The Thesis Committee for William Jared Wall
Certifies that this is the approved version of the following thesis:

A Microsimulation Analysis of the Mobility Impacts of Intersection Ramp Metering

APPROVED BY
SUPERVISING COMMITTEE:

Supervisor:

____
C. Michael Walton

____
Jing (Peter) Jin

A Microsimulation Analysis of the Mobility Impacts of Intersection Ramp Metering

by

William Jared Wall, B.S.Civ.Eng.

Thesis

Presented to the Faculty of the Graduate School of

The University of Texas at Austin

in Partial Fulfillment

of the Requirements

for the Degree of

Master of Science in Engineering

The University of Texas at Austin

December 2013

Dedication

To my amazing wife Lauren-Leigh, whose love, support, patience, and sacrifice have made all of this possible.

Acknowledgements

I would like to acknowledge Dr. Peter Jin, not only for developing the initial concept of Intersection Ramp Metering, but also for his assistance and guidance throughout this thesis.

Abstract

A Microsimulation Analysis of the Mobility Impacts of Intersection Ramp Metering

William Jared Wall, M.S.E.

The University of Texas at Austin, 2013

Supervisor: C. Michael Walton

Urban freeway demand that frequently exceeds capacity has caused many agencies to consider many options to reduce congestion. A series of solutions that falls under the Active Traffic Management (ATM) banner have shown promising potential. Perhaps the most popular ATM strategy is ramp metering. Ramp metering involves limiting the access of vehicles to freeways at an entrance ramp. By doing this, freeway throughput, speeds, and travel time reliability can be increased, while the number of traffic incidents can be decreased. This study examines the application of an innovative ramp metering strategy, Intersection Ramp Metering (IRM), at a section of Loop 1 in Austin, TX. IRM implements the ramp metering function at the intersection immediately upstream of the entrance ramp, rather than on the ramp itself. A microsimulation analysis of this application is performed in VISSIM, and the results confirm that freeway throughput (+10%), and system average travel time (-14%), can be improved, as well as several other performance measures.

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Chapter 1: Introduction

Urban freeways are becoming increasingly congested during peak hours. Many of these facilities are approaching or have already reached capacity. Particularly, the travel periods during the morning and afternoon peak hours see significant congestion. Agencies all across the country have been exploring many different options to ease this congestion. There are only three basic ways congestion can be lessened: increasing capacity, improving system operations, or reducing demand. Historically, the solution to congestion has been to increase capacity. This led to the construction and expansion of the freeway system that we currently have. However, demand has been constantly increasing and many of these expansions merely keep congestion at constant levels. Many cities across the country have expanded their freeways near the absolute limit, yet congestion is still increasing. Now that system expansion is becoming an increasingly difficult option, transportation officials must now look to improving operations and reducing demand as the primary approach toward easing this congestion.

One heavily researched and tried and true method of improving operations is Active Traffic Management (ATM). Many agencies are now turning to various ATM strategies to help alleviate congestion and improve system performance. ATM involves the implementation of intelligent transportation systems to dynamically manage traffic operations based on prevailing conditions to alter traffic flow and driver behavior [1]. It can be helpful toward alleviating both recurrent and non-recurrent congestion. Typically applied toward urban freeways, most ATM strategies can delay the onset of or reduce the duration of congestion, and in some cases prevent it entirely. One of the primary objectives of such systems is to improve travel time reliability. An additional windfall for many ATM strategies is that they concurrently increase system throughput and improve highway safety by reducing the number and severity of crashes.

The travel time reliability of a roadway segment is dictated by the amount and character of congestion that that particular stretch of road experiences. The indices that describe travel time reliability are measures of the variation in travel times across a segment for different users. The higher this variation, the less reliable that segment will be for producing consistent travel times.

The designed goal of many ATM strategies is to improve flow along a freeway, either by implementing a control system on the mainlines or at a freeway entrance ramp. One of the more commonly used ATM strategies implemented at entrance ramps is ramp metering. Ramp metering involves controlling the access of vehicles entering a segment of freeway. With only a traffic control signal, entrance ramp flows can be easily adjusted by the presiding agency. By carefully adjusting the rate at which vehicles can enter the freeway, several operational benefits can be obtained.

At uncontrolled ramps, random vehicle arrivals and large platoons arriving from surface street intersections complicate merging and weaving movements and cause a traffic breakdown that has the potential to propagate upstream. These locations are considered bottlenecks for the mainlines of a freeway. By guaranteeing a uniform arrival rate and breaking up these vehicle platoons, ramp metering will mitigate or in some cases even eliminate the traffic breakdown that occurs at these points [2]. This uniform arrival rate helps facilitate simpler weaving and merging movements. When a ramp metering system is properly calibrated, it has the potential to increase freeway volumes, increase overall travel speeds, reduce incident rates, and decrease fuel consumption and vehicular emissions [3].

The first ramp metering system was implemented on the Eisenhower Expressway in Chicago, Illinois in 1963. It consisted of a traffic officer standing on the ramp and sending vehicles to the freeway one at a time at a predetermined rate. By the 1970's, cities such as Minneapolis, Minnesota began using permanent traffic control devices. These early devises were pre-timed with fixed metering rates [4]. The 1990's saw a rise in the use of traffic responsive systems, which yielded much better system performance results. Today, more sophisticated ramp metering systems are being widely used in at least 29 cities in the United States [5].

Austin, Texas suffers from some of the worst freeway congestion in the nation, according to researchers at the Texas Transportation Institute. They estimate that Austin commuters spend an average of 44 hours a year stuck in traffic. These delays add up to an annual cost of $930 per commuter. Austin has the worst planning time index score in the state, at 4.26. This means that for a trip that would take 30 minutes under uncongested conditions, commuters should allow themselves 2 hours and 8 minutes to be assured to making to their destination in time. Austin's planning time index score was the 6th worst of the 101 largest U.S. cities included in the study. It is apparent that an ATM strategy that could help lessen this congestion would be beneficial to Austin [6].

1.1 – Problem Statement

In this research, a novel ramp metering approach is introduced. It seeks to facilitate the ramp metering process at the intersection immediately upstream of the entrance ramp, as opposed to the ramp itself. This process is called Intersection Ramp Metering (IRM). This method will have limited applicability, specifically to where ramp geometries do not allow for a traditional ramp metering system, and where there is plenty of queue storage at the intersection approach. This technique could be further beneficial, in that it could reduce the cost associated with having to install a separate traffic control hardware system on the ramp. Instead, it uses the existing hardware at the intersection. The additional cost could be eliminated altogether if the intersection controller is sophisticated enough to facilitate a ramp metering algorithm and could be connected to detectors on the freeway mainlines, if these detectors already exist. However, if detectors need to be installed, this would not be cost incurred over traditional ramp metering systems because mainline detectors are needed for each. Like traditional ramp metering systems, this system strives to improve travel conditions on the freeway by carefully controlling the dispatch of vehicles onto the freeway.

In addition to the description of the system is an initial evaluation of its effectiveness. It is important to remember that the freeway and performance effects of IRM are not intended to be superior to existing ramp metering techniques; instead it is intended to be equally as effective as the existing systems. The evaluations consist of calibrated VISSIM model simulations, based on an existing intersection in Austin, TX, using real peak hour traffic data collected from that site. Several different evaluation scenarios are presented, and recommendations are made regarding the results.

1.2 – Thesis Summary

This thesis introduces and evaluates a novel ramp metering approach where the control is applied at the immediate upstream intersection instead of the ramp itself. Chapter 2 summarizes general active traffic management strategies as well as several ramp metering control algorithms and traffic signal control timing optimization strategies. Chapter 3 describes the methodology of the research. Chapter 4 presents the development of the microsimulation model. Chapter 5 discussed the design of the experiment. Chapter 6 presents the simulation results of the research. Chapter 7 discusses recommendations and conclusions based on these results as well as future research approaches that should be taken.

Chapter 2: Literature Review

2.1 Summary of Prevailing ATM Technologies

Active traffic management is defined by the Federal Highway Administration (FHWA) as the "ability to dynamically manage recurrent and non-recurrent congestion based on prevailing and predicted traffic conditions." Specifically, its intention is to improve trip reliability, maximize the effectiveness of a system, and improve safety through the use of systems integrated with technology. Crucial to the effectiveness of ATM is that it features the automation of dynamic deployment, rather than deployment by human operators. Most ATM strategies work by simply influencing driver behavior in a way that will improve system operations. The most popular ATM strategies are summarized in Table 2.1 below [1].

Table 2.1 - Popular ATM Technologies

ATM Strategy	**Concept**	**Used on Freeway or Arterial**	**Control System**	**Traffic Impact**
Adaptive Ramp Metering	Traffic Signals on freeway onramps dynamically control the rate of vehicles dispatched from ramp based on mainline freeway conditions	Freeway	Traffic Control Signal, Capable Signal Controller, Mainline and Ramp Detectors	Improved Freeway flow and travel speeds, reduced delay

Table 2.1 (continued) – Popular ATM Technologies

ATM Strategy	Concept	Used on Freeway or Arterial	Control System	Traffic Impact
Adaptive Traffic Signal Control	Continuously monitors arterial traffic conditions and adjusts intersection timing (phase lengths, cycle lengths, offsets, etc.) to achieve predetermined objective (minimize delay, maximize flow, etc.). Often monitors traffic well upstream of intersection so that arriving traffic patterns are known.	Arterial	Traffic Control Signal, Capable Signal Controller, traffic detectors at intersection and upstream	Increase in throughput, decrease in delay and queue lengths
Dynamic Junction Control	Dynamically controls lane assignments at freeway onramps and off-ramps based on prevailing freeway and arterial conditions. For example, when exiting volumes are high relative to through freeway volumes, the right freeway lane could be designated as exit-only. If entering volumes are high relative to existing freeway volumes, a lane drop could be implemented upstream of the entrance ramp to allow entering vehicles an additional acceleration lane	Freeway	Lane Assignment Indicators, detectors on mainlines and ramps and/or arterials	Increase in throughput, decrease in delay
Dynamic Lane Reversal	Reverses travel direction on lanes to dynamically allocate directional capacities based on prevailing traffic conditions.	Freeway or Arterial	Lane Assignment Indicators, detectors on mainlines	Increase in capacity
Dynamic Lane Use Control	Dynamically opens or closes lanes based on existing freeway conditions. It can be used in advance of recurrent bottlenecks or incidents. Advanced warning is provided to assist merging movements upstream of the lane closure	Freeway	Lane Status Indicators	Increase in throughput, decrease in delay

Table 2.1 (continued) – Popular ATM Technologies

ATM Strategy	Concept	Used on Freeway or Arterial	Control System	Traffic Impact
Dynamic Merge Control	Dynamically manages the merging of vehicles based on prevailing travel conditions. It can provide guidance and merging instructions well upstream of a merge point	Freeway	Merge Control Signage	Increase in throughput, decrease in delay
Dynamic Shoulder Lanes	Dynamically allows usage of road shoulder to increase the roadway capacity during congested periods. It can be used either during recurrent bottlenecks or unexpected conditions such as incidents.	Freeway	Shoulder Opening Status Signage	Increase in capacity
Dynamic Speed Limits	Dynamically changes speed limits based on prevailing freeway traffic or weather conditions. Can be freeway wide or individual lane assigned limits,	Freeway	Speed Limit Signage and mainline detectors	Uniform Speed
Queue Warning	Dynamically displays warnings of downstream queues or bottlenecks in an attempt to lessen shockwaves and reduce rear-end crashes	Freeway	Queue Warning Signage	Uniform Speed
Transit Signal Priority	Dynamically adjusts intersection signal timings upon the arrival of a transit bus, by either extending the green interval or bringing up the green interval sooner, in an attempt to lessen the number of intersections the bus will stop at.	Arterials	Bus detection sensors, Capable Traffic Controllers	Decrease in transit travel time

2.2 Ramp metering

Ramp metering control is implemented on freeway entrance ramps. While the control strategies and logic may be unique for different systems, the basic concept is the same. By installing ramp meters, engineers can control the rate that vehicles are allowed to enter a freeway, reduce freeway demand, and break up vehicle platooning caused by releases from upstream signals [2]. Figure 2.1 below shows the basic layout for either a single lane or dual-lane ramp metering system.

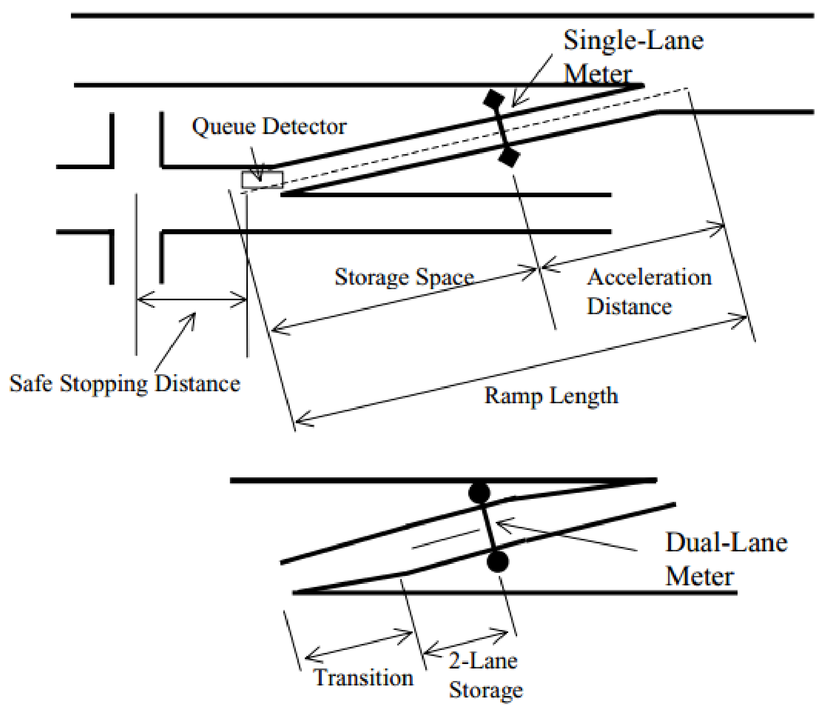

Figure 2.1 - Ramp Meter Geometric Characteristics

It can be seen above that the meter is typically placed on the ramp, where sufficient acceleration distance for freeway-bound vehicles can be provided. It is also important that the meter not be located so far up the ramp that there will not be enough storage space for the queuing vehicles.

The FHWA, in their Ramp Management and Control Handbook, recommends agencies consider the following 6 elements before determining their ramp metering strategy [7]:

- Geographic extent – the ramp metering will either be isolated around one or several ramps, or it will be a part of a larger coordinated system
- Approach – pre-timed or traffic responsive
- Metering algorithm – logic used to determine the metering rate
- Queue management – how ramp queues will be held to an acceptable length
- Flow control – how vehicles will be dispatched from the ramp (one at a time or several at a time)
- Signing – how drivers will know if the system is on or off

Each of these six elements is important in determining how a ramp metering system will be implemented. However, the importance of each of these can be ranked differently by different individuals.

Determining whether or not a pre-timed or traffic responsive approach will be best depends on the type of congestion that is occurring, and the amount of detectors that are available. In general, the traffic benefits of a traffic responsive system are greater, but so are the capital costs. If the congestion is predictable and nearly always recurrent, a pre-timed system may be an acceptable approach. This is particularly true when there are no detectors on the ramp or on the mainline near the ramp. If the agency determines that installing detectors in these locations is not feasible, then a traffic responsive system would not be possible. Furthermore, there are two degrees of traffic responsive systems. One will switch on the metering system when the traffic reaches a critical point. In this system, the ramp metering will begin at a predetermined rate. Another type of traffic responsive system, and the one that is used by a majority of modern agencies, is one in which the ramp metering rate itself is determined by traffic conditions. Typically the ramp discharge rate will be lower when there is more congestion on the mainline [7].

There are several metering algorithms that are commonly used in practice, and several more that have been developed but have yet to be implemented. Each algorithm utilizes a different control logic, and each has different singular objectives. However, the goal of each is to lessen freeway congestion and improve system performance. In general, the more complex an algorithm is, the more sophisticated the controller hardware must be and more detectors that would be required. This would make the system more susceptible to equipment failures and would make it more expensive [8]. For this reason, many agencies prefer the simple, yet very well performing, techniques.

There are two different types of ramp metering strategies, local and coordinated. Coordinated systems will typically use local control algorithms on the ramp level, and include some further control logic that seeks to weave each ramp's operations together to achieve optimal operations throughout a freeway corridor. The following are three of the most popular local control strategies, ALINEA, Demand Capacity, and Occupancy Control [9].

2.2.1 ALINEA

One of the more common algorithms for ramp metering is called Asservissement LINéaire d'Entrée Autoroutière, or simply, ALINEA. ALINEA is a very popular local feedback ramp metering strategy. It has been used extensively with little tweaking since its introduction in the early 1990's. It is still used widely throughout Europe with much success [10]. The intent of the ALINEA system is to maximize mainline flow by maintaining a desired occupancy level. Hence, it only requires one detector per lane, which will be located downstream of the entrance ramp. Because mainline occupancy is the only determinate for a standard ALINEA system, no detectors are required on the entrance ramp itself. The control algorithm for ALINEA calculates metering rates that will be applied to achieve the desired mainline occupancy. The following equation is used to calculate the ramp metering rates in ALINEA [11]:

$$r(t) = r(t-1) + K_r[O_{des} - O_{dn}(t)]$$

Where $r(t)$ is the ramp metering rate at time step t, O_{des} is the desired occupancy, which is typically the critical occupancy, where the freeway's flow is maximized. Typical values for the desired occupancy range from 18% to 31% [12]. O_{dn} is the measured downstream occupancy at time t, $r(t-1)$ is the metering rate from the previous time period, and K_r is a regulatory parameter [3]. A value of 70 vehicles per hour has been used extensively for Kr with much success [13]. One of the key operational advantages of ALINEA is both the simplicity of the control algorithm and the minimal detector requirements. Figure 2.2 depicts the required detector locations for ALINEA. The detectors should be placed such that vehicles dispatched from the ramp will reach the detector within the determined time-step for this equation to hold true [3]. Oftentimes, the available detector location will be the key parameter in determining the time-step length.

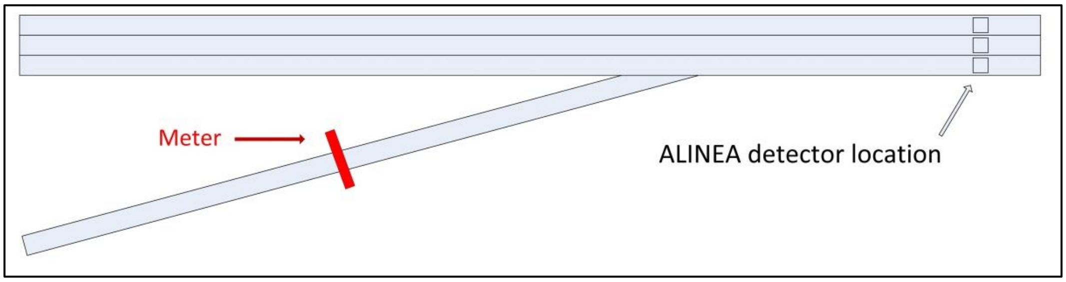

Figure 2.2 - Required Detector Locations for ALINEA

The ALINEA technique is a tried and true method that has performed well over many simulations and field implementations with minimal adjustments. Papageorgiou et al. lists the many benefits of ALINEA, which includes its simplicity due to only one control equation and variable, its low implementation cost, its efficiency, and its flexibility due to the fact that the desired occupancy level can be adjusted at any time, either automatically or manually.

Furthermore, since ALINEA is a feedback control philosophy, it attempts to avoid freeway congestion before it occurs, rather than control it after it occurs, which is what is done in feed-forward control philosophies. By the feedback approach being based on downstream measurements rather than upstream measurements, it is theoretically more suitable for controlling downstream conditions than a feed-forward approach, such as Demand Capacity or Occupancy Control [13]. Consider the following control diagram on the left side of Figure 2.3, which details the feed-forward approach philosophy.

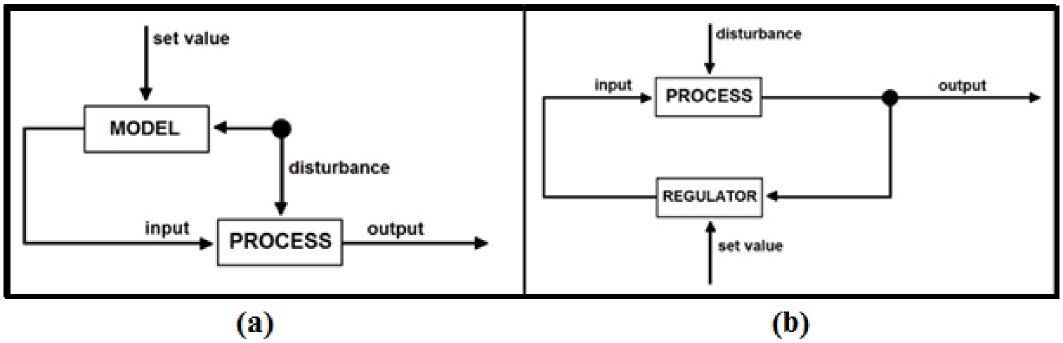

Figure 2.3 – Comparison between Feed-Forward (a) and Feedback Control (b)

This control diagram represents the logic of the feed-forward and feedback approaches. The feed-forward system is regulated by a set value for a parameter; which in the case for our example is the desired mainline flow. The process output would be mainline flow, which we desire to be equal to the freeway capacity. The process input would be the onramp flow, which is dictated by our calculated ramp metering rate. The disturbance would be the mainline upstream conditions. In order to achieve the desired output, this type of system measures the disturbances and applies what the model calculated to be the appropriate input to be combined with the disturbance to achieve the desired output. For this reason this type of approach is often called disturbance compensation. Because of the presence of immeasurable disturbances, this structure is highly sensitive. Furthermore, the success of this system is reliant on a highly accurate model, because the output is never measured and the model is never readjusted to reach a more suitable output. A more robust approach is the feedback control approach, which is utilized by the ALINEA algorithm [11]. This approach is represented by the control diagram on the right side of Figure 2.3.

In this approach, the output is considered rather than the disturbance. We still have our set value acting as the regulator, but in this instance it is modifying the controllable input to keep the output as close to the desired value as possible. By the input being consistently readjusted based on the output, we can be certain that we are producing a stronger result. In the previous feed-forward approach, the output is never measured; therefore we have to trust the accuracy of our model and the measured disturbances in order to be assured we are near our desired output. In the feedback approach, we are updating the input based on the output, and can therefore be certain we are nearing the desired output assuming only modest changes in the disturbance [13].

Two popular feed-forward approaches are Demand Capacity and Occupancy Control [8]. Because these approaches employ a feed-forward approach, the freeway occupancy is measured upstream of the merge point. The detector locations are shown in Figure 2.4 below.

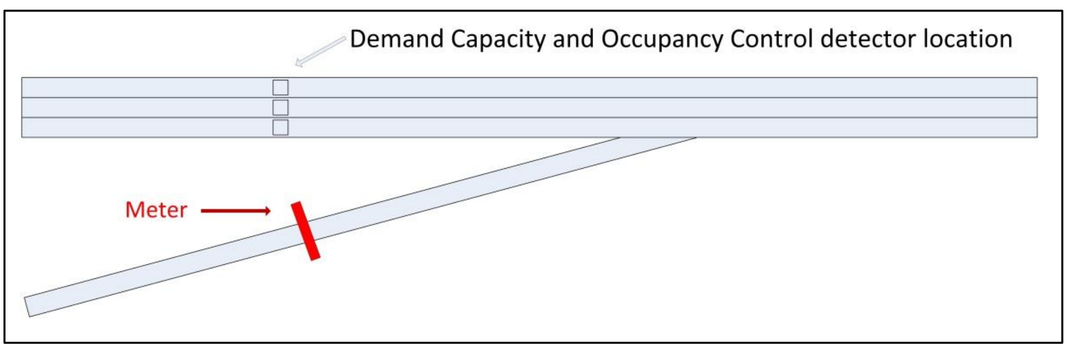

Figure 2.4 - Required Detector Locations for Demand Capacity and Occupancy Control

2.2.2 Demand Capacity

The Demand Capacity strategy (DC) utilizes the following equation to determine ramp metering rates:

$$r(t) = \begin{cases} q_{cap} - q_{up}(t-1), & O_{up}(t-1) \leq O_{cr} \\ r_{min}, & else \end{cases}$$

Where q_{cap} is the capacity of the downstream freeway segment, q_{up} *(t – 1)* is the upstream freeway flow at time t-1, O_{up} *(t-1)* is the upstream freeway occupancy, O_{cr} is the critical downstream occupancy where freeway flow is at its maximum, and r_{min} is the minimum allowable ramp flow.

This strategy seeks to add to the upstream flow the amount of ramp flow necessary to reach the downstream capacity. If the last measured upstream occupancy is greater than the critical occupancy, the system reverts to sending the lowest allowable flow [10]. The r_{min} is a parameter that would be determined by the agency implementing the system, and would typically be a function of the ramp queue length, storage capacity, and vehicle arrival rate.

2.2.3 Occupancy Control

Occupancy control (OCC) is a special form of the DC strategy that assumes a linear relationship between the occupancy and flow at a point on the freeway is maintained up until the critical occupancy is reached. This relationship can be described by the following equation:

$$q_{up} = \frac{v_f * O_{up}}{g}$$

Where v_f is the freeway free-flow speed and g is the g-factor that converts occupancy to density. This formula yields an estimation of q_{up} based on the measured occupancy, which can, under certain circumstances, reduce implementation costs [13]. With this estimation having been made, the metering rate is the same rate that is used in DC when the measured occupancy is under the critical value, which is shown in the following formula:

$$r(k) = q_{cap} - \frac{v_f}{g} * O_{up}(t-1)$$

This is a simpler version of the DC strategy, but is even more inaccurate due to the assumption of linearity from the fundamental diagram [10].

These control algorithms describe how vehicles are dispatched from a single ramp during the ramp metering process. Papageorgiou et al. argues that ALINEA is superior to DC and OCC because of the fact that occupancy is the controlled variable, rather than volume, because traffic volumes over a detector will be the same for congested and light traffic [13]. Another advantage cited is that the critical occupancy is less sensitive to weather and outside influences than the capacity is.

When multiple ramps are considered together, a coordinated system may be developed. Most coordinated systems use one of the above mentioned local ramp metering algorithms at the individual ramp level. The following describes several popular coordinated ramp metering strategies.

2.2.4 ZONE

The ZONE algorithm was originally used in Minneapolis, Minnesota. This system divides a freeway segment into various zones, between 3 to 6 miles in length, which are characterized by boundaries an upstream free-flow section and a downstream bottleneck section. The objective is to maximize throughput through these bottleneck locations [14]. The algorithm seeks to control volumes within the zone. The control equation is:

$$M + F = X + B + S - (A + U)$$

Where M is the total volume of the metered ramps in the zone, F is the total metered freeway to freeway ramp volumes, X is the total off-ramp volumes, B is the volumes of the downstream bottleneck section at capacity (usually assumed to be approximately 2,200 vehicles per hour per lane), S is the space available within the zone, which is estimated based on mainline occupancy, A is the measured volumes at the upstream free-flow section, and U is the total measured volumes of the non-metered ramps. In this equation, M and F are variables that can be controlled; all others are either measured or pre-set. For each individual meter, two metering rates are calculated. One is a local occupancy control algorithm, which is an overriding mechanism intended for non-recurring congestion. The other is a system-level metering rate that is determined by comparing the five measured variables (X, B, S, A, and U) with a series of thresholds based partly on historic peak hour traffic volumes [15].

Although the system was continuously amended to improve performance, public skepticism over the effectiveness of ramp metering began to rise along with the rise in demand throughout the 1990's. Eventually, it was mandated that for an 8 week period in

2000, the entire Minneapolis ramp meter system would be shut off, and before and after data compared.

The findings from the mandatory shutdown confirmed that the ZONE metering system reduced freeway congestion, freeway delays, fuel consumption, vehicular emissions, and number of incidents. At the same time, it increased freeway throughput and average speed and it optimized freeway merging [16].

2.2.5 Stratified ZONE

Although the ramp-meter holiday proved the system wide benefits of ramp metering, there was still some controversy over onramp wait times. This forced the Minnesota DOT to consider the ramp queues, and a new system was developed for the region called stratified ramp metering [15]. The stratified ramp metering system still incorporates the basic concept of ZONE, but it also factors in ramp demand and queue sizes. The control objectives are listed as:

1. Control flow into a zone so that the capacity is not exceeded
2. Limit ramp wait times to below the predetermined value

In order to achieve these objectives, the system utilizes a hierarchal control structure with two tiers. The first involves the zone itself. All ramp meters within a zone are assigned an allocated proportion of zonal capacity, based on their respective ramp demands. The ramp metering rates are based upon this allocation, provided they are within the predetermined range of acceptable metering rates, which is determined to be 1,714 vph to 240 vph. In the case where a ramp meter is contained within two overlapping zones, the most restrictive metering rate is always used. The release rates and control logic are very similar to those used by the ZONE system.

The second tier in the hierarchy involves the ramp itself. Two ramp status variables are introduced, ramp demand and minimum release rate. Ramp demand is the hourly flow rate of vehicles wishing to enter a ramp, and is measured by a detector located at the far upstream end of the ramp and another just past the stop bar of the ramp. The minimum release rate is time-varying and is calculated from the ramp queue lengths. This variable is important in ensuring that the ramp wait times are below the predetermined value. The formula used for calculating the minimum rate is:

$$r_{min} = \frac{N}{T_{max}}$$

Where N is the number of vehicles in the queue and T_{max} is the predetermined maximum allowable ramp wait time for a vehicle, which is 2 minutes for a freeway-to-freeway ramp and 4 minutes for a standard local access ramp. Because the calculation of ramp demand requires an upstream ramp detector, stratified ramp metering has a greater number of detector requirements than ZONE. A schematic depicting the ramp level detector requirements for each system is shown in Figure 2.5 below.

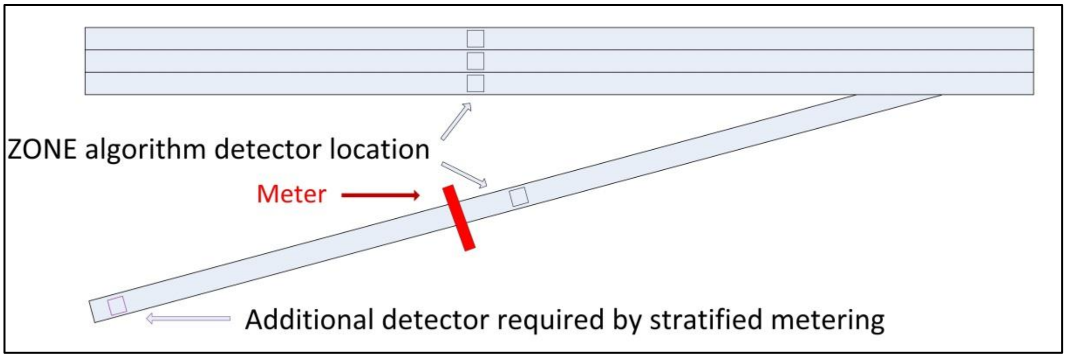

Figure 2.5 - Detector Requirements for ZONE and Stratified Ramp Metering

When the proposed release rate determined in the first tier drops below the minimum release rate determined in the second tier, the minimum release rate overrides until the proposed rate is once again greater than the minimum. If the override feature is triggered and is maintained throughout several control cycles, the minimum release rate will gradually increase until it reaches the ramp flushing rate of 1,714 vehicles per hour. If this scenario occurs, it will have a negative impact on freeway conditions.

The stratified system has met its objectives of improving ramp wait times, but this is achieved at the detriment of several mainline performance measures. However, results have shown that total system performance is still improved over the non-metering case [15].

Selection of a ramp metering algorithm is an important step in implementing a ramp metering system. Each agency, upon deciding that they want to use ramp meters, will need to select the algorithm that best suits their desired objectives, and is best compatible with the available hardware and controllers.

2.2.6 Queue management

Queue management is an important issue, and one of the main reasons the Minnesota DOT switched from ZONE to Stratified ZONE as mentioned above. In the year 2000, Minneapolis performed the famous experiment where the ramp metering system was shut off for an 8 week period. The catalyst for this experiment was largely public outrage over the effectiveness of the ramp metering systems, which primarily stemmed from long onramp wait times. The study ultimately concluded that ramp metering provided an overall benefit in both safety and mobility to freeway facilities, but it also highlighted that onramp wait times were often unbearably long during the ramp metering operation. These conclusions led to the development of the stratified ramp control algorithm, which factors ramp queues in the calculation of the ramp metering rate. A successful ramp metering system should never have queues that back up to the point where they adversely affect surface street traffic. In fact, many systems include detectors at the far upstream end of an entrance ramp that will turn off the ramp metering system when it reaches a certain occupancy level. This process is known as ramp flushing [3].

Ramp flushing policies are usually implemented to ensure that ramp metering does not adversely affect the performance of surface street intersections. However, much research suggests that not only does ramp flushing degrade the performance of ramp metering; it may even cause the system to perform worse than a non-metered ramp would. Several simulation results have shown that when ramp flushing is permitted, the benefits of traffic responsive ramp metering are minimal [3]. It is recommended that if ramp demand is greater than the meter capacity for any time period throughout normal operations, then a ramp meter that permits flushing should not be installed.

2.2.7 Flow control

There are three popular strategies for characterizing the flow of vehicles through a ramp metering system. The first, and most popular, is a single-lane one car per green strategy. This approach allows one car to enter the freeway for every ramp metering cycle. The second approach is a single-lane multiple car per green strategy. This is also known as platoon or bulk metering. Here, two or more vehicles are allowed to enter the freeway each metering cycle. This approach is less effective towards improving freeway conditions, because it essentially recreates small platoons. Also, this approach does not necessarily increase the capacity of metering systems, because the longer greens times require longer yellow times. Therefore there are not as many ramp metering cycles, hence the lack of any significant increase in capacity. Finally, is dual-lane metering. This consists of two adjacent lanes on the freeway entrance ramp, which reduces to one lane before the freeway merge. The dispatch pattern for this dual-lane approach allows the first vehicle in one lane to go, followed by the first vehicle from the next lane. Because a vehicle from one lane can be released while a vehicle in the other lane is still coming to a stop at the meter, this approach can sustain about 90% more capacity than a single lane approach [2].

2.2.8 Signing

Signing for ramp metering systems should be done such that users will be able to quickly and easily identify how the system works and when it is on. The FHWA's Manual on Uniform Traffic Control Devices requires that only 6 standards be followed when implementing ramp metering systems, which are listed below as shown in Section 4I.02 and 4I.03 [17]:

- Ramp control signals shall meet all of the standard design specifications for traffic control signals, except as otherwise provided in this Section.
- The signal face for freeway entrance ramp control signals shall be either a two-section signal face containing red and green signal indications or a three-section signal face containing red, yellow, and green signal indications.
- If only one lane is present on an entrance ramp or if more than one lane is present on an entrance ramp and the ramp control signals are operated such that green signal indications are always displayed simultaneously to all of the lanes on the ramp, then a minimum of two signal faces per ramp shall face entering traffic.
- If more than one lane is present on an entrance ramp and the ramp control signals are operated such that green signal indications are not always displayed simultaneously to all of the lanes on the ramp, then one signal face shall be provided over the approximate center of each separately-controlled lane.
- Ramp control signals shall be located and designed to minimize their viewing by mainline freeway traffic.
- The RAMP METERED WHEN FLASHING sign shall be supplemented with a warning beacon (see Section 4L.03) that flashes when the ramp control signal is in operation.

2.2.9 Geometric Requirements

The installation of a ramp meter signal on an entrance ramp should only be attempted if the location meets certain geometric requirements. In their Roadway Design Manual, the Texas Department of Transportation (TxDOT) provides requirements for minimum ramp length [2].

First, there must be sufficient stopping distance from the upstream intersection to the back of the queue. This point coincides with location of a queue detector. TxDOT's design criteria suggests that for a 35 mph design speed, no less than 240 ft. would be desired as the stopping distance. This value was calculated using the American Association of State Highway and Transportation Official's (AASHTO) stopping sight distance equation [18].

Additionally, there must be sufficient storage space on the ramp itself for vehicles queuing to get on the freeway. This storage space is between the queue detector and the meter itself. For a single lane meter, the required storage distance in feet is:

$$L = .820V + 0.000244V^2 \qquad V \leq 1600\ vph$$

Where L is the required storage distance in feet, and V is the expected peak-hour demand in vehicles per hour.

Finally, there must also be enough space provided to allow vehicles to accelerate comfortably from a stop to a safe merging speed by the time they reach the freeway merge point. The AASHTO values for this distance based on merging speed and ramp grade are shown in Table 2.2 below.

Table 2.2 - Required Distance between Meter and Freeway Merge Point (ft)

Merging Speed (mph)	Ramp Grade (%)		
	-3	**0**	**3**
37	295	367	492
43	417	518	682
50	591	748	1027
56	814	1060	1529
62	1086	1450	2182

2.3 Signal Timing

When adjustments are being made to existing signal timings, certain justifications must be made and certain objectives must be targeted. Because driver behavior is such an integral part of traffic engineering, there is never one correct optimal approach for timing an intersection. Typically, however, certain principals are always followed. One, the total number of phases should be minimized. This is to reduce the total amount of lost time, or the time that no vehicles are being processed through an intersection. The more often a phase changes, the more clearance intervals there will have to be, and the more lost time that will be incurred. Additionally, the analyst should always seek to maximize the amount of movements that can be served per phase, if possible. The fewer approaches that are stopped at an intersection, the fewer the vehicles that will be experiencing stopped delay. In general, signal timing approaches are designed to minimize the amount of control delay inflicted upon drivers.

2.3.1 Webster's Optimal Cycle Length

One of the more popular signal timing approaches was developed by F.V. Webster in 1958. He sought to create a timing scheme in which average control delays could be minimized [19]. In this, he developed one of the first widely used, accurate measures for control delay. His estimates were based on quantifying the lost time caused by the starting-up of vehicles in a queue, and the slowing of vehicles as the yellow indication is displayed. These variances in flow contribute to control delay, in addition to the stopped delay incurred during the red interval. A demonstration of these delays can be seen in Figure 2.6 below.

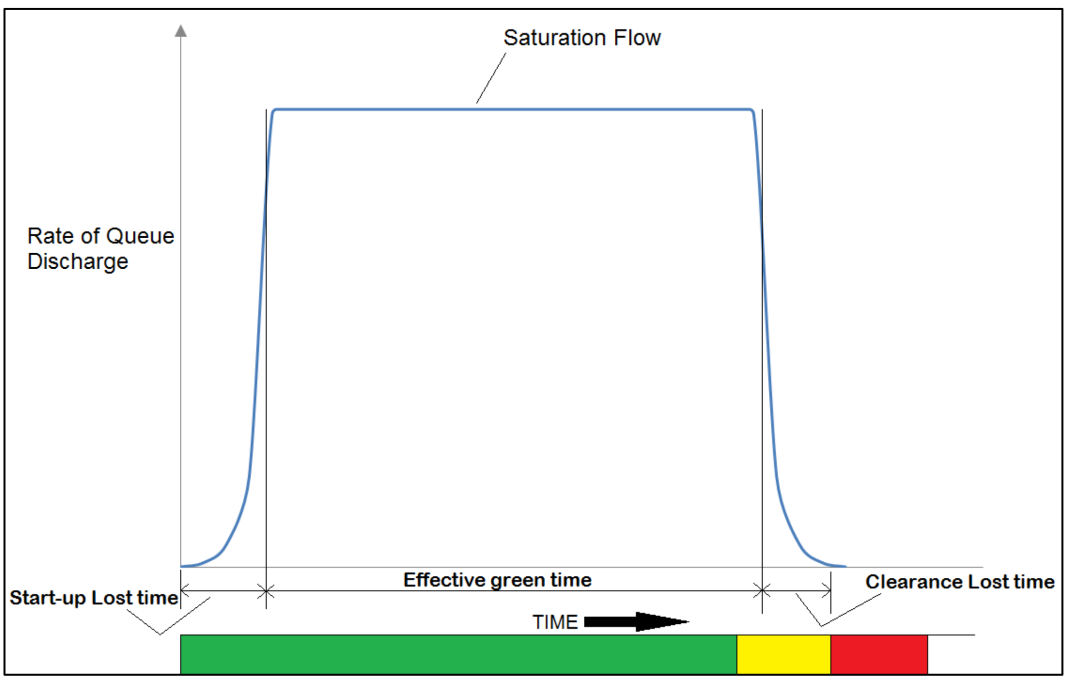

Figure 2.6 - Variation in Queue Discharge Rate During Phase Interval in Saturated Conditions

In addition to quantifying the lost times, this approach allows us to quantify the effective green time per phase. Based on these principals, Webster was able to estimate average delay through theoretical assumptions and computer simulations, which is shown in the following equation:

$$d = \frac{c\left(1-\frac{g}{c}\right)^2}{2\left[1-\left(\frac{g}{c}\right)x\right]} + \frac{x^2}{2q(1-x)} - 0.65\left(\frac{c}{q^2}\right)^{\frac{1}{3}} x^{2+5\left(\frac{g}{c}\right)}$$

Where d is the average delay per vehicle for this particular approach in seconds, C is the cycle length in seconds, g is the effective green time in seconds, x is the degree of saturation (this is the ratio of actual flow to maximum possible flow), and q is the arrival rate in vehicles per second.

The first two terms are purely theoretical. The first term quantifies delay based on a period of uniform arrivals into the intersection. The second term allows for stochastic variability during periods where a Poisson arrival pattern and a constant processing rate through the intersection can be assumed. The final term is empirically derived from the simulation results. Its inclusion makes the model fit the observed results. This final term makes up only about 10% of the total delay.

From this equation, a relationship between delay and cycle length can be deduced. An approximate representation of this relationship is shown in Figure 2.7 below. It can easily be seen that there is a certain value for cycle length where the total delay is minimized.

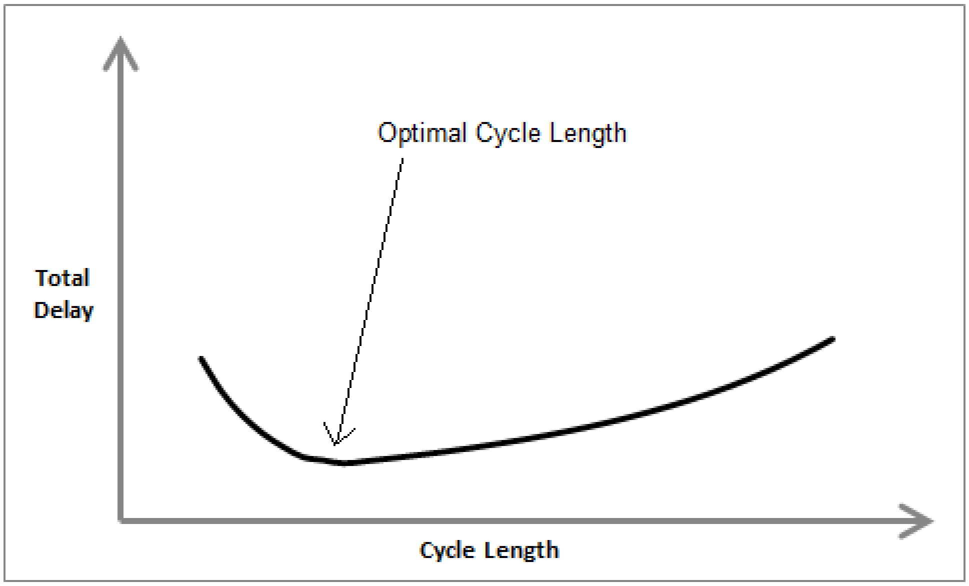

Figure 2.7 - Approximate Relationship between Delay and Cycle Length

This value for optimal cycle length can be found by differentiating the delay equation with respect to cycle length and setting it to equal zero. Webster published an approximated value of this result, citing the true solution as too complicated for practical applications. Instead, he developed an approximation for optimal cycle length, which is:

$$C_{opt} = \frac{1.5L + 5}{1 - \sum\left(\frac{CLV_i}{S_i}\right)}$$

Where C_o is the optimal cycle length in seconds, L is the sum of lost time for all phases in seconds (typically assumed to be the sum of all red and yellow intervals), CLV_i is the critical lane volume for approach i, and s_i is the saturation flow for approach i (typically 1800 vphpl). The critical lane flow is the flow of the lane with the highest volumes for that phase, accounting for left and right turn equivalencies.

Once the total cycle length is calculated, phase lengths can be calculated based on the relative critical lane volumes, as shown in the following equation:

$$g_i = \left(\frac{CLV_i}{\sum_{i=1}^{n} CLV_i}\right)\left[(C_{opt}) - \left(\sum_{i=1}^{n}(y_i + r_i)\right)\right]$$

Where gi is the green time for phase i, yi is the yellow time for phase i, and r_i is the red time for phase i. This relationship basically states that the green time for a cycle should be divided amongst a cycle based on the relative volumes for each phase.

2.3.2 Timing Strategies for Diamond Interchanges

Diamond interchanges present a unique signal timing situation. Because of the existence of frontage roads, Texas utilizes a slightly different control scheme for diamond interchanges than other jurisdictions. The phasing plan is dictated by the geometric allowances of the intersection, which in turn is dictated by the land use pattern adjacent to the interchange. The key distinction between the different classificational functions of diamond interchanges is the spacing between the two intersections. These classifications are as follows:

- Conventional Diamond – Intersection spacing is greater than 800 ft. These interchanges are most commonly found in rural areas and are typically stop sign controlled.
- Compressed Diamond – Interchange spacing is between 400 and 800 ft. These interchanges are commonly found in suburban areas, and are

typically signal controlled. The two intersections do not have to use an interconnected signal system.

- Tight Diamond – Interchange spacing is less than 400 ft. These intersections are typically found in urban areas, and will be signal controlled. Because of the close proximity of the two interchanges, it is practically required that they be designed as one system.

Researchers at the Texas Transportation Institute summarized guidelines that are used for timing such intersections in Texas [20]. Basically, for compressed and tight diamonds, there are two different control strategies that are used, the Texas Three-Phase and the Texas Four-Phase Control Strategies. The Three-Phase Strategy is best used when there is sufficient storage between the two intersections, typically a compressed diamond. This strategy does not seek to ensure that the space between intersections will be cleared at the end of each phase. A diagram of the Texas Three-Phase Control Strategy is shown in Figure 2.8 below.

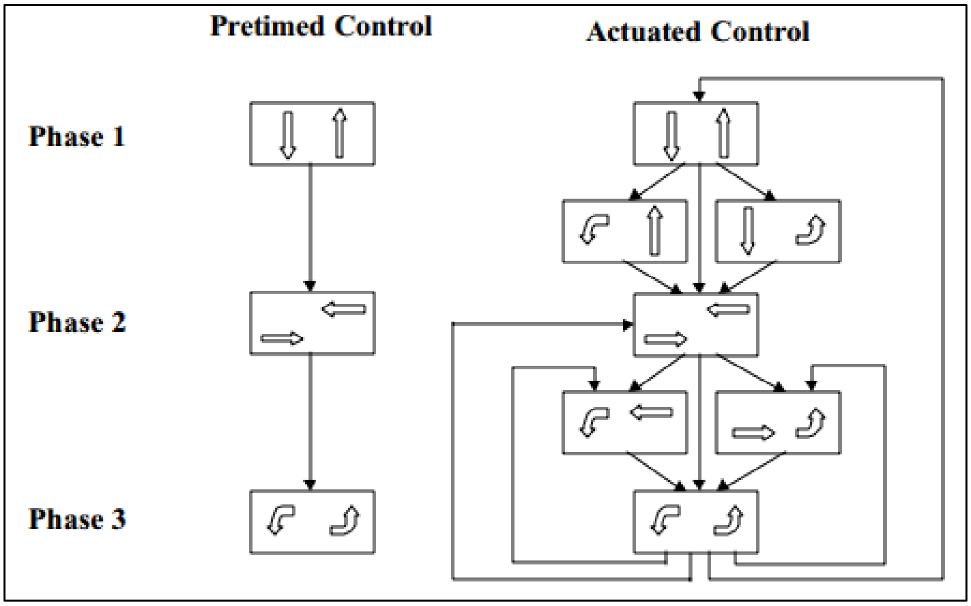

Figure 2.8 - Texas Three-Phase Control Strategy

When the intersection spacing is tight and there is little to no storage space in the area between the two intersections, the phasing pattern should be arranged so that this space will be cleared at the end of each phase. This is where the Texas Four-Phase Strategy comes in. It utilizes a split-phasing pattern, with overlaps between frontage road and arterial phases to prevent excess lost time. The control diagram for this strategy is shown below in Figure 2.9.

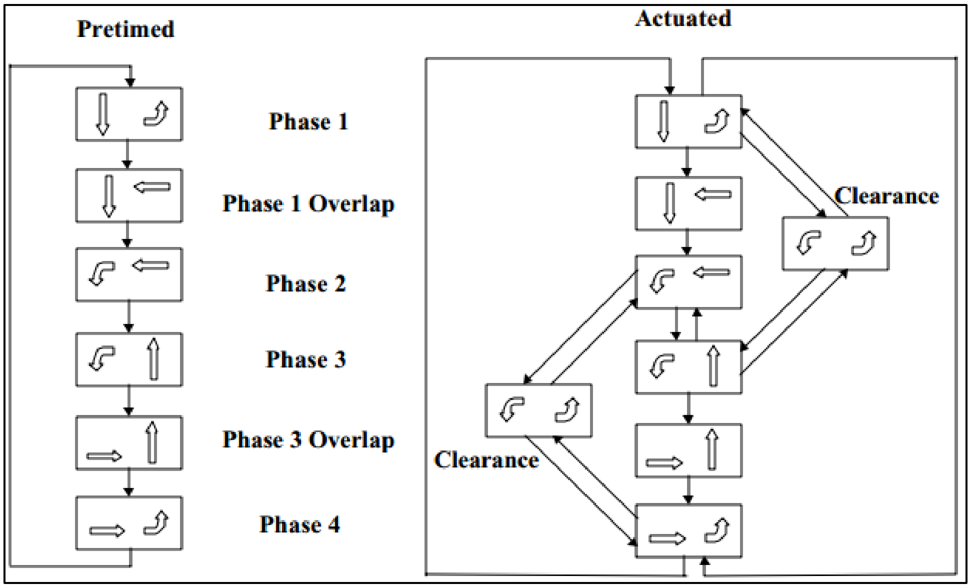

Figure 2.9 - Texas Four-Phase Control Strategy

Also, a technique to calculate phase splits is presented. The National Electrical Manufacturers Association (NEMA) phase numbering system for a diamond interchange is shown in Figure 2.10 below.

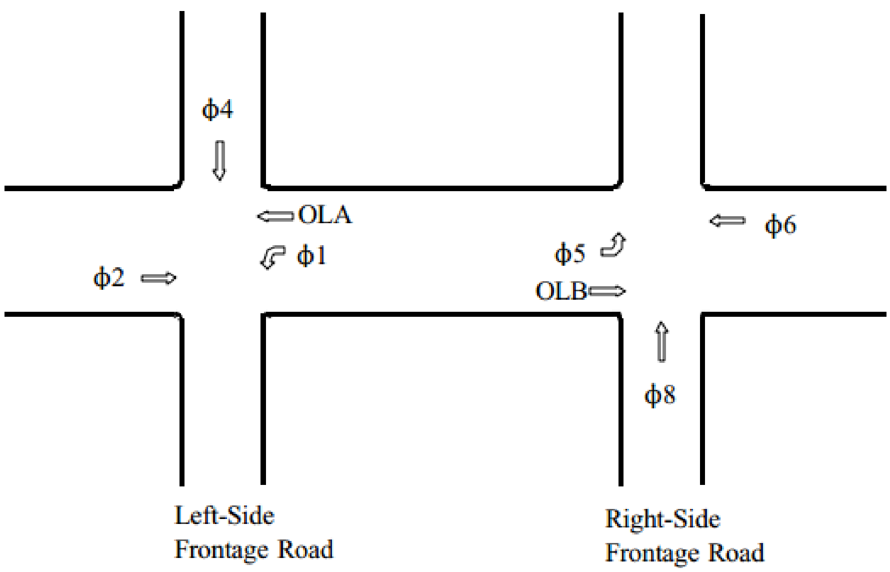

Figure 2.10 - NEMA Phase Numbering

The first step in calculating the phase splits is calculating the overlap. The overlap calculation is as follows:

$$\boldsymbol{\Phi} = \boldsymbol{\Phi}_{LR} + \boldsymbol{\Phi}_{RL}$$

Where Φ is the overlap time, Φ_{LR} is the travel time from the left intersection to the right intersection minus two seconds, and Φ_{RL} is the travel time from the right intersection to the left intersection minus two seconds. The following table, Table 2.3, is provided to assist in the determination of travel times between intersections.

Table 2.3 - Appropriate Travel Times Between Intersections

Design Speed (mph)	Link Distance (feet)														
	100	150	200	250	300	350	400	450	500	550	600	650	700	750	800
20	5	7	9	10	12	14	16	17	19	21	22	24	26	28	29
25	5	7	8	9	11	12	13	15	16	18	19	20	22	23	24
30	5	7	8	9	10	11	13	14	15	16	17	18	19	20	22
35	5	7	8	9	10	11	12	13	14	15	16	17	18	19	20
40	5	7	8	9	10	11	12	13	14	14	15	16	17	18	19
45	5	7	8	9	10	11	12	13	14	14	15	16	17	17	18

Once the overlap time has been determined, and a cycle length has been pre-determined, the phase splits using the Texas Four-Phase Strategy can be calculated using the following equation:

$$\phi_i = \frac{y_i}{y_2 + y_4 + y_6 + y_8}(C + \boldsymbol{\Phi} - 4l) + l \qquad i = 2, 4, 6, 8$$

Where, ϕ_i equals the phase time in seconds for phase i, y_i is the CLV to saturation flow ratio for phase i, C is the cycle length in seconds, and l is the lost time per phase, in seconds.

Then the odd numbered phases can be calculated as follows:

$$\phi_1 = \phi_6 + \phi_8 - \mathbf{\Phi}$$

$$\phi_5 = \phi_2 + \phi_4 - \mathbf{\Phi}$$

Once the phase durations have been calculated, the lost time per phase must be subtracted to get the green time per phase. This approach will give the recommended phase intervals for the interchange.

Chapter 3: Methodology

3.1 Intersection Ramp Metering System Design

The intent of this research is to design a system that can be implemented with little to no additional infrastructure requirements. This system would facilitate a modified ALINEA ramp metering process, using only existing traffic control signals. Figure 3.1 below shows the interchange layout as well as the required detector locations and the existing phasing sequence for the metered intersection, which is the one shown on the right side of the image.

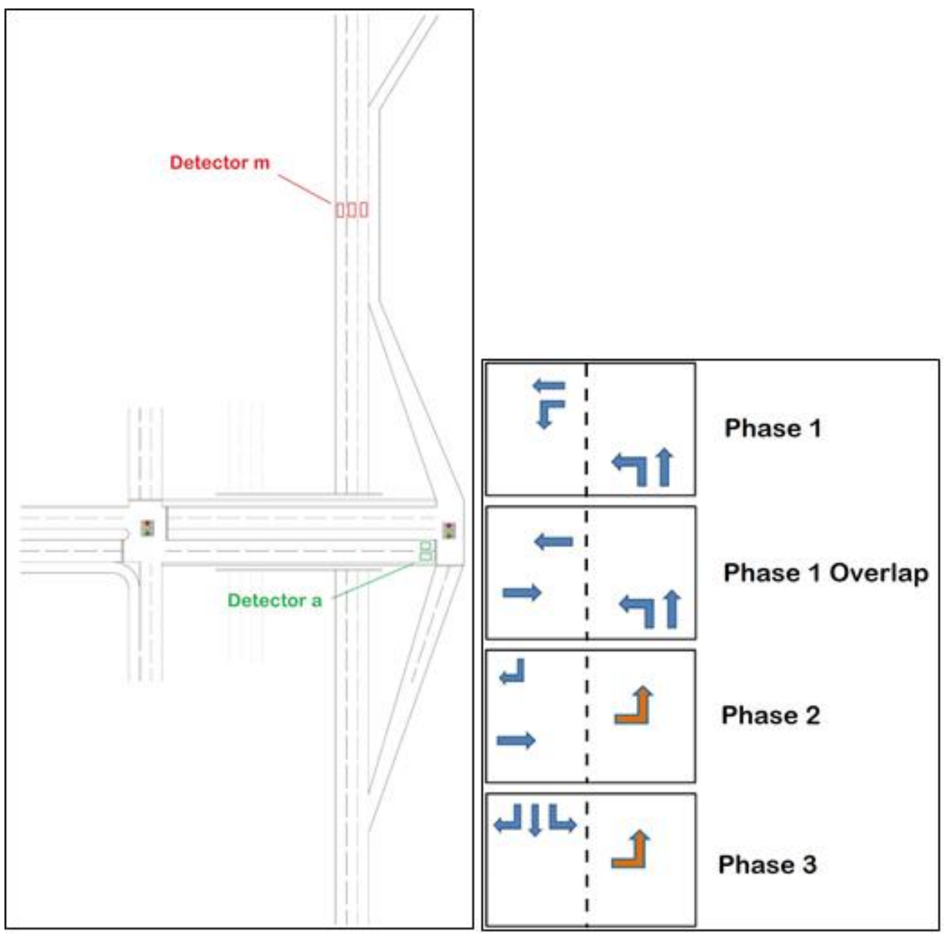

Figure 3.1 - Intersection System Design and Phase Plan

The interchange analyzed in this research is classified as a tight diamond interchange, because the intersection spacing is about 340 ft. Due to this classification, it can be seen above that the intersection is controlled with a modified Texas Four-Phase Control Strategy. This phasing plan is modified because the arterial does not continue beyond the right side of the diamond in the above diagram. Because of this, there is no westbound approach, and no need for what was Phase 2 in the conventional Texas Four-Phase Strategy.

In this particular application, although the ALINEA algorithm does not require a detector at the entrance ramp (or in this case Detector a at the intersection approach), the model code used requires detection of a vehicle for the metering system to be activated. That is the purpose of Detector a above. The mainline detector, Detector m, is required for any ALINEA application, and should be located at some point downstream of the merge point.

The IRM algorithm implements metered control during a particular display in the phasing sequence. The metered movements are depicted by the orange arrows in Figure 3.1 above. All normal movements are depicted by blue arrows. In this particular situation, because measured volumes for the northbound through movement were considerably low, this movement is not metered. However, for similar intersections, this movement could also be metered if the volumes warrant it. The proposed phasing sequence will follow the same pattern as the existing sequence, which, shown in the diagram above, is the Texas Four-Phase Strategy.

3.2 The Intersection ALINEA Algorithm

The ALINEA algorithm can be modified to account for multiple signal heads using the same downstream detector data, and the inclusion of a maximum and minimum rate. The system is implemented using the following control logic:

$$\boldsymbol{r_1(t)} = \begin{cases} \boldsymbol{r_{off}}, & \boldsymbol{if\ OCC_m < OCC_{m,threshold}} \\ \boldsymbol{r_1(t-1) + K_r\left[OCC_{m,des} - OCC_m(t)\right]}, & \boldsymbol{if\ OCC_m \geq OCC_{m,threshold}} \end{cases}$$

Where $r_1\ (t)$ is the current ramp metering rate, OCC_m is the measured occupancy of the downstream detector, $r_1\ (t\text{-}1)$ is the metering rate during the previous interval, K_r is the regulatory parameter which is set at 70 vehicles per hour, $OCC_{m,des}$ is the desired mainline occupancy, and $OCC_m\ (t)$ is the current measured mainline occupancy. The metering rate is further bound by the predetermined maximum and minimum metering rates.

The signal control logic converts the ramp metering rate into a signal display. It does this by calculating the red interval and cycle length for each interval. Since the green interval is fixed in our approach, the cycle length will always be the sum of the fixed green interval and the calculated red interval. To determine the red time in the ramp metering cycle, T_r for the ramp metered approach, the control logic utilizes the following equation:

$$\mathbf{T_r = \frac{3600}{r_1(t)C/g} * n - T_G = \frac{3600g}{r_1(t)C} * n - T_G}$$

Where n is the number of lanes, C is the cycle length, g is the intersection green time for the metered approach, and T_G is the predetermined green time for the IRM signal. This formula dictates the rate at which vehicles are dispatched to the ramp. At the same time, it could also be set to ensure that queues for the onramp will not back up and cause an adverse effect on traffic not wishing to access the freeway. If the occupancy of a detector placed at the point where queues should not stretch beyond is greater than the determined threshold, it can be assumed that the queues are backing up into the adjacent intersection. If the queue reaches this length, then the phase 1 metering rate could be increased to its maximum value, and the traffic is dispatched to the ramp at that rate until the detector occupancy is measured to be lower than the maximum value. While this process, known as "ramp flushing", will lessen the effectiveness of our ramp metering procedure, it is occasionally a necessary element of the system design. It is not feasible to imagine an implementation of this system where onramp queues that are blocking other movements will be acceptable. Because of this, the availability of storage space at the intersection is an important parameter in determining the success of the IRM system. While this particular application had plenty of storage space for onramp vehicles, a queue flushing scenario is included in the simulation.

Chapter 4 Model Implementation, Calibration, and Validation

A base-case model was created first, and it was calibrated to best resemble the conditions experienced during the data collection period. The measured volume inputs were put in, incremented into 5 minute intervals. The measured routing decisions were put in as well, also incremented into 5 minute intervals. The routing decisions were entered as a proportion of all approaching vehicles selection a certain route. These routing proportions are considered static and do not change throughout any scenario in this research. The car following behavior of vehicles in the model as well as desired speed decisions were tweaked until the model most accurately displayed what was observed in the field. The model was validated when the bottlenecking caused by the platooning vehicles that enter the weaving section from the Far West onramp could be accurately replicated. For the metered models, the ramp metering control was overlapped on top of the existing pre-timed signal control. The metering logic was introduced using the VISSIM Vehicle Actuated Programing (VAP). The VAP interprets the coded control logic and translates them into the simulated signal control.

4.1 Performance Measures of Freeway Facilities

The success of the IRM system will be based upon the changes it causes in certain freeway performance measures. These measures are the same as those that are commonly used when evaluating freeway operations.

4.1.1 Flow, Speed, Throughput, and Travel Times

Network-wide values of delays, throughputs, speeds, and travel times will be averaged and reported. The analyst can use the results of these values to estimate the initial success of the system. However, before a system can be deemed a success, it must be verified that certain other conditions are met as well.

In addition to studying the effects on the whole network, individual vehicle paths should be examined as well. Average travel time (in seconds) and total throughput (in number of vehicles) for 4 separate paths in the network will be collected. The paths are:

- Freeway – These are vehicles that exclusively travel along the freeway segment throughout their period in the network
- Southbound Frontage Road – these are the vehicles that enter from the north of the network, and continue through the interchange to the southbound frontage road
- Far West Offramp – these are the vehicles that exit at Far West Blvd, and continue westbound on that road.
- Far West Onramp – these are the vehicles that enter the network heading eastbound on Far West Blvd, and continue onto the northbound freeway. These vehicles will be metered during the IRM implementation

Also, values for average flow (in vehicles per hour per lane) and average speed (in miles per hour) will be measured for those vehicles that are traveling through the weaving segment of the freeway in the network. These values are aggregated into 5 minute intervals, and the average across that interval is the value reported. Typically, an analyst would want to see values of these two measures as high as possible, but of equal or greater importance is the amount of variation within these values. A good system will not only produce high flows and speeds, but also relatively constant flows and speeds throughout the simulation period. This lack of variation leads to an increase in travel time reliability.

4.1.2 Travel Time Reliability

Average speed and travel times are a good measure of evaluating freeway performance, but they do not tell the whole story. Because of the variations in individual travel times over a segment, each driver does not experience the average travel time. In these instances, driver perception is often as important as or more important than the average values. One of the simplest measures of travel time reliability is the 95th Percentile travel time. This measure will give an indication of how bad travel delay will be on the heaviest travel days. This measure is also easily understood by travelers, and can be used on traveler information systems.

Another good way to quantify the effects of variability is to use travel time reliability indices [21]. These indices give a representation of the variation in travel times. Two of the most commonly used travel time reliability indices are planning time index and the buffer index. The buffer index (B.I.) is a measure of the amount of extra time, or buffer time, that most travelers will allow themselves to ensure on-time arrivals. The extra time accounts for any unanticipated delay. The B.I. is reported as a percentage of the average travel time that should be added to the trip to account for unexpected delay. It is calculated as follows:

$$B.I. = \frac{95^{th} Percentile\ Travel\ Time - Avergage\ Travel\ Time}{Average\ Travel\ Time}$$

The planning time index (P.T.I.) is a measure of the total travel time that should be planned when a buffer time is added in. The key distinction between the P.T.I. and the B.I. is that the P.T.I. quantifies both expected and unexpected delay, while the B.I. only quantifies unexpected delay. The P.T.I is a comparison of the free-flow travel time to the near worse-case travel time. It is reported as amount of the free-flow time that should be planned for a high-priority trip, like an airline departure or a medical appointment [6]. For example, if a trip is 30 minutes during free-flow conditions, a P.T.I of 1.5 means that a traveler would need to allot 45 minutes for the trip (1.5 x 30 minutes). The P.T.I is calculated as:

$$P.T.I. = \frac{95^{th} Percentile\ Travel\ Time}{Free\ Flow\ Travel\ Time}$$

4.2 Model Implementation

For the IRM implementation, the VAP used was developed by researchers at the Texas Transportation Institute for the purpose of simulating several different types of metering algorithms in VISSIM [3]. This particular code has several parameters that can be adjusted by the modeler in order to optimize the performance of the system. In the IRM implementations, the VAP parameters are optimized so that total network delay would be minimized and system throughput would be maximized in each scenario. In this control logic, the ALINEA metering rate equation could be bound by both a minimum and maximum allowable rate. Furthermore, a mainline occupancy threshold could be set, which turns on the metering system only when that threshold is reached.

Chapter 5 Experimental Design

5.1 Data Collection

In this study, a location that frequently experiences bottlenecks due to an onramp and subsequent weaving segment is analyzed. A VISSIM model was calibrated to replicate the actual conditions on Loop 1 northbound at the interchange of Far West Boulevard in Austin, Texas, for a typical afternoon peak period. The P.M. peak is typically the busiest time of day for this particular section of roadway. The VISSIM network was created to represent approximately 1 mile of the NB mainlines, including a weaving segment between the Far West Blvd. onramp and the Anderson Lane offramp. The two surface street intersections at Far West Blvd. and both the NB and SB frontage roads were also included. This network is shown in Figure 5.1 below.

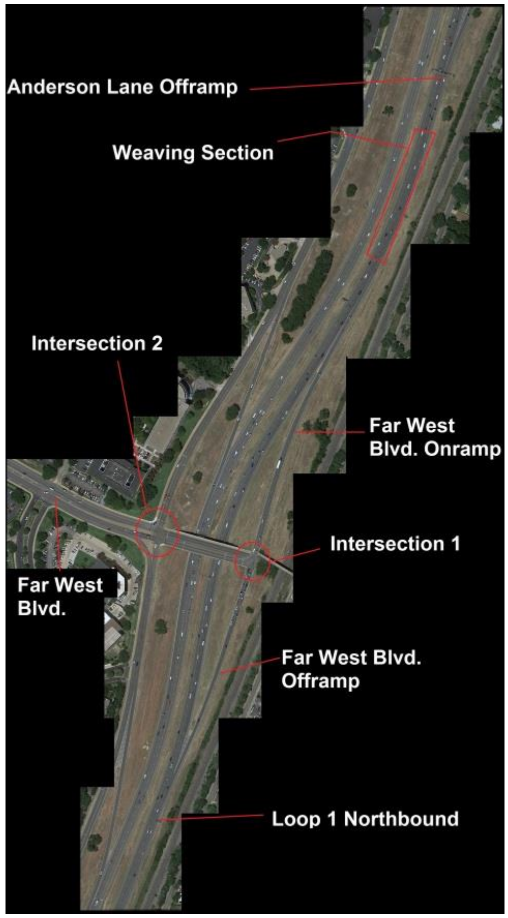

Figure 5.1 Analysis Area

Traffic counts taken from a one and one half hour video of the July 10, 2013 p.m. peak-hour traffic at the site was used to calibrate the base model. In addition to traffic volumes, signal timings and routing decisions were also collected from the video. The collected data was summarized into 16 five-minute intervals for the simulation. The two major intersections each were filmed independently. Additionally, another camera was fixed on the freeway mainlines downstream of the intersection, just before the merge point from the Far West Onramp. Mainline counts and the Anderson Lane exit counts were taken from this video.

The cameras used in the data collection were placed on tripods on the bridge between the two intersections. Two cameras were faced towards each intersection, and the other was pointed towards the downstream end of the freeway, as shown in Figure 5.2 below.

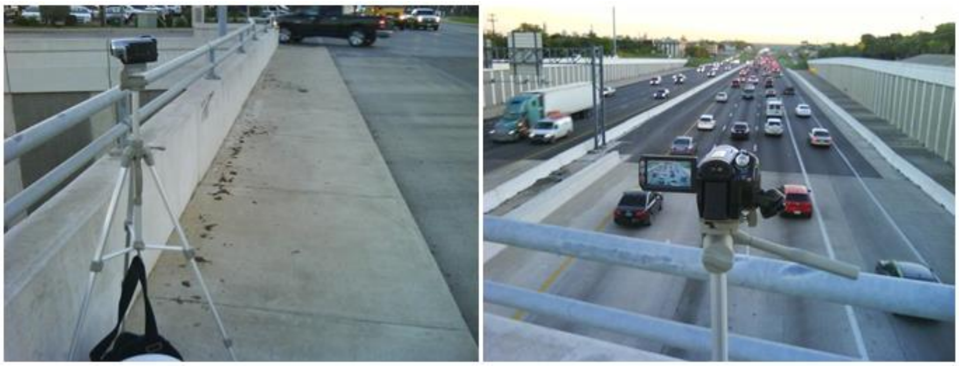

Figure 5.2 – Data Collection Equipment

Figures 5.3, 5.4, and 5.5 show the locations of the cameras on the bridge, as well as the locations of the camera operators. The cameras depicted in green were aimed at the intersections, and the cameras depicted in red were aimed at the freeway.

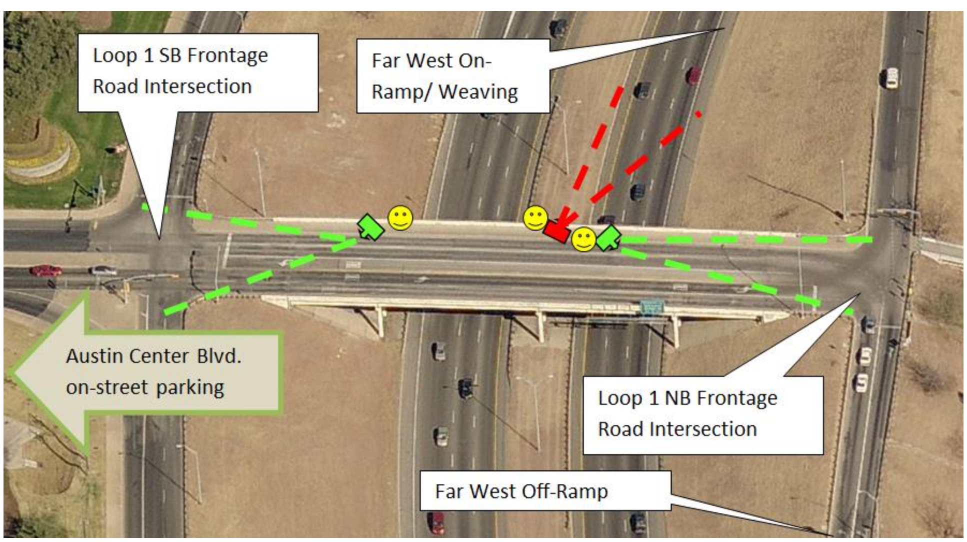

Figure 5.3 – Aerial of Data Collection Location

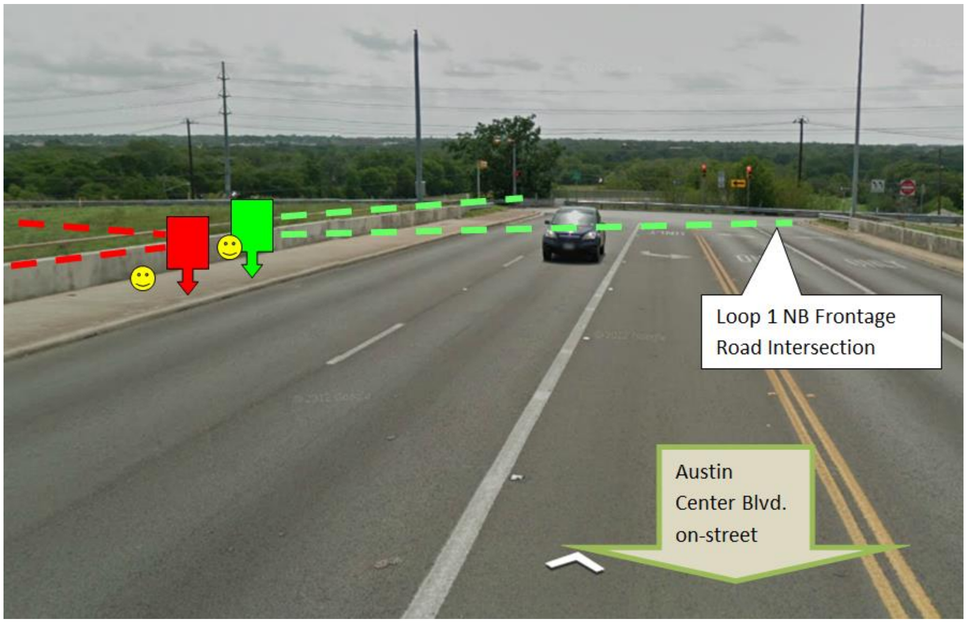

Figure 5.4 – View of Intersection 1

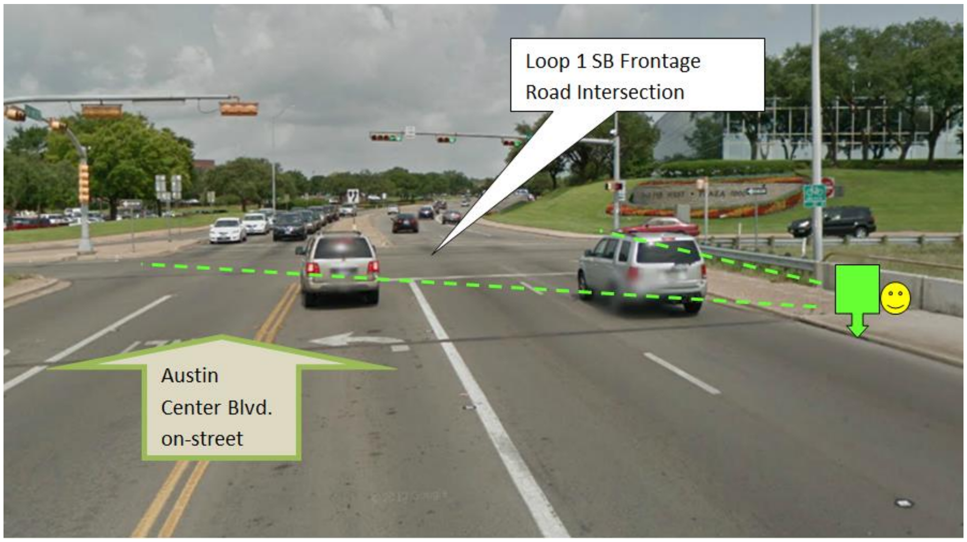

Figure 5.5 – View of Intersection 2

The camera aimed at the freeway was positioned such that volume counts could be made before the merge point from the Far West onramp, and so that vehicles exiting to Anderson Lane could be counted as well. A screenshot of the video produced with this camera is shown in Figure 5.6 below.

Figure 5.6 - View From Bridge

By watching a playback of the videos, a counting procedure is done using software that registers each input and the time on the video that it occurred. For instance, while watching the playback of the video, when a northbound left is observed, the analyst would push 1 on a keyboard, when a northbound through would occur, the analyst would push 2 on the keyboard. The traffic counting software exports a spreadsheet that registers the number pushed for each count and the time on the video that it occurred. From this, volume counts can be compiled. The data compiled was summarized into 5 minute intervals. Figure 5.7 below shows the peak interchange hourly counts that were collected, as well as the peak 15 minute counts within that hour.

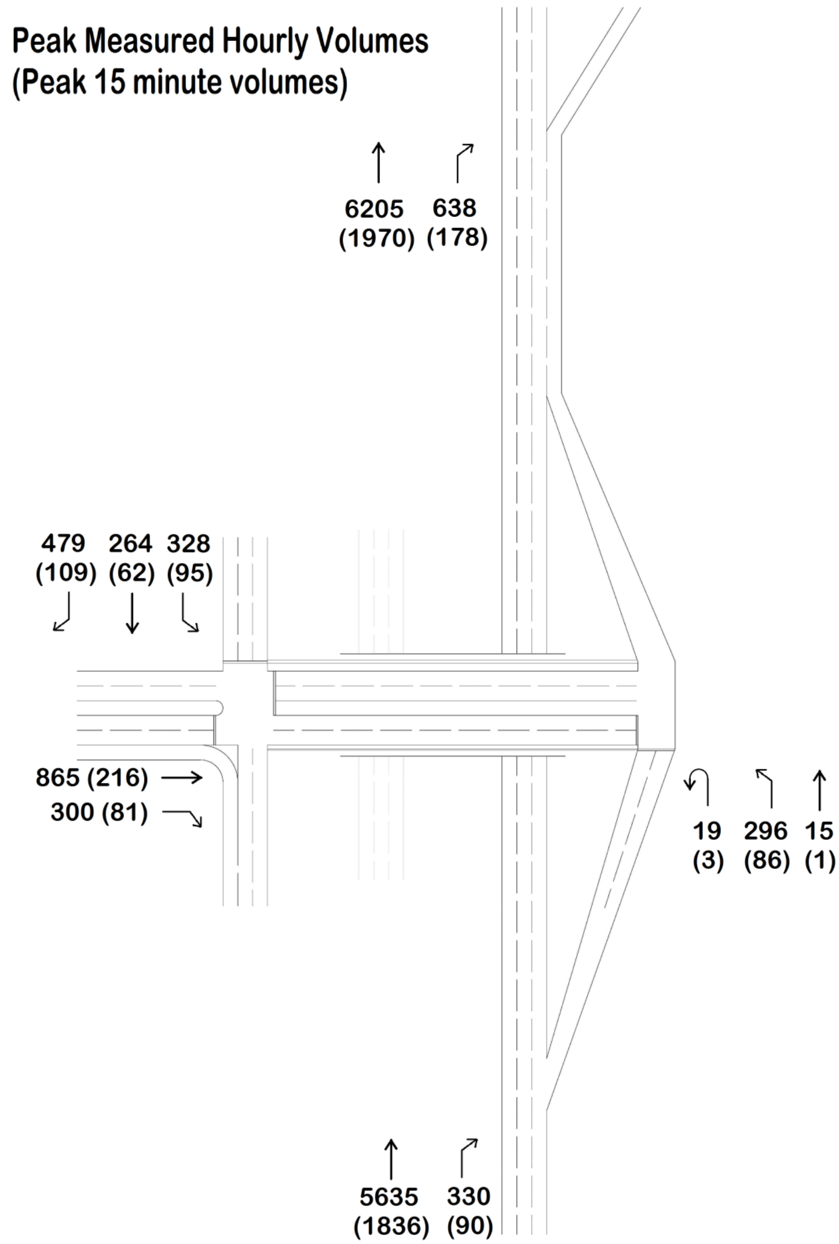

Figure 5.7 - Measured Volumes

From these peak hourly and peak 15 minute volumes, the peak hour factor (PHF) for the intersection traffic can be computed. The PHF is a representation of the variation in volumes over the course of the peak hour. This metric has a range of 0.25 to 1, where lower values represent more variation in volumes and higher values represent less variation in volumes. The PHF for this interchange is computed as follows:

$$PHF = \frac{\sum[Hourly\ flow\ through\ intersection]}{4 * (\sum[Peak\ 15\ minute\ flow\ through\ intersection])} = \frac{2566}{4 * (653)} = .98$$

Because the PHF is so close to 1, it can be assumed that volumes do not show very much variation throughout the peak hour.

Also from the video, average flows over various time intervals could be determined. In this research, 5-minute intervals were used for the simulation evaluation. Observed flows from the videos for critical areas are shown in Figure 5.8 below.

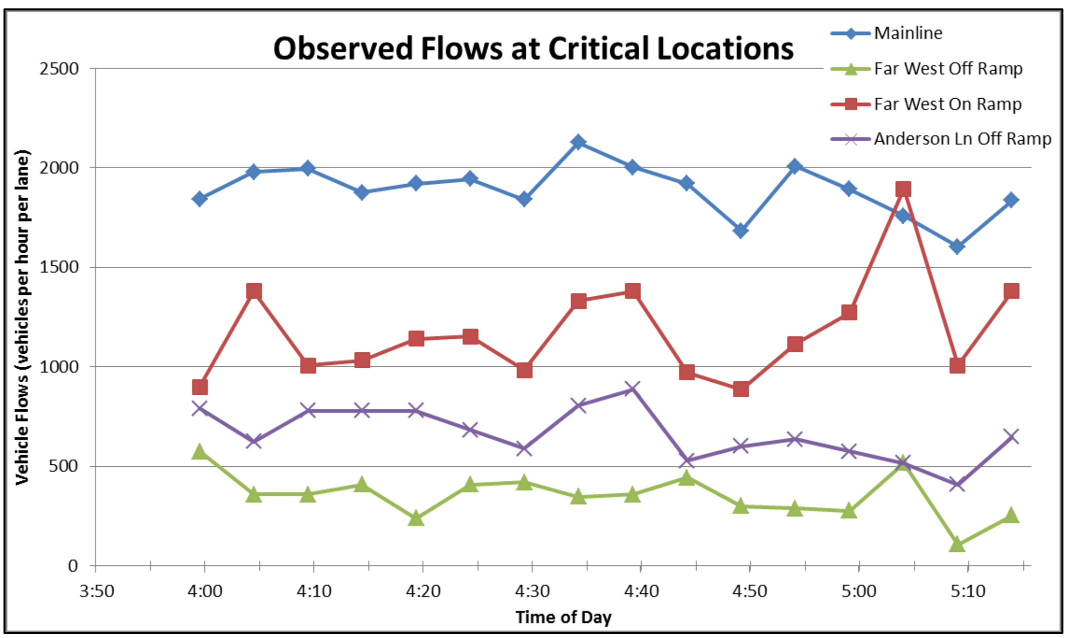

Figure 5.8 - Observed Traffic Flows

From the same data, the phasing sequence, cycle length, and phase intervals can be calculated. Since this intersection is pre-timed, most intervals remained constant throughout the duration of the data collection effort. The only changes to these intervals were caused by a few pedestrian actuations. However, it was determined that since pedestrian volumes were so low, it would not be necessary to consider them in this experiment. The measured signal timings and phasing is shown in Figure 5.9 below.

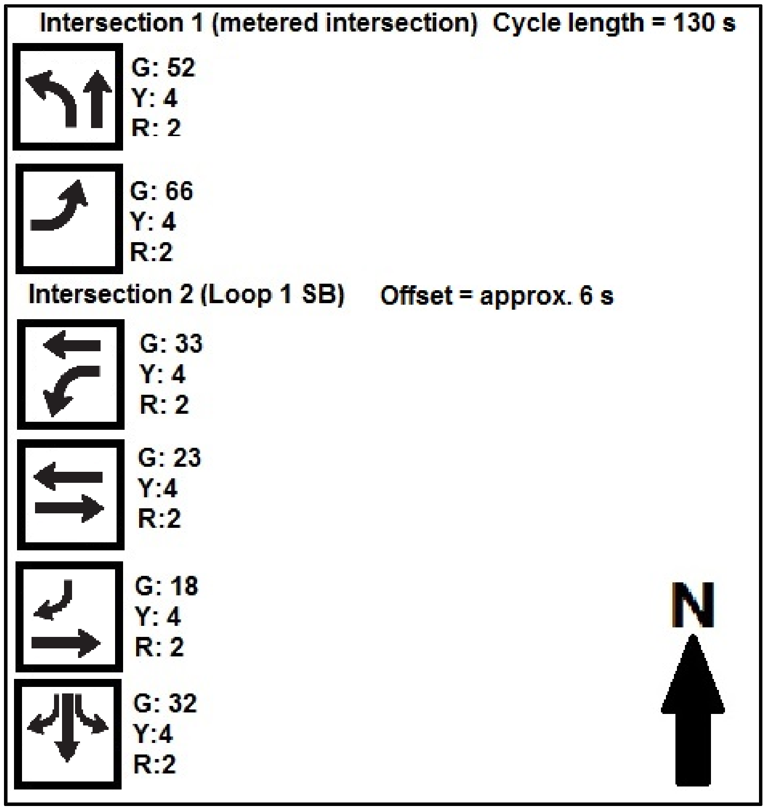

Figure 5.9 - Observed Interchange Timings

The above represents the modified Texas Four-Phase strategy. Like many other intersections in Austin, this one features a relatively long cycle length, at 130 seconds.

5.2 Evaluation Scenarios

In evaluating the proposed system in the research, attention will be separately placed on the freeway conditions and the surface street intersection conditions. Each of the following scenarios will be evaluated by their effects on both the freeway and the surface streets. To fully understand the potential benefits of the IRM system, several different scenarios will be evaluated in micro-simulation. Each evaluation alternative is listed below:

- Base Case Alternative – this is the do-nothing case
- Scenario X – in this alternative, the onramp will be closed so that the best possible benefit to the freeway can be determined.
- Scenario 1 – implementing IRM without additional timing or phasing adjustments
- Scenario 2 – implementing IRM with several timing adjustments
- Scenario 3 – Base Case with the same timing/phasing plan as Alternative 2
- Additionally, Queue Clearance scenarios will be analyzed with the final settings for Scenario 1 and 3

Each of these scenarios will be modeled in the calibrated VISSIM network, and certain output data will be compared. In each scenario, the IRM signal will be controlled by the Intersection ALINEA algorithm, which is laid down in the VAP code.

Chapter 6: Result Analysis

6.1 Model Calibration Results

An important aspect of model validation in this particular experiment is ensuring that the mainline flow breakdowns caused by the bottlenecking behavior at the weaving section that were observed in the field could be replicated in the model. Driver behavior parameters were adjusted until the model best mimicked the observed conditions. Figure 6.1 below shows the comparison of the observed flows to the base case modeled flows, in vehicles per hour per lane (vphpl).

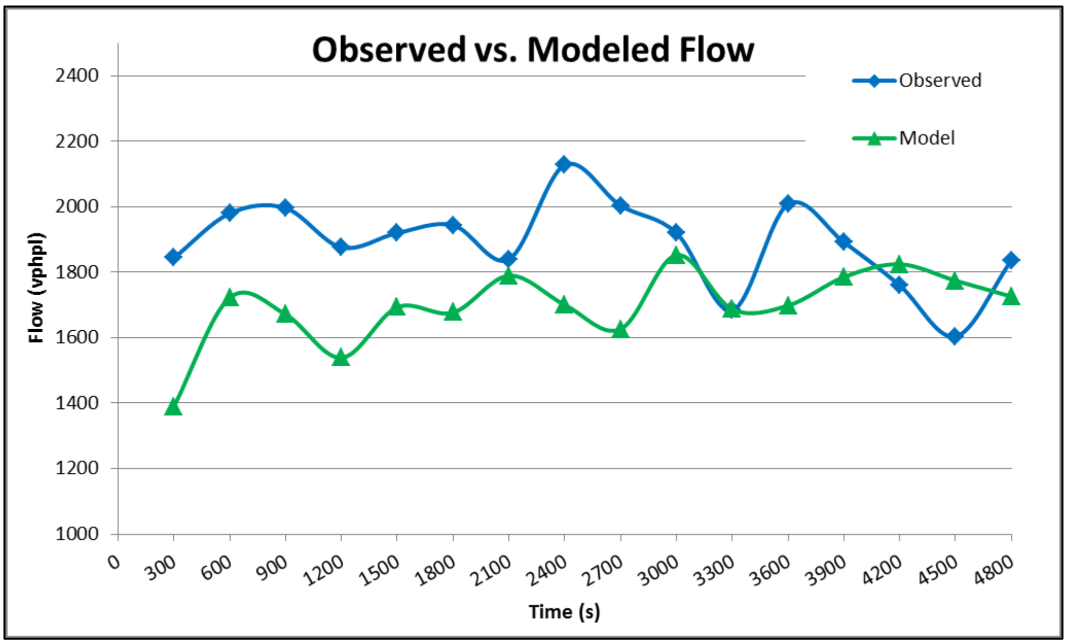

Figure 6.1 - Observed Freeway Flows vs. Modeled Freeway Flows

Although not technically a continuous function, the graph has smoothed lines to help visualize the trends in vehicle flow. The modeled location is about 100 ft. downstream of the freeway merge point. This point was selected because this is the same point on the freeway where the ALINEA algorithm seeks to control the occupancy. This location is so close to the merge point that differences in the backward shockwave speed between the field data and modeled results could be minimized.

Crucially, our model is able to replicate the flow breakdowns with relatively high accuracy. Each flow curve shows three major breakdowns. The first happens between about 600 seconds and 1200 seconds (15 to 20 minutes) in each, and with similar (100-200 vphpl) quantities. The second major breakdown occurs at 2400 s (40 minutes) in the field data, and about 5 minutes sooner in the model. The final major breakdown occurs at 3600s (60 minutes) in the field, and about 10 minutes earlier in the model. Each flow decline lasts about 10 minutes in the model, and 15 minutes in the field results. In general, flows recover in the model a little sooner than the field data shows. The speed at which the flows recover is also quite similar in each scenario, with flows increasing up to 300 vphpl in a 5 minute period.

6.2 IRM System Parameter Optimization Results

In preliminary research for this project, it was found that eliminating the maximum rate (by setting it at an unattainable number, 26,000 vph) and setting the occupancy threshold only 5% below the optimum occupancy was the best approach. Neither of these factors were very sensitive, and the model would produce similar results for varying values for each. The minimum metering rate was very sensitive towards the overall system performance. At very high occupancy levels, the algorithm will nearly stop the metered traffic without setting a reasonable minimum rate. This would cause extreme backups on the surface street intersections and greatly degrade system performance. In the preliminary research, it was found that setting this rate at 900 vph would both keep ramp queues in check, and still not hamper the freeway performance.

Historically, the optimal occupancy is the most important adjustable parameter in the ALINEA equation. Previous research found that the optimal range for the values is from 18% to 31%. In this particular instance, an optimal occupancy level of 30% was selected as a starting point for the evaluations. The next important adjustable parameter was the green interval. This essentially determines the number of vehicles that can proceed through each cycle. It was determined in this research that a green interval of 2.0 seconds would be best. This allows 2-3 vehicles per green. When any more vehicles than that were dispatched, they would ultimately start forming a queue to enter the mainlines at the downstream end of the ramp. This would effectively eliminate the benefit of the metering system.

The final adjustable constant was the ALINEA constant itself. Previous research has found that using a value of 70 vph was ideal, and that this value was not immensely sensitive in the model. This value was adjusted several times, but it was never determined that a value other than 70 vph would ever improve the model. The evaluation timestep can also be adjusted. In an ALINEA system, it is important that dispatched vehicles are able to reach the downstream detector within the timestep. It was determined that 60 seconds is the appropriate timestep for this model. Table 6.1 below lists all adjustable parameters and their initial evaluation values.

Table 6.1 - Initial VAP Parameter Settings

Parameter	Initial Setting
K_r	70 vph
Desired Mainline Occupancy	0.30
Mainline Occupancy Threshold	0.25
Maximum Rate	26,000 vph
Minimum Rate	900 vph
Green Interval	2.0 sec

In each evaluation scenario that involves the IRM system, the above values are used as a starting point for the evaluation. Since there is little to no previous research on how to optimize the ALINEA control parameters to meet system objectives, a trial and error approach is used. In each simulation run, a single parameter will be slightly adjusted, and the results evaluated. This process will be continued until it is believed that the best possible results are achieved. A flow chart is kept of the VAP adjustments and the corresponding network and freeway performance results for each evaluation scenario involving IRM. The flow charts can be found in Appendix A. The final optimal settings for each scenario are included in the evaluation scenario section below.

The locations of the mainline detectors were also moved around throughout several simulations and the results were compared. It was found that the optimal location of these detectors is the point where the weaving movements can feasibly begin. This correlated with the point just after where the solid white line becomes a broken white line on the parallel acceleration lane.

6.3 Scenario Implementation and Optimization Results

Each evaluation scenario was modeled based off of the original calibrated VISSIM network. Each scenario was developed and evaluated independently of the others.

6.3.1 Base Case Alternative

In this case, the initial calibrated model is simulated without any adjustments to signal timing, volume inputs, routing decisions, or driver behavior parameters. This is the scenario that best replicates what is experienced in the field. The results from this analysis will serve as the benchmark which all other scenarios will be measured against. Each IRM model is calibrated so that it should achieve the best possible results over the Base Case Scenario.

6.3.2 Scenario X

In this scenario, the entrance ramp from Far West Blvd is deleted from the network before the simulation is run. In this case, vehicles from the intersection are not able to reach the freeway. This is done to evaluate the "best possible" scenario for the freeway. Without the interference from merging traffic, the bottlenecks should be reduced and the exiting movements to Anderson Ln. should be achieved easier. Network results from this scenario will not be collected. Rather, only freeway performance results will be collected and included in the analysis. These results will show us the absolute ceiling for freeway improvements through IRM.

6.3.3 Scenario 1

In this case, the IRM system will be added to the existing network and no additional signal timing alterations will be made. The final optimal parameter settings are shown in Table 6.2 below. The flow chart showing parameter adjustments and results is in the Appendix.

Table 6.2 - Optimal VAP Parameter Settings – Scenario 1

Parameter	Optimal Setting
K_r	70 vph
Desired Mainline Occupancy	0.30
Mainline Occupancy Threshold	0.25
Maximum Rate	26,000 vph
Minimum Rate	850 vph
Green Interval	1.5 sec

6.3.4 Scenario 2

In this case, IRM is included. Additionally, the interchange will be retimed to see if any additional benefits can be gained. In determining cycle length it becomes important to specify the goals the evaluator wants to achieve. In general, the approach that is taken is a delay minimizing approach. When implementing the IRM approach, it must be determined which movements through the intersections will be impacted, and by how much. Naturally, the ramp bound traffic will incur increased delay with such a system, but it must be determined how much extra delay is too much. Also, how much the non-ramp bound vehicles are punished by the IRM system must be determined. In adjusting the timing for this particular approach, any negative effects to the non-ramp bound vehicles due to the IRM are sought to be eliminated. While the additional incurred delay to the ramp bound traffic is attempted to be checked by modifications to the ramp metering rate. The optimal cycle length used in this scenario is based on Webster's delay minimizing approach. The calculations are shown as follows:

$$C_0 = \frac{1.5L + 5}{1 - (\frac{CLV_1}{S} + \frac{CLV_2}{S} + \frac{CLV_\#}{S})} = \frac{1.5(18) + 5}{1 - (\frac{252}{1800} + \frac{432}{1800} + \frac{525}{1800})} \approx 100\ seconds$$

Once the cycle length was calculated, the desired phase splits should be determined. Using the phase split calculations for the Texas Four-Phase diamond interchange timing strategy, the phase lengths were calculated as shown below:

ϕ_2 = 39 s
ϕ_4 = 47 s
ϕ_6 = 5 s
ϕ_8 = 25 s

Since there is no phase 6, the 5 seconds allotted to that phase were redistributed to the other phases based on the relative amount of green time they were already assigned. The corrected phase lengths are shown below.

ϕ_2 = 41 s
ϕ_4 = 49 s
ϕ_6 = 0 s
ϕ_8 = 26 s

When implemented into the signal control, these timings will look as they are represented in Figure 6.2 below.

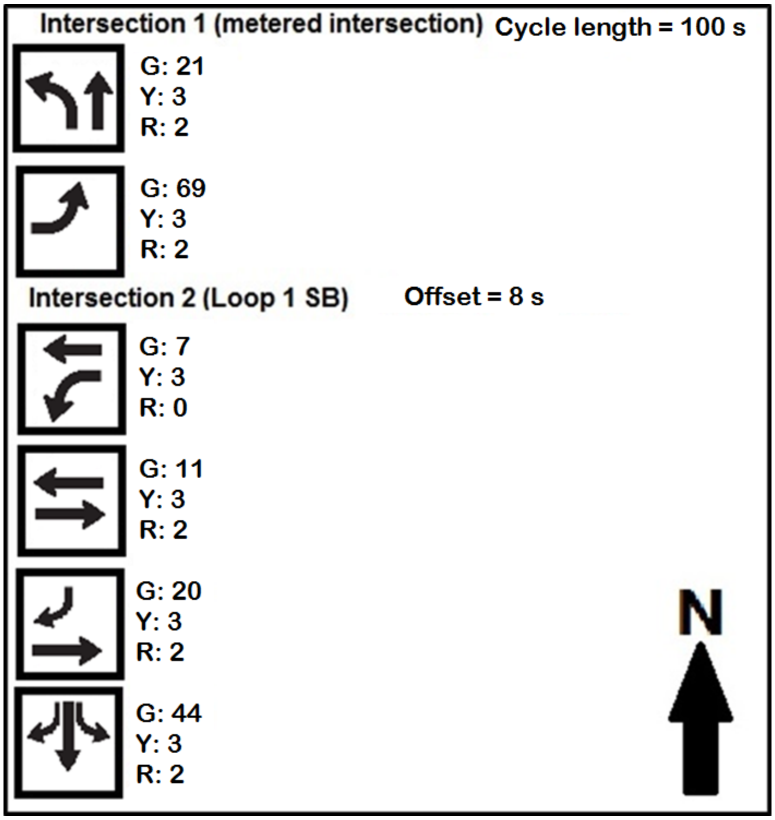

Figure 6.2 - Interchange Timings for Scenario 2

The optimal VAP settings are shown in Table 6.3 below. Ultimately, most of the optimal parameter settings found in Scenario 1 were also optimal in Scenario 2.

Table 6.3 - Optimal VAP Parameter Settings – Scenario 2

Parameter	Optimal Setting
K_r	70 vph
Desired Mainline Occupancy	0.30
Mainline Occupancy Threshold	0.25
Maximum Rate	26,000 vph
Minimum Rate	900 vph
Green Interval	2.0 sec

6.3.5 Scenario 3

In this scenario, the intersection timing adjustments made in Scenario 2 will be kept, but the IRM system will be shut off. This is to determine if the network-wide changes measured in Scenario 2 should be attributed to the changes in signal timing, or to the IRM system.

6.3.6 Queue Clearance Scenarios

The final model settings for Scenarios 1 and 3 will also be evaluated with queue flushing enabled. These results will be compared to the Base Case and Scenario 1 and 3. To implement the queue flushing in the model, an additional detector must be installed. When the measured occupancy of this detector reaches 0.90, it will trigger the meter to turn off, until the occupancy is back within the allowable limits. The detector location in this scenario was placed so that queues backing up on Far West Blvd. would not block the upstream intersections and driveways. This location corresponded with a point about 400 ft. from the Southbound Frontage Road intersection, as shown in Figure 6.3.

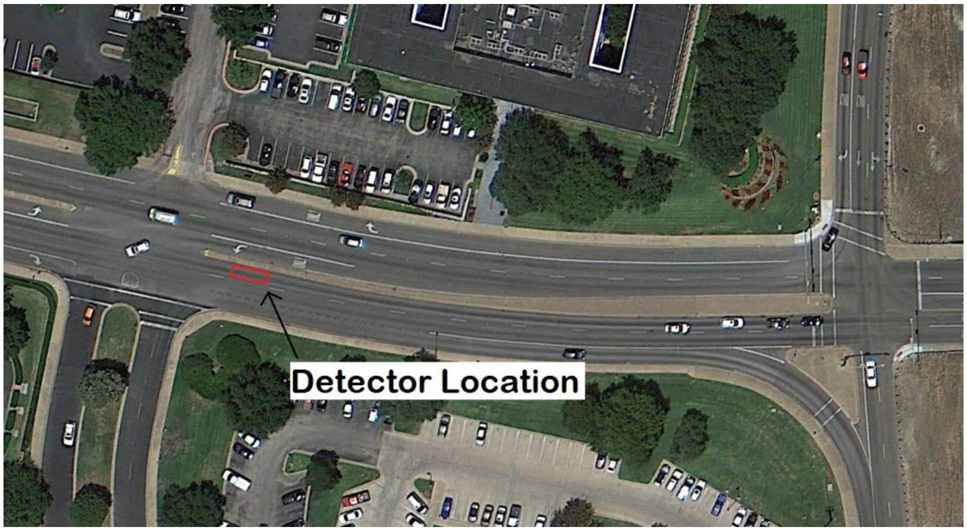

Figure 6.3 - Queue Clearance Detector Placement

This placement still allows for a significant amount of queuing, with 730 ft. of storage space between this position and the meter. Assuming average spacing for vehicles in a queue to be 20 ft., this would allow about 73 vehicles to queue.

6.4 Modeled Freeway Flow and Speed Characteristics

One of the best ways of evaluating freeway performance is by measuring the volume flow rate and mainline speeds. A good way to evaluate the effectiveness of any change to the system is to compare it with the results from the Base Case (assumed to be the worst feasible scenario) and the Ramp Closed (assumed to be the best possible scenario). Figure 6.4 below shows the comparison between the Base Case and Ramp Closed freeway flows and speeds.

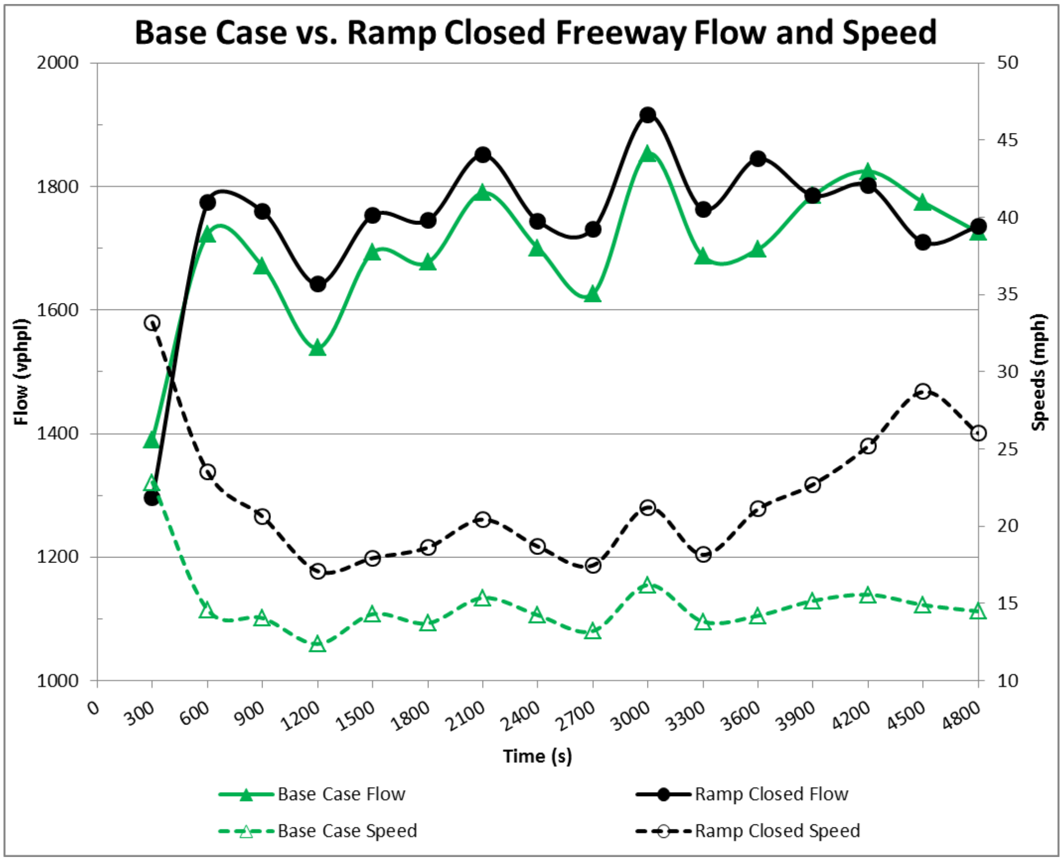

Figure 6.4 - Measured Avergage Freeway Flows and Speeds for Base Case and Scenario X

The chart above practically shows the best and worst case scenarios for freeway conditions. It can be seen that there is not much variation in flows between the two scenarios. This is likely because of the interference in flow caused by exiting vehicles at Anderson Lane. For speeds, there seems to be a larger variation. The ramp closed scenario allows for speeds on average of about 5 mph higher than the base case. Below, Figures 6.5, 6.6, and 6.7 show the measured freeway flows and speeds for each scenario. Scenarios 1, 2, and 3, are compared to the Base Case and the Ramp Closed Scenarios.

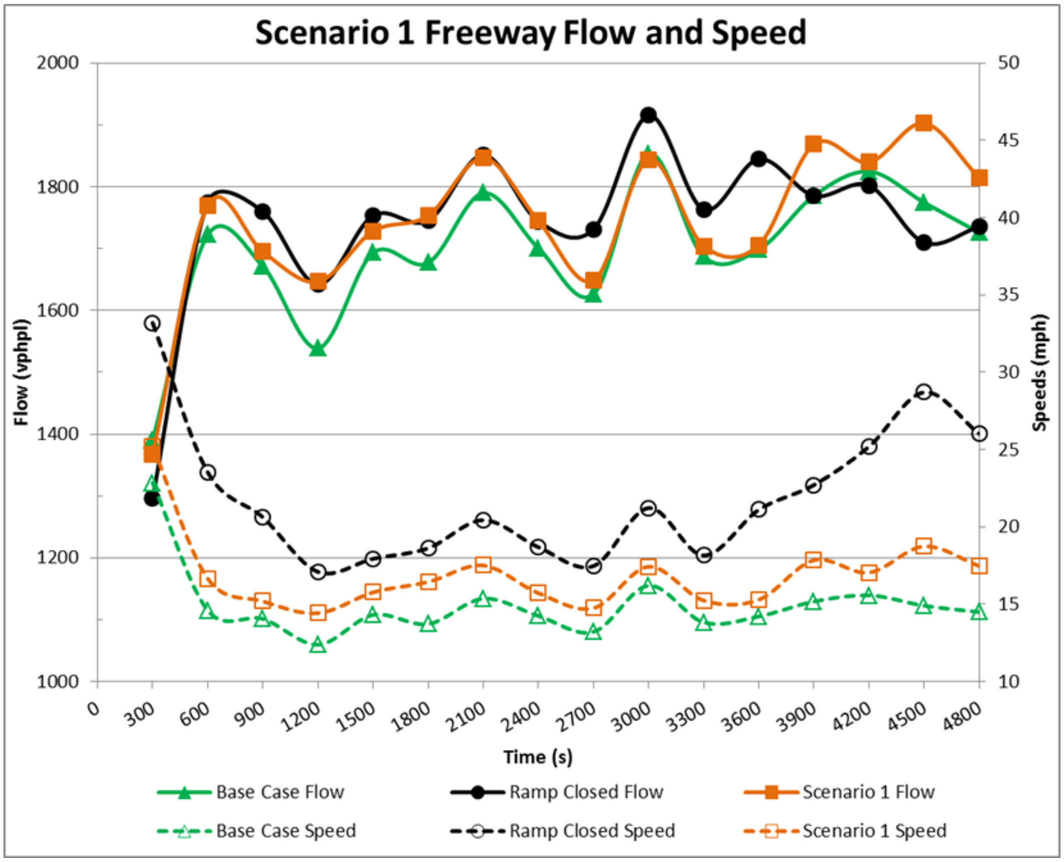

Figure 6.5 - Measured Average Freeway Flows and Speeds for Scenario 1

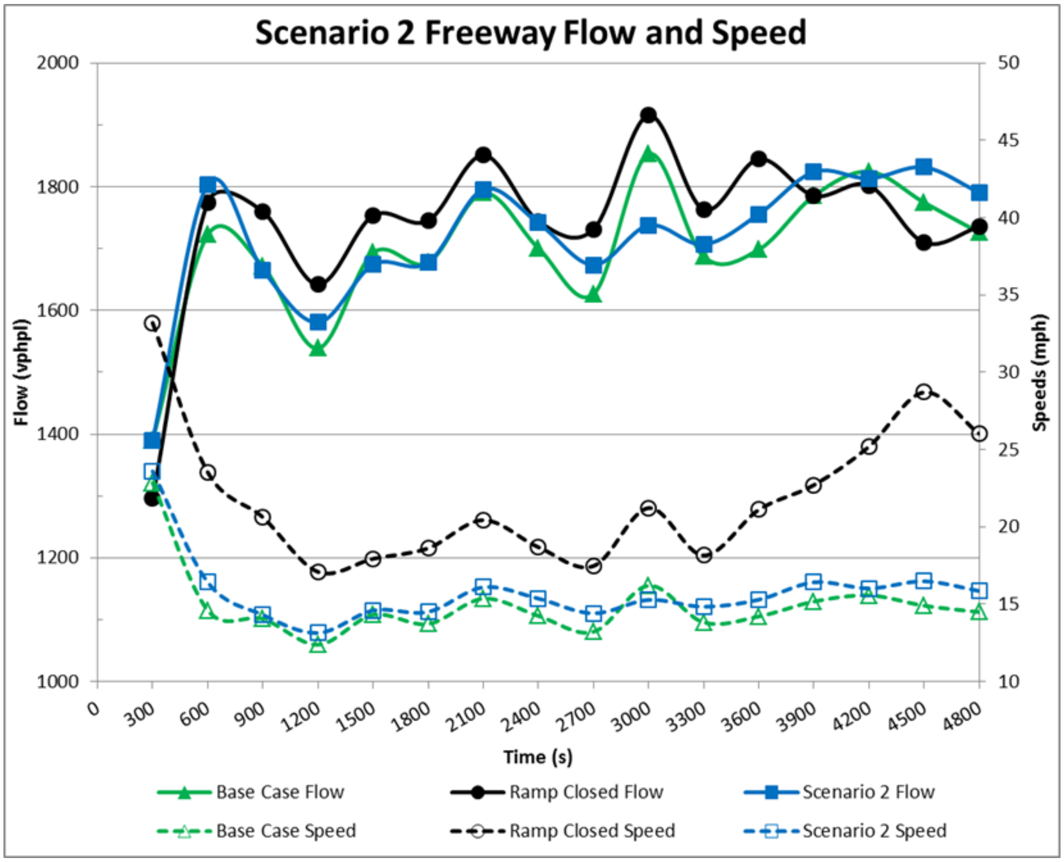

Figure 6.6 - Measured Average Freeway Flows and Speeds for Scenario 2

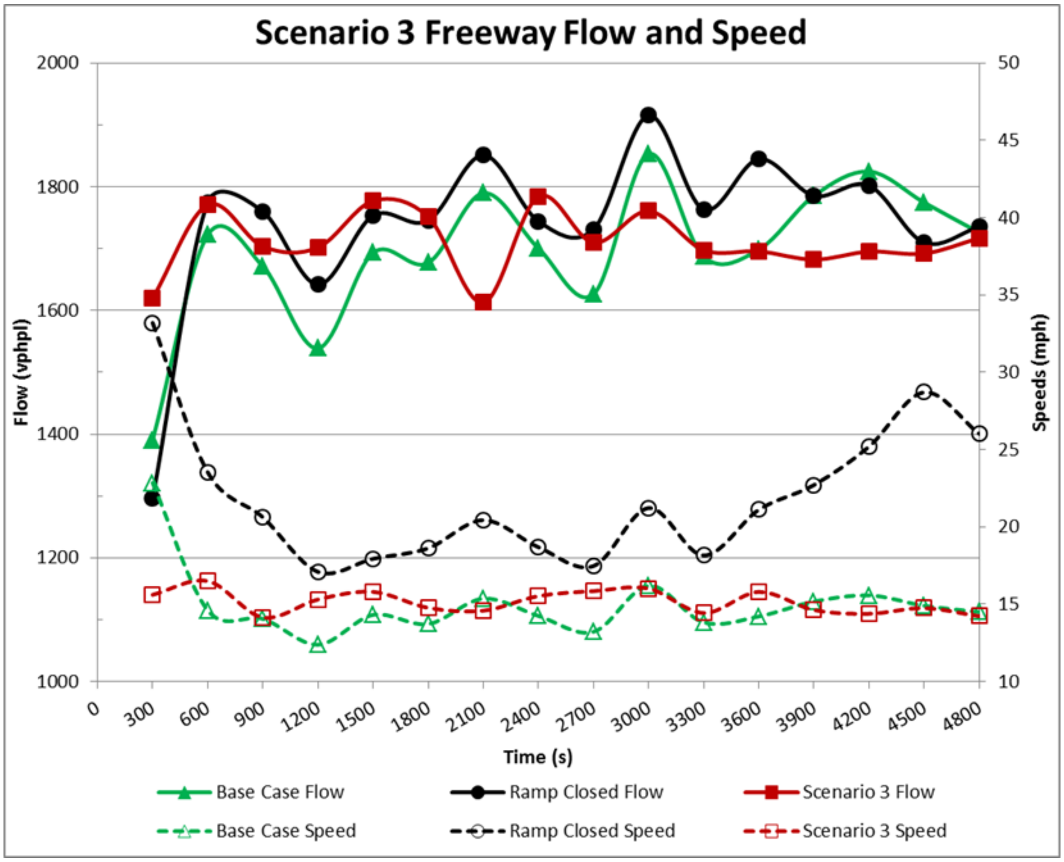

Figure 6.7 - Measured Average Freeway Flows and Speeds for Scenario 3

Scenario 1 is the scenario that best replicates the ramp closed freeway conditions. During the first half of the simulation, Scenario 1 keeps the freeway flows practically at the ramp closed levels. Toward the latter half of the simulation, the flow rate drops back to base case levels before ultimately spiking at the end. It can be seen that not only does this scenario delay the onset of the flow breakdowns; the breakdowns also recover quicker than they do in the Base Case Scenario. Scenario 2 also delays the onset of flow breakdowns, but it does not do so as quickly or to the extent that was seen in Scenario 1. However, Scenario 2 has less variation amongst the flows over the entire simulation period, and shows flow recovery occurring before it does in Scenario 1. In both of these measures Scenario 2 outperforms Scenario 1.

Scenario 3, by incorporating a shorter cycle length than the Base Case Scenario actually improves freeway flows without the use of a meter. This is achieved by dispatching smaller platoons of vehicles more often rather than large platoons less often. Scenario 3 shows a similar average flow across the simulation period as the Base Case Scenario, but the variation in flows is greatly reduced. Both Scenario 1 and 3 appear to smooth the variation in flows throughout the simulation period. It appears from the charts that both scenarios, particularly Scenario 1, bring the Base Case and Ramp Closed curves together. Even in cases where the overall flows were less than the Base Case, the slope of the flow curves are less. This can have a significant impact on travel time reliability.

It can be seen that the IRM scenarios not only increase freeway speeds, but they also reduce the variations in speed across the simulation period. Scenario 2 in particular maintains nearly constant freeway speeds after the onset of congestion about ten minutes in. Even Scenario 3 improves the average speed and lessens the variation, mostly because the shorter cycle length employed at the intersection more evenly distributes the large platoons dispatched from the interchange than the longer cycle length in the Base Case did. It seems apparent that even in the absence of ramp metering, shortening upstream cycle lengths can improve freeway conditions.

Based upon a review of the freeway impacts, it would be determined that Scenario 1 is the preferred scenario. But network-wide conditions must be considered as well.

Because of the needed balance between freeway and arterial performance, explained in more detail later, Scenario 2 is superior.

6.5 Network Performance

The metering system was analyzed based on the changes in delay, throughput, speed, and travel time for the entire system and for the freeway. In general, ramp metering systems hope to improve the performance of the freeway while not hindering the performance of the surface streets. Before and after values of system wide results are shown in Table 6.4 below.

Table 6.4 - System-wide Results (% change)

	Base Case	**Scenario X**	**Scenario 1**	**Scenario 2**	**Scenario 3**
Average Delay (s) / veh	161.6	N/A	166.7 (3%)	155.1 (-4%)	150.2 (-7%)
Throughput (# vehicles)	8420	N/A	8241 (-2%)	8372 (-1%)	8565 (2%)
Average Speed (mph)	12.5	N/A	13.0 (4%)	13.3 (6%)	13.2 (6%)
Total Travel Time (hr)	553.4	N/A	545.3 (-1%)	528.1 (-5%)	531.8 (-4%)
Total Delay (hr)	402.7	N/A	400.2 (-1%)	380.8 (-5%)	379.3 (-6%)

Color Coding Scheme*

More than 25% worse	5% to 25% worse	2% to 4% worse	1% worse to 1% better	2% to 4% better	5% to 25% better	More than 25% better

* - better or worse compared to Base Case

It can be seen that purely in terms of network improvements, no scenario was more effective than Scenario 3, which merely adjusted intersection timings. However, it is important to remember that the ramp metering scenarios are supposed to significantly improve freeway performance, while having little to no improvement to the system as a whole. The above results validate part of this statement. While the adjustments of signal timing can lessen the negative effects of IRM (Scenario 2), implementing the system into the Base Case timing plan will slightly increase average delay and throughput, as shown in Scenario 1. It was mentioned in previous sections that although Scenario 1 provided the best freeway improvements, it is not the preferred scenario because of the negative arterial impacts. The above network-wide results show an indication of that. Scenario 2 is intended to be a "system optimal" solution, rather than a "freeway optimal" solution, and thus it provides system-wide improvements over Scenario 1.

6.6 Freeway and Arterial Performance

It is important to ensure that the improvements to the freeway are worth any negative changes in system-wide performance. As Table 6.5 below shows, the values of average travel time and total throughput for the mainline vehicles are dramatically improved when the intersection metering system is implemented, but some arterial movements can be negatively impacted.

Table 6.5 Freeway and Arterial Performance Results (% change)

Freeway Performance						
Path		Base Case	Scenario X	Scenario 1	Scenario 2	Scenario 3
Freeway	Avg. Travel Time (s)	234.5	125.0 (-47%)	176.5 (-25%)	201.1 (-14%)	232.5 (-1%)
	Throughput (# vehicles)	5019	6371 (27%)	5873 (17%)	5530 (10%)	5044 (0%)

Arterial Performance						
Path		Base Case	Scenario X	Scenario 1	Scenario 2	Scenario 3
Southbound Frontage Road	Avg. Travel Time (s)	102.1	N/A	336.7 (230%)	94.6 (-7%)	59.4 (-42%)
	Throughput (# vehicles)	268	N/A	202 (-25%)	281 (5%)	311 (16%)
Far West Offramp	Avg. Travel Time (s)	104.0	N/A	85.4 (-18%)	100.9 (-3%)	112.8 (8%)
	Throughput (# vehicles)	314	N/A	361 (15%)	341 (9%)	314 (0%)
Metered Onramp	Avg. Travel Time (s)	343.8	N/A	899.1 (162%)	642.4 (87%)	316.7 (-8%)
	Throughput (# vehicles)	911	N/A	332 (-64%)	477 (-48%)	912 (0%)

Color Coding Scheme*						
More than 25% worse	5% to 25% worse	2% to 4% worse	1% worse to 1% better	2% to 4% better	5% to 25% better	More than 25% better

* - better or worse compared to Base Case

It can be seen from above, that while Scenario 1 shows tremendous improvements to the freeway, with a 25% decrease in average travel times and a 17% increase in throughput, its negative impacts to the southbound frontage road traffic, and the excessive onramp queue times may not make this a feasible scenario. Scenario 2 was tweaked so that the two uninvolved paths (southbound frontage road and far west offramp), would not be negatively affected. In the final calibration of this scenario, these two paths actually show travel time improvements of 7% and 3%, and throughput improvements of 5% and 9%, respectively.

In Scenario 1, the SB Frontage Road average travel time is increased because of vehicles queuing up in the left lane to get to the metered approach. Also, the Far West offramp average travel time is so improved because the freeway bottleneck is no longer backing up beyond the Far West offramp.

A trade-off must be made between freeway performance and arterial performance, as it is difficult to implement a metering system that improves both. The goal of this model is to produce average travel time improvements of around 15% for the freeway while keeping average travel time for the entire network the same. Any increases in arterial travel times could be considered a necessary sacrifice to improve the freeway. However, as the results above show, these increases in travel times can be limited to the metered offramp. The models were calibrated such that the non-metered movements would be affected minimally, while the metered movements would bear the burden of the increased delay and travel times, within an acceptable range. Intuitively, the metered movements will always see an increase in travel times. Here the travel times are significantly increased, which could be considered reasonable considering the improvements made to the freeway bottleneck. The Scenario 2 timing scheme was designed so that the additional travel time for ramp bound vehicles would be kept below 5 minutes.

6.7 Travel Time Reliability

The travel time reliability measures mentioned previously were also calculated from the simulation results for the freeway segment. The results for each scenario are shown in Table 6.6 below. To calculate the P.T.I., free-flow travel time through the freeway was needed, which in this case is determined to be 51.6 seconds.

Table 6.6 - Travel Time Reliability Results (% change)

	Base Case	**Scenario X**	**Scenario 1**	**Scenario 2**	**Scenario 3**
Average travel time (s)	583.4	125.0 (-47%)	176.5 (-25%)	201.1 (-14%)	232.5 (-1%)
95th Percentile Travel Time (s)	440.2	189.0 (-57%)	261.0 (-41%)	313.8 (-29%)	434.2 (-1%)
Buffer Index	88%	51% (-42%)	48% (-45%)	56% (-36%)	87% (-1%)
Planning Time Index	8.5	3.7 (-57%)	5.1 (-41%)	6.1 (-29%)	8.4 (-1%)

Color Coding Scheme*

More than 25% worse	5% to 25% worse	2% to 4% worse	1% worse to 1% better	2% to 4% better	5% to 25% better	More than 25% better

* - better or worse compared to Base Case

The significant effects of the IRM implementation are shown in the travel time reliability improvement. Both scenarios showed significant improvements in both travel time reliability indices. These results confirm what was hypothesized before, that the lessening of flow and speed variations across the simulation period would improve the travel time reliability. The buffer index of 88% for the Base Case shows that the time needed to travel is nearly double the free-flow travel time. The two IRM scenarios bring the B.I. to 48% and 56%, this would be a savings of 10 – 12 minutes for a trip that takes

30 minutes under free-flow conditions. The decreases in planning time index are even more pronounced. The P.T.I. of 8.5 for the Base Case suggests that for a trip normally taking 30 minutes, travelers would need to allow themselves 4 hours and 15 minutes to be guaranteed an on time arrival. The IRM P.T.I.'s of 5.1 and 6.1 mean that a savings of 1 hour and 12 minutes to 1 hour and 42 minutes can be achieved for the same trip. Stated in these terms, the travel time reliability impacts seem very crucial.

6.8 With Queue Clearance

When queue flushing is permitted, the following network wide results are collected, as shown in Table 6.7.

Table 6.7 – System-wide Impacts of Queue Flushing (% change)

	Base Case	**Scenario 1**	**Scenario 1 Queue Flush**	**Scenario 2**	**Scenario 2 Queue Flush**
Average Delay (s) / veh	161.6	166.7 (3%)	167.4 (4%)	155.1 (-4%)	163.4 (1%)
Throughput (# vehicles)	8420	8241 (-2%)	8491 (1%)	8372 (-1%)	8513 (1%)
Average Speed (mph)	12.5	13.0 (4%)	12.2 (-2%)	13.3 (6%)	12.4 (-1%)
Total Travel Time (hr)	553.4	545.3 (-1%)	571.6 (3%)	528.1 (-5%)	561.9 (2%)
Total Delay (hr)	402.7	400.2 (-1%)	420.3 (4%)	380.8 (-5%)	410.5 (2%)

Color Coding Scheme*

More than 25% worse	5% to 25% worse	2% to 4% worse	1% worse to 1% better	2% to 4% better	5% to 25% better	More than 25% better

* - better or worse compared to Base Case

It is interesting to note that in areas where the IRM did not improve over the Base Case, the queue clearance scenarios did not greatly degrade from the IRM scenarios.

However, queue clearance has eliminated all of the network wide gains as a result of the IRM. Freeway travel time reliability results were also collected, as shown in Table 6.8.

Table 6.8 – Travel Time Reliability Impacts of Queue Flushing (% change)

	Base Case	**Scenario 1**	**Scenario 1 Queue Flush**	**Scenario 2**	**Scenario 2 Queue Flush**
Average travel time (s)	583.4	176.5 (-25%)	227.0 (-61%)	201.1 (-14%)	230.1 (-61%)
95th Percentile Travel Time (s)	440.2	261.0 (-41%)	431.8 (-2%)	313.8 (-29%)	429.7 (-2%)
Buffer Index	88%	48% (-45%)	90% (2%)	56% (-36%)	87% (-1%)
Planning Time Index	8.5	5.1 (-41%)	8.4 (-1%)	6.1 (-29%)	8.3 (-2%)

Color Coding Scheme*

More than 25% worse	5% to 25% worse	2% to 4% worse	1% worse to 1% better	2% to 4% better	5% to 25% better	More than 25% better

* - better or worse compared to Base Case

It can be seen in the table above that nearly all of the improvements in the travel time reliability indexes gained in the IRM scenarios are practically eliminated in the queue flushing scenarios. Anytime large platoons are allowed to be dispatched onto the freeway, travel time reliability will suffer. This is extremely apparent in the case above.

Chapter 7: Conclusion and Future Research

7.1 Conclusion and Recommendations

In conclusion, a new ramp metering strategy, intersection ramp metering, is introduced in this research. This system is well suited to implement a ramp metering procedure where existing infrastructure and roadway geometries may not allow for a standard ramp metering system. This system is shown to improve both freeway travel times and throughput, while not having an overly detrimental effect on the surrounding arterial network. This control system is implemented using the VISSIM VAP program on a calibrated model representing an actual freeway segment with recurring bottlenecks. The results show a decrease in freeway travel times by 14% and an increase in freeway throughput by 10%, while the overall network shows a decrease in travel time of 5% and a decrease in throughput of 1%.

The major contributions of this study include the following:

- Proposed the idea of intersection ramp metering to address the deployment issue of ramp metering on urban freeway with limited right of way and ramp geometry.
- Developed optimization models and framework for adjusting system design parameters.
- Conducted simulation studies to evaluate the mobility and reliability performance of the proposed system with the base case model calibrated with field traffic flow and signal timing data.

Based on the evaluation results, the following recommendations can be made regarding the deployment strategies of the proposed intersection ramp metering system. If considering the improvements made to the freeway only, Scenario 1, where the IRM system is implemented without any signal adjustments, is the best alternative. However, the negative impacts on the un-involved paths would likely eliminate this scenario from field implementation. Scenario 2, where the IRM implementation also included adjustments to the intersection signal timing, was calibrated with these items in mind. The final results for Scenario 2 suggest that freeway improvements can be achieved, while concurrently the un-involved paths have improved conditions. If this system were to be implemented in the field, this scenario would likely be the best scenario.

7.2 Future Research Paths

This research presents the most simplified application of IRM. Future research should focus on better ways to integrate signal timing with the IRM operation, and the implementation of such a system on different geometric intersection alignments. The IRM system is designed to be able to be implemented in nearly any intersection upstream of a freeway entrance ramp. This network would be tweaked in any of the following scenarios:

7.2.1 With a 4-Legged intersection

If there were a westbound leg into the metered intersection, as there typically is in the field the system can easily be adapted to accommodate the additional traffic. If it is determined that the approach volumes destined for the freeway are significant, those movements could also be metered during their appropriate phases. Consider the following scenario when multiple movements are metered. In this case, imagine the same intersection modeled previously, except it has a westbound approach with enough traffic turning on to the freeway to warrant metering. In this case, the control logic would be based on the following:

$$r_1(t) = \begin{cases} r_{off} & if\ OCC_a > OCC_{a,max} \\ r_{1,2}(t-1) + K_r\left[OCC_{m,des} - OCC_m(t)\right] & if\ OCC_a < OCC_{a,max} \end{cases}$$

$$r_2(t) = \begin{cases} r_{off} & if\ OCC_b > OCC_{b,max} \\ r_{1,2}(t-1) + K_r\left[OCC_{m,des} - OCC_m(t)\right] & if\ OCC_b < OCC_{b,max} \end{cases}$$

Where OCC_a is the measured occupancy of detector "a", $OCC_{a,max}$ is the predetermined maximum allowable occupancy on detector "a" (which is set to ensure that onramp queues do not affect other traffic movements), $r_1\ (t)$ is the ramp metering rate during phase 1, $r_{1,max}$ is the maximum allowable ramp metering rate during phase 1, $r_{1,2}\ (t\text{-}1)$ is the metering rate during the previous interval (whether the measurement occurred during phase 1 or 2), OCC_b is the measured occupancy of detector b, $OCC_{b,max}$ is the predetermined maximum allowable occupancy on detector b, and $r_2\ (t)$ is the ramp metering rate during phase 2.

Since each approach may require different regulating parameters, each metering signal would be guided by its own VAP logic in a simulation. For instance, if the westbound approach does not have nearly the amount of storage as the eastbound approach, it may be necessary to include ramp flushing in the logic for the westbound traffic, as well as increasing the green interval and the minimum metering rate for this movement alone. Because each approach and their regulatory parameters are independent of the others, the entire system can be optimized based on the individual approaches. This system could similarly be further expanded to include intersections with three metered approaches.

7.2.2 With a Typical Frontage Road

The particular intersection modeled in this research did not include a true frontage road. However, for a majority of the intersections in the state of Texas, the system would have to be implemented on a location that includes a frontage road. In this scenario, one of the through lanes for the frontage road approach would have to be converted into a metered lane only. Consider the example shown below in Figure 7.1 of the intersection of the Interstate 35 N frontage road with 12th Street in downtown Austin.

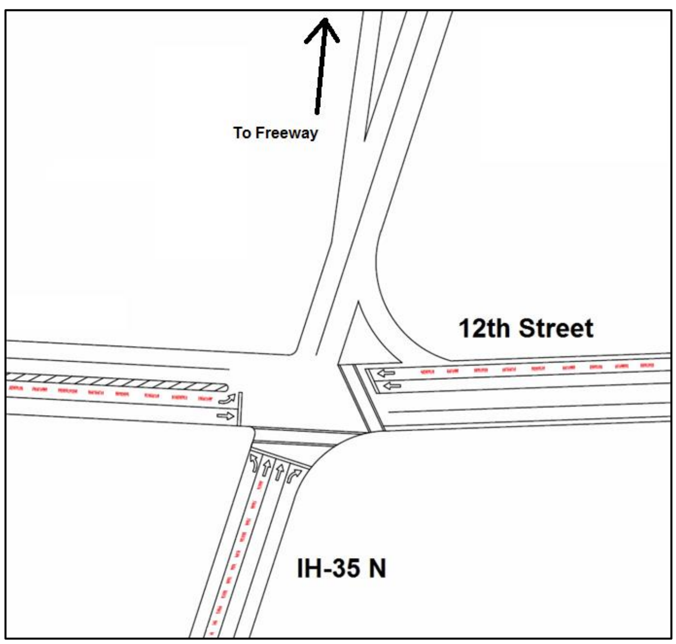

Figure 7.1 - Example Frontage Road Intersection Metering Layout

Here the dashed lines represent the lanes that could potentially be metered. Complicating this scenario is the fact that the left turn lane for the NB approach would still have to be operational, while at the same time many through vehicles will stay on the frontage road, so vehicles queued for the freeway would have vehicles moving on both sides of them. These queues would have to be carefully managed so that they would not get so long that they would complicate the lane selection procedure for vehicles approaching the intersection. It would likely be that the minimum metering rate for this approach would be higher than it would be in the other approaches. Further studies will simulate intersections with this layout.

7.2.3 With Ramp Gap Control

Because most ramp metering systems are implemented just close enough to the freeway to allow for metered vehicles to accelerate to an appropriate speed and merge in with freeway traffic, these systems are able to effectively control the rate at which the metered vehicles arrive. However, for an intersection ramp metering system, the metering occurs at the far upstream end of the ramp. In this scenario, differences in driver behavior cause the rate at which vehicles are metered at the intersection to be slightly different than the rate at which they arrive at the freeway merge point. In this scenario, it would be helpful to implement a form of gap control that ensures that the vehicles will not be platooning by the time they reach the merge point. This could be carried out by a series of regulatory signs that tell motorists to leave a certain gap between them and the vehicle in front of them, such as one car length. An example of such a sign is shown in Figure 7.2 below.

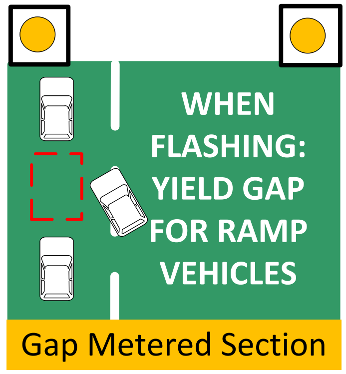

Figure 7.2 - Gap Control Sign

This concept would simply make the metering operation more effective, as it would improve the accuracy of the ramp metering rate.

7.2.4 With Adaptive Cycle Length Calculations

The effectiveness of IRM could be even further enhanced if adaptive traffic signal control functions were included in the control logic. An ideal situation would be a traffic responsive calculation of cycle length and phasing splits. This could be implemented based on Webster's optimal cycle length equation. Here, the optimal cycle length C_o can be determined based on the revised Webster's optimal cycle length equation.

$$\mathbf{C_0} = \frac{\mathbf{1.5L + 5}}{\mathbf{1 - \sum_p (V_p/S)}} = \frac{\mathbf{1.5L + 5}}{\mathbf{1 - r_D}/S}$$

Where L is the total lost time for all phases, S is the saturation flow rate (assumed to be 1800 vphpl), and V_p is the volume of the critical movement which is assumed to the volume of the metered approach, and r_D is the current ramp metering rate within the allowable range. Here, there is a key difference in the calculation of the lost time since vehicles under ramp metering control need to start from a stop in every ramp metering cycle. The total lost time can be calculated as the following:

$$\mathbf{L} = \frac{\mathbf{g}}{\mathbf{T_r} + \mathbf{T_G}} * \mathbf{l_0} + \sum_{\mathbf{p}} \mathbf{l_0}$$

Where g is the green time assigned to the ramp metering approach, l_0 is the start-up lost time which is assumed to be two seconds per vehicle, and p indicates other minor phases. Combining the previous two equations we get:

$$\mathbf{L} = \frac{\mathbf{g}}{\frac{\mathbf{3600} * \mathbf{n}}{\mathbf{r_D C_0 / g}}} \mathbf{l_0} + \mathbf{l_0} = \left(\frac{\mathbf{r_D C_0}}{\mathbf{3600} * \mathbf{n}} + \mathbf{1}\right) \mathbf{l_0}$$

Substituting L in and solving for C_0, we can get the optimal cycle length for IRM, which becomes the following function with respect to r_D

$$\mathbf{C_{IRM}} = \frac{\mathbf{1.5} * \mathbf{l_0} + \mathbf{5}}{\mathbf{1} - \frac{\boldsymbol{r_D}}{\boldsymbol{S}} - \frac{\mathbf{1.5}\boldsymbol{r_D}}{\mathbf{3600}\boldsymbol{n}}}$$

This equation can be used to determine the cycle length of the intersection on which the ramp metering signal is applied. This relationship could be tied into the VAP logic for a simulation analysis.

Appendix

Example VAP Code [3], this was used for Scenario 1

```
PROGRAM RampMeter; /** Ramp meter for Peak direction **/

CONST /** select ALGORITHM to run **/

    Algorithm = 1, /** 1 - ALINEA; 2 - Fixed; 3 - No Meter; 4 - Ramp
closure**/
    QueueOverRide = 0, /** 1 - queue override; 0 - no queue override
**/
    QueueCountInterval = 5,
    OccupancyInterval = 1,
    GreenInterval = 1.5,
    KR = 70, /** ALINEA constant **/
    MaxRate = 26000,
    MinRate = 850,
    FixedRate = 900, /* used for fixed metering, Can only model rates
400, 450, 515, 600, 720, 900, 1200 */
    RedInterval = 1.5,
    TransitionPeriod = 60,
    NumberofDetectors = 2,  /** total num. of downstream detectors **/
    dd1 = 11, dd2 = 12, /**downstream detector numbers**/
    NumberMeterLane = 2,
    d_Presence1 = 2, /** presence detector-Lane 1 **/
    QueueDetector_Advance = 1,
    Occupancy_Opt = 0.30, /** optimal or target occupancy **/
    Occupancy_Threshold = 0.25, /** threshold to metering **/
    Queue_Threshold = 0.90, /** for ramp queue detection **/

/* Data Collection Parameters */
StartTime = 0,
EndTime = 4800;

/**********************************************************************/
SUBROUTINE ALINEA;

  IF CountTimer = OccupancyInterval THEN
    TRACE (variable (MeterPrevious));
    IF OccupancyInterval = 1 THEN             /** set interval to 1-
sec, for report purposes **/
      AverageOcc := (Occup_rate (dd1) + Occup_rate
(dd2))/NumberofDetectors;
      AvgOccup_DownStreamDet := AverageOcc;
    ELSE
      AvgOccup_DownStreamDet := Occup_DetDownStream /
(OccupancyInterval);
    END;
    IF AvgOccup_DownStreamDet < Occupancy_Threshold THEN
      MeterRate := 80000;
    ELSE
```

```
      MeterRate := MeterPrevious + KR*(Occupancy_Opt -
AvgOccup_DownStreamDet)*100;
    END;
    IF MeterRate >= MaxRate THEN
      MeterRate := MaxRate;
      RedInt := (3600/MeterRate)*NumberMeterLane - GreenInterval;
      MeterPrevious := MeterRate;
    ELSE
      IF MeterRate <= MinRate THEN
        MeterRate := MinRate;
        RedInt := (3600/MeterRate)*NumberMeterLane - GreenInterval;
        MeterPrevious := MeterRate;
      ELSE
        RedInt := (3600/MeterRate)*NumberMeterLane - GreenInterval;
        MeterPrevious := MeterRate;
      END;
    END;
    /**SumVeh := front_ends(dd1) + front_ends(dd2);**/
    SumVeh := rear_ends(dd1) + rear_ends(dd2);
    FlowRate := (SumVeh/OccupancyInterval) * 3600;
    /** TRACE (variable); **/
    TRACE (variable (AvgOccup_DownStreamDet, FlowRate));
    TRACE (variable (MeterRate, RedInt));
     /** TRACE (variable (AvgOccup_DownStreamDet)); **/
    RESET(CountTimer);
    Occup_DetDownStream := 0;
    clear_rear_ends(dd1);
    clear_rear_ends(dd2);
  ELSE
    AverageOcc := (Occup_rate (dd1) + Occup_rate
(dd2))/NumberofDetectors;
    Occup_DetDownStream := Occup_DetDownStream + AverageOcc;
  END.
/*****************************************************************/
SUBROUTINE FixedMeter;
  MeterRate := FixedRate;
  /* RedInt := (3600/MeterRate)*NumberMeterLane - GreenInterval. */
  RedInt := RedInterval.
/*****************************************************************/
SUBROUTINE NoMeter;
  IF CountTimer = OccupancyInterval THEN
    IF OccupancyInterval = 1 THEN /** set interval to 1-sec, for report
purposes **/
      AverageOcc := (Occup_rate (dd1) + Occup_rate
(dd2))/NumberofDetectors;
      AvgOccup_DownStreamDet := AverageOcc;
    ELSE
      AvgOccup_DownStreamDet := Occup_DetDownStream /
(OccupancyInterval); /** **/
    END;
    SumVeh := rear_ends(dd1) + rear_ends(dd2);
    FlowRate := (SumVeh/OccupancyInterval) * 3600;
    /** TRACE (variable); **/
    TRACE (variable (AvgOccup_DownStreamDet, FlowRate));
```

```
    /**TRACE (variable (MeterRate, RedInt)); **/
    /** TRACE (variable (AvgOccup_DownStreamDet)); **/
    RESET(CountTimer);
    Occup_DetDownStream := 0;
    clear_rear_ends(dd1);
    clear_rear_ends(dd2);
   ELSE
    AverageOcc := (Occup_rate (dd1) + Occup_rate
(dd2))/NumberofDetectors;
    Occup_DetDownStream := Occup_DetDownStream + AverageOcc;
   END.
SUBROUTINE RampClose;
  RedInt := 1000000;
  sg_red(1);
  IF CountTimer = OccupancyInterval THEN
   IF OccupancyInterval = 1 THEN /** set interval to 1-sec, for report
purposes **/
    AverageOcc := (Occup_rate (dd1) + Occup_rate
(dd2))/NumberofDetectors;
    AvgOccup_DownStreamDet := AverageOcc;
   ELSE
    AvgOccup_DownStreamDet := Occup_DetDownStream /(OccupancyInterval);
   END;
   SumVeh := rear_ends(dd1) + rear_ends(dd2);
   FlowRate := (SumVeh/OccupancyInterval) * 3600;
   /** TRACE (variable); **/
   TRACE (variable (AvgOccup_DownStreamDet, FlowRate));
   TRACE (variable (MeterRate, RedInt));
   /** TRACE (variable (AvgOccup_DownStreamDet)); **/
   RESET(CountTimer);
   Occup_DetDownStream := 0;
   clear_rear_ends(dd1);
   clear_rear_ends(dd2);
  ELSE
   AverageOcc := (Occup_rate (dd1) + Occup_rate
(dd2))/NumberofDetectors;
   Occup_DetDownStream := Occup_DetDownStream + AverageOcc
  END.
/*******************************************************************/
SUBROUTINE MeterOperation;
/**********************************/
/**** METERING OPERATIONS ****/
/**********************************/
/*Single-lane meter */

TRACE (variable (QueueSpill,FlushFlagCurrent));
TRACE (variable (FlushFlagPrevious,TransitionTimer));
IF (t_green(1) >= GreenInterval) OR (Occupancy(d_Presence1) <=0) THEN
  IF (QueueOverRide AND QueueSpill) THEN
    MeterPrevious := MaxRate; /** Do not start red if queuespill and
with override policy **/
    IF (SimuTime >= StartTime) AND (SimuTime < EndTime) THEN
          TotalMeterFlushTime := TotalMeterFlushTime + 1;
          TRACE (variable (SimuTime,TotalMeterFlushTime));
```

```
    END;
  ELSE
    /* No queue spill. Meter flush stops after transition */
    IF TransitionFlag = 0 THEN /* Not in transition */
      sg_red(1);
    ELSE
      IF TransitionTimer >= TransitionPeriod THEN
        sg_red(1);
        Stop(TransitionTimer);
        Reset(TransitionTimer);
        TransitionFlag := 0;
      END;
    END;
   END;
END;
IF (t_red(1) >= RedInt) THEN /*Red has the desired metering rate */
  IF Occupancy(d_Presence1) > 0 THEN
    sg_green(1);
  END;
END.

/*****************************/
/**** This is the main routine ****/
/*****************************/

START(QueueTimer);
START(CountTimer);
SimuTime := SimuTime + 1;

IF QueueTimer = (QueueCountInterval + 1) THEN
  AvgOccup_AdvanceQueueDet := Occup_AdvanceQueueDet /
QueueCountInterval;
  QueueSpill := AvgOccup_AdvanceQueueDet >= Queue_Threshold;
  FlushFlagPrevious := FlushFlagCurrent;
  IF QueueSpill THEN
    FlushFlagCurrent := 1;
  ELSE
    FlushFlagCurrent := 0;
  END;
  IF (FlushFlagPrevious = 1) AND (FlushFlagCurrent = 0) THEN
    Start(TransitionTimer);
    TransitionFlag := 1;
  END;

  RESET (QueueTimer);
  Occup_AdvanceQueueDet := 0;
ELSE
  Occup_AdvanceQueueDet := Occup_AdvanceQueueDet +
Occup_rate(QueueDetector_Advance);
END;

IF Algorithm = 1 THEN
  GOSUB ALINEA;
  GOSUB MeterOperation;
```

```
ELSE
  IF Algorithm = 2 THEN
    GOSUB FixedMeter;
    GOSUB MeterOperation;
  ELSE
    IF Algorithm = 3 THEN
      GOSUB NoMeter;
    ELSE
      IF Algorithm = 4 THEN
        GOSUB RampClose;
      END;
    END;
  END;
END.
```

Also, the flow charts of changes in VAP code and corresponding simulation results for each IRM scenario were tracked. These flow charts are shown below. To help fit on the page, the flow charts are split into 3 tables. In the first, the simulation parameter settings for each run are shown. Blank cells mean that the above parameter was kept the same for that run. In the second table, the simulation results for each run are shown. In the third table, general comments made after each run are recorded. The final parameter selections are shown in **bold**.

Scenario 1

Run	OCC_{des}	OCC Threshold	Min Rate	Green Interval
1	0.30	0.25	900	2.0
2	0.29			
3	0.31			
4	0.30	0.26		
5		0.24		
6		0.25	850	
7			800	
8			950	
9			**850**	**1.5**
10				1.0
11				2.5

Run	Avg. Delay/Veh (s)	Throughput (# veh)	Avg. Speed (mph)	Total Travel Time (hr)	Total Delay (hr)	Freeway Avg. Travel Time (s)	Freeway Throughput (# veh)
1	166.2	8251	12.9	547.4	401.9	184.2	5764
2	167.5	8226	12.9	547.9	402.9	183.7	5763
3	170.5	8200	12.6	554.5	409.9	187.3	5724
4	168.0	8254	12.8	551.6	406.1	184.8	5774
5	169.2	8228	12.7	552.5	407.5	186.2	5753
6	167.1	8209	12.9	545.5	400.8	177.7	5838
7	167.1	8209	12.9	545.5	400.8	177.7	5838
8	170.1	8214	12.7	554.0	409.2	187.5	5734
9	**166.7**	**8241**	**13.0**	**545.3**	**400.2**	**176.5**	**5873**
10	170.5	8182	12.8	551.7	407.6	176.7	5879
11	166.4	8240	12.9	547.0	401.7	183.09	5768.8

Run	After Notes
1	Good results, try lowering des occ
2	No performance improvements, try a higher des occ
3	Even worse, go back to 0.30, now adjust occ threshold
4	No performance improvements, try a lower threshold
5	Still no performance improvements, return to .25, try a lower min rate
6	Good improvements, try going even lower
7	Same, try increasing
8	Worse, go back to 850 and adjust green interval
9	**This is the best so far, let’s try to further restrict the green interval**
10	No freeway improvements and overall system is worse, try a higher green interval
11	Good system-wide results, but no freeway improvements, return to run 9

Scenario 2

Run	Cycle Length	Green Interval	Minimum Rate
1	**100**	**2**	**900**
2	90		
3	115		
4		1.5	
5	100		
6		2	850

Run	Avg. Delay/Veh (s)	Throughput (# veh)	Avg. Speed (mph)	Total Travel Time (hr)	Total Delay (hr)	Freeway Avg. Travel Time (s)	Freeway Throughput (# veh)
1	**155.1**	**8372**	**13.3**	**528.1**	**380.8**	**201.1**	**5530**
2	159.8	8330	13.0	536.9	390.3	201.9	5514
3	155.9	8358	13.2	530.2	383.0	203.9	5497
4	157.8	8317	13.2	530.9	384.6	196.6	5583
5	158.0	8345	13.2	533.5	386.6	193.0	5631
6	157.3	8370	13.3	533.4	386.1	192.2	5648

Run	SB Frontage Rd. Avg. Travel Time (s)	SB Frontage Rd. Throughput (# veh)	Metered Onramp Avg. Travel Time (s)	Metered Onramp Throughput (# veh)	Far West Offramp Avg. Travel Time (s)	Far West Offramp Throughput (# veh)
1	**94.6**	**281**	**642.4**	**477**	**100.9**	**341**
2	121.2	272	642.8	487	98.5	343
3	90.7	283	629.0	489	105.1	340
4	111.8	266	708.1	430	104.5	345
5	139.9	261	708.6	430	98.6	348
6	140.59	262.3	709.59	429.4	98.07	348.4

Run	Notes
1	**Try shorter cycle, raise sat flow to 1900**
2	Not good, try sat flow = 1700
3	Neither, try lowering G from 2.0 to 1.5 to improve freeway flow
4	SB frontage TT too high, return to 100 sec cycle, keep 1.5 sec green
5	SB frontage even worse, return to 2.0 green, lower min rate to 850
6	SB frontage no better, return to run 1

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