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Alexander Skabardonis and Xiao-Yun Lu, California PATH, UC Berkeley

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16. ABSTRACT
The independent operations of freeway ramp meters and the adjacent arterial intersection traffic signals often causes queue spill back on the freeway on-ramps and the surface street network that result in activation of queue override, which negates the benefits of ramp metering. This is due to the lack of coordination between the two traffic control systems which are usually operated by different agencies. Field measurements at a real-world test site show that queue override reduces the freeway bottleneck capacity by 10%. A control strategy for coordinating freeway ramp metering and arterial traffic signals was developed, field implemented, and evaluated in this study. The algorithm takes available on-ramp storage into account and dynamically reduces the cycle length of the feeding intersection signal control in order to avoid on-ramp queue spill-back and mitigate unnecessary delay in the conflicting directions. The proposed algorithm was tested in the morning peak during a four-month period. Observations in the field suggest that the proposed control was able to significantly reduce travel time and delay on freeways while preventing on-ramp queue spill-back to arterials.

17. KEY WORDS
Integrated corridor management (ICM), Ramp Metering (RM), Local Responsive Ramp Metering (LRRM), Vehicle-Miles-Traveled (VMT), Vehicle-Hours-Traveled (VHT), Freeway efficiency (Q) or Average speed.

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Coordination of Freeway Ramp Meters and Arterial Traffic Signals Phase II-B – Field Implementation

David Kan
Xiao-Yun Lu
John Spring, David Nelson
Alexander Skabardonis

California PATH Research Report
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Abstract

The independent operations of freeway ramp meters and the adjacent arterial intersection traffic signals often causes queue spillback on the freeway on-ramps and the surface street network that result in activation of queue override, which negates the benefits of ramp metering. This is due to the lack of coordination between the two traffic control systems which are usually operated by different agencies. Field measurements at a real-world test site show that queue override reduces the freeway bottleneck capacity by 10%. A control strategy for coordinating freeway ramp metering and arterial traffic signals was developed, field implemented, and evaluated in this study. The algorithm takes available on-ramp storage into account and dynamically reduces the cycle length of the feeding intersection signal control in order to avoid on-ramp queue spillback and mitigate unnecessary delay in the conflicting directions. The proposed algorithm was tested in the morning peak during a four-month period. Observations in the field suggest that the proposed control was able to significantly reduce travel time and delay on freeways while preventing on-ramp queue spillback to arterials.

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Executive Summary

The Problem

Integrated corridor management (ICM) for highway facilities, comprised of freeways and adjacent arterial streets, offers considerable potential for managing both traffic congestion and reducing the adverse environmental impacts of transportation.

Current freeway traffic management, mostly Ramp Metering (RM), and the relevant arterial/surface street intersection traffic signals control are usually operated by different agencies with most likely different or conflict interests rather than coordinated and optimized for mitigating the overall traffic congestions.

During peak hours, freeway ramp metering restricts the flow of on-ramp traffic entering the freeway mainline, to mainline traffic that create traffic congestion in the merging area. Operating in isolation, RM intends to prevent capacity drop under the conditions of high on-ramp demand.

Operating independently from the RM, the traffic controller of the arterial intersection feeding into the onramp usually fail to recognize that the metered on-ramp is oversaturated. Instead, with increasing demand, arterial traffic signals provide long cycles (with long green durations) and progressively timing with adjacent intersection signals (along the major arterial) to maximize the flow. Under high demand, this leads to platoons of arterial traffic advancing into the oversaturated onramps, which leads excessive queues at the on-ramps and then spillback to the feeding intersection. This will definitely reduce the throughput, or even cause gridlock, of the overall network including both freeway corridor and relevant arterials.

The Objectives

The objective of this project is to develop, implement, and field test coordination strategy between arterial signal control and the freeway RM to improve the overall traffic throughput. This is accomplished through coordinating arterial signal timing with the on-ramp queue volume and storage capacity so that the feeding-flow into the freeway considers the onramp storage limit and RM rate. This coordination intends to prevent freeway on-ramp queue oversaturation, queue spillback and RM queue override so that the freeway could also keep at higher flow.

What We Have Done

System Scope: Freeway and Arterial

This project conducted field tests the coordination strategy along a four-mile long 4-lane corridor of I-680 Northbound (including Capitol Ave. as the parallel arterial) during the morning peak. The selected site spans from Alum Rock Ave. to Berryessa Rd. in San Jose, California. There are three recurrent bottlenecks on this stretch of I-680; they are located near the on-ramps from Berryessa Rd. (2-lane), McKee Rd. (3-lane), and Alum Rock Ave. (3-lane). At all three bottlenecks have high volumes of demand during the morning peak (7:30-9:30 AM).

Ramp Metering Strategies On Freeway Corridor (Local Responsive And Coordinated)

Caltrans District 4 is currently using local traffic responsive ramp metering at all on-ramps in this corridor. The metering rates are assigned according to various thresholds of freeway mainline occupancies immediately upstream of the merging or weaving area.

In the latter portion of the field test, an optimal Coordinated Ramp Metering (CRM) algorithm was implemented, which was implemented before on SR-99 NB corridor in Caltrans District 3 by the project team. The CRM algorithm determines the metering rates based the Cell Transmission Model of the freeway corridor with the traffic speed and density in each section as the state parameters. The objective function for the optimal control is to minimize the total travel time and maximizes the total travel distance, subjected to appropriate constraints. The density is estimated from the 30-second loop detector data on occupancy, flow and speed obtained from the San Jose Traffic Management Center (TMC).

Arterial Intersection Signal Strategies

Our developed signal control strategy is similar to that of oversaturated signals in the following sense: The signalized intersections has simultaneous offset (zero offset) when it is congested downstream of the major intersection with access to the freeway on-ramp. This allows the residual queue at the downstream intersection to dissipate in time, otherwise, large platoons of vehicles would arrive from upstream intersection before the downstream residual queues are served, which can fill up the limited queue storage space and cause spillback upstream. Furthermore, most users of the arterials adjacent to the freeways travel a short segment to access the nearest freeway on-ramp rather than a long segment of the arterial, thus uninterrupted progression and maximum bandwidth for a long stretch of arterial is not appropriate.

For this field test, the above signal control strategy was applied to the intersections of Capitol Ave., and McKee Rd. and Capitol Ave. and Berryessa Rd. The intersection of Capitol Ave. and Alum Rock Ave. was not included in this signal control implementation due to the lack of queue spillback at the on-ramp.

Coordination Strategies

The following simple and easy strategy for the coordination of freeway ramp metering and arterial traffic signal was implemented and field-tested. The coordination strategy manages the on-ramp queues at the adjacent arterial, facilitated by system integration through wireless communication between the intersection signal controller and onramp metering controller. This coordination strategy actively monitors the queue length on the freeway on-ramp during ramp metering operation and actively reduces the cycle length of the nearby arterial signalized intersections to prevent long green times and therefore long platoons of traffic from entering the on-ramp to avoid queue spillback.

Field Implementation

The field implementation of coordination of ramp metering and arterial traffic signals was conducted every weekday morning peak (7:00 AM to 10:00 AM) from Monday April 29, 2019 to Friday August 23, 2019. Four different scenarios were tested during this four-month test period:

- Scenario 1: Caltrans District 4 local responsive ramp metering (see section 3.4 for details) without coordination of ramp metering and arterial traffic signals (baseline)

- Scenario 2: Caltrans District 4 local responsive ramp metering (see section 3.4 for details) with coordination between ramp metering and arterial traffic signals
- Scenario 3: PATH developed CRM (See Appendix A for details) without coordination between ramp metering and arterial traffic signals
- Scenario 4: PATH developed CRM (See Appendix A for details) with coordination between ramp metering and arterial traffic signals

The field test lasted a total of 17 weeks, which included time spent on fine tuning, resolving unforeseen circumstances such as vandalized equipment, and further schedule extension due to low traffic demand.

Total travel time (TTT), total delay (TD), and vehicle-miles traveled (VMT) were used to determine the effectiveness of the control strategies developed in this project to improve the day-to-day operation on the freeway corridor and the relevant arterials. These measures of effectiveness are all derived from loop detector data provided by Caltrans.

The on-ramp queue detector occupancy data (at each on-ramp) were used to determine of the presence or severity of on-ramp queue spillback after introducing the coordination of freeway ramp metering and arterial traffic signals. Further data collection using arterial detectors were not conducted due to the fact that City of San Jose does not store arterial detector data for arterial performance evaluation purposes.

Test Results

The on-ramp queue detector occupancy data (at each on-ramp) were used to determine of the presence or severity of on-ramp queue spillback. Further data collection using arterial detectors were not conducted due to the fact that City of San Jose failed to save the arterial detector data for performance evaluation purposes.

Results from the first half of the study suggest that the proposed coordinating strategy reduced the freeway delay by up to 11.49% (Scenario 2 vs. Scenario 1). Similarly, the second half of the field test (with the coordinated ramp metering algorithm) demonstrated a 5.92% (Scenario 4 vs. Scenario 3) reduction in delay.

In addition, comparison of on-ramp queue detector occupancy data (with and without coordination) demonstrated that the proposed coordination strategy was also able to prevent on-ramp queue spillback regardless of the ramp metering strategy implemented, thereby mitigating the interference on arterials as well during ramp metering operation.

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Chapter 1. Introduction

1.1 Problem Statement

The implementation of integrated corridor management (ICM) on highway facilities, that are comprised of freeways and adjacent arterial streets, offers considerable potential to managing traffic congestion as well as reducing the adverse environmental impacts of transportation. However, current freeway control systems (mostly on-ramp ramp metering) and the traffic signals on adjacent arterial intersections (facilitating freeway access) operate independently under day-to-day recurrent traffic conditions. This independence results in conflicts that compromise the ability of the ramp control system or the intersection control system to mitigate congestion.

A ramp meter restricts the flow of on-ramp traffic entering the freeway, if the mainline occupancy is high. This restriction in traffic flow reduces the conflicts between vehicles entering the highway, via the ramp, and vehicles traveling the mainline highway. Such conflicts cause bottlenecks and a mainline capacity drop. Ramp metering, therefore, maximizes the capacity of the freeway (mainline and on-ramp) during periods of high demand. There are limits to this approach, however.

Successful operation of ramp metering depends on effective management of the queue of vehicles awaiting highway access. The ramp metering systems monitors queue length. When it grows the ramp metering system dynamically modifies vehicles access rate to insure that the queue does not spill back on the adjacent arterial. Should the vehicle queue extend into the adjacent arterial, the metering system allows all vehicles in the queue unmetered access on to the highway. This creates conflicts at the vehicle merge point and congestion on the highway.

Arterial traffic signals facilitate the access of arterial traffic on to the highway. These signals, today, operate independently of ramp-meter control system. This approach is problem-free under conditions of light traffic.

During periods of high demand, arterial traffic signals maximize arterial capacity by providing long cycles, with long green duration, in combination with progressively coordinating traffic signals along the major arterial. This strategy creates long platoons of vehicles – that travel down the arterial and onto the freeway on-ramp. Platoons of arterial traffic oversaturated freeway on-ramps by creating excessive queues that spillback on the adjacent arterial. Queue spillback will not only impede the conflicting directions of the arterial traffic, it will also trigger queue override at the metered on-ramps, which releases more on-ramp queue onto the freeway and further reduces its capacity.

1.2 Project Objectives

The objectives of this research project is to develop and implement a control algorithm to manage the entry of vehicles on the freeway corridor when both an on-ramp meter and an arterial traffic signal control vehicle access to the highway. The project will focus on signal timing changes of the arterial traffic signal control strategies. There are three arterial signal control and coordination strategies under consideration (Local Responsive or Coordinated) Ramp Metering, arterial intersection traffic signal control, and the coordination between the two control systems.

1.3 Concept of Operation (ConOps)

The ConOps scheme is illustrated in Figure 1.1 and Figure 1.2, below. New arterial control algorithms are hosted on a new PATH arterial traffic control computer, which is located in the San Jose Traffic Management Center (SJ TMC). These algorithms require data from the intersection traffic detectors, and will obtain this data through an interface with the existing SJ TMC computer, rather than directly linking with controllers in the field.

PATH's freeway ramp management (RM) control computer will be located in PATH Head Quarter at Richmond Field Station and will directly link with the existing RM controllers through wireless modem. This may require that existing RM controllers be upgraded to 2070 controllers, which offer a convenient interface for data acquisition and direct control. The existing arterial traffic management (ATM) control computer, located in Caltrans District 4 (D4) TMC, will act as a monitor and a supervisor. Should we experience any operational problems, the ATM computer will disable the new coordinated ramp metering (CRM) signal and reinstate the pre-existing default RM control strategy.

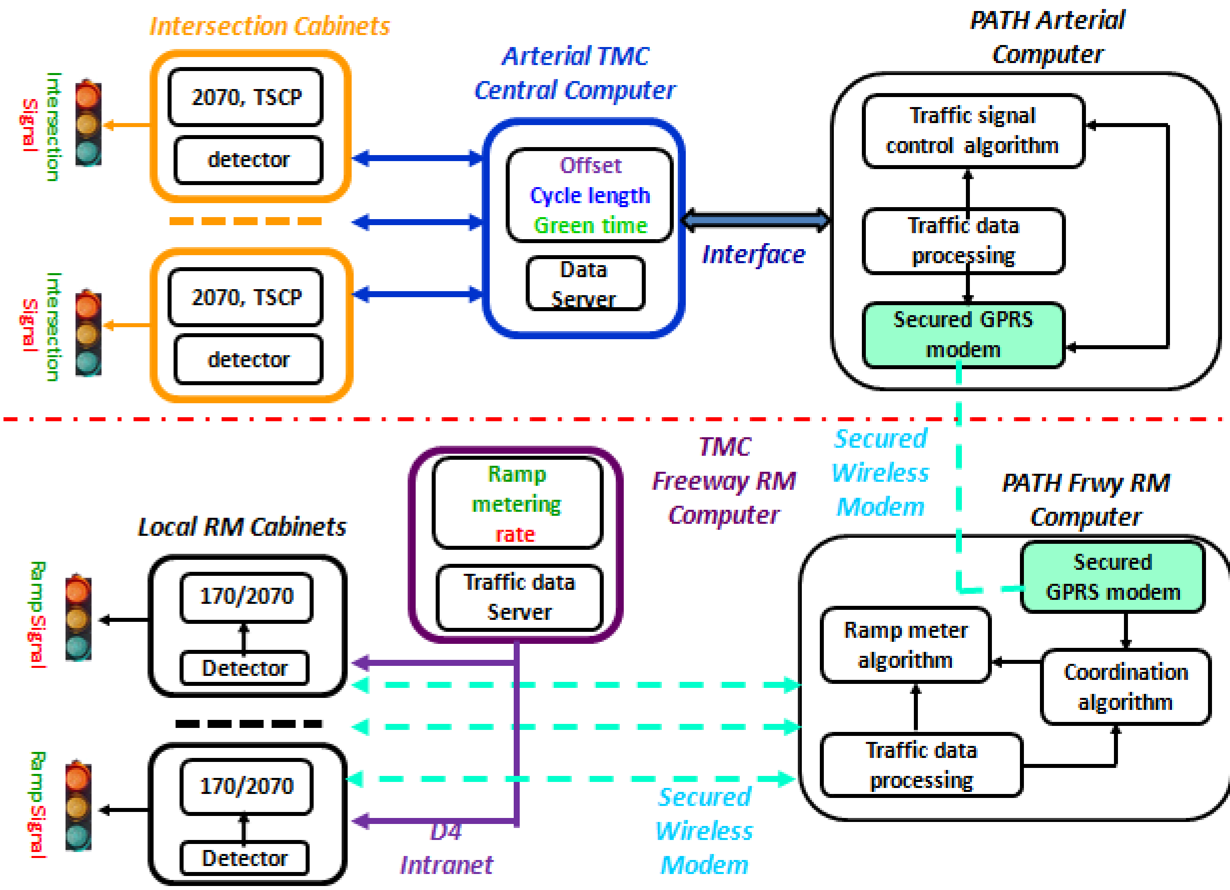

Figure 1.1 Block diagram and data flow of ConOps

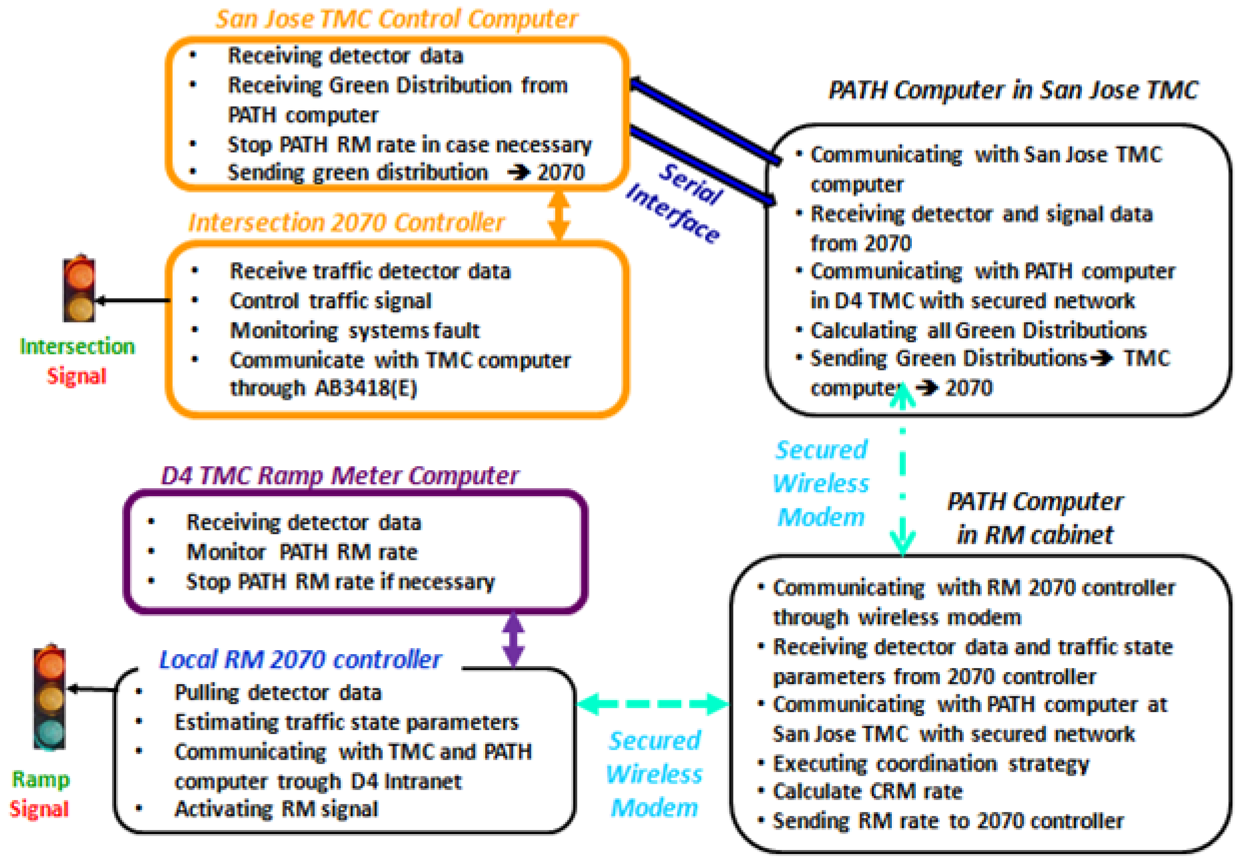

Figure 1.2 Functional diagram of ConOps

1.4 Report Organization

Chapter 2 provides an overview of recent research, in the area of coordinating freeway ramp metering and arterial traffic signals. Chapter 3 describes the test site selection process, and presents the characteristics of the selected site. Chapter 4 describes the three control strategies developed and the recommended control strategy to be tested in simulation and the field. Chapter 5 details the microscopic simulation model, its calibrated procedures, and the results of the simulations tests. Chapter 6 presents the results from a field study on the effect of queue override on the freeway discharge rate, performed to supplement the simulation results. The final chapter 6 summarizes the study findings and recommends the appropriate control strategy to be tested in the upcoming field implementation.

Chapter 2. Literature Review

A review of existing research literature reveals a primary focus on the development of optimization algorithms and routing models for integrated control of a freeway-arterial corridor system – with emphasis on non-recurrent (incident related) congestion. Two secondary themes also emerged first – control strategies for freeway interchanges to avoid off-ramp queue spillback, and algorithms that prevent overflow on metered ramps that adversely affect arterial operations under recurrent congestion. Representative summaries of the primary and secondary focus of recent research are presented below.

2.1 Freeway Traffic Diversion

Several studies address ramp and arterial signal coordination to successfully divert freeway traffic onto adjacent arterials, in an effort to respond to (mostly) non-recurrent conditions such as incidents on freeways. Such scenarios typically involve how to effectively utilize the spare capacity of the adjacent arterials in the event of temporary freeway capacity reduction, in order to prevent major freeway breakdown. They do not address how to efficiently coordinate metered on-ramps and adjacent arterials.

Recently, the Federal Highway Administration released a manual for coordinated freeway and arterial operation [1] but the document does not provide any control strategies for coordinated operation of freeway and arterial. This manual outlined the practical issues including, institutional barriers, technological challenges, and the integration of intelligent transportation systems. The manual also provided examples of freeway-arterial corridors that have implemented coordinated operation schemes; however, these examples only show how local arterials can be coordinated with the freeway in the event of an incident – to help divert some freeway traffic.

In separate studies, control strategies were developed for incident-related freeway traffic diversion. These strategies adjust ramp metering rates and arterial traffic signal timing in order to facilitate the transfer of high volumes of freeway traffic from the freeway, on the adjacent parallel arterial and then back on to the freeway – immediately downstream of a bottleneck. Tian et al [2] proposed a traffic-responsive coordination strategy, based on real-time queue detection on the freeway. This approach would extends the green times of freeway off-ramps and signals on the parallel arterial, and then maximizes ramp meter rates of the downstream on-ramps. It was shown to be effective for freeway-arterial corridors with consecutive diamond interchanges. In addition, the work by Zhang et al [3] tested a similar approach at a corridor with various configurations of freeway interchanges. Other related research include: an optimization-based coordination strategy that minimizes corridor level delay during incident diversion [4]; an empirical study of the effect of dynamic traveler information on freeway traffic diversion [5]; and, a control strategy for diverting traffic from the freeway to the adjacent arterials with significant spare capacity, in the event of periodic freeway capacity reduction [6].

2.2 Off-ramp Bottleneck

Several studies investigated the queue spillback of off-ramp freeway traffic onto the freeway mainline. Off-ramp bottlenecks are created by inefficient signal timing at the downstream end of the freeway off-ramp, where it intersects with the arterial. Recently Yang et al [7] proposed conditional signal priority for off-ramp traffic in order to mitigate the impact of off-ramp spillback on freeway performance, and this was enhanced in [8] by incorporating downstream arterial signal progression to quickly discharge the off-ramp queue and further reduce the impact of off-ramp spillback.

2.3 Coordination of Freeway On-ramp and Adjacent Arterial

Several studies addressed the inefficient control of freeway on-ramp metering and its impact upon nearby arterial corridor – which facilitate freeway access in the day-to-day recurrent conditions. However, no general control strategies have been implemented.

Tian et al [9] proposed an algorithm to reduce the frequency of queue override in an effort to maintain maximum freeway capacity - without imposing significant penalty on the arterial. This approach is mostly empirical and suggests that queue override should only be activated when spillbacks are detected on both the on-ramp and a length of the surface street upstream of the on-ramp. An evaluation of the approach showed limited benefit at an isolated intersection/freeway on-ramp but the study failed to recognize that the arterial signals near the freeway on-ramps are timed inefficiently.

Recker et al [10] developed a system-wide optimization model for ramp metering and traffic signals, based on stochastic queuing theory. Improvements were observed, after implementing the control strategy at a network of freeways and arterials. These improvements, however, are a result of using a more efficient ramp metering control, rather than coordination of ramp metering and traffic signals. Additionally, the proposed approach requires solving non-linear optimization in real time, which is computationally intensive and not feasible in most situations.

Other research efforts focused only on control of isolated signalized intersections at or adjacent to freeway on-ramps. For example, Li and Tao [11] proposed a signal optimization model for an arterial at an isolated freeway interchange using the cell transmission model; but, they neglected to include ramp metering in their algorithm.

In a recently completed PATH project, Su et al [12] developed a signal optimization model that takes the ramp meter rate and on-ramp queue length into account, for an isolated diamond interchange. A brief field test was conducted to show that coordination of freeway ramp metering and arterial traffic signals was technologically feasible. However, the algorithm developed is not applicable at the corridor level. It does not consider the importance of selecting the appropriate cycle length (when the freeway on-ramp becomes oversaturated and has limited queue storage space) and the impact of queue override.

Chapter 3. Test Site Selection

This chapter describes the process of test site selection and the characteristics of the site selected for testing the control strategies for coordination of freeway on-ramp metering and arterial signal control.

3.1 Site Selection Criteria

The selected site should be representative of typical freeway corridor segments (4 to 6 mile long) with at least one adjacent arterial facilitating freeway access. The selected corridor should satisfy the following criteria:

(1) The freeway corridor should have 3 to 5 freeway-arterial interchanges;

(2) The freeway corridor must not contain any freeway-freeway interchanges;

(3) At least one recurrent bottleneck must be observed during either the morning or the evening peak hours, preferable in only one direction of the freeway;

(4) The recurrent bottleneck(s) must be caused by the high on-ramp demand;

(5) Under recurrent conditions, the bottlenecks observed along the freeway corridor must be isolated (free-flow conditions at the upstream and downstream ends of the corridor);

(6) The physical capacity of a section is fixed except for lane reduction caused by lane closure due to incident/accident;

(7) The freeway corridor must have low frequency of incidents that contribute to non-recurrent delay;

(8) The length of the freeway on-ramps should not be too short or too long (ideally, they should accommodate 30 to 50 queued vehicles);

(9) The corridor must contain at least one parallel arterial adjacent to the freeway;

(10) The parallel arterial(s) must connect the arterials that have interchanges with and are perpendicular to the freeway;

(11) The parallel and perpendicular arterials adjacent to the freeway should be primarily used to facilitate freeway access;

(12) High demand from arterial to freeway should be the main cause of arterial congestion;

(13) No more than 5 major (with higher demand) signalized intersections along the parallel arterial;

(14) There should not be high concentration of pedestrians crossing the arterial or bicyclists impeding the arterial traffic flow;

(15) No active work zones on the freeway and the arterial;

(16) Satisfactory detector health and properly functioning ramp meters and traffic signals;

(17) Cooperation between the jurisdictions responsible for the operation and maintenance of the freeway ramp metering and arterial traffic signals control systems;

(18) The selected site must be supported by centralized data acquisition and control system in order to coordinate freeway ramp-metering and arterial traffic signals for field implementation

3.2 Candidate Sites

Six candidate test sites were identified based on extensive data analysis and input from Caltrans and the project panel. The maps of the candidate sites are shown in Figures 3.1 through 3.6.

3.2.1 Candidate Site #1: I-80 Northbound PM Peak

Segment of interest: Central Ave. to Pinole Valley Rd.

Parallel arterial: San Pablo Ave.

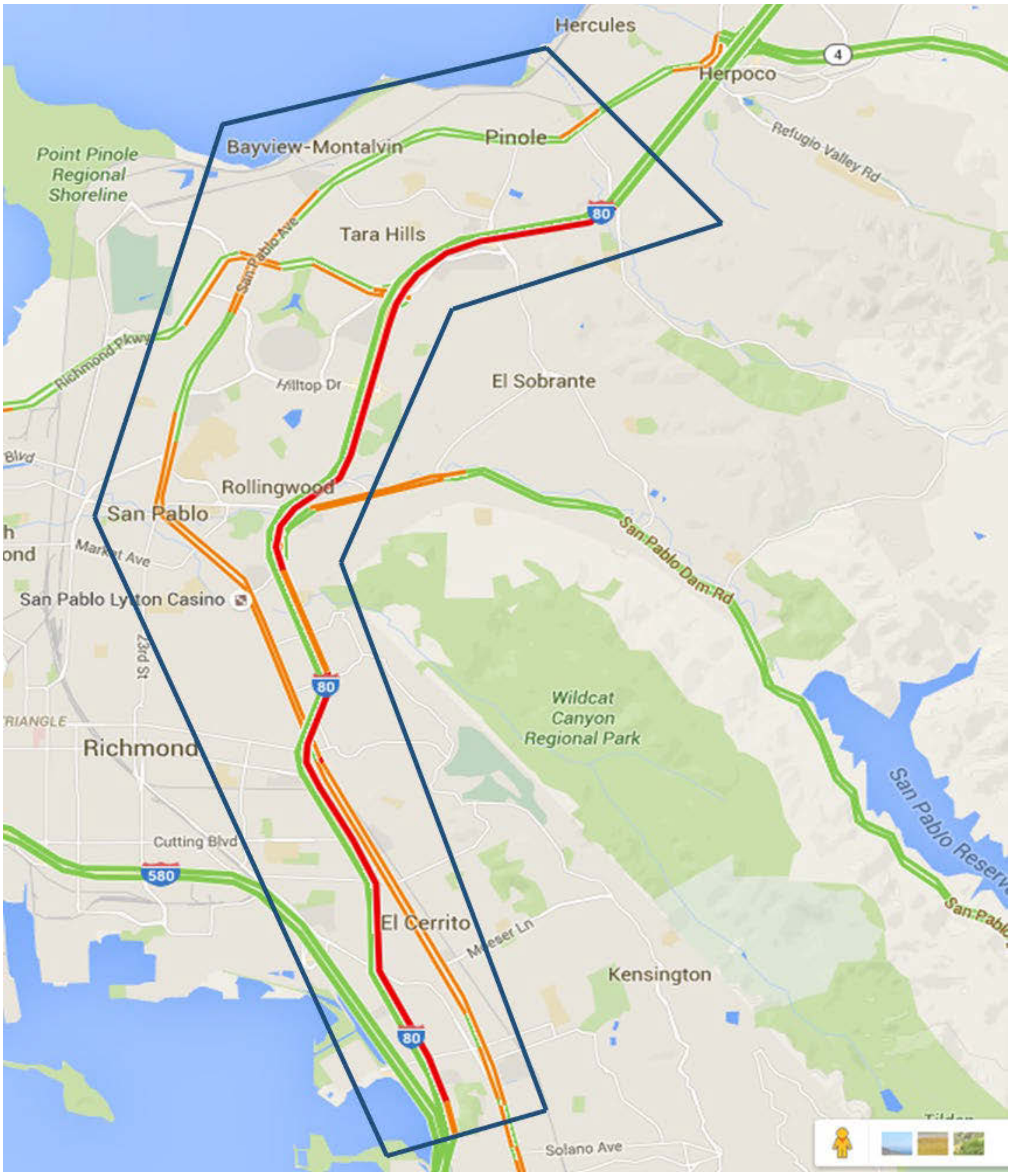

Figure 3.1 Map of I-80 Northbound PM Peak and San Pablo Ave.

3.2.2 Candidate Site #2: I-680 Northbound AM Peak

Segment of interest: Capitol Expy. To Berryessa Rd.

Parallel arterial: Capitol Ave.

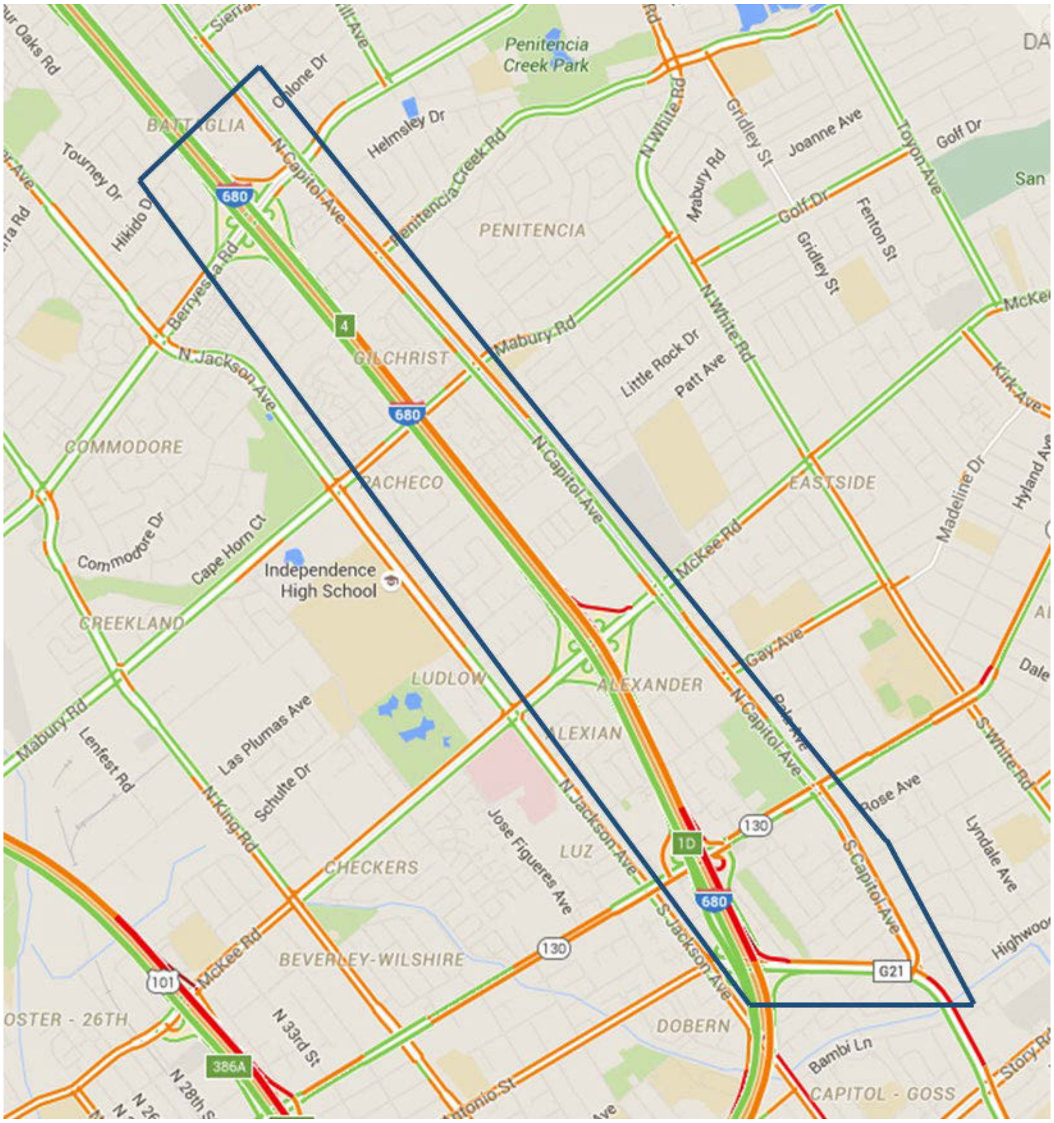

Figure 3.2 Map of I-680 Northbound AM Peak and Capitol Ave.

3.2.3 Candidate Site #3: I-680 Northbound AM Peak

Segment of interest: E. San Antonio St./Capitol Expy. to Berryessa Rd.

Parallel arterial: Jackson Ave.

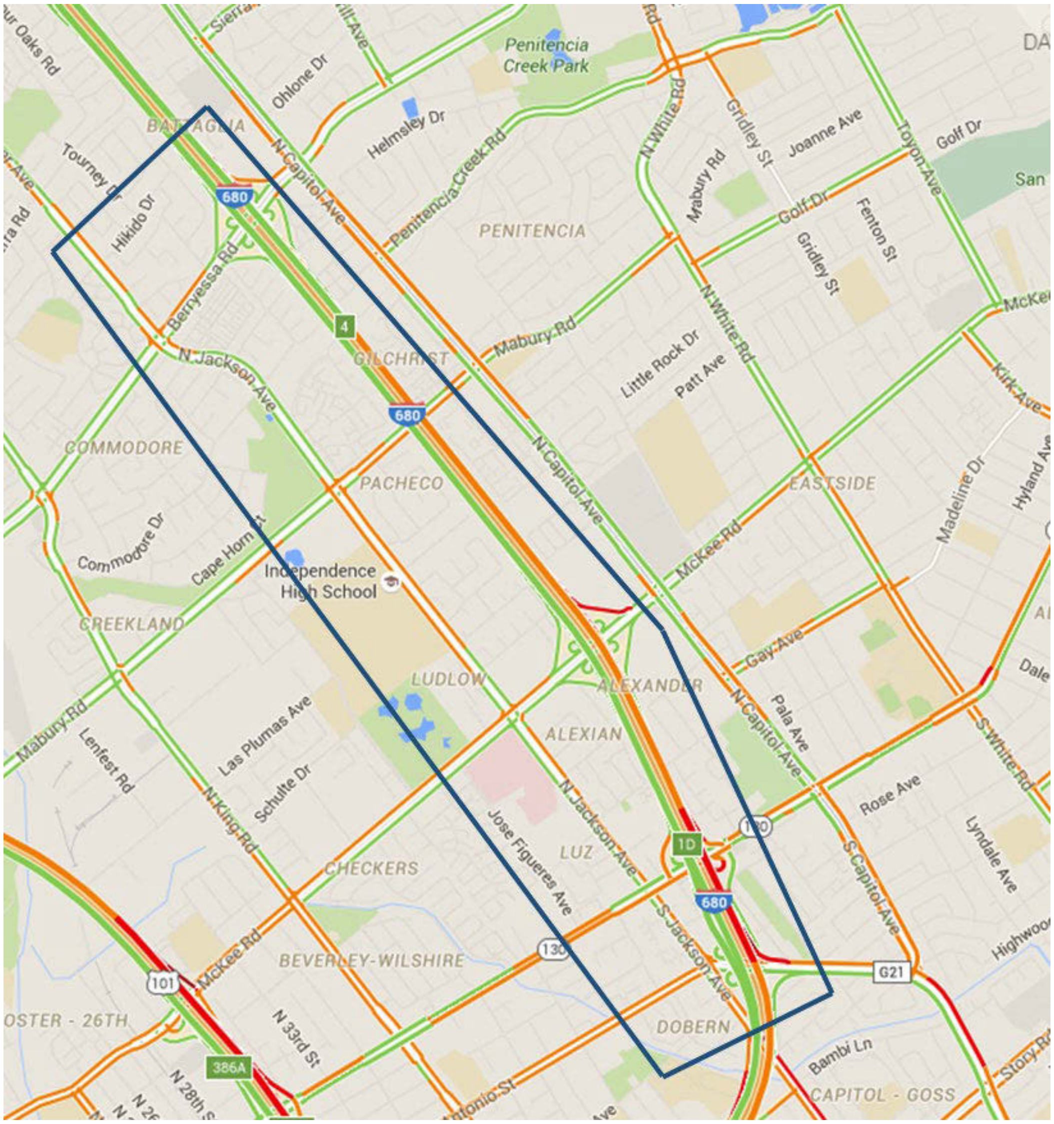

Figure 3.3 Map of I-680 Northbound AM Peak and Jackson Ave.

3.2.4 Candidate Site #4: SR-87 Northbound AM Peak

Segment of interest: Branham Ln. to W. Alma Ave.

Parallel arterial: Almaden Expy.

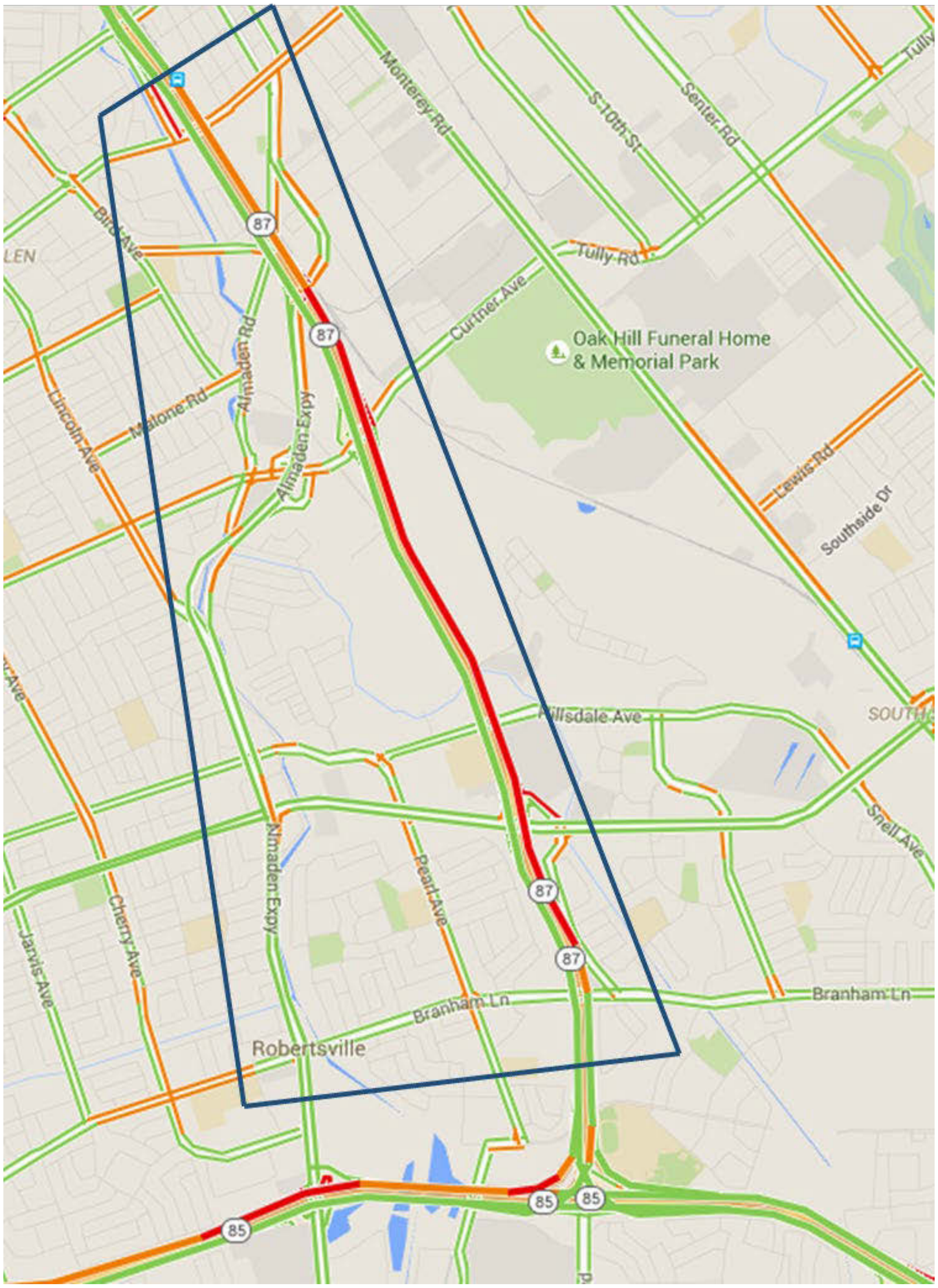

Figure 3.4 Map of SR-87 Northbound AM Peak and Almaden Expy.

3.2.5 Candidate Site #5: US-101 Northbound AM Peak & PM Peak

Segment of interest: Wilfred Ave. to Baker Ave.

Parallel arterial: Santa Rosa Ave.

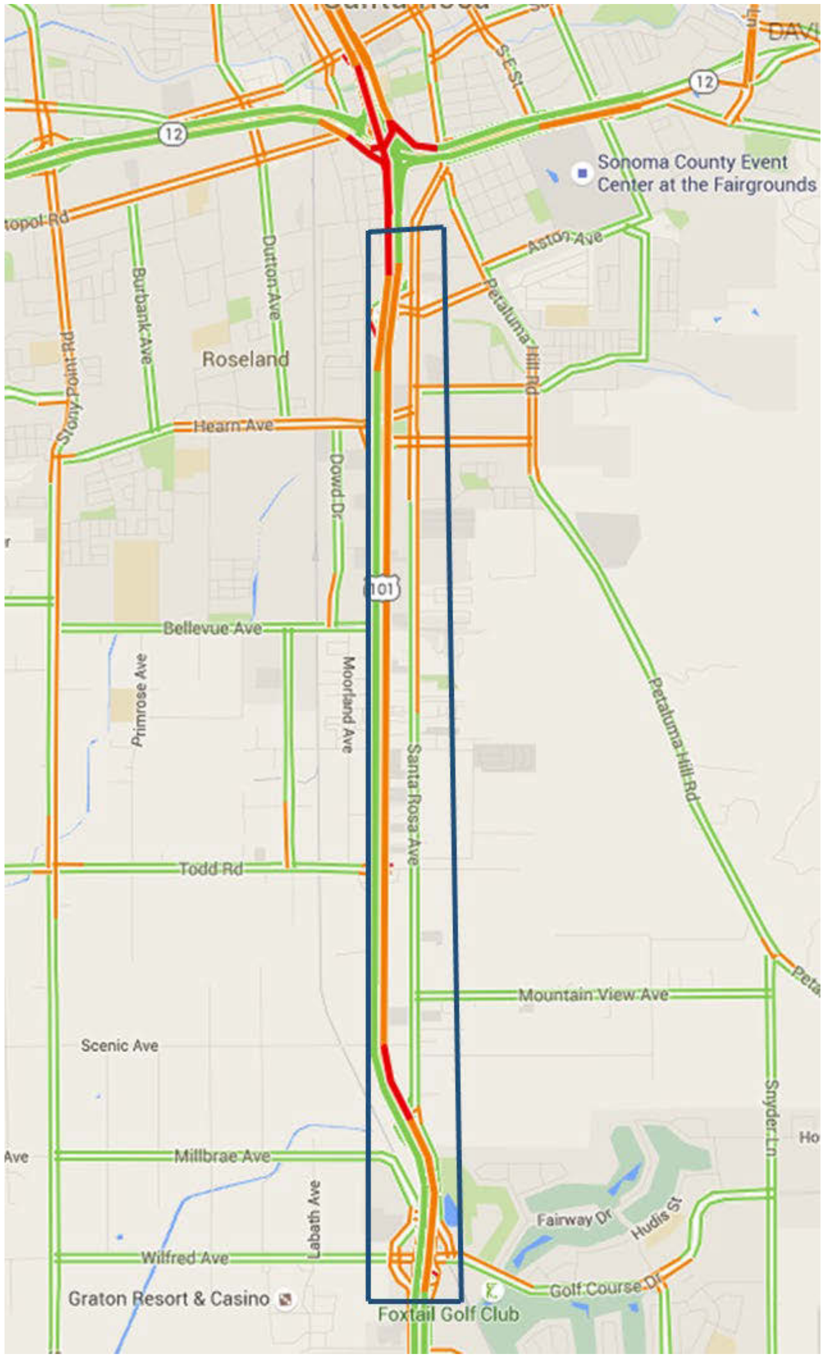

Figure 3.5 Map of US-101 Northbound AM/PM Peak and Santa Rosa Ave.

3.2.6 Candidate Site #6: SR-4 Westbound AM Peak

Segment of interest: Railroad Ave. to Willow Pass Rd.

Parallel arterial: Leland Rd.

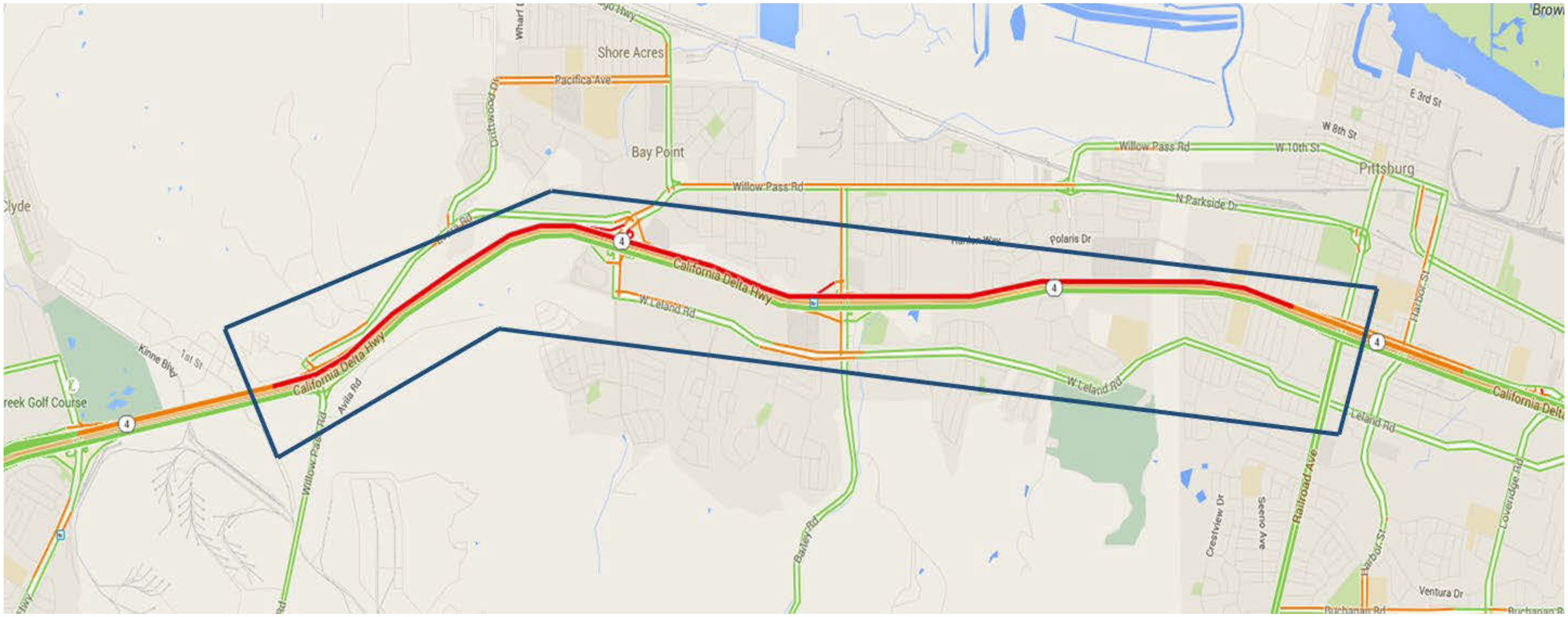

Figure 3.6 Map of SR-4 Westbound AM Peak and Leland Rd

The six candidate sites, illustrated above, were further reduced to two sites: I-680 Northbound AM Peak (with Capitol Ave. as the parallel arterial) and I-680 Northbound AM Peak (with Jackson Ave. as the parallel arterial) after careful consideration and discussion with the project panel. The 4 other candidate sites were not selected by the project team, due to lack of coordination from the city-level jurisdictions in charge of the arterial traffic signal operations.

3.3 Recommendation

After careful consideration, the project team selected I-680 Northbound AM Peak with Capitol Ave. as the parallel arterial. While the two final candidate sites do share the same segment of freeway and both arterials have desirable road geometries – the site with Capitol Ave. as the parallel arterial emerged as being more desirable field test site, for the following reasons:

- A majority of the traffic heading onto the congested northbound direction of the freeway approaches from the east side of the freeway, which is the area surrounding Capitol Ave.
- The City of San Jose advised that several upgrades for Jackson Ave., including pedestrian signals and narrower road geometries, are being planned. In addition, the project team expressed concern about the impact of transit signal priority (TSP) (implemented for light rail vehicles operating in the median of Capitol Ave.). However, the relatively low frequency (every 15 minutes) and the field observations help us confirm that it is not problematic.

Lastly, the project team was required to adjust the scope of the project in order to account for institutional barriers in Santa Clara County. The most upstream signalized intersection was removed from the study site, as was the most upstream freeway bottleneck (at Capitol Expy., on-ramp). The finalized study site is shown in Figure 3.7.

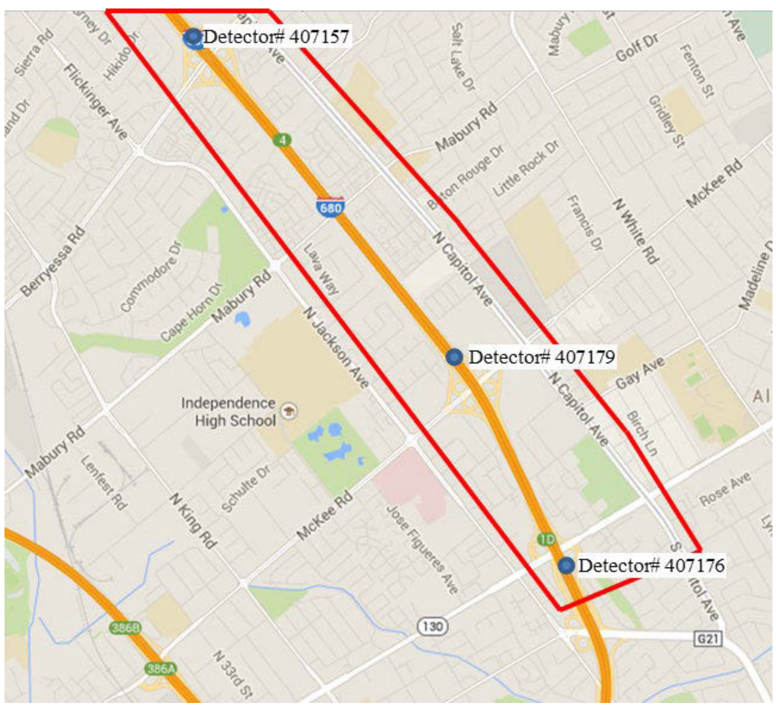

Figure 3.7 Map of Updated Study Site and Freeway Detector Locations

3.4 Details of the Selected Site

As shown in Figure 3.7, the selected test site is a 4-mile section of I-680 from Alum Rock Ave. to Berryessa Rd. in San Jose, California. There are three recurrent bottlenecks on this stretch of I-680; they are located near the on-ramps from Berryessa Rd., McKee Rd., and Alum Rock Ave. High on-ramp demand (entering the northbound freeway mainline during the morning peak (7:30-9:30 AM) is responsible for all three of the bottlenecks.

Peak hour traffic has grown on this northbound section of I-680. The section of highway under study connects this growing employment center to the densely populated, residential region of the southern section of the City of San Jose. There are many trips between the densely populated residential areas surrounding San Jose, in the south, to the employment centers in Fremont and Milpitas in the north, during the morning peak period.

This section of the freeway has 4 lanes in each direction, whereas the parallel arterial Capitol Ave., as well as Alum Rock Ave. and Berryessa Rd. all have 2 lanes in each direction. McKee Rd. has 3 lanes in each direction. Typically, merging traffic from the arterial causes average speed to decrease to about 20 mph near the Alum Rock on-ramp, 30 mph near the McKee on-ramp, and 40 mph near the Berryessa on-ramp. Refer to Figure 3.8 for a contour plot of average speeds of the selected freeway segment during a typical morning peak, and Figures 3.9-3.11 for flow and speed time series of each bottleneck during a typical morning peak.

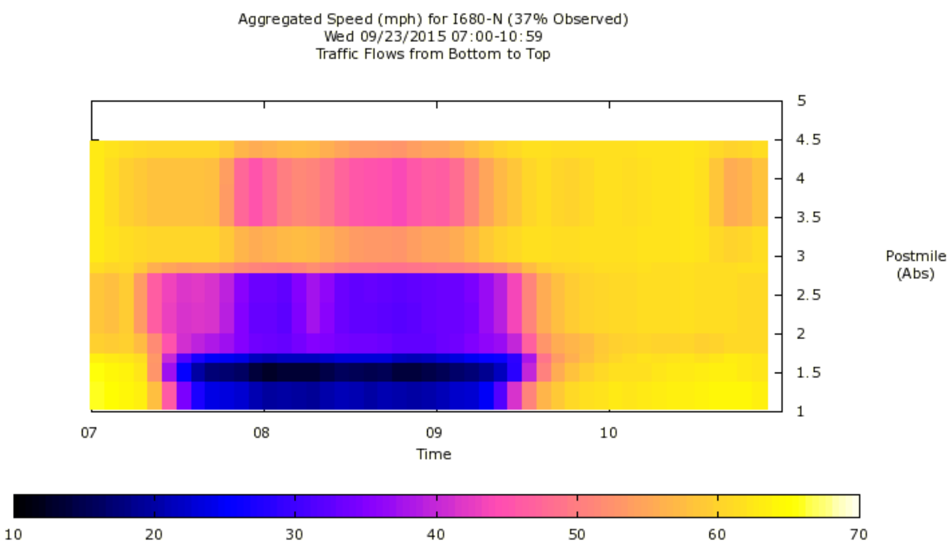

Figure 3.8 Speed Contour Plot of I-680 Northbound (Alum Rock Ave. to Berryessa Rd.)

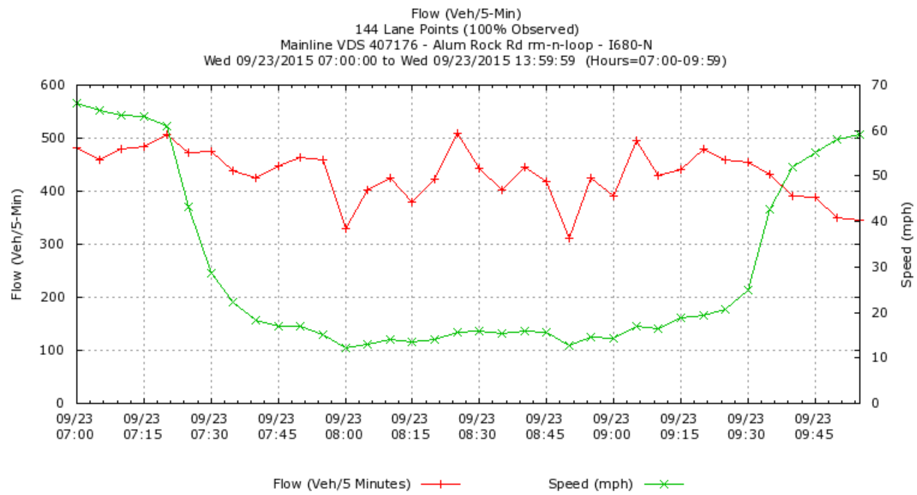

Figure 3.9 Flow and Speed of I-680 Northbound near Alum Rock Ave. On-ramp

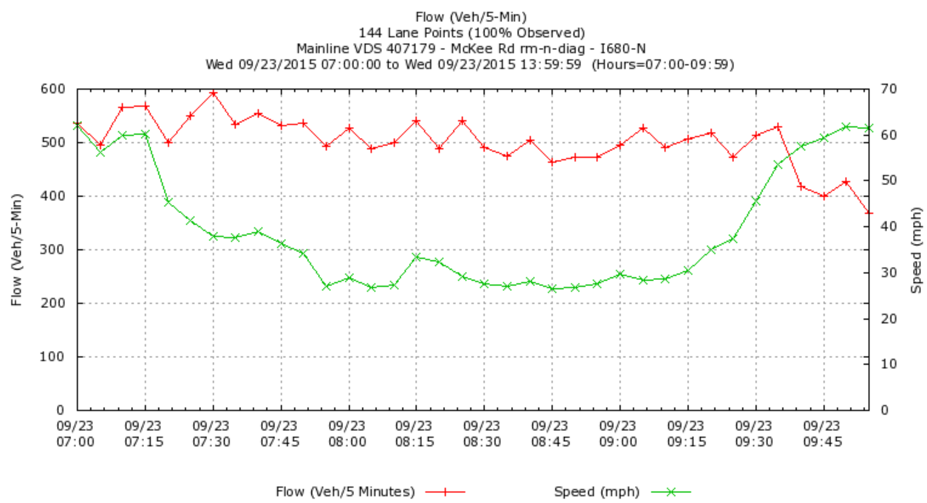

Figure 3.10 Flow and Speed of I-680 Northbound near McKee Rd. On-ramp

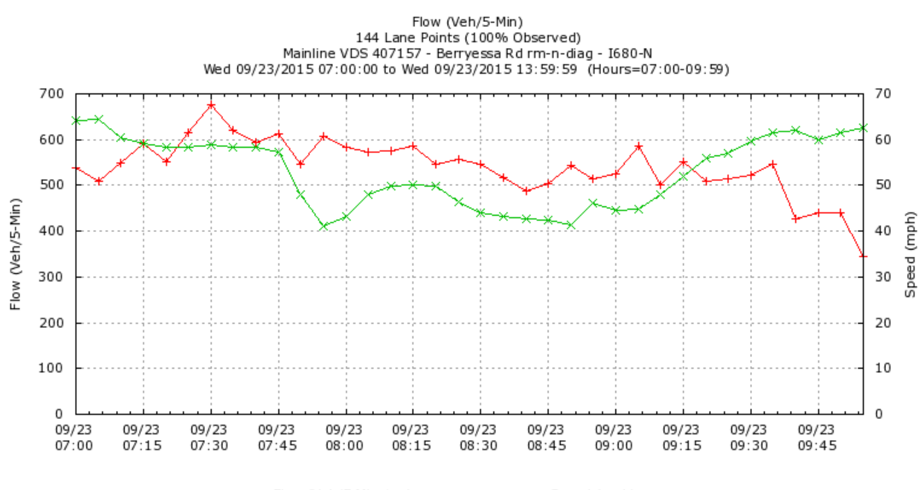

Figure 3.11 Flow and Speed of I-680 Northbound near Berryessa Rd. On-ramp

All of the on-ramps in this corridor are metered and the ramp meters operate under the local responsive demand-capacity approach. The metering rates are assigned based on various thresholds of freeway mainline occupancies immediately upstream of the merging or weaving area. The metering rates and their respective occupancy thresholds for each on-ramp are shown in Table 3.1 and Table 3.2.

Table 3.1: Alum Rock Ave. AM Peak Ramp Metering Rates

Time of Day	Alum Rock Ave. (loop)		Alum Rock Ave. (diagonal)	
	Mainline Occupancy	Meter Rate	Mainline Occupancy	Meter Rate
6:00 – 7:00 AM	⩽ 3%	No metering	⩽ 4%	No metering
	3% to 12%	900 vph/lane	4% to 14%	900 vph/lane
	⩾ 12%	300 vph/lane	⩾ 14%	560 vph/lane
7:00 – 7:30 AM	⩽ 3%	No metering	⩽ 4%	No metering
	3% to 5%	900 vph/lane	4% to 7%	900 vph/lane
	⩾ 5%	300 vph/lane	⩾ 7%	480 vph/lane
7:30 – 9:00 AM	⩽ 3%	No metering	⩽ 4%	No metering
	3% to 5%	900 vph/lane	4% to 7%	900 vph/lane
	⩾ 5%	400 vph/lane	⩾ 7%	480 vph/lane
9:00 – 10:00 AM	⩽ 10%	No metering	⩽ 12%	No metering
	10% to 12%	900 vph/lane	12% to 14%	900 vph/lane
	⩾ 12%	360 vph/lane	⩾ 14%	560 vph/lane

Table 3.2: McKee Rd. and Berryessa Rd. AM Peak Ramp Metering Rates

Time of Day	McKee Rd.		Berryessa Rd.	
	Mainline Occupancy	Meter Rate	Mainline Occupancy	Meter Rate
6:00 – 7:00 AM	⩽ 4%	No metering	⩽ 3%	No metering
	4% to 14%	900 vph/lane	3% to 14%	900 vph/lane
	⩾ 14%	420 vph/lane	⩾ 14%	420 vph/lane
7:00 – 7:15 AM	⩽ 4%	No metering	⩽ 3%	No metering
	4% to 6%	900 vph/lane	3% to 5%	900 vph/lane
	⩾ 6%	400 vph/lane	⩾ 5%	560 vph/lane
7:15 – 7:30 AM	⩽ 4%	No metering	⩽ 3%	No metering
	4% to 6%	900 vph/lane	3% to 5%	900 vph/lane
	⩾ 6%	560 vph/lane	⩾ 5%	560 vph/lane
7:30 – 7:45 AM	⩽ 4%	No metering	⩽ 3%	No metering
	4% to 6%	900 vph/lane	3% to 5%	900 vph/lane
	⩾ 6%	720 vph/lane	⩾ 5%	600 vph/lane
7:45 – 8:00 AM	⩽ 4%	No metering	⩽ 3%	No metering
	4% to 6%	900 vph/lane	3% to 5%	900 vph/lane
	⩾ 6%	720 vph/lane	⩾ 5%	650 vph/lane

8:00 – 8:15 AM	⩽ 4%	No metering	⩽ 3%	No metering
	4% to 6%	900 vph/lane	3% to 5%	900 vph/lane
	⩾ 6%	600 vph/lane	⩾ 5%	600 vph/lane
8:15 – 8:30 AM	⩽ 4%	No metering	⩽ 3%	No metering
	4% to 6%	900 vph/lane	3% to 5%	900 vph/lane
	⩾ 6%	400 vph/lane	⩾ 5%	600 vph/lane
8:30 – 9:00 AM	⩽ 4%	No metering	⩽ 3%	No metering
	4% to 6%	900 vph/lane	3% to 5%	900 vph/lane
	⩾ 6%	400 vph/lane	⩾ 5%	510 vph/lane
9:00 – 10:00 AM	⩽ 12%	No metering	⩽ 12%	No metering
	12% to 14%	900 vph/lane	12% to 14%	900 vph/lane
	⩾ 14%	420 vph/lane	⩾ 14%	450 vph/lane

The signalized intersections of this corridor operate with time of day (TOD) coordinated actuated timing plans. The existing cycle lengths are relatively long (130 to 160 seconds) and the signal-timing plan provides progression (or green wave- A vehicle with the posted speed, once catch the green of the upstream section, is likely catch the green of all downstream traffic signals along the corridor.) to the northbound direction, which experiences higher demand. When a vehicle encounter green signals at successive signalized intersections, this coordinated signal timing is termed progressions. Refer to Tables 3.12 to 3.15 for turning movement volumes and signal timing plans of the four major signalized intersections during a typical weekday morning peak.

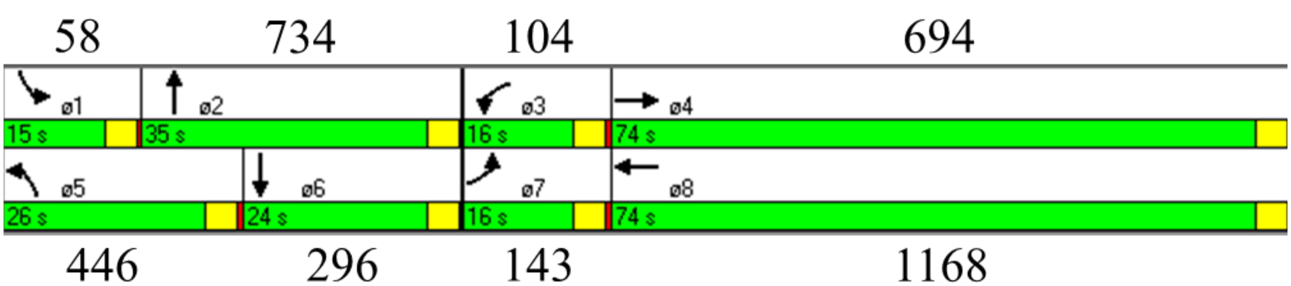

Figure 3.12 Signal Timing Plan and Typical Volumes at Capitol Ave. & Alum Rock Ave.

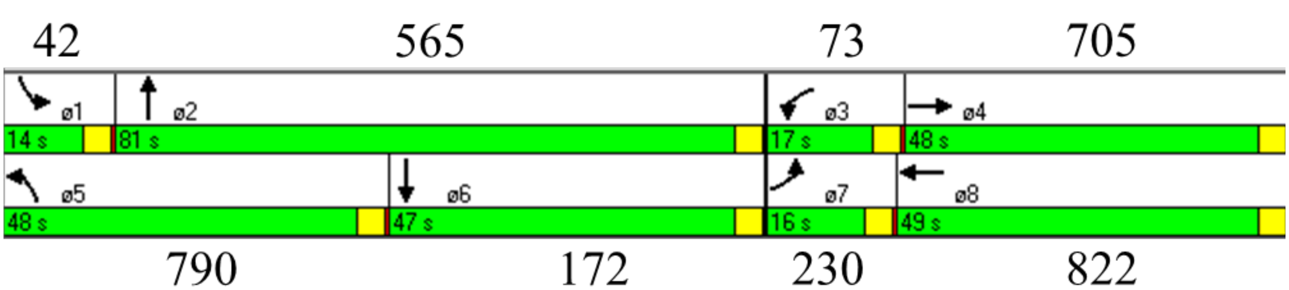

Figure 3.13 Signal Timing Plan and Typical Volumes at Capitol Ave. & Berryessa Rd.

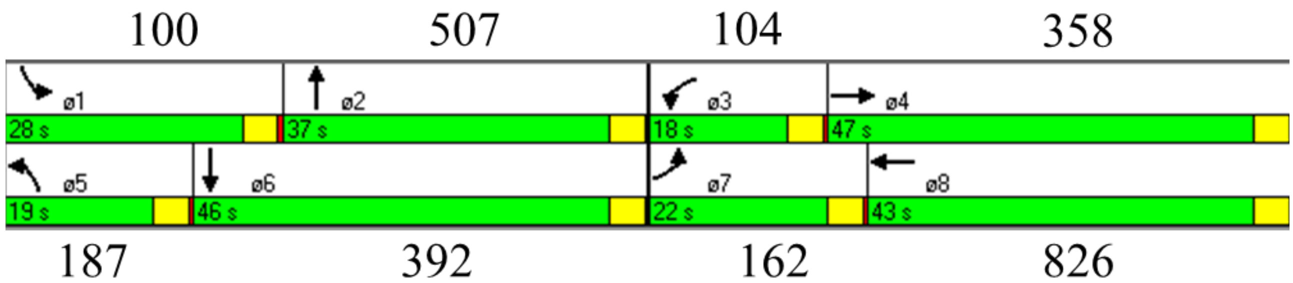

Figure 3.14 Signal Timing Plan and Typical Volumes at Capitol Ave. & Mabury Rd.

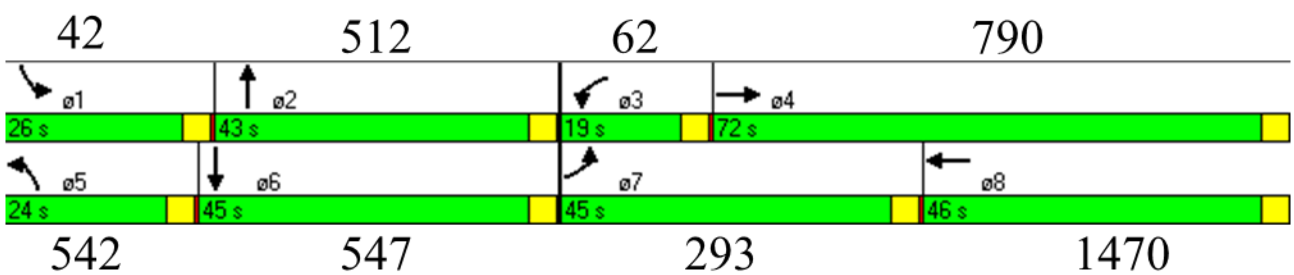

Figure 3.15 Signal Timing Plan and Typical Volumes at Capitol Ave. & McKee Rd.

Chapter 4. Proposed Control Strategies for Coordination

A conceptually simple control strategy that is easy to implement, is proposed to resolve the problem of inefficient control of freeway ramp metering and arterial traffic signals. Currently, arterial signals fail to recognize oversaturation of metered on-ramps. These signals employ long cycle lengths and provide progression to the heavier direction. These three factors (independent operation of arterial signals, with long cycle times and progression) cause the activation of queue override by the ramp meter, creating a bottleneck to traffic flow and then congestion on the mainline freeway.

The proposed algorithm is an extension of a previously developed control algorithm, by PATH, for coordinating a single freeway metered on-ramp with an adjacent isolated signalized intersection. [12]

4.1 Freeway Ramp Metering

The proposed arterial signal control strategy will be integrated into two different ramp-metering strategies: local traffic responsive and coordinated ramp metering strategies. Caltrans District 4 is currently implementing local traffic responsive ramp metering at all of the on-ramps in the selected corridor. This ramp-metering algorithm is based on the demand-capacity approach. The metering rates are assigned according to various thresholds of freeway mainline occupancies immediately upstream of the merging or weaving area. Detailed ramp operations are discussed in the previous section, on site selection.

In the latter portion of the field test (proposed by this report) the coordinated ramp-metering algorithm replaced the local responsive ramp-metering algorithm. The coordinated ramp-metering algorithm uses a simulation model to determine the on-ramp metering rate on each freeway section to minimize the total travel time and maximizes the total distance traveled - subject to appropriate constraints.

The simulation model employed the coordinated ramp-metering algorithm is based on the cell transmission macroscopic model (CTM). CTM predicts macroscopic traffic behavior on a given corridor by evaluating the flow and density at finite number of intermediate points at different time steps. The number of vehicles in each cell (segment of freeway between adjacent on/off-ramps) are derived from flow and density calculations, in each time step. Flow and density data are initially determined from the loop detector data, which are collected at 30 seconds and stored at the Traffic Management Center (TMC). The coordinated ramp metering algorithm requires real time data for mainline flow, speed, and occupancy (to estimate density), as well as on-ramp and off-ramp flow.

The proposed control strategy, for coordinating arterial traffic signals and freeway ramp metering, is not limited to any specific freeway ramp-metering algorithm. The ramp-metering algorithm does not require modification, except for the queue override function that releases on-ramp queues onto the freeway to prevent arterial spillback. Detailed description of the coordinated ramp-metering algorithm can be found in Appendix A.

Once more efficient arterial control is implemented, adjacent on-ramp meters should no longer experience queue spillback – and no longer implement queue override. With an updated control system, the arterial traffic signals will now be award of the on-ramp saturation levels and would compensate for the negative impact of on-ramp queues – through modified signal timing. Among the positive impacts is the ability to mitigate the capacity drop that is a result of on-ramp metering queue override (releasing ramp vehicle queue inventory onto the freeway).

4.2 Arterial Traffic Signals

There have been numerous proposed mathematical models for estimating delay at signalized intersections. The most commonly used approach is the approximate steady state expression of delay proposed by Webster. The delay of each phase or turning movement at a signalized intersection is the following:

$$d = \frac{c[1-(g/C)]^2}{2[1-(g/C)x]} + \frac{x^2}{2v(1-x)} - 0.65\left(\frac{C}{v^2}\right)^{\frac{1}{2}} x^{2+5(g/C)} \tag{4.17}$$

where,

d: average delay per vehicle (sec)
C: cycle length (sec)
g: effective cycle length (sec)
x: degree of saturation (sec)
v: arrival rate (veh/sec)

The first two terms correspond to the delay modelled using an *M/D/1* queuing system, which has Poisson arrival, deterministic service time, and single server. The last term, in equation 4.17, corresponds to a correction term calibrated based on field data, in order to better reflect the real world conditions.

Furthermore, the effective green can be expressed as the following:

$$g = \frac{y}{Y} \cdot (C - L) \tag{4.18}$$

where,

y: ratio of arrival rate and saturation flow
Y: sum of y's of the cycle
L: total lost time (sec)

Substitute Equation 4.18 into Equation 4.17, the delay can expressed as a function of cycle length. An optimization model that minimizes the overall delay of the signalized intersection can be formulated to determine the optimal cycle length C_o:

$$C = \frac{1.5L + 5}{1 - Y} \tag{4.19}$$

Given the optimal cycle length, the corresponding effective green times can be determined using Equation 4.19. Additionally, offsets can be tuned to provide progression to the heavier direction if the signal timing plan is developed for an arterial corridor. Such approach has been widely used by the local transportation agencies and has been incorporated into commercial software for developing signal timing plans. Despite the popularity, this approach does not provide efficient signal timing plans for arterial intersections adjacent to saturated on-ramps with queue storage constraints.

During peak hours, the freeway ramp meter restricts the flow of the on-ramp in order to prevent merge-related, capacity drop on the freeway. This limits the capacity of the on-ramp, and with the higher demand for on-ramp access from the adjacent arterial, excess accumulation can be observed at the on-ramp and eventually propagates to the arterial intersection. This situation exists, because arterial traffic signals and freeway ramp meters currently operate independent of each other. The arterial traffic signal timing plans, outlined in Equation 4.17 to 4.19, do not consider the freeway on-ramp downstream and are developed under the assumption that the arterials do not have queue storage constraints. As a result, for relatively high demand during the peak hours, the optimal cycle length is relatively long in order to maximize capacity and reduce start-up lost time.

Current signal timing design operates long green times that feed the freeway on-ramp. Due to on-ramp storage limits, portions of arterial signal green times cannot be effectively utilized – as the on-ramp queue storage space fills up before the green duration terminates. This causes vehicles to block the intersection and imposes unnecessary delay on the conflicting movements. This overflow of vehicles activates queue override, which releases vehicles from the on-ramp to the freeway in order to mitigate queue spillback at the arterial. Unfortunately, activating queue override introduces more merging traffic to the freeway mainline, which triggers more lane changes and speed reduction in the median lane, and can result in capacity drop on the freeway.

Signalized arterial intersections, near the freeway on-ramps, have characteristics that are similar to a series of adjacent signalized intersection in over-saturated conditions. Signal timing plans for arterials, adjacent to freeway on-ramps, should be similar to signal timing plans of over-saturated arterials. Long cycle lengths and long green durations are not feasible, under these conditions. Cycle lengths and green durations should consider the on-ramp queue storage space and should be designed to avoid queue spillback [17].

To illustrate the improved signal timing approach, consider a signalized intersection with 4 phases. In most real world cases, not every phase serves the turning movements that have on-ramp access. Thus, in this example, consider phase 1 and phase 2 as the turning movements feeding the on-ramp, and the remaining phases do not have on-ramp access.

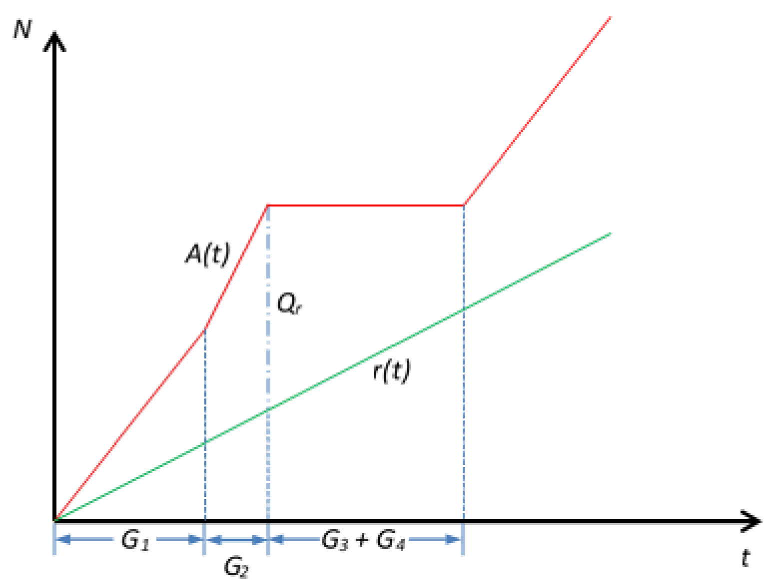

Figure 4.1 Queueing Diagram of Freeway On-ramp during a Signal Cycle

The queuing diagram in Figure 4.1 illustrates the upstream arrival patterns *A(t)*, downstream departure rate *r(t)*, and the excess accumulation of the freeway on-ramp when ramp metering is active and the demand from the upstream arterial exceeds the capacity of the metered on-ramp. Based on the queuing diagram, when the freeway on-ramp is metered at rate $r(t)$, the high arrival rate from phase 1 of the upstream signalized intersection results in excess accumulation on the on-ramp. Similarly, the excess accumulation grows after phase 2 begins. Since the arrival rates from both phase 1 and phase 2 are higher than the ramp meter rate, the green time for phase 1 and phase 2 must terminate at or before the excess accumulation reaches the maximum on-ramp queue storage capacity Q_r. The subsequent phases, which do not feed the on-ramp, can be served earlier, and when the subsequent phases terminate, a portion of the on-ramp will be available for storing the excess accumulation from phase 1 and phase 2 in the next cycle. This results in a relatively shorter cycle length. If longer cycle lengths were used thus the green times for phase 1 and phase 2 were terminated after the excess accumulation reaches the maximum on-ramp queue storage capacity, the arrival curve $A(t)$ afterwards must be parallel to the departure curve with slope $r(t)$, the ramp meter rate. This is because the queue storage capacity of the on-ramp cannot be exceeded, thus the queue must be stored at the upstream arterial. In other words, if long green durations were provided to phase 1 and phase 2, vehicles from phase 1 and phase 2 would discharge at rates lower than the saturation flow rate for some portion of the cycle, which lead to lower capacity of the intersection. This can also impose unnecessary delay in the conflicting directions. The mathematical expressions for this concept of signal control are described in the next few paragraphs.

4.2.1 On-Ramp Excess Vehicle Accumulation

First, the excess accumulation of the on-ramp must be estimated. Using the ramp meter rate $r(t)$ that is updated each time step t, the on-ramp excess accumulation at time step t can be determined based on the following process:

$$Q(0) = 0$$

$$Q(1) = Q(0) + A(1) - D(1)$$

$$Q(2) = Q(1) + A(2) - D(2)$$

$$\vdots$$

$$Q(t) = Q(t-1) + A(t) - D(t)$$

$Q(t)$ is the on-ramp excess accumulation at the end of time step t, $A(t)$ is the number of arrivals from upstream of the on-ramp during time step t, and $D(t)$ is the number of departures from ramp meter during time step t. The arrivals and departures can be measured by the loop detectors at the upstream and downstream ends of the freeway on-ramp, respectively. The on-ramp excess accumulation should be updated at the end of every cycle in order to perform real time control.

4.2.2 Arterial Signal Cycle Length: Algorithm to Manage Ramp Queue

Equation 4.20 is developed using the queuing diagram in Figure 4.1, to ensure that the green time for phase 1 and phase 2 must terminate at or before the excess accumulation reaches the maximum on-ramp queue storage capacity Q_r, therefore imposes an upper limit for the cycle length:

$$Q(t-1) + G_1 \cdot s_1 \cdot \beta_1 + G_2 \cdot s_2 \cdot \beta_2 - G_1 \cdot r(t) - G_2 \cdot r(t) \leq Q_r \qquad (4.20)$$

where,

G_1: effective green time of phase 1
G_2: effective green time of phase 2
s_1: saturation flow of phase 1
s_2: saturation flow of phase 2
β_1: percentage of demand of phase 1 that access the on-ramp
β_2: percentage of demand of phase 2 that access the on-ramp
$Q(t-1)$: residual on-ramp queue from the previous cycle.
$r(t)$: ramp metering rate

Similar to Equation 4.18, the effective green times can be expressed as functions of cycle length. Thus, G_1 and G_2 are expressed as the following:

$$G_1 = \frac{y_1}{Y} \cdot (C - 4l) \tag{4.21}$$

$$G_2 = \frac{y_2}{Y} \cdot (C - 4l) \tag{4.22}$$

where,

y_1: ratio of arrival rate and saturation flow of phase 1
y_2: ratio of arrival rate and saturation flow of phase 2
Y: sum of y's of the cycle
C: cycle length (sec)
l: lost time of each phase (sec)

Substitute Equations 4.21 and 4.22 into equation 4.20, equation 4.20 can be expressed in terms of cycle length. Solving for cycle length in terms of the rest of the variables, the upper limit of cycle length is the following:

$$C \leq \frac{[Q_r - Q(t-1) + r(t) \cdot 2l] \cdot Y + 4l \cdot \left[\sum_{i=1,2} s_i \beta_i y_i - \sum_{i=1,2} r(t) y_i\right]}{\left[\sum_{i=1,2} s_i \beta_i y_i - \sum_{i=1,2} r(t) y_i\right]} \tag{4.23}$$

The upper limit of the cycle length must be updated at the end of every cycle in order to perform real time control that coordinates with freeway ramp metering.

4.2.3 Arterial Signal Operational Cases

Case 1: Sufficient Queue Storage
If the on-ramp has sufficient queue storage space, the upper limit of the cycle length is typically higher than the optimal cycle length determined using Webster's formula or other similar methods, thus in this case, the cycle length remains unchanged.

Case 2: Insufficient Queue Storage
When there is limited on-ramp queue storage, the upper limit of the cycle length is typically lower than the optimal cycle length determined using Webster's formula (or other similar methods). In such a case, the upper limit takes precedence in order to reduce the cycle length and prevent queue spillback at the on-ramp and its nearby arterial intersections. With the new cycle length, the effective green times are still determined using Equation 4.18.

4.2.4 Arterial Signal Dual Ring Operation

Many signalized intersection are timed using the dual ring structure. A typical example shown in Figure 4.2, which illustrates a four-leg signalized intersection with 8 phases - each phase represents a turning movement. Phases 1 and 3 correspond to the leading left turns while phases 6 and 8 correspond to the lagging left turns, and the rest of the phases represent the through movements. There is a center barrier to prevent conflicts among turning movement of the north-south and the east-west directions.

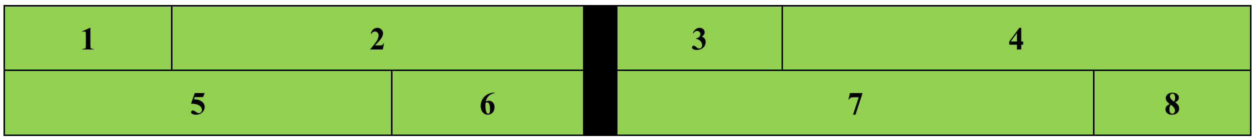

Figure 4.2 Typical Dual Ring Structure of a Four-leg Intersection

As an example, phase 1 and phase 2 correspond to the turning movements feeding the on-ramp. Similarly, Figure 4.3 illustrates the upstream arrival patterns, downstream departure rate, and the excess accumulation of the freeway on-ramp when ramp metering is active and the demand from the upstream arterial exceeds the capacity of the metered on-ramp. The methodology for determining the upper limit of the cycle length is the same as before, with the exception of how the effective green times are computed.

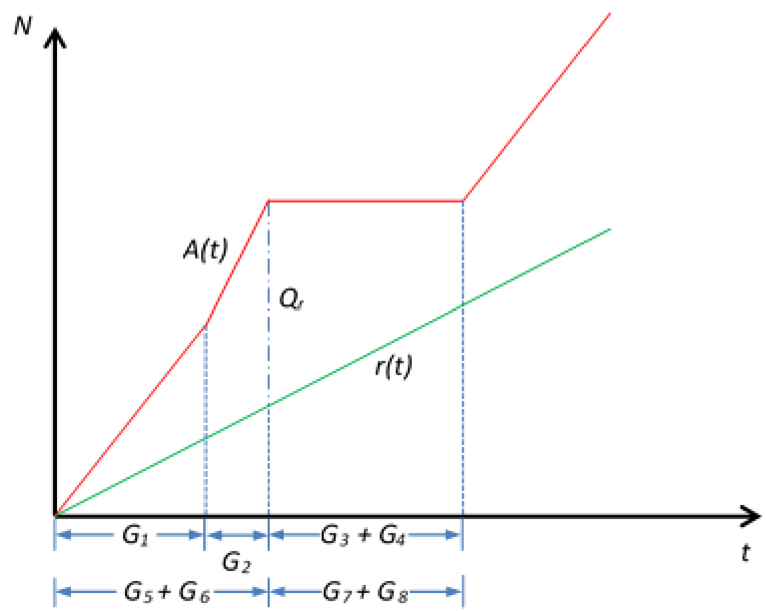

Figure 4.3 Queueing Diagram of Freeway On-ramp during a Signal Cycle (Dual Ring)

Cycle Time for Dual Ring Operation

To formulate the expression for the effective green times, let y_L denote the ratio of arrival rate and saturation flow of all phases to the left of the barrier, as illustrated in Figure 4.2 (conceptually the same as Figure 4.1). Similarly, let y_R denote the ratio of arrival rate and saturation flow of all phases to the right of this same barrier. They are defined as the following:

$$y_L = \max\,(y_1 + y_2, y_5 + y_6) \tag{4.24}$$

$$y_R = \max\,(y_3 + y_4, y_7 + y_8) \tag{4.25}$$

Where y_i denotes the ratio of arrival rate and saturation flow of phase i, for $i = 1,2, \dots, 7,8$.

Then, the effective green times for phase 1 and phase 2 are defined as the following:

$$G_1 = \frac{y_1}{y_1 + y_2} \cdot \frac{y_L}{y_L + y_R} \cdot (C - 4l) \tag{4.26}$$

$$G_2 = \frac{y_2}{y_1 + y_2} \cdot \frac{y_L}{y_L + y_R} \cdot (C - 4l) \tag{4.27}$$

For the remaining phases, the effective green times are:

$$G_3 = \frac{y_3}{y_3 + y_4} \cdot \frac{y_R}{y_L + y_R} \cdot (C - 4l) \tag{4.28}$$

$$G_4 = \frac{y_4}{y_3 + y_4} \cdot \frac{y_R}{y_L + y_R} \cdot (C - 4l) \tag{4.29}$$

$$G_5 = \frac{y_5}{y_5 + y_6} \cdot \frac{y_L}{y_L + y_R} \cdot (C - 4l) \tag{4.30}$$

$$G_6 = \frac{y_6}{y_5 + y_6} \cdot \frac{y_L}{y_L + y_R} \cdot (C - 4l) \tag{4.31}$$

$$G_7 = \frac{y_7}{y_7 + y_8} \cdot \frac{y_R}{y_L + y_R} \cdot (C - 4l) \tag{4.32}$$

$$G_8 = \frac{y_8}{y_7 + y_8} \cdot \frac{y_R}{y_L + y_R} \cdot (C - 4l) \tag{4.33}$$

Substitute Equations 4.26 and 4.27 into Equation 4.20, Equation 4.20 can be expressed in terms of cycle length. Solving for cycle length in terms of the rest of the variables, the upper limit of cycle length is the following:

$$C \leq \frac{[Q_r - Q(t-1) + r(t) \cdot 2l] \cdot (y_L + y_R)}{\left[\frac{(s_1 \cdot \beta_1 - r(t)) \cdot y_1 y_L}{y_1 + y_2} + \frac{(s_2 \cdot \beta_2 - r(t)) \cdot y_2 y_L}{y_1 + y_2}\right]} \tag{4.34}$$

If phase 2 and phase 3, instead of phase 1 and phase 2, correspond to the turning movements feeding the on-ramp, then the upper limit of the cycle length becomes:

$$C \leq \frac{[Q_r - Q(t-1) + r(t) \cdot 2l] \cdot (y_L + y_R)}{\left[\frac{(s_2 \cdot \beta_2 - r(t)) \cdot y_2 y_L}{y_1 + y_2} + \frac{(s_3 \cdot \beta_3 - r(t)) \cdot y_2 y_L}{y_3 + y_4}\right]} \tag{4.35}$$

Similar to previous discussions, the upper limit of the cycle length must be updated at the end of every cycle in order to perform real time control that coordinates with freeway ramp metering. The effective green times can be computed using Equations 4.26 through 4.33.

4.2.5 Coordination of the Signals on the Arterial as Oversaturated

Lastly, adjacent arterial signals should not be coordinated to ensure uninterrupted flow and progression, on the arterial, because every major intersection is capacity-constrained – due to the on-ramp nearby. Thus, coordination of adjacent signals should be similar to the coordination of oversaturated signals.

As with signals on oversaturated arterials, signals with adjacent over saturated freeway ramps should incorporate a simultaneous offset (zero offset) when the on-ramp is congested. Offset is defined as the difference in time from a reference point in the cycle at the upstream intersection to the same point in the cycle at the downstream intersection. This reference point is usually taken to be the beginning of the main street green. This would allow the residual queue at the downstream intersection to dissipate in time, otherwise, large platoons of vehicles would arrive from upstream intersection before the downstream residual queues are served, which can fill up the limited queue storage space and cause spillback upstream.

Most users of the arterials that are adjacent to freeways travel a short segment to access the nearest freeway on-ramp rather than a long segment of the arterial. Thus progression for a long stretch of arterial is not appropriate. In such case, each signalized intersection should be timed independently to avoid assigning the longest cycle length to all of the signalized intersections, which can impose heavy delays to the conflicting directions and cause queue spillback when queue storage is constrained downstream.

Overall, the above approaches are intended to prevent queue spillback at the on-ramp and further upstream on the arterial. This mitigates any potential penalties imposed on the conflicting directions of the arterial traffic and eliminate the need for queue override at the metered on-ramps.

4.2.6 Modifications for Implementation

The proposed algorithm effectively reduced the cycle length at the signalized intersections near the on-ramp. The existing cycle length is 160 second while the new cycle length under the proposed algorithm would be 120 second (while keeping the same distribution of green times for each phase). However, other practical concerns such as minimum pedestrian crossing times and minimum green times for individual phases were taken into consideration after the discussions with city of San Jose traffic engineers. Thus, the new cycle length would be 145 second after implementing the proposed control strategy; this is still shorter than the existing 160 second cycle (or longer if demand is higher and triggers actuated signals to use maximum green times) and more effective for mitigating queue spillback.

Chapter 5. Evaluation of Proposed Coordination Strategies

The field implementation of proposed strategy for coordination of ramp metering and arterial traffic signals was conducted every weekday morning peak (7:00 AM to 10:00 AM) from Monday April 29, 2019 to Friday August 23, 2019. Four different scenarios were tested during this four-month test period:

- Scenario 1: Caltrans District 4 local responsive ramp metering (see section 3.4 for details) without coordination of ramp metering and arterial traffic signals (baseline)
- Scenario 2: Caltrans District 4 local responsive ramp metering (see section 3.4 for details) with coordination between ramp metering and arterial traffic signals
- Scenario 3: PATH developed coordinated ramp metering (See Appendix A for details) without coordination between ramp metering and arterial traffic signals
- Scenario 4: PATH developed coordinated ramp metering (See Appendix A for details) with coordination between ramp metering and arterial traffic signals

5.1 Field Test Schedule

Testing and evaluation were not conducted for the entirety of this four-month period due to unforeseen events, such as broken ramp metering

Details regarding the field test schedule and events are shown below:

Week 1: Monday, April 29 – Friday, May 3

- Data collection for the scenario 1 (Caltrans District 4 local responsive ramp metering without coordination of ramp metering and arterial traffic signals) officially began
- Fine tuning of software for coordinating Caltrans District 4 ramp metering controller and City of San Jose traffic signal controller, outside of the morning peak (7:00 AM to 10:00 AM)

Week 2: Monday, May 6 – Friday, May 10

- Data collection for the scenario 1 (Caltrans District 4 local responsive ramp metering without coordination of ramp metering and arterial traffic signals) continued
- Fine tuning of software for coordinating Caltrans District 4 ramp metering controller and City of San Jose traffic signal controller, outside of the morning peak (7:00 AM to 10:00 AM)
- The algorithm for coordinating Caltrans District 4 ramp metering controller and City of San Jose traffic signals was ready for deployment at the end of the week

Week 3: Monday, May 13 – Friday, May 17

- Data collection was paused; ramp metering not generating correct rates due to ramp metering controller configuration error
- Ramp metering controller configuration error was resolved on Wednesday May 15

- Implementation and data collection for scenario 2 (Caltrans District 4 local responsive ramp metering with coordination of ramp metering and arterial traffic signals) began on Thursday May 16

<u>Week 4: Monday, May 20 – Friday, May 24</u>

- Implementation and data collection for scenario 2 (Caltrans District 4 local responsive ramp metering with coordination of ramp metering and arterial traffic signals) continued

<u>Week 5: Monday, May 27- Friday, May 31</u>

- Implementation and data collection for scenario 2 (Caltrans District 4 local responsive ramp metering with coordination of ramp metering and arterial traffic signals) continued
- During the entire week, landscaping activity downstream of the Berryessa Rd. interchange blocked a part of the right lane and the entire right-side shoulder on Northbound Interstate 680, and this caused a bottleneck downstream of the test site. Due to queue spillback from this bottleneck, the data could not be used for the analysis

<u>Week 6: Monday, June 3 – Friday, June 7</u>

- Implementation and data collection for scenario 2 (Caltrans District 4 local responsive ramp metering with coordination of ramp metering and arterial traffic signals) continued
- During the entire week, landscaping activity downstream of the Berryessa Rd. interchange continued to block a part of the right lane and the entire right-side shoulder on Northbound Interstate 680, and this caused a bottleneck downstream of the test site. Due to queue spillback from this bottleneck, the data could not be used for the analysis.

<u>Week 7: Monday, June 10 – Friday, June 14</u>

- Implementation and data collection for scenario 2 (Caltrans District 4 local responsive ramp metering with coordination of ramp metering and arterial traffic signals) continued
- The ramp metering signal at McKee Rd. on-ramp was struck by a vehicle on prior to the morning peak of Tuesday June 11 but was fixed by the morning peak of Thursday June 13, which made two days of data invalid for analysis
- During the entire week, landscaping activity downstream of the Berryessa Rd. interchange continued to block a part of the right lane and the entire right-side shoulder on Northbound Interstate 680, and this caused a bottleneck downstream of the test site. Due to queue spillback from this bottleneck, the data could not be used for the analysis.

<u>Week 8: Monday, June 17 – Friday, June 21</u>

- Implementation and data collection for scenario 2 (Caltrans District 4 local responsive ramp metering with coordination of ramp metering and arterial traffic signals) continued June 17
- Coordinated ramp metering cannot be implemented due to the ramp metering controller’s inability to update ramp metering rates using real time detector data

- The issue with the ramp metering controller was resolved shortly before the morning peak of Tuesday June 18 and implementation and data collection for scenario 3 (PATH developed coordinated ramp metering without coordination of ramp metering and arterial traffic signals) began
- During the entire week, landscaping activity downstream of the Berryessa Rd. interchange continued to block a part of the right lane and the entire right-side shoulder on Northbound Interstate 680, and this caused a bottleneck downstream of the test site. Due to queue spillback from this bottleneck, the data could not be used for the analysis.

<u>Week 9: Monday, June 24 – Friday, June 28</u>

- Implementation and data collection for scenario 3 (PATH developed coordinated ramp metering without coordination of ramp metering and arterial traffic signals) continued
- The landscaping activity downstream of the Berryessa Rd. interchange was no longer present

<u>Week 10: Monday, July 1 – Friday, July 5</u>

- Implementation and data collection for scenario 3 (PATH developed coordinated ramp metering without coordination of ramp metering and arterial traffic signals) continued
- The landscaping activity downstream of the Berryessa Rd. interchange was no longer present

<u>Week 11: Monday, July 8 – Friday, July 12</u>

- Implementation and data collection for scenario 3 (PATH developed coordinated ramp metering without coordination of ramp metering and arterial traffic signals) continued
- The landscaping activity downstream of the Berryessa Rd. interchange was no longer present

<u>Week 12: Monday, July 15 – Friday, July 19</u>

- Implementation and data collection for scenario 3 (PATH developed coordinated ramp metering without coordination of ramp metering and arterial traffic signals) continued
- A mainline detector cabinet was vandalized shortly before the morning peak of Tuesday, July 16 and the ramp metering system could no longer operate. Caltrans District 4 began repairing and replacing the lost and damaged equipment. Monday, July 15 was the last day of implementation and data collection for scenario 3 (PATH developed coordinated ramp metering without coordination of ramp metering and arterial traffic signals)

<u>Week 13: Monday, July 22 – Friday, July 26</u>

- Caltrans District 4 continued to repair and replace the lost and damaged equipment in the mainline detector cabinet.

<u>Week 14: Monday, July 29 – Friday, August 2</u>

- Caltrans District 4 continued to repair and replace the lost and damaged equipment in the mainline detector cabinet.

Week 15: Monday, August 5 – Friday, August 9

- Caltrans District 4 continued to repair and replace the lost and damaged equipment in the mainline detector cabinet until Wednesday, August 7
- Implementation and data collection for scenario 4 (PATH developed coordinated ramp metering with coordination of ramp metering and arterial traffic signals) began on Thursday, August 8

Week 16: Monday, August 12 – Friday, August 16

- Implementation and data collection for scenario 4 (PATH developed coordinated ramp metering with coordination of ramp metering and arterial traffic signals) continued

Week 17: Monday, August 19 – Friday, August 23

- Implementation and data collection for scenario 4 (PATH developed coordinated ramp metering with coordination of ramp metering and arterial traffic signals) continued
- The field test officially ended at 10:00 AM on Friday, August 23

In addition to the days with the unforeseen events listed above, the following days were not considered in the data processing due to unusually low demand and presence of major incidents/accidents (according to PeMS records):

- Friday, June 28
- Monday, July 1
- Tuesday, July 2
- Wednesday, July 3
- Thursday, July 4
- Friday, July 5
- Thursday August 15
- Tuesday August 20
- Wednesday August 21
- Thursday August 22
- Friday August 23

5.2 Field Test Measures of Effectiveness and data collection.

Total travel time, total delay, and vehicle-miles traveled (VMT) were used to determine the effectiveness of the proposed control strategies for coordinate freeway ramp metering and arterial traffic signals in improving the day-to-day operation on the Interstate 680 freeway. All of which were measured using the loop detector data provided by Caltrans and can be found using the PeMS system [19]. We expect the improved freeway operation due to the proposed coordination of freeway ramp metering arterial traffic signals would decrease the total travel time and the total delay while maintaining the same or increase VMT.

The on-ramp queue detector occupancy data (at each on-ramp) will be used to determine the presence of, or severity of, on-ramp queue spillback after introducing the coordination of freeway ramp metering and arterial traffic signals. Higher occupancy (40% or above) indicates the presence of on-ramp queue spillback that could interfere with the arterial operation. Further data collection using arterial detectors were not conducted due to the fact that City of San Jose does not store arterial detector data for arterial performance evaluation purposes; the detectors are mainly used for vehicle detection when operating the actuated signals.

5.3 Field Test Results and Discussion

Table 5.1 shows that the proposed coordination of freeway ramp metering and arterial traffic signal strategy test reduced the total travel time and total delay by 5.79% and 11.49%, respectively. In both scenarios, the days in which data were valid showed similar total vehicle-miles traveled (VMT), as shown in Figures 5.1 to 5.17 (Some days were unavailable due to detector error). The aggregated distance traveled in the figures represent the total distance traveled by all vehicles (in all lanes) observed during the morning peak. As time progresses, the total distance traveled (or VMT) increases and the cumulative distance traveled (or VMT during the entire morning peak) can be shown to be greater when the proposed coordination strategy is implemented. As a result, both travel time and delay improved while the freeway throughput remained relatively the same.

Table 5.1: Travel Time and Delay on Northbound Interstate 680 under Local Responsive Ramp Metering (before and after coordination of freeway ramp metering and arterial traffic signals)

	Date of Testing	**Total Travel Time (veh-hr)**	**Total Delay (veh-hr)**
Scenario 1: Caltrans District 4 local responsive ramp metering without coordination of ramp metering and arterial traffic signals	Monday, April 29	5.98	3.37
	Tuesday, April 30	6.56	3.95
	Wednesday, May 1	6.53	3.92
	Thursday, May 2	6.46	3.85
	Friday, May 3	4.89	2.28
	Monday, May 6	5.25	2.64
	Tuesday, May 7	5.96	3.34
	Wednesday, May 8	3.12	0.50
	Thursday, May 9	5.16	2.55
	Friday, May 10	2.76	0.14
	Average	**5.27**	**2.65**
Scenario 2: Caltrans District 4 local responsive ramp metering with coordination of ramp	Thursday, May 16	4.15	1.54
	Friday, May 17	5.87	3.26
	Monday, May 20	4.61	2.00
	Tuesday, May 21	5.34	2.73

metering and arterial traffic signals	Wednesday, May 22	6.26	3.65
	Thursday, May 23	4.91	2.30
	Friday, May 24	3.58	0.96
	Average	**4.96**	**2.35**
	% Change	**-5.79%**	**-11.49%**

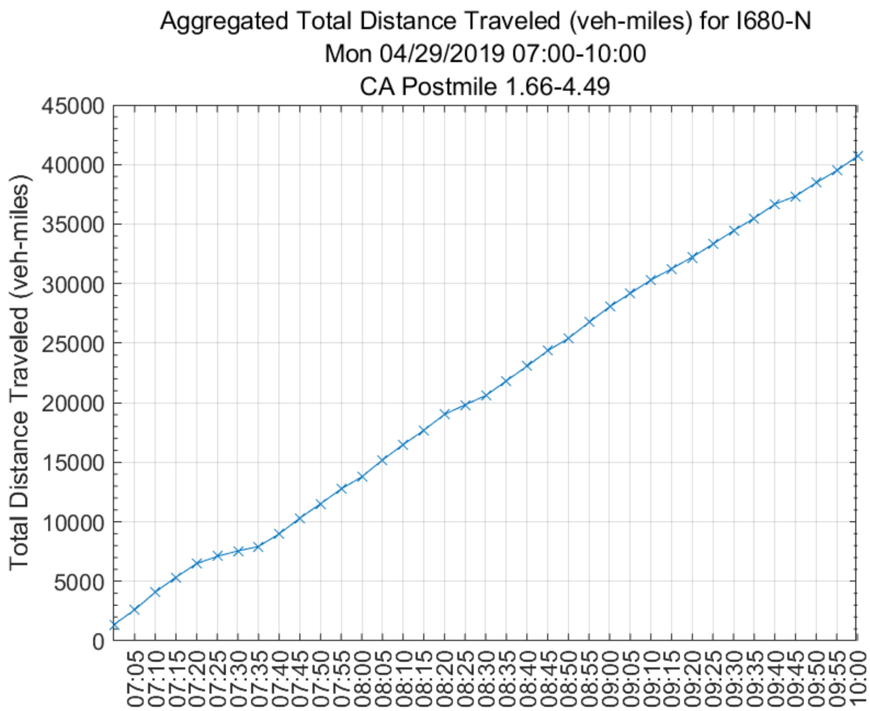

Figure 5.1 Total Distance Traveled on Northbound Interstate 680 (Monday April 29, 2019, Scenario 1: Caltrans District 4 local responsive ramp metering without coordination of ramp metering and arterial traffic signals)

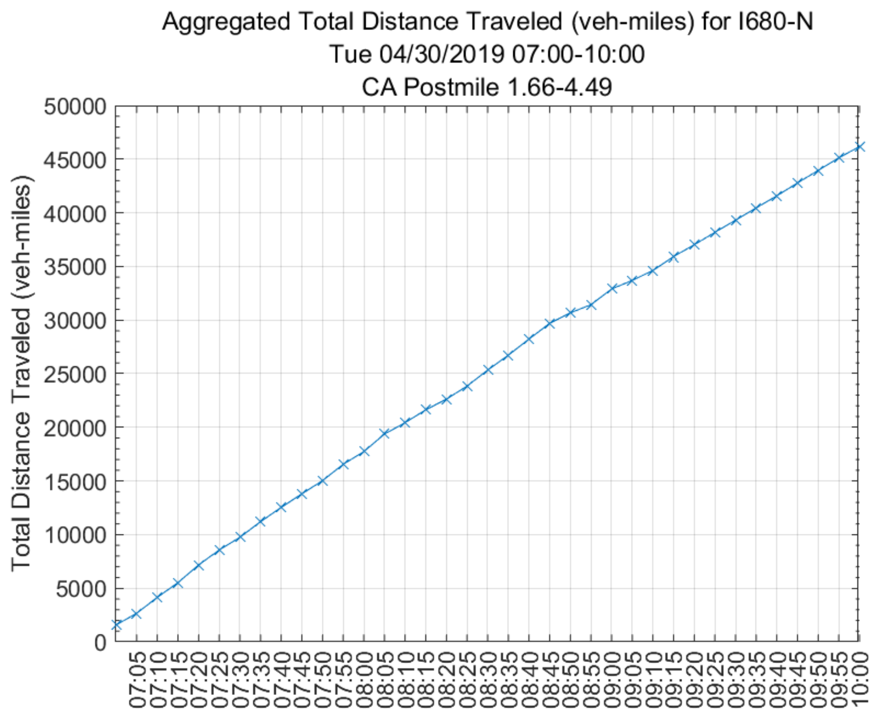

Figure 5.2 Total Distance Traveled on Northbound Interstate 680 (Tuesday April 30, 2019, Scenario 1: Caltrans District 4 local responsive ramp metering without coordination of ramp metering and arterial traffic signals)

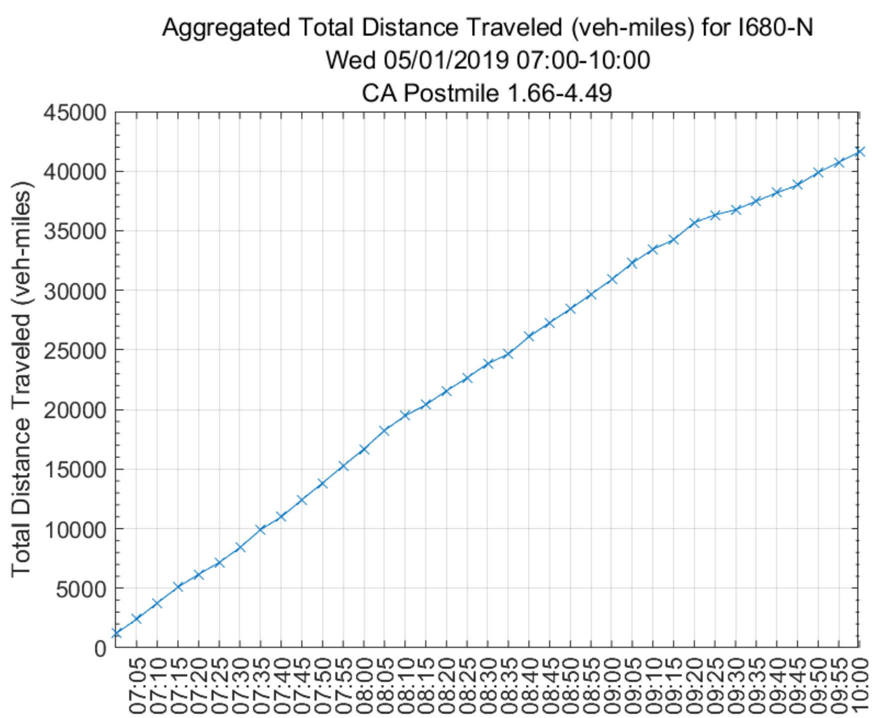

Figure 5.3 Total Distance Traveled on Northbound Interstate 680 (Wednesday May 1, 2019, Scenario 1: Caltrans District 4 local responsive ramp metering without coordination of ramp metering and arterial traffic signals)

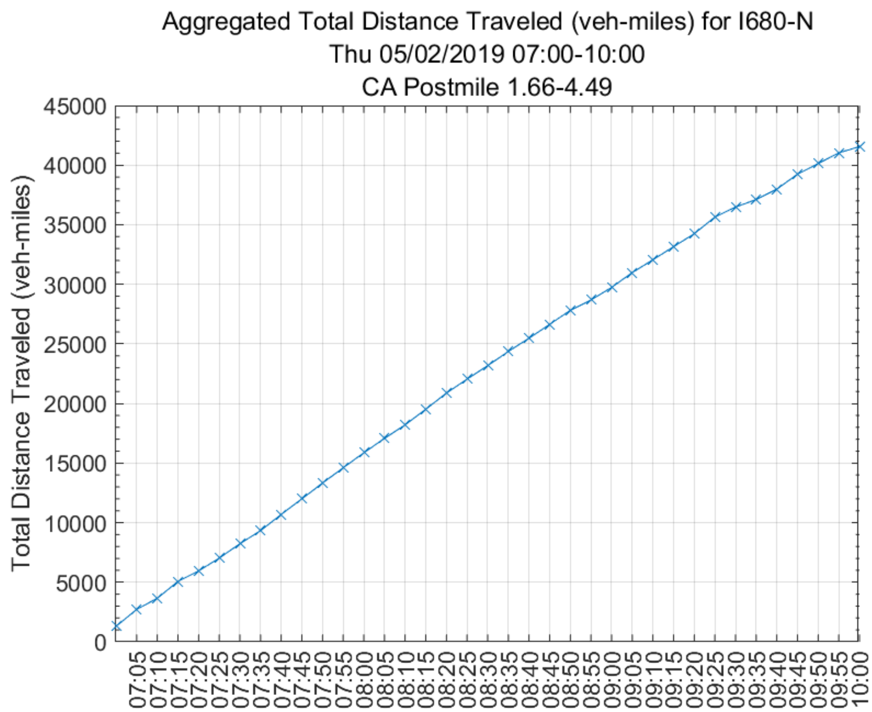

Figure 5.4 Total Distance Traveled on Northbound Interstate 680 (Thursday May 2, 2019, Scenario 1: Caltrans District 4 local responsive ramp metering without coordination of ramp metering and arterial traffic signals)

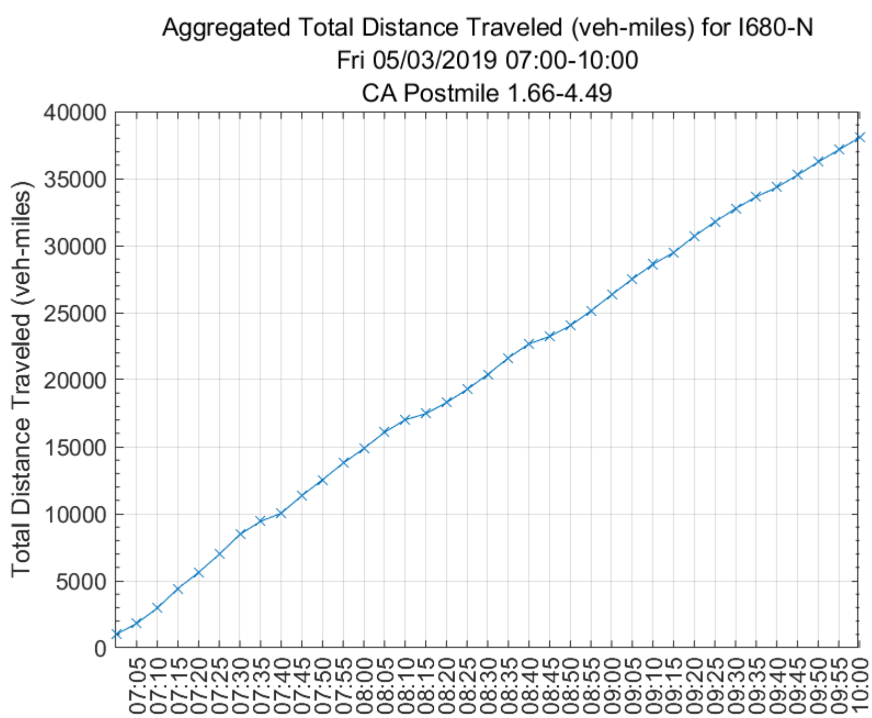

Figure 5.5 Total Distance Traveled on Northbound Interstate 680 (Friday May 3, 2019, Scenario 1: Caltrans District 4 local responsive ramp metering without coordination of ramp metering and arterial traffic signals)

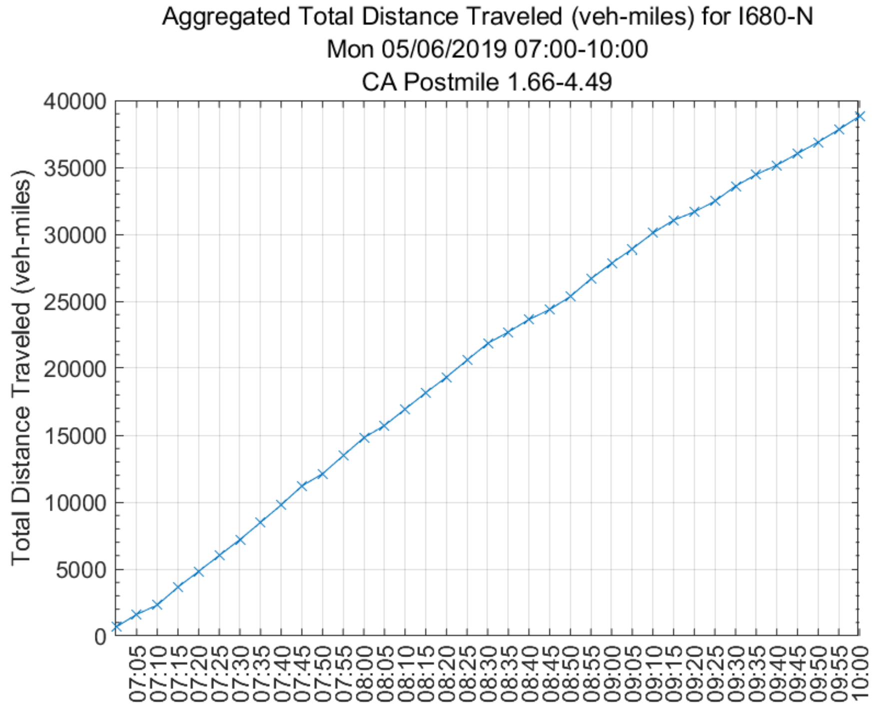

Figure 5.6 Total Distance Traveled on Northbound Interstate 680 (Monday May 6, 2019, Scenario 1: Caltrans District 4 local responsive ramp metering without coordination of ramp metering and arterial traffic signals)

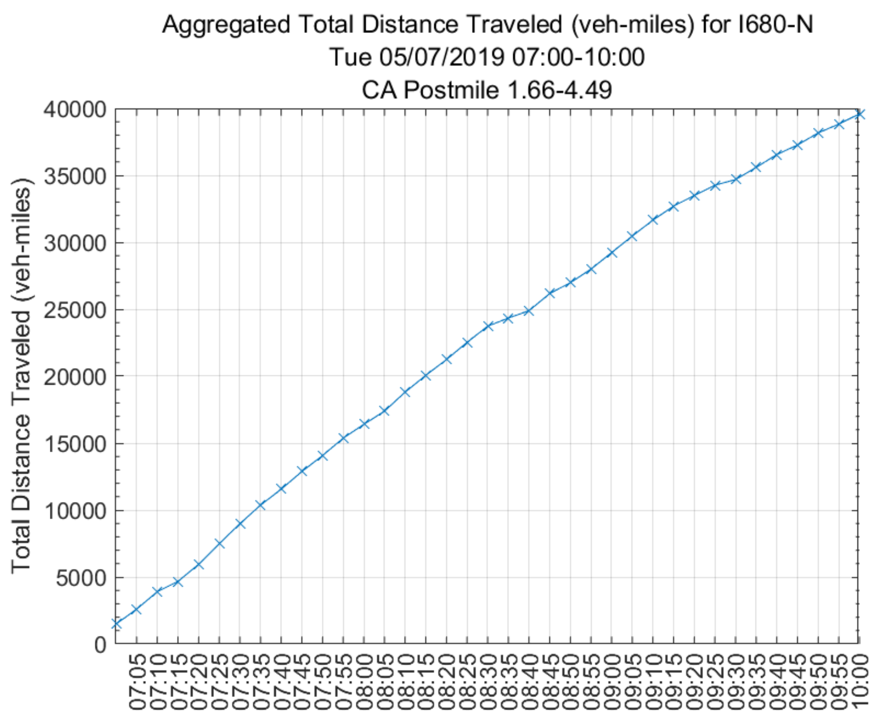

Figure 5.7 Total Distance Traveled on Northbound Interstate 680 (Tuesday May 7, 2019, Scenario 1: Caltrans District 4 local responsive ramp metering without coordination of ramp metering and arterial traffic signals)

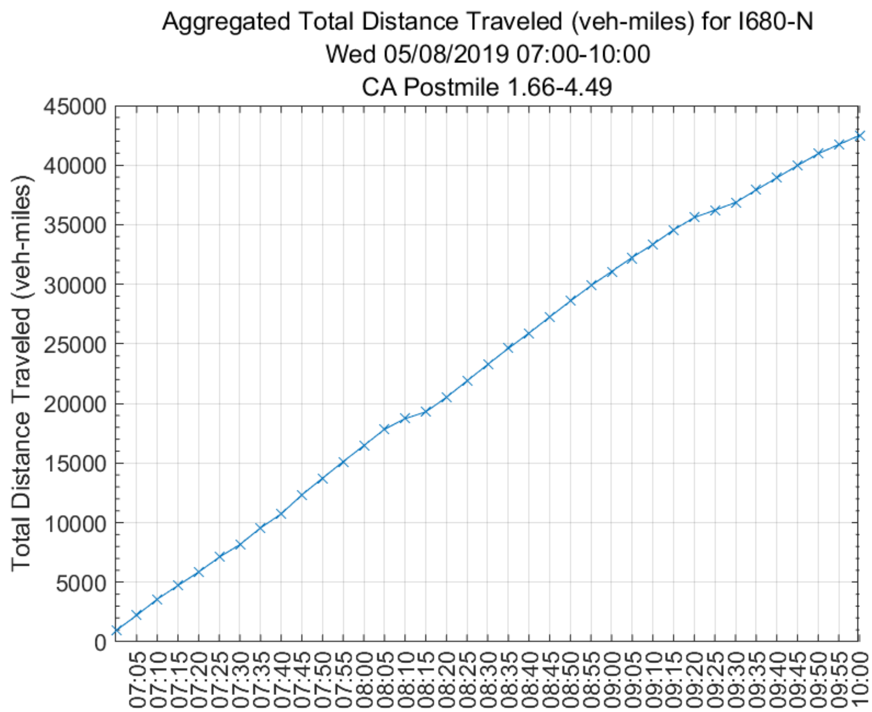

Figure 5.8 Total Distance Traveled on Northbound Interstate 680 (Wednesday May 8, 2019, Scenario 1: Caltrans District 4 local responsive ramp metering without coordination of ramp metering and arterial traffic signals)

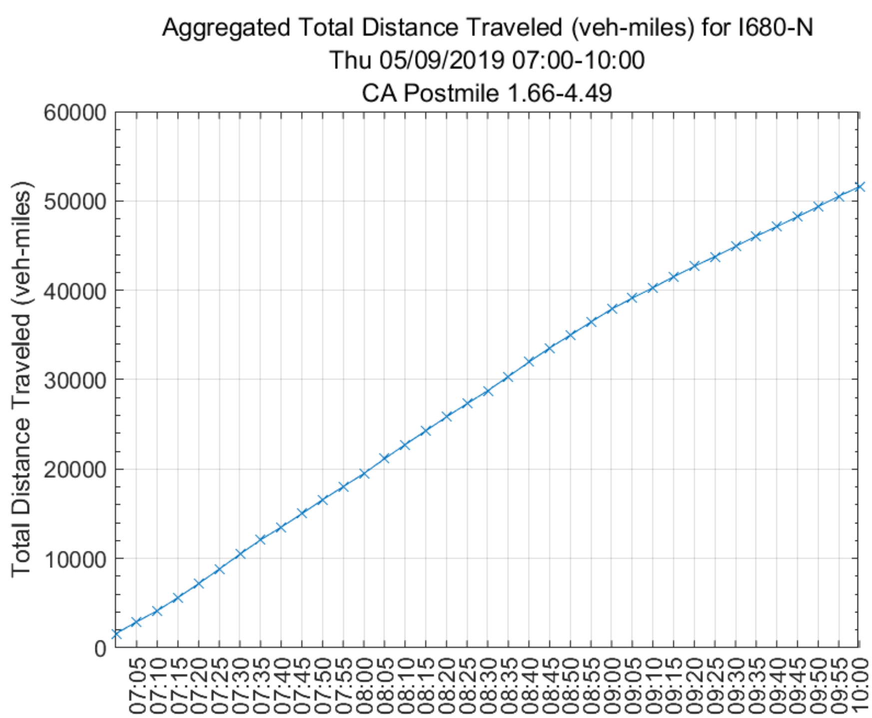

Figure 5.9 Total Distance Traveled on Northbound Interstate 680 (Thursday May 9, 2019, Scenario 1: Caltrans District 4 local responsive ramp metering without coordination of ramp metering and arterial traffic signals)

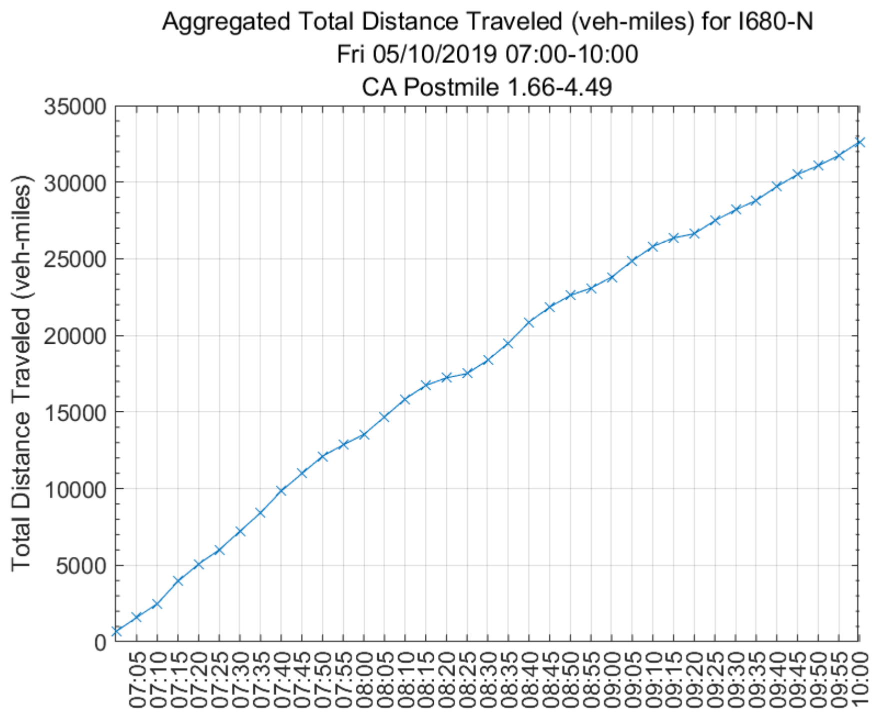

Figure 5.10 Total Distance Traveled on Northbound Interstate 680 (Friday May 10, 2019, Scenario 1: Caltrans District 4 local responsive ramp metering <u>without</u> coordination of ramp metering and arterial traffic signals)

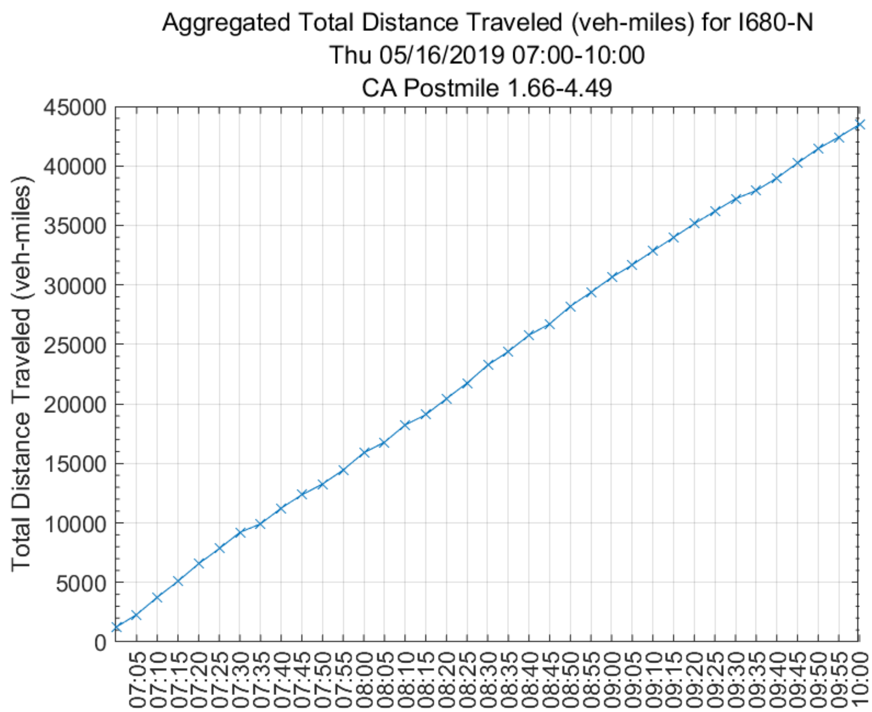

Figure 5.11 Total Distance Traveled on Northbound Interstate 680 (Thursday May 16, 2019, Scenario 2: Caltrans District 4 local responsive ramp metering <u>with</u> coordination of ramp metering and arterial traffic signals)

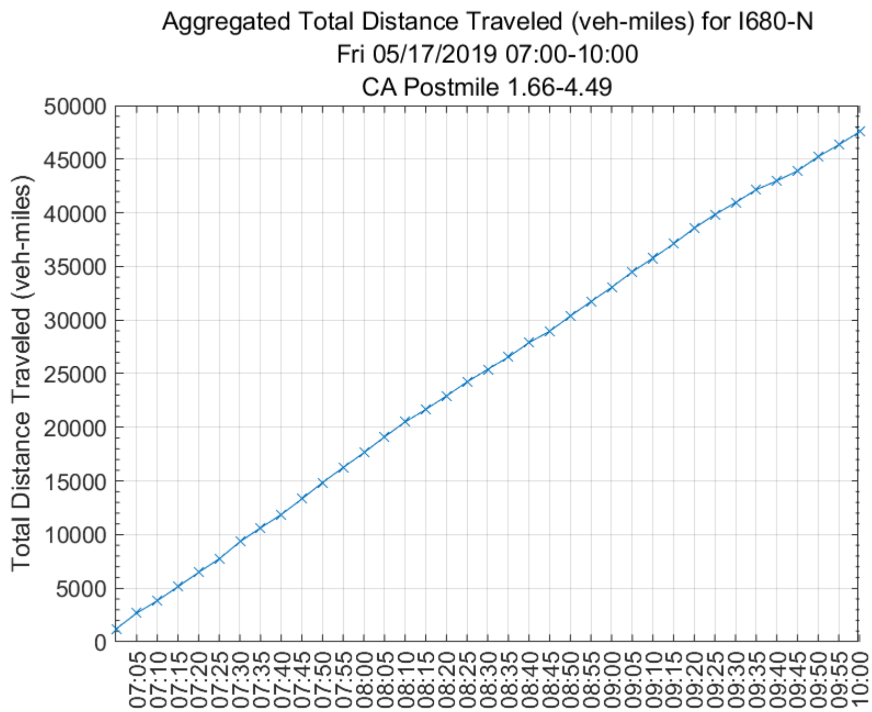

Figure 5.12 Total Distance Traveled on Northbound Interstate 680 (Friday May 17, 2019, Scenario 2: Caltrans District 4 local responsive ramp metering <u>with</u> coordination of ramp metering and arterial traffic signals)

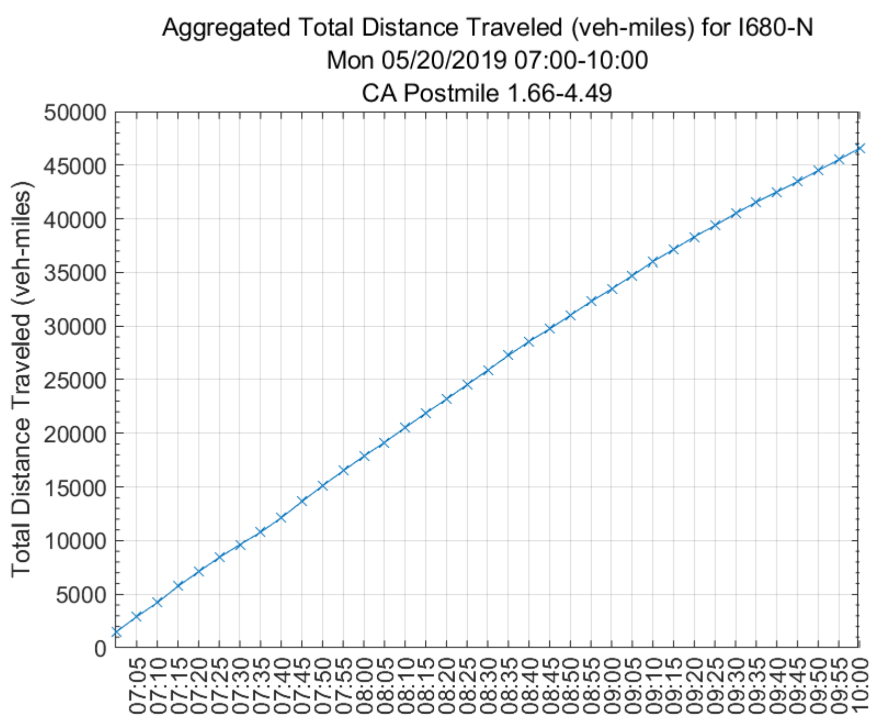

Figure 5.13 Total Distance Traveled on Northbound Interstate 680 (Monday May 20, 2019, Scenario 2: Caltrans District 4 local responsive ramp metering <u>with</u> coordination of ramp metering and arterial traffic signals)

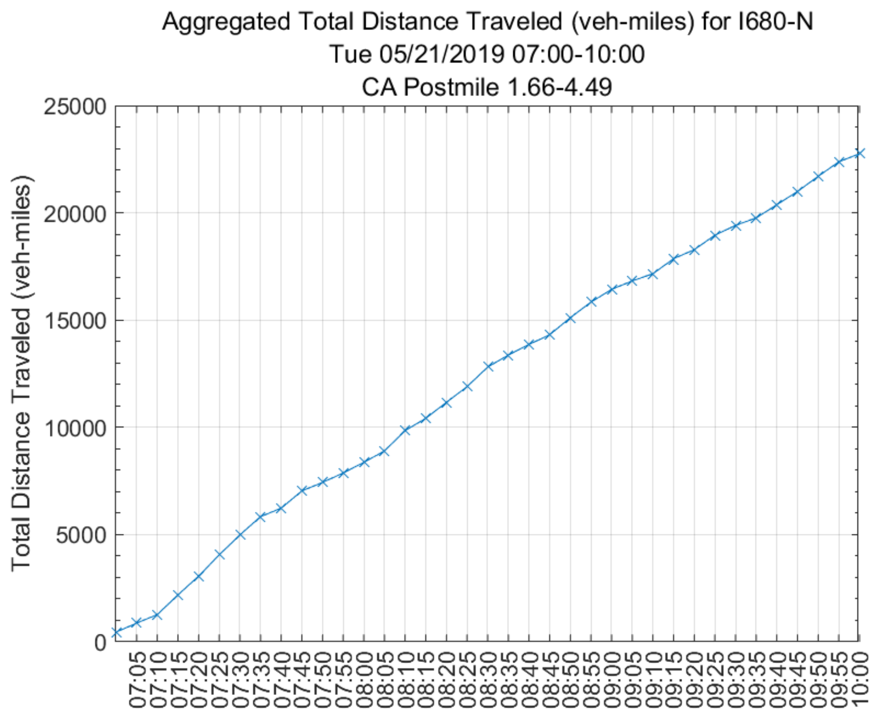

Figure 5.14 Total Distance Traveled on Northbound Interstate 680 (Tuesday May 21, 2019, Scenario 2: Caltrans District 4 local responsive ramp metering <u>with</u> coordination of ramp metering and arterial traffic signals)

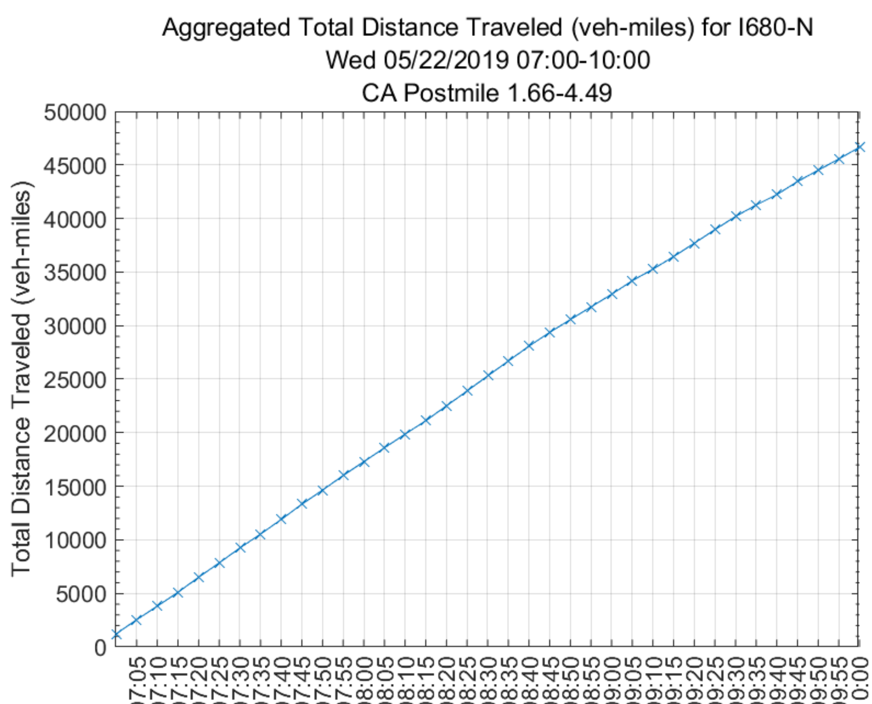

Figure 5.15 Total Distance Traveled on Northbound Interstate 680 (Wednesday May 22, 2019, Scenario 2: Caltrans District 4 local responsive ramp metering <u>with</u> coordination of ramp metering and arterial traffic signals)

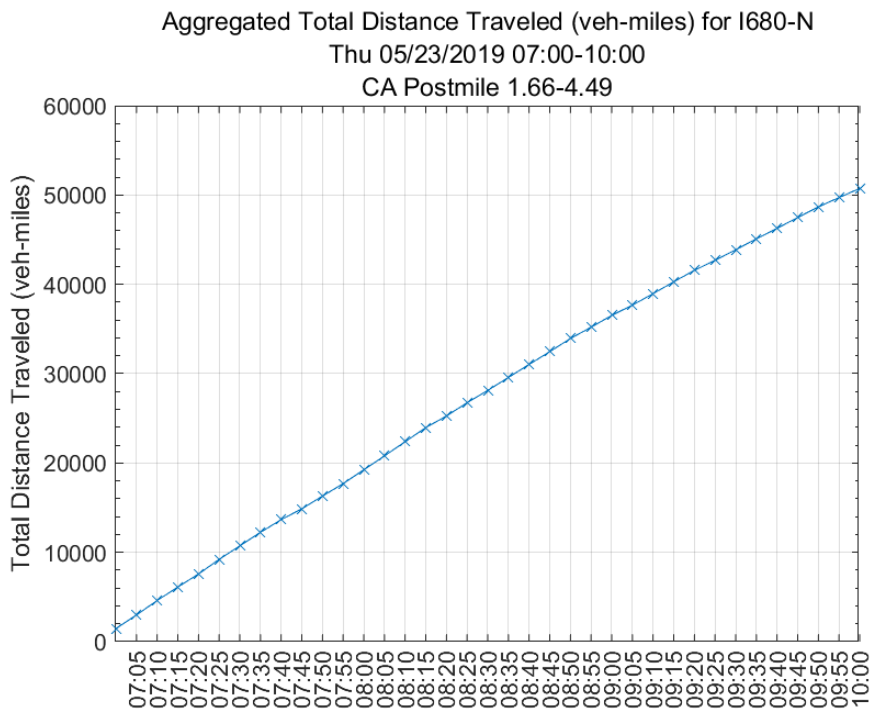

Figure 5.16 Total Distance Traveled on Northbound Interstate 680 (Thursday May 23, 2019, Scenario 2: Caltrans District 4 local responsive ramp metering with coordination of ramp metering and arterial traffic signals)

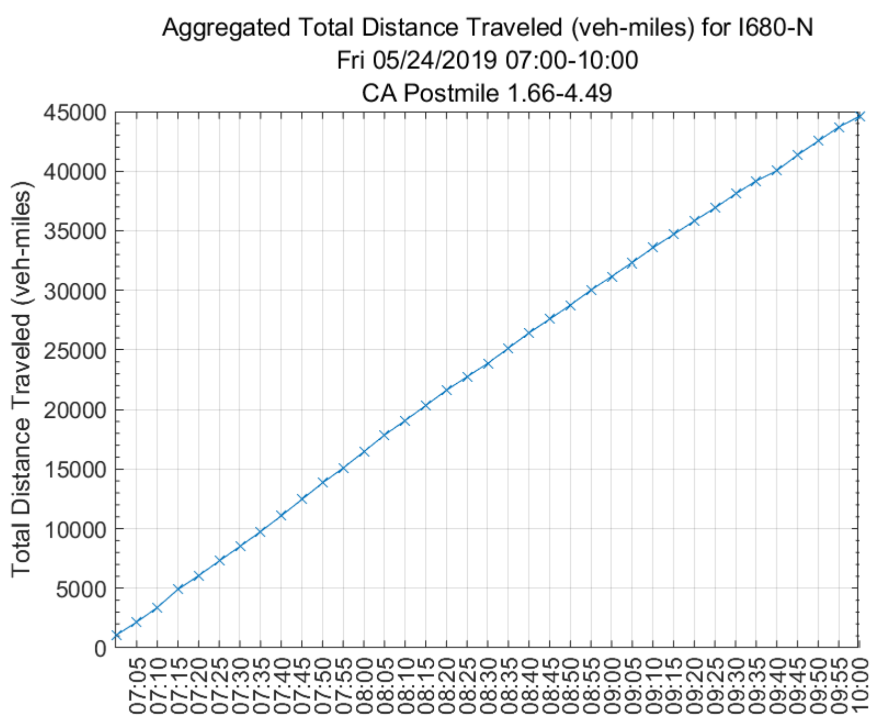

Figure 5.17 Total Distance Traveled on Northbound Interstate 680 (Friday May 24, 2019, Scenario 2: Caltrans District 4 local responsive ramp metering with coordination of ramp metering and arterial traffic signals)

Similarly, despite using a different ramp metering algorithm (coordinated ramp metering) in the second half of the field test, the data demonstrated similar improvement in freeway performance. Table 5.2 shows that the proposed coordination of freeway ramp metering and arterial traffic signal strategy test reduced the total travel time and total delay by 2.75% and 5.92%, respectively. In both scenarios, the days in which data were valid showed similar total vehicle-miles traveled (VMT), as shown in Figures 5.18 to 5.34 (Some days were unavailable due to detector error). As a result, both travel time and delay improved while the freeway throughput remained relatively the same.

Table 5.2: Travel Time and Delay on Northbound Interstate 680 under Coordinated Ramp Metering (before and after coordination of freeway ramp metering and arterial traffic signals).

	Date of Testing	**Total Travel Time (veh-hr)**	**Total Delay (veh-hr)**
Scenario 3: PATH developed coordinated ramp metering without coordination of ramp metering and arterial traffic signals	Monday, June 24	4.41	1.80
	Tuesday, June 25	6.13	3.52
	Wednesday, June 26	5.26	2.65
	Thursday, June 27	9.45	6.84
	Monday, July 08	2.91	0.30
	Tuesday, July 09	2.72	0.11
	Wednesday, July 10	2.84	0.23
	Thursday, July 11	5.45	2.84
	Friday, July 12	2.73	0.12
	Monday, July 15	6.94	4.33
	Average	**4.88**	**2.27**
Scenario 4: PATH developed coordinated ramp metering with coordination of ramp metering and arterial traffic signals	Thursday, Aug 08	4.23	1.62
	Friday, Aug 09	2.72	0.11
	Monday, Aug 12	3.55	0.93
	Tuesday, Aug 13	6.49	3.88
	Wednesday, Aug 14	6.27	3.66
	Friday, Aug 16	4.45	1.84
	Monday, Aug 19	5.53	2.92
	Average	**4.75**	**2.14**
	% Change	**-2.75**	**-5.92**

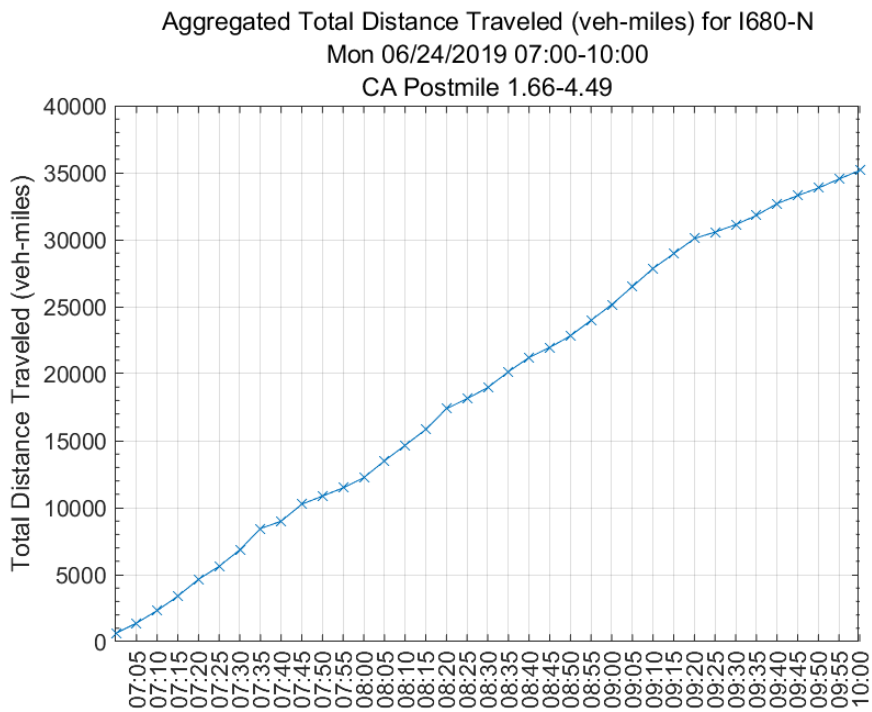

Figure 5.18 Total Distance Traveled on Northbound Interstate 680 (Monday June 24, 2019, Scenario 3: PATH developed coordinated ramp metering without coordination of ramp metering and arterial traffic signals)

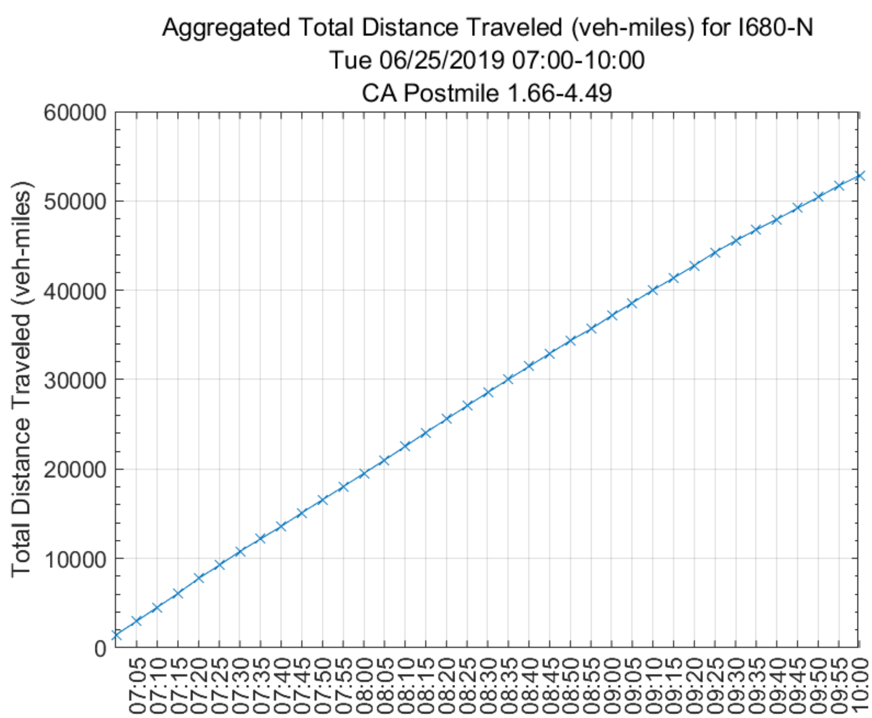

Figure 5.19 Total Distance Traveled on Northbound Interstate 680 (Tuesday June 25, 2019, Scenario 3: PATH developed coordinated ramp metering without coordination of ramp metering and arterial traffic signals)

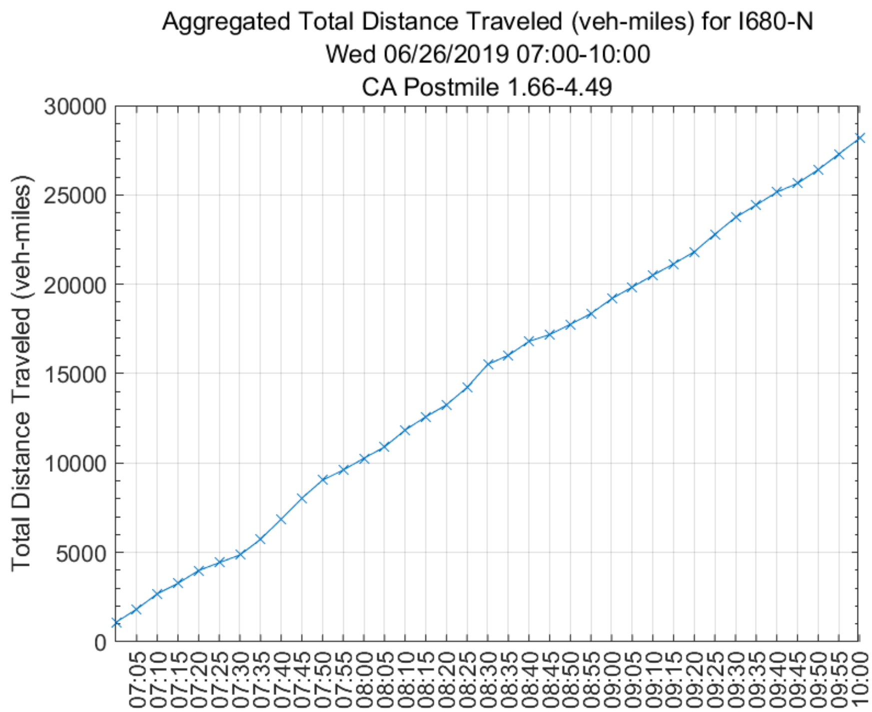

Figure 5.20 Total Distance Traveled on Northbound Interstate 680 (Wednesday June 26, 2019, Scenario 3: PATH developed coordinated ramp metering without coordination of ramp metering and arterial traffic signals)

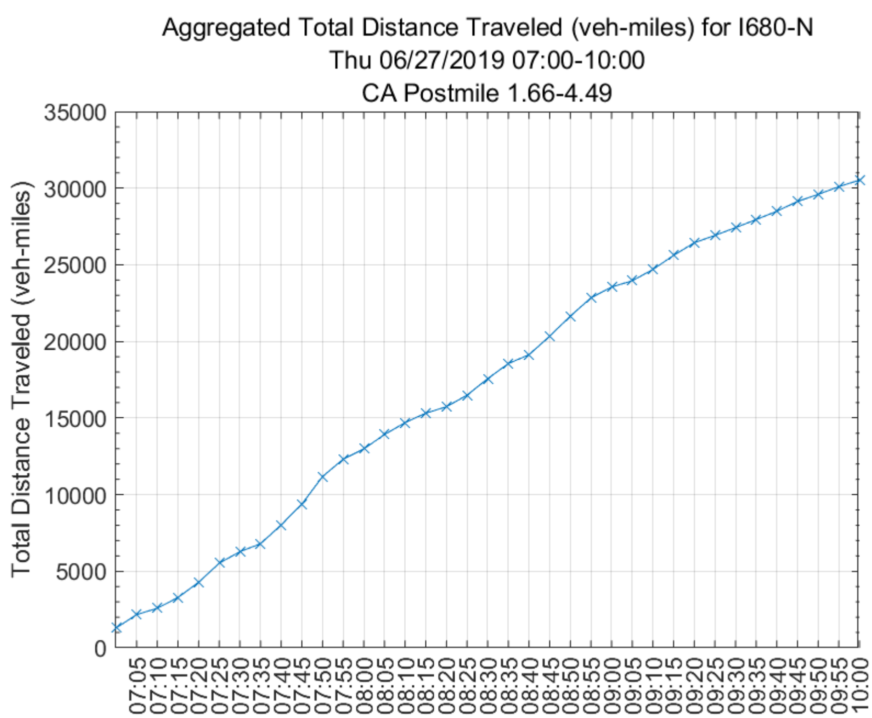

Figure 5.21 Total Distance Traveled on Northbound Interstate 680 (Thursday June 27, 2019, Scenario 3: PATH developed coordinated ramp metering without coordination of ramp metering and arterial traffic signals)

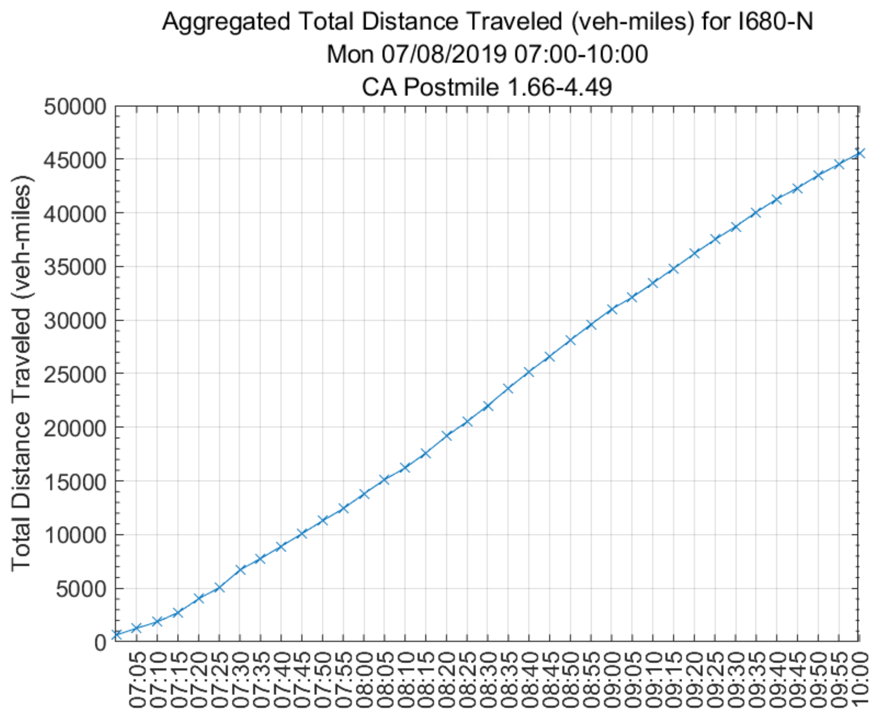

Figure 5.22 Total Distance Traveled on Northbound Interstate 680 (Monday July 8, 2019, Scenario 3: PATH developed coordinated ramp metering without coordination of ramp metering and arterial traffic signals)

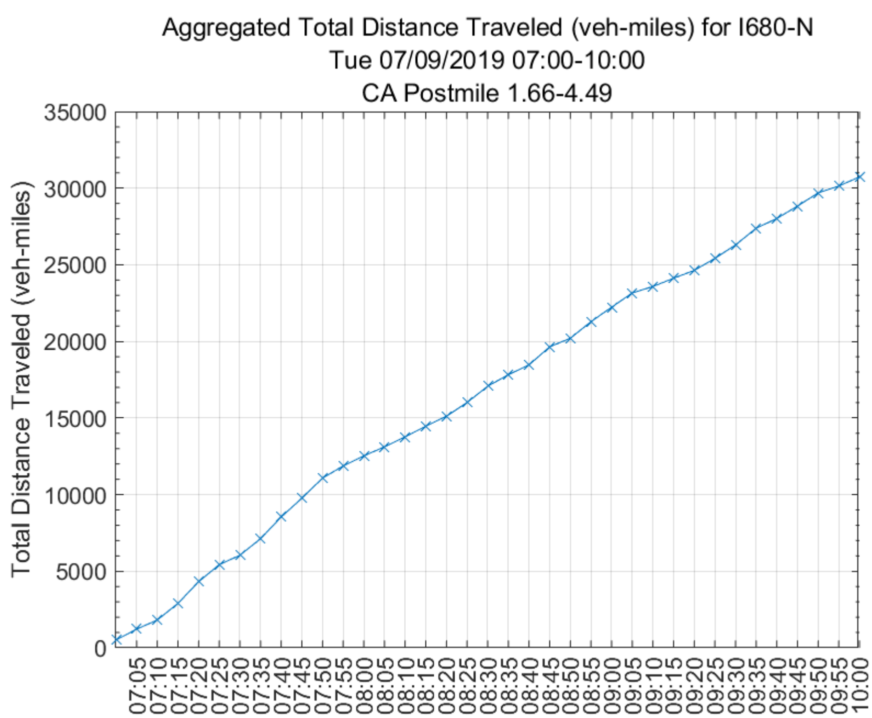

Figure 5.23 Total Distance Traveled on Northbound Interstate 680 (Tuesday July 9, 2019, Scenario 3: PATH developed coordinated ramp metering without coordination of ramp metering and arterial traffic signals)

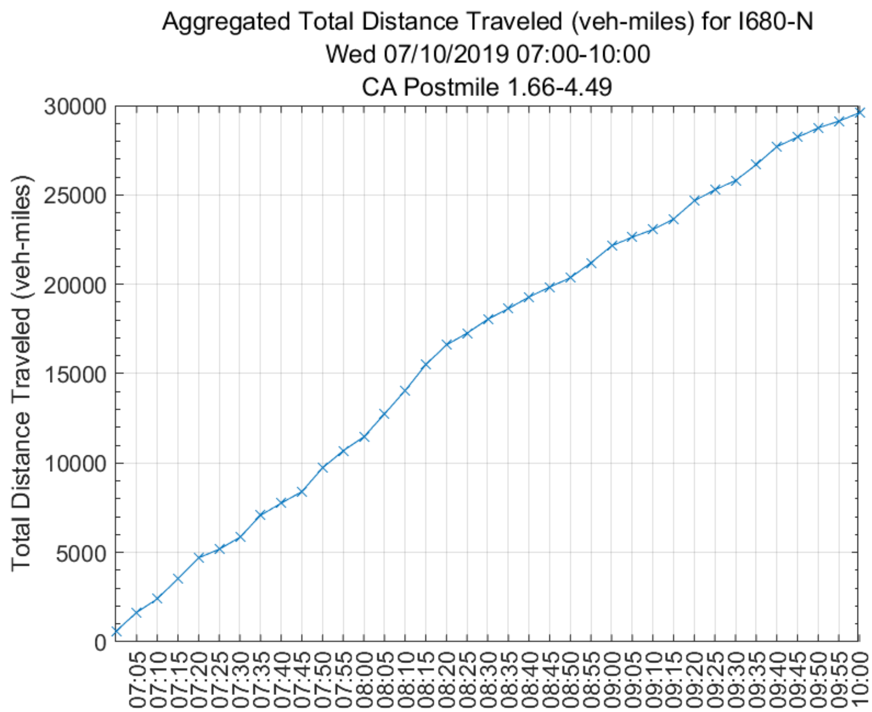

Figure 5.24 Total Distance Traveled on Northbound Interstate 680 (Tuesday July 10, 2019, Scenario 3: PATH developed coordinated ramp metering without coordination of ramp metering and arterial traffic signals)

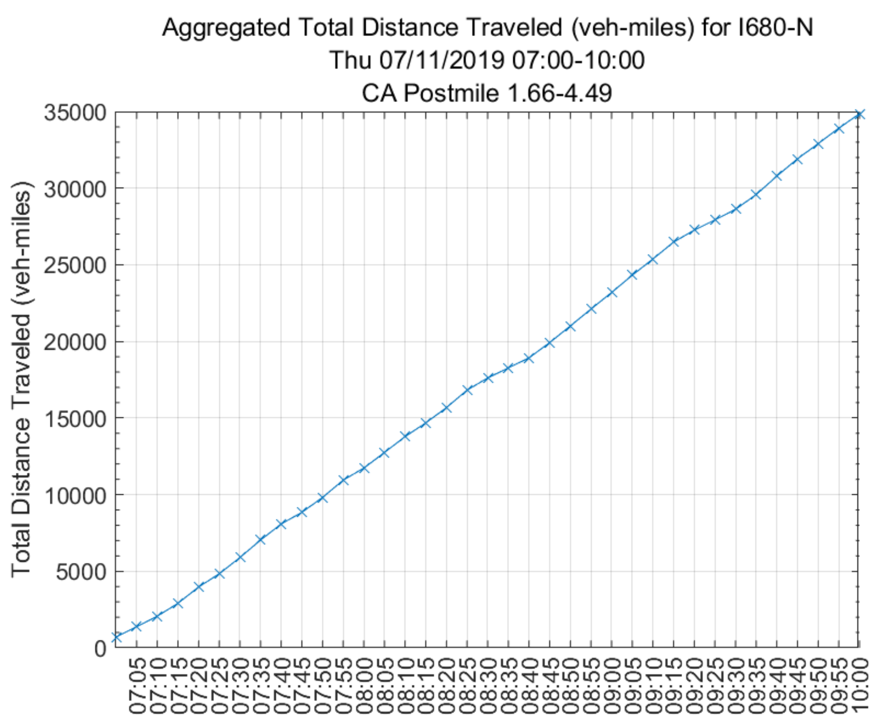

Figure 5.25 Total Distance Traveled on Northbound Interstate 680 (Thursday July 11, 2019, Scenario 3: PATH developed coordinated ramp metering without coordination of ramp metering and arterial traffic signals)

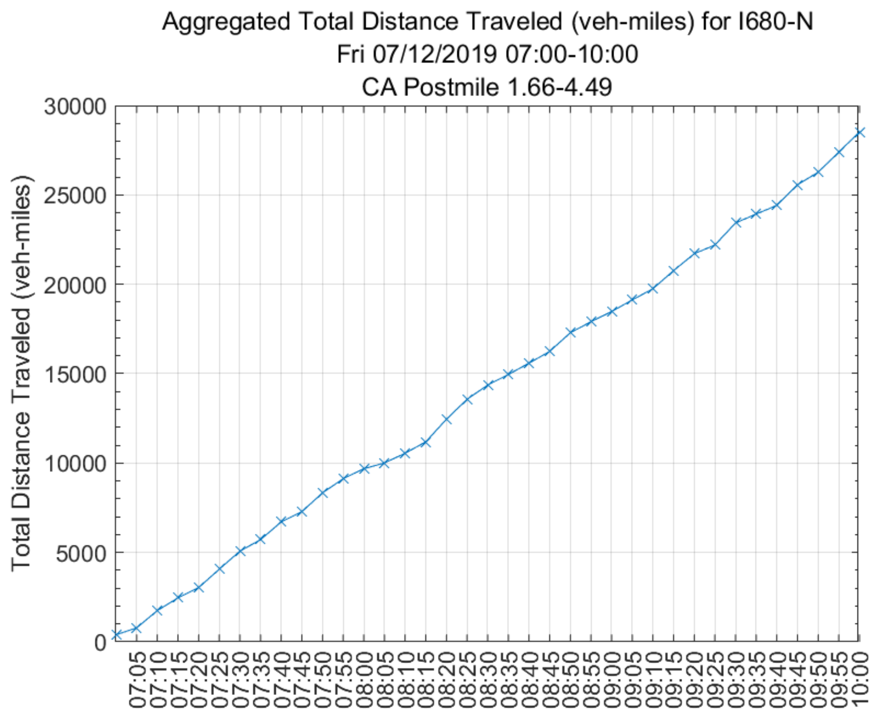

Figure 5.26 Total Distance Traveled on Northbound Interstate 680 (Friday July 12, 2019, Scenario 3: PATH developed coordinated ramp metering without coordination of ramp metering and arterial traffic signals)

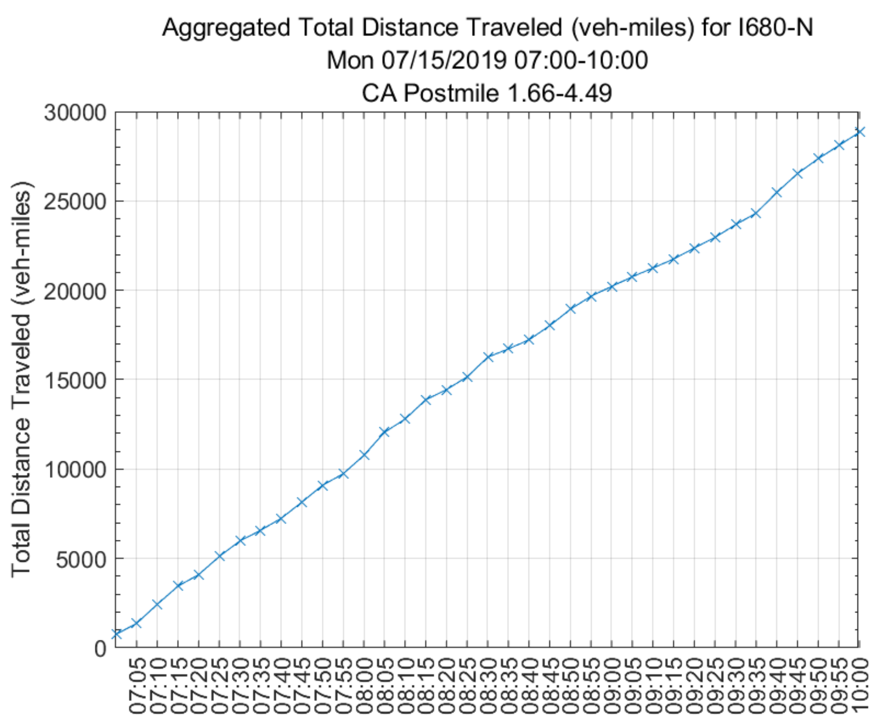

Figure 5.27 Total Distance Traveled on Northbound Interstate 680 (Monday July 15, 2019, Scenario 3: PATH developed coordinated ramp metering without coordination of ramp metering and arterial traffic signals)

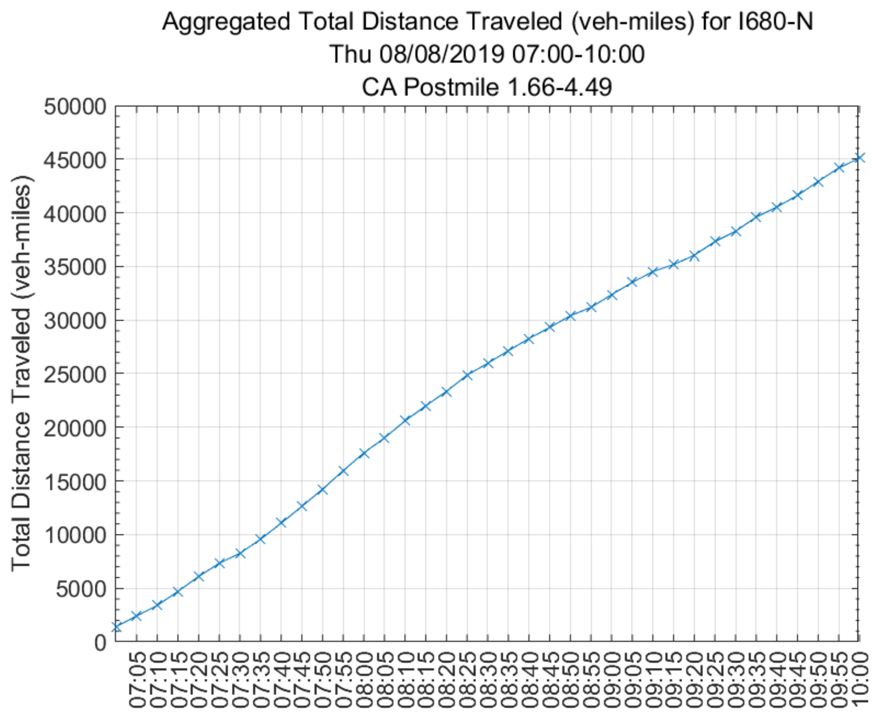

Figure 5.28 Total Distance Traveled on Northbound Interstate 680 (Thursday August 8, 2019, Scenario 4: PATH developed coordinated ramp metering <u>with</u> coordination of ramp metering and arterial traffic signals)

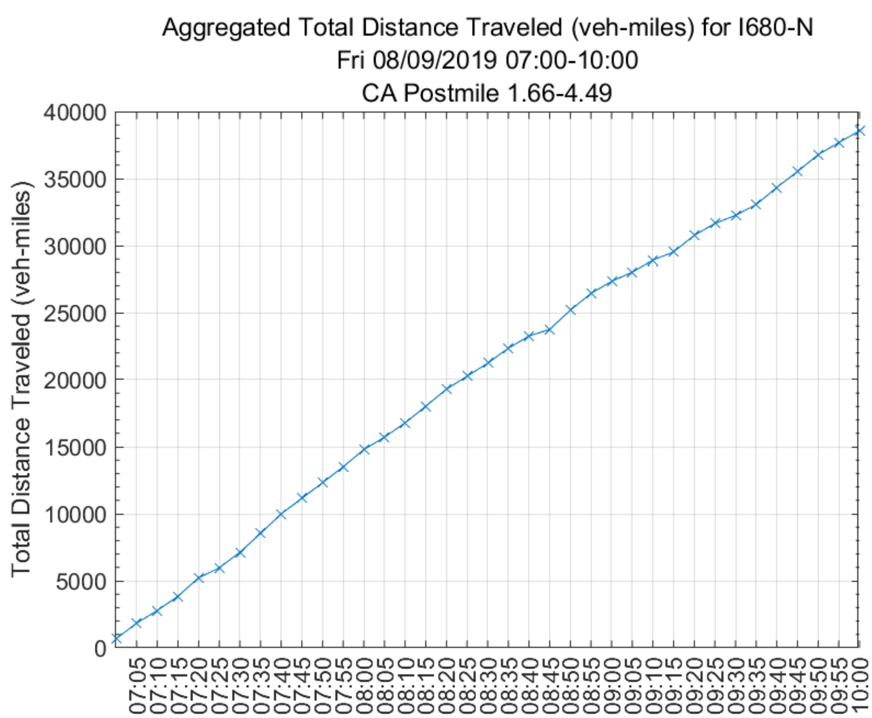

Figure 5.29 Total Distance Traveled on Northbound Interstate 680 (Friday August 9, 2019, Scenario 4: PATH developed coordinated ramp metering <u>with</u> coordination of ramp metering and arterial traffic signals)

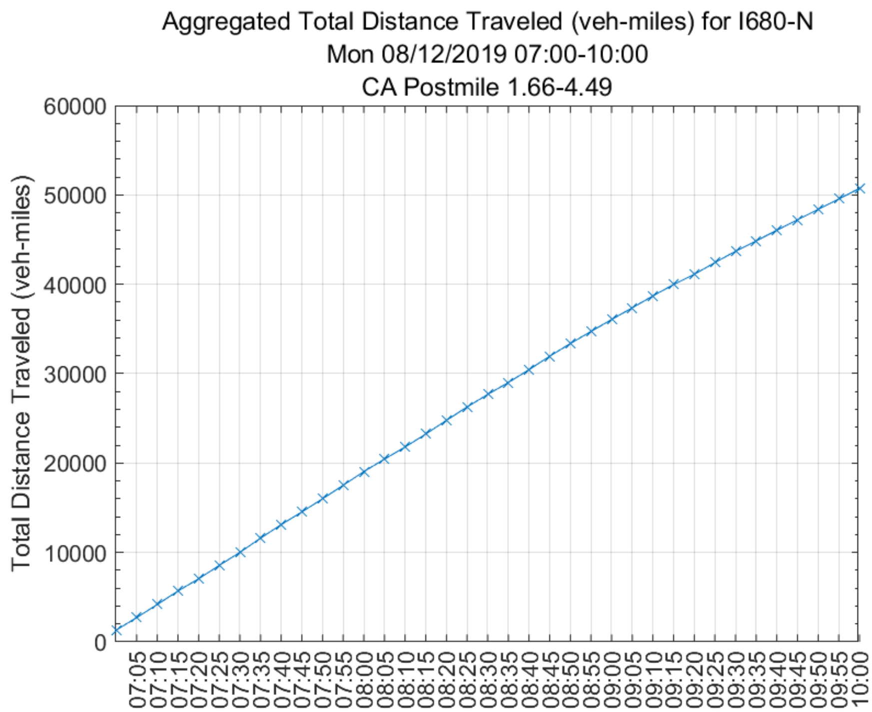

Figure 5.30 Total Distance Traveled on Northbound Interstate 680 (Monday August 12, 2019, Scenario 4: PATH developed coordinated ramp metering <u>with</u> coordination of ramp metering and arterial traffic signals)

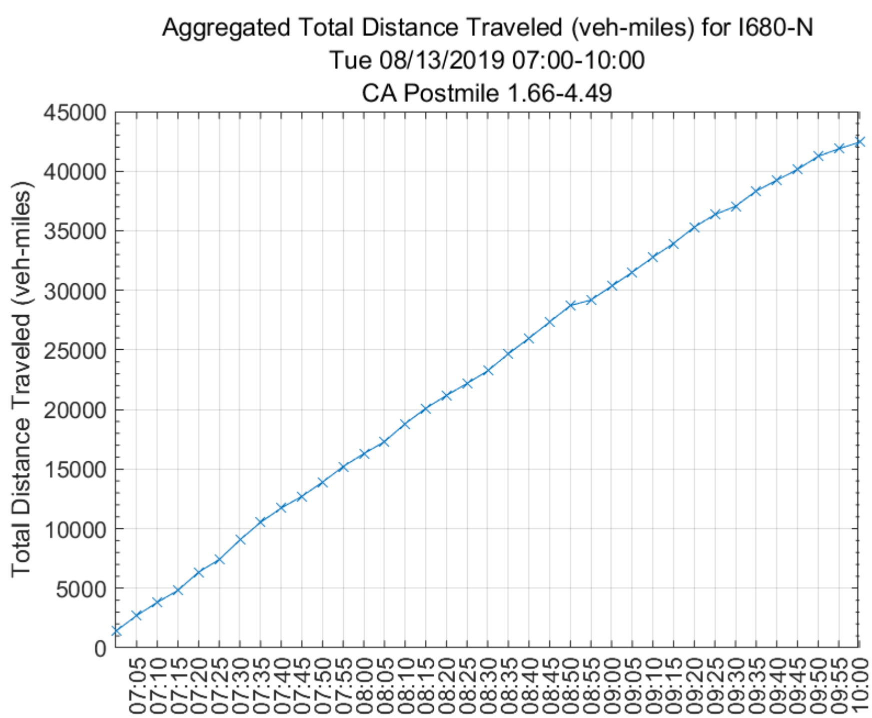

Figure 5.31 Total Distance Traveled on Northbound Interstate 680 (Tuesday August 13, 2019, Scenario 4: PATH developed coordinated ramp metering <u>with</u> coordination of ramp metering and arterial traffic signals)

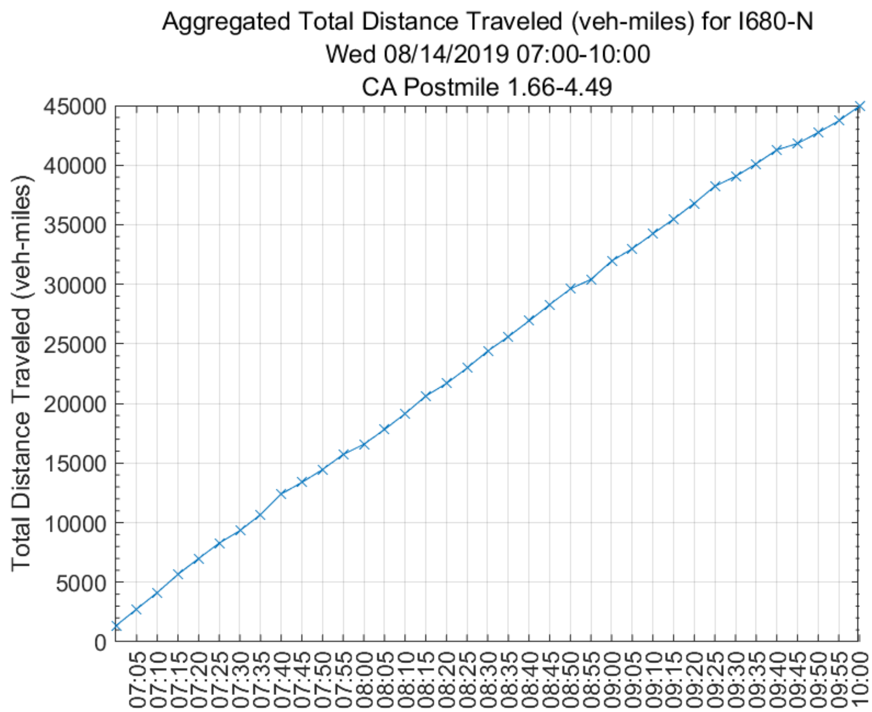

Figure 5.32 Total Distance Traveled on Northbound Interstate 680 (Wednesday August 14, 2019, Scenario 4: PATH developed coordinated ramp metering with coordination of ramp metering and arterial traffic signals)

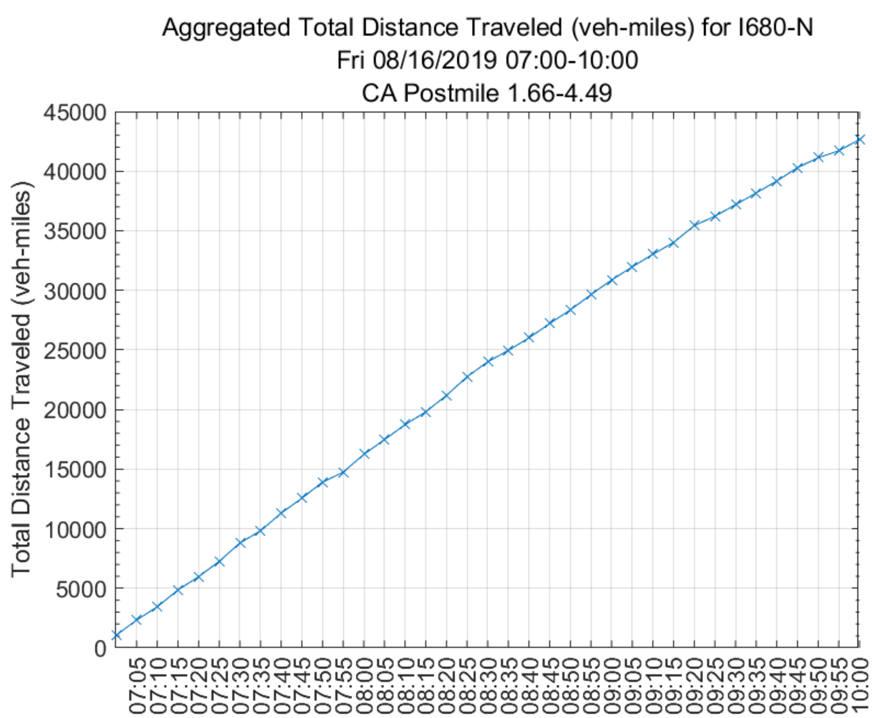

Figure 5.33 Total Distance Traveled on Northbound Interstate 680 (Friday August 16, 2019, Scenario 4: PATH developed coordinated ramp metering with coordination of ramp metering and arterial traffic signals)

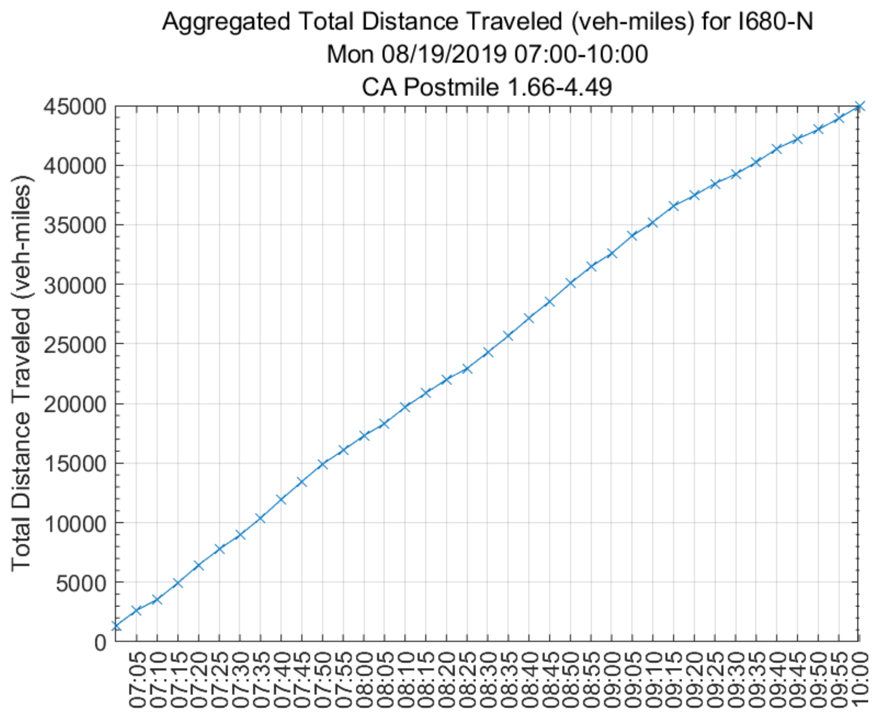

Figure 5.34 Total Distance Traveled on Northbound Interstate 680 (Monday August 19, 2019, Scenario 4: PATH developed coordinated ramp metering with coordination of ramp metering and arterial traffic signals)

In addition to the improved freeway operation, the arterial spillback was mitigated based on the observed data from on-ramp queue detectors. Figures 5.35 to 5.41 show the on-ramp queue detector occupancy that were available for the days considered for data analysis. Many days of the field test did not have available on-ramp queue detector data due to detector errors and malfunction. As shown in Figures 5.36 and 5.37, when scenario 3 was tested, the on-ramp queue detector shows extremely high occupancy at both McKee Rd. on-ramp and Berryessa Rd. on-ramp. Following the coordination between freeway ramp metering and arterial traffic signals in scenario 4, Figures 5.40 and 5.41 show that the on-ramp queue detector occupancy was significantly reduced at both the McKee Rd. on-ramp and the Berryessa Rd. on-ramp. In Figures 5.40 and 5.41 (corresponding to scenario 4), none of the on-ramp queue detectors showed higher than 40% occupancy, which is the threshold for queue spillback and the ramp metering controller to activate queue override and relax ramp metering rates to alleviate queue spillback. However, Figures 5.36 and 5.37 showed that the queue detector occupancies were higher than 40%, which was a result of queue spillback in scenario 3, when coordination of freeway ramp metering and arterial traffic signals was absent. This shows that the proposed control strategy for coordination of freeway ramp metering and arterial traffic signals prevent queue spillback.

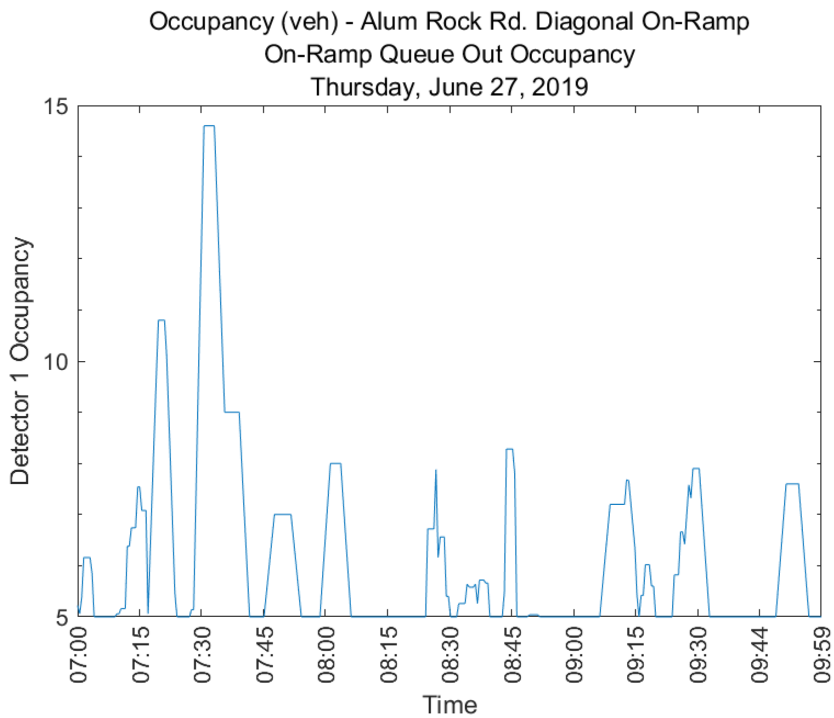

Figure 5.35 Alum Rock Ave. On-Ramp Queue Detector Occupancy (Thursday June 27, 2019, Scenario 3: PATH developed coordinated ramp metering <u>without</u> coordination of ramp metering and arterial traffic signals)

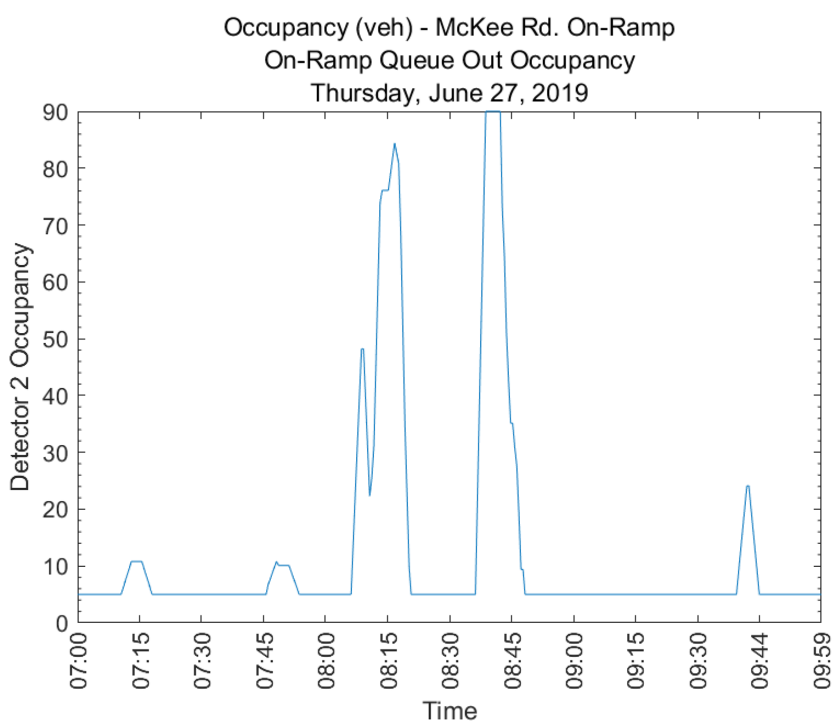

Figure 5.36 McKee Rd. On-Ramp Queue Detector Occupancy (Thursday June 27, 2019, Scenario 3: PATH developed coordinated ramp metering <u>without</u> coordination of ramp metering and arterial traffic signals)

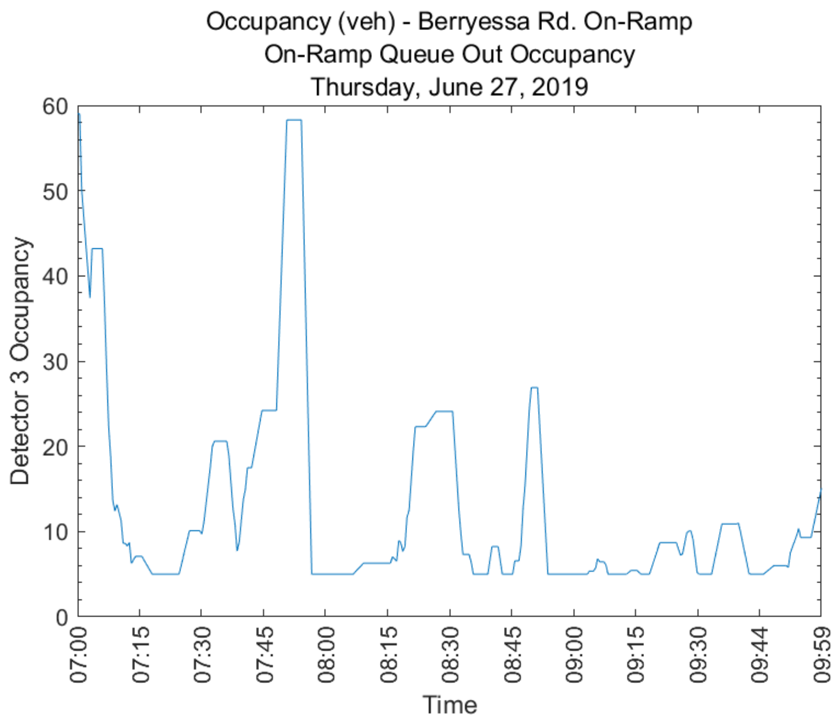

Figure 5.37 Berryessa Rd. On-Ramp Queue Detector Occupancy (Thursday June 27, 2019, Scenario 3: PATH developed coordinated ramp metering without coordination of ramp metering and arterial traffic signals)

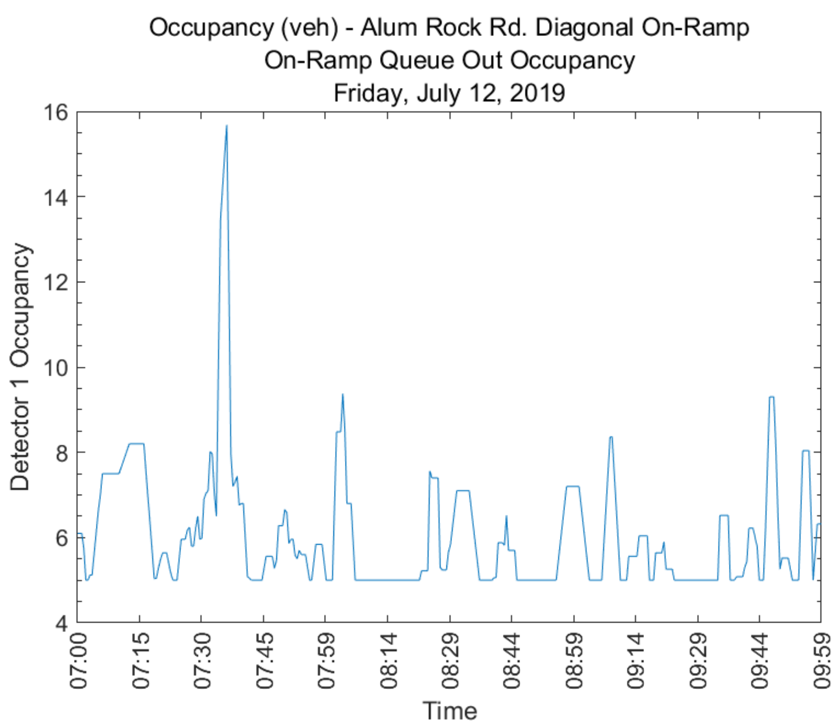

Figure 5.38 Alum Rock Ave. On-Ramp Queue Detector Occupancy (Friday July 12, 2019, Scenario 3: PATH developed coordinated ramp metering without coordination of ramp metering and arterial traffic signals)

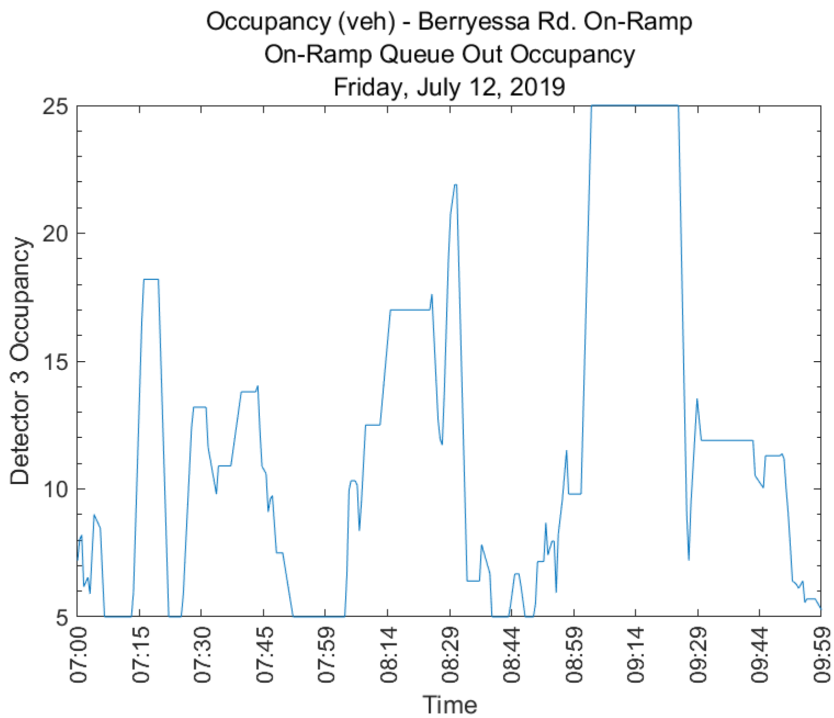

Figure 5.39 Berryessa Rd. On-Ramp Queue Detector Occupancy (Friday July 12, 2019, Scenario 3: PATH developed coordinated ramp metering without coordination of ramp metering and arterial traffic signals)

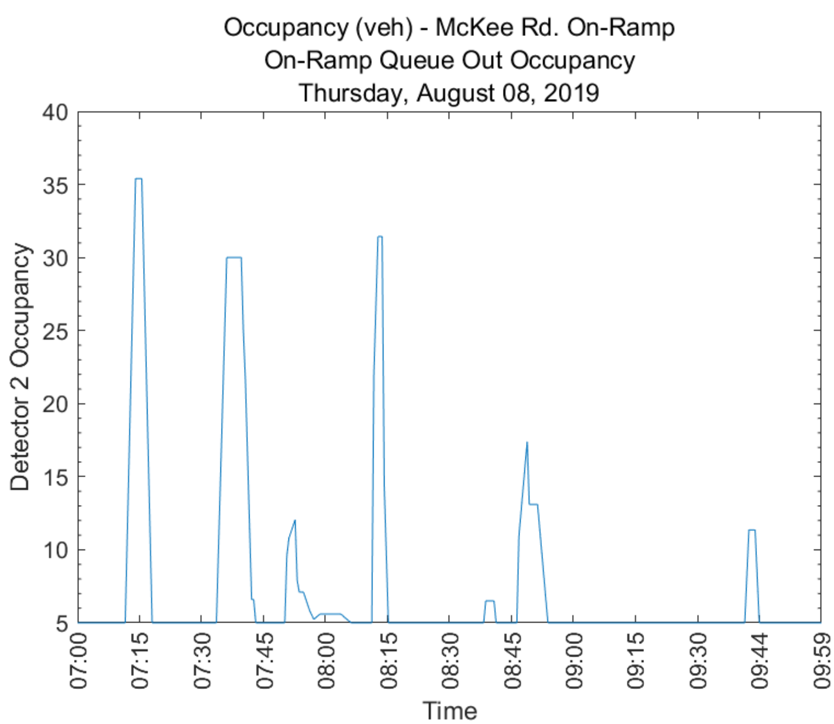

Figure 5.40 McKee Rd. On-Ramp Queue Detector Occupancy (Thursday August 8, 2019, Scenario 4: PATH developed coordinated ramp metering with coordination of ramp metering and arterial traffic signals)

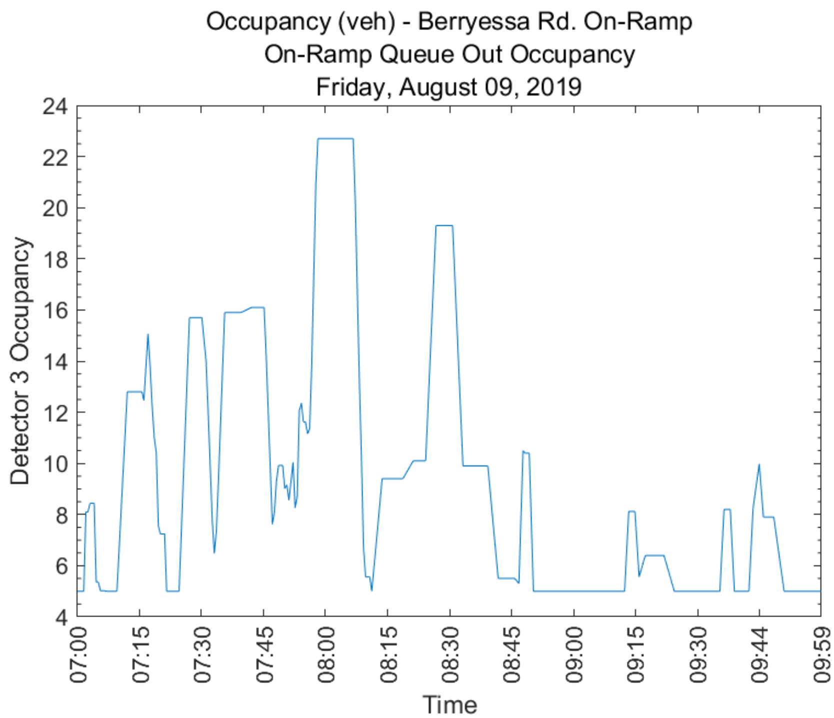

Figure 5.41 Berryessa Rd. On-Ramp Queue Detector Occupancy (Friday August 9, 2019, Scenario 4: PATH developed coordinated ramp metering <u>with</u> coordination of ramp metering and arterial traffic signals)

Chapter 6. Conclusions

6.1 Summary of the Study Findings

The objectives of the research project described in this report is to develop and implement a coordination and control strategy to manage the entry of vehicles with onramp metering and signal timing changes at the intersections along adjacent arterial(s). The feature of the developed coordination strategy between freeway ramp metering and the relevant arterial intersection traffic signal control is the simplicity and effectiveness, which intends to better use the on-ramp storage capacity. The report describes the research performed in phase II of the project: field implementation and evaluation of a proposed algorithm.

A section of the I-680 Northbound freeway with parallel arterial Capitol Ave. in the city of San Jose was selected as the test site. The AM peak was selected as the analysis period.

Three control strategies (RM, arterial signal control, and the coordination of the two) were developed for coordinated operation of ramp meters and adjacent arterial traffic signals. The selected algorithm considered available on-ramp storage capacity and dynamically reduces arterial signal cycle length in order to avoid on-ramp queue spillback and mitigate the delay in the conflicting directions of the arterial traffic.

The proposed algorithm was tested in the field from Monday April 29, 2019 to Friday August 23, 2019. The first field test was conducted using the local traffic responsive ramp metering algorithm currently implemented in Caltrans District 4 while the second test was conducted using the CRM algorithm developed by UC Berkeley's California PATH (this algorithm had been implemented in Caltrans District 3 in the past).

The field test of coordination of ramp metering and arterial traffic signals was conducted every weekday morning peak (7:00 AM to 10:00 AM). Four different scenarios were tested during this four-month test period:

- Scenario 1: Caltrans District 4 local responsive RM without coordination of ramp metering and arterial traffic signals (baseline)
- Scenario 2: Caltrans District 4 local responsive RM with coordination between ramp metering and arterial traffic signals
- Scenario 3: PATH developed CRM without coordination between ramp metering and arterial traffic signals
- Scenario 4: PATH developed CRM with coordination between ramp metering and arterial traffic signals

TTT, TD, and VMT were used to determine the effectiveness of the coordination and control strategies to improve the day-to-day operation on the Interstate 680 freeway.

This field study suggested that the proposed algorithm for coordinating freeway ramp metering and arterial traffic signals reduced the freeway delay by up to 11.49% (Scenario 2 vs. Scenario 1). Similarly, the second half of the field test (with the coordinated ramp metering algorithm) demonstrated a 5.92% (Scenario 4 vs. Scenario 3) reduction in delay. The performance evaluation for the freeway traffic is based on PeMS data, which is independent for the data collected by the PATH team. Freeway operations improved under both ramp metering algorithm tested. In addition, the new CRM algorithm was also able to prevent on-ramp queue spillback, thereby mitigating the interference on arterials during ramp metering operation.

The on-ramp queue detector occupancy data were used to determine of the presence or severity of on-ramp queue spillback. Further data collection using arterial detectors were not conducted due to the fact that City of San Jose did not save the arterial detector data for performance evaluation, which was a mistake of PATH project team since the team did not check that during the test. This is a lesson for any field operational tests.

6.2 Recommendations

Based on the results and findings of the field implementation, it is recommended that further tests be conducted during different times of the year with more fluctuations in traffic demand and on another larger and more complex freeway corridors to assess the longer-term feasibility and robustness of this proposed coordination and control strategies. Future research could include a model based integrated control strategy for both freeway RM and arterial traffic signal timing with the on-ramp traffic as the link between the two subjected to the onramp storage capacity limit. Also, there is a trade-off balance issue to be resolved as to which one has higher priority for s given traffic scenarios: freeway or arteria. This would require an integrated network-wide system optimal control with both freeways and relevant arterials involved.

Lessons learned and experience gained from this project and any further field implementations and tests could provide important technical support to the guidelines for integrated and coordinated operation of freeways and arterials in the future.

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Appendix A

The coordinated ramp-metering algorithm uses a simulation model to determine the traffic speed and density on each freeway section at each time step (*1, 2*). It then determines the metering rates to minimize the total travel time and maximizes the total distance traveled subject to appropriate constraints (*1, 2*).

The simulation model is based on the cell transmission macroscopic model that estimates the number of vehicles in each cell (segment of freeway between adjacent on/off-ramps) using density in each time step (*1, 2*). The density is determined from the 30 second detector data on flow and speed that are collected and stored at the TMC. This is a valid assumption because this freeway corridor is equipped with dual loop detectors that can accurately measure vehicle speed.

The freeway is divided into N segments such that each segment i has at most one on-ramp and one off-ramp. The following variables are defined for each segment i at each time step k:

$\rho_i(k)$: Mainline density of segment i at time step k (veh/segment)

ρ_i^J: Jam density of segment i (veh/segment)

$f_i(k)$: Mainline flow of vehicles leaving the upstream segment i, moving to the downstream segment $i+1$, at time step k (veh/time step)

$\bar{f}_i(k)$: Measured mainline flow (veh/time step)

F_i: Mainline capacity of segment i (veh/time step)

$w_i(k)$: Number of vehicles on the on-ramp corresponding to segment i, at time step k (veh)

w_i^J: Jam density of the on-ramp corresponding to segment i (veh/segment)

$r_i(k)$: Metering rate; number of vehicles entering segment i through its corresponding on-ramp at time step k, determined by the controller (veh/time step)

r_i^m: Minimum allowable metering rate for the on-ramp i (veh/period)

r_i^o: Maximum allowable metering rate for the on-ramp i (veh/period)

$d_i(k)$: Estimated/measured demand at the on-ramp corresponding to segment i at time step k (veh/time step)

$s_i(k)$: Flow at the off-ramp corresponding to segment i at time step k (veh/time step)

$v_i(k)$: Time mean speed of vehicles in segment i at time step k (segment/time step)

$u_i(k)$: Space mean speed of vehicles in segment i at time step k (segment/time step)

T: Time step

λ_i: Number of lanes in segment i (dimensionless)

L_i: Length of mainline segment i

L_i^o: Queue storage capacity of on-ramp i (number of vehicles)

By the law of conservation, the dynamics of freeway mainline are described by the evolution of mainline density $\rho_i(k)$ over time:

$$\rho_i(k+1) = \rho_i(k) + \frac{T}{\lambda_i L_i}\left(f_{i-1}(k) + r_i(k) - f_i(k) - s_i(k)\right) \tag{1}$$

Since density is related to space mean speed $u_i(k)$, the traffic flow of each time step can be expressed as:

$$f_i(k) = \lambda_i \rho_i(k) u_i(k) \tag{2}$$

Where space mean speed $u_i(k)$ is assumed to be given. Substituting equation 2 into equation 1 gives a linearized equation:

$$\rho_i(k+1) = \rho_i(k) + \frac{T}{\lambda_i L_i}\left(\lambda_{i-1}(k)\rho_{i-1}(k)u_{i-1}(k) + r_i(k) - \lambda_i(k)\rho_i(k)u_i(k) - s_i(k)\right) \tag{3}$$

Similarly the evolution of on-ramp queue is described by the following conservation equation:

$$w_i(k+1) = w_i(k) + T\left(d_i(k) - r_i(k)\right) \tag{4}$$

Suppose that there are n_i fixed sensors (loop detectors) on segment i and $\bar{v}_l(k)$ is individual vehicle speeds (measured speed) from each sensor, the time mean speed is computed by:

$$v_i(k) = \frac{1}{n_i}\sum_{l=1}^{n_i} \bar{v}_l(k) \tag{5}$$

Assuming stationary conditions, the space mean speed can be computed from $\bar{v}_l(k)$, using a harmonic mean of the measurements:

$$u_i(k) = \frac{1}{\frac{1}{n_i}\sum_{l=1}^{n_i}\frac{1}{\bar{v}_l(k)}} \tag{6}$$

Constraints

The freeway is subject to constraints in maximum and minimum mainline density, on-ramp length, and ramp metering rate. These constraints are formulated as the following inequalities:

$$0 \le w_i(k) \le L_i^O w_i^J \tag{7}$$

$$r_i^m \le r_i(k) \le min\left\{d_i(k), r_i^O, \lambda_i\left(F_i - \bar{f}_{i-1}(k)\right), \lambda_i u_i(k)\left(\rho_i^J - \bar{\rho}_i(k)\right)\right\} \tag{8}$$

$$0 \leq \rho_i(k) \leq min\{\rho_i^J, \phi(u_i(k))\} \tag{9}$$

Equation 7 relates to the on-ramp queue storage capacity. Equation 8 ensures the minimum and maximum allowable ramp metering rates are satisfied; the lower bound of on-ramp metering rate r_i^m is maintained at 300 veh/hr to prevent oversaturation. The upper bound of on-ramp metering rate is the minimum of the following four terms: the on-ramp demand, the maximum allowable metering rate, spare capacity on the mainline under free-flow conditions: $\lambda_i\left(F_i - \bar{f}_{i-1}(k)\right)$, and space capacity on the mainline under congested conditions: $\lambda_i u_i(k)\left(\rho_i^J - \bar{\rho}_i(k)\right)$. Equation 9 is an indirect constraint on ramp metering rate through the density dynamics. The function $\phi(u_i(k))$ describes the speed versus density, which is obtained from an empirical study of traffic speed drop.

Objective Function

The algorithm tries to minimize the total time spent (TTS) and maximize the total traveled distance (TTD):

$$TTS(k) = T\sum_{j=0}^{Np-1}\sum_{i=1}^{N} L_i\lambda_i\rho_i(k+j) + \alpha_w T\sum_{j=0}^{Np-1}\sum_{i=1}^{N} w_i(k+j) \tag{10}$$

where the first term of TTS is also called total travel time (TTT), the second term of TTS represents time delay due to on-ramp queue, and α_w is the on-ramp weighting parameter.

TTD is defined as

$$TTD(k) = T\sum_{j=0}^{Np-1}\sum_{i=1}^{N} L_i\lambda_i f_i(k+j) + T\sum_{j=0}^{Np-1} L_N\lambda_N f_N(k+j) \tag{11}$$

For tractability, these two objective functions are combined into a single cost function

$$J = TTS - TTD_\alpha \tag{12}$$

where subscript α represents positive weighting parameters for each segments. Choosing the weighting parameters $\alpha_{TTD,N} \gg \alpha_{TTD,0} > 0$ emphasizes maximizing the flow on the most downstream segment N and equation 12 can be written as

$$\begin{aligned} J = T\sum_{j=0}^{Np-1}\sum_{i=1}^{N} L_i\lambda_i\rho_i(k+j) + \alpha_w T\sum_{j=0}^{Np-1}\sum_{i=1}^{N} w_i(k+j) \\ - \alpha_{TTD,0}T\sum_{j=0}^{Np-1}\sum_{i=1}^{N} L_i\lambda_i f_i(k+j) - \alpha_{TTD,N}T\sum_{j=0}^{Np-1} L_N\lambda_N f_N(k+j) \end{aligned} \tag{13}$$

The objective function in equation 12 was chosen for the following reasons: TTS is related to vehicle hours traveled (VHT), and TTD is related to vehicle miles traveled (VMT). Both VHT and VMT are available in the PeMS archive, and therefore, it is convenient for any freeway user to evaluate the ramp metering control performance by accessing this open data base. In addition, minimizing TTS may discourage vehicles from entering the freeway in order to prioritize the freeway mainline speed. Minimizing negative TTD is equivalent to maximizing (positive) TTD, which ensures that the freeway is fully utilized at capacity. Therefore, to minimize the difference between TTS and TTD is to formulate the problem as a non-zero sum game. The overall effect of minimizing the objective function J is to minimize VHT and maximize VMT. In addition, since freeway average speed is defined as VMT/VHT, minimizing the cost function J also leads to higher freeway speeds.

Implementation

The coordinated ramp metering algorithm requires real time data for mainline flow, speed, and occupancy (to estimate density), as well as on-ramp and off-ramp flow. However, the on-ramp and off-ramp detectors were unable to generate any data at the time when the coordinated ramp metering algorithm was developed. As a result the algorithm currently in practice relies on on-ramp and off-ramp flow data generated from the previous years for the corresponding day, week, and month, as agreed upon by Caltrans District 3. Shortly after the trial test period of the coordinated ramp metering algorithm, Caltrans District 3 was able to resolve the technical issue and the on-ramp and off-ramp detectors were able to generate data. This coordinated ramp metering algorithm will adopt real time on-ramp and off-ramp data as part of the ramp metering control in the future, and PATH, the original developed of the algorithm, and District 3 are working together to resolve such shortcoming.

Figure 5 shows the overall system structure of the coordinated ramp metering system (*1, 2*). The red arrow indicates that the loop detector on the freeway sends 30-second field data (flow, speed, occupancy) to the PATH computer, which is installed in Caltrans District 3 TMC and is directly linked with its intranet for data acquisition (every 30 seconds), processing, traffic state parameter estimation, calculating the optimal metering rate, and sending it to the corresponding on-ramp for activation. The blue arrow starting the PATH computer sends the calculated optimal ramp metering rate to all of the cabinets (Universal Ramp Metering System (URMS) controller in the field). The yellow arrow indicates that each cabinet (URMS) sends its corresponding metering rate to the appropriate metering traffic light. The intranet connection with the 2070 controllers in the field used fixed IP addresses. This is simple setup requires no third party support and allows the PATH computer to access all raw data unaltered by any third party system. Lastly, this setup prevents any delay when receiving and transferring data.

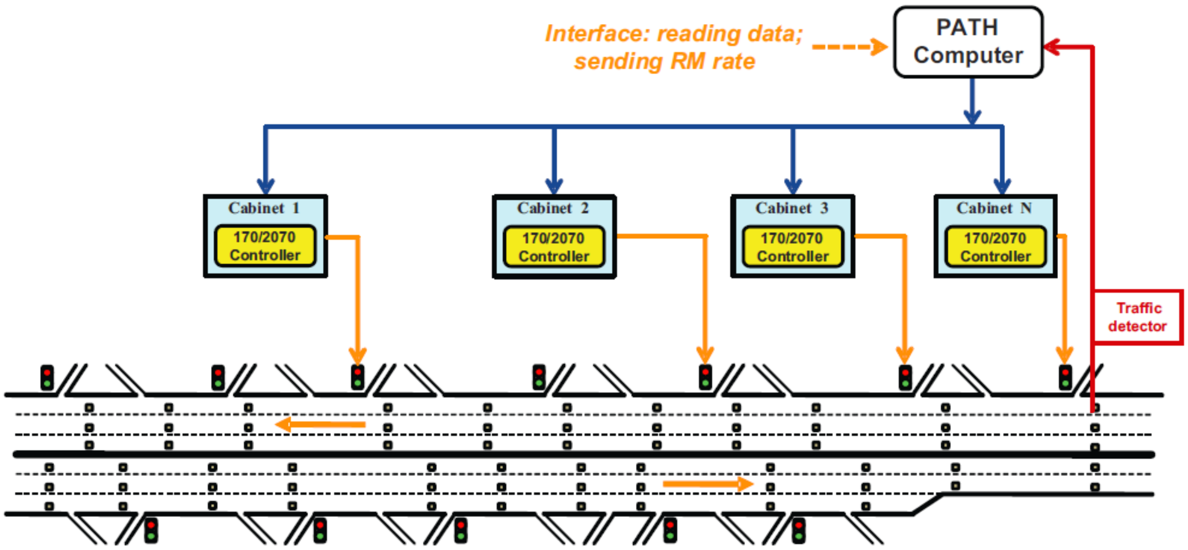

Figure A.1 Interface between ramp metering computer and controllers (*1*).

This coordinated ramp metering algorithm was not applied to all of the freeway on-ramps on SR99. Five of the on-ramps (upstream section, including Elk Grove Blvd., Eastbound Laguna Blvd., Westbound Laguna Blvd., Eastbound Sheldon Rd., and Westbound Sheldon Rd. on-ramps) currently implements local responsive ramp metering only (*1*). The on-ramp demand at these upstream on-ramps are relatively low (less than 500 veh/hr). PATH has found that the coordinated ramp metering algorithm does not generate different ramp metering rates at these five on-ramps, as compared to those generated by local responsive ramp metering (*2*).

Lastly, the analysis should take into account the downstream freeway-to-freeway interchange at US 50, which could cause queue spillback on SR99 when US 50 (which does not employ coordinated ramp metering) becomes overly congested.

Previous Evaluation

A brief evaluation was conducted immediately following the implementation of the coordinated ramp metering algorithm (*1, 2*). The data for the performance evaluation were obtained from PeMS (*3*) for the time period prior to and after the introduction of the coordinated ramp metering algorithm. For the prior case, the evaluation selected the entire month of October, 2015 and the first week of November, 2015. For the after studies, the evaluation selected the same timeframe of 2016. The evaluation was conducted during weekday morning and evening peaks. The evaluation selected three performance metrics:

- Vehicle-Miles-Traveled (VMT): indication of freeway bottleneck capacity
- Vehicle-Hours-Traveled (VHT): indication of delay
- Freeway efficiency (Q) or Average speed: ratio of VMT and VHT

Table 1.2 summarizes the before and after comparison using the above performance metrics. The bold text indicates improvement while the *italic text* indicates deterioration.

Table 1.2 Summary of SR99 morning and evening peak hourly performance

	6-7 AM	7-8 AM	8-9 AM	3-4 PM	4-5 PM	5-6 PM
2015 VMT	80118.58	74488.19	71804.62	78513.72	75687.66	69856.27
2016 VMT	83900.23	79286.52	75408.84	80324.92	76949.2	72439.83
2015 VHT	2324.12	2020.70	1366.42	1331.05	1305.34	1180.34
2016 VHT	2210.69	1973.29	1398.04	1410.57	1317.57	1206.86
2015 Q	34.47	36.86	52.54945	58.99	57.98	59.18
2016 Q	37.95	40.18	53.93897	56.95	58.40	60.02
ΔVMT	**4.72 %**	**6.442 %**	**5.019 %**	**2.307 %**	**1.667 %**	**3.698 %**
ΔVHT	**−4.881 %**	**−2.346 %**	*2.314 %*	*5.974 %*	*0.937 %*	*2.247 %*
ΔQ	**10.093 %**	**8.999 %**	**2.644 %**	*−3.460 %*	**0.723 %**	**1.420 %**

Throughout the entire morning peak VMT increased by 5.39% on average, VHT decreased by 1.64% on average, and Q increased by 7.25% on average (*1*). However, the coordinated ramp metering algorithm was less effective in the evening peak; during the entire evening peak, VMT increased by only 2.56% on average, VHT increased by only 3.04% on average, and Q decreased by 0.44% on average (*1*).

References for Appendix A

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