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Evaluating Ramp Metering and Variable Speed Limits to Reduce Crash Potential on Congested Freeways using Micro-Simulation

by

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

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ABSTRACT

Recent research at UCF into defining surrogate measures for identifying crash prone conditions on freeways has led to the introduction of several statistical models which can flag such conditions with a good degree of accuracy. Outputs from these models have the potential to be used as real-time safety measures on freeways. They may also act as the basis for the evaluation of several intervention strategies that might help in the mitigation of risk of crashes.

Ramp Metering and Variable Speed Limits are two approaches which have the potential of becoming effective implementation strategies for improving the safety conditions on congested freeways. This research evaluates both these strategies in different configurations and attempts to quantify their effect on risk of crash on a 9-mile section of Interstate-4 in the Orlando metropolitan region. The section consists of 17 Loop Detector stations, 11 On-ramps and 10 off-ramps. PARAMICS micro-simulation is used as the tool for modeling the freeway section. The simulated network is calibrated and validated for 5 minute average flows and speeds using loop detector data. Feedback Ramp Metering algorithm, ALINEA, is used for controlling access from up to 7 on-ramps. Variable Speed Limits are implemented based on real-time speed conditions prevailing in the whole 9-mile section. Both these strategies are tested separately as well as collectively to determine the individual effects of all the parameters involved. The results have been used to formulate and recommend the best possible strategy for minimizing the risk of crashes on the corridor.

The study concluded that Ramp Metering improves the conditions on the freeway in terms of safety by decreasing variance in speeds and decreasing average occupancy. A
safety benefit index was developed for quantifying the reduction in crash risk and it indicated that an optimal implementation strategy might produce benefits of up to 55%. The condition on the freeway section improved with increase in the number of metered ramps. It was also observed that shorter signal cycles for metered ramps were more suitable for metering multiple ramps. Ramp Metering at multiple locations also decreased the segment wide travel-times by 5% and was even able to offset the delays incurred by drivers at the metered on-ramps.

Variable Speed Limits (VSL) were individually not as effective as ramp metering but when implemented along with ramp metering, they were found to further improve the safety on the freeway section under consideration. By means of a detailed experimental design it was observed that the best strategy for introducing speed limit changes was to raise the speed limits downstream of the location of interest by 5 mph and not affecting the speed limits upstream. A coordinated strategy - involving simultaneous application of VSL and Ramp Metering - provided safety benefits of up to 56% for the study section according to the safety benefit index. It also improved the average speeds on the network besides decreasing the overall network travel time by as much as 21%.
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1  CHAPTER 1- INTRODUCTION

Automobile safety has been an issue ever since automobiles were invented. The first human fatality in an automobile crash occurred in 1896. Ever since then, the dependence on the automobile has increased and so has the exposure to these incidents. Since 1967 National Transportation Safety Board (NTSB) has been actively legislating changes that make vehicles and their operations safer for the general public. However the number of crashes taking place has been increasing. According to National Center for Statistics and Analysis (NCSA), there were 43,005 fatal crashes on American Roads in 2002.

Although efforts have been made to make the vehicles safer for passengers and pedestrians, still there is a need to provide systems that are responsive to situations which might eventually lead to a crash. Research has been on for sometime for defining measures to quantify risk of a crash and develop measures to predict the occurrence of crashes or crash prone characteristics in the traffic stream. Research at UCF (Abdel-Aty et al., 2004; 2005) has focused on these issues and surrogate statistical measures have been developed that would be able to flag crash prone conditions and thereby allow for intervention. This study is related to 2 such possible measures which can be implemented because of the advancement in integrated and intelligent traffic systems. Ramp Metering and Variable Speed Limits have been in the domain of use of traffic engineers for many years and they have often been found useful in increasing throughput and preventing recurring and non recurring congestion. This study looks at these factors from an implementation for safety point of view. Lee et al. (2004) and Dilmore (2005) have already contributed to this work and had successfully shown the utility of Ramp Metering and Variable Speed Limits separately in improving safety on different freeway sections.
This study significantly tries to enhance the work in two directions. It considers the application of both these strategies over a whole simulated road network of Interstate-4 and unlike previous studies the ITS applications are implemented over multiple locations and their effect observed over the whole network. The study also explores the possibility of coordination of these strategies with one another in an attempt to maximize their utility in mitigating the risk of crashes. It is the premise of the author that a strategy that combines all two of the above mentioned 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 technologies.

The study thus has three major objectives which are summarized below:

- 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.
2 CHAPTER 2- LITERATURE REVIEW

2.1 Crash Prediction

Madanat and Liu (1995) came up with an incident likelihood prediction model using loop data as input. The focus of their research was to enhance existing incident detection algorithms with likelihood of incidents. A Binary logit methodology was used for analysis. They concluded that merging section, visibility and rain are statistically the most significant factors for crash likelihood prediction.

Lee et al. (2002) introduced the concept of “crash precursors” and hypothesized that the likelihood of crash occurrence is significantly affected by short-term turbulence of traffic flow. They came up with factors such as speed variation along the length of the roadway (i.e. the difference between the speeds upstream and downstream of the crash location) and also across the three lanes at the crash location. A crash prediction model was developed using log-linear analysis. In a later study (Lee et al., 2003), they continued their work along the same lines and modified this model. They concluded that variation of speed has relatively longer term effect on crash potential rather than density and average speed difference between upstream and downstream ends of roadway sections. In one of their most recent related studies Lee et al. (2004) proposed the application of these models and estimated real-time crash potential. The main focus of this study was to reduce the crash potential obtained from the model through different control strategies of variable speed limits (VSL). PARAMICS Micro-simulation was used to evaluate the effect of Ramp metering on the model at a single location (Lee et al. 2005).
Oh et al. (2001) used the Bayesian classifier to categorize the two possible traffic flow conditions. A more detailed analysis of patterns in crash characteristics as a function of real-time traffic flow was done by Golob and Recker (2001, 2004). The methodology used was non-linear (nonparametric) canonical correlation analysis (NLCCA) with three sets of variables. Golob et al. (2004, a) also showed that certain traffic flow regimes are more conducive to traffic crashes than the others. Of the eight traffic flow regimes found to exist on the six freeways in Orange County (California), the study found that nearly 76% of all crashes occurred in the four traffic regimes that existed in conditions when flow was nearing congestion or already congested. This link between traffic congestion and freeway crashes was also noted by Zhang et al. (2005) in a study that explored the relationship between crashes, weather conditions, and traffic congestion. Park and Ritchie (2004) showed that the lane-changing behavior and presence of long vehicles with-in a freeway section has significant impact on section speed variability.

Various modeling methodologies have been explored e.g., Probabilistic neural network (PNN) (Abdel-Aty and Pande, 2005), matched case-control Logistic Regression (Abdel-Aty et al. 2004), multi-layer perceptron (MLP)/radial basis function (RBF) neural network architectures (Pande, 2003) and Generalized Estimation Equation (Abdel-Aty and Abdalla, 2004). The data for these studies were collected from 13.2-mile central corridor of Interstate-4 in Orlando. Abdel-Aty and Pande (2005) used Probabilistic Neural Networks (PNN) as the classification algorithm and demonstrated the feasibility of predicting crashes at least 10-minutes prior to their occurrence.

Findings from the aforementioned studies pointed towards potential application of real-time traffic data in the field of traffic safety. Abdel-Aty et al.(2004) argued that the
accuracy of real-time crash prediction model may be increased if the model utilizes information on traffic flow characteristics for both crash and non-crash cases while controlling for other external factors (thereby implicitly accounting for factors such as the geometry and location). It was proposed that this can be achieved using a within-stratum analysis of a binary outcome variable $Y$ (crash or non-crash) as a function of traffic flow variables $X_1, X_2, \ldots, X_k$ from matched crash-non-crash cases where a matched set (stratum) can be formed using crash site, time, season, day of the week, etc., so that the variability due to these factors is controlled. In epidemiological studies, this is known as matched case-control analysis while each case refers here to a crash and control to a non-crash. This model is used as indicator of risk of crash and a detailed description is included in Section 3.3.

### 2.2 Micro-simulation for Testing Alternatives

Recent advances in the field of computing have landed a very significant tool in the hands of Traffic Engineers, that of simulation. A well defined simulation model provides planners with the tool for testing strategies that might be too expensive or difficult to implement in the real life scenarios. Dahlgreen (2001) talks about the decision making process and the role that simulation can play in it. One of the important observations made in the paper is that, simulation should be oriented towards application- i.e. should be carried out when there is a chance of the use of results of simulation in the decision making process. The benefit of Simulation can only be derived effectively if there is an exhaustive calibration and validation process carried out before hand that makes the simulation accurate and close to the actual conditions on the road.
Research into the development of simulations has resulted in dramatic advances in the improvement of reliability of simulations. Micro-simulations now come very close in modeling inter-vehicle interactions. With the large amount of traffic data available, it is possible to calibrate the microscopic simulations and validate their results for any application.

Many studies now rely on Simulation to test and prove new theories and enhance the validity of the results. Chu et al. (2004) evaluate the potential benefits of ITS strategies like Local and Coordinated Ramp Metering under incident scenarios. A section of corridor in Irvine, California is simulated in PARAMICS. ITS strategies in their study include incident management, local adaptive ramp metering, coordinated ramp metering, traveler information systems, and their combinations. Based on a calibrated model developed and described in Chu et al. (2003a, 2003b), these different scenarios are tested in the microscopic simulation and five measures of effectiveness (MOE’s) are compared for current scenarios and test scenarios to demonstrate the use of Micro-simulation in evaluating the different scenarios.

In another study, Chu et al. (2003c), a similar approach is used on the same network to evaluate different adaptive ramp metering strategies. Hasan et al. (2001) conducted a similar study on evaluating ramp metering using MITSIM as their simulation tool.

Ben-akiva et al. (2001) use MITSIMLab to evaluate Freeway control. In this work, different freeway control strategies are evaluated. A study is undertaken on the pros and cons of different strategies and recommendations made based on the simulated test scenarios.
2.2.1 Micro-simulation for safety

Besides serving as test-beds for evaluation strategies of ITS application, Micro-simulation has also been used in assessment of safety impacts and traffic patterns as a result of these applications of ITS strategies. Paio et al. (2004) used Micro-simulation in their study towards showing the overall effects of deployment of Intelligent Speed Adaptations (ISA). They show that ISA improves the safety on the freeway by decreasing speeds and also decrease variations in speed. Their assessment is based on simulation of these scenarios in AIMSUN.

Lee et al. (2004; 2005) used PARAMICS to assess the safety benefits of Variable Speed Limits and Ramp Metering. Both the studies used the same segment of freeway and simulated a small section of I-880. The crash prediction model used was also developed by the authors in Lee et al. (2003) and all these studies are part of PhD thesis of Lee (2004). The results of the study show that there is an improvement in the safety of the freeway section when Variable Speed Limits or Ramp Metering is used. However, the testing was done over a very small simulated stretch of Hayward I-880 in California. The study used a 2.5 km long stretch which included 1 onramp, 4 detectors and 3 Variable Message Signs. Also, the study stops at treating Variable Speeds and ramp metering separately.

Dilmore (2005) uses a longer stretch of freeway from I-4 in Orlando to test the effects of Variable Speed Limits and come up with strategies that lead to a maximum drop in the risk of a crash at any location. The study showed that Variable Speed Limits
could have a positive effect on the safety of freeways and laid out some rules as to the implementation of Variable Speeds in order to optimize the benefits.

These studies however treated single locations for measurement of safety and did not look at system wide benefits from implementation of either of the strategies. Abdel-Aty et al. (2006) proved that the treatment of single locations in the study by Dilmore (2005) leads to occurrence of Crash Migration in space.

2.3 Variable Speed Limits

Hoogen et al. (1994) published results of an experiment performed with the application of variable speed control on motorways in the Netherlands. They found that the differences in volume, speed, and occupancy between and within lanes became smaller and variations also decreased when variable speed control was implemented. Their primary purpose was to attempt at decreasing the differences between existing average speeds and the speed limits posted. In a follow-up to this study, Smulders et al. (1998) presented the results from the first stage of the plan by the Ministry of Transport, Public works and Water Management, Netherlands, to implement Speed Control. The investigation stage provided the results from a discussion by experts. When asked to suggest Speed Controls for Safety Benefits and increasing throughputs- one group suggesting implementation for safety- suggested implementation of speed limits closer to desired speeds whereas the second group suggested implementation for throughput- and this group suggested lower speeds.

Rämä (1999) investigated the effects of weather-controlled speed limits and signs on driver behavior on a 14-km-long highway in Finland. She observed that when the variable speed limits are enforced, mean speed and standard deviation of speed were
reduced due to a reduction in the highest speeds. Ha et al. (2003) also found similar results from their field experiment of Automated Speed Enforcement (ASE). They observed that speed, speed variance, and the percentage of short time headways were significantly reduced a few kilometers ahead of an ASE station where warning signs are posted. They also concluded that smaller speed variation eventually resulted in the reduction of accident frequency and fatality based on historical accident records. Sailer et al. (1999) proved theoretically and empirically that free-flow speeds decrease with speed limits. Their model provides the guideline of predicting the impact of variable speed limits on the distribution of traffic speeds. Hegyi et al. (2002) enhanced the performance of variable speed limits through the coordination with ramp metering. Although their assumptions associated with driver compliance with variable speed limits were simplified, they demonstrated potential benefits of variable speed limits in minimizing total travel time.

Park and Yadlapati (2003) evaluated a number of variable speed limit control logics at work zones using the VISSIM Microscopic simulation model. They used the minimum safety distance equation as a surrogate measure of safety. With varying driver’s compliance rates in the simulation, they found that variable speed limits could be beneficial in improving both mobility and safety at work zones. Borrough (1997) reported that enforcement of Variable Speed Limits in England led to decrease in number of crashes. Speed limits were adjusted in response to the level of congestion on the M25, one of the most congested freeways in England. Using variable message signs (VMS) and loop detectors measuring traffic density and speed, speed limits were lowered in increments as congestion increased. Speed cameras were used to enforce the speed limits,
which resulted in more than 26,000 fines. The study found that motorists were more inclined to keep to their lane 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, resulting in smoother traffic flow which actually increased average travel times of traffic. Results show that traffic crashes decreased by 28% during the 18 months of operation.

Zhichai et al. (2004) tested different VSL scenarios aimed at relieving demand driven congestion (due to the increase of traffic volume) and supply driven congestion (due to the road’s geometric condition or traffic accident). A computer simulation was carried out including the causes of congestion and several factors that lead to the instability of freeway traffic flow such as small time headway, large speed variance, and frequent disturbances. They simulated scenarios of variable speed limits systems that can control the freeway traffic flow. The simulation results indicated that the VSL benefits are obvious when the traffic volume is equal to or greater than 2800 veh/h while resolving demand driven congestion in one direction on a freeway with two lanes. The benefits included- increase of the served traffic volume, travel time savings and reduction of speed deviation. While resolving supply driven congestion, the results indicated that VSL can- reduce the queue time, reduce number of stops, avoid congestion and save travel time, when the traffic volume is equal to or greater than 2000 veh/h for the same freeway configuration. Their findings also showed that VSL delays the occurrence of congestion at higher volumes but can’t prevent it from happening altogether.

All these studies primarily targeted congestion and travel time as their final objectives and did not consider safety as a primary objective. Lee et al. (2004) dealt with
the safety benefits of using Variable Speed Limits by using Variable Message Signs. The study found that variable speed limits can reduce average total crash potential 25% by temporarily reducing speed limits during risky traffic conditions. However, the study was not performed with real data. This meant that the research could not reflect flexible scenarios and thus could not be used in optimization scenarios.

Dilmore (2005) starts off from where this study ends by using Real traffic data from I-4 in Central Florida and tests for the effects of Variable Speed Limits at locations with higher speeds. The study showed that the best effects of VSL on safety can be obtained by inducing high drops in speed limits upstream of the risk prone location and even increasing the speed limits downstream of this location. However, the testing in this study- although done with real data - considered only single stations and there is no network wide coordinated strategy tested or provided as an end result.

### 2.4 Ramp Metering

Ramp metering is one of the most frequently used methods of freeway control intended to reduce congestion. Ramp meters are special traffic signals on a freeway on-ramp that allow one vehicle or a platoon of vehicles to enter the freeway. The first use of ramp control was on the Eisenhower Expressway (I-290) in Chicago, Illinois, in 1963, where a police officer directed the traffic to allow one vehicle to enter at a time, at a predetermined rate. Today, ramp metering has evolved and expanded, and is used throughout the US, with notable applications in Minnesota, California, New York, and Washington state. Ramp metering is also becoming popular in Europe, with applications including Amsterdam, Paris, and Glasgow.
Ramp Metering evolved early with fixed signal metering which used only pre-timed signals to allow vehicles at constant time intervals onto the freeway. However, a number of Ramp Metering Strategies have evolved which make use of real time traffic conditions on the freeway to determine the signal timing and so forth.

The Ramp metering controllers have evolved into local or isolated meters, and area-wide or coordinated. Local ramp meters take into effect only the traffic conditions at a single ramp, whereas coordinated ramp meters take into account traffic conditions over an area. Chu et al. (2004) studied the effects of different ramp metering algorithms on recurrent congestion using Micro-simulation PARAMICS. The algorithms evaluated by them included ALINEA, Bottleneck, etc. Ben-akiva et al. (2001) evaluated FLOW and ALINEA using MITSIMLab

2.4.1 Ramp Metering Algorithms

Masher et al. (1975) and Koble et al. (1980) proposed two ramp metering algorithms, Demand Capacity algorithm and Percent Occupancy algorithm, respectively. The former (Masher et al., 1975) works by measuring the occupancy ($O_o$) downstream from the location where on-ramp meets the freeway. If it exceeds the critical occupancy ($O_c$), congestion is assumed to exist, and the metering rate is set to the minimum rate ($R_{min}$). The critical occupancy is the maximum occupancy that is desired on the freeway section. Occupancy values higher than critical occupancy lead to highly congested traffic conditions. If the value of occupancy at downstream location ($O_o$) does not exceed the critical occupancy, the volume is measured upstream of the merge ($q_{in}$), and the metering rate is set to the difference between the downstream capacity ($q_{cap}$) and the upstream volume. Equation 2-1 provides the logic for the algorithm provided by Masher et al.
The metering rate $R$ can be defined as the measure of the number of vehicles that have to be allowed to enter the freeway within one cycle (time interval) on the meter. This is used as the measure for evaluating the measure of green time for every signal cycle.

**Equation 2-1**

\[
O_o \leq O_c \\
R = \max(q_{cup} - q_{in}, R_{\min}) \\
O_o > O_c \\
R = R_{\min}
\]

The latter (Koble et al., 1980) uses occupancy measurements from only upstream of an on-ramp to identify and measure congestion. The critical occupancy is measured using historical data (Hadj-Salem et al., 1988). This algorithm involves 2 constants: $K_1$ is the capacity flow, and $K_2$ is a constant based on slope of a straight line approximation of the un-congested part of the fundamental diagram. Equation 2-2 provides the logic for the algorithm provided by Koble et al. (1980). $O_{in}$ is the occupancy upstream from the location of on ramp merge on the mainline. The variable $k$ shows the time interval for calculation of metering or upstream occupancy, $O_{in}$.

**Equation 2-2**

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

The feed-forward nature of these algorithms meant that each output of metering rate was independent of the previous calculation. This was overcome by use of Feed-back Algorithms like ALINEA proposed by Papageorgiou et al. (1991). ALINEA is based on the linear quadratic (LQ) feedback law.

The equation used for calculation of metering rate at every time step $t$ is provided in Equation 2-3.
The metering rate is also a function of the difference between the measured occupancy \( (O[t]) \) at time \( t \) and a target set critical occupancy \( (O_c) \). \( K_R \) is a regulator parameter. \( \delta t \) is the length of interval at which each reevaluation of the algorithm is done. In practical scenarios this would be the signal cycle of the ramp meter. ALINEA is one of the most commonly used and one of the most effective algorithms, and will be described in greater detail in Section 5.2.

Recently, there has been research to enhance the ALINEA algorithm. Oh and Sisiopiku (2001) proposed a modified version, known as MALINEA. MALINEA addresses two main disadvantages to ALINEA. The first is that although ALINEA optimizes the occupancy downstream of the entrance ramp, congestion can still occur upstream of the ramp. The second is that the optimal detector location can be difficult to determine. MALINEA measures the upstream occupancy and accepts as parameters a regulator parameter, the slope of the curve relating the downstream and upstream occupancies, and the time lag between the upstream and downstream measurements.

Smaragdis and Papageorgiou (2003) expanded the applications of ALINEA-based algorithms. The traditional ALINEA algorithm requires occupancy measurements on downstream detectors, which unfortunately are not always available. FL-ALINEA is an algorithm that uses flow measurements from downstream detectors, rather than occupancy measurements. Its formula is identical to the formula used for traditional occupancy-based ALINEA, except that it measures flow, and tries to reach a set point flow rather than set point occupancy. However, when the occupancy is over the critical
occupancy, the metering rate is set to the minimum rate, since the freeway is already over capacity.

UP-ALINEA uses occupancy measurements, but from upstream detectors, and estimates the downstream occupancy. This is useful in cases where a feed-forward algorithm (such as demand-capacity or percent-occupancy) was previously used.

UF-ALINEA is a variation based on upstream flow measurements. This algorithm simply uses the sum of the upstream flow and the ramp flow to estimate the downstream flow. X-ALINEA/Q is referred to wherever any of the modified ALINEA algorithms are used with queue control. All of these algorithms, except for X-ALINEA/Q are less efficient than the traditional ALINEA algorithm, but are useful when downstream occupancy measurements are not available.

Bogenberger et al. (2001) proposed a nonlinear approach for designing traffic responsive and coordinated ramp control using a self adapting fuzzy system and evaluated it using FREQ model. They found the results to be satisfying and an implementation of this approach is under way.

2.4.2 Field Studies

Several recent field evaluations of ramp metering centered on the Minneapolis-St. Paul metro area, in Minnesota. MnDOT currently uses a coordinated ramp metering algorithm that divides the freeway into zones. Cambridge Systematics (2001) estimates that ramp metering saves the motoring public in the Twin Cities, Minnesota $40 Million annually, increasing mainline mean freeway speeds from 46 mph to 53 mph, and significantly reducing accidents.
Unfortunately, despite the fact that ramp metering can have great benefits, the public often opposes the use of ramp metering. For this reason, MnDOT needed evidence that ramp metering is beneficial. Two highways were selected for evaluation: Trunk Highway 169, a circumferential highway; and I-394, a downtown highway. Traffic was studied in March, 2000. The ramp meters were shown to reduce total travel time between 6% and 16%, with speeds increasing between 13% and 26%. Because of the improved traffic flow, traffic stops on ramps were cut to one third when ramp metering was implemented. Ramp metering was also shown to reduce both fuel consumption and pollutant emissions by between 2% and 47% (Hourdakis and Michalopoulos, 2002). However, subsequent research has shown there might be scenarios where ramp metering might inclusion emissions.

Ramp metering field evaluations have also been done in Europe. ALINEA and several other local ramp metering algorithms were tested on the Boulevard Peripherique in Paris, France, as well as the A10 Motorway in Amsterdam, Netherlands (Papageorgiou et al., 1997). These tests showed ALINEA to be the superior local ramp metering algorithm. ALINEA was also compared to METALINE, a coordinated algorithm. ALINEA and METALINE showed similar results for recurrent congestions, although METALINE is by far more difficult to set up and calibrate (Papageorgiou et al., 1997).

2.4.3 Minnesota Shut-off Experiment

The Minnesota state legislature passed a bill requiring ramp meters to be shut off for evaluation for eight weeks in the fall of 2000. Prior to this experimental shut off the MnDOT used a coordinated zone algorithm for Ramp Metering. This experiment showed that shutting off the ramp meters increased congestion and increased accidents, and
changed travel patterns. However, on certain test sites, the ramp meters were also shown to significantly increase travel time for short trips, despite improving travel time for longer trips. Because of this, MnDOT decided to focus on equity, rather than simply improving mainline efficiency (Levinson et al., 2002). The executive summary for the report (Minnesota DOT, 2001) however points out the annual benefits of ramp metering. Significant amongst them is the fact that in the absence of ramp metering, the peak period crashes on the main line and ramps increased by 26 percent.

### 2.4.4 Simulation Testing

Although field testing can be a useful method of evaluating ramp metering algorithms, it has many limitations. Field testing can be expensive, difficult, and time consuming. The impossibility of changing detector locations in real time can limit flexibility. Also, it is difficult to isolate the effect of ramp metering from other uncontrollable factors, such as weather, incidents, construction, or changes in traffic patterns. For these reasons, traffic simulation has become a valuable tool used as an alternative to field evaluation.

A macroscopic traffic simulator known as METANET was used to study the Boulevard Peripherique, in Paris, France. The test was to compare ALINEA and METALINE. The results showed that the two algorithms performed similarly for recurring congestion, although METALINE performed slightly better for non-recurring congestion (Papageorgiou et al, 1991). Kwon et al. (2001) used a macroscopic simulator at the University of Minnesota in order to compare a coordinated algorithm from each of the three classes: the incremental algorithm used in Colorado; the zone algorithm used in Minnesota, and the fuzzy logic algorithm used in Seattle, Washington. Because the
Minnesota algorithm did not use queue control, it resulted in the most restrictive metering rates, the lowest amount of mainline congestion, but the longest ramp queues. In contrast, the Denver algorithm and the Seattle fuzzy logic algorithms both showed that queue control can reduce the mainline efficiency. Furthermore, this test showed that the fuzzy logic algorithm is very sensitive to the weights used for each rule.

The PATH program at the University of California (Zhang et al., 2001) used PARAMICS to compare four algorithms: ALINEA, Bottleneck, Zone, and SWARM. The tests showed that all of the algorithms tested improve traffic flow. Also, there was very little difference in the performance of each algorithm. Hasan (1999) and Ben-akiva(2001) used MITSIMLab to study ramp metering on the Central Artery / Tunnel (Big Dig) network, and compared the local strategy ALINEA with the coordinated strategy FLOW. The results showed that ramp metering deteriorated system performance at low demands, and that coordination was only effective at very high demand levels. However, ramp metering almost always improved the mainline traffic flow. He also showed that queue control always improved system performance, and that coordination significantly improved performance when a bottleneck existed downstream of the on-ramp.

2.5 Scope of Study

The discussion of literature above shows that evaluation of ramp metering algorithms and variable/dynamic speed limits have been attempted using Micro-simulation. Studies like Chu et al. (2003, 2004), Hasan et al. (2001), Lee et al. (2004), Smaragdis et al. (2002), etc. have shown that this approach of testing ITS strategies using Micro-simulation has a number of advantages and many significant results can be obtained from such an approach. In light of the fact that such an approach for testing ITS
strategies holds useful research integrity and reliability, it seems appropriate that the objectives of testing the effects of Ramp Metering and Variable Speed Limits on safety of freeways could be satisfied through the use of an appropriate simulation model.

United States Department of Transportation published a report on Surrogate Safety Measures from Traffic Simulation Models (FHWA, 2003). The report compares 8 different simulation models for compatibility and concluded that PARAMICS, VISSIM and AIMSUN outperform others (viz. CORSIM, WATSIM, HUTSIM, TEXAS, INTEGRATION) in terms of capabilities, fidelity and reliability when dealing with surrogate safety measures. This study, being exactly the type of research mentioned in the report, is necessary to consider the pros and cons provided in the study as part of the simulation model choice. The main factors where the three better labeled models outdo others are listed below.

- Ability to obtain detailed state variable information on each vehicle on time scales with better than second-by-second accuracy
- Ability to interface with other software
- A well equipped Application Programming Interface (API) library with current research being carried out to enhance the existing codes and performance.

While either of PARAMICS, VISSIM or AIMSUN could be used for modeling and obtaining surrogate safety measures with a high degree of fidelity, PARAMICS is chosen based on its proven scalability and presence of flexible API libraries associated specifically with issues like Variable Speed Limits and Ramp Metering.
There have been limited studies relating the effects of Ramp Metering Strategies or Variable Speed Limits with safety measures on freeways. The works of Lee et al. (2004, 2005), Dilmore (2005) and Abdel-Aty et al. (2006) have addressed this issue for single locations and in an exclusive manner for each strategy. This study takes this analysis to a different level both in terms of scale and strategy. While a network is considered for analysis of the effects of these strategies, the strategies are themselves implemented at multiple locations in a variety of combinations. The objective is to find the best possible combination of available ITS resources that could help in crash risk mitigation. ALINEA is rated as one of the best algorithms in terms of alleviating congestion especially in cases of recurrent congestion (Chu et al., 2004 and Papageorgiou et al., 1998). It compares favorably with other ramp metering algorithms like BOTTLENECK, FLOW, etc. when considering cases for travel time, speeds and flows (Chu et al., 2004, Scariza, 2003, Lee et al. 2005). As such it provides an ideal choice for testing the benefits of ramp metering in terms of safety without the need for developing complicated implementation algorithms at the outset.

The literature review thus provided three important directions for this study in terms of providing the confidence in results from Micro-simulation, proving the scalability and acceptability of PARAMICS for use in such a study as this and providing detailed information about the possible alternatives for Ramp Metering and Variable Speed Limit strategies that have shown encouraging results for traffic and safety related parameters.
3 CHAPTER 3- METHODOLOGY

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 Hayward 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- Atty et al., 2006).

In light of the previous research it became necessary to test the effectiveness of the ITS strategies like Ramp Metering and Variable Speed Limits. Also, it is necessary 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 roadway. This process would be described separately in Chapter 4. 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.
Dilmore (2005) in his work 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). This study wanted to explore the possibility of implementing variable speed limits in the low speed scenario also. 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 3-1. 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 3-1. 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 3.1.2.
Table 3-1 Station Classification for I-4 East Bound based on Speed Regime

<table>
<thead>
<tr>
<th>Station</th>
<th>Hourly Mean Speed</th>
<th>Classification</th>
</tr>
</thead>
<tbody>
<tr>
<td>33</td>
<td>35.432</td>
<td>Low Speed</td>
</tr>
<tr>
<td>34</td>
<td>27.324</td>
<td>Low Speed</td>
</tr>
<tr>
<td>35</td>
<td>24.7180</td>
<td>Low Speed</td>
</tr>
<tr>
<td>36</td>
<td>22.763</td>
<td>Low Speed</td>
</tr>
<tr>
<td>37</td>
<td>34.3059</td>
<td>Low Speed</td>
</tr>
<tr>
<td>38</td>
<td>30.1890</td>
<td>Low Speed</td>
</tr>
<tr>
<td>39</td>
<td>23.8280</td>
<td>Low Speed</td>
</tr>
<tr>
<td>40</td>
<td>16.2494</td>
<td>Low Speed</td>
</tr>
<tr>
<td>41</td>
<td>18.890</td>
<td>Low Speed</td>
</tr>
<tr>
<td>42</td>
<td>25.67226</td>
<td>Low Speed</td>
</tr>
<tr>
<td>43</td>
<td>28.5642</td>
<td>Low Speed</td>
</tr>
<tr>
<td>44</td>
<td>35.663</td>
<td>Low Speed</td>
</tr>
<tr>
<td>45</td>
<td>36.240</td>
<td>Low Speed</td>
</tr>
<tr>
<td>46</td>
<td>35.080</td>
<td>Low Speed</td>
</tr>
<tr>
<td>47</td>
<td>36.802</td>
<td>Low Speed</td>
</tr>
<tr>
<td>48</td>
<td>35.432</td>
<td>Low Speed</td>
</tr>
<tr>
<td>49</td>
<td>34.675</td>
<td>Low Speed</td>
</tr>
</tbody>
</table>
3.1 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.

3.1.1 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:

**Equation 3-1**

\[
\text{logit} \left( p_j(x_{ij}) \right) = \alpha_j + \beta_1 x_{1ij} + \beta_2 x_{2ij} + \ldots + \beta_k x_{kij}
\]

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

Note that the intercept term \( \alpha_j \) 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 $\alpha_1, \alpha_2, \ldots, \alpha_N$ (Collett, 1991). So the effects of matching variables cannot be estimated and Equation 3-1 cannot be used to estimate crash probabilities. However, the values of the $\beta$ parameters that maximize the conditional likelihood function would also be estimates of $\beta$ coefficients in Equation 3-1. 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 prediction purposes under this matched crash-non-crash analysis. Consider two observation vectors $x_{1j} = (x_{11j}, x_{21j}, \ldots, x_{k1j})$ and $x_{2j} = (x_{12j}, x_{22j}, \ldots, x_{k2j})$ 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 3-1 and its form is shown in equation 3-2.

**Equation 3-2**

$$\log \left\{ \frac{p(x_{1j})/[1-p(x_{1j})]}{p(x_{2j})/[1-p(x_{2j})]} \right\} = \beta_1(x_{11j} - x_{12j}) + \beta_2(x_{21j} - x_{22j}) + \ldots + \beta_k(x_{k1j} - x_{k2j})$$

The right hand side of equation 3-2 depends only on $\beta_j$, therefore the estimate for log odds ratio may be obtained using estimated $\beta$ 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 $\bar{x}_{2j} = (\bar{x}_{12j}, \bar{x}_{22j}, \bar{x}_{32j}, \ldots, \bar{x}_{k2j})$ 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 on average by equation 3-3.

**Equation 3-3**

\[
\log \left( \frac{p(x_{1j})/[1 - p(x_{1j})]}{p(x_{2j})/[1 - p(x_{2j})]} \right) = \beta_1(x_{11j} - \bar{x}_{12j}) + \beta_2(x_{21j} - \bar{x}_{22j}) + \ldots + \beta_p(x_{k1j} - \bar{x}_{k2j})
\]

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 $> 37.5$mph) and low speed regime (average speeds $< 37.5$mph).

### 3.1.2 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 3-1 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 and 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 3-1. As can be seen in Figure 3-1, 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 3-1 Detector Numbering on I-4](image)

The first letter in the variable name can be A, S or CV. 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.

### 3.1.3 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 $\text{LogCVSF2}$ and $\text{LogCVSF3}$ have positive model coefficients implying highly varying speeds around the crash location. It indicates frequent formation and dissipation of queues. Also, with $\text{LogCVSF2}$ having larger and far more significant hazard ratio than $\text{LogCVSF3}$, 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 $\text{logCVSF2}$ and $\text{logCVSF3}$ 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 $\text{LogCVSF2}$ is also coupled with low standard deviation in volume (indicated by negative coefficient of $\text{SVF2}$) 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 $\text{AOH3}$ and $\text{AOE2}$, 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 3-4.

Equation 3-4

\[
\text{Risk Index} = 2.64827 \log(\text{CVSF}_2) + 0.88842 \log(\text{CVSF}_3) + 1.33966 \log(\text{AOE}_2) + 0.97766 \log(\text{AOH}_3) - 0.43603 \text{SVF}_2
\]

3.1.4 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 \( \log(\text{AOF}_2) \) 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 3-5.

**Equation 3-5**

$$Risk\_Index = -0.93423 \times LogAOF2 + 1.14584 \times LogAOH3 - 0.22878 \times SVH2 - 0.10055 \times AVG2 + 0.05932 \times AVE3$$

### 3.2 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 shows the measure of Effectiveness for the East Bound Direction. A detailed explanation of the study corridor is presented in Section 4-3.

Equation 3-6

\[
(\text{MOE})_i = \frac{1}{n} \left( \sum_{j=1}^{n} \sum_{t=1}^{360} (\text{RiskIndex}) \right)
\]

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.

3.3 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 3.3.1, 3.3.2, and 3.3.3.
3.3.1 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.

3.3.1.1 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 Section 5.1.
3.3.1.2 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 3-2. 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 Chapter 5, feedback ramp metering works much better than pre-timed, so a detailed exploration of parameters was of little value. Figure 3-2 represents the levels of each factor that were considered for analysis of pre-timed metering.

Table 3-2 Cases for Pre-Timed Metering Evaluation

<table>
<thead>
<tr>
<th>Case</th>
<th>Cycle Length</th>
<th>Green time/ Phase</th>
<th>Number of Ramps Metered</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>50</td>
<td>25</td>
<td>1</td>
</tr>
<tr>
<td>2</td>
<td>50</td>
<td>15</td>
<td>1</td>
</tr>
<tr>
<td>3</td>
<td>25</td>
<td>10</td>
<td>1</td>
</tr>
<tr>
<td>4</td>
<td>25</td>
<td>15</td>
<td>1</td>
</tr>
<tr>
<td>5</td>
<td>50</td>
<td>25</td>
<td>7</td>
</tr>
<tr>
<td>6</td>
<td>50</td>
<td>15</td>
<td>7</td>
</tr>
<tr>
<td>7</td>
<td>25</td>
<td>10</td>
<td>7</td>
</tr>
<tr>
<td>8</td>
<td>25</td>
<td>15</td>
<td>7</td>
</tr>
</tbody>
</table>
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 500-667 vph. 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.

### 3.3.2 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 2-3, 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 3-3 and Figure 3-4.

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 3-4 and Table 3-3.

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 2-3) 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 3-3.

![Figure 3-3 Q-K relationship with critical density marked](image_url)

*Figure 3-3 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 3-3 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 3-3 and is graphically depicted in Figure 3-4. Table 3-3 shows a total of 48 cases of parameter combinations used for optimization.

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

<table>
<thead>
<tr>
<th>Module2</th>
<th>Number of Ramps Metered</th>
<th>Cycle Length</th>
<th>Critical Occupancy</th>
<th>Critical Occupancy</th>
<th>Critical Occupancy</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1</td>
<td>25s</td>
<td>0.17</td>
<td>0.2</td>
<td>0.23</td>
</tr>
<tr>
<td></td>
<td>1</td>
<td>50s</td>
<td>0.17</td>
<td>0.2</td>
<td>0.23</td>
</tr>
<tr>
<td>$K_R=70$</td>
<td>3</td>
<td>25s</td>
<td>0.17</td>
<td>0.2</td>
<td>0.23</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>50s</td>
<td>0.17</td>
<td>0.2</td>
<td>0.23</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>25s</td>
<td>0.17</td>
<td>0.2</td>
<td>0.23</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>50s</td>
<td>0.17</td>
<td>0.2</td>
<td>0.23</td>
</tr>
<tr>
<td></td>
<td>7</td>
<td>25s</td>
<td>0.17</td>
<td>0.2</td>
<td>0.23</td>
</tr>
<tr>
<td></td>
<td>7</td>
<td>50s</td>
<td>0.17</td>
<td>0.2</td>
<td>0.23</td>
</tr>
<tr>
<td></td>
<td>1</td>
<td>25s</td>
<td>0.17</td>
<td>0.2</td>
<td>0.23</td>
</tr>
<tr>
<td></td>
<td>1</td>
<td>50s</td>
<td>0.17</td>
<td>0.2</td>
<td>0.23</td>
</tr>
<tr>
<td>$K_R=120$</td>
<td>3</td>
<td>25s</td>
<td>0.17</td>
<td>0.2</td>
<td>0.23</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>50s</td>
<td>0.17</td>
<td>0.2</td>
<td>0.23</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>25s</td>
<td>0.17</td>
<td>0.2</td>
<td>0.23</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>50s</td>
<td>0.17</td>
<td>0.2</td>
<td>0.23</td>
</tr>
<tr>
<td></td>
<td>7</td>
<td>25s</td>
<td>0.17</td>
<td>0.2</td>
<td>0.23</td>
</tr>
<tr>
<td></td>
<td>7</td>
<td>50s</td>
<td>0.17</td>
<td>0.2</td>
<td>0.23</td>
</tr>
</tbody>
</table>
3.3.3 **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. The factors effecting Variable Speed Limits as found from an analysis of work by Dilmore (2005) shows that the important factors for us to consider 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 3-4, 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 3-4. All the different variables and their levels are indicated in Figure 3-5.
Table 3-4 Split Plot Experimental Design

<table>
<thead>
<tr>
<th>Block Single</th>
<th>Gradual</th>
<th>Gradual</th>
<th>Sudden</th>
<th>Sudden</th>
<th>Gradual</th>
<th>Gradual</th>
<th>Sudden</th>
<th>Sudden</th>
</tr>
</thead>
<tbody>
<tr>
<td>Metering</td>
<td>S1</td>
<td>S2</td>
<td>S1</td>
<td>S2</td>
<td>S3</td>
<td>S4</td>
<td>S3</td>
<td>S4</td>
</tr>
<tr>
<td>Block Network</td>
<td>Gradual</td>
<td>Gradual</td>
<td>Sudden</td>
<td>Sudden</td>
<td>Gradual</td>
<td>Gradual</td>
<td>Sudden</td>
<td>Sudden</td>
</tr>
<tr>
<td></td>
<td>S1</td>
<td>S2</td>
<td>S1</td>
<td>S2</td>
<td>S3</td>
<td>S4</td>
<td>S3</td>
<td>S4</td>
</tr>
<tr>
<td>Block Single</td>
<td>Gradual</td>
<td>Gradual</td>
<td>Sudden</td>
<td>Sudden</td>
<td>Gradual</td>
<td>Gradual</td>
<td>Sudden</td>
<td>Sudden</td>
</tr>
<tr>
<td>No-metering</td>
<td>S1</td>
<td>S2</td>
<td>S1</td>
<td>S2</td>
<td>S3</td>
<td>S4</td>
<td>S3</td>
<td>S4</td>
</tr>
<tr>
<td>Block Network</td>
<td>Gradual</td>
<td>Gradual</td>
<td>Sudden</td>
<td>Sudden</td>
<td>Gradual</td>
<td>Gradual</td>
<td>Sudden</td>
<td>Sudden</td>
</tr>
<tr>
<td></td>
<td>S1</td>
<td>S2</td>
<td>S1</td>
<td>S2</td>
<td>S3</td>
<td>S4</td>
<td>S3</td>
<td>S4</td>
</tr>
</tbody>
</table>

Experimental Setup:

![Tree Diagram](image)

Figure 3-5 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 value as high magnitude speed changes might
cause unnecessary braking and speed differentials. 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 Tables 3-5, 3-6 and 3-7 in section 3.8.

3.4 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 3-6 Quantification of benefits (x-axis – time, y-axis – Risk Index)

Figure 3-6 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.
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 is 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:

\[
\text{Safety Benefits Index} = \frac{\left| (\text{Area under Test Case} - \text{Area under Base Case}) \right|}{\left| \text{Area under Base Case} \right|}
\]

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).
3.5 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.

3.5.1 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).

### 3.5.2 Plans

The plans file is used to code in the specific signal plan for the node in question. Using the guidelines from Quadstone (2002) 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.

3.6 Variable Speed Limits

Variable speed limits in PARAMICS can be implemented in two main ways. Dilmore (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 throughout 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 prediction 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 3-8.

![Figure 3-8 Nomenclature for stations of interest](image)

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.
3.7 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.
### 3.8 Experimental Design Cases

Table 3-5 Pre-timed metering case numbers and variable values

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Table 3-6  Feed-back Metering case numbers and variable values

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Table 3-7 VSL and FB Metering case numbers and variable values

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In an age where we are dealing with a very high penetration of advanced computing into every aspect of research, Transportation and traffic research has also seen an increased use of this activity. The penetration is not just limited to data collection, synchronization and analysis but also in prediction and control. Simulations have become an integral element in the research community. Increasingly simulations are becoming training and optimization tools replacing the need to conduct numerous real world experiments. Traffic Simulation Models provide us with a similar capability to integrate research and implementation. Additionally, with the presence of numerous amounts of data available from hardware like inductive loops, cameras, manual counts etc., it is also possible to make accurate designs for these models and be able to validate them.

Micro-simulation is one such Simulation Model (Others being Macroscopic and Mesoscopic, Boxill and Yu, 2000). Micro-simulation requires large amounts of data for model building but it also provides the ability to test numerous scenarios. With the introduction of Advanced Traffic Management and Information Systems (ATMIS), Micro-simulation provides probably the best tool for testing the system performance as well as its optimized usage. This additionally does away with the cost of introducing test systems into live traffic streams.

As indicated earlier, traffic simulation models are becoming an increasingly important tool for traffic control. Simulators are needed, not only to assess the benefits of ITS in planning mode, but also to generate scenarios, optimize control, and predict network behavior at the operational level. They can also give the traffic engineer an overall picture of the traffic and the ability to assess current problems and test possible
solutions immediately. Experimental or new techniques can be tried and tested without any disruption to traffic in a real network.

Traffic simulation models can be classified as either microscopic, mesoscopic, or macroscopic. Microscopic models continuously or discretely predict the state of individual vehicles. Microscopic measures include individual vehicle speeds and locations. Macroscopic models aggregate the description of traffic flow. Macroscopic measures of effectiveness can be speed, flow and density. Mesoscopic models have aspects of both macro and microscopic models. In addition, simulation models can be classified by functionality, i.e. signal, freeway, or integrated. A number of these models like Synchro, CORSIM, etc are in frequent usage for various reasons.

4.1 Functions of Traffic Simulation Models for ITS

The *Actions de Préparation, d’accompagnement et du suivi* (APAS, 1995) assessment of road transport models and system architectures has identified four main uses for simulation models in ITS:

1. Simulating networks including interaction between vehicles and new responsive control and information systems.
2. Short term forecasting.
3. Enhancing assignment models.
4. Providing inputs to car driving simulations.

This research will deal with the first in this list of functions.
4.2 Surrogate Safety Measures in Simulation Models

The USDOT Report on Surrogate Safety measures in simulation models (USDOT, 2003) states that among the micro-simulations available for research and use, PARAMICS, VISSIM and AIMSUN provide the best interface to derive surrogate safety measures from simulation. This could be based on the superiority of these simulations in 4 primary areas over packages such as CORSIM, SIMTRAFFIC, WATSIM, HUTSIM, TEXAS and INTEGRATION.

- Ability to obtain detailed state variable information on each vehicle on time scales with better than second-by-second accuracy
- Capability for interface to other external codes and hardware control designs
- Existing API libraries which are being updated and refined by researchers
- Ability to model all features related to surrogate safety analysis with high fidelity

Most other packages either lack one or more of these capabilities or are still in the development stage for such a use. The fact that this study uses at one time or the other, each of the advantages that are specified above, the choice of simulation software was restricted to one of these. Each of PARAMICS, VISSIM and AIMSUN has their advantages and disadvantages and either would be a good pick. PARAMICS scores over the others in the availability of a better API library for easier development of desired application codes. Besides, PARAMICS development is leading to an extensibility of input and output processors, which could prove to be one of its biggest advantages in the future. The only main disadvantage that PARAMICS had is its reliance on Origin
Destination Matrices for deriving the traffic volumes on links. The extent of this study allows us to ignore dynamic route assignment and hence, to a large extent avoid this disadvantage.

In light of the benefits of PARAMICS and its compatibility with the objectives of this study, it was chosen as the simulation of choice.

4.3 Network Description

The overall simulation was done for a 19 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.

4.4 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 prediction models we describe in the Section 3.1 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 4-1 depicts a flow chart showing the operational sequence for the Calibration and Validation process and the process of testing different scenarios.
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. This section is shown in Figure 4-2 as study section.

4.5 Data Preparation

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.
4.5.1 AutoCAD

Knowing that the I-4 corridor will be drawn in PARAMICS and that the program accepts .dxf files, an overlay was drawn in AutoCAD and then saved as a .dxf file. To avoid 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 Systems Simulation (CATSS) office at the University of Central Florida (UCF).

The final network drawing of the corridor is shown in Figure 4-2. It also shows the study section that will be used for calibration as well as the starting and ending station numbers within the section. The shaded region points out the highly congested downtown area of Orlando. The study section that we are concerned with extends from Station 33 to 49, about 9 miles of Interstate-4.
Figure 4-2 Interstate-4 with study section, major roads and detector stations shown.
4.6 Calibration of the OD Matrix

4.6.1 Data Sources

To construct the Origin Destination Matrix of the I-4 corridor six sources were 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 conducted by URS, and the 2003 Florida Standard Urban Transportation Model Structure (FSUTMS) model output. Hourly ramp counts for I-4 were also obtained from URS through Shankar Ramasamy at the Center for Advanced Transportation System Simulation at UCF. 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 process of estimating the data for fourth lane would be too large and error prone, making the method undesirable.

A non-parametric analysis is used to compare the data sources to see if they were 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 from 2002 was significantly higher than the FDOT data, which was unexpected. The data was considered unreliable and hence not used. 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.
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 were associated with off ramps were given zeros. Next, all columns associated with on ramps were given zeros. Then, all indices associated with on ramps and off ramps on different roads (different directions on the interstate) were given zeros. Finally, all off ramps behind a given on ramp were given zeros.

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

1. When rows and columns both were 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 were repeated.

It should be noted that the correction steps were performed at all other ramps besides the East-West Expressway first and then the East-West Expressway counts were adjusted.

### 4.6.2 Remaining Error

The remaining error, the difference between the peak hour ramp counts and the OD matrix, after adjustment, is 4.10% 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 (for the whole EB and WB network), in terms of vehicles per hour, may be seen in Appendix.

All the work related to data collection and authentication was in Sections 4.5 and 4.6 were carried out by Jeremy Dilmore. A detailed description can be found in Dilmore (2005).

### 4.7 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 4-4 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 was warehoused at UCF until February, 2004. The source of crash and geometric characteristics data for the freeway is *FDOT* (Florida Department of Transportation) intranet server.
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.

4.7.1 Missing Data

The loop data extracted from available data in UCF 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 4-1.
Equation 4-1

CS = 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

A sample for calculation of Center Lane speed data for station in East Bound direction is provided in Table 4-1. 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 5 minute speeds in left lane etc.

Detailed Models for all stations and parameters are listed in Appendix.
Table 4-1 Regression Coefficients for Stations 34 through 48 for Center Lane Speed Imputation

<table>
<thead>
<tr>
<th>Station</th>
<th><em>RMSE</em></th>
<th>Intercept</th>
<th>LS</th>
<th>LV</th>
<th>LO</th>
<th>LS_LS</th>
<th>LV_LS</th>
<th>LV_LV</th>
<th>LO_LS</th>
<th>LO_LV</th>
<th>LO_LO</th>
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<td>0.026667</td>
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</table>
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)

**Equation 4-2**

\[
GEH(S) = \sqrt{\frac{(F_y - F_i)^2}{(F_y + F_i)/2}}
\]

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 –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 4-4) 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. A scatter plot of the available data classed by categories can be found in Appendix. This period was then used for all further calculations as it presented the best prospect of data availability and consistency.
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 also the fact that one hour speed averages didn’t make any sense.

Dilmore (2005) used the same network in his work for testing the effects of VSL on this corridor. However, significant emphasis wasn’t paid in the 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.

### 4.7.2 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.

4.8 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 4-2. 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%< error < 20%, > 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 4-2 Sample Profile for Calibration

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<th>5 min. Time Interval</th>
<th>Detector 33</th>
<th>Detector 34</th>
<th>Detector 35</th>
<th>Detector 36</th>
<th>Detector 37</th>
<th>Detector 38</th>
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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 Figure 4-5 through 4-12. 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 4-3 shows the difference between actual counts and simulated counts for every five minute interval. Figure 4-5 and 4-6 show 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 4-3. Also notable is the larger errors towards the end of the simulation between time intervals 22 to 30. Figure 4-6, 4-7 and 4-8 show that the errors at these stations are much larger. This can also be seen in Table 4-3 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 4-8, 4-9, 4-10 and 4-11. Station 49
shows larger errors for the all the first 22 time intervals as is seen in Table 4-3. 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 this errors in flows. This part is dealt with in the Section 4.9.
Figure 4-5 Comparison of mainline counts Station 33 and 34 EB (x axis- number of 5 min. intervals)
Figure 4-6 Comparison of mainline counts Station 35 and 36 EB (x axis- number of 5 min. intervals)
Figure 4-7 Comparison of mainline counts Station 37 and 38 EB (x axis- number of 5 min. intervals)
Figure 4-8 Comparison of mainline counts Station 40 and 41 EB (x axis- number of 5 min. intervals)
Figure 4-9 Comparison of mainline counts Station 42 and 43 EB (x axis- number of 5 min. intervals)
Figure 4-10 Comparison of mainline counts Station 44 and 45 EB (x axis- number of 5 min. intervals)
Figure 4-11 Comparison of mainline counts Station 46 and 47 EB (x axis- number of 5 min. intervals)
Figure 4-12 Comparison of mainline counts Station 48 and 49 EB (x axis- number of 5 min. intervals)
Table 4-3 Flow errors for five minutes Stations 33- 49. (T- 5 min Interval)

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4.9 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 Section 4-8 showed in Table 4-3 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 4-13 and 4-14 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 with 10% of the actual speed. Figure 4-13 and 4-14 show that the speeds at stations 36, 37 and 40 are almost always much higher than the speeds in the actual conditions.
Figure 4-13 Speed comparisons for Stations 36 and 37 with 10% error bars
Figure 4-14 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 4.9.1. 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. This process is explained in Section 4.9.2.

4.9.1 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/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 4-15 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 4-15 Speed comparisons for Stations 38 and 40 with 95% CI bars](image-url)
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 4-15 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 way over posted speeds. This meant that every time the vehicles emerged
from congestion, they tended to speed up leading to higher overall 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 queuing 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 4-4 were tested along with adjustments in end zones of OD.

Table 4-4 Cases Tested for Calibration Parameters

<table>
<thead>
<tr>
<th>Case</th>
<th>Headway</th>
<th>Reaction Time</th>
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<tr>
<td>8</td>
<td>0.50 sec</td>
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</table>

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). The only difference being the value of reaction time. The fact that these values provided the best possible queuing behavior confirmed the conclusion made by Dilmore (2005) in the verification process regarding 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.

4.9.1.1 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.

4.9.1.2 Accepted Scenario

The final accepted scenario is presented in Figures 4-16 to 4-31. Figures 4-16 to 4-31 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 4-5. This table follows directly from Table 4-3 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 4-5 Station wise errors in flows at end of calibration (T- 5 min Interval)

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Figure 4-16 Flow and Speed profile for Station 33 at end of Calibration
Figure 4-17 Flow and Speed profile for Station 34 at end of Calibration
Figure 4-18 Flow and Speed profile for Station 35 at end of Calibration
Figure 4-19 Flow and Speed profile for Station 36 at end of Calibration
Figure 4-20 Flow and Speed profile for Station 37 at end of Calibration
Figure 4-21 Flow and Speed profile for Station 38 at end of Calibration
Figure 4-22 Flow and Speed profile for Station 40 at end of Calibration
Figure 4-23 Flow and Speed profile for Station 41 at end of Calibration
Figure 4-24 Flow and Speed profile for Station 42 at end of Calibration
Figure 4-25 Flow and Speed profile for Station 43 at end of Calibration
Figure 4-26 Flow and Speed profile for Station 44 at end of Calibration
Figure 4-27 Flow and Speed profile for Station 45 at end of Calibration
Figure 4-28 Flow and Speed profile for Station 46 at end of Calibration
47 Flows

47 Speeds

Figure 4-29 Flow and Speed profile for Station 47 at end of Calibration
Figure 4-30 Flow and Speed profile for Station 48 at end of Calibration
Figure 4-31 Flow and Speed profile for Station 49 at end of Calibration
5 CHAPTER 5- RESULTS

5.1 Ramp Metering

Chapter 3 provides 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 3 measures were defined in Sections 3.1, 3.2, 3.4. These measures were represented by numbers and 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 Prediction 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

Equation 5-1

\[ Risk\_Index = 2.64827 \log(CVS)_{F2} + 0.88842 \log(CVS)_{F3} + 1.33966 \log(AOE)_{E2} + 0.97766 \log(AOH)_{H3} - 0.43603 SVF_{2} \]

Moderate to High Speed Model

Equation 5-2

\[ Risk\_Index = -0.93423 \log(AOF)_{F2} + 1.14584 \log(AOH)_{H3} - 0.22878 SVH_{2} - 0.10055 AVG_{2} + 0.5932 AVE_{3} \]

The terms in the notation follow a specific notation that is followed throughout 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 is defined in Figure 3.8.

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

2\(\rightarrow\) 5 - 10 minute interval before the time of interest

3\(\rightarrow\) 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 3.2 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.
Equation 5-3

\[(MOE)_i = \frac{1}{n} \left( \sum_{j=1}^{n} \sum_{t=1}^{360} (RiskIndex) \right)\]

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

\(j\) = Seed Number (20)

\(t, n\) = 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 3.4. 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 Index = \[\left\{ \frac{|(Area \ under \ Test \ Case \ - \ Area \ under \ Base \ Case)|}{|Area \ under \ Base \ Case|} \right\}\]

Station wise comparisons are done using just the Safety Benefits for just that station and are calculated as illustrated below.

Safety Benefits Index = \[\left\{ |(Area \ under \ Test \ Case \ - \ Area \ under \ Base \ Case) | \right\}\]

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.
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.

![43 Risk Index](image)

**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 Figures 5-5 however shows an improvement in the risk index as comparison to the case in Figure 5-4.
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 3.2. 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 500-667 vph. 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 600 vph 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 3-6. 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.
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
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 from Section 2.4.1 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.
Equation 5-4

\[ R(t) = R(t - \delta t) + K_r \times (\hat{O} - O(t)) \]

\( \delta t \) 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 \( \delta t \)
- 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 2000 ft 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 setup 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 3.3.2 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 3-6. 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, as described in Section 3.4. 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](image)

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. Dilmore (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.
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.

![Detector 33-49/ 5 Ramps](image)

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.
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 prediction provided in Equation 5-1. From Equation 5-1, it is obvious 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, 24, 25 and 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, 24, 25, 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, 28 and 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
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 prediction 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 $K_R$

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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, 15, 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

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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 3-4. 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. For example, the value of these benefits was found to be 585.83. These numbers could be compared across stations to compare station wise benefits. 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

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<th>Area Case21-X</th>
<th>Station Benefit</th>
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<td>-21047.6</td>
<td>-32434.2</td>
<td><strong>11386.68</strong></td>
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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>
<thead>
<tr>
<th>Case</th>
<th>Area under Curve</th>
<th>Area b/n Curves</th>
<th>Benefit Index</th>
</tr>
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<td>M2_5</td>
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<td>M2_6</td>
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<td>-29140.06992</td>
<td>7991.16992</td>
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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 3.4). 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.
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 Fig 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.

![Safety Benefits by Station](image)

**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 throughout 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.
Equation 5-5

\[ TT = \sum_{j=1}^{17} \sum_{i=1}^{30} \left( \frac{l_i}{s_{ij}} \right) \]

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 (y-axis – time in minutes)**

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 accumulated 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 wise 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 the 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 3.3. Dilmore (2005) had reached the conclusion that Variable Speeds when implemented in the case of low speeds does not show a significant difference 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 Section 5.4.1. The Application Programming Interface in PARAMICS was used to implement variable speeds. However, significant difference exists in the way Variable Speed Limits in PARAMICS was implemented by Dilmore (2005) and this research.

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. The current research 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:
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.

Dilmore (2005) 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 15mph 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

\[
\text{Average 5 min speeds} = \begin{cases} 
\leq 35\text{mph} \\
35\text{mph} < \text{speed} \leq 45\text{mph} \\
45\text{mph} < \text{speed} \leq 55\text{mph} \\
> 55\text{mph}
\end{cases}
\]
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 works of Lee (2004) and Dilmore (2005), where both research works used either one of them 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). Dilmore (2005) 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, Dilmore (2005) did not look at congested situations but looked at locations with only high speeds. The variable parameters in this study therefore became

- The strategy used for variable speed limits across all locations
- Number of ramps that were metered along with implementing Variable Speed Limits
- Magnitude of speed changes i.e. 5mph upstream and downstream or 10mph upstream (2 stations) and 10 mph downstream.

The 24 cases that resulted from this experimental design (Refer Table 3-8) could be split into three main sections. 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 10mph 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 by Dilmore (2005), in that Variable Speeds 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.

![33-49/ Case 1-8 Module 3](image)

**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, 43, 44 and 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 to 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 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.

![RM vs RM+VSL for 3 ramps](image)

**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 to 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 to 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 figures from 5-52 to 5-45 and 5-48 to 5-51, it is very 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 et al., 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.
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

Dilmore (2005) presented in his work the conclusion that variable speed limits did not perform in the low speed regimes. However, the study in that case 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 speeds found to be ineffective. Although this makes sense if the speeds of the cars are in the very low conditions, 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 34, 35, 36 and 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 5.4.6.

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, 37, 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, 37, 38, 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 by Dilmore (2005). 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 overall 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.
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 seem Case 12 and Case 18 dominate the benefits.
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.
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.
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 5-54. 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**

<table>
<thead>
<tr>
<th></th>
<th>TT</th>
<th>Max. variance</th>
<th>CI Minimum</th>
<th>CI Maximum</th>
</tr>
</thead>
<tbody>
<tr>
<td>Base</td>
<td>154925.9</td>
<td>3525.1</td>
<td>151400.8</td>
<td>158451</td>
</tr>
<tr>
<td>Module 3 Case</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>12</td>
<td>139096.1</td>
<td>2664.33</td>
<td>136431.8</td>
<td>141760.4</td>
</tr>
<tr>
<td>Module 3 Case</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>18</td>
<td>121685.9</td>
<td>2235.4</td>
<td>119450.5</td>
<td>123921.3</td>
</tr>
</tbody>
</table>

**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.

Equation 5-6

\[ \text{Risk} = 1.21405 \log \text{CVSF2} + 0.02466 \text{AOG2} - 0.19124 \text{SVG2} \]

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.
6 CHAPTER 6- CONCLUSIONS

This research 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 overall 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 overall travel times which were found to be better than even the case where all 7 ramps in our network were metered.

### 6.1 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 and Lane Change control could be explored individually as well as to complement each other. Extensive research is however needed to explore the driver behavior part in these strategies. For instance, how to warn the drivers best about impending speed limit changes or how to issue warning about the safety risk on the corridor, etc. Micro-simulation as a tool will provide a very good starting point for these studies and field implementation would be required to evaluate this research.
REFERENCES


