The Thesis Committee for Michael DeGaspari
Certifies that this is the approved version of the following thesis:

Evaluation of the Mobility Impacts of Proposed Ramp Metering and Merge Control Systems: An Interstate 35 Case Study

APPROVED BY
SUPERVISING COMMITTEE:

Supervisor:

C. Michael Walton

Jing (Peter) Jin

Evaluation of the Mobility Impacts of Proposed Ramp Metering and Merge Control Systems: An Interstate 35 Case Study

by

Michael DeGaspari, B.E.

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Dedication

To my parents, whose support and guidance have helped me to reach far and grab hold of my dreams.

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Abstract

Evaluation of the Mobility Impacts of Proposed Ramp Metering and Merge Control Systems: An Interstate 35 Case Study

Michael DeGaspari, M.S.E.

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Supervisor: C. Michael Walton

Increasing demand on freeway facilities is a major challenge facing urban areas in the United States and throughout the world. Active Traffic Management (ATM) strategies can be used to increase the performance of these facilities through improved operations without the significant expenditure associated with adding capacity. One ATM strategy that has been widely deployed in the current state of practice is ramp metering, which controls the traffic demand placed on a freeway. Merge control strategies are less prevalent and largely undeveloped. This study examines the recurrently congested northbound section of Interstate Highway 35 that approaches downtown Austin, Texas. Using the VISSIM microsimulation platform, a model of this segment was developed and calibrated to reflect current peak-hour congestion. Within this model, ramp metering and merge control technologies were implemented. The impacts on traffic throughput, speed and travel time for each of these proposed systems are evaluated.

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Chapter 1: Introduction

Transportation providers are faced with the challenge of providing for a growing level of traffic under a constrained budget. This is due to falling or stagnant revenue levels, and an extreme growth in the level of congestion on the nation's roads during the past few decades. In many cases, DOT entities are unable to add capacity by expansion or new construction to the most stressed portions of the road network, urban freeways, due to insufficient budget or lack of available right-of-way. Because of this, many DOTs are turning to traffic operations solutions such as active traffic management (ATM) as a way of mitigating urban freeway congestion in a cost-effective manner.

Active traffic management strategies involve the implementation of intelligent transportation systems that influence and regulate the flow of vehicles on a freeway. The goal of these strategies is to improve the safety and operation of a road that has exceeded its capacity. In evaluating the impact investment in these technologies has made on the performance of the road network, it is necessary to examine how key metrics are affected. Transportation providers are charged with two main tasks. First, the facilities they build and maintain must be safe for users. Second, these facilities must provide to users a maximum amount of utility for a given level of investment. Therefore, to meet these two goals, DOT entities must ensure that any active traffic management techniques implemented on highway facilities maintain or improve the roadway's safety while improving indicators of system effectiveness, such as travel time reliability.

A roadway's travel time reliability is an indicator of the level of congestion experienced on it. A measure of travel time reliability attempts to quantify the variability of the travel time users experience along the same route at different times. Many studies have been published that show an increase in the level of safety and a decrease in the number of fatalities and accidents on the roads on which ATM strategies have been implemented. However, the number of highways with ATM worldwide remains comparatively small. While some forms of active traffic management attempt to regulate the flow of traffic already on highways, such as variable speed limit techniques, others attempt to improve the safety and throughput of the merging/weaving sections seen at onramp locations. Freeway facilities are set apart from other roadway classifications by their requirement for access control. In the state of practice, this control is achieved by limiting motorist entrance and egress from the facility to specially constructed onramps and off-ramps. A form of active traffic management that specifically attempts to address these merging and weaving sections is ramp metering. Ramp metering technologies have gained acceptance and are increasingly implemented on freeway facilities throughout the world. Other methodologies for actively controlling the merging and weaving sections at onramps are still being proposed and evaluated. By comparing metrics relating a facility's efficiency before and after implementing these control devices, DOTs can make better decisions about making ATM investments.

1.1: RESEARCH BACKGROUND: ACTIVE TRAFFIC MANAGEMENT

Active traffic management systems utilize a suite of intelligent transportation systems technologies. Typically, these ATM systems integrate roadway sensors that measure the speed and volume of traffic on the highway, variable message signs that communicate dynamically changing rules or messages to motorists, and a control algorithm that determines which messages should be displayed under given traffic conditions. There are a number of different control strategies that can be implemented on ATM systems (Mirshahi, et al., 2007). These include queue warning algorithms, variable speed limit (VSL) control, and dynamic lane assignment. All of these methodologies utilize different strategies to improve the flow of traffic.

A VSL system replaces the static posted speed limit with a speed limit that is dynamically adjusted. The control logic of a VSL system will dynamically adjust the speed limit along a roadway broken into discrete segments. It will analyze the speed and traffic flow along each of these segments. Using this information, the control algorithm will adjust the speed limit along each segment of the highway in order to smooth the transition between free-flow traffic upstream and congestion downstream. This change in speed limits will prevent shockwave impacts, which result in the formation of excessive queues. By utilizing VSL strategies, the capacity of existing highways can be expanded without physically widening the right-of-way. This can be an effective strategy for increasing the capacity of urban highways, which experience bottleneck scenarios during peak commuting hours. FHWA recommendations for VSL systems focus on ensuring adequate sensor coverage so that the internal model in the control algorithm reflects the actual traffic conditions. Additionally, there is a focus on ensuring that motorists always have the dynamically changing rules in their field of vision.

Variable speed limit systems effectively force vehicles to travel at similar speeds. This reduces the occurrence of small headways between vehicles following each other. This also reduces the variability of speeds across lanes. Both of these reductions help to limit the number of collisions between vehicles, increasing the safety of the road and also delays due to accidents. In addition, forcing vehicles to travel at similar speeds also results in a reduction in the variability of the gaps left between vehicles (Varaiya & Kurzhanskiy, 2010). Without large, inefficient gaps left by drivers traveling well below or above the speed limit, the total volume of the traffic on the roadway can increase. One congestion issue that plagues highways, particularly during peak hours, is queue formation. Queues form on highways when a segment has a downstream output volume that is less than the upstream input volume. A VSL system can be used to increase the speed of the downstream end of the segment or decrease the speed of the upstream end of the segment. By modifying these conditions, the queue will dissipate.

Although VSL systems are gaining popularity overseas, adoption rates in the United States have been slow. In some cases, early field deployments in the US have been advisory, and the dynamically changing limits are not enforced (Nissan & Koutsopoulosb, 2011). This is done over concerns of limited public acceptance, or legal statutes that do not provide for a dynamic speed limit (Sisiopiku, 2001). Examples of states with advisory VSL systems include Oregon, Utah and Minnesota. In other states, such as New Jersey and Washington, speed limits posted by VSL systems are enforceable. Because not all of the VSL systems nationally are enforced, it is difficult to determine the effectiveness of these systems.

Queue warning systems are often deployed in conjunction with VSL systems. Sensors in the roadway detect when a queue develops, and a display system alerts motorists upstream to reduce their speed. The aim is to sufficiently reduce the speed of vehicles upstream to decrease the vehicular flow into the queued segment. This will hasten the dissipation of the queue, and will also prevent rear-end collisions caused by motorists braking too quickly when approaching the queue. Active traffic management systems typically utilize variable message signs positioned above each lane of traffic in order to communicate information and dynamic roadway rules. For VSL systems, this means that the dynamic speed limit is posted above each lane. This has two purposes. First, it helps to harmonize speeds across different lanes by not allowing motorists to treat different lanes as “slow lanes” or “passing lanes.” Second, it allows each variable message sign to include warnings about individual lane closures. In a situation where a collision has occurred, this allows authorities to immediately notify traffic upstream of a lane closure and direct motorists to clear lanes. This is known as dynamic lane control.

Active traffic management systems that include real-time VSL controls can be used to mitigate congestion. Field tests of these systems have revealed that they are effective in situations where bottlenecks cause the special distribution of the traffic speed on the highway to exhibit dramatic reductions from free-flow speeds to congested and stop-and-go levels (Chang, Park, & Paracha, 2011). By using VSL strategies, the transition between free-flow speed and queue situations are smoothed, which increases the average speed and reduces overall travel times on recurrently congested roadway segments. This also increases total output.

Ramp metering systems work by managing the overall demand placed upon a highway facility by an onramp. Ramp meters use traffic signals at freeway onramps to allow single or dual vehicles to merge onto the mainline with a small delay between cars. This helps to minimize conflicts due to lack of acceptable gaps and queues spilling from the merging section onto the mainline and the frontage road. By reducing the amount of vehicles entering the facility at any given time, this helps to improve highway safety.

1.2: Problem Statement

This project will evaluate the impact on congestion and travel reliability indexes of two ATM technologies deployed to improve highway merging at onramps. The ATM technologies to be investigated are ramp metering and gap metering. Both of these technologies focus on active control of the merging and weaving sections of freeways. Ramp meters regulate the flow of traffic onto a facility, limiting the number of vehicles competing for available gaps on the mainline and ensuring that merging vehicles are travelling at lower speeds during congested hours. Gap metering is a novel active traffic management technique proposed by Jin et al. which focuses on modulating traffic flow on the mainline in order to increase the supply of gaps available to merging traffic. Gap metering also attempts to make the appearance of gaps in the mainline traffic flow more predictable. These two techniques will be evaluated separately and together.

The evaluation will be conducted using a VISSIM traffic micro-simulation of the northbound I-35 corridor between SH-71 and Lady Bird Lake in Austin, Texas. This segment of freeway experiences heavy traffic during the AM peak. In order to ensure the simulations reflect the expected morning peak conditions, video data of the highway and its frontage road was collected and processed. In simulation, four scenarios will be considered: base case, ramp metering only, gap metering only, and both ramp and gap metering operating in conjunction. Each of these scenarios was considered under both peak hour and off-peak hour conditions. The results of these simulations will be compared using volumetric throughput and travel time equitability indexes. Based on the performance of the facility under each of these cases, recommendations will be made for the implementation of an active traffic management system along the route.

1.3 Thesis Summary

This thesis uses a microsimulation of a section of the I-35 corridor in Austin, Texas to evaluate different active traffic management strategies for merging sections on freeways. Chapter 2 discusses the existing state of practice of ramp metering and active merge control technologies, as well as the proposed merge control strategy of Gap Metering. Chapter 3 lays out the experimental framework for this evaluation. Chapter 4 is a discussion of the results from the various microsimulation runs. Chapter 5 lays out recommendations for facility improvement based on the results of this study.

Chapter 2: Literature Review—ATM Strategies for Weaving & Merging Sections

2.1: Ramp Metering State of Practice

One of the first methods to emerge in the field of active highway control is ramp metering. Ramp meters utilize existing technologies to control access onto a freeway's mainline. Ramp metering systems typically utilize detection units on a ramp and on the mainline in order to ascertain the demand and current additional capacity of a freeway. They then use this information to control the rate at which new vehicles are allowed to access the freeway and merge onto the mainline. By regulating the traffic entering the freeway, ramp meters smooth the flow of traffic to avoid traffic breakdowns. Ramp metering strategies help to break down platoons of traffic attempting to gain access to freeway facilities. By replacing a continuous flow of vehicles from an onramp with individual vehicles with larger headways, ramp meters improve merging behaviors and help to prevent the incidence of recurrent bottlenecks.

2.1.1: Ramp Metering Operation

Ramp meters are control devices placed at freeway onramps (Chaudhary, Tian, Messer, & Chu, 2004). The design of ramp meters has three operational objectives. First, ramp meters control the number of vehicles allowed to enter the freeway. Second, they also reduce demand on the freeway. Additionally, ramp meter implementation aids in breaking up traffic platoons that form from queuing at upstream signal heads. By achieving these operational objectives, ramp meters serve to manage the demand placed on the mainlines of freeway facilities during peak periods. By ensuring that the traffic volume wishing to merge onto a mainline section remains below the freeway's bottleneck capacity, ramp meters can improve mainline speed and throughput. This is achieved by effectively trading mainline delay that results from mergers with queuing delay at ramp meter signal heads. This introduction of a controlled delay to vehicles intending to merge onto the freeway essentially levies a cost on freeway use, reducing demand to use the freeway mainline for shorter trips during peak hours. In addition, by reducing merging demand, conflicts occurring at freeway merger points are reduced, thus improving facility safety.

Chaudhary et al. note that urban freeway facilities experience their highest levels of congestion during peak hours. Much of this congestion is the result of longer distance commuting to and from workplaces. When freeways experience extreme congestion and exhibit traffic flow breakdown, their ability to move high volumes of traffic falls off significantly. Because ramp metering installations serve to incur a delay cost on users, Chaudhary et al. suggest that they can be effectively used if deployed along highly congested bottleneck sections of freeways. In this way, motorists traveling along the corridor within the bottleneck section will be encouraged to avoid the freeway mainline due to the additional ramp delay. This will make them more likely to take alternative routes along surface streets, thus relieving a small amount of demand upon the facility in the worst congested areas. Because travelers who enter the freeway facility far upstream at uncontrolled ramps do not experience this delay, the freeway's capacity is used to favorably move motorists with longer trips into and beyond core areas. This shifting of delay only works if all onramps along and slightly beyond a bottleneck section have ramp metering treatments applied.

2.1.2: Ramp Meter System Design

In 2000, the Texas Department of Transportation updated its roadway design manual to include standard references and criteria for ramp meters (TxDOT, 2000). This report drew upon a number of sources, including the ramp metering design standards of other states. The manual defines three types of ramp metering systems available for implementation as part of the state of practice in Texas. The first type is a single-lane, one car per green ramp meter. This system allows for a single car to enter the freeway mainline during each signal cycle. The system can have a predefined cycle length of 4.5 seconds, resulting in a capacity of 800 vph. The second type is a single-lane, multiple cars per green ramp meter. In this design, a sign posted alongside the signal head notifies drivers how many vehicles may proceed per green, and the control system is supplemented by multiple queue detectors. This system has a variable cycle length of between 6-6.5 seconds, and can handle up to 1200 vph. The third type of ramp metering system allows for dual-lanes on the onramp. The signal heads do not display simultaneous greens, but instead allow two queues to form and take turns for the acceleration zone.

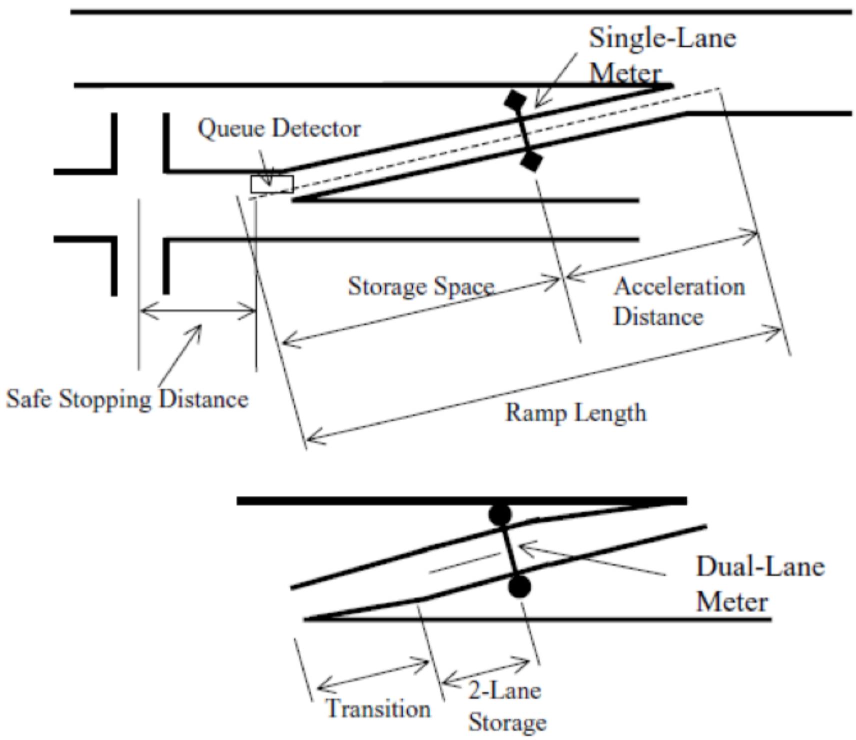

Figure 1: Ramp Meter Design Schematic (TxDOT 2000)

There are several key design aspects to be taken under consideration in ramp meter deployment. First, the acceleration distance a vehicle needs to merge with free-flow traffic from the stop bar must be considered. Insufficient acceleration lengths result in a safety hazard. Second, adequate queue storage must be provided upstream of the ramp meter signal head. If adequate queue storage is not present, queues may propagate through the local street network, potentially reaching the next exit upstream of the onramp. Finally, sufficient stopping distance must be provided for vehicles which have been discharged from upstream intersections. Figure 1 illustrates the key design aspects for high quality ramp metering systems. The recommended horizontal clearances are 22 ft. for single-lane ramp meters, 28 ft. for curbed dual-lane ramp meters, and 32 ft. for uncurbed ramp meters. The length requirement for the queue storage can be calculated based on the following empirical equation for each lane:

$$L = 0.25V - .0000742V^2\ , where\ V \leq 1600\ vph\ (eqn. 1)$$

where L is the required length for ramp metering system on the ramp, and V is the ramp flow. The TxDOT Ramp Metering study also provides guidelines regarding the required distance from meter to the merging point and the stopping site distance between the end of metered queue and the upstream intersection.

2.1.3: Impact of Ramp Metering on Traffic Flow

Oner describes the impact that the installation of ramp metering devices has on the distribution of vehicular headways (Oner, 2011). By comparing the observed headways at four unsignalized and two signalized ramp locations in Ohio, a distribution of the interarrival time (IAT) was constructed. IAT distributions at the unsignalized ramp locations were found to be very similar to the IAT distributions of the corresponding mainline traffic. In contrast, the IAT distributions at metered ramp locations were substantially different from the mainline distributions at corresponding locations. The distributions showed that ramp meter locations typically demonstrate headway distributions that skew toward shorter time headways than unsignalized ramps.

Zhang and Levinson evaluated the impact that continuous use of ramp metering along freeway facilities has upon the capacity of recurrent bottleneck sections (Zhang & Levinson, 2010). By evaluating the traffic flow at bottleneck sections with adjacent ramp meters at 27 individual locations in Minneapolis-St. Paul, the impact that ramp metering implementation has on these sections can be established using empirical data taken over a seven week period. Zhang and Levinson determined that ramp metering systems at bottleneck segments increase the capacity of the mainline at the bottleneck in three ways. First, they postpone the traffic flow breakdown that occurs at bottleneck locations, sometimes eliminating them entirely. The study measured an average 73% increase in the pre-queue transition period. Second, the ramp meters allowed mainlines to accommodate higher flows during the pre-queue transition period than without, resulting in an average 2% increase in traffic volumes. Third, the flow rates for the queue discharge after traffic flow breakdown was an average of 3% higher than without ramp meters. Therefore, ramp meters can be an effective solution for increasing the throughput of freeways with sections of recurrent congestion during peak hours.

2.1.4: Previous Case Study—Minneapolis-St. Paul

One of the most noteworthy examples of testing the effectiveness of ramp metering systems occurred in Minneapolis and St. Paul, Minnesota (Levinson & Zhang, 2006). Political pressure led to the requirement for a “ramp meter holiday” during which the system would be turned off. Data from this off period would be compared to before and after, in order to determine system effectiveness. MNDOT began implementing ramp metering strategies in the metropolitan area in the 1970s, slowly expanding the system over the years. After the ramp meter holiday, data from the experiment was analyzed according to seven performance measures. These were mobility, equity, productivity, consumer surplus, accessibility, travel time variation, and travel demand response. The main determination of the study was that while ramp metering systems were beneficial to mainline traffic, vehicles on ramps could be subjected to long queue times, impacting performance for these users. Levinson and Zhang call for a ramp meter control algorithm that also optimizes delay for queued vehicles, resulting in greater system equity.

The eight-week ramp meter holiday took place during the fall of 2000 (Cambridge Systematics, Inc., 2001). During this period, the ramp meter signal infrastructure was set to flashing yellow mode. This is consistent with system operation during off-peak hours. Observations during the trial period were compared to system performance prior to the meters being turned off. According to the data, there was a nine percent average traffic volume reduction and a fourteen percent peak traffic volume reduction on freeways during this period. However, traffic volumes on parallel arterials during the same period did not change. The study also found that the decrease in ramp delays was not sufficient to offset the additional delay on the mainline facilities. The ramp meters were found to result in an annual system wide savings of over 25 thousand hours. The elimination of ramp metering was also found to halve the travel time reliability of the system, resulting in 2.6 million additional hours of unexpected annual delay.

Switching off the system also resulted in an increase in peak period crashes by 26 percent. In the Twin City area, the ramp meters are responsible for an annual savings of over 1000 crashes. The ramp metering system was also found to be responsible for saving over 1000 tons of emissions and 5.5 million gallons of fuel per year. The calculated savings of all of these effects totaled $40 million per year, approximately 15 times greater than the cost of the system, and making it the highest performing component of the area's congestion management system.

2.2: Merge Control State of Practice

While ramp metering strategies focus on limiting the influx of vehicles onto a highway in order to improve traffic conditions due to new mergers, other active control strategies have been developed to address safety and mobility issues at other types of merger zones (Pesti, Wiles, Chu, Songchitruksa, Shelton, & Cooner, 2008). Several different dynamic merge control strategies are being investigated to determine their effectiveness at merger points due to temporary lane closures for roadwork. The merger situations that arise at lane closures are different from those that are seen at freeway onramp weaving sections. In situations of lane closure, traffic has slowed and motorists exhibit queuing behavior as they approach the merge point. Mergers at onramps, on the other hand, occur at higher speeds. The merge control strategies developed for lane closure situations attempt to regulate two types of merging behaviors: early mergers and late mergers. Aggressive drivers will take advantage of the less congested closed lane to pass as many mainline vehicles as they can until the latest possible merging opportunity. An excess number of late mergers poses safety issues near the merge point, and can increase the risk of collision due to unexpected merging behavior. Problems also emerge when too many drivers exhibit conservative behavior and attempt to merge into open lanes as early as possible. This results in the remaining capacity of the closed lane being underutilized. While this early merger behavior can help to reduce the demand for lane changes close to the merging point, an excessive number of early mergers ineffectively uses lane capacity. Dynamic merge control strategies influence driver decisions to reach a balance between early and late merging behaviors.

In low traffic flow conditions, early merging behaviors can reduce the likelihood of traffic flow breakdown due to merging conflicts caused by high speed merging. Conversely, situations with high traffic flow and low speeds are optimal for a higher proportion of late merging behavior. Early test systems for merge control implementation take advantage of these tradeoffs. Merge control installations typically include detectors of both speed and volume along the mainline and the terminating lane (Pesti, Wiles, Chu, Songchitruksa, Shelton, & Cooner, 2008). A variable message sign is used to regulate merging vehicle behavior. One such system was developed by the Michigan Department of Transportation, and is called the Dynamic Early Lane Merge Traffic Control System (DELMTCS). The system is deployed at merger zones for temporary roadwork closures. It uses dynamically changing "no-passing zones," which attempt to minimize late lane mergers and aggressive behavior. In addition, it minimizes delay experienced in the tapering road section. A second family of dynamic merge strategies encourages late mergers, directing motorists to wait until the lane terminates to merge with the mainline. These directions are supplemented by instructions for drivers to "Take Turns" at the merge point. Such systems are employed by PennDOT, MnDOT, and MDOT.

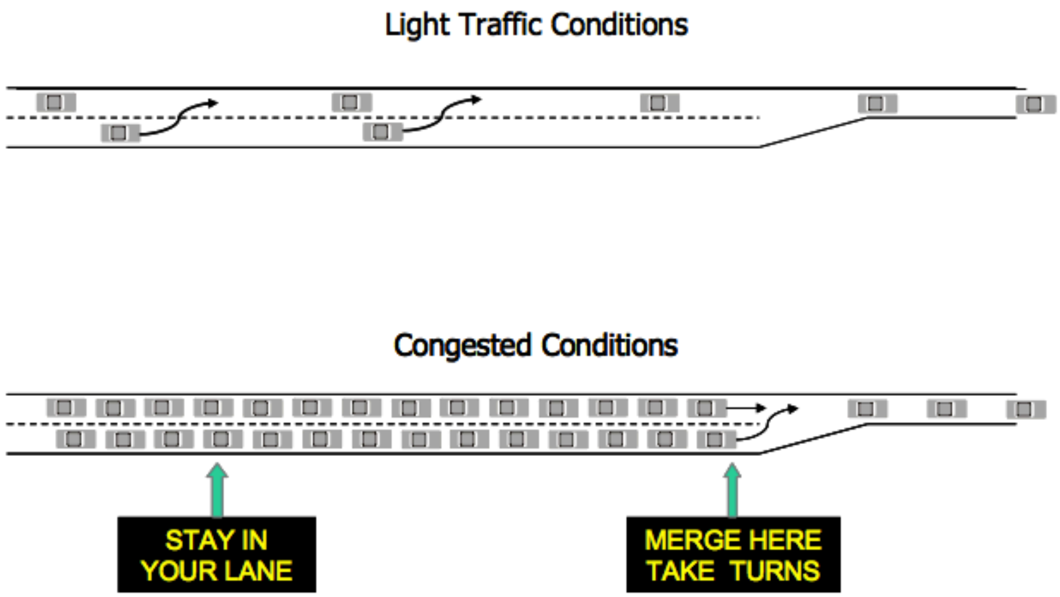

Figure 2: Dynamic Merge (Pesti et al, 2008)

Figure 2 shows an example schematic of dynamic merge deployment at a constriction due to road construction. The figure displayed is an example of late merge deployment (Pesti, Wiles, Chu, Songchitruksa, Shelton, & Cooner, 2008). Dynamic message signs notify drivers how they should modify their behavior. By having merging maneuvers take place at a predetermined location in a non-random order, a higher throughput can be achieved and the total delay for the system can be reduced. The system deployed in the schematic has the potential to change between late merge and early merge operations simply by changing the DMS controllers.

Early merge strategies at work site constrictions work well under light traffic conditions. This is because a lower vehicular density allows drivers to more easily find gaps in traffic for merging maneuvers. When the demand along a segment increases and exceeds the capacity of the downstream constriction, however, the congestion causes queuing behavior to emerge. This queuing produces a shockwave in the flow of traffic, which reduces roadway safety by increasing the likelihood of rear-end collisions. This is especially true when the traffic shockwaves caused by bottlenecks propagate far upstream, past the point of visibility of the constriction. If drivers have not yet seen advance warning signs of lane closure due to construction, they may be unprepared to make sudden collision avoidance actions.

Problems at bottleneck sections also emerge when drivers who execute late merging behaviors use unused capacity in the closing lane. When this happens in an uncontrolled scenario, queued drivers in the open lane may become upset by passing cars merging late and avoiding delays. The result is an inequitable distribution of delay for vehicles along the freeway section which is determined solely by driver aggression. When late merging dynamic merge systems are deployed, it encourages all drivers to make use of available lane capacity regardless of an individual driver's aggressive or cautious behaviors. Late merge systems are best deployed during peak hours. Work sites may use DMS systems to switch between late and early merge dynamic merge systems based on time of day and facility demand.

The late and early dynamic merge systems described so far are optimized for unexpected bottleneck conditions, such as those that result from lane closures due to construction. State of practice has so far yielded few active merge control technologies ideal for recurrent bottleneck sections, such as those that result from freeway constriction due to lack of right-of-way or other geometric factors. However, bottleneck segments are common among existing urban highways, particularly in older facilities that have experienced extreme growth in their demand since their original construction. A potential application for dynamic merge control systems exists on freeway sections that pass through or terminate in central business districts. This is because the high number of trip destinations in these areas may result in bottlenecks due to queue spillback from exit ramps.

A dynamic merge control technology that directly deals with merging and weaving zones where a freeway onramp intersects the mainline utilizing lane control technology has been implanted in Europe (Texas Transportation Institute, 2012). This technique, known as junction control, dynamically closes mainline lanes upstream of a merge point and yields an exclusive lane from the mainline to the merging ramp traffic downstream of the merge point. This strategy is illustrated in Figure 3.

Figure 3: Junction Control (Texas Transportation Institute, 2012)

The technology utilizes individual overhead lane control signs indicating to oncoming traffic whether their lane is closed downstream. The highway facility operator can deploy this technology in order to modify access to a facility depending on fluctuating demand. If a freeway facility has a particularly high demand at an onramp compared to the mainline, this technology can be used to give priority to onramp traffic. This can help minimize delay caused by a bottleneck at the onramp, and also prevent queues from propagating through the surface street network. This technology is particularly well suited to deployment at onramps or freeway mergers where there are groups of multiple lanes joining at the merge point. If a freeway onramp has two lanes, the innermost lane can be closed during periods of low demand, effectively making it a one lane onramp. Under this case, priority would be given to mainline traffic. When ramp demand increases, for instance during the peak commuting hours, priority can be given to ramp traffic. When proper operational procedures are adopted to ensure that the system prioritizes the upstream section with the highest level of demand, the resulting traffic behavior can reduce mean travel times and increase mean speeds across both trunk links. This lane control technology also helps to minimize the number of collisions due to merging maneuvers, because the merging traffic has a dedicated downstream lane during peak periods. By minimizing the number of potential conflicts, the number of accidents can be reduced. Some of the hurtles needed to be overcome before widespread adoption of these practices include driver education. Presently, motorists in the US are unfamiliar with active control on freeway facility mainlines, and may resist a perceived signalization of freeway segments.

2.3: Active Merge Control—Gap Metering System Design

While ramp metering strategies focus on limiting the influx of vehicles onto a highway in order to improve traffic conditions due to new mergers, other active control strategies have been developed to address safety and mobility issues at other types of merger zones. Several different dynamic merge control strategies are being investigated to determine their effectiveness at merger points due to temporary lane closures for roadwork. This study evaluates a new kind of active merge control technology called Gap Metering.

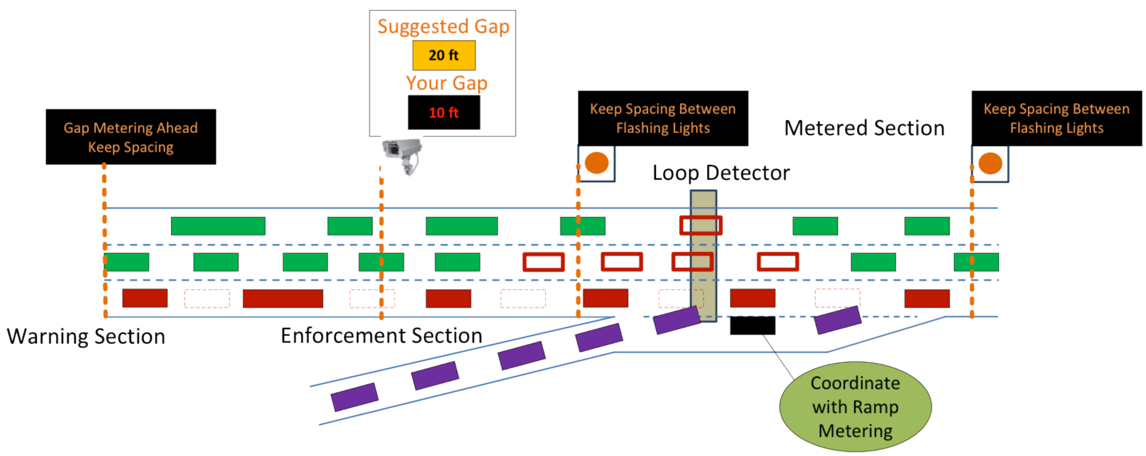

Figure 4: Gap Metering Reference Schematic (Jin, 2012)

Gap metering works by influencing mainline drivers to modify their behavior in order to smooth merging activities on a freeway (Jin, 2012). Upstream of a merging and weaving section, a detector determines the current gap spacing of approaching mainline traffic. Once the number and size of gaps drop below an acceptable threshold, the control system activates. A combination of visual cues and dynamic message signs advises motorists approaching the merging zone to leave a one-vehicle gap in front of their vehicle. A secondary detector can be used to observe and report compliance. The gap provided by the driver should be large enough for a merging vehicle to make a lane change without either the ramp or mainline traffic changing speed. By utilizing this technique to ensure more homogenous merging behavior, additional delays caused by bottlenecking at weaving sections can be controlled.

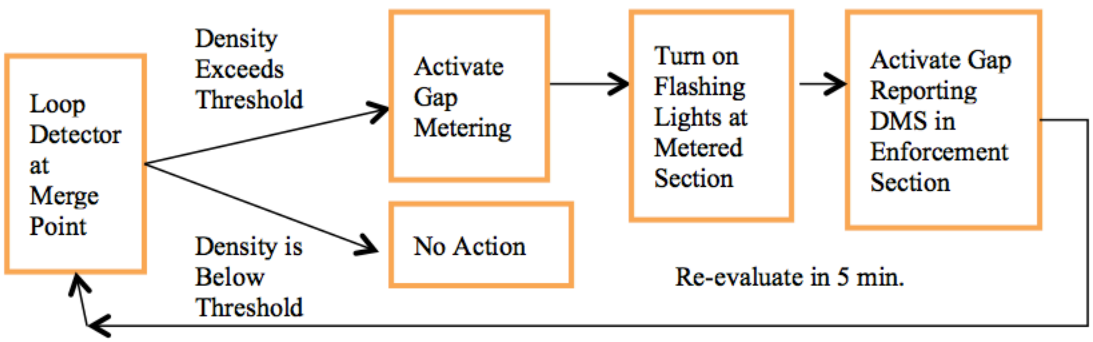

Figure 5: Gap Meter System Operation

As seen in Figure 4, the gap metering system has three major sections. Prior to the merging point, traffic on the mainline approaches a warning section. A dynamic message sign or static sign with flashing indicator notifies drivers that they are entering a freeway segment with gap metering in place. Drivers are made aware that in addition to their driving behavior being subject to speed control through the speed limit, their car-following behavior will also be regulated. This will cause drivers to pay special attention to the spacing gap they are leaving between themselves and the vehicle ahead of them. Next, drivers enter the enforcement section. Here, a detection unit measures the spacing between vehicles traveling on the mainline. One potential method for enforcement is to use a dynamic messaging sign to report to drivers their spacing. This could work in a similar way to DMS systems that report vehicular speeds alongside a posted speed limit. By giving drivers feedback, they will be influenced to adjust their spacing. A static or dynamic sign will display the recommended vehicular gap at that location. This gap will be determined by the section's geometry and detected speed. It may be either dynamic or static.

Finally, drivers will enter the metered section. The beginning and end of this section will be clearly marked. One way to mark this section is with flashing lights with static signs. Signage will indicate to drivers that they should not change the spacing with the vehicle ahead of them. This metered section will begin before the merge point, and will end after the merging and weaving zone has been passed. By ensuring that drivers keep their spacing in this section, after they had adjusted the available gaps in the previous section, the system will provide gaps for merging traffic. A loop detector at the merge point will measure the volume and occupancy of the facility. When the occupancy rises above the point of providing a minimum gap, the system will activate. By providing adequate gaps along the mainline for merging traffic from the ramp, the system will allow ramp traffic to effectively "zipper" onto the mainline. In addition, by ensuring that mainline traffic adjusts its spacing before reaching the merge point, the system allows drivers to react and change their speeds and spacing more gradually. This reduces the chance of traffic flow breakdown occurring, and therefore helps to offset the possibility that a shockwave will propagate upstream.

The gap metering design that Jin proposes includes four major parameters that need to be taken into account for system implementation (Jin, 2012). These are:

- **Lanes Metered**: When implemented at a merge point, a gap metering system can be made to either apply only to the rightmost general-purpose lane, or for all lanes along the mainline approach. Signage on roadside DMS systems as well as overhead gantries can be used in order to indicate which lanes are under metered control. Additionally, a system can be implemented to alternate between no merge control, gap metering on the rightmost lane only, and gap metering across all lanes dynamically, in such a way that different congestion levels will trigger a different system behavior.
- **Gap Size:** Individual implementations of the gap metering concept may vary the size of the yielding space metered mainline vehicles are expected to yield. Already, the direction to drivers to leave a one vehicle gap is open to a wide degree of interpretation of the required spacing for one vehicle to merge in front. This is why the aforementioned feedback system in the enforcement section is important; it aids drivers in adjusting their spacing until it is approximately uniform. Besides a spacing headway method for feedback, a time headway could be alternatively suggested. The system must be able to adjust its spacing requirements in order to reflect different needs for gap acceptance at various locations. If facility

geometry allows for enough acceleration space such that merging vehicles can match the speed of the mainline traffic, uniform spacing requirements can be applied. If the difference in speeds between merging and mainline traffic is significant, however, the system would require a higher gap length in order to allow additional room for merging vehicle acceleration.

- **Yielding Strategy**: Individual implementation of gap metering systems may vary the yielding strategy they advise to mainline drivers. Drivers in metered lanes may chose to adjust their spacing after allowing a single merging vehicle in front, closing the gap and not permitting additional vehicles to merge. Alternatively, the system may advise mainline drivers to readjust their post-merge gap to permit other vehicles to merge onto the mainline. This can be achieved with DMS systems that alternatively instruct drivers to "Allow One Vehicle in Front," or to "Keep One Vehicle Gap" for the length of the merging section.
- **Compliance Rate**: In addition, the rate at which mainline drivers comply to gap metering instructions may vary significantly, causing the system to have different performance impacts based on the proportion of drivers following gap metering instructions. The feedback sign showing drivers their gap alongside the gap distance required by the system is intended to boost driver compliance rates in much the same way that DMS-base vehicular speed feedback signs do. However, if gap metering is not a

legally enforceable control technique, it may be that drivers will have a low compliance rate due to the lack of consequences for disobeying system instructions. Alternatively, license plate readers may be installed in the enforcement section to make note of vehicles complying or ignoring system instructions. If penalties can not be applied to drivers who consistently ignore gap metering system instructions, then perhaps an adequate motivator would be the application of a small credit to the tolls of vehicles with high compliance rates.

Enforcement of the gap metering concept can be difficult, and it faces several hurtles. First, while DOT agencies have the authority to regulate speed, the enforcement of a spacing requirement such as the one employed in gap metering is unprecedented. It is possible that new legislation would need to be passed before such a system could be implemented, granting DOTs this authority. It is worth noting that gap metering falls into a category of active traffic management previously unexplored. There are three main quantitative descriptors that can be applied to traffic flow on a freeway facility. They are speed, volume, and density. Other ATM systems such as variable speed limits and queue warning systems help to actively regulate traffic speed. Ramp metering systems control the volume of traffic accessing a freeway facility, placing an upper bound on the number of vehicles allowed entering. Gap metering can be thought of as a way of actively regulating the density of traffic on the freeway along certain segments. Preliminary studies indicate that not all mainline traffic must adhere to the gap metering instructions in order for a benefit to be seen (Jin, 2012). The level of adhering drivers may be as low as 10% to 15%, and a substantial increase in facility performance can still be identified.

Chapter 3: Evaluation Methodology

3.1: Experimental Design

3.1.1: Study Site

The northbound section under study of I-35 has a straight alignment for the majority of its length, and curves westward right before the river crossing. Four East-West surface streets cross the facility along this section. Starting from the south, they are Woodward Street, East Oltorf Street, Woodland Avenue, and East Riverside Drive. There is a parallel frontage road along the entire length of the facility. The mainline section evaluated includes three onramps. One is from an interchange with State Highway 71, one is immediately south of East Oltorf Street, and one is immediately south of Woodland Avenue. Additionally, there are four off ramps along the studied section. There is one immediately north of Woodward Street, one north of East Oltorf Sreet, one north of Woodland Avenue, and one north of East Riverside Drive.

This section has been of particular interest for evaluating potential congestion relief systems because it represents a significant bottleneck on the approach to the Austin central business district. Congestion in Austin is a persistent problem that results in costly and time consuming delays for the city's commuters. The Capitol Area Metropolitan Planning Organization has published data showing that the AADT along I-35 along the evaluation section was as high as 177 thousand vehicles per day in 2009 (CAMPO, 2009). In the same year, the Urban Mobility Report ranked Austin as 15th in the nation for most congestion delay (Lomax, 2010 Urban Mobility Report, 2011). Previously, master's candidates Lily Aung and Jonathan Martk used the same evaluation section of 1-35 to determine the impacts of ATM systems such as a queue warning system and a variable speed limit system (Aung, 2011) (Markt, 2011).

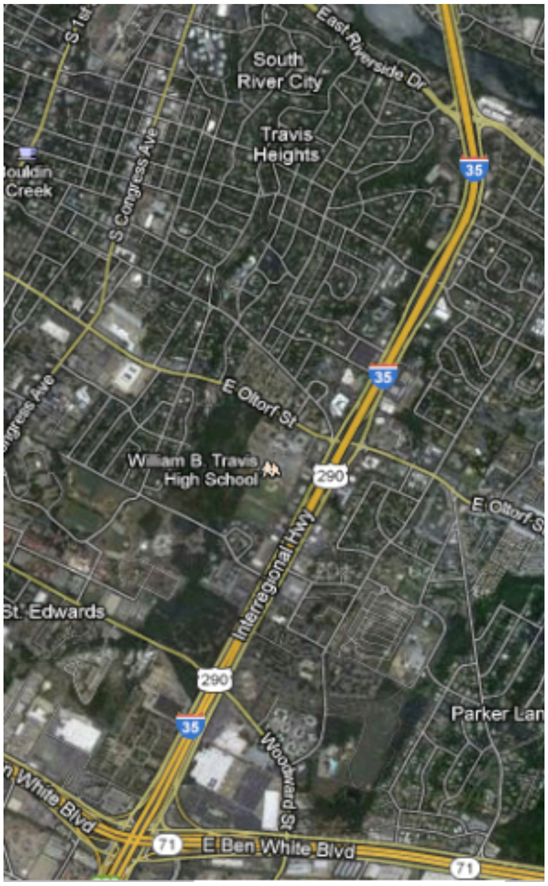

Figure 6: Section Satellite View (Courtesy Google Maps)

During the construction of the VISSIM model, individual network links were overlaid onto a satellite photo to ensure geometric accuracy. The positions of existing signal heads were replicated, and signal time plans were recorded on-site and input into the model. Because there is a lack of installed sensors along the length of the facility, traffic flow data was collected by video. During a single morning peak commuting period, 2 hours of footage of the facility's mainline and each of the onramps, off ramps, and turning movements at adjacent intersections were recorded. Once processed, this data provided route splits for each of the decision points in the simulation network.

3.1.2: Data Collection Effort

Although a high level of interest exists for the examination of the I-35 facility between State Highway 71 and East Riverside Drive, there is a severe lack of accurate data characterizing traffic flow along this section. While the installation of a modern ramp control system or merge control system along this section of the facility would require a significant investment in detector technology, preliminary evaluation of these systems also requires the wealth of data that such detectors would provide. Due to the lack of detection along this section, an effort was made to record traffic behavior during the morning peak along the northbound route. Special effort was made to ensure that the data collected encapsulated the beginning of the morning peak commuting period, including the transition from free-flow to traffic flow break-down. By using data from this period to evaluate ramp control and merge control technologies, the impact these systems would have on peak period traffic could be ascertained.

In order to obtain accurate data for this section of the I-35 corridor in a cost effective manner, video surveillance was used. By using portable digital camcorders mounted on tripods, operators were able to record the traffic flows. When reviewing the captured footage, virtual detectors were established within the frame. When a vehicle passed over the virtual detector during playback, it's presence was manually recorded.

While this method of video processing does not provide the occupancy data a conventional loop detector would, the volume data recorded can be used at any time period resolution. This is because the processed video data established a timestamp for each vehicle passing through the virtual detector zone.

Table 1 shows a list of each of the 11 video camera locations used during the data collection effort. All of the camera locations had recorders active during the same time period in order to relay an accurate profile of the facility's traffic demand.

Camera No.	Location
1	Woodward Off-ramp
2	Woodward On-Ramp
3	Woodward Frontage
4	Oltorf On-Ramp
5	Oltorf Off-Ramp
6	Oltorf Frontage
7	Woodland On-Ramp
8	Woodland Off-Ramp
9	Woodland Frontage
10	Riverside Off-Ramp
11	Riverside Frontage

Table 1: Video Detector Camera Location

The recordings were made on Wednesday, April 11, 2012. The camera operators coordinated the start time for recording footage, and video footage was captured from 7:00 AM to 9:00 AM. In addition to observing traffic flows on the mainline of the freeway's northbound section, the cameras also captured turning movements at major intersections along the frontage road. The data acquired from this effort aids in several ways. First, it provides information about the existing traffic volumes seen on the corridor, including an indication of the shape of the peak period's demand. Second, the data recorded during this period indicates the amount of demand placed on each onramp to the facility mainline. The data also indicates the amount of traffic diverting from the mainline to the frontage road. This is of particular interest, because preliminary examination of the facility indicated that while a heavy amount of traffic demand is seen on the freeway mainline, the signalized frontage road is under capacity. Because the frontage road extends for the length of the I-35 corridor through the city of Austin, diverting traffic from the mainline could help relieve the bottleneck conditions seen as the corridor approaches the central business district. Importantly, the frontage road maintains three lanes for most of its length along this section, and the geographic bottleneck of the river crossing actually features a dedicated four-lane bridge for the frontage road. Active traffic control strategies such as ramp metering and merge control could help to modify driver behavior to more fully utilize this capacity. Figure 7 shows the extent of the data collection effort with the locations of each camera station. Figures 8 and 9 show the camera locations in detail, with field-of-vision and virtual detector locations.

Figure 7: Video Recording Camera Locations

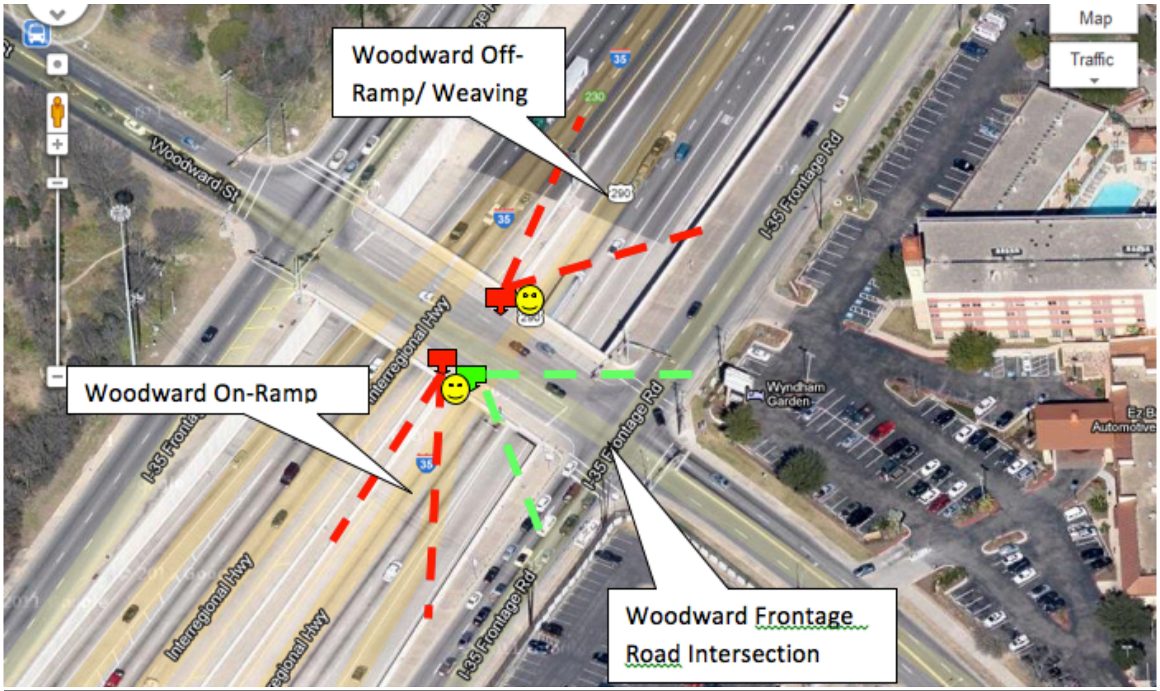

Figure 8: Woodward Street Camera Detail

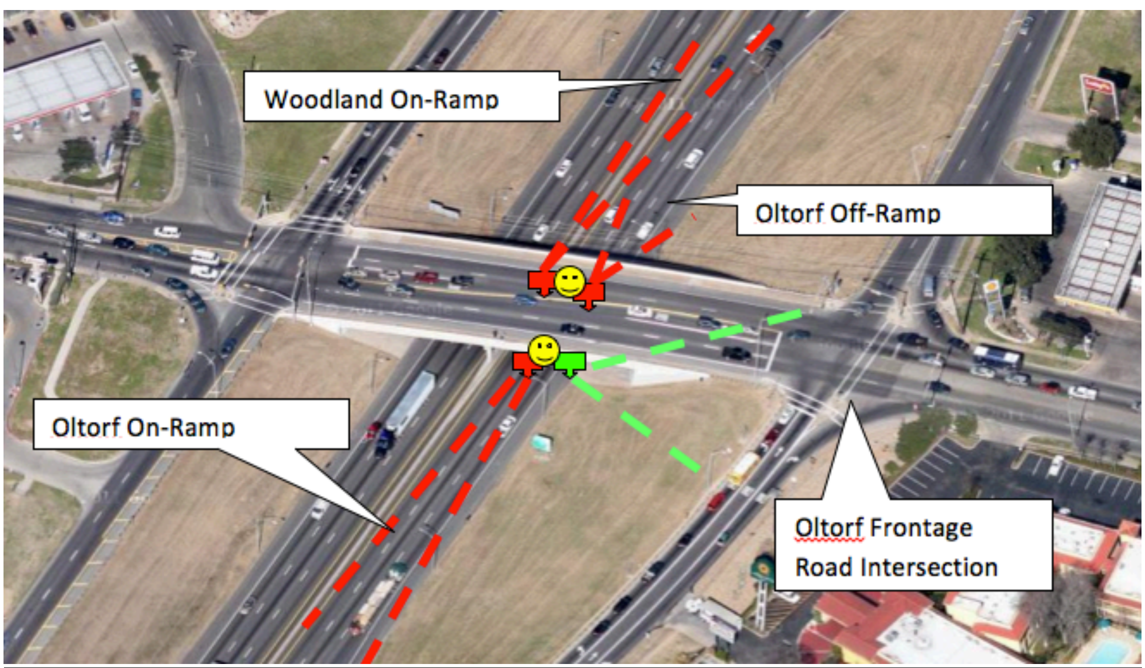

Figure 9: Oltorf Street Camera Detail

The data collection exercise revealed the current traffic patterns during the morning peak along the northbound section of I-35. From this data, the volumes along the mainline as well as on the freeway onramps and off-ramps were determined. Figures 10-12 show the observed mainline volumes, onramp volumes, and off ramp volumes as the morning peak period progressed. The decline in mainline flow starting at about 7:20 AM indicates the beginning of traffic flow breakdown. At this point, the high demand placed on the facility in combination with the bottleneck section at the northern boundary of the section stresses the freeway's capacity and pushes it into a F level of service.

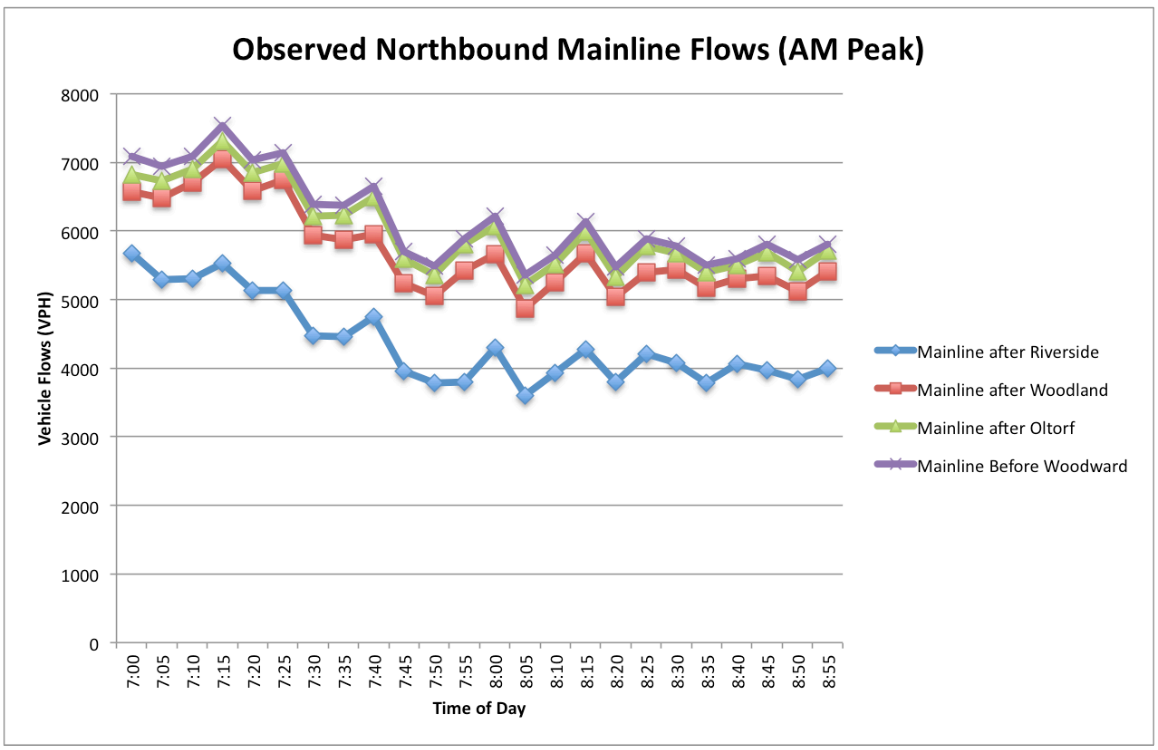

Figure 10: Observed Northbound Mainline Flows (AM Peak)

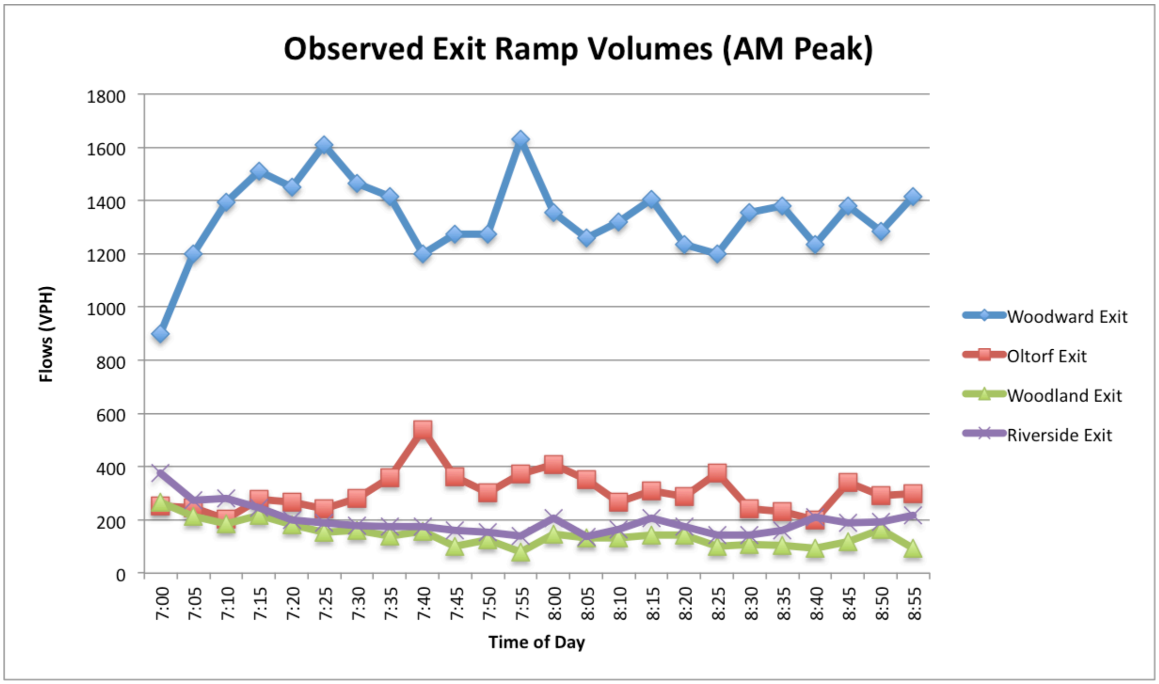

Figure 11: Observed Exit Ramp Volumes (AM Peak)

Figure 11 shows the observed exit ramp volumes for the four exit locations along the evaluation section of the freeway. All of these exits merge with the facility's signalized frontage road. As clearly seen in the data, the three southernmost exits along the northbound section demonstrate relatively consistent demand volumes throughout the peak period. These are the three exit ramps located along the northernmost portion of the evaluation section as it approaches the central business district. The highest demand of this northern group of ramps is seen at the East Oltorf Street exit, which serves as a major east-west collector for the surrounding neighborhood. The East Riverside Drive exit serves a large east-west arterial servicing the portion of the city south of the river. The highest off-ramp demand seen along the evaluation section is at the Woodward Street exit. This likely indicates that drivers exiting at this point do not have a destination in the central business district, and are moving from the freeway before reaching the bottleneck section. This is the first exit for the northbound section of I-35 after it crosses State Highway 71.

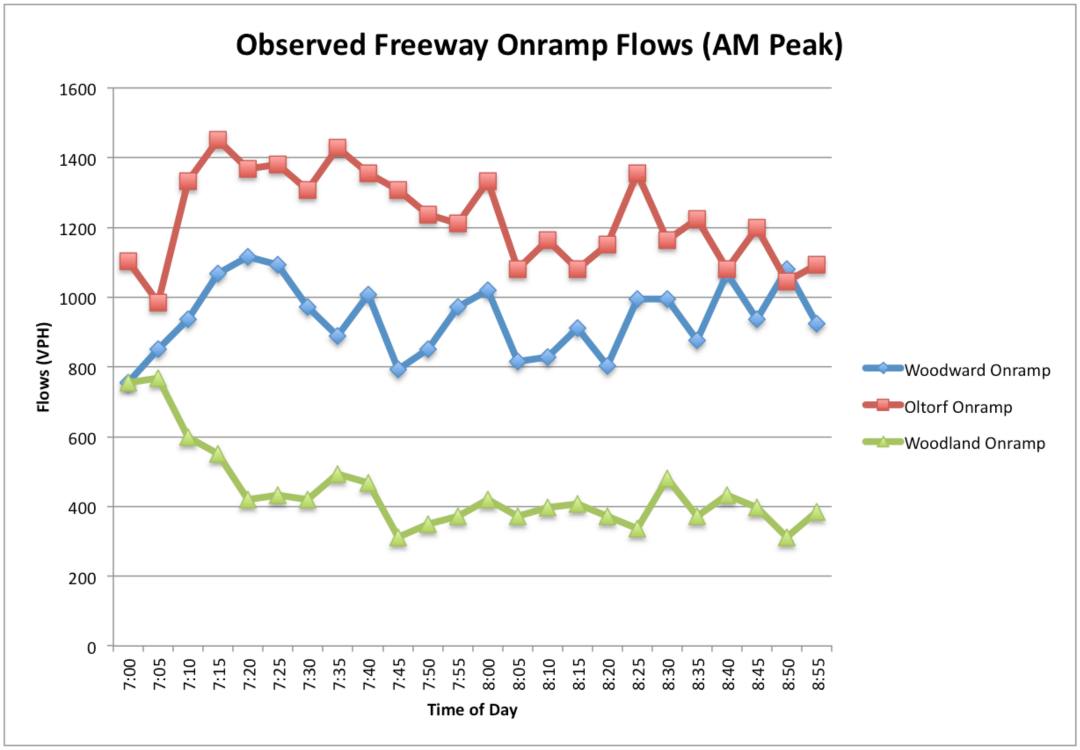

Figure 12: Observed Freeway Onramp Flows (AM Peak)

Figure 12 indicates the level of demand placed on the onramps and exit ramps for the northbound section of the facility. During the observation period, the highest level of demand from oncoming vehicles accessing the freeway came from the onramp from State Highway 71 at Woodward Street and the onramp preceding East Oltorf Street. The level of demand these ramps see is consistent throughout the morning peak period, and ranges between approximately 800 and 1200 vehicles per hour. Because the existing TXDOT design criteria for ramp metering establishes 800 vehicles per hour as an ideal ramp candidate for single lane, single car per green ramp metering, it was determined that the implementation of ramp meters at these locations would be beneficial. During the observed peak hour period, the ramp demand at the Woodland Avenue onramp was approximately 800 vehicles per hour. However, as the peak commute period progressed, this volume declined due to traffic flow breakdown along the freeway mainline. Because of the high level of demand during ideal traffic flow conditions, it was determined that the Woodland Avenue onramp would also make a good candidate for ramp meter evaluation. By helping to regulate the demand levels of oncoming vehicles onto the freeway facility, it is possible that a delay in traffic flow breakdown could be achieved. This would allow the Woodland Avenue onramp to process a higher number of vehicles during the early part of the peak commuting period. Because of the relative onramp demands observed at these three locations, they were all selected for evaluation of ramp metering and merge control systems in microsimulation.

3.1.3: Model Implementation of Ramp Metering and Merge Control

In order to evaluate the efficacy of ramp metering and gap metering along the section of I-35, both of these control strategies were introduced into the base model in VISSIM. Ramp metering and gap metering were implemented at each of the three onramps on the section. They were implemented in the model in such a way that ramp metering and gap metering could each be evaluated alone and in combination.

Ramp meter implementation into the VISSIM model was achieved by using standard program elements. At each onramp, a signal head was implemented. Immediately upstream of the signal head, a detector was placed to determine the presence of a queue. Downstream, in the merging/weaving zone, a detector was placed in each of the two rightmost mainline lanes. These detectors serve to determine the level of traffic on the mainline, informing the system when ramp metering should be implemented. After passing a certain traffic density threshold across these detectors, the ramp metering system would be turned on. The signal head allows one car per green during this interval. Adequate spacing was ensured for both queue storage lengths and acceleration zones. Figures 13, 14, and 15 show each of the ramp metering systems as implemented in the VISSIM model.

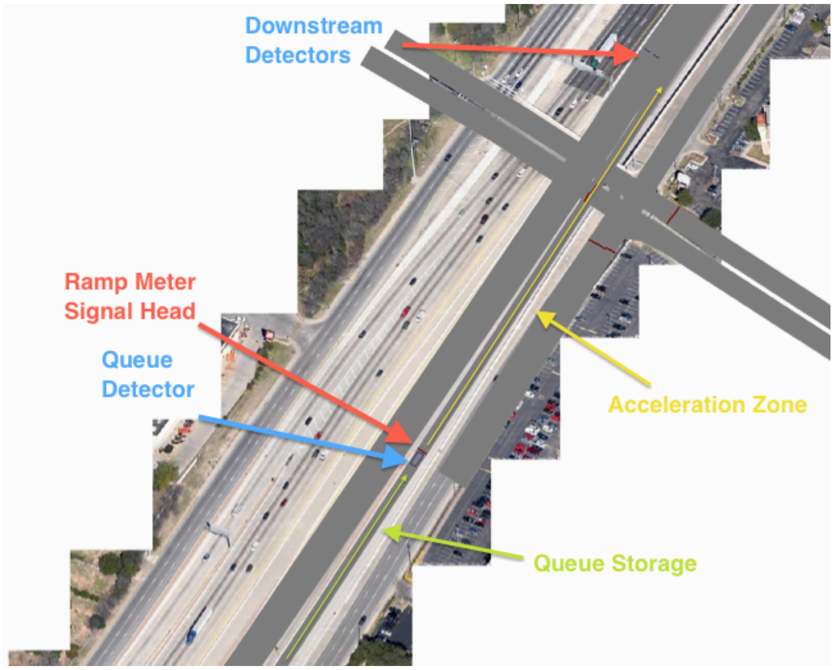

Figure 13: State Highway 71 Ramp Meter Schematic

Of the three onramps along the northbound evaluation section of the freeway, the onramp from State Highway 71 required the least modification for the implementation of a ramp metering system. More than 300 ft. of queue storage space is available, which is more than adequate than the demand level indicated in the data collection exercise. After the ramp meter signal head, 720 ft. of acceleration space is provided. Upstream of the merge point, the mainline has four lanes. Four mainline lanes are available from the merge point until the freeway diverges at the next off-ramp. This provides a 600 ft. long merging and weaving section.

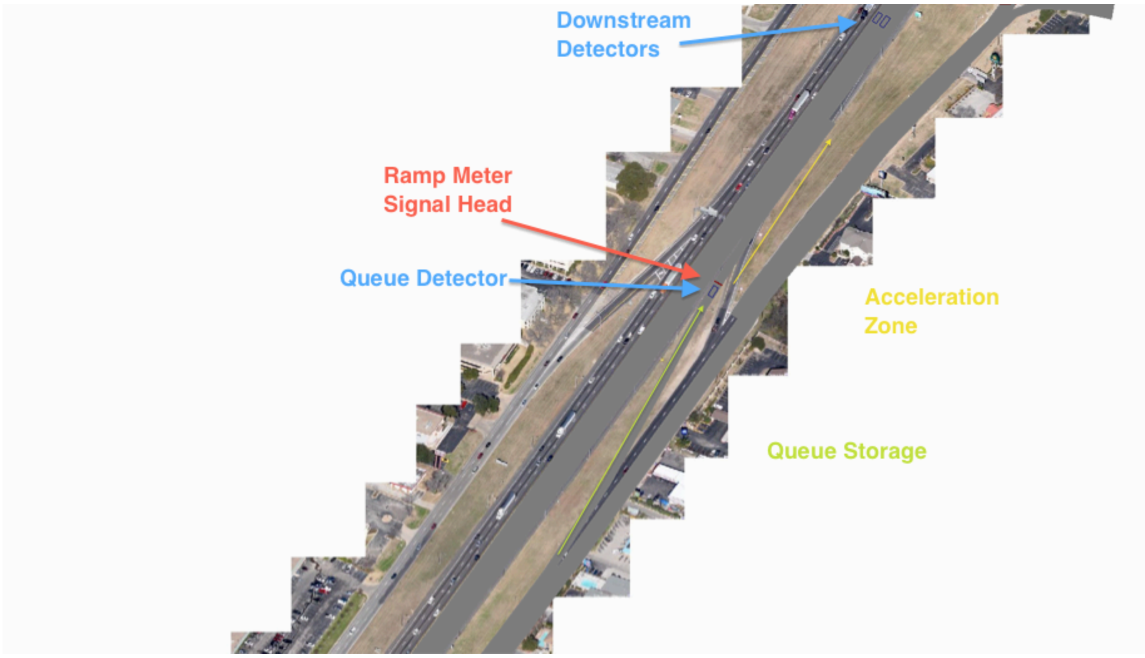

Figure 14: East Oltorf Street Ramp Meter Schematic

The next onramp is immediately before the service road intersects East Oltorf Street. During the data collection exercise, this ramp saw the highest level of demand. Geometric reconfiguration of the ramp and service road was required to provide adequate storage space. While the existing freeway geometry provides less than 200 ft. of queue storage, which would result in spillover onto the frontage road, the modified geometry has 700ft. of queue storage available. After the ramp meter signal head, 370 ft. of acceleration space is provided. Upstream of the merge point, the mainline has three lanes. The mainline keeps this width downstream of the merge point. The short merging and weaving section provided makes an ideal candidate location to evaluate the effectiveness of the gap metering concept.

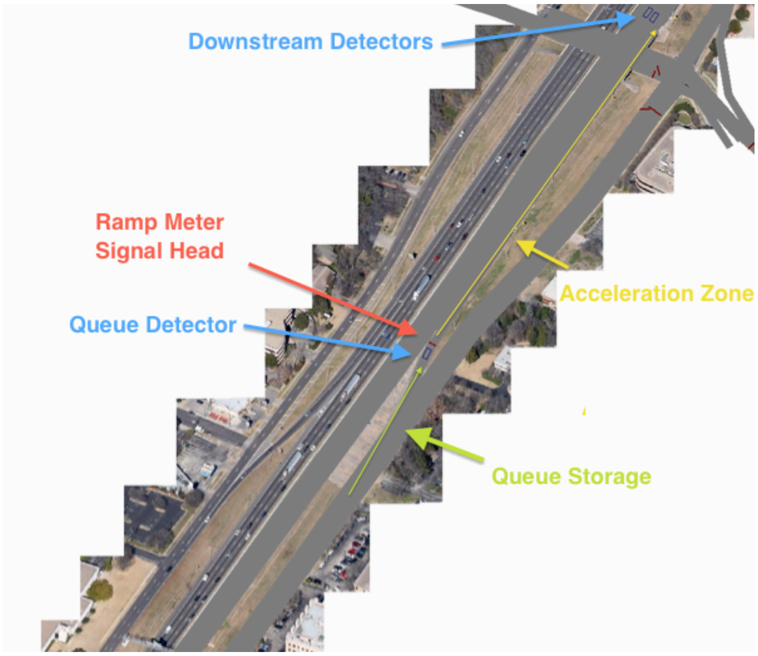

Figure 15: Woodward Avenue Ramp Meter Schematic

At the onramp immediately before Woodward Avenue, no major geometric changes to the network were needed. The frontage road at this location is three lanes wide, with a dedicated lane on the left that feeds the freeway onramp. Within the VISSIM network, link modifications were made in order to restrict lane changing behavior along this section. These modifications effectively extend the ramp onto the service road for a short distance, mimicking a striped median preventing lane changes in the real world. This was done in order to increase the available queue storage length to 400 ft. After the ramp meter signal head, 450 ft. is available as an acceleration zone. Finally, after the merge point, a fourth lane is provided in a weaving section that is longer than 1100 ft.

Because this location is the onramp closest to the central business district and bottleneck section in the north, significant queue spillover is seen during the morning peak.

In order to meet the requirements for ramp metering systems, several modifications were made to the link properties in the VISSIM model. The changes made to the network should not be considered final designs, but preliminary modifications for evaluation purposes only. No changes were made for the ramp meter present at the onramp from State Highway 71, because the large flyover length provides for more than adequate queue storage and acceleration space. At the E. Oltorf Street ramp meter, a geometric change was made to the ramp, making it longer. This provides more space for queue storage and an acceleration zone downstream of the signal head. The link geometry of the Woodward Avenue ramp was not changed, however. Instead, the striping on the frontage road was changed so that the leftmost lane became exclusively queue storage for the ramp metering system immediately upstream of the ramp's location. The ramp meter control algorithm was implemented in the VISSIM API. It utilizes downstream detectors at the merge point to determine whether mainline traffic flow has exceeded a predefined occupancy. After that occupancy level is reached, the metering system turns on. As an evaluation case, single car per green meters were implemented along the evaluation segment of the freeway. The control algorithm utilized for the ramp metering mechanisms in the simulation is a slightly modified version of the one provided by default in the VISSIM package. The code for this program can be found in the appendix. It uses a 4 second cycle time to determine ramp meter signal head activation.

Because gap metering is a novel concept, its implementation into the VISSIM model was achieved through the VISSIM API. A separate vehicle group was implemented in the program. The relative size of this group was 10% of the total vehicle input, reflecting an assumed compliance rate with the gap metering system. Program limitations prohibited an exact model of the active control device described as part of the gap metering system design. Instead, the group of gap metering vehicles were assigned unique behaviors which mimic adherence to gap metering instructions. As these vehicles pass through the merging/weaving section immediately downstream of the merge point, they adjust their minimum acceptable spacing to 25 feet, thereby leaving a gap between following vehicles on the mainline large enough for an acceptable merge. After they pass the end of the weaving area, the special class of vehicles return their minimum acceptable spacing to the default value of 5 feet.

For this study, four different cases were evaluated. A no action case without gap meters or ramp meters functions as a base scenario against which to measure each of the alternatives. In the second scenario, ramp meters were considered on their own. In the third, ramp meters were considered in conjunction with gap meters. In the fourth scenario, gap meters were considered on their own. Each of these four scenarios was considered under both peak-hour and off-peak conditions, resulting in eight simulation types. In the model, peak hour congestion conditions were achieved by lowering the speed vehicles could exit the north end of the model to 12 mph. In the off-peak scenarios, the speed was raised to 50 mph. While this is not a free-flow value, it reflects the bottlenecking behavior seen at the north end of the study area.

3.2: PERFORMANCE MEASURES OF FREEWAY FACILITIES

Several different performance measures were used to evaluate each of the implementation scenarios under consideration. For each of the eight scenarios, the same types of output data were generated. Output presentation included network-level measurements of travel time, delay, speed, and throughput over the course of the simulation, as well as string-level data, which provides average speed, volume, and occupancy in 60-second intervals for each 100 ft. long segment on the network.

3.2.1: Speed

The speed of the vehicles passing through the simulation network is measured in two ways. First, a network-wide file describes the average speed of vehicles taking a particular route for the entire length of that route. This speed data is taken as an average across the duration of the simulation. From this speed, the average travel time for each route across the entire duration of the simulation can be determined. Because 10 simulation runs were conducted for each scenario, mean speeds and travel times can be compared using statistical tests to determine the impact that each of the alternatives has had. In addition, string level data is provided for each of the links on the simulation network. String data displays the average speed across each 100 ft. segment of each link in the network. This average value is given for each 60 seconds of the 2-hour simulation time. By evaluating these speed and travel time values for different routes and at different locations along the evaluation section, the impacts that the ATM strategies under consideration can be determined.

3.2.2: Throughput

In addition, the impacts that the systems under evaluation have upon throughput are important because of the necessity to relieve the bottleneck situation at the north end of the evaluation section. For this evaluation, the traffic volume was analyzed in two ways. First, individual route sections were defined along the simulation network. These routes were the mainline route and the frontage road route. Each of these route volumes reflects the total number of vehicles traveling the entire length of the evaluation segment along this route. For example, the mainline volumes will reflect the total number of vehicles traveling on the mainline that entered at the south end of the network, continued without taking an exit ramp, and left the simulation network at the north end. Similarly, the frontage road throughput volumes reflect the total number of vehicles entering the network at the southern end of the frontage road, and leaving the network at the northern end of the frontage road, without merging onto the freeway mainline or turning onto a different street. The second way volumes were evaluated was by tracking the average volumes for each route as the simulation time progressed. By pinpointing when the facility volume begins to decline, the impact that the proposed active traffic management systems have on delaying the breakdown of traffic flow can be evaluated.

3.2.3: Travel Time Reliability

There are many different ways to measure the impact that congestion has upon the performance of a road network. Travel delays caused by congestion have a marked impact on the behavior of drivers and their ability to anticipate the amount of time their trips will take. An important tool for evaluating the impact of congestion on an urban area is travel time reliability. Transportation providers conventionally quantify the level of congestion on urban routes using measures such as delay, risk of delay, mean speed, vehicle hours traveled, or volume-to-capacity ratios. In particular, volume-to-capacity ratios (typically expressed as a Level Of Service) compare the number of vehicles using a facility with the number of vehicles it was designed to accommodate. While these measures reflect the roadway's overall performance, it does not take into account the experience of individual drivers. In order for transportation providers to measure the impacts of congestion mitigations strategies from the perspective of highway users, travel time reliability must be quantified (Bureau of Transportation Statistics, 2003). Using travel time reliability statistics, transportation providers can better communicate the needs for investments in transportation projects. In addition, by disseminating information about the travel time reliability of a route to the public, DOTs can better inform travelers about the best options for their transportation needs.

Travel time reliability is defined as the consistency or dependability of travel times. Measures of travel time reliability can quantify the variability of travel times along a route measured either on a day-to-day basis or across different times of the day. The

FHWA defines four different ways of measuring travel time reliability (FHWA, 2005). All of these methods attempt to relate the longer travel times experienced by users to the average travel time. Each of these indexes communicates the variability of travel time in a different way.

The first measure of travel time reliability is the 95th percentile travel time. This is measured along a specific route. This value gives a rough idea to commuters of the longest travel time they can reasonably expect to experience along a given route. Because the 95th percentile travel time is specific to a route, it is not useful in comparing different routes or for evaluating the reliability of the entire network. The 95th percentile travel time is also known as the planning time, because commuters can plan to arrive at their destination within the planning time 19 out of 20 times. The second measure of travel time reliability is the planning time index. The planning time index is a relation between the planning time and the free flow time along the route. The planning time index is defined as:

$$Planning\ time\ index = \frac{95^{th}\ percentile\ travel\ time}{freeflow\ travel\ time}\ (eqn. 2)$$

The third measure is the buffer index, which relates the buffer time to the mean travel time. Buffer time is defined as the difference between the 95th percentile travel time and the mean travel time. The buffer index is defined as:

$$Buffer\ Index\ (\%) = \frac{95^{th}\ percentile\ travel\ time - average\ travel\ time}{average\ travel\ time}\ (eqn. 3)$$

The planning time index and the buffer index can be used to relate the travel time reliability of different routes or of the same route under different conditions. This is because the two indexes take into account the best case and typical travel times (Lomax, Schrank, & Turner, SELECTING TRAVEL RELIABILITY MEASURES, 2003). The fourth way of measuring travel time reliability is to define a threshold of either travel time or speed and report the proportion of times that the conditions along a route exceeds that threshold. This value is difficult to use when relating the reliability of travel time along different routes or for the same route under different conditions. Rather, it is best used to communicate to users the reliability of a single route.

The impacts that the implementation of intelligent transportation systems on congestion can be measured using travel time reliability. The use of travel time reliability measures instead of the traditional level of service indicators was proposed by Chen et al. (Chen, Skabardonis, & Varaiya, 2003). This study suggested that the implementation of an Advanced Traveler Information System along a corridor could report unexpected delays along a corridor, therefore attracting drivers to other routes and improving reliability. Lyman and Bertini assert that travel time reliability measures are underused by transportation planning, and should be used as standard indexes of assessing congestion along a corridor (Lyman & Bertini, 2008). Conventional vehicle-capacity measures of congestion will reveal different priorities for improvement than travel time reliability measures. Additionally, travel time reliability measures are ideal for evaluating the impact that operational strategies have on congestion, including operational changes of roadway rules due to the implementation of active traffic management technologies.

Chapter 4: Results and Analysis

In order to evaluate the impact that each of the active traffic management strategies had on the performance of the freeway segment, several different measurement criteria were used. Special attention was given to the impacts the systems under consideration had on vehicular speed, traffic volumes, and travel times. Because the northbound I-35 corridor features both mainline and frontage road sections, a major point of consideration was whether the installation of ramp or gap metering systems would have an adverse impact on the performance of the frontage road. Because of the limited capacity at the north end of the bottleneck section, a disproportionately negative impact on frontage road performance could bring down overall corridor performance even as the mainline improves. Therefore, the same measurement techniques were used on both the mainline and frontage road in order to determine each system's relative impact on each. As noted before, four different ATM scenarios were considered for both peak period and off-peak period traffic. For each scenario, ten random-seed runs were performed in VISSIM. The data shown below represents the average for each of those 10 runs.

4.1: Vehicular Speed

The two graphs below show the average speed of vehicles along the mainline during the peak hour for each of the different cases evaluated. As seen, the average vehicular speed declines as the peak commuting period begins, starting at approximately 7:10 AM. This rapid slowdown in average vehicular speed along the length of the evaluation segment is indicative of the shockwave from the bottleneck section at the north end of the segment propagating upstream. The Figure 16 compares average vehicular speed on the mainline with and without the implementation of a ramp metering system. Mainline speeds when the ramp metering system is in place are slightly higher as a result of the lower level of vehicular interaction at merge points due to lower ramp demand.

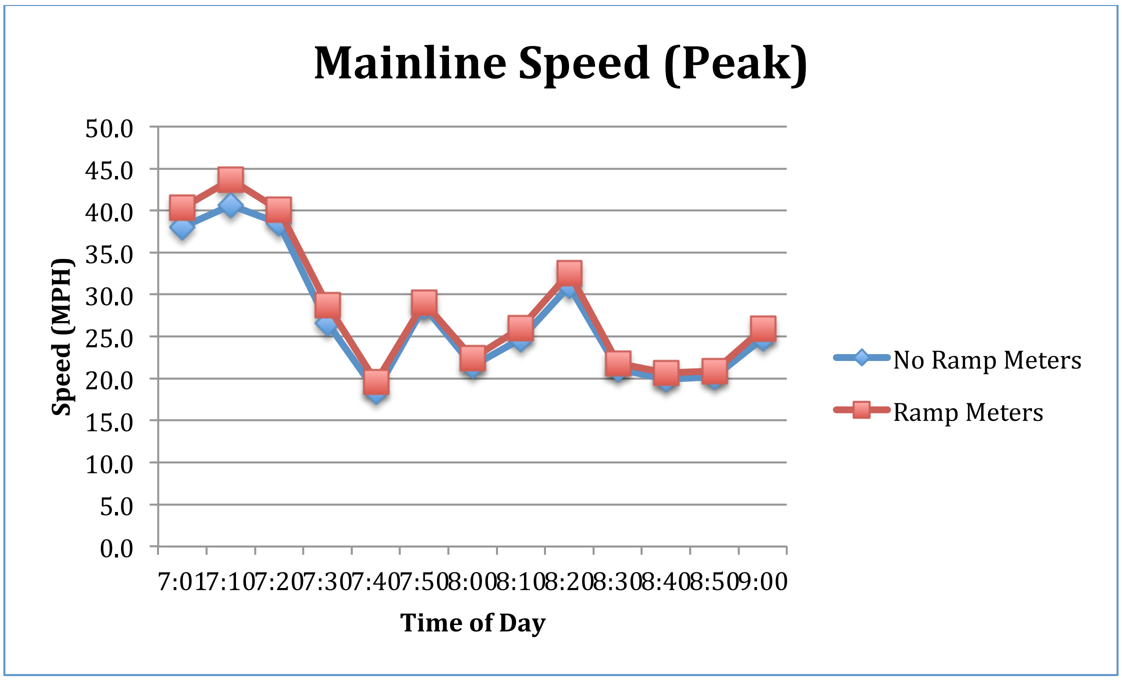

Figure 16: Average Vehicular Speed Ramp Meter Vs. Base Case, Peak Period

Figure 17 shows the average vehicular speed along the mainline for the base case, the scenario with both ramp meter and gap meter implementation, and the scenario with gap meter implementation only. As shown, the average speed also declines early in the peak period due to shockwave propagation upstream. However, it is observed that the gap metering systems have a positive impact on vehicular speed later in the peak period, leading to an earlier recovery of traffic flow.

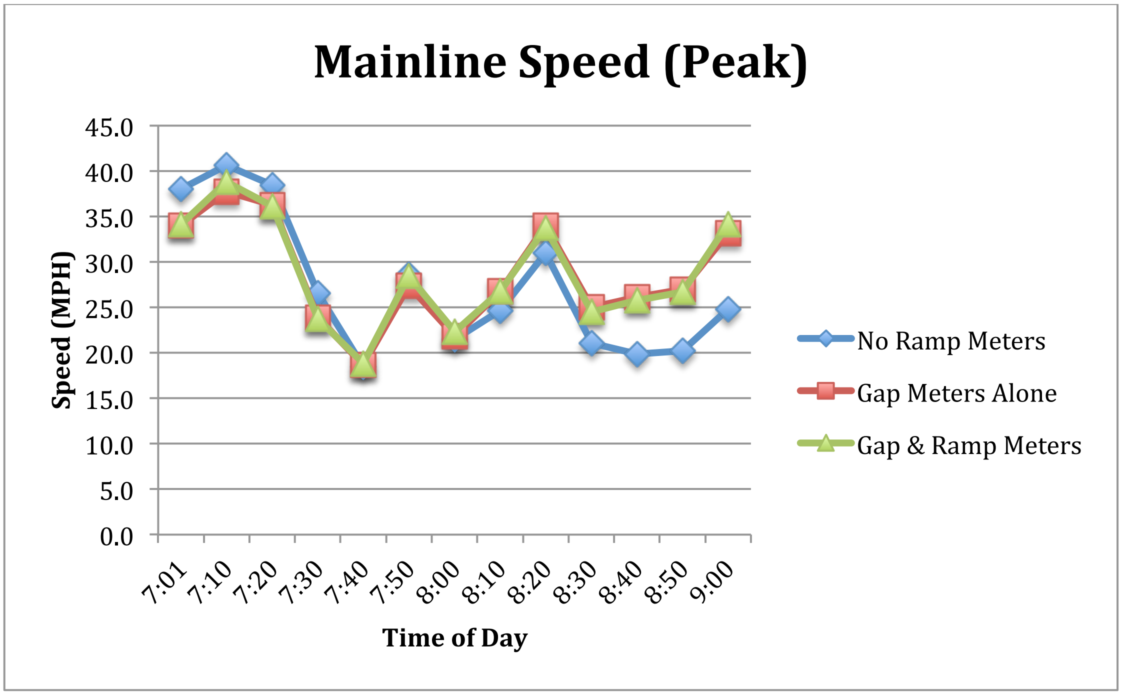

Figure 17: Average Vehicular Speed for Gap Metering Cases vs. Base Case

Figures 18 and 19 below show the impacts each of the implementation scenarios under consideration has on the average speed along the frontage road. Because the frontage road is signalized, much of the delay seen along this route is due to stoppages and queuing at signal heads. The average speed along the frontage road is virtually unchanged between the base and ramp metering only cases. For the gap metering cases, a slight improvement is seen in the frontage road vehicular speed during the latter half of the peak period. This could be due to a lower amount of delay due to queue spillover from the ramps. Because the gap metering system allows for merging traffic to be processed more efficiently, fewer stoppages occur due to drivers being unable to find an acceptable gap. As flow along the mainline recovers, this positive effect is magnified.

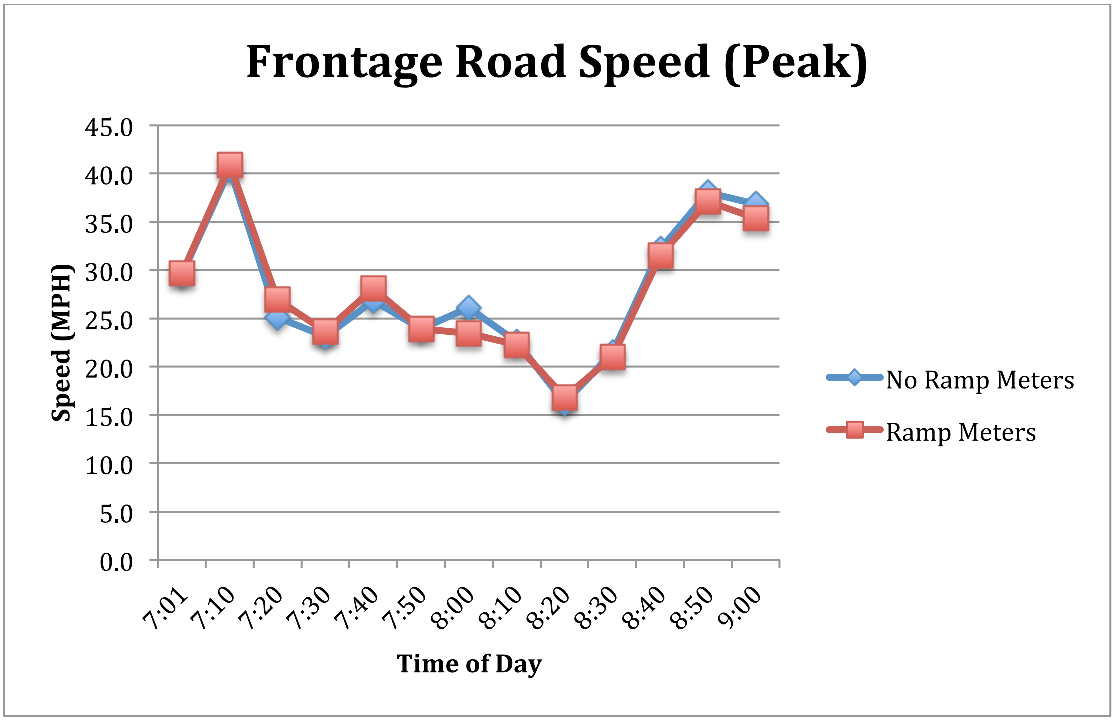

Figure 18: Average Vehicular Speed for Ramp Metering Case vs. Base Case

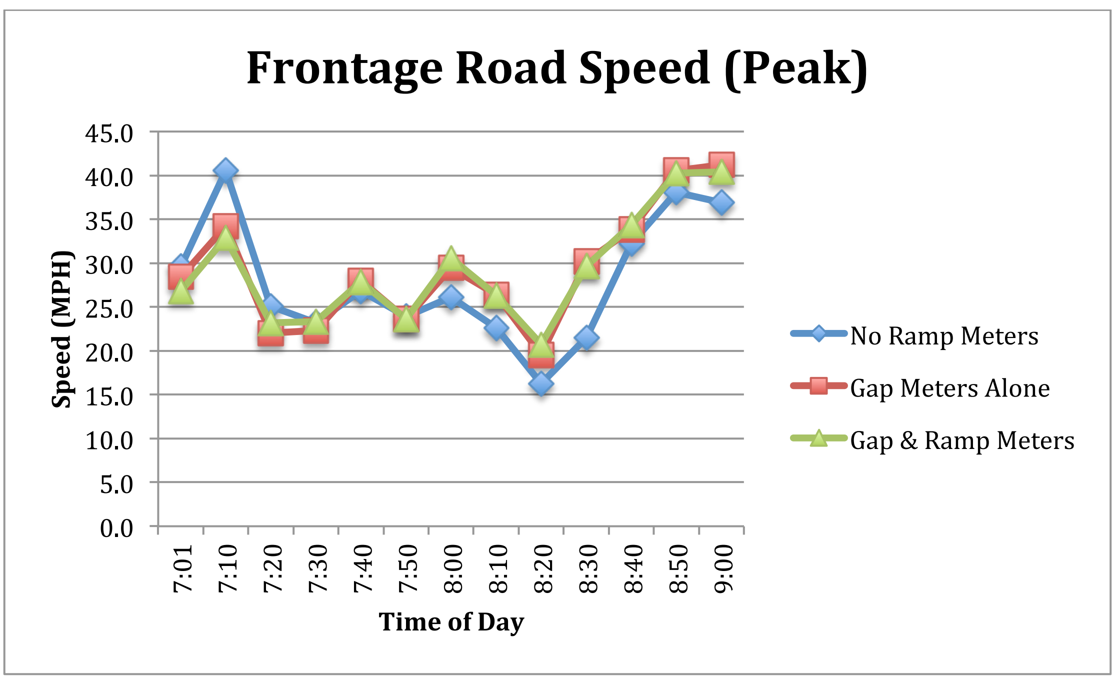

Figure 19: Average Vehicular Speed for Gap Metering Cases vs. Base Case

Figures 20 through 23 below show the impacts that each of the systems under consideration have on the northbound segment of I-35 during mid-day off-peak hours. This condition was simulated by using the same volumetric inputs to the simulation network and lifting the bottleneck constraints at the north end of the evaluations segment. The implementation of a ramp metering system does not have a large impact on the average speed of the mainline, and no discernable pattern can be seen as the period progresses. A moderate reduction in mainline speed is seen with the implementation of gap meters during the off peak period. This is because drivers adhering to gap metering instructions are slowing down to increase their vehicular headway. Under conditions representing traffic flow breakdown, this increased headway provides a necessary gap that is otherwise unavailable to merging traffic. Under higher speeds, however, this results in a disproportionate reduction of average speed. The situation observed when both gap and ramp meters are deployed simultaneously during off-peak hours shows a severe reduction in mainline traffic speed.

Moderate reductions are seen in frontage road speeds for off-peak deployment of ramp metering systems. This is because of the unnecessary queue spillover that occurs due to restrictions on ramp demand during this period. This is also true of frontage road speeds under simultaneous gap metering and ramp metering deployment. The implementation of a gap metering system alone does not result in significant reduction of the traffic speed along the frontage road. This may indicate that while there is a slight impact on the frontage road, gap metering systems would not result in queuing on ramps.

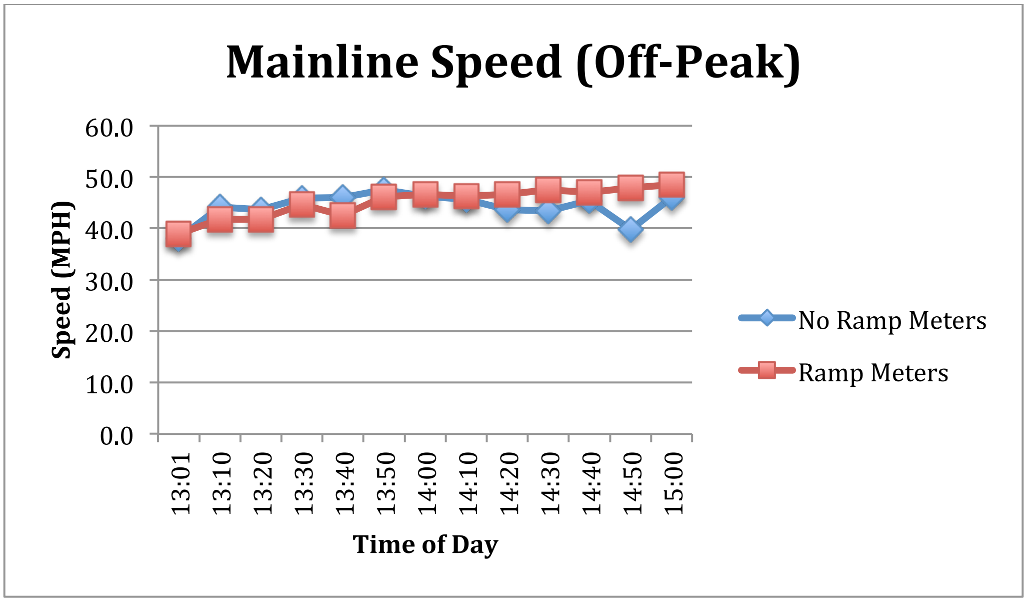

Figure 20: Average Vehicular Speed Ramp Meters vs. Base Case (Off-Peak)

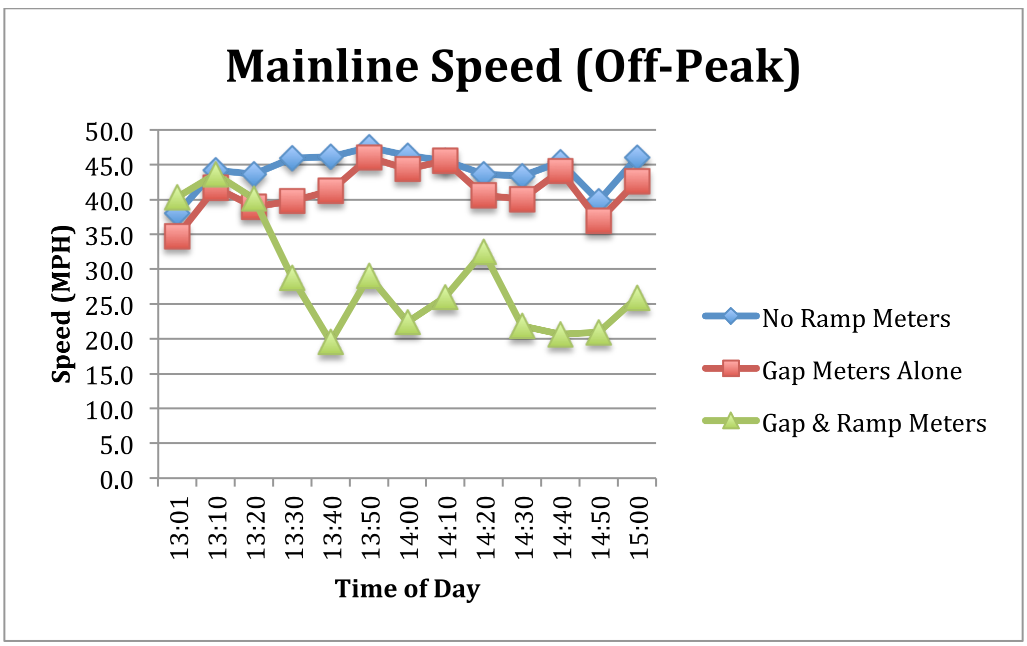

Figure 21: Average Vehicular Speed Gap Metering Cases vs. Base Case (Off Peak)

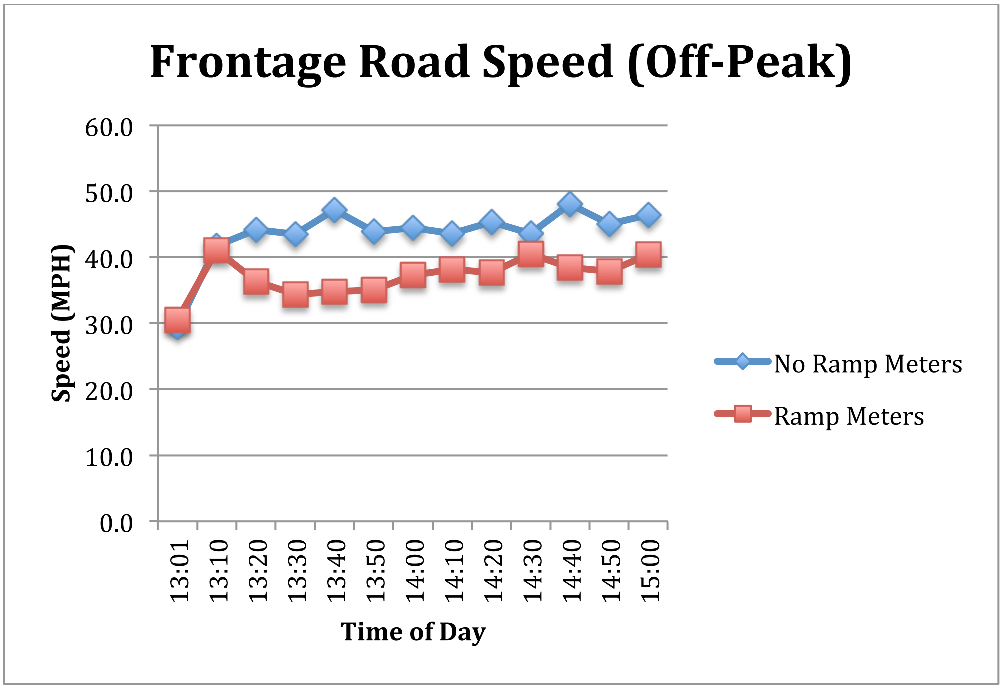

Figure 22: Average Vehicular Speed Ramp Metering Case vs. base Case (Off Peak)

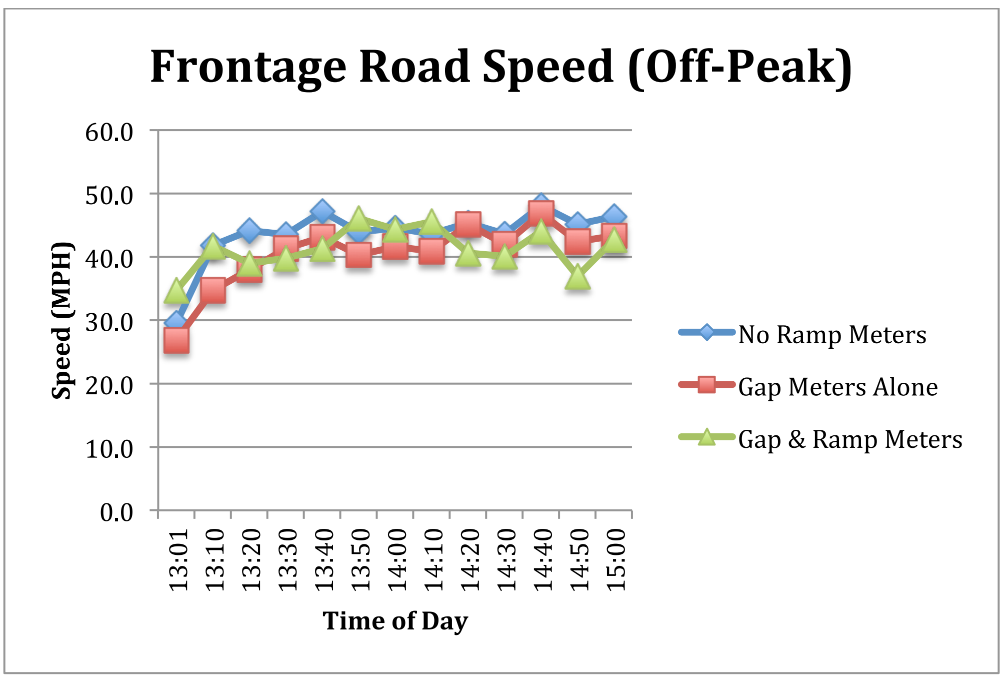

Figure 23: Average Vehicular Speed Gap Metering Cases vs. Base Case (Off Peak)

4.2: Average Segment Volumes

Because none of the active traffic management strategies under consideration in this study have the capability of increasing the capacity of either the freeway or the frontage road, the impact that these techniques have on the hourly volume for each segment is negligible. In order to calculate the average volume for the mainline and frontage road segments, post-processing was applied to the VISSIM output. The primary output format from the VISSIM simulations was a series of string data. For each link of interest on the simulation network, a minute-by-minute account of the segment's volume was presented in vehicles per hour. The average volumes for the mainline and frontage roads were then determined by performing a weighted average of each of the segment's component links based on link length.

As seen in Figures 24 through 29, none of the control strategies examined had any effect on traffic volumes during the peak period. Flows are displayed at 10-minute intervals. It is clearly seen that the flow rates are highly variable on both the mainline and also the frontage roads during this period, although the changes are consistent between implementation scenarios. This is because during the peak period, the breakdown of traffic flow results in the propagation of shockwaves upstream. As these shockwaves move upstream, vehicles alternate between motion and a queued state. This results in the instantaneous flow rate at certain points along the length of the facility dropping to zero. Taken in aggregate, the average flow rate along the length of the facility will fluctuate between a minimum and maximum value, observed here as between approximately 1500 vehicles/hour and 2600 vehicles/hour respectively.

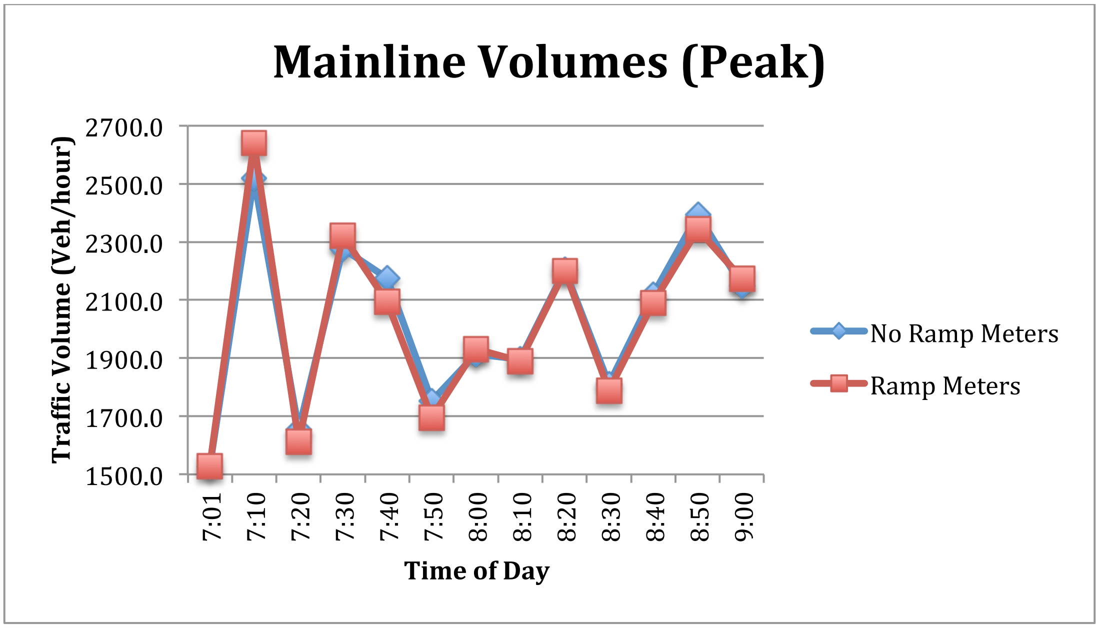

Figure 24: Average Mainline Volumes Ramp Metering Case vs. Base Case

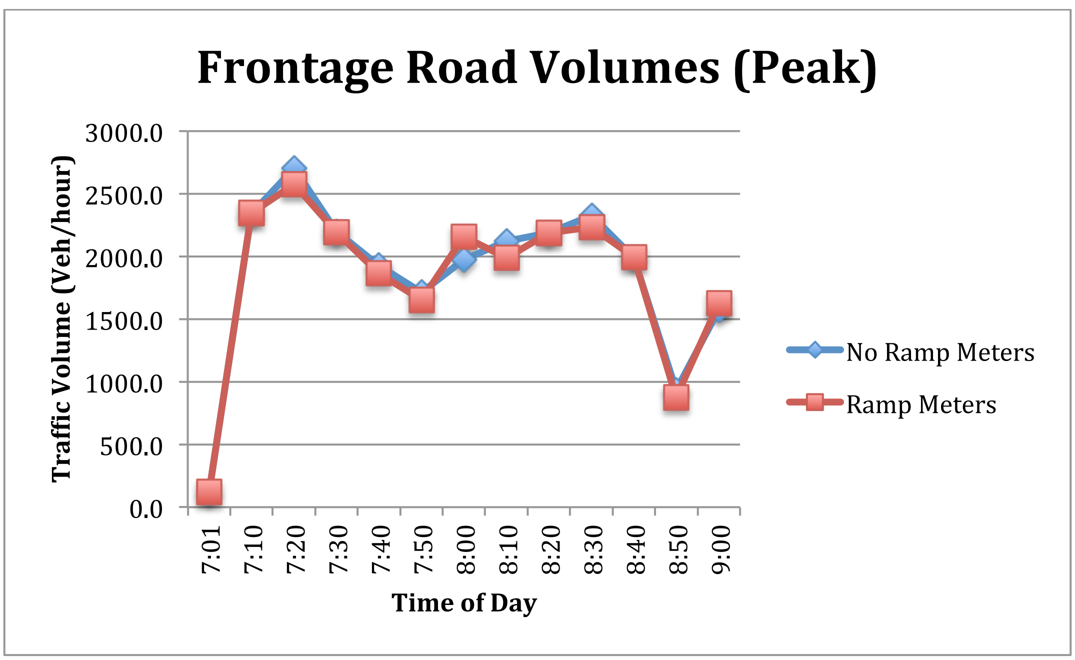

Figure 25: Average Frontage Road Volumes Ramp Metering Case vs. Base Case

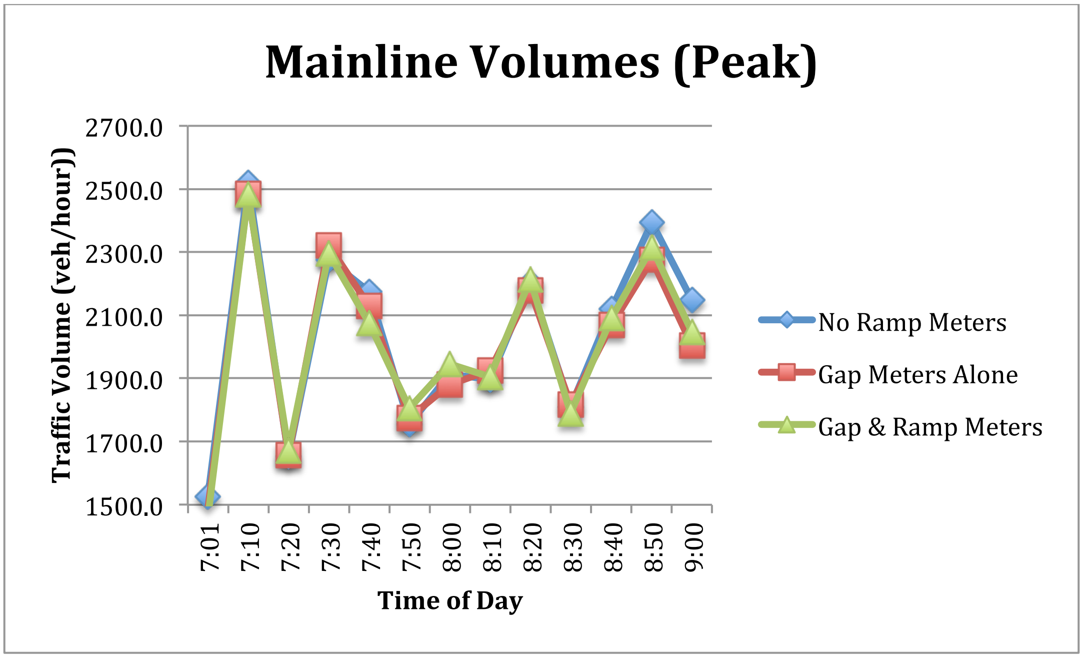

Figure 26: Mainline Volumes Gap Metering Cases Vs. Base Case, Peak Hours

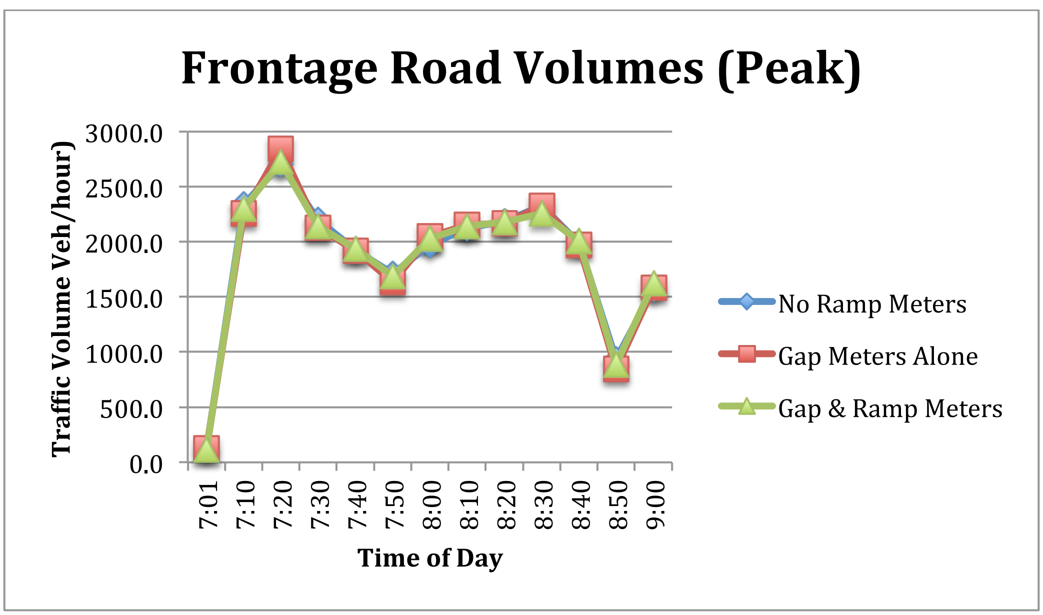

Figure 27: Frontage Road Volumes Gap Metering Cases Vs. Base Case, Peak Hours

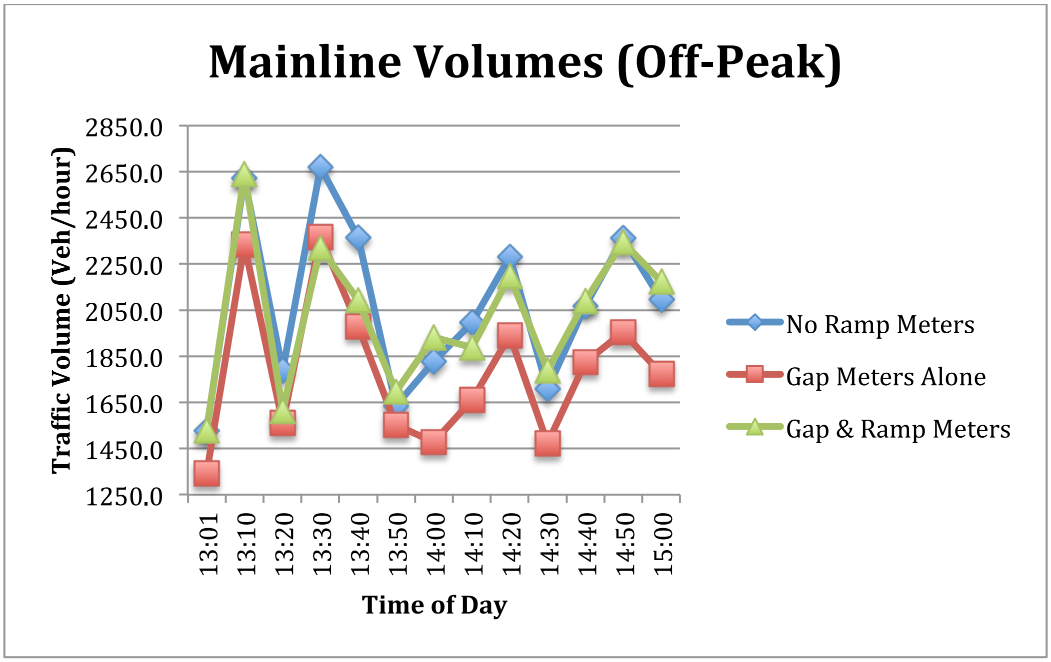

Figure 28: Mainline Volumes Gap Metering Cases Vs. Base Case, Off-Peak Hours

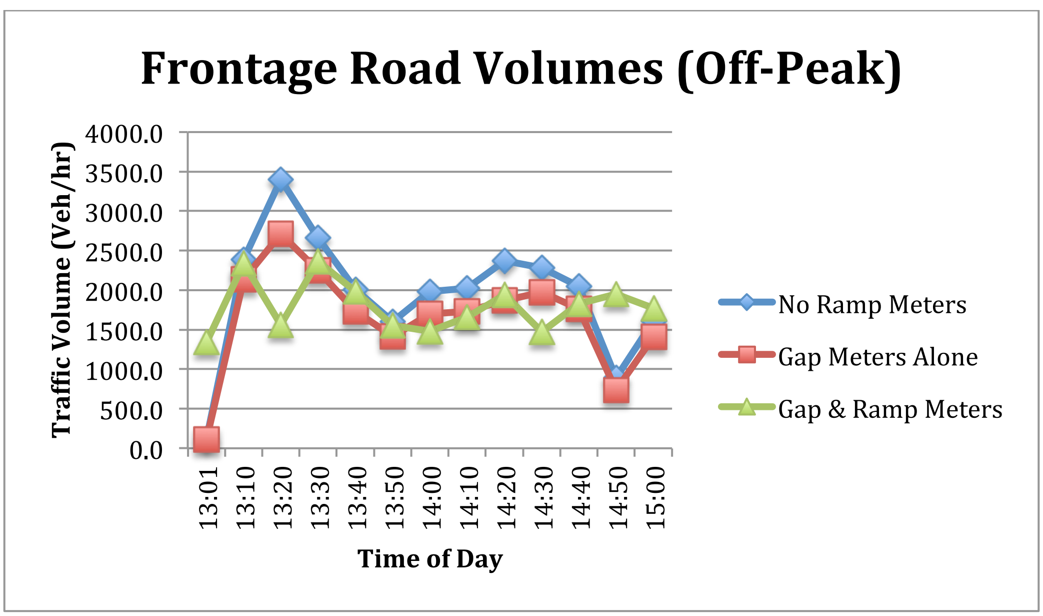

Figure 29: Frontage Road Volumes Gap Metering Cases Vs. Base Case, Off-Peak Hours

The presence of a ramp metering system during off-peak hours has a large impact on overall throughput, however. Due to the restriction on vehicles attemting to use the onramps imposed by ramp metering systems, smaller bottlenecks along the mainline section at merge points were elimitated or greatly reduced. This resulted in a much higher throughput for the mainline of the northbount evaluation section, as seen in the figure below. However, this postitive impact on mainline flow had a proprotionally large impact on the flow of the frontage road. Because of the excess queueing on the freeway entrances resulting from ramp meter system activation, the flow along the frontage road decreased to well below 1000 vehicles per hour. This is a very low rate because the frontage road is at least 2 lanes for its entire length along this segment, and at some points it is three lanes wide. The effects of queue spillover during off-peak hours indicate that ramp meters should never be deployed outside of peak commuting periods.

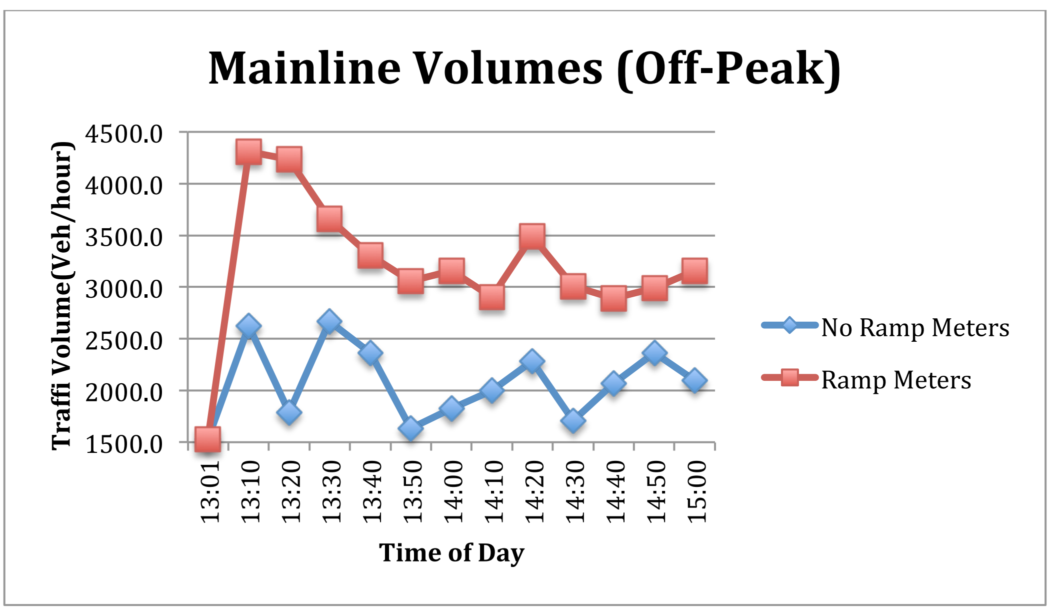

Figure 30: Average Flow Rate Ramp Metering Case vs. Base Case (Off Peak)

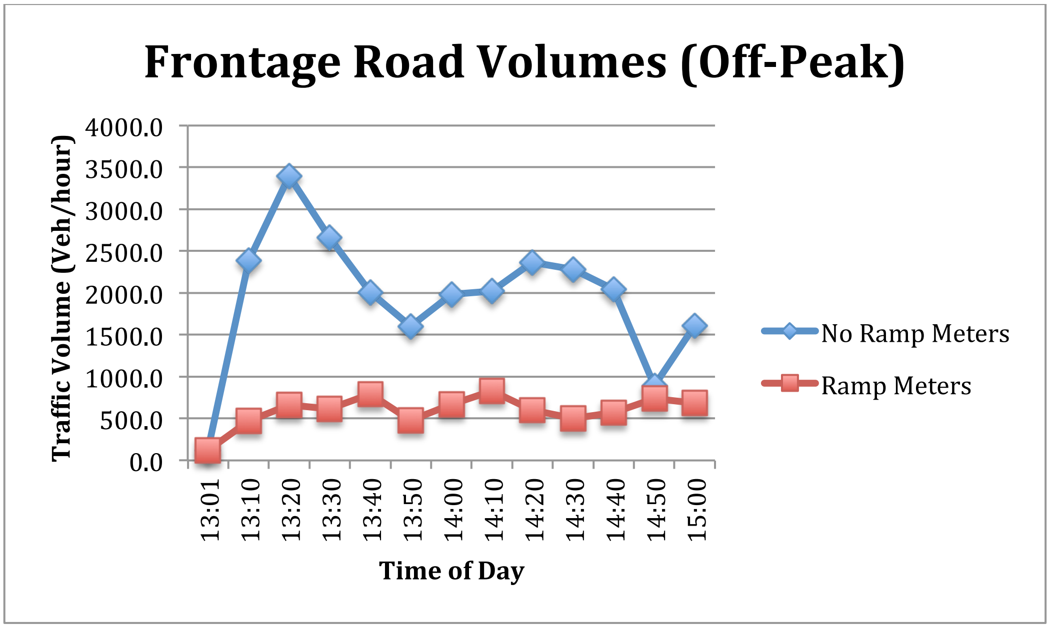

Figure 31: Average Flow Rate Ramp Metering Case vs. Base Case (Off-Peak)

4.3: Impact on Travel Times

Based on the definitions of travel time reliability measures discussed in Chapter 2, a metric by which to measure the equitability of travel times between two parallel routes was developed. This measure is based on the delay each route experiences. Because traffic along the mainline and frontage roads are affected in different manners by ramp metering and merge control technologies, the differences in travel times experienced along these routes must be evaluated. Using the concept of travel time reliability as a template, a Travel Time Equitability index is defined as:

$$TTE = \frac{|Frontage\ Road\ Delay - Mainline\ Delay|}{Average\ Delay} \quad (eqn. 4)$$

By applying this metric to the delays observed under each of the simulation scenarios, the varying impact that each of the control technologies under consideration has can be evaluated. These changes are summarized in Table 2.

		Frontage Road Delay (sec)	Mainline Delay (sec)	Average Delay (sec)	TTE
Peak Hour	Base Case	340	519	429.5	0.42
	Ramp Meters Only	344	483	413.5	0.34
	Gap Meters Only	325	344	334.5	0.06
	Both Ramp & Gap Meters	313	392	352.5	0.22
Off Peak	Base Case	109	17.5	63.25	1.45
	Ramp Meters Only	160	19.5	89.75	1.57
	Gap Meters Only	188	35	111.5	1.37
	Both Ramp & Gap Meters	128	33	80.5	1.18

Table 2: Travel Time Equity

The value of travel time equity represents the relative difference in delay experienced between two routes. A smaller value represents a higher level of equity between travel times along a route. Therefore, any operational strategies intended to improve the efficiency along the evaluation corridor will shrink the travel time equity value. By doing so, it will ensure that any changes made to the facility operation will improve performance of the mainline at the expense of the frontage road. By making the two parallel routes more equitable in their performance, the additional, unused capacity represented by the frontage road will be more likely to absorb excess demand along the route. As seen in Table 2, all of the operational strategies for ramp and merge control had a positive impact on the TTE value during peak hours. Simulations showed that without any operational changes along the evaluation section, the Travel Time Equity index comparing mainline and frontage road performance was 0.44, skewing toward better performance along the frontage road. All of the ATM strategies under evaluation yielded improvements during the peak commuting period, with gap metering implementation alone showing the highest level of equality between route delays. As seen with the other measures of performance, implementation during off-peak hours is unadvisable.

4.4: NETWORK LEVEL MEASUREMENTS

During the data collection effort along the study corridor, it was noted that while the mainline facility exhibited signs of an F level of service, there was excess capacity on the frontage road. Even though the frontage road is signalized, it remains a viable alternative route to the mainline under congested conditions. Accordingly, this study examines the impacts each of the control scenarios has upon both the mainline and the frontage road. To achieve accurate results, each scenario was run with 10 random seeds. The results displayed below reflect the average of each of these 10 runs. The inclusion of 10 runs controls for variance and statistical bias in the results.

The previously displayed results were generated from string output from the VISSIM model. While the string output focuses on the performance of individual links, tools within the program were also used to generate network level impacts for each of the evaluation scenarios. Four major measures of performance were used to evaluate each scenario on the network level. These were delay, throughput, average vehicular speed, and travel time. Because ramp and merge control will have an effect on the flow of traffic from the frontage road to the facility mainline, these values were taken separately along the mainline and along the frontage road.

The delay statistic is given in average seconds per vehicle. Delay is defined as the difference between travel time and free flow travel time. The throughput statistic refers to the total number of vehicles that traveled a particular route for the entire length of the facility. This number is shown for the entire two-hour simulation time. The mainline throughput only measures the number of vehicles entering the network at the southernmost part of the mainline, and exiting the network at the northernmost part of the mainline. The frontage road throughput only measures the number of vehicles entering the network at the southernmost part of the frontage road, and exiting the network at the northernmost part of the frontage road. Vehicles taking other routes are not included in this number. For example, a vehicle entering the simulation network at the southern end of the mainline and exiting the freeway facility before reaching the northern end of the mainline where the bottleneck condition occurs would not be counted in route throughput. Similarly, a vehicle entering the freeway facility at any of the three on-ramp locations and exiting at the north end of the mainline would not be counted in the route throughput either. Speed and travel time are given as averages across these two routes for the two-hour simulation duration.

Each scenario was compared against the no action case for each of these measures. The tables below summarize the average measurement for each of the ten runs for each scenario. Each of the runs for each scenario is used to compare the scenarios and determine if a difference is statistically significant. A paired-samples T test with an alpha level of 0.05 was used to determine whether differences between the base case and each of the alternative cases are statistically significant. The results of these tests are summarized in the tables below. If the percent change column is shaded green, the alternative has a statistically significant effect on performance that is beneficial. If the percent change column is shaded red, the alternative has a statistically significant effect on performance that is detrimental.

Table 3 shows the results for the four scenarios for peak hour conditions. As seen, the implementation of a ramp metering system during peak commuting hours along the northbound bottleneck section of I-35 would make small but significant improvements on overall network performance. The ramp metering system resulted in a 7% reduction in delay along the mainline route. This meant a decrease in overall travel time of 6%. In addition, the number of vehicles able to take the northbound mainline route increased slightly by 1%. All of these results had a statistical significance. Impacts along the network level flow of the frontage road were negligible, and did not demonstrate statistical significance.

The implementation of a gap metering system at each of the merging/weaving segments along the evaluation route would also have a net beneficial result during peak hours. The network level output showed a reduction in delay along the mainline of 24%, and a corresponding reduction in overall travel times along the mainline of 19%. This was accompanied by a small 3% reduction in overall route throughput. However, this does not mean that the mainline handled a smaller overall volume, because this amount may be made up for in increased merging rates from the on-ramps. Additionally, a 24% increase in average vehicle speed was observed. No significant change was seen in the network level evaluation criteria for the frontage road. Even better improvements were seen with a combined ramp metering and gap metering system. This implementation scenario resulted in a 34% reduction in delay along the mainline, and a 27% reduction in overall travel time. A 38% increase in average vehicle speed was also seen along the mainline. This scenario also has the distinction of being the only one to positively influence network level performance along the frontage road route as well. A small but significant decrease in delay of 4% was observed along the frontage road, with a 3% decrease in overall travel time. In addition, a 3% increase in average vehicular speed along this route was also observed.

Peak Hour	Mainline			Frontage		
Ramp Meters Only	Base Case	Ramp Meters Only	% Change	Base Case	Ramp Meters Only	% Change
Delay (seconds)	519	483	-7%	340	344	1%
Throughput (vehicles)	4047	4074	1%	83	81	-2%
Speed (mph)	12.78	13.52	6%	17.6	17.5	-1%
Travel Time (seconds)	649	613	-6%	470	474	1%

Peak Hour	Mainline			Frontage		
Ramp and Gap Meters	Base Case	Ramp and Gap Meters	% Change	Base Case	Ramp and Gap Meters	% Change
Delay (seconds)	519	344	-34%	340	325	-4%
Throughput (vehicles)	4047	3970	-2%	83	75	-10%
Speed (mph)	12.78	17.6	38%	17.6	18.2	3%
Travel Time (seconds)	649	474	-27%	470	455	-3%

Peak Hour	Mainline			Frontage		
Gap Meters Only	Base Case	Gap Meters Only	% Change	Base Case	Gap Meters Only	% Change
Delay (seconds)	519	392	-24%	340	313	-8%
Throughput (vehicles)	4047	3937	-3%	83	75	-10%
Speed (mph)	12.78	15.8	24%	17.6	18.7	6%
Travel Time (seconds)	649	523	-19%	470	443	-6%

Table 3: Peak Hour Results

Table 4 shows results for the four scenarios for off-peak conditions. The base, no-action scenario was compared against the ramp metering only, both ramp and gap metering, and gap metering only cases for the off-peak, no bottleneck scenario. During the off-peak period, none of the control implementation scenarios examined in this study performed favorably. Ramp metering implementation during off-peak hours resulted in a 1% decrease in overall route throughput along the mainline. In addition, the resulting queue spillback onto the frontage road saw a 47% increase in overall delay along the frontage road route. This was accompanied by an 18% reduction in average vehicular speed and a 21% increase in overall travel time.

Scenarios featuring gap metering control techniques fared even worse. Implementing gap metering only resulted in a 89% increase in delay along the mainline route, with a corresponding 11% increase in overall travel time. In addition, this scenario resulted in a decrease in route throughput by 9%. Average vehicle speed along the mainline route was decreased by 10%. There were also severe negative impacts on the frontage road. This route displayed a 17% increase in overall delay and a 8% increase in total travel time. In addition, the frontage road route had a decrease in total throughput of 8% and average vehicle speed of 7%. The implementation of a coordinated gap and ramp metering system resulted in a doubling of delay along the mainline. This was accompanied by an increase in total route travel time by 12%, with a reduction of throughput and speed by 10% and 11%, respectively. The frontage road also saw severely negative impacts from a dual ramp and gap metering system. The frontage road route had an overall increase in delay by 72%, as well as an increase in overall travel time by 33%. Vehicles traveling along the frontage road route had a reduction in total throughput by 17%, and an average decrease in speed by 25%.

Off Peak	Mainline			Frontage		
Ramp Meters Only	Base Case	Ramp Meters Only	% Change	Base Case	Ramp Meters Only	% Change
Delay (seconds)	17.5	19.5	11%	109	160	47%
Throughput (vehicles)	4468	4408	-1%	86	82	-5%
Speed (mph)	56.2	55.4	-1%	34.6	28.5	-18%
Travel Time (seconds)	147	149	1%	239	290	21%

Off Peak	Mainline			Frontage		
Ramp and Gap Meters	Base Case	Ramp and Gap Meters	% Change	Base Case	Ramp and Gap Meters	% Change
Delay (seconds)	17.5	35	100%	109	188	72%
Throughput (vehicles)	4468	4028	-10%	86	71	-17%
Speed (mph)	56.2	50.2	-11%	34.6	26.1	-25%
Travel Time (seconds)	147	165	12%	239	318	33%

Off Peak	Mainline			Frontage		
Gap Meters Only	Base Case	Gap Meters Only	% Change	Base Case	Gap Meters Only	% Change
Delay (seconds)	17.5	33	89%	109	128	17%
Throughput (vehicles)	4468	4077	-9%	86	79	-8%
Speed (mph)	56.2	50.7	-10%	34.6	32.1	-7%
Travel Time (seconds)	147	163	11%	239	258	8%

Table 4: Off-Peak Results

Chapter 5: Conclusion

By examining the results from the peak hour scenarios, it is determined that implementation of both ramp metering and merge control along the studied segment of I-35 would be tremendously beneficial in terms of reducing delay, and increasing speeds. The best performance was seen by the scenario utilizing both ramp and gap metering alongside each other. This includes a 34% reduction in delay and 38% increase in speeds along the mainline. A more modest benefit was seen along the frontage road. The gap metering only scenario yielded a delay reduction of 24%. The implementation of a ramp metering system alone would reduce delay by 7%, and increase speeds along the facility by 6%. In none of the peak hour scenarios is the total vehicular throughput affected greatly. This is because during peak hours, the facility is already experiencing a level of service of F. While these alternatives reduced delay and congestion, they do not result in additional capacity.

All of the alternative scenarios yielded poorer results for the off-peak simulations. Across the board, implementation of either control technology resulted in additional delay and reduced throughput compared to the base scenario of no action. Therefore, it is recommended that if active control strategies are implemented, they should be limited in their operation to peak hours. During non-peak hours, they have a detrimental effect. This is true even though activation of the tested ramp metering system required a threshold density to be reached. When this threshold was passed during the non-peak period, ramp meter operation still had a negative effect. Therefore, activation of the ATM systems should only occur during peak demand periods.

The results of this study indicate that further attention should be given to the feasibility and potential benefits of the implementation of a combined ramp meter and active merge control system for Austin's highways. One avenue of evaluation could be the safety impacts that such a system would have. Additionally, expansion of the simulation network to include more controlled onramps would indicate whether system-wide gains could be made from these operational improvements. The concept of gap metering is worth further evaluation as a candidate for addition to the state of practice of merge control. While the gap metering strategy shows operational improvements in terms of travel speeds and delay reduction, the assumptions the system's simulation is based on may not hold. In particular, the assumption of a 10% compliance rate to gap metering instructions requires further evaluation. Study is needed to determine the potential driver response to the proposed control system.

Both the data collection effort and the model indicate that currently, the I-35 northbound frontage road is underutilized. Because the I-35 corridor and river crossing represent a major bottleneck of capacity entering the Austin central business district, making the best use of this capacity should be a priority for TXDOT. Future study may include potential operational strategies that encourage drivers to leave the mainline facility in favor of the frontage road. The results of this study indicate that ramp metering and merge control system implementation could significantly improve traffic operation along this corridor.

Appendix

VisVAP Code for Ramp Metering System (Modified from default provided in VISSIM package)

```
PROGRAM RampMetering; /*
D:\VISSIM\Daten\__Training\VAP_RampMetering.214\RampMetering.vv */
CONST
      MAX_LANE = 2,
      KR = 70,
      OCC_OPT = 0.29;
/* ARRAYS */
ARRAY
      detNo[ 2, 1 ] = [[11], [12]];
/* SUBROUTINES */
/* PARAMETERS DEPENDENT ON SCJ-PROGRAM */
      IF( prog_aktiv = 1 ) AND ( prog_aktiv0vv <> 1 ) THEN
       prog_aktiv0vv := 1;
       DT := 1;
      ELSE IF( prog_aktiv = 2 ) AND ( prog_aktiv0vv <> 2 ) THEN
       prog_aktiv0vv := 2;
       DT := 1;
      END END;
/* EXPRESSIONS */
      Demand := Detection( 2 );
/* MAIN PROGRAM */
S00Z001:  IF NOT init THEN
S01Z001:    init := 1;
S01Z002:    Set_sg( 1 , off )
      END;
S00Z004:  cyc_sec := cyc_sec + 1;
S00Z005:  IF cyc_sec >= cyc_length THEN
S01Z005:    cyc_sec := 0
      END;
S00Z007:  Set_cycle_second( cyc_sec );
S00Z008:  laneNo := 1;
S00Z010:  IF laneNo <= MAX_LANE THEN
S01Z010:    IF detNo[ laneNo, 1 ] > 0 THEN
S02Z010:      oout := oout + Occup_rate( detNo[ laneNo, 1 ]);
S02Z011:      laneNo := laneNo + 1;
         GOTO S00Z010
       END
```

```
      END;
S00Z013:   timer_dc := timer_dc + 1;
S00Z014:   IF timer_dc = (60 * DT) THEN
S01Z014:     timer_dc := 0;
S01Z015:     qRamp := (Front_ends( 9 )); Clear_front_ends( 9 );
S01Z016:     oout := oout / MAX_LANE / (60*DT);
S01Z017:     cqRamp := qRamp + KR * (OCC_OPT - oout);
S01Z018:     cyc_length := 60*DT / cqRamp;
S01Z019:     oout100 := oout * 100; RecVal( 1, oout100 );
S01Z020:     oout := 0
      END;
S00Z023:   IF cyc_length < 4 THEN
S01Z023:     Set_sg( 1 , off )
      ELSE
S00Z024:     IF Demand THEN
S01Z024:       IF cyc_sec = 0 THEN
S02Z025:         Set_sg( 1 , redamber );
S02Z026:         cyc_sec := 0
        ELSE
S01Z025:         IF T_red( 1 ) >= cyc_length-3 THEN
          GOTO S02Z025
         ELSE
S00Z027:           IF Current_state( 1, redamber ) THEN
S01Z027:             Set_sg( 1 , off )
          ELSE
S00Z028:             IF Current_state( 1, off ) THEN
S01Z028:               IF NOT (cyc_length < 4) THEN
S01Z029:                 Set_sg( 1 , amber )
            END
           ELSE
S00Z030:               IF Current_state( 1, amber ) THEN
S01Z030:                 Set_sg( 1 , red )
            END
           END
          END
         END
        END
       ELSE
        GOTO S00Z027
       END
      END;
S00Z032:   RecVal( 2, cyc_length );
```

```
S00Z033:    qRampHour := qRamp * 60 / DT; RecVal( 3, qRampHour )
PROG_ENDE:    .
/*---------------------------------------------------------------------------*/
```

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