Coordination of Freeway Ramp Metering and Arterial Traffic Signals

Xingan (David) Kan
(Corresponding Author)
Department of Civil and Environmental Engineering
Institute of Transportation Studies
109 McLaughlin Hall, University of California, Berkeley, CA 94720
Phone: +1(408) 613-7638, Email: david.kan@berkeley.edu

Xiao-Yun Lu, Ph.D.
Partners for Advanced Transportation Technology
Institute of Transportation Studies
Richmond Field Station Building 452, Richmond, CA 94804
Phone: +1(510) 665-3644, Email: xiaoyun.lu@berkeley.edu

Alexander Skabardonis, Ph.D.
Department of Civil and Environmental Engineering
Institute of Transportation Studies
109 McLaughlin Hall, University of California, Berkeley, CA 94720
Phone: +1(510) 642-9166, Email: skabardonis@ce.berkeley.edu

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ABSTRACT

Currently, most freeway ramp metering systems and adjacent signalized arterials operate independent of each other. The independent operation can significantly reduce the performance of both types of highway facilities because it often results in providing green times to vehicles heading toward the freeway when the metered on-ramps become full and the queue spillback affects the operation on the arterial and diminishes the performance of the metering system. This paper proposes a control strategy for coordinating arterial traffic signals with freeway ramp metering. Freeway ramp meters are controlled using the ALINEA control strategy, and arterial traffic signals are controlled using a signal optimization algorithm that considers the available on-ramp and arterial queue storage capacity. An experiment conducted using a calibrated microsimulation model of a freeway corridor in San Jose, California shows that the proposed coordination strategy reduced delay on the freeway and the arterials.

INTRODUCTION

In the current state of practice, traffic controls at freeway on-ramps and arterial intersections are operated independently. During peak hours, this independent operation can significantly degrade the traffic performance of both highway facilities. If the arterial traffic signal control does not take into account the on-ramp metering rates and available queue storage, it would provide green times to the movements heading toward the on-ramp even if the on-ramp queue storage space is not available; this would create queue spillbacks affecting the arterial operations and would also affect the freeway on-ramp metering system operation. Recently, corridor management is increasingly considered for reducing congestion in urban areas. The efficient coordinated control of freeway ramp meters and adjacent signalized arterial is essential for the successful operation of travel corridors.

This paper proposes a model-based rather than an ad hoc coordination strategy for ramp meters and arterial traffic signals, which has been tested on a freeway corridor with a parallel arterial using microscopic simulation. The corridor level study considers coordinated intersections instead of isolated signals so is appropriate for corridor management, and the model-based approach allows the realistic assessment of performance on each component of the system.

The rest of the paper is organized as follows: The next section presents a literature review of on-ramp metering and arterial traffic signal coordination models and strategies. Next, the proposed strategy is presented. The following section describes the application of the proposed strategy through simulation in a real-life freeway corridor. The last section summarizes the study findings and discusses next steps in the ongoing research.

LITERATURE REVIEW

Approaches for coordinated operation of freeway ramp metering and arterial traffic signal control can be broadly divided into two groups: model based or non-model-based. Model-based methods usually come with optimization. Papageorgiou (1) presented a design approach of integrated control for traffic corridors based on the store-and-forward model. This method could formulate a traffic network control problem with any topology and various traffic control approaches, such as ramp metering, signal control and route guidance, as a linear optimization problem.

In the non-model-based strategies, a common method is to switch between different operations according to real-time traffic situation and some criteria. Kwon et al (2) proposed an adaptive control to coordinate the ramp meters and the adjacent intersection signals without prediction of demand. In this approach, vehicle counts and presence from loop detectors were used to compute congestion index for each links on arterial or freeway. Based on that, an adaptive ramp meter rate was designed to balance the congestion level of freeway and arterial, and an adaptive intersection control was used to balance that of phases by adjusting green times. The study in (3) with microscopic traffic simulation proposed four operational strategies to integrate freeway ramp meters and arterial traffic signals, namely local coordination, area wide ramp metering coordination, diversion and congestion. The work developed a list of sixteen control tactics which were claimed could achieve coordination under most traffic situations.

Tian el at (4) developed an integrated control strategy of a system with freeway and arterial intersection in a configuration of a diamond interchange. Local coordinated and diversion strategy, off ramp priority, on-ramp priority, and inhibit metering tactics were used under different traffic conditions. The objective is to evaluate the performance of the coordination between freeway adaptive local traffic responsive ramp metering and arterial adaptive interchange signal timing. Simulation showed that traffic operation performance was significantly improved compared to non-adaptive control and static ramp metering control. Similar idea was presented in Zhang's approach (5). Zhang proposed a simple locally synchronized control by adding three operations, on-ramp priority, off-ramp priority, and intersection internal metering, to the normal actuated signal control and ALINEA. Simulation results showed that the performance of this simple method could be comparable to global optimal control algorithm in some cases.

The work in (6, 7) further analyzed the dynamic interaction between the feeding intersection, on-ramp queue, and freeway mainline traffic nearby. In particular, they focused on the modeling of freeway traffic. The model was very complicated, which caused concerns in the implementation. In addition, the field implementation of coordination is often challenging because of controller hardware and software issues, communications and agency cooperation (9).

Han and Reiss (8) proposed a strategy to relieve problems in on-ramp caused by platoon type of feeding flows from intersection, which intended to individually release vehicles from the on-ramps into freeway through metering. A two-level variable metering rate to reduce delay at a ramp meter signal was investigated. The problem was modeled as minimizing total ramp delay.

In the report (10), the coordination between ramp metering with its direct feeding intersection's time of day signals was discussed. Two numerical algorithms were proposed. The global strategy aimed at optimizing corridor performance while taking into account all control elements and the traffic conditions. In the work of (11), the strategy proposed to use the default traffic parameters of the 170 or 2070 controllers, i.e. phase minimums, maximums and gap settings for practical feasibility. It is noted that those control variables are different from green split, cycle length, and off-set, as used in modern ATCS. An adaptive ad hoc method is developed in (12) for practical coordination of freeway local ramp metering and the feeding intersection traffic signal: the metering module continuously adjusts the adaptive policy depending on the congestion level of the adjacent intersection, while the intersection module determining signal phases explicitly reflects the traffic conditions at the ramp areas.

Zhang (13) proposed a methodology for traffic signal coordination of arterials adjacent to freeways; the offsets were determined based on the approaches outlined in TRANSYT-7F but were tuned to account for diversion of arterial traffic destined for freeway on-ramps, and when simulated, delay on both the arterial and freeway were reduced.

Lastly, the study in (14) presented a model-based coordination strategy that takes ramp meter rate and on-ramp queue length as the input, and determines the green duration for each movement by balancing the demand/capacity ratio through optimization. The coordination strategy was tested at an isolated signalized intersection of an arterial and a congested freeway on-ramp; delay at the intersection was effectively reduced but the applicability of the study is limited due to the size of the study site. Thus, there is a need for a large study site, and this paper will discuss the simulation tests using similar approaches in (14) but at a corridor level instead of an isolate intersection.

PROPOSED COORDINATION STRATEGY

The goal of this study is to coordinate the control of two subsystems, on-ramp metering of a freeway corridor and signal control at the feeding intersections of relevant arterial(s) with a proper strategy which can possibly improve overall traffic condition. This strategy should consider the limitation of existing systems, like detectors, traffic controller, control plans, and phase configuration. Installation of extra detectors or signal lights is not an option. It is clear that the interactions between the freeway and the arterial(s) are on the on-ramps and off-ramps. Therefore, the control of flows and queues at the ramps are the crux for the coordination of the two subsystems.

Since the intersection congestion is usually caused by the congestion on freeway, improving freeway traffic situation might help to reduce the intersection queue length. It is necessary to have a ramp metering design that could prevent freeway breakdown and maximize flow. This design also needs to consider the queue propagation at the interchange and queue spillovers or blockage in the arterial.

Along the arterials parallel to the freeway, left turns and right turns at the intersections with arterials perpendicular to the freeway serve major flows heading toward the freeway on-ramp. Similarly, through movements of the perpendicular arterials carry large volumes of traffic heading towards the freeway. Thus it is preferable to give long green durations to them before the on-ramp gets full. By doing this, on-ramp storage can be more effectively used to prevent long queues at intersections and to reduce delay. When the on-ramp becomes full, green durations of these movements should be reduced, which could be used by other movements. Otherwise, some portion of the extra green durations would be wasted due to the lack of on-ramp storage space. The same idea applies to the parallel arterial's intersections upstream of the left an rights turns that lead to on-ramp access. These are the main ideas for the coordination. In this sense, the signal control has to wisely distribute green times to intersection movements taking into account the condition at on-ramp. The current actuated control fails to do this because it extends green as long as there is a vehicle actuating the detector (until max out), disregarding whether the on-ramp can take the vehicle or not. This problem can also be observed along the arterials leading toward the freeway on-ramp; the current actuated control fails to consider the downstream arterial queue caused by congested on-ramps in the downstream direction, thus some portions of the extra green duration would be wasted due to the inability to send more vehicles towards the freeway on-ramp downstream. This means less green time for the conflicting movements and thus increases delay for the arterials. Moreover, many conflicting movements carry large volumes of freeway off-ramp traffic, and less green time and greater delay for them would lead to spillback of off-ramp queues onto the freeway, and affect freeway performance.

The proposed control/coordination strategy has two parts, ramp metering control and intersection signal control. Ramp metering rate is updated every 30 seconds, at same frequency as the detection measurement. The ramp metering control adopted is UP ALINEA (traffic measurement is located immediately upstream of the merging area) with queue-overwrite. Intersection signal control takes the information of ramp meter rate, on-ramp queue length as the input, and the downstream queue length, and determines green duration for each movement through an optimization. In this way, the coordination has been incorporated in the intersection timing. Details about the two controls are in the following subsections.

Metering

Model parameters:
k: time step index
$r(k)$: metering rate at the k-th interval
$\hat{o}$: desired occupancy, usually takes the value of critical occupancy
$o_{in}(k)\big(o_{out}(k)\big)$: occupancy masured at upstream (downstream) of on-ramp, at the k-th interval
$\tilde{o}_{out}(k)$: estimation of $o_{out}(k)$
K_R: regulator gain
$q_r(k)\big(q_{in}(k)\big)$: on-ramp (mainline upstream) flow at the k-th interval
$\lambda_{in}(\lambda_{out})$: number of lanes, upstream (downstream) of on-ramp
w: shockwave speed
α, γ: tuning parameters
L: section length

Detail of ALINEA can be found in (15). Equation (2) shows how the metering rate is updated.

$$r(k) = r(k-1) + K_R[\hat{o} - o_{out}(k-1)] \tag{2}$$

The ramp metering rate is updated every cycle such that the freeway occupancy near the merging area is maintained at a desirable value. However, ALINEA requires detectors located downstream of the merging area instead of upstream of the freeway merging area, which is the current setup. Thus, it is necessary to adopt an extended version, UP ALINEA, which estimate the occupancy downstream of the merging area based on the occupancy measured upstream of the merging area, and the flows of the freeway mainlines and on-ramps. The estimation algorithm is described as follows:

If the upstream occupancy is not greater than the critical occupancy, the downstream occupancy would be:

$$\tilde{o}_{out}(k) = \alpha o_{in}(k)\left(1 + \frac{q_r(k)}{q_{in}(k)}\right)\left(\frac{\lambda_{in}}{\lambda_{out}}\right) \tag{3}$$

Otherwise, the downstream occupancy would be estimated with the following:

$$\tilde{o}_{out}(k) = \gamma \tilde{o}'_{out}(k) + (1-\gamma)\tilde{o}'_{out}(k-1) \tag{4}$$

$$\tilde{o}'_{in}(k) = \tilde{o}_{in}(k) \cdot \frac{\lambda_{in}}{\lambda_{out}} + \frac{100L}{w\lambda_{out}} \cdot q_r(k) \tag{5}$$

In this study, assume $\alpha = 1$, $\gamma = 0.2$, w = -15km/h, and $\hat{o}$ = 25%.

Due to some safety and policy issues, the metering rate from ALINEA will be truncated if it is outside the range of 400vph to 900 vph. To avoid queue propagation, it is necessary to release vehicle (with a release rate of 700 vph) when queue spillback is detected.

Intersection Signal Optimization

Notations that may be used in this section are as follows:
i: phase index
j: additional phase index for phase prioritization
k: intersection index
m: number of phases between phase *i* and the first phase (assumed to be the through movement) of the cross street
t: time index
r: on-ramp index
$g_{ik}(t)$: green time assigned to phase *i* of intersection *k*, a decision variable
C: network-wide common cycle length
$q_{ik}(t)$: queue length of phase *i* of intersection *k*
$d_{ik}(t)$: demand of phase *i* of intersection *k*
$f_{sat,ik}$: saturation flow of phase *i* of intersection *k*
$o_{ik}(t)$: offset between phase *i* of intersection *k* and the first phase of the downstream intersection, a decision variable
$o_{i+m,k}(t)$: the offset between the first phase (through movement) of the cross street and the first phase (through movement) of this cross street at the downstream intersection
l_{acc}: starting lost time due to acceleration
$q_{i,k+1}(t)$: queue length of phase *i* of the downstream intersection
$G_{ik,min}$: minimum green time of phase *i* of intersection *k*
AR: downstream section of the phase is an arterial
L: downstream arterial link
s_L: length of the downstream link (in number of vehicles)
R: downstream section of the phase is a freeway on-ramp
RA_r: available queue storage space (in number of vehicles) of on-ramp *r*
LT: left turn phase
μ_{ijk}, v_{ijk}, δ, ε: tuning parameters

The control strategy for signalized intersections is intended to determine the optimal green durations $g_{ik}(t)$. The control strategy distributes the green durations according to the desired green time of each phase with efficient use of the on-ramp storage and downstream link storage. Given the cycle length C, demand rate $d_{ik}(t)$, queue length $q_{ik}(t)$, and discharge flow rate $f_{sat,ik}$, the term

$$\frac{q_{ik}(t) + d_{ik}(t) \cdot C}{f_{sat,ik}}$$

estimates the desired green time of a particular phase. By minimizing the first term in equation 6, with $\mu_{ijk} = v_{ijk} = 1$, all of the phases would have the same ratio of desired and assigned green durations. The offset causes the downstream intersection to initiate green at a later time, thus during the time between the beginning of green of phase *i* of intersection *k* and the beginning of green of the corresponding phase in the downstream intersection (on the same link), the flow through phase i of intersection k is: $f_{sat,ik} \cdot [o_{ik}(t) + l_{acc}]$. The second term in equation 6 penalizes on the difference between the feeding volume and the amount of available space behind the queue at the downstream intersection. The term $\sum_{i \in R} f_{sat,ik} \cdot g_{ik}(t)$ is the maximum feeding volume into the on-ramp, and the third term in equation (6) penalizes on the differences between the feeding volume and the available on-ramp storage space. μ_{ijk} and ν_{ijk} are used for phase prioritization, and δ and ε are used to scale the absolute values of the second and the third terms, respectively, to the level of the first term in equation 6, which is a ratio.

$$
\begin{aligned}
Min \sum_{k} \Bigg\{ \sum_{i \neq j} \mu_{ijk} & \left| \frac{g_{ik}(t)}{\frac{q_{ik}(t) + d_{ik}(t) \cdot C}{f_{sat,ik}}} - \nu_{ijk} \left(\frac{g_{jk}(t)}{\frac{q_{jk}(t) + d_{jk}(t) \cdot C}{f_{sat,jk}}} \right) \right| \\
& + \delta \left| \sum_{i \in AR} \left(f_{sat,ik} \cdot [o_{ik}(t) + l_{acc}] - \left[s_L - \sum_{i \epsilon L} q_{i,k+1}(t) \right] \right) \right| \\
& + \varepsilon \left| \sum_{r} \left(\sum_{i \in R} f_{sat,ik} \cdot g_{ik}(t) - RA_r \right) \right| \Bigg\}
\end{aligned}
\tag{6}
$$

Equations 7 to 10 are the constraints that address several practical limitations; equation 7 ensures that the minimum green time related to traffic safety is satisfied, equations 8 and 9 ensures that the dual ring structure is followed, and equation 9 prevents the assignment of separate offset decision variables for left turn phases.

$$g_{ik}(t) \geq G_{ik,min} \tag{7}$$

$$g_1(k) + g_2(k) = g_5(k) + g_6(k) \tag{8}$$

$$\sum_{i=1-4 \text{ } or \text{ } 5-8} g_{ik}(t) = C \tag{9}$$

$$o_{ik}(t) = \sum_{i}^{i+m} g_{ik}(t) + o_{i+m,k}(t), \forall i \in LT \tag{10}$$

The discussion in the two paragraphs above assumes that we know the cycle length in advance. Typically, this is the network-wide optimal cycle length used in the coordinated actuated arterial traffic signals.

APPLICATION

A microsimulation test was conducted to determine the effectiveness of the above coordination strategy. A site was carefully selected, calibrated using real world data, and modeled in microscopic simulation.

Site Selection

For the analysis of freeway ramp meter and arterial traffic signal coordination strategy, a medium-sized freeway segment was selected, along with its parallel arterials. Various site selection criteria were considered; the complexity of the system should be manageable, thus the freeway segment and its parallel arterial should not be longer than 5 miles, and the parallel arterial should have no more than 5 major signalized intersections; the site must have recurrent congestion due to strong interaction between freeway and arterial traffic in peak hours and be isolated, thus it should not exhibit characteristics such as backward propagation of downstream queues, congestion due to freeway to freeway exchanges, and excess upstream demand; other criteria include sufficient queue storage space, cooperation among jurisdictions, and good detector data quality.

As shown in Figure 1, the site selected for our study is a 4 mile section of I-680 from Capitol Expy. to Berryessa Rd. in San Jose, California. Due to merging traffic from the Capitol Expy., Alum Rock Ave, McKee Rd, on-ramps and high volumes of off-ramp traffic at Berryessa Rd., recurrent congestion during the morning peak (7:30-8:30 AM) was observed in the northbound direction. Capitol Ave. is the parallel arterial immediately east of the freeway and it channels most of the traffic heading onto the northbound direction of I-680 during the morning peak period.

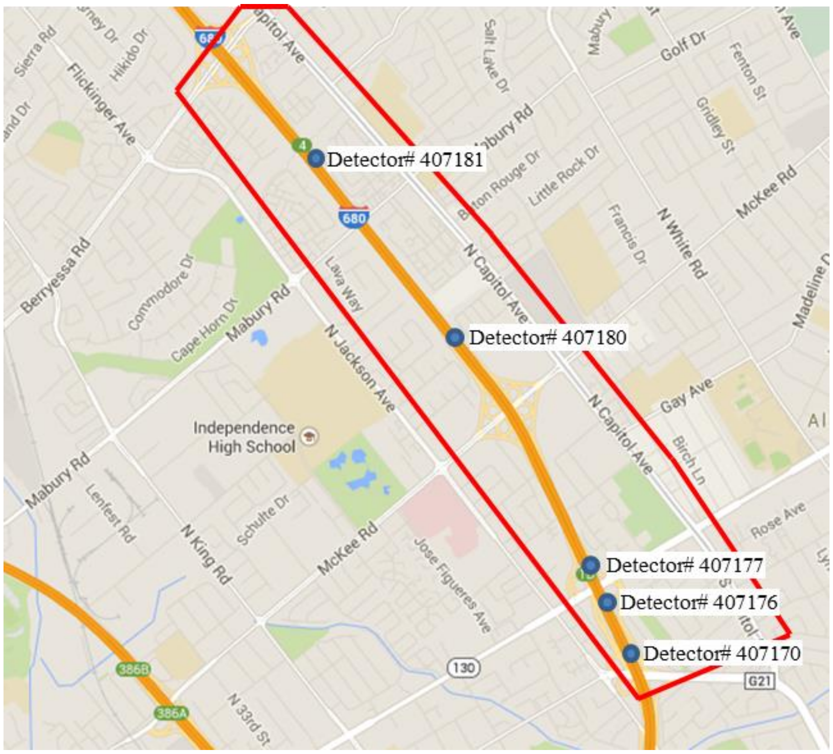

Figure 1 Study site: I-680-Capitol Ave and detector locations.

Model Application and Calibration

The selected site was coded into the AIMSUN (16) microscopic simulation model. For the freeway, loop detector data (speed and flow) aggregated over 5 minutes for Wednesday, April 15, 2015 were obtained from the PeMS system (17) and used for model input and calibration. On-ramp meters operate under the ALINEA strategy and the arterial traffic signals operate with time of day (TOD) coordinated actuated timing plans. The timing plans were obtained from Caltrans and the city of San Jose. For the arterial intersections, hourly turning moment traffic counts collected by the city of San Jose on representative weekdays were used. Data on arterial performance (speeds, travel times) were not available at the time of the study but for future work, these will be collected at each 5 minute interval of the morning peak hour. Twenty replications of the simulation model runs with different random number seeds were made for cases with and without the coordination of ramp meters and arterial traffic signals.

The model was calibrated to existing conditions prior to the evaluation of the proposed control strategy. The predicted flows and speeds at selected locations on the freeway mainline were compared with real traffic measurements in every 5 minutes to assess the accuracy of the simulation model in representing observed conditions. For flows, we need at least 85% of the flows to be acceptable and GEH<5 (18). According to this criterion, simulated flow quantity is said to be acceptable if it satisfies the requirement below.

Link flow quantity

- If 700vph < real flow < 2700vph, simulated flow has an error within 15%;
- If real flow < 700vph, simulated flow has an error within 100vph;
- If real flow > 2700vph, simulated flow has an error within 400vph.

The GEH statistic is computed as

$$GEH(k) = \sqrt{\frac{2[M(k) - C(k)]^2}{M(k) + C(k)}} \tag{1}$$

where:
$M(k)$ is the simulated flow in time interval k
$C(k)$ is the corresponding field measured flow in time k.

A satisfactory calibration requires that on average of all detectors, for at least 85% of time points k, the flow is to satisfy the condition $GEH(k) < 5$. For speed, the relative root mean squared error (RRMSE) of simulated speed values are required to be 15% or lower, on average of all detectors. For arterial hourly flows, GEH < 5 must be satisfied for at least 85% of the turning movement at the major intersections.

Tables 1 and 2 summarize the calibration results for the five detectors along the four mile stretch of Northbound I-680, as well as the hourly flows of the major arterial intersections. It can be seen that on average, for the freeway, the simulated flows and speeds satisfy the calibration criteria. Similarly, the simulated flows at major arterial intersections satisfy the calibration criterion. Calibration was not performed in the southbound freeway direction because of the low volume during the morning peak analysis periods.

Table 1 Calibration of Flows

Freeway: 5-min flows of I-680 Northbound					
Detector ID	Target	Cases	Cases Met	% Met	Target Met?
407170	GEH < 5 for > 85% of time k	12	12	100%	Yes
407176	GEH < 5 for > 85% of time k	12	10	83.33%	No
407177	GEH < 5 for > 85% of time k	12	11	91.67%	Yes
407180	GEH < 5 for > 85% of time k	12	8	66.67%	No
407181	GEH < 5 for > 85% of time k	12	11	91.67%	Yes
Overall	**GEH < 5 for > 85% of time *k***	**60**	**52**	**86.67%**	**Yes**
Arterial: hourly flows					
Target		Cases	Cases Met	% Met	Target Met?
GEH < 5 for > 85% of the turning movements at the major intersections		32	37	86.49%	Yes

Table 2 Calibration of Freeway Speeds

Time Interval	Detector ID #									
	407170		407176		407177		407180		407181	
	Obs.	Sim.	Obs.	Sim.	Obs.	Sim.	Obs.	Sim.	Obs.	Sim.
7:30 - 7:35	36.5	28.4	30.5	20.3	39.9	30.8	46.7	46.9	59.2	51.4
7:35 - 7:40	30.4	28.1	24.4	20.6	35.8	30.7	43.1	46.4	57.5	50.8
7:40 - 7:45	24.8	27.8	20.1	20.4	32.6	30.8	40.9	46.4	58.9	50.7
7:45 - 7:50	22.7	26.8	17.1	20.0	31.1	30.6	41.2	45.0	59.7	50.3
7:50 - 7:55	20.7	25.5	15.8	19.2	30.0	30.5	40.2	44.0	59.8	49.1
7:55 - 8:00	21.4	24.3	15.4	18.5	30.3	30.2	38.6	42.4	60.0	48.0
8:00 - 8:05	21.9	24.0	17.3	18.3	30.7	29.9	35.5	40.6	59.0	46.6
8:05 - 8:10	20.7	21.7	17.3	17.5	29.8	29.7	35.3	39.7	58.2	46.0
8:10 - 8:15	20.8	20.6	17.0	17.1	30.1	29.0	34.7	38.8	57.9	45.2
8:15 - 8:20	20.3	19.8	15.9	16.6	29.8	28.5	35.2	38.5	55.7	44.7
8:20 - 8:25	19.3	19.6	15.3	16.6	30.1	28.6	36.1	38.1	56.4	44.5
8:25 - 8:30	21.2	19.9	17.9	16.8	31.3	28.3	38.7	38.1	56.7	44.5
RRMSE	12.52%		14.94%		8.68%		9.73%		18.57%	
Target	<15%		<15%		<15%		<15%		<15%	
Target Met?	Yes		Yes		Yes		Yes		No	
Overall	**12.89%**									
Target	**RMSSE<15%**									
Target Met?	**Yes**									

Simulation Results and Discussions

Changes in delay and total distance travelled as a result of coordinating ramp meters and arterial traffic signals are shown in Table 3. The parallel arterial, Capitol Ave, experienced reduction in average delay in both directions; while the northbound direction experienced very small reduction in average delay, the southbound direction experienced significant reduction in average delay. This is because the southbound direction conflicts with the left turn movements from Capitol Ave to the major cross streets that have immediate access to the freeway, and by reducing the green times to these left turn movements when the on-ramps are full and reallocating the green times to the conflicting southbound through movements, the southbound traffic spent less time waiting at the signalized intersections. Similarly, eastbound directions of the cross streets, which conflict with the left turning traffic attempting to access the freeway on-ramps, also experienced significant reduction in average delay, and this can be explained by the same idea. Furthermore, many vehicles in the eastbound direction come from the freeway off-ramp, thus the reduction in delay for the eastbound direction could also reduce delay and spillback at the freeway off-ramps, which can improve the performance of the freeway mainline. This is evident in the increase of total distance travelled on the freeway, which indicates that the flow was increased; moreover, the total delay increased but by less than 1/3 of the percent increase in total distance traveled, which shows that the average delay reduced slightly. However, slight increase in average delay was experienced by the westbound direction (towards the on-ramp) of the cross streets, but this was outweighed by the performance improvements of both the conflicting directions of arterial street and the freeway.

Table 3 Summary of Simulation Results.

<table>
<tr><th></th><th colspan="2">Before Coordination</th><th colspan="2">After Coordination</th><th colspan="2">% Difference</th></tr>
<tr><td colspan="7">Arterial Performance</td></tr>
<tr><td colspan="7">Average Delay of Parallel Arterial* (min/veh)</td></tr>
<tr><td>Capitol Ave NB</td><td colspan="2">7.55</td><td colspan="2">7.40</td><td colspan="2">-1.95%</td></tr>
<tr><td>Capitol Ave SB</td><td colspan="2">2.05</td><td colspan="2">1.79</td><td colspan="2">-12.73%</td></tr>
<tr><td colspan="7">Average Delay of Cross Street (sec/veh)</td></tr>
<tr><td>Alum Rock WB</td><td colspan="2">34.96</td><td colspan="2">36.57</td><td colspan="2">4.62%</td></tr>
<tr><td>Alum Rock EB</td><td colspan="2">9.52</td><td colspan="2">8.01</td><td colspan="2">-15.88%</td></tr>
<tr><td>McKee WB</td><td colspan="2">10.04</td><td colspan="2">10.62</td><td colspan="2">5.80%</td></tr>
<tr><td>McKee EB</td><td colspan="2">2.03</td><td colspan="2">1.34</td><td colspan="2">-34.10%</td></tr>
<tr><td>Berryessa WB</td><td colspan="2">9.95</td><td colspan="2">11.23</td><td colspan="2">12.86%</td></tr>
<tr><td>Berryessa EB</td><td colspan="2">7.71</td><td colspan="2">6.71</td><td colspan="2">-12.86</td></tr>
<tr><td colspan="7">Freeway Performance</td></tr>
<tr><td></td><td>Total Delay (veh-hr)</td><td>Total Distance Traveled (veh-miles)</td><td>Total Delay (veh-hr)</td><td>Total Distance Traveled (veh-miles)</td><td>Change in Total Delay</td><td>Change in Total Distance Traveled</td></tr>
<tr><td>I-680 NB**</td><td>169.88</td><td>13749.05</td><td>171.73</td><td>14220.10</td><td>1.09%</td><td>3.43%</td></tr>
</table>

* Entire stretch of parallel arterial

** The uncongested southbound direction was not examined due to its low volume and the lack of traffic entering the southbound direction from the arterials

CONCLUSIONS AND RECOMMENDATIONS

This study developed a ramp meter and arterial traffic signal coordination strategy that improved the performance of both the freeway and the arterial. The coordination of freeway ramp meters and arterial traffic signals was achieved by optimizing green distributions of arterial traffic signals such that the green time for each movement is balanced according to its demand while taking into account the available spaces on the freeway on-ramp as well as the downstream section of the arterial.

The proposed coordination strategy was tested and compared with the independent operation of ramp meters and arterial traffic signals. Simulation results show that both the freeway and the arterials experienced reduction in delay, and the freeway achieved higher flow.

Future work will perform sensitivity analysis for changes in traffic volumes, and will include freeway incidents in the corridor. Lastly, field tests of the coordination strategy will be conducted.

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