I, Karthekul Kumar Allam M.S., hereby submit this original work as part of the requirements for the degree of Master of Science in Civil Engineering.

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Simulation-Based Integrated Control Algorithm for Controlling Shockwave Propagation on Freeways and Queue Spillback at On-ramps

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Simulation-Based Integrated Control Algorithm for Controlling Shockwave Propagation on Freeways and Queue Spillback at On-ramps

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ABSTRACT

Heavy traffic flow entering the freeway mainline can cause or worsen congestion on the freeway mainline due to interrupted on-ramp traffic. In order to address this problem, Freeway ramp metering systems are usually deployed to improve the traffic flow on urban freeways. These systems are attended to improve the flow on the freeway by regulating the on-ramp rate entering the freeway. However, amongst all the possible factors contributing to traffic congestion on freeways and arterials connected via ramps, lack of coordination in the operation of various system components is a major source of inefficiency at freeway and ramp conjunctions. Various ramp metering algorithms have been developed in an attempt to solve the unsystematic problem in coordinating the control of freeway, and arterials. Nevertheless, most of the algorithms control actions were selected based on either pre-defined plans or detected traffic condition. Implementation of the algorithms on the field is also a concern because of its excessive data requirements of these algorithms. Furthermore, existence of a range of flow rates in synchronized flow traffic can cause breakdown due to internal perturbations. Many algorithms failed to address the issue of worsening freeway congestion due to upstream traffic at the on-ramp.

This research will formulate an integrated ramp metering control algorithm, using real-time traffic measurements. Multiple priority objectives are explicitly set up to delay the congestion due to internal perturbations, maintain the throughput of freeway, and to prevent on-ramp vehicles from overflowing into arterials. This algorithm is also formulated to minimize the delays and control the shockwave propagation from the merge location of the on-ramp and the freeway.
In order to measure the effectiveness, the proposed algorithm is compared with a traffic responsive algorithm, such as ALINEA (Asservissement LINéaire d’Entrée Autoroutiere), and proposed recommendations accordingly. Considering the importance of gap acceptance and internal perturbations in the current research, calibration of the simulation model was conducted. For this purpose, video data was collected capturing the traffic at the merge location of the freeway and on-ramp, from which the accepted gaps and headways of the vehicles merging into the freeway have been extracted. This was given as an input to the VISSIM simulation model for calibration.

The simulation test results indicate that the proposed integrated ramp metering algorithm is more effective than the ALINEA and Fixed Time ramp metering algorithms in terms of reducing the system delays and travel time on the freeway, and minimizing the freeway breakdown. Additionally, the proposed algorithm works efficiently at the ramp traffic flow of no more than 580 veh/hr. In other words, the ALINEA likely outperforms the proposed algorithm as the ramp traffic exceeds 580 veh/hr.

The contribution of the research will be reflective of the following aspects: 1) developing a method with a supportive algorithm for minimizing the shockwave propagation by dampening shockwave formation of mainline freeway traffic; 2) developing a method with a supportive algorithm to reduce mainline freeway shockwaves by dispatching vehicles from the on-ramp at a flow rate that can best fit in the observed gaps to be available from the freeway traffic; 3) identifying the minimum accepted gaps at the merge location from video observation to develop appropriate gap acceptance parameters; and 4) developing an integrated computing system to provide a fundamental platform for further functionality expansion in the future for study of a multiple-ramp situation at a freeway system.
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CHAPTER 1:
PROBLEM STATEMENT

1.1 Background

Due to increase in traffic flow entering the freeway mainline, there is often a very high congestion on the freeway mainline and this congestion is mainly due to interrupted traffic from the on-ramp traffic (Arnold, 1998). According to urban mobility report Texas Transportation Institute (2012), congestion caused urban Americans to spend an extra 5.5 billion hours on the road and also purchase an extra 2.9 billion gallons of fuel. To mitigate this congestion, ramp meters have been deployed with traffic signals on the ramp, and these traffic signals are used to control the rate of vehicles entering the freeway from a local street or connector (Jacobson, Stribiak, and Nelson, 2006). Ramp meters usually benefit the freeway mainline traffic by decreasing the impact of merging vehicles and thereby causing the smooth traffic flow. This is usually achieved by the storage of vehicles on the on-ramp; however, the storage capacity of the on-ramp reaches its threshold when there is a difference between the arrival rate to the on-ramp and the metering discharge from the ramp.

Though the ramp meters were proved useful in traffic operations, the operation issue still exists on the ramp meters. The freeways, ramp meters and arterials are considered as all individual components rather than a single element. One of the reason might be the jurisdictional issues i.e. the freeways and ramp meters could be part of city or counties, whereas, freeways and the ramp metering system is maintained by Department of Transportation. During the peak hours, it often requires limited entry of traffic on metered ramp from the on-ramps to the freeways,
resulting in long queues on the on-ramps. The traffic released from the arterial signals that feed the on-ramp would also aggravate the queue spillback effect (Wu and Jin, 2008). This argument had been frequently used against ramp meters.

Various algorithms, model based and non-model based, have been developed to address the problem of coordinating and integrating the control of freeway and arterial systems. Based on the scope and application of ramp metering, ramp metering control algorithms may be categorized into Isolated, Coordinated and Integrated algorithms. Only those algorithms which were implemented in the field are discussed below.

1.2 Isolated Algorithms

In isolated ramp-metering algorithms, ramp metering rate for an on-ramp is determined based on its local traffic conditions. Traffic parameters such as speed, occupancy, flow, travel speed, and occasionally queue over-flow on the metered ramp are obtained by the local traffic detectors. This section discusses about Asservissement LINeaire d’Entrée Autoroutiere, i.e. ALINEA (Papageorgiou et al., 1997), and Zone algorithms (Stephanedes, 1994), which were the two most extensively implemented isolated algorithms.

1.2.1 Zone-Algorithm

The Zone algorithm was first deployed in Minnesota DOT. The important aspects of this algorithm are the proper division of the zones, calculation of the density, estimation of the bottleneck capacity. In this algorithm, the freeway is divided into several zones, and each entry ramp is affiliated with a zone. Each zone is comprised of freeways, arterials, on-ramps and off-ramps, metered freeway-freeway connectors, un-metered on-ramps, metered on-ramps. Typically, the
upstream zone functions as free flow conditions and the downstream zone of the segment functions as the bottleneck of the section. The algorithm is typically based on an equality equation. For each zone, the algorithm is formulated to maintain the condition of mainline under certain level of density by controlling the inflow from the on-ramps and freeway connectors. The collective ramp metering rate obtained from the algorithm is re-distributed to other on-ramps using a pre-defined ramp metering factor. Further adjustment to the metering rate can be made based on environmental factors and other considerations. Levinson and Zhang (2002) evaluated the system for various performance measures such as mobility, travel time and delay. The algorithm proved advantageous and demonstrated consistent results when compared to other studies. However, when the equity equation was analyzed, the performance of the algorithm was relatively low. It was found out that the benefits occurred at the expense of short trips and ramp metering was actually increasing the travel time of the vehicles over the freeway mainline. Another significant drawback of the algorithm is that it did not consider the stochastic nature of the traffic flow and would underperform during the non-recurrent congestions.

1.2.2 ALINEA Algorithm

ALINEA is an isolated algorithm which was developed at the technical university of Munich (Chu and Yang, 2003) and was successfully implemented in several locations worldwide. It basically works on a simple feedback loop structure. It was effective when deployed in field for its function of preventing the freeway flow from being saturated by maintaining the occupancy at a desired level. ALINEA is discussed in more details in chapter 4.
1.3 Integrated Algorithms

Integrated ramp metering algorithms are those that optimize the surface street signals in conjunction with freeways, highways, and arterials to minimize the delays and travel time on freeways and arterials. Integrated algorithms have definite control objective(s) which are linked to the control action. The objectives are usually to minimize the delays, travel time, and maximize the speed and throughput of the entire system. Constraints like maximum ramp flow and allowable queue on the on-ramp will also play as key elements in deciding the ramp metering rates. As in many other algorithms, the computation of ramp metering rates in this algorithm can also vary depending upon various scenarios and situations considered in the algorithm. Conceptually this type of algorithm is most effective because of its capability to handle both arterials and freeway systems with various modeling constraints and was proved to be theoretically robust.

1.3.1 METALINE Algorithm

METALINE (Papageorgiou et al., 1990) algorithm is an extension of the local control algorithm ALINEA. The control logic of METALINE is proportional to integrated state feedback. The metering rate of each ramp is computed based on the changes in measured occupancy of each freeway segment under METALINE control. The major challenge to the successful implementation of METALINE is the proper choice of the control matrices and target occupancy vector. Conversely, it is reported that METALINE performed better than ALINEA against non-recurrent congestion but failed to show advantage in recurrent congestion (Papageorgiou and Kotsialos, 2002).
Similarly, local linear programming based algorithm (Yoshino et al., 1995) is among the oldest in research and practice. It was widely used before automatic control based dynamic algorithms were introduced. It works on maximizing the weighted sum of the ramp flow where weights are chosen by the user. Nevertheless, the performance is heavily dependent on the O-D data and the algorithm is oblivious to the variation of travel time in its computational analysis.

1.3.2 Fuzzy logic Algorithm

Fuzzy Logic ramp metering algorithm (Taylor and Meldrum, 1998) is an integrated, traffic-responsive ramp metering algorithm. This algorithm was first developed at the University of Washington and was first deployed in 1999 by Washington Department of Transportation (WsDOT). It was test implemented on few on-ramps on I-405 and was then deployed on whole I-405 after the evaluation results proved fuzzy logic algorithm to be effective in reducing the delays and improving the travel time and level of serviceability in comparison with the bottleneck algorithm.

Fuzzy Logic ramp metering implementation first began in 1989 in Netherlands (Taylor and Meldrum, 1998) and nine ramp meters were deployed on site by 1995. Evaluation results showed improved freeway and the arterials conditions. The travel time and speed on the freeway was improved substantially when compared to other algorithms.

In the Fuzzy Logic, the empirical knowledge about the traffic flow is converted into various fuzzy rules depending upon the size of the network. The traffic variables such as occupancy, flow rate, queue and ramp queue measured from loop detectors are categorized into finite textual classes, such as very small, small, big, and very big. Then, a set of rules is run on
the inputs which determine metering levels; e.g., IF occupancy is small AND ramp queue is small THEN high metering rate. The metering levels are then converted back from textual classes to numeric ramp metering rate and applied on the field.

In a way fuzzy logic algorithm is an expert and powerful system when the proper rules are used. The calibration of the rules also plays an important role in effective functioning of the algorithm. However, even though the algorithm was implemented and proved effective, Zhang and et al. (2001) noted that the robustness of this algorithm heavily relies on proper selection of rules, which can be quite complex for global-level control. Finally, calibration of parameters is a very huge task and in complex networks it becomes very difficult to calibrate the rules. It was also observed that the algorithm underperforms when the traffic conditions change over time.

1.3.3 Multiple-objective, Integrated Large-scale, and Optimized System (MILOS)

Algorithm

MILOS is an acronym for Multiple-objective, Integrated Large-scale, and Optimized System. The MILOS is an integrated control algorithm which considers the interaction surface street system and freeway system. The interactions are usually governed by the formulating a multi-objective solution methodology to address all the set objectives.

MILOS is a multi-objective hierarchical freeway on-ramp control system that decomposes the large-scale freeway ramp-metering problem into a series of optimization problems. As the traffic control parameters change, the optimization problem are resolved to continually adjust the control strategy to the real time behavior of the system. In addition to the optimization, a predictive scenario based optimization scheme is also implemented in the
algorithm to mitigate the unpredictability of the future system state. This prediction scenario system was implemented in real time to address the next short term stochastic disturbance.

MILOS has two levels of hierarchy. 1) Area Coordinator; and 2) Predictive scenario based subsystem. The metering rate is then computed from the both the levels and a suitable ramp metering rate for the prediction scenario is applied in the system.

As a very latest attempt, Papamichail et al. (2009) developed a model predictive algorithm hierarchical control approach for coordinating ramp metering of freeway networks. Local control strategy ALINEA and optimal control tool AMOC were incorporated into the network.

Thus, many of these studies incorporate a group of equations either to constrain freeway conditions to freeway flow or to avoid dealing with freeway mainline traffic dynamics (Zhang and Levinson, 2004). Few heuristic algorithms exist but are only appropriate for particular case study and small scale networks (Lovell and Dagnzo, 2000). Many integrated and coordinated algorithms are observed to be computationally very complex (Wang and Zhang, 2013). After an ample research and examination of previous studies on ramp meters, it is noticed that ramp metering algorithms have expanded from local to coordinated and then further to integrated. Accordingly, the mathematical models used in the algorithms have become more complicated and augmented the problem of implementing them in the field.

Furthermore, it was observed that there exists a range of flow rate in the free flow where traffic breakdown can occur due to internal perturbations. Consequently, control algorithms developed could not prevent the traffic break down from appearing in the vicinity of the on-ramp
merge area even though their control objective is achieved (Helbing, 2007). The lack of the ability of the upstream detectors to detect the queue would worsen the traffic congestion. The ramp meter on the on-ramp would be allowing the vehicles to merge on to the freeway causing a consecutive shockwave to propagate upstream of on-ramp.

In earlier studies, traffic conditions were observed at the upstream and the downstream of the freeway bottleneck located near the busy on-ramp (Cassidy and Rudjanakanoknad, 2006). It was observed that due to congestion at the bottleneck, shockwaves are induced from the point of bottleneck and travels through the upstream and the down-stream of the bottleneck. In the current research shockwave travelling through the upstream is being analyzed. Shockwaves are analyzed using conventional methods such as queuing theory and shockwave analysis.

As an effort to address the problems discussed above, this research aims at formulating an integrated ramp metering control algorithm using real time traffic measurements. These traffic measurements can be obtained from detectors. Three control functions are explicitly set up to improve the system performance, which includes reducing freeway mainline congestion, adequately using on-ramp storage capacities, and preventing the congestion at arterials, delaying the propagation of shockwave by dampening it.

1.4 Problems Identified

In summary, the study problems have been identified as follows:

1. Traffic breakdown is observed beyond upstream detectors due to internal perturbations.
   Also, many algorithms have failed to detect the amplifying problem of congestion.
2. Many algorithms, which were non-responsive to the breakdown at the upstream, are at a potential risk of causing multiple shockwaves formation originating from merge location.

3. Queue spillback on to arterials was observed to be the most common problem in many control algorithms resulting in congestion and safety concerns.

4. Many algorithms (WSDOT fuzzy, SWARM, and MILOS etc.) were proved to be complex to calibrate and dependent on the quality of input data such as O-D tables, and previously predicted demands. Below mentioned are examples of two such integrated algorithms.

- SWARM algorithm operates on two levels: 1) Local control system, and 2) Global control system. This algorithm requires O-D information as input. Pham (2002) evaluated the SWARM algorithm increases the mainline throughput by 11% and reduced the delay by 16% when compared to Bottleneck and METALINE algorithms. The algorithm was proven to be complex to implement because of its data requirements and computational analysis.

- MILOS: Hierarchical freeway on-ramp control system which decomposes large scale ramp metering problems into optimization problems. This algorithm is very data intensive. It requires O-D estimates, recurring incidents and flow profile data for computation. The MILOS algorithm increased the throughput by 44% and reduced the delay by 24% when compared with other PC-RT algorithms.

The remainder of this document is organized as follows: In chapter 2, goal of the research is described, focusing on the objectives achieved in the research; In chapter 3, the traffic condition of the selected location is described, focusing on its road geometry and traffic demand which lead to the congestion; In chapter 4, the literature review regarding various algorithm
identified and the draw backs of those algorithms were discussed; In chapter 5, the proposed control algorithm is explained in detail; In chapter 6, the case study location is described and signal, detector configuration is explained. The algorithm is evaluated on the calibrated simulation model; and in chapter 7, the results and conclusions are discussed.
CHAPTER 2:
RESEARCH GOAL AND OBJECTIVES

The motivation of this research is to examine a more refined control algorithm which maximizes the ramp metering effect by increasing the throughput on the freeways and simultaneously decreasing the formation of the queues over the on-ramp. Theoretically, integrated arterial and freeway operation control strategy is likely to have the potential to meet the concerned issues. Accordingly, multiple priority objectives are developed to meet the issues. The algorithm can be assessed by comparing it with already developed algorithms such as ALINEA.

2.1 Goal

The goal of this research is to develop an integrated control strategy which integrates the ramp meter and arterial signal systems to mitigate the freeway congestion and maintains the throughput of freeway.

2.2 Objectives

In order to achieve the aforementioned goal, the primary focus of the research is placed on exploring an alternative algorithm addressing the internal perturbations, delay at the arterials, on-ramp, and throughput of the freeway. Effect of upstream detectors is not considered. The approach is believed to produce better results but is not conclusive of the same at this point of time. Accordingly, a micro-simulation-based approach will be developed with an attempt to
achieve the following objectives of optimizing the proposed integrated ramp and arterial signal system.

- To minimize the delay associated with arterials and freeway and on-ramp.
- To increase/maintain the throughput on the freeway.
- To decrease travel time on the freeway.
- To minimize queue spillback into arterials.

Therefore, it’s worthwhile to explore the analytical procedures for analyzing the components of integrated system.
CHAPTER 3:  
SCOPE OF THE RESEARCH

3.1 Scope of the Study Site

The Study sites would be at the on-ramp of the freeway with a decent on-ramp and freeway traffic volume. The selection of the sites should meet the following criteria:

1) On-ramp should directly merge into the freeway without an additional merge lane.
2) Presence of an arterial signal closer to the on-ramp governing the traffic coming on to on-ramp.

![Image of case study site, I-71S connecting Kenwood Road and ramp meter on on-ramp.]

Based on the above criteria, I-71 South bound and the on-ramp connecting Kenwood road is considered as the study site for the current research, as shown in Figure 1.

I-71 is a typical freeway in the Cincinnati area that bears heavy traffic and Kenwood road is an extensively used route to connect Kenwood Towne Centre and I-71S. In order to evaluate the effect of integrated ramp metering system, a freeway section from Kenwood road until
steward road is considered. The geographical scope of this study is shown in Figure 1. The study site location is a half interchange with NB off-ramp, SB on-ramp and is located just 11 miles from the Central Business District (CBD). The Montgomery road interchange is one of the busiest interchanges to the upstream of the study site location because of the nearby Kenwood Towne Centre Mall. Figure 2 shows the broader view of the study site location from the CBD.

![Map of the study site location and Central Business District](image)

*Figure 2. Distance of the study site location from central business district*
CHAPTER 4:  
RELATED WORK AND LITERATURE REVIEW

4.1 Introduction

Due to profuse increase in the population over years, there has been an increase in travel demand too thereby increasing in the flow of vehicles over the roadways. Many corridors in the country are frequently congested, with the worst congestion problems usually arising during the morning and evening peak periods (Schrank and Lomax, 1999). Due to the bottlenecks formed at the merge location, shockwaves are induced into the both upstream and downstream of the traffic (Das and Levinson, 2004). The ramp metering traffic control measure for the corridor aims to improve flows on both freeway and arterial streets during peak periods. However, due to lack of coordination of the ramp meters and arterial signal systems there is an impending queue spillback on the arterial systems which could show a negative effect on traffic. Many coordinated and integrated algorithms have been developed, but have failed to prove effective and were difficult to implement. In this chapter, coordination strategies and their mechanism and limitations have been described in detail.

4.2 Traffic Responsive Metering

There are basically four types of ramp-metering operations based on the level of complexity of the control algorithm: pre- time, local traffic responsive, coordinated freeway ramp metering, and integrated freeway/surface street system. Pre-time ramp metering control is proved effective only in recurrent congestion (Saito et al., 2003) and about 60% of the traffic delays are caused due to non-recurrent congestion (Cambridge Systematics Inc., 1990). Therefore, the incapability of dealing with the non-recurrent congestion degrades the purpose of
the pre-timed ramp meters. To mitigate this problem, traffic responsive algorithm have been deployed and were proved to be effective when compared to pre-time meters as they were using the real time traffic measurements and, hence are both responsive to both recurrent and non-recurrent congestion.

Based on the control scopes, the algorithms are divided into local and coordinated ramp metering algorithms. Local traffic responsive algorithms consider inputs from local traffic detectors, while coordinated ramp metering uses inputs from other traffic detectors too which may belong to upstream and downstream detectors of freeway. The ALINEA was the first ramp metering strategy based on the classical feedback control theory, and has been deployed successfully in various cities since 1990. In the current study, only the ALINEA is used to measure the effectiveness of the proposed algorithm.

4.3 ALINEA Algorithm

The ALINEA algorithm is a type of isolated ramp metering. In an isolated ramp metering algorithms, the ramp metering rate for an on-ramp is computed by considering the local traffic-conditions such as speed, occupancy, flow, etc. Papageourgiou et al. (1991) proposed a rather simple linear feedback type of model called ALINEA. The ALINEA is a closed-loop algorithms which is based on the feedback structure obtained from the main-line loop detectors. The feedback control logic dynamically maintains the mainline occupancy level below the target occupancy level by resisting the inflow from on-ramps. The ALINEA uses equation 1 for deriving the ramp metering algorithm.
\[ r(t) = r(t - 1) + K_R \ast (O_{desired} - O_{downstream}(t)) \]  \hspace{1cm} (1)

Where,

- \( r(t) \) = Measured metering rate of time interval T;
- \( K_R \) = Regular parameter;
- \( O_{desired} \) = Desired occupancy at the downstream detector; and
- \( O_{downstream} \) = Occupancy at the downstream detector.

Figure source: Papageourgiou et al (1991)

**Figure 3. Detector configuration of the ALINEA algorithm**

The most advantageous feature of the algorithm is that it is insensitive to the certain range of values of occupancy and reacts smoothly to slight errors thus establishing the traffic flow. Figure 3 shows the detector requirement for the ALINEA. The primary purpose of the ALINEA is to stabilize the freeway traffic at high throughput without the breakdown of the
freeway. Studies have shown that this ramp metering strategy is more preferable than that of many other traffic responsive strategies.

Many other local traffic responsive algorithms were also proposed such as DC (Demand-Capacity) strategy (Masher et al., 1975) and OCC (Occupancy) strategy which are based on the feed forward rejection schemes. These feed forward rejection schemes are usually based on mainstream measurements of flow and occupancy of upstream of the on-ramp. Field studies indicated that ALINEA have proven superiority over DC, OCC and other control strategies.

4.4 Integrated and Coordinated Ramp Metering Algorithms

Integrated ramp metering algorithms have been developed to address different issues and problems in congestion, mainly associated with freeway traffic. Isolated ramp metering being associated mainly with the local traffic conditions paves a way for new issues which are to be addressed. Below mentioned are the researches previously done to coordinate and integrate the ramp metering and arterial systems. The main objective of integrated operating system is to reduce the travel time and increase the throughput of the freeway. Ramp metering rate is decided by optimizing the travel time and throughput by considering various factors as maximum allowable queue and freeway capacity. Integrated ramp metering strategies are considered to be more effective and appealing as their capability of handling various metering and modelling constraints.

However, these algorithms are invariably more complex and more demanding in computation. Many algorithms have been developed based on integrated ramp metering strategy, out of which few are mentioned below.
4.4.1 METALINE Algorithm

Dikaki and Papagerogiou (1994) proposed METALINE, an integrated coordinated algorithm which is an extended version of ALINEA algorithm. METALINE is coordinate form of ALINEA, which uses occupancy and control gain matrices to return the ramp metering rates. The algorithm is governed by following the below equation to compute the ramp metering rate. The ramp metering rate for a particular ramp is calculated based on mainline traffic flow, control matrices K1 and K2, and the change in measured occupancy at each segment. The K1 control matrix is used to tune the sensitivity of the detector occupancies to the current ramp metering location. K2 control factor is used to tune the contribution of critical detectors to the ramp metering rates at each location. The variation of occupancy from critical occupancy for each segment that has a controlled on-ramp is governed by equation:

$$\vec{r}(k) = \vec{r}(k-1) - K_1(\vec{\sigma}(k) - \vec{\sigma}(k-1)) - K_2(\vec{O}(k) - \vec{O}^c)$$

(2)

Where,

$$\vec{r}(k) = \text{Vector of metering rates for the controlled ramps at time step } k;$$

$$\vec{\sigma}(k) = \text{Vector of measured occupancies within the directional freeway segment at time step } k;$$ and

$$\vec{O}, \vec{O}^c = \text{Measured and desired occupancy of ramps respectively.}$$

Though METALINE algorithm is an extension of ALINEA, Papageorgiou and Kotsialos (2002) had proved that METALINE performs better under non-recurrent congestion but it showed no advantage over recurrent congestion.
4.4.2 System Wide Area Ramp Metering (SWARM) Algorithm

SWARM (System Wide Area Ramp Metering) was first developed under a contract with California Department of Transportation (Paesani et al., 1997). The algorithm was implemented in Orange county of California. SWARM algorithm is a traffic responsive algorithm where the freeway network is divided into different zones based on their locations of bottlenecks and their capacities. These zones contains multiple on and off-ramps in between, SWARM algorithm works on two levels.

SWARM-1 which works on the global network i.e. entire network system based on forecasted densities at the bottlenecks locations. SWARM-2 works with respect to local traffic conditions near each on-ramp. The densities are calculated from performing the linear regression on a set of data collected from the current and past and applying the kalman filter process in order to incorporate the non-linear term in the regression model. The ideal optimized ramp metering is then selected from SWARM-1 or SWARM-2 based on the pre-defined restrictive ramp metering in the algorithm.

4.4.3 Helper Algorithm

Helper algorithm is basically composed of local traffic responsive algorithm with added control over ride feature. Within a freeway corridor usually a set of location group of on-ramp meters are selected and then the pre-defined ramp metering rates are applied to all the local on-ramp meters. The local conditions like occupancy are measured using the traffic detectors. The algorithm also monitors the queue detectors and demand detectors for any on-ramp queue spillback causing delays on on-ramp and arterials.
If an on-ramp meter is operating at a minimum ramp metering rate and the occupancy and speed at the detectors reach the threshold value then the algorithm flags the on-ramp location as “critical” and the central over ride is run at the location. This eventually decreases the throughput of the freeway mainline and decreases the performance of the system.

4.4.4 Bottleneck Algorithm

Bottleneck algorithm was first developed by Washington department of Transportation and was implemented in the north of Seattle (Jacobsen et al., 1989). This algorithm was proved to more effective as it reduced the travel time by 48% and the increased the volume by 86%. Similar to SWARM algorithm, the bottleneck algorithm also computes the ramp metering rate in two levels. One based on local conditions and the other based on global conditions. The local ramp metering algorithm is computed by calculating the difference between the demand and capacity of the local on-ramp. The capacity is calculated from the loop detector data and historic data available for the on-ramp. The global ramp metering rate is when the storage and occupancy of the location increases its threshold occupancy. The final ramp metering rate is then computed by the pre-defined ramp metering rates.

Messer, Blake, Tian Z (Messer et al., 2006) developed a generalized system for coordination between ramp meters and upstream signal controllers and applied the system to a case study using a modeled Diamond Interchange. They used the ALINEA traffic responsive ramp-metering algorithm as developed by Papageourgiou et al. (1991). However, their research did not use the data from the traffic signals to influence metering rates.

Zhang and Wang (Zhang and Wang, 2013) have demonstrated a coordinated ramp metering algorithm mitigating freeway. As many algorithms have been difficult to implement on
the field due big heuristic algorithms, they have proposed a simple algorithm with hierarchical control scheme; a three priority layer structure which is easy to implement on the field. The proposed algorithm was inspired from this algorithm.

Additionally, his study was mainly concentrated on the queue spillback on the on-ramp and impact of queue flushes on the freeway. The impact of shockwaves on congestion at the merge location was ignored in research. Besides, it is observed that the most of integrated and coordinated algorithms were not only difficult to implement but lacked the dynamic allocation of the green time at the arterial signal systems. In addition to this, all the integrated algorithms failed to address the internal perturbation caused at the upstream of on-ramps.

Kerner (2006) and Helbing (2007) reported that a range of flow rate exists in free flow where traffic breakdown can occur due to the internal perturbations. Consequently, the ALINEA was unable to completely prevent traffic breakdown from appearing in the vicinity of the on-ramp merge area even though its control objective is achieved (Kerner et al., 2006).

Once traffic breaks down in the merge area upstream from the acceleration lane drop, multiple congestion traffic flow phases may emerge. Some of congestion phases are not in the scope of the ALINEA’s sensors since they propagate upstream of on-ramp (Kerner et al., 2006). Figure 4 shows the grey regions that denotes the formation of metastable traffic and region and the region above dotted line indicates the breakdown.
Figure 4. Illustration of velocity V and flow Q as function of vehicle density

There is a stable traffic flow, when the velocity changes little with the density. More specifically, there is stable traffic below some critical density $\rho_{c1}$ and above some critical density $\rho_{c4}$ (see Figure 4). For medium traffic densities between two critical densities $\rho_{c2}$ and $\rho_{c3}$, traffic flow is linearly unstable, i.e., even small perturbations can grow and cause a breakdown of traffic flow. In the intermediate density ranges $\rho_{c1} < \rho < \rho_{c2}$, and $\rho_{c3} < \rho < \rho_{c4}$, one finds metastable traffic, i.e., sufficiently small perturbations will fade away, while large enough ones will grow and cause a breakdown of traffic flow. The value of $Q_{out}$ falls into the metastable regime between $\rho_{c1}$ and $\rho_{c2}$. It depends on the maximum density and the average time gap, i.e., on the weather conditions, the truck fraction, and lane.

Currently most of the ramp metering algorithms are not designed to tackle the internal perturbations caused beyond the upstream detectors. Therefore this poses a potential threat of forming multiple shockwaves upwards from the merge area. In addition to that, Shockwaves are analyzed and measures are taken to control the propagation upstream. The speed and length of
the shockwave is analyzed to accomplish that. Below is the brief description about shockwave and its characteristics.

4.5 Shockwave Propagation

Shockwaves are homogenous high density areas of freeway traffic flow appearing at the bottleneck and propagating backward, leading to a congestion phenomenon. Accurate estimation of queues and delays is an important part of estimating the traffic characteristics. Two methods have been commonly used by researchers to estimate the queues and the delays i.e. input-output model and the shockwave analysis (Rakha and Zhang, 2005).

The simple input-out model determines the traffic queues with difference between the total arrivals and total departures of a traffic facility. This model estimates the vertical queues without the space dimension. Conversely, shockwave analysis method describes traffic queues through the changes in traffic density along the roadway section and estimates the vertical queue in both time and space dimension. However, it has been proved that both the theories yield almost same results during the congestion and dissipation process (Rakha and Zhang, 2005).

In the current research, shockwave analysis is considered in modeling the flow and queues in the shockwave. Nam and Drew et al. (1998) had done quite substantial study on both the methods and their consistency. In order to calculate the speed of the shockwave propagating from the bottle neck, consider a flow and density of the approaching traffic stream are denoted as $q_{n1}$ and $d_{n1}$ respectively and those of the queuing flow are denoted as $q_q$ and $d_q$, respectively. For a segment of road way of length $l$, the speed of a shockwave is equivalent to difference in flow divided by the difference in density between the two flow regimes (Nam and Drew et al., 1998) as the following equation:
\[ W_u = \frac{q - q_{n1}}{d_q - d_{n1}} \]  

Where,

\( W_u \) = Speed of the shockwave backward upstream shockwave;

\( q_{n1} \) = Flow of the approaching traffic stream; and

\( d_{n1} \) = Density of the approaching traffic stream.

Similarly, the speed of the shock forward recovery wave, which is the basically the shockwave caused while the discharging the queue is given by (Nam and Drew, 1998):

\[ W_d = \frac{q_{n2} - q}{d_{n2} - d_q} \]  

Where,

\( W_d \) = Speed of the shockwave backward upstream shockwave;

\( q_{n2} \) = Flow of the reduced approaching flow; and

\( d_{n2} \) = Density of the reduced approaching flow.

Furthermore, from the shockwave analysis where the queue length caused may be calculated using the following equation:

\[ L = -W_u(t_1 - t_0) \]  

Where,

\( t_1 \) = Time when the queue occurs, when the traffic demand drops \( q_{n2} \) to \( q_{n1} \); and

\( t_0 \) = Time in the absence of congestion.
These equations would be used in calculating the speed and length of shockwave in methodology. Conversely, detecting the shockwave and measuring the length of the shockwave from the loop detectors is not as accurate because of the common equipment malfunction of loop detectors and communication faults. In addition to that, many detectors which are present on the freeway age more than 15 years and have worn out causing malfunctioning (Chen and Kwon, 2003). Therefore, the alternative technologies have been explored to devise a surrogate to the loop detectors.

Yang and Chen (Yang and Chen, 2014) have developed a method to calculate the length and speed of the shockwaves in a freeway segment using video cameras. Distributed low-angle video cameras are placed on the roadway section on either sides of the freeway. In the visible region between the opposite cameras placed at the start and end of the freeway segment, the cameras detect the shockwave through a duplex flexible windows fused with adaboost cascade classifiers. This helps to dynamically calculate the weight of the measurement noise. In the blind region, the speed changes of the vehicles entering and leaving the blind region is used to estimate the shockwave position.

In addition to the loop detectors, many technologies were developed to detect the traffic and estimate the shockwave parameters. The following are few technologies which can be used for the purpose.

**Radar Technology:** This SDR radar traffic classified uses the latest radar technology to provide accurate speed, count and classification of passing vehicles. The product is designed to meet the increasing demand in traffic for accurate, reliable vehicle detection technology. Radar can serve as a suitable surrogate for the loop detectors. Placing two opposite radars on the freeway segment would provide enough data to calculate the length and speed of the shockwave.
**Covert Technology:** Covert Technology is an extension to the radar technology. In addition to the safety benefits available in the radar technology, this product does not suffer the obvious visibility range issues. This gives the radar technology a major advantage when collecting true vehicle speeds. Using this technology the products records the speed, count and classification of vehicles in both the directions proving to be cost effective.

**E-Tube –Wireless Counter:** This is an innovative technology developed to provide a cost-effective sensor data for traffic. The sensor data transmits the data when the vehicle wheel hits the sensor. The energy generated from each vehicle hit is enough to power the traffic sensor without any cables.

These above mentioned technologies may be used to as a surrogate to loop detector data. The literature review indicates that a significant number of studies have been done in this field. Therefore, it is important to have a better understanding of the system operational characteristics and system operations when integration strategies are sought. However, there are few areas which can be addressed. This research focuses on addressing those problems.
CHAPTER 5: METHODOLOGY

5.1 Approach

As discussed earlier in chapter 2, the goal and objectives of this research mainly concentrate on exploring the causes of congestion such as gap factor at the freeway and merge areas, queue spillback at the on-ramps, minimizing arterial delays and delaying the freeway breakdown due to internal perturbations. Desirable ramp metering strategies should be adopted to address the freeway congestion promptly and optimally that regulates the traffic entering the on-ramp capacities and prevents on-ramp traffic from entering into freeway traffic flow. Many algorithms ignore the negative impacts of ramp meters, leading to insufficient congestion responses from system perspective (WSDOT, 2008).

As an effort to address the problems discussed above, the current research aims at formulating ramp metering control algorithm, using real-time traffic measurements. Multiple priority objectives are explicitly setup to improve system performance and to delay the freeway breakdown.

These quantity-based objectives are measurable with simulation output datasets and are helpful in tackling the freeway congestion by delaying the formation of internal perturbations. These objectives sufficiently regulate the space on on-ramp by dynamic green time allocation at arterials and dampen the upward shockwave formed at the upstream detectors. As we know, quick and effective response is crucial for congestion control because traffic queues can form up quickly but take longer time to discharge. Once traffic speed drops, freeway throughput will reduce by 5%-10% (Papageorgiou et al., 2003). The proposed ramp metering approach can
dynamically assemble ramp meter to work together and effectively coordinate the individual meter rates. This approach minimizes time to clear the congestion by leveraging the response strengths. To better present the new understanding, Figure 5 shows the outline of the steps involved. The proposed algorithm and the priority objectives will be explained in the following sections.

Figure 5. Steps in methodology

The proposed methodology for algorithm involves mainly three important tasks –

Task 1: Site investigation and video data collection;
Task 2: Developing an algorithm based on the three governing objective functions with a well calibrated simulation model; and

Task 3: Evaluating the performance of algorithm by comparing it with other algorithms such as ALINEA based on various factors such as delay, throughput is the final task.

Priority objective functions play a vital role in the performance of algorithm and proper implementation of empirical equations. These functions are necessary and should be well coded in the algorithm. Flowchart of the proposed algorithm is provided later in the chapter (Figure 7).

Before describing the details of the proposed algorithm, a typical layout of a freeway and on-ramp section, including the detector configurations is considered to setup the ground for the sensor data.

A typical layout of freeway section, as shown in Figure 6, includes the setup of both the upstream and downstream detector configurations effect. This setup is used to describe the methodology in the development of the research. Single loop detectors and dual loop detectors, the most extensively used traffic sensors are used in the study. Single loop detectors are used on both the freeway and the on-ramp.

Typically, loop detector stations are spaced around half a mile from each other in the dense corridors (Wang et al., 2009). Each on-ramp is employed with three loop detectors: queue detector, demand detector, and merge detector as illustrated in Figure 6. Detailed configuration of the detector and ramp metering system will be provided later in the next chapter (in Figure 9). On-ramp vehicle queue lengths, number vehicles on on-ramp, and flow rate on the on-ramp are measureable from these detectors which enable the proposed ramp metering algorithm to minimize the queue spillback by utilizing the available on-ramp queue storage.
In the past researches, traffic density has been used as the sole variable to measure the congestion on the freeway. However, using the density \( (d) \) alone for congestion detection may result in frequent mal-functioning of the algorithm because it is possible to observe a relatively high density value when many vehicles are traveling at a high speed (Zhang and Wang, 2013). Therefore a relatively high \( d \) value does not necessarily have a congestion scenario. Therefore, both speed \( (s) \) and density \( (d) \) are used for congestion detection. In the current research, as density is not directly measurable from loop detectors, lane occupancy is used as surrogate in practice (Zhang and Wang, 2013). The freeway section which is used as upstream detector station is located at \( k \) and downstream detector is located at location \( k+1 \). Traffic volume and lane occupancy can be measured at each detector location.

Therefore, density can be calculated by the empirical equation (Zhang and Wang, 2013)
\[
d(k,j) = \frac{O(k,j)}{l(k,j)}
\]  

(5)

Where,

\[
O(k,j) = \text{Occupancy at the dual loop detectors; and}
\]

\[
l(k,j) = \text{Length of the detectors;}
\]

Therefore, the congestion is detected if

\[
d(j,k) > d_{cr} \text{ and } s(j,k) < s_{cr}
\]

Here \(d_{cr}\) is the pre-specified critical density value and \(s_{cr}\) is the chosen critical speed value.

When other types of traffic sensors, such as radar sensor, dual loop or camera detectors (Chin and Fan, 2006), are used for congestion detection, lane occupancy and traffic speed may be directly measured.

5.2 First Priority Objective

Assuming the congestion is detected between stations \(k\) and \(k+1\) (upstream and downstream detectors) at interval \(j\), the excess demand over the roadway section \(k\) can be calculated as:

\[
\Delta D(k,j+1) = V(k,j+1)\Delta T + Q(k,j) + F_{on}(k,j+1)\Delta T - V(k+1,j+1)\Delta T
\]  

(7)

Where,

\[
\Delta D (k,j+1) = \text{Excess demand over the discharging capacity;}
\]

\[
Q(k,j) = \text{Number of queued vehicles when congestion is detected at time } k;
\]

\[
F_{on} (k,j+1) = \text{Traffic detector measured on-ramp inflow rate; and}
\]

\[
V(k,j+1) = \text{Mainstream flowrate measured at station i.}
\]
The upper limit of $Q(k, j)$ can be estimated by quantifying the number of vehicles within a freeway section (Zhang and Wang, 2013). When congestion is detected promptly, $Q(k, j)$ should be small and can be estimated using the mainstream volumes at Station $k$ and $k+1$ and corresponding on-ramp volumes as follows:

$$Q(k, j) \approx \sum V_{in}(k, x) - \sum V_{out}(k, x) = Q(k, j_0 - 1) + V_{in}(k, j) - V_{out}(k, j)$$

(8)

Where,

- $j_0$ = Time interval when congestion starts to form;
- $V_{in}$ = Volume of traffic entering this section through the mainstream and on-ramps measured at Station k in vehicles per time interval; and
- $V_{out}$ = Volume of vehicles per interval leaving the section measured by Station k+1.

The time between the time intervals $j_0$ and $j$ should be no shorter than the typical free-flow travel time under incident-free conditions over the link.

To prevent vehicles from further queuing up and finally have the queued vehicles discharged, $\Delta D$ must be controlled less than zero. Taking this condition into Equation (6) and re-arranging the terms yield

$$F_{on}(k, j + 1) \times T < V(k + 1, j + 1) \times T - V(k, j + 1) \times T - Q(k, j)$$

(9)

Where,

- $Q(k, j)$ = Number of queued vehicles when congestion is detected at time k;
- $F_{on}(k, j + 1)$ = Traffic detector measured on-ramp inflow rate; and
- $V(k, j + 1)$ = Mainstream flowrate measured at station i.
Equation (3) is considered the first priority control objective, indicating that the on-ramp flow rate must be constrained under a certain level to effectively address the congestion. While developing the algorithm, it is made sure that the objective function is satisfied. This is achieved by controlling the ramp metering rate on the on-ramp.

Vehicles are allowed to merge into the freeway only when a minimum headway is found on the rightmost lane which merges with on-ramp (Bunker and Troutbeck 2003). This strategy not only helps in controlling the on-ramp flow into freeway but also delays the formation of internal perturbations in the traffic at the upstream of the on-ramp. Therefore, when a congestion is detected i.e. when the density and speed reaches a critical value, ramp metering rate is regulated by allowing a vehicle only when a gap is found. The vehicle is then discharged from the on-ramp to meet the gap and have a relatively smooth merge into freeway. When the ramp metering setting satisfies the inequality, it is believed that freeway congestion will be mitigated for a period and the queued vehicles will be discharging appropriately. The discharge time however depends on the how much smaller the $\Delta D$ is.

5.3 Second Priority Objective

In the course of implementing the first priority objective, the ramp metering rate is regulated to satisfy the inequality. The potential threat of queue spillback while regulating the ramp metering rate should also be addressed. This is can be achieved by the good use of the storage space available on the on-ramp without over flowing into the arterial streets.
This regulation is accomplished through the secondary priority objective function as follows:

\[ D_r(j+1) - r_{on}(j+1) \times T + Q_r(j) \leq C_{r-on} \]  

(10)

Where,

\[ D_r(j+1) = \text{Traffic demand on on-ramp at interval } j+1, \text{ which can be estimated from the} \]

\[ \text{demand measured at the previous interval (Xie et al., 2007);} \]

\[ Q_r(j) = \text{The on-ramp stored vehicles at the end of interval } j; \text{ and} \]

\[ C_r = \text{The storage capacity of on-ramp (i.e. the number of vehicles that can be stored} \]

\[ \text{on the on-ramp).} \]

Through this inequality group, the maximum storage capacity of an on-ramp can be considered in deciding its metering rate and potential negative impacts can be avoided due to its vehicle spillover to local streets. The importance of this constraint is considered secondary for the entire system operations. The algorithm then checks for the internal perturbations (third objective) as shown in Figure.7 and then comes up with an optimal metering rate.

However, at high volume traffic conditions, the on-ramp storage reaches maximum and the vehicles may potentially start to spillover onto the arterial. Regulating the traffic feeding the on-ramps is necessary to mitigate the spillover caused at the on-ramp. This is accomplished by holding the phases which feed the on-ramp. If the system detects that the ramp meter is not in immediate danger of becoming oversaturated, the phases are served. When queues are detected on metered ramp, the signal would hold particular phases, either internal left-turn phases or the main-street, arterial through phases, depending on the intermediate queue conditions on the arterial street.
The disadvantage of holding the internal phases is that the arterial through traffic would be stopped and unnecessary delays to the traffic would occur. To avoid that, the green time is dynamically allocated to various phases of the arterials by providing minimum delays at the arterials.

5.4 Constraints

To theoretically integrate such an equity consideration in ramp metering, a constraint condition group can be established to enforce the metering rates of its demands. Setting these constraint conditions can systematically complement the objective function effectively. Also, a feasible ramp metering rate should be between zero and the minimum of either the on-ramp demand or the on-ramp storage capacity (number of vehicles). A single-lane metered on-ramp that allows one vehicle per green can serve up to 900 veh/hr. If two vehicles are allowed per green, then a single-lane metered on-ramp can serve from 1,100 to 1,200 veh/hr. (Highway Capacity Manual 2010) indicate that an uncontrolled single-lane on-ramp may have a throughput capacity of 1800 to 2200 vehicles per hour. Hence, a maximum capacity of 1200 vehicles per hour per lane is utilized in this study.

5.5 Third Objective

The primary purpose of the third objective is to minimize the impact of shockwave propagation on formation of congestion on mainline freeway. Helbing et al. (2006) reported that there exists a range of flow rate in free flow where traffic breakdown can occur due to the internal perturbations. This breakdown and the congestion can occur upstream to the merge location. Subsequently, the previous algorithms developed were unable to prevent traffic breakdown from appearing in the vicinity of the on-ramp merge area even though their control
objective is achieved. Many algorithms start functioning with the normal ramp meter rate that they were designed. Vehicles are allowed to merge into the freeway, which aggravates the congestion because of the impending vehicle discharging at the upstream of on-ramp. A platoon of vehicles starts discharging at the upstream of the on-ramp which would have a potential risk of causing multiple shockwave propagation. This is promptly discussed in this chapter and incorporated in the algorithm.

This process continues until the upstream location of the merge location is occupied resulting a disastrous traffic congestion. Therefore, the time required to discharge the complete queue is calculated using the equations mentioned in the literature. The flow chart of the proposed algorithm is shown in Figure 7. This objective is aimed at formulating the mechanism and mitigating the formation of multiple shockwaves yielding a ripple effect in the traffic congestion.
This is accomplished by dampening the formation of the new shockwave at the merge location. Dampening a shockwave can be divided into three types that can be applied in traffic flow theory.
5.5.1 Types of Dampening

a. Critically Damped

A system is said to be critically damped when it returns to equilibrium as quickly as possible and it reaches steady state. System reaches critically damped state when the algorithm is responsive to the congestion at downstream, upstream and beyond upstream detectors. This is achieved by providing the optimal ramp metering without causing congestion or any possibility of shockwave propagation formation. Critically damped system is not practically possible as no algorithm cannot prevent the congestion completely and maintain a balanced stable system.

b. Under Damped

In under damped system, the system oscillates and gradually becomes stable. The system becomes stable at a slower pace than the critically damped system. It is believed that many algorithms are under damped as they are not responsive to the congestion beyond upstream detectors and the ramp metering algorithms were not designed accordingly. The vehicles are allowed to merge into freeway, forming a shockwave and causing oscillating shockwave propagation.

c. Over Damped

In over damped system, the system returns to the stable state without oscillating. There will be no oscillation observed in the over damped system, which is accomplished by holding the ramp metering rate and by allowing the queue to dissipate from the upstream. Shockwave propagation will not be observed in the freeway but serious queue spillback will be observed in the system causing the congestion and safety problems.
Figure 8 shows the graphical representation of the dampening systems. Under-damped algorithms ramp metering rate would oscillate between low ramp metering rates to high ramp metering rate in order to reduce the congestion at the merge location as shown in Figure 8.

When congestion is detected at the merge location, the ramp metering rate is decreased (higher delays on on-ramp). Conversely, the ramp metering rate is increased when the ramp volumes reaches maximum causing congestion at the arterials. Many current algorithms are believed to be under-damped algorithms. The present research adopts an under-damped algorithm strategy aiming to achieve critically damped situation. Figure 8 illustrates typical types of dampening.

Critically damped is highly impractical when applied to traffic flow theory. This helps in reducing the formation of shockwave and congestion as much as possible at the merge location. Formation of internal perturbations is detected by the speed of vehicles at the upstream detectors. If the speed of the vehicles is below the critical speed, it is understood that the queue is caused due to internal perturbations of the synchronized traffic breakdown (FHWA, 2009).

The length of the queue is obtained from the equations from chapter 3. The queueing flow and queueing density and density of traffic at the upstream detectors are obtained from loop detectors. The time (seconds) required to damp the shockwave produced would then be calculated and will be given as an input to algorithm. The damping of the shockwave is triggered by increasing or holding the ramp meter rate high until the time where the queue is dissipated from the merge location.
The time until the maximum maintained ramp meter rate is calculated by the arrival rate to the queue, length of the queue and speed of the shockwave as given in the following equation:

\[
\text{Time (}\mu\text{)} = \frac{L_{\text{max}}}{W_d}, \mu \text{ being the damping factor.} \quad (II)
\]

Thus, considering all the factors from the third objective function, the optimal ramp metering rate is obtained. Then the algorithm loops back to measurement of the density and speed and a new optimal ramp metering rate is obtained as when required.

In order to provide a proof of concept study, case study site location is selected and the algorithm is implemented in a simulated environment. The algorithm is evaluated for performance using selected measures of effectiveness. The detailed simulation case study is discussed in the next chapter.
CHAPTER 6:
CASE STUDY AND SIMULATION MODEL

6.1 Study Site Location and Scenarios

In this study, a VISSIM-based simulation model was developed to simulate freeway ramp metering system operations. The ramp metering simulation model is established for one freeway segment and arterial along Kenwood Road on I-71 South. The selection criteria for this segment also include: recurrent and non-recurrent congestion. This is frequently observed along this segment which provides a flexible platform to examine the proposed algorithm performance. To be capable of testing the coordinated control mechanism, an arterial signal, on-ramp with detector configuration is included in the simulation model.

A geographic map of this freeway segment is shown below in Figure 1 and Figure 2 in chapter 2. The detector configuration and the signal system are shown in the Figure 9. The length of the corridor at the study site location is about 3.6 miles, and there are 3 lanes. While the posted speed limit is 55 mph, the free flow speed generally ranges from around 55 to 65 mph. The Size of the detectors on on-ramp and freeways is 6 x 6 ft. (FHWA and ADOT). Figure 9 shows the location of the ramp meter and length of the acceleration length on the selected roadway segment.

The VISSIM simulation model is configured based on the real roadway geometric features, ramp signal settings, traffic demands, and operational patterns mentioned above. The simulation model is calibrated using ground-truth traffic data collected at the case study location site. The archived GPS traffic speed measurements were available from the data collection. Additionally, volume and occupancy measurements were obtained from ARCADIS US
Company. The volume data was obtained for years 2014, 2019, 2035. Availability of volumes for different years has provided the flexibility to check the performance of the simulation model for varying volumes. In addition to that video data was also collected at the on-ramp and Euclid road bridge for data into simulation model. Virtual loop detectors are employed in the simulation model to match their real-world locations on the freeway segment, including the advance queue detectors at the entrance of each on-ramp, and merge detectors at each signal head. The models’ reliability is checked by comparing simulation outputs with observed field data at each loop station along the freeway corridor and ramps.

**Figure 9. Detector and signal configuration of case study**
Considering the importance of the gap acceptance in the present study, more importance is levied on the calibration of model. Our simulation model was calibrated following the guidelines of Arizona Department of Transportation. The traffic inputs, routing decisions, and driver behavior parameters are adjusted to minimize the difference between simulation outputs and ground-truth data. When the simulation outputs are reasonably consistent with the ground-truth traffic volume and speed measurements, the model’s calibration requirement is satisfied. Calibration of the gap acceptance is described in detail, later in this chapter. In order to accomplish the modelled integration control strategy, micro simulation is developed in VISSIM by using the suitable driving behavior parameters which suits the site conditions.

Two scenarios are considered in this research:

**Scenario: 1** – This scenario is considered as the base scenario where the site is tested for the ramp meter deployed on the on-ramp. The algorithm is simulated on the real-time roadway geometric features, signal settings, traffic demands and the operational moments. This scenario is mainly used to measure the performance of ramp metering for various algorithms on the ground conditions. The left turn feeding the on-ramp at the case study site locations remained un-signalized replicating the real world conditions. The assumptions considered in the scenario were that the traffic signal at the off-ramp of I-71NB is considered as a fixed time traffic signal, replicating the ground conditions.

**Scenario: 2** – This scenario is used to test for the signalized left-turn feeding the on-ramp with the ramp meter deployed on the on-ramp. The algorithm is also simulated on the real-time roadway geometric features, signal settings, traffic demands and operational moments. The vehicle operational moments from the left-turn feeding the on-ramp is coordinated signalized control. The signal is coordinated between the ramp meter and the arterial signal. The proposed
algorithm controls the left turn signal when the on-ramp space has reached the threshold occupancy.

The assumptions considered in the scenario were mainly, the traffic signal at the off-ramp of I-71NB is considered as a fixed time traffic signal, replicating the ground conditions. The effect of the upstream on-ramps is negligent on the current ramp meter being considered.

6.2 Measure of Effectiveness

This algorithm is then compared with traffic responsive algorithms such as ALINEA to measure the effectiveness of the proposed algorithm and recommendations have been proposed accordingly.

The performance metrics used for comparing the integrated algorithm with other algorithm:

- Mainline Freeway Travel time.
- Mainline Freeway Throughput, speed & Delay
- Arterial and Ramp & Delay
- Ramp Throughput & Delay

The proposed methodology for accomplishing the aforementioned scenarios consists of mainly four main tasks: 1) data Collection and post processing of the data; 2) calibration and Validation of the VISSIM simulation model using the collected data; 3) developing ALINEA algorithm and calculating the change in the parameters that were considered for the evaluation with respect to the use of ALINEA; and 4) developing the proposed integrated algorithm and evaluating the best suitable scenario on the basis of measure of effectiveness. Based on the obtained results, few recommendations have been made.
6.3 Case Study Site Investigation and Data Collection

As discussed in chapter 3, based on the historic data we have and through the preliminary analysis, the region between On-ramp connecting Kenwood Road is selected out of many critical locations identified. Accordingly the data is collected at the Kenwood Road of the I-71 South bound. The main area of focus would be from the on-ramp connecting from Kenwood road and I-71 South bound until the on-ramp connecting the Steward road and I-71 South bound. The volume counts of the traffic were obtained from ARCADIS.US Company. The Daily Hourly Volume (DHV) of the study area has been provided by the company which serves as the volume input for the research. The data collection is performed primarily to obtain the factors to validate the simulation model. Volume (15 minute interval) of the traffic entering the on-ramp is obtained from video and minimum acceptable time gap at the merge is obtained at the merge location.

6.4 Video Data Collection

6.4.1 Deployment of Data Collection Devices

Two cameras are deployed in this study. The first camera, Camera 1, was placed on the turf at the entrance of on-ramp to record arterial traffic entering the on-ramp. Figure 10 shows the location of the video cameras and the field of view for each camera. The orientation of Camera 1 on the turf was placed in such a way that the vehicles making a left turn at the arterial are adequately captured. Figure 11 shows the field of view of the video from Camera 1 which is placed at Kenwood road.

Another camera, Camera 2, was placed on Euclid road. Cones were placed on the shoulder lane of I-71 South for a span of 300ft with 50ft spacing between the cones as show in Figure 12. While counting the acceptance gaps of the vehicles, virtual lines are drawn on the
freeway based on the cones to facilitate the measurement of the critical headway in terms of distance. The field of view of the video from Camera 2 with sample markers is shown in Figure 12. This will help in calculating the time gap taken for vehicles to merge into the freeway.

Figure 10. Locations of cameras placed on the Kenwood Road and Euclid Road
6.4.2 Data Collection Period

Video data was collected on the AM-peak hour and PM-peak hour for a duration of about two and half hours

- 05/13/2015 from 7:00 am to 9:30am and
- 05/12/2015 from 4:00pm to 6:30pm

6.4.3 Data Obtained from video:

- 15 minute volume of traffic to calibrate volume of traffic entering freeway.
- Minimum acceptable time gap
- Volume: Manual counting of the videos was done to obtain 15 minute volume traffic. The collected volume was used for the calibration of the simulation model.
6.5 GPS Data Collection

Meanwhile, the GPS-based floating car data will be collected during peak hours. Multiple floating car runs were carried out every 30 minutes during the observation period. The runs will be carried over in the peak hours specified. The GPS data logger was switched on at Exit 11 of I-71 South. From Exit 11 the car was driven until Exit 10 of I-71 South is reached. The car was driven by following the traffic and by not being too aggressive or passive. Overtaking of the vehicles was also avoided. For consistency purpose, car was driven most of the time in the middle lane. Floating car runs were carried at least 3 times. Effectively, the data collection route is the floating runs between the exit-11 to Exit 10.

The runs were carried depending on the weather conditions. Timings were noted at the respective critical locations and notes were prepared if there was any unusual activity observed.
on the freeway. Table 1 shows the sample GPS trip log of the 10/15/2013.

Table 1. Sample log of the floating car runs

<table>
<thead>
<tr>
<th>Trip Number</th>
<th>Start time at Exit 44</th>
<th>Time at Camera</th>
<th>Speed (mph)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Trip 1</td>
<td>3:02PM</td>
<td>3:04PM</td>
<td>58.4</td>
</tr>
</tbody>
</table>

The current study site contains two arterial intersections on either side of the freeway on Kenwood road closer to on-ramp for I-71S as shown in Figure 1. Therefore, the intersection was observed in the peak hours and the cycle lengths for the major streets and the minor streets have been evaluated. It is found out that the minor streets have negligible amount of traffic when compared to the major street.

6.6 Validation and Calibration of the Model

6.6.1 Method

A microscopic traffic simulation model must be calibrated and validated before its application for real problem-solving practice. The calibration involves checking the model results against observed data and adjusting parameters until the model results fall within an acceptable range of error. The observed data included roadway traffic volume and travel time data. The following models embedded in microscopic traffic simulation software such as VISSIM are generally needed to be calibrated:

- Range of desired speed changes and distribution,
- Gap acceptance at the merge location
- Priority rules for merging and diverging traffic,
- Diving behavior models,
The network coding errors are major source of abnormal vehicular movements (Chu & Liu, 2004). Such errors can be found at any time during the process of the calibration. Accordingly, fixing network coding errors is an important task throughout the whole calibration process. In this study, range of desired speed changes and distribution, priority rules for merging and diverging traffic, diving behavior models involved in the VISSIM model are calibrated by comparing with available traffic flow volume, spot speed, and travel time at critical locations on both mainline freeway and on-ramps (Oregon Department of Transportation, 2011).

Critical locations are usually referred to the key points on roadway network which could impact the whole road network, such as bottleneck locations and merge points (Columbia River Crossing, 2006).

In addition to the aforementioned factors, gap acceptance parameter plays an important part in intersection simulation models. Mostly, when a traffic network is congested, drivers at the merge locations tend to be aggressive by not yielding to the traffic on the freeway. Therefore, modelling the simulation model for the existing conditions is challenging to produce unless more aggressive parameters are used to calibrate the model.

Critical headway represents the minimum time taken by the vehicle from on-ramp to merge safely into freeway. Usually, critical headway depends upon the driving behavior parameters, geometric layout, vehicle flow rate, speed, and vehicle characteristics. Accordingly, critical headway can be calculated by the accepted gaps of the vehicles merging into the freeway from on-ramp. Gaps between vehicles are typically quantified in two ways: Time gap and Space gap.

Time gap is the time elapsed by the front end of the two successive vehicles to reach the same point on the road. The time gap (time headway) is of two types: Accepted gaps and
Rejected gaps. Accepted gaps are those gaps where the vehicle from on-ramp utilizes the gap between vehicles on freeway to merge safely. On the other hand, rejected gaps are those gaps where the vehicles may not have enough opportunity to merge into the freeway and rejects the gap eventually. Vehicles would wait or look for an acceptable gap to merge safely into the freeway. There can be many rejected gaps but there will only be one accepted gap for a vehicle.

Space gap is the distance between the rear of the vehicle and front of the vehicle at a point of time. All traffic entering a merge location from on-ramp has to make a decision whether there is sufficient time and space for it to enter the intersection safely. The vehicles on the approaching stream assess the gaps on the conflicting stream. They decide based on their perception, if the time and space gap is too small to cross or large enough to safely maneuver their vehicle. In the former case they would reject the gap and speed up or slow down to merge into a different gap unless the traffic is stopped on the mainline. In the latter case they would accept the gap and merge into the circulating stream.

In this case study, acceptance gaps of the individual vehicles have been extracted from the video data collected at the case study site location. A data pool of all the accepted gaps was obtained by marking the vehicles in the video using the markers placed on the merge location. The percentage of error in extracting the data from the video is calculated by comparing it to the video taken from a different camera angle. The range of accepted gaps was extracted from the video manually and the critical headway is analyzed by plotting the gap acceptance curves of the vehicles.

6.6.2 Interpreting the Gap Acceptance Curves

The primary objective of the gap acceptance study is to find the critical headway which
will be utilized to calibrate the VISSIM simulation model. The default critical headway value in
VISSIM does not necessarily represent the real conditions. Therefore, the default values are
modified and the real-time critical headway values are used for the VISSIM simulation. Once the
gap acceptance data is collected and analyzed, distribution of the accepted and rejected gaps
would be obtained. Curves representing the frequency of rejection and acceptance of gaps are
then plotted. Three methods of analysis were investigated. These include:

- Raff’s critical gap;
- An “equal” overlapping area gap and;
- 50th Percentile of critical acceptance gap curve

50th Gap Acceptance Percentile curve method is used in the current research to obtain the
critical headway. In congestion periods, there can be many rejected gaps but only there will only
be a single accepted gap for a vehicle. Therefore, plotting the accepted gaps produced ‘an
exponential’ type curve. The 50th percentile gap acceptance was selected as it represents at least
half of the vehicles which definitely were accepted. Any gaps generated to the left of 50th
percentile distribution had a very low probability of acceptance. Minimum acceptable gap is
obtained by marking the 50th percentile in the curve. Minimum acceptable critical gap is then
entered in the VISSIM to calibrate the model. According to the historical traffic volume and the
GPS data and field observation, congestions were found in merge influence area is the on-ramp
from Kenwood road. The VISSIM model validation is done following the guidelines of Oregon
Department Transportation and the procedure is described as follows.

6.6.3 Validation of Traffic Volume

The GEH statistic approach is used to represent the goodness-of-fit of a model which was
invented by Geoffrey E. Havers (GEH). It is the principle used for comparing the traffic volumes between simulation and observation. The GHE statistic for a link is computed by using the following equation:

\[
GEH = \sqrt{\frac{2(m-c)^2}{(m+c)}}
\]  

(9)

Where,

\[m = \text{Output traffic volume from the simulation model (vph)}; \text{ and}\]

\[c = \text{Input traffic volume (vph}).\]

Table 2 summarizes the statistic guidelines of the GEH statistic of comparing the real time volumes and the simulated volumes. If the GEH statistic is less than 5, then the model is considered to be robust. If the GEH statistic is greater than 10, the model is considered to be unacceptable for consideration and its results cannot be used for the analysis. Table 3 shows the sample data of the GEH statistic which is adopted in this research.

**Table 2. GEH statistic guidelines**

<table>
<thead>
<tr>
<th>Criteria</th>
<th>Acceptance Targets</th>
</tr>
</thead>
<tbody>
<tr>
<td>GEH statistic &lt; 5.0</td>
<td>Acceptable fit</td>
</tr>
<tr>
<td>GEH &gt; 5.0</td>
<td>Unacceptable fit</td>
</tr>
</tbody>
</table>

**Table 3. Sample traffic volume validation result**

<table>
<thead>
<tr>
<th>Link ID</th>
<th>Actual Volume</th>
<th>Accepted Range of Volume</th>
<th>VISSIM Simulated Volume</th>
<th>GEH Statistic</th>
<th>Pass/Fail</th>
</tr>
</thead>
<tbody>
<tr>
<td>23726</td>
<td>5981</td>
<td>5083~6878</td>
<td>5664</td>
<td>3.9</td>
<td>Pass</td>
</tr>
</tbody>
</table>

6.6.4 Validation of Spot Speed

Once the volume output is validated for VISSIM Model, the driving behavior of vehicles
has to be calibrated to replicate the real time conditions. In order to achieve this, the primary step is to validate the spot speeds of the vehicles. Spot speeds on the freeway are obtained using the GPS data collected from floating car runs. Few critical locations have been considered all along the stretch of the freeway and the spot speeds observed at those locations should be less than 10% of the actual speed identified using the GPS data collected. Table 4 shows the sample data of the spot speed observed at those locations.

**Table 4. Sample comparison between observed speed and modeled speed**

<table>
<thead>
<tr>
<th>Link ID</th>
<th>Speed from GPS</th>
<th>Accepted Percentage</th>
<th>Accepted Range</th>
<th>VISSIM Simulated Speed</th>
<th>Pass/Fail</th>
</tr>
</thead>
<tbody>
<tr>
<td>8</td>
<td>52.74</td>
<td>±10%</td>
<td>47.47-58.01</td>
<td>40.41*</td>
<td>Fail</td>
</tr>
</tbody>
</table>

* The value does not satisfy the criteria

6.6.5 Validation of Travel Time

Travel time sections are setup at the various locations in the network to find the time take to travel in various peak hours in the simulation model. GPS data was used to obtain the ground travel time conditions. Calibration of the travel time should also be met at the peak hour volume at all the identified critical locations of the network, thereby, validating all the network parameters. Modeled travel time should be within the range of 15% of the actual travel time that was recorded from the GPS data. Table 5 shows the sample data of the travel time for validation.

**Table 5. Sample comparison between observed travel time and modeled travel time**

<table>
<thead>
<tr>
<th>Link ID</th>
<th>Travel time from GPS</th>
<th>Accepted percentage</th>
<th>Accepted range</th>
<th>VISSIM simulated travel time</th>
<th>Pass/Fail</th>
</tr>
</thead>
<tbody>
<tr>
<td>8</td>
<td>40.7</td>
<td>±15%</td>
<td>36.63-44.8</td>
<td>44.2</td>
<td>Pass</td>
</tr>
</tbody>
</table>
Using the calibrated simulation model, simulation experiments are conducted to examine the proposed algorithm’s performance under a variety of traffic condition. VISSIM COM interface is used to develop the algorithm using C# language and VISSIM VAP.
CHAPTER 7: RESULTS AND CONCLUSIONS

7.1 Data Acquisition and Analysis of Data

An extensive study has been conducted to provide proof of concept for the proposed methodology. The major task of the research was to explore and understand the integrated coordinated algorithm and compare its effectiveness with other algorithms. The primary task is to collect adequate data and its post analysis. As discussed earlier, this research needs the Video data and GPS data and the vehicle volumes as the inputs to the simulation model. Therefore, GPS data was collected by running the multiple car floating runs across the section of the freeway. The GPS-based floating car data was collected during peak hours. The spot speeds and the travel time were analyzed at the critical locations based on the data collected.

There were five reference points considered to evaluate the data collected. These five points were considered based on the merge areas and non-merge areas. Links in the network model have been divided accordingly to make the evaluations simpler. Table 6 shows the spot speeds obtained at the five different locations.

Table 6. Location and GPS co-ordinates

<table>
<thead>
<tr>
<th>S.NO</th>
<th>Location</th>
<th>Latitude</th>
<th>Speed (mph)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Before on-ramp merge-Kenwood road</td>
<td>39'11'39&quot;</td>
<td>58.11</td>
</tr>
<tr>
<td>2</td>
<td>After on-ramp merge-Kenwood road</td>
<td>39'11'35&quot;</td>
<td>60.50</td>
</tr>
<tr>
<td>3</td>
<td>Between exit-11 and exit-10</td>
<td>39'11'18&quot;</td>
<td>58.93</td>
</tr>
<tr>
<td>4</td>
<td>Before on-ramp merge steward road</td>
<td>39'11'1&quot;</td>
<td>59.40</td>
</tr>
<tr>
<td>5</td>
<td>After on-ramp merge- steward road</td>
<td>39'10'48&quot;</td>
<td>60.65</td>
</tr>
</tbody>
</table>
Accordingly, the links in the VISSIM were divided such that the each of this reference points constitute a link. Therefore, a complete five links have been created in the network model for the analysis.

7.2 Calibration and Validation of the model

Gap acceptance curve is plotted utilizing the extracted data from the video. An exponential type curve is obtained as shown below in Figure.13. 50\textsuperscript{th} percentile was calculated by interpolating the graph. The observed minimum acceptable headway from the graph was observed to be 2.96 seconds. In reality, the gaps accepted will be dependent on various factors like geometric layout, vehicle types and traffic conditions. In addition to the critical headway, which is the time taken for the rear vehicle to traverse to the same point, distance travelled by the vehicle is also calculated. These values were utilized to vary the driving behavior lane change and following parameters in VISSIM.

![Gap Acceptance Curve](image_url)

**Figure 13. Gap acceptance curve at merge location**
VISSIM microsimulation model was built using observed gap acceptance values and well validated following the DOT guidelines.

7.2.1 Validation of Traffic Volume

Table 7. Validation of traffic volume

<table>
<thead>
<tr>
<th>LinkID</th>
<th>Accepted Range of Traffic Volume (veh/hr)</th>
<th>Accepted Range of Traffic Volume (veh/hr)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Simulation Time (seconds)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>900</td>
</tr>
<tr>
<td>7</td>
<td>5083~6878</td>
<td>5714</td>
</tr>
<tr>
<td>6</td>
<td>4476~6056</td>
<td>4985</td>
</tr>
<tr>
<td>42</td>
<td>3812~5158</td>
<td>4213</td>
</tr>
<tr>
<td>53</td>
<td>322~436</td>
<td>356</td>
</tr>
<tr>
<td>33</td>
<td>370~501</td>
<td>394</td>
</tr>
<tr>
<td>37</td>
<td>352~477</td>
<td>399</td>
</tr>
<tr>
<td>8</td>
<td>479~649</td>
<td>511</td>
</tr>
</tbody>
</table>

Traffic volume is one of the important factors while simulating a network. The validation of traffic volume for the simulation model is shown above in Table 7. As discussed in earlier chapters, the criterion depends on GEH statistic. Below is the GEH statistic table.

Table 8. GEH statistic values for traffic volume calibration

<table>
<thead>
<tr>
<th>LinkID</th>
<th>VISSIM Simulated Volume</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Simulation Time (seconds)</td>
</tr>
<tr>
<td></td>
<td>900</td>
</tr>
<tr>
<td>7</td>
<td>3.5</td>
</tr>
<tr>
<td>6</td>
<td>3.9</td>
</tr>
<tr>
<td>42</td>
<td>4.1</td>
</tr>
<tr>
<td>53</td>
<td>1.2</td>
</tr>
<tr>
<td>33</td>
<td>2.1</td>
</tr>
</tbody>
</table>

* The value does not satisfy the criteria
As shown in Table 8, the GEH statistic of all the links have been passed, therefore the simulation model is calibrated and validated for the volume.

7.2.2 Validation of Travel Time

The travel time is validated with accepted percentage of 15%. The comparison between observed travel time and modeled travel time can be shown below in Table 9. Thus, the validation result of the travel time conforms to the validation standard. As the simulated travel time has been in the accepted range, the model is calibrated and validated for Travel time.

Table 9. Comparison between observed travel time and modeled travel time

<table>
<thead>
<tr>
<th>Link ID</th>
<th>GPS-based Travel Time (sec)</th>
<th>Accepted Percentage</th>
<th>Accepted Range</th>
<th>Simulation Time (seconds)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>900</td>
</tr>
<tr>
<td>7</td>
<td>40.7</td>
<td>±15%</td>
<td>36.63-44.8</td>
<td>43.8</td>
</tr>
<tr>
<td>6</td>
<td>64</td>
<td>±15%</td>
<td>57.6-70.4</td>
<td>62.9</td>
</tr>
<tr>
<td>53</td>
<td>27</td>
<td>±15%</td>
<td>24.3-29.7</td>
<td>30.3</td>
</tr>
<tr>
<td>33</td>
<td>27.5</td>
<td>±15%</td>
<td>24.8-30.3</td>
<td>27</td>
</tr>
<tr>
<td>37</td>
<td>30</td>
<td>±15%</td>
<td>27-33</td>
<td>32.5</td>
</tr>
<tr>
<td>8</td>
<td>22.4</td>
<td>±15%</td>
<td>20.2-24.6</td>
<td>20.4</td>
</tr>
</tbody>
</table>

* The value does not satisfy the criteria

7.2.3 Validation of Spot Speed

After volume output is validated for VISSIM Model, the next step is to match spot speeds. As it is shown in Table 10, there are two speed values that do not meet the criteria, in other words, 87.5% spot speeds that satisfy the criteria which are larger than the threshold. Thus, the validation results of spot speed conform to the validation standard. As the results suggest in
Table 10, the simulated speed values fall in the acceptable range. The model is calibrated and validated for speed.

**Table 10. Comparison between observed speed and modeled speed**

<table>
<thead>
<tr>
<th>LinkID</th>
<th>Speed from GPS (miles/hr)</th>
<th>Accepted Percentage</th>
<th>Accepted Range (miles/hr)</th>
<th>VISSIM Simulated Speed</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Simulation Time (seconds)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>900</td>
</tr>
<tr>
<td>7</td>
<td>52.74</td>
<td>±10%</td>
<td>47.47-58.01</td>
<td>44.82*</td>
</tr>
<tr>
<td>6</td>
<td>59.22</td>
<td>±10%</td>
<td>53.30-65.14</td>
<td>58.23</td>
</tr>
<tr>
<td>42</td>
<td>56.43</td>
<td>±10%</td>
<td>50.79-62.07</td>
<td>58.17</td>
</tr>
<tr>
<td>53</td>
<td>57.19</td>
<td>±10%</td>
<td>51.47-62.91</td>
<td>58.98</td>
</tr>
</tbody>
</table>

* The value does not satisfy the criteria

**7.3 CASE STUDY RESULTS**

**7.3.1 Delay of Freeways and Arterials**

In Table 11, we can see that considerable improvements were achieved by the proposed ramp metering algorithm over the ALINEA and Fixed Time ramp metering. This is demonstrated by the significant reduction in average vehicle delay. For example, in Scenario 1, the average delay of On-ramp, the first on-ramp upstream of the location, is 8.91 seconds per vehicle with the ALINEA and 9.4 seconds with Fixed Time ramp metering system. The average delay on on-ramp decreases to 2.19 seconds per vehicle with ALINEA and 2.68 seconds per vehicle with Fixed Time ramp metering when compared to the proposed ramp metering algorithm. This corresponds to an improvement of 24.5% and 29% respectively. Traffic throughput maintains almost at the same level. In addition to that, the average delay of the freeway corridor decreases from 42.3 seconds per vehicle with fixed time ramp metering to 30.1 seconds per vehicle in the
proposed ramp metering algorithm. The average delay of the overall system reduces by 10.20 seconds per vehicle in Scenario 1 and 10.3 seconds per vehicle in Scenario 2 when compared to ALINEA and 15.72 seconds and 16.1 seconds when compared to Fixed Time ramp metering.

Table 11. Delay of freeways and arterials

<table>
<thead>
<tr>
<th>Time Period: 7:00 AM - 8:00AM</th>
<th>Proposed Algorithm</th>
<th>ALINEA</th>
<th>Fixed</th>
<th>Reduction in delay</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Average Delay (secs)</td>
<td>Through -put (veh/hr)</td>
<td>Average Delay (secs)</td>
<td>Through -put (veh/hr)</td>
</tr>
<tr>
<td>Scenario 1 (without signalized left turn)</td>
<td>3.1</td>
<td>6138</td>
<td>40.2</td>
<td>6122</td>
</tr>
<tr>
<td>Freeway</td>
<td>6.72</td>
<td>499</td>
<td>8.91</td>
<td>492</td>
</tr>
<tr>
<td>On-ramp</td>
<td>30.1</td>
<td>5149</td>
<td>37.4</td>
<td>5134</td>
</tr>
<tr>
<td>Scenario 2 (with signalized left turn)</td>
<td>5.2</td>
<td>392</td>
<td>8.3</td>
<td>378</td>
</tr>
</tbody>
</table>

7.3.2 Throughput and Speed of Freeway

The throughput and the speed on the freeway is higher than ALINEA when compared to the proposed algorithm as shown below in Figure 14 and Figure 15. However, during high-volumes (traffic breakdown), the algorithm underperforms compared to ALINEA. Proposed algorithm works efficiently only until the On-ramp traffic volume reaches 580 veh/hr. Queue spillback into the arterials was also observed at the on-ramp when the on-ramp traffic volume is more than 580 veh/hr. As shown in Figure 14 and Figure 15, Throughput and speed of the freeway is observed to decrease after the ramp volume exceeded 580 veh/hr. ALINEA works
better after the on-ramp volume exceeds 580 veh/hr. In other words, the ALINEA likely outperforms the proposed algorithm as the ramp traffic exceeds 580 veh/hr.

Figure 14. Throughput of the freeway mainline under three ramp metering strategies

Figure 15. Speed of the freeway mainline under three ramp metering strategies
Meanwhile, the throughput and speed of Fixed Time ramp metering is less than ALINEA and the proposed ramp metering algorithm during all the volumes.

7.4 CONCLUSIONS

Freeway ramp metering systems have been successfully proved to improve the traffic flow on urban freeways. These systems improve the flow on the freeway by regulating the rate at which vehicles are allowed to enter freeway. However, amongst all the possible factors contributing to traffic congestion on freeways and arterials, lack of coordination in the operation of various system components is one of the major sources causing inefficiency. Most of the integrated algorithms which were developed to bridge the gap had their control actions selected based on either pre-defined plans or detected traffic condition. Additionally, implementation of the algorithm was complicated because of the excessive data requirements of the integrated algorithms. This research develops an integrated control strategy which integrates the ramp metering with arterial signal systems, in order to mitigate the freeway congestion and maintaining the throughput at the same time. The major contributions can be summarized as follows:

1.) The innovation this research will be reflective of developing an integrated control strategy using a multi-objective approach which can to minimize the delay associated with local arterials, mainline freeway, and on-ramp, decrease the travel time on the freeway and minimize the possibility of queue spillback into arterials.

2.) Unlike many integrated algorithms which have excess and complicated data requirements, the proposed algorithm is primarily uses only loop detector data.
3.) The proposed integrated algorithm considered the effect of traffic breakdown caused due to internal perturbations at the merge location of the freeway and on-ramp which was not considered in many integrated algorithms. The impact of the propagation of shockwaves from the merge location was minimized by dampening the shockwave.

4.) The proposed algorithm was successful in delaying the traffic breakdown due to internal perturbations and was effective in decreasing the travel time and delays by 24% and 18% respectively.

5.) The proposed integrated system potentially provides a fundamental platform for further functionality expansion for study by considering cluster of off ramps and on-ramps on the freeway integrating with arterial signal intersections.

Apart from these, the calibration of the simulation model has been given a big focus in this research study. The accepted gaps of the synchronized and congested flow were given as input to the simulation model for effective results.
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APPENDIX I: ALINEA ALGORITHM

PROGRAM RampMetering;

/* D:\VISSIM\Daten\__Training\VAP_RampMetering.214\RampMetering.vv */

CONST
  MAX_LANE = 3,
  KR = 70,
  OCC_OPT = 0.29;

/* ARRAYS */
ARRAY     detNo[ 3, 1 ] = [[3], [4], [5]];

/* SUBROUTINES */

/* PARAMETERS DEPENDENT ON SCJ-PROGRAM */
IF( prog_aktiv = 1 ) AND ( prog_aktiv0vv <> 1 ) THEN
  prog_aktiv0vv := 1;
  DT := 1;
ELSE IF( prog_aktiv = 2 ) AND ( prog_aktiv0vv <> 2 ) THEN
  prog_aktiv0vv := 2;
  DT := 1;
END END;

/* EXPRESSIONS */
Demand := Detection( 1 );

/* MAIN PROGRAM */
S00Z001: IF NOT init THEN
  S01Z001: init := 1;
  S01Z002: Set_sg( 1, off )
END;
S00Z004: cyc_sec := cyc_sec + 1;
S00Z005: IF cyc_sec >= cyc_length THEN
  S01Z005: cyc_sec := 0
END;
S00Z007: Set_cycle_second( cyc_sec );
S00Z008: laneNo := 1;
S00Z010: IF laneNo <= MAX_LANE THEN
S01Z010: IF detNo[ laneNo, 1 ] > 0 THEN
  S02Z010: out := out + Occup_rate( detNo[ laneNo, 1 ]);
S02Z011: laneNo := laneNo + 1;
GOTO S00Z010
S00Z013: \[ \text{timer}_\text{dc} := \text{timer}_\text{dc} + 1; \]
S00Z014: \[ \text{IF} \ \text{timer}_\text{dc} = (60 \ast \text{DT}) \ \text{THEN} \]
S01Z014: \[ \text{timer}_\text{dc} := 0; \]
S01Z015: \[ \text{qRamp} := \text{(Front}_\text{ends}(2)); \text{Clear}_\text{front}_\text{ends}(2); \]
S01Z016: \[ \text{oout} := \text{oout} / \text{MAX}_\text{LANE} / (60\ast\text{DT}); \]
S01Z017: \[ \text{cqRamp} := \text{qRamp} + \text{KR} \ast (\text{OCC}_\text{OPT} - \text{oout}); \]
S01Z018: \[ \text{cyc}_\text{length} := 60\ast\text{DT} / \text{cqRamp}; \]
S01Z019: \[ \text{oout100} := \text{oout} \ast 100; \text{RecVal}(1, \text{oout100}); \]
S01Z020: \[ \text{oout} := 0 \]
END;
S01Z021: \[ \text{TRACE (variable (cyc}_\text{length));} \]
S00Z023: \[ \text{IF} \ \text{cyc}_\text{length} < 4 \ \text{THEN} \]
S01Z023: \[ \text{Set}_\text{sg}(1, \text{off}); \]
ELSE
S00Z024: \[ \text{IF} \ \text{Demand} \ \text{THEN} \]
S01Z024: \[ \text{IF} \ \text{cyc}_\text{sec} = 0 \ \text{THEN} \]
S02Z025: \[ \text{Set}_\text{sg}(1, \text{redamber}); \]
S02Z026: \[ \text{cyc}_\text{sec} := 0 \]
ELSE
S01Z025: \[ \text{IF} \ T\text{red}(1) >= \text{cyc}_\text{length}-3 \ \text{THEN} \]
GOTO S02Z025
ELSE
S00Z027: \[ \text{IF} \ \text{Current}_\text{state}(1, \text{redamber}) \ \text{THEN} \]
S01Z027: \[ \text{Set}_\text{sg}(1, \text{off}); \]
ELSE
S00Z028: \[ \text{IF} \ \text{Current}_\text{state}(1, \text{off}) \ \text{THEN} \]
S01Z028: \[ \text{IF} \ \text{NOT} (\text{cyc}_\text{length} < 4) \ \text{THEN} \]
S01Z029: \[ \text{Set}_\text{sg}(1, \text{amber}); \]
END
ELSE
S00Z030: \[ \text{IF} \ \text{Current}_\text{state}(1, \text{amber}) \ \text{THEN} \]
S01Z030: \[ \text{Set}_\text{sg}(1, \text{red}); \]
END
END
END
ELSE
GOTO S00Z027
END
END;
S00Z032: RecVal( 2, cyc_length );

S00Z033: qRampHour := qRamp * 60 / DT; RecVal( 3, qRampHour )

PROG_ENDED:

/*---------------------------------------------------------------*/
using System;
using System.Collections.Generic;
using System.ComponentModel;
using System.Data;
using System.Drawing;
using System.Linq;
using System.Text;
using System.Threading.Tasks;
using System.Windows.Forms;
using VISSIMLIB;
using System.Collections;
using System.IO;
using Integrated_Test;
using System.Threading;
using System.Runtime.InteropServices;
using Integrated_Simulation_Class;

namespace Integrated_Test
{
    public partial class IntegratedAlgorithm : Form
    {
        bool stopsim = false;

        public IntegratedAlgorithm()
        {
            InitializeComponent();
        }

        static void Desktop()
        {
            Shell32.Shell objShel = new Shell32.Shell();

            // Show the desktop
            ((Shell32.IShellDispatch4)objShel).ToggleDesktop();
        }

        private void startButton_Click(object sender, EventArgs e)
        {
            if (!stopsim)
            {
                for (int z = 0; z < 2; z++)
                {
                    SigDataClass sds = new SigDataClass();
                    Vissim vissim = new Vissim();
                    String Path_network = "C:\Users\Manasa\Dropbox\AECOM\Thesis\Proposed_Algorithm\Proposed_Algorithm.inpx";
                    vissim.LoadNet(Path_network);

                    //Simulation parameters
                }
            }
        }
    }
}
ISimulation sim = vissim.Simulation;
int SimPeriod = 0;
vissim.Simulation.set_AttValue("SimPeriod", sds.End_of_simulation);
vissim.Simulation.set_AttValue("SimRes", sds.resolution);
//vissim.Simulation.set_AttValue("SimPeriod", End_of_simulation);
vissim.Simulation.set_AttValue("RandSeed", sds.randomseed);
//sim.Resolution = 4;
//int steps = sim.Resolution;
//sim.Period = 5400;
//sim.RunContinuous();
System.Console.WriteLine("Hello, World!");

//Adjust Random Seed
textBox2.Text = (Convert.ToInt32(textBox2.Text) + z).ToString();
sim.RandomSeed = Convert.ToInt32(textBox2.Text);
sim.RandomSeed = 5;

//Set evaluations to on
vissim.Net.Evaluation.set_AttValue("Delay", true);
vissim.Net.Evaluation.set_AttValue("TRAVELTIME", true);
vissim.Net.Evaluation.set_AttValue("QUEUECOUNTER", true);
vissim.Net.Evaluation.set_AttValue("COMPILED", true);
IEvaluation eval = vissim.Evaluation;
eval.set_AttValue("DELAY", true);
eval.set_AttValue("TRAVELTIME", true);
eval.set_AttValue("QUEUECOUNTER", true);
vissim.Net.DelayMeasurements.SetAllAttValues("COMPILED",true);
vissim.Net.DelayMeasurements.set_AttValue("COMPILED", true);
vissim.Net.DelayMeasurements.set_AttValue("FILE", true);
vissim.Net.DelayMeasurements.TravelTimeEvaluation.set_AttValue("COMPILED", true);
vissim.Net.Evaluation.TravelTimeEvaluation.set_AttValue("FILE", true);

//signal data initialization
int sigcount = vissim.Net.SignalControllers.Count;

//Get signal controllers
ISignalControllerCollection scs = vissim.Net.SignalControllers;
ISignalController SignalController1 = vissim.Net.SignalControllers.get_ItemByKey(1);
ISignalController SignalController2 = vissim.Net.SignalControllers.get_ItemByKey(2);

int sigNo = 0;

//Initialization of the attributes
#region Initialization
foreach (ISignalController sig in scs) {

foreach (ISignalGroup sg in sig.SGs)
{
    sg.set_AttValue("STATE", 1);
}

sds.intNo = sigNo;
sds.timer1 = 0;
sds.timer2 = 0;
sds.pres1 = false;
sds.pres2 = false;
sds.updateProg = 1;
sds.greenLeftMajor = 10 * sds.steps;
sds.yellow1Major = 13 * sds.steps;
sds.greenthroughMajor = 52 * sds.steps;
sds.yellow2Major = 55 * sds.steps;
sds.greenthroughMinor = 77 * sds.steps;
sds.yellow1Minor = 80 * sds.steps;
sds.maxcycle = 102 * sds.steps;

// Initialization ramp
sds.rampcycle[0] = sds.rampcycle[0] * sds.steps;

#endregion Initialization

int count = 1;
sds.timer1 = sds.steps;
sds.timer2 = sds.steps;
sds.ph1 = 1;
sds.ph2 = 5;
// while (count < (vissim.Simulation.get_AttValue("SimPeriod")) * vissim.Simulation.get_AttValue("SimRes"); && !stopsim)
while (count < (vissim.Simulation.get_AttValue("SimPeriod")) * vissim.Simulation.get_AttValue("SimRes"))
{
    try
    {
        // variables
        double satSpeed;
        double satOccupancy = 0.0000;
        double tempHeadway;

        // Detectors
        IDetector dets_101 = vissim.Net.Detectors.get_ItemByKey("101");
        IDetector dets_102 = vissim.Net.Detectors.get_ItemByKey("102");
        IDetector dets_103 = vissim.Net.Detectors.get_ItemByKey("103");
        IDetector dets_201 = vissim.Net.Detectors.get_ItemByKey("201");
        IDetector dets_202 = vissim.Net.Detectors.get_ItemByKey("202");

        // code...
IDetector dets_203 = vissim.Net.Detectors.get_ItemByKey("203");
IDetector dets_301 = vissim.Net.Detectors.get_ItemByKey("301");
IDetector dets_302 = vissim.Net.Detectors.get_ItemByKey("302");
IDetector dets_303 = vissim.Net.Detectors.get_ItemByKey("303");
IDetector dets_1 = vissim.Net.Detectors.get_ItemByKey("1");
IDetector dets_2 = vissim.Net.Detectors.get_ItemByKey("2");
IDetector dets_3 = vissim.Net.Detectors.get_ItemByKey("3");
IDetector dets_4 = vissim.Net.Detectors.get_ItemByKey("4");

satSpeed = vissim.Net.Detectors.get_ItemByKey("203").get_AttValue("SPEED");
satOccupancy = vissim.Net.Detectors.get_ItemByKey("203").get_AttValue("OCCUPANCY");
tempHeadway = dets_203.get_AttValue("HEADWAY");

#region DetectionInitialization
if (satSpeed < 30 && satOccupancy > 0.25) {
    int time = vissim.Simulation.get_AttValue("SimSec");
    int time_check = time_check+time;
    int time_interval = time_check+ 5*60*4;
    if(time_interval== time_check) {
        sds.FreewayVolume_NInterval = sds.volume_freeway;
        sds.OnrampVolume_NInterval = sds.volume_onramp;
        sds.ShockVolume_NInterval = sds.shockwave_volume;
        sds.first_flag = Firstpriorityobjective(sds);
        tempHeadway = dets_203.get_AttValue("HEADWAY");
        if(sds.first_flag == false) {
            if(tempHeadway < 1) {
                sds.pres1 = false;
                int i = sds.rampcycle[1];
                sds.rampcycle[1] = i + sds.extramp * sds.steps;
            }
            else {
                break;
            }
        }
    }
    else {
        List<int> Freeway_volume = new List<int>();
        List<int> Onramp_volume = new List<int>();
        List<int> Shockwave_volume = new List<int>();
        List<int> Shockwave_occupancy = new List<int>();
        Freeway_volume = vissim.Net.Detectors.get_ItemByKey("101").get_AttValue("GapTm");
    }
}
#endregion
Onramp_volume = vissim.Net.Detectors.get_ItemByKey("4").get_AttValue("GapTm");
Shockwave_volume = (vissim.Net.Detectors.get_ItemByKey("401").get_AttValue("GapTm") + vissim.Net.Detectors.get_ItemByKey("402").get_AttValue("GapTm") + vissim.Net.Detectors.get_ItemByKey("403").get_AttValue("GapTm"));
Shockwave_occupancy = (vissim.Net.Detectors.get_ItemByKey("401").get_AttValue("OCCUPANCY") + vissim.Net.Detectors.get_ItemByKey("402").get_AttValue("OCCUPANCY") + vissim.Net.Detectors.get_ItemByKey("403").get_AttValue("OCCUPANCY"));

ds.volume_freeway = Freeway_volume.Count() * 2;
s.shockwave_volume = Shockwave_volume.Count() * 3;
ds.shockwave_occupancy = Shockwave_occupancy.Count() / 3;
ds.density_approaching = sds.shockwave_occupancy / 6;
ds.density_Napproaching = sds.Shockwave_Noccupancy / 6;

maxQ = (double)vissim.Net.QueueCounters.get_ItemByKey(sds.QC_number).get_AttValue("QLenMax(Avg, Avg)");

sds.second_flag = Secondpriorityobjective(sds);
if(sds.second_flag == false)
{
  sds.pres1 = false;
  int i = sds.rampcycle[1];
  sds.cycle[1] = sds.greenthroughMajor * sds.steps;
}

sds.dampening = thirdObjective(sds);
if (sds.dampening > 0)
{
  sds.pres1 = false;
  int i = sds.rampcycle[1];
  sds.rampcycle[1] = i + sds.extramp * sds.steps;
}

// Write the string to a file.
System.IO.StreamWriter file = new System.IO.StreamWriter("C:\Users\Manasa\Dropbox\UC\Integrated Simulation\test.txt");
file.WriteLine(sds.dampening);
file.Close();

//Check Signal Lights

sds = IRampLights(sds, SignalController1, SignalController1);
sds = IArterialLights(sds, SignalController1,
SignalController1);

sds.timer1 += 1;
sds.timer2 += 1;

//

}  
try {
    File.Move(pathTextBox.Text + "sim.rsz", pathTextBox.Text + @"RunData\sim_ACT_" + textBox2.Text + ".rsz");
}  
catch (Exception ex) {
    MessageBox.Show(ex.Message);
}  
try {
    File.Move(pathTextBox.Text + "sim.stz", pathTextBox.Text + @"RunData\sim_algo" + textBox2.Text + ".stz");
}  
catch (Exception ex) {
    MessageBox.Show(ex.Message);
}  
try {
    File.Move(pathTextBox.Text + "sim.vlr", pathTextBox.Text + @"RunData\sim_algo" + textBox2.Text + ".vlr");
}  
catch (Exception ex) {
    MessageBox.Show(ex.Message);
}  
try {
    File.Move(pathTextBox.Text + "sim.vlz", pathTextBox.Text + @"RunData\sim_algo" + textBox2.Text + ".vlz");
}  
catch (Exception ex) {
    MessageBox.Show(ex.Message);
private SigDataClass IRampLights(SigDataClass sds, ISignalController SignalController1, ISignalController SignalController2)
{
    //double tempHeadway = 0;
    if (sds.timer1 == sds.steps)
    {
        SignalController1.SGs.get_ItemByKey(1).set_AttValue("STATE", 1);
    }
    else if (sds.timer1 < sds.rampcycle[0])
    {
        SignalController1.SGs.get_ItemByKey(1).set_AttValue("STATE", 1);
    }
    else if ((sds.timer1 > sds.rampcycle[0]) && (sds.timer1 < sds.rampcycle[1]))
    {
        SignalController1.SGs.get_ItemByKey(1).set_AttValue("STATE", 3);
    }
    else if (sds.timer1 > sds.rampcycle[1])
    {
        SignalController1.SGs.get_ItemByKey(1).set_AttValue("STATE", 1);
        sds.timer1 = sds.steps;
    }
    return sds;
}

private SigDataClass IArterialLights(SigDataClass sds, ISignalController SignalController1, ISignalController SignalController2)
{
    if (sds.timer2 <= sds.greenLeftMajor && sds.ph1 == 1 && sds.ph2 == 5)
    {
        SignalController2.SGs.get_ItemByKey(1).set_AttValue("STATE", 3);
    }
    else if (sds.timer2 >= sds.greenLeftMajor && sds.timer2 < sds.yellow1Major)
    {
        SignalController2.SGs.get_ItemByKey(1).set_AttValue("STATE", 4);
    }
    else if (sds.timer2 > sds.yellow1Major && sds.timer2 < sds.greenthroughMajor)
    {
        SignalController2.SGs.get_ItemByKey(1).set_AttValue("STATE", 1);
        SignalController2.SGs.get_ItemByKey(2).set_AttValue("STATE", 3);
    }
    else if (sds.timer2 >= sds.greenthroughMajor && sds.timer2 < sds.yellow2Major)
    {
        SignalController2.SGs.get_ItemByKey(2).set_AttValue("STATE", 4);
    }
}
```csharp
}
}
else if (sds.timer2 >= sds.yellow2Major && sds.timer2 < sds.greenthroughMinor)
{
    SignalController2.SGs.get_ItemByKey(2).set_AttValue("STATE", 1);
    SignalController2.SGs.get_ItemByKey(3).set_AttValue("STATE", 3);
}
else if (sds.timer2 == sds.greenthroughMinor)
{
    SignalController2.SGs.get_ItemByKey(3).set_AttValue("STATE", 4);
}
else if (sds.timer2 < sds.maxcycle && sds.timer2 < sds.yellow1Minor &&
         sds.timer2 > sds.greenthroughMinor)
{
    SignalController2.SGs.get_ItemByKey(3).set_AttValue("STATE", 4);
}
else if (sds.timer2 >= sds.maxcycle)
{
    SignalController2.SGs.get_ItemByKey(3).set_AttValue("STATE", 1);
    System.Console.WriteLine("Completion of one cycle=" + sds.timer2);
    sds.timer2 = sds.steps;
}
else if (sds.first_flag = false)
{
    SignalController2.SGs.get_ItemByKey(3).set_AttValue("STATE", 1);
}
return sds;

Boolean Firstpriorityobjective(SigDataClass sds)
{
    sds.first_flag = (sds.volume_onramp < sds.FreewayVolume_NInterval -
                      sds.volume_freeway);
    return sds.first_flag;
}

Boolean Secondpriorityobjective(SigDataClass sds)
{
    sds.second_flag = (sds.Onramp_capacity < (sds.OnrampVolume_NInterval -
                                             sds.volume_onramp + (sds.maxQ));
    return sds.second_flag;
}

int thirdObjective(SigDataClass sds)
{
    sds.speed_dampening = Convert.ToInt32((sds.ShockVolume_NInterval -
                                             sds.shockwave_volume) / (sds.density_approaching - sds.density_Napproaching));
    return sds.dampening;
}
```