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Employment of the Traffic Management Lab for the Evaluation and Improvement of Stratified Metering Algorithm--Phase III

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16. Abstract (Limit: 200 words)

The evaluation results (done in Phase II) demonstrated that the SZM strategy was generally beneficial. However, they also revealed that freeway performance degraded by reducing the ramp delays. Therefore, it is desired to improve the effectiveness of the current SZM control. There are two objectives in this study. One objective is to improve the control logic of current SZM strategy. This is accomplished through an estimation algorithm for the refined minimum release rate. The simulation results indicate that the improved SZM strategy is very effective in postponing and decreasing freeway congestion while resulting in smoother freeway traffic flow compared to the SZM strategy. The second objective of this project is to improve the current queue size estimation. Depending on the counting error of queue and passage detectors, freeway ramps are classified into three different categories, and different methods are applied respectively for improved queue size estimation. The surveillance video data were recorded and used to verify the improvement of the proposed methods. The results indicate that the proposed methods can greatly improve the accuracy of queue size estimation compared with the current methodology. Also, the proposed method was evaluated by the micro-simulation. The simulation results indicate the performance of freeway mainline is significantly improved. And the total system performance is better than the original SZM control.

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Employment of the Traffic Management Lab for the Evaluation and Improvement of Stratified Metering Algorithm – Phase III

Final Report

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Executive Summary

Minnesota's Stratified Zone Metering (SZM) Strategy, as a successor of the ZONE metering algorithm, has been deployed in the Minneapolis/Saint Paul area since early 2002. The SZM strategy aims maximize freeway capacity utilization while keeping ramp delays below a predetermined threshold. Although preliminary evaluation results confirm that the SZM strategy is generally beneficial, they also reveal that freeway performance declines by reducing the ramp delays. Therefore, improving the effectiveness of the current SZM control is a desired goal.

There are two objectives in this study. One objective is to improve the control logic of current SZM strategy without compromising the ramp queue and delay constraints. This is accomplished through an estimation algorithm for the refined minimum release rate. Both the SZM and the improved SZM strategy are tested in a sophisticated microscopic simulator at TH169-NB and I-94EB test sites and under varying demand scenarios. The simulation results indicate that the improved SZM strategy is very effective in postponing and decreasing freeway congestion while resulting in smoother freeway traffic flow compared to the SZM strategy. It is also demonstrated that the effectiveness of the improved SZM logic depends on test site characteristics and demand patterns.

The second objective is to improve the current queue size estimation. To calculate in real time the on-ramp waiting times, accurate queue size estimation is crucial because inaccurate queue size can result in maximum wait time violations or reduce the quality of mainline traffic flow by releasing excessive vehicles into mainline. Such inaccuracies have been identified in the field and are caused by detection errors. In this report, the accuracy of the current SZM ramp queue size estimation method, which utilizes a uniform and pre-calibrated regression equation, is improved. Depending on the counting error of queue and passage detectors, freeway ramps are classified into three different categories, and different methods are applied respectively for improved queue size estimation. For Class I ramps, exhibiting minor counting error, a Conservation Model estimating queue size based on queue (Input) and passage detector counts (Output) is applied; For Class II ramps, which contain only significant counting error on passage detectors, the Conservation Model is still applied but the traffic counts of passage detectors are replaced by the so-called "Green Counts", which are calculated by the release rate of each ramp; Finally for Class III ramps, in which both the queue and passage detectors have significant error, two models were developed: the site-specific Regression and the Kalman Filtering Model. To verify the improvement of the proposed methods, surveillance video data were recorded and used to compare actual and estimated queue sizes. The results indicate that the proposed methods can greatly improve the accuracy of queue size estimation compared with the current methodology. And the simulation results indicate that by improving queue size estimation, the performance of freeway mainline is significantly improved. And the total system performance is better than the original SZM control.

Part I: Introduction and Background

Chapter 1 Introduction

1.1 Problem Statement

Ramp Metering, one of Traffic Management's tools, has been recognized as an effective way of relieving freeway congestion. It has been recommended to the U.S. Federal Highway Administration as the No. 1 tool to address freeway congestion, excluding additional capacity (Cambridge Systematics, 2004).

Ramp Metering has been deployed for over 30 years and is presently employed in a number of urban areas in North America. By 1995, ramp meters had been installed and operated in 23 metropolitan areas in the U.S. Of these, 11 cities have a system of more than 50 ramp meters, including Minneapolis-St. Paul (Piotrowicz G. and Robinson J. 1995).

Ramp metering regulates the flow of traffic entering the freeways during the peak periods. It limits the amount of traffic entering the freeway so that the demand at bottlenecks does not exceed capacity. When properly implemented and operated, ramp metering can benefit the freeway system by smoothing the overall flow of freeway traffic, decreasing travel times and increasing safety on the freeway. However, ramp metering also has potential drawbacks such as excessive ramp delays and long queues as well as high cost of installation and maintenance.

Over the years, several types of ramp meter strategies have been developed to meet different objectives of the engineers in charge of implementation. These strategies range from simple to complex ones and are trying to fully maximize the benefits of ramp control while keeping the disadvantages at a minimum.

The Minnesota Department of Transportation (Mn/DOT) started to implement ramp metering in 1970 on southbound I-35E north of downtown St. Paul. Since then, approximately 430 ramp meters have been installed and used to help manage the flow of traffic through bottlenecks and help merge traffic onto freeways in the Twin Cities Metropolitan Area. Mn/DOT has successfully implemented an integrated control scheme called zone control since 1972 and refined it substantially since then (Lau, 1996). The objective of the ZONE metering strategy is to maximize the utilization of freeway capacity without any constraints on ramp queues. It was thereby not uncommon for ramp vehicles to experience delays as long as 10 to 15 minutes during peak periods (Cambridge Systematics, 2001). In 2003, due to excessive ramp delays, a new strategy called the Stratified Zone Metering (SZM) was designed and implemented on all Twin Cities freeways (Lau, 2001). The new strategy is still focused on maximizing freeway throughout and avoiding bottleneck congestion, but also aims at keeping the maximum delays at all metered ramp below a predetermined threshold.

Implementation and evaluation of the new strategy demonstrated that the new strategy meets its maximum ramp delay objective and is effective in dissipating ramp queues faster. However, ramp delays are shifted to the mainline, thereby reducing the quality of mainline flow, as evidenced by the increase in the total mainline delays and the number of stops at test sites (Xin et al., 2004). In spite of this, the new strategy is still beneficial versus the no control alternative, but often marginally, especially when demands increase.

These results led to a search for potential improvement to the SZM control without compromising or significantly jeopardizing the maximum ramp delay constraint. One improvement presented in this report are based on better balancing the freeway performance and ramp delay objectives in a way that postpones and reduces the severity of bottleneck congestion, thereby increasing throughput, reducing freeway delays and improving system performance. The proposed improvements are implemented and evaluated at 2 test sites over several days representing different geometries and demand conditions. Evaluation results suggest that they are very effective in improving the system performance. The effects of more accurate online queue size estimation which is critical to the SZM strategy is also explored in this study.

The other improvement without compromising the maximum waiting time at each entrance ramp in this report is to improve queue size estimation. Currently, a uniform pre-calibrated regression equation is used to estimate queue size at all ramps. However, recent study (Feng, 2005) indicates that the current queue size estimation model underestimates queue size at certain ramps leading to maximum wait time violations; and more frequently, in other ramps, the model overestimates queue size, thereby releasing more vehicles into mainline by accelerating the onset of congestion. Further more, close examination which used traffic simulation proved that the accurate queue size estimation can greatly improve performance of mainline traffic flow. Based on these findings, Mn/DOT decided to improve the accuracy of ramp queue estimation. The goal of this study is to address this objective. Depending on the counting error of queue and passage detectors, freeway ramps were classified into three categories, and different methods were applied respectively for improved queue size estimation. The evaluation results presented here suggest that the improved methodologies are very effective as compared to the current queue size estimation in the SZM control.

1.2 Research Objectives

The objectives of this research include: (1) To improve the SZM control logic and provide tangible evidence of the improved performance; (2) To improve the queue size estimation. One of the preconditions of this research is to maintain the basic framework of the SZM. In summary, the key objectives are to:

- Develop a general improvement methodology to make the Twin Cities' New Stratified Zone Metering strategy more effective without compromising the maximum ramp delay objective. This requires thorough investigation of the SZM control logic and exploration of potential improvement alternatives.

- Evaluate the effectiveness of the proposed improvements at two typical test sites in Twin Cities Metropolitan Area. The evaluation should consider varying demand levels and different geometry characteristics.
- Explore the effects of more accurate queue size estimation on the SZM strategy and compare the resulting benefits with those of the proposed improved logic.
- Propose new efficient queue size estimation methodologies and test these methodologies using real video data.
- Evaluate the effectiveness of the proposed queue size estimation at two typical test sites in Twin Cities Metropolitan Area by micro-simulation.

1.3 Organization of Report

Part I of the report presented the general statement of the problem and reviews the literature on most of the state of the art ramp control strategies including Minnesota's New Stratified Zone Metering (SZM). Part II includes the methodologies of improvement of the control logic and evaluation of the improvement. Part III includes the new queue size estimation methodologies and the evaluation of new methodologies. And Part IV summarizes the finding with concluding remarks and future research suggestions.

Chapter 2 Background and Literature Review

The benefits of ramp metering depend largely on the controller/algorithm. In this chapter, the most common and practical ramp control algorithms will be presented. Most of real-life ramp metering deployments are essentially empirical. It should be noted that, despite lack of analytical formulations due to the empirical nature, optimal (or near optimal) operational performance of ramp metering could still be achieved through field fine-tuning by the conventional trial and error approaches.

According to the control structure involved, ramp control strategies can generally be classified into two categories, i.e., isolated ramp control and coordinated ramp control. The isolated ramp control is based on locally measured traffic conditions with the metering rates independent of each other; whereas in coordinated control, the metering rates are jointly coordinated in order to achieve a system-level objective.

ZONE algorithm, bottleneck algorithm, fuzzy control logic and SZM strategy will be briefly described in the chapter. They are representative examples of empirical design with successful large-scale field implementations.

2.1 Review of the Fieldale Ramp Control Algorithms

Ramp metering has been applied since 1963 in Chicago. The first metered ramp was installed in Chicago on the Eisenhower Expressway in 1963. This first ramp was controlled by a police officer, who stopped traffic and released vehicles on the ramp one at a time at a predetermined rate. With the advent of Intelligent Transportation Systems (ITS), ramp metering has become a key component of Advanced Traffic Management Systems (ATMS). Currently ramp meters are in operation in several metropolitan areas in North America (Piotrowicz et al., 1995 and Robinson et al. 1989).

2.1.1 Pros and Cons of Ramp Meters

Depending on the type of the hardware, strategies used by the implementing agencies and the geometric configuration of freeway, ramp and alternative arterial, the potential benefits of ramp meters may include:

1. Improve mainline traffic flow and increases in freeway productivity;
2. Reduce the number of stops on the freeway mainline, the fuel consumption and the emission of the pollutants;
3. Reduce the impacts of recurring congestion due to heavy traffic demand therefore delaying or preventing the occurrence of freeway slow speed operations;
4. Break the vehicle platoons and promoting easier and safer merging from ramps;
5. Encourage motorists on shorter trips to use arterials when alternative arterials exist and encourage motorists to shift travel times or change travel modes.

On the other hand, ramp meters also have some negative effects on the freeway/ramp system. Disadvantages of ramp metering include:

1. Increase the delays and waiting time on the ramps: although the overall system travel time may be improved and overall emissions are reduced, ramps experience increases in delay time and emissions.
2. Extend the ramp queues to the arterials: the ramp queue can back up onto the arterials and interrupt the local traffic if there is no mechanism controlling the ramp queues.
3. Negative effects on alternate routes: the ramp delays encourage motorists to use the arterials, which is actually a desired effect for shorter trips. This may increase the traffic volume on the alternative arterials significantly and result in congestion on the alternative arterials.
4. Inequity issues: the users of the ramps will experience long delays while everyone else on the freeway experiences very little delay.
5. Increase the fuel consumption and the emission of the pollutants on the ramps.

Other disadvantages like the like expensive cost of ramp metering system and the possible increase of accidents on the ramps also limit the overall benefit of the ramp metering system.

2.1.2 Types of Ramp Metering Algorithms

The benefits of ramp metering depend largely on the controllers. Controllers are the software or algorithm that meters use in controlling the ramp inflows. Depending on the goals and objectives of the implementing agencies, several types of ramp meter controllers can be pursued. Several factors influence how agencies choose the best control strategy for their cities, but the decision is mainly driven by the public, local politicians, and geometric conditions of the ramps and the freeway system.

The oldest and simplest form of ramp meter controller is pre-timed or fixed-time. These controllers would choose a metering rate based on the time of day, and would be based on historical data. These controllers cannot respond changes in traffic patterns because they don not use the real-time data. If demand pattern is unusual or an incident occurs, the pre-timed metering rate may be very ineffective. On the other hand, new controller technology allows for more sophisticated metering, where metering can adapt to the changes in mainline and ramp traffic conditions. These ramp meter controllers are called the traffic responsive controllers. Today, most of the ramp meter controllers are traffic responsive controllers.

Traffic responsive ramp metering controllers require real-time traffic surveillance. A variety of devices are available for traffic surveillance, but the most common device is the loop detector which is being deployed in many us cities. The typical measurements of the loop detector are traffic flow, speed, and occupancy.

According to the scope of the controllers, the traffic responsive ramp metering algorithms can be divided into two broad categories: local or isolated, and area-wide or coordinated.

- Local or isolated control: Local ramp metering control only takes into account the traffic conditions near a single ramp when calculating the metering rate for that ramp. The metering rate at one ramp does not take into account metering rates and traffic conditions at other ramps.

• Coordinated or Area-wide control: Coordinated ramp metering control/algorithm considers several ramp meters as a system and optimizes traffic flow over an area, rather than just at a single ramp.

The most well-known local ramp metering strategies are the demand-capacity (DC) strategy (Masher et al., 1975), the occupancy (OCC) strategy (Masher et al., 1975) and ALINEA (Papageorgiou et al., 1991). Papageorgiou et al., 1997 found that ALINEA performed significantly better as compared to DC and OCC strategies in several comparative field-evaluations.

1. ALINEA

ALINEA (Asservissement LINeaire d'Entree Autroutiere) was developed by engineers at the Technical University of Munich (Papageorgiou et al, 1991). A simple equation is used to calculate the metering rate:

$$r(k) = r(k-1) + K_R(\hat{o} - o_{out}(k-1)) \qquad (2\text{-}1)$$

Where

k=1, 2, …. is the discrete time index.

$r(k)$ is the ramp release rate to be implemented during control period k;

$r(k-1)$ is the ramp release rate implemented during control period k-1;

$o_{out}(k-1)$ is the measured downstream freeway occupancy(averaged over all lanes) for control period k-1;

K_R is a regulator parameter;

$\hat{o}$ is the desired value for downstream occupancy, typically but not necessarily $\hat{o} = o_{cr}$. (o_{cr} is the critical occupancy, which corresponds to downstream freeway occupancy when the downstream freeway flow rate reaches capacity)

From the equation it can be seen that ALINEA is a local-feedback control algorithm. It adjusts the metering rate to keep the freeway occupancy downstream of the ramp at a desired level. ALINEA is one of the most commonly used and one of the most effective algorithms.

Since the first application of ALINEA, several research studies have been conducted to enhance ALINEA. Oh and Sisiopiku(2001) proposed MALINEA, which improves the ALINEA by addressing the two disadvantages of ALINEA. The first is that the congestion could happen upstream of the on-ramp even if the occupancy downstream of the on-ramp can be kept to the desired occupancy. The second is that the determination of the optimal downstream detector station could be very difficult. Smaragdis and Papageorgiou (2003) extended the applications of

ALINEA strategy by developing AD-ALINEA and AU-ALINEA strategies. AD-ALINEA strategy is favorable when the desired occupancy cannot be determined in advance or is subject to real-time change of traffic conditions. AU-ALINEA is an upstream-measurement based version of AD-ALINEA when the lack of measurement devices happen downstream of the on-ramp.

The detailed description of the other two popular local ramp metering strategies (DC and OCC strategies) can be found in Masher et al. (1975).

Although the local ramp metering algorithms like ALINEA are widely used, the current trend is toward the coordinated algorithms. Coordinated algorithms are designed to optimize traffic flow over a section of the freeway rather than just a single ramp, in order to achieve greater efficiency. According to Chu et al. (2001), coordinated algorithms can be further divided into three classes: incremental or cooperative algorithms; bottleneck or competitive algorithms; and integral algorithms.

For cooperative ramp metering algorithms, only one set of metering rate is computed for each on-ramp. Further metering rate adjustment may be applied based on system traffic conditions to avoid the congestion on the freeway and the long queue at the critical ramps. The typical integral algorithms are: Minnesota Zone algorithm and Denver helper ramp algorithm (Lipp et al., 1991) etc.

For the competitive algorithms, several sets of metering rates are computed based on both local and system-wide traffic conditions, and the most restrictive one will be chose as the field rate. The final metering rate is also subject to the further adjustment if there is. The typical integral algorithms are: The Seattle Bottleneck algorithm and the SWARM algorithm (Paesani et al., 1997) etc.

For the integral ramp metering algorithms, there is one or several explicit control objectives linked to the control logic. The objective could be minimizing the travel time, or maximizing the system throughput. Several constraints such as the maximum allowable ramp queue and freeway bottleneck capacity are included in the control logic as well .The metering rates for each ramp is optimized at the system level. Further adjustment may be also applied to the optimal rate as in cooperative and competitive algorithms. The typical integral algorithms are: Fuzzy logic algorithm (Taylor and Meldrum, 1998) etc.

Only one fieldable ramp metering algorithm for each coordinated algorithm category will be presented due to the scope of the thesis.

2. Minnesota Zone Algorithm

Zone algorithm was introduced in the Minneapolis/St. Paul area along I-35 East in 1970. It divides the freeway into zones. Zone is defined as a unidirectional freeway section with length 3 to 6 miles. The upstream boundary of a zone is usually a free-flow area. The downstream boundary is a bottleneck where the demand/capacity ratio is highest in that zone.

The basic concept of the zone algorithm is to balance the volume of the traffic entering and leaving the zone. For each 30-second control interval, the zone conservation equation is used to calculate the sum of the volume from the metered ramps. The zone conservation equation can be expressed as the following:

$$A + U + M + F = X + B + S \qquad (2\text{-}2)$$

Where

A is the upstream mainline volume, a measured variable;

U is the sum of the volumes from non-metered ramps, a measured variable;

M is the sum of the volume from local access metered ramps, a control variable;

F is the sum of the volumes from freeway to freeway metered ramp, a control variable;

X is the sum of exit ramp volumes, a measured variable;

B is the downstream bottleneck capacity, a constant;

S is the space available within the zone for the entering traffic, a variable calculated based on the occupancy of mainline detectors.

The variables in Eq.(2-2) are measured in real time every 30 seconds; but an aggregation of 5 minutes is applied to even out big swings without losing prevailing traffic patterns. It should be noted that the five-minute value for B is normally set to be constant, equaling 1/12 of the highest recorded hourly flow rate from the past 15 days.

Each individual variable in Eq.(2-2) also has a target value (noted by *t*). The zone conservation equation written in the target form is:

$$M_t + F_t = X_t + B_t + S_t - A_t - U_t \qquad (2\text{-}3)$$

The target values are derived from historical data in the past 15 days except S_t , which is set to zero, indicating no space available in target condition.

Each metered ramp is assigned six metering rates and one of them will be selected as the final metering rate. The selection is based on the comparison of the real-time *M*+*F* to a series of thresholds. These thresholds are in the format of $\lambda_1 M_t + \lambda_2 F_t$, wherein λ_1 and λ_2 are the predetermined coefficients. Specifically, for local access ramps, the corresponding coefficients are 1.5, 1.3, 1.1, 0.9, 0.7 and 0.5; while for freeway-to-freeway ramps, they are 1.25, 1.15, 1.05, 1.05, 0.95, 0.85 and 0.75.

The ZONE algorithm adopts an occupancy control mechanism to apply local adjustments to the final metering rate. Each metered ramp is associated with a certain number of downstream freeway detector stations. The occupancy based metering rate for each ramp is based upon the highest occupancy measurement of the corresponding downstream freeway detector stations. The reason for this adjustment is to eliminate the temporary negative effect of an incident. If an incident occurs and produces a temporary bottleneck downstream of a metered ramp, a more restrictive metering rate is needed for such ramps to prevent further breakdown.

Finally, the most restrictive metering rate of the above two would be selected for field implementation.

The evaluation results (Cambridge Systmatics, 2001) show that the ZONE metering strategy is very effective in improving freeway throughout, increasing freeway speed and reducing freeway congestion delays. It also results in excessive ramp delays which led to the public dissatisfaction and shutdown study. The Minnesota Stratified Zone Metering strategy was developed to be consistent with the public concerns about the waiting time on the ramp. Detailed control philosophy will be presented in 2.2.

3. Bottleneck Algorithm

The Seattle Bottleneck algorithm (Jacobsen et al., 1989) was developed by the Washington Department of Transportation (WSDOT), and had been used to control a portion of I-5, north of the Seattle Central Business District.

The basic principle of the bottleneck algorithm is to ensure the flow at any predetermined bottleneck location does not exceed the capacity.

For each ramp, two metering rates will be calculated: the local metering rate and the bottleneck metering rates, the more restrictive of these two will be selected as the final metering rate. The local metering rate is the difference between the downstream capacity and the upstream flow rate. The upstream flow rate is determined based on the historical flow-occupancy relationship. For the bottleneck metering rate, bottleneck locations on the freeway will first be identified. Each bottleneck is associated with an influence zone which may contain one or more metered ramps. Each metered ramp has a weighting factor associated with it. The weighting factor is determined both by the distance of the ramp from the bottleneck and the historical demand on the ramp. In order for the bottleneck algorithm to be invoked, two conditions must be met. The first condition is that the downstream occupancy must be greater than a certain occupancy threshold, indicating that the freeway section is operating above capacity. The second condition is that the freeway section must be storing vehicles, meaning that the sum of the vehicles entering the section and entering via on-ramps must be greater than the sum of the vehicles exiting the section and leaving via off-ramps. If both conditions are met, the metering rate reduction for section i for time interval t+1 is determined as follows:

$$U_{i,t+1} = (q_{i,t}^{IN} + q_{i,t}^{ON}) - (q_{i,t}^{OUT} + q_{i,t}^{OFF}) \tag{2-4}$$

The volume reduction calculated using Eq.(2-4), and the weighting factors, are then used to calculate the bottleneck metering rate reduction for each ramp within the influence zone. The bottleneck metering rate for each ramp is then calculated by subtracting the bottleneck metering rate reduction from the measured on-ramp flow during the previous interval. Because influence zones may overlap, each ramp may have more than one bottleneck metering rate associated with it. In that case, the most restrictive bottleneck metering rate is chosen. Finally, either the most restrictive bottleneck metering rate, or the local metering rate, which ever is more restrictive, is selected.

The bottleneck algorithm uses a two step queue control process. The first part is queue adjustment. When the queue for a ramp reaches a certain length, the metering rate for that ramp is increased slightly. The second step is advance queue override. When the queue reaches its maximum permissible length, the ramp meter is shut off.

The Seattle Bottleneck algorithm is conceptually one of the best heuristic ramp metering algorithms implemented in the field. It is real-time, coordinated, yet logically simple (based on supply-demand and flow conservation) and flexible (only a few adjustable parameters). Field operations with this control also show remarkable improvement in traffic conditions.

4. Fuzzy Logic Algorithm

The fuzzy logic algorithm was first implemented on 126 ramps in the greater Seattle area (Taylor and Meldrum, 1998).

The fuzzy logic algorithm use the rule based logic to incorporate human expertise. In general, it contains three major steps:

1. Fuzzification: In this step, the quantitative inputs (such as occupancy) are translated into a set of classes or linguistic variables (eg. small, medium, big). For each input, the dynamic range, distribution and shape of these fuzzy classes can be tuned.

2. Rule evaluation: This step is the heart of the control strategy. The rules are several if-then statements similar to human expertise. For a given rule premise, a fuzzy class of metering rates is determined (eg. If the downstream occupancy is small, then the metering rate is small). Each rule is associated with a rule weight which reflects the importance of this rule. By adjusting these rule weights, the different performance objectives can be balance. After this step, several rule outcomes (the metering rate classes) will be produced for each on-ramp.

3. Defuzzification: the last step in the fuzzy logic algorithm is to determine a numerical metering rate for each based on different the rule outcomes. This step involves converting a set of metering rates class to a single metering rate. There are several methods for this conversion. The centroid method is used in Seattle fuzzy logic algorithm.

Empirical tests show that this algorithm performs better compared to other algorithms. This algorithm appears to be well suited to ramp metering because of the 5 main reasons.

1. It can handle the inaccurate input information from the loop detector and

2. It can balance the different conflictive objective simultaneously.

3. It also doesn't require the complex system modeling

4. It can be tuned to achieve different performance objectives.

5. It can anticipate the congestion problem and adopt appropriate actions before the congestion happens.

But the performance of this algorithm depends largely on the tuning process, in which the dynamic range limits for the linguistic variable and the rule weight will be refined. This disadvantage makes the algorithm very difficult to configure and limits the practical value in the field.

2.2 Minnesota Stratified Zone Metering Overview

The Stratified Zone Metering (SZM) control strategy is one of competitive ramp metering algorithms. It inherited the basic concept and functionalities of the ZONE algorithm, but it is quite different from the Zone. The transition from the ZONE algorithm to the SZM algorithm signifies the shift of emphasis away from freeway flow to a trade-off between freeway traffic and ramp vehicles. The detailed description of the new ramp control strategy is presented in the following subsections.

2.2.1 Control Philosophy

The control objective of the SZM is to maximize the throughput at bottlenecks during control period subject to the following two constraints:

1. Zone Capacity constraint: The total entering volumes do not exceed the zone capacity

2. Maximum waiting time constraint: The waiting time for each vehicle on any metered on-ramp can't exceed a predetermined maximum during control time period

In reflection of the constraints, there are two levels of design in the SZM algorithm: the zone level design and the ramp level design.

2.2.2 Data Processing

The functionality of SZM control strategy is entirely dependent on real time 30 second occupancy and volume data from the loop detectors in the metro area. Unlike occupancy, volume counts are discrete and when converted to hourly rates these discontinuities blow up resulting in a flow rate function with noise. Hence, all hourly flow rates need to be smoothed by a floating average to capture overall trends. Smoothing in SZM algorithm is done according to the following equation:

$$F_t = F_{t-1} + K*(G_t - F_{t-1}) \qquad (2\text{-}5)$$

Where

t = 1, 2, 3…is the sampling index;

F_t and F_{t-1} are the smoothed flow rates for the current and previous sampling intervals respectively;

G_t is the current unsmoothed hourly flow rate;

K is the demand filter factor (0.15).

The above equation is used in processing the A (the measured upstream mainline volume), X (the total measured off-ramp volume) and U (the total measured non-metered entrance ramp volume) in each zone.

2.2.3 Ramp-level Design

In SZM control strategy, acceptable range of metering rates is determined to be between 240 veh/hr and 1714 veh/hr. The upper bound of this range is set to 1714veh/hr for two consideration: (a) 4 seconds is the minimum reasonable cycles length for a one-lane ramp with one vehicle released pre green; (b) controlled ramps in twin-cities metropolitan area are designed to have two lanes before the ramp meter but transitioned back to one lane before merging into freeway. The lower bound of acceptable metering rates is determined to be 240veh/hr because 15 seconds is the maximum waiting time the first motorist in the ramp queue would tolerate before significantly violation occurs. Metering rates should always be adjusted to this range.

Besides the range of metering rates above, each controlled on- ramp associates with two variables: the ramp demand and the minimum release rate.

2.2.3.1. Ramp Demand

Ramp demand (D) is defined as the hourly flow rate of the vehicles desiring to enter the ramp. It is a key variable in the control logic and is calculated from the smoothed hourly flow rate based on the 30 second volume counts from detectors. On each ramp, typically two types of detectors are deployed to measure the ramp demand in real time; a queue detector at the upstream end of the ramp and a passage detector immediately downstream to the ramp meter. Ramp demand is typically from a queue detector: the 30-second volume measurement is first converted into an hourly flow rate (Q), then the hourly flow rate is smoothed by Eq.(2-5) in which the initial value is set to 240veh/hr (the lower bound of acceptable metering rates).

$$D_t = D_{t-1} + K * (Q_t - D_{t-1}) \tag{2-6}$$

Where

D_t and D_{t-1} are the smoothed ramp demands (veh/hr) for the current and previous sampling intervals respectively;

Q_t is the converted hourly flow rate;

K is the demand filter factor (0.15).

The control logic also includes two special cases to obtain the ramp demand when the queue detector is not available or the measurement is not accurate.

(a) queue detector is not available

When the queue detector is not available or malfunctioning, the passage detector volume counts will be used as to calculate the ramp demand. In order to prevent excessive queuing, *the passage flow rate* (P) converted from the 30 second volume count is increased by a factor to estimate ramp demand. This factor is called the *Passage Compensate factor (* K_c*).* Then the hourly flow rate is smoothed by Eq.(2-7).

$$D_t = D_{t-1} + K_p * (K_c * P_t - F_{t-1}) \tag{2-7}$$

Where

D_t and D_{t-1} are the smoothed ramp demands (veh/hr) for the current and previous sampling intervals respectively;

P_t is the converted hourly flow rate measured from the passage detector;

K_c is the Passage Compensate factor (1.15)

K_p is flow filtering factor(0.20).

(b) The measurement from queue is not accurate

When the ramp queue extends beyond its queue detector, the queue detector no longer gives an accurate measurement of the ramp demand. Such a condition is identified from the high occupancy measurements at the queue detector. Hence, whenever queue detector occupancy exceeds an empirically determined threshold ***(*** $\boldsymbol{O_{threshold}}$*: 25%)*, a 30-second *step increment in ramp demand (Δ: 150 veh/hr)* is added to the smoothed flow rate.

$$D_t = D_{t-1} + \Delta \tag{2-8}$$

Where

D_t and D_{t-1} are the smoothed ramp demands (veh/h) for the current and previous sampling intervals respectively;

Δ is step increment in ramp demand (150veh/hr);

This will result in ramp demand reaching the upper bound of acceptable metering rates in less than 10 sampling period (5minutes).

2.2.3.2 The minimum release rate

The minimum release rate $R_{\min}$ is another variable associated with each controlled on-ramp. The reason of employing $R_{\min}$ is to keep the waiting time on the ramp lower than the maximum allowed for each ramp. To ensure that the last vehicle in the queue on each ramp will not wait more than $T_{\max}$, the *minimum release rate* $R_{\min}$ for each controlled ramp is designed based on Eq.(2-9).

$$R_{\min} = \frac{N}{T_{\max}} \qquad (2\text{-}9)$$

Where

N is the queue size on the ramp (in vehicles)

$T_{\max}$ is the maximum allowed waiting time for each ramp: 2 minutes for freeway to freeway ramp and 4 minutes for local access ramp.

The queue size N in Eq.(2-5) is calculated from an empirical equation based upon *the ramp queue density (Q_D)* and *the ramp queue storage length (Q_L).* Preliminary analysis indicates that this queue size estimation is biased and can result in maximum wait time violations or, more often, in overestimating the queue size resulting in reduced system performance.

$$N = Q_D * Q_L \qquad (2\text{-}10)$$

Where

Q_L is the queue storage length in feet between the ramp meter and the queue detector(a pre-calibrated parameter for each ramp)

Q_D is the queue density estimated using a smoothed metering release rate called the *accumulated release rate* (R_a *).*

$$Q_d = 206.715 - 0.03445 * R_a \qquad (2\text{-}11)$$

Where

R_a is a smoothed metering release rate resulting from the release rate.

If the queue detector is located too far upstream of the ramp (the farther upstream a queue detector's location, the longer the ramp queue storage length would be), *the queue size* (N) would be overestimated. To prevent this, a *queue probability* is introduced as the ratio of *passage flow rate* over the *accumulated release rate*. The final minimum release rate should be the result of Eq.(2-9) multiplied by *queue probability*.

$$\text{Queue\_prob} = \min(1, \frac{P_t}{R_{a,t}}) \tag{2-12}$$

$$R_{\min} = \frac{N}{T_{\max}} * \text{Queue\_prob} \tag{2-13}$$

The minimum release rate will be subject to the two adjustments whenever necessary.

- **Adjustment 1**

The minimum release rate calculated from Eq.(2-9) would be lower than it really should be if the ramp queue backs up over the queue detector. Under such situation, the minimum release rate for the meter must be set to be greater than the ramp's demand so that the queue won't spill spread into adjacent local streets. This rule also applies in case of queue detector malfunction as an extra safeguard to prevent longer waiting time than desired.

$$R_{\min} = \max(R_{\min}, \text{ramp demand}) \tag{2-14}$$

- **Adjustment 2**

Minimum release rate is never set higher than the *passage demand* as long as the queue detector occupancy is below the predetermined threshold.

$$R_{\min} = \min(P_t, R_{\min}) \tag{2-15}$$

Minimum release rate determined as above should be in acceptable range of metering rates between 240veh/hr and 1714veh/hr. All metering rates should be adjusted accordingly if not within this range.

2.2.4 Zone Level Design/Zone Balance

Zone Balance is the central element of Stratified Zone Metering control. A Zone is defined as a continuous stretch of freeway with mainline detector stations as end points. It is identified as a group of consecutive mainline stations with number of stations in a zone varying from two to seven. Thus, the entire freeway segment is divided into groups of zones containing 2, 3…7 consecutive stations. Each such Zone group constitutes a Layer. As there are zones of six different sizes, six layers can be identified one for each zone size. Table 2.1 gives a detailed illustration of layers for the freeway segment of TH169-NB. In other words, all mainline stations on the entire freeway are grouped in sets of two, three, and so on up to seven, and all consecutive zones with same number of stations are said to form a layer. As it can be readily seen, zones overlap with zones of other sizes (refer to figure 2.1). The concept behind choosing the maximum number of stations in a zone to be seven is that it is believed that to alleviate a bottleneck, controlling meters within a distance of 3 miles (stations are approximately half a mile apart) is sufficient for the next control interval of 30 seconds.

This Zone-Layer structure enables SZM to achieve a system wide control. Moreover, unlike its predecessor, identification of potential bottlenecks is not required in the SZM control due to an extensive overlap of zones.

Once the zone-layer structure is built, the next step is to process what is known as the zone balance. Zone balance is an inequality which reflects the zone capacity constraint of SZM: to maintain the number of vehicles entering a zone (input) less than that leaving the zone (output). In terms of the possible inputs and output flows within a given zone, the zone inequality takes the form as:

$$M + A + U \le B + X + S$$

i.e.,

$$M \le B + X + S - A - U \tag{2-16}$$

Where

M is the metered entrance ramp flow (controlled by the Algorithm)

A is the measured upstream mainline flow

U is the total measured unmetered entrance ramp flow

X is the total measured exit ramp flow

B is the downstream mainline capacity

S is the spare capacity on the mainline

Upstream mainline flow A, unmetered entrance ramp U and exit ramp flow X are smoothed based on Eq. (2-5).

The *downstream mainline capacity (B)* is the expected mainline capacity at that location. It is calculated based on the capacity estimate of rightmost lane ***(C_R)*** and the capacity estimate for other lanes ***(C_O).*** Specifically,

$$DownstreamMainlineCapacity(B) = C_R + (NumberOfLanes - 1) * C_O \qquad (2\text{-}17)$$

The *Spare capacity (S)* is introduced to measure the unoccupied capacity in the zone so that the metering rates for the ramp meters that are affected by this zone, can be less restrictive than otherwise. More specifically, spare capacity is calculated as,

$$S = (FullZoneDensity - CurrentZoneDensity) * NumberofLane * Speed \qquad (2\text{-}18)$$

where the *FullZoneDensity* is 32veh/mile/lane and the *CurrentZoneDensity* is estimated from the loop detector measurement.

The process of distributing a zone's maximum allowed metered input M among its metered ramps is known as zone processing. Under Stratified Zone Metering, zones are processed sequentially based on layers; starting from the first zone in the first layer to the last zone in the sixth layer. For each zone in this sequence, the zone processing is done as follows:

i) Calculate the total allowed metered entrance ramp input M into the zone using Eq.(16).

ii) Calculate the sum of the demands from all the metered ramps within the zone

$$\sum_i D = D_1 + D_2 + D_3 + ... + D_n \qquad (2\text{-}19)$$

where *n* is the number of metered ramps within the zone

iii) Propose a weighted release rate *(* R_i^p *)* for each metered ramp, in proportion to the individual ramp demand *(D_i)*

$$R_i^p = M * \left[\frac{D_i}{\sum_i D_i} \right] \forall \; i = 1, 2, 3 \ldots n \qquad (2\text{-}20)$$

iv) All metered ramps in the zone, at this moment, should have *minimum release rate (r_{min}* from Eq.2-8*)*, a *release rate* proposed from a previous zone processing and the new *proposed release rate (* R_i^p from Eq.2-14*)*. The initial value of the *release rate* is set to the *Maximum release rate (**R_{max}** : 1714 veh/hr)* and may get modified as the zones are processed. The *proposed release rate* R_i^p is compared with the *minimum release rate* and *release rate* for each ramp meter and such a comparison results in zone balance. If the *proposed rate* is less than the *minimum release rate*, the zone balance is reduced by the difference while if the *proposed rate* is greater than the *release rate*, the zone balance is increased by the difference.

v) If the zone balance is below zero, each meter that reduced the zone balance gets it finalized release rate as the minimum release rate. Otherwise, the release rates of all the meters that increased the balance remain unchanged. Then the zone is processed again excluding the finalized meters and deducting their respective release rates from the total allowed metered input *M*. This iterative process continues until a zero zone balance is achieved.

The procedures of the zone processing are demonstrated in figure 2.2. This zone processing is done sequentially for all zones in all layers and this finalizes the release rates of all metered ramps as *field rates* for the next 30-second control interval.

2.2.5 Broken Zone Identification and Correction

A zone controlling one or several meters is considered "broken" when the sum of the release rates for the entire zone's meters is less than its M calculated from Eq.(2-16). A zone becomes "broken" if another zone, processed later, makes one or more of its meters more restrictive. A broken zone must be fixed; otherwise the release rates of some meters controlled by it would be more restrictive than necessary.

The broken zones are identified by scanning all the zones in reverse order that the zones are processed, e.g. from last zone in layer 6 to first zone in layer 1. And once a broken zone is identified, all meters affected by this will have their release rate set to 1714veh/h and all the zones are processed again, from first zone in layer 1 to last zone in layer 6. This iterative fixing process will terminate when no more broken zones can be identified.

Table 2.1 Stratified Zone Metering Example (TH-169 NB)

Location Layer	Layer 1	Layer 2	Layer 3	Layer 4	Layer 5	Layer 6
76th St	A Zone1-1	A Zone 2-1	A	A	A	A
Exit ...	X	X	X	X	X	X
Valley View Rd	B A Zone1-2	S A Zone 2-2	S A	S A	S A	S A
... Meter	M	M M	M M	M M	M M	M M
69th St	B A Zone1-3	B S A Zone 2-3	S S A	S S A	S S A	S S A
EB Exit ...	X	X X	X X X	X X X	X X X	X X X
T.H.62	B A Zone1-4	B S A Zone 2-4	B S S A	S S S A	S S S A	S S S A
... EB Meter	M	M M	M M M	M M M M	M M M M	M M M M
... HOV Bypass	U	U U	U U U	U U U U	U U U U	U U U U
... WB Exit	X	X X	X X X	X X X X	X X X X	X X X X
... WB Meter	M	M M	M M M	M M M M	M M M M	M M M M
Exit ...	X	X X	X X X	X X X X	X X X X	X X X X
Bren Rd	B A Zone 1-5	B S A Zone 2-5	B S S A	B S S S A	S S S S A	S S S S A
... Meter	M	M M	M M M	M M M M	M M M M M	M M M M M
... HOV Bypass	U	U U	U U U	U U U U	U U U U U	U U U U U
Exit ...	X	X X	X X X	X X X X	X X X X X	X X X X X
Lincoln Dr	B A Zone 1-6	B S A Zone 2-6	B S S A	B S S S A	B S S S S A	S S S S S
... Meter	M	M M	M M M	M M M M	M M M M M	M M M M M
Exit ...	X	X X	X X X	X X X X	X X X X X	X X X X X
Excelsior Blvd	B A Zone 1-7	B S A Zone 2-7	B S S A	B S S S A	B S S S S	B S S S S
... Meter	M	M M	M M M	M M M M	M M M M	M M M M
... HOV Bypass	U	U U	U U U	U U U U	U U U U	U U U U
Exit to T.H.7	X	X X	X X X	X X X X	X X X X	X X X X
Van Buren Way	B A Zone 1-8	B S A Zone 2-8	B S S A	B S S S	B S S S	B S S S
T.H.7	B A Zone 1-9	B S A Zone2-9	B S S	B S S	B S S	B S S
... Meter	M	M M	M M	M M	M M	M M
36th St	B A Zone 1-10	B S	B S	B S	B S	B S
... Meter	M	M	M	M	M	M
Exit ...	X	X	X	X	X	X
Minnetonka Blvd	B	B	B	B	B	B

A - Upstream station, X - Exit ramp, B - Downstream station, M - Metered ramp, U - Unmetered ramp

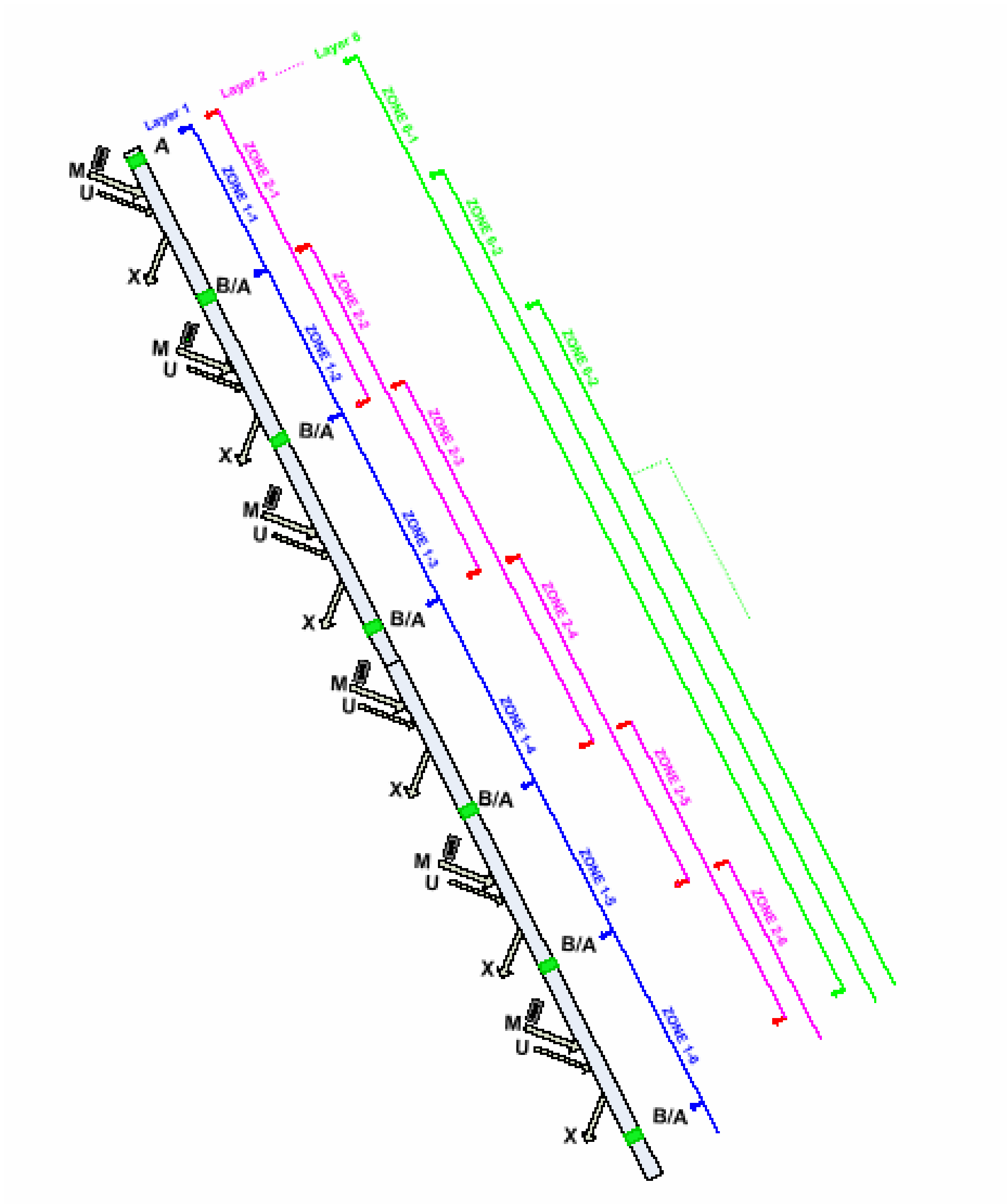


Figure 2.1 Zone-Layer Structure of Stratified Zone Metering

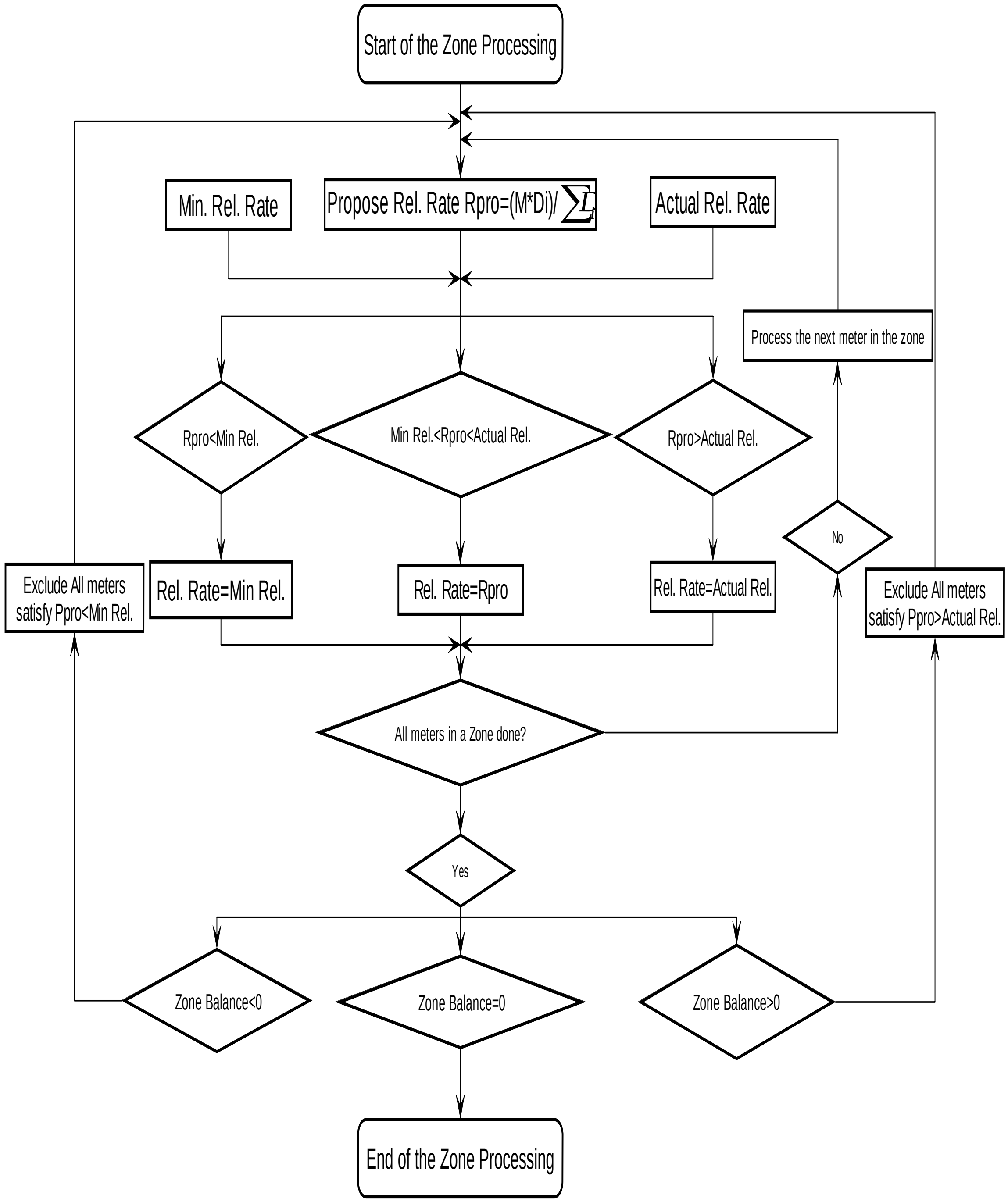


Figure 2.2 Flowchart for Zone Processing

Part II: Improving the Control Logic of the SZM Strategy

Chapter 3 Methodology

Close examination of the performance of the SZM algorithm revealed that soon after the commencement of the control period, the capacity constraint is overridden by the maximum ramp delay one, thereby resulting in freeway congestion and higher system delays as compared to the ZONE strategy.

This is caused not only by high ramp demand and/or the very restrictive (4 minutes) maximum ramp delay constraint, but also because of the unnecessary high minimum release rates determined by the SZM logic to satisfy the ramp delay constraints.

In this chapter, a refined minimum release rate is defined and determined. The refined minimum release rate is more responsive to the actual demand and freeway conditions. The improved logic proposed here is based on the refined minimum release rate and aims to postpone and decrease the onset of the freeway congestion.

3.1 Preliminary Assessment and Motivation

In the current Stratified Zone Metering (SZM) control algorithm, the objective of the Stratified Ramp Control is to regulate the ramp metering rates maximizing the throughput at bottlenecks subject to the following two constraints:

1. Zone Capacity constraint: The total entering volumes do not exceed the zone capacity

2. The maximum waiting time/delay constraint: The waiting time for each vehicle on any metered on-ramp can't exceed a predetermined maximum during control time period

The two constraints are often in conflict with each other; in such case, the maximum delay constraint has higher priority than the zone capacity constraint. This means that the maximum delay constraint has to be satisfied at any time even by violating the zone capacity constraint. In order to satisfy the two control constraints, the SZM logic calculates two metering rates for each ramp at the beginning of every 30-second control interval: the minimum release rate and the proposed release rate.

Minimum Release Rate

The primary public concern was excessive long wait times at the ramps. Consequently, one of the constraints of the SZM is to keep the wait time for each vehicle lower than a predefined maximum. To realize this, a minimum release rate is calculated for each metered ramp based on:

$$R_{\min} = \frac{N}{T_{\max}} \tag{3-1}$$

Where N is the ramp queue size; $T_{\max}$ in is the predetermined maximum waiting time, 240 seconds (4 minutes, 8 control intervals) for local access ramps and 120 seconds (2 minutes, 4 control intervals) for freeway-to-freeway ramps. The rate implemented in the field for each ramp meter must always be greater than or equal to the minimum release rate.

Proposed Release Rate

The second constraint in the SZM logic is the zone capacity one. This constraint ensures that the number of vehicles entering a zone (input) is less than that the one leaving the zone (output). The total metered entrance ramp flow (M) is determined by the zone inequality. A proposed release rate for each ramp within a zone is calculated based on M:

$$R_i^p = M * \left[\frac{D_i}{\sum_i D_i} \right] \forall \; i = 1, 2, 3 \ldots n \tag{3-2}$$

where n is the number of metered ramps in a zone;

D_i is the ramp demand for ramp i;

M is the total metered entrance ramp flow resulting from Eq.(2-16).

The process of distributing the total ramp volume (M) over all metered ramps of a zone is called zone processing. The zone processing is performed for all zones in layers 1 to 7. As zones are overlapping, each ramp is assigned multiple proposed release rates which reflect the zone capacity constraint on their corresponding zones. The final field release rate for each ramp for the next 30-second control interval is the higher between the minimum release rate and the most restrictive proposed release rate. It can be shown that if the sum of the minimum release rates for all ramps in a zone is greater than M, all the field rates of these ramps are set to the minimum release rate. In such case, the zone capacity constraint for this zone is violated since the zone input will be higher than the zone output. As a result of having the ramp delay constraint override the zone capacity one, the freeway flow breaks down rapidly.

In the assessment study, it was observed that during the metering time, most of the time the release rate for each ramp would be set to the minimum release rate and the zone capacity constraint would be violated. This means that shortly after the control starts, the waiting time constraint becomes binding and override the zone capacity constraint, therefore result in serious break down on the freeway mainline and increasing system delays. This is the main reason why the freeway performance in SZM had been greatly reduced as compared to the ZONE alternative. This is caused not only by high ramp demand and/or the very restrictive maximum waiting time constraint, but also because of the unnecessary high minimum release rates determined by the SZM logic to satisfy the ramp delay constraints.

As it can be seen, the $R_{\min}$ in Eq.(3-1) is an average rate during $T_{\max}$ time intervals; there are 8 control intervals for local access ramp and 4 control intervals for freeway to freeway ramp. Suppose this rate is calculated for time t, and is implemented in the field for a period of $T_{\max}$, the last vehicle (Nth vehicle) at time t will wait for exactly $T_{\max}$ on the ramp. As stated above, using this average release rate for each ramp can make the recurring congestion happen early on the freeway. The freeway will soon break down because of the high inputs from the ramps and the freeway performance will be highly jeopardized.

The ramp demand for each ramp varies from time to time during the control period and the ramp demand pattern varies from one ramp to another. So in this case, instead of choosing the average rate as minimum release rate for each ramp, we can also select a more restrictive rate which satisfies the maximum waiting time constraint and does not violate the zone inequality. The freeway performance will be improved due to the restriction of in-flow from the ramps. The ramp performance would not deteriorate much because we could compensate the restriction by releasing more vehicles in the freeway in the future since we have a better traffic conditions on freeway than the SZM algorithm. Following this, the freeway congestion can be delayed in time and the freeway performance will be greatly enhanced.

3.2 Summary of the Improved Control Logic

One of the shortcomings of the current SZM logic is that it is memoryless, i.e., it does not take into account any of its previous decisions. Furthermore, it has no look forward mechanism to postpone the commencement of the dominance of the ramp delay constraint over the zone capacity one. The proposed improvement aims in postponing the commencement of this dominance by taking into account recent past decisions and determining more restrictive minimum release rates which keep the zone capacity constraint satisfied. However, these more restrictive minimum release rates can lead to temporary fast growth of ramp queues. Due to the very limited queue storage space on freeway-to-freeway ramps the proposed improvement only applies to the local access ramps. Specifically, for each local access ramp, the proposed improvement takes into account the decisions made during the past eight 30-second control intervals (eight is the maximum waiting time for local access ramps divided by 30 seconds) and apportions the maximum individual vehicle delay along the next 8 consecutive control intervals in a manner that delays the onset of freeway congestion. The latter is achieved by tracking the accumulated delay of the last vehicle in the queue at the end of each of the preceding 8 control intervals. This is accomplished whenever minimum release rates are to be determined for each ramp in a zone sequentially within each layer. The logic can be summarized by the following steps. Detailed description of this logic and its implementation are too are presented next.

Step 1: Obtain current measurements of volume and occupancy at time t from the related freeway and ramp detectors.

Step 2: Denote N_i as the last vehicle in the queue at each time increment t-i, where i=1, 2…8. For each N_i, the following equation is used to calculate the average minimum release rate.

$$\frac{X_i}{T_{\max} - W_i}, \text{ i=1, 2 ….8} \qquad (3\text{-}3)$$

Where

X_i is the current position of vehicle N_i

$T_{\max}$ is the maximum allowable waiting time

W_i is the accumulated wait time of vehicle N_i up to the current time. W_i is the difference between the arrival time of vehicle N_i and the current time.

Step 3: Select the highest value of the 8 rates calculated in step 2 as the overall average minimum release rate for this ramp. This rate, if enforced, will even out large swings and substantially reduce the ramp queue.

Step 4: For each vehicle N_i from the equation below calculate the minimum release rate so that the remaining waiting time of N_i in the ramp is $T_{\max}$ -W_i .

$$\frac{[X_i - ((\frac{T_{\max} - W_i}{30}) - 1) * R_{\max} * \frac{30}{3600})]}{\frac{30}{3600}}, \text{ i=1, 2 ….8} \qquad (3\text{-}4)$$

where $R_{\max}$ is the maximum allowable metering rate for each ramp (1714veh/h)

These rates are more restrictive than the corresponding average minimum release rates of step 2.

Step 5: Select the highest value of the 8 rates calculated in step 4. If enforced, this rate will improve the freeway mainline traffic conditions but result in faster growth of the ramp queue.

Step 6: Selection of the final minimum release rate.

If the summation of the average minimum release rates selected in step 3 for all ramps in a zone is less than M (the total allowable entrance ramp flow), the average minimum release rates will be selected as the final minimum release rates for these ramps (This implies that the zone's capacity constraint can be satisfied by the average minimum release rates). If not, the rates of step 5 will be selected as the final minimum release rates for these ramps to keep the zone capacity constraint satisfied.

The final minimum release rates from step 6 are then be fed into the zone processing procedure. The zone processing is accomplished sequentially for all zones in all layers and finalizes the release rates of all metered ramps as field rates for the next 30-second control interval.

3.3 Detailed Description of the Improved Control Logic

The improvement introduced here aims to address the problem of how to enhance the mainline performance during peak while keeping the ramp delays below the desired thresholds. The improvement is based on the principle of postponing and decreasing congestion.

To satisfy the waiting time constraint and keep the zone capacity constraint alive as long as possible, the refined minimum release rate for each ramp at current control interval needs to be determined This rate is more restrictive than the minimum release rate produced from Eq.(3-1) and it is only active when the zone inequality is potentially violated by the ones determined from Eq.(3-1). It should also guarantee that the last vehicle in the ramp queue at any interval will wait for no more than the predetermined threshold. In order to find this rate, the last vehicle in the ramp queue at any time has to be tracked until it enters the freeway. At the beginning of each control interval, we need to check the waiting time of the tracked vehicle and the additional time it needs to enter the freeway so that the total waiting time for this vehicle will not exceed the predetermined threshold. The refined minimum release rate is determined by the additional time it takes for the vehicle to merge into the freeway. By using this rate, the waiting time constraint can be satisfied and the mainline performance will be improved. Although this will end up with more vehicles on the ramps, the ramp delay incurred by those vehicles can be compensated by releasing more vehicles into the freeway in future control periods. Therefore, overall, the ramp performance will not deteriorate that much. The refined minimum release rate can lead to temporary fast growth of ramp queue. Due to the very limited queue storage space on freeway-to-freeway ramps, the proposed improvement only applies to the local access ramps.

3.3.1 Notation

The notation conventions below are used in explaining the improvement to the SZM control. This improvement only utilizes the metered local access ramps.

i: index of the metered local access ramps;

j, t: index of the control interval during the metering time

J: the total number of control intervals during the metering time

k: index of the metered freeway to freeway ramps

l: free index

m: index of the zone in a layer

n: index of layer

$I_{m,n}$: the total number of metered local access ramps in zone m within layer n

$K_{m,n}$: the total number of metered freeway to freeway ramps in zone m within layer n

$M_{j,m,n}$: the total metered local access ramp volume for zone m within layer n at the beginning of the j^{th} control interval (in veh/hr)

$F_{j,m,n}$: the total metered freeway to freeway ramp volume for zone m within layer n at the beginning of the j^{th} control interval (in veh/hr)

$A_{j,m,n}$: the measured upstream mainline volume for zone m within layer n at the beginning of the j^{th} control interval (in veh/hr)

$B_{m,n}$: downstream bottleneck capacity for zone m within layer n (in veh/hr)

$X_{j,m,n}$: the total exit ramp volumes for zone m within layer n at the beginning of the j^{th} control interval (in veh/hr)

$S_{j,m,n}$: spare capacity (space available within the zone) for zone m within layer n at the beginning of the j^{th} control interval (in veh/hr)

Q_j^i : the queue size at the beginning of the j^{th} control interval on ramp *i*.(in vehicles)

V_t^i : the last vehicle in the queue at the beginning of the t^{th} control interval on ramp i

$N_{t,j}^i$: The number of vehicles before V_t^i including V_t^i itself in the queue at the beginning of the j^{th} control interval (t≤j). If V_t^i has already merged into freeway before the j^{th} control interval, then $N_{t,j}^i$ =0. It is obvious that $0 \leq N_{t,j}^i \leq Q_j^i$. (in vehicles)

WT_t^i : total waiting time for V_t^i on ramp *i* (depends on the context, in seconds or control intervals)

$W_{t,j}^i$: before the j^{th} control interval, the time V_t^i has be waiting on the ramp *i* (in seconds or control intervals)

$WE_{t,j}^i$: the expected waiting time for V_t^i during and after the j^{th} control interval. If $WE_{t,j}^i \leq 0$, this means V_t^i has already merged into freeway before the j^{th} control interval (in seconds or control intervals)

$R_{avg,j}^i$: the average minimum release rate for ramp *i* in the j^{th} control interval, which is calculated from Eq.(21) for ramp *i* at the j^{th} control interval (in veh/hr)

$R_{avg,t,j}^i$: the average minimum release rate during and after control interval j needed for vehicle V_t^i to wait for no more than $T_{\max}$ amount of time. For all t≤j, $R_{avg,t,j}^i = N_{t,j}^i \ /(T_{\max} - W_{t,j}^i)$. When t=j, $W_{t,j}^i$ =0, $R_{avg,j,j}^i = Q_j^i \ / T_{\max} = R_{avg,j}^i$ (in veh/hr)

$R_{ref,j}^i$: the refined minimum release rate for ramp *i* in the j^{th} control interval, which the smallest minimum release rate needed for ramp *i* at the j^{th} control interval (in veh/hr)

$R^i_{field,j}$: the field rate for ramp *i* in the *j*th control interval (in veh/hr)

R_{MIN} : lower bound of the acceptable metering rate: 240veh/hr.

R_{MAX} : upper bound of the acceptable metering rate: 1714veh/hr

$R^i_{acc,j}$: the acceptable release rate for ramp *i* in the *j*th control interval (in veh/hr)

$MIN^i_{t,j}$: the least release rate needed for ramp *i* in the *j*th control interval to ensure V^i_t to wait for no more than $T_{\max}$, t=j-7,…., j. The calculation of these 8 release rates will be introduced shortly.

3.3.2 Determination of the Refined Minimum Release Rate

Suppose the current control interval is the *j*th control interval, finding $R^i_{ref,j}$ is equivalent to minimize $R^i_{acc,j}$:

$R^i_{ref,j}$ =min ($R^i_{acc,j}$)

Subject to:

1. $0 \le WT^i_t = W^i_{t,j} + WE^i_{t,j} \le T_{\max}$ (240 seconds or 8 control intervals), $1 \le t \le J$.

2. $R_{MIN} \le R^i_{acc,j} \le R_{MAX}$

Corollary 1:

For the constraint 1 above, the current control interval is the *j*th control interval. For any t>j, WT^i_t will be ensured by $R^i_{ref,t}$ to be less than $T_{\max}$. In such cases, $R^i_{ref,j}$ has nothing to do with WT^i_t as long as the constraint 2 is satisfied.

Corollary 2:

For any t that satisfies $1 \le t \le j-8$(suppose j>=9 for now), the vehicle V^i_t must have merged into freeway before the beginning of the *j*th control interval.(If V^i_t is still on ramp i, $W^i_{t,j}$ will be j-t number of control intervals. Since j-t>=j-(j-8)=8, so $W_{t,j}^{i}$ >=8 control intervals. This will contradict to $0 \le WT_t^i = W_{t,j}^i + WE_{t,j}^i \le T_{\max}$).

Update the constraint 1 above, the minimization problem has changed to:

$R_{ref,j}^{i}$ =min ($R_{acc,j}^{i}$)

Subject to:

1. $0 \le WT_t^i = W_{t,j}^i + WE_{t,j}^i \le T_{\max}$ (240 seconds or 8 control intervals), j-7$\le$t$\le$j.

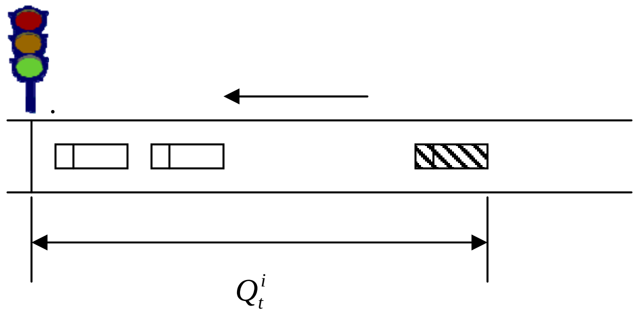
2. $R_{MIN} \le R_{acc,j}^{i} \le R_{MAX}$

If j$\le$7, then constraint 1 changes to:

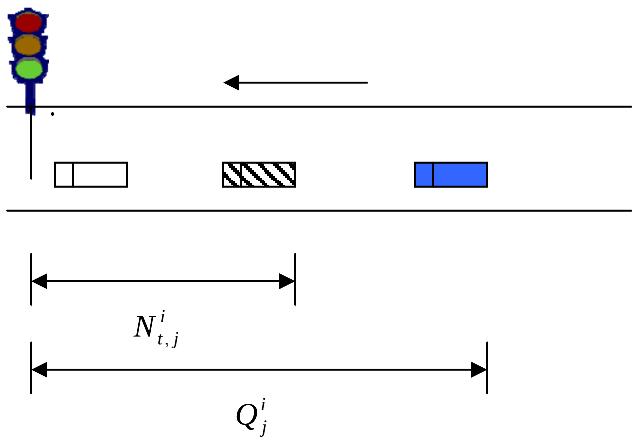
1a. $0 \le WT_t^i = W_{t,j}^i + WE_{t,j}^i \le T_{\max}$ (240 seconds or 8 control intervals), 1$\le$t$\le$j.

For convenience, now suppose j>7. $W_{t,j}^{i}$ is the waiting time V_t^i has spent on the ramp before the *j*th control interval. Vehicle V_t^i appeared on ramp *i* at the beginning of the *t*th control interval. So if it is still on ramp i, $W_{t,j}^{i}$ will be j-t control intervals. If it already merged into freeway before the *j*th control interval, the corresponding inequality constraint will be eliminated from constraint 1. We still consider the general case which assumes this vehicle is still on the ramp. So $WE_{t,j}^i = WT_t^i - W_{t,j}^i \le$ 8-(j-t) control intervals, which is the expected waiting time for V_t^i during and after the *j*th control interval. Here $WE_{t,j}^i$ >0, because this vehicle is still on ramp i. In order to connect $WE_{t,j}^i$ with $R_{acc,j}^{i}$, $N_{t,j}^{i}$ has to be determined beforehand because $R_{acc,j}^{i}$ is the metering rate that can make $N_{t,j}^{i}$ number of vehicles merge into freeway in $WE_{t,j}^i$ amount of time. $N_{t,j}^{i}$ (The number of vehicles before V_t^i including V_t^i itself in the queue at the beginning of the *j*th control interval) can be calculated based on arrival/departure information on ramp *i* from the *t*th to the *j*th control interval.

At the beginning of the *t*th control interval, V_t^i is the last vehicle in the queue of ramp *i* and the corresponding queue size is Q_t^i:



At the beginning of the j^{th} control interval, there are $N_{t,j}^i$ -1 number of vehicles in front of V_t^i in the queue of the j^{th} control interval:



The difference between Q_t^i and $N_{t,j}^i$ (Q_t^i - $N_{t,j}^i$) is the number of vehicles that have merged into freeway from ramp *i* from the t^{th} to the j^{th} control interval. Thus:

$$Q_t^i - N_{t,j}^i = \sum_{l=t}^{j-1} R_{field,l}^i \ *30/3600 \text{ (in vehicles)}$$

So $N_{t,j}^i = Q_t^i - (\sum_{l=t}^{j-1} R_{field,l}^i \ *30/3600)$

$R_{acc,j}^i$ is the metering rate which can make $N_{t,j}^i$ number of vehicles merge into freeway in $WE_{t,j}^i$ (=WT_t^i - $W_{t,j}^i \leq$ 8-j+t) amount of time. The minimum $R_{acc,j}^i$ is achieved when making exact $N_{t,j}^i$ vehicles enter the freeway in the largest amount of available time. This means that the necessary condition for $R_{ref,j}^i = R_{acc,j}^i$ is $WE_{t,j}^i$ =8-j+t for some t. We don't know which t can make the $R_{acc,j}^i$ minimized. But for any control interval t, if we make the exact $N_{t,j}^i$ number of vehicles merge into freeway in 8-j+t control intervals starting from the j^{th} control interval from ramp i, the $R^i_{acc,j}$ can only be minimized when the release rates for other control intervals except the j^{th} reach R_{MAX} .

So the necessary condition changes to:

$$\text{For some t, } R^i_{ref,j} = \frac{[N^i_{t,j} - (\sum_{l=1}^{7-j+t} R_{\max} * \frac{30}{3600})]}{\frac{30}{3600}} \text{ ; j-7} \le \text{t} \le \text{j} \qquad (3\text{-}5)$$

Define $MIN^i_{t,j}$ =[$N^i_{t,j}$ -($\sum_{l=1}^{7-j+t} R_{\max}$ *30/3600)]/(30/3600) as the least release rate needed for ramp *i* in the j^{th} control interval to ensure that V^i_t wait for no more than $T_{\max}$.(j-7 ≤ t ≤ j). Because for some t, the $R^i_{ref,j} = MIN^i_{t,j}$ and $R^i_{ref,j}$ is the minimum of $R^i_{acc,j}$, $R^i_{ref,j}$ will be the maximum of the $MIN^i_{t,j}$ (j-7 ≤ t ≤ j). ($MIN^i_{t,j}$ is not necessarily to be monotonically increasing or decreasing with respect to t) If for some t, vehicle V^i_t has already merged into freeway, then $MIN^i_{t,j}$ will be $R_{\min}$. For any t, $MIN^i_{t,j}$ has to be restricted to the range of [$R_{\min}$, $R_{\max}$].

3.3.3 Development of the Improved Control Logic

The idea to improve the SZM is to use the refined minimum release rates when the when the using average minimum release rates will violate the zone capacity constraint so that the freeway breakdown could be avoid temporarily. In the future, these ramps using the refined minimum release rates should increase the minimum release rates since these ramps have more cars left on the ramps compared to the SZM strategy.

Step 1 of the improved algorithm is to divide the control period into J control intervals. After the ramp control takes place, if the zone inequality is satisfied for all the zones within different layers, keep the original ramp control algorithm. The improvement algorithm will only be evoked when the zone inequality is not satisfied for any zone within any layer.

Suppose this condition has been met in zone m in layer n at the jth control interval. This means for the local access ramps in such zone:

$$\sum_i R^i_{avg,j} > M_{j,m,n} = B_{j,m,n} + X_{j,m,n} + S_{j,m,n} - A_{j,m,n} - U_{j,m,n} - F_{j,m,n} \text{, i = 1,}\ldots\text{, } I_{m,n} \text{ ;}$$

$R^i_{avg,j}$ is the average minimum release rate for ramp *i* in the j^{th} control interval. At the same time, $N^i_{t,j}$ (The number of vehicles before V^i_t including V^i_t itself in the queue at the beginning of the j^{th} control interval) and $MIN^i_{t,j}$ (the least release rate needed for ramp *i* in the j^{th} control interval to ensure V^i_t to wait for no more than $T_{\max}$) will be updated for each t in [j-7,j].

Step 2 is to update $R^i_{ref,j}$ based on $N^i_{t,j}$ and $MIN^i_{t,j}$ for i=1,…,$I_{m,n}$ .The $R^i_{avg,j}$ will be overridden by $R^i_{ref,j}$ for ramp i. These refined ramp minimum release rates need to be updated for ramp *i* in all the zones within any layer. This overriding will make the zone inequality satisfied; therefore the actual congestion will be avoided for one control interval.

Step 3 is to propose the new weighted release rate *(* R^p_i *)* for each metered ramp based on these new minimums, in proportion to the individual ramp demand *(*D_i*)*.

Step 4 is to compare R^p_i with the *minimum release rate* and *release rate* for each ramp meter. Such a comparison will reflect the maximum waiting time constraint and the zone balance.

Step 3 and Step 4 are the zone processing steps. These steps will be done sequentially for all zones in all layers. This finalizes the release rates of all metered ramps as *field rates* for the j^{th} control interval. The ramp meters are grouped into two categories according to the relationships between their field rates and the average minimum release rate. The first category is the ramps with $R^i_{avg,j} \le R^i_{field,j}$, these ramps are the normal ramps. The second category is the ramps with $R^i_{avg,j} > R^i_{field,j}$. These ramps are called the delayed ramps because these ramps release fewer vehicles into freeway compared to the SZM algorithm. The freeway mainline performance is improved because these delayed ramps have switched the delay from the mainline to themselves.

Step 5 is to update $R^i_{avg,t,j+1}$ for $(j+1)^{th}$ control interval for the delayed ramps for t:

j-6 ≤ t ≤ j. For the delayed ramps, we have to try to release more vehicles into the freeway in the future control intervals because we have kept more vehicles on these ramps compared to using the SZM algorithm. So the maximum of these $R^i_{avg,t,j+1}$ (j-6 ≤ t ≤ j) and $R^i_{avg,j+1}$ calculated from Eq.(3-1) at the beginning of the $(j+1)^{th}$ control interval will be used as $R^i_{avg,j+1}$ for the delayed ramps. Therefore, the delayed ramp might release more vehicles into the freeway in future control intervals. Although we keep more vehicles on the ramps at first, we compensate this kind of ramp delay by releasing more vehicles into freeway in the future control interval. Overall, the ramp MOEs will not be decreased a lot.

Step 2 to Step 5 will be repeated for all the control intervals after the j^{th} during metering period.

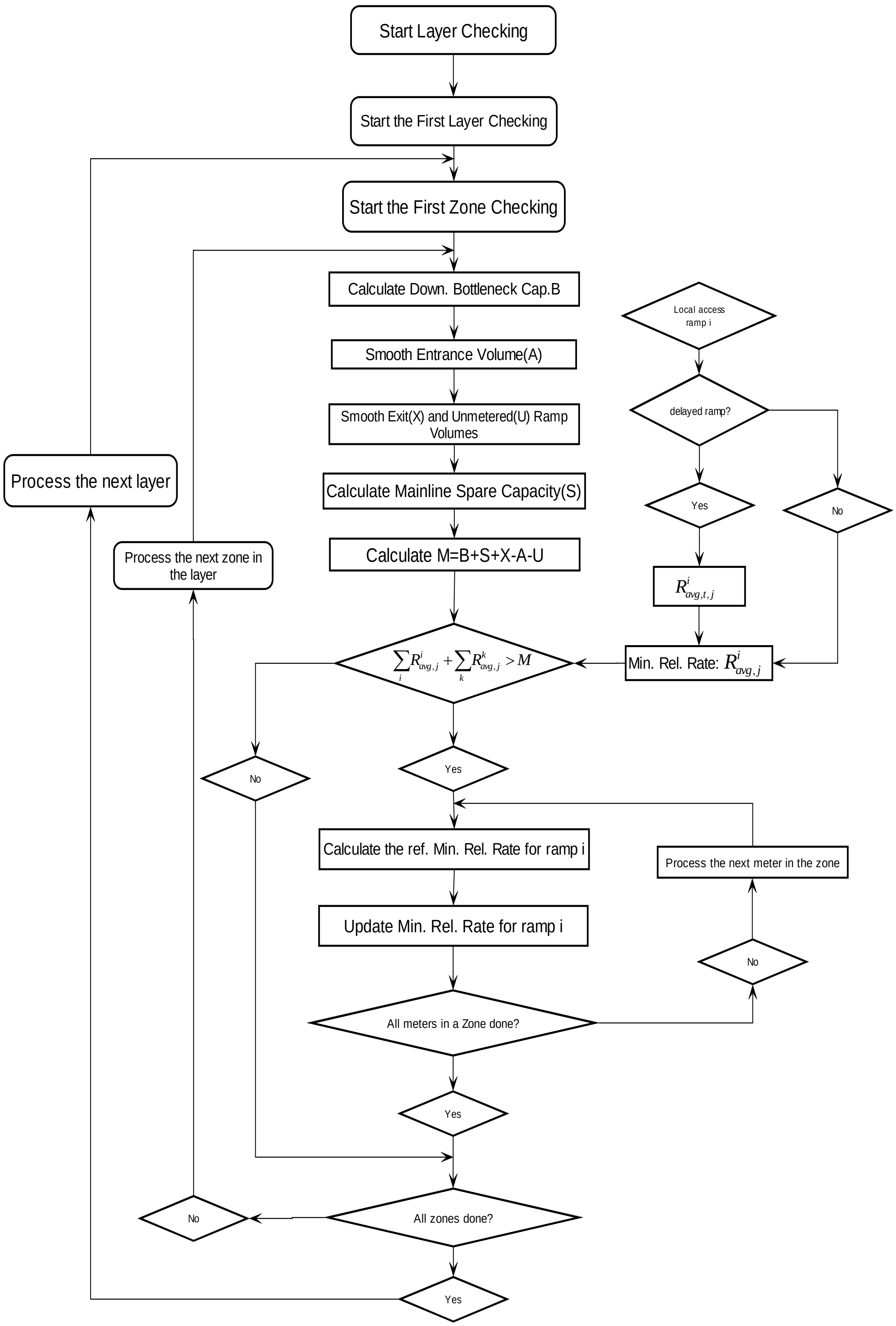


Figure 3.1 Flowchart for Step 1 and 2 of the improved SZM

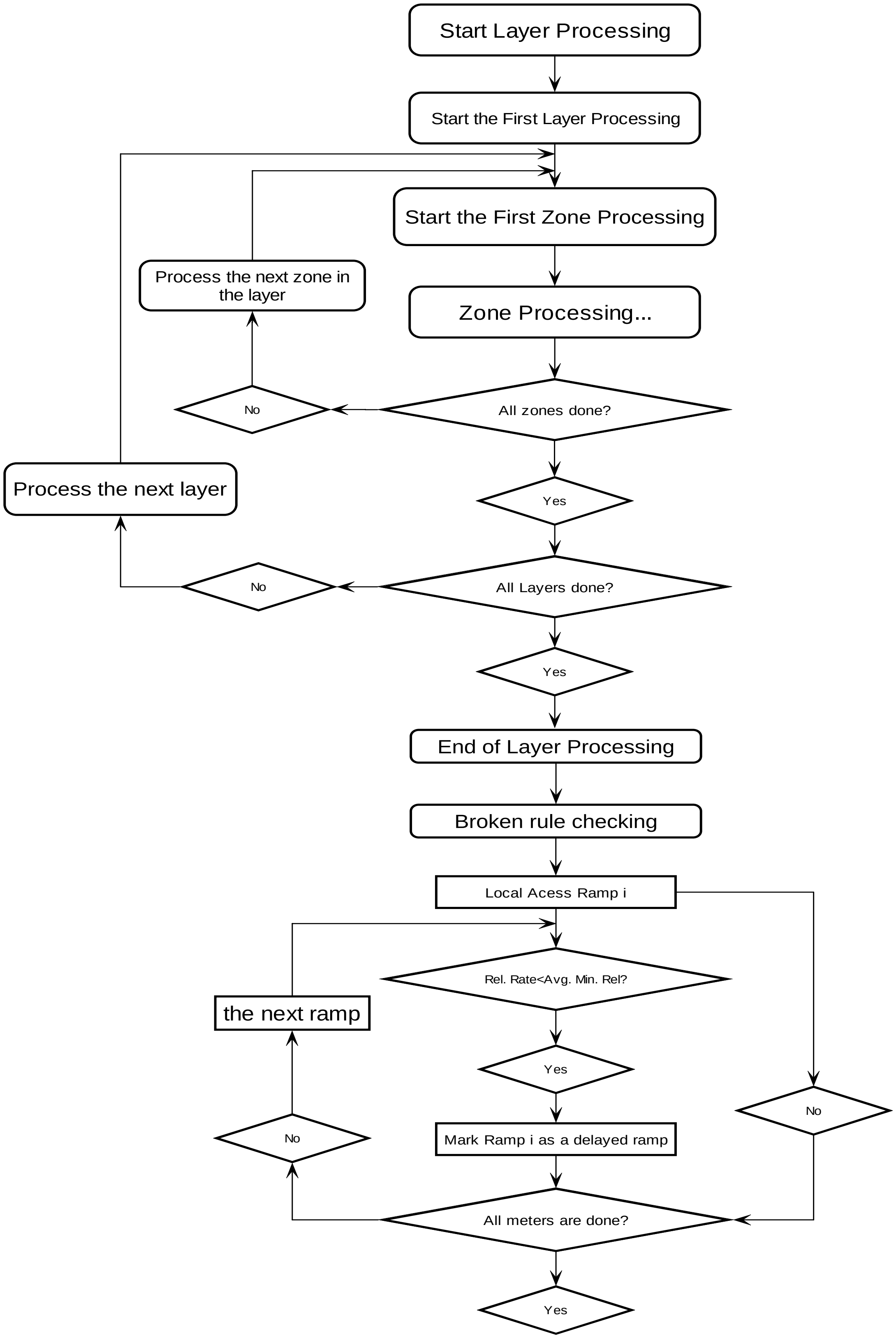


Figure 3.2 Flowchart for Step 3, 4 and 5 of the improved SZM

Chapter 4 Evaluation Methodology

One of the most important objectives of this research is testing and assessing the effectiveness of improved SZM strategy. Due to the cost, time and safety considerations, the proposed improvements could not be directly implemented and evaluated in the field. Thus the only widely acceptable feasible alternative was simulation, which was adopted here.

Selection of a suitable simulator was therefore essential for evaluating the proposed improvements. In this study, the AIMSUN simulator was used based on earlier experience, suitability, reputation as well as its proven record in testing traffic control and management systems including ramp metering. The original and enhanced SZM ramp control strategies were implemented and integrated with AIMSUN through a previously developed Control Plan Interface.

Once the simulator was selected, the appropriate measures of effectiveness (MOEs) for ramp metering evaluation should be identified and extracted. The test sites for carrying out the simulation experiments were selected based on certain criteria. The traffic data necessary for building realistic simulation models was collected for the test sites selected. Finally the simulation model was calibrated to obtain a good match between actual and simulated fundamental measurements (e.g., Flow and Speed) by fine tuning the global and local parameters of the microscopic simulator.

4.1 AIMSUN Overview

AIMSUN was selected to test and assess the effectiveness of the proposed improvement because it can model complex geometry of realistic large-scale urban networks and provide interface for integrating user-defined freeway control strategies. A description of the simulator can be found in Barceló et al.(1994).

AIMSUN (Advanced Interactive Microscopic Simulator for Urban and Non-Urban Networks) is a microscopic simulator which is designed as a tool for traffic analysis. AIMSUN is capable of dealing with different traffic network and has been proven to be very beneficial for traffic engineers to design and assess the traffic system. AIMSUN is a part of the GETRAM Simulation Environment (Generic Environment for Traffic Analysis and Modeling). GETRAM Simulation Environment includes AIMSUN simulator, TEDI (a traffic network graphical editor), a network database, a module for storing results and an API (Application Programming Interface) and the GETRAM Extension. The API can interact with the traffic assignment models and other simulation models.

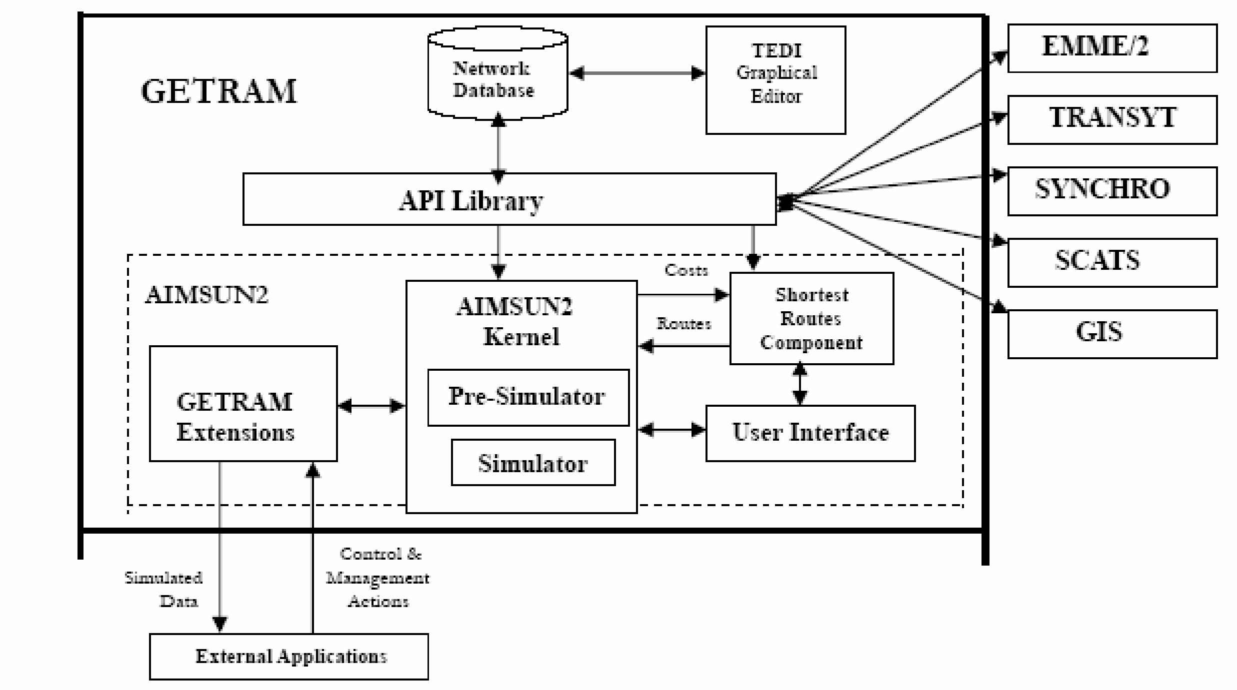


Figure 4.1 Conceptual structure of GETRAM/AIMSUN

AIMSUN simulator consists of 5 modules: User Interface, Pre-Simulator Module, Simulation Module, Shortest Route Computation Module and GETRAM Extensions module. GETRAM Extension is a special module which is a DLL Library of functions. These functions allow the user to access and manipulate the AIMSUN internal data during the simulation period. Therefore, this module can be used to emulate the external sophisticated control applications such as traffic sign, VMS's and ramp metering control logic. Figure 4.1 presents an overall functional structure of AIMSUN and its integration with GETRAM Environment.

AIMSUN Traffic Modeling

In AIMSUN, the simulation time is split into small time intervals called simulation cycles or simulation steps. Each simulation step, the position and speed of every vehicle in the system in updated according to vehicle movement models. Once such information of all vehicles have been updated, vehicles scheduled to arrive during this cycle will be introduced into the system and the next vehicle arrival times to the system will be generated. The vehicle arrival times are based on the travel demand and certain distribution. The vehicle movement models in AIMSUN include car following model, lane changing model and other special models. The car following model implemented in AIMSUN is an ad hoc development of the Gipps model (Gipps, 1986). The lane changing behavior in AIMSUM is also a development of the Gipps model. It is modeled as a decision process which approximates the driver's behavior by analyzing the necessity of a lane change, the desirability of a lane change and the feasibility for a lane change in sequence. The feasibility for a lance change is checked by a simple gap acceptance model which examines whether a gap is acceptable or not. Other special models contain the on-ramp model, off-ramp model and Look Ahead model. On-ramp model is a special lane changing model which applies to vehicles trying to merging into the freeway mainline from an on-ramp lane. The off-ramp model is a special lane changing model which applies to vehicle attempting to exit the freeway mainline via an off-ramp. The Look Ahead model can capture the next two lane changing motivations in two sections for a vehicle; therefore it guarantees this vehicle will reach the appropriate lane on time.

AIMSUN can also model the traffic control and management. AIMSUN takes into account different types of traffic control: traffic signals, give-away signs (yield and stop signs) and ramp metering. On the other hand, AIMSUN also can implement traffic management actions such as the Variable Message Signs (VMS) which can affect the traffic behavior. For the traffic signals and ramp metering, certain traffic control plan has to be specified to define the Signal Group definition, the timing for each phase and the timings or flows for all ramp meters.

Simulation Experiment I/O

The inputs required by the AIMSUN are geometry layout (with or without traffic control) and traffic demand data. The geometry layout is composed of a set of sections which contain different traffic measures. The sections are connected through nodes. The traffic control includes traffic signals, traffic signs and ramp metering. The traffic demand data can be defined in either the traffic flows at each section or the O/D matrix. The traffic flow is favorable in this study since the O/D information is often unavailable. The traffic demand data of flow includes the traffic composition data, flow at each input sections and turning proportion at each sections.

The output AIMSUN provides includes the animation of the simulation, statistical results and detection data. The statistical result provides the statistical measures such as Flow, Speed, Density, Travel and Delay time. These traffic measures can be aggregated at different levels defined by the user: for the whole system, for each section, etc. The detection data provides the detector measures for each detection interval specified by the user. The statistical results and the detection data can be either exported to the ASCII file or to a database.

AIMSUN like other simulators does not provide a unique output for a given experiment. The emulation of a complex traffic system involved a lot of randomness. Each run of the same simulation scenario is called a replication. A replication only provides a possible behavior for the modeled system; therefore it is only one of the feasible results of the model. The final results should be obtained through several replications.

The random seed is the only parameter related to the replication. This parameter must be an integer number and it used as the initial seed for the pseudo-random number generator. The pseudo-random generator is used to sample real numbers uniformly distributed between 0 and 1. A simulation experiment should consist of a set of replications of the same scenarios with the same traffic network, traffic demand, traffic control plan and a set of modeling parameters. Different seeds will be used for each of the replication; therefore the average results of several simulation runs can reflect average traffic condition of a specific scenario. The former study has shown that 10 replications are enough to keep the average simulation result within the 95% CI of the sample mean.

GETRAM Extension

GETRAM Extension is a special module in AIMSUN. It enables the external application to communicate with GETRAM/AIMSUN. It provides several functions which allow the external application to access the internal data of AIMSUN during simulation run time. The external application (user defined) then feed this data to in a control plan and decides the appropriate control actions will be applied to the system. Finally the external application sends the control actions to the AIMSUN model, which will emulate the corresponding operation through certain model components such as traffic signals, VMS's and ramp meters. The process of information exchange between AIMSUN and the external application is shown in figure 4.2.

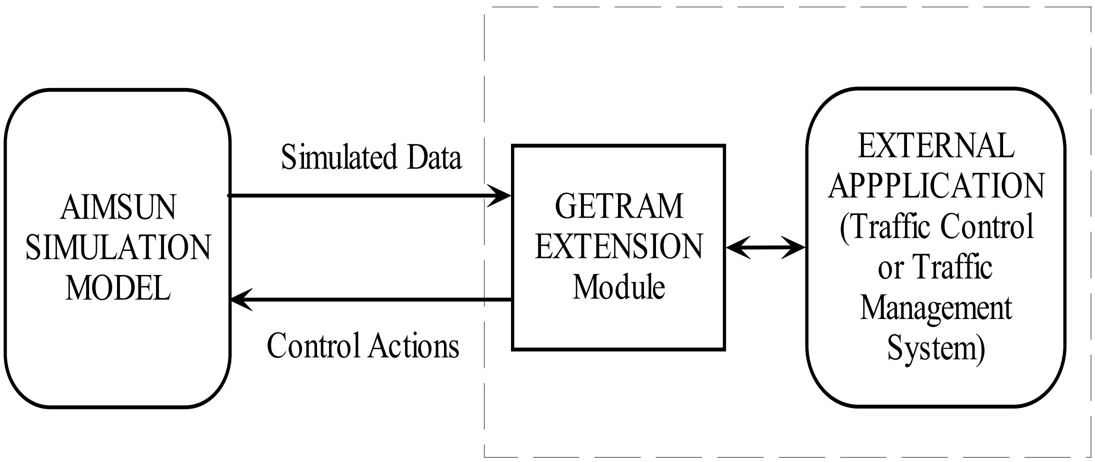


Figure 4.2 Communication between AIMSUN and the external application

In order to communicate with the AIMSUN Simulation models, the Getram Extension defines six high level functions. GetExtLoad, GetExtInit, GetExtManage, GetExtPostManage, GetExtFinish and GetExtUnLoad.

(1) *GetExtLoad():* It is called when the getram extension is loaded by AIMSUN
(2) *GetExtInit():* It is called when AIMSUN starts the simulation and can be used to initialize whatever the getram extension needs
(3) *GetExtManage* (***floa****t time,* ***float*** *timeSta, float timeTrans,* ***float*** *acicle*): This is called every simulation step at the beginning of the cycle, and can be used to request detector measures, vehicle information and interact with junctions, metering and VMS in order to implement the control and management policy. This function takes four parameters: time, timeSta, timeTrans and acicle.
(4) *GetExtPostManage (****floa****t time,* ***float*** *timeSta, float timeTrans,* ***float*** *acicle):* This function has the similar form and functionalities as the function *GetExtManage* (***floa****t time,* ***float*** *timeSta, float timeTrans,* ***float*** *acicle*). But it is called in every simulation step at the end of the cycle.
(5) *GetExtFinish():* It is called when AIMSUN finish the simulation and can be used to finish whatever the getram extension needs
(6) *GetExtUnLoad():* It is called when the Getram extension is unloaded by AIMSUN.

The Getram Extension module can be implemented using a DLL written in C++ or a script file in Python. These five functions are incorporated in the implementation file GetExt.cxx. GetExt.cxx and certain header and library files are compiled together using any C++ complier to form the DLL (Dynamic Link Library) file. The user can modify the file GetExt.cxx , fill in the functions GetExtLoad(), GetExtIni(), GetExtManage(….), GetExtPostManage(….), GetExtFinish() and GetExtUnload() and add some other files if necessary.

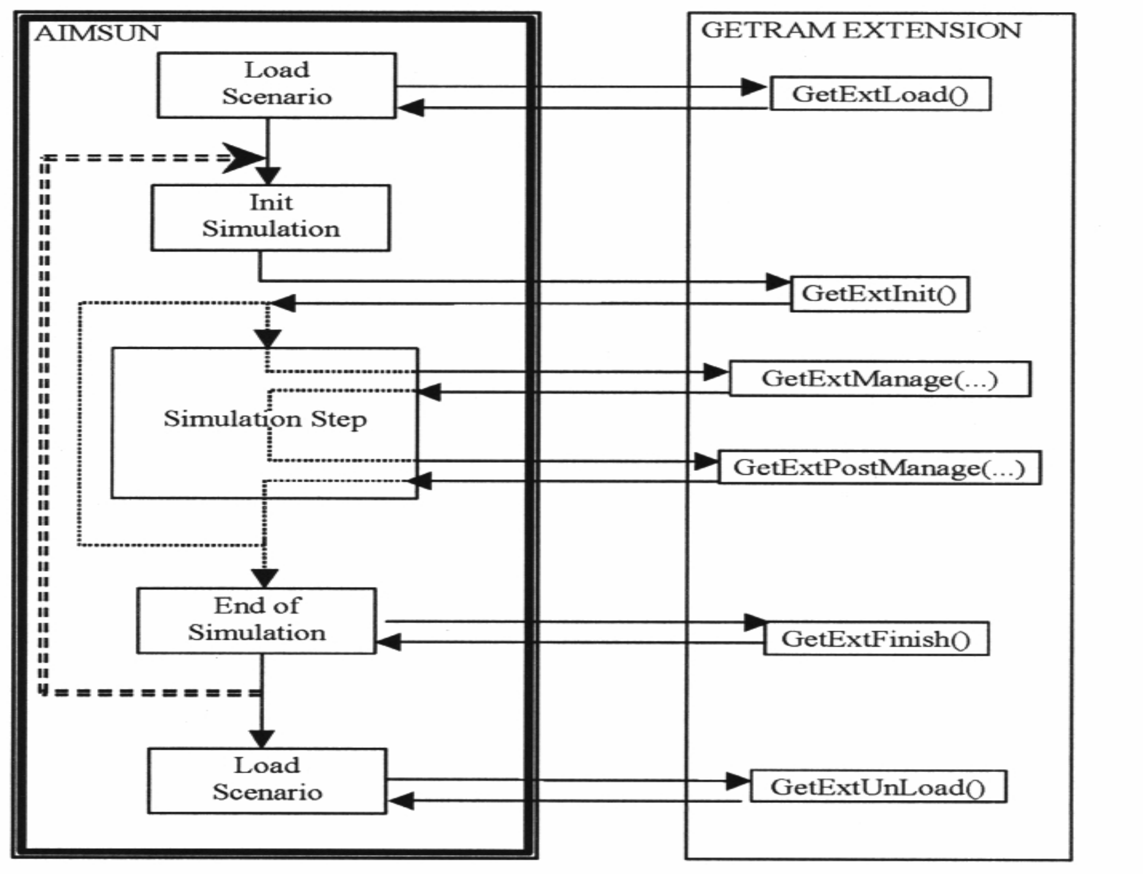


Figure 4.3 Interaction between the extension functions and the simulation model

Figure 4.3 graphically depicts the interaction between the six high level GETRAM extension functions and AIMSUN simulation model.

4.2 Emulation of SZM and Improved SZM Ramp Control

In order to emulate the SZM and improved SZM ramp control in AIMSUN simulator, the corresponding control logic should be incorporated into the six high level functions in Getram Extension.

The flow of Getram Extension with SZM Control logic as well as the interaction with AIMSUN simulator is shown in Figure 4.4.

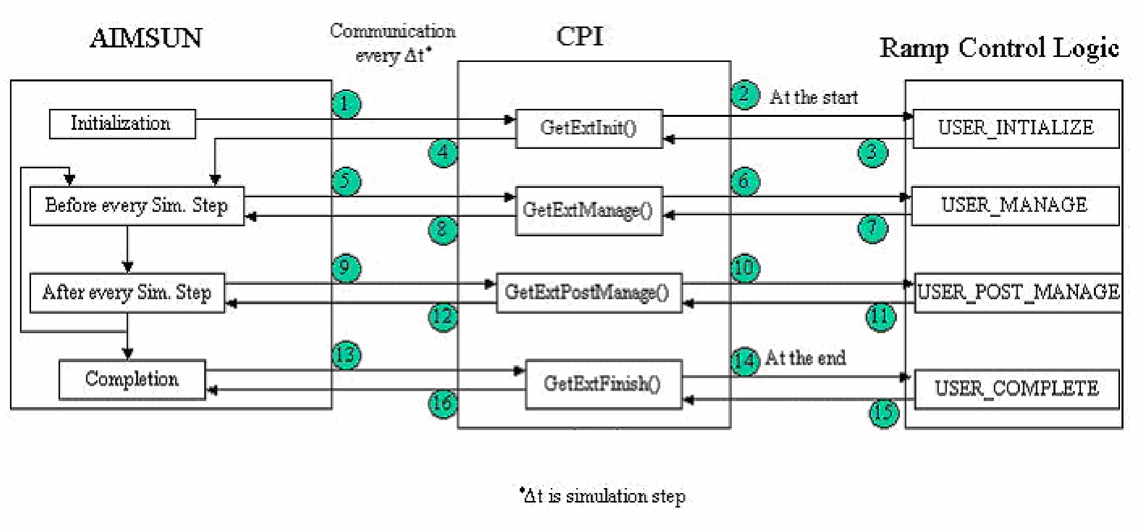


Figure 4.4 CPI interaction with the Simulator and the Ramp Control Logic

The circles numbered from 1 to 16 represent the steps of the control flowing between the corresponding components. For simplicity, the prefix "circle" is omitted while describing the process.

Once the simulation starts, the first function invoked is GetExtInit(). In this function, first the input data such as parameters for ramp control are loaded from several input text files. Appropriate data structure such as the detector maps, station maps and meter maps are created and initialized in this function. Then, in the user-defined function USER_INITIALIZE the data structures required by the ramp control logic are created and initialized. The default ramp metering rates are returned at this stage.

Once the initialization is done and at the beginning of each simulations step, the function GetExtManage() will be called(step 5). In this function USER_MANAGE function is implemented to emulate the SZM control logic. For every 30 seconds control interval, first the runtime simulation data such as the detector measures is feed in the control logic. Then according to the ramp control logic, applicable metering rates to be implemented are calculated. Finally, the Getram extension will return the metering rates to the simulator for implementation (step 8).

After the function GetExtManage() and the simulation has simulated for one simulation step, the Getram Extension is passed on to the function GetExtPostManage (step 9). The function USER_POST_MANGE() (step 10) is implemented in this function to finish the tasks needed at the end of the simulation steps. Functions GetExtManage() and GetExtPostManage() will be called for every simulation step.

At end of the simulation, the function GetExtFinish() is called. In this function, the data structure created for the control logic will be cleared up. The function USER_FINISH() is implemented in this function to complete the tasks need when the simulation finishes.

The DLL is developed in VC 6.0 using Microsoft Foundation Classes. The DLL has been extensively debugged and tested to ensure that the algorithm emulates exactly the SZM control logic. Since the control logic is independent of the site chrematistics, several configuration files which contain the site information are necessary. The three necessary configuration files for each site are rulefile.txt, ramps.txt and stat.txt.

Rulefile.txt

In the stratified zone-metering algorithm each segment of the freeway with mainline detector station as end points forms a zone. The number of stations in a zone varies from two to seven. Zones with the same number of stations are grouped as a layer. To identify the layer and zone information for a certain site, the configuration file rulefile.txt is necessary. This file is prepared by user before the simulation. It provides a sequence of all the mainline detector stations from the upstream to the downstream. This enables easy identification of all the zones and layers. The file also provides the ramp information between two consecutive mainline detector stations. This facilitates the metering rate calculation in the control logic.

The following syntax needs to be maintained in this configuration file:

- The basic format of each line is:

  String_indentifier BLANK or TAB: TAB string TAB string…

- Each string identifier ends with a colon (:);
- The spacing between the colon and the identifier name can be arbitrary; but it is so chosen that, an indentation is preserved;
- A double asterisk character (' ** ') designates a mainline station entry;
- In case of multiple entries to an identifier, a spacing of one tab between entries is maintained;
- In case of no entry to an identifier, a blank line remains;
- In the last line of rulefile.txt, '###END_OF_RULEFILE###' is used to mark the end of file.

A sample rulefile.txt below shows the syntax to be followed:

Sample configuration file rulefile.txt
**MAINLINE STATION : 440 Metered Ramps : 3C1

Unmetred station :
Exit station : 1783
**MAINLINE STATION : 441
Metered Ramps : 3C2 3C3
Unmetred station : 1918
Exit station : 1939 1941
**MAINLINE STATION : 442
… …

The Ramps.txt:

The ramps configuration file provides ramp name, ramp type, the IDs of the detectors on the on-ramps and ramp length. The syntax for this configuration file includes:

- The entries should be as shown below:

  String TAB string TAB string TAB string TAB string

- The sequence of the corresponding entries should be: Ramp name, Ramp type queue detector ID, passage detector ID, ramp length
- No spacing after the colon;
- In case of no entry being appropriate, "none" is used as the string;
- In case of no queue detector, the ramp length needs to be set to 1 foot;

A sample ramps.txt below shows the syntax to be followed:

Sample configuration file ramps.txt
Ramp_name/type/queue_station/passage_station/ramp_length:T.H.7 3B3 L TH7 1914 655 Ramp_name/type/queue_station/passage_station/ramp_length:36th St.

3B4	L	36thSt	1917	442
Ramp_name/type/queue_station/passage_station/ramp_length:Minnetonka Blvd				
3B5	L	Minnetonka	1924	322
Ramp_name/type/queue_station/passage_station/ramp_length:Cedar Lake Rd				
3B6	L	none	1928	1
………				

Stat.txt:

The station configuration file provides detector station name and the IDs of the detectors in corresponding station. The syntax for this configuration file includes:

- The entries should be as shown below:

  String TAB string TAB string TAB …

- The first string should be the detector station name. The strings after the first one should be the IDs of the detectors in this station. The strings are separated by the TAB keys.

A sample stat.txt below shows the syntax to be followed:

Sample configuration file stat.txt			
760	3031	3032	
3033	3033		
759	3028	3029	3068
ValleyView	ValleyView-1	ValleyView-2	
1_ValleyView	1_ValleyView		
2_ValleyView	2_ValleyView		
TH62EB	TH62EB-1	TH62EB-2	
1_TH62EB	1_TH62EB		
2_TH62EB	2_TH62EB		

……..

4.3 Selection of MOEs

The performance of ramp control strategies can be evaluated from a number of perspectives, e.g., freeway safety, traffic flow/travel time, energy consumption and the environment, etc. Depending on the evaluation objective, generally one or more of the above is selected. The performance MOEs considered in this study are:

(1) The performance MOEs for the freeway mainline; this includes

1.1 The Performance MOEs for the freeway safety

- the number of stops occurred in the freeway mainline;
- average number of stops per vehicle

1.2 The performance MOE of freeway traffic flow and travel time;

- freeway total travel time
- freeway total travel/ total mileage traveled;
- freeway total delays;
- average freeway delay time per vehicle;
- total freeway traffic volumes;
- freeway average speed;

(2) The performance MOEs for all the metered ramp; this includes:

2.1 The general performance MOEs for all the metered ramps

- total ramp travel time for all metered ramps;
- total ramp travel/ total mileage traveled for all metered ramps;

- total ramp delays for all metered ramps;
- average ramp delay for all metered ramps;
- total ramp traffic volumes

2.2 The performance MOEs for ramp queue and waiting time for each individual metered ramp:

- average wait time experienced by vehicles serviced by the ramp;
- maximum wait time experienced by vehicles serviced by the ramp;
- total ramp delay
- average queue size on the ramp;
- maximum queue size on the ramp;

(3) The performance MOEs for freeway and ramp system; this includes:

- total system travel time;
- system average speed;

(4) The performance MOEs for the energy consumption and the environment impact. This includes

- Estimating fuel consumption measured in gallons.
- Estimating pollutant emissions in terms of carbon monoxide (CO), nitrides of oxygen (NO_x) and hydrocarbons (HC);

The selected measures of effectiveness and their definitions are presented in table 4.1. These measures of effectiveness will be used to assess the incremental change between the two scenarios (the SZM control and the improved SZM control).

As stated in 4.1, the original outputs from AIMSUN are the set of measurements at different aggregation levels in spatial (either the entire system or each section/turning) or temporal terms (the whole duration of simulation, or a regular collection interval defined by the user). In this study, original simulation outputs in terms of section statistics will be extensively used for exacting the required measures of effectiveness.

Table 4.1 Performance Measures of Effectiveness for Ramp Metering

Category		Measure of Effectiveness	Definition
Freeway Mainline	Freeway Safety	**Total Number of Stops**	Total number of stops experienced by all the vehicles while traveling on the freeway mainline
		Number of Stops Per Veh	Average number of stops per vehicle while traveling on the freeway mainline
	Freeway Traffic Flow	**Total Freeway Travel Time**	Total travel time accumulated by all the vehicles while traveling in freeway mainline (vehicle-hours)
		Total Freeway Delay	Total delay time accumulated by all the vehicles while traveling in freeway mainline (vehicle-hours)
		Average Freeway Delay	Average delay time per vehicle while traveling in freeway mainline (minutes/vehicle)
		Total Freeway Travel	Total number of vehicle-miles traveled in freeway mainline
		Volume	Total number of vehicles that have been serviced by freeway
Metered Ramps	Total Ramps	**Total Ramp Travel Time**	Total travel time accumulated by all the vehicles while traveling on ramps (vehicle-hours)
		Total Ramp Delay	Total delay time accumulated by all the vehicles while traveling on ramps (vehicle-hours)
		Average Ramp Delay	Average delay time per vehicle while traveling on ramps (minutes/vehicle)
	Individual Ramp	**Max Ramp Wait Time**	Maximum wait time experienced by vehicles while traveling the ramp under study (minutes)
		Mean Ramp Wait Time	Average wait time per vehicle while traveling the ramp under study (minutes)
		Max Ramp Queue Size	Maximum number of vehicles in queue on the ramp under study
		Mean Ramp Queue Size	Average number of vehicles in queue on the ramp under study
Fuel Consumption &Environment Impacts		**Fuel Consumption**	Total fuel consumed in gallons
		Pollutants Emissions	Total emissions in kilograms for CO,HC and NOx , respectively

4.4 Test Sites and Data Acquisition

4.4.1 Test Site Selection Criteria

In coordination with the Mn/DOT TMC traffic engineers, two test sites were selected having geometric properties and traffic characteristics that are representative of the Twin Cities freeway network to the extent possible. The following criteria were used in choosing the sites:

- geographic representation within the Twin Cites metropolitan area;
- level of congestion;
- representative length;
- upstream and downstream boundary conditions
- ease of traffic data collection;
- availability of alternate routes.

Through discussions with the Mn/DOT engineers at the TMC, three test sites were selected having geometric properties and traffic characteristics that are representative of the Twin Cities freeway network to the extent possible. The following criteria were used in choosing the sites:

I. Representative of Twin Cities freeway network

The Twin Cities freeway network includes typical geometric configurations such as weaving sections, lane drop locations, high volume entrance ramps, high volume exit ramps, etc. An effort was made to select sites which included most of these features but also avoided those that had too many weaving sections and complex geometric sections within a short span. Also, the freeways can be classified into one of the following general categories: radial, circumferential, central business district connector i.e. connecting the Minneapolis and St. Paul downtown districts. The selected sites should represent at least two of the categories.

II. Level of congestion

One of the major objectives of ramp metering is to ease freeway congestions. Test sites of various congestion levels are essential for testing ramp metering effectiveness. Hence the test sites need to be selected such that they cover at least two of three identified congestion levels, i.e., low traffic, moderately heavy traffic and very heavy traffic.

III. Representative length

The ramp metering strategy under study is the recently deployed Stratified Zone Metering. This strategy is based on dividing the freeway into zones and regulating the zone entering volumes. As described in the earlier chapter, Stratified Zone control strategy requires at least seven stations (3.0 miles) to define a complete layer. Thus, assuming a minimum of two completer layers a minimum length of 6 miles is essential.

IV. Upstream and downstream boundary conditions

If recurrent traffic congestion exists beyond the freeway segment to be simulated (i.e., either the upstream or downstream end of the freeway segment, or both, are subject to recurrent traffic congestion), essential difficulty would arise in calibrating the corresponding simulation model because the boundary conditions can not be controlled. In this case, the simulation process could deviate from what actually occurred in reality. Because of this, the sites to be selected must have the boundaries free of congestion so that the simulation model can be accurately calibrated replicating the real situation.

V. Ease of traffic data collection

As the Mn/DOT ramp metering algorithms rely heavy on real-time detector data, it is essential that the sites selected should have most of its mainline detectors in working conditions for successful calibration. In addition, all entrance and exit detectors must be operational, so that the boundary demand conditions can be well defined.

VI. Availability of alternate routes

One of the goals of this research is to have the flexibility to expand a test site(s) to include arterials associated with that freeway for future research. This would allow a study of the effects of diverted freeway-bound traffic to adjoining arterials due to ramp metering and evaluation of the impacts of ramp control on the corridor as a whole. Through information about the traffic diversion due to metering is not currently available, the selected sites have alternate routes and thus, corridor simulation is also possible once the data becomes available.

4.4.2 Test Site Descriptions

Two test sites were selected in this study to evaluate the effectiveness of the improvement methodology: TH169-NB and I94WB. These tested were chosen because the geometric properties and traffic characteristics of these two sites are very representative of the Twin Cities freeway network. The detailed description of these two sites is the following:

TH 169 Northbound

This site is a segment of TH-169 Northbound circumferential freeway. This test section is about 12 mile long starting from the interchange with I-494 and ending at 63rd Avenue North. The upstream and downstream boundaries are free of congestion. Most of the test section is two lanes on the mainline. It has 10 weaving areas, 24 entrance ramps and 25 exit ramps. The 23 metered ramps contain 4 HOV bypasses and 2 freeway-to-freeway ramps connecting from TH 62 and I-394.

I-94 Eastbound

This site is a segment of I-94 Eastbound. This test section is about 11 miles long starting from the interchange with I-394 and ending at 9rd St. Off ramp. The upstream and downstream boundaries are also free of congestion. During peak hours, this test section is often severely congested due to the heavy traffic and the complex geometry. It includes 6 weaving areas, 14 exit ramps and 19 entrance ramps of which 4 are unmetered.

The positions of these two sites in the twin cities freeway network are shown in figure 4.5. Their geographical characteristics are summarized in table 4.2.

Table 4.2 Geometric Properties of Selected Test Sites

Characteristics	**TH-169NB**	**I-94EB**
Direction	Northbound	Eastbound
Length (miles)	12	11
Upstream Boundary	I-494 interchange	I-394 interchange
Downstream Boundary	63rd Avenue North	9th St. Off ramp
Metering Period	AM and PM	AM and PM
Metered Entrance Ramps	23	15
Unmetered Entrance Ramps	1	4
Off ramps	25	14
Weaving Sections	10	6
Lane Drop Locations	1	3
Geometric Complexity	Medium	Complex
Level of Congestion	Medium	High

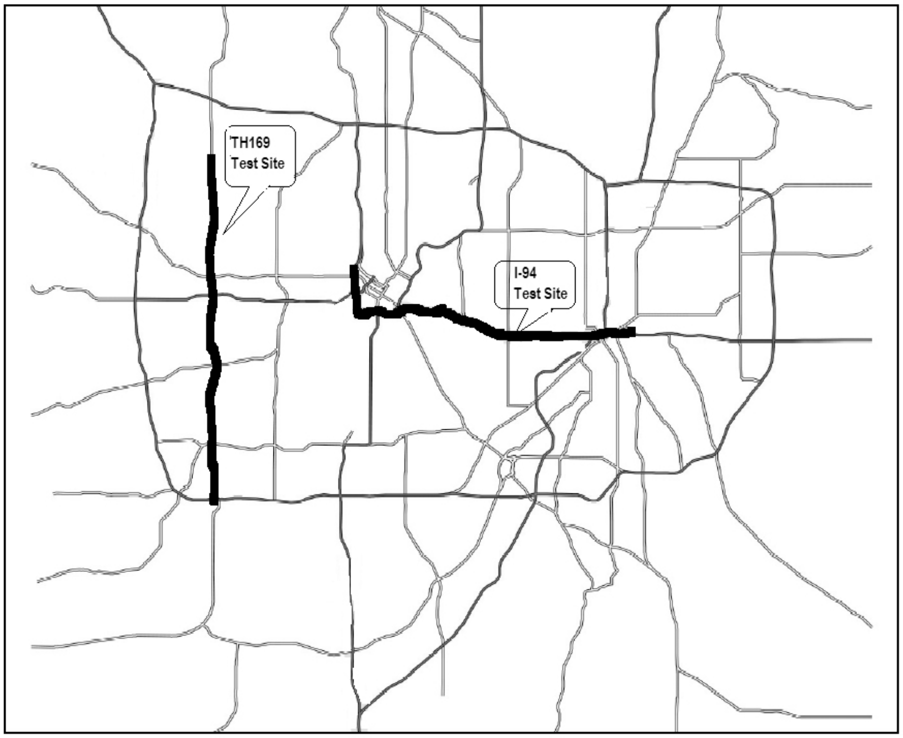


Figure 4.5 Two Selected Test Sites: TH-169NB and I-94EB

The selected test dates are November 08, 2000, November 13, 2000 and November 27,2000 for TH-169NB; October 26, 2000, November 01, 2000 and November 27,2000 for I-94EB. The dates were specifically selected during the ramp meter shutdown period to ensure the calibrated simulation models have no systematic bias to a particular set of control parameter values. Afternoon peak was selected as the test sites experience more severe congestion. In order to include the entire congestion cycle each simulation experiment was conducted from 14:00 to 20:00, while the SZM control period is from 15:00 to 18:00.

4.4.3 Simulation Data Acquisition

As stated in 4.1, two types of information are generally required in order to build the simulation models of the test sites: geometry layout and the traffic demand data.

The geometric layout consists of the physical geometry properties of the freeway such as traffic flow direction, number of lanes on the mainline, width of the lanes, length of the mainline between ramps, length of the entrance ramps, length of acceleration and deceleration lanes, location of the detectors and ramp meters, etc. The Digital Orthophoto QuarterQuadrangles (DOQQs) are used as background to build the geometry layout of the test sites. DOQQs are high resolution black and white aerial photos, 3.75' x 3.75', which cover the entire Twin Cities Metropolitan Area.

The traffic demand is generated from traffic flow data due to the unavailability of the O/D information. The traffic demand data of flow includes the traffic composition data, flow at each input sections and turning proportion at each sections. These data can all be obtained through the traffic detecting system. The traffic detecting/monitoring system currently implemented by Mn/DOT consists of loop detectors (traffic sensors) and closed circuit television (CCTV) cameras. Mn/DOT deployed nearly 230 CCTV cameras along 210 miles freeway in the metro-area. The traffic composition data, i.e., the respective percentage for each of the vehicle types constituting the traffic flow can exacted from the real-time videos. The traffic flow data used in this study is assumed to be composed of three vehicles types, i.e., car, single-unit truck and Semi-trailer. The corresponding percentage is 95%, 2.5% and 2.5%.

Apart from the camera system, approximately 3,700 loop detectors are deployed on freeway mainline, entrance ramps as well as exit ramps. The loop detector usually collects two types of traffic data: volume and occupancy. Volume is a measurement of the number of vehicles that have passed over the detector. Each detector measures a lane volume and the sum of all detector volumes in a station gives the total traffic volume crossing that location. Occupancy is measured as the percentage of time during which a loop detector is occupied by a vehicle. Typical deployment of detectors is illustrated in figure 4.6, which contains two mainline detectors (1942 and 1943), two entrance ramp detectors (1944 and 1945) and one exit ramp detector (1946). In this example, detector 1944 positioned after the stop line, is commonly referred to as passage detector as it measures the volume passing the ramp meter; while detector 1945 on the upstream end of the ramp, is known as queue detector since it is deployed for the purpose of measuring the entering demand whilst detecting queue spillback on the ramp.

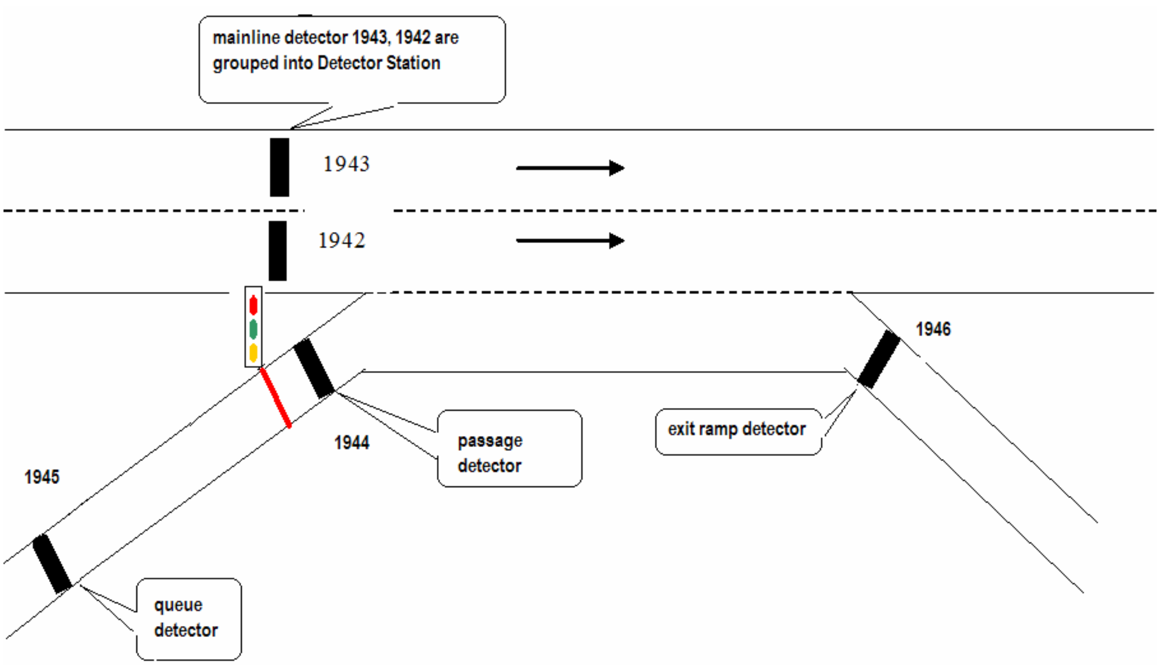


Figure 4.6 Typical Mainline, Entrance/Exit Ramp, and Detectors

The loop detector data is aggregated every 30 seconds and transmitted to the RTMC (Regional Transportation Management Center). These 30-second data are used to compute the flow at each input sections and turning proportion at each section.

Entrance demands are retrieved from loop detector volume measurement. The queue detector located upstream of each ramp directly measures the entering demand. However, care must be taken in checking the consistency of detector data as loop detector might malfunction and fail to give vehicle counts or occupancy. This means, the data record with a zero volume but a non-zero occupancy or vice-versa should be filtered out as the two measurements are not consistent with each other. Whenever necessary, a visual comparison of the traffic volume (or occupancy) on consecutive days should be carried out to check if the flow patterns were similar.

Turning percentages of the mainline volumes at exit ramps are important for the micro-simulator to replicate the actual traffic flow process. In this study, the turning percentage is determined from the ratio of mainline volume to exit volume. This is illustrated graphically in figure 4.7, where the turning percentage of the mainline flow at the exit ramp is computed as:

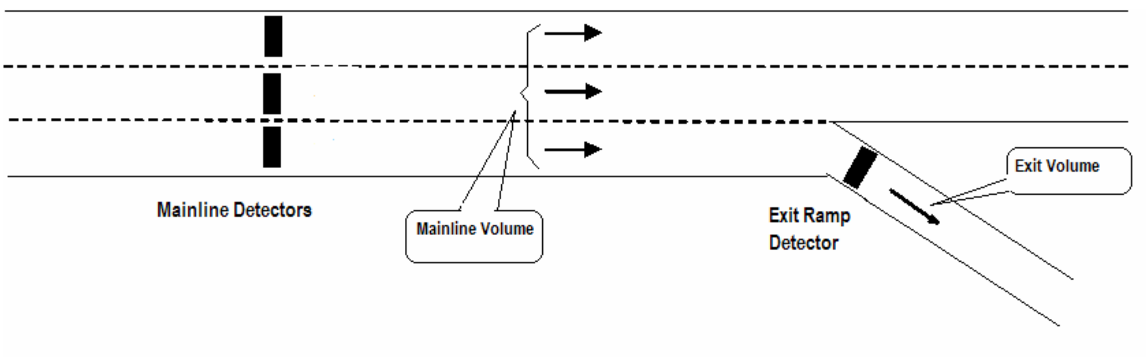


Figure 4.7 Mainline Detectors and Exit Ramp Detector

$$P = \frac{V_{exit}}{V_{mainline}} \tag{4-1}$$

where:

P represents the turning percentage of mainline volume exiting from the off-ramp;

V_{exit} represents the volume recorded by the exit ramp detector during a prescribed time interval;

$V_{mainline}$ represents the volume recorded by the mainline detectors during a prescribed time interval.

4.5 Simulation Model Calibration

Once the geometric and traffic data were used to build the simulation models of the test sites, the next step was to calibrate them. Simulation model calibration is the process of obtaining a good match between actual and simulated fundamental measurements (e.g., Flow and Speed) by fine tuning the global and local parameters of the microscopic simulator. In this study the calibration methodology proposed by Hourdakis et al. (2000) was followed resulting in very satisfactory statistical match. For instance, by comparing actual and simulated volumes on mainline detector stations, the correlation coefficient was high ranging from 0.90-0.98 at both test sites, while similar scores were obtained for other test metrics (Thiel's coefficients, etc) and speed contours.

Table 4.3 Goodness of Fit for TH-169NB Mainline Station Volumes (14:00-20:00)

Statistic / Test days	Root Mean Square Error %	Correlation coefficient	Theil's Inequality Coefficient	Theil's Bias Proportion	Theil's Variance Proportion	Theil's Covariance Proportion
Nov 08	8.84	0.91	0.00424	0.00591	0.05422	0.93987
Nov 13	8.75	0.92	0.00364	0.04477	0.00545	0.94978
Nov 27	8.81	0.93	0.00383	0.00837	0.02409	0.96753

Table 4.3 summarizes the calibration test results for the TH-169NB site. For all three days the overall statistical scores from the final simulation of this site are presented. In the table, these scores represent 5-minute volume comparison in all mainline detector stations. As table 4.3 suggests, through this systematic calibration and validation methodology, very high accuracy was achieved.

Chapter 5 Evaluation Results of Improved Ramp Control Logic

This chapter summarizes the simulation results. The comparison results of the improved SZM vs SZM (both are based on the current empirical queue size estimation) are summarized in section 5.1.

The sites for testing the above scenarios are TH169-NB and I-94EB, being selected in line with the criteria presented in Chapter 4. These two sites possess representative geometric traits and demand patterns; hence the results can be extrapolated to the entire freeway system in the Twin Cities area. Also, the time period for testing both scenarios is determined to be PM, i.e., from 14:00 to 20:00 (metered period is from 15:00 to 18:00), as this covers the time prior to and after peak-hour congestion. Furthermore, in order to perform the simulation experiments in uniform operational conditions, three testing dates were selected for each test site from the ramp shutdown period in 2000, during which ramp metering was inactive. For TH169-NB test site, the degrees of congestion for the three test dates are: NOV 08, 2000 (moderate congestion), NOV 13, 2000(moderate congestion) and NOV 27, 2000 (severe congestion). For the I-94EB test site, the degrees of congestion for the three test dates are: OCT 26, 2000 (severe congestion), NOV 01, 2000 (moderate congestion) and NOV 27, 2000 (moderate congestion).

The comparison results of improved SZM vs. SZM are summarized in tables 5.1(entire simulation period results, 14:00-20:00) and 5.2 (metering period results, 15:00-18:00). Detailed quantitative results are provided in the Appendix. Both tables present the MOEs' percentage change occurred with the improved SZM control, i.e., a positive percentage change in an MOE indicates this MOE increases with the improved SZM control and vice versa. Each table includes results for both test sites (TH169-NB and I-94EB) and all six test days (each site with three days). It should be noted that as similar findings can be derived from both tables; thus for brevity we will focus on table 5.2 and summarize the major results for metering period only.

Table 5.1 Percentage Change for Major MOEs (Improved SZM control over SZM control) Entire Simulation Period (2:00pm to 8:00pm)

Categories		% Change	TH-169NB			I-94EB		
			NOV 08	**NOV 13**	**NOV 27***	**OCT 26***	**NOV 01**	**NOV 27**
Freeway MOEs (Mainline)	Total Number of Stops		-9.47%	-17.43%	-16.24%	-4.53%	-6.87%	-12.08%
	Number of Stops Per Veh		-9.47%	-17.43%	-16.24%	-4.53%	-6.87%	-12.08%
	Total Freeway Travel Time (veh-hours)		-2.01%	-1.40%	-3.72%	-1.32%	-2.31%	-2.37%
	Total Freeway Travel (veh-miles)		-0.02%	-0.02%	0.15%	0.01%	-0.02%	0.17%
	Total Freeway Delay (veh-hours)		-7.48%	-7.28%	-9.56%	-4.07%	-6.79%	-9.21%
	Average Freeway Delay (min/veh)		-7.48%	-7.28%	-9.56%	-4.07%	-6.79%	-9.21%
	Volume(vehicles serviced by freeway)		0.00%	0.00%	0.00%	0.00%	0.00%	0.00%
	Average Speed (mile/hour)		2.03%	1.39%	4.03%	1.35%	2.35%	2.61%
Ramp MOEs	Total Ramp Travel Time (veh-hours)		-3.21%	-14.39%	-5.19%	-12.63%	-11.99%	-20.20%
	Total Ramp Travel (veh-miles)		0.00%	0.00%	0.00%	0.00%	0.00%	0.00%
	Total Ramp Delay (veh-hours)		-3.42%	-24.42%	-7.38%	-20.16%	-19.38%	-33.17%
	Average Ramp Delay (min/veh)		-3.42%	-24.42%	-7.38%	-20.16%	-19.38%	-33.17%
	Volume (vehicles entered from ramps)		0.00%	0.00%	0.00%	0.00%	0.00%	0.00%
System MOEs	Total System Travel Time (veh-hour)		-2.10%	-2.14%	-3.80%	-2.44%	-3.24%	-4.14%
	Average System Speed (mile/hour)		2.13%	2.16%	4.11%	2.51%	3.33%	4.49%
	Fuel Consumption (gallons)		-3.67%	-3.01%	-4.38%	-1.81%	-3.70%	-3.67%
	Pollutants Emissions (kgs)	CO	-2.46%	-1.81%	-3.88%	-1.98%	-3.18%	-3.50%
		HC	-2.17%	-1.71%	-3.56%	-2.00%	-3.30%	-3.82%
		NO_x	-3.09%	-2.33%	-4.83%	-2.23%	-3.72%	-3.94%

*** The most severely congested day on a test site**

Table 5.2 Percentage Change for Major MOEs (Improved SZM control over SZM control)
Metering Period (3:00pm to 6:00pm)

Categories	% Change		TH-169NB NOV 08	TH-169NB NOV 13	TH-169NB NOV 27*	I-94EB OCT 26*	I-94EB NOV 01	I-94EB NOV 27
Freeway MOEs (Mainline)	Total Number of Stops		-9.57%	-19.62%	-16.17%	-4.67%	-6.66%	-11.90%
	Number of Stops Per Veh		-9.57%	-19.62%	-16.17%	-4.66%	-6.66%	-11.89%
	Total Freeway Travel Time (veh-hours)		-3.08%	-2.46%	-5.10%	-2.14%	-3.14%	-3.30%
	Total Freeway Travel (veh-miles)		0.00%	-0.04%	0.32%	0.00%	0.20%	0.55%
	Total Freeway Delay (veh-hours)		-7.71%	-9.06%	-9.47%	-4.28%	-6.63%	-8.95%
	Average Freeway Delay (min/veh)		-7.71%	-9.06%	-9.47%	-4.27%	-6.63%	-8.94%
	Volume (vehicles serviced by freeway)		0.00%	0.00%	0.00%	-0.01%	0.00%	-0.01%
	Average Speed (mile/hour)		3.17%	2.47%	5.72%	2.19%	3.46%	4.01%
Ramp MOEs	Total Ramp Travel Time (veh-hours)		-2.60%	-18.97%	-5.93%	-15.01%	-14.13%	-24.57%
	Total Ramp Travel (veh-miles)		-0.03%	-0.03%	-0.04%	-0.05%	-0.02%	-0.10%
	Total Ramp Delay (veh-hours)		-3.13%	-23.96%	-6.76%	-19.76%	-18.97%	-32.64%
	Average Ramp Delay (min/veh)		-3.13%	-23.96%	-6.76%	-19.75%	-18.98%	-32.64%
	Volume (vehicles entered from ramps)		0.00%	0.00%	0.00%	-0.01%	0.00%	-0.01%
System MOEs	Total System Travel Time (veh-hour)		-3.03%	-3.66%	-5.15%	-3.69%	-4.43%	-6.01%
	Average System Speed (mile/hour)		3.13%	3.75%	5.77%	3.83%	4.85%	6.97%
	Fuel Consumption (gallons)		-4.29%	-4.67%	-4.72%	-2.28%	-3.95%	-3.70%
	Pollutants Emissions (kgs)	CO	-3.39%	-3.00%	-4.99%	-2.41%	-3.28%	-4.08%
		HC	-3.00%	-2.91%	-4.60%	-2.71%	-3.66%	-4.56%
		NO_x	-3.97%	-3.72%	-5.76%	-2.56%	-3.62%	-4.51%

* The most severely congested day on a test site

5.1 Freeway Traffic Flow, Travel Time and Delay

As indicated in table 5.2, *total freeway mainline travel time* is reduced under the improved SZM control as compared to the original. The reduction can be attributed to the postponed and decreased freeway congestion favoring freeway performance. The reduction varies from 2.14% to 5.10% depending on the test site geometries as well as the demand conditions. At the less geometrically complex TH-169NB, the reduction varies from 2.46% to 5.10%. The largest reduction appears on the day with the highest demand. For I-94EB, which is a Central Business District (CBD) freeway with complex geometric layout and generally higher demands, the reduction is less pronounced varying from 2.14% to 3.30%. The least reduction occurs on the severely congested day. The difference between the trends of improvement on these two test sites can be explained by the origins of the majority of the demand. On TH-169NB, most of the demand comes from the ramps; in contrast on I-94EB, most of the demand originates at the upstream boundary of the freeway mainline.

In addition to the above favorable results, the *freeway average speed* in both test sites increases under the improved SZM strategy. The increase varies from 3.17% to 5.72% on TH-169NB and 2.19% to 4.01% on I-94EB. Furthermore, the improved SZM strategy is more effective in smoothing out the freeway flows. This is shown in figure 5.1, where the Nov 27, 2000 density patterns of I-94EB resulting from both the SZM and improved SZM are plotted. This figure indicates that the improved SZM is more capable of smoothing out the freeway traffic flow and is therefore more effective in eliminating freeway delays as compared to the SZM. Specifically, as shown in the figure, although the location and time of the bottlenecks does not change, the duration of the congestion has shrunk with the improved SZM.

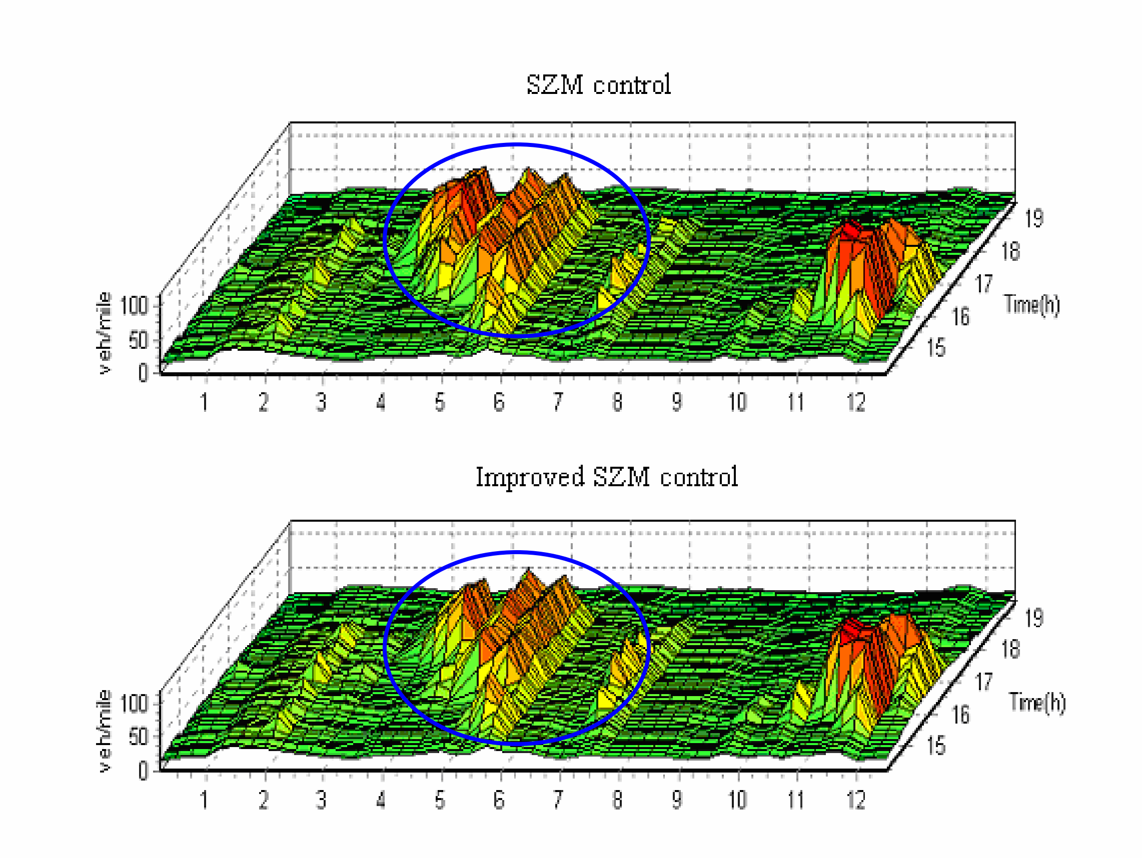


Figure 5.1 Density Patterns: Improved SZM vs SZM alternative

I-94EB, Nov 27, 2000

Regarding the *total freeway delay,* table 5.2 indicates that it is significantly reduced under the improved SZM control. Specifically, on TH-169NB, the total freeway delay is reduced by 7.71% to 9.47%. While on I-94EB, the total freeway delay is reduced by 4.28% to 8.95%.

5.2 Total Ramp Travel Time and Total Ramp Delay

With respect to ramp travel times, table 5.2 also indicates that the *total ramp travel times* are reduced significantly with the improved SZM control as compared to the original in both sites and all test days. This reduction is due to the improved traffic conditions on the mainline which can accept more inputs from the ramps. On TH-169NB, the reduction in total ramp travel time varies from 2.60% to 18.97%; while on I-94EB, the reduction is more significant varying from 14.13% to 24.57%.

The *total ramp delays* are greatly reduced as well under the improved SZM control. The reduction can be as much as 32.64%. These findings suggest that that the improved SZM can effectively improve freeway traffic conditions while simultaneously reducing delays on both the freeway and the ramps. The results also indicate that the improved SZM not only is effective for the TH-169NB (less congested and simpler geometrically), but also for I-94EB (a CBD freeway with complex geometric layout and generally higher demands).

5.3 Total System Travel Time

Due to the reduction of the total freeway travel time and the total ramp travel time, the *total system travel time* are significantly reduced under the improved SZM control. This show the improved SZM can effectively enhance the freeway traffic condition and reduce the delay at the system level on the two test sites. The reduction varies from 3.03% to 6.01%.

Freeway Safety

It is also suggested in table 5.2 that, for both test sites, *the total number of stops* and *number of stops per vehicle* on the freeway mainline decreases significantly. The percentage decrease varies from 4.66% to 19.62% depending on the test site characteristics and the demand conditions. The reduction is a direct result of the increased smoothness of the mainline flow. Based on known associations between accident rates and speed variance (Garber et al., 1989 and Oh et al., 2001), increasing flow smoothness should result in reduction of accident likelihood. Therefore in general, it can be inferred that the improved SZM logic increases traffic safety as compared to the original.

To be specific, for TH169-NB, which is a circumferential freeway with less geometric complexities, the reductions of two MOEs are very significant. For example, the percentage reductions of the total number of stops for the three test dates are 9.57%, 19.62% and 16.17% respectively. The improved SZM is not only beneficial to the moderately congested dates (Nov 08, 2000 and Nov 13, 2000), but also to the severely congested days (Nov 27, 2000).

For I-94EB, which is a CBD freeway with complex geometric layout and generally higher demand, the trend is similar, but the percentage reductions are less significant.

Energy Consumption and Environmental Impacts at System Level

As illustrated in table 5.2, in general the improved SZM has positive effects on the *Fuel Consumption* and *Pollutant Emissions*. The improved SZM strategy can save 2.28% to 4.72% fuel and eliminate the emission of pollutant by 2.71% to 5.76%. These are the direct consequences of the reduced system travel time and total number of stops on the freeway mainline.

For the TH169-NB, the fuel consumption is decreased by 4.29%~4.72%. The pollutant emissions are decreased by 2.91%~5.76%. For the I-94EB, the fuel consumption is decreased by 2.28%~3.70%. The pollutant emissions are decreased by 2.41%~4.56%, the most severely congestion day yields the largest fuel consumption decrease.

5.4 MOEs for Each Individual Ramp

Table 5.3 presents the effects of the SZM and the improved SZM on ramp wait times and ramp queue sizes for all metered entrance ramps on I-94EB for NOV 27, 2000. As indicated in this table, although violations of maximum ramp delay still occur under both the SZM and the improved SZM due to the inaccurate queue size estimation logic, the violation magnitude is almost negligible under the improved SZM strategy and much lower than the original one.

Table 5.3 MOEs for Ramp Performance on I-94EB (Improved SZM control over SZM control)
NOV, 27 2000 (3:00pm to 6:00pm)

MOE / Ramps	Average Ramp Wait Times (minutes)		Max Ramp Wait Times (minutes)		Total Ramp Delay (vehicle-hours)		Average Queue Size (vehicles)		Max Queue Size (vehicles)	
	SZM	ISZM	SZM	ISZM	SZM	ISZM	SZM	ISZM	SZM	ISZM
Lyndale Ave.	0.12	0.23	0.57	0.59	2	3	1	1	3	4
Hennepin Ave.	1.52	1.05	2.71	3.06	85	58	21	14	44	39
5th Ave	2.76	1.33	6.12*	4.13*	60	29	14	7	29	24
6 Street	1.37	0.84	4.93*	4.01*	39	24	10	6	44	41
Cedar Ave	1.38	1.27	2.98	2.88	28	26	6	6	13	13
Riverside Ave.	3.17	1.93	5.56*	3.99	87	53	20	12	34	30
Huron Blvd	0.24	0.28	0.82	0.95	6	7	1	2	7	12
Cretin Ave.	1.59	0.87	3.91	3.28	40	22	9	5	19	20
Snelling Ave.	0.61	0.66	2.25	1.84	21	23	5	6	24	21
Lexington Ave	2.09	1.34	5.76*	4.55*	57	37	13	8	31	30
Dale Street	0.05	0.10	0.59	1.19	1	2	1	1	7	11
Marion Street	0.04	0.04	0.52	0.49	1	1	1	1	8	8

*The maximum allowed ramp wait time is violated

Part III: Improving Queue Size Estimation for the SZM Strategy

Chapter 6 Methodology

6.1 Ramp Classification

A typical deployment of ramp detectors is shown in Figure 6.1, which contains two kinds of detectors. The detector positioned after the stop line is commonly referred to as passage detector as it measures the volume passing the ramp meter. The other detector which is located on the upstream end of the ramp is known as queue detector since it is deployed for the purpose of measuring the entering demand and detecting queue spillback on the ramp. If the traffic volumes can be measured accurately by both passage and queue detector, the queue size can be easily estimated through the flow conservation, as described in the following.

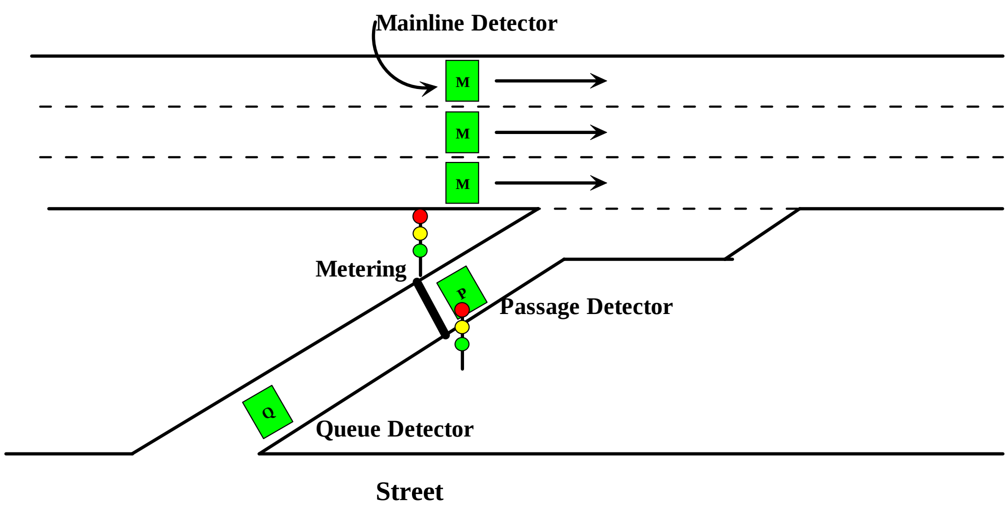


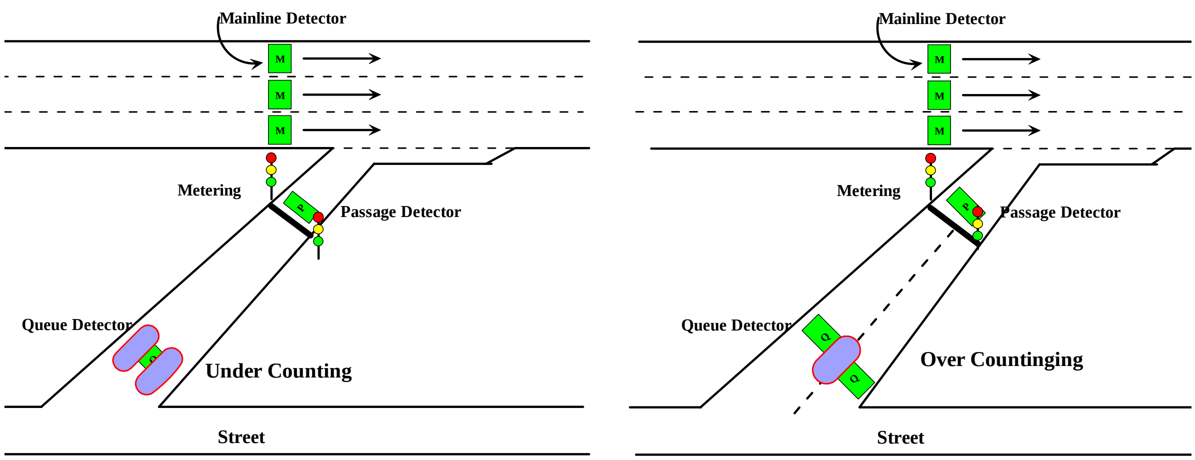
Figure 6.1 Location of Detectors

Let U_k denote the number of vehicles that pass the queue detector at the kth discrete-time interval ($k = 1,2,3\cdots$), V_k the number of vehicles that pass the passage detector at the kth discrete-time interval, and x_k denotes the number of vehicles that stay between the queue detector and passage detector, i.e. the queue size, the following difference equation, termed conservation equation, is derived:

$$x_k = x_{k-1} + (U_k - V_k) = x_{k-1} + \Delta f_k \tag{6-1}$$

Where: $\Delta f_k = U_k - V_k$

However, Δf_k can not be perfectly determined. False and missed counts contribute errors to the measurement of U_k and V_k. According to the experiences of Mn/DOT's engineers and the comparison of real data and detector data, it was found that there are about 31.0% metered ramps that have biased queue detector counts and about 6.8% metered ramps that have over or under count on passage detectors. The reason for under counting is that the detector is not wide enough that two vehicles can pass the detector at the same time which leads to miss counting the vehicles (See Figure 6.2 (a)). The reason for over counting is that the two detectors are so close that one vehicle can cover the two detectors at the same time (See Figure 6.2 (b)).



(a) Under Counting **(b) Over Counting**

Figure 6.2 The Reasons of Counting Error

Considering Mn/DOT's request that the methodology for queue size estimation should be as simple as possible, it is straightforward to apply conservation equation (Eq.6-1) to estimate queue size between queue detector and passage detector for those ramps that have no or small counting error. According to the comparison of real data and detector data, more than 60% ramps can apply the conservation method (Based on our research on two test sides).

Still, 40% ramps cannot use the conservation method (Based on our research on two test sides). According to the different sources of counting error, these ramps can be classified into two categories: one for the ramps with counting error on passage detector only and the other for the ramps with counting errors on both queue and passage detectors. Different methodologies should apply respectively. The detailed description is in the following sections.

6.2 Methodologies for Queue Size Estimation

As described above, the ramps are categorized into three categories according to the different reasons of counting error: Class I for those ramps with error-free or minor counting error (The threshold of “minor error” will be explained in next chapter), Class II for the ramps with passage detector counting error and Class III for the ramps with counting errors from both queue and passage detectors. Three different methods are applied for different categories of ramps.

6.2.1 Method I: Conservation Model for Class I Ramps

For ramps with error-free or minor counting error, the conservation model is simple and efficient. Practically, the model can be described as follows:

$$x_k = \begin{cases} \max(0, x_{k-1} + U_k - V_k), & \text{if } Occ\_queue_k < 25\% \\ Max\_queue, & \text{if } Occ\_queue_k \geq 25\% \end{cases} \qquad (6\text{-}2)$$

where:

x_k is the queue size at the kth discrete-time interval ($k = 1,2,3\cdots$)

U_k is the number of vehicles that pass the queue detector at the kth discrete-time interval

V_k is the number of vehicles that pass the passage detector at the kth discrete-time interval

Max_queue is the maximum queue size for different ramps

Occ_queue_k is the occupancy for queue detector.

As shown in Eq. (6-2), the nonnegative constraint is to ensure that the queue size is larger than zero. The congestion constraint is used to deal with the condition that the queue extends beyond the queue detector. Once the occupancy of queue detector is greater than occupancy threshold of 25%, which means that the queue spill out, maximum queue size is used, which is the maximum number of vehicles between queue detector and passage detector, to replace the current queue size.

6.2.2 Method II: Conservation Model for Class II Ramps

For the ramps with passage detector error while queue detectors are accurate, the conservation model can be still applied, in which the erroneous count from passage detector is replaced by "Green Count". The "Green Count", which is decided by the release rate, is the maximum number of vehicles that can pass through the ramp metering. The current SZM strategy can produce the "Green Count" at every discrete time period (30 seconds). Generally, the Green Count is larger than or equal to the traffic counts of passage detector. Therefore, if the passage detector has error, the traffic counts of passage detectors are replaced by the green counts and still apply conservation law to estimate the accurate queue size. Considering that sometimes the green counts are larger than the passage detector, the nonnegative constraint should be added. The detailed function is as follows:

$$x_k = \begin{cases} \max(0, x_{k-1} + U_k - G_k), & \text{if } Occ\_queue_k < 25\% \\ Max\_queue, & \text{if } Occ\_queue_k \geq 25\% \end{cases} \tag{6-3}$$

where:

x_k, U_k are the same as defined at Equation (6-2)

G_k is the Green Count at the kth discrete-time interval.

6.2.3 Method III: Two Models for Class III Ramps

The greatest problem of queue size estimation is for ramps with significant counting errors from queue detectors. As revealed in the literature, how to estimate the number of vehicles between two detectors with the counting error (i.e. queue size, or how to estimate the density between two detectors) have been studied before. Most of them dealt with the situation on the mainline freeway with the assumption of homogeneous traffic flow and applied Kalman Filtering or Extended Kalman Filtering to estimate the vehicle counts, density, or travel time between two detectors with counting error (Gazis and Knapp, 1971; Szeto and Gazis, 1972; Gazis and Szeto, 1974; Kurkjian et al.1980; Gazis and Liu, 2003; Chu et al, 2005). The research of Kurkjian et al (1980) considered the situation of inhomogeneous traffic flow, but their method can only be applied for off-line case. So far no studies have been found for the on-ramp queue size estimation. Because there is metering between queue detectors and passage detectors, the traffic flow is inhomogeneous. And on-line estimate of queue size needs to be calculated for each discrete time period. Therefore, the existing research results cannot be easily extended to our case.

Data available for queue size estimation includes traffic counts of queue detector and passage detector, occupancy of queue detector, and passage detector and release rate or the "Green Count" at each time period. Based on the analysis of real queue size on each ramp and traffic counts and occupancy of passage and queue detectors, it is hypothesized that some statistical relationship may exist between the queue size and traffic counts and occupancy, although the traffic counts and occupancies are faulty. And once the queue size can be represented by traffic counts and occupancy, Kalman Filtering is applied to estimate the queue size due to its efficient and capability to estimate the state of a dynamic system from a series of incomplete and noisy measurements. Therefore, two models are described below, one is the regression Model and the other is the Kalman Filtering Model.

Regression Model:

Many different forms of regression model can be used to fit the relationship between the queue size and the traffic counts and occupancy of queue and passage detectors. In this paper a simple linear regression model is applied due to the simplicity. Because the current state of ramp queue is related that in the last time period, the information of queue and passage detectors in the last period is also used. The detailed description of Regression Model is as following:

$$\begin{aligned} x_k = {} & a*Vol\_queue_k + b*Vol\_passage_k + c*Occ\_queue_k + \\ & d*Occ\_passage_k + e*Vol\_queue_{k-1} + f*Vol\_passage_{k-1} + \\ & g*Occ\_queue_{k-1} + h*Occ\_passage_{k-1} \end{aligned} \qquad (6\text{-}4)$$

Where:

$x_k \geq 0$ is the queue size at the kth discrete-time interval ($k = 1,2,3\cdots$)

Vol_queue_k is the traffic count of queue detector at the kth time interval

Occ_queue_k is the occupancy of queue detector at the kth time interval

$Vol\_passage_k$ is the traffic count of passage detector at the kth time interval

$Occ\_passage_k$ is the occupancy of passage detector at the kth time interval

a, b, c, d, e, f, g, h are the parameters in regression model.

Kalman Filtering Model:

In general, Kalman Filtering Model consists of two equations, one called state equation, which represents the change from state $k-1$ to state k; the other is measurement equation, which represents the relationship between measurement and the real value. Kalman Filtering requires that these two equations are independent, which means that the information used in state equation should be independent with the information of measurement equation. In this case, the conservation equation (Eq.6-1) can treat as state equation, which represents the change of queue size from state $k-1$ to state k. The state equation uses the information of traffic counts of queue detectors and passage detectors. However, building the measurement equation is not a trivial task. Some research used rough counts to build the measurement equation (Gazis and Knapp, 1971), but in our case, there is no such kind of information. Kurkjian et al (1980) built the linear relationship of occupancy and density and used occupancy to build measurement equation. However, this method is only feasible for homogeneous traffic flow. Although the Kurkjian et al (1980) method cannot be applied directly, this research gives some ideas to build the measurement equations. Because only information of traffic counts and occupancy is available, and traffic counts are used to build the state equation, the measurement equation only can be built by using occupancy. Therefore, we first fit between the real queue size and the occupancy data from queue and passage detectors, and then use the regression model to build the measurement equation. The state equation is combined (i.e. conservation equation) with the measurement equation to build the Kalman Filtering Model:

$$\begin{cases} x_k = x_{k-1} + U_k - V_k + \xi_k & (6-5.1) \\ Z_k = f(Occ\_queue_k, Occ\_passage_k) + \varepsilon_k & (6-5.2) \end{cases}$$

Where:

$x_k \geq 0, U_k, V_k$ are the same as defined before

ξ_k is the noise of state equation at the kth time interval

ε_k is the noise of state equation at the kth time interval

Occ_queue_k is occupancy of queue detector at the kth time interval

$Occ\_passage_k$ is occupancy of passage detector at the kth time interval

Z_k is the queue size calculated by occupancy of queue and passage detector

$f(Occ\_queue_k, Occ\_passage_k)$ is the function of occupancy of queue and

passage detector, which is used to estimate the queue size.

Now, the problem is to appropriately fit the model, and build the functional form of $f(Occ\_queue_k, Occ\_passage_k)$ between the real queue size and occupancy data of queue and passage detectors. As mentioned before, a simple linear model is used. The detailed model is as following:

$$f(Occ\_queue_k, Occ\_passage_k) = a1 * Occ\_queue_k + b1 * Occ\_passage_k \\ + c1 * Occ\_queue_{k-1} + d1 * Occ\_passage_{k-1} \quad (6\text{-}5.3)$$

Where:

$a1, b1, c1, d1$ are parameters.

The equations (6-5.1), (6-5.2) and (6-5.3) form the Kalman Filtering Model. The detailed description of Kalman Filtering along with the solution are no described here and can be found in Kalman (1960) and Myers and Tapley (1976).

Chapter 7 Evaluation Results for Improved Ramp Queue Estimation

7.1 Test Sites and Problematic Ramps Identification

Two test sites were selected in this study because their geometric properties and traffic characteristics are representative in Twin Cities freeways: TH-169 NB and I-94EB. The detailed information of these two test sites can be found in section 4.4.

Due to the cost and time considerations, only ramps with counting errors were taken as test ramps among 29 metered ramps (17 from TH-62 NB, 12 from I-94 EB). Therefore, the next step would be to identify the problematic ramps among the 29 ramps.

Identification of Problematic Ramps:

One of the methods to identify the problematic ramps with counting error of detectors is to analyze existing volume data between queue and passage detectors. A method termed "h *ratio*" is applied to test whether the detectors have counting error and how big the error is. The basic idea of "h *ratio*" is to compare the traffic counts of passage detector (Output) and traffic counts of queue detector (Input). If there is no counting error, h should be equal or very close to 1 according to the conservation law. This study compares the each 15 minutes traffic counts of passage and queue detectors during 2-8pm. The formula is used as following:

$$h = \frac{\text{15 min. passage detector counts (Outout)}}{\text{15 min. queue detector counts (Input)}} \qquad \text{(6-6)}$$

If h is very close to 1 and fluctuates around 1, it means the detectors have no or minor counting error and this kind of ramps belongs to Class I. If h is always larger than 1 or less than 1, systematic counting error may exist and these ramps belong to Class II or Class III ramps. In this study, if h is less than 1.02 or larger than 0.98, (i.e. the counting error is less than 2%), these ramps are considered error-free ramps and belong to Class I ramps. Otherwise, these ramps belong to Class II or Class III ramps. In order to separate Class II and Class III ramps, real traffic counts of queue detectors measured by video are compared with traffic counts recorded by queue detectors. The analysis result is described in Table 7.1. The result indicates that there are about 10 ramps (among 29 ramps) that the h *ratio* is larger than 1.02 or less than 0.98. Also, for testing Method I, one metered ramp with little error ($h \le 0.98$) is added.

7.2 Data Collection

The data required in this study includes the real queue size, which is used for fitting Regression Model and Kalman Filtering Model, and the real traffic counts of queue detector, which is used for classifying Class II and Class III ramps. In order to measure these data, the video data is used. Surveillance cameras in Mn/DOT's Regional Transportation Management Center (RTMC) were used to record the queue size and traffic counts on each ramp. Due to the locations of camera, among 11 ramps, only 8 ramps could be recorded (See Table 7.1). And by comparing the real traffic counts of queue detector from the video and traffic counts recorded by queue detectors, it was found that two ramps belong to Class II ramps. On these two ramps, the traffic counts of queue detectors are correct while the traffic counts of passage detector are biased. These two ramps can apply Method II to estimate the queue size.

Table 7.1 Problematic Ramp Identification (By h ratio)

Ramp		Mean	Stdev	Maximum	Minimum	Statement	Ramp Type	Record
Th-169 NB	TH62EB	**0.91**	0.17	1.04	0.46	Over count	**Class II**	Recorded
	Bren Road	**1.06**	0.10	1.15	0.91	Undercount	Class III	Recorded
	Excelsior	**1.27**	0.21	1.74	1.03	Undercount	Class III	**Non-Recorded**
	I394WB	**1.08**	0.11	1.35	0.99	Undercount	Class III	Recorded
	Bettycrocker	**1.04**	0.12	1.40	0.89	Undercount	Class III	**Non-Recorded**
	Plymouth	**1.06**	0.10	1.35	0.97	Undercount	Class III	Recorded
I-94 EB	Hennepin	**1.26**	0.19	1.58	0.94	Undercount	Class III	Recorded
	6th ST	**1.28**	0.54	2.73	0.62	Undercount	Class III	**Non-Recorded**
	Cretin	**0.84**	0.05	1.00	0.82	Over count	Class III	Recorded
	Lexington	**1.02**	0.09	1.25	0.86	Undercount	**Class I**	Recorded
	Dale	**0.98**	0.04	1.03	0.82	Over count	**Class II**	Recorded

Table 7.2 Comparison of Three models for Three Classes Ramps

Ramp		*h ratio*	Model I: Conservation Model		Model II: Regression Model		Model III: Kalman Filtering Model		Mn/DOT's Algorithm	
			Mean	St.dev	Mean	St.dev	Mean	St.dev	Mean	St.dev
Class I	Lexington	1.02	-0.17	1.40	0.88	3.26	-0.13	2.35	14.668	2.6903
Class II	TH62EB	0.91	-0.44	2.10	0.42	4.37	2.75	7.45	9.47	5.09
	Dale	0.98	-0.25	0.94	-1.12	7.62	-1.89	8.68	17.92	10.49
Class III	Bren Road	1.06	6.00	5.46	0.77	4.70	-0.14	5.04	9.27	5.98
	I394WB	1.08	8.98	5.18	-1.99	3.14	-1.21	4.51	0.64	4.50
	Plymouth	1.06	6.78	4.04	0.68	2.19	-0.39	2.38	-0.18	3.51
	Hennepin	1.26	3.80	4.90	-0.31	4.38	-1.62	3.85	18.47	5.50
	Cretin	0.84	4.99	4.88	0.98	3.87	-0.08	4.17	3.22	5.65

7.3 Test Results of Ramp Queue Estimation

After classification of ramps and recording the real queue size, the proposed methods can be tested. In order to compare the different methods, all three models are applied (Conservation Model, Regression Model and Kalman Filtering Model) to three classes of ramps.

Class I Ramps:

One Class I Ramp, Lexington ramp on I-94EB, has been tested. Table 7.2 presents the statistic data, including mean and standard deviation of error, of different methods. The data indicates that all three new methods are significantly better than Mn/DOT's current methodology. Figure 7.1 uses the accumulative probability of error (The X axis is the error and the Y axis is the accumulative probability of error) to describe that the three new methods are better than Mn/DOT's method. Figure 7.1 indicates that the Conservation Model is better than two other models, Regression Model and Kalman Filtering Model. This conclusion is also supported by Figure 7.2, which compares the estimations of queue size of different methods and real queue size measured by video. All these results suggest that for Class I ramps, Conservation Model is the best one for queue size estimation.

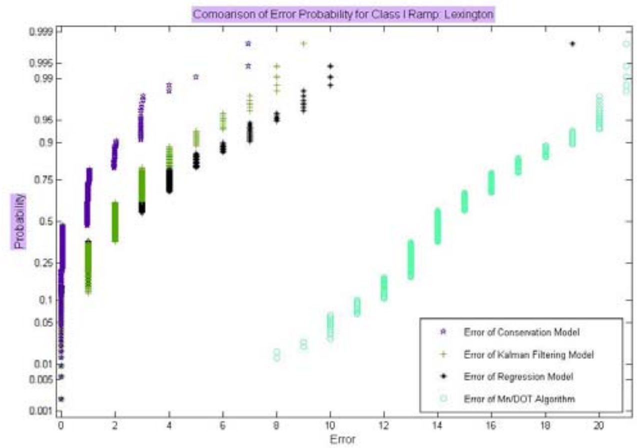


Figure 7.1 Comparison of Error Probability for Class I Ramps: Lexington

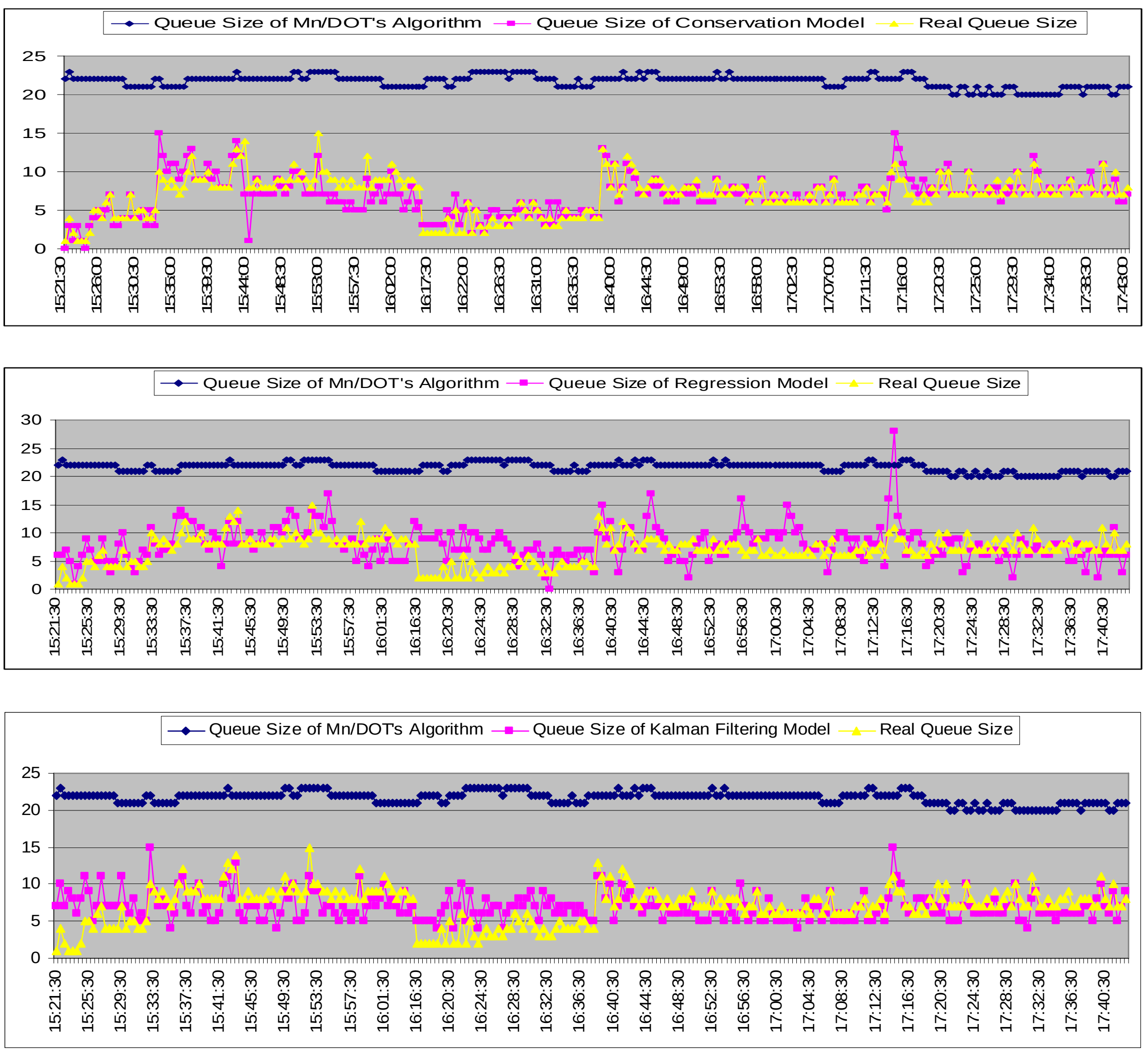


Figure 7.2 Comparison of Queue Size Estimation for Class I Ramps: Lexington

Class II Ramps

For the second class ramps, green counts are used to replace the traffic counts of passage detector and apply Conservation Model again. Two ramps, TH62EB on TH-169 NB and Dale on I-94 EB, belong to this class and are tested. The comparisons are described in Table 7.2, Figure 7.3, Figure 7.4, Figure 7.5 and Figure 7.6. Table 7.2 indicates that the error of Conservation Model is the smallest and Figure 7.3-7.6 suggest that the estimated queue size by Conservation Model matches the real queue size very well and evidently better than the other two models.

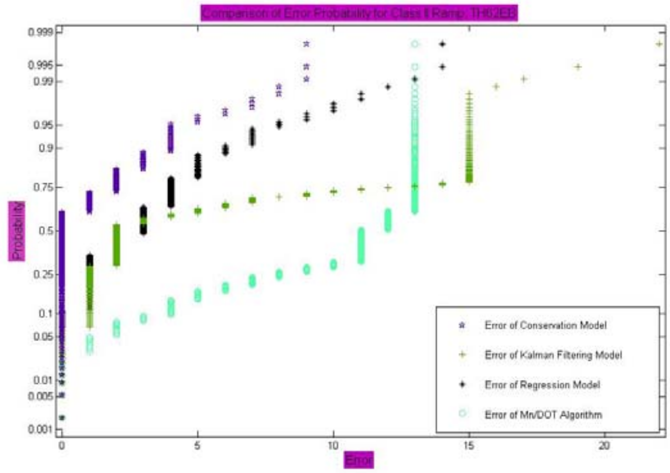


Figure 7.3 Comparison of Error Probability for Class II Ramps: TH62EB

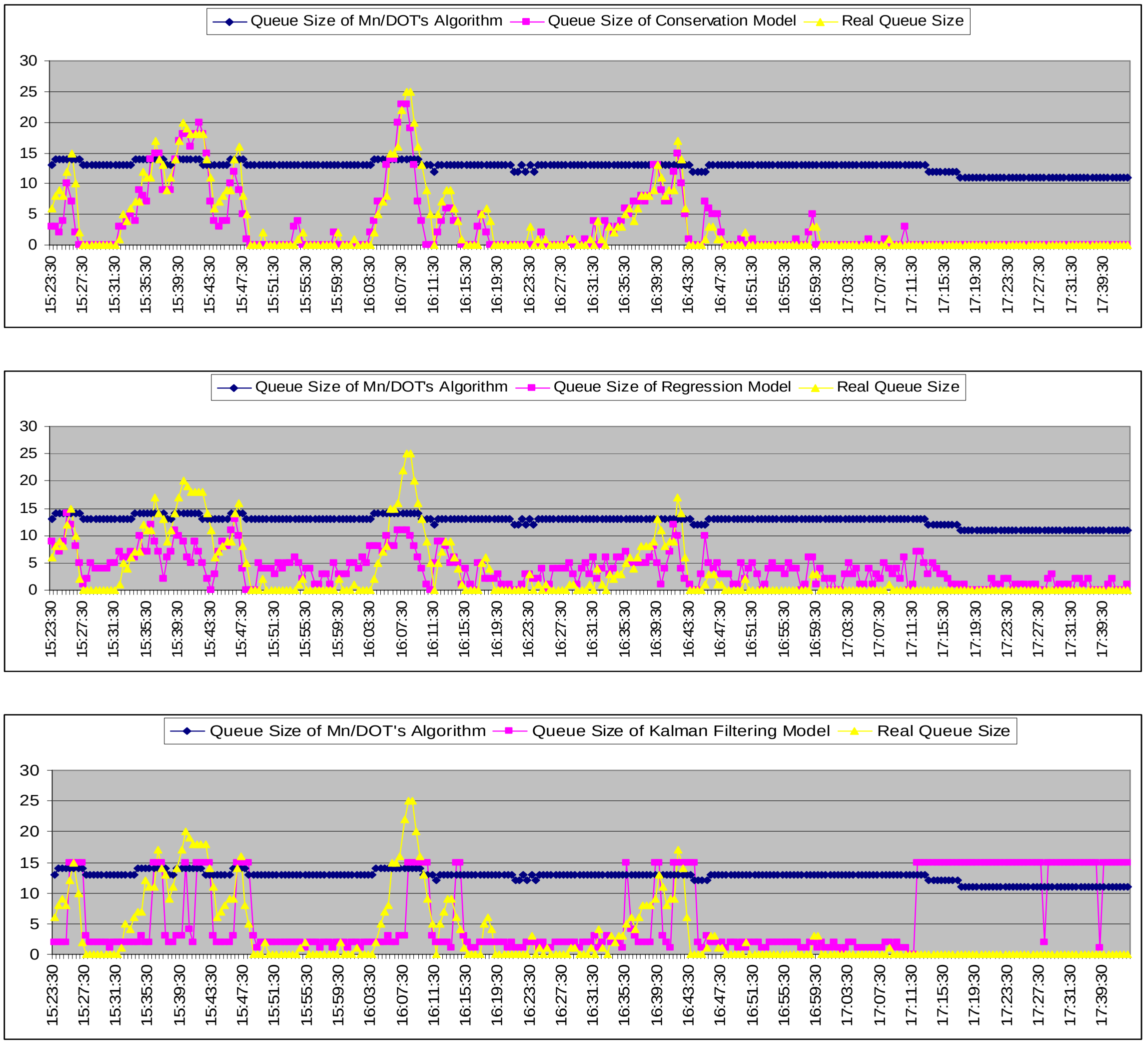


Figure 7.4 Comparison of Queue Size Estimation for Class II Ramps: TH62EB

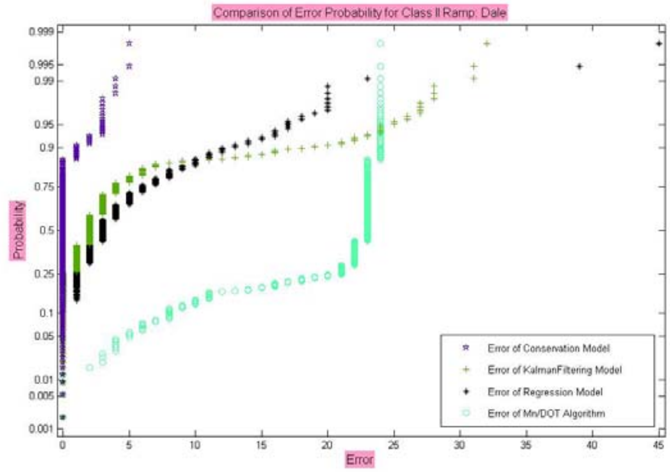


Figure 7.5 Comparison of Error Probability for Class II Ramps: Dale

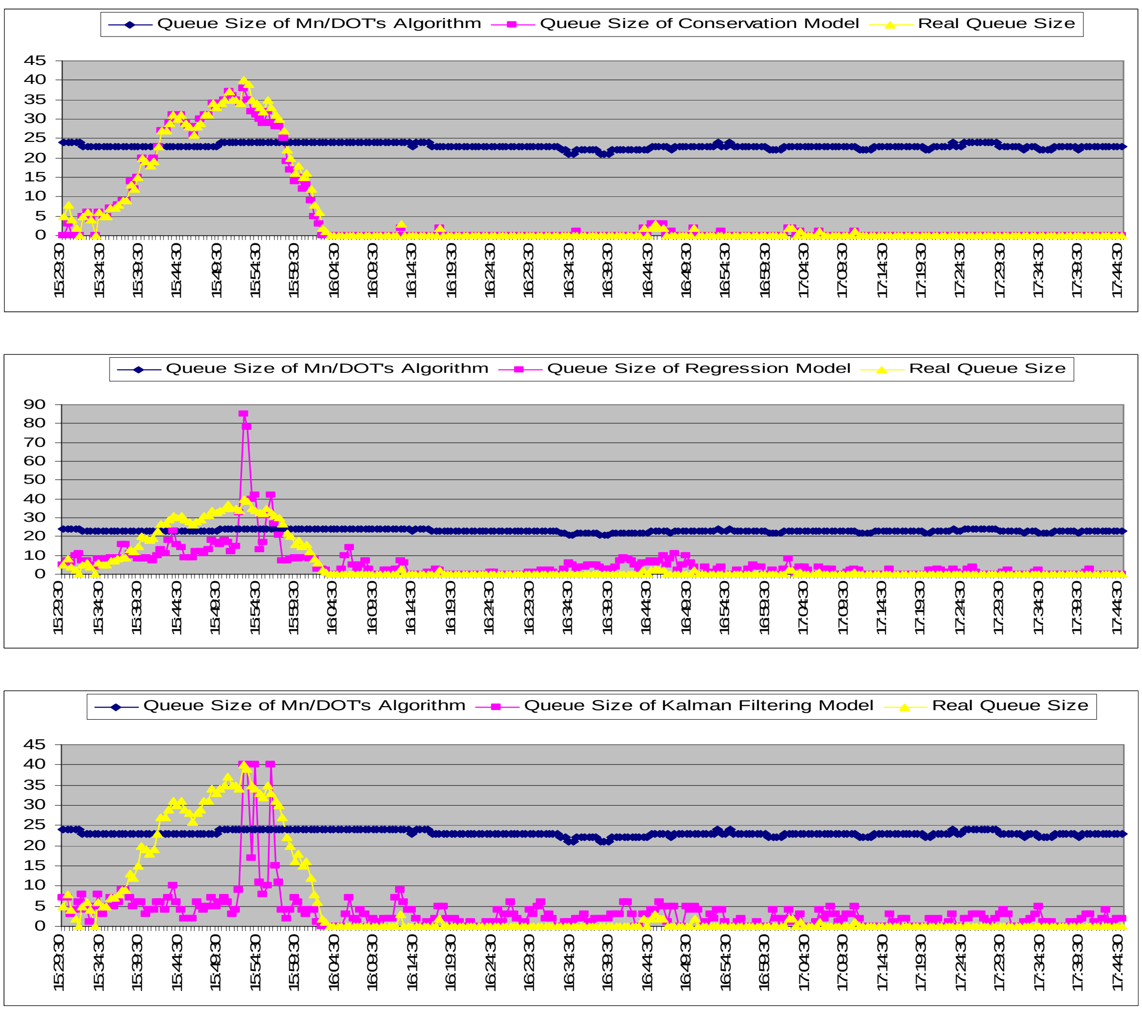


Figure 7.6 Comparison of Queue Size Estimation for Class II Ramps: Dale

Class III Ramps:

Five Class III Ramps are tested and the results of two representative ramps are explained here. One is Plymouth Ramp on TH169NB, which has relative small error ($h\ ratio$ equals 1.06), the other is Cretin Ramp on I94EB, which has significant error ($h\ ratio$ equals 0.84). Plymouth Ramp represents the ramp with the relative small error which is less than 10% but larger than 2% while Cretin Ramp has a significant error larger than 10%. Also, the Cretin Ramp is a classical ramp which is over counting. The comparison results are described in Table 7.2 and Figure 7.7~7.10. The mean and standard deviation of error in Table 7.2 indicate that for class III ramps, both the Regression Model and Kalman Filtering Model can estimate good results while the Conservation Model cannot produce accurate queue size. This conclusion also can be seen from Figure 7.9 and Figure 7.10 which compare the estimations of queue size of different models and the real queue size. One thing that needs to explain here is that for I-394WB Ramp and Plymouth Ramp, the mean error of the queue size estimated by Mn/DOT's current algorithm in Table 7.2 is very small. However, it does not mean that this algorithm can estimate the accurate queue size. Actually, the queue size estimated by Mn/DOT's current method almost is a constant. This is described in Figure 7.8 and Figure 7.10. The blue line is the queue size estimated by Mn/DOT. It almost is a straight line. The comparisons of queue sizes in Figure 7.8 and Figure 7.10 indicate that the two new models: Regression Model and Kalman Filtering Model can estimate the accurate queue sizes which are very close to real queue size. After carefully comparing Regression Model and Kalman Filtering Model, it indicates that the Kalman Filtering Model is a little better that Regression Model, but the differences are not significant. However, the Kalman Filtering Model needs some other parameters, such as the noise of state equation and the noise of measurement equation. Therefore, the Regression Model is more favorable because of its simplicity.

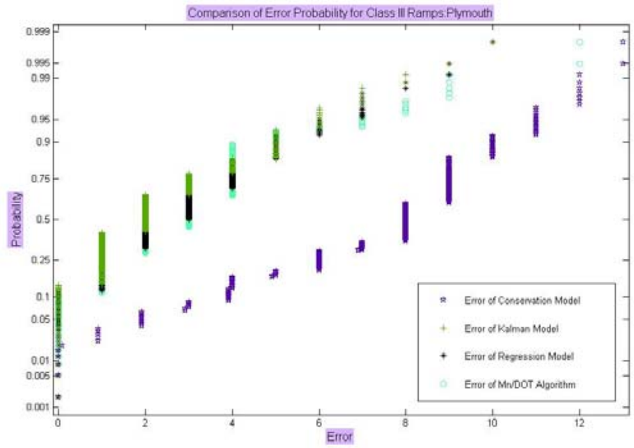


Figure 7.7 Comparison of Error Probability for Class III Ramps: Plymouth

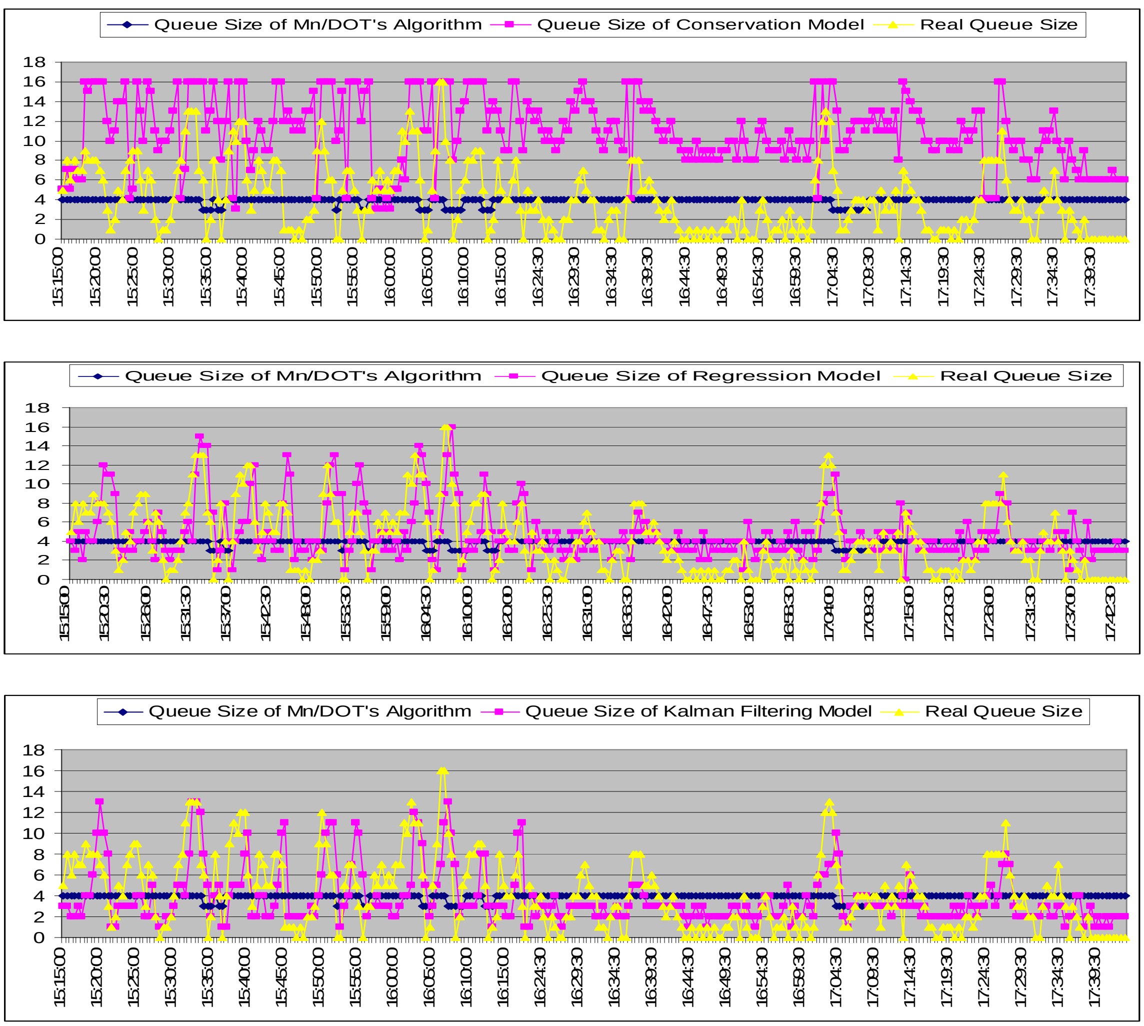


Figure 7.8 Comparison of Queue Size Estimation for Class III Ramps: Plymouth

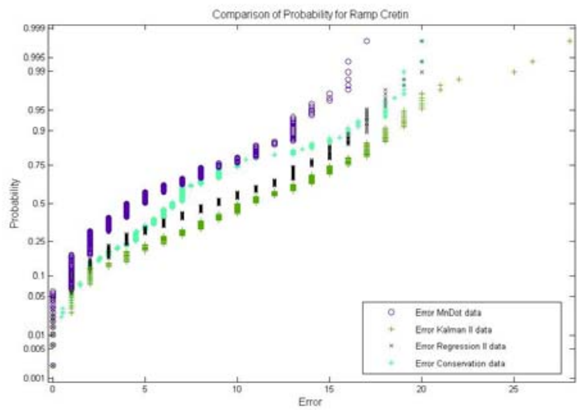


Figure 7.9 Comparison of Error Probability for Class III Ramps: Cretin

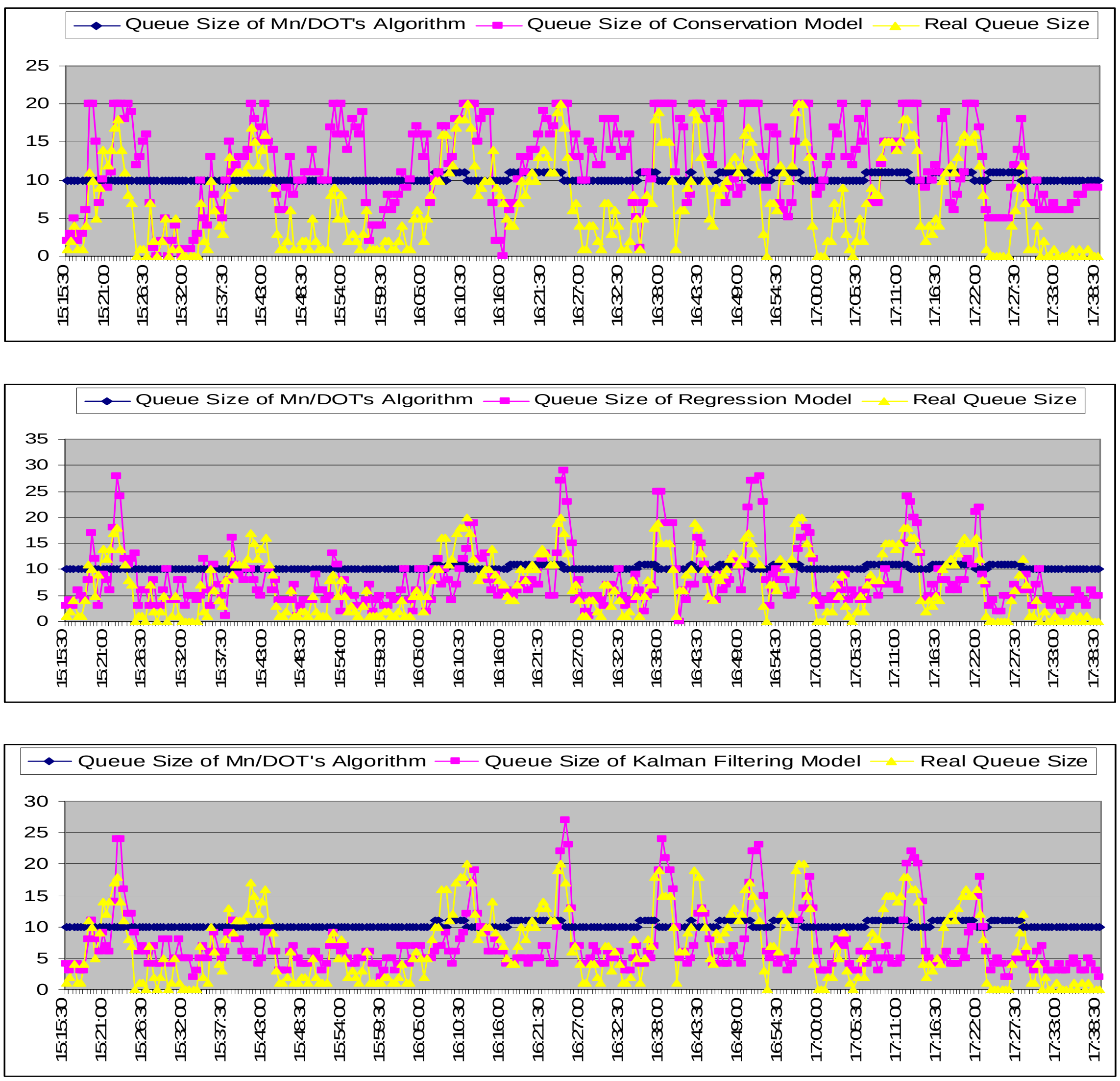


Figure 7.10 Comparison of Queue Size Estimation for Class III Ramps: Cretin

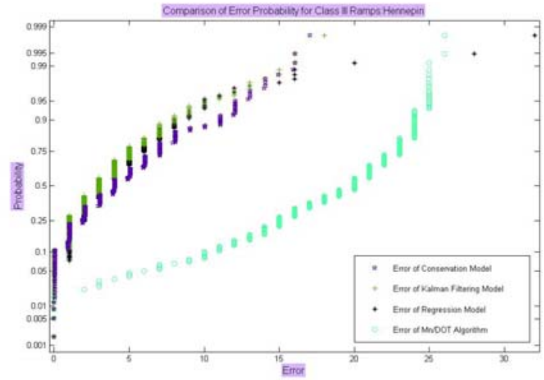
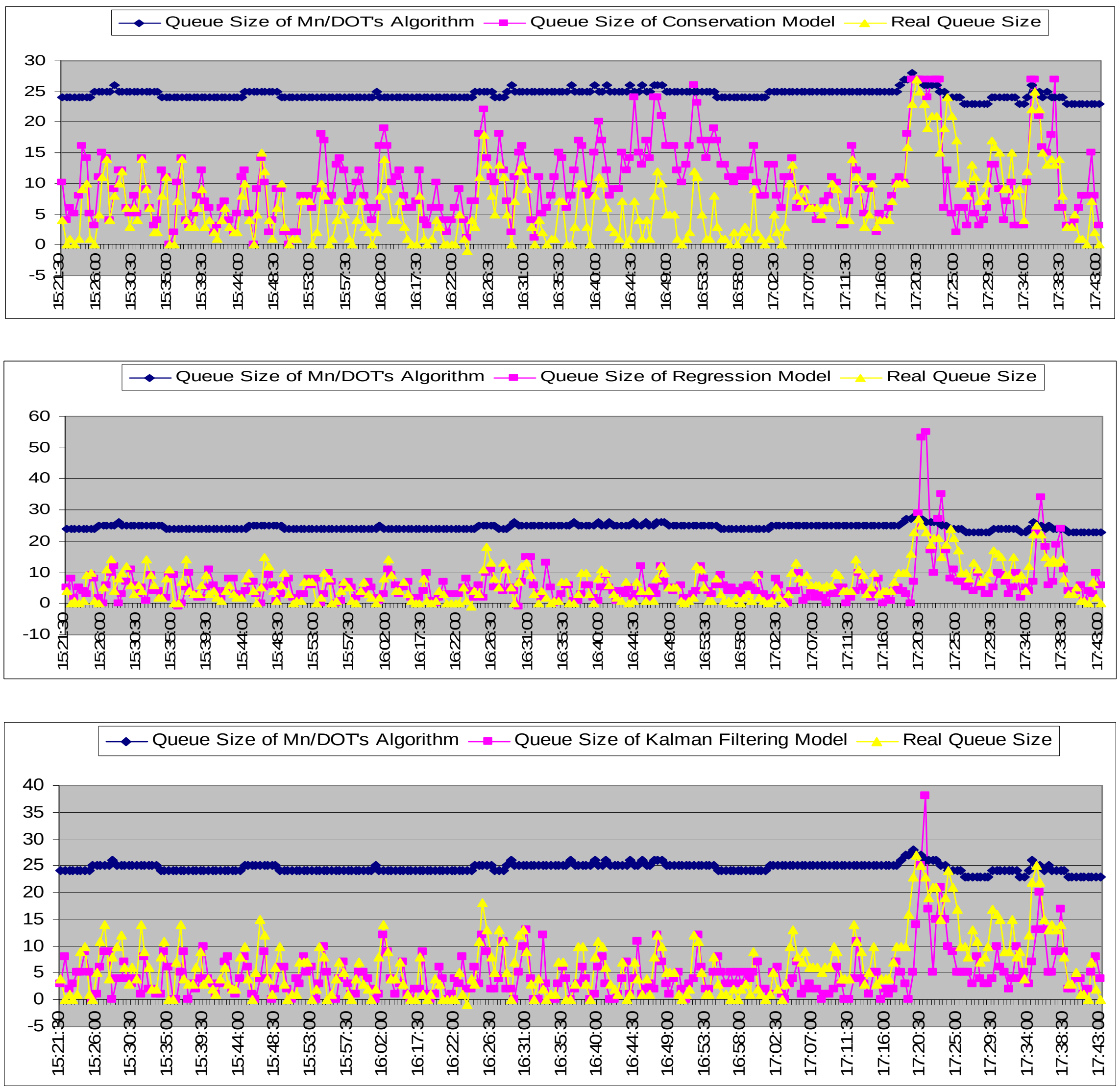


Figure 7.11 Comparison of Error Probability for Class III Ramps: Hennepin



7.12 Comparison of Queue Size Estimation for Class III Ramps: Hennepin

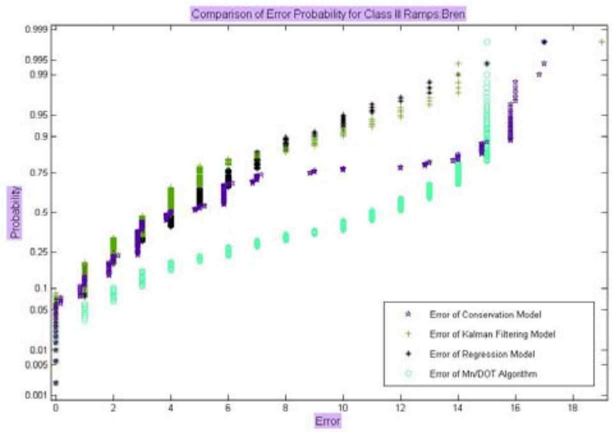
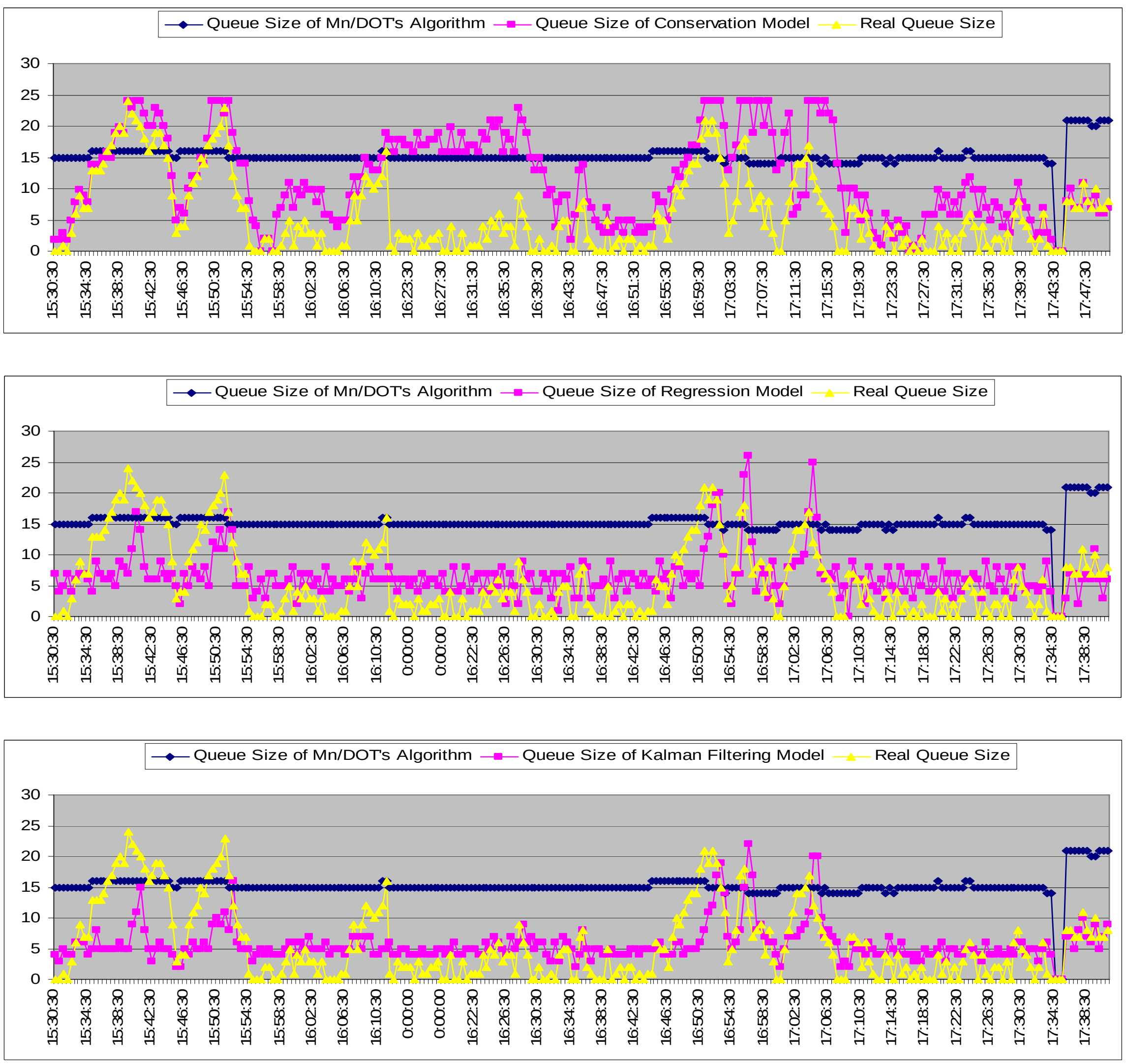


Figure 7.13 Comparison of Error Probability for Class III Ramps: Brenn



7.14 Comparison of Queue Size Estimation for Class III Ramps: Brenn

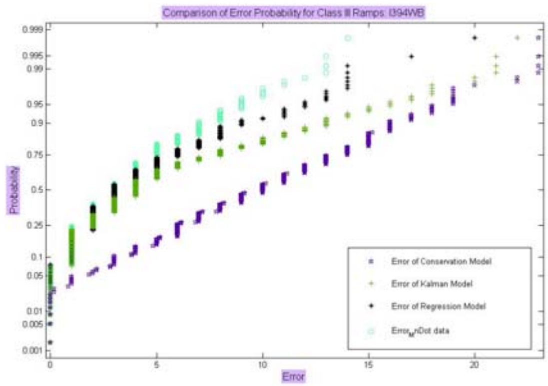
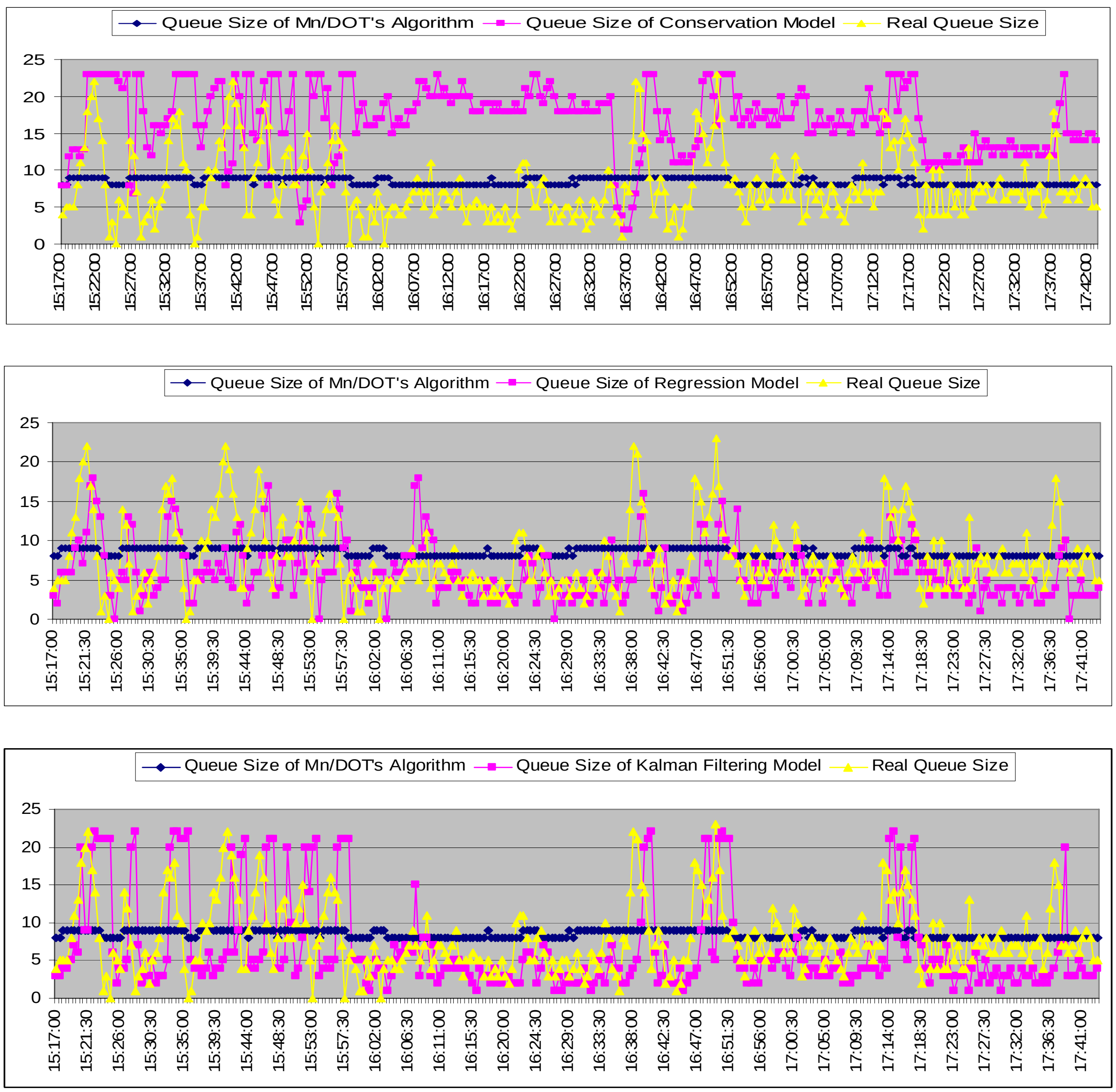


Figure 7.15 Comparison of Error Probability for Class III Ramps: I394WB



7.16 Comparison of Queue Size Estimation for Class III Ramps: I394WB

Chapter 8 System Evaluation with Improved Ramp Queue Estimation

After testing the accuracy of improved queue size estimation algorithm, micro-simulator was applied to assess the effectiveness of improved methodologies for queue size estimation. In order to save the cost and time, the same two test sites – TH169NB and I-94EB and the same test time were selected to do testing.

In the testing, MOEs (measures of effectiveness), including the total freeway mainline delay, total freeway mainline travel time, travel speed etc., are selected to evaluate the performance of both the original and the enhanced SZM strategies. The test results for the entire simulation period (14:00-20:00) and for the metering period (15:00-18:00) are shown in the Table 8.1 and 8.2. These tables show the percentage change between the two scenarios: the SZM control and the enhanced SZM control. The base case for the comparison is the original SZM control. Thus, a positive percentage change means that this MOE increased with the enhanced SZM strategy and vice versa. Following are the analyses for PM peak hours (15:00-18:00).

As indicated in Table 8.2, the performance of freeway mainline is greatly improved. For example, *the total number of stops* on the freeway mainline decreases significantly. The percentage decrease varies from 13.62% to 30.22% for three test days on TH169NB. And for I94EB, the stop times decrease about 2~3%. The reduction indicates that the improved SZM control significantly increases the smoothness of the mainline flow. Based on known associations between accident rates and speed variance (Garber, et. Al., 1989; Oh, C., et al., 2001), increasing flow smoothness should result in reduction of accident likelihood. Another important MOE index is *total freeway mainline travel time.* As suggested in these two tables, the reduction of total travel time varies from 2.64% to 8.46% for TH169 NB and from 1.01% to 1.70% for I94EB. The reduction on TH169 NB is much higher than I94EB because of more congestion on the I94EB. The reduction indicates that the improved methodology decreases the congestions. Also, this table indicated the similar trend for the *total freeway delay.* It is significantly reduced under the improved SZM control. Specifically, the total freeway delay is reduced to 20.84% for Nov. 08, TH169NB. And for OCT 26 on I94EB, the reduction is 3.20%. In addition to the above favorable results, the *freeway average speed* in both test days improves under the improved SZM strategy. The improvement varies from 2.74% to 9.34% for TH169NB and from 1.06 to 1.66 for I94EB. All these test results show that the improved SZM control with new bottleneck capacity estimation significantly improve the performance of mainline freeway.

For the performance of ramp system, different test sites have different results. For TH169NB, the MOEs, including *total ramp travel time, total ramp delay* and *average ramp delay* increased, which mean that the performance of ramp system decreased. These results are reasonable. It is a tradeoff of the significant improvement of freeway mainline system. However, different scenarios happened on I94EB. The *total ramp travel time* decreased from 2.93% to 9.08% and the *total ramp delay* decreased from 4.78% to 12.94%, which mean that the improved SZM control increases the performance of ramp system. The possible reason for different testing results is because of the different traffic patterns in these two test sites. It is more congested on I94EB than on TH169NB. Therefore, the original SZM control often overestimates the queue size for TH169NB while underestimates the queue size for I94EB. Therefore, the improved queue size estimation algorithm increased the delay of ramps for TH169NB and decreased the ramp delay for I94EB. Another important ramp MOEs is maximum waiting time on each ramp as shown in Table 8.3, 8.4, 8.5 and 8.6. From these tables, we can see, although the improved SZM control increased the delay on each ramp, the maximum waiting time is still less than the 4 minutes at most of the time. Therefore, the new algorithm did not compromise the maximum delay constraint.

The testing results also indicate that the total system performance increased. During PM peak hours, the *total system travel time* decreased from 0.50% to 2.03% and the *average system speed* increased from 0.69% to 2.16% for TH169NB. And for I94EB, the *total system travel time* decreased from 1.26% to 2.47% and the *average system speed* increased from 1.32% to 2.46%. Other MOEs, such as *fuel Consumption and pollutants emissions*, have significantly decrease, which means that the new algorithm improves the air condition.

Table 8.1 Percentage Change for Major MOEs for Queue Size Estimation (Enhanced SZM control with improved queue size estimation over SZM control) Metering Period (2:00pm to 8:00pm)

Categories	% Change		TH-169NB			I-94EB		
			NOV 08	NOV 13	NOV 27*	OCT 26*	NOV 01	NOV 27
Freeway MOEs (Mainline)	Total Number of Stops		-29.63%	-16.09%	-13.46%	-3.20%	-1.25%	-2.36%
	Number of Stops Per Veh		-29.63%	-16.09%	-13.46%	-3.20%	-1.25%	-2.36%
	Total Freeway Travel Time (veh-hours)		-5.41%	-1.47%	-3.36%	-1.15%	-0.87%	-0.70%
	Total Freeway Travel (veh-miles)		-0.02%	0.02%	0.09%	-0.01%	-0.01%	-0.02%
	Total Freeway Delay (veh-hours)		-19.78%	-8.10%	-8.65%	-3.44%	-2.37%	-2.46%
	Average Freeway Delay (min/veh)		-19.78%	-8.10%	-8.65%	-3.44%	-2.37%	-2.46%
	Volume (vehicles serviced by freeway)		0.00%	0.00%	0.00%	0.00%	0.00%	0.00%
	Average Speed (mile/hour)		5.70%	1.51%	3.57%	1.15%	0.87%	0.68%
Ramp MOEs	Total Ramp Travel Time (veh-hours)		48.92%	17.80%	50.23%	-6.69%	-1.82%	-2.44%
	Total Ramp Travel (veh-miles)		0.01%	0.01%	0.02%	0.00%	0.00%	0.00%
	Total Ramp Delay (veh-hours)		65.62%	29.23%	76.44%	-12.83%	-3.80%	-5.67%
	Average Ramp Delay (min/veh)		65.62%	29.23%	76.44%	-12.83%	-3.80%	-5.67%
	Volume (vehicles entered from ramps)		0.00%	0.00%	0.00%	0.00%	0.00%	0.00%
System MOEs	Total System Travel Time (veh-hour)		-1.43%	-0.39%	-0.49%	-1.63%	-0.94%	-0.83%
	Average System Speed (mile/hour)		1.44%	0.41%	0.58%	1.64%	0.94%	0.82%
	Fuel Consumption (gallons)		-8.82%	-3.22%	-4.20%	-1.41%	-1.16%	-1.13%
	Pollutants Emissions (kgs)	CO	-4.12%	-0.75%	-1.89%	-1.51%	0.08%	-0.93%
		HC	-2.92%	-0.40%	-1.01%	-1.31%	-0.04%	-0.95%
		NO	-5.38%	-1.13%	-2.65%	-1.70%	0.17%	-1.07%

Table 8.2 Percentage Change for Major MOEs for Queue Size Estimation (Enhanced SZM control with improved queue size estimation over SZM control) Metering Period (3:00pm to 6:00pm)

Categories		% Change	TH-169NB			I-94EB		
			NOV 08	**NOV 13**	**NOV 27***	**OCT 26***	**NOV 01**	**NOV 27**
Freeway MOEs (Mainline)	Total Number of Stops		-30.22%	-17.93%	-13.62%	-2.76%	-1.67%	-2.24%
	Number of Stops Per Veh		-30.22%	-17.93%	-13.64%	-2.76%	-1.61%	-2.24%
	Total Freeway Travel Time (veh-hours)		-8.46%	-2.64%	-4.88%	-1.70%	-1.48%	-1.01%
	Total Freeway Travel (veh-miles)		0.09%	0.03%	0.18%	-0.07%	-0.06%	0.04%
	Total Freeway Delay (veh-hours)		-20.84%	-10.23%	-9.25%	-3.20%	-2.86%	-2.30%
	Average Freeway Delay (min/veh)		-20.85%	-10.23%	-9.31%	-3.20%	-2.81%	-2.30%
	Volume (vehicles serviced by freeway)		0.00%	0.00%	0.00%	0.00%	-0.05%	0.00%
	Average Speed (mile/hour)		9.34%	2.74%	5.30%	1.66%	1.44%	1.06%
Ramp MOEs	Total Ramp Travel Time (veh-hours)		59.80%	24.74%	66.19%	-9.08%	-2.93%	-3.78%
	Total Ramp Travel (veh-miles)		0.01%	0.03%	0.04%	0.00%	-0.07%	0.00%
	Total Ramp Delay (veh-hours)		66.10%	29.87%	77.63%	-12.94%	-4.78%	-5.75%
	Average Ramp Delay (min/veh)		66.10%	29.87%	77.05%	-12.94%	-4.72%	-5.75%
	Volume (vehicles entered from ramps)		0.00%	0.00%	0.00%	0.00%	-0.06%	0.00%
System MOEs	Total System Travel Time (veh-hour)		-2.03%	-0.71%	-0.50%	-2.47%	-1.59%	-1.26%
	Average System Speed (mile/hour)		2.16%	0.74%	0.69%	2.46%	1.56%	1.32%
	Fuel Consumption (gallons)		-10.87%	-5.06%	-5.21%	-1.74%	-1.65%	-1.14%
	Pollutants Emissions (kgs)	CO	-5.71%	-1.25%	-2.58%	-1.65%	0.30%	-0.94%
		HC	-4.10%	-0.71%	-1.36%	-1.71%	0.08%	-0.97%
		NO	-7.15%	-1.83%	-3.40%	-1.78%	0.37%	-1.07%

Table 8.3 MOEs for Ramp Performance on TH169NB Queue Size Estimation (Enhanced SZM control with improved queue size estimation over SZM control) NOV, 08 2000 (3:00pm to 6:00pm)

MOE / Ramps	Average Ramp Wait Times (minutes)		Max Ramp Wait Times (minutes)		Total Ramp Delay (vehicle-hours)		Average Queue Size (vehicles)		Max Queue Size (vehicles)	
	SZM	QSZM	SZM	QSZM	SZM	QSZM	SZM	QSZM	SZM	QSZM
36thSt	0.14	1.07	0.47	2.49	2	14	0	3	3.85	10.20
BettyCrocker	0.22	0.78	1.08	2.25	3	11	1	2	8.20	14.15
Brenn	0.52	0.47	1.72	1.61	17	15	4	3	18.40	18.60
Cedar	0.15	0.15	0.57	0.57	1	1	0	0	4.75	4.75
Excelsior	1.51	1.39	4.45*	3.23	46	43	10	10	26.70	25.60
I394EB	0.25	1.60	1.05	3.36	5	35	1	8	8.25	21.80
I394WB	1.30	1.97	2.42	4.07*	48	73	11	17	23.30	27.70
Lincoln	0.18	0.17	0.50	0.41	1	1	0	0	4.55	3.85
MedicineLake	2.38	1.59	6.53*	2.96	43	29	10	6	27.85	17.10
Minnetonka	0.16	0.56	0.46	2.40	2	6	0	1	3.35	9.10
Plymouth	0.23	0.55	0.40	1.45	5	11	1	2	5.70	6.45
TH55EB	0.14	1.04	0.38	2.55	2	13	0	3	4.35	13.50
TH55WB	0.10	0.79	0.31	1.42	1	11	0	3	2.85	7.05
TH62EB	0.27	0.36	1.22	1.40	10	13	2	3	17.55	17.30
TH62WB	0.09	0.53	0.47	1.55	3	18	0	4	7.00	18.45
TH7	0.19	1.56	0.85	3.18	4	36	1	8	6.70	20.55
ValleyView	0.26	0.18	2.43	2.29	9	6	2	1	21.25	19.75

Table 8.4 MOEs for Ramp Performance on TH169NB Queue Size Estimation (Enhanced SZM control with improved queue size estimation over SZM control) NOV, 27 2000 (3:00pm to 6:00pm)

MOE / Ramps	Average Ramp Wait Times (minutes)		Max Ramp Wait Times (minutes)		Total Ramp Delay (vehicle-hours)		Average Queue Size (vehicles)		Max Queue Size (vehicles)	
	SZM	QSZM	SZM	QSZM	SZM	QSZM	SZM	QSZM	SZM	QSZM
36thSt	0.11	0.64	0.46	2.40	1	9	0	2	2.80	9.80
BettyCrocker	0.32	0.73	3.01	2.56	5	10	1	2	18.15	18.20
Brenn	0.40	0.37	1.37	1.43	12	11	3	2	16.30	16.95
Cedar	0.12	0.12	0.21	0.21	1	1	0	0	2.00	2.00
Excelsior	0.90	0.87	3.79	3.30	26	25	6	6	24.50	24.35
I394EB	0.73	1.44	3.62	3.22	16	32	4	7	24.25	24.45
I394WB	0.75	1.82	2.42	4.06*	27	65	6	15	23.30	24.80
Lincoln	0.20	0.21	1.15	0.78	2	2	0	0	6.60	6.15
MedicineLake	1.76	1.46	6.23*	2.69	29	24	7	5	23.05	14.15
Minnetonka	0.14	0.42	0.39	2.32	2	5	0	1	3.35	7.05
Plymouth	0.19	0.17	0.36	0.36	4	3	1	1	5.85	5.10
TH55EB	0.14	0.85	0.45	2.28	2	11	0	2	4.75	11.70
TH55WB	0.13	0.39	0.42	1.02	2	6	0	1	4.30	7.30
TH62EB	0.25	0.25	1.11	1.36	8	8	2	2	15.60	15.45
TH62WB	0.07	0.38	0.41	1.55	2	12	0	3	7.15	17.50
TH7	0.11	1.34	0.45	3.38	3	31	0	7	4.95	20.55
ValleyView	0.15	0.06	1.24	0.54	5	2	1	0	12.90	7.70

*The maximum allowed ramp wait time is violated

Table 8.5 MOEs for Ramp Performance on I-94EB Queue Size Estimation (Enhanced SZM control with improved queue size estimation over SZM control) Oct, 26 2000 (3:00pm to 6:00pm)

MOE / Ramps	Average Ramp Wait Times (minutes)		Max Ramp Wait Times (minutes)		Total Ramp Delay (vehicle-hours)		Average Queue Size (vehicles)		Max Queue Size (vehicles)	
	SZM	QSZM	SZM	QSZM	SZM	QSZM	SZM	QSZM	SZM	QSZM
Lyndale Ave.	0.09	0.30	0.39	1.50	1	4	0	1	3.10	6.95
Hennepin Ave.	0.57	0.58	1.86	2.04	37	37	9	9	35.55	37.10
5th Ave	1.86	1.18	5.00*	2.87	41	26	10	6	28.50	18.85
6 Street	1.67	1.46	4.39*	3.82	87	76	22	19	55.95	55.70
Cedar Ave	1.12	0.68	3.66	2.94	25	15	6	4	17.25	13.25
Riverside Ave.	2.74	1.81	5.81	3.27	75	50	17	11	33.65	24.20
Huron Blvd	0.24	0.85	0.91	2.03	7	24	1	6	9.80	18.15
Cretin Ave.	1.08	1.32	2.45	2.93	31	38	7	9	17.25	18.85
Snelling Ave.	0.18	0.16	1.66	1.66	7	6	2	2	20.90	21.85
Lexington Ave	0.63	0.35	2.21	1.27	23	13	5	3	23.70	18.00
Dale Street	0.70	0.72	3.26	3.87	15	15	3	4	22.05	23.55
Marion Street	0.94	0.74	3.56	3.27	29	23	6	5	27.85	27.50

* The maximum allowed ramp wait time is violated

Table 8.6 MOEs for Ramp Performance on I-94EB for Queue Size Estimation (Enhanced SZM control with improved queue size estimation over SZM control) NOV, 27 2000 (3:00pm to 6:00pm)

MOE / Ramps	Average Ramp Wait Times (minutes)		Max Ramp Wait Times (minutes)		Total Ramp Delay (vehicle-hours)		Average Queue Size (vehicles)		Max Queue Size (vehicles)	
	SZM	QSZM	SZM	QSZM	SZM	QSZM	SZM	QSZM	SZM	QSZM
Lyndale Ave.	0.08	0.91	0.25	2.30	1	13	0	3	2.50	8.55
Hennepin Ave.	0.52	1.05	2.28	3.16	29	58	7	14	38.40	39.55
5th Ave	2.37	1.18	5.88*	3.01	52	26	12	6	28.50	18.65
6 Street	1.19	0.72	4.56*	4.90*	34	21	9	5	42.90	45.60
Cedar Ave	1.40	0.76	2.98	2.02	28	15	6	3	13.25	12.20
Riverside Ave.	3.12	0.59	5.73*	1.44	85	16	19	4	33.95	12.40
Huron Blvd	0.24	2.13	0.82	6.48*	6	56	1	13	6.85	29.45
Cretin Ave.	0.58	0.95	2.04	2.56	15	24	3	5	18.10	18.60
Snelling Ave.	0.01	2.31	0.12	4.15	0	81	0	19	4.40	36.15
Lexington Ave	0.11	1.21	1.12	4.14	3	33	1	7	15.15	29.90
Dale Street	0.17	0.10	2.00	1.39	3	2	1	0	10.30	12.20
Marion Street	0.14	0.03	2.17	0.31	4	1	1	0	21.60	5.10

*The maximum allowed ramp wait time is violated

Chapter 9 System Evaluation with Improved Ramp Control Logic and Queue Estimation

The final evaluation part is to assess the overall improvement. Two improvements, control logic and queue size estimation, are combined and evaluated by the same micro-simulator, AIMSUM. The same test sites and test time are selected. And the same MOEs are selected to evaluate the performance of both the original and the comprehensive enhanced SZM strategies. The test results are shown in the Table 9.1 and 9.2. Again, these tables show the percentage change between the two scenarios: the SZM control and the enhanced SZM control. The base case for the comparison is the original SZM control. Thus, a positive percentage change means that this MOE increased with the enhanced SZM strategy and vice versa. Following are the analyses for PM peak hours (15:00-18:00).

The simulation test results show that the performance of freeway mainline is significantly improved. In detail, the percentage of *total number of stops* decrease varies from 23.85% to 27.74% for three test days on TH169NB. And for I94EB, this number decreased from 3.73% to 14.71% during PM peak hour. The reduction indicates that the improved SZM control significantly increases the smoothness of the mainline flow and increases the safety for freeway mainline system. Also, as suggested in Table 9.2, the *total freeway mainline travel time* reduced from 3.52% to 8.94% for TH169 NB and from 1.71% to 3.50% for I94EB. The *total freeway delay* is significantly reduced under the improved SZM control. For example, the total freeway delay is reduced to 20.22% for Nov. 08, TH169NB. And for Nov 27 on I94EB, the reduction is as high as 9.18%. In addition to the above favorable results, the *freeway average speed* in both test days improved under the improved SZM strategy. The improvement varies from 3.68% to 10.22% for TH169NB and from 1.62% to 4.24% for I94EB. All these test results show that the improved SZM control with new bottleneck capacity estimation significantly improve the performance of mainline freeway.

However, one negative result is that the combined algorithm decreased the performance of ramp system. Both these two tables indicate that almost all the MOEs of ramp system has decreased, such as the *total ramp travel time, total ramp delay* and *average ramp delay*. But these results are reasonable. As we know, our objective is to improve the efficiency of freeway system as much as possible without compromising the predetermined threshold of the maximum waiting time at each entrance ramp. As shown in Table 9.3~9.6, the maximum waiting time for each vehicle is still less than 4 minutes for most of the test results. Therefore, the new algorithm achieved our goals.

Also, the testing results indicate that the total system performance increased. During PM peak hours, the *total system travel time* decreased from 1.55% to 4.87% and the *average system speed* increased from 1.60% to 5.49% for TH169NB. And for I94EB, the *total system travel time* decreased from 0.50% to 1.22% and the *average system speed* increased from 0.69% to 1.81%. It is a significant improvement. From system point, we say, the new algorithm is very efficient. Other MOEs, such as *fuel Consumption and pollutants emissions*, have significantly decrease, which means that the new algorithm decreases the air pollution.

Table 9.1 Comprehensive Evaluation: Percentage Change for Major MOEs (Comprehensive improved SZM control over SZM control) Metering Period
(2:00pm to 8:00pm)

Categories	% Change		TH-169NB			I-94EB		
			NOV 08	NOV 13	NOV 27*	OCT 26*	NOV 01	NOV 27
Freeway MOEs (Mainline)	Total Number of Stops		-27.20%	-21.25%	-25.42%	-4.11%	-6.94%	-14.54%
	Number of Stops Per Veh		-27.20%	-21.25%	-25.42%	-4.11%	-6.94%	-14.54%
	Total Freeway Travel Time (veh-hours)		-5.29%	-1.89%	-6.07%	-1.18%	-2.25%	-2.47%
	Total Freeway Travel (veh-miles)		-0.02%	0.03%	0.17%	0.00%	-0.02%	0.13%
	Total Freeway Delay (veh-hours)		-19.18%	-10.72%	-15.74%	-3.57%	-6.49%	-9.44%
	Average Freeway Delay (min/veh)		-19.18%	-10.72%	-15.74%	-3.57%	-6.49%	-9.44%
	Volume (vehicles serviced by freeway)		0.00%	0.00%	0.00%	0.00%	0.00%	0.00%
	Average Speed (mile/hour)		5.57%	1.96%	6.65%	1.17%	2.28%	2.67%
Ramp MOEs	Total Ramp Travel Time (veh-hours)		36.29%	16.78%	42.81%	4.67%	16.58%	14.79%
	Total Ramp Travel (veh-miles)		0.00%	0.02%	0.01%	0.00%	0.00%	0.00%
	Total Ramp Delay (veh-hours)		49.20%	27.76%	65.18%	9.19%	36.60%	31.89%
	Average Ramp Delay (min/veh)		49.20%	27.76%	65.18%	9.19%	36.60%	31.89%
	Volume (vehicles entered from ramps)		0.00%	0.00%	0.00%	0.00%	0.00%	0.00%
System MOEs	Total System Travel Time (veh-hour)		-2.25%	-0.45%	-3.46%	-0.64%	-0.77%	-1.13%
	Average System Speed (mile/hour)		2.28%	0.89%	3.76%	0.64%	0.76%	1.28%
	Fuel Consumption (gallons)		-8.41%	-4.19%	-7.54%	-1.17%	-2.66%	-2.03%
	Pollutants Emissions (kgs)	CO	-5.13%	-1.66%	-5.37%	-1.00%	-1.41%	-1.72%
		HC	-3.90%	-1.21%	-4.28%	-0.76%	-1.49%	-1.57%
		NO	-6.63%	-2.38%	-6.92%	-1.20%	-1.79%	-2.09%

* The most severely congested day on a test site

Table 9.2 Comprehensive Evaluation: Percentage Change for Major MOEs (Comprehensive improved SZM control over SZM control) Metering Period
(3:00pm to 6:00pm)

Categories \ % Change			TH-169NB			I-94EB		
			NOV 08	NOV 13	NOV 27*	OCT 26*	NOV 01	NOV 27
Freeway MOEs (Mainline)	Total Number of Stops		-27.74%	-23.85%	-25.75%	-3.73%	-6.90%	-14.71%
	Number of Stops Per Veh		-27.74%	-23.85%	-25.75%	-3.73%	-6.95%	-14.71%
	Total Freeway Travel Time (veh-hours)		-8.24%	-3.52%	-8.94%	-1.71%	-3.06%	-3.50%
	Total Freeway Travel (veh-miles)		0.06%	0.03%	0.36%	-0.06%	0.26%	0.57%
	Total Freeway Delay (veh-hours)		-20.22%	-13.69%	-16.84%	-3.32%	-6.36%	-9.18%
	Average Freeway Delay (min/veh)		-20.22%	-13.69%	-16.84%	-3.32%	-6.41%	-9.18%
	Volume (vehicles serviced by freeway)		0.00%	0.00%	0.00%	0.00%	0.05%	0.00%
	Average Speed (mile/hour)		9.04%	3.68%	10.22%	1.62%	3.43%	4.24%
Ramp MOEs	Total Ramp Travel Time (veh-hours)		45.27%	24.35%	57.22%	6.83%	25.15%	21.01%
	Total Ramp Travel (veh-miles)		-0.01%	0.00%	-0.01%	0.00%	0.07%	-0.01%
	Total Ramp Delay (veh-hours)		49.88%	29.30%	66.99%	9.36%	38.58%	32.42%
	Average Ramp Delay (min/veh)		49.88%	29.30%	66.99%	9.36%	38.50%	32.42%
	Volume (vehicles entered from ramps)		0.00%	0.00%	0.00%	0.00%	0.06%	0.00%
System MOEs	Total System Travel Time (veh-hour)		-3.20%	-1.55%	-4.87%	-0.76%	-0.50%	-1.22%
	Average System Speed (mile/hour)		3.36%	1.60%	5.49%	0.69%	0.77%	1.81%
	Fuel Consumption (gallons)		-10.39%	-6.66%	-9.36%	-1.43%	-2.68%	-1.48%
	Pollutants Emissions (kgs)	CO	-7.25%	-2.85%	-7.31%	-1.01%	-1.17%	-1.34%
		HC	-5.54%	-2.11%	-5.88%	-0.96%	-1.20%	-1.07%
		NO	-9.02%	-3.92%	-8.87%	-1.15%	-1.52%	-1.69%

* The most severely congested day on a test site

Table 9.3 MOEs for Ramp Performance on TH169NB for Comprehensive Evaluation
(Comprehensive improved SZM control over SZM control) NOV, 08 2000
(3:00pm to 6:00pm)

MOE / Ramps	Average Ramp Wait Times (minutes)		Max Ramp Wait Times (minutes)		Total Ramp Delay (vehicle-hours)		Average Queue Size (vehicles)		Max Queue Size (vehicles)	
	SZM	CSZM	SZM	CSZM	SZM	CSZM	SZM	CSZM	SZM	CSZM
36thSt	0.14	1.04	0.47	2.77	2	13	0	3	3.85	13.55
BettyCrocker	0.22	0.87	1.08	2.43	3	12	1	3	8.20	16.15
Brenn	0.52	0.52	1.72	1.80	17	17	4	4	18.40	19.15
Cedar	0.15	0.14	0.57	0.57	1	1	0	0	4.75	4.75
Excelsior	1.51	1.06	4.45*	3.88	46	32	10	7	26.70	26.25
I394EB	0.25	1.62	1.05	3.34	5	35	1	8	8.25	21.90
I394WB	1.30	1.41	2.42	3.43	48	52	11	12	23.30	27.55
Lincoln	0.18	0.17	0.50	0.36	1	1	0	0	4.55	3.30
MedicineLake	2.38	1.25	6.53*	3.03	43	22	10	5	27.85	18.05
Minnetonka	0.16	0.53	0.46	2.28	2	6	0	1	3.35	9.45
Plymouth	0.23	0.23	0.40	0.39	5	4	1	1	5.70	5.70
TH55EB	0.14	0.93	0.38	3.06	2	12	0	3	4.35	15.70
TH55WB	0.10	0.79	0.31	2.02	1	11	0	3	2.85	9.85
TH62EB	0.27	0.43	1.22	1.42	10	16	2	4	17.55	17.20
TH62WB	0.09	0.61	0.47	1.57	3	21	0	5	7.00	18.75
TH7	0.19	1.52	0.85	3.63	4	35	1	8	6.70	22.85
ValleyView	0.26	0.32	2.43	2.72	9	11	2	2	21.25	20.75

*The maximum allowed ramp wait time is violated

Table 9.4 MOEs for Ramp Performance on TH169NB (Comprehensive improved SZM control over SZM control) NOV, 27 2000
(3:00pm to 6:00pm)

MOE / Ramps	Average Ramp Wait Times (minutes)		Max Ramp Wait Times (minutes)		Total Ramp Delay (vehicle-hours)		Average Queue Size (vehicles)		Max Queue Size (vehicles)	
	SZM	CSZM	SZM	CSZM	SZM	CSZM	SZM	CSZM	SZM	CSZM
36thSt	0.11	0.74	0.46	3.00	1	10	0	2	2.80	11.25
BettyCrocker	0.32	0.80	3.01	3.05	5	11	1	3	18.15	18.75
Brenn	0.40	0.36	1.37	1.44	12	11	3	2	16.30	17.25
Cedar	0.12	0.12	0.21	0.21	1	1	0	0	2.00	2.00
Excelsior	0.90	0.66	3.79	2.59	26	19	6	4	24.50	23.65
I394EB	0.73	1.40	3.62	3.53	16	31	4	7	24.25	26.25
I394WB	0.75	1.44	2.42	3.23	27	52	6	12	23.30	24.65
Lincoln	0.20	0.17	1.15	0.69	2	1	0	0	6.60	6.00
MedicineLake	1.76	1.53	6.23*	3.17	29	25	7	6	23.05	15.75
Minnetonka	0.14	0.46	0.39	2.32	2	6	0	1	3.35	7.80
Plymouth	0.19	0.18	0.36	0.35	4	4	1	1	5.85	5.35
TH55EB	0.14	0.86	0.45	2.90	2	11	0	3	4.75	13.80
TH55WB	0.13	0.46	0.42	1.32	2	7	0	1	4.30	8.20
TH62EB	0.25	0.25	1.11	1.46	8	8	2	2	15.60	15.40
TH62WB	0.07	0.39	0.41	1.60	2	12	0	3	7.15	17.80
TH7	0.11	1.21	0.45	4.14*	3	29	0	6	4.95	22.90
ValleyView	0.15	0.17	1.24	1.76	5	5	1	1	12.90	17.00

*The maximum allowed ramp wait time is violated

Table 9.5 MOEs for Ramp Performance on I-94EB (Comprehensive improved control over SZM control) OCT, 26 2000
(3:00pm to 6:00pm)

MOE / Ramps	Average Ramp Wait Times (minutes)		Max Ramp Wait Times (minutes)		Total Ramp Delay (vehicle-hours)		Average Queue Size (vehicles)		Max Queue Size (vehicles)	
	SZM	CSZM	SZM	CSZM	SZM	CSZM	SZM	CSZM	SZM	CSZM
Lyndale Ave.	0.09	1.16	0.39	3.20	1	16	0	4	3.10	13.50
Hennepin Ave.	0.57	0.90	1.86	2.64	37	58	9	14	35.55	42.35
5th Ave	1.86	1.26	5.00*	3.34	41	28	10	7	28.50	21.05
6 Street	1.67	1.16	4.39*	3.72	87	60	22	15	55.95	50.40
Cedar Ave	1.12	0.32	3.66	1.00	25	7	6	2	17.25	8.80
Riverside Ave.	2.74	2.00	5.81	4.08*	75	55	17	13	33.65	27.55
Huron Blvd	0.24	2.02	0.91	4.09*	7	58	1	13	9.80	30.30
Cretin Ave.	1.08	1.13	2.45	2.60	31	33	7	7	17.25	20.65
Snelling Ave.	0.18	1.71	1.66	4.53*	7	67	2	16	20.90	35.50
Lexington Ave	0.63	0.91	2.21	2.90	23	33	5	8	23.70	21.60
Dale Street	0.70	0.25	3.26	2.37	15	5	3	1	22.05	16.55
Marion Street	0.94	0.25	3.56	2.82	29	7	6	2	27.85	26.15

*The maximum allowed ramp wait time is violated

Table 9.6 MOEs for Ramp Performance on I-94EB (Comprehensive improved SZM control over SZM control) NOV, 27 2000
(3:00pm to 6:00pm)

MOE / Ramps	Average Ramp Wait Times (minutes)		Max Ramp Wait Times (minutes)		Total Ramp Delay (vehicle-hours)		Average Queue Size (vehicles)		Max Queue Size (vehicles)	
	SZM	CSZM	SZM	CSZM	SZM	CSZM	SZM	CSZM	SZM	CSZM
Lyndale Ave.	0.08	0.19	0.25	1.91	1	3	0	1	2.50	7.20
Hennepin Ave.	0.52	0.26	2.28	2.39	29	15	7	4	38.40	38.40
5th Ave	2.37	1.68	5.88*	3.91	52	37	12	9	28.50	21.40
6 Street	1.19	0.80	4.56*	4.55*	34	23	9	6	42.90	42.60
Cedar Ave	1.40	1.25	2.98	2.29	28	25	6	6	13.25	12.10
Riverside Ave.	3.12	2.06	5.73*	3.63	85	56	19	13	33.95	23.05
Huron Blvd	0.24	2.43	0.82	5.35*	6	64	1	15	6.85	26.70
Cretin Ave.	0.58	0.54	2.04	2.09	15	14	3	3	18.10	18.00
Snelling Ave.	0.01	0.02	0.12	0.25	0	1	0	0	4.40	5.60
Lexington Ave	0.11	0.08	1.12	0.65	3	2	1	0	15.15	9.10
Dale Street	0.17	0.14	2.00	1.72	3	3	1	1	10.30	9.60
Marion Street	0.14	0.14	2.17	2.20	4	4	1	1	21.60	20.65

* The maximum allowed ramp wait time is violated

Part IV: Concluding Remarks

Chapter 10 Conclusions

10.1 General Conclusions

Two improvements are proposed in this project. One is to improve the control logic of the currently operational SZM ramp control strategy, and the other is to improve the accuracy of the current queue size estimation. For control logic improvement, the shortcoming of the current control logic lies in the implementation of unnecessarily high minimum release rates during the control period leading to faster onset of freeway congestion. The improved control logic postpones the commencement of congestion by refining the minimum release rate for each ramp. Both the original and the improved SZM strategy were tested successfully through advanced microscopic simulation on two typical freeway test sites for several days representing varying demand and congestion levels in Minneapolis/St. Paul metropolitan area. The simulation results show that the proposed enhancements to the SZM control logic decrease freeway congestion by postponing its onset and by reducing its severity at bottlenecks. Quantitatively the improved logic resulted in moderate to high improvements in freeway and system total travel time, freeway speed and energy consumption. However, the improvements in total ramp delays and total number of stops on the freeway were much more significant. The results are consistent for both simulation periods (3:00 to 6:00pm and 2:00pm to 8:00pm) on both test sites and all test dates.

For queue size estimation, three improved methods are presented in this report based on different categories of ramp errors. For Class I ramps, characterized by minor volume detection error (less than 2%), a simple Conservation Model was found to be sufficient in estimating the queue size. For Class II ramps, exhibiting significant counting error in passage detectors but queue detectors accurate, results indicate that the conservation model can still be applied but the traffic counts of passage detectors need to be replaced by the “Green Counts”. For Class III ramps which have significant errors (larger than 2%) on queue and passage detectors, two different models are proposed in this paper: a Regression Model and a Kalman Filtering Model. By comparing with the ground truth data, the test results suggest that the improved methodologies work very well and greatly improve the accuracy of queue size estimation. Also, the new model is evaluated by micro-simulation. The simulation results indicate that the improved algorithm significantly improve the performance of freeway mainline, while, as a trade-off, the delay in ramp system is increased. However, the total system performance has increased and the new SZM control did not compromise the predetermined threshold of the maximum waiting time at each entrance ramp.

10.2 Future Research Suggestions

There are still some potential improvements to current SZM control based on previous research:

1. Optimize the SZM Parameters

There are 20 control parameters in the current Stratified Zone Metering (SZM) algorithm. Tuning these parameters can significantly affect the system performance of the SZM algorithm and these effects on the system performance depend on test sites and demand levels. Therefore in this project we will study and analyze the effects of the control parameters on the freeway system performance (Sensitivity Analysis); explore the appropriate optimization methodology which is effective and credible in searching the optimal parameter values; streamline the parameter optimization methodology and incorporate it with rigorous microscopic simulation. More important, in order to implement easily, we will explore the optimal common parameters for all test sits and all test time.

2. Develop a methodology for determining location-dependent bottleneck capacity

Bottleneck capacity in SZM control actually refers to downstream capacity of a working zone. It may or may not be a real bottleneck. The bottleneck capacity is used in determining the spare capacity inside a zone. In the present form of the strategy, uniform values are used for this parameter which doesn't reflect real-time traffic conditions and location-dependent geometric characteristics. Thus, it is recommended for future studies to develop a methodology for determining location-dependent bottleneck capacity. This involves extensive work including analysis of the dynamics of traffic flow near real bottlenecks and the exploration of the feasibility of using alternative traffic measurements to estimate the bottleneck capacity.

3. Comprehensive evaluation of the all the improvements

Till now, several researches about how to improve the effectiveness of current SZM control have been done separately (Beegala et al., 2005; Feng et al., 2005; Liu et al., 2006). However, there are no combinations of these researches. Therefore, the future research will focus on how to combine these improved methodologies and gain the biggest effectiveness.

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