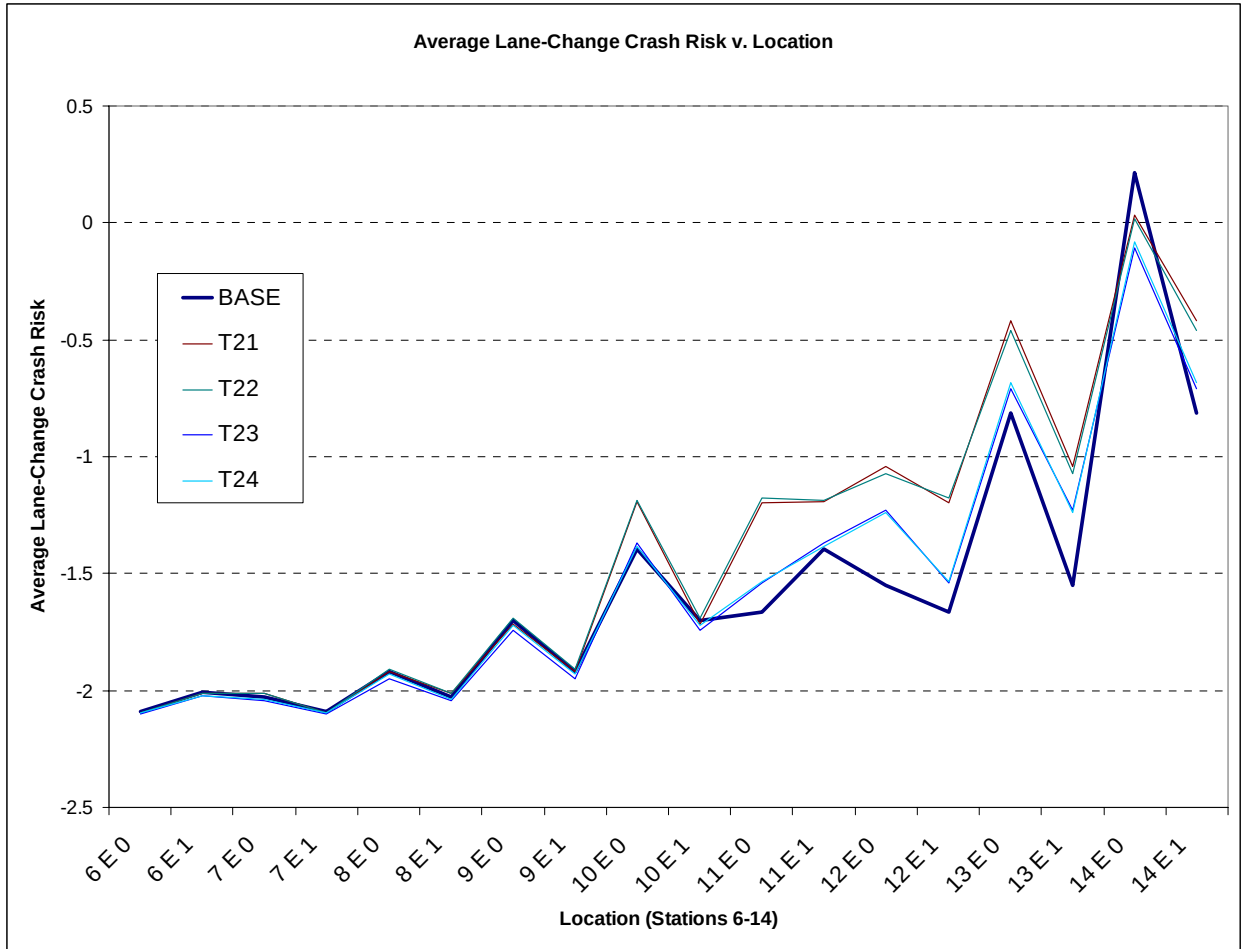


Figure 9-40: Average Lane-Change Crash Risk v. Location for Treatments 21-24 for 1st Section - Magnified

Table 9-21 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 9-21: 1st Section: Summary of Difference in Lane-Change Crash Risk for
Treatments 17-24**

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

9.3.3.2.2 Segment 2: Stations 15 – 21

The 2nd segment occurs over stations 15-21. Figures 9-41 and 9-42 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.

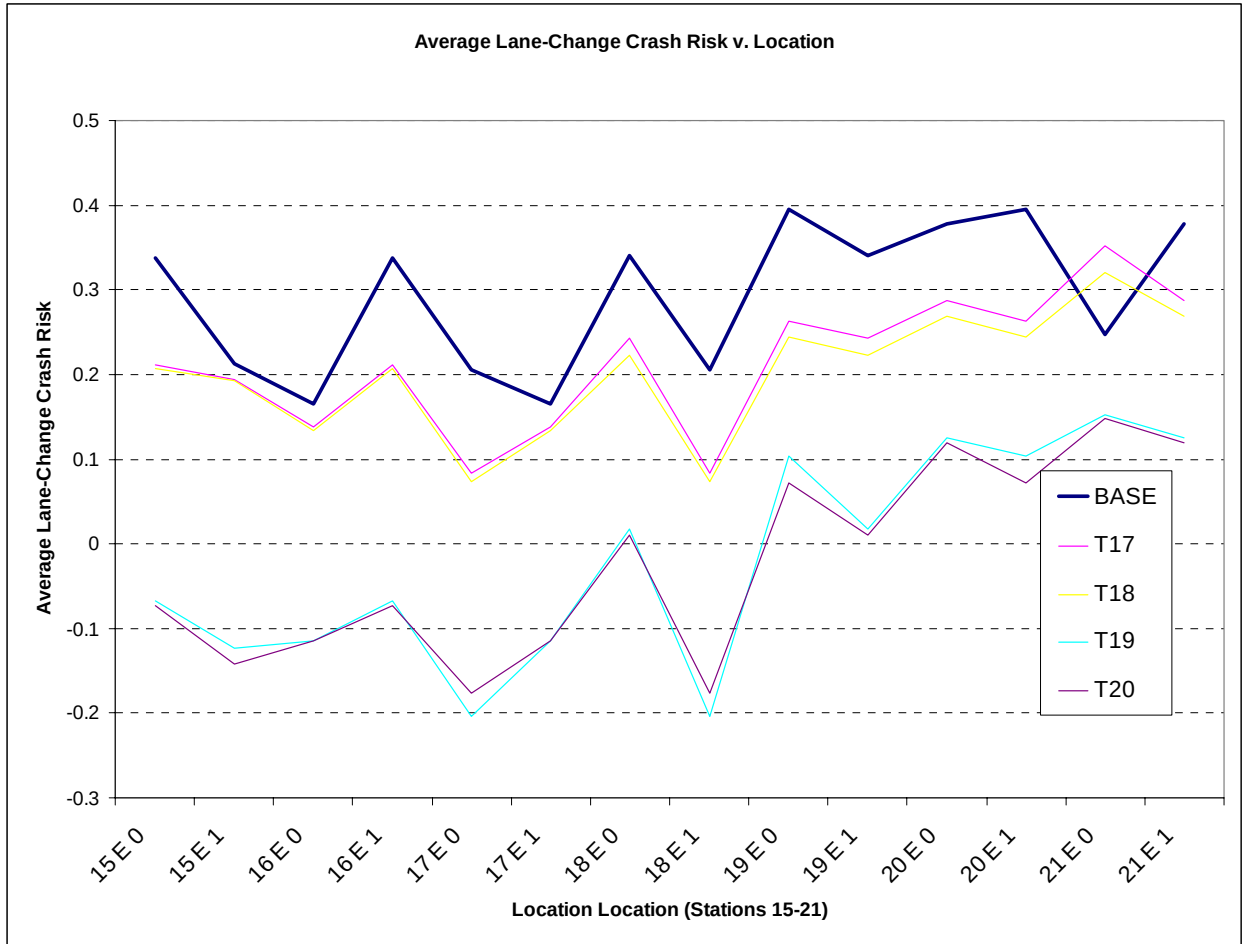


**Figure 9-41: Average Lane-Change Crash Risk v. Location for Treatments 17-20
for 2nd Section - Magnified**

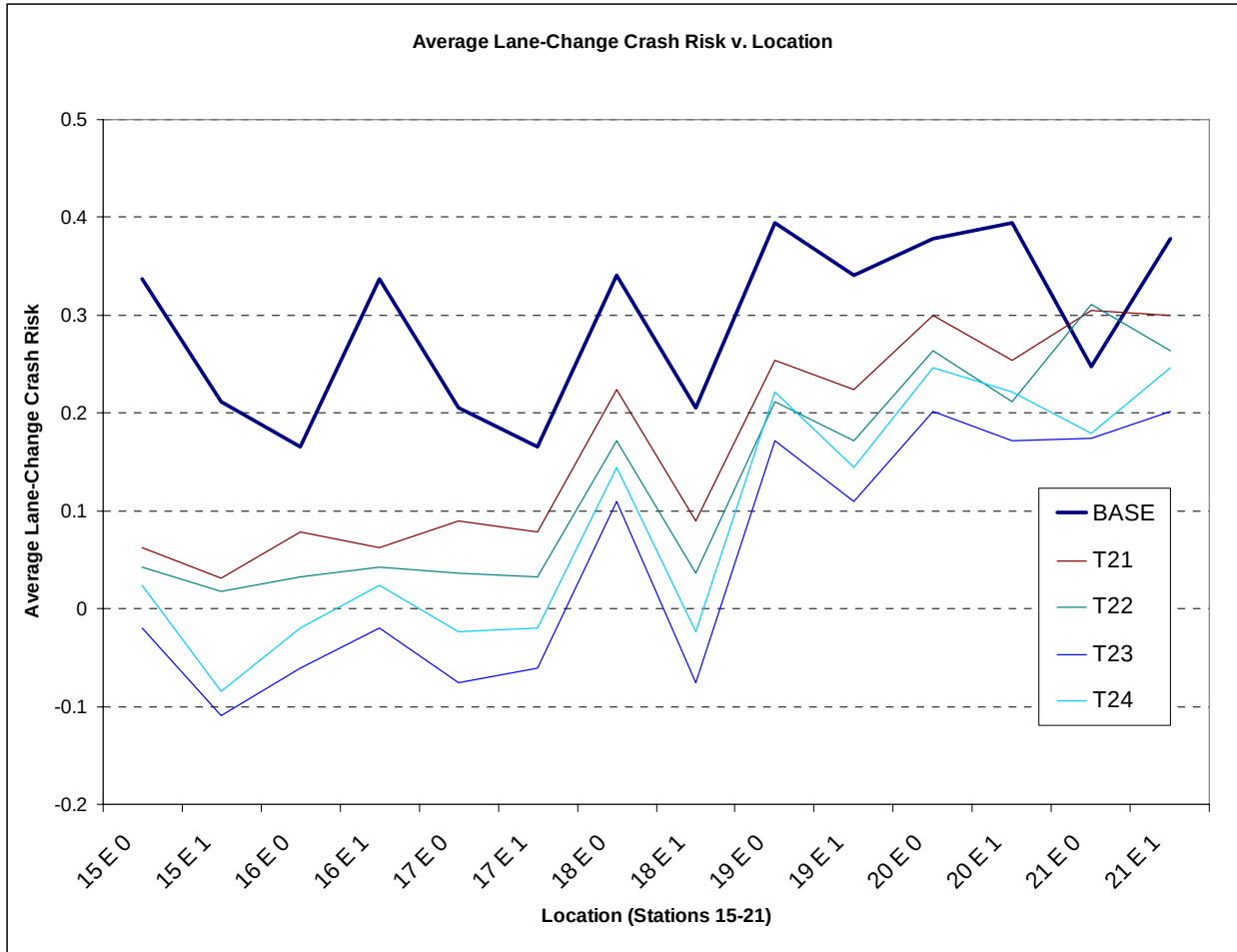


Figure 9-42: Average Lane-Change Crash Risk v. Location for Treatments 21-24 for 2nd Section - Magnified

Table 9-22 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 9-22: 2nd Section: Summary of Difference in Lane-Change Crash Risk for Treatments 17-24

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

9.3.3.2.3 Segment 3: Stations 32 - 52

The 3rd segment occurs over stations 32-52. Figures 9-43 and 9-44 magnify stations 32-52 for Treatments 17-20 and 21-24, respectively.

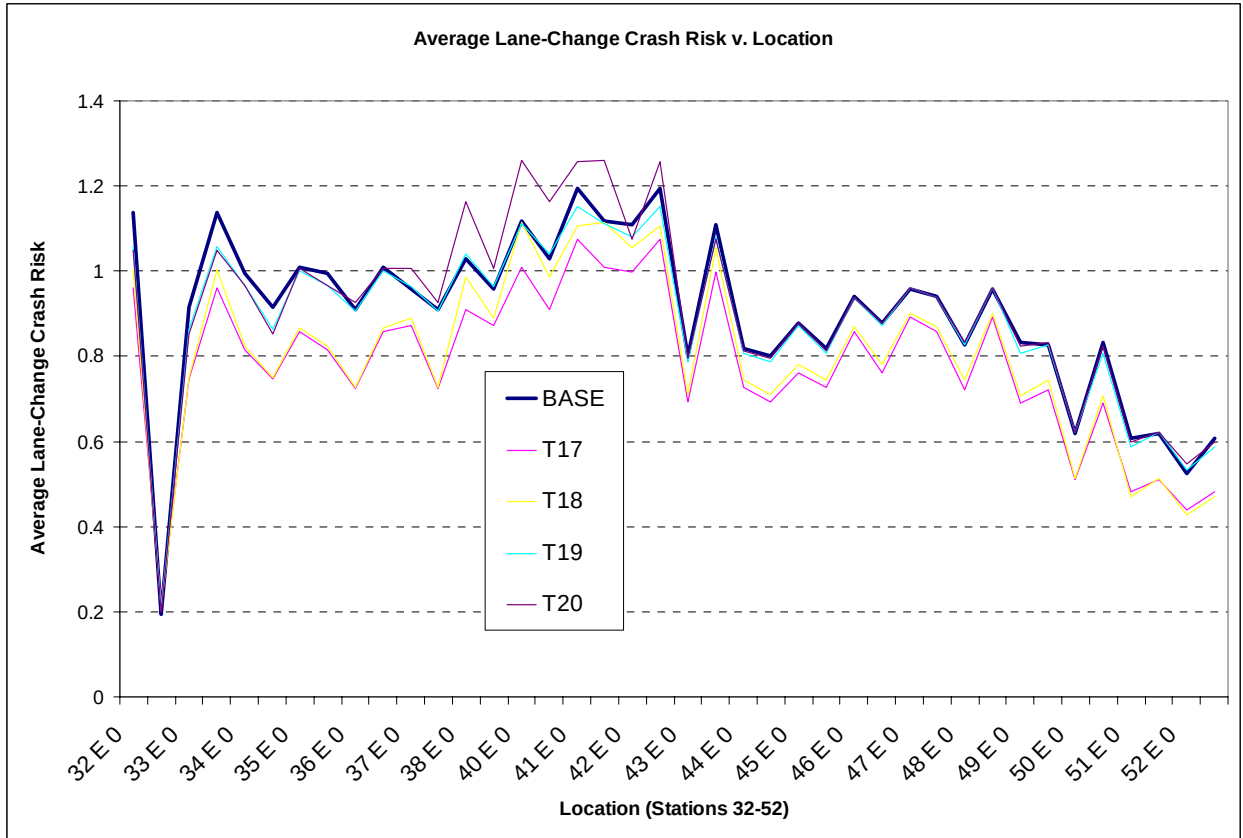


**Figure 9-43: Average Lane-Change Crash Risk v. Location for Treatments 17-20
for 3rd Section - Magnified**

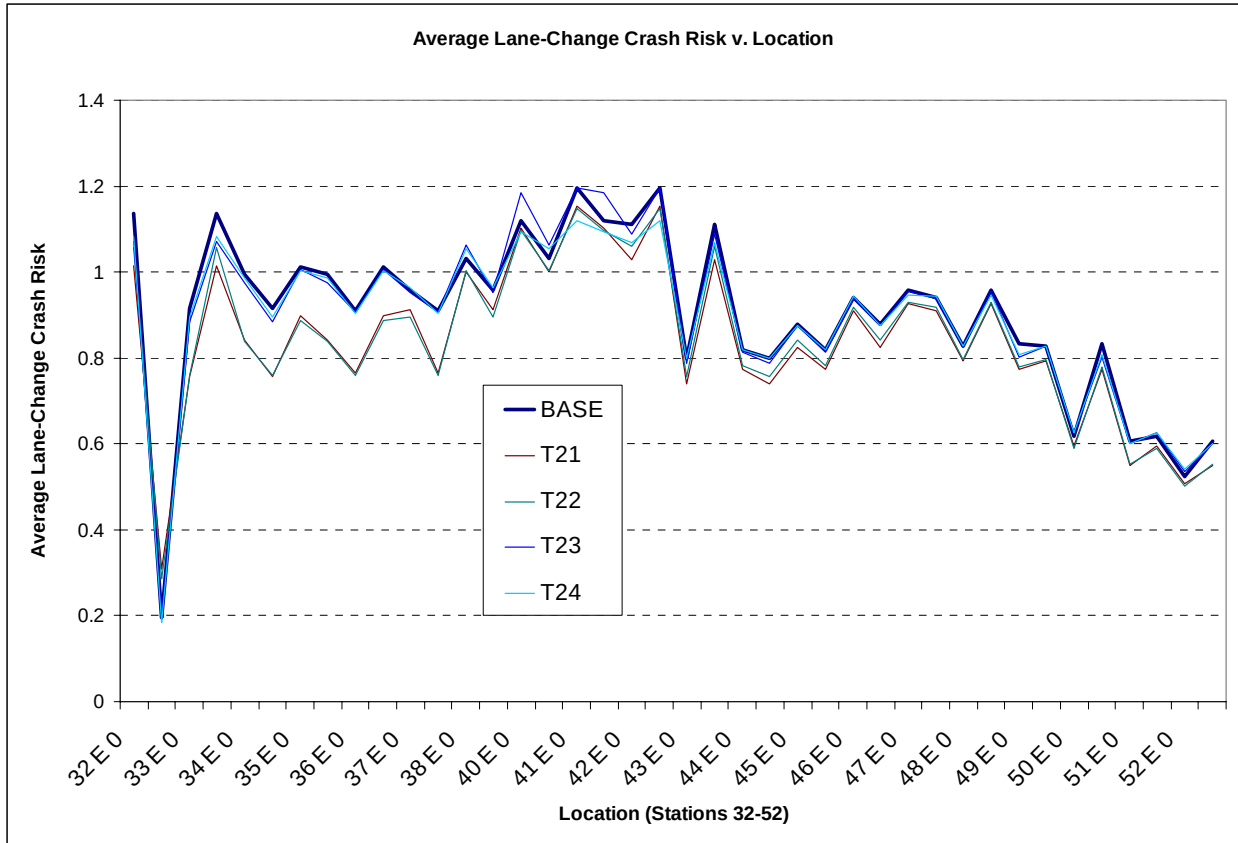


Figure 9-44: Average Lane-Change Crash Risk v. Location for Treatments 21-24 for 3rd Section - Magnified

Table 9-23 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 9-23: 3rd Section: Summary of Difference in Lane-Change Crash Risk for Treatments 17-24

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

9.3.3.2.4 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 9-24: Cumulative Lane-Change ORCI Values for Treatments 17-24

	Base	T17	T18	T19	T20	T21	T22	T23	T24
Cumulative ORCI	---	0.727	0.109	4.191	3.122	1.304	1.699	2.966	2.538

The lane-change crash profile over the 5-min time steps for Treatment 19 at Station 18 E 1 is shown in Figure 9-45. 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.

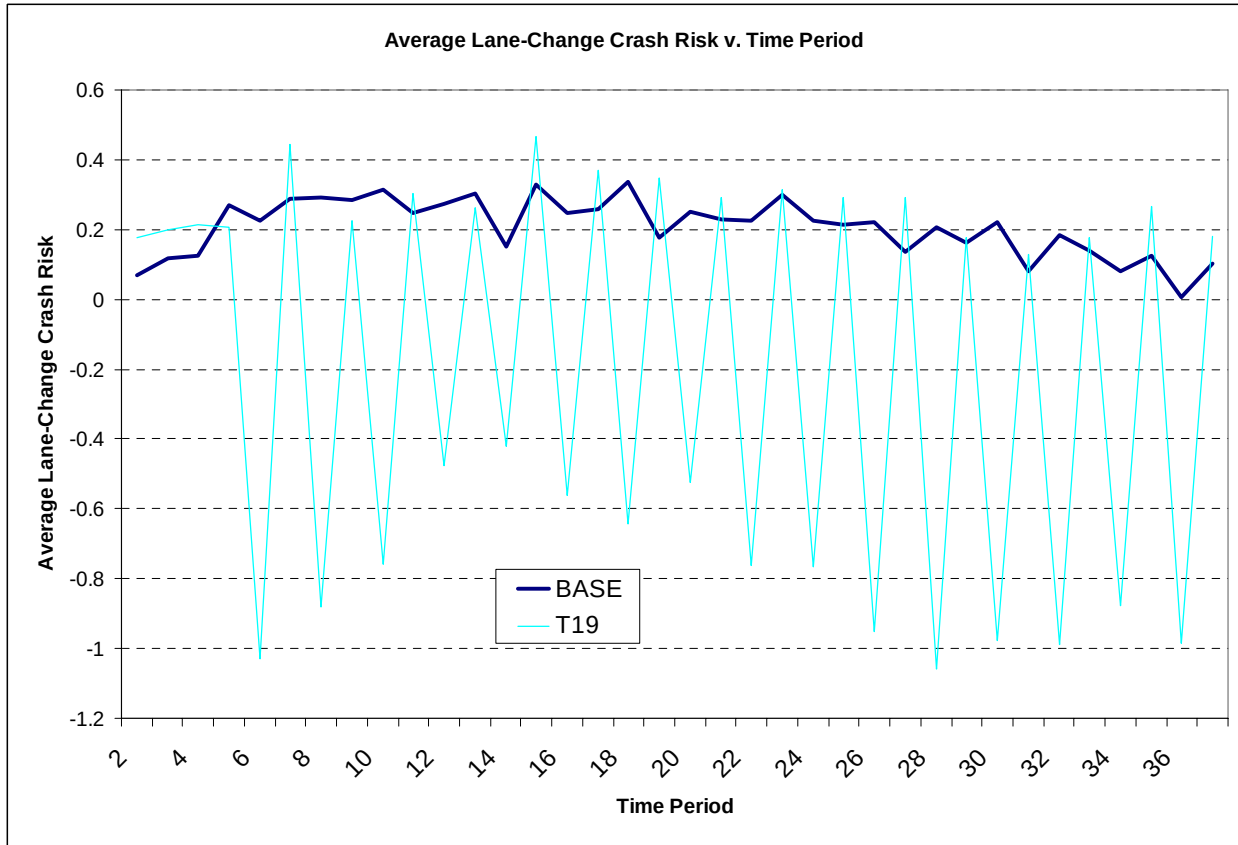


Figure 9-45: Lane-Change Crash Profile at Station 18 E 1 for Treatment 19

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.

9.3.3.3 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. These travel times were compared with the travel times collected from the BASE case replications, as shown in Table 9-25.

Table 9-25: Analysis of Travel Time for Treatment 19

	Test Case ID	
	Base	T19
Average Travel Time (vehicle-hours)	16391.447	16451.873
Travel Time Increase	---	60.425
Percent Increase	---	0.369
T-Statistic (Significance of Change)	---	0.875

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.

9.4 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.

9.4.1 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 9-26 displays the different factors that make up Treatments 01-08, which are reviewed in this section.

Table 9-26: Description of Treatments 01-08

Treatment ID	Speed Change Implementation	Speed Zone Threshold	Speed Change Distance	Speed Change Time Period
1	-10	5 mph	Speed Zone	5 min
2	-10	5 mph	Speed Zone	10 min
3	-10	5 mph	1/2 Speed Zone	5 min
4	-10	5 mph	1/2 Speed Zone	10 min
5	-10	2.5 mph	Speed Zone	5 min
6	-10	2.5 mph	Speed Zone	10 min
7	-10	2.5 mph	1/2 Speed Zone	5 min
8	-10	2.5 mph	1/2 Speed Zone	10 min

9.4.1.1 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 9-46 shows the average rear-end crash risk plotted over each location for Treatments 01-08.

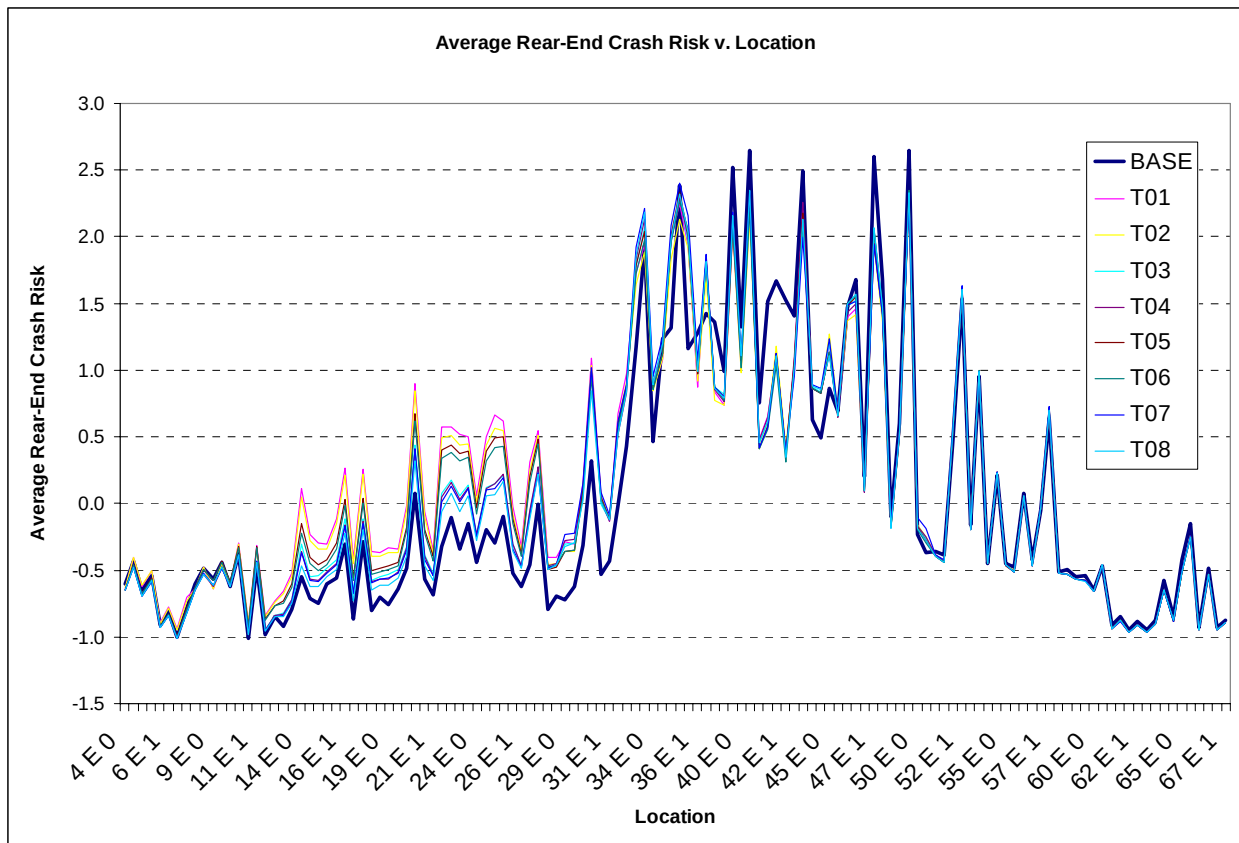


Figure 9-46: Average Rear-End Crash Risk v. Location for Treatments 01-08

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.

9.4.1.1.1 Segment 1: Stations 10 – 34

Figure 9-47 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.

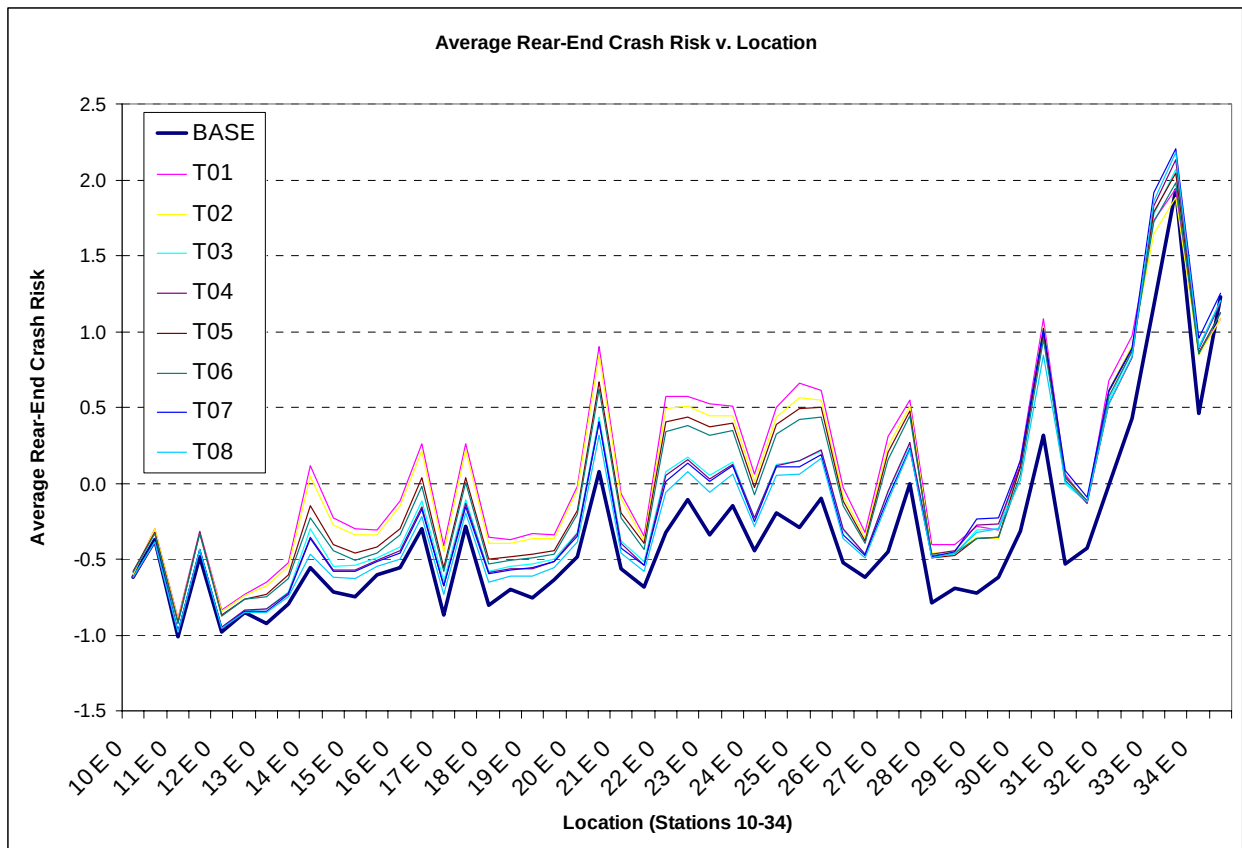


Figure 9-47: Average Rear-End Crash Risk v. Location for Treatments 01-08 for 1st Segment

Table 9-27 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 is 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 9-27: 1st Segment: Summary of Difference in Rear-End Crash Risk for Treatments 01-08

	Test Case ID								
	Base	T01	T02	T03	T04	T05	T06	T07	T08
Average Crash Risk (Stations 10 to 34)	-0.347	0.101	0.060	-0.100	-0.106	0.009	-0.021	-0.102	-0.147
Crash Risk Benefit	---	-0.448	-0.407	-0.248	-0.241	-0.357	-0.326	-0.246	-0.201
T-Statistic (Significance of Change)	---	3.723	3.418	2.029	1.962	2.929	2.704	1.961	1.621
ORCI	---	-22.407	-20.351	-12.388	-12.056	-17.835	-16.304	-12.277	-10.033

9.4.1.1.2 Segment 2: Stations 38 – 43

Figure 9-48 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 9-28. 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.

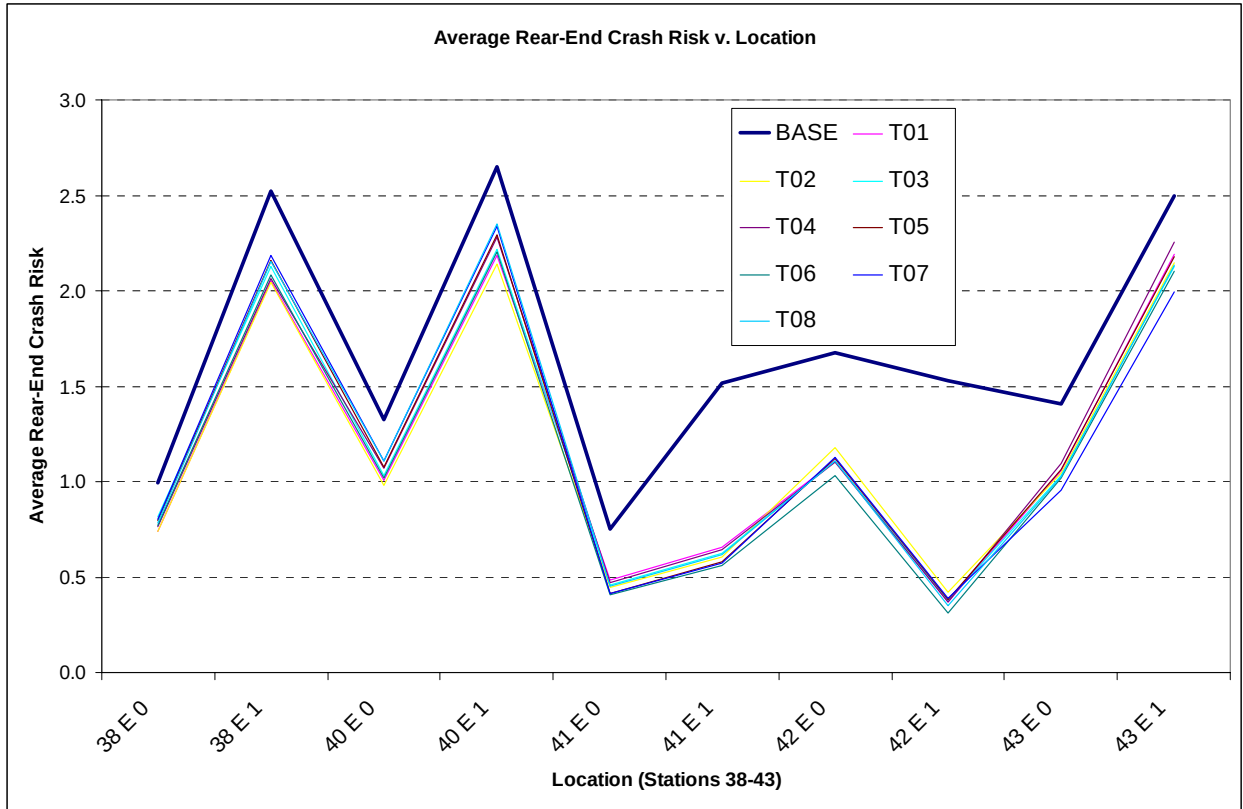


Figure 9-48: Average Rear-End Crash Risk v. Location for Treatments 01-08 for 2nd Segment

Table 9-28: 2nd Segment: Summary of Difference in Rear-End Crash Risk for Treatments 01-08

	Test Case ID								
	Base	T01	T02	T03	T04	T05	T06	T07	T08
Average Crash Risk (Stations 38 to 43)	1.685	1.185	1.175	1.192	1.212	1.205	1.151	1.189	1.208
Crash Risk Benefit	---	0.500	0.510	0.494	0.473	0.480	0.534	0.496	0.477
T-Statistic (Significance of Change)	---	1.642	1.690	1.613	1.523	1.530	1.730	1.597	1.522
ORCI	---	5.004	5.103	4.936	4.733	4.799	5.341	4.962	4.769

9.4.1.1.3

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 9-47. The cumulative ORCI values for each treatment is shown in Table 9-29, 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 9-29: Cumulative Rear-End ORCI Values for Treatments 01-08

	Base	T01	T02	T03	T04	T05	T06	T07	T08
Cumulative ORCI	---	-17.40	-15.25	-7.45	-7.32	-13.04	-10.96	-7.31	-5.26

9.4.1.2 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 9-49 shows the average lane-change crash risk plotted over each location for Treatments 01-08.

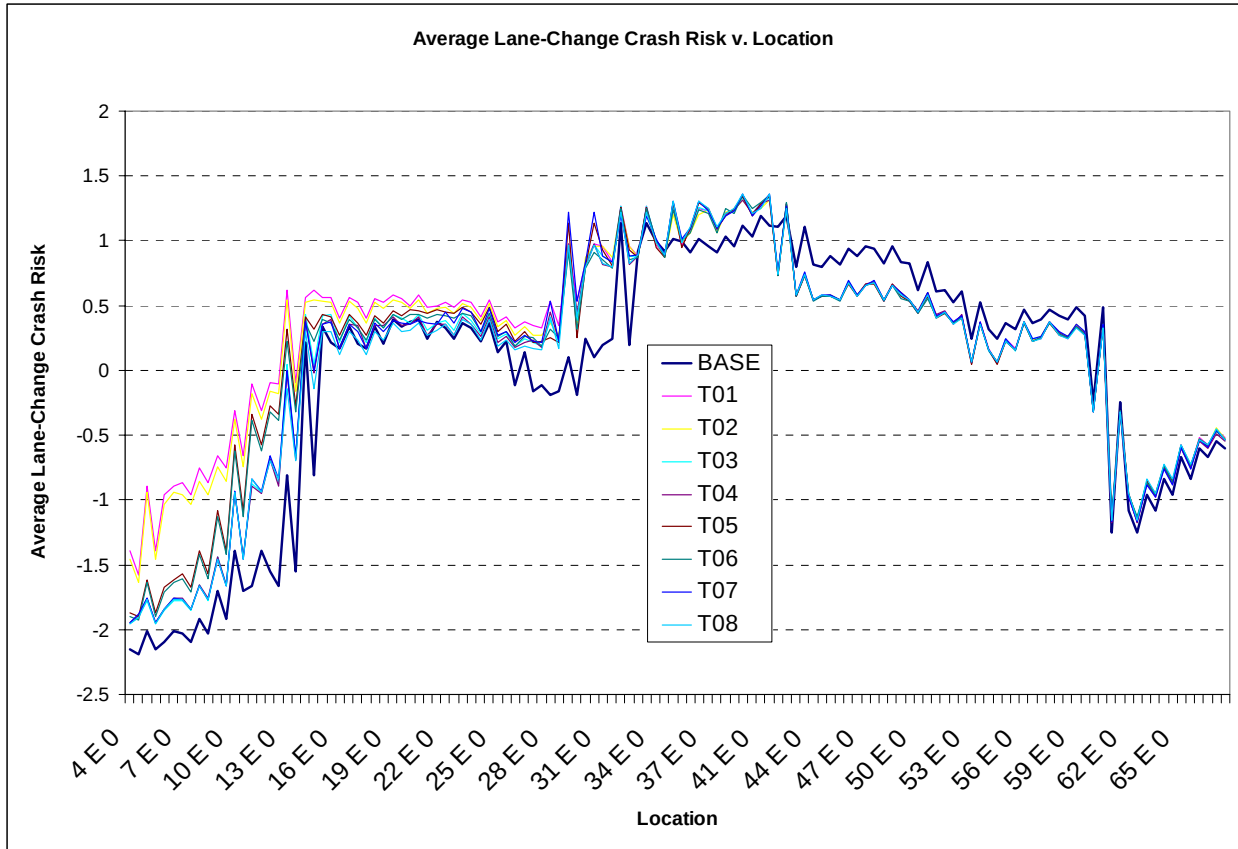


Figure 9-49: Average Lane-Change Crash Risk v. Location for Treatments 01-08

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.

9.4.1.2.1 Segment 1: Stations 4 - 40

Figure 9-50 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.

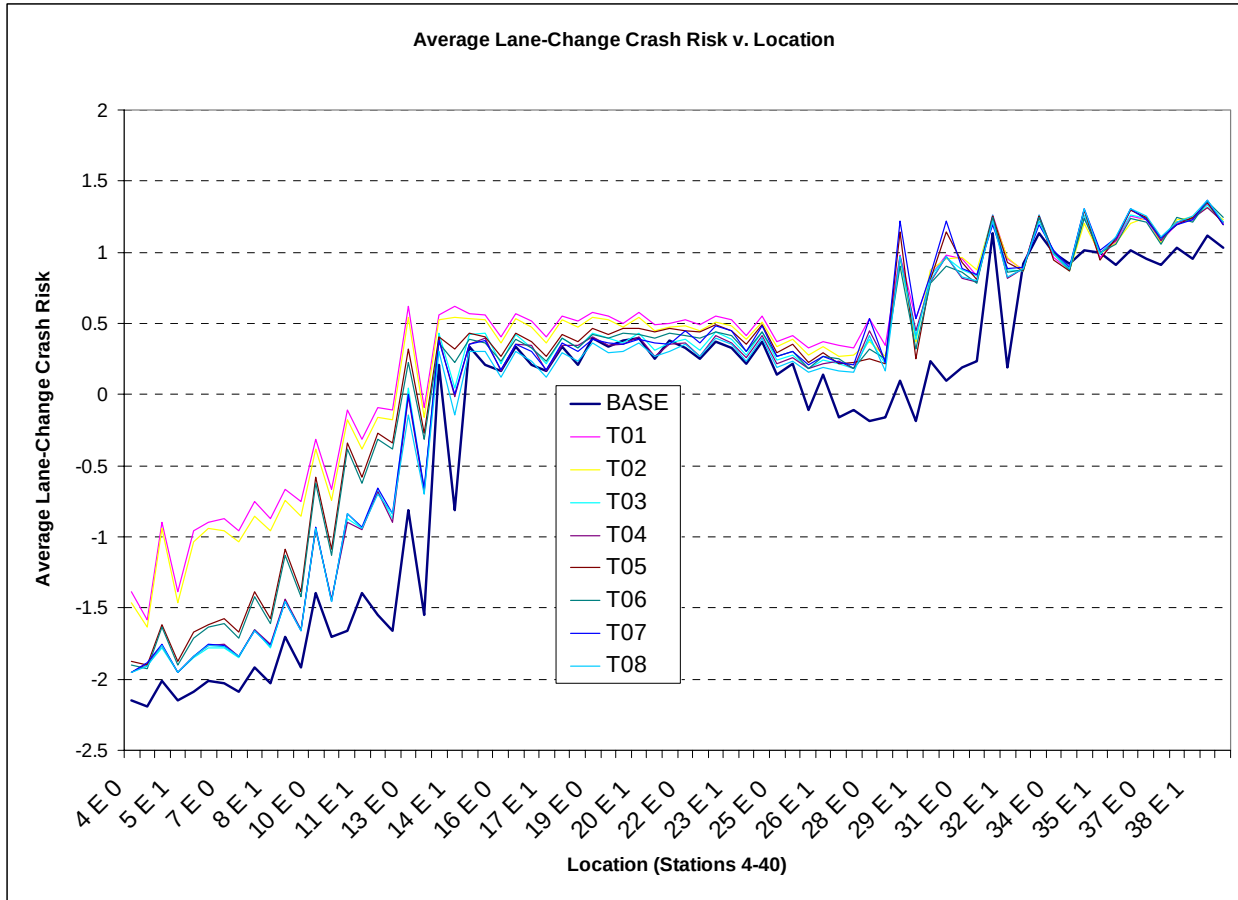


Figure 9-50: Average Lane-Change Crash Risk v. Location for Treatments 01-08 for Segment 1

Table 9-30 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 9-30: 1st Segment: Summary of Difference in Lane-Change Crash Risk for Treatments 01-08

	Test Case ID								
	Base	T01	T02	T03	T04	T05	T06	T07	T08
Average Crash Risk (Stations 4 to 40)	-0.208	0.343	0.300	0.068	0.058	0.166	0.133	0.081	0.037
Crash Risk Benefit	---	-0.551	-0.508	-0.276	-0.266	-0.374	-0.341	-0.288	-0.245
T-Statistic (Significance of Change)	---	3.590	3.281	1.587	1.530	2.229	2.031	1.653	1.413
ORCI	---	-39.637	-36.591	-19.892	-19.129	-26.946	-24.546	-20.767	-17.652

9.4.1.2.2 Segment 2: Stations 42 - 50

Figure 9-51 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 9-31 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.

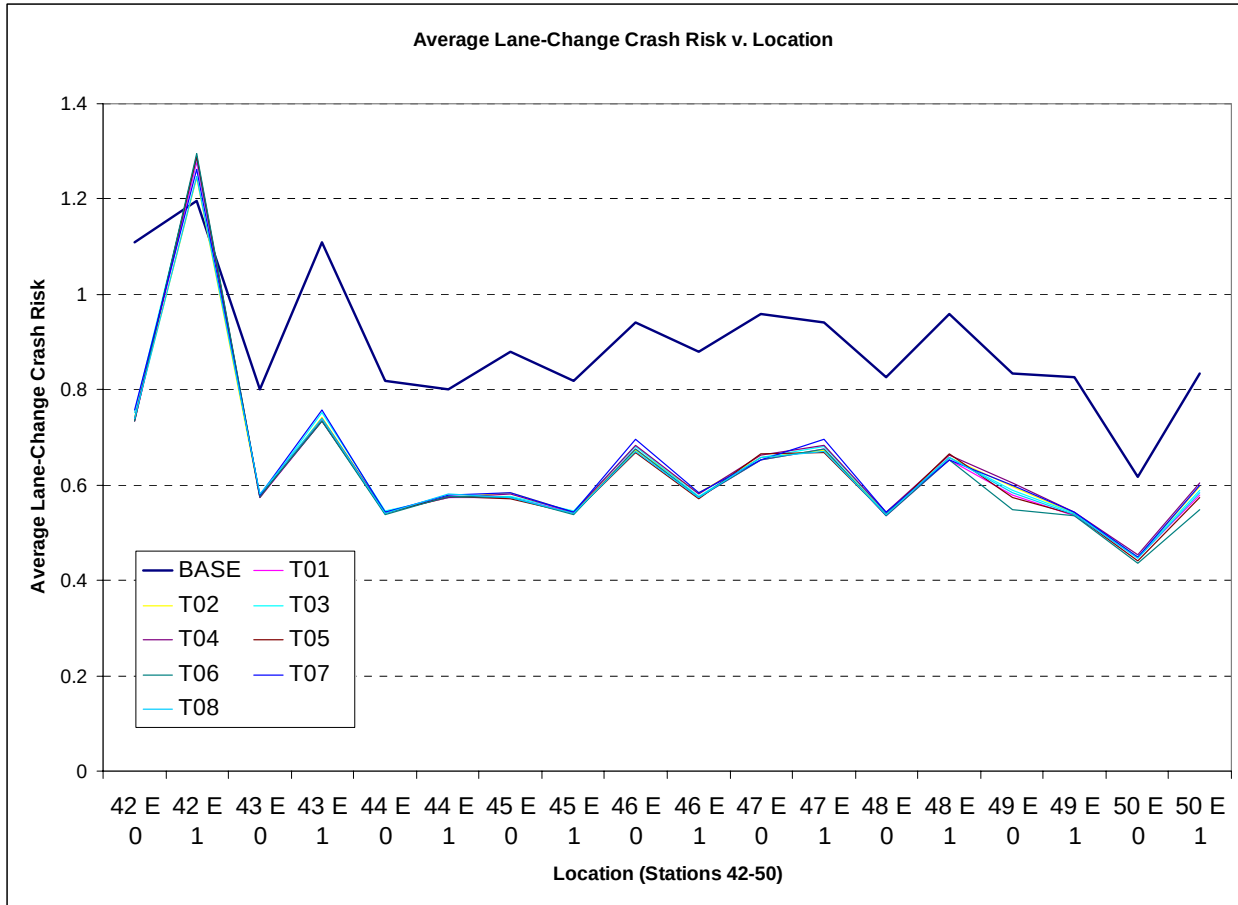


Figure 9-51: Average Lane-Change Crash Risk v. Location for Treatments 01-08 for Segment 2

Table 9-31: 2nd Segment: Summary of Difference in Lane-Change Crash Risk for Treatments 01-08

	Test Case ID								
	Base	T01	T02	T03	T04	T05	T06	T07	T08
Average Crash Risk (Stations 42 to 50)	0.897	0.637	0.640	0.642	0.645	0.637	0.633	0.646	0.637
Crash Risk Benefit	---	0.260	0.257	0.255	0.252	0.260	0.264	0.251	0.260
T-Statistic (Significance of Change)	---	4.934	5.020	4.806	4.776	4.885	4.901	4.818	5.050
ORCI	---	4.678	4.627	4.582	4.542	4.680	4.747	4.525	4.679

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 9-32 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 9-32: Cumulative Lane-Change ORCI Values for Treatments 01-08

	Base	T17	T18	T19	T20	T21	T22	T23	T24
Cumulative ORCI	---	-34.96	-31.96	-15.31	-14.59	-22.27	-19.80	-16.24	-12.97

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.

9.4.2 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 9-33 displays the different factors that make up Treatments 01-08, which are reviewed in this section.

Table 9-33: Description of Treatments 09-16

Treatment ID	Speed Change Implementation	Speed Zone Threshold	Speed Change Distance	Speed Change Time Period
9	-5	5 mph	Speed Zone	5 min
10	-5	5 mph	Speed Zone	10 min
11	-5	5 mph	1/2 Speed Zone	5 min
12	-5	5 mph	1/2 Speed Zone	10 min
13	-5	2.5 mph	Speed Zone	5 min
14	-5	2.5 mph	Speed Zone	10 min
15	-5	2.5 mph	1/2 Speed Zone	5 min
16	-5	2.5 mph	1/2 Speed Zone	10 min

9.4.2.1 Rear-End Crash Risk

The rear-end crash risk was analyzed for Treatments 09-16. Figure 9-52 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.

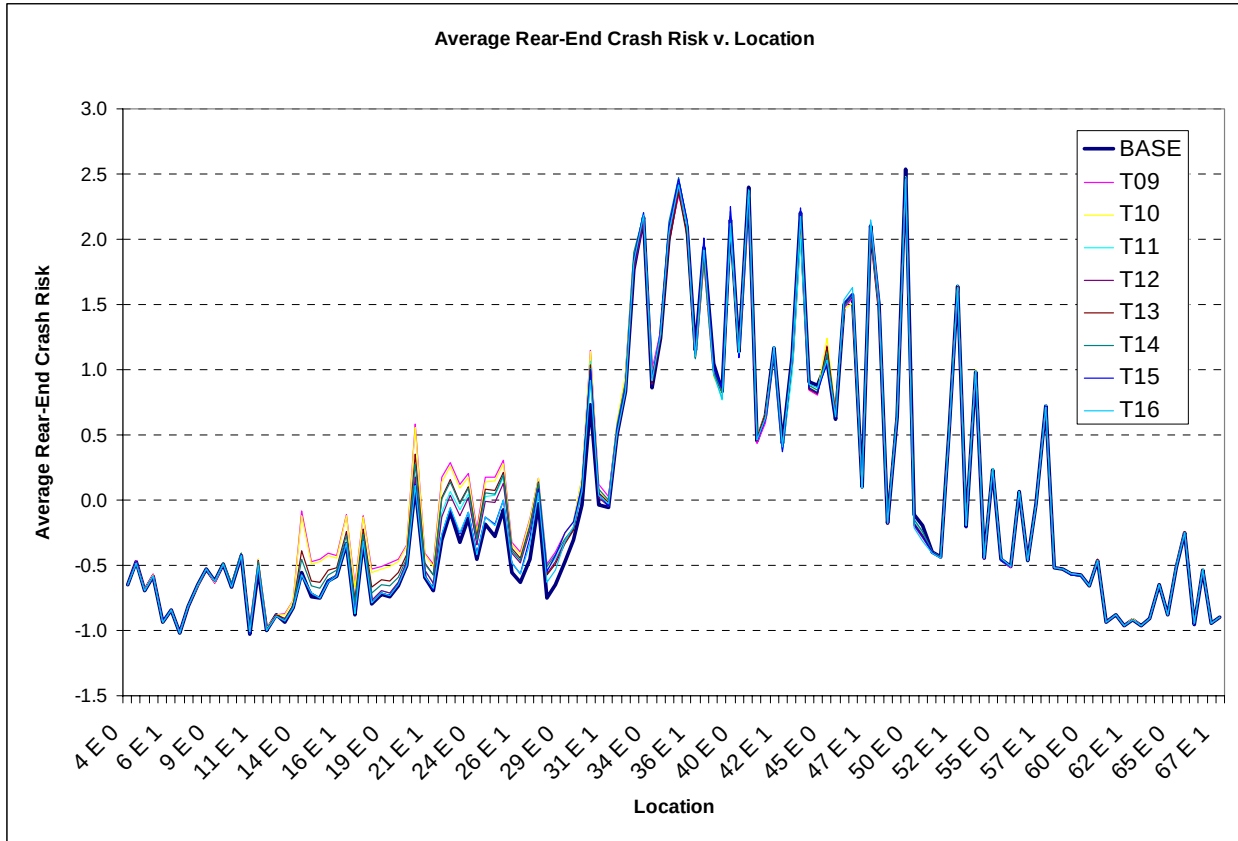


Figure 9-52: Average Rear-End Crash Risk v. Location for Treatments 09-16

9.4.2.1.1 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 9-53 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 9-34 summarizes the t-tests performed for each treatment against the BASE case. As shown, none of these changes are significant.

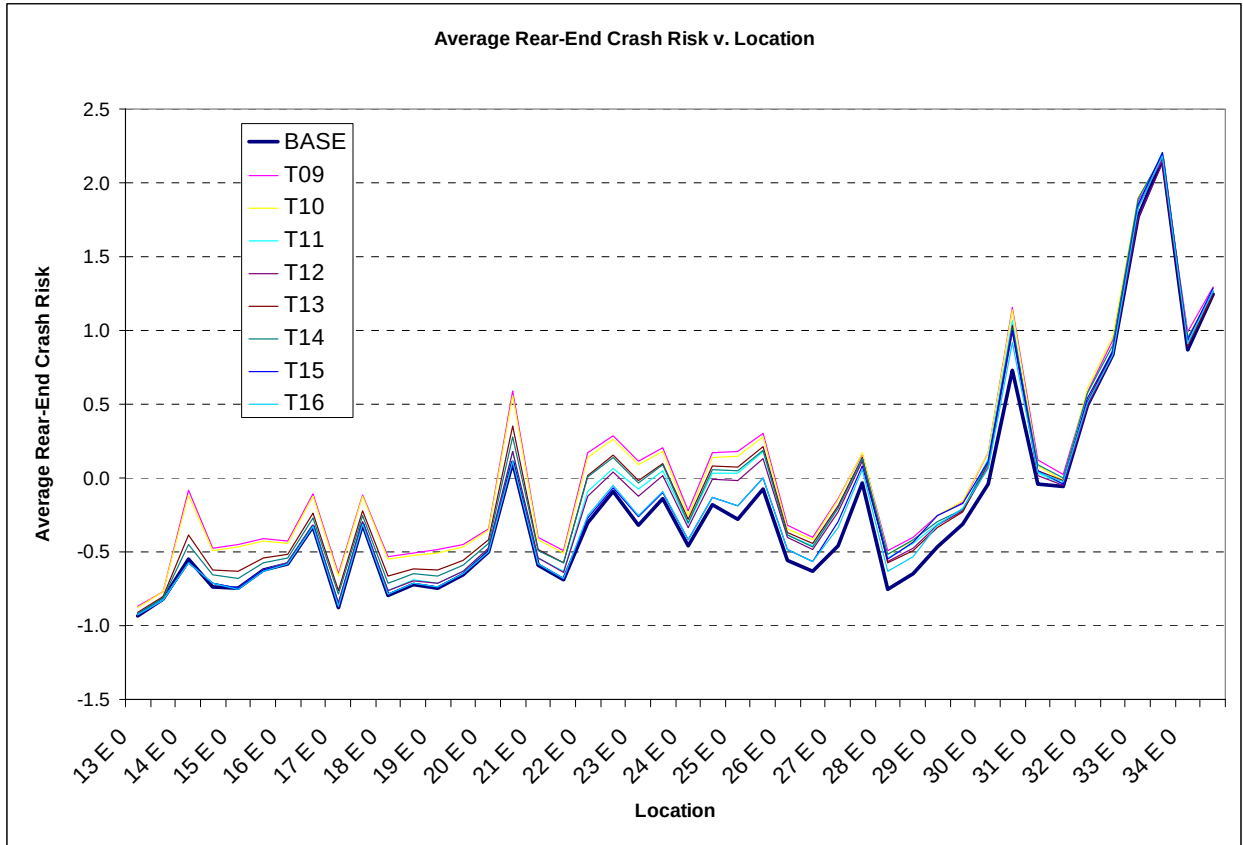


Figure 9-53: Average Rear-End Crash Risk v. Location for Treatments 09-16 for Segment 1

Table 9-34: 1st Segment: Summary of Difference in Rear-End Crash Risk for Treatments 09-16

	Test Case ID								
	Base	T09	T10	T11	T12	T13	T14	T15	T16
Average Crash Risk (Stations 13 to 34)	-0.201	0.034	0.020	-0.101	-0.121	-0.061	-0.068	-0.144	-0.158
Crash Risk Benefit	---	-0.236	-0.221	-0.100	-0.081	-0.141	-0.133	-0.057	-0.044
T-Statistic (Significance of Change)	---	1.632	1.523	0.673	0.550	0.965	0.905	0.384	0.295
ORCI	---	-10.368	-9.712	-4.394	-3.545	-6.185	-5.866	-2.512	-1.919

The second segment occurs between stations 36 and 50 and is positively affected by the application of Treatments 09-16. Figure 9-54 shows the average rear-end crash risk for each treatment over the length of segment 2. Table 9-35 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.

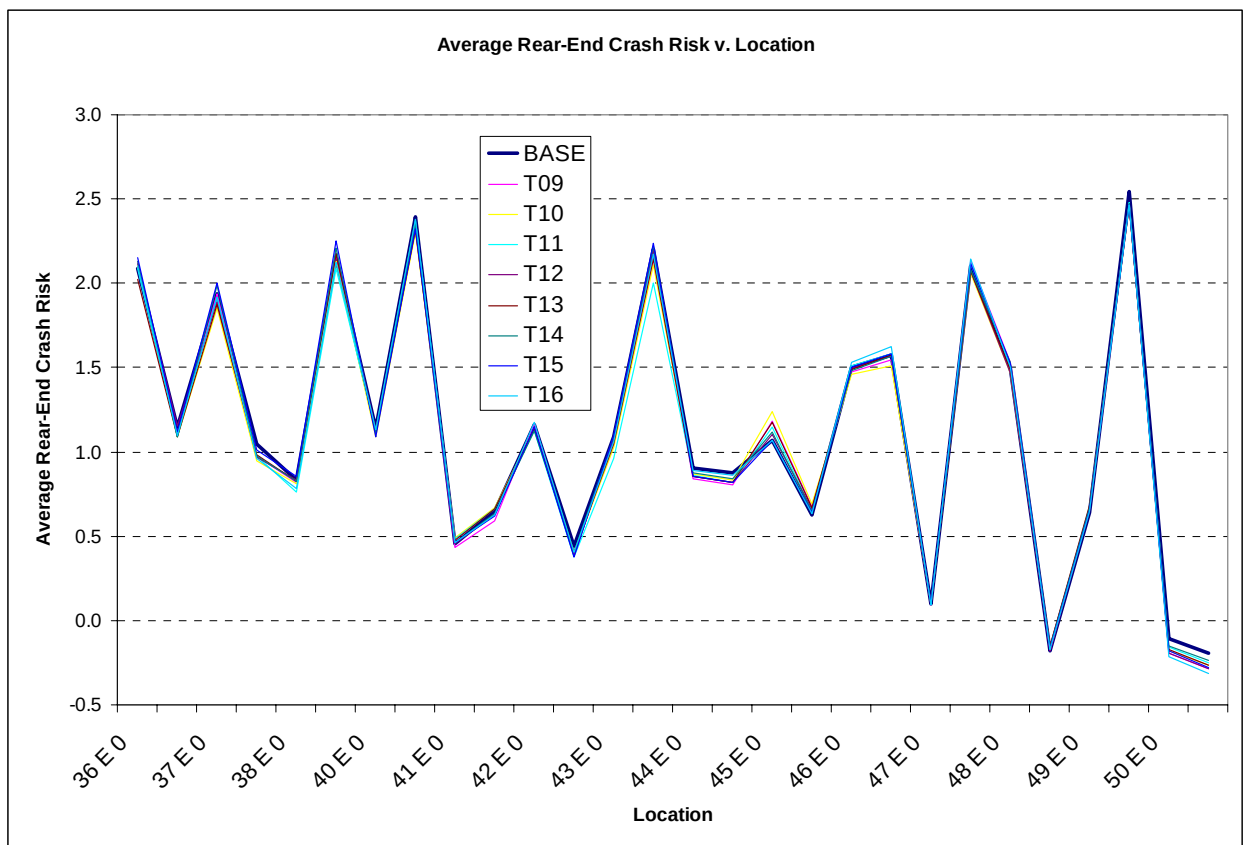


Figure 9-54: Average Rear-End Crash Risk v. Location for Treatments 09-16 for Segment 2

Table 9-35: 2nd Segment: Summary of Difference in Rear-End Crash Risk for Treatments 09-16

	Test Case ID								
	Base	T09	T10	T11	T12	T13	T14	T15	T16
Average Crash Risk (Stations 36 to 50)	1.131	1.112	1.104	1.104	1.110	1.109	1.124	1.124	1.113
Crash Risk Benefit	---	0.019	0.027	0.027	0.021	0.022	0.007	0.008	0.018
T-Statistic (Significance of Change)	---	0.089	0.129	0.130	0.099	0.104	0.035	0.036	0.085
ORCI	---	0.525	0.750	0.759	0.586	0.605	0.205	0.212	0.506

9.4.2.1.3 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 9-36 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 9-36: Cumulative Rear-End ORCI Values for Treatments 09-16

	Base	T09	T10	T11	T12	T13	T14	T15	T16
Cumulative ORCI	---	-9.842	-8.963	-3.635	-2.960	-5.580	-5.660	-2.300	-1.413

9.4.2.2 Lane-Change Crash Risk

The lane-change crash risk was analyzed for Treatments 09-16. Figure 9-55 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.

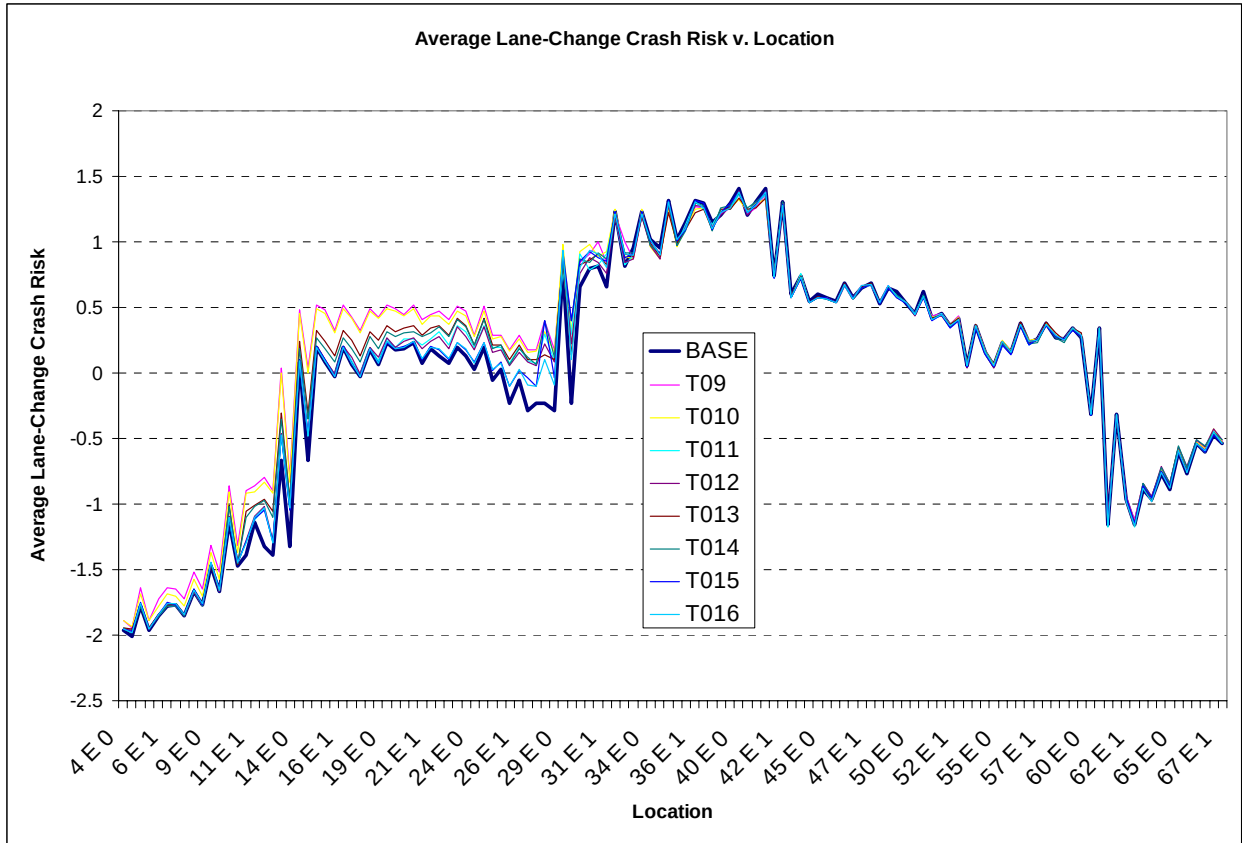


Figure 9-55: Average Lane-Change Crash Risk v. Location for Treatments 09-16

9.4.2.2.1 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 9-56 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 9-37 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.

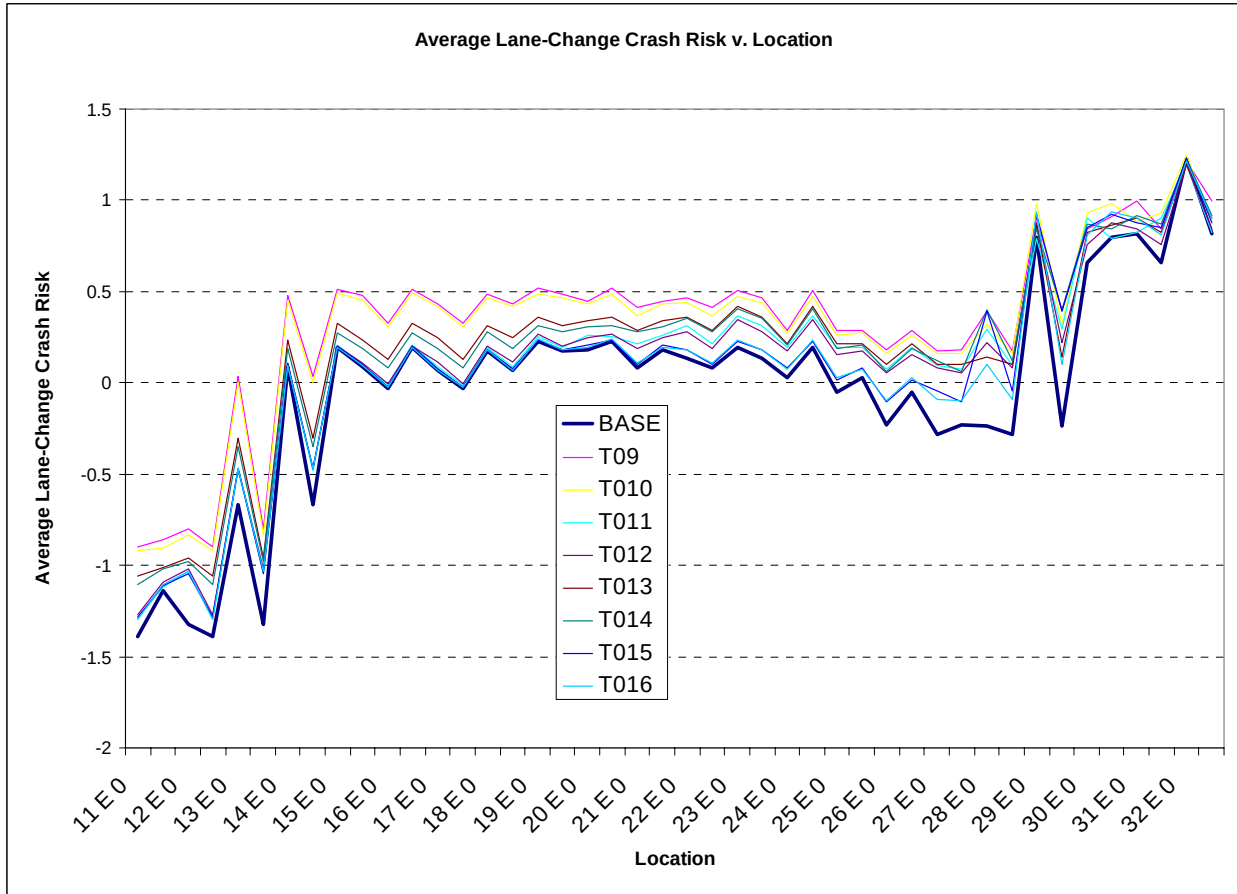


Figure 9-56: Average Lane-Change Crash Risk v. Location for Treatments 09-16 for Segment 1

Table 9-37: 1st Segment: Summary of Difference in Lane-Change Crash Risk for Treatments 09-16

	Test Case ID								
	Base	T09	T10	T11	T12	T13	T14	T15	T16
Average Crash Risk (Stations 11 to 32)	-0.024	0.326	0.306	0.118	0.109	0.189	0.180	0.079	0.063
Crash Risk Benefit	---	-0.351	-0.331	-0.142	-0.133	-0.214	-0.204	-0.103	-0.088
T-Statistic (Significance of Change)	---	2.999	2.805	1.151	1.086	1.782	1.684	0.831	0.708
ORCI	---	-15.429	-14.553	-6.266	-5.863	-9.400	-8.979	-4.553	-3.861

The second segment occurs between stations 33 and 42 and is positively affected by the application of Treatments 09-16. Figure 9-57 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 9-38 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.

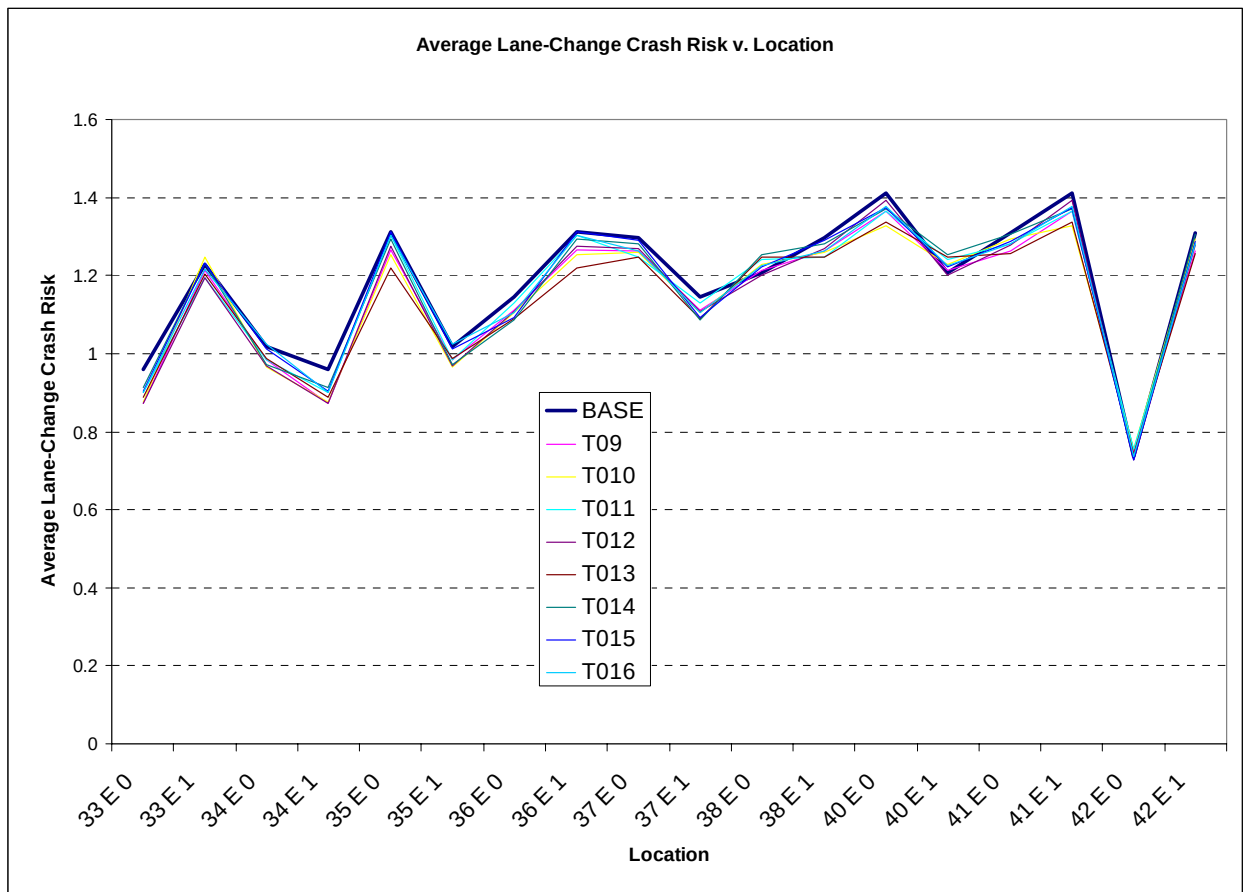


**Figure 9-57: Average Lane-Change Crash Risk v. Location for Treatments 09-16
for Segment 2**

Table 9-38: 2nd Segment: Summary of Difference in Lane-Change Crash Risk for Treatments 09-16

	Test Case ID								
	Base	T09	T10	T11	T12	T13	T14	T15	T16
Average Crash Risk (Stations 33 to 42)	1.182	1.146	1.146	1.159	1.147	1.138	1.162	1.164	1.161
Crash Risk Benefit	---	0.036	0.037	0.023	0.035	0.044	0.020	0.018	0.021
T-Statistic (Significance of Change)	---	0.602	0.608	0.391	0.562	0.750	0.326	0.299	0.345
ORCI	---	0.653	0.658	0.422	0.627	0.793	0.361	0.329	0.374

9.4.2.2.3 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 9-39 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 9-39: Cumulative Lane-Change ORCI Values for Treatments 09-16

	Base	T09	T10	T11	T12	T13	T14	T15	T16
Cumulative ORCI	---	-14.775	-13.895	-5.844	-5.236	-8.607	-8.618	-4.224	-3.488

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.

9.4.3 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 9-40 displays the different factors that make up Treatments 17-24, which are reviewed in this section.

Table 9-40: Description of Treatments 17-24

Treatment ID	Speed Change Implementation	Speed Zone Threshold	Speed Change Distance	Speed Change Time Period
17	-5/+5	5 mph	Speed Zone	5 min
18	-5/+5	5 mph	Speed Zone	10 min
19	-5/+5	5 mph	1/2 Speed Zone	5 min
20	-5/+5	5 mph	1/2 Speed Zone	10 min
21	-5/+5	2.5 mph	Speed Zone	5 min
22	-5/+5	2.5 mph	Speed Zone	10 min
23	-5/+5	2.5 mph	1/2 Speed Zone	5 min
24	-5/+5	2.5 mph	1/2 Speed Zone	10 min

9.4.3.1 Rear-End Crash Risk

The effects that Treatments 17-24 have on rear-end crash risk are reviewed in this section. Figures 9-58 and 9-59 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.

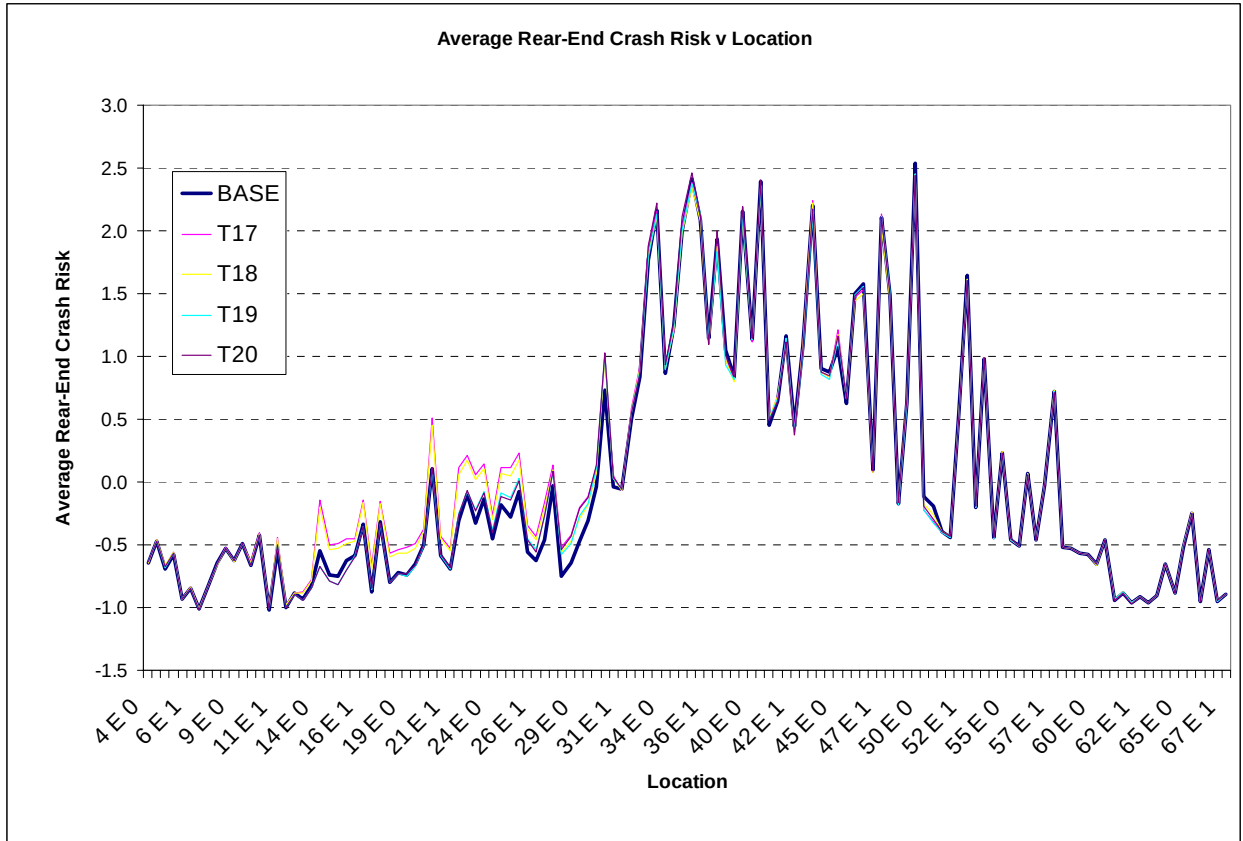


Figure 9-58: Average Rear-End Crash Risk v. Location for Treatments 17-20

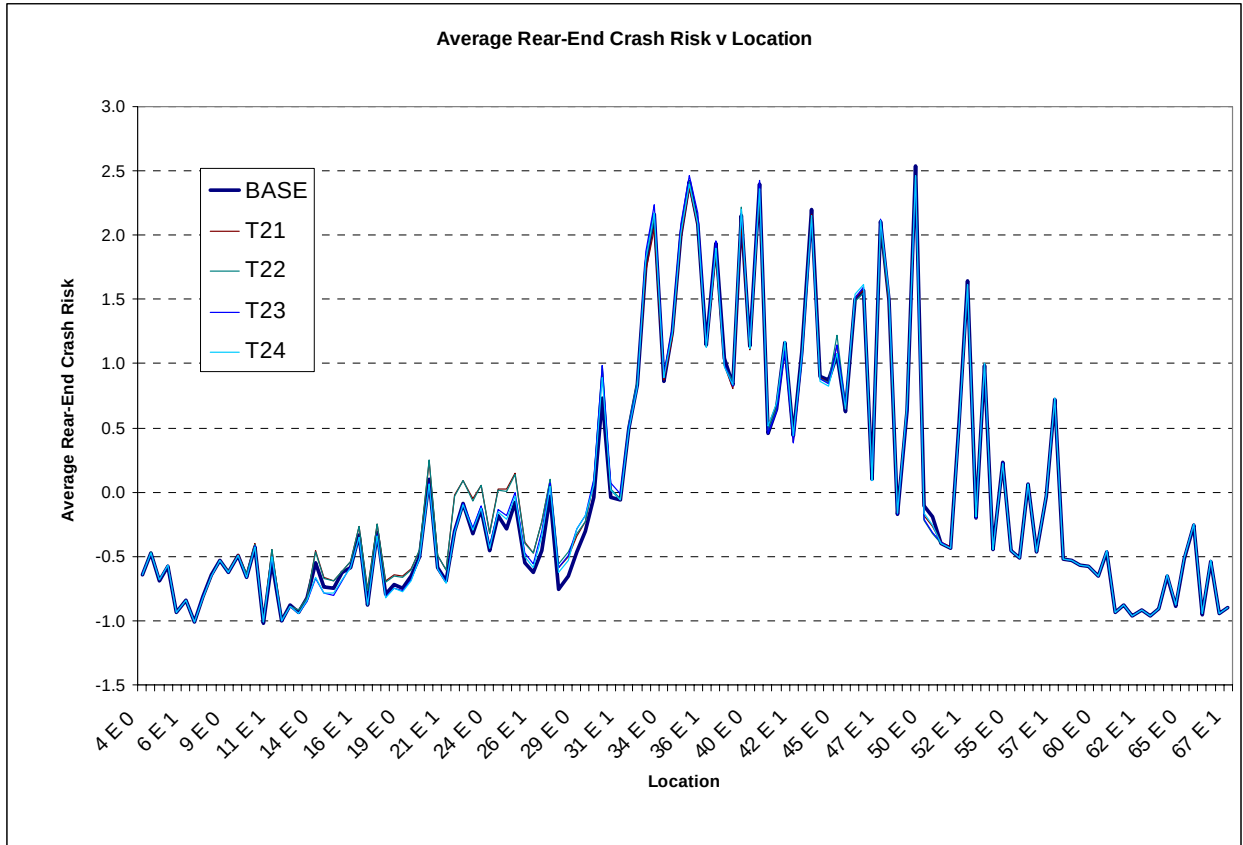


Figure 9-59: Average Rear-End Crash Risk v. Location for Treatments 21-24

9.4.3.1.1 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 9-60 and 9-61 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.

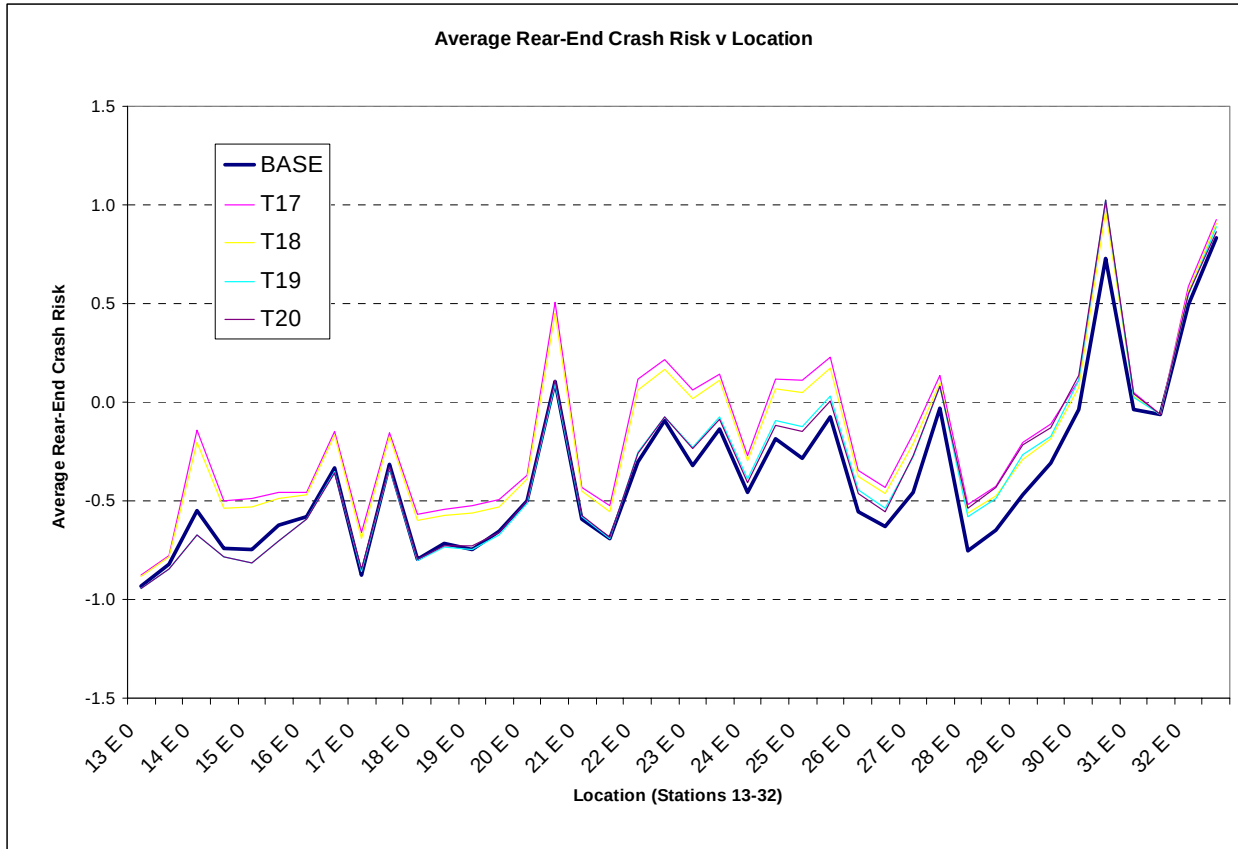


Figure 9-60: Average Rear-End Crash Risk v. Location for Treatments 17-20 for Segment 1

Table 9-41 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.

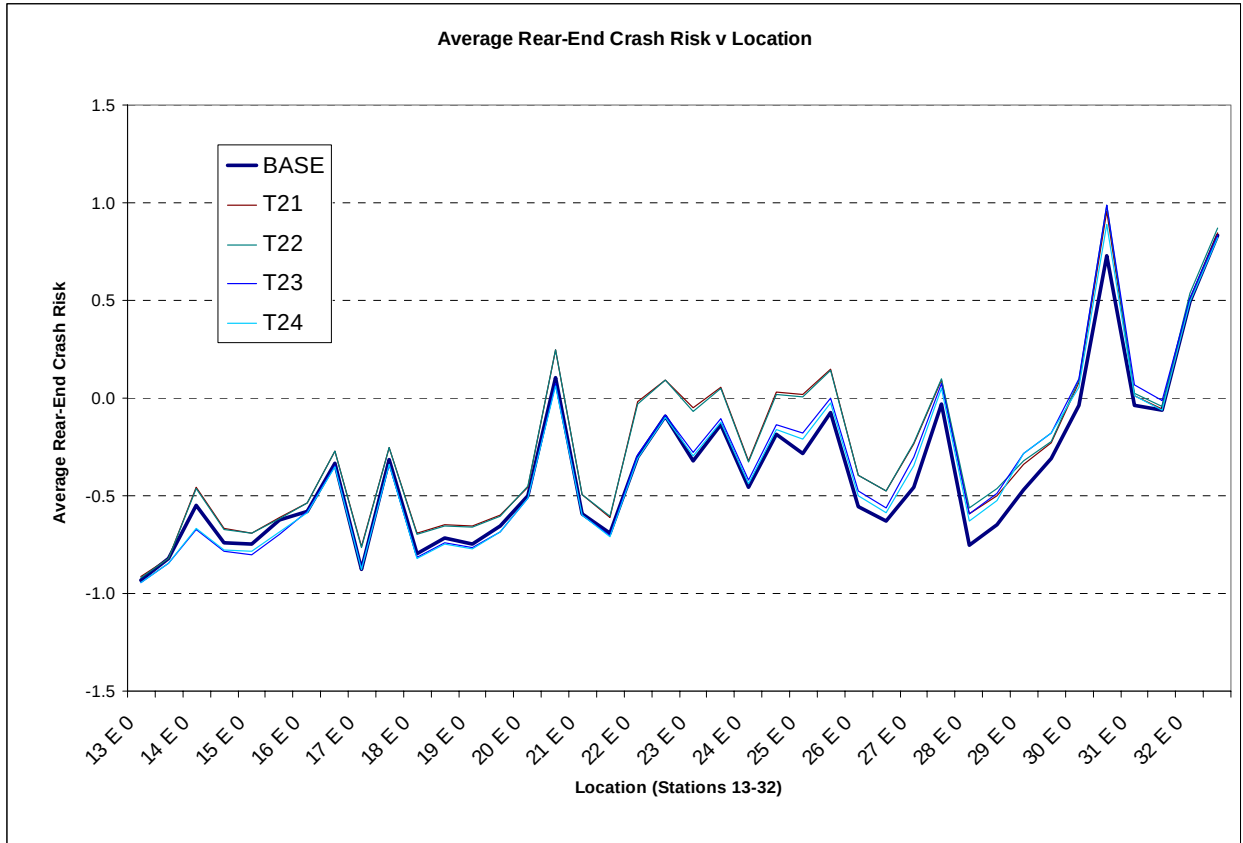


Figure 9-61: Average Rear-End Crash Risk v. Location for Treatments 21-24 for Segment 1

Table 9-41: 1st Segment: Summary of Difference in Rear-End Crash Risk for Treatments 17-24

	Test Case ID								
	Base	T17	T18	T19	T20	T21	T22	T23	T24
Average Crash Risk (Stations 13 to 32)	-0.372	-0.158	-0.194	-0.323	-0.318	-0.256	-0.254	-0.336	-0.353
Crash Risk Benefit	---	-0.214	-0.178	-0.050	-0.054	-0.116	-0.119	-0.036	-0.020
T-Statistic (Significance of Change)	---	2.284	1.912	0.516	0.560	1.237	1.255	0.378	0.211
ORCI	---	-8.559	-7.131	-1.993	-2.157	-4.654	-4.752	-1.449	-0.796

Across the 2nd segment, the differences in rear-end crash risk are positive. Figures 9-62 and 9-63 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.

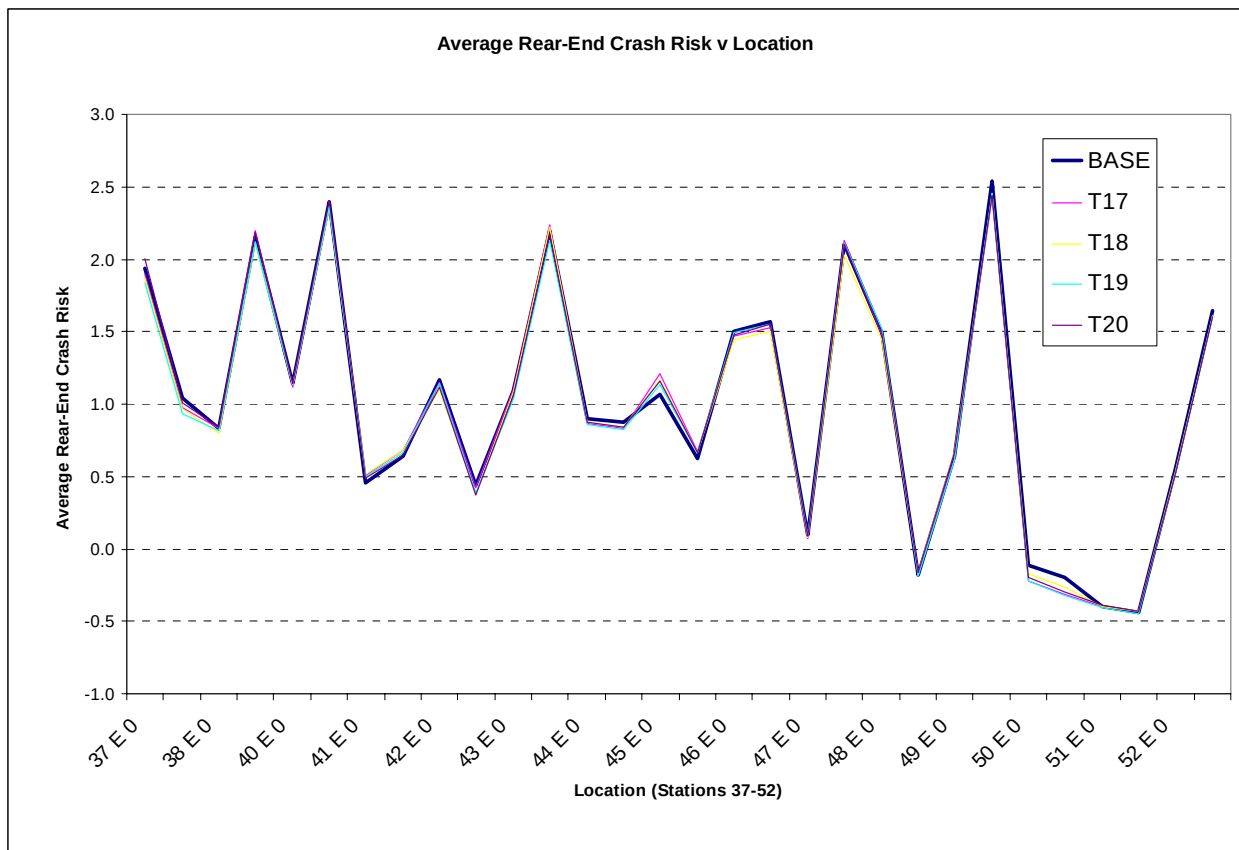


Figure 9-62: Average Rear-End Crash Risk v. Location for Treatments 17-20 for Segment 2

Table 9-42 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.

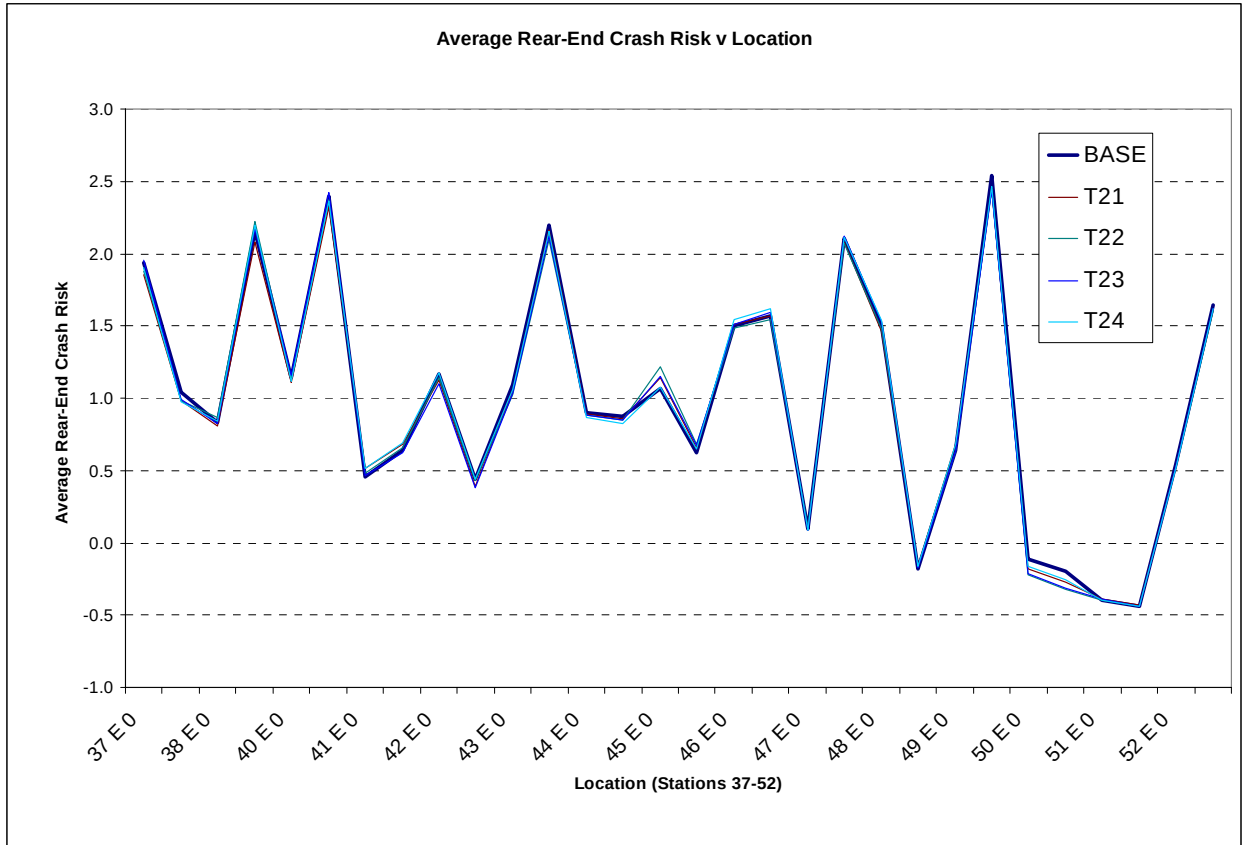


Figure 9-63: Average Rear-End Crash Risk v. Location for Treatments 21-24 for Segment 2

Table 9-42: 2nd Segment: Summary of Difference in Rear-End Crash Risk for Treatments 17-24

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

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 9-43 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 9-43: Cumulative Rear-End ORCI Values for Treatments 17-24

	Base	T17	T18	T19	T20	T21	T22	T23	T24
Cumulative ORCI	---	-8.115	-6.458	-1.133	-1.813	-4.050	-4.224	-1.011	-0.609

9.4.3.2 Lane-Change Crash Risk

The lane-change crash risk is next to be analyzed for the implementation of VSL Treatments 17-24. Figures 9-64 and 9-65 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.

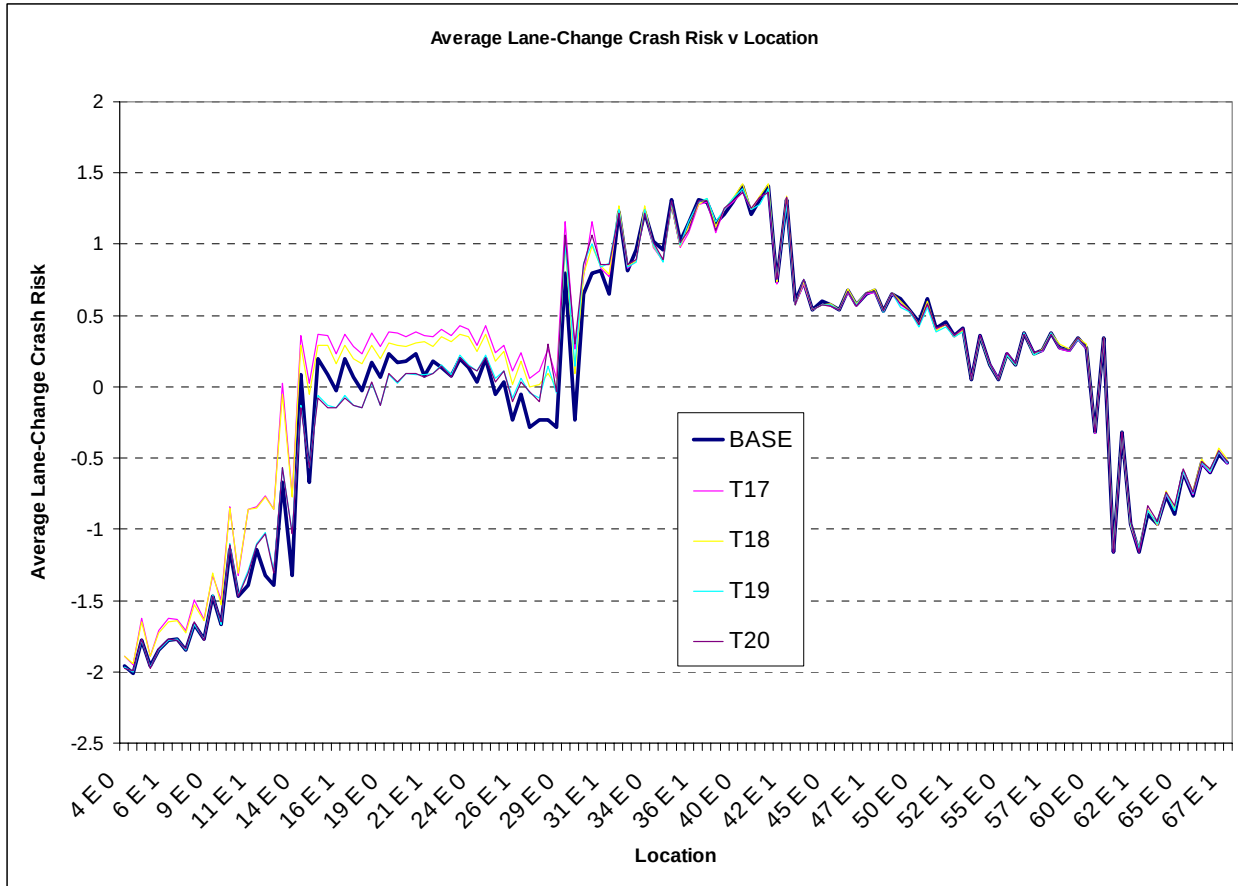


Figure 9-64: Average Lane-Change Crash Risk v. Location for Treatments 17-20

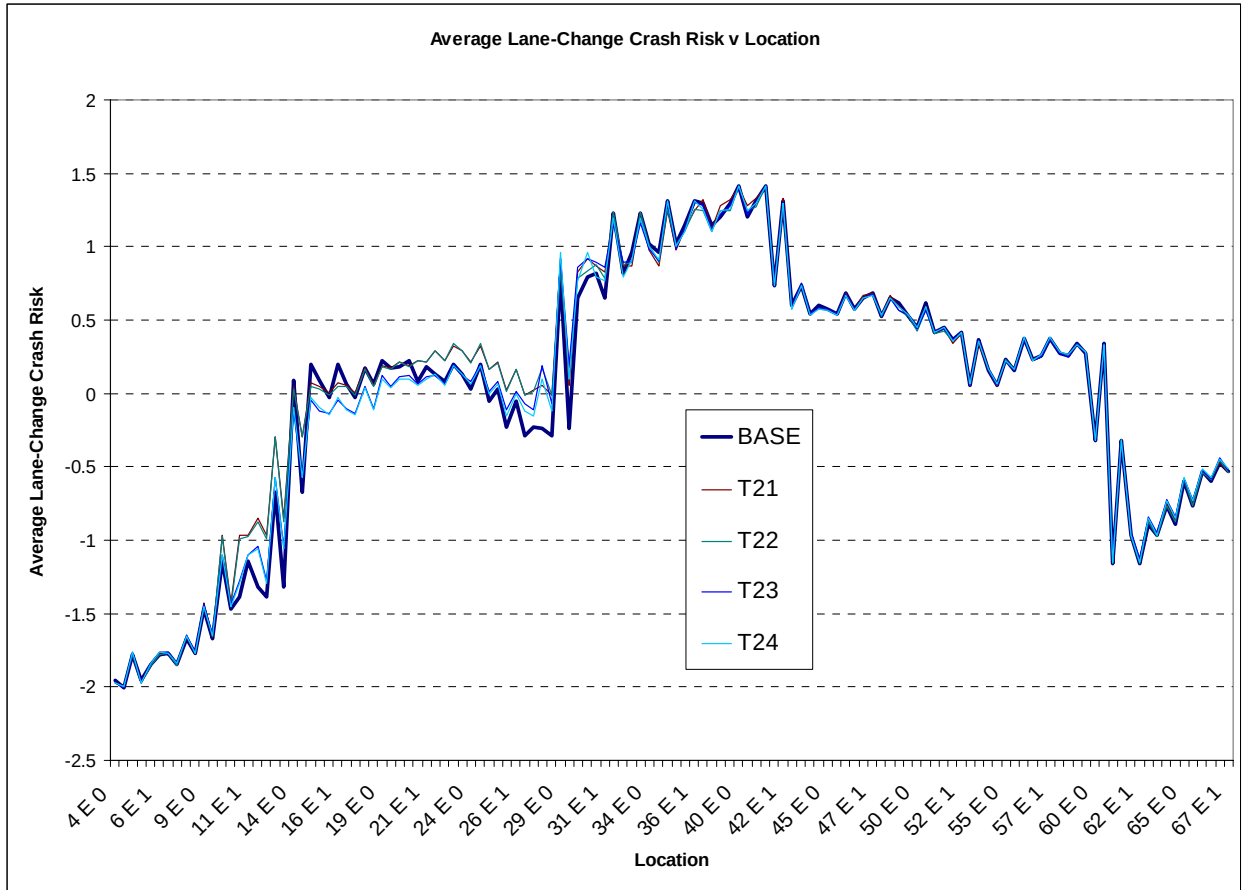


Figure 9-65: Average Lane-Change Crash Risk v. Location for Treatments 21-24

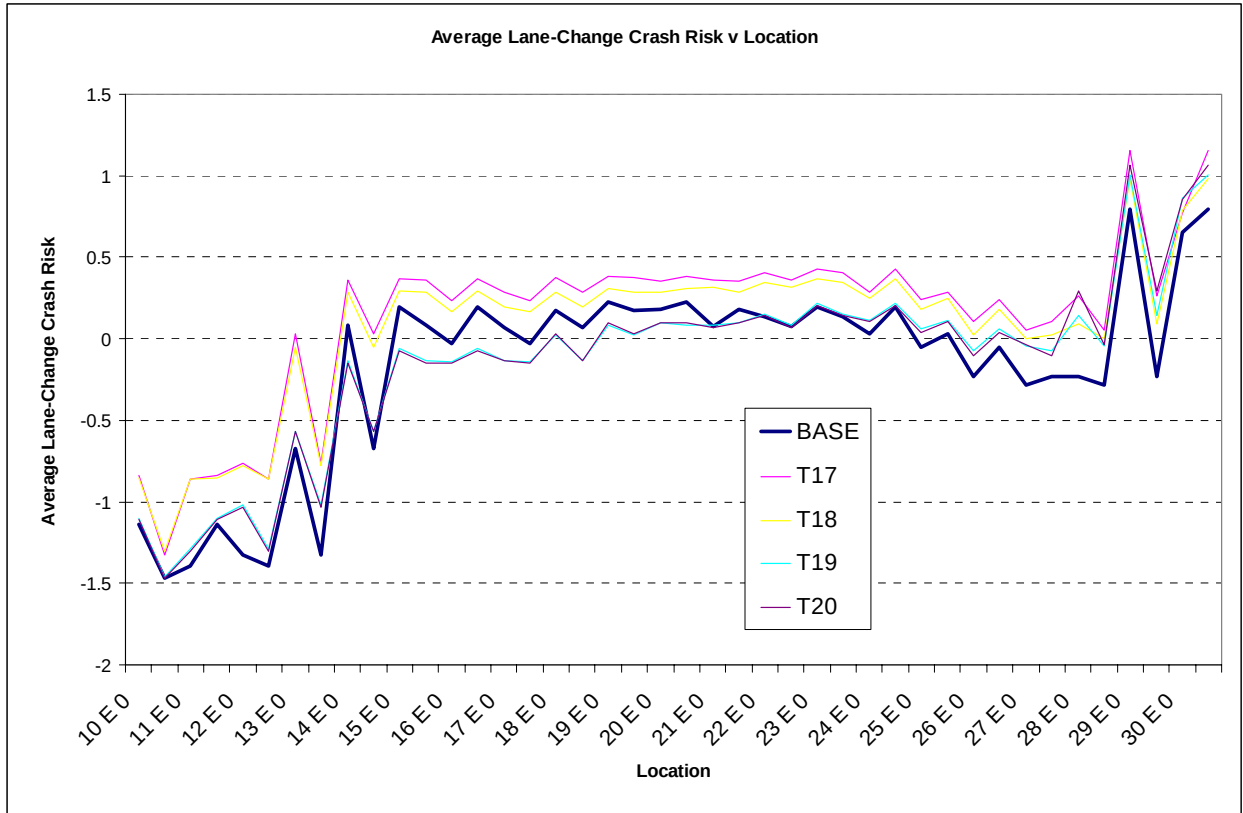


Figure 9-66: Average Lane-Change Crash Risk v. Location for Treatments 17-20 - Magnified

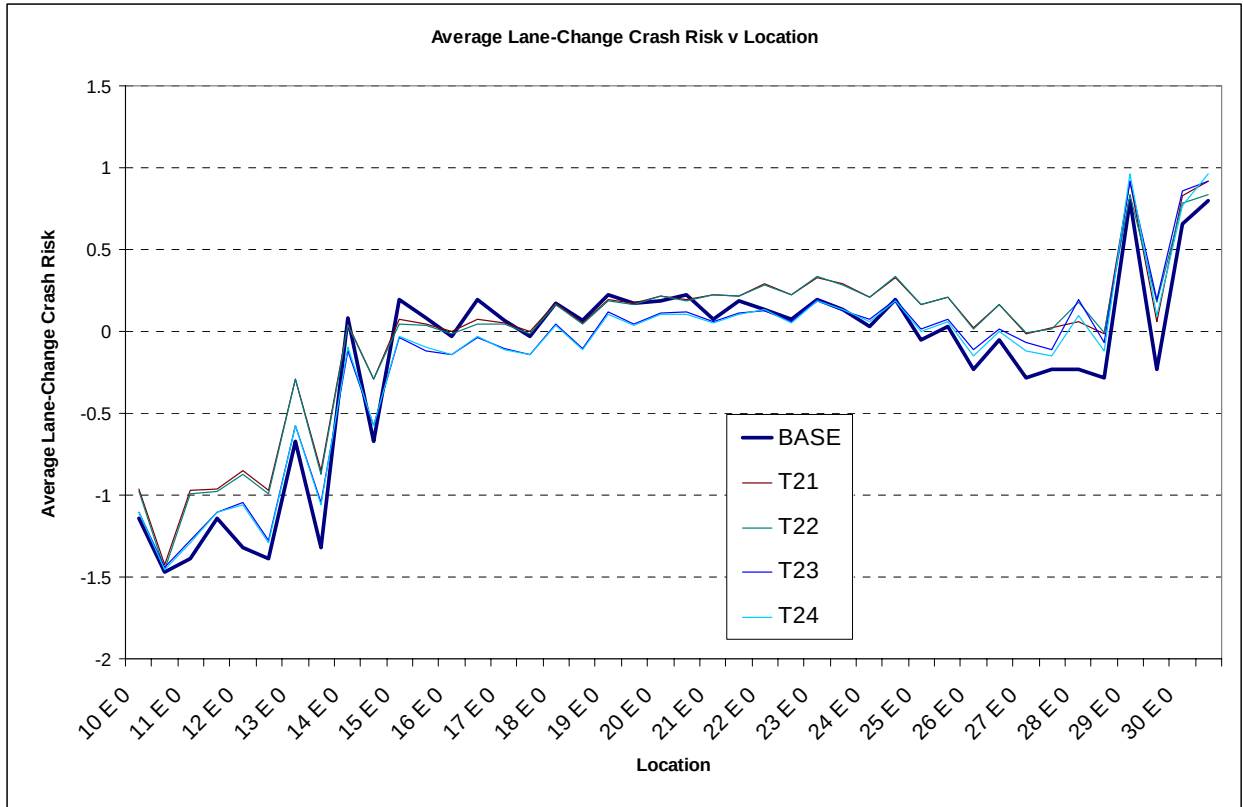


Figure 9-67: Average Lane-Change Crash Risk v. Location for Treatments 21-24 – Magnified

Figures 9-66 and 9-67 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 9-44, 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.

Table 9-44: Summary of Difference in Lane-Change Crash Risk for Treatments 17-24

	Test Case ID								
	Base	T17	T18	T19	T20	T21	T22	T23	T24
Average Crash Risk (Stations 10 to 30)	-0.172	0.140	0.075	-0.137	-0.134	-0.020	-0.027	-0.143	-0.156
Crash Risk Benefit	---	-0.312	-0.246	-0.034	-0.037	-0.152	-0.145	-0.028	-0.016
T-Statistic (Significance of Change)	---	2.556	2.070	0.274	0.293	1.261	1.202	0.226	0.124
ORCI	---	-13.090	-10.348	-1.448	-1.565	-6.379	-6.073	-1.186	-0.652

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 drawn in Chapter 3 that VSL may not be effective in highly congested situations. Other ITS strategies must be used to reduce crash risk in congested situations, such as ramp metering. It is in accordance with the conclusions from Chapter 5 (Also see Dhindsa, 2005) that VSL in conjunction with ramp metering could help to reduce crash risk in highly congested situations.

9.5 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.

9.5.1 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 the Section 9.5.2.

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 worst effect on crash risk, and Treatment 19 was found to be the most beneficial in the 80% loading condition.

9.5.1.1 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 9-45.

Table 9-45: Percentage Change in Rear-End Crash Risk Factors from the BASE Case

Treatment	AOF2	ASF2	ASG2
02	6.66	-3.71	-3.71
19	-0.98	2.23	2.03

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.

9.5.1.2 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 9-46.

Table 9-46: Percentage Change in Lane-Change Crash Risk Factors from the BASE Case

Treatment	ASW2	ASU2	AOW2	ADALOU2
02	-3.64	-3.59	5.10	2.44
19	1.95	2.31	-1.11	-5.25

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

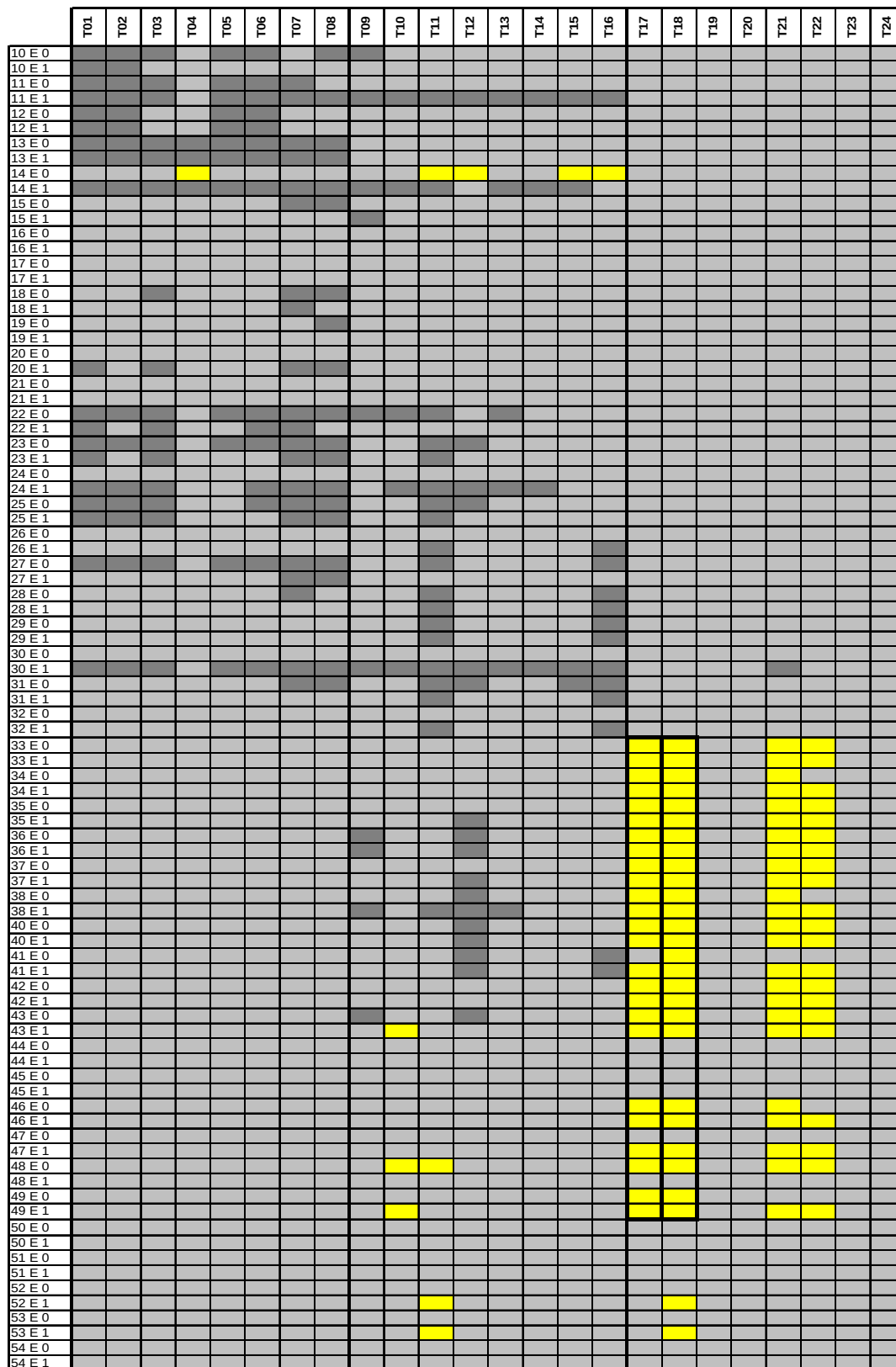
occupancy downstream was increased, and the difference in occupancy across the different lanes was also increased. In Treatment 19, however, where a significant reduction of lane-change crash risk was observed, the average speeds were increased, the average occupancy was decreased, and the difference 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.

9.5.2 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 9-68 and 9-69 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. 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 the crash migration phenomenon is present in Treatments 01-16. However, crash migration is resisted with the implementation of Treatments 17-24. 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.



	T01	T02	T03	T04	T05	T06	T07	T08	T09	T10	T11	T12	T13	T14	T15	T16	T17	T18	T19	T20	T21	T22	T23	T24
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Figure 9-69: 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 9-70 and 9-71 show the trends that each treatment had on the rear-end and lane-change crash risk, respectively. These figures show that crash risk migration is much more present in the 80% loading scenario than the 60% loading scenario. When comparing Figures 9-70 and 9-71 to Figures 9-68 and 9-69, respectively, the negative effect of crash migration is much more prevalent in the upstream areas of the network. This alludes to the supposition that crash migration 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 9-71, Treatment 19's resistance to crash migration allows it to outperform Treatments 17 and 18, even though they experience two segments of significant benefit. This is a very interesting trend, and it is also shown to carry through into the 90% loading scenario.

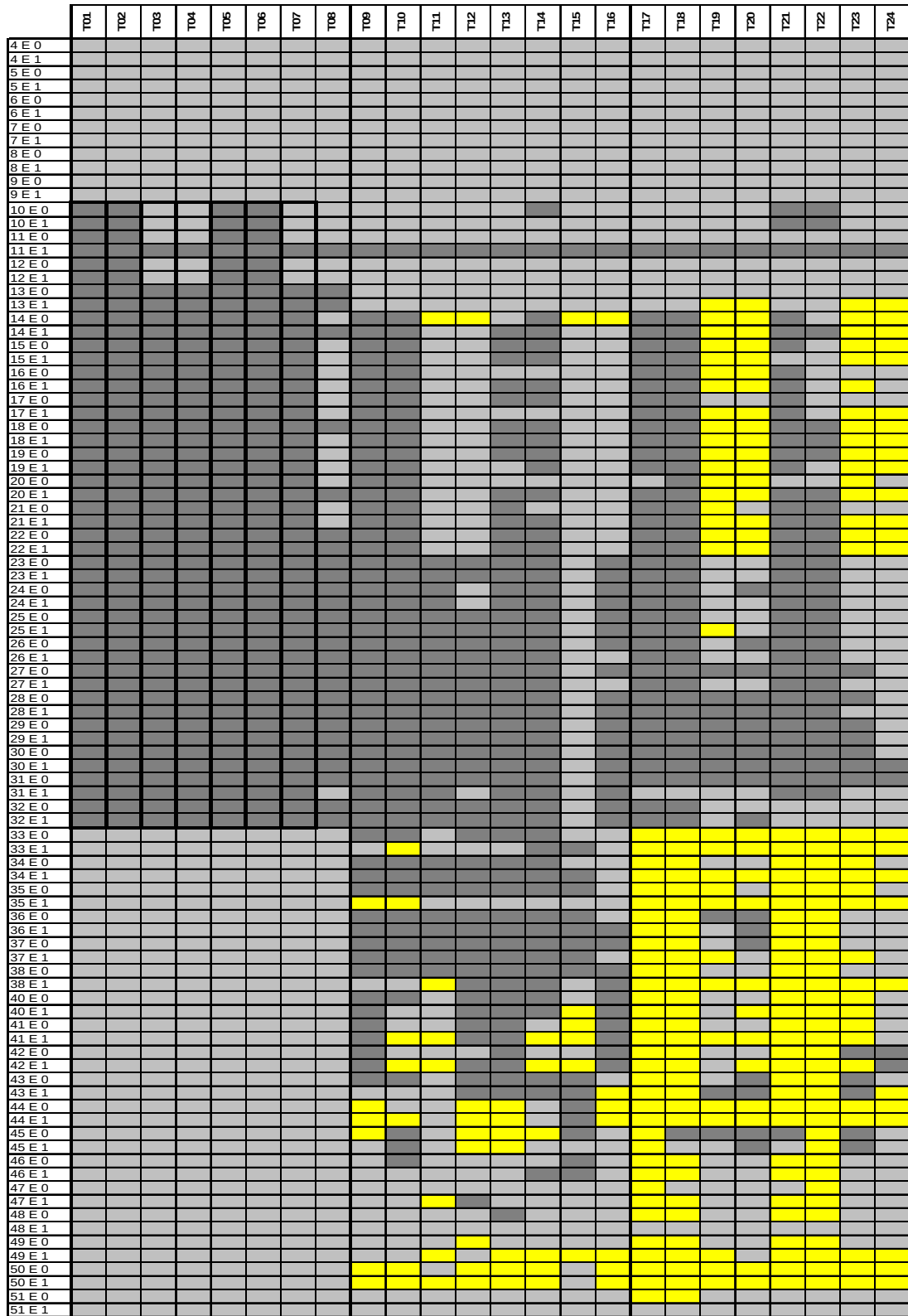


Figure 9-70: Trend of Affected Locations for Rear-End Crash Risk at 80% Loading

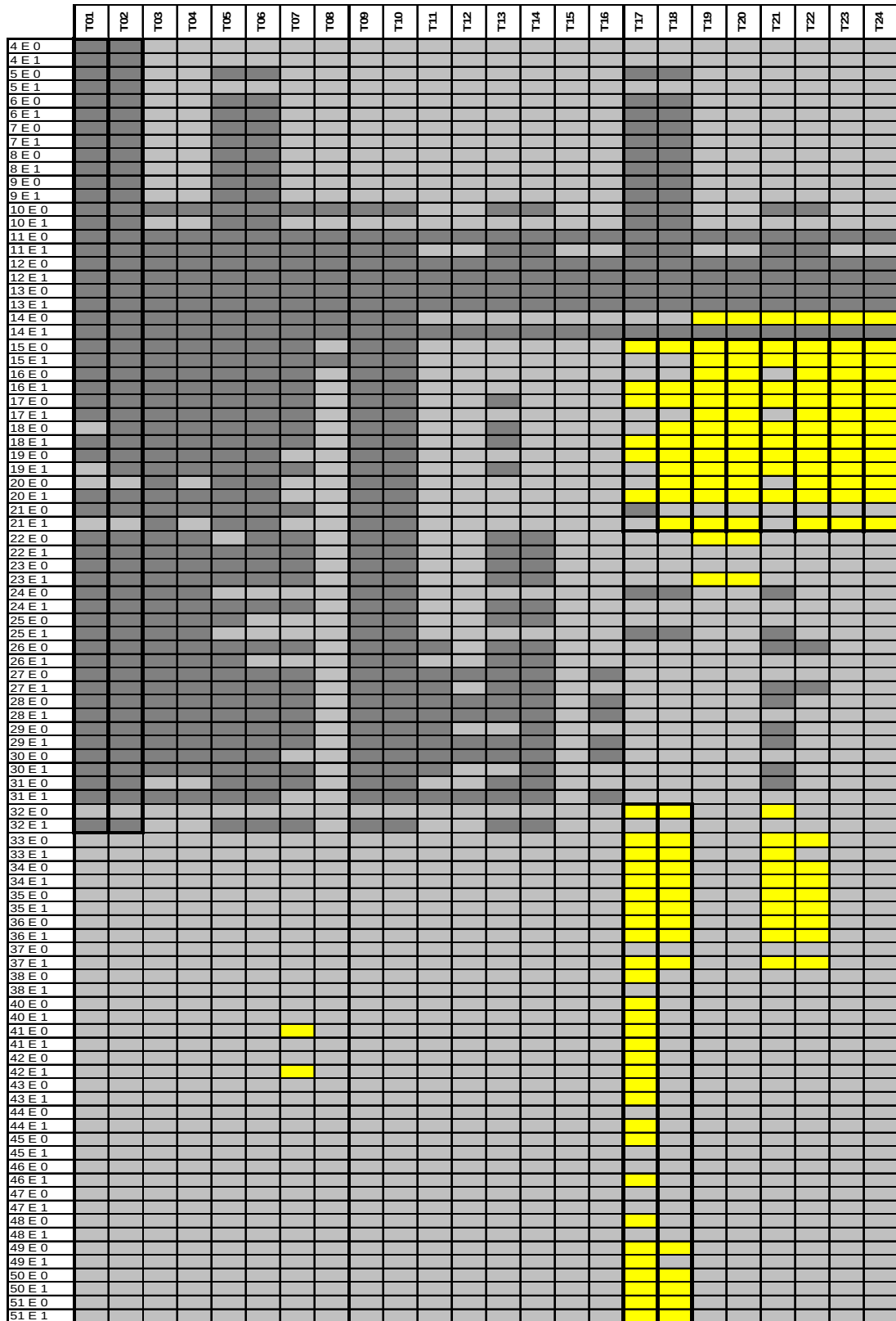


Figure 9-71: Trend of Affected Locations for Lane-Change Crash Risk at 80% Loading

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 9-72 and 9-73 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 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 9-73 with 9-71.

From Figure 9-72, no treatment was found to have a significant benefit to the rear-end crash risk. As reviewed in Section 9.4, 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 9-73, 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 cash risk in the surrounding areas. Nevertheless, it is interesting to note the evidence in Figures 9-72 and 9-73 of Treatments 19, 20, 23 and 24 being more resistant to crash migration than any other treatments, as noted in the 80% loading scenario.

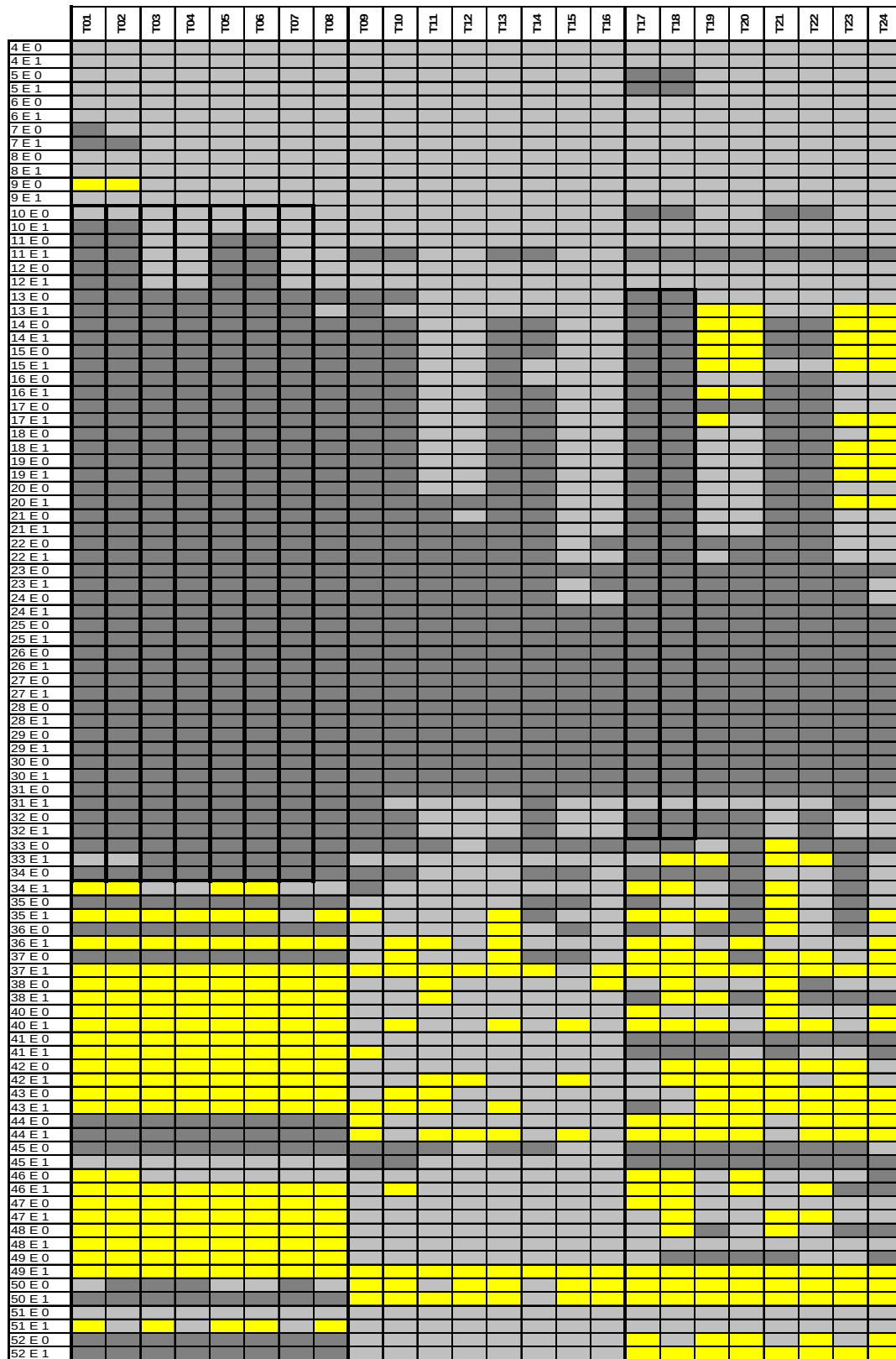


Figure 9-72: Trend of Affected Locations for Rear-End Crash Risk at 90% Loading

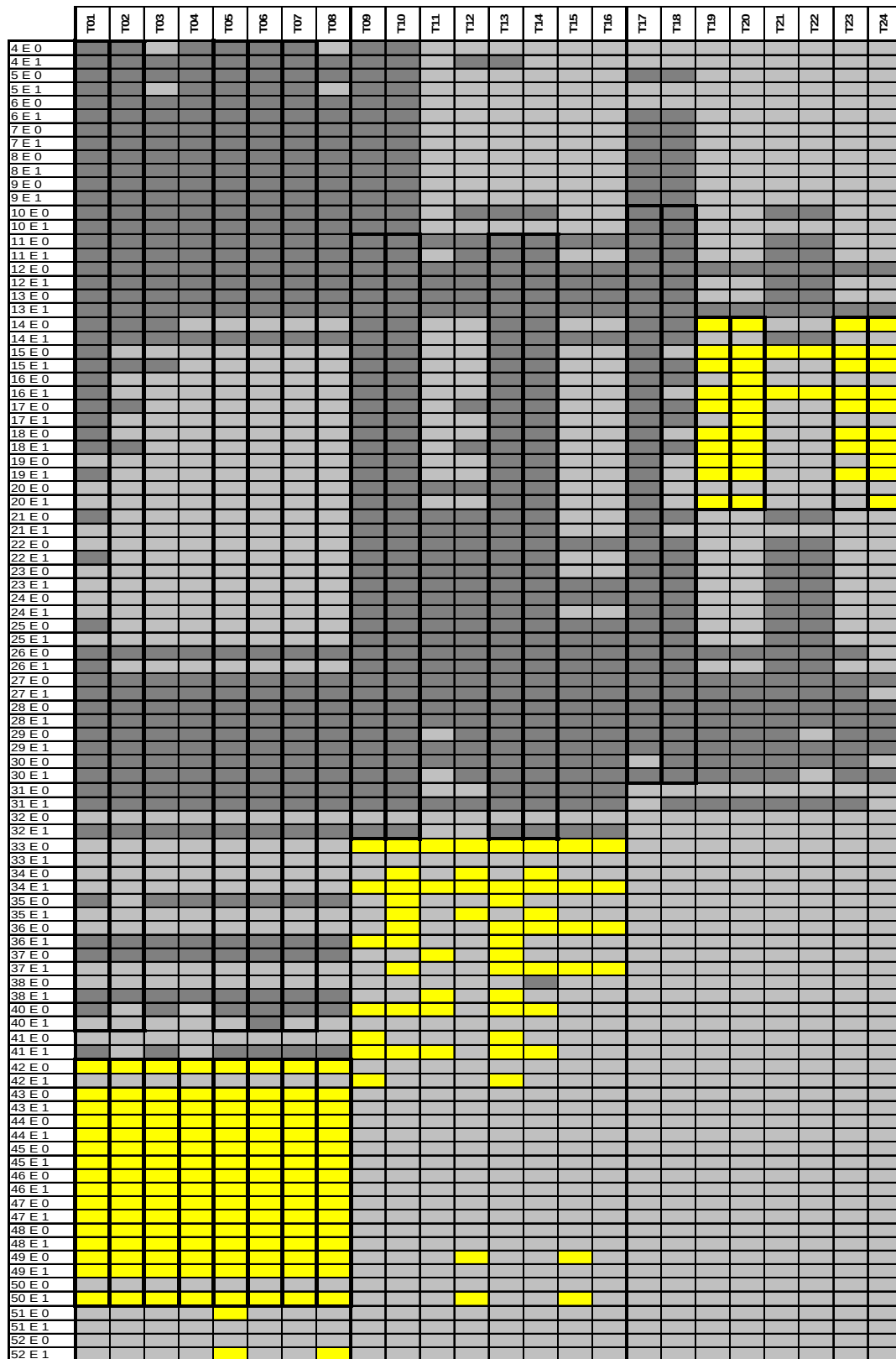


Figure 9-73: Trend of Affected Locations for Lane-Change Crash Risk at 90% Loading

9.5.3 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 intensity 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.
- 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 the results from Dilmore (2005).

9.6 CONCLUSIONS

This chapter 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 through Orlando, FL was simulated using the PARAMICS micro-simulation program. Twenty-four separate, network-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.

CHAPTER 10

FINAL CONCLUSIONS AND AVENUES FOR FUTURE RESEARCH ADVANCEMENTS

The research described in this report includes extensive analysis of historical crash and archived loop detector data for real-time crash risk assessment in Volume I and then evaluations of ITS strategies for reduction in crash risk in a microscopic simulation environment in Volume II. The innovative work in this research started from generic models developed using the within stratum matched sampling models. The work was then expanded to disaggregate analysis of crashes by type so that different loop data patterns associated with rear-end crashes; lane-change crashes as well as single vehicle crashes may be identified. It was found that while we can develop measures that can be used to relate risk of rear-end and lane-change related crashes with the loop detector data; a similar measure of crash risk can not be developed for single vehicle crashes. The report provides detailed computer codes in the Appendices for real-time crash risk assessment as well as APIs for assessing crash risk in the microscopic simulation environment. The codes detailed in the Appendices are referenced appropriately throughout this report and may be integrated with the RTMC process. More details on the future research effort with the crash risk assessment were provided in Chapter 9—Volume I. This chapter focuses on the future scope for the evaluation of ITS strategies for their crash risk reduction potential. This research has shown multiple strategies in which variable speed limits, ramp metering, and route diversion could benefit the safety of urban freeways.

Future research should address the transferability and related issues for the ITS strategies recommended in this report. It 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. However, with the experience in data analysis research group should be able to achieve that in lesser time and effort for other corridors around the state. These strategies could then be repeated at different volume loading conditions for the other corridor and compared to the conclusions of this study.

Further analysis could be done on this network by trying different strategies, especially with some strategies recommended by the FDOT ITS staff. Further research could also include the logistics of implementing these strategies in the field. Another way to expand on this study is to observe the effects of combining the strategies used here with other previously tested ITS strategies. In Chapter 7 (Also see Gayah, 2006) we 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 recommendations from Chapter 7 for route diversion or ramp metering with variable speed limits recommendations provided in Chapter 9 might prove to be very successful in the reduction of crash risk on freeways.

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APPENDICES (VOLUME II)

Appendix II-A

OD Matrix

demand period 1																	
matrix count 1																	
divisor 1.0000																	
matrix 1																	
	to 1	to 5	to 7	to 9	to 11	to 12	to 14	to 16	to 17	to 19	to 23	to 25	to 26	to 28	to 31	to 32	to 34
from 2	0	0	145	0	0	0	0	0	0	0	0	0	0	0	0	0	0
from 3	0	0	533	0	0	0	0	0	0	0	0	0	0	0	0	0	0
from 4	0	0	0	0	218	0	211	0	0	127	0	143	0	72	0	0	42
from 6	0	0	0	0	8	0	70	0	0	64	0	109	0	12	0	0	14
from 7	0	614	0	0	744	0	643	0	0	402	0	425	0	229	0	0	289
from 8	190	0	529	0	0	0	0	0	0	0	0	0	0	0	0	0	0
from 10	0	0	0	0	0	0	46	0	0	151	0	152	0	73	0	0	62
from 13	221	0	841	29	0	0	0	0	0	0	0	0	0	0	0	0	0
from 15	0	0	0	0	0	0	0	0	0	145	0	98	0	82	0	0	56
from 18	0	0	0	0	0	0	0	0	0	0	0	29	0	52	0	0	29
from 20	196	0	570	115	0	169	0	0	0	0	0	0	0	0	0	0	0
from 21	0	0	0	0	0	0	0	0	0	0	0	14	0	32	0	0	45
from 22	172	0	486	107	0	175	0	13	17	0	0	0	0	0	0	0	0
from 24	0	0	0	0	0	0	0	0	0	0	0	0	0	14	0	0	18
from 27	127	0	375	91	0	107	0	13	18	0	4	0	0	0	0	0	0
from 29	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	9
from 30	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2
from 33	102	0	277	85	0	105	0	17	24	0	11	0	7	0	2	0	0
from 35	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
from 37	113	0	327	52	0	94	0	71	30	0	15	0	11	0	4	2	0
from 38	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
from 40	52	0	103	25	0	62	0	11	16	0	8	0	56	0	3	2	0
from 41	65	0	180	31	0	39	0	19	20	0	11	0	56	0	3	3	0
from 42	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
from 46	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
from 47	49	0	84	28	0	84	0	14	46	0	11	0	63	0	4	4	0
from 49	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
from 51	11	0	57	28	0	51	0	54	45	0	59	0	64	0	4	4	0
from 53	60	0	539	130	0	167	0	174	150	0	103	0	101	0	53	120	0
from 56	60	0	39	47	0	61	0	87	126	0	119	0	35	0	7	7	0
from 59	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
from 60	4	0	22	32	0	41	0	67	154	0	114	0	41	0	65	6	0
from 64	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
from 65	3	0	11	44	0	57	0	74	104	0	100	0	36	0	88	29	0
from 67	2	0	2	87	0	114	0	150	169	0	73	0	74	0	17	120	0
from 69	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
from 73	391	0	1172	281	0	371	0	165	366	0	261	0	224	0	178	269	0

Appendix II-B
Code from PARAMICS Files

FOR IMPLEMENTING MULTIPLE LINKS FILES
Profile File

Demand Profile

Period Count 3

Divisor 13.33

Interval 5

period 1 start 15:45:00

82	76	78			
79	91	83	85	87	99
91	93	94	106	98	100

period 2 start 17:00:00

98	97	95	93	92	100
88	87	95	83	81	80
78	76	75	73	71	70

period 3 start 18:30:00

78	76	65	63	82	62

CONFIGURATION USED

Configuration File

start time 15:45:00
simulation time 03:15:00
demand weight 100.0
seed 1234
split seed
demand matrix tuning level 0
generator 0
loop length 6.560 ft
speed memory 1
closest origin vehiclepark disabled
closest destination vehiclepark enabled
file time "-"
curve speed factor 1.000
amber time 3.000
speed drift 5
maximum diversion 300
right hand drive
units us
timestep detail 2
mean headway 1.00
mean reaction time .42
cost coefficients 1.000, 0.000 mins per mile, 0.000
queue speed 8.000 mph
queue distance 9.000 ft
weight heavy 2.000 ton
feedback 00:00:00
feedback smoothing factor 0.500
feedback decay factor 0.995
perturbation disabled

FOR IMPLEMENTING PREEMPTION

Plans File

plan count 4

```
plan 1 definition
loops 1
parameters 0
if (init)
{
    variable;
}
if (count [1] type [17] >= 1)
{
    green1 = 0; ##
    green2 [2] = 40;
}
```

```
plan 2 definition
loops 1
parameters 0
if (time = 1)
{
    green2 [2] = 0;
    clear [1];
}
```

```
plan 3 definition
loops 1
parameters 0
if (init)
{
    variable;
}
if (count [1] type [17] >= 1)
{
    green1 = 0; ##
    green2 [3] = 40;
}
```

```
plan 4 definition
loops 1
parameters 0
if (time = 1)
{
    green2 [3] = 0;
}
```

```
clear [1];  
}
```

Appendix II-C

8 Scenarios for the Low Speed Cases

Case 1

	t-15min	t-10min	t-5min	t	t+5min
x+2mi	0	-5	-5	-5	-5
x+1.5mi	-5	-10	-10	-10	-5
x+1mi	-5	-10	-10	-10	-5
x	-5	-10	-10	-10	-5
x+.5mi	-5	-5	-5	-5	-5

Case 2

	t-15min	t-10min	t-5min	t	t+5min
x+2mi	0	0	0	0	0
x+1.5mi	-5	-5	-5	-5	-5
x+1mi	-5	-10	-10	-10	-5
x	-5	-10	-10	-10	-5
x+.5mi	-5	-5	-5	-5	-5

Case 3

	t-15min	t-10min	t-5min	t	t+5min
x+2mi	0	-5	-10	-10	-5
x+1.5mi	-5	-10	-15	-15	-5
x+1mi	-5	-10	-15	-15	-5
x	-5	-10	-15	-15	-5
x+.5mi	-5	-5	-5	-5	-5

Case 4

	t-15min	t-10min	t-5min	t	t+5min
x+2mi	0	0	-5	-5	0
x+1.5mi	0	-5	-10	-10	-5
x+1mi	-5	-10	-15	-15	-5
x	-5	-10	-15	-15	-5
x+.5mi	-5	-5	-5	-5	-5

Gradual Changes

Case 5

	t- 15min	t- 10min	t-5min	t	t+5min
x+2mi	0	0	0	0	0
x+1.5mi	0	-10	-10	-10	-10
x+1mi	0	-10	-10	-10	-10
x	0	-10	-10	-10	-10
x+.5mi	0	0	0	0	0

Case 6

	t- 15min	t- 10min	t-5min	t	t+5min
x+2mi	0	0	0	0	0
x+1.5mi	0	0	0	0	0
x+1mi	0	-10	-10	-10	-10
x	0	-10	-10	-10	-10
x+.5mi	0	0	0	0	0

Case 7

	t- 15min	t- 10min	t-5min	t	t+5min
x+2mi	0	-5	-5	-5	-5
x+1.5mi	0	-15	-15	-15	-15
x+1mi	0	-15	-15	-15	-15
x	0	-15	-15	-15	-15
x+.5mi	0	0	0	0	0

Case 8

	t- 15min	t- 10min	t-5min	t	t+5min
x+2mi	0	0	0	0	0
x+1.5mi	0	-5	-5	-5	-5
x+1mi	0	-15	-15	-15	-15
x	0	-15	-15	-15	-15
x+.5mi	0	0	0	0	0

Abrupt Changes

Appendix II-D

Speed Imputation Models

Table for Center Lane speed from left lane parameters imputation

Station	RMSE	Intercept	LS	LV	LO	LS_LS	LV_LS	LV_LV	LO_LS	LO_LV	LO_LO
34	2.91058	1.3221	0.22122	0.44714	0.089	-0.00209	0.00025	-0.00081	-0.00079	-0.00176	-0.00044
35	3.19748	2.3967	0.14828	0.79445	0.15767	-0.00083	-0.00934	0.008336	-0.00035	-0.01457	-0.00106
36	3.12997	2.6661	0.37562	0.09138	0.03357	-0.00338	-0.00144	0.004993	-0.00448	0.009051	-0.00031
37	3.00552	1.9958	0.27731	0.81319	-0.09035	-0.00265	-0.00215	-0.01349	-0.00335	-0.00157	0.001707
38	3.67759	-0.4948	0.17951	0.00392	0.32989	-0.00191	0.007927	0.029115	-0.00484	-0.01363	-0.00122
40	3.00043	2.0769	0.2566	0.8436	-0.02109	-0.00266	-0.00355	-0.0045	-0.00105	-0.00502	0.000652
41	2.98917	4.1159	0.26778	0.31974	0.02673	-0.00424	0.007498	-0.00542	0.00074	-0.00012	-0.00025
42	2.534	-0.8392	0.33625	0.59397	0.05972	-0.0039	-0.00744	0.005282	0.001149	-0.00725	0.000284
43	2.8742	3.8127	0.05531	0.76977	0.19096	0.00113 1	-0.01954	0.016652	0.011234	-0.02368	-0.00222
44	2.89625	10.0568	0.14951	0.4749	-0.21161	-0.00258	-0.00136	0.01191	-0.00183	-0.00267	0.002277
45	2.88515	11.3037	0.01627	0.49128	-0.2905	0.00039 2	-0.01385	0.017905	0.011488	-0.01423	0.003331
46	2.75551	-2.1841	0.33398	1.13039	0.07447	-0.00381	-0.00369	-0.00862	-0.00233	-0.00825	0.000675
47	3.24766	4.7248	0.2044	0.6977	0.22492	-0.00376	0.006478	-0.01569	-0.00191	-0.00812	-0.00193
48	2.73402	8.125	0.02581	0.8615	-0.54043	-0.00186	0.003375	-0.02244	0.006128	0.008468	0.006099

Table for Center Lane volume from left lane data imputation

Station	RMSE	Intercept	LS	LV	LO	LS_LS	LV_LS	LV_LV	LO_LS	LO_LV	LO_LO
34	6.4682	9.5559	0.0135	1.98401	-0.59656	0.012772	-0.03744	0.031231	0.006897	-0.0254	0.008259
35	8.5776	22.276	0.71235	0.07119	-0.84815	-0.00338	0.015291	0.015245	-0.01278	-0.00214	0.012004
36	5.8745	-1.1069	1.03`346	-0.59828	0.12001	-0.00043	-0.0009	0.02494	-0.00834	0.014123	-0.00059
37	7.8069	0.8885	0.38509	1.69633	-0.51676	0.010188	-0.01772	-0.00974	-0.00482	0.005322	0.00669
38	12.0179	16.3157	1.13505	-0.38067	-0.16028	-0.00743	0.02129	-0.03644	-0.0146	0.024457	0.001625
40	9.3866	-3.1172	0.87824	0.65216	-0.14445	0.00093	0.004276	-0.01192	-0.02297	0.019416	0.004441
41	6.5219	-1.2012	1.3248	-0.92748	0.22652	-0.00312	0.007069	0.021107	-0.01641	0.020411	-0.00204
42	7.525	-21.3096	2.69798	-1.65025	0.92191	-0.02205	0.041008	0.007314	-0.04809	0.031075	-0.00625
43	8.8967	9.9595	1.34689	0.15663	-0.54603	-0.0127	0.029138	-0.01907	-0.05179	0.049441	0.009304
44	4.7489	-16.9913	1.27354	0.80499	0.13681	-0.00153	0.005898	-0.03815	-0.00853	0.014394	0.000524
45	9.8202	-47.2398	3.32845	1.44276	1.01223	-0.02999	0.015669	-0.01665	-0.06298	0.003984	-0.00284
46	7.0798	-21.9014	1.92251	2.86147	-0.34803	-0.00907	-0.02979	-0.00217	-0.01591	-0.02868	0.010984
47	7.2548	10.0684	1.46145	-0.43827	0.23353	-0.00939	0.032291	-0.02245	-0.06303	0.068477	-0.00472
48	5.8375	-24.6009	2.02121	-0.17482	0.30031	-0.0104	0.016676	-0.00416	-0.03215	0.026667	0.002903

Appendix II-E

VB Codes

API for implementing VSL

```
/* -----
 * Paramics Programmer API (paramics-support@quadstone.com)
 * Quadstone Ltd.          Tel: +44 131 220 4491
 * 16 Chester Street       Fax: +44 131 220 4492
 * Edinburgh, EH3 7RA, UK  WWW: http://www.paramics-online.com
 * ----- */
#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 scenarios for applications according to speed regime*/

#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

/* define variables and pointers*/

const char *g_ParamFile = "api_example7";
static char **g_LinkLookup = NULL;
static char **g_BeaconLinks = NULL;
static char **g_LoopLinks = NULL;
static int nLinks = 0;
static int nLoops = 0;
static int nBeacons = 0;
static float *g_Speeds1 = NULL;
static float *g_Speeds2 = NULL;
static float *g_Speeds3 = NULL;
static float *g_UpSpeeds1 = NULL;
static float *g_UpSpeeds2 = NULL;
static float *g_UpSpeeds3 = NULL;
static float *g_DownSpeeds1 = NULL;
static float *g_DownSpeeds2 = NULL;
```

```

static float *g_DownSpeeds3 = NULL;
static float *g_AvgUpSpeed = NULL;
static float g_StimlateTime = 0.0;
static float g_RestrictionDistance = 0.0;
static float *g_BaseSpeeds = NULL;
static int count = 0;
static float *g_SumSpeeds1 = NULL;
static float *g_SumSpeeds2 = NULL;
static float *g_SumSpeeds3 = NULL;
static float *g_SumUpLaneSpeeds = NULL;
static float *g_SumDownLaneSpeeds = NULL;
static int *g_SpeedLimit = NULL;
/* -----
 * call qpx_NET_postOpen once when the full network has been read into modeller
 * ----- */
void qpx_NET_postOpen(void)
{
    int i;
    char *link_ref;

    qps_GUI_printf("\nParamics Programmer API: Using VMS and Detectors\n");

    nLinks = qpg_NET_links();
    nLoops = qpg_NET_detectors();
    nBeacons = qpg_NET_beacons();

    /* now allocate memory for all dynamic objects in our plugin */
    pp_allocate_memory();

    if (nLoops != nBeacons)
    {
        /* warning as each VMS beacon should ideally only have one loop
         * associated with it */
        qps_GUI_printf("\nWARNING: Number of Beacons and Loops found in network
\n"
                        "          do not match, this may cause problems !\n");
    }

    /* now collect information about the Detectors/VMS objects in our network */
    pp_check_beacons();
    pp_check_loops();

    /* at this point we know the location of each beacon and each loop in our
     * network, now we must assume that a beacon will require information from
     * at least one detector on the same link to work, so check that this is

```

```

    * true */
    for (i = 0; i < nBeacons; i++)
    {
        link_ref = g_BeaconLinks[i];
        if (!pp_beacon_loop_match(link_ref))
        {
            qps_GUI_printf("\nERROR: Beacon on link %s has no corresponding "
                           "detector.\n", link_ref);
            return;
        }
    }
    /* now collect existing speed limits */
    pp_collect_initial_speeds();
}

/* -----
 * Read/Verify parameters file
 * ----- */
void qpx_CFG_parameterFile(char *filename,int count)
{
    qps_GUI_printf("\nParameters file %s has %d variables\n", filename, count);
}

/* -----
 * Read/Store/Update API Variables
 *
 * This call is made for each of the parameters supplied in the plugin
 * parameter file
 * ----- */
void qpx_GUI_parameterValue(char *filename,int index,char *label,float value)
{
    /* check file name */
    if (strcmp(filename, g_ParamFile) == 0)
    {
        /* store/update variables */
        if (strcmp(label, "Required Stimulus Period (Mins.)" == 0)
            g_StimulateTime = value * 60.0 ;
        if (strcmp(label, "Speed Restriction Distance (ft.)" == 0)
            g_RestrictionDistance = value;
    }
}

```



```

/* -----
 * Called once for every time step - The main body of our plugin.
 * ----- */
void qpx_NET_timeStep(void)
{
    int i;
    int j;
    int k;
    int t;
    int m;
    int n;
    int o;
    LINK* linkD;
        LINK* linkE;
        LINK* linkF;
        LINK* linkG;
        LINK* linkH;
        LINK* linkP;
        LOOP* loopplane1;
        LOOP* loopplane2;
        LOOP* loopplane3;
        DETECTOR* detec;
    char* link_ref_nameD;
        char* link_ref_nameE;
        char* link_ref_nameF;
        char* link_ref_nameG;
        char* link_ref_nameH;
    int loop_count;
    int loop_lanes;
    int loop_index;
        int LCVS;
        int LQ;
        int LCOVV;
    float ave_flow;
    float ave_speed;
        float Occ1;
        float Occ2;
        float Occ3;
        float SumStdev;
        float CVS;
        float Q;
        float D;
        float AvgSpeed1;
        float AvgSpeed2;
        float AvgSpeed3;
        float AvgUpSpeed;

```

```

float AvgDownSpeed;
float AvgUpDownSpeed;
float SpeedLimit;
float SumUpSpeeds1;
float SumUpSpeeds2;
float SumUpSpeeds3;
float upstreams;
float downstreams;
float AvgSpeed;

/* as the timestep is usually > 1 we should check that we are in
* a whole second first */
if (qpg_CFG_simulationTime() - (float)floor((double)qpg_CFG_simulationTime()) >
0.0)
    return;
    o = nBeacons - 10;
/* we are in a whole second so query each of the detectors associated
* with the VMS beacons */
for (i = 0; i < nBeacons; i++)
{
    /* link data */
    link_ref_nameF = g_BeaconLinks[i];
    linkF = qpg_NET_link(link_ref_nameF);
    detec = qpg_NET_detector(link_ref_nameF);
    looplane1 = qpg_DTC_multipleLoop(detec, 1);
    looplane2 = qpg_DTC_multipleLoop(detec, 2);
    looplane3 = qpg_DTC_multipleLoop(detec, 3);

    /*qps_GUI_printf("\ni = %d: link %s", i, link_ref_name);*/

    /* now collect the average flow for the detector(s) associated
    * with the current link */
    loop_lanes = 0;

    /* the number of detectors on the link associated with the current
    * beacon */
    loop_count = qpg_LNK_detectors(linkF);

    for (k = 0; k < loop_count; k++)
    {
        /* get the network index of the loop(s) associated with
        * the the current VMS beacon on the current link */
        loop_index = qpg_LNK_detectorIndexByIndex(linkF, k+1);
        /* the number of lanes the current detector covers */
        /*qps_GUI_printf(" :Detector %d (%s)\n", (i+1) ,

```

```

qpg_DTI_name(qpg_LNK_detectorIndexByIndex(linkF, 1))); */

    loop_lanes += qpg_DTI_lanes(loop_index);

    for(j = 0; j < loop_lanes; j++)
        {ave_flow      +=      qpg_DTI_flow(loop_index,      j+1,
APILOOP_SMOOTHED);}

        /* get the speed on each lane of each detector */
        g_Speeds1[i]      =      qpg_DTI_speed(loop_index,      1,
APILOOP_SMOOTHED);
        g_Speeds2[i]      =      qpg_DTI_speed(loop_index,      2,
APILOOP_SMOOTHED);
        g_Speeds3[i]      =      qpg_DTI_speed(loop_index,      3,
APILOOP_SMOOTHED);
    }

}

/*LOOP* qpg_DTC_multipleLoop(DETECTOR* detector, int lane);*/

/* Calculate crash precursors */
if(((int)qpg_CFG_simulationTime()) % 30 == 0)
{

    t=(((int)qpg_CFG_simulationTime()-56700)/30);
    for( j= 21 ; j < 29 ;j++)
    {
        i = 2 * j;
        link_ref_nameD = g_BeaconLinks[i+3];
        linkD = qpg_NET_link(link_ref_nameD);
        link_ref_nameE = g_BeaconLinks[i+1];
        linkE = qpg_NET_link(link_ref_nameE);
        link_ref_nameF = g_BeaconLinks[i-1];
        linkF = qpg_NET_link(link_ref_nameF);
        link_ref_nameG = g_BeaconLinks[i-3];
        linkG = qpg_NET_link(link_ref_nameG);
        link_ref_nameH = g_BeaconLinks[i-5];
        linkH = qpg_NET_link(link_ref_nameH);

        detec = qpg_NET_detector(link_ref_nameF);

        looplane1 = qpg_DTC_multipleLoop(detec, 1);
        looplane2 = qpg_DTC_multipleLoop(detec, 2);
    }
}

```

```

looplane3 = qpg_DTC_multipleLoop(detec, 3);

if( t > 0)
{
    AvgSpeed1 = 0.0;
    AvgSpeed2 = 0.0;
    AvgSpeed3 = 0.0;

    g_UpSpeeds1[t] = g_Speeds1[i]*2.24;
    g_UpSpeeds2[t] = g_Speeds2[i]*2.24;
    g_UpSpeeds3[t] = g_Speeds3[i]*2.24;

    g_Occ1[t]      =      qpg_DTL_occupancy(      looplane1,
APILOOP_SMOOTHED);
    g_Occ2[t]      =      qpg_DTL_occupancy(      looplane2,
APILOOP_SMOOTHED);
    g_Occ3[t]      =      qpg_DTL_occupancy(      looplane3,
APILOOP_SMOOTHED);
    /*qps_GUI_printf("\n Occupancy on three lanes is %f %f
%f\n",g_Occ1[t],g_Occ2,g_Occ3);*/
    g_DownSpeeds1[t] = g_Speeds1[i-2]*2.24;
    g_DownSpeeds2[t] = g_Speeds2[i-2]*2.24;
    g_DownSpeeds3[t] = g_Speeds3[i-2]*2.24;

    g_UpLaneSpeeds[t]                                     =
(g_UpSpeeds1[t]+g_UpSpeeds2[t]+g_UpSpeeds3[t])/3;
    g_DownLaneSpeeds[t]                                   =
(g_DownSpeeds1[t]+g_DownSpeeds2[t]+g_DownSpeeds3[t])/3;
    qps_GUI_printf("\n Up and Down speed for every 30 s is
%d %d at i=%d\n",(int)g_UpLaneSpeeds[t],(int)g_DownLaneSpeeds[t],i);

    g_SumUpLaneSpeeds[i]=g_SumUpLaneSpeeds[i]+g_UpLaneSpeeds[t];

    g_SumDownLaneSpeeds[i]=g_SumDownLaneSpeeds[i]+g_DownLaneSpeeds[t];

    if(((int)qpg_CFG_simulationTime()) % 300 == 0)
    {
        link_ref_nameD = g_BeaconLinks[i+3];
        linkD = qpg_NET_link(link_ref_nameD);
        link_ref_nameE = g_BeaconLinks[i+1];
        linkE = qpg_NET_link(link_ref_nameE);
        link_ref_nameF = g_BeaconLinks[i-1];
        linkF = qpg_NET_link(link_ref_nameF);
        link_ref_nameG = g_BeaconLinks[i-3];
        linkG = qpg_NET_link(link_ref_nameG);
        link_ref_nameH = g_BeaconLinks[i-5];
    }
}

```

```

linkH = qpg_NET_link(link_ref_nameH);

AvgUpSpeed=g_SumUpLaneSpeeds[i]/10;
AvgDownSpeed=g_SumDownLaneSpeeds[i]/10;
qps_GUI_printf("\n Up and Down 5 minutes
average is %f %f at i= %d\n",AvgUpSpeed, AvgDownSpeed,i);
Q=AvgUpSpeed-AvgDownSpeed;
qps_GUI_printf("\n Q = %f at i=%d\n",Q,i);
m=qpg_LNK_lanes(linkF);
qps_GUI_printf("\n number of lanes =%d\n",m);

if (Q > 0 || Q < 0)
{
    if ((int)AvgDownSpeed <= 35)
    {
        qps_GUI_printf("\n Less than 35 and
speed diff = %f at i=%d\n",Q,i);
        qps_GUI_printf("\n Original Speed
Limit at this link is %d\n",(int)g_BaseSpeeds[pp_link_id_lookup(linkF) - 1]);
        /*pp_adjust_speed(linkD, SPEED_F,
i+4);
        pp_adjust_speed(linkE, SPEED_D,
i+2);*/
        for( n = 0; n < m; n++)
        {qps_LNK_speedcontrol( linkF,
n+1, 0);qps_GUI_printf("\n speed control removed\n");}

        /*g_SpeedLimit[i]=40;*/
        /*pp_adjust_speed(linkD,
SPEED_A, i+3);
i+1);*/
1);
i-3);
i-5);

        pp_adjust_speed(linkE, SPEED_B,
pp_adjust_speed(linkF, SPEED_C, i-
pp_adjust_speed(linkG, SPEED_D,
pp_adjust_speed(linkH, SPEED_E,

        /*g_SpeedLimit[i+4]=45;
g_SpeedLimit[i+2]=45;*/
        /*g_SpeedLimit[i]=40;
        /*g_SpeedLimit[i-2]=40;
g_SpeedLimit[i-4]=50;

```

```

/*qps_GUI_printf("\n Speed Limit at
this link reset to %d\n",(int)g_BaseSpeeds[pp_link_id_lookup(linkF) - 1]);*/
}
else if((int)AvgDownSpeed > 35 &&
(int)AvgDownSpeed <= 45)
{
qps_GUI_printf("\n More than 35
Less than 45, speed diff = %f\n",Q);
qps_GUI_printf("\n Original Speed
Limit at this link is %d\n",(int)g_BaseSpeeds[pp_link_id_lookup(linkF) - 1]);
for( n = 0; n < m; n++)
{qps_LNK_speedcontrol( linkF,
n+1, 0);qps_GUI_printf("\n speed control removed\n");}
/*pp_adjust_speed(linkD,
SPEED_B, i+3);
pp_adjust_speed(linkE, SPEED_C,
pp_adjust_speed(linkF, SPEED_D, i-
1);
pp_adjust_speed(linkG, SPEED_E,
i-3);
pp_adjust_speed(linkH, SPEED_F, i-
5);
/*g_SpeedLimit[i+4]=55;
g_SpeedLimit[i+2]=50;
g_SpeedLimit[i]=45;
g_SpeedLimit[i-2]=55;
g_SpeedLimit[i-4]=60;
/*qps_GUI_printf("\n Speed Limit at
this link reset to %d\n",(int)g_BaseSpeeds[pp_link_id_lookup(linkF) - 1]);*/
}
else if((int)AvgDownSpeed > 45 &&
(int)AvgDownSpeed <= 55)
{
qps_GUI_printf("\n More than 45
Less than 55, speed diff = %f\n",Q);
/*qps_GUI_printf("\n Original Speed
Limit at this link is %d\n",(int)g_BaseSpeeds[pp_link_id_lookup(linkF) - 1]);
qps_LNK_speedcontrol( linkF, 1, 0);
qps_LNK_speedcontrol( linkF, 2, 0);
qps_LNK_speedcontrol( linkF, 3,
0);*/
for( n = 0; n < m; n++)

```

```

n+1, 0);qps_GUI_printf("\n speed control removed\n");}
/*pp_adjust_speed(linkD,
SPEED_C, i+3);
i+1);*/
1);
3);
i-5);

/*g_SpeedLimit[i+4]=55;
g_SpeedLimit[i+2]=55;
g_SpeedLimit[i]=55;
g_SpeedLimit[i-2]=60;
g_SpeedLimit[i-4]=60;
/*qps_GUI_printf("\n Speed Limit at
this link reset to %d\n",(int)g_BaseSpeeds[pp_link_id_lookup(linkF) - 1]);*/

}
else if((int)AvgDownSpeed > 55)
{
qps_GUI_printf("\n huge speeds
going on buddy, speed diff = %f\n",Q);
qps_GUI_printf("\n Original Speed
Limit at this link is %d\n",(int)g_BaseSpeeds[pp_link_id_lookup(linkF) - 1]);
for(n = 0; n < m; n++)
{qps_LNK_speedcontrol( linkF,
n+1, 0);qps_GUI_printf("\n speed control removed\n");}
/*pp_adjust_speed(linkD,
SPEED_D, i+3);
i+1);*/
1);
i-3);
i-5);

/*g_SpeedLimit[i+4]=65;
g_SpeedLimit[i+2]=65;
g_SpeedLimit[i]=65;
g_SpeedLimit[i-2]=65;

```

```

        g_SpeedLimit[i-4]=65;

        /*qps_GUI_printf("\n Speed Limit at
this link reset to %d\n",(int)g_BaseSpeeds[pp_link_id_lookup(linkF) - 1]);*/

    }
}

    g_SumUpLaneSpeeds[i] = 0.0;
    g_SumDownLaneSpeeds[i] = 0.0;
}

    }
}

}

/* -----
* Allocate memory for all dynamic objects
* ----- */
void pp_allocate_memory(void)
{
    int i;

    /* for each object clear old memory, and allocate new */

    if (g_BaseSpeeds != NULL) free(g_BaseSpeeds);
    g_BaseSpeeds = calloc(sizeof(float), nLinks);

    if (g_Speeds1 != NULL) free(g_Speeds1);
    g_Speeds1 = calloc(sizeof(float), nLinks);

    if (g_Speeds2 != NULL) free(g_Speeds2);
    g_Speeds2 = calloc(sizeof(float), nLinks);

    if (g_Speeds3 != NULL) free(g_Speeds3);
    g_Speeds3 = calloc(sizeof(float), nLinks);

    if (g_UpSpeeds1 != NULL) free(g_UpSpeeds1);
    g_UpSpeeds1 = calloc(sizeof(float), nLinks);
    /*g_UpSpeeds1 = malloc((nLinks) * sizeof(float *));
    for (i = 0; i < (nLinks); i++)
        g_UpSpeeds1[i] = malloc((nLinks) * sizeof(float)); */

    if (g_UpSpeeds2 != NULL) free(g_UpSpeeds2);
    g_UpSpeeds2 = calloc(sizeof(float), nLinks);

```



```

if (g_UpSpeeds3 != NULL) free(g_UpSpeeds3);
g_UpSpeeds3 = calloc(sizeof(float), nLinks);

if (g_DownSpeeds1 != NULL) free(g_DownSpeeds1);
g_DownSpeeds1 = calloc(sizeof(float), nLinks);

if (g_DownSpeeds2 != NULL) free(g_DownSpeeds2);
g_DownSpeeds2 = calloc(sizeof(float), nLinks);

if (g_DownSpeeds3 != NULL) free(g_DownSpeeds3);
g_DownSpeeds3 = calloc(sizeof(float), nLinks);

if (g_AvgUpSpeed != NULL) free(g_AvgUpSpeed);
g_AvgUpSpeed = calloc(sizeof(float), nLinks);

if (g_UpLaneSpeeds != NULL) free(g_UpLaneSpeeds);
g_UpLaneSpeeds = calloc(sizeof(float), nLinks);

if (g_DownLaneSpeeds != NULL) free(g_DownLaneSpeeds);
g_DownLaneSpeeds = calloc(sizeof(float), nLinks);

if (g_SpeedLimit != NULL) free(g_SpeedLimit);
g_SpeedLimit = calloc(sizeof(int), nLinks);

if (g_LinkLookup != NULL) free(g_LinkLookup);
g_LinkLookup = calloc(sizeof(char), nLinks);

if (g_BeaconLinks != NULL) free(g_BeaconLinks);
g_BeaconLinks = calloc(sizeof(char), nLinks);

if (g_LoopLinks != NULL) free(g_LoopLinks);
g_LoopLinks = calloc(sizeof(char), nLinks);
}

/* -----
 * Collect the initial links speeds throughout the network and build a
 * lookup table of Link index No/Names
 * ----- */
void pp_collect_initial_speeds(void)
{
    int i;

    qps_GUI_printf("\nCollecting data about existing link speed limits.\n");

```

```

    for (i = 0; i < nLinks; i++)
    {
        g_BaseSpeeds[i] = qpg_LNK_speedlimit(qpg_NET_linkByIndex(i+1));
        qps_GUI_printf("\nexisting link speed limits at link %d is
%d.\n",i,(int)g_BaseSpeeds[i]);
        g_LinkLookup[i] = qpg_LNK_name(qpg_NET_linkByIndex(i+1));
    }
}

/* -----
 * Collect information on beacons in our network
 * ----- */
void pp_check_beacons(void)
{
    int i;
    int j;
    LINK* linkF = NULL;

    /* tell the user about the VMS signs we found in our network */
    if (nBeacons <= 0)
    {
        /* no VMS beacons in our network ? */
        qps_GUI_printf("\nWARNING: No VMS beacons found in current network.\n");
        return;
    }
    else
    {
        /* report and store data */
        qps_GUI_printf("\n%d Beacons found in current network.\n", nBeacons);

        for (i = 0; i < nLinks; i++)
        {
            linkF = qpg_NET_linkByIndex(i+1);
            if (qpg_LNK_beacons(linkF) > 0)
            {
                /* this link has a beacon(s) */
                qps_GUI_printf("\n :Link %s has %d beacons(s)\n",
                    qpg_LNK_name(linkF),qpg_LNK_beacons(linkF));
                /* report the details of each beacon */
                for (j = 0; j < qpg_LNK_beacons(linkF); j++)
                {
                    qps_GUI_printf(" :Beacon %d (%s)\n", (j+1) ,
qpg_BCI_name(qpg_LNK_beaconIndexByIndex(linkF, j+1)));
                    /* store the link name associated with each beacon index */

```

```

        g_BeaconLinks[(qpg_LNK_beaconIndexByIndex(linkF,
j+1) - 1)] = qpg_LNK_name(linkF);
    }
    qps_GUI_printf("\nexisting link speed limits at this link %d is
%d.\n",i,qpg_LNK_speedlimit(linkF));

    }
    linkF = NULL;
}
}
}
}

```

```

/* -----
* Collect information on loops in our network
* ----- */
void pp_check_loops(void)
{
    int i = 0;
    int j = 0;
    LINK* linkF;

    /* tell the user about the loop detectors we found in our network */
    if (nLoops <= 0)
    {
        /* no detectors found in our network ? */
        qps_GUI_printf("\nWARNING: No loop detectors found in current network.\n");
        return;
    }
    else
    {
        /* report and store data */
        qps_GUI_printf("\n%d Loop Detectors found in current network.\n", nLoops);

        for (i = 0; i < nLinks; i++)
        {
            linkF = qpg_NET_linkByIndex(i+1);
            if (qpg_LNK_detectors(linkF) > 0)
            {
                /* this link has a detector(s) */
                qps_GUI_printf("\n :Link %s has %d detector(s)\n",
                    qpg_LNK_name(linkF),qpg_LNK_detectors(linkF));
                /* report the details of each detector */
                for (j = 0; j < qpg_LNK_detectors(linkF); j++)
                {

```

```

        qps_GUI_printf(" :Detector %d (%s)\n", (j+1),
        qpg_DTI_name(qpg_LNK_detectorIndexByIndex(linkF, j+1)));
        /* store the link name associated with each loop index */
        g_LoopLinks[(qpg_LNK_detectorIndexByIndex(linkF,
j+1) - 1)] = qpg_LNK_name(linkF);
    }
}
linkF = NULL;
}
}
}

/* -----
* Match at least one loop on the same link as a beacon
* ----- */
Bool pp_beacon_loop_match(char *linkC)
{
    int i = 0;
    Bool found = FALSE;

    while(i < nLoops && !found)
    {
        if (g_LoopLinks[i] == linkC)
        {
            found = TRUE;
        }
        i++;
    }
    return found;
}

/* -----
* Apply the required speed restriction / adjustments
* Note: this function can be used with the SPEED_NORM flag to remove
* any currently imposed speed restrictions.
* ----- */
void pp_adjust_speed(LINK* linkC, int flagS, int indexB)
{
    /* hex color ID for VMS signs */
    int hex_red = 0x000001ff;
    int hex_amber = 0x001aa6ff;
    int hex_green = 0x0035dd6b;
    int m;
    int n;

```

```

float dist = 0.0;
float orgSpeed;
LINK* linkD;
    LINK* linkE;
    LINK* linkP;
    LINK* linkG;
    LINK* linkH;
Bool stopLoop;
char* link_ref_name = qpg_LNK_name(linkC);

/* The distance we apply our speed restriction over will include 'linkC'
 * i.e. the link the VMS sign is located on and then any following links
 * ensuring that the total restriction length is <= the 'g_RestrictionDistance'
 * parameter defined by the user.*/

switch (flagS)
{
    case SPEED_A:
        /* restore link speeds to original */
        qps_BCI_colour(indexB, hex_green);
        qps_BCI_message(indexB, " Warning          \n"
                           "    Drive Safely  \n"
                           "        35 mph    ");
        /* restore original speeds*/
        linkP = linkC;
        m = qpg_LNK_lanes(linkP);
        dist = qpg_LNK_length(linkP);
        qps_LNK_speedlimit(linkP, (float)35);
        qps_GUI_printf("\n 35 Mph. Speed restriction imposed from link %s, for
        %4.0f %s. at %s.\n",
            link_ref_name, g_RestrictionDistance, qpg_UTL_distanceUnitLabel(),
            qpg_UTL_integerToTimeString((int)qpg_CFG_simulationTime()));
        break;

    case SPEED_B:
        /* restore link speeds to original */
        qps_BCI_colour(indexB, hex_green);
        qps_BCI_message(indexB, " Warning          \n"
                           "    Drive Safely  \n"
                           "        40mph    ");
        /* restore original speeds*/
        linkP = linkC;
        m = qpg_LNK_lanes(linkP);
        dist = qpg_LNK_length(linkP);
        qps_LNK_speedlimit(linkP, (float)40);

```

```

/* report */
qps_GUI_printf("\n40 Mph. Speed restriction imposed from link %s, for %4.0f
%s. at %s.\n",
    link_ref_name, g_RestrictionDistance, qpg_UTL_distanceUnitLabel(),
    qpg_UTL_integerToTimeString((int)qpg_CFG_simulationTime()));
break;

case SPEED_C:
/* restore link speeds to original */
qps_BCI_colour(indexB, hex_green);
qps_BCI_message(indexB, " Warning      \n"
    "    Drive Safely  \n"
    "        45mph    ");
/* restore original speeds*/
linkP = linkC;
m = qpg_LNK_lanes(linkP);
dist = qpg_LNK_length(linkP);
qps_LNK_speedlimit(linkP, (float)45);
/* report */
qps_GUI_printf("\n45 Mph. Speed restriction imposed from link %s, for %4.0f
%s. at %s.\n",
    link_ref_name, g_RestrictionDistance, qpg_UTL_distanceUnitLabel(),
    qpg_UTL_integerToTimeString((int)qpg_CFG_simulationTime()));
break;

case SPEED_D:
/* restore link speeds to original */
qps_BCI_colour(indexB, hex_green);
qps_BCI_message(indexB, " Warning      \n"
    "    Drive Safely  \n"
    "        50mph    ");
/* restore original speeds*/
linkP = linkC;
m = qpg_LNK_lanes(linkP);
dist = qpg_LNK_length(linkP);
qps_LNK_speedlimit(linkP, (float)50);
/* report */
qps_GUI_printf("\n50 Mph. Speed restriction imposed from link %s, for %4.0f
%s. at %s.\n",
    link_ref_name, g_RestrictionDistance, qpg_UTL_distanceUnitLabel(),
    qpg_UTL_integerToTimeString((int)qpg_CFG_simulationTime()));
break;

case SPEED_E:
/* restore link speeds to original */

```

```

qps_BCI_colour(indexB, hex_green);
qps_BCI_message(indexB, " Warning      \n"
                    "    Drive Safely \n"
                    "        55mph   ");

/* restore original speeds*/
linkP = linkC;
m = qpg_LNK_lanes(linkP);
dist = qpg_LNK_length(linkP);
qps_LNK_speedlimit(linkP, (float)55);
/* report */
qps_GUI_printf("\n55 Mph. Speed restriction imposed from link %, for %4.0f
%, at %,.\n",
               link_ref_name, g_RestrictionDistance, qpg_UTL_distanceUnitLabel(),
               qpg_UTL_integerToTimeString((int)qpg_CFG_simulationTime()));
break;

case SPEED_F:
/* restore link speeds to original */
qps_BCI_colour(indexB, hex_green);
qps_BCI_message(indexB, " Warning      \n"
                    "    Drive Safely \n"
                    "        60mph   ");

/* restore original speeds*/
linkP = linkC;
m = qpg_LNK_lanes(linkP);
dist = qpg_LNK_length(linkP);
qps_LNK_speedlimit(linkP, (float)60);
/* report */
qps_GUI_printf("\n60 Mph. Speed restriction imposed from link %, for %4.0f
%, at %,.\n",
               link_ref_name, g_RestrictionDistance, qpg_UTL_distanceUnitLabel(),
               qpg_UTL_integerToTimeString((int)qpg_CFG_simulationTime()));
break;
case SPEED_G:
/* restore link speeds to original */
qps_BCI_colour(indexB, hex_green);
qps_BCI_message(indexB, " Warning      \n"
                    "    Drive Safely \n"
                    "        65 mph   ");

/* restore original speeds*/
linkP = linkC;
m = qpg_LNK_lanes(linkP);
dist = qpg_LNK_length(linkP);
qps_LNK_speedlimit(linkP, (float)65);
/* report */

```

```

        qps_GUI_printf("\n65 Mph. Speed restriction imposed from link %s, for %4.0f
%s. at %s.\n",
        link_ref_name, g_RestrictionDistance, qpg_UTL_distanceUnitLabel(),
        qpg_UTL_integerToTimeString((int)qpg_CFG_simulationTime()));
        break;

    case SPEED_H:
        /* restore link speeds to original */
        qps_BCI_colour(indexB, hex_green);
        qps_BCI_message(indexB, " Warning          \n"
        "    Drive Safely  \n"
        "        70mph    ");

        /* restore original speeds*/
        linkP = linkC;
        m = qpg_LNK_lanes(linkP);
        dist = qpg_LNK_length(linkP);
        qps_LNK_speedlimit(linkP, (float)70);
        /* report */
        qps_GUI_printf("\n70 Mph. Speed restriction imposed from link %s, for %4.0f
%s. at %s.\n",
        link_ref_name, g_RestrictionDistance, qpg_UTL_distanceUnitLabel(),
        qpg_UTL_integerToTimeString((int)qpg_CFG_simulationTime()));
        break;

    case SPEED_I:
        /* restore link speeds to original */
        qps_BCI_colour(indexB, hex_green);
        qps_BCI_message(indexB, " Warning          \n"
        "    Drive Safely  \n"
        "        75 mph    ");

        /* restore original speeds*/
        linkP = linkC;
        m = qpg_LNK_lanes(linkP);
        dist = qpg_LNK_length(linkP);
        qps_LNK_speedlimit(linkP, (float)75);
        /* report */
        qps_GUI_printf("\n60 Mph. Speed restriction imposed from link %s, for %4.0f
%s. at %s.\n",
        link_ref_name, g_RestrictionDistance, qpg_UTL_distanceUnitLabel(),
        qpg_UTL_integerToTimeString((int)qpg_CFG_simulationTime()));
        break;

    case SPEED_NORM:
        /* restore link speeds to original */
        qps_BCI_colour(indexB, hex_green);
        qps_BCI_message(indexB, "          \n"

```



```

                "    Drive Safely  \n"
                "                ");
/* restore original speeds*/
linkP = linkC;
m = qpg_LNK_lanes(linkP);
dist = qpg_LNK_length(linkP);
orgSpeed = g_BaseSpeeds[pp_link_id_lookup(linkP) - 1];
qps_LNK_speedlimit(linkP, orgSpeed);
qps_GUI_printf("\nSpeed restriction removed from link %s, for %4.0f %s. at
%s.\n",
                link_ref_name, g_RestrictionDistance, qpg_UTL_distanceUnitLabel(),
                qpg_UTL_integerToTimeString((int)qpg_CFG_simulationTime()));
break;
    }
}

```



```

ActiveWorkbook.Close

Call AddOne

Next

End Sub
Sub AddOne()
    Workbooks.Add

End Sub

Sub dataacc(filename As String, sheetname As String, seed As Integer, i As Integer, k
As String, net As String)
    Dim insertsheet As String
    Dim seed1 As String
    Dim j As Integer
    Dim seed2 As String

    ActiveWorkbook.Worksheets.Add
    With ActiveSheet.QueryTables.Add(Connection:=Array( _
        "OLEDB;Provider=Microsoft.Jet.OLEDB.4.0;Password=""";User ID=Admin;Data
Source=C:\Documents and Settings\ADhindsa\Desktop\Runs test case" _
        , _
        "s\Accum\Run " + net + ".xls;Mode=Share Deny Write;Extended
Properties=""HDR=NO;"";Jet OLEDB:System database=""";Jet OLEDB:Registry
Path=""";Je" _
        , _
        "t OLEDB:Database Password=""";Jet OLEDB:Engine Type=35;Jet
OLEDB:Database Locking Mode=0;Jet OLEDB:Global Partial Bulk Ops=2;Jet " _
        , _
        "OLEDB:Global Bulk Transactions=1;Jet OLEDB:New Database
Password=""";Jet OLEDB:Create System Database=False;Jet OLEDB:Encrypt Dat" _
        , _
        "abase=False;Jet OLEDB:Don't Copy Locale on Compact=False;Jet
OLEDB:Compact Without Replica Repair=False;Jet OLEDB:SFP=False" _
    ), Destination:=Range("A1"))
    .CommandType = xlCmdTable
    .CommandText = sheetname
    .Name = filename
    .FieldNames = True
    .RowNumbers = False
    .FillAdjacentFormulas = False
    .PreserveFormatting = True
    .RefreshOnFileOpen = False
    .BackgroundQuery = True

```

```

.RefreshStyle = xlInsertDeleteCells
.SavePassword = False
.SaveData = True
.AdjustColumnWidth = True
.RefreshPeriod = 0
.PreserveColumnInfo = True
.SourceDataFile = _
"C:\Documents and Settings\ADhindsa\Desktop\Runs test cases\Accum\Run " + net
+ ".xls"
.Refresh BackgroundQuery:=False
End With
j = i - 25
seed1 = j
seed2 = i
insertsheet = "Sheet" + seed1
ActiveSheet.Name = insertsheet
Sheets(insertsheet).Select
ActiveSheet.Name = sheetname
Rows("1:1").Select
Selection.Delete Shift:=xlUp
Selection.Insert Shift:=xlDown
Columns("A:A").Select
Selection.Delete Shift:=xlToLeft
Range("A1").Select
ActiveCell.FormulaR1C1 = "LS"
Range("B1").Select
ActiveCell.FormulaR1C1 = "CS"
Range("C1").Select
ActiveCell.FormulaR1C1 = "RS"
Range("D1").Select
ActiveCell.FormulaR1C1 = "LV"
Range("E1").Select
ActiveCell.FormulaR1C1 = "CV"
Range("F1").Select
ActiveCell.FormulaR1C1 = "RV"
Range("K1").Select
ActiveCell.Value = "Av_speed"
Range("K2").Select
ActiveCell.FormulaR1C1 = "=AVERAGE(RC[-10]:RC[-8])"
Range("K2").Select

Selection.AutoFill Destination:=Range("K2:K361"), Type:=xlFillDefault
Range("K2:K361").Select

Range("L1").Select
ActiveCell.FormulaR1C1 = "flow"

```

```

Range("L2").Select
ActiveCell.FormulaR1C1 = "=SUM(RC[-8]:RC[-6])"
Range("L2").Select
Selection.AutoFill Destination:=Range("L2:L361")
Range("L2:L361").Select

```

```

Range("M1").Select
ActiveCell.FormulaR1C1 = "simspat5"

```

```

Range("M11").Select
ActiveCell.FormulaR1C1 = "=AVERAGE(R[-9]C[-2]:RC[-2])"
Range("M21").Select
ActiveCell.FormulaR1C1 = "=AVERAGE(R[-9]C[-2]:RC[-2])"
Columns("M:M").ColumnWidth = 12
Range("M2:M21").Select
Selection.AutoFill Destination:=Range("M2:M361")
Range("M2:M361").Select

```

```

Range("N1").Select
ActiveCell.FormulaR1C1 = "flow_at5"
Range("N11").Select
ActiveCell.FormulaR1C1 = "=SUM(R[-9]C[-2]:RC[-2])"
Range("N21").Select
ActiveCell.FormulaR1C1 = "=SUM(R[-9]C[-2]:RC[-2])"
Range("N2:N21").Select
Selection.AutoFill Destination:=Range("N2:N361"), Type:=xlFillDefault
Range("N2:N361").Select

```

```

    Call blanksacc
End Sub
Sub blanksacc()
'
' blanksacc Macro
' Macro recorded 8/9/2005 by Albinder Dhindsa
'

```

```

Columns("M:N").Select
Selection.SpecialCells(xlCellTypeBlanks).Select
Selection.Delete Shift:=xlUp
ActiveWindow.SmallScroll Down:=-9
Range("M2:N7").Select
Selection.Delete Shift:=xlUp
ActiveWindow.SmallScroll Down:=-6

```

```
Range("O1").Select
ActiveCell.FormulaR1C1 = "time"
Range("O2").Select
ActiveCell.FormulaR1C1 = "16:30"
Range("O3").Select
ActiveCell.FormulaR1C1 = "16:35"
Range("O2:O3").Select
Selection.AutoFill Destination:=Range("O2:O31")
Range("O2:O31").Select
ActiveWorkbook.Save
End Sub
```

Accumulation of Calibration Data Part 2

Sub Calibrationpartdeux()
,

' Calibrationpartdeux Macro

' Macro recorded 8/10/2005 by Albinder Dhindsa
,

Dim numruns As Integer

Dim startstn As Integer

Dim stopstn As Integer

Dim filename As String

Dim sheetname As String

Dim network As String

Dim seed As Integer

Dim i As Integer

Dim stn As Integer

Dim k As String

Dim stn1 As String

Dim p As Integer

Dim Name As String

Dim stn2 As String

Dim stn3 As Integer

Dim startstring As String

Dim stopstring As String

Dim q As Integer

Dim l As Integer

numruns = (InputBox("what is the number of runs you wish to accumulate over?"))

startstn = (InputBox("what is the starting station number?"))

stopstn = (InputBox("what is the stopping station number?"))

network = (InputBox("network string name?"))

startstring = startstn

stopstring = stopstn

Name = "Station" + startstring

ActiveSheet.Name = Name

For stn = startstn To stopstn

 'filename = "Run " + k

 seed = 0

 stn1 = stn

 If stn = 39 Then GoTo 100

For i = 11 To numruns

 k = i

 q = i - 10

 seed = seed + 1

```

sheetname = "Detect" + stn1 + "_seed_" + k + "$$"
filename = "Calibrationdataaccumulation" + k
p = 1 + ((q - 1) * 15)
Worksheets(Name).Activate
Call Dataget(filename, sheetname, network, seed, i, p, k, q)

Next

ActiveWorkbook.SaveAs filename:= _
"C:\Documents and Settings\ADhindsa\My
Documents\I4LoopData\Calibration\accbystation" + network + startstring + stopstring +
".xls" _
, FileFormat:=xlNormal, Password:="", WriteResPassword:="", _
ReadOnlyRecommended:=False, CreateBackup:=False

stn3 = stn + 1
stn2 = stn3
Name = "Station" + stn2
Set NewSheet = Worksheets.Add
NewSheet.Name = "Station" + stn2

100 Next
End Sub

Sub Dataget(filename As String, sheetname As String, network As String, seed As
Integer, i As Integer, p As Integer, k As String, q As Integer)

Dim l As Integer

With ActiveSheet.QueryTables.Add(Connection:=Array( _
"OLEDB;Provider=Microsoft.Jet.OLEDB.4.0;Password=""";User ID=Admin;Data
Source=C:\Documents and Settings\ADhindsa\My Documents\I4LoopDat" _
"a\Calibration\" + network + "\calibrationdataaccumulation" + k +
".xls;Mode=Share Deny Write;Extended Properties="""HDR=YES;""";Jet OLEDB:Syste"
_
"m database=""";Jet OLEDB:Registry Path=""";Jet OLEDB:Database
Password=""";Jet OLEDB:Engine Type=35;Jet OLEDB:Database Locking Mode" _
"=0;Jet OLEDB:Global Partial Bulk Ops=2;Jet OLEDB:Global Bulk
Transactions=1;Jet OLEDB:New Database Password=""";Jet OLEDB:Create " _
"System Database=False;Jet OLEDB:Encrypt Database=False;Jet OLEDB:Don't
Copy Locale on Compact=False;Jet OLEDB:Compact Without Re" _
, "plica Repair=False;Jet OLEDB:SFP=False"), Destination:=Cells(1, p))

```



```

.CommandType = xlCmdTable
.CommandText = sheetname
.Name = "calibrationdataaccumulation" + k
.FieldNames = True
.RowNumbers = False
.FillAdjacentFormulas = False
.PreserveFormatting = True
.RefreshOnFileOpen = False
.BackgroundQuery = True
.RefreshStyle = xlInsertDeleteCells
.SavePassword = False
.SaveData = True
.AdjustColumnWidth = True
.RefreshPeriod = 0
.PreserveColumnInfo = True
.SourceDataFile = _
"C:\Documents and Settings\ADhindsa\My Documents\I4LoopData\Calibration\" +
network + "\"calibrationdataaccumulation" + k + ".xls"
.Refresh BackgroundQuery:=False
End With

End Sub

```

Accumulation of MCI from Risk Index Files by station

Sub RiskIndexEval()

,

' RiskIndexEval Macro

' Macro recorded 8/15/2005 by Albinder Dhindsa

,

Dim numruns As Integer

Dim startstn As Integer

Dim stopstn As Integer

Dim filename As String

Dim sheetname As String

Dim network As String

Dim seed As Integer

Dim i As Integer

Dim stn As Integer

Dim k As String

Dim stn1 As String

Dim p As Integer

Dim Name As String

Dim stn2 As String

Dim stn3 As Integer

Dim startstring As String

Dim stn4 As String

Dim newname As String

Dim s As Integer

numruns = (InputBox("what is the number of runs you wish to accumulate over?"))

startstn = (InputBox("what is the starting station number?"))

stopstn = (InputBox("what is the stopping station number?"))

network = (InputBox("network string name?"))

For stn = startstn To stopstn

If stn = 39 Then GoTo 100

startstring = stn

 Name = "Station" + startstring

 'filename = "Run " + k

 seed = 0

 stn1 = stn

Worksheets.Add

ActiveSheet.Name = Name

For i = 1 To numruns

 k = i

 seed = seed + 1

 sheetname = "Detect" + stn1 + "_seed_" + k

 filename = "Run " + k

```

        p = 2 * i - 1
        Call Datacall(filename, sheetname, network, seed, i, p, k, Name)

    Next
100 Next

ActiveWorkbook.SaveAs filename:= _
    "C:\Documents and Settings\ADhindsa\My Documents\I4LoopData\Risk Indices" +
network + ".xls", _
    FileFormat:=xlNormal, Password:="", WriteResPassword:="", _
    ReadOnlyRecommended:=False, CreateBackup:=False
    s = 0
    For stn = startstn To stopstn
    If stn = 39 Then GoTo 200
    stn4 = stn
    newname = "Station" + stn4
    s = s + 1
    Call sumrisks(newname, numruns)
    Call Riskstationgraphs(newname, s)
200    Next
ActiveWorkbook.SaveAs filename:= _
    "C:\Documents and Settings\ADhindsa\My Documents\I4LoopData\Risk Indices" +
network + ".xls", _
    FileFormat:=xlNormal, Password:="", WriteResPassword:="", _
    ReadOnlyRecommended:=False, CreateBackup:=False

End Sub

Sub AddOne()
    Workbooks.Add

End Sub

Sub Datacall(filename As String, sheetname As String, network As String, seed As
Integer, i As Integer, p As Integer, k As String, Name As String)

    ChDir "C:\Documents and Settings\ADhindsa\My Documents\Riska\" + network
    Workbooks.Open filename:= _
        "C:\Documents and Settings\ADhindsa\My Documents\Riska\" + network + "\Run "
+ k + ".xls"
    Sheets(sheetname).Select
    Range("L1:M360").Select
    Range(Selection, Selection.End(xlDown)).Select
    Range(Selection, Selection.End(xlDown)).Select
    Selection.Copy

```

```
Windows("Book1").Activate
Sheets(Name).Select
Cells(1, p).Select
Selection.PasteSpecial Paste:=xlPasteValues, Operation:=xlNone, SkipBlanks _
:=False, Transpose:=False
Application.CutCopyMode = False
Windows("Run " + k + ".xls").Activate
ActiveWindow.Close

End Sub
```

Accumulating risk and speed data from Runs by station

Sub riskcomparison()

,

' riskcomparison Macro

' Macro recorded 9/24/2005 by Albinder Dhindsa

,

' Keyboard Shortcut: Ctrl+v

,

Dim numruns As Integer

Dim startstn As Integer

Dim stopstn As Integer

Dim filename As String

Dim sheetname As String

Dim network As String

Dim seed As Integer

Dim i As Integer

Dim stn As Integer

Dim k As String

Dim stn1 As String

Dim p As Integer

Dim Name As String

Dim stn2 As String

Dim stn3 As String

Dim startstring As String

Dim stopstring As String

Dim q As Integer

Dim l As Integer

Dim netname As String

startstn = (InputBox("what is the starting station number?"))

stopstn = (InputBox("what is the stopping station number?"))

seed = (InputBox("what is the seed number?"))

netname = (InputBox("what is the network name?"))

stn3 = seed

startstring = startstn

stopstring = stopstn

Sheets.Add

Sheets.Add

For q = startstn To stopstn

stn2 = q

l = q - 32

Name = "Detect" + stn2 + "_seed_" + stn3

Sheets(Name).Select

Columns("L:L").Select

Selection.Copy

```

Sheets("Sheet2").Select
Columns(1).Select
Selection.PasteSpecial Paste:=xlPasteValues, Operation:=xlNone, SkipBlanks _
:=False, Transpose:=False
Next
Sheets("Sheet2").Select
Range("A2:Q29").Select
Selection.Delete Shift:=xlUp
Range("A1").Select
ActiveCell.FormulaR1C1 = "M2_19"
Range("A1").Select
Selection.Copy
Range("B1").Select
ActiveSheet.Paste
Range("A1:B1").Select
Application.CutCopyMode = False
Selection.AutoFill Destination:=Range("A1:Q1"), Type:=xlFillDefault
Range("A1:Q1").Select
Range("A1").Select
ActiveCell.FormulaR1C1 = netname + "_33"
Range("B1").Select
ActiveCell.FormulaR1C1 = netname + "_34"
Range("A1:B1").Select
Selection.AutoFill Destination:=Range("A1:Q1"), Type:=xlFillDefault
Range("A1:Q1").Select
End Sub

```

Appendix II-F

ROUTE DIVERSION AND RAMP METERING CODE

Route Diversion API Code

```
static int diversion1 = 80;
static int diversion2 = 0;
static int diversion3 = 0;
static int diversion4 = 0;
static int diversion5 = 0;
int count1 = 0;
int count2 = 0;
int count3 = 0;
int count4 = 0;
int count5 = 0;
int qpo_RTM_decision(LINK* link, VEHICLE* vehicle)
{
    if (qpg_VHC_origin(vehicle) == 25 || qpg_VHC_origin(vehicle) == 82 ||
    qpg_VHC_origin(vehicle) == 35)
    {
        if (link == qpg_NET_linkByIndex(1414) && qpg_VHC_origin(vehicle) == 25 &&
        (qpg_VHC_destination(vehicle) >= 39 && qpg_VHC_destination(vehicle) <= 58))
        {
            if (rand()%100 < diversion1)
            {
                count1++;
                qps_GUI_printf("Diversion 1 %i\n", count1);
                return 2;
            }
            else return 0;
        }
        else if (link == qpg_NET_linkByIndex(1209) && qpg_VHC_origin(vehicle) == 25 &&
        (qpg_VHC_destination(vehicle) >= 39 && qpg_VHC_destination(vehicle) <= 58))
        {
            if (rand()%100 < diversion2 && (qpg_VHC_destination(vehicle) < 123))
            {
                count2++;
                qps_GUI_printf("Diversion 2 %i\n", count2);
                return 2;
            }
            else return 0;
        }
        else if (link == qpg_NET_linkByIndex(1132) && qpg_VHC_origin(vehicle) == 82 &&
        (qpg_VHC_destination(vehicle) == 3 || (qpg_VHC_destination(vehicle) >= 96 &&
        qpg_VHC_destination(vehicle) <= 121)))
        {
            if (rand()%100 < diversion3)
            {
                count3++;
                qps_GUI_printf("Diversion 3 %i\n", count3);
```



```

    return 2;
}
else return 0;
}
else if (link == qpg_NET_linkByIndex(1248) && qpg_VHC_origin(vehicle) == 82 &&
(qpg_VHC_destination(vehicle) == 3 || (qpg_VHC_destination(vehicle) >= 96 &&
qpg_VHC_destination(vehicle) <= 121)))
{
    if (rand()%100 < diversion4)
    {
        count4++;
        qps_GUI_printf("Diversion 4 %i\n", count4);
        return 3;
    }
    else return 0;;
}
else if (link == qpg_NET_linkByIndex(1329) && qpg_VHC_origin(vehicle) == 35 &&
(qpg_VHC_destination(vehicle) >= 39 && qpg_VHC_destination(vehicle) <= 58))
{
    if (rand()%100 < diversion5)
    {
        count5++;
        qps_GUI_printf("Diversion 5 %i\n", count5);
        return 1;
    }
    else return 0;
}
else return 0;
}
else return 0;
}
}

```

Sample of “Phases” File used for ALINEA Ramp Metering

use plan 1
on node 811 phase 1
with loops
 Detector68 lane 1
 Detector68 lane 2
 Detector68 lane 3

with parameters
30 ##1 cycle length
0.17 ##2 critical occupancy
10.5 ##3 minimum green
27 ##4 maximum green
0 ##5
0 ##6
0 ##7
0 ##8
0 ##9
0 ##10
0 ##11
0 ##12
0 ##13
0 ##14
1 ##15
0 ##16
27 ##17

Sample of “Plans” File used for ALINEA Ramp Metering

plan count 1

plan 1 definition

loops 3

parameters 17

if (init) { fixed; }

if (parameter[15] < 2 * parameter[1])

{

parameter[5] = occupancy[1] running;

parameter[6] = occupancy[2] running;

parameter[7] = occupancy[3] running;

if (parameter[5] > 0.5) {parameter[5] = 0.5;}

if (parameter[6] > 0.5) {parameter[6] = 0.5;}

if (parameter[7] > 0.5) {parameter[7] = 0.5;}

parameter[11] = parameter[11] + parameter[5];

parameter[12] = parameter[12] + parameter[6];

parameter[13] = parameter[13] + parameter[7];

parameter[15] = parameter[15] + 1;

green2=parameter [16];

green3=parameter [16];

}

else

{

parameter[14] = (parameter[11] + parameter[12] + parameter[13]) / (3 * parameter[1]);

if (parameter[14] > 1) {parameter[14] = 1;}

parameter[8] = parameter[5];

parameter[9] = parameter[6];

parameter[10] = parameter[7];

report parameter[14];

```
parameter[15] = 1;  
parameter[8] = 0; parameter[9] = 0; parameter[10] = 0; parameter[11] = 0; parameter[12]  
= 0; parameter[13] = 0;
```

```
parameter[16] = parameter[17] + parameter[1] / 730 * 70 * (parameter[2] -  
parameter[14]);
```

```
report parameter[16];
```

```
    if ( parameter [16] < parameter [3] )  
    {  
        parameter [16] = parameter [3];  
    }  
    if ( parameter [16] > parameter [4] )  
    {  
        parameter [16] = parameter [4] ;  
    }
```

```
green2=parameter [16];  
green3=parameter [16];
```

```
report parameter[16];
```

```
parameter[17] = parameter[16];
```

```
}
```

Zone Algorithm API Code

```
#include <stdlib.h>
#include <stdio.h>
#include <string.h>
#include <math.h>
#include <limits.h>

#include "programmer.h"
#include "plugin_p.h"

static int deteccount[15];
static int deteccountold[15];
static int detecflow[15];
static float smoothedflow[15];
static float smoothedflowold[15];
static float averagespeed[15];

static int oncount[9];
static int oncountold[9];
static int onflow[9];
static float onsmoothedflow[9];
static float onsmoothedflowold[9];

static int offcount[11];
static int offcountold[11];
static int offflow[11];
static float offsmoothedflow[11];
static float offsmoothedflowold[11];

static float proprate[9];

static float M;
static float rate[9];
static float minrate[9];

static int cycle = 30; // change me!!!

DETECTOR* det1;
NODE* n[9];
LINK* deteclink[15];
float density[15];
```

```

void zone1();
void zone2();
void zone3();

void qpx_NET_postOpen(void)
{
    int i;

    qps_GUI_printf("API: Ramp Metering\n");

    qps_GUI_printf("%s\n",qpg_DTC_name(qpg_NET_detectorByIndex(64)));

    n[0] = qpg_NET_nodeByIndex(765);
    n[1] = qpg_NET_nodeByIndex(758);
    n[2] = qpg_NET_nodeByIndex(759);
    n[4] = qpg_NET_nodeByIndex(188);
    n[5] = qpg_NET_nodeByIndex(460);
    n[6] = qpg_NET_nodeByIndex(761);
    n[7] = qpg_NET_nodeByIndex(199);
    n[8] = qpg_NET_nodeByIndex(202);

    qps_SIG_action(n[0], 1, 2, API_ACTION_MAXIMUM_GREEN,
API_ACTIONMODE_SET, cycle);
    qps_SIG_action(n[0], 1, 2, API_ACTION_MINIMUM_GREEN,
API_ACTIONMODE_SET, 0);
    qps_SIG_action(n[0], 2, 1, API_ACTION_MAXIMUM_GREEN,
API_ACTIONMODE_SET, cycle);
    qps_SIG_action(n[0], 2, 1, API_ACTION_MINIMUM_GREEN,
API_ACTIONMODE_SET, 0);

    qps_SIG_action(n[1], 1, 2, API_ACTION_MAXIMUM_GREEN,
API_ACTIONMODE_SET, cycle);
    qps_SIG_action(n[1], 1, 2, API_ACTION_MINIMUM_GREEN,
API_ACTIONMODE_SET, 0);
    qps_SIG_action(n[1], 2, 1, API_ACTION_MAXIMUM_GREEN,
API_ACTIONMODE_SET, cycle);
    qps_SIG_action(n[1], 2, 1, API_ACTION_MINIMUM_GREEN,
API_ACTIONMODE_SET, 0);

    qps_SIG_action(n[2], 1, 2, API_ACTION_MAXIMUM_GREEN,
API_ACTIONMODE_SET, cycle);
    qps_SIG_action(n[2], 1, 2, API_ACTION_MINIMUM_GREEN,
API_ACTIONMODE_SET, 0);

```

```
qps_SIG_action(n[2], 2, 1, API_ACTION_MAXIMUM_GREEN,  
API_ACTIONMODE_SET, cycle);  
qps_SIG_action(n[2], 2, 1, API_ACTION_MINIMUM_GREEN,  
API_ACTIONMODE_SET, 0);
```

```
qps_SIG_action(n[4], 1, 2, API_ACTION_MAXIMUM_GREEN,  
API_ACTIONMODE_SET, cycle);  
qps_SIG_action(n[4], 1, 2, API_ACTION_MINIMUM_GREEN,  
API_ACTIONMODE_SET, 0);  
qps_SIG_action(n[4], 2, 1, API_ACTION_MAXIMUM_GREEN,  
API_ACTIONMODE_SET, cycle);  
qps_SIG_action(n[4], 2, 1, API_ACTION_MINIMUM_GREEN,  
API_ACTIONMODE_SET, 0);
```

```
qps_SIG_action(n[5], 1, 2, API_ACTION_MAXIMUM_GREEN,  
API_ACTIONMODE_SET, cycle);  
qps_SIG_action(n[5], 1, 2, API_ACTION_MINIMUM_GREEN,  
API_ACTIONMODE_SET, 0);  
qps_SIG_action(n[5], 2, 1, API_ACTION_MAXIMUM_GREEN,  
API_ACTIONMODE_SET, cycle);  
qps_SIG_action(n[5], 2, 1, API_ACTION_MINIMUM_GREEN,  
API_ACTIONMODE_SET, 0);
```

```
qps_SIG_action(n[6], 1, 2, API_ACTION_MAXIMUM_GREEN,  
API_ACTIONMODE_SET, cycle);  
qps_SIG_action(n[6], 1, 2, API_ACTION_MINIMUM_GREEN,  
API_ACTIONMODE_SET, 0);  
qps_SIG_action(n[6], 2, 1, API_ACTION_MAXIMUM_GREEN,  
API_ACTIONMODE_SET, cycle);  
qps_SIG_action(n[6], 2, 1, API_ACTION_MINIMUM_GREEN,  
API_ACTIONMODE_SET, 0);
```

```
qps_SIG_action(n[7], 1, 2, API_ACTION_MAXIMUM_GREEN,  
API_ACTIONMODE_SET, cycle);  
qps_SIG_action(n[7], 1, 2, API_ACTION_MINIMUM_GREEN,  
API_ACTIONMODE_SET, 0);  
qps_SIG_action(n[7], 2, 1, API_ACTION_MAXIMUM_GREEN,  
API_ACTIONMODE_SET, cycle);  
qps_SIG_action(n[7], 2, 1, API_ACTION_MINIMUM_GREEN,  
API_ACTIONMODE_SET, 0);
```

```
qps_SIG_action(n[8], 1, 2, API_ACTION_MAXIMUM_GREEN,  
API_ACTIONMODE_SET, cycle);  
qps_SIG_action(n[8], 1, 2, API_ACTION_MINIMUM_GREEN,  
API_ACTIONMODE_SET, 0);
```

```

qps_SIG_action(n[8], 2, 1, API_ACTION_MAXIMUM_GREEN,
API_ACTIONMODE_SET, cycle);
qps_SIG_action(n[8], 2, 1, API_ACTION_MINIMUM_GREEN,
API_ACTIONMODE_SET, 0);

```

```

for (i = 0; i < 15; i++)

```

```

{
    deteccount[i] = 0;
    deteccountold[i] = 0;
    detecflow[i] = 0;
    smoothedflowold[i] = 0;
    smoothedflow[i] = 0;

```

```

    oncount[i] = 0;
    oncountold[i] = 0;
    onflow[i] = 0;
    onsmoothedflow[i] = 0;
    onsmoothedflowold[i] = 0;

```

```

    offcount[i] = 0;
    offcountold[i]=0;
    offflow[i]=0;
    offsmoothedflow[i]=0;
    offsmoothedflowold[i]=0;

```

```

    deteclink[i] = qpg_DTC_link(qpg_NET_detectorByIndex(i*2 + 62));

```

```

    density[i] = 0;

```

```

}

```

```

rate[0] = 2000;
rate[1] = 2000;
rate[2] = 2000;
rate[4] = 2000;
rate[5] = 2000;
rate[6] = 2000;
rate[7] = 2000;
rate[8] = 2000;

```

```

minrate[0] = 670;
minrate[1] = 188;

```



```

minrate[2] = 222;
minrate[4] = 301;
minrate[5] = 540;
minrate[6] = 295;
minrate[7] = 455;

}

void qpx_NET_timeStep(void)
{
    int i;
    int g;
    float meterrate;

    if ((double)qpg_CFG_simulationTime() -
        (float)floor((double)qpg_CFG_simulationTime())>0.0)
        return;

    if ((int)qpg_CFG_simulationTime() % cycle == 1)
    {
        if (rate[0] < 1900)
        {
            meterrate = rate[0] * cycle / 1900;
            qps_SIG_action(n[0], 1, 2, API_ACTION_CURRENT_GREEN,
                API_ACTIONMODE_SET, (int)meterrate);
            qps_GUI_printf("SIGNAL ACTIVATED 0 %f %f %f %i\n",
                qpg_CFG_simulationTime(), rate[0], onsmoothedflow[0], (int)meterrate);
        }
        else
        {
            meterrate = cycle;
            qps_SIG_action(n[0], 1, 2, API_ACTION_CURRENT_GREEN,
                API_ACTIONMODE_SET, (int)meterrate);
        }

        if (rate[1] < 1900)
        {
            meterrate = rate[1] * cycle / 1900;
            qps_SIG_action(n[1], 1, 2, API_ACTION_CURRENT_GREEN,
                API_ACTIONMODE_SET, (int)meterrate);
            qps_GUI_printf("SIGNAL ACTIVATED 1 %f %f %f %i\n",
                qpg_CFG_simulationTime(), rate[1], onsmoothedflow[1], (int)meterrate);
        }
        else
    }

```

```

{
    meterrate = cycle;
    qps_SIG_action(n[1], 1, 2, API_ACTION_CURRENT_GREEN,
API_ACTIONMODE_SET, (int)meterrate);
}

if (rate[2] < 1900)
{
    meterrate = rate[2] * cycle / 1900;
    qps_SIG_action(n[2], 1, 2, API_ACTION_CURRENT_GREEN,
API_ACTIONMODE_SET, (int)meterrate);
    qps_GUI_printf("SIGNAL ACTIVATED 2 %f %f %f %i\n",
qpg_CFG_simulationTime(), rate[2], onsmoothedflow[2], (int)meterrate);
}
else
{
    meterrate = cycle;
    qps_SIG_action(n[2], 1, 2, API_ACTION_CURRENT_GREEN,
API_ACTIONMODE_SET, (int)meterrate);
}

if (rate[4] < 1900)
{
    meterrate = rate[4] * cycle / 1900;
    qps_SIG_action(n[4], 1, 2, API_ACTION_CURRENT_GREEN,
API_ACTIONMODE_SET, (int)meterrate);
    qps_GUI_printf("SIGNAL ACTIVATED 4 %f %f %f %i\n",
qpg_CFG_simulationTime(), rate[4], onsmoothedflow[4], (int)meterrate);
}
else
{
    meterrate = cycle;
    qps_SIG_action(n[4], 1, 2, API_ACTION_CURRENT_GREEN,
API_ACTIONMODE_SET, (int)meterrate);
}

if (rate[5] < 1900)
{
    meterrate = rate[5] * cycle / 1900;
    qps_SIG_action(n[5], 1, 2, API_ACTION_CURRENT_GREEN,
API_ACTIONMODE_SET, (int)meterrate);
    qps_GUI_printf("SIGNAL ACTIVATED 5 %f %f %f %i\n",
qpg_CFG_simulationTime(), rate[5], onsmoothedflow[5], (int)meterrate);
}
else
{

```

```

    meterrate = cycle;
    qps_SIG_action(n[5], 1, 2, API_ACTION_CURRENT_GREEN,
API_ACTIONMODE_SET, (int)meterrate);
}

if (rate[6] < 1900)
{
    meterrate = rate[6] * cycle / 1900;
    qps_SIG_action(n[6], 1, 2, API_ACTION_CURRENT_GREEN,
API_ACTIONMODE_SET, (int)meterrate);
    qps_GUI_printf("SIGNAL ACTIVATED 6 %f %f %f %i\n",
qpg_CFG_simulationTime(), rate[6], onsmoothedflow[6], (int)meterrate);
}
else
{
    meterrate = cycle;
    qps_SIG_action(n[6], 1, 2, API_ACTION_CURRENT_GREEN,
API_ACTIONMODE_SET, (int)meterrate);
}

if (rate[7] < 1900)
{
    meterrate = rate[7] * cycle / 1900;
    qps_SIG_action(n[7], 1, 2, API_ACTION_CURRENT_GREEN,
API_ACTIONMODE_SET, (int)meterrate);
    qps_GUI_printf("SIGNAL ACTIVATED 7 %f %f %f %i\n",
qpg_CFG_simulationTime(), rate[7], onsmoothedflow[7], (int)meterrate);
}
else
{
    meterrate = cycle;
    qps_SIG_action(n[7], 1, 2, API_ACTION_CURRENT_GREEN,
API_ACTIONMODE_SET, (int)meterrate);
}

if (rate[8] < 1900)
{
    meterrate = rate[8] * cycle / 1900;
    qps_SIG_action(n[8], 1, 2, API_ACTION_CURRENT_GREEN,
API_ACTIONMODE_SET, (int)meterrate);
    qps_GUI_printf("SIGNAL ACTIVATED 8 %f %f %f %i\n",
qpg_CFG_simulationTime(), rate[8], onsmoothedflow[8], (int)meterrate);
}
else
{
    meterrate = cycle;

```

```

    qps_SIG_action(n[8], 1, 2, API_ACTION_CURRENT_GREEN,
API_ACTIONMODE_SET, (int)meterrate);
}

}

if ((int)qpg_CFG_simulationTime() % 30 == 0)
{

    for (i=0; i<15; i++)
    {
        smoothedflowold[i] = smoothedflow[i];
        deteccountold[i] = deteccount[i];
        deteccount[i] = qpg_DTI_count(i*2+62, 1, 0) + qpg_DTI_count(i*2+62, 2, 0) +
qpg_DTI_count(i*2+62, 3, 0);
        detecflow[i] = deteccount[i] - deteccountold[i];
        g = detecflow[i] * 120;
        smoothedflow[i] = (g - smoothedflowold[i])*0.15 + smoothedflowold[i];

        density[i] = 5280 * ((qpg_DTI_occupancy(i*2+62,1,APILOOP_SMOOTHED) +
qpg_DTI_occupancy(i*2+62,2,APILOOP_SMOOTHED) +
qpg_DTI_occupancy(i*2+62,3,APILOOP_SMOOTHED)) / 3 / 300 * detecflow[i]) / 14;

//  qps_GUI_printf("%f%f%f\n",
qpg_DTI_occupancy(i*2+62,1,APILOOP_SMOOTHED), density[i]);

        averagespeed[i] = (qpg_DTI_speed(i*2+62,1,APILOOP_SMOOTHED) +
qpg_DTI_speed(i*2+62,2,APILOOP_SMOOTHED) +
qpg_DTI_speed(i*2+62,3,APILOOP_SMOOTHED) )*2.2369 / 3;
    }

    for (i=0; i < 9; i++)
    {
        onsmoothedflowold[i] = onsmoothedflow[i];
        oncountold[i] = oncount[i];
        oncount[i] = qpg_DTI_count(i+135,1,0);
        onflow[i] = oncount[i] - oncountold[i];
        g = onflow[i] * 120;
        onsmoothedflow[i] = (g - onsmoothedflowold[i]) * .15 + onsmoothedflow[i];

    }

    for (i=0; i < 11; i++)

```

```

{
    offsmoothedflowold[i] = offsmoothedflow[i];
    offcountold[i] = offcount[i];
    offcount[i] = qpg_DTI_count(i+144,1,0);
    offflow[i] = offcount[i] - offcountold[i];
    g = offflow[i] * 120;
    offsmoothedflow[i] = (g - offsmoothedflowold[i]) * .15 + offsmoothedflow[i];
}

// qps_GUI_printf("%i %f %f\n", offflow[3], offsmoothedflowold[3],
offsmoothedflow[3]);

}

if ((int)qpg_CFG_simulationTime() % cycle == 0)
{

// CHANGE THIS zone1() zone2() zone3() also change CYCLE variable

    zone1();

}
}

[0] * M / (onsmoothedflow[0] + onsmoothedflow[1]);
rate[1] = M - rate[0];

if (rate[0] < minrate[0])
{
    rate[0] = minrate[0];
}
if (rate[1] < minrate[1])
{
    rate[1] = minrate[1];
}
}

void zone24()
{
    float maxdens;
    float minspeed;
    float S;

    if (max(density[5], max(density[4], density[3])) < 32)

```

```

{
    maxdens = max(density[5], max(density[4], density[3]));
    minspeed = min(averagespeed[5], min(averagespeed[4], averagespeed[3]));

    S = (32 - maxdens)*(minspeed*3);
}
else
{
    S = 0;
}

```

```

M = 1800 + 2100 + 2100 + offsmoothedflow[4] + offsmoothedflow[3] + S -
smoothedflow[3] - onsmoothedflow[3];

```

```

rate[2] = M;

```

```

if (rate[2] < minrate[2])
{
    rate[2] = minrate[2];
}
}

```

```

void zone61()
{
    float maxdens;
    float minspeed;
    float S;
    float demand;

    if(max(density[0], max(density[1], max(density[2], max(density[3], max(density[4],
density[5]))))) < 32)
    {
        maxdens = max(density[0], max(density[1], max(density[2], max(density[3],
max(density[4], density[5])))));
        minspeed = min(averagespeed[0], min(averagespeed[1], min(averagespeed[2],
min(averagespeed[3], min(averagespeed[4], averagespeed[5])))));

        S = (32 - maxdens) * (minspeed * 3);
    }
    else
    {
        S = 0;
    }
}

```

```

M = 6000 + offsmoothedflow[4] + offsmoothedflow[3] + offsmoothedflow[2] +
offsmoothedflow[1] + offsmoothedflow[0] + S - smoothedflow[0] - onsmoothedflow[3];

```

```

qps_GUI_printf("%f%f%f ", -1*smoothedflow[0], S, M);

```

```

demand = onsmoothedflow[0] + onsmoothedflow[1] + onsmoothedflow[2];

```

```

rate[0] = onsmoothedflow[0] * M / demand;
rate[1] = onsmoothedflow[1] * M / demand;
rate[2] = onsmoothedflow[2] * M / demand;

```

```

if (rate[0] < minrate[0])
    rate[0] = minrate[0];
if (rate[1] < minrate[1])
    rate[1] = minrate[1];
if (rate[2] < minrate[2])
    rate[2] = minrate[2];

```

```

qps_GUI_printf("%f%f%f\n", rate[0], rate[1], rate[2]);

```

```

}

```

```

void zone62()

```

```

{
    float maxdens;
    float minspeed;
    float S;
    float demand;

```

```

    if(max(density[10], max(density[9], max(density[8], max(density[7], max(density[6],
density[5]))))) < 32)

```

```

    {
        maxdens = max(density[10], max(density[9], max(density[8], max(density[7],
max(density[6], density[5])))));
        minspeed = min(averagespeed[10], min(averagespeed[9], min(averagespeed[8],
min(averagespeed[7], min(averagespeed[6], averagespeed[5])))));

```

```

        S = (32 - maxdens) * (minspeed * 3);
    }
    else
    {
        S = 0;
    }

```

```
M = 6000 + offsmoothedflow[5] + offsmoothedflow[6] + offsmoothedflow[7] + S -
smoothedflow[5];
```

```
qps_GUI_printf("%f%f%f ", -1*smoothedflow[5], S, M);
```

```
demand = onsmoothedflow[4] + onsmoothedflow[5] + onsmoothedflow[6] +
onsmoothedflow[7];
```

```
rate[4] = onsmoothedflow[4] * M / demand;
rate[5] = onsmoothedflow[5] * M / demand;
rate[6] = onsmoothedflow[6] * M / demand;
rate[7] = onsmoothedflow[7] * M / demand;
```

```
if (rate[4] < minrate[4])
    rate[4] = minrate[4];
if (rate[5] < minrate[5])
    rate[5] = minrate[5];
if (rate[6] < minrate[6])
    rate[6] = minrate[6];
if (rate[7] < minrate[7])
    rate[7] = minrate[7];
```

```
qps_GUI_printf("%f%f%f%f\n", rate[4], rate[5], rate[6], rate[7]);
```

```
}
```

```
void zone63()
```

```
{
    float maxdens;
    float minspeed;
    float S;
```

```
if(max(density[10], max(density[11], max(density[12], max(density[13], density[14])))
< 32)
```

```
{
    maxdens = max(density[10], max(density[11], max(density[12], max(density[13],
density[14]))));
    minspeed = min(averagespeed[10], min(averagespeed[11], min(averagespeed[12],
min(averagespeed[13], averagespeed[14]))));
```

```
    S = (32 - maxdens) * (minspeed * 3);
}
else
{
    S = 0;
```



```

}

M = 6000 + offsmoothedflow[8] + offsmoothedflow[9] + offsmoothedflow[10] + S -
smoothedflow[10];

qps_GUI_printf("%f%f%f ", -1*smoothedflow[10], S, M);

rate[8] = M;

if (rate[8] < minrate[8])
    rate[8] = minrate[8];

qps_GUI_printf("%f\n", rate[8]);
}

void largezone()
{
    float maxdens;
    float minspeed;
    float S;
    float demand;

    if(max(density[14], max(density[13], max(density[12], max(density[11],
max(density[10], max(density[9], max(density[8], max(density[7], max(density[6],
max(density[5], max(density[4], max(density[3], max(density[2], max(density[1],
density[0])))))))))))) < 32)
    {
        maxdens = max(density[0], max(density[1], max(density[2], max(density[3],
max(density[4], max(density[5], max(density[6], density[7])))))));
        maxdens = max(maxdens, max(density[13], max(density[8], max(density[9],
max(density[10], max(density[11], max(density[12], density[14])))))));

        minspeed = min(averagespeed[0], min(averagespeed[1], min(averagespeed[2],
min(averagespeed[3], min(averagespeed[4], min(averagespeed[5], min(averagespeed[6],
averagespeed[7])))))));
        minspeed = min(minspeed, min(averagespeed[13], min(averagespeed[8],
min(averagespeed[9], min(averagespeed[10], min(averagespeed[11],
min(averagespeed[12], averagespeed[14])))))));

        S = (32 - maxdens) * (minspeed * 3);
    }
}

```

```

else
{
    S = 0;
}

```

```

M = 6000 + offsmoothedflow[0] + offsmoothedflow[1] + offsmoothedflow[2] +
offsmoothedflow[3] + offsmoothedflow[4] + offsmoothedflow[5] + offsmoothedflow[6]
+ offsmoothedflow[7] + offsmoothedflow[8] + offsmoothedflow[9] +
offsmoothedflow[10] + S - smoothedflow[0];

```

```

qps_GUI_printf("%f%f%f", -1*smoothedflow[0], S, M);

```

```

demand = onsmoothedflow[0] + onsmoothedflow[1] + onsmoothedflow[2] +
onsmoothedflow[3] + onsmoothedflow[4] + onsmoothedflow[5] + onsmoothedflow[6] +
onsmoothedflow[7] + onsmoothedflow[8];

```

```

rate[0] = onsmoothedflow[0] * M / demand;
rate[1] = onsmoothedflow[1] * M / demand;
rate[2] = onsmoothedflow[2] * M / demand;
rate[3] = onsmoothedflow[3] * M / demand;
rate[4] = onsmoothedflow[4] * M / demand;
rate[5] = onsmoothedflow[5] * M / demand;
rate[6] = onsmoothedflow[6] * M / demand;
rate[7] = onsmoothedflow[7] * M / demand;
rate[8] = onsmoothedflow[8] * M / demand;

```

```

if (rate[0] < minrate[0])
    rate[0] = minrate[0];
if (rate[1] < minrate[1])
    rate[1] = minrate[1];
if (rate[2] < minrate[2])
    rate[2] = minrate[2];
if (rate[3] < minrate[3])
    rate[3] = minrate[3];
if (rate[4] < minrate[4])
    rate[4] = minrate[4];
if (rate[5] < minrate[5])
    rate[5] = minrate[5];
if (rate[6] < minrate[6])
    rate[6] = minrate[6];
if (rate[7] < minrate[7])
    rate[7] = minrate[7];
if (rate[8] < minrate[8])
    rate[8] = minrate[8];

```

```

qps_GUI_printf("%f %f %f %f %f %f %f %f %f\n", rate[0], rate[1], rate[2], rate[3] ,
rate[4], rate[5], rate[6], rate[7] , rate[8]);

}
*/

void zone1()
{
float maxdens;
float minspeed;
float S;
float demand;

if(max(density[0], max(density[1], max(density[2], max(density[3], max(density[4],
density[5]))))) < 32)
{
maxdens = max(density[0], max(density[1], max(density[2], max(density[3],
max(density[4], density[5]))));
minspeed = min(averagespeed[0], min(averagespeed[1], min(averagespeed[2],
min(averagespeed[3], min(averagespeed[4], averagespeed[5]))));

S = (32 - maxdens) * (minspeed * 3);
}
else
{
S = 0;
}

M = 6000 + offsmoothedflow[4] + offsmoothedflow[3] + offsmoothedflow[2] +
offsmoothedflow[1] + offsmoothedflow[0] + S - smoothedflow[0] - onsmoothedflow[3];

qps_GUI_printf("%f %f %f ", -1*smoothedflow[0], S, M);

demand = onsmoothedflow[0] + onsmoothedflow[1] + onsmoothedflow[2];

rate[0] = onsmoothedflow[0] * M / demand;
rate[1] = onsmoothedflow[1] * M / demand;
rate[2] = onsmoothedflow[2] * M / demand;

if (rate[0] < minrate[0])
rate[0] = minrate[0];
if (rate[1] < minrate[1])
rate[1] = minrate[1];
if (rate[2] < minrate[2])
rate[2] = minrate[2];

```

```

qps_GUI_printf("%f%f%f\n", rate[0], rate[1], rate[2]);

}

void zone2()
{
    float maxdens;
    float minspeed;
    float S;
    float demand;

    if(max(density[4], max(density[5], max(density[6], max(density[7], max(density[8],
max(density[9], density[10])))))) < 32)
    {
        maxdens = max(density[4], max(density[5], max(density[6], max(density[7],
max(density[8], max(density[9], density[10])))))));
        minspeed = min(averagespeed[4], min(averagespeed[5], min(averagespeed[6],
min(averagespeed[7], min(averagespeed[8], min(averagespeed[9],
averagespeed[10])))))));

        S = (32 - maxdens) * (minspeed * 3);
    }
    else
    {
        S = 0;
    }

    M = 6000 + offsmoothedflow[4] + offsmoothedflow[5] + offsmoothedflow[6] +
offsmoothedflow[7] + S - smoothedflow[4];

    qps_GUI_printf("%f%f%f ", -1*smoothedflow[0], S, M);

    demand = onsmoothedflow[4] + onsmoothedflow[5] + onsmoothedflow[6] +
onsmoothedflow[7];

    rate[4] = onsmoothedflow[4] * M / demand;
    rate[5] = onsmoothedflow[5] * M / demand;
    rate[6] = onsmoothedflow[6] * M / demand;
    rate[7] = onsmoothedflow[7] * M / demand;

    if (rate[4] < minrate[4])
        rate[4] = minrate[4];
    if (rate[5] < minrate[5])
        rate[5] = minrate[5];
    if (rate[6] < minrate[6])
        rate[6] = minrate[6];

```

```

if (rate[7] < minrate[7])
    rate[7] = minrate[7];

qps_GUI_printf("%f%f%f%f\n", rate[4], rate[5], rate[6], rate[7]);
}

void zone3()
{
    float maxdens;
    float minspeed;
    float S;
    float demand;

    if(max(density[10], max(density[9], max(density[8], max(density[7], max(density[6],
    max(density[5], max(density[4], max(density[3], max(density[2], max(density[1],
    density[0])))))))) < 32)
    {
        maxdens = max(density[0], max(density[1], max(density[2], max(density[3],
        max(density[4], max(density[5], max(density[6], density[7])))))));
        maxdens = max(maxdens, max(density[8], max(density[9], density[10])));

        minspeed = min(averagespeed[0], min(averagespeed[1], min(averagespeed[2],
        min(averagespeed[3], min(averagespeed[4], min(averagespeed[5], min(averagespeed[6],
        averagespeed[7])))))));
        minspeed = min(minspeed, min(averagespeed[8], min(averagespeed[9],
        averagespeed[10])));

        S = (32 - maxdens) * (minspeed * 3);
    }
    else
    {
        S = 0;
    }

    M = 6000 + offsmoothedflow[0] + offsmoothedflow[1] + offsmoothedflow[2] +
    offsmoothedflow[3] + offsmoothedflow[4] + offsmoothedflow[5] + offsmoothedflow[6]
    + offsmoothedflow[7] + S - smoothedflow[0];

    qps_GUI_printf("%f%f%f", -1*smoothedflow[0], S, M);

    demand = onsmoothedflow[0] + onsmoothedflow[1] + onsmoothedflow[2] +
    onsmoothedflow[3] + onsmoothedflow[4] + onsmoothedflow[5] + onsmoothedflow[6] +
    onsmoothedflow[7];

    rate[0] = onsmoothedflow[0] * M / demand;

```

```
rate[1] = onsmoothedflow[1] * M / demand;  
rate[2] = onsmoothedflow[2] * M / demand;  
rate[3] = onsmoothedflow[3] * M / demand;  
rate[4] = onsmoothedflow[4] * M / demand;  
rate[5] = onsmoothedflow[5] * M / demand;  
rate[6] = onsmoothedflow[6] * M / demand;  
rate[7] = onsmoothedflow[7] * M / demand;
```

```
if (rate[0] < minrate[0])  
    rate[0] = minrate[0];  
if (rate[1] < minrate[1])  
    rate[1] = minrate[1];  
if (rate[2] < minrate[2])  
    rate[2] = minrate[2];  
if (rate[3] < minrate[3])  
    rate[3] = minrate[3];  
if (rate[4] < minrate[4])  
    rate[4] = minrate[4];  
if (rate[5] < minrate[5])  
    rate[5] = minrate[5];  
if (rate[6] < minrate[6])  
    rate[6] = minrate[6];  
if (rate[7] < minrate[7])  
    rate[7] = minrate[7];
```

```
qps_GUI_printf("%f%f%f%f%f%f%f%f\n", rate[0], rate[1], rate[2], rate[3] ,  
rate[4], rate[5], rate[6], rate[7]);  
}
```

Appendix II-G

PARAMICS API CODE Speed Zone

VSL Homogeneous Speed Zones API Code

```
#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_SpeedZone[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);

/* -----
* 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();

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]));

} */

}

/* -----
* Called once for every time step (actually every second) - The main body of our plugin.
* ----- */
void qpx_NET_second(void)
{

int i;
int x;
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* loopplane1;
LOOP* loopplane2;
LOOP* loopplane3;
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 */

linkFspeed1[station][timeincrement] = g_Speeds1[i]*2.2369;
linkFspeed2[station][timeincrement] = g_Speeds2[i]*2.2369;
linkFspeed3[station][timeincrement] = g_Speeds3[i]*2.2369;

/*      qps_GUI_printf("%f ", linkFspeed1[station][timeincrement]);
qps_GUI_printf("%f ", linkFspeed2[station][timeincrement]);
qps_GUI_printf("%f ", linkFspeed3[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 (linkFspeed1[station][t] != linkFspeed1[station][t+1])
{
numspeed1++;
speedtotal1 += linkFspeed1[station][t+1];
}

if (linkFspeed2[station][t] != linkFspeed2[station][t+1])
{
numspeed2++;
speedtotal2 += linkFspeed2[station][t+1];
}

if (linkFspeed3[station][t] != linkFspeed3[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;

//      qps_GUI_printf("\n%f      Avg.      Speed      at      Detector      %i\n",
linkF5minavgspeed[station], station);

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

downstreamspeed[station] = linkF5minavgspeed[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("%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("\nSpeedZone %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

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

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;
}

//      qps_GUI_printf("speedchange_distance    %f    for    Speed    Zone    %i\n",
Speedchange_distance[r], r);

```

```

//      VSL Implementation

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);
qps_LNK_speedlimit(links[71], speed);
qps_LNK_speedlimit(links[72], speed);
break;
case 67:
qps_LNK_speedlimit(links[73], speed);
qps_LNK_speedlimit(links[74], speed);
break;
case 69:
qps_LNK_speedlimit(links[75], speed);
qps_LNK_speedlimit(links[76], speed);
qps_LNK_speedlimit(links[77], speed);
qps_LNK_speedlimit(links[78], speed);
break;
case 71:
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);
break;
case 73:
qps_LNK_speedlimit(links[84], speed);
qps_LNK_speedlimit(links[85], speed);

```

```

qps_LNK_speedlimit(links[86], speed);
break;
case 75:
qps_LNK_speedlimit(links[87], speed);
qps_LNK_speedlimit(links[88], speed);
qps_LNK_speedlimit(links[89], speed);
qps_LNK_speedlimit(links[90], speed);
break;
case 77:
qps_LNK_speedlimit(links[91], speed);
qps_LNK_speedlimit(links[92], speed);
qps_LNK_speedlimit(links[93], speed);
break;
case 79:
qps_LNK_speedlimit(links[94], speed);
qps_LNK_speedlimit(links[95], speed);
qps_LNK_speedlimit(links[96], speed);
break;
case 81:
qps_LNK_speedlimit(links[97], speed);
qps_LNK_speedlimit(links[98], speed);
qps_LNK_speedlimit(links[99], speed);
break;
case 83:
qps_LNK_speedlimit(links[100], speed);
qps_LNK_speedlimit(links[101], speed);
break;
case 85:
qps_LNK_speedlimit(links[102], speed);
qps_LNK_speedlimit(links[103], speed);
break;
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:
qps_LNK_speedlimit(links[131], speed);
qps_LNK_speedlimit(links[132], speed);
qps_LNK_speedlimit(links[133], speed);
qps_LNK_speedlimit(links[134], speed);
break;

```

```

case 117:
qps_LNK_speedlimit(links[135], speed);
qps_LNK_speedlimit(links[136], speed);
break;
case 119:
qps_LNK_speedlimit(links[137], speed);
qps_LNK_speedlimit(links[138], speed);
break;
case 121:
qps_LNK_speedlimit(links[139], speed);
break;
case 123:
qps_LNK_speedlimit(links[140], speed);
break;
case 125:
qps_LNK_speedlimit(links[141], speed);
break;
case 127:
qps_LNK_speedlimit(links[142], speed);
qps_LNK_speedlimit(links[143], speed);
break;
case 129:
qps_LNK_speedlimit(links[144], speed);
break;
case 131:
qps_LNK_speedlimit(links[145], speed);
break;
case 133:
qps_LNK_speedlimit(links[146], speed);
qps_LNK_speedlimit(links[147], speed);
qps_LNK_speedlimit(links[148], speed);
qps_LNK_speedlimit(links[149], speed);
qps_LNK_speedlimit(links[150], speed);
break;

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

}

```