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16. ABSTRACT

This project has focused on field implementation and testing of a Coordinated Ramp Metering (CRM) algorithm at California State Route 99 Northbound corridor in Sacramento between Calvine Road and the SR50 interchange after 12th Ave. It is a 9 mile long corridor with 11 onramps to which the CRM algorithm has been applied. After refining the CRM algorithm using a microscopic simulation of the test site, the project team worked closely with Caltrans Headquarters Division of Traffic Operation and District 3 Freeway Operation for (a) finalizing the ConOps (b) real-time data acquisition from 2070 controllers in the field and establishing correct data mapping between field detectors and the controller in the CRM algorithm; (c) implementing the CRM algorithms as real-time code running on a PATH computer; (d) estimating realtime traffic state parameters; (e) system integration of all software modules and hardware components; (f) conducting three weeks of dry-run tests, two weeks of progressive switching-on, system tuning and preliminary test, and five weeks of extensive testing and data collection; and (h) accomplishing performance analysis with PeMS data. By comparing the VMT (Vehicle Miles Travelled), VHT (Vehicle Hours Travelled), and the ratio VMT/VHT during field tests in 2017 with data from 2016 in the same period, during the AM peak hours, VMT/VHT was increased by 7.25% on average, which indicated traffic improvement. During the PM peak hours, VMT/VHT decreased by 0.44% on average, which meant no traffic improvement. The reason was that the traffic was not congested most of the time in PM hours. This suggests that the CRM algorithm tested could be more effective for congested traffic.

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Abstract

This project has focused on field implementation and testing of a Coordinated Ramp Metering (CRM) algorithm that is based on a simplified optimal control approach. The test site was the California State Route 99 Northbound (SR99 NB) corridor in Sacramento between Calvine Road and the SR50 interchange after 12^{th} Ave (Abs. PostMile 290.454 - 299.467). It is a 9 mile long corridor with 11 onramps to which the CRM algorithm has been applied. After refining the CRM algorithm using a microscopic simulation of the test site, the project team worked closely with Caltrans Headquarters Division of Traffic Operation and District 3 Freeway Operation for (a) finalizing the ConOps, which was the blueprint for the overall structure of the project; (b) real-time data acquisition from 2070 controllers in the field and establishing correct data mapping between field detectors and the controller in the CRM algorithm; (c) implementing the CRM algorithms as real-time code running on a PATH computer; (d) estimating real-time traffic state parameters; (e) system integration of all software modules and hardware components; (f) conducting three weeks of dry-run tests (without control actuation), two weeks of progressive switching-on, system tuning and preliminary test, and five weeks of extensive testing and data collection; and (h) accomplishing performance analysis with PeMS data. By comparing the VMT (Vehicle Miles Travelled), VHT (Vehicle Hours Travelled), and the ratio VMT/VHT (defined as system efficiency in PeMS; interpreted as *average speed*) during field tests in 2017 with data from 2016 in the same period, during the AM peak hours (6:00AM - 9:00AM), VMT/VHT was increased by 7.25% on average, which indicated traffic improvement. During the PM peak hours (3:00PM - 6:00PM), VMT/VHT decreased by 0.44% on average, which meant no traffic improvement. The reason was that the traffic was not congested most of the time in PM hours. This suggests that the CRM algorithm tested could be more effective for congested traffic.

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List of Acronyms and Abbreviations

ALINEA	(Asservissement LINéaire d'Entrée Autoroutière) A local occupancy based traffic responsive ramp metering algorithm
ATM	Active Traffic Management
BN	Bottleneck
Caltrans	California Department of Transportation
CMS	Changeable Message Sign
CRM	Coordinated Ramp Metering
CTM	Cell Transmission Model
EAR	Exploratory Advanced Research
EB	East Bound, (WB, SB, NB: West Bound, South Bound, North Bound)
FD	Fundamental Diagram
FHWA	Federal Highway Administration
FLOW	A coordinated ramp metering algorithm
GEH	after the name of Geoffrey E. Havers, who created a statistics test similar to a chi-squared test
GPL	General Purpose Lane
HERO	HEuristic Ramp metering coOrdination
HOV	High Occupancy Vehicle
LARM	Local Adaptive Ramp Metering
LP	Linear Programming
LQI	Linear Quadratic Control with Integral Action
LRRM	Local Responsive Ramp Metering
LWR	Lighthill-Witham-Richards
METANET	A second order traffic model including speed and density as state variables
MPC	Model Predictive Control
NGSIM	Next Generation Simulation
OFR	Exit ramp
ONR	Entrance ramp

OpenStreetMap	A map of the world, an open source
PATH	California Partners for Advanced Transportation Technology
PeMS	Performance Measurement System
PM	Postmile
RM	Ramp Metering
RMSE	Root Mean Square Error
RMSP	Root Mean Square Percentage
RTMC	Regional Traffic Management Center
SR	State Route
SVO	Speed, Volume, Occupancy
SWARM	System-wide Adaptive Ramp Metering
TD	Total Delay
TMC	Traffic Management Center
TOD	Time-of-Day
TOPL	Tools for Operations Planning
TNOS	Total Number of Stops
VMT	Vehicle Miles Travelled
TTS	Total Time Spent
TTT	Total Travel Time
VDS	Vehicle Detector System
VHT	Vehicle Hours Travelled
VII	Vehicle Infrastructure Integration
VMS	Variable Message Signs
VMT	Vehicle Miles Travelled
VSA	Variable Speed Advisory
VSL	Variable Speed Limit

Executive Summary

This report documents the work conducted in the project: Field Test of Coordinated Ramp Metering (CRM). The test site was California State Route 99 Northbound (SR99 NB) in Sacramento between Elk Grove and the SR50 interchange after 12th Ave (CA Post-Mile 10 - 32.767; or Abs. Post-Mile 284.57 - 299.467). It is a 13 mile long corridor with 16 onramps and 11 off-ramps. System modeling, real-time data acquisition and traffic state parameter estimation have been conducted for the corridor. However, the CRM algorithm has been applied to the downstream 11 onramps (between Calvine Road and 12th Ave) for both AM peak hours (6:00am-9:00am) and PM peak hours (3:00pm-6:00pm). The upstream 5 onramps still used the default Local Responsive Ramp Metering (LRRM). The 11 downstream CRM controlled onramps included:

- Calvine Road EB, WB
- Mack Road EB, WB
- Florin Road EB, WB
- 47th Ave EB, WB
- Fruitridge Road EB, WB
- 12th Ave

The CRM algorithm uses a method called Model Predictive Control which is a simplified Optimal Control since it only considers a finite time horizon. The algorithm was simulated based on a well-calibrated microscopic simulation model for SR99 NB 13 mile section using the Aimsun traffic simulator. *Optimal* here means that the proposed CRM algorithm calculates the ramp metering rate for maximizing VMT (Vehicle-Miles-Traveled) and minimizing VHT (Vehicle-Hours-Traveled). The CRM algorithm is essentially different from Local Responsive Ramp Metering (LRRM) which determines RM rate of an onramp only based on local mainline occupancy/flow measurement of its immediate upstream detector. The CRM algorithm determines RM rate by looking at mainline occupancy/flow of the whole corridor, the demand at all onramps and the out-flow from off-ramps. The optimization is for the whole corridor. Intuitively, the implemented algorithm intends to control the SR99 NB corridor as a long discharging stretch in the sense that the downstream should not be more congested than the upstream traffic on average. It is believed that this is the best way to release the congested traffic faster.

The main tasks of this project included: fine tuning the CRM algorithm through simulation for SR99 NB in AM peak traffic; refinement of the ConOps for the overall system structure; algorithm implementation as real-time code on a PATH control computer located at the Caltrans District 3 RTMC and directly linked with the 2070 traffic controllers in the field through Caltrans intranet; real-time data acquisition and data mapping; real-time data cleansing, imputation and traffic state parameter estimation; conducting dry-runs (running the algorithm with real-time field data as input but without activation of control); software and hardware system integration; preliminary test and system tuning; extensive test and "after" scenario data collection; evaluation of the performance with PeMS data; and writing this final report to document all the algorithms and system developed, lessons learned and experience gained in the project and making recommendations.

With support from Caltrans HQ Traffic Operations Division and District 3 RTMC traffic engineers, a very simple ConOps was adopted: a PATH computer located in the D3 RTMC directly linked with 2070 controllers in the field through the Caltrans intranet. Every 30s, it polled traffic detector data, estimated traffic state parameters, calculated the optimal CRM rate for each onramp, and sent it back to the individual 2070 controllers for activation. URMS was the application software on each 2070 controller. This is the simplest way for system interface. The advantages are obvious: it is simple and direct (avoided any interface with middleware), and reliable in both data acquisition and control activation.

After system setup, the project team worked closely with Caltrans District 3 RTMC engineers on 2070 controllers in the field and established a mapping between URMS data, actual location of the traffic detectors and the model of the CRM algorithm. After that, the project team had a clear picture about the relation of each URMS controller, its IP address and location, and its loop detector cards which the loop detectors in the field were wired with. This was a critical step in the preparation for field testing. If this mapping had been incorrect, the overall traffic observed from the real-time data in the algorithm and the RM rate calculation would have been wrong.

The implemented CRM algorithm on the PATH computer mainly contained three modules: real-time traffic data acquisition, traffic state estimation, and real-time CRM algorithm. The real-time traffic acquisition module was the interface of the PATH computer with URMS on the 2070 controllers in the field. It collected all raw traffic data (flow, occupancy, and speed), and then put them in a database. These collected raw traffic data were then fed into the traffic state parameter estimation modules for processing: imputing missed data, filtering the noise and estimating traffic state parameters such as density. The estimated traffic state parameters for the overall corridor were then used for an optimal CRM rate calculation for each onramp. The real-time CRM algorithm was based on a mathematical model of the freeway corridor. After all the software components were built, the PATH computer conducted dry runs for three weeks, which actually ran all the processes mentioned above except that the RM rate was not sent to the RM signal for activation. Instead, all the traffic state parameters and RM rate were saved to files for analysis. Those saved data were carefully checked to make sure every part of the system worked correctly and robustly in the sense that even if there was some loop detector data fault, the historical data for the same time of a day would be used, and therefore would not affect the CRM calculation significantly. In case there was a problem with the PATH computer, the 2070 controller in the field would automatically activate the default LRRM.

After the dry run, the project team had a meeting with the project panel and made a presentation in the middle of September 2016. A decision was made on how and when to switch on the CRM for field tests. On the day when the system was switching on, all the core members of the project panel including Caltrans HQ and D3 freeway operation engineers and PATH project team were present and witnessed the moment of CRM switching on. We spent two weeks for progressive switching on and minor system tuning. In the first week, the project team tightly monitored the CRM system, tuned the algorithm and observed the traffic through D3 freeway traffic video systems as well as Google Traffic. The project panel was updated every day on the status of the tests with presentation slides which included traffic state parameter estimation, traffic pattern, and CRM rate for each onramp. From the second week, the panel was updated every week with similar information. The presentation slides also included the comparison of CRM rate with default LRRM rate which was calculated on the PATH CRM computer.

After having confirmed that the overall system was running correctly and the CRM rate calculation was reasonable, five weeks of extensive tests and data collection on the "after" scenario were conducted with close monitoring of the CRM rate for each onramp. Traffic information and the comparison of the CRM rate with the default Local Responsive Ramp Metering rate were regularly updated to the project panel for feedback. After the five-week tests, PeMS hourly VHT and VMT data were used for evaluation of the performance. It is noted that PeMS data are completely independent from the data in the PATH CRM computer obtained directly from the 2070 controllers in the field. By doing so, the project team intended to achieve an objective performance evaluation. The data for the same period of weekdays in October 2015 and October 2016 have been used for comparison in performance analysis, i.e. corresponding weekdays were compared. For example, Tuesday was compared with Tuesday. This comparison should be reasonable since the AM peak traffic on the freeway corridor was mainly commuters going to work in Sacramento. To address traffic demand fluctuations and differences, the ratio VMT/VHT was used as the performance parameter, which was defined as the "*efficiency*" (Q value) in PeMS and could be understood as the average speed of the freeway traffic. The increase of the ratio indicates system performance improvement. It is believed that this ratio can more reasonably accommodate traffic demand changes since both VMT and VHT are accounted for.

The performance evaluation over the five weeks of data showed that ratio VMT/VHT was increased by 7.25% on average for AM peak hours (6:00am – 9:00am) which usually had congested traffic. For PM peak hours (3:00pm – 6:00pm), the ratio VMT/VHT was decreased by 0.44% on average, which was within the statistical error margin. It meant that the CRM algorithm could not improve PM traffic. The reason could be that the traffic was not congested most of the time in the PM peak hours, so the CRM algorithm could not improve it. This observation suggests that the CRM algorithm could be effective for congested traffic caused by high demand.

During the test, Caltrans District 3 RTMC traffic engineers were also closely monitoring the traffic in the corridor on a daily basis and provided support. Based on their observation through operation and performance parameter of PeMS, they agreed on the performance analysis of the project team. RTMC engineers also made the following request after the performance analysis: (a) to continue using the CRM control as the daily operation for the SR99 NB corridor; (b) to develop a computer interface for the CRM algorithm so that Caltrans freeway traffic engineers could more conveniently apply it to other similar freeway corridors.

Chapter 1. Introduction

This research report documents the work performed under California Department of Transportation contract 65A0537 for the project titled "Field Test of Coordinated Ramp Metering (CRM)".

The project was sponsored by the California Department of Transportation (Caltrans) and undertaken by the California Partners for Advanced Transportation Technology (PATH). Officially, the project duration was from 06/30/2014 to 6/30/2016.

Most Ramp Metering (RM) operations in California are fixed by Time-of-Day (TOD) or locally responsive to occupancy measurements immediately upstream of the entrance ramp merge. The locally responsive ramp metering strategy adjusts the ramp metering rate to improve traffic flow at the entrance ramp merge area. Traffic on each section of a freeway affects each other dynamically: downstream section flow depends on the demand flow from its upstream, and downstream congestion could back-propagate to the upstream, corridor CRM can go further by coordinating the entrance ramp flow of relevant sections such that the whole corridor could achieve better throughput and accommodate more traffic. CRM has been studied in analysis and simulation in several previous works [1] [2] [3], which have indicated some potential in reducing freeway congestion at recurrent bottleneck locations. These concepts need to be tested in the field to determine whether the projected benefits could be achieved in practice in California. If the results of field testing are favorable, it could provide the basis for future widespread adoption of CRM control strategies to further improve mobility and safety and reduce energy and emissions impacts of freeway congestion.

Freeway corridor traffic flow is limited by bottleneck flow. If the section upstream of a bottleneck is congested, the bottleneck flow will drop well below its capacity. A logical approach to maximize recurrent bottleneck flow is to create a discharge section immediately upstream of the bottleneck.

The objective of this project was to conduct field implementation, test and evaluation of a newly developed CRM algorithm [2] [4]. The main tasks of this project are listed as follows:

- fine tuning CRM algorithm through simulation for SR99 NB AM Peak traffic
- algorithm implementation as real-time code on a PATH control computer located at the Caltrans District 3 RTMC
- system integration with 2070 traffic controllers in the field through District 3 RTMC intranet
- establishing a mapping between URMS data, actual location of the traffic detectors and the model of the CRM algorithm
- data cleansing, filtering and imputation based on field raw loop detector data
- real-time traffic state parameter estimation
- conducting dry run (with real-time traffic data input and CRM rate calculation and without activation)
- progressive activation of the CRM control and control parameter tuning to confirm that the CRM algorithm was working reasonably
- conducting extensive tests and "after" scenario data collection
- objective performance analysis based on PeMS hourly VMT and VHT data
- writing up final report to document: (a) system developed; (b) the CRM algorithm implemented; (c) lessons learned and experiences gained for the CRM algorithm implementation and test; (d) performance analysis of the algorithm for field operation; (e) the limit or requirement on the application of the algorithm; and (f) recommendations for the next step for wider range application on other similar freeway corridors.

The test site selected is a 13 mile long stretch: California State Route 99 Northbound (SR99 NB) in Sacramento between Elk Grove and 12th Ave (CA Post-Mile 10 to 32.767, or Abs. Post-Mile 284.57 - 299.467) with 16 onramps and 11 off-ramps. Further downstream of the 12th Ave is the interchange with SR50. This corridor is relatively isolated in the sense that the most downstream traffic at the interchange with SR50 was not congested most of the time although it is congested sometimes. After careful traffic analysis, the project team decided to apply the CRM algorithm only for the downstream 11 onramps and leave the upstream 5 onramps still using the original Local Responsive Ramp Metering strategy. Such a consideration was based on three reasons: (a) the traffic demands on the downstream onramps were relatively higher; (b) the overall system was shorter and therefore simpler; and (c) most importantly, improvement of traffic downstream would naturally improve the traffic upstream. Practical test results indicated that this decision was correct.

The operation time periods on workdays were the same as before: AM peak hours (6:00am-9:00am) and PM peak hours (3:00pm-6:00pm). The following Table 1-1 shows the onramps of SR99 NB section under study and the RM strategy applied:

Table 1-1 Entrance ramp list of the test corridor with applied RM strategy

Entrance Onramp ID	**Street Name**	**RM Strategy**		**Entrance Ramp ID**	**Street Name**	**RM Strategy**
1	Elk Grove	LRRM		9	Mack Rd WB	CRM
2	Laguna Blvd EB	LRRM		10	Florin Rd EB	CRM
3	Laguna Blvd WB	LRRM		11	Florin Rd WB	CRM
4	Sheldon Rd EB	LRRM		12	47th Ave EB	CRM
5	Sheldon Rd WB	LRRM		13	47th Ave WB	CRM
6	Calvine Rd EB	CRM		14	Fruitridge Rd EB	CRM
7	Calvine Rd WB	CRM		15	Fruitridge Rd. WB	CRM
8	Mack Rd EB	CRM		16	12th Ave	CRM

In Table 1-1, the entrance ramp ID is in sequence from upstream to downstream. All the mainline section ID corresponds to the entrance ramp ID in the sense that the section is the one immediately upstream of the entrance ramp.

The CRM tested is significantly different from LRRM in the sense that LRRM determines the RM rate of an onramp only based on local mainline occupancy/flow of its immediate upstream detector, while CRM determines the RM rate by looking at mainline occupancy/flow of the whole corridor, the demand at all onramps, and out-flow from off-ramps. The CRM algorithm implemented and tested in this project was based on a simplified version of *optimal control*, called *model predictive control*. It is a linear traffic model with the assumption that the average speed of each section can be measured. This was the case for the test site since all URMS of the 2070 controllers in the field provided reasonably good speed estimation based on dual loop traffic detector data. The control objective is to minimize the total VHT and to maximize the total VMT of the freeway corridor. This is a non-zero sum game approach. It is clear that VHT should be reduced and VMT should be increased for traffic improvement. However, one could not simply limit restrictively the number of vehicles entering the freeway since the demand is very high in AM peak hours along the SR99 NB corridor. Instead, it is necessary to encourage more vehicles getting into the freeway, which can be implemented by properly increasing VMT. Therefore, practical optimal freeway corridor traffic control should minimize a weighted difference: VHT-αVMT in general, where α is a positive number and it converts the unit of VMT into that of VHT. It is noted that minimizing (-VMT) is equivalent to maximizing VMT, which means encouraging more vehicles getting into the freeway. Intuitively, the implemented algorithm intends to control the SR99 NB corridor as a long discharging section in the sense that the downstream should not be more congested than the upstream traffic on average. It is believed that this is the best way to increase the throughput of the overall traffic.

The following chapters (Chapter 2 through Chapter 4) have documented all main findings of the field test. The rest of this report is organized as follows. Chapter 2 describes the mathematical formulation of the CRM algorithm. Traffic detector data acquisition and traffic state parameter estimation are presented in Chapter 3. Chapter 4 is for the concept of operations (ConOps), system set-up, algorithm implementation, and progressive test procedure. Chapter 5 presents PeMS data analysis for performance evaluation, and summary of field test results. In the last chapter, Chapter 6, some remarks on the project and some recommendations have been made which are necessary for the implementation of the algorithm in other freeway corridors and for further research for extending the algorithm to freeway networks with multiple freeway corridors.

Chapter 2. Coordinated Ramp Metering Algorithm

This chapter documents the real-time implementation of CRM algorithm developed in the former FHWA EAR program supported project [2]. The algorithm is essentially based on Optimal Control. Since an optimal control need to consider an infinite time interval, it is not practical for implementation. Therefore, a simplified Model Predictive Control (MPC) [4] is used as an approximation, which only needs to consider a finite time interval, i.e. to predict traffic based a model for finite number of future steps. The time interval we used for control is 30s which is the same as the field traffic data updated time interval on 2070 controller. As indicated in the name, MPC needs a rigorous dynamic traffic model which can describe the changes of traffic along the corridor. It is obvious that the faithfulness of the model would affect the performance of the control. This can be achieved reasonably well thanks to the availability of the real-time data from the field, which update the model to the current status.

2.1 CRM Design with Model Predictive Control

The main points of MPC can be summarized as follows:

- The system in consideration needs to have a dynamical mathematical model with all the state variables estimated or measured;
- The control problem is usually formulated as an optimal control with a proper objective function with the model plus appropriate constraint; as default, the optimal control problem is formulated in an infinite time horizon;
- The problem is then simplified by assuming a finite look ahead time horizon on which the system dynamics are discretized;
- Correspondingly, the objective function and the constraints are also discretized in the finite time horizon; as a consequence, the optimal control problems has been simplified as a sequential optimization rolling with the time;
- At each time step, the system model is used to predict the system states in the given finite time horizon;

- Optimization is conducted at each time step; for the control variable obtained in the finite time horizon, the first one corresponding to the first time step is actually applied to the system for feedback control.

2.2 Modeling

The CRM algorithm uses a simplified version of *Optimal Control*, called *Model Predictive Control* based a Cell Transmission Model which is linear with the assumption that average traffic speed for each freeway section are measured. This assumption is reasonable since the 2070 controller running URMS as reasonably good speed estimation from dual loop traffic detector stations in the field. This section introduces the model actually used for CRM control design in this project.

2.2.1 Nomenclature

<u>*Model Parameters*</u>

$m -$ link index; $M -$ Critical VSL Control link index; *M+1* discharge link index;

$k -$ time index

$L_m -$ length of link *m*

$N_p -$ prediction steps for each *k* in Model Predictive Control *State and Control Variables*

$q_m(k)$ - estimated mainline flow at time *k*

$\rho_m(k) -$ density of link *m* at time *k*

$r_m(k) -$ metering flow rate (veh/hr), control variable

<u>*Measured or Estimated Traffic State Parameters*</u>

$\overline{q}_m(k-1) -$ flow at time *k-1,* measured

$\overline{v}_m(k) -$ *time mean speed* at fixed sensor location within link *m* at time *k,* measured

$u_m(k) -$ *distance ean speed* of the link *m*, estimated

$\overline{\rho}_{M+1} -$ discharge link density, measured/estimated

$s_m(k) -$ total exit ramp flow of a link (veh/hr), measured

d_m − demand from entrance ramp m, measured or estimated

Q_m − mainline capacity of link m, known

Q_b − bottleneck capacity flow, known

$Q_{m,o}$ − entrance ramp m capacity, known

$L_{m,o}$ − entrance ramp m length, known;

V_f − free-flow speed, known

O_c − critical occupancy, known

ρ_c − critical density, known

Here, each link is considered as one cell for simplicity. It is assumed that each link has exactly one on-ramp but may contain more than one exit ramp.

The first equation in (Eq. 2.1) is the conservation of flow. It is linear since the speed variables $u_{m-1}(k)$ and $u_m(k)$ can be estimated from the sensor detection in the field. Such linearization and decoupling bring great advantages to control design.

2.2.2 Dynamical Model of The System

The following linearized density and entrance ramp queue dynamics model are adopted:

$$\begin{aligned} \rho_m(k+1) &= \rho_m(k) + \frac{T}{L_m\lambda_m}\left(\lambda_m\rho_{m-1}(k)u_{m-1}(k) - \lambda_m\rho_m(k)u_m(k) + r_m(k) - s_m(k)\right) \\ w_m(k+1) &= w_m(k) + T\cdot\left[d_m(k) - q_{m,o}(k)\right] \end{aligned} \quad \text{(Eq. 2.1)}$$

2.2.3 Constraints

The following constraints (Eq. 2.2) are adopted for CRM design.

$$\begin{aligned} &0 \le w_m(k) \le L_m^{(r)}\cdot\rho_J \\ &0 \le r_m(k) \le \min\left\{d_m(k), Q_{m,o}, \lambda_m\left(Q_m - \bar{q}_{m-1}(k)\right), \lambda_m u_m(k)\cdot\left(\rho_J - \bar{\rho}_m(k)\right)\right\} \\ &0 \le \rho_m(k) \le \min\left\{\rho_J, \varphi\left(u_m(k)\right)\right\} \end{aligned} \quad \text{(Eq. 2.2)}$$

The first inequality constraint in (Eq. 2.2) is the entrance ramp queue length limit; the second one is the direct constraints on RM rate, which is the minimum of the four terms in the braces: the entrance ramp demand, entrance ramp capacity; the last two terms are space available in the mainline. $\lambda_m \left(Q_m - \overline{q}_{m-1}(k) \right)$ is likely assumed in free-flow case, and $\lambda_m u_m(k) \cdot \left(\rho_J - \overline{\rho}_m(k) \right)$ is likely assumed in congestion. The third one is an indirect constraint on RM rate through the density dynamics. $\varphi\left(u_m(k) \right)$ is the curve of a specified traffic speed drop probability contour as indicated in Figure 2-1, with three flow contours for reference. For a given acceptable traffic drop probability, the contour gives an upper bound for the feasibility region.

In MPC design, at time step k, RM rate is to be determined over the predicted time horizon $k+1,...,k+N_p$:

$$r = \left[r_1(k+1),...,r_1(k+N_p),...,r_M(k+1),...,r_M(k+N_p) \right]^T \quad \text{(Eq. 2.3)}$$

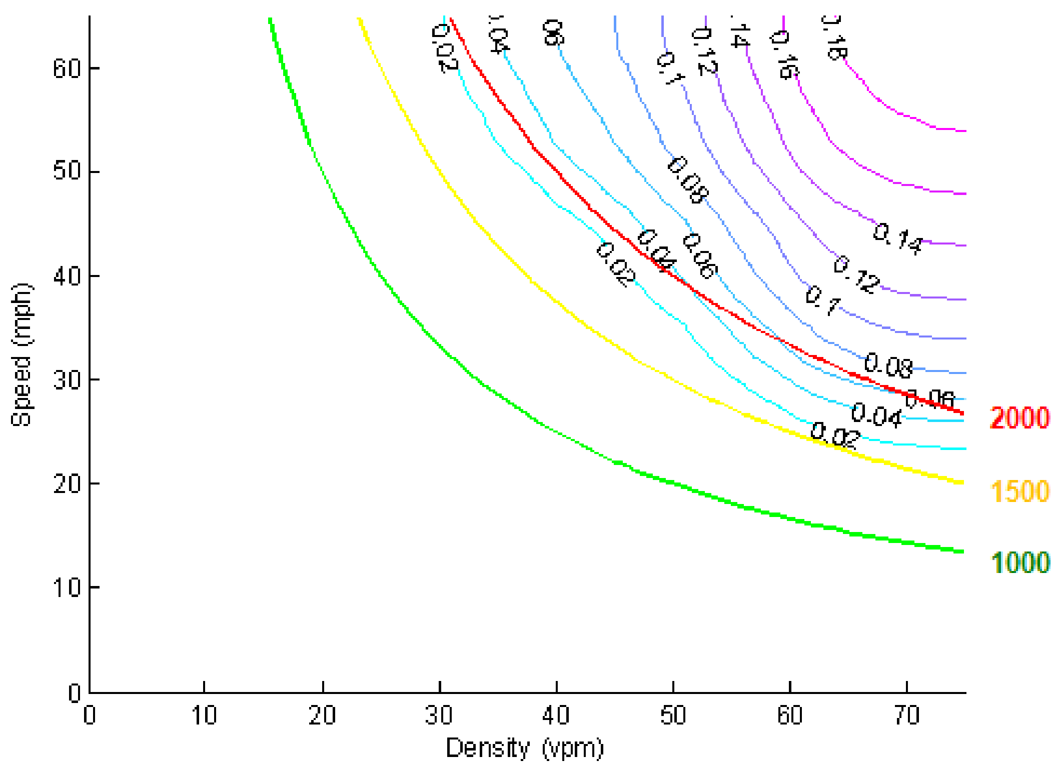

Figure 2-1. Empirical traffic speed drop probability contour vs. flow contour

2.2.4 Objective Function

The following objective function is used at time step *k* over the predictive time horizon:

$$
\begin{aligned}
&J = TTS - TTD \\
&TTS = T\sum_{j=1}^{N_p}\sum_{m=1}^{M} L_m \lambda_m \rho_m\left(k+j\right) \qquad \text{(TTT)} \\
&+\alpha_w T\sum_{j=1}^{N_p}\sum_{o} w_o\left(k+j\right) \quad \text{(Time Delay Due to Onramp Queue)} \qquad \text{(Eq. 2.4)} \\
&TTD = \alpha_{TTD,0} T\sum_{j=1}^{N_p}\sum_{m=1}^{M-1} \lambda_m L_m q_m\left(k+j\right) + \alpha_{TTD,M} T\sum_{j=1}^{N_p} \lambda_M L_M q_M\left(k+j\right) \\
&\alpha_{TTD,M} >> \alpha_{TTD,0} > 0
\end{aligned}
$$

Minimizing *J* is equivalent to minimize TTS (or density), and maximize VMT (to maximize mainline flow). Choosing $\alpha_{TTD,M} >> \alpha_{TTD,0}$ emphasizes maximizing the flow on link *M*.

The reasons for choosing this objective function are as follows: in practice, TTS (TTT) is related to VHT and TTD is related to VMT. Minimizing TTS may discourage vehicles get into the freeway so that the mainline could have better flow when the mainline density is higher. To minimize negative TTD is equivalent to maximize TTD which is to encourage vehicle get into the freeway. Therefore, to minimize the difference of the two is somehow intended to formulate the problem as a non-zero sum game. The overall effect of minimizing this objective function J leads to minimize VHT and maximize VMT. It is important to note that the units of the two system performance parameters are different. To put them in the same objective function, the coefficient choice need to be appropriate.

2.2.5 Algorithm Modification by Queue Override

Beside the systematic consideration in optimization process with entrance ramp queue length taken into account, the entrance ramp queue has been further taken into consideration for onramps with very high demands. If the queue reaches 85% of the entrance ramp, then the meter will be green for at least 10[s] which is to make sure the queue has been adequately flushed.

2.3 Implementation of the CRM Algorithm

2.3.1. Section division, Sensor Locations, Number of Lanes

Section division: to use the linearized CTM for CRM algorithm development, it is necessary to divide the road network into sections. Since all the onramps are metered for the system concerned, the road network is divided into sections according to sections according to the sensor locations: in general, section boundaries are at the mid-point between the entrance ramp merge point and the its immediate upstream sensor location. With this division principle, the overall system has the following components:

- 12 sections: corresponding to 11 onramps (not all section contains a onramp)
- 11 entrance onramps
- 10 exit ramps

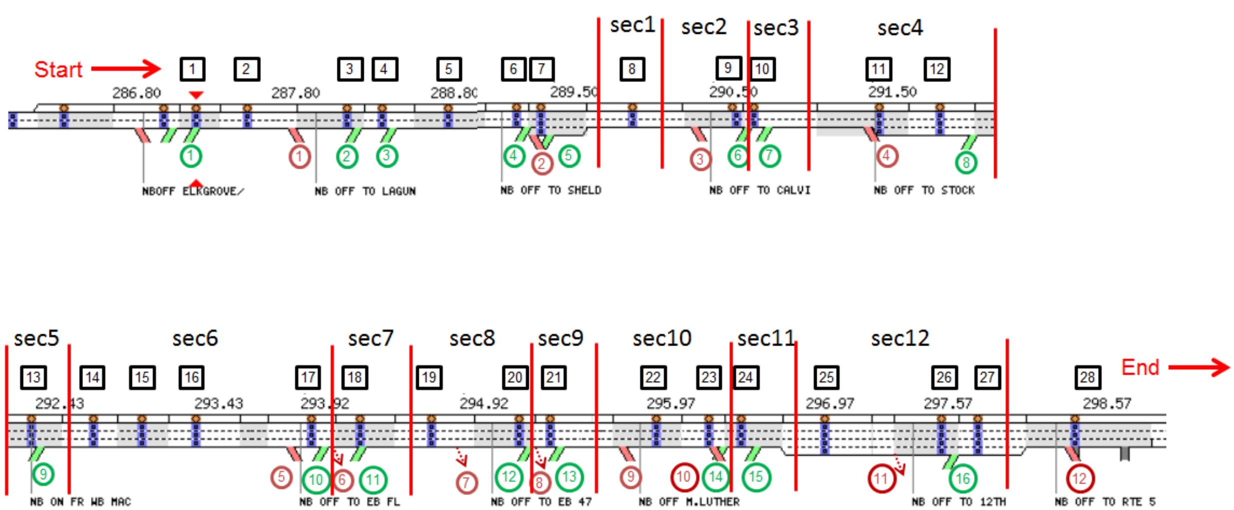

Figure 2-2. The section division configuration and VDS location of the SR99 N test site

The section division configuration of the SR99 N test site is illustrated in Figure 2-2, where numbered black boxes are mainline VDS from upstream to downstream, numbered green circles are onramp VDS from upstream to downstream, and numbered red circles are offramp VDS from upstream to downstream. Since the first 5 onramp are not controlled by the CRM algorithm, they are excluded from the section division.

It is noted that this does not mean that all the onramps are to be coordinated. Instead, only a subset of entrance ramp meters is coordinated. The main points for the selection of RM for coordination include:

- Demand is high enough so that its flow into the system would significantly affect the overall traffic and its queue would affect overall system VHT;
- To be coordinated onramps are located close enough: if two groups of onramps are far separated and their traffic rarely affects each other, it does not make sense to coordinate them; instead, it would be simpler to just operate them separately;

The following is a list of onramps (11 in total) from upstream to downstream that the project preliminarily selected for coordination:

- Calvine EB, WB (the blue circle 6 and 7 in Figure 2-2)
- Mack Road EB, WB (the blue circle 8 and 9 in Figure 2-2)
- Florine EB, WB (the blue circle 10 and 11 in Figure 2-2)
- 47th Ave EB, WB (the blue circle 12 and 13 in Figure 2-2)
- Fruitridge EB, WB (the blue circle 14 and 15 in Figure 2-2)
- 12th Ave (the blue circle 16 in Figure 2-2)

Number of Lanes: With the section division above in mind, an immediate question is how to determine the number of lanes since it is a model parameter in (Eq. 2-1 ~ Eq. 2-4). The reason is that the number of lanes in each section may not be homogeneous. To resolve this problem, we used the distance-based weighted number of lanes for each section. This is done as follows. Assuming that a section with length L_m is divided into two subsections: the first has $\lambda_{m,1}$ lanes with lengths $L_{m,1}$; and the second has $\lambda_{m,2}$ lanes with length $L_{m,2}$ $(L_m = L_{m,1} + L_{m,2})$. Now a *composite number of lanes* λ_m is determined as follows:

$$\lambda_m = \frac{L_{m,1}\lambda_{m,1} + L_{m,2}\lambda_{m,2}}{L_m} \quad \text{(Eq. 2.5)}$$

It is noted that: (a) a composite number of lanes for a section could be a decimal; (b) such a number is inconsistent with density estimation across the section; (c) this method could be applied to a section with more than two subsections with different number of lanes.

Sensor Locations:

As shown in Figure 2-2, the locations of sensors used for RM are immediately upstream of the entrance ramp. The simulation model created sensors at similar locations. There are two ways to create sensors in Aimsun microsimulation: either lane-by-lane or one sensor (such as loop detector) across all lanes. For model calibration above, lane-by-lane sensors are used since it is necessary to distinguish between GP lanes and the HOV lane. After model calibration, RM does not need to distinguish flows between lanes. Therefore, cross-lane single sensors are used for convenience.

2.3.2 Traffic State Parameters

In Eq. 2-1 ~ Eq. 2-4, there are three traffic state parameters: density, speed and entrance ramp queue length. Since the problem here is for RM only with speed control, we can use sensor measured speed [5] to replace the unknown with known values. Strictly speaking, the speed $u_m(k)$ at time step *k* is a distance mean speed, while a sensor can only measure at a point to get time mean speed. For this reason, it is necessary to convert time mean speed at a point into a distance mean speed with the *harmonization mean* as follows:

$$u_m(k) = \frac{1}{\frac{1}{m}\sum_{i=1}^{m}\frac{1}{\overline{v}_{m,i}(t_i)}} \qquad \text{(Eq. 2.6)}$$

where $\overline{v}_{m,i}(t_i)$ is the measured speed at the point sensor during time interval k, and all the time points $\{t_0, t_1, ..., t_m\}$ fall into this time interval. Clearly, to get proper distance mean speed, the sampling rate at the fixed detector should be much higher. However, in practice, one can just use time mean speed to replace the distance mean speed for operation.

- ***2.3.3 Lane-wise Metering***

In Aimsun microsimulation, an entrance ramp with multiple lanes has to be set with a single metering rate which controls all the lanes, essentially, with flow control of all lanes together. However, this is different from what is in the field for California highways, where each lane of a metered entrance ramp has an individual meter including the HOV lane. Besides, the green time intervals of different lanes are shifted to avoid time-space conflicts of vehicles from different lanes at the merge after the meter. It is clear that this is more efficient for vehicles entering the freeway with a lane merge after metering. To resolve this problem, we used the following techniques. The lanes upstream of the meter have been divided into independent roads with one lane each. In this way, each road can be metered individually. The demand for GPL of an entrance ramp has been randomly distributed between the GPL and that for the HOV lane still kept as it should be. Then the total flow of all the lanes is used in the optimization process to determine the RM rate. After the optimization process, the desired total flow (metering rate) is obtained for each entrance ramp. Such desired total flow is then split between lanes according to the percentage of measured flow with respect to the total measured flow at the entrance ramp upstream. It is noted that such a process is necessary to simulation development but not necessary for field implementation since metering in the field is automatically split between lanes and activated individually.

2.3.4 Parameter Section in Modeling

The model in (Eq. 2.1-2.4) has several parameters that need to be determined. Those values are listed in the following Table 2-1.

Table 2-1 Model Parameter Selection for Simulation

Parameters	ρ_J	$\alpha_{TTD,M}$ $\alpha_{TTD,0}$	α_w	T
Values	200 [Veh/Ln]	2.0	6.5	30 s

2.3.5 Field Default Ramp Metering

The field default RM strategy in current operation is occupancy-based *Local Adaptive Ramp Metering (LARM)*. The RM plan was obtained from Caltrans District 3 freeway traffic engineers. As an example, Table 2-2 shows the field Local Adaptive RM strategy actually in operation for morning hours at the onramps of WB Mack Road and EB Florin Road. For each location, the third column is the metering rate and the fourth column is the occupancy threshold which is directly measured by the loop detector in the mainline immediately upstream of the entrance ramp. Similar strategy for sensor locations and ramp metering rate are implemented in microscopic simulation as the default case.

2.3.6 Practical Control Strategy in Simulation

For constructing a complete test site, the whole test segment with 16 onramps had been built in Aimsun microsimulation before field implementation. Although the network built for Aimsun microsimulation includes 16 onramps, the upstream 5 onramps (Elk Grove Blvd, EB Laguna Blvd, WB Laguna Blvd, EB Sheldon Rd, and WB Sheldon Rd) still use the field default LRRM control. Only the downstream 11 onramps (Calvine EB and WB, Mack Road EB and WB, Florin EB and WB, 47th Ave EB and WB, Fruitridge EB and WB, 12th Ave) are coordinated with the Optimal CRM strategy presented above. This mixed control strategy of Local Adaptive RM and the proposed Optimal CRM strategy had been implemented in Aimsum microsimulation as well, which was agree with the control strategy of field implementation. All of the entrance onramp ID are listed in Table 1-1.

For entrance ramp HOV lanes, the RM rate always use the maximum lane rate at 950 [veh/hr], which applies to both control strategies: LRRM and Optimal CRM. The LRRM rates for the 5 upstream onramps were obtained from Caltrans District 3 RTMC. Therefore, they were in agreement with what was in operation in the field. The LRRP and CRM activation all use One- Car-Per-Green strategy in the field as they were before. The project team did not change the activation strategy.

Table 2-2 Field Operational Local Responsive RM (LRRM) Strategy of WB Mack Road and EB Florin Road in AM hours

WB Mack Road

	Level	Meter Rate (VPH)	Thresholds		
			Occ	MLFlow (VPH)	
	1	900	5.0	850	0
	2	853	6.1	925	0
	3	806	7.1	1000	0
	4	759	8.2	1075	0
	5	712	9.3	1150	0
	6	665	10.4	1225	0
	7	618	11.4	1300	0
AM	8	570	12.5	1375	0
	9	523	13.6	1450	0
	10	476	14.6	1525	0
	11	429	15.7	1600	0
	12	382	16.8	1675	0
	13	335	17.9	1750	0
	14	288	18.9	1825	0
	15	240	20.0	1900	0

EB Florin Road

	Level	Meter Rate (VPH)	Thresholds		
			Occ	MLFlow (VPH)	
	1	1000	6.0	1300	0
	2	966	6.3	1346	0
	3	932	6.5	1392	0
	4	898	6.8	1439	0
	5	863	7.0	1485	0
	6	829	7.3	1532	0
	7	795	7.5	1578	0
AM	8	760	7.8	1625	0
	9	726	8.0	1671	0
	10	692	8.3	1717	0
	11	658	8.5	1764	0
	12	623	8.8	1810	0
	13	589	9.0	1857	0
	14	555	9.3	1903	0
	15	520	9.5	1950	0

2.3.7 Entrance Ramp Queue Overwrite

The following entrance ramp queue overwrite scheme has been used jointly with the Optimal CRM algorithm. The queue detector is located about 15% distance to the upstream end of the entrance ramp. The schematic overwrite algorithms is as follows:

- If the occupancy of the queue detector is over 70%, then use the maximum lane RM rate 950 [veh/hr] for 3 cycles (or 1.5 minutes)
- If the occupancy of the queue detector continues to be higher than 70%, then this maximum lane RM will remain.
- Using queue overwrite release rates 900vph for one car per green and 1100 vph or lower for two cars per green.

It has been observed from simulation that this strategy can effectively reduce the queue end to the downstream of the queue detector.

Besides, to avoid queue spills back to arterial at 12th Ave onramp, the minimum ramp metering rate used is the same as the original LRRM. Also, the actual CRM rate is very similar to what previous implemented for LRRM at this location,

2.3.8. Onramp Demand and Off-Ramp Flow

The input to the freeway corridor is the flow at the most upstream mainline and from the onramps. Since the onramp data and off-ramp flow were not available at the time of field test, the project team used PeMS 5 min historical data of week in September 2017 as the prediction. This approximation is not accurate but reasonable since the AM peak hour demand of that stretch were mainly commuters. Test result indicated that this approach was reasonable.

2.4 Conclusion

This section has documented the proposed coordinated ramp metering (CRM) algorithm and its implementation, which is an optimization control approach based on a linearized cell transmission model (CTM) with on-ramp queue dynamic model. Both the CTM and the queue model are formulated based on the conservation of vehicles principle. The control objective is to minimize total VHT and to maximize the total VMT, and therefore, we consider a weighted combination of TTS and TTD as the objective. In addition, the constraints of the system are also modeled. These constraints are mainline capacity, on-ramp capacity, and the minimum and maximum ramp metering rate. With the linear system model, linear constraints, and linear objective function, the CRM algorithm becomes a linear programming (LP) problem at each time step and can be solved very fast in real-time by the well-known *Simplex Method*.

In order to prevent the on-ramp queue spill back to local street, an entrance ramp queue override scheme is also implemented in this project as an auxiliary control strategy of the proposed CRM algorithm. The last onramp meter at 12th Ave should not be too restrictive to avoid traffic spilling back to the arterial. The CRM rate was adjusted to be close to the LRRM rate at this location.

Chapter 3. Real-time Traffic Data Preparation

Traffic state parameter estimation is very critical to the implementation of the CRM algorithm since it represents the current status of the freeway corridor traffic situation. Correct control would need a correct estimation of traffic state. However, since the raw traffic data from the loop detectors in the field have flaws [5] which may be caused by many factors: uncertainties of traffic, loop detector characteristics (such as sensitivity level setting, connection between detector card and in-lane circuit, vehicle types, etc.), and data passing process through network connecting the 2070 controllers in the field and Caltrans District 3 RTMC. Therefore, a robust method for obtaining reasonably good traffic state parameters from the noisy and/or even faulty data is necessary. This section describes how the field data were processed for this purpose.

3.1 Test Site and Traffic Situation

The objective of this project is to conduct limited field testing of a newly developed control algorithm for Coordinated Ramp Metering in the first stage. Therefore, test site selection criteria [2] are proposed mainly based on the characteristics and infrastructure requirements of CRM. Since Changeable Message Signs (CMS) could be added for VSL testing to any site with proper type of bottleneck, site selection criteria also include the factors related to VSL, mainly, the bottleneck types.

Traffic of SR99 NB between Elk Grove in Elk Grove city and SR50 interchange near 12^{th} Ave in Sacramento city has been analyzed. This corridor has recurrent bottlenecks in AM peak hours due to high flow of commuters to the city of Sacramento for work in the morning.

This report focuses on SR99 NB between PM 285~305 (between Stockton Blvd and SR50-Interchange downstream of 12^{th} Ave) as shown in Figure 3-1. Loop detector has some improvement. Besides, entrance ramp and exit ramp data are available from PeMS now, which will be very useful for system modeling and simulation. However, some sensors speed estimations are still not available, but flow data of some lanes are available, which are important for system analysis and RM control.

3.1.1 Road Geometry and Sensor Location:

The overall road map of the section in consideration is shown in Figure 3-1, and the lane/entrance ramp/exit ramp geometry and sensor locations are shown in Figure 3-2.

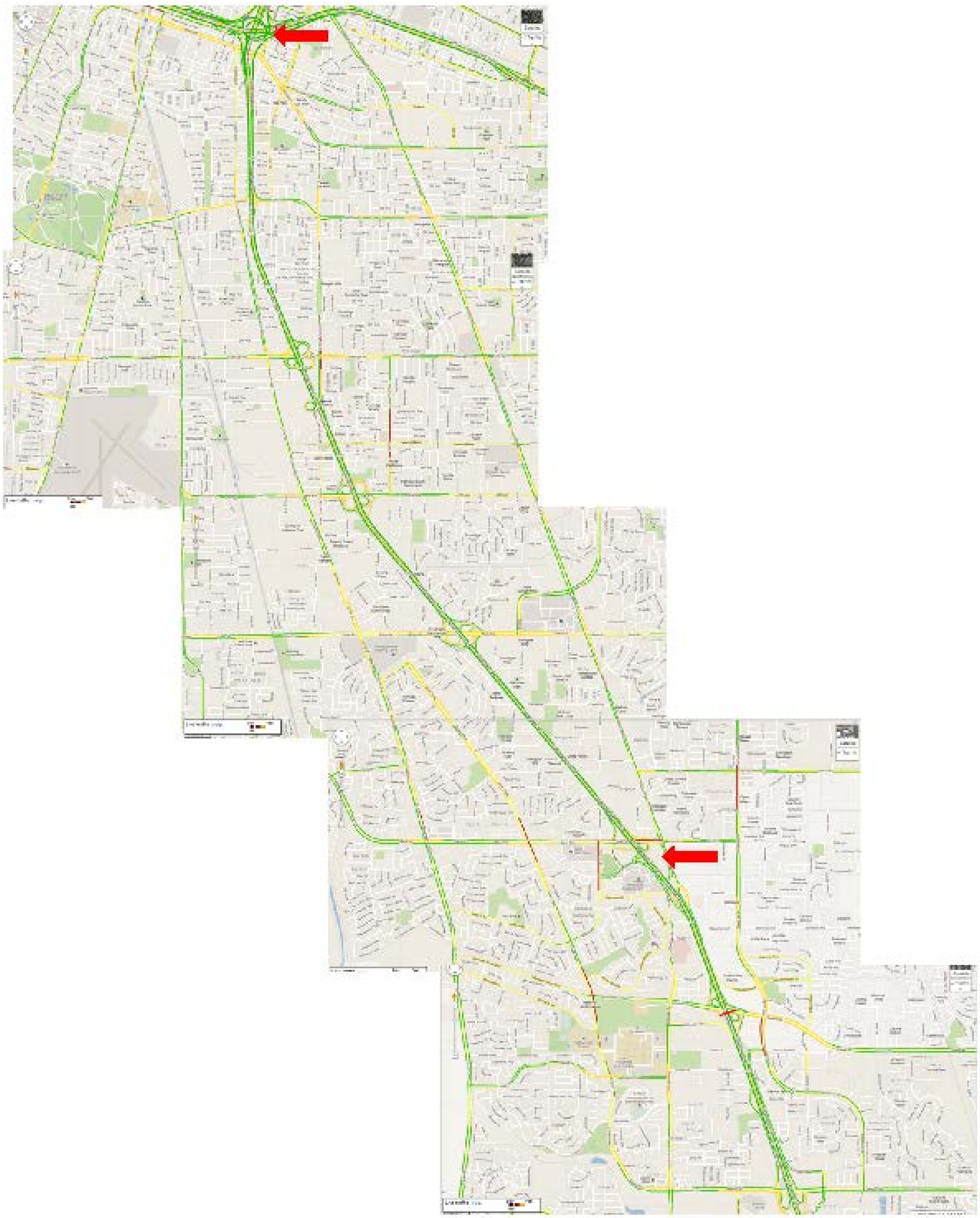

Figure 3-1. Road map of SR99 between 12th Ave and SR50 interchange

In Figure 3-1, the potential candidate bottleneck and the downstream bottleneck are indicated with red arrows. In Figure 3-2, only the candidate bottleneck is indicated with a red spot, which is near the 47th Ave entrance ramp.

In addition to mainline sensors, entrance ramp flows are also available now from PeMS, which were not there before. This will benefit traffic analysis and RM control. Some exit ramp flow is also available. Historical entrance ramp and exit ramp data are available back to May 2010.

3.1.2 Bottleneck Location Observation from Macroscopic Contour Plot

Macroscopic contour plot of the traffic data on 10/19/12 is shown in Figure 3-3, from which, it can be observed that there are two bottlenecks in the range of PM 286 ~ PM 299, which are very close. If downstream traffic is very heavy, they could be combined as one.

This is consistent for 2012 and 2013 data. Therefore, the coordination should include the whole section. The overall system should be controlled through TMC to reduce interface with individual and communication between onramps.

3.1.3 More Detail Traffic Analysis Using VDS Raw Data

Identified Major Bottlenecks (all activates in AM traffic) [2]:

(1) PM 298.5: downstream congestion caused by diverging traffic to US 50 EB and WB. This one may back-propagate to upstream bottleneck at PM 296.54.

(2) PM 296.54: middle congestion caused by merging traffic from Fruitridge Rd (EB and WB). Two on-ramps (one from EB and one from WB) are close. The merging lane doesn't drop (until the split of SR 99 and S Sacramento Freeway; i.e., there's a lane addition. Thus, congestion is light at this location, but it becomes more severe as it propagates upstream passing on-ramps from 47th Ave (at PM 295.7).

(3) PM 290.76: upstream congestion caused by merging traffic from Calvine Rd. (EB and WB). The congestion is light. It starts earlier and may merge with congestion from downstream bottleneck congestion back-propagation. This will need further investigation. Its road geometry may be interesting: the entrance ramp leads to an added lane extended to the exit to E. Stockton Blvd.

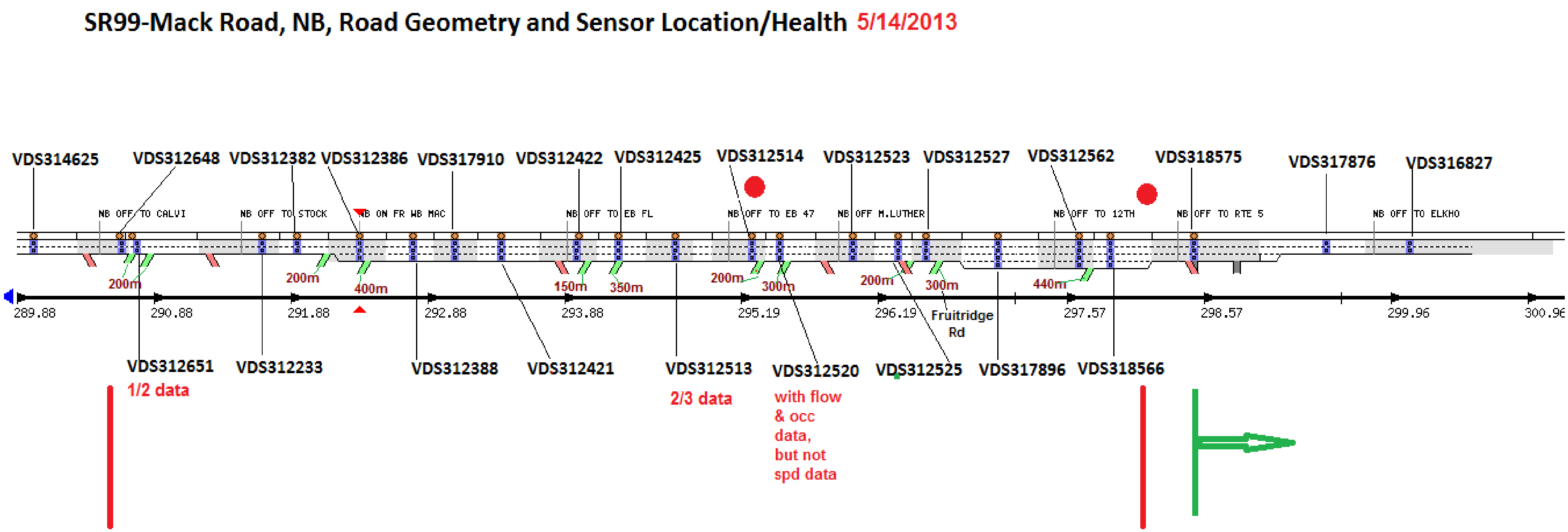

Figure 3-2. Postmile (PM), lane geometry, entrance ramp/exit ramp info, and sensor locations and health

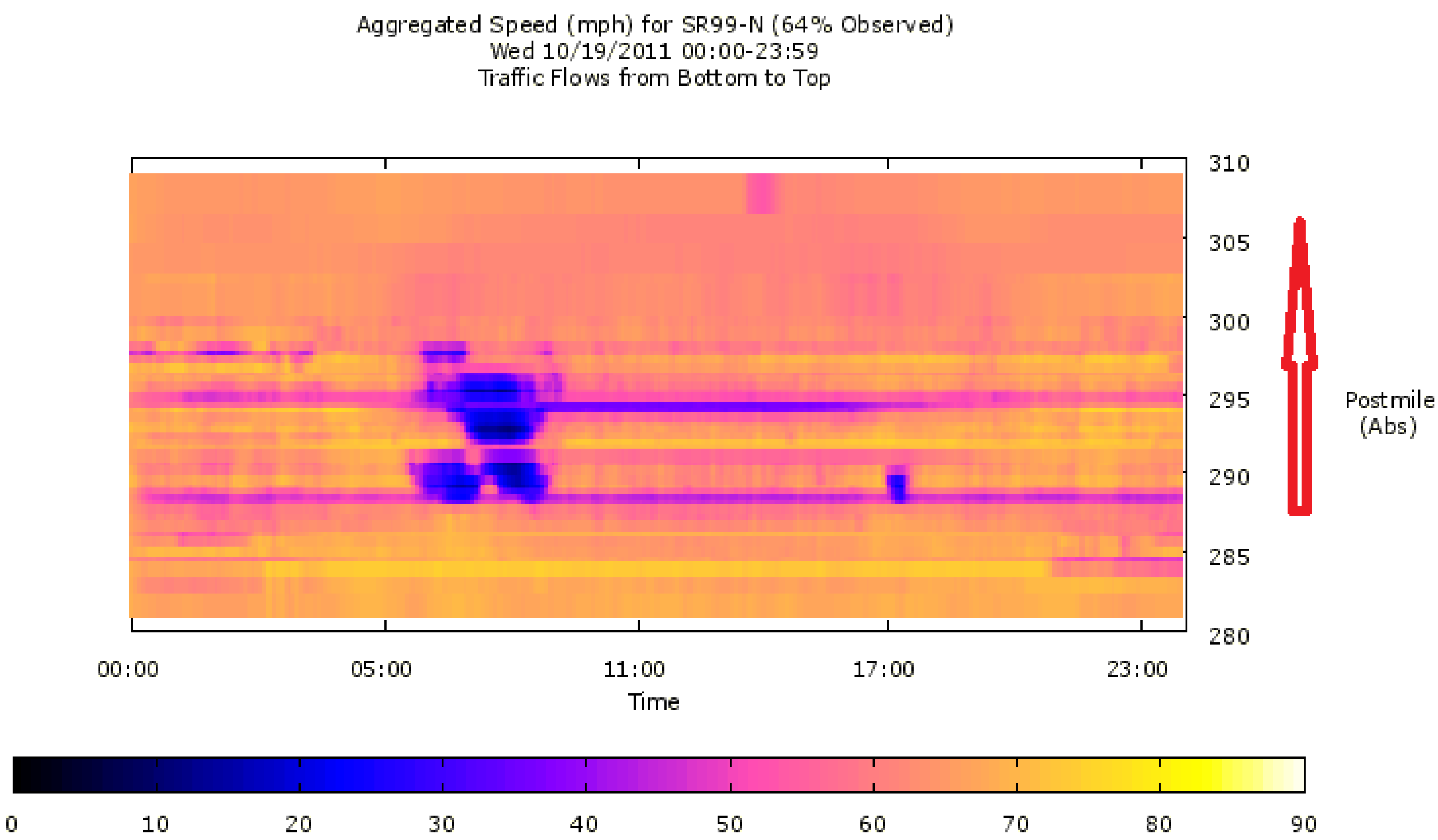

Figure 3-3. SR99 NB AM peak recurrent bottleneck location on and affected range, time interval, and intensity

3.2 Raw Field Data

The real-time traffic data is obtained from the URMS 2070 controllers that installed on SR99 Northbound in 30 second or shorter sampling time. There are 28 controllers in the field, which contains 28 mainline vehicle detector stations (VDS), 16 on-ramps VDS and 12 off-ramp VDS. A VDS contains several loop detectors and the number of loop detectors depends on the number of lanes in the location of VDS. PeMS provide the VDS configuration of the test site, which is illustrated in Figure 3-4.

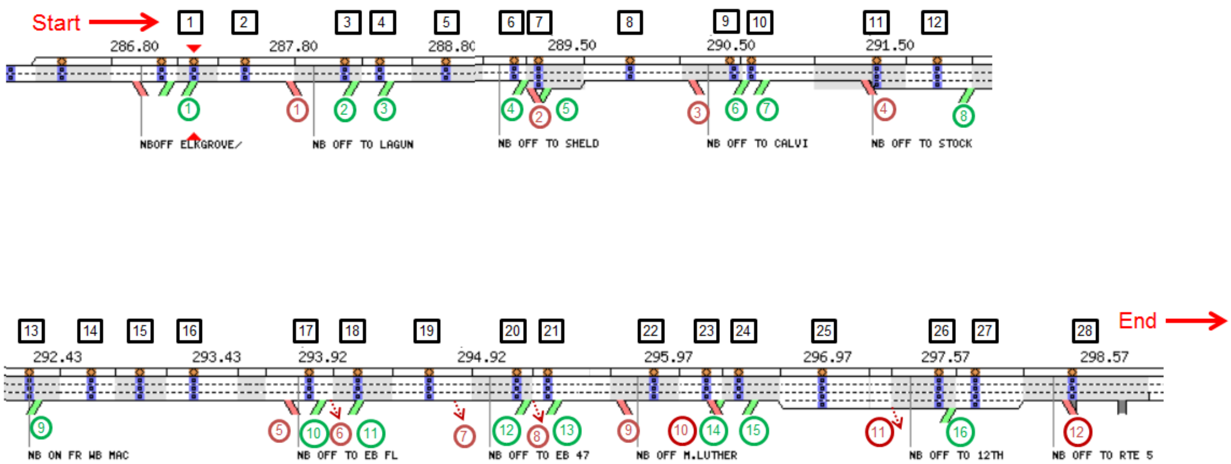

Figure 3-4. VDS configuration of SR99 Northbound test site

The data collected from the field mainline, on-ramp and off-ramp contains the following:

- Mainline GP lane data lane by lane: flow, occupancy, density and speed
- Mainline HOV lane data: flow, occupancy, density and speed
- On-ramp GP lane data lane by lane
    - Passage detector: flow, occupancy
    - Demand detector: flow and occupancy
    - Queue detector: flow and occupancy
- On-ramp HOV by pass lane data: flow and occupancy
- Off-ramp data lane by lane: flow and occupancy

3.3 Potential Data Problems

These data is used for ramp metering control, monitor the freeway traffic state and evaluate control performance. However, there are several possible detector failure modes when collecting raw data. The possible detector failure modes are

(1). Data missing:

The individual detector is not reporting any data and consecutive zeros are collected when pulling data from the 2070 controllers. One possible case is that in the same VDS, some loop detectors are sending valid nonzero data, but others are sending consecutive zeros.

(2). Invalid data:

The data is nonzero but is value is abnormal. For example, the data contains outliers if the measurement has a sudden deviation from a normal value (the data is deviate too far from the sample mean).

(3). Disconnection with 2070 controllers:

The communication between PATH computer and 2070 controllers could suddenly disconnect with each other. Possible reasons are that the controller in the field is down or the firmware version of controller in the field is not compatible with PATH computer.

Therefore, it is necessary to build an algorithm to clean raw data in real-time such that traffic state parameters becomes more reliable for controller.

3.4 Data Cleansing Procedures

The data cleansing procedures contains four steps

(a) Data aggregation over lanes

In this step, the loop detectors of a VDS are aggregated into one.

The flow aggregation over lanes are computed by

$$f_i(k) = \sum_{j=1}^{n_i} f_{i,j}(k) \tag{Eq. 3.1}$$

where $f_{i,j}(k)$ is the flow measurement at VDS i of its loop detector j at time k, $f_i(k)$ is the aggregated flow at VDS i, and n_i is the number of loop detectors in VDS i (usually n_i equals to the number of lanes in the section where VDS i is installed).

The speed aggregation over lanes are computed by harmonic speed, that is

$$v_i(k) = n_i \left(\sum_{j=1}^{n_i} \frac{1}{v_{i,j}(k)} \right)^{-1} \quad \text{(Eq. 3.2)}$$

where $v_{i,j}(k)$ is the speed measurement at VDS i of its loop detector j at time k, $v_i(k)$ is the aggregated flow at VDS i, and n_i is the number of loop detectors in VDS i (usually n_i equals to the number of lanes in the section where VDS i is installed).

The vehicle density is derived from flow and speed aggregation

$$\rho_i(k) = \frac{f_i(k)}{v_i(k)} \quad \text{(Eq. 3.3)}$$

The occupancy aggregation is computed by weighted average with speed measurement

$$o_i(k) = \frac{\sum_{j=1}^{n_i} v_{i,j}(k) o_{i,j}(k)}{\sum_{j=1}^{n_i} o_{i,j}(k)} \quad \text{(Eq. 3.4)}$$

where $o_{i,j}(k)$ is the speed measurement at VDS i of its loop detector j at time k, $o_i(k)$ is the aggregated flow at VDS i, and n_i is the number of loop detectors in VDS i (usually n_i equals to the number of lanes in the section where VDS i is installed).

(b) Data aggregation over sections

In this step, the data is aggregated into section data since the control algorithm uses cell transmission model (CTM). In the CTM setting, a freeway is partitioned into several sections and each section contains at most one on-ramp and one off-ramp. In order to implement CTM, each section must contain at least one VDS such that the traffic information in the section can be obtained. Figure 2 illustrates the definition of sections that encoded in the control algorithm.

The flow aggregation over sections are computed by

$$q_n(k) = \frac{1}{c_n} \sum_{i=1}^{c_n} f_i(k) \qquad \text{(Eq. 3.5)}$$

where $q_n(k)$ is the aggregated flow in section n at time k and c_n is the number of VDS in section n.

The speed aggregation over sections are computed by

$$V_n(k) = \frac{1}{c_n} \sum_{i=1}^{c_n} v_i(k) \qquad \text{(Eq. 3.6)}$$

where $V_n(k)$ is the aggregated speed in section n at time k and c_n is the number of VDS in section n.

The density aggregation over sections are computed by

$$K_n(k) = \frac{1}{c_n} \sum_{i=1}^{c_n} \rho_i(k) \qquad \text{(Eq. 3.7)}$$

where $K_n(k)$ is the aggregated density in section n at time k and c_n is the number of VDS in section n.

The occupancy aggregation over sections are computed by

$$\omega_n(k) = \frac{1}{c_n} \sum_{i=1}^{c_n} o_i(k) \qquad \text{(Eq. 3.8)}$$

where $\omega_n(k)$ is the aggregated occupancy in section n at time k and c_n is the number of VDS in section n.

If the traffic information in a section is not available, then the traffic information is that section will be estimated by the average of its upstream and downstream section. The section definition is in Figure 2-2.

(c) Data filtering

In this step the data in each section are filtered by the method of moving average. The advantage of data filtering is that the data become less noisy after filtering. Suppose the length of data window is n_p, then

The filtered flow $q_n(k)$ is

$$\bar{q}_n(k) = \frac{1}{n_p} \sum_{i=0}^{n_p-1} q_n(k-i) \quad \text{(Eq. 3.9)}$$

The filtered speed $V_n(k)$ is

$$\bar{V}_n(k) = \frac{1}{n_p} \sum_{i=0}^{n_p-1} V_n(k-i) \quad \text{(Eq. 3.10)}$$

The filtered density $K_n(k)$ is

$$\bar{K}_n(k) = \frac{1}{n_p} \sum_{i=0}^{n_p-1} K_n(k-i) \quad \text{(Eq. 3.10)}$$

The filtered occupancy $\omega_n(k)$ is

$$\bar{\omega}_n(k) = \frac{1}{n_p} \sum_{i=0}^{n_p-1} \omega_n(k-i) \quad \text{(Eq. 3.11)}$$

3.5 Data Imputation

Up to this stage, most of the mainline data can be obtained in acceptable quality. However, most of on-ramp/off-ramp flow and occupancy data in demand detector are not available. There are two strategies for recovering those missing on-ramp/off-ramp.

(a) Method of flow balance

Suppose an on-ramp flow data is missing or not available, it is possible to estimate the missing on-ramp flow value from its adjacent mainline and off-ramp VDS data by flow balance

$$r_i(k) = f_{i,down}(k) + s_{i,down}(k) - f_{i,up}(k) \quad \text{(Eq. 3.12)}$$

where $r_i(k)$ is the estimated on-ramp flow at location i, $f_{i,down}(k)$ is the mainline flow measurement in the immediate downstream of location i, $f_{i,up}(k)$ is the mainline flow measurement in the immediate upstream of location i, and $s_{i,down}(k)$ is the off-ramp flow measurement in the immediate downstream of location i.

Similarly, suppose an off-ramp flow data is missing or not available, it is possible to estimate the missing off-ramp flow value from its adjacent mainline and on-ramp VDS data by flow balance

$$s_i(k) = f_{i,up}(k) + r_{i,up}(k) - f_{i,down}(k) \qquad \text{(Eq. 3.13)}$$

where $s_i(k)$ is the estimated off-ramp flow at location i, $f_{i,down}(k)$ is the mainline flow measurement in the immediate downstream of location i, $f_{i,up}(k)$ is the mainline flow measurement in the immediate upstream of location i, and $r_{i,up}(k)$ is the on-ramp flow measurement in the immediate upstream of location i.

(b) PeMS historical data

PeMS provide cleaned historical five minutes data for a freeway. Those historical data can also be used as a compensation of missing data. A set of flow data of each on-ramp/off-ramp is coded in the control algorithm (look up table) as an alternative option for compensating the missing on-ramp or off-ramp data.

The data cleansing procedures are illustrated as the flow chart in Figure 3-5.

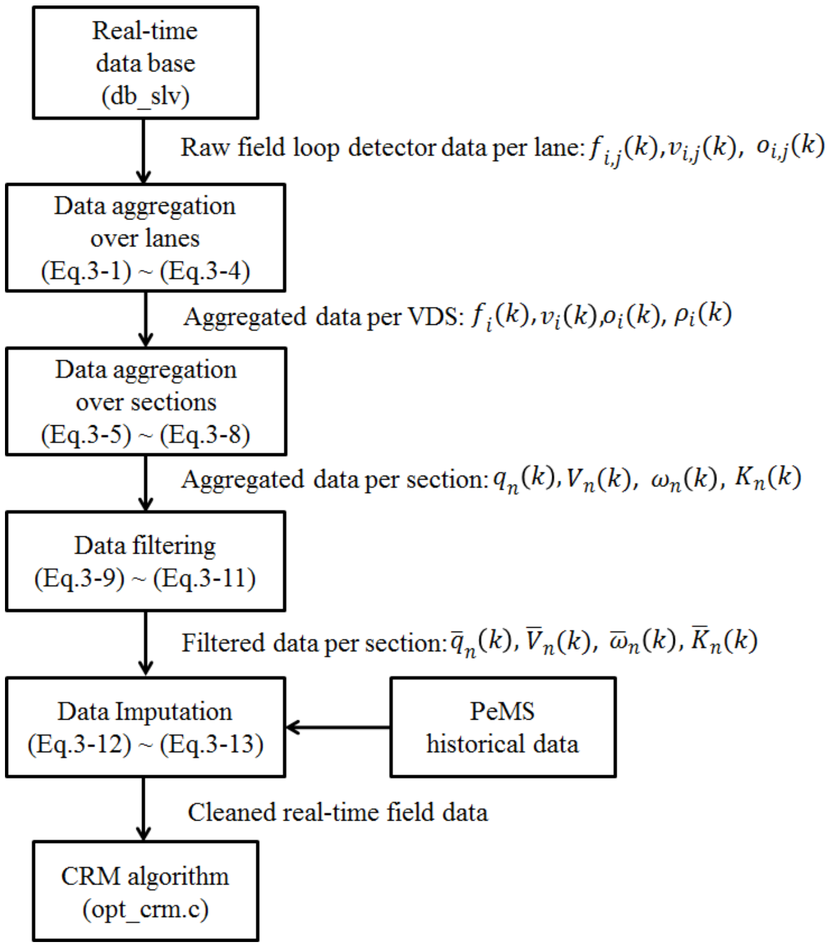

Figure 3-5. Data cleansing procedures as a flow chart

3.6 Data Cleansing Results

Figure 3-6 to Figure 3-14 show the results of data cleansing for flow, speed, density and occupancy. One can observe that the missing data (for example, flow data in VDS 15, 21, and 22 shown in Figure 3-6) at each VDS can be recovered when the VDS data (lane by lane data) are aggregated into section data. In addition, the data noise is reduced after data aggregation.

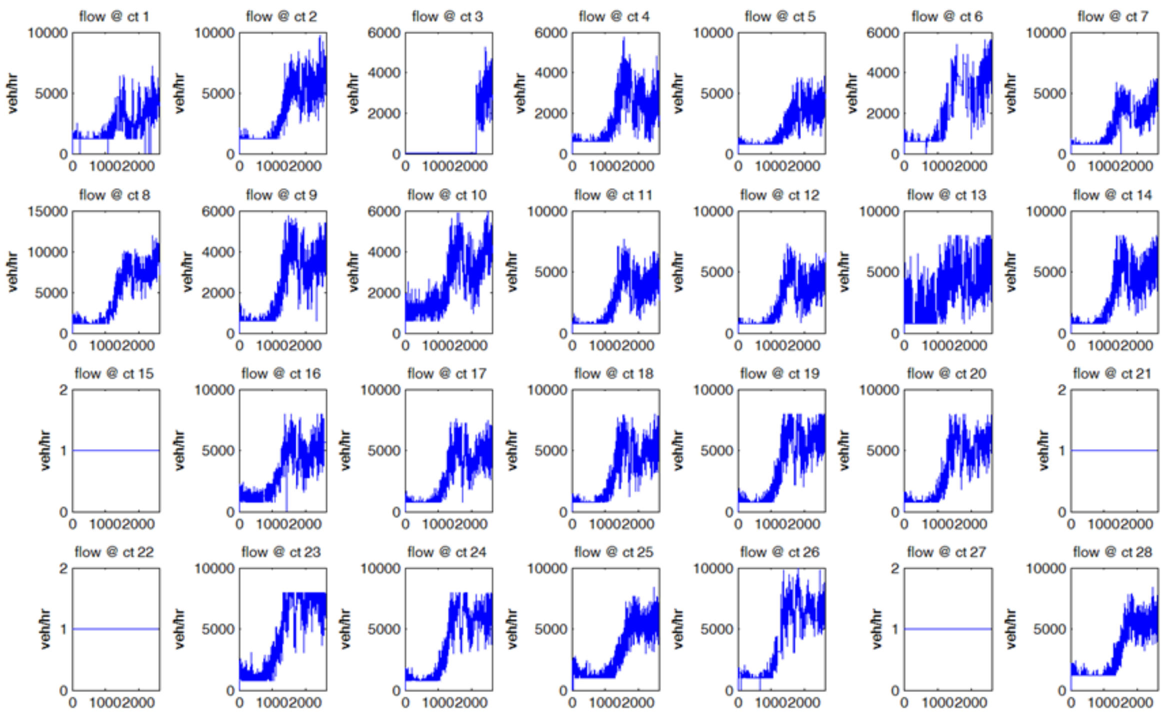

Figure 3-6. Aggregated mainline flow data in each VDS: each subfigure shows lane by lane aggregation of flow data for each VDS as numbered from 1 to 28 in Figure 3-4.

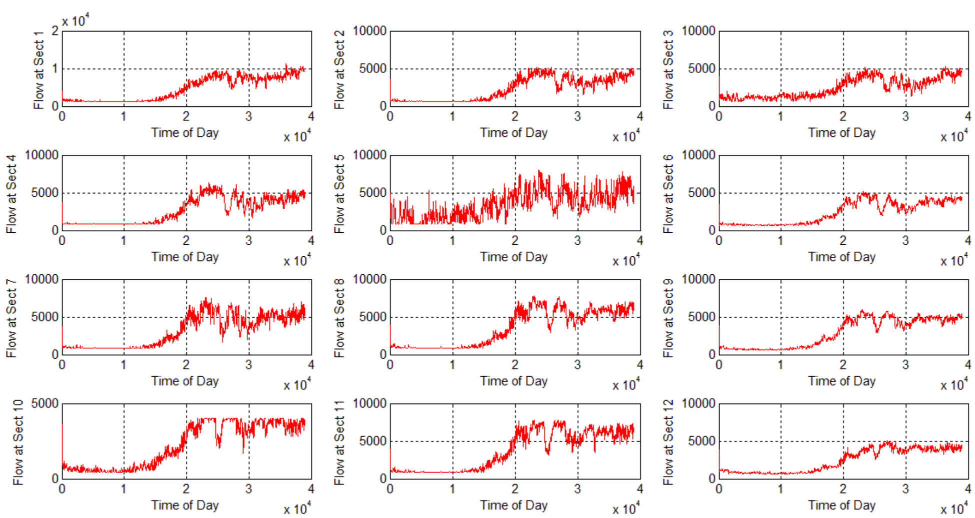

Figure 3-7. Aggregated mainline flow data in each section: each subfigure shows aggregation of flow data for each section as numbered from 1 to 12 in Figure 2-2.

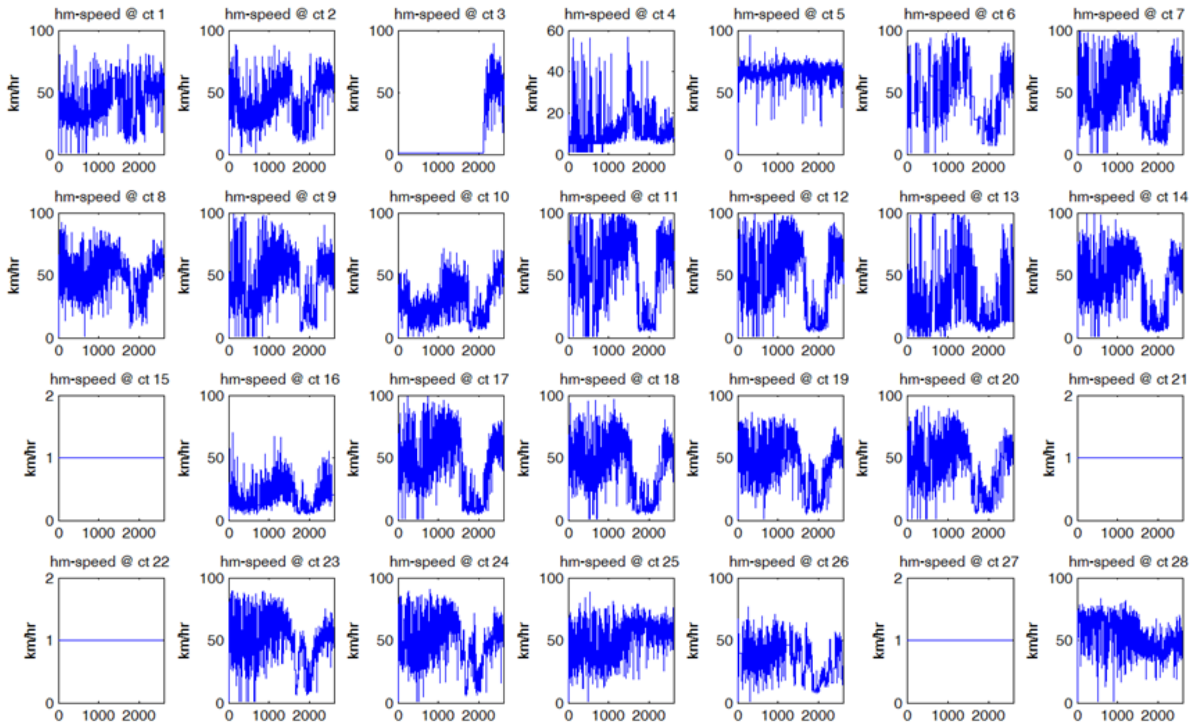

Figure 3-8. Aggregated mainline speed data in each VDS: each subfigure shows lane by lane aggregation of speed data for each VDS as numbered from 1 to 28 in Figure 3-4.

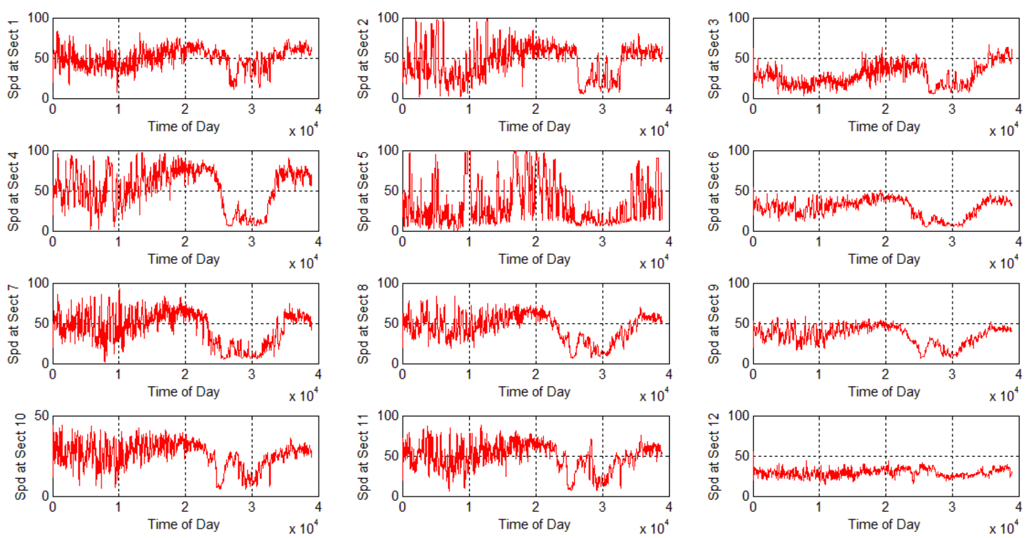

Figure 3-9. Aggregated mainline speed data in each section: each subfigure shows aggregation of speed data for each section as numbered from 1 to 12 in Figure 2-2.

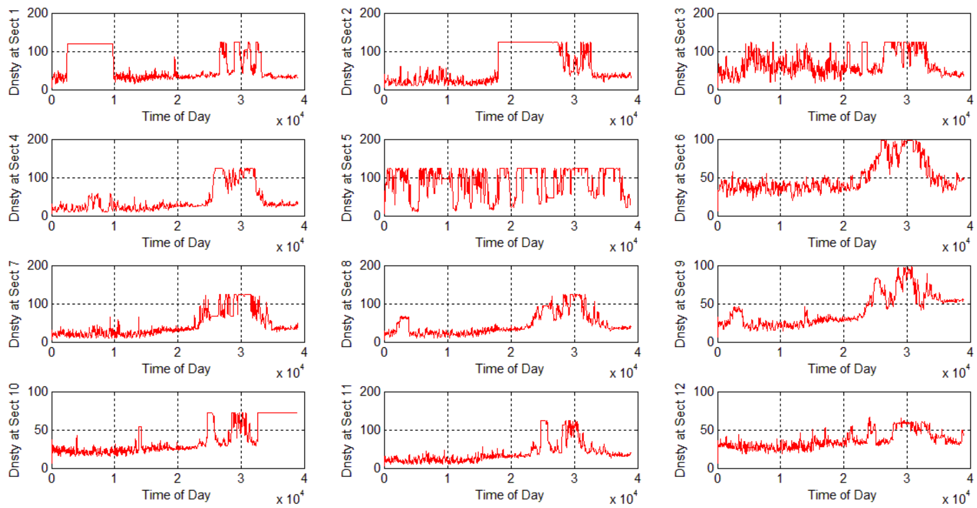

Figure 3-10. Aggregated mainline density data in each section: each subfigure shows aggregation of mainline density data for each section as numbered from 1 to 12 in Figure 2-2.

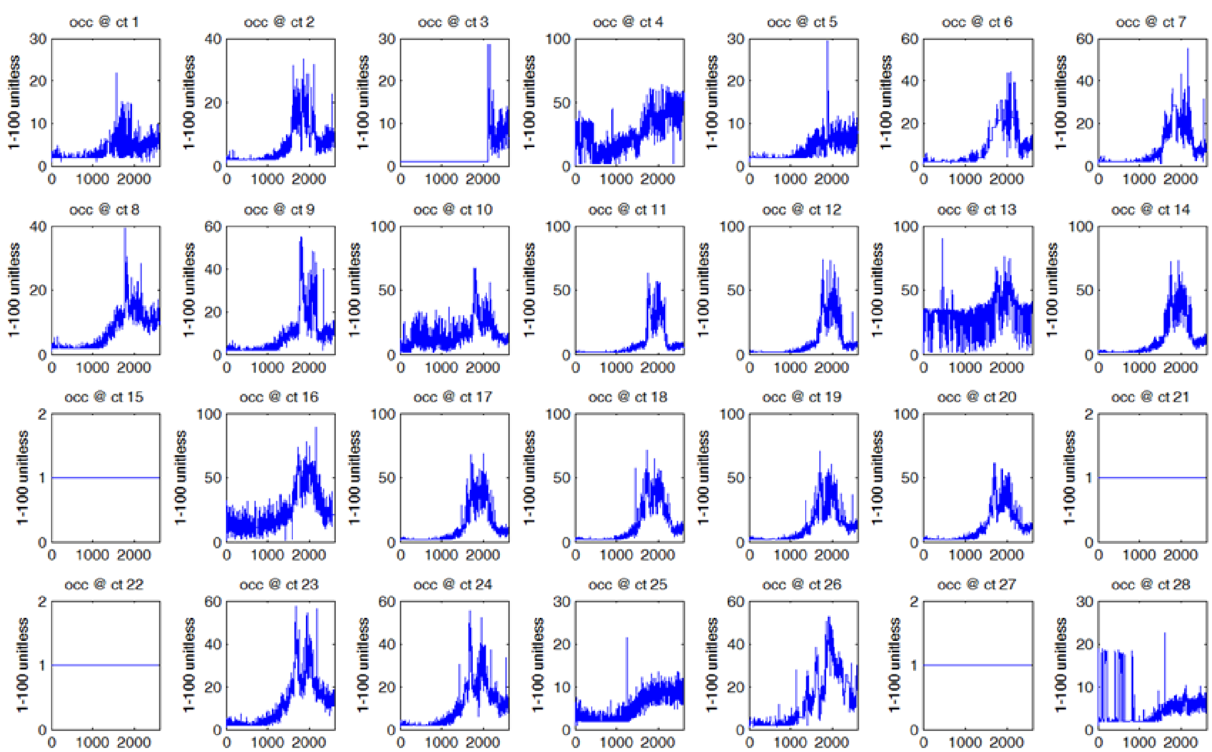

Figure 3-11. Aggregated mainline occupancy data in each VDS: each subfigure shows lane by lane aggregation of occupancy data for each VDS as numbered from 1 to 28 in Figure 3-4.

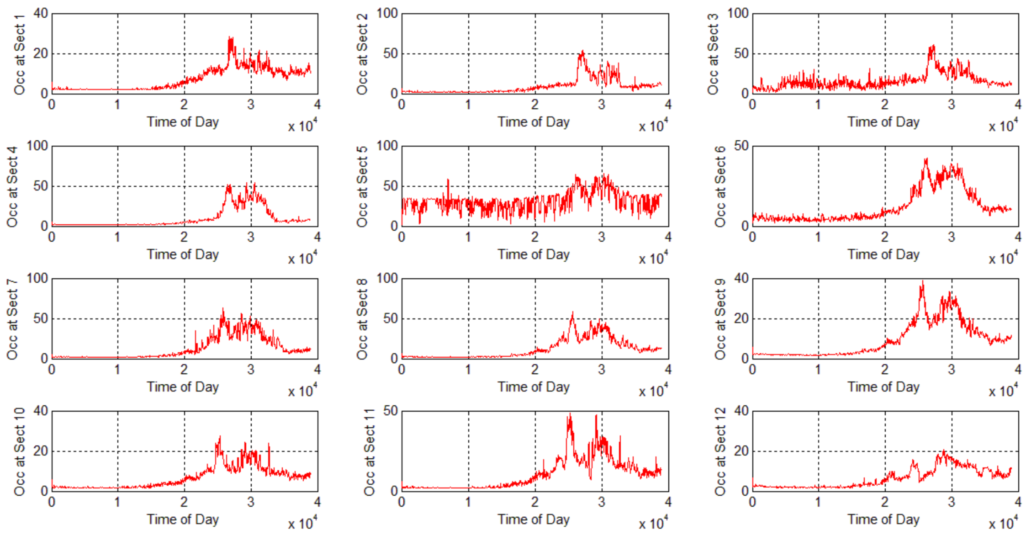

Figure 3-12. Aggregated mainline occupancy data in each section: each subfigure shows aggregation of occupancy data for each section as numbered from 1 to 12 in Figure 2-2.

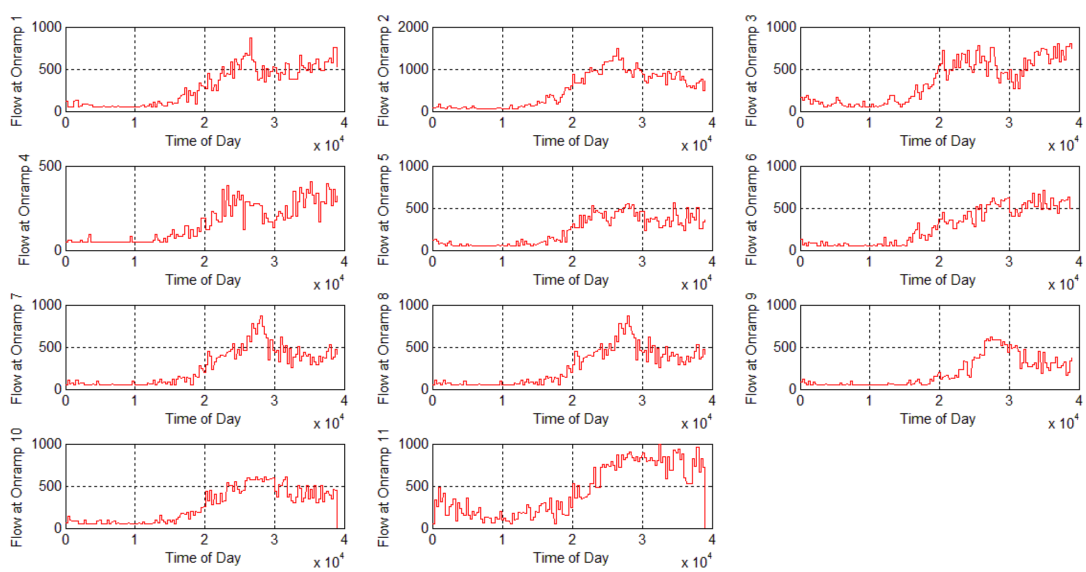

Figure 3-13. Aggregated on-ramp flow in each section: subfigures from left to right are on-ramp flow data of Calvine EB and WB, Mack Road EB and WB, Florin EB and WB, 47th Ave EB and WB, Fruitridge EB and WB, 12th Ave.

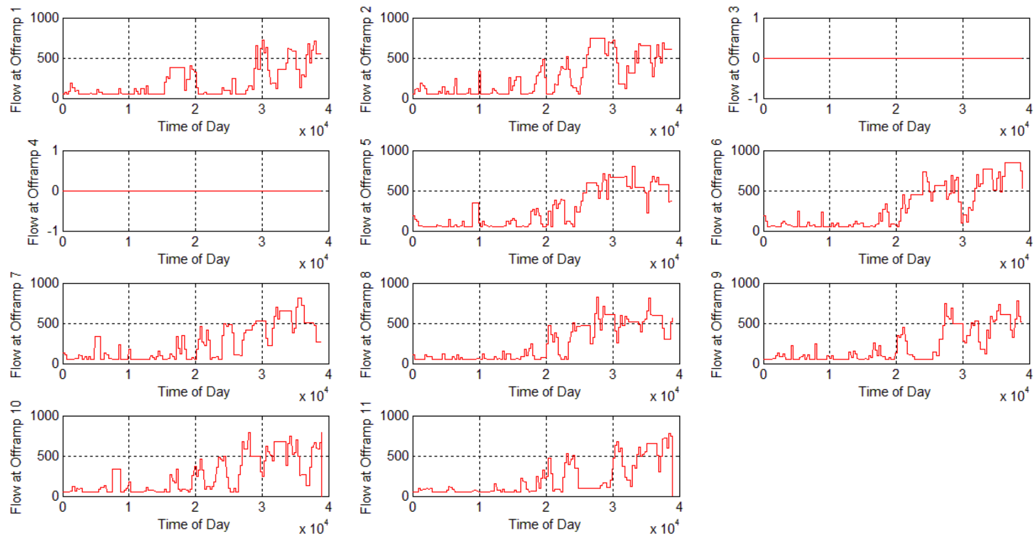

Figure 3-14. Aggregated off-ramp flow in each section: Subfigures 1 is the data of off-ramp #3, Subfigures 2 is the data of off-ramp #4, subfigures 3 and 4 has no data since there are no off-ramps, Subfigures 5-11 is the data of off-ramp from #5 to #11. The off-ramp index is illustrated as the red circle in Figure 3-4.

3.7 Conclusion

This section has demonstrated how the real-time traffic data is processed for use by the CRM control algorithm. The traffic data used in this project can be divided into three categories: mainline data, on-ramp data, and off-ramp data. Mainline data contains lane-by-lane flow, occupancy, and speed for both general purpose and HOV lanes. On-ramp data contains lane-by-lane flow and occupancy. Off-ramp data contains flow and occupancy. All data are collected from URMS 2070 controllers every 30 seconds. However, the real-time raw data obtained from URMS controllers may not be acceptable since the data could be missing, invalid, noisy or have other data faults, which were not suitable to feed into the CRM algorithm directly. Therefore, a series of data cleaning procedures were conducted. The first step was to aggregate the data over lanes, which gives data for each VDS. Then, according to the section configuration defined in the CRM algorithm, these VDS data are aggregated into data for each section. The last step is to remove noise from the data and impute missing or invalid (abnormal) data. Moving average method was adopted for data filtering. If the missing or invalid data were found to be temporary, they were replaced with the data from its adjacent lane or upstream and downstream detector according to flow balance law. If data was missing or appeared to be abnormal for a long time period, PeMS historical data was used as replacement.

Chapter 4. Field Implementation

Field implementation software development can be divided into the following parts: (a) PATH computer is physically located in Caltrans District 3 RTMC directly linked with 2070 controllers in the field through network for real-time raw data polling and logging; (b) data processing and traffic state parameter estimation; (c) CRM rate calculation; and (d) sending CRM rate back to 2070 controller for activation. This chapter will focus on (a) and (c), particularly on how the CRM algorithm was implemented.

4.1 Traffic Characteristics of Test Site

Since the onramp queue detector did not have correct data, it is helpful to know onramp queue situation based on the observations of the local freeway traffic engineers. Another important piece of information is the HOV lane utility since it is important for disseminating the total RM rate of an onramp into individual lanes if it has more than one lane. The following Table 4-1 shows some qualitative information about onramp traffic characteristics and ramp metering facilities which was actually used in the implementation. The information was provided by Caltrans District 3 freeway traffic engineers. The following three items in the table are emphasized, which are most important for CRM algorithm tuning:

- HOV lane Utility
- Onramp demand in peak hours
- Probability of queue spill over to arterial

4.2. ConOps

Figure 4-1 shows the overall system structure of the CRM system and signal flow of the system. The red arrow starting from the loop detector on the freeway to PATH computer in the figure is the measurement of all available field data (flow, speed, occupancy). The blue arrow starting from PATH computer to all cabinets (URMS controller in the field) in the figure is the calculated optimal ramp metering rate by the proposed algorithm. The yellow arrow in the figure starting from each cabinet (URMS) to its corresponding ramp metering traffic lights is the on-ramp metering light control signal. PATH CRM computer is located in Caltrans District 3 RTMC directly link with its intranet for data acquisition, processing, traffic state parameter estimation, calculating optimal RM rate, and sending it to the corresponding onramps activation.

Table 4-1. Qualitative information about onramp traffic characteristics

Onramp ID	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**
Onramp name	**Calvine EB**	**Calvine WB**	**Mack Rd EB**	**Mack Rd WB**	**Florin EB**	**Florin WB**	**47Th St EB**	**47Th St WB**	**Fruitridge EB**	**Fruitridge WB**	**12th St.**
# of Lanes	**2**	**3**	**2**	**2**	**1**	**2**	**2**	**2**	**1**	**1**	**1**
HOV lane Utility	15	15	15	15	No HOV	15	15	15	No HOV	No HOV	No HOV
Onramp demand in peak hours (unitless)	moderate	moderate	moderate	low	No HOV	Moderate	Low	moderate	low	high	high
Probability of queue spill over to arterial (%)	10	30	10	10	10	10	0	10	0	60	95
# of metering lights (unitless)	**1**	**2**	**1**	**1**	**1**	**1**	**1**	**1**	**1**	**1**	**1**
Metering Time AM (hour)	6:00-9:00	6:00-9:00	6:00-9:00	6:00-9:00	6:00-9:00	6:00-9:00	6:00-9:00	6:00-9:00	6:00-9:00	6:00-9:00	6:00-9:00
Metering Time PM (hour)	No	No	3:00-6:00	3:00-6:00	3:00-6:00	3:00-6:00	3:00-6:00	3:00-6:00	3:00-6:00	3:00-6:00	3:00-6:00

PATH CRM computer collect traffic data and aggregation it in every 30 seconds. The benefits of using this system structure are the following. The intranet connection with 2070 controllers in the field used fixed IP addresses. Such an implementation scheme is obviously very advantageous. First of all, it is very simple and direct. Secondly, there is no middle ware in between PATH CRM computer and 2070 controllers in the field; therefore, it is not necessary for support of any third party. Thirdly, PATH computer can access all the raw field data which were not changed by any middle ware; therefore the data were trustable. Fourthly, such a direct link practically avoided any delays and data passing errors caused by middle ware/system(s).

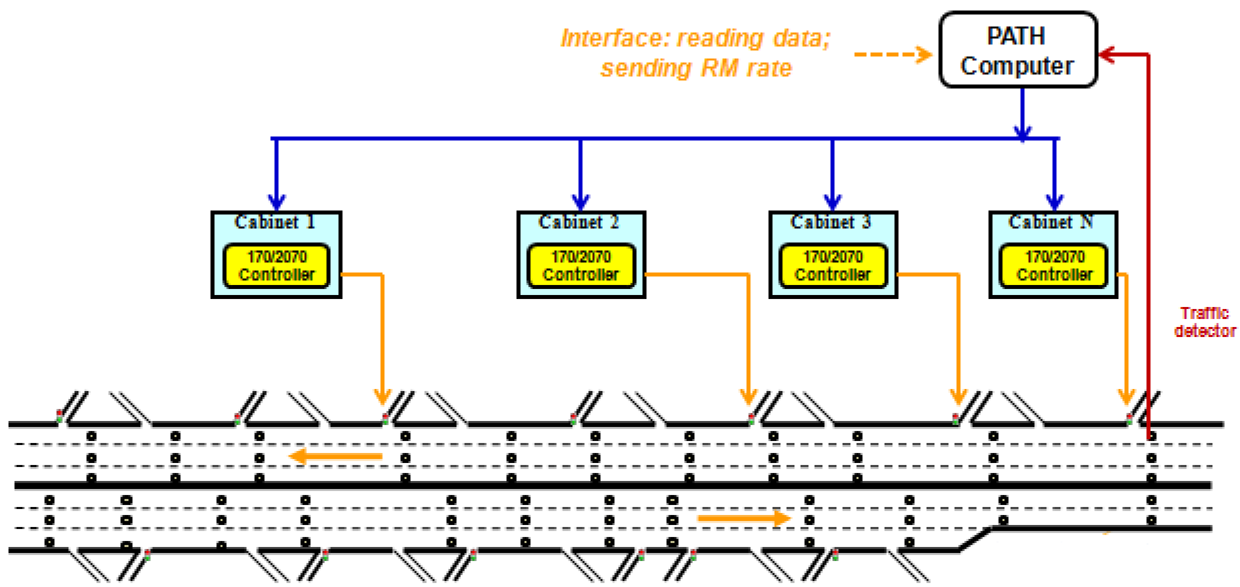

Figure 4-1. System ConOps: Directly interface with TMC RM Computer for traffic data retrieving and CRM; PATH computer is for data processing and calculation for RM rate; real-time data were obtained from 2070 controller every 30s; and CRM rate also sent back for activation every 30s.

4.3 System Software

Software for the Coordinated Ramp Metering (CRM) project was constructed in two layers: low-level interfacing to the field controllers, and mid-level control via a control algorithm. This section describes the low-level interfacing and control.

There are 28 2070 controllers acquiring mainline and ramp data. The corridor of California State Route 99 NB section that was chosen for the project was northbound SR99 from Elk Grove to the Interstate 50 interchange. This corridor was divided into 16 mainline sections with 16 onramps, 5 of which were under LRRM and 11 downstream onramps were under CRM control.

At the lowest level, each 2070 controller was running the Universal Ramp Metering System, version 2.10 (URMS v.2.10). Its job is to acquire loop data and control the ramp metering lights using some control algorithm (e.g. Time Of Day, local Traffic Responsive Plans, PATH Coordinated Ramp Metering algorithm).

There are five possible sources of control for URMS. In order of priority, they are: Manual (MAN), Communications (COM), Interconnect (INT), Time-based control (TBC), and Default (DEF). The CRM control interface uses the Interconnect port for data acquisition and control; the Caltrans District 3 TMC uses the Communications port. Since the COM port has a higher priority than the INT port, this means that the TMC can send a command that will override the same command sent by the PATH system. This ability to override the PATH system was designed as a safety feature in case something went wrong with the PATH system. Manual control is set on the front panel of the 2070 controller, and is used by field engineers to temporarily change ramp metering parameters. Its control times out after six hours. Time-based control is used by the time-of-day tables to set metering rates according to a preset rate table, and the default control is used when time-of-day control is inactive. The reader is referred to the URMS User Manual, Caltrans Document URMS-2070-UM-015, for a complete description of the URMS control system.

The PATH CRM computer, physically located in the District 3 headquarters, was connected to the Caltrans intranet as a test system. Doing so isolated the PATH system from the vagaries (and security problems) of the internet. It communicates with the 28 controllers using Ethernet messages whose protocol was the URMS messaging written by David Wells of Caltrans Headquarters. Following is a description of the software running on the CRM computer (see Figure 4-2 below).

urms.c

urms.c communicates directly with the field controllers with TCP sockets. It sequentially requests a connection and polls its controller for detector data, including flow and occupancy of the mainline, off-ramp, and queue detectors, and flow data for the onramp detectors. Polling for detector data occurs every 30 seconds, since that is the update rate of detector data in all of the controllers. If a change in the metering rate is requested by the PATH control algorithm (in opt_crm.c), this new metering rate is sent to the URMS controller. A separate instance of urms.c is started up for each URMS controller, so when the software is all running, there are 28 instances of urms.c

db_slv

The PATH control algorithm and urms.c "talk" to each other via a publish/subscribe database called db_slv. POSIX messaging is used to send data to, and receive data from, a memory pool that is registered with the Linux operating system. This inter-process communication allows the system engineer to start up different processes independently. db_slv is started up first to establish the memory pool, then separate urms.c processes are started up for each controller. urms.c requests memory allocation for its data structures, and sends a unique number (a "database variable") that db_slv uses when it receives a message for a particular data structure. During runtime operation, any process can request data from db_slv using the database variable. As a practical matter to prevent race conditions, only one process usually writes to a given database variable. The system engineer can also set up a software signal that will trigger processes to read a database variable when its value changes. In this way, a process that does calculations using the data can wait until a new set of data is available before it runs its calculation.

opt_crm.c

opt_crm.c contains the control algorithm that optimized all of the metering rates in the SR99 corridor. It reads the detector data from all of the controllers from db_slv and optimizes the metering rates for the controlled onramp metering lights. Then it sends these metering rates back to the appropriate database variables in db_slv. The act of writing the database variables triggers urms.c to read the new metering rate from db_slv and send it to its controller.

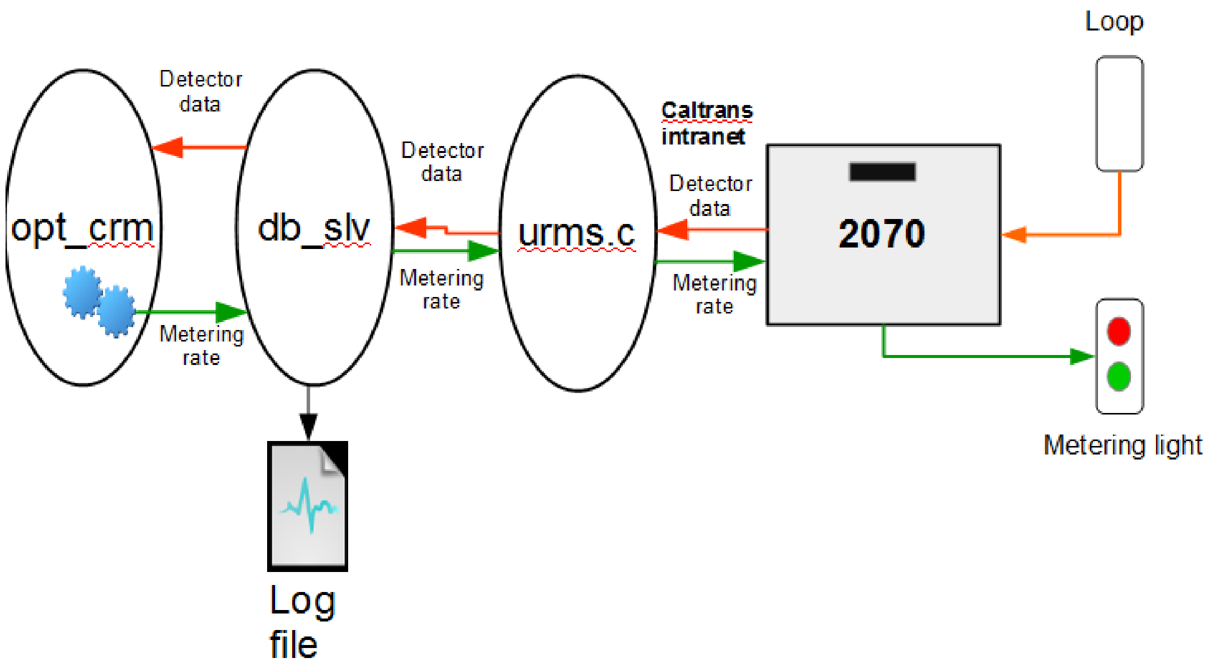

Figure 4-2. Software architecture of Coordinated Ramp metering

In Figure 4-2, loop data is acquired by the 2070 controller. urms.c polls the 2070 for loop data and passes them along to db_slv. opt_crm polls db_slv for all controller data and calculates optimum set of metering rates and writes them to db_slv. This database write triggers urms.c to read the metering rates from db_slv and pass them on to the 2070 controller, which then sends them to the metering lights.

4.4 Raw Traffic Data Acquisition

In the development of the real-time traffic data acquisition system, the team learned through experiences that the real-time data preparation would take much longer time than previously planned. The main difficulties were found to be: (a) the mapping between loop detector ID and the actual positions in the field, which need to be correct; (b) onramp queue detector data were not available, and the team had to use average historical PeMS data to approximate it; and (c) data health: it was very common that detectors in some lanes or the whole detector station across all lanes had heath issues; therefore, data correction, imputation, and filtering was absolutely necessary to ensure proper system performance. The project team used about four weeks for Dry-

4.5 Progressive Implementation

Dry-run (without control) and the week of Sept 19-25 for switching on the field test with the project panel and preliminary tuned the CRM algorithm. The formal tests were started from Sept 26 onwards although some minor tuning was still conducted in the second week. Then the algorithm was finalized and extensive data collections had started from then. The project team kept updating the project panel daily in the first week and weekly afterwards. The update information included comparison of ramp metering rates of CRM and original Local Responsive Ramp Metering for both AM & PM peak hours, and some other traffic state parameters of the freeway corridor. The project team started performance analysis from the week of Nov 1st 2016. We used six weeks PeMS data of this year (as “after” scenario) and the same period of last year (as “before” scenario) for the analysis. The performance parameter used was the ratio of total Vehicle Miles Travelled (VMT) and total Vehicle Hours Traveled (VHT). This ratio could be interpreted as the efficiency of the highway. The performance analysis was accomplished on Nov 11, 2016. The project team proposed to present the test results to the project panel on Nov 18, 2016 in Caltrans District 3 RTMC.

4.6 Monitoring of CRM Rate

To make sure the CRM algorithms were executed correctly, the project team tightly monitored the 2070 controllers in the filed remotely thanks to the intranet of Caltrans. The following is an example which shows what information of the 2070 controllers could be observed remotely (Figure 4-3). The information which can be obtained from the 2070 controller include: onramp name, machine time, field RM ID, control scheme actually activated (i.e. LRRM or CRM, current RM rate), cycle count, etc. this information can be used to tell if the CRM algorithm is activated and clue of fault if is not activated.

Besides, LRRM and CRM rates were compared for everyday during the tests, which was reported to the project panel. Figure 4-4 and Figure 4-5 are example of such comparison for AM and PM peaks respectively. The comparisons of other days have been listed in Appendix 1.

Figure 4-3. Remote monitoring of 2070 controllers in the field

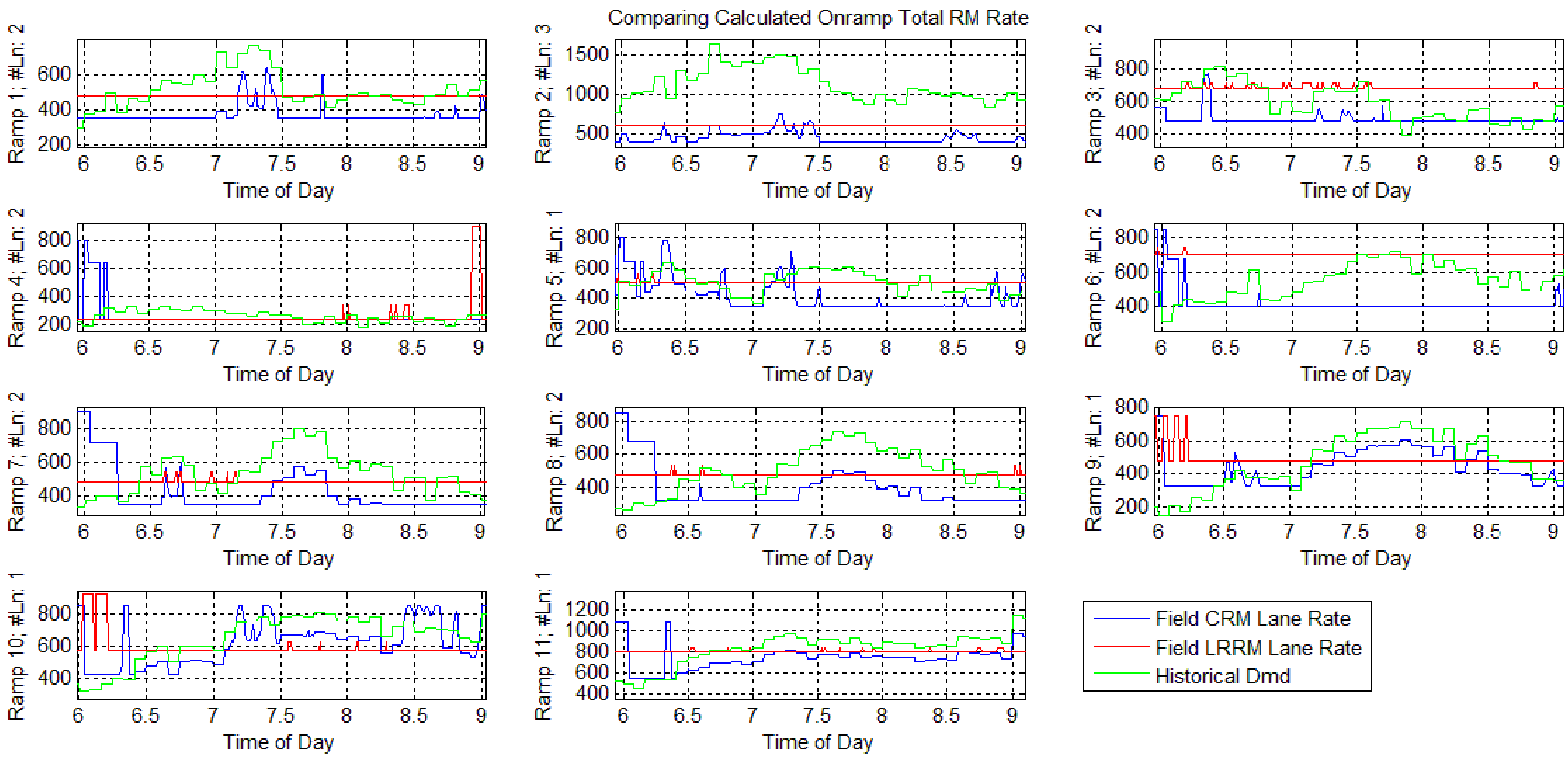

Figure 4-4. Comparison of LRRM and CRM rate for AM peak hours on 10/12/2016 Wednesday

It can be observed from Figure 4-4 that the RM rates for LRRM (**red**) and CRM (**blue**) control strategies are quite different for AM peak hours except Ramp 2 (Calvine WB) and Ramp 11 (12th Street).

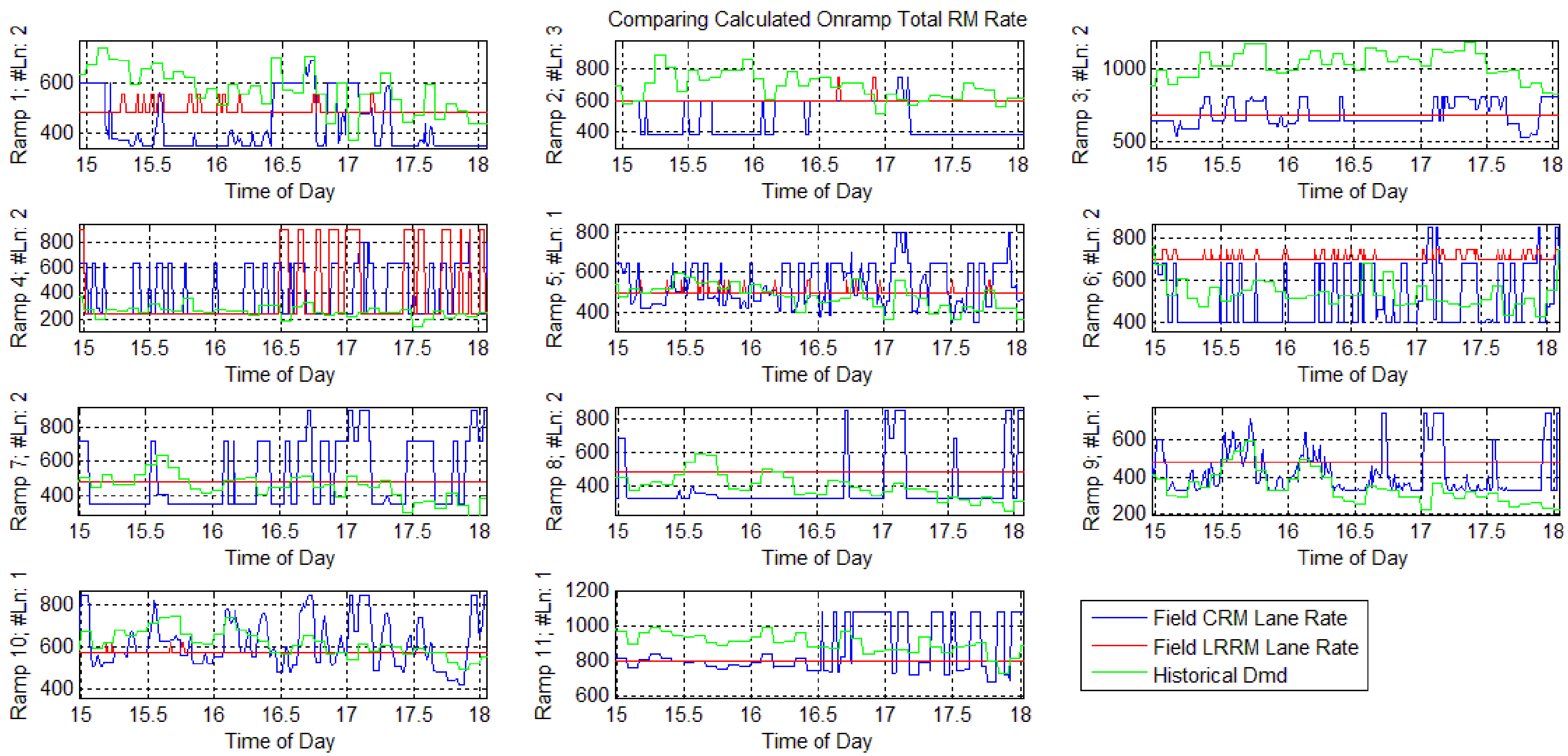

Figure 4-5. Comparison of LRRM and CRM rate for PM peak hours on 10/12/2016 Wednesday

It can be observed from Figure 4-5 that the RM rates for LRRM (**red**) and CRM (**blue**) control strategies are closer than AM peak hours in the sense that LRRM rate is approximately the average of the CRM rate except Onramp 1 (Calvine EB) , Onramp 2 (Calvine WB), and Onramp 6 (Florin WB).

4.7. About Onramp Demand Data and Off-Ramp Flow Data

The demand for each onramp and flow from the off-ramp are very important for CRM implementation since it tells how much traffic needs to be handled from a given onramp to avoid traffic spills back to arterials or surface streets, and the number of vehicles leaving the off-ramps. Although, this project did not have an active coordination between freeway RM and arterial intersection traffic signal controls, the CRM algorithm was taking care of this issue in a simple way: if the demand from an onramp is too high, the corresponding CRM rate would be slightly higher. This however will sacrifice the overall performance of the system. To implement this functionality, it is necessary to have flow and occupancy data from the queue detector for the onramps. Since the queue detector data for the test site was not available, the project team used 5mon average flow of a typical week of the same month last year as an approximation. Since the traffic patterns in the test site were very similar for all the workdays, which was mainly the state government staff as commuters towards work in AM peak hours, this approximation proved to be reasonable. The following Figure 4-6 depicts the demand flow for all the onramps, Figure 4-7 depicts the flow from off-ramps.

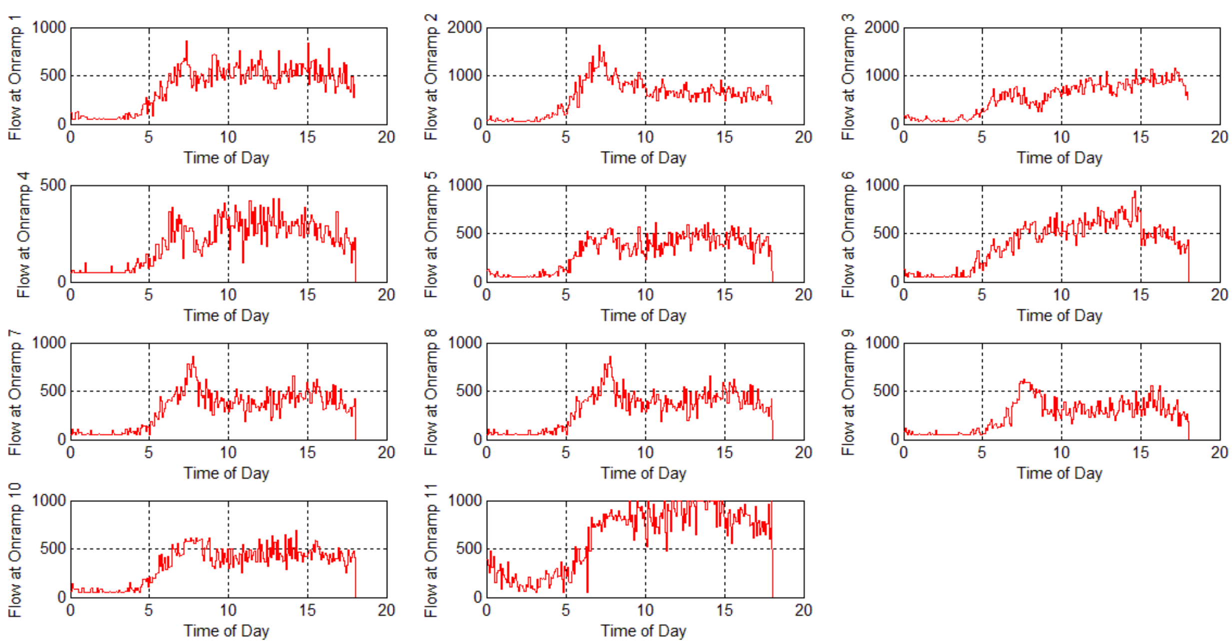

Figure 4-6. PeMS 5min historical onramp demand data averaged overall typical five workdays

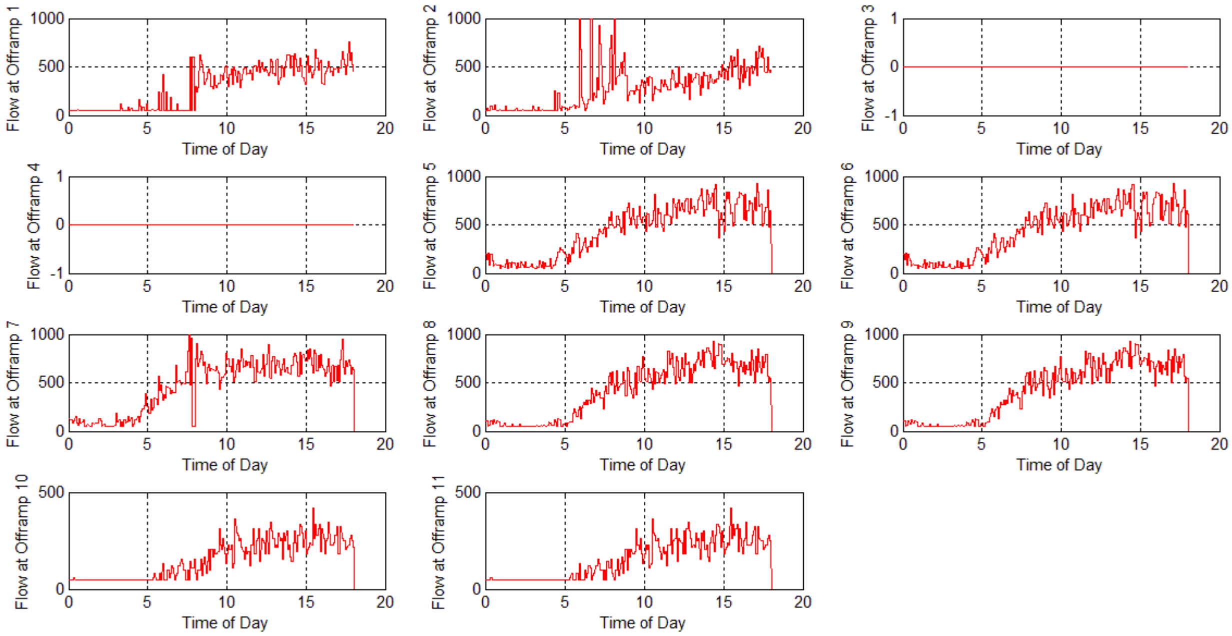

Figure 4-7. PeMS 5min historical off-ramp demand data averaged overall typical five workdays

The off-ramp 3 and 4 were all zeros since the off-ramp of section 3 and section 4 did not exist.

4.8 Monitoring of Queue Length

The proposed CRM algorithm changes the ramp metering light and it also influence the queue dynamics at each onramp. Since the queue detector in the test site is not available, the actual queue length during the field test cannot be measured directly. Although the queue length can be estimated by other information around the onramp: onramp flow, demand, and its adjacent mainline flow, the accuracy of queue length estimation is very limited. In order to overcome this equipment limitation, Google Map is used for monitoring the queue condition at each onramp. Besides, Google Map also provides freeway incident and accident information in real-time, which help us understand if there is an event influence the field test. The observation was made from 6:00 AM to 9:00 AM every 20 minutes to 30 minutes during the field test. Figure 4-8 is an example of queue monitoring and it is obtained near Mack Road onramp at 7:24 AM on 10/19/2016. The other monitoring of queue length by Google Map on 10/19/2016 Wednesday is provided in Appendix 3. The observation of queue length by Google Map indicates that the queue at each onramp will not split back to its adjacent local street and arterial. The onramp storage is used without excess its capacity.

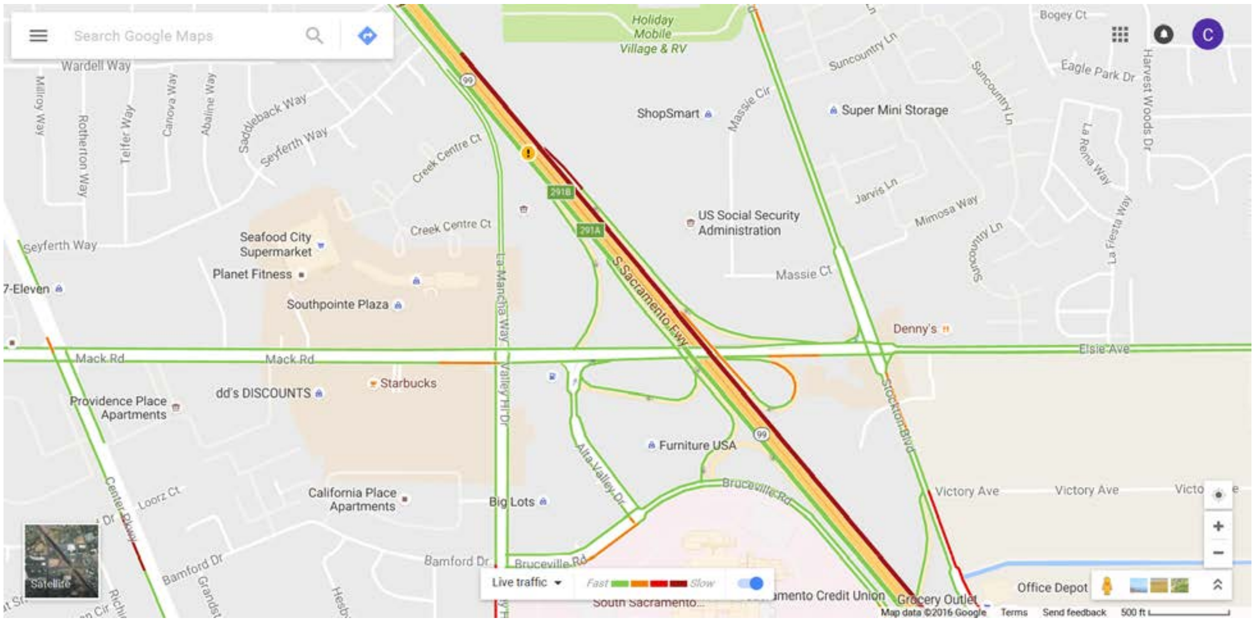

Figure 4-8. Monitoring of queue length near Mack Road onramp at 7:24 AM on 10/19/2016

4.9 Conclusion

This section describes the ConOps of the system, system software in the PATH computer and observations of the field test. The system software contains three parts: real-time traffic data acquisition module, traffic state estimation module, and real-time CRM algorithm. The ConOps of the system is described as the following. The PATH computer located in Caltrans District 3 RTMC is directly connected with each 2070 controller running URMS in the field through the District 3 intranet. The PATH computer actively polled all traffic data every 30s. These raw real-time data collected from the field were stored on the hard disk of the PATH computer and then processed by data cleaning procedures. Then, the processed data were sent to the core CRM algorithm for the calculation of the optimal ramp metering rate for each on-ramp. Those ramp metering rates were then sent back to the 2070 controller at each on-ramp for setting the corresponding RM traffic signal. The ramp metering rate was also updated every 30 seconds.

The field implementation started with a two week dry run which was then followed by a progressive implementation test and then extensive formal field test and "after" scenario data collection. During the progressive implementation test, the project team examined all the data flow, signal, hardware and software in the system to make sure the system working as expected, particularly, traffic data and ramp metering rate. The formal field test was started from September 26, 2016 and some minor tuning of control parameters was still conducted in the first two weeks of field test. During the field test, all traffic data and ramp metering rates were stored in a database, closely monitored by the project team, and regularly reported to the project panel. Since most of on-ramp queue detectors were not available during the test period (9/19/16 – 11/4/16), the queue length ground truth could not be measured. In order to monitor the ground truth of queue length and observe the impact of ramp metering on the local streets, the project team regularly observed the real-time traffic situation through Google Maps for all the 11 controlled on-ramps. The observations showed that the queue of each metered on-ramp did not spill back to the local street during the AM traffic peak hours (6:00AM-9:00AM), which indicated that the CRM rates for all the onramps were reasonable.

Chapter 5. CRM Field Test Performance Analysis

In order to evaluate the performance of the CRM algorithm objectively, the project team purposely used PeMS hourly VHT and VMT data since this data was independent from what we collected directly from the 2070 controllers in the field. The PeMS data archive is shown in Figure 5-1.

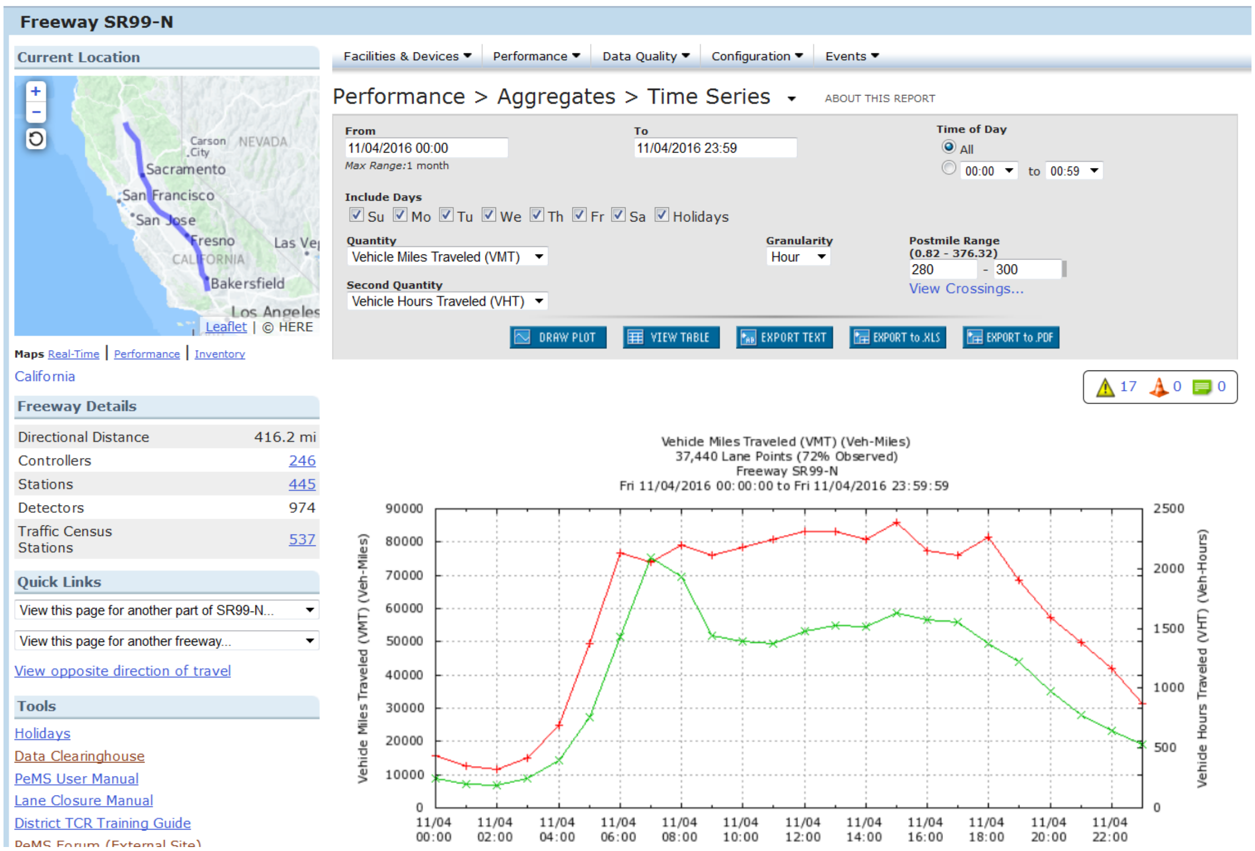

Figure 5-1. Hourly Data Source in PeMS: the red line is VMT and the green line is VHT

By doing so, the project team intend to obtain the performance results as objective as possible. The same period, i.e. those of October 2015 and 2016, data were used: corresponding weekday was compared, e.g. Tuesday compared with Tuesday, which is reasonable since the traffic pattern for commuters are very similar for the same working days. To address demand fluctuation and difference, VMT/VHT was used as the performance parameters, which could be understood as the average speed. Note that this parameter was defined in PeMS as the *system efficiency*.

5.1 Performance Indexes of Freeway System

The goal of coordinate ramp metering (CRM) control is to improve the freeway system efficiency by regulating the number of vehicles entering the freeway mainline from the on-ramp. Two performance measures are used to evaluate the system efficiency. They are Vehicle-Miles Traveled (VMT) and the Vehicle-Hours Traveled (VHT). The definition of VMT is that the sum of distance (in unit of miles) traveled by each vehicle on the given section of freeway over a given time period. VMT is the same as the concept of total travel distance (VMT). Consider a freeway is partitioned into n segments with length L_i for the i-th segment and each segment contains at least one loop detector. VMT can be computes as

$$VMT(t) = \sum_{i=1}^{n} VMT_i(t) \qquad \text{(Eq. 5.1)}$$

where

$$VMT_i(t) = f_i(t) L_i$$

and $f_i(t)$ is the flow at the i-th segment.

The definition of VHT is that the sum of all trip times (in unit of hours) spent by each vehicle on the given section of freeway over a given time period. VHT is the same as the concept of total travel time (VHT). The definition of VHT is

$$VHT(t) = \sum_{i=1}^{n} VHT_i(t) \qquad \text{(Eq. 5.2)}$$

where

$$VHT_i(t) = \frac{f_i(t) L_i}{v_i(t)}$$

and $v_i(t)$ is the speed at the i-th segment.

In the CRM control algorithm, the control objective is regulating the ramp metering rate such that the vehicles entering the freeway can either maximize VMT or minimize VHT. Therefore, the freeway efficiency can be defined as

$$Q(t) = \frac{VMT(t)}{VHT(t)} \tag{Eq. 5.3}$$

From the definition of Q, VMT is in the numerator of Q value and VHT is in the denominator of Q value, increasing VMT or decreasing VHT can make Q increase, which is consistent with the control objective: maximize VMT or minimize VHT. Therefore, higher Q values not only indicate the control performance is better, but also indicate the freeway efficiency is better.

There is another way to interpret the Q value. Since the unit of Q equals to the unit of flow over vehicle density (f/ρ), the freeway efficiency can also be interpreted as the average speed of all trips of the freeway during a period of time. Higher Q values indicate the drivers on the freeway gain higher speed on average, therefore, higher Q values means high freeway efficiency. In addition, higher VMT values indicate the freeway can be used by more drivers in the traffic engineer's point of view. Lower VHT values indicate the driver can spend less time while travel through the freeway. Increasing VMT or decreasing VHT can make Q increase, which is equivalent with increasing the freeway usage or reducing the waste of travel time. Therefore, Q is an index of freeway efficiency for both traffic engineer's and driver's point of view.

5.2 Evaluation of Performance Indexes

Before the CRM control was installed in the field, the traffic data of the test site is collected and analyzed to understand the original freeway traffic characteristic and performance. The data of the test site is also collected during the field test. Therefore, the comparison of the freeway traffic between before field test and after field test can be made by investigating the percentage improvement. VMT, VHT and Q are three performance of freeway, their percentage improvement are defined as following.

The percentage of improvement of VMT is defined as

$$\Delta VMT = \frac{VMT_{new} - VMT_{old}}{VMT_{old}} \tag{Eq. 5.4}$$

where VMT_{new} is the VMT value after the field test and VMT_{old} is the VMT value before the field test. If ΔVMT is a positive value, then it means VMT_{new} is greater than VMT_{old} and it indicates that the usage of the freeway is increased since the VMT value is increased after the field test, which means the new control method in the field test makes the freeway accommodate more traffic demand than before the field test. Therefore, a positive ΔVMT means the usage of freeway is improved and it is favorable.

The percentage of improvement of VHT is defined as

$$\Delta VHT = \frac{VHT_{new} - VHT_{old}}{VHT_{old}} \tag{Eq. 5.5}$$

where VHT_{new} is the VHT value after the field test and VHT_{old} is the VHT value before the field test. If ΔVHT is a negative value, then it means VHT_{new} is less than VHT_{old} and it indicates that the travel time of the freeway is decreased since the VHT value is decreased after the field test, which means the new control method in the field test makes the driver spend less travel time on the freeway on average than before the field test. Therefore, a negative ΔVHT means the travel time of the freeway is improved and it is favorable.

The percentage of improvement of Q is defined as

$$\Delta Q = \frac{Q_{new} - Q_{old}}{Q_{old}} \tag{Eq. 5.6}$$

where Q_{new} is the Q value after the field test and Q_{old} is the Q value before the field test. If ΔQ is a positive value, then it means Q_{new} is greater than Q_{old} and it indicates that efficiency of the freeway is increased since the Q value is increased after the field test, which means the new control method in the field test makes the average speed of traffic on freeway and the efficiency of freeway increase than before the field test. Therefore, a positive ΔQ means the speed and efficiency on the freeway is improved and it is favorable.

5.3 Performance Evaluation of Field Test

The data source of performance evaluation of field test is obtained from PeMS. The data of the stretch of test site SR99 Northbound from 280 Postmile to 300 Postmile is used. The sampling time of performance index VMT, VHT, and Q is in hour and it is the minimal sampling time provided by PeMS. The duration of data before field test is the weekday from the whole October to the first week of November in 2015. The duration of data during the field test is the weekday from the whole October to the first week of November in 2016. The AM ramp metering activation time is from 6:00 AM to 9:00 AM. The PM ramp metering activation time is from 3:00 PM to 6:00 PM. Note that the field test started from September 19, 2016 Wednesday. The traffic data in the duration from September 19, 2016 to September 30, 2016 is not used since the traffic engineers was adjusting system parameters during the beginning of the field test. After the duration of system tuning, the traffic characteristic becomes representative and those data is meaningful for analysis. The performance index in same day of week in 2015 and 2016 are compared. Table 5-1 listed the day that we used to make day by day comparison.

Table 5-1 The weekday before and during the field test

	Monday	Tuesday	Wednesday	Thursday	Friday
1st week	10/5/2015 10/3/2016	10/6/2015 10/4/2016	10/7/2015 10/5/2016	10/8/2015 10/6/2016	10/9/2015 10/7/2016
2nd week	10/12/2015 10/10/2016	10/13/2015 10/11/2016	10/14/2015 10/12/2016	10/15/2015 10/13/2016	10/16/2015 10/14/2016
3rd week	10/19/2015 10/17/2016	10/20/2015 10/18/2016	10/21/2015 10/19/2016	10/22/2015 10/20/2016	10/23/2015 10/21/2016
4th week	10/26/2015 10/24/2016	10/27/2015 10/25/2016	10/28/2015 10/26/2016	10/29/2015 10/27/2016	10/30/2015 10/28/2016
5th week	11/2/2015 10/31/2016	11/3/2015 11/1/2016	11/4/2015 11/2/2016	11/5/2015 11/3/2016	11/6/2015 11/4/2016

A quick way to observe the VMT improvement is to plot the VMT versus Q data for both before and after the field test. Figure 5-2 shows the VMT versus Q distribution, where circles are the data from AM traffic and crosses are data from PM traffic. The figure shows that the circles are more scatter than the crosses, which means the VMT values have larger variation rage during the AM traffic than it during the PM traffic. The circles are classified into two colors: red circle are the data in 2016 (data in field test) while blue circles are the data in 2015 (data before field test). Comparing with the blue circle data cluster, the red circle cluster lies on the right direction of VMT axis and upward direction of Q axis. This difference of location of data cluster indicates the improvement of freeway efficiency Q and VMT since more red circles moves toward positive Q and positive VMT direction and it means the field test increase the freeway efficiency (average speed) Q by increasing VMT (usage of freeway/increasing demand). One the other hand, the scatter of crosses in the figure is more concentrate than circles, which means the PM traffic has no much change after the CRM control. Therefore, the CRM control increases both the freeway efficiency and usage during AM traffic more significantly than it during PM traffic.

The VHT improvement can also be observed by plotting VHT versus Q for both and before the field test, which is shown in Figure 5-3. In this figure, circles are the data from AM traffic and crosses are data from PM traffic. It shows that the circles are more scatter than the crosses, which means the VHT values have larger variation rage during the AM traffic than it during the PM traffic (the variation of travel time in AM is larger than it in PM). Comparing with the blue circle data cluster, the red circle cluster slightly moves to the left direction of VHT axis and to the upward direction of Q axis. This difference of location of data cluster indicates the improvement of freeway efficiency Q and VHT since more red circles moves toward positive Q and negative VHT direction and it means the field test increase the freeway efficiency Q by decreasing VHT (travel time) . One the other hand, the scatter of crosses in the figure is more concentrate than circles, which means the PM traffic has no much change after the CRM control. Therefore, the CRM control slightly decreases the VHT (travel time) during AM traffic while the VHT during PM traffic does not have significate change.

Figure 5-2 and Figure 5-3 give the general trend of change of freeway performance before and after field test. The details of all VMT, VHT, and Q data collected in 2015 (before the field test) and in 2016 (during the field test) are listed in Appendix 2.

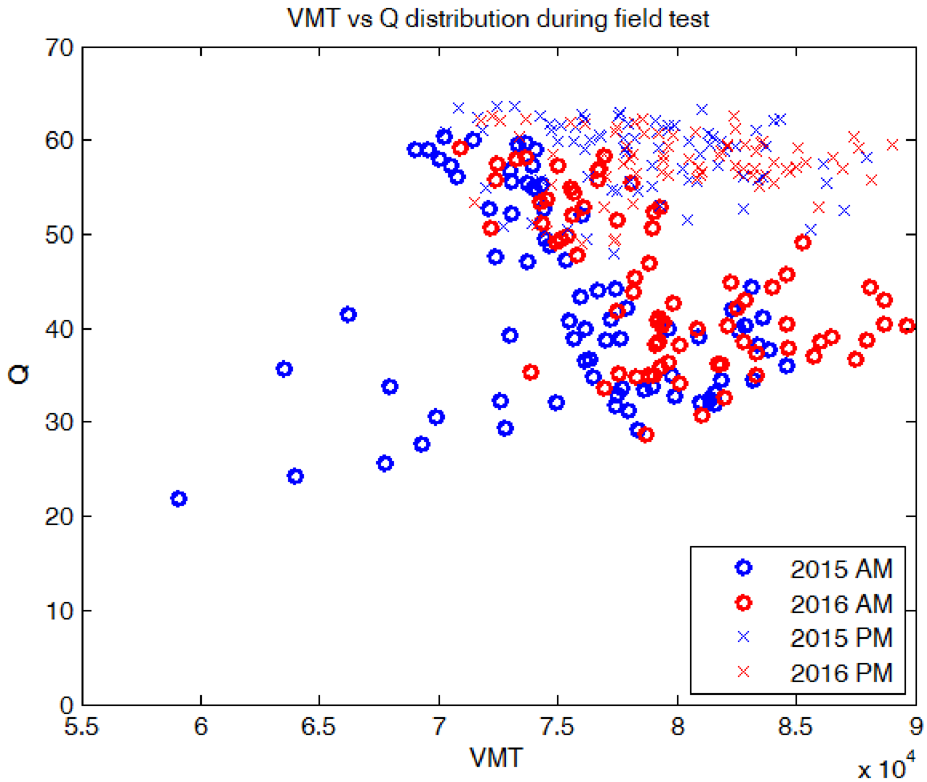

Figure 5-2. VMT versus Q distribution: Blue and red circles are AM traffic data in 2015 and 2016, respectively. Blue and red crosses are PM traffic data in 2015 and 2016, respectively.

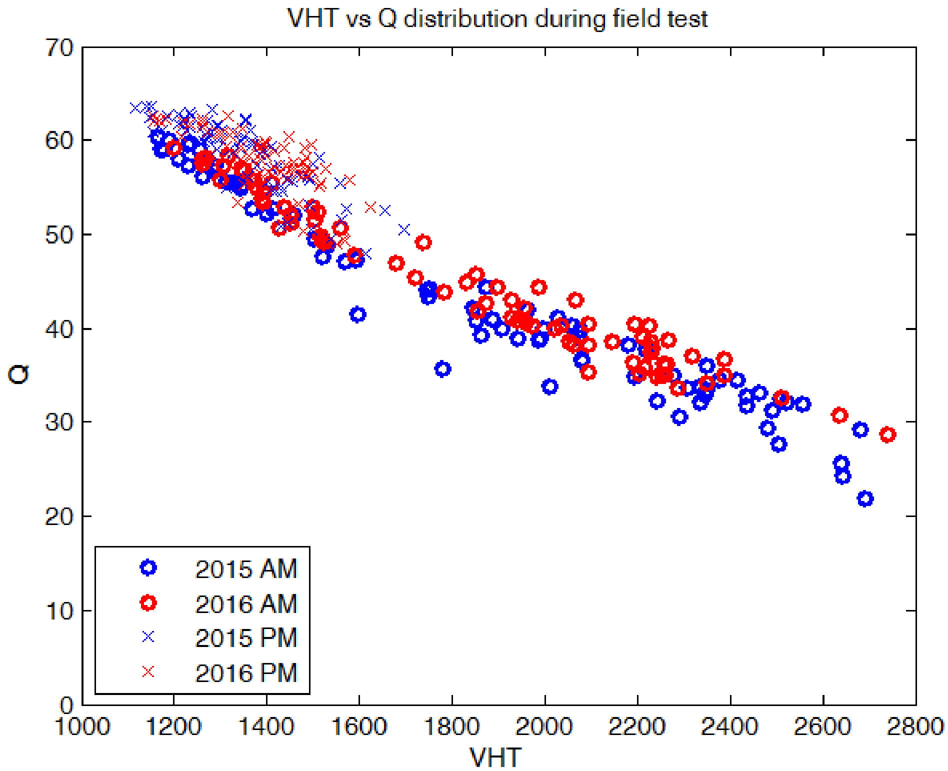

Figure 5-3. VHT versus Q distribution: Blue and red circles are AM traffic data in 2015 and 2016, respectively. Blue and red crosses are PM traffic data in 2015 and 2016, respectively.

5.4 Conclusion

A summary of the comparison of PeMS data on VMT, VHT, and Q before and after the field test is shown in Table 5-2. In order to evaluate the overall performance of each performance index, the average of VMT, VHT and Q over all time duration in AM and PM has been calculated as follows.

The average of %ΔVMT in the AM peak is computed as (4.72 %+6.442 %+5.019 %)/3=5.39%. The average of % ΔVHT in AM is computed as (-4.881 %-2.346 %+2.314 %)/3=1.64%. The average of % ΔQ in AM is computed as (10.093 %+8.999 %+2.644 %)/3=7.25%.

The average of %ΔVMT in PM is computed as (2.307 %+1.667 %+3.698 %)/3=2.56%. The average of % ΔVHT in PM is computed as (5.974 %+0.937 %+2.247 %)/3=3.04%. The average of % ΔQ in PM is computed as (-3.460 %+0.723 %+1.420 %)/3=0.44%.

Therefore, we have average improvement of VMT, VHT, and Q in both AM and PM traffic.

AM ramp metering performance is summarized as following:

- VMT (vehicle-miles traveled) is increased by 5.39% on average.
- VHT (vehicle-hours traveled) is decreased by 1.64% on average.
- Q (freeway efficiency) is increased by 7.25% on average.
- Since VMT and Q have significant increase, CRM in AM peak has improved the traffic.

PM ramp metering performance is summarized as following:

- VMT (vehicle-miles traveled) is increased by 2.56% on average,
- VHT (vehicle-hours traveled) is increased by 3.04% on average, and
- Q (freeway efficiency) is decreased by 0.44% on average.
- Since the change of both VMT and Q are marginal, CRM in the PM peak cannot improve traffic.

Table 5-2 Summary of both AM and PM performance comparison

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

Chapter 6. Concluding Remarks and Future Research

The following remarks and recommendations are based on our experiences in the execution of the project. It includes the applicability, extendibility, and limits of the CRM algorithm developed and tested.

6.1 Concluding Remarks

RM is the most widely used freeway traffic congestion mitigation means in California, which essentially controls the onramp traffic demand into the freeway. However, the current RM strategy for freeway operation is mainly LRRM – it does not consider the traffic further upstream and further downstream. Since traffic along different sections of the freeway corridor affects each other if the demands are high, to achieve better system performance, it is necessary to coordinate the metering rate along a freeway corridor to balance the entrance flow so that the mainline throughput could be improved. How long a freeway corridor should be coordinated, or the scope of the system, needs be determined based on the overall traffic situation: mainline most upstream traffic demand, demand from onramps, out-flow from off-ramps, road geometry, and distances between onramps. It only makes sense to coordinate onramps that are close enough so that their traffic affects each other.

This project implemented and field tested the CRM algorithm developed in a previous project [2]. The CRM algorithm used a simplified optimal control strategy, called model predictive control. Thanks to the real-time traffic data from the field and the corresponding traffic state parameters estimated at each time step, it is only necessary to solve a linear programing (LP) problem, which is simple for implementation. The objective function is the trade-off between total VMT and total VHT of the freeway corridor. Intuitively, the CRM algorithm intended to control the corridor traffic as a long discharge section in the sense that the downstream section traffic should not be more congested than the upstream section traffic if the overall demand is high. Field test results indicated that this approach did improve overall traffic along the corridor.

The test site, SR99 NB from Elk Grove to the SR 50 interchange has 16 onramps and 11 off-ramps. The implementation only controlled the downstream 11 onramps, which included all the bottlenecks. The most upstream 5 onramps were still controlled by the default LRRM strategy. For easy implementation, Caltrans HQ and District 3 Freeway Operations engineers suggested a simple system set-up by putting a project computer at the District 3 RTMC and directly linking it with the 2070 traffic controllers which run URMS in the field through the District 3 intranet for real-time data acquisition and control. This approach proved to be very simple, efficient and robust. The project did not add any extra sensors, nor any other equipment except an industrial computer running Linux as real-time operating system. To avoid any negative impact on the corridor traffic, the project team adopted a progressive implementation process including a dry run, cautious progressive switching on the control and tight monitoring and regular reporting to the project panel about the metering rate for each onramp and the corridor traffic situations.

To get an objective evaluation of the performance of the algorithm, PeMS hourly VHT and VMT data were collected for 5 weeks during the extensive tests. The performance parameter used for evaluation was the ratio VMT/VHT which could be interpreted as the average speed (or efficiency as defined in PeMS [6]). It is believed that this ratio is objective and could reasonably accommodate traffic demand fluctuations.

According to the data in Table 5-2, the improvement of freeway performance index is summarized as follows: for AM peak (6:00am-9:00am) traffic, CRM algorithm improved the traffic by 7.25%. For PM peak (3:00pm-6:00pm) traffic, VMT/VHT decreased by 0.44% on average, which was marginal and indicated that CRM algorithm could not improve the traffic. The reason was that the traffic was already in free-flow most of the time in PM peak hours. Therefore, CRM could not do much to improve the traffic if it was not congested in the first place. The results show that the CRM algorithm would be effective in improving congested traffic.

6.2 Recommendations

It is noted that the test conducted was for a single corridor of medium size with more than one bottleneck and with RM on all the onramps. Most importantly, the most downstream traffic at the interchange with SR50 (after 12th Ave.) in both directions did not back-propagate to SR99 NB most of the time in the AM peak hours. Otherwise, the algorithm would not work, which is a limitation to the algorithm that was tested. Clearly, if the system includes more than one freeway corridor and other traffic flows affect each other through the interchanges, it is necessary to use RM to control the traffic of all the freeway corridors and the traffic through the interchange(s) and to properly balance the traffic density over the whole network involved. Further research would be necessary to extend the CRM algorithm to freeway traffic networks involving multiple freeway corridors.

The tested algorithm, however, could be applied to a relatively isolated freeway corridor similar to the SR99 NB corridor with multiple bottlenecks. For doing so conveniently, it is necessary to develop a User Interface software, e.g. a Linux or Windows based application. This was initially suggested by Caltrans District 3 RTMC traffic engineers at the End Project Meeting on 11/18/16. The initial consideration suggests that the software should have but not be limited to have the following functionalities for more convenient application to other similar freeway corridors:

- model a given freeway corridors by dividing the freeway into section according to the location of the onramps
- identify bottlenecks along the corridor, particularly the most downstream bottleneck
- set up the data link from the field 2070 controller and the CRM computer for data polling
- build a proper data mapping between the raw traffic detector data from 2070 controllers and mainline sections, onramps and off-ramps
- check automatically if the mapping is built correctly
- process raw data robustly to generate traffic state parameters even if there are some temporary data faults
- set lower and upper bounds of the CRM rate according to daily operation experience
- log raw traffic data and state parameters in database for further analysis
- select parameters for system tuning – the number of parameters to be tuned should be minimized
- monitor the CRM rate of any onramp and give warning/alarm to the CRM operation engineers through email or other ways if any part of the control system goes wrong or the calculated CRM rate is not executed properly at any onramp for any reason

- evaluate the performance of the system with aforementioned index using PeMS data

Those functionalities will hide all the complications and allow Caltrans freeway traffic engineers to use the CRM control algorithms more conveniently. Therefore, the project team would join Caltrans District 3 to propose a new project for developing such software.

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Appendix 1. RM Rates Comparison for LRRM and CRM

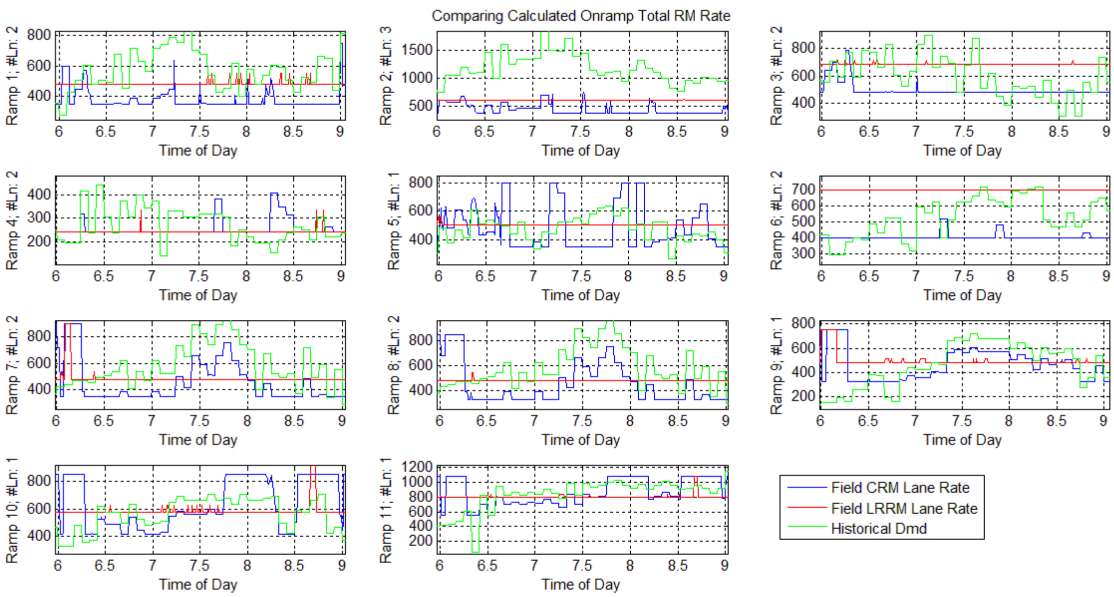

Figure A1-1. Comparison of LRRM and CRM rate for AM peak hours on 09/27/2016 Tuesday

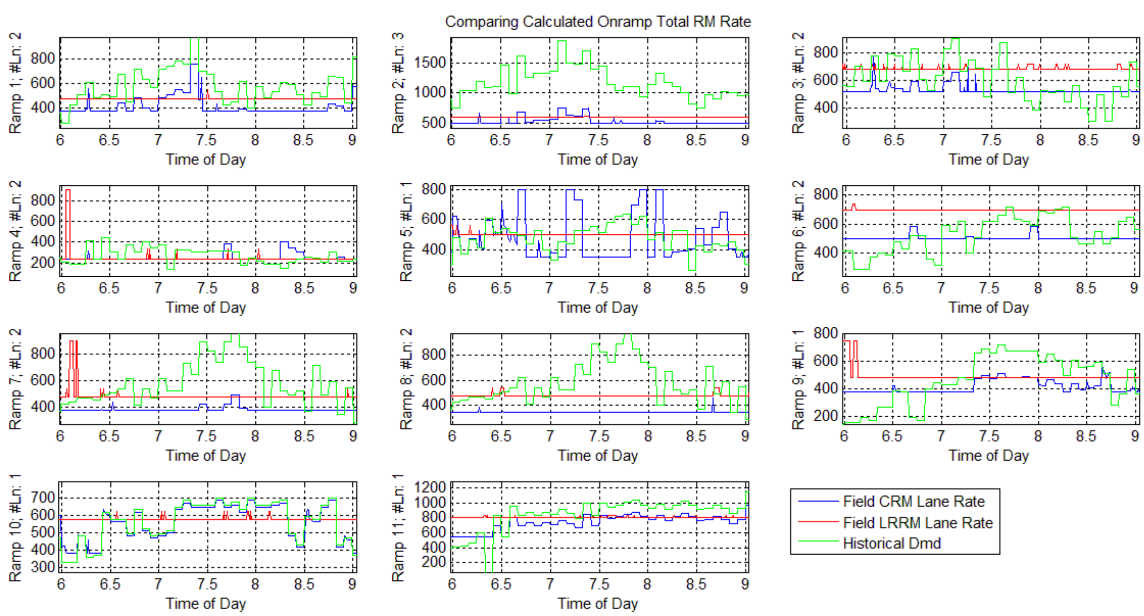

Figure A1-2. Comparison of LRRM and CRM rate for AM peak hours on 09/28/2016 Wednesday

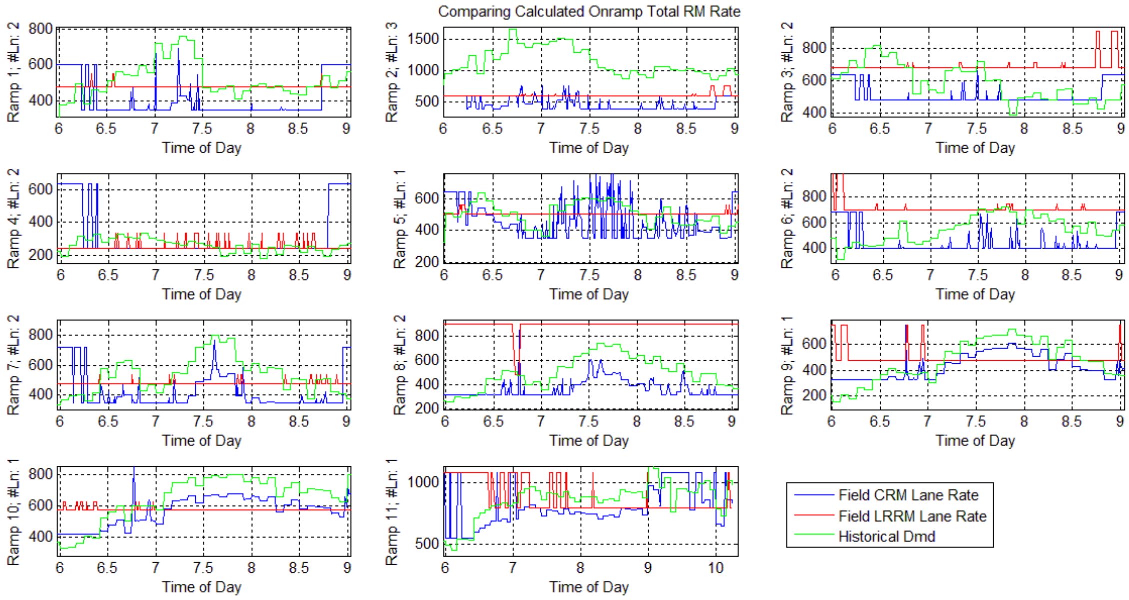

Figure A1-3. Comparison of LRRM and CRM rate for AM peak hours on 09/29/2016 Thursday

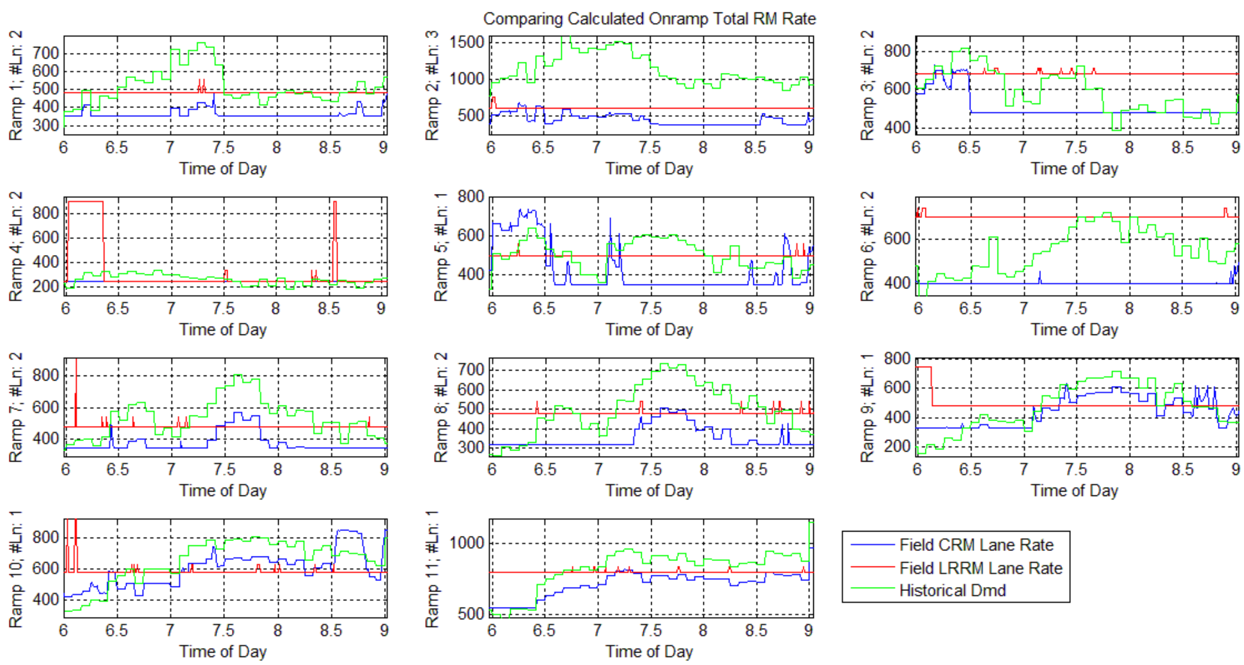

Figure A1-4. Comparison of LRRM and CRM rate for AM peak hours on 09/30/2016 Tuesday

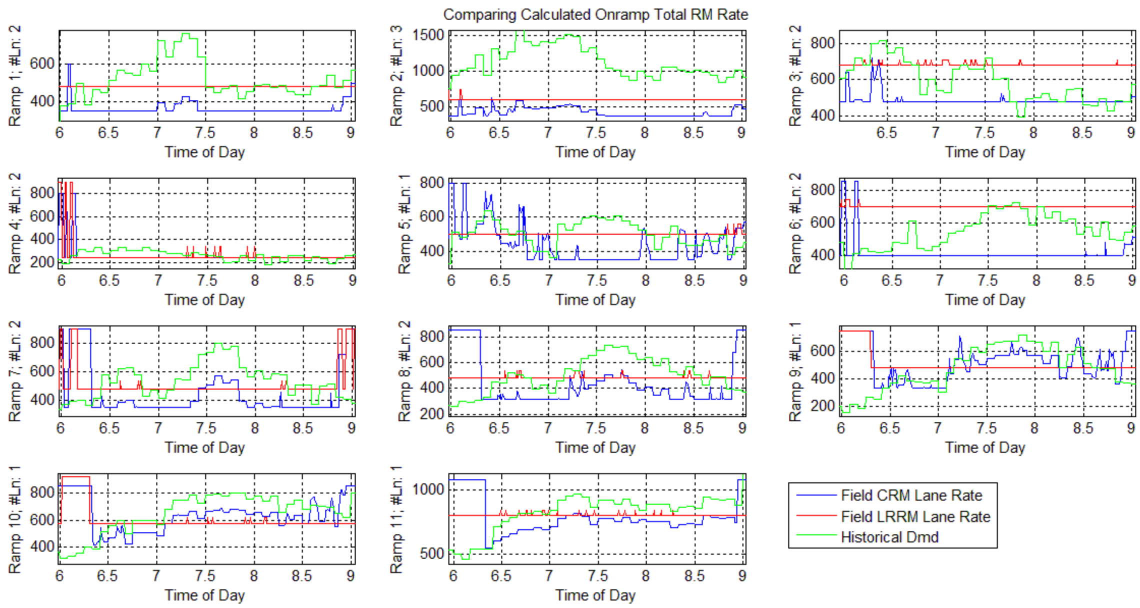

Figure A1-5. Comparison of LRRM and CRM rate for AM peak hours on 10/03/2016 Monday

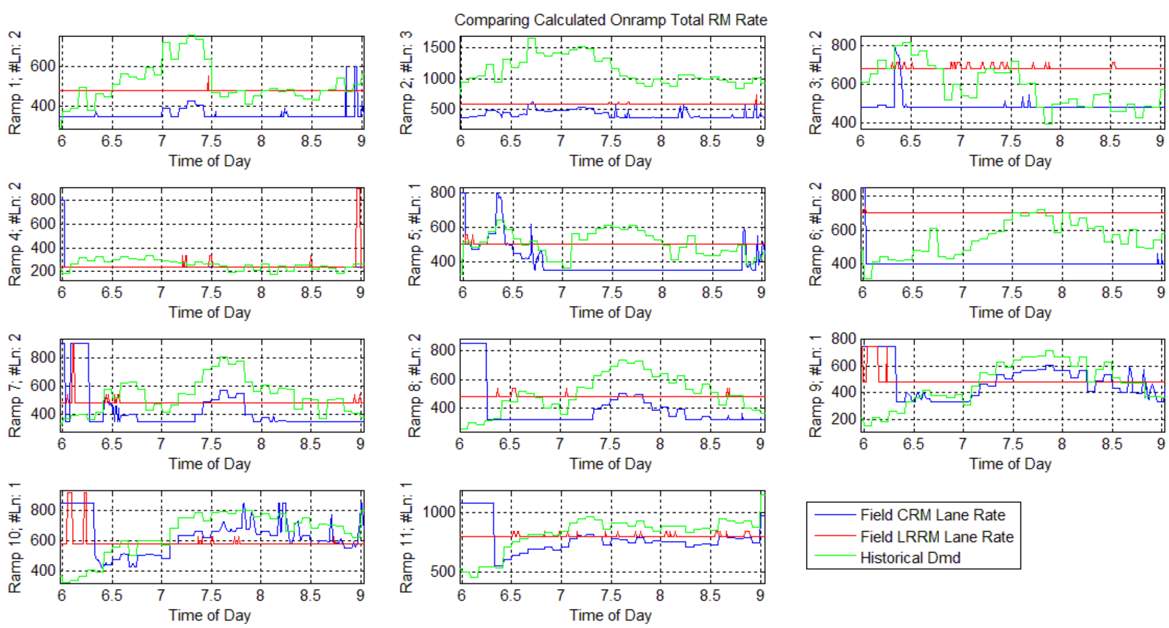

Figure A1-6. Comparison of LRRM and CRM rate for AM peak hours on 10/04/2016 Tuesday

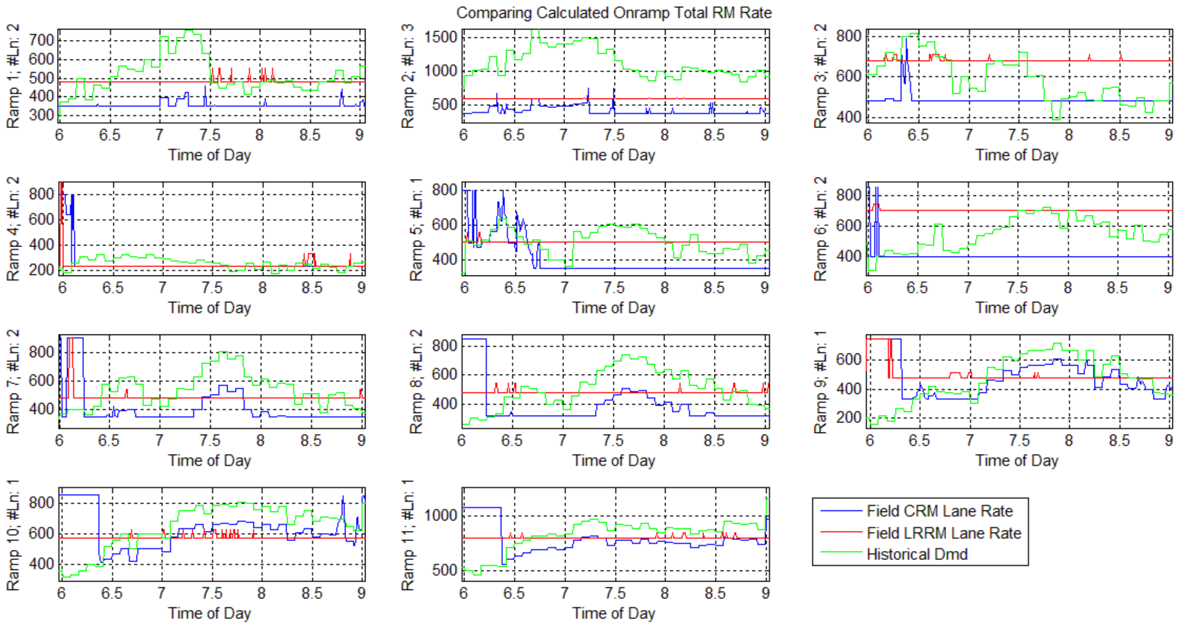

Figure A1-7. Comparison of LRRM and CRM rate for AM peak hours on 10/05/2016 Wednesday

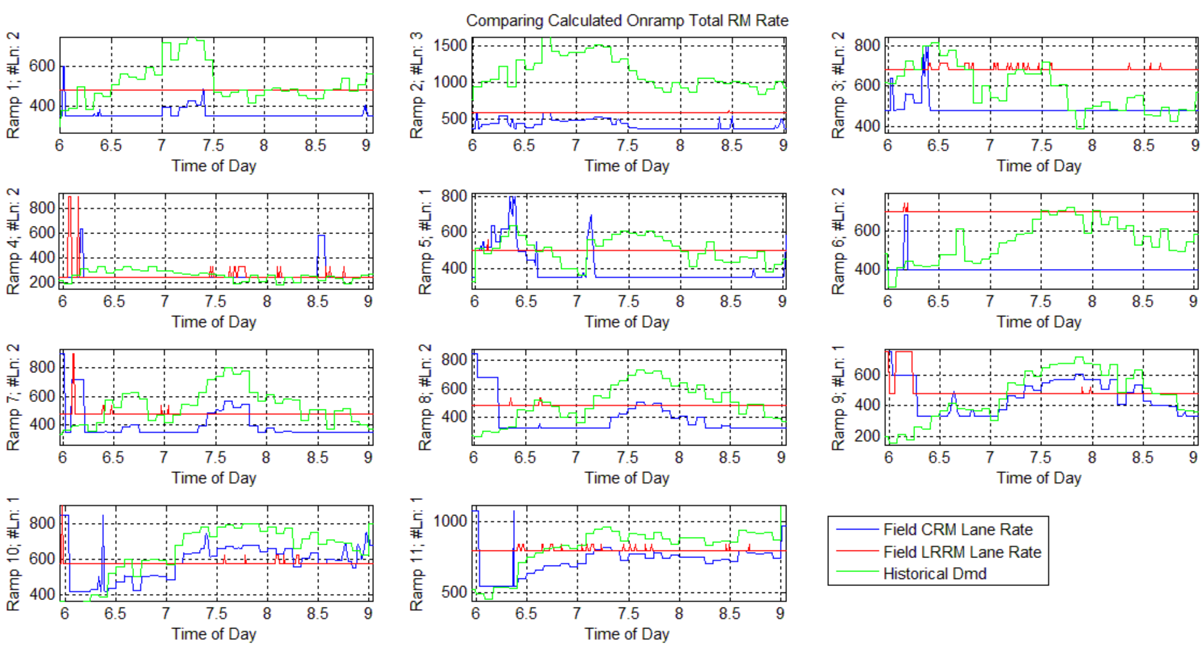

Figure A1-8. Comparison of LRRM and CRM rate for AM peak hours on 10/06/2016 Thursday

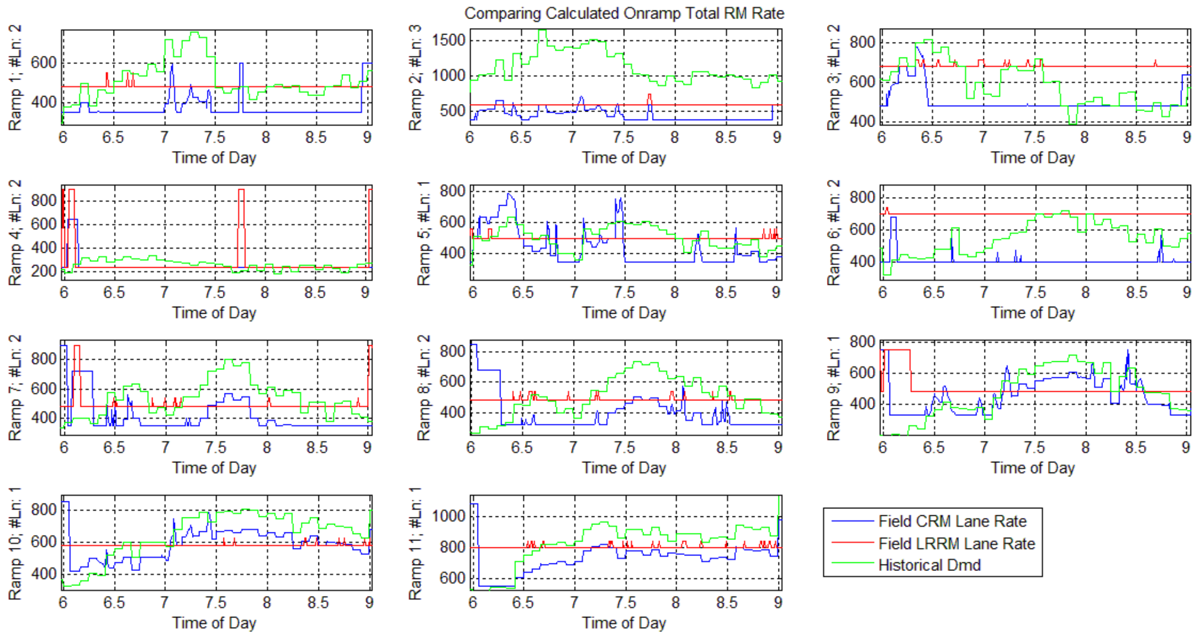

Figure A1-9. Comparison of LRRM and CRM rate for AM peak hours on 10/07/2016 Friday

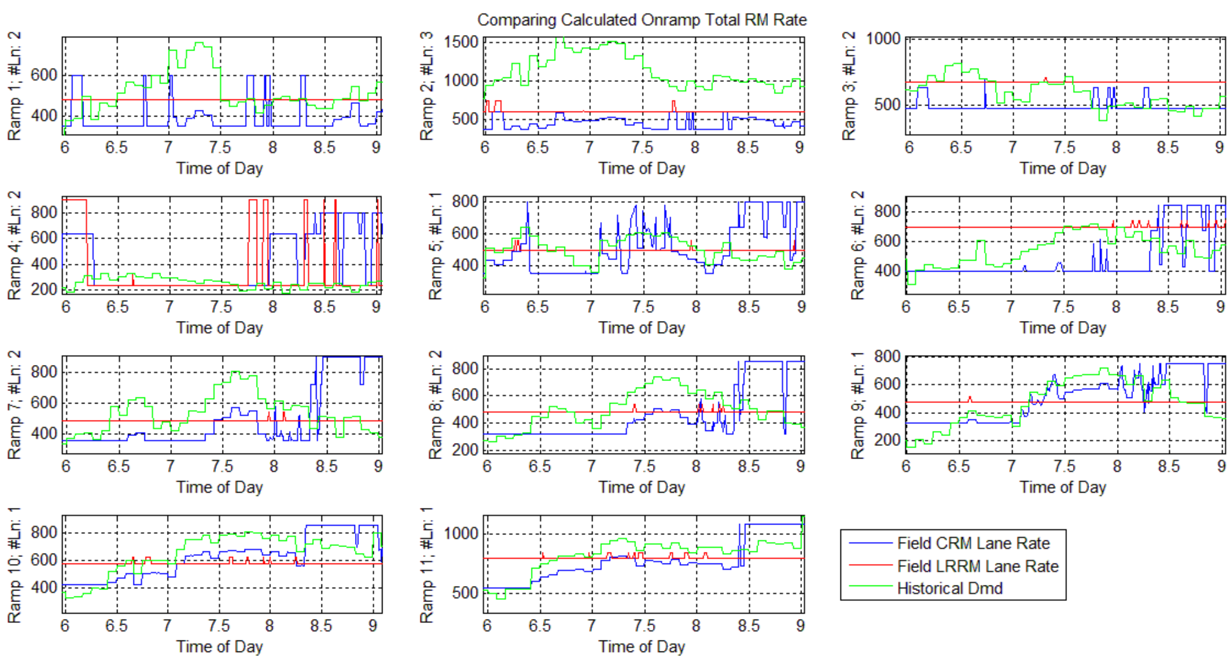

Figure A1-10. Comparison of LRRM and CRM rate for AM peak hours on 10/10/2016 Monday

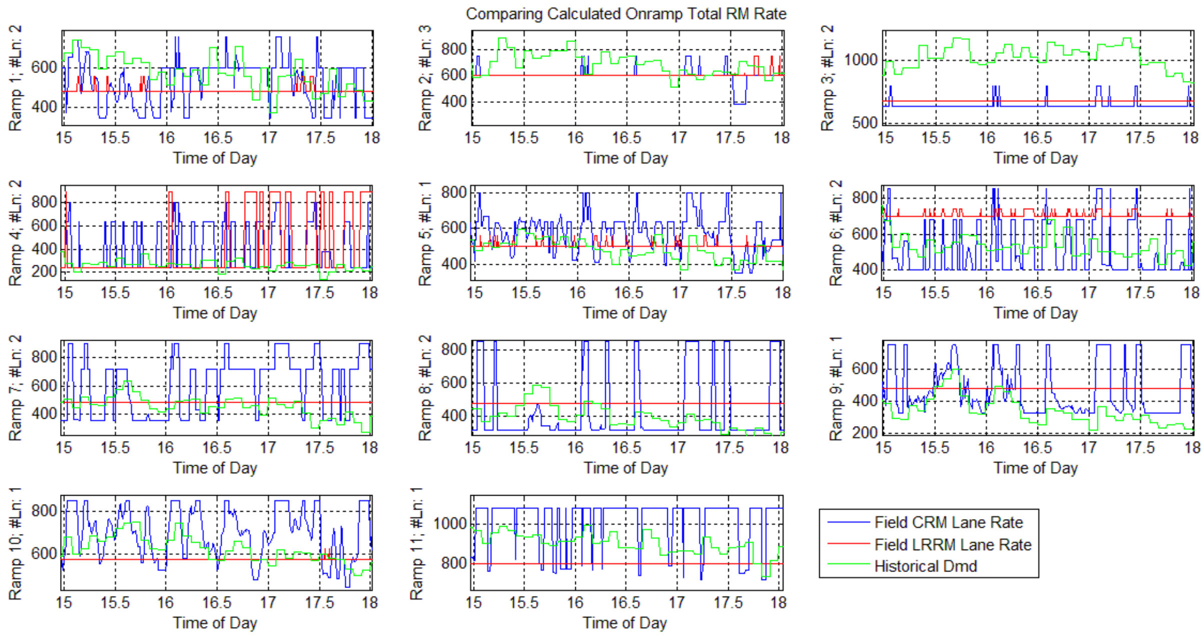

Figure A1-11. Comparison of LRRM and CRM rate for PM peak hours on 10/10/2016 Monday

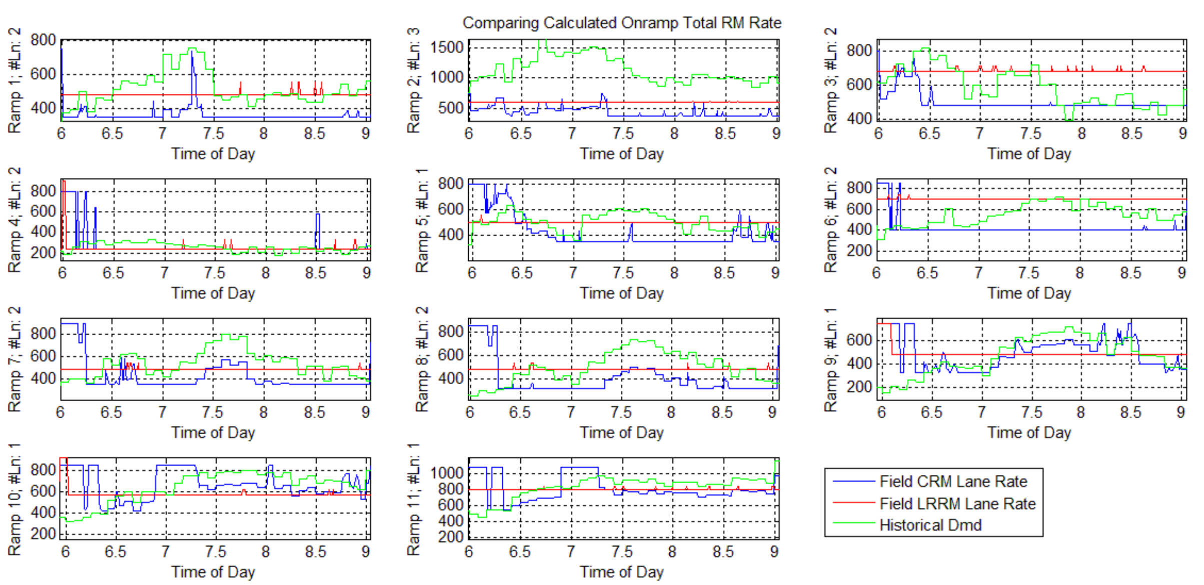

Figure A1-12. Comparison of LRRM and CRM rate for AM peak hours on 10/11/2016 Tuesday

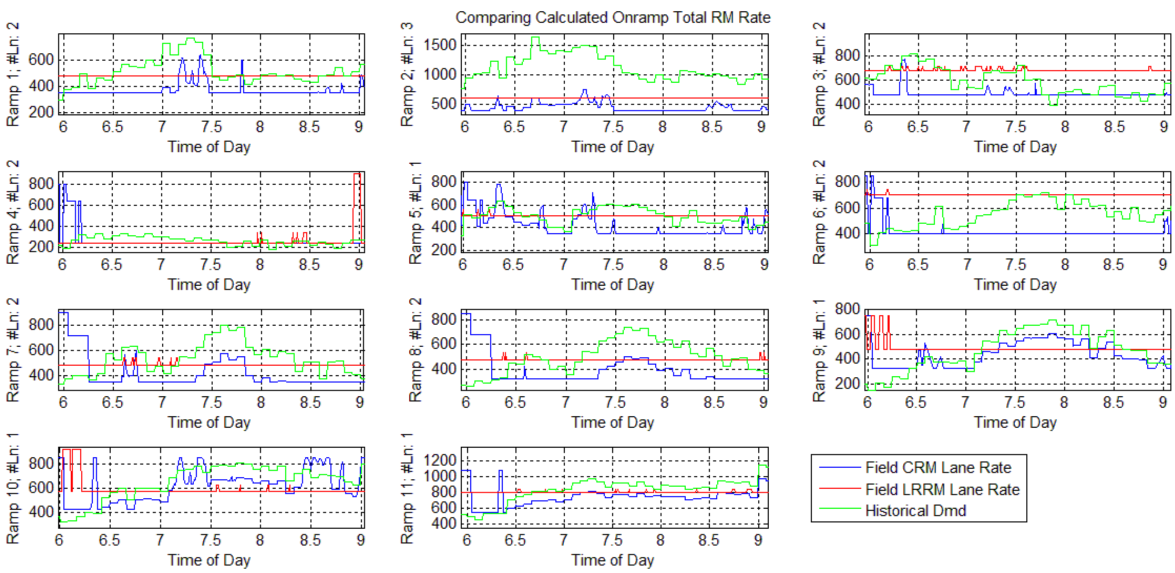

Figure A1-13. Comparison of LRRM and CRM rate for AM peak hours on 10/12/2016 Wednesday

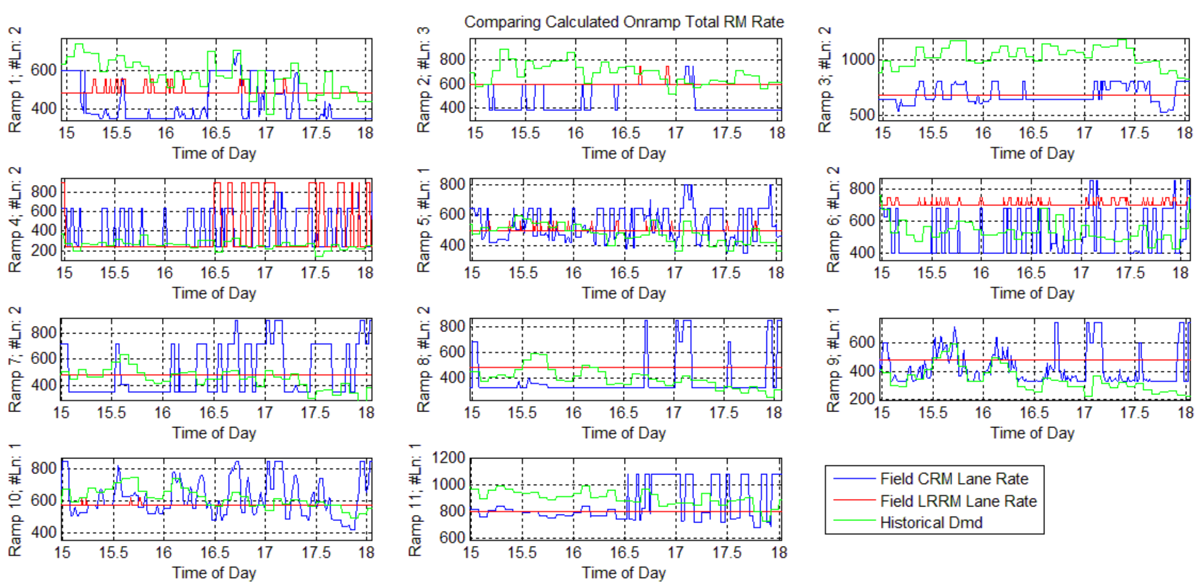

Figure A1-13. Comparison of LRRM and CRM rate for PM peak hours on 10/12/2016 Wednesday

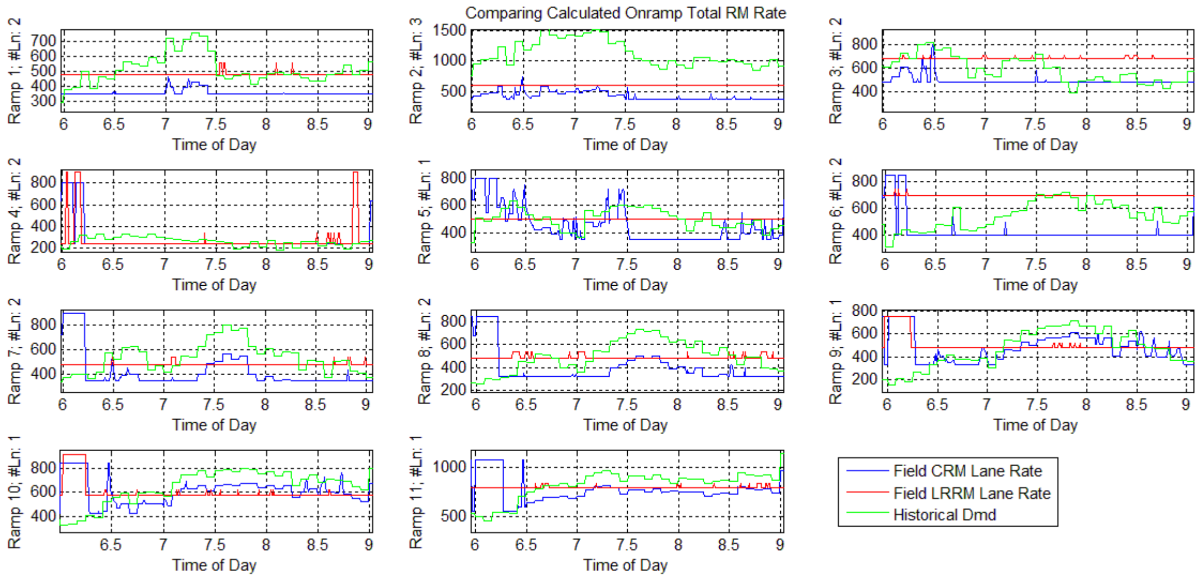

Figure A1-14. Comparison of LRRM and CRM rate for AM peak hours on 10/13/2016 Thursday

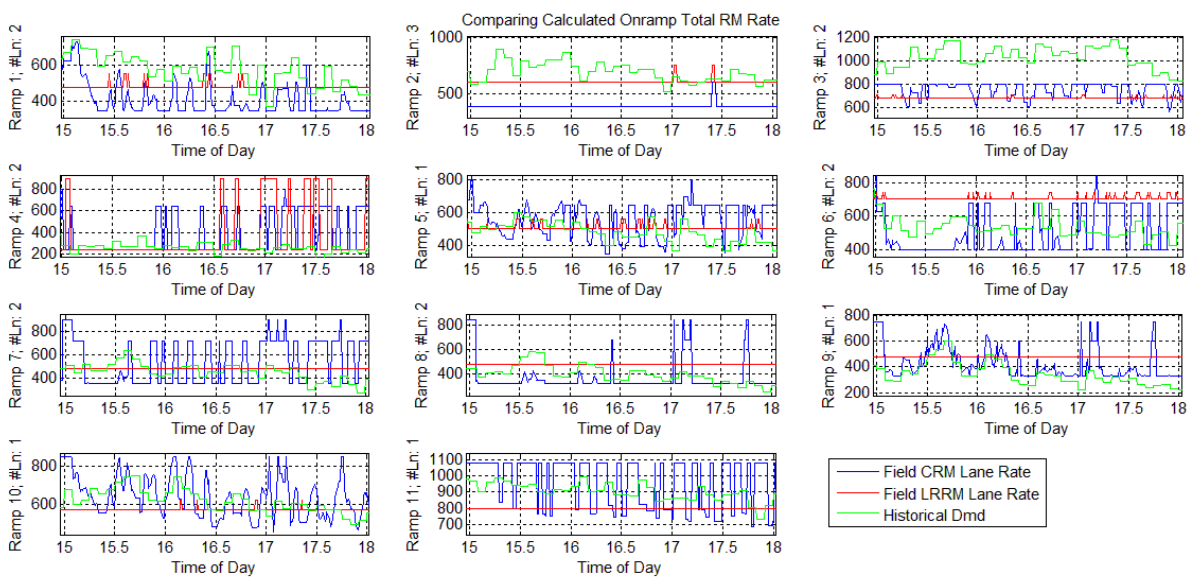

Figure A1-15. Comparison of LRRM and CRM rate for PM peak hours on 10/13/2016 Thursday

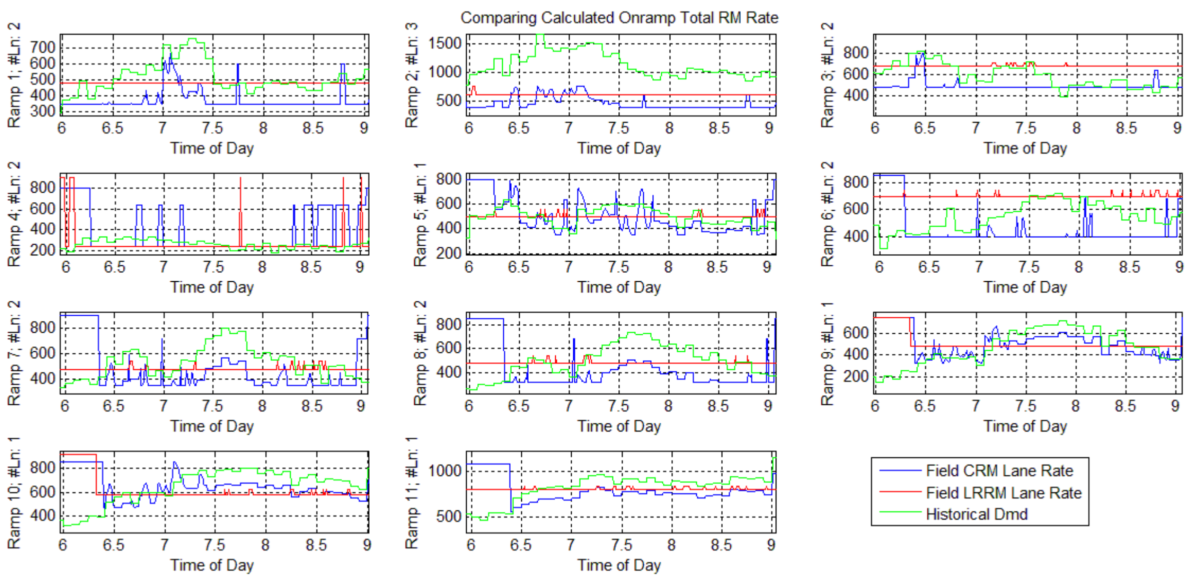

Figure A1-16. Comparison of LRRM and CRM rate for AM peak hours on 10/14/2016 Friday

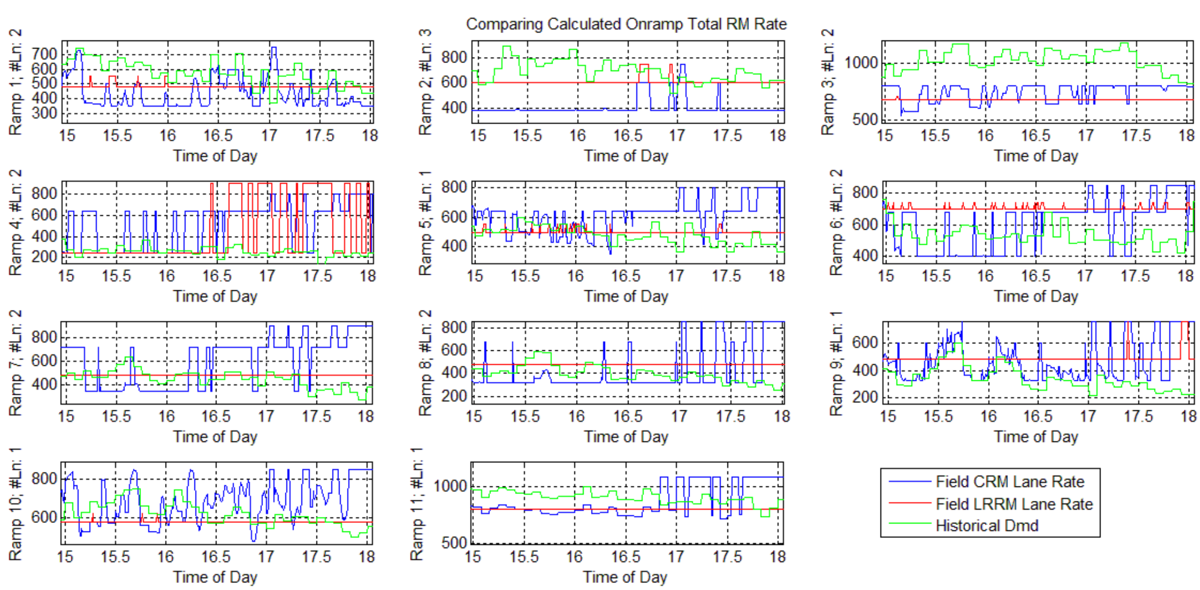

Figure A1-17. Comparison of LRRM and CRM rate for PM peak hours on 10/14/2016 Friday

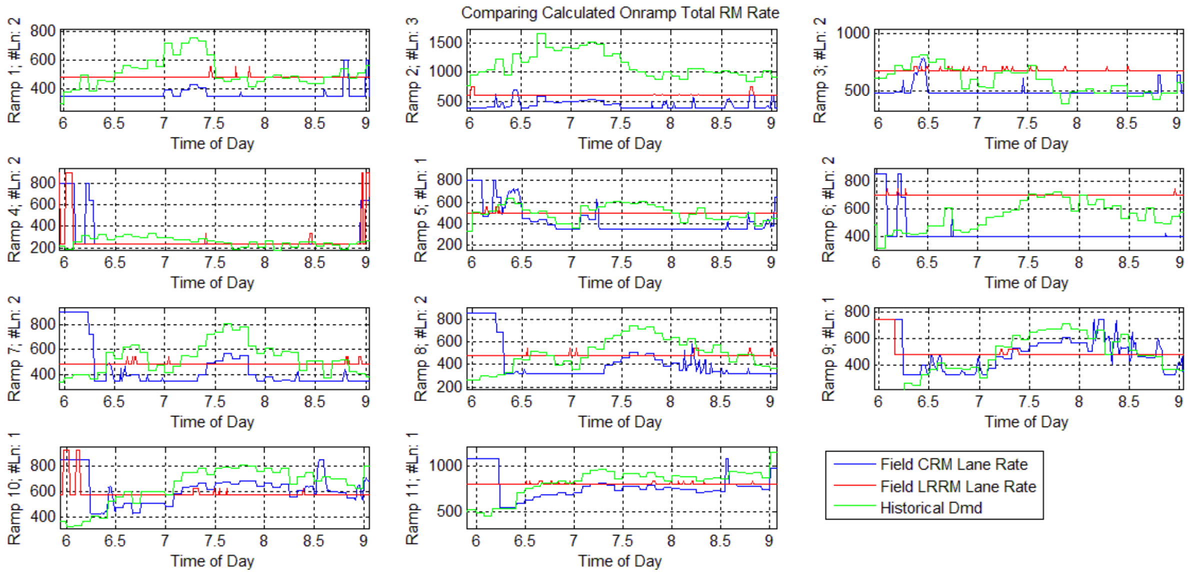

Figure A1-18. Comparison of LRRM and CRM rate for AM peak hours on 10/17/2016 Monday

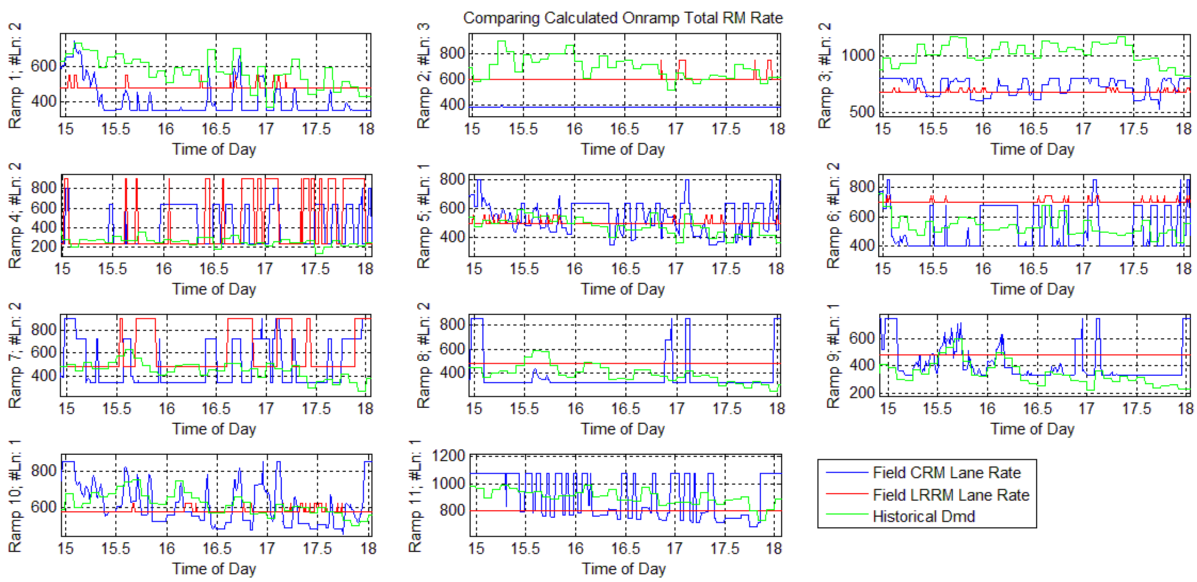

Figure A1-19. Comparison of LRRM and CRM rate for PM peak hours on 10/17/2016 Monday

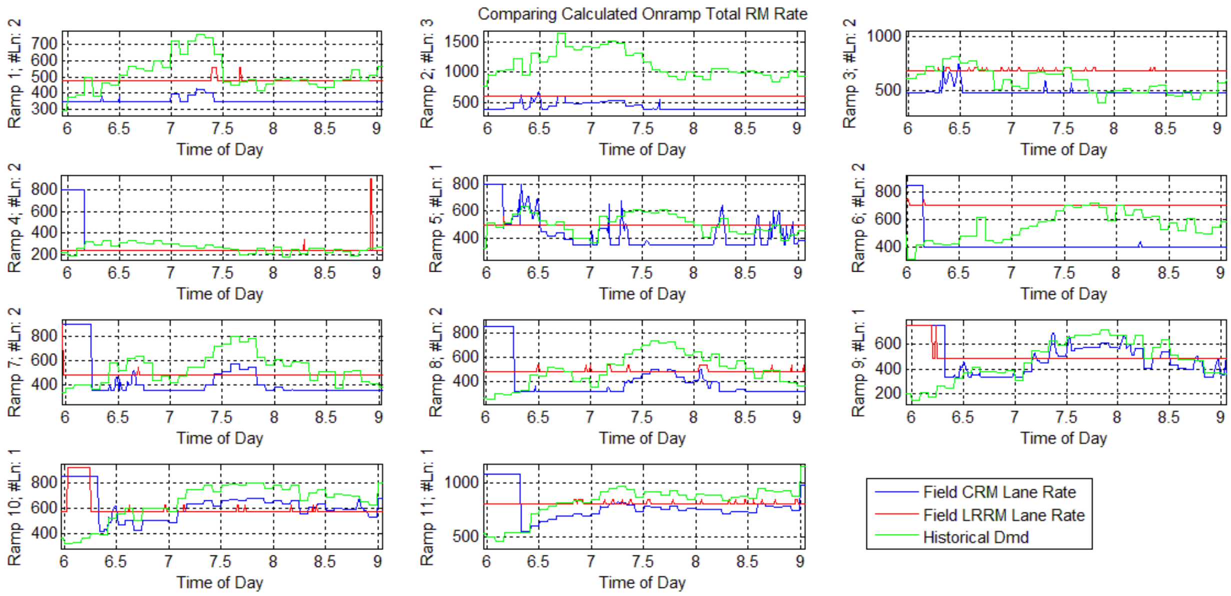

Figure A1-20. Comparison of LRRM and CRM rate for AM peak hours on 10/18/2016 Tuesday

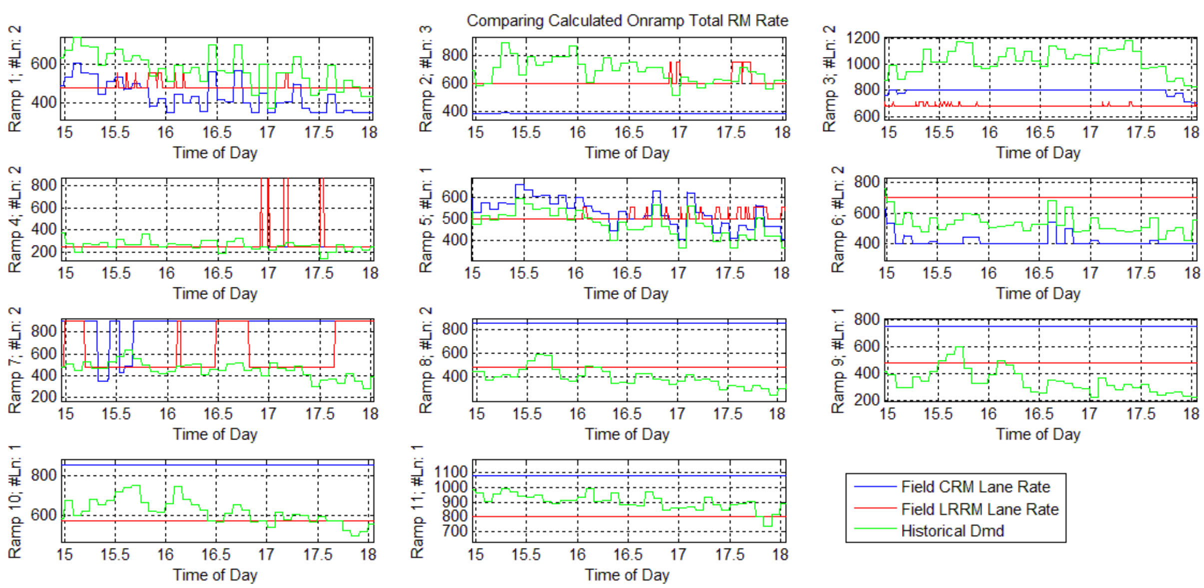

Figure A1-21. Comparison of LRRM and CRM rate for PM peak hours on 10/18/2016 Tuesday

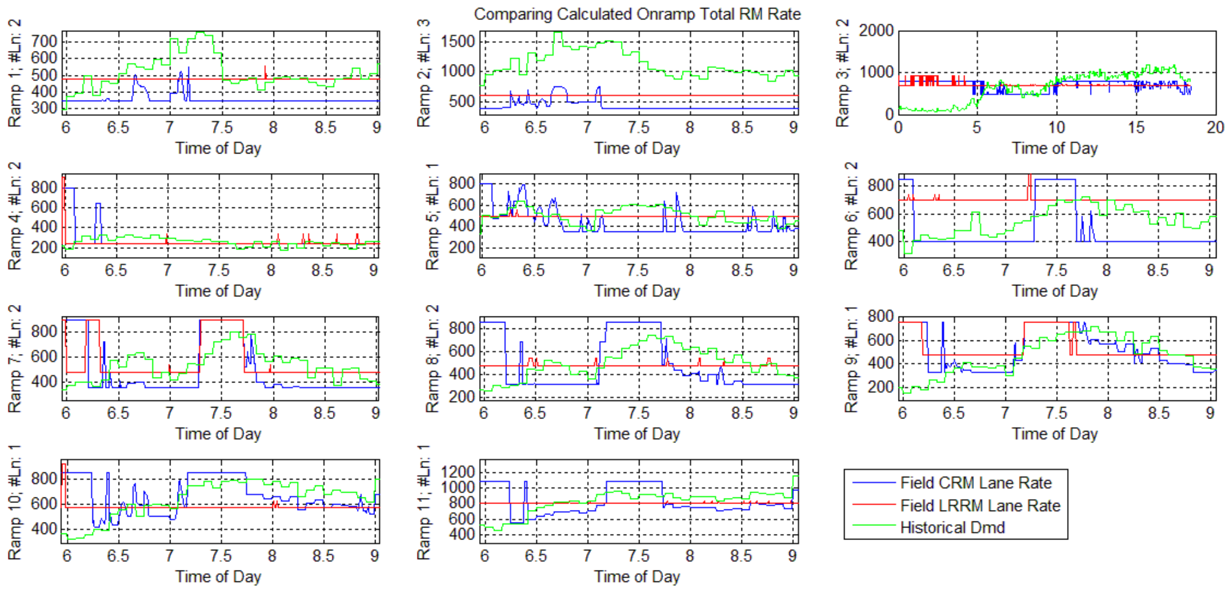

Figure A1-22. Comparison of LRRM and CRM rate for AM peak hours on 10/19/2016 Wednesday

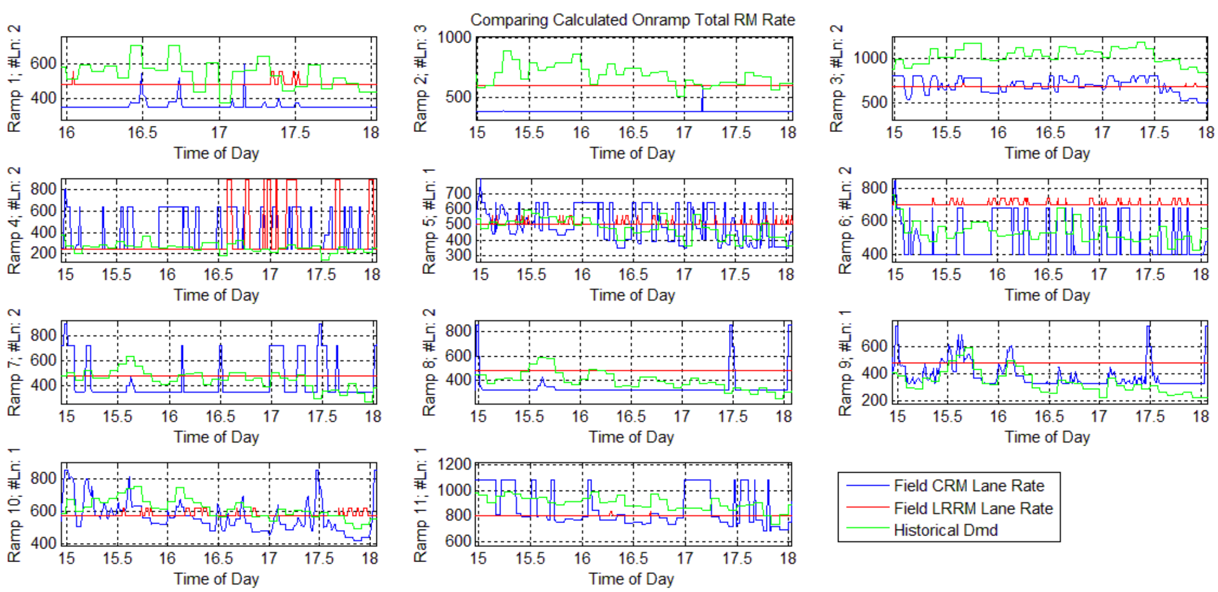

Figure A1-23. Comparison of LRRM and CRM rate for PM peak hours on 10/19/2016 Wednesday

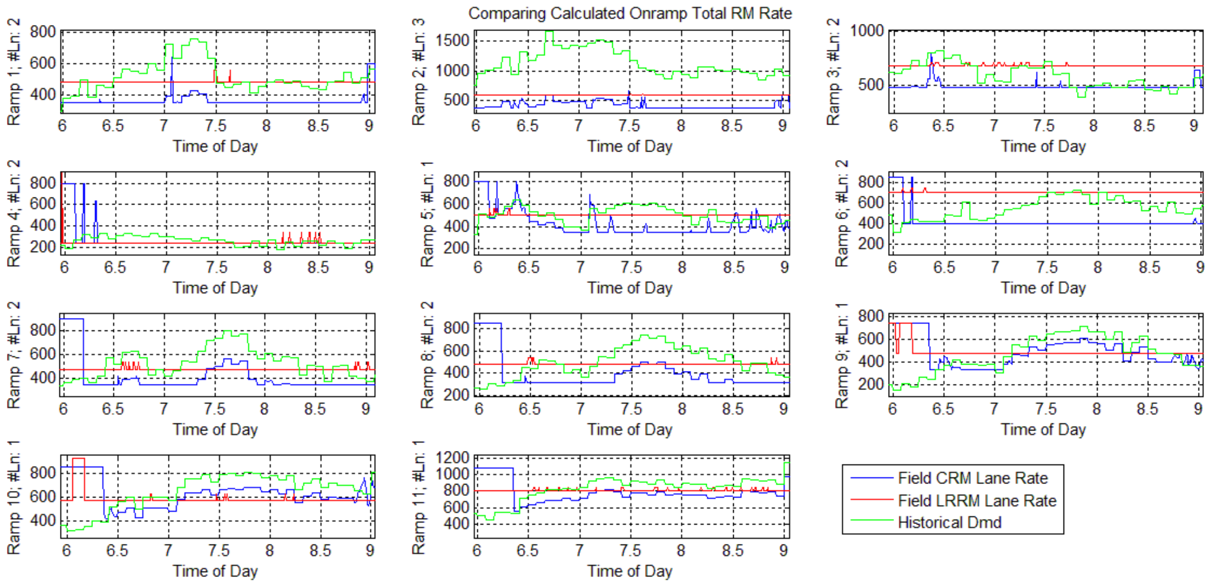

Figure A1-24. Comparison of LRRM and CRM rate for AM peak hours on 10/20/2016 Thursday

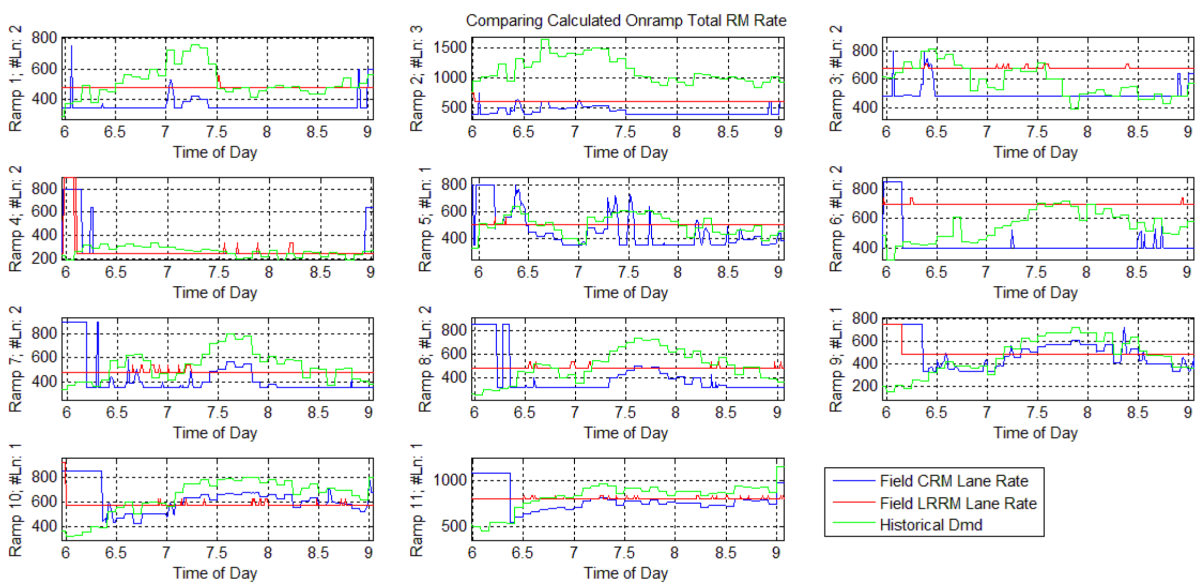

Figure A1-25. Comparison of LRRM and CRM rate for AM peak hours on 10/21/2016 Friday

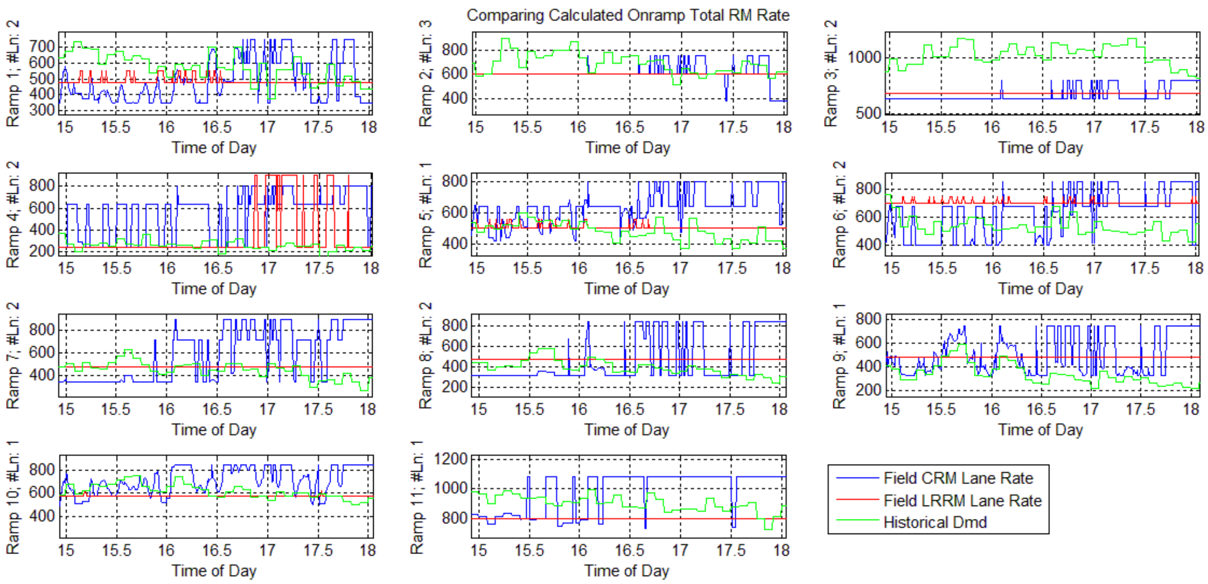

Figure A1-26. Comparison of LRRM and CRM rate for PM peak hours on 10/21/2016 Friday

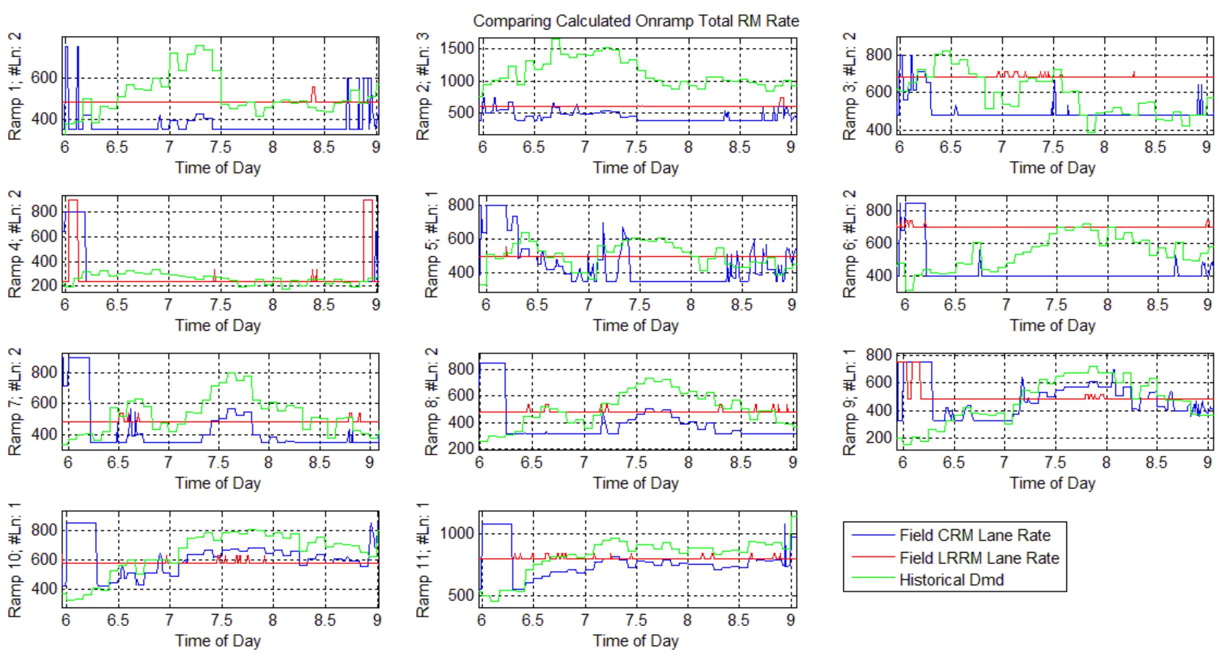

Figure A1-27. Comparison of LRRM and CRM rate for AM peak hours on 10/24/2016 Monday

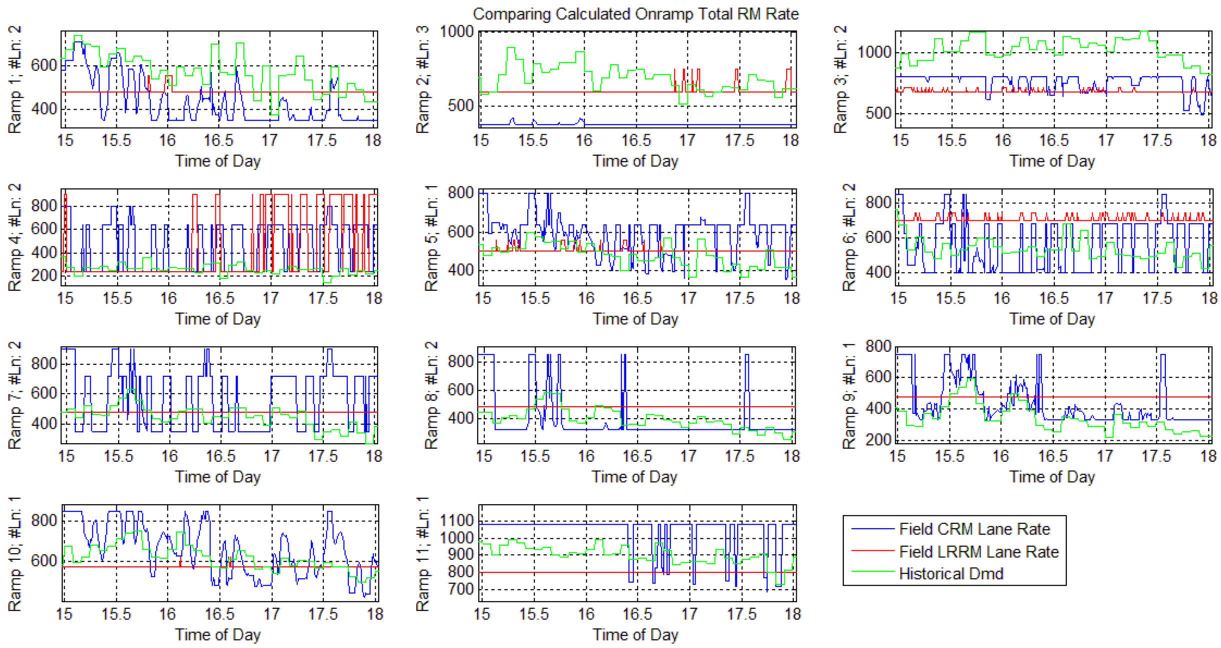

Figure A1-28. Comparison of LRRM and CRM rate for PM peak hours on 10/24/2016 Monday

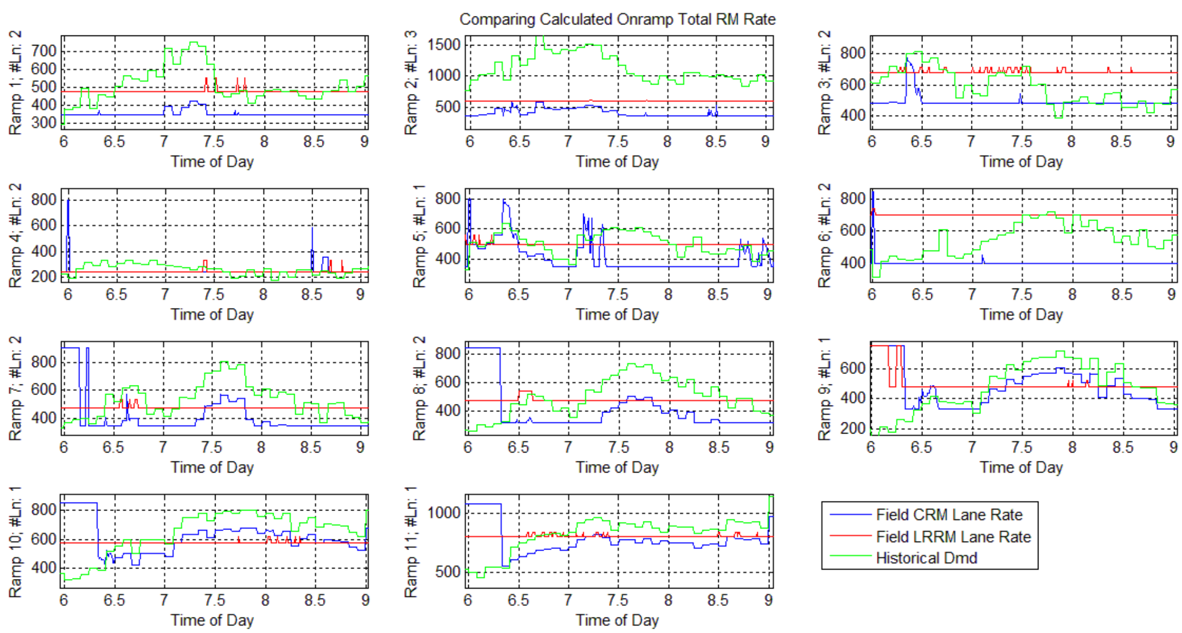

Figure A1-29. Comparison of LRRM and CRM rate for AM peak hours on 10/25/2016 Tuesday

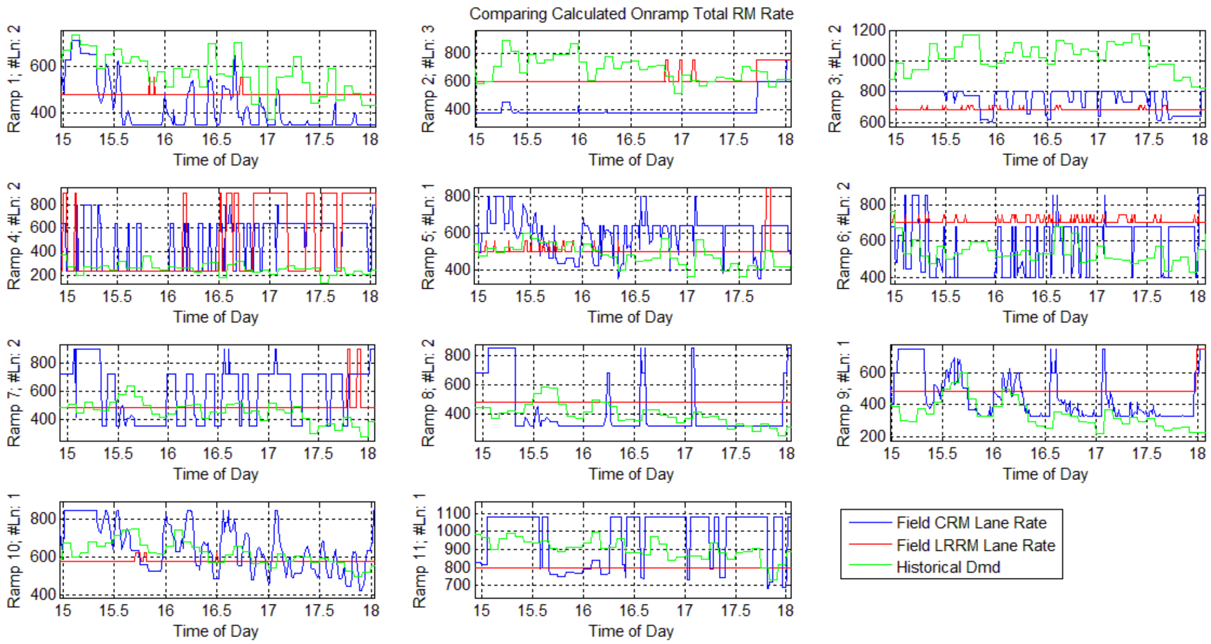

Figure A1-30. Comparison of LRRM and CRM rate for PM peak hours on 10/25/2016 Tuesday

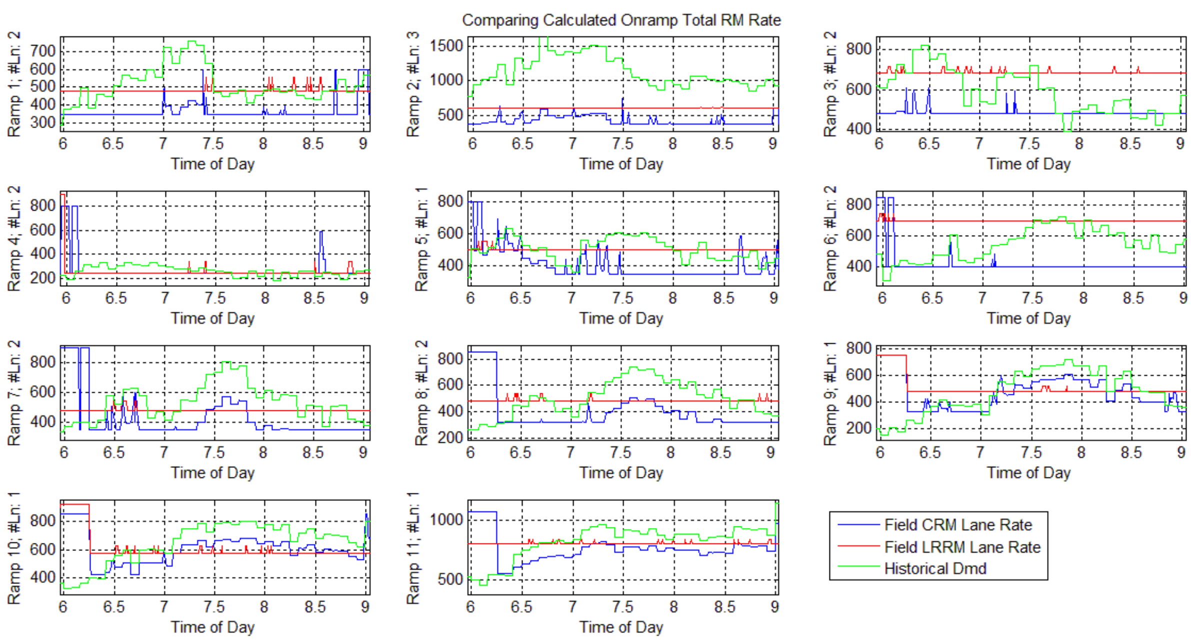

Figure A1-31. Comparison of LRRM and CRM rate for AM peak hours on 10/26/2016 Wednesday

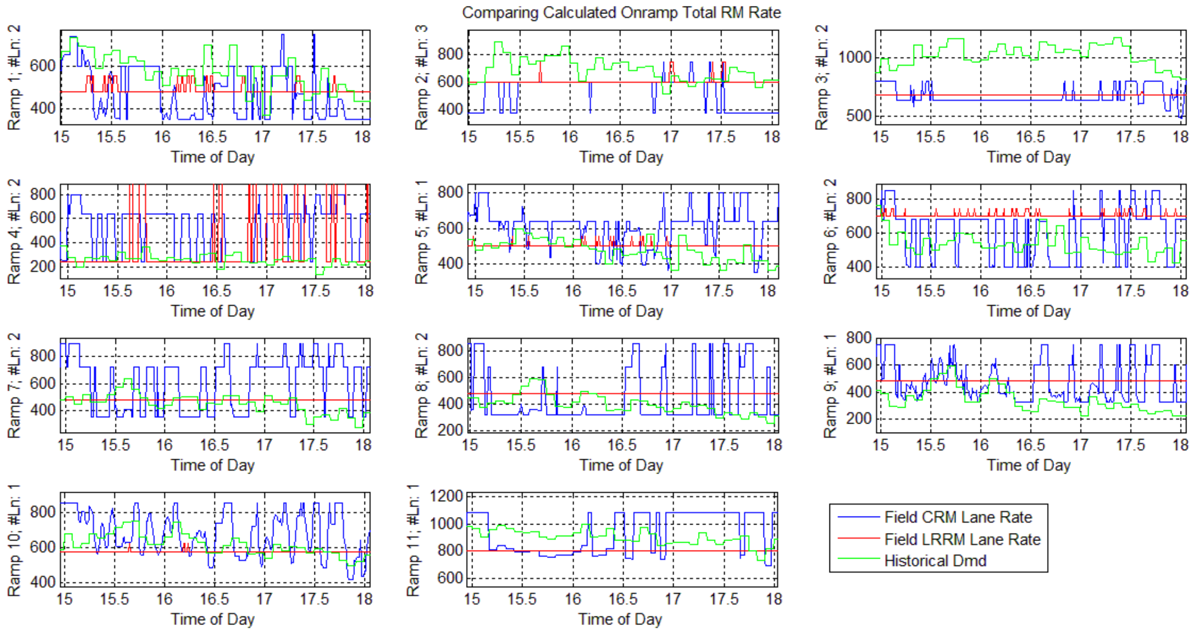

Figure A1-32. Comparison of LRRM and CRM rate for PM peak hours on 10/26/2016 Wednesday

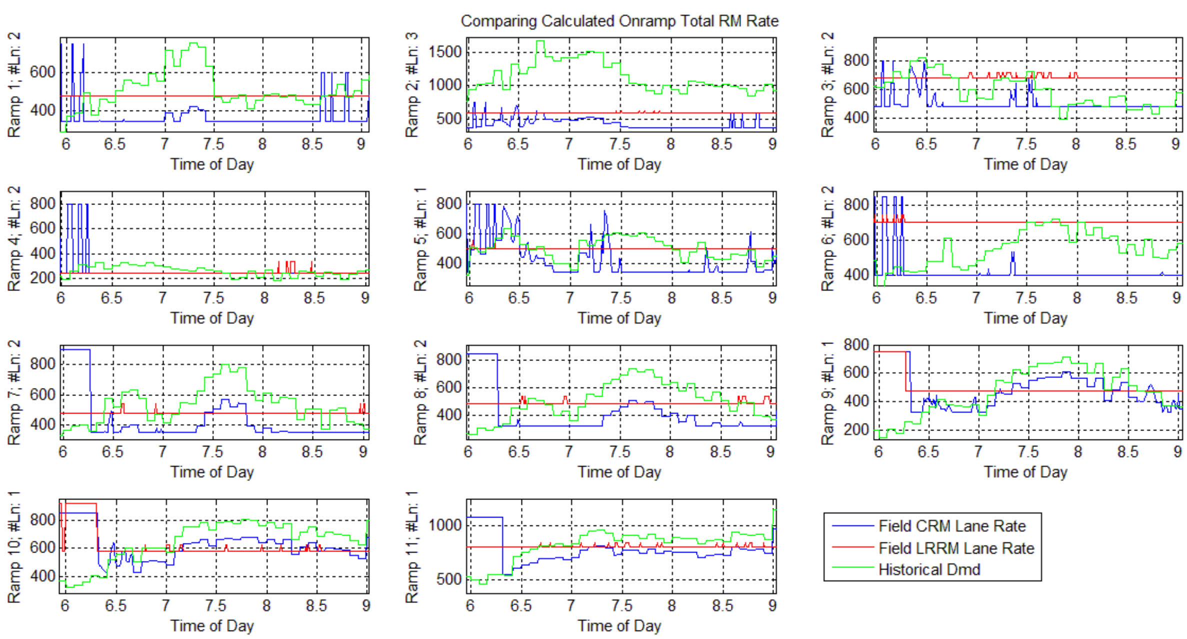

Figure A1-33. Comparison of LRRM and CRM rate for AM peak hours on 10/27/2016 Thursday

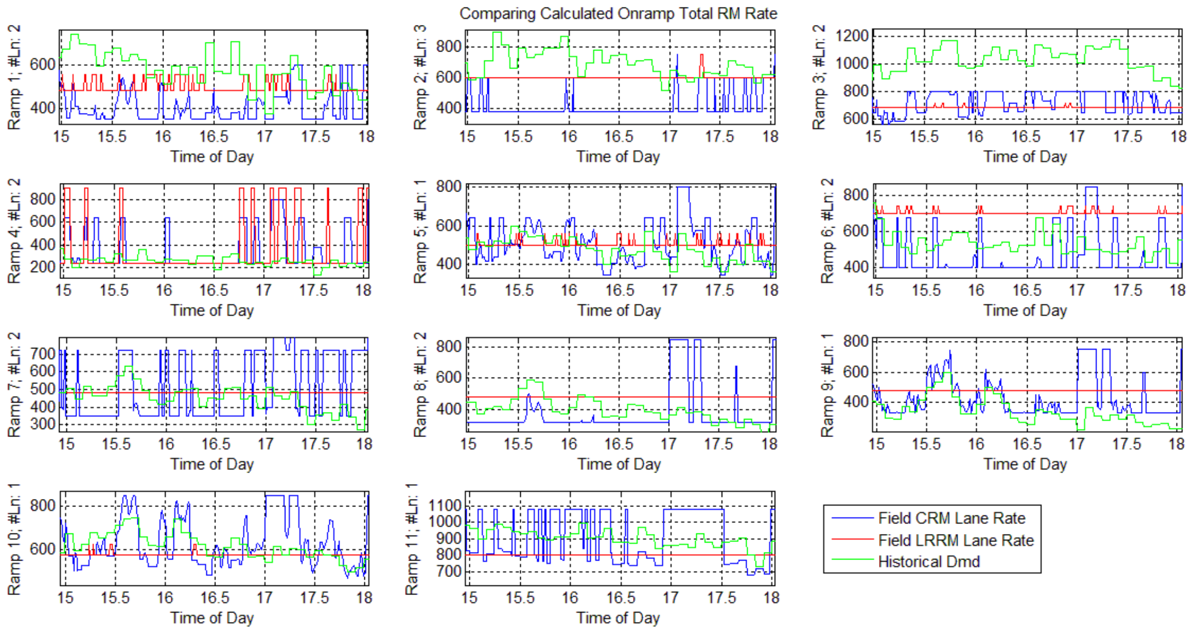

Figure A1-34. Comparison of LRRM and CRM rate for PM peak hours on 10/27/2016 Thursday

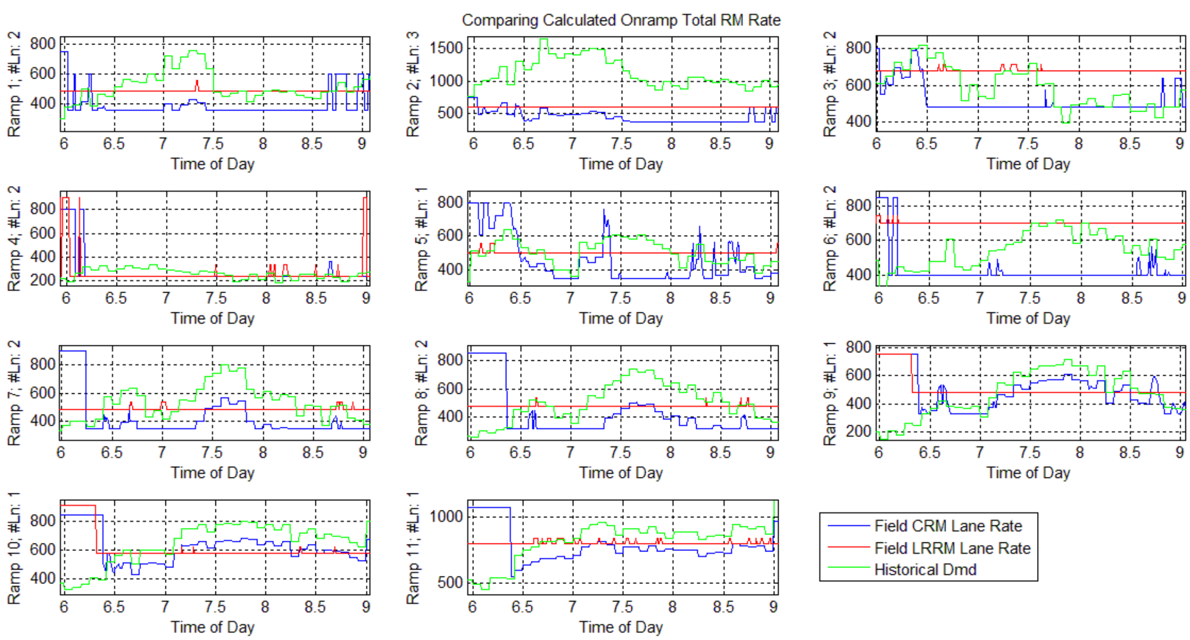

Figure A1-35. Comparison of LRRM and CRM rate for AM peak hours on 10/28/2016 Friday

Appendix 2. Traffic Data Analysis for Performance Analysis

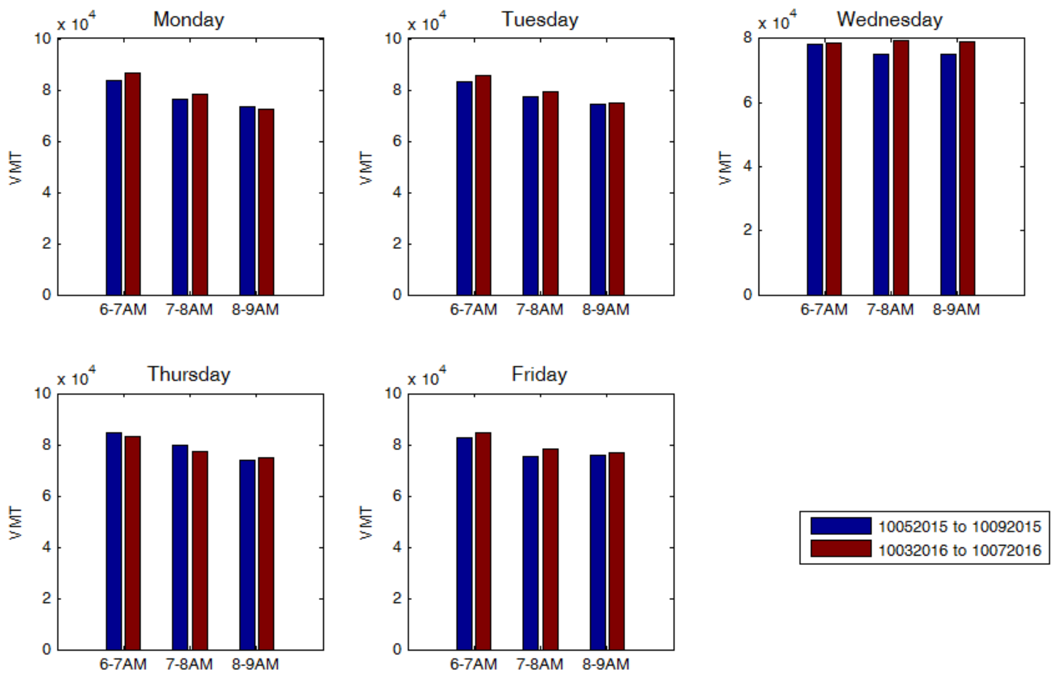

Figure A2-1. October Week1 AM VMT

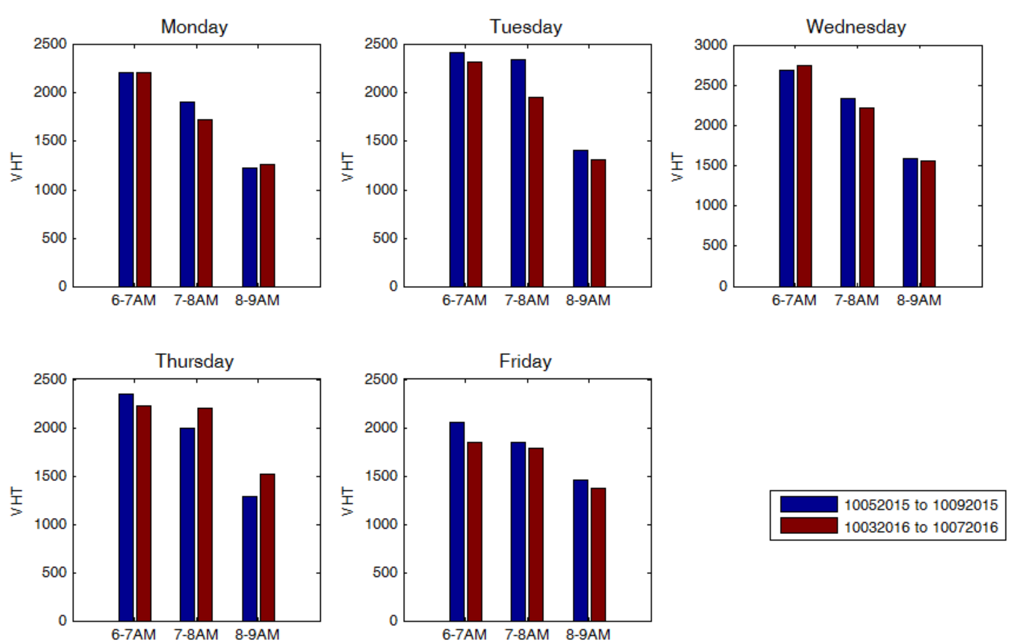

Figure A2-2. October Week1 AM VHT

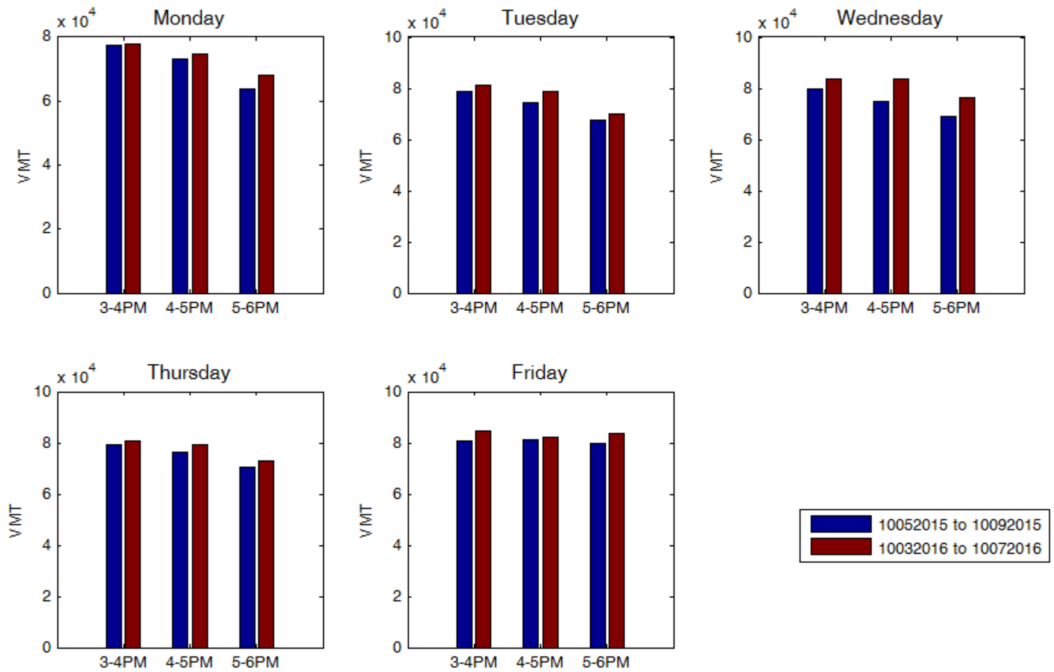

Figure A2-3. October Week1 PM VMT

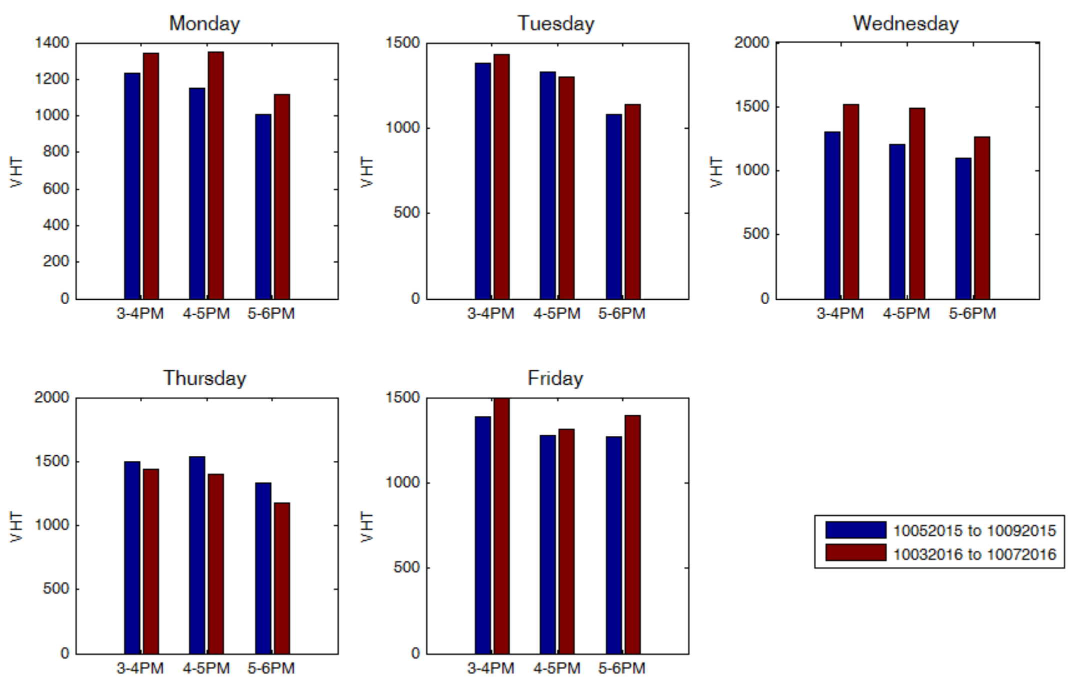

Figure A2-4. October Week1 PM VHT

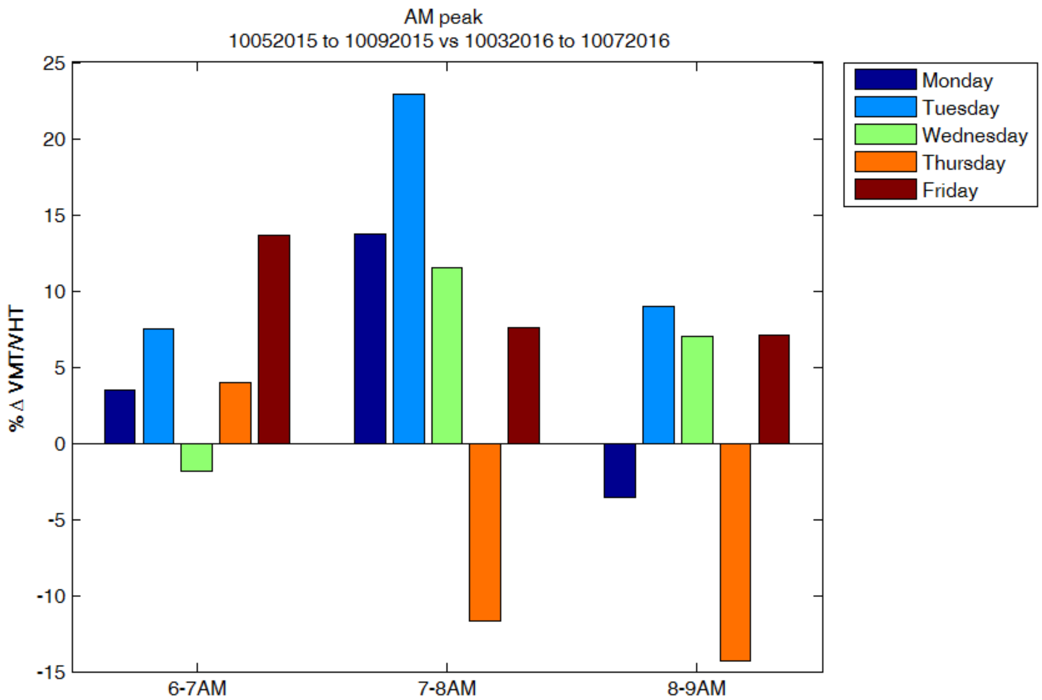

Figure A2-5. October Week1 AM %ΔQ

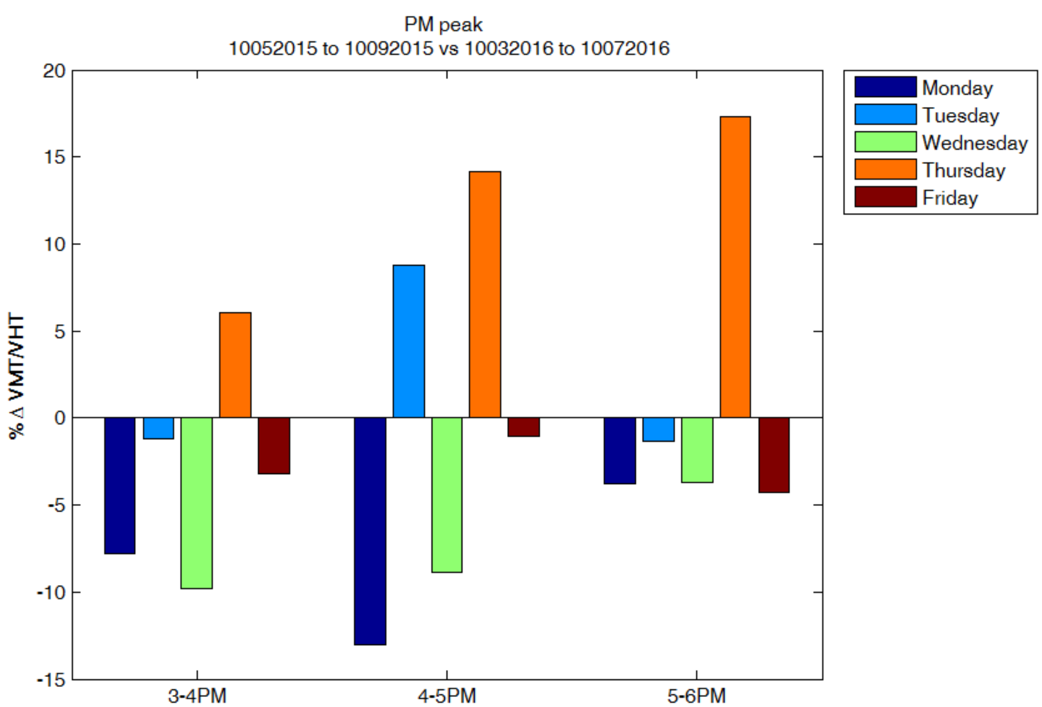

Figure A2-6. October Week1 PM %ΔQ

Table A2-1. October Week1 data

Monday

Table: 10052015 vs 10032016 AM peak performance comparison

Morning peak	6-7AM	7-8AM	8-9AM	Avg.	Std.
10052015 VMT	83861.70	76134.50	73336.90	77777.70	5451.42
10052015 VHT	2218.80	1905.90	1231.00	1785.23	504.83
10052015 VMT/VHT	37.80	39.95	59.58	45.77	12.00
10032016 VMT	86441.70	78207.60	72425.30	79024.87	7043.85
10032016 VHT	2209.50	1721.70	1261.00	1730.73	474.31
10032016 VMT/VHT	39.12	45.42	57.43	47.33	9.30
%improvement	3.510%	13.713%	-3.593%	4.544%	8.699%

Table: 10052015 vs 10032016 PM peak performance comparison

Evening peak	3-4PM	4-5PM	5-6PM	Avg.	Std.
10052015 VMT	77599.70	73152.80	63881.60	71544.70	6999.00
10052015 VHT	1234.10	1151.40	1005.20	1130.23	115.91
10052015 VMT/VHT	62.88	63.53	63.55	63.32	0.38
10032016 VMT	78016.30	74691.10	68274.10	73660.50	4952.19
10032016 VHT	1345.20	1351.90	1116.80	1271.30	133.84
10032016 VMT/VHT	58.00	55.25	61.13	58.13	2.94
%improvement	-7.766%	-13.040%	-3.804%	-8.203%	4.634%

Tuesday

Table: 10062015 vs 10042016 AM peak performance comparison

Morning peak	6-7AM	7-8AM	8-9AM	Avg.	Std.
10062015 VMT	83160.20	77466.70	74353.80	78326.90	4465.77
10062015 VHT	2415.40	2347.40	1411.50	2058.10	561.00
10062015 VMT/VHT	34.43	33.00	52.68	40.04	10.97
10042016 VMT	85742.40	79395.00	74988.70	80042.03	5405.97
10042016 VHT	2317.30	1957.90	1306.60	1860.60	512.33
10042016 VMT/VHT	37.00	40.55	57.39	44.98	10.89
%improvement	7.470%	22.878%	8.951%	13.100%	8.501%

Table: 10062015 vs 10042016 PM peak performance comparison

Evening peak	3-4PM	4-5PM	5-6PM	Avg.	Std.
10062015 VMT	78929.80	74247.40	67367.20	73514.80	5816.01
10062015 VHT	1377.90	1331.70	1082.60	1264.07	158.84
10062015 VMT/VHT	57.28	55.75	62.23	58.42	3.38
10042016 VMT	81025.20	78604.30	69792.60	76474.03	5911.54
10042016 VHT	1431.70	1296.30	1136.60	1288.20	147.72
10042016 VMT/VHT	56.59	60.64	61.40	59.55	2.58
%improvement	-1.203%	8.759%	-1.322%	2.078%	5.786%

Wednesday

Table: 10072015 vs 10052016 AM peak performance comparison

Morning peak	6-7AM	7-8AM	8-9AM	Avg.	Std.
10072015 VMT	78343.10	74938.50	75305.60	76195.73	1868.71
10072015 VHT	2679.90	2335.40	1592.20	2202.50	555.90
10072015 VMT/VHT	29.23	32.09	47.30	36.21	9.71
10052016 VMT	78659.30	79277.40	78918.20	78951.63	310.40
10052016 VHT	2741.00	2216.10	1559.20	2172.10	592.13
10052016 VMT/VHT	28.70	35.77	50.61	38.36	11.19
%improvement	-1.835%	11.485%	7.015%	5.555%	6.779%

Table: 10072015 vs 10052016 PM peak performance comparison

Evening peak	3-4PM	4-5PM	5-6PM	Avg.	Std.
10072015 VMT	79589.80	74640.80	69159.50	74463.37	5217.41
10072015 VHT	1301.80	1208.80	1101.80	1204.13	100.08
10072015 VMT/VHT	61.14	61.75	62.77	61.89	0.82
10052016 VMT	83424.30	83749.60	76525.60	81233.17	4080.12
10052016 VHT	1512.50	1488.00	1265.80	1422.10	135.91
10052016 VMT/VHT	55.16	56.28	60.46	57.30	2.79
%improvement	-9.784%	-8.850%	-3.685%	-7.440%	3.285%

Thursday

Table: 10082015 vs 10062016 AM peak performance comparison

Morning peak	6-7AM	7-8AM	8-9AM	Avg.	Std.
10082015 VMT	84586.90	79645.30	73933.40	79388.53	5331.39
10082015 VHT	2350.50	1997.10	1288.50	1878.70	540.81
10082015 VMT/VHT	35.99	39.88	57.38	44.42	11.39
10062016 VMT	83302.20	77558.40	74894.00	78584.87	4297.06
10062016 VHT	2227.10	2202.40	1523.50	1984.33	399.28
10062016 VMT/VHT	37.40	35.22	49.16	40.59	7.50
%improvement	3.938%	-11.698%	-14.326%	-7.362%	9.874%

Table: 10082015 vs 10062016 PM peak performance comparison

Evening peak	3-4PM	4-5PM	5-6PM	Avg.	Std.
10082015 VMT	79318.60	76174.80	70271.50	75254.97	4593.16
10082015 VHT	1492.70	1537.60	1328.00	1452.77	110.36
10082015 VMT/VHT	53.14	49.54	52.92	51.86	2.02
10062016 VMT	80973.10	79361.90	73167.80	77834.27	4120.79
10062016 VHT	1436.50	1403.40	1178.90	1339.60	140.15
10062016 VMT/VHT	56.37	56.55	62.06	58.33	3.24
%improvement	6.080%	14.147%	17.290%	12.506%	5.783%

Friday

Table: 10092015 vs 10072016 AM peak performance comparison

Morning peak	6-7AM	7-8AM	8-9AM	Avg.	Std.
10092015 VMT	82826.10	75473.70	75991.60	78097.13	4103.58
10092015 VHT	2058.30	1852.00	1458.60	1789.63	304.68
10092015 VMT/VHT	40.24	40.75	52.10	44.36	6.70
10072016 VMT	84583.10	78114.40	76637.20	79778.23	4226.18
10072016 VHT	1850.20	1782.00	1373.10	1668.43	258.03
10072016 VMT/VHT	45.72	43.84	55.81	48.45	6.44
%improvement	13.607%	7.564%	7.129%	9.434%	3.621%

Table: 10092015 vs 10072016 PM peak performance comparison

Evening peak	3-4PM	4-5PM	5-6PM	Avg.	Std.
10092015 VMT	80956.20	81037.00	79938.10	80643.77	612.46
10092015 VHT	1384.50	1280.40	1270.90	1311.93	63.02
10092015 VMT/VHT	58.47	63.29	62.90	61.55	2.68
10072016 VMT	84752.30	82372.00	83740.30	83621.53	1194.59
10072016 VHT	1497.80	1314.90	1390.80	1401.17	91.89
10072016 VMT/VHT	56.58	62.65	60.21	59.81	3.05
%improvement	-3.230%	-1.020%	-4.275%	-2.841%	1.662%

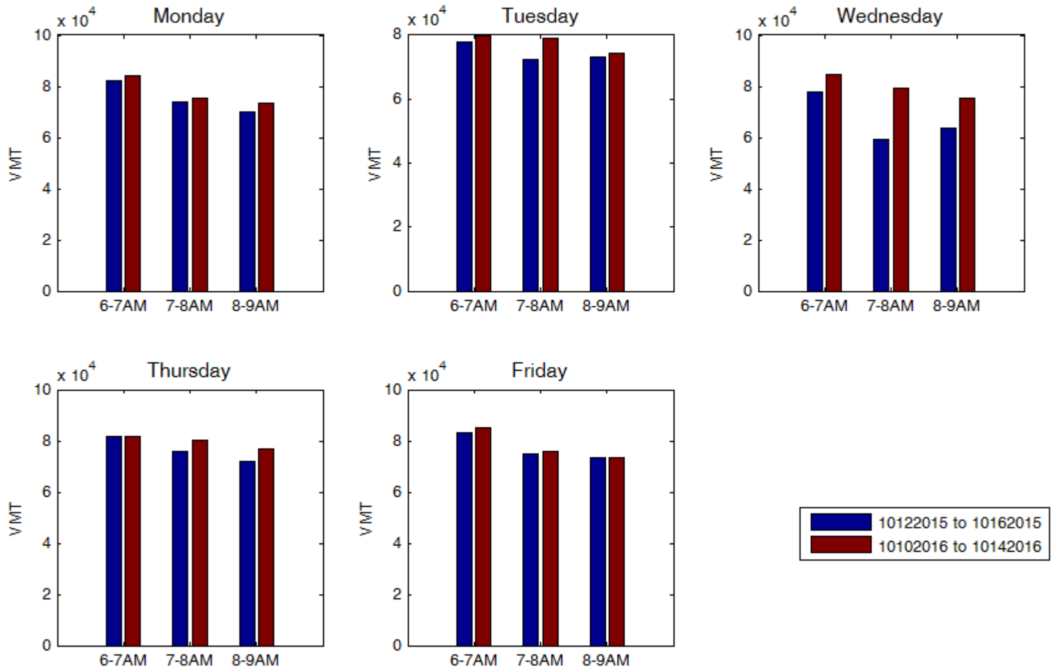

Figure A2-7. October Week2 AM VMT

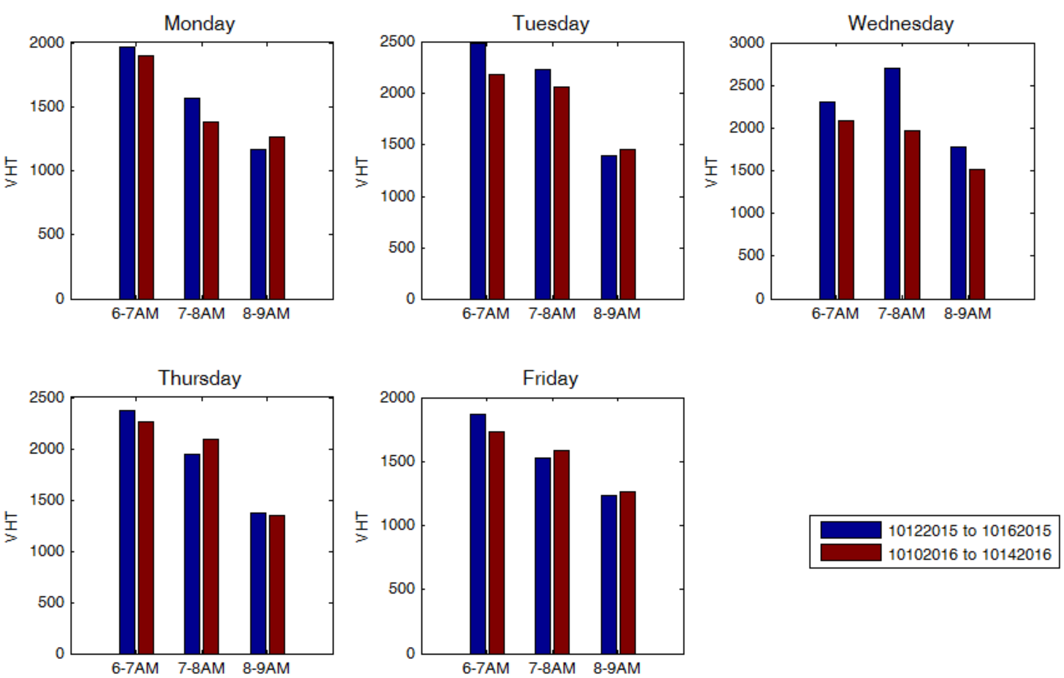

Figure A2-8. October Week2 AM VHT

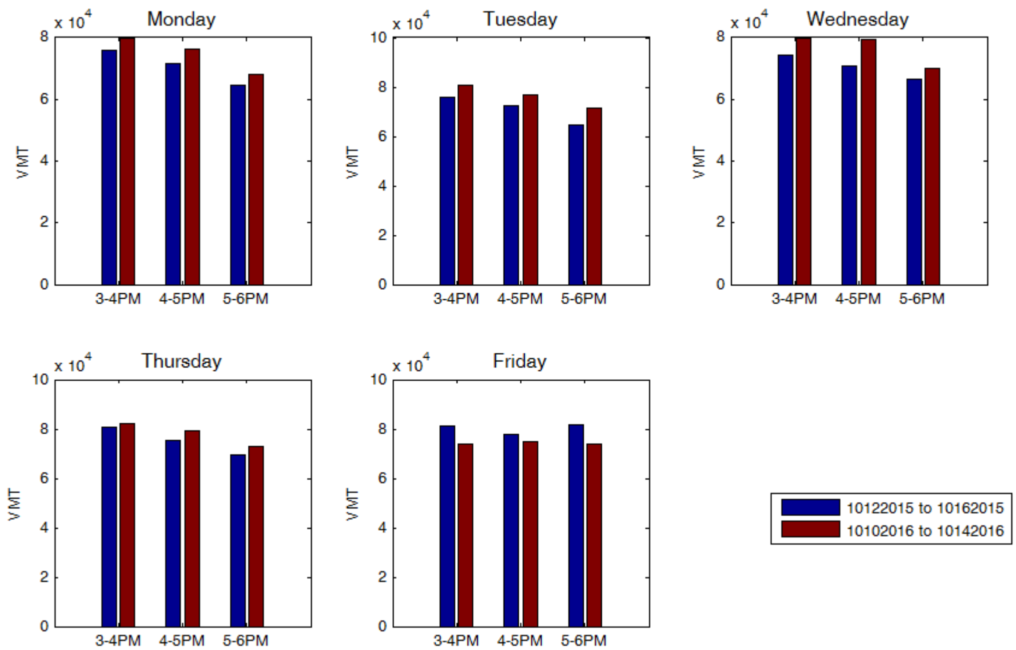

Figure A2-9. October Week2 PM VMT

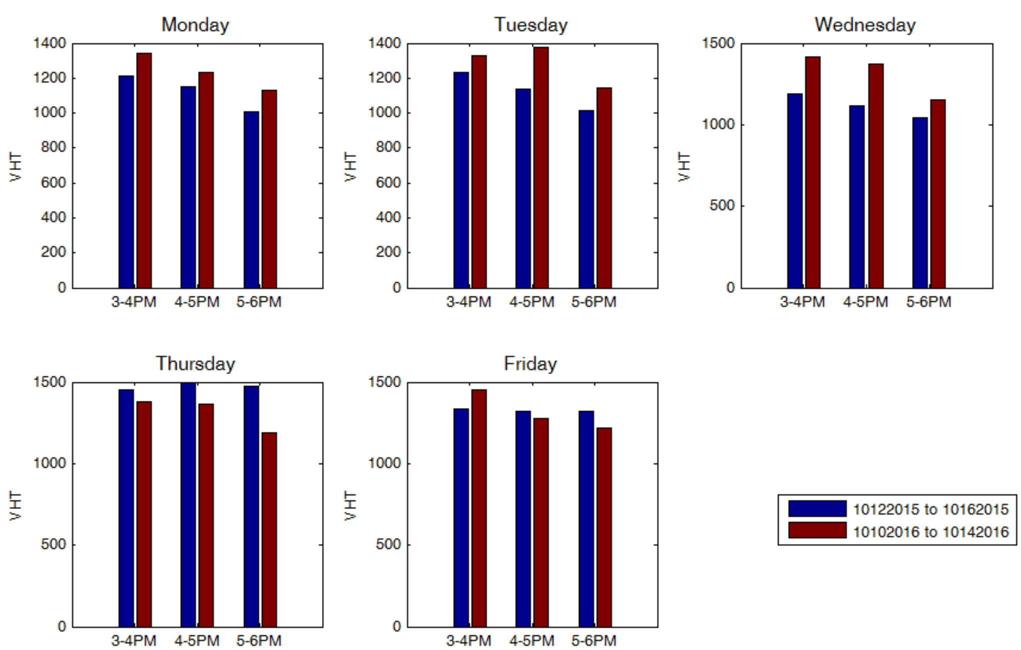

Figure A2-10. October Week2 PM VHT

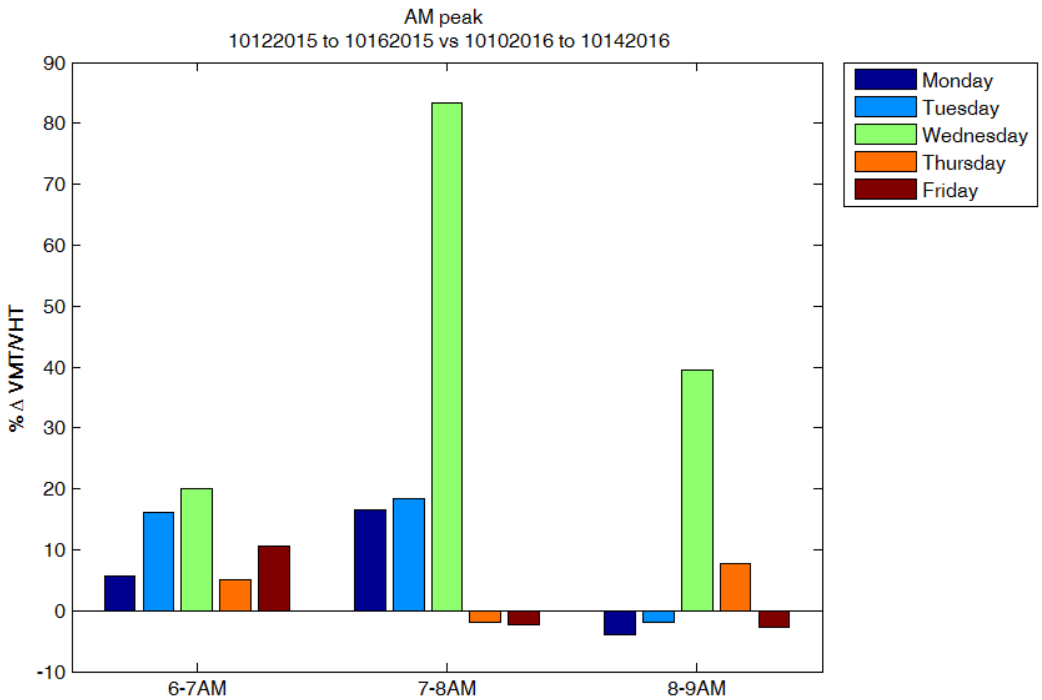

Figure A2-11. October Week2 AM %ΔQ

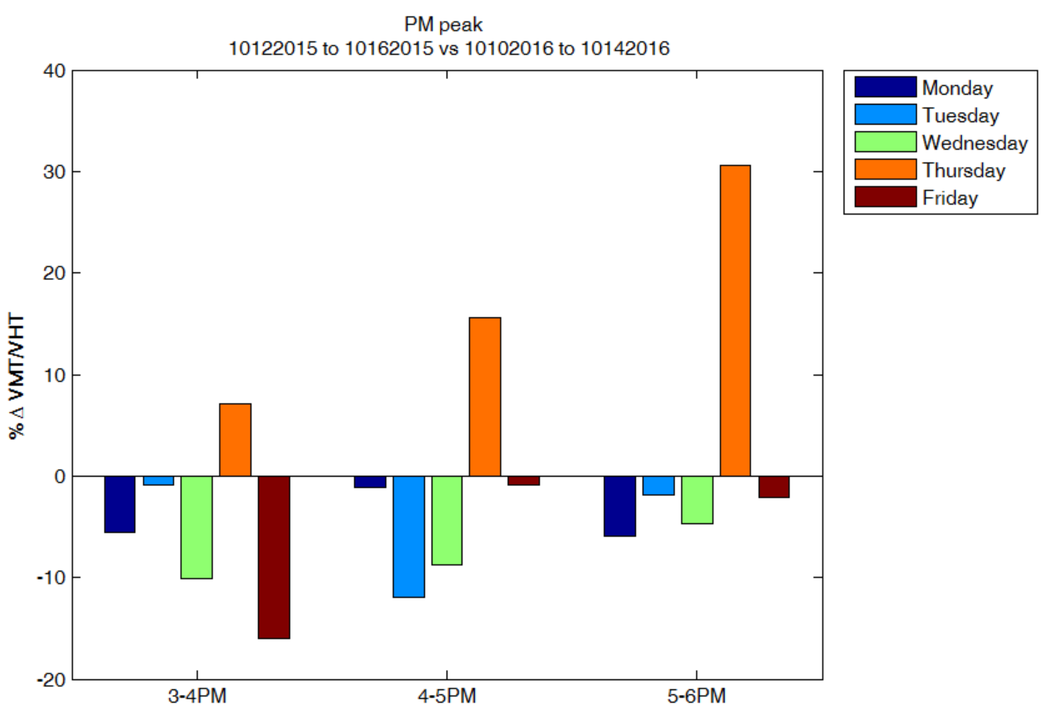

Figure A2-12. October Week2 PM %ΔQ

Table A2-2. October Week2 data

Monday

Table: 10122015 vs 10102016 AM peak performance comparison

Morning peak	6-7AM	7-8AM	8-9AM	Avg.	Std.
10122015 VMT	82295.50	73723.00	70213.20	75410.57	6215.42
10122015 VHT	1962.10	1567.60	1162.60	1564.10	399.76
10122015 VMT/VHT	41.94	47.03	60.39	49.79	9.53
10102016 VMT	83986.80	75536.90	73206.80	77576.83	5672.13
10102016 VHT	1896.60	1377.30	1262.30	1512.07	337.94
10102016 VMT/VHT	44.28	54.84	57.99	52.37	7.18
%improvement	5.580%	16.617%	-3.971%	6.075%	10.303%

Table: 10122015 vs 10102016 PM peak performance comparison

Evening peak	3-4PM	4-5PM	5-6PM	Avg.	Std.
10122015 VMT	75982.60	71640.30	64393.40	70672.10	5854.95
10122015 VHT	1209.40	1147.90	1006.80	1121.37	103.87
10122015 VMT/VHT	62.83	62.41	63.96	63.07	0.80
10102016 VMT	79620.30	76137.90	68162.30	74640.17	5874.00
10102016 VHT	1342.60	1234.60	1132.70	1236.63	104.96
10102016 VMT/VHT	59.30	61.67	60.18	60.38	1.20
%improvement	-5.608%	-1.185%	-5.913%	-4.236%	2.646%

Tuesday

Table: 10132015 vs 10112016 AM peak performance comparison

Morning peak	6-7AM	7-8AM	8-9AM	Avg.	Std.
10132015 VMT	77952.00	72566.90	73011.90	74510.27	2988.92
10132015 VHT	2489.50	2243.30	1398.70	2043.83	572.10
10132015 VMT/VHT	31.31	32.35	52.20	38.62	11.77
10112016 VMT	79646.00	79051.40	74298.00	77665.13	2931.14
10112016 VHT	2190.10	2064.00	1451.40	1901.83	395.15
10112016 VMT/VHT	36.37	38.30	51.19	41.95	8.06
%improvement	16.141%	18.399%	-1.933%	10.869%	11.144%

Table: 10132015 vs 10112016 PM peak performance comparison

Evening peak	3-4PM	4-5PM	5-6PM	Avg.	Std.
10132015 VMT	75691.30	72457.50	64479.30	70876.03	5770.88
10132015 VHT	1233.00	1138.20	1011.00	1127.40	111.39
10132015 VMT/VHT	61.39	63.66	63.78	62.94	1.35
10112016 VMT	80795.20	77024.40	71552.20	76457.27	4647.53
10112016 VHT	1327.40	1374.30	1142.60	1281.43	122.50
10112016 VMT/VHT	60.87	56.05	62.62	59.85	3.41
%improvement	-0.848%	-11.960%	-1.812%	-4.873%	6.156%

Wednesday

Table: 10142015 vs 10122016 AM peak performance comparison

Morning peak	6-7AM	7-8AM	8-9AM	Avg.	Std.
10142015 VMT	77689.30	59086.50	63496.10	66757.30	9720.73
10142015 VHT	2307.30	2692.80	1778.40	2259.50	459.07
10142015 VMT/VHT	33.67	21.94	35.70	30.44	7.43
10122016 VMT	84574.30	79326.40	75410.10	79770.27	4598.20
10122016 VHT	2091.80	1972.20	1515.20	1859.73	304.31
10122016 VMT/VHT	40.43	40.22	49.77	43.47	5.45
%improvement	20.077%	83.309%	39.393%	47.593%	32.403%

Table: 10142015 vs 10122016 PM peak performance comparison

Evening peak	3-4PM	4-5PM	5-6PM	Avg.	Std.
10142015 VMT	74262.90	70853.10	66469.30	70528.43	3906.93
10142015 VHT	1185.50	1116.00	1041.90	1114.47	71.81
10142015 VMT/VHT	62.64	63.49	63.80	63.31	0.60
10122016 VMT	79624.10	79451.50	70192.50	76422.70	5396.20
10122016 VHT	1413.90	1372.30	1154.10	1313.43	139.55
10122016 VMT/VHT	56.32	57.90	60.82	58.34	2.29
%improvement	-10.101%	-8.808%	-4.665%	-7.858%	2.840%

Thursday

Table: 10152015 vs 10132016 AM peak performance comparison

Morning peak	6-7AM	7-8AM	8-9AM	Avg.	Std.
10152015 VMT	81864.10	75649.00	72069.30	76527.47	4956.14
10152015 VHT	2376.40	1940.10	1367.30	1894.60	506.09
10152015 VMT/VHT	34.45	38.99	52.71	42.05	9.51
10132016 VMT	81864.90	80090.80	76676.30	79544.00	2637.16
10132016 VHT	2263.60	2094.80	1350.20	1902.87	486.01
10132016 VMT/VHT	36.17	38.23	56.79	43.73	11.36
%improvement	4.984%	-1.947%	7.740%	3.592%	4.991%

Table: 10152015 vs 10132016 PM peak performance comparison

Evening peak	3-4PM	4-5PM	5-6PM	Avg.	Std.
10152015 VMT	80843.10	75384.80	69453.40	75227.10	5696.49
10152015 VHT	1455.40	1494.90	1477.70	1476.00	19.80
10152015 VMT/VHT	55.55	50.43	47.00	50.99	4.30
10132016 VMT	82115.10	79390.80	73056.60	78187.50	4647.59
10132016 VHT	1379.80	1362.20	1190.10	1310.70	104.81
10132016 VMT/VHT	59.51	58.28	61.39	59.73	1.56
%improvement	7.139%	15.573%	30.608%	17.773%	11.888%

Friday

Table: 10162015 vs 10142016 AM peak performance comparison

Morning peak	6-7AM	7-8AM	8-9AM	Avg.	Std.
10162015 VMT	83109.00	74651.80	73620.40	77127.07	5206.11
10162015 VHT	1872.50	1530.30	1232.40	1545.07	320.31
10162015 VMT/VHT	44.38	48.78	59.74	50.97	7.91
10142016 VMT	85281.30	75785.70	73649.80	78238.93	6191.66
10142016 VHT	1736.50	1589.00	1268.10	1531.20	239.49
10142016 VMT/VHT	49.11	47.69	58.08	51.63	5.63
%improvement	10.650%	-2.231%	-2.776%	1.881%	7.599%

Table: 10162015 vs 10142016 PM peak performance comparison

Evening peak	3-4PM	4-5PM	5-6PM	Avg.	Std.
10162015 VMT	81294.00	77815.80	81589.00	80232.93	2098.49
10162015 VHT	1337.90	1319.00	1317.90	1324.93	11.24
10162015 VMT/VHT	60.76	59.00	61.91	60.56	1.47
10142016 VMT	74090.00	74791.70	73986.90	74289.53	437.93
10142016 VHT	1451.90	1279.30	1221.00	1317.40	120.07
10142016 VMT/VHT	51.03	58.46	60.60	56.70	5.02
%improvement	-16.018%	-0.904%	-2.121%	-6.347%	8.397%

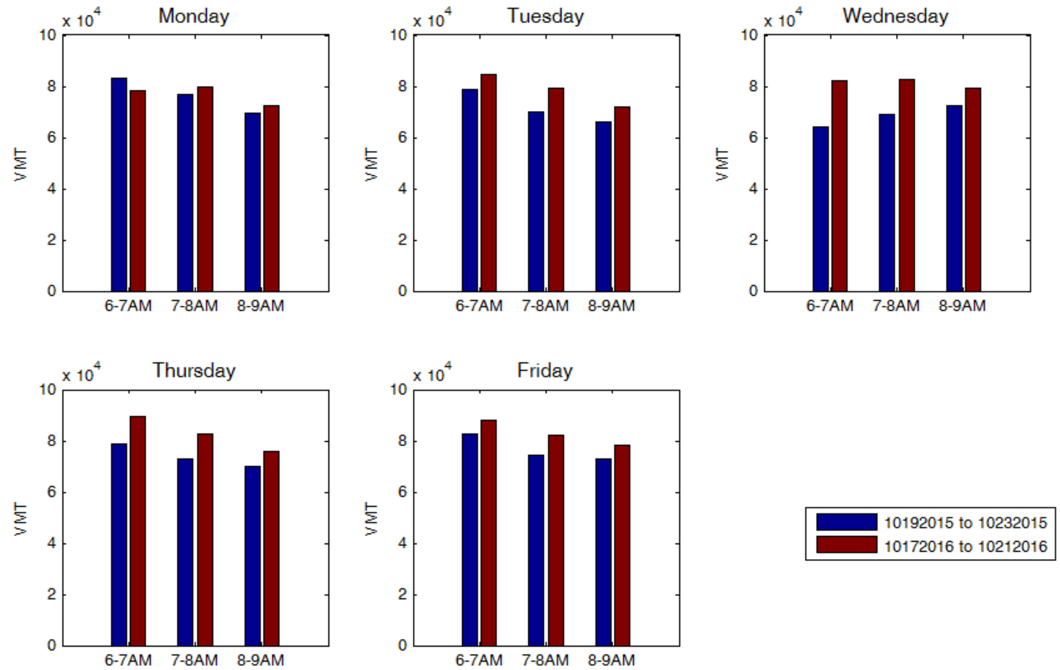

Figure A2-13. October Week3 AM VMT

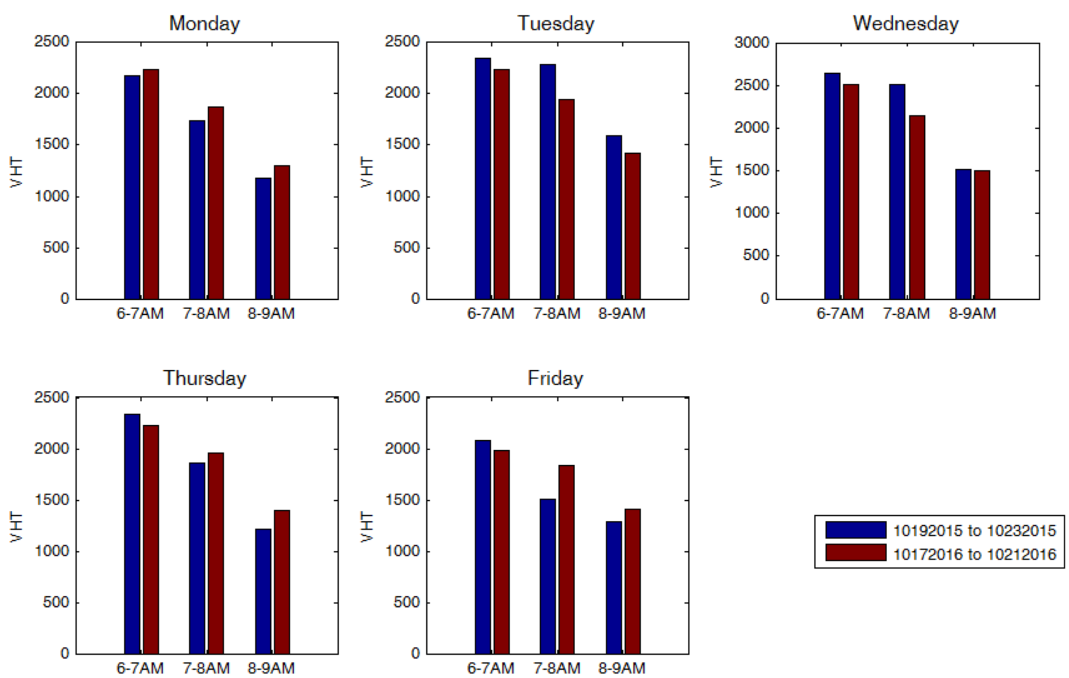

Figure A2-14. October Week3 AM VHT

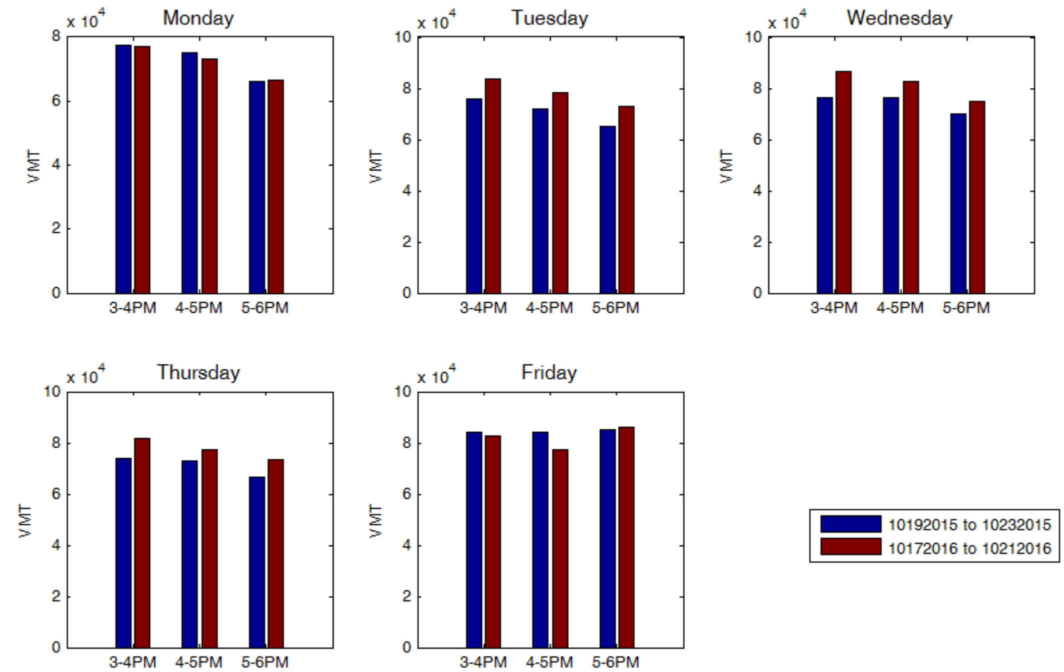

Figure A2-15. October Week3 PM VMT

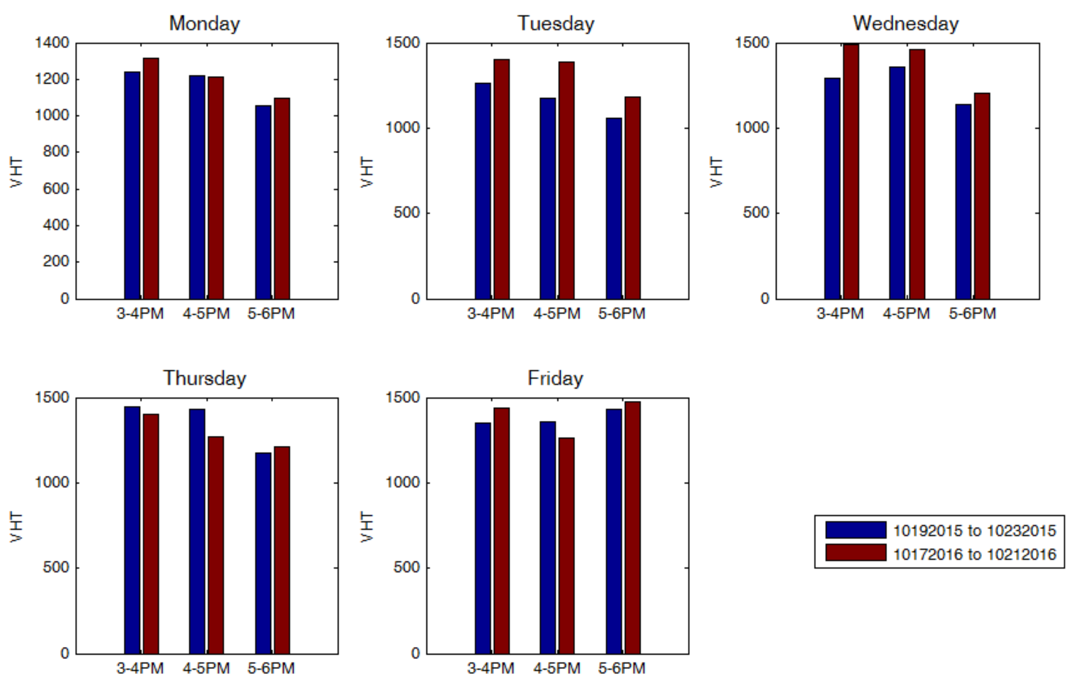

Figure A2-16. October Week3 PM VHT

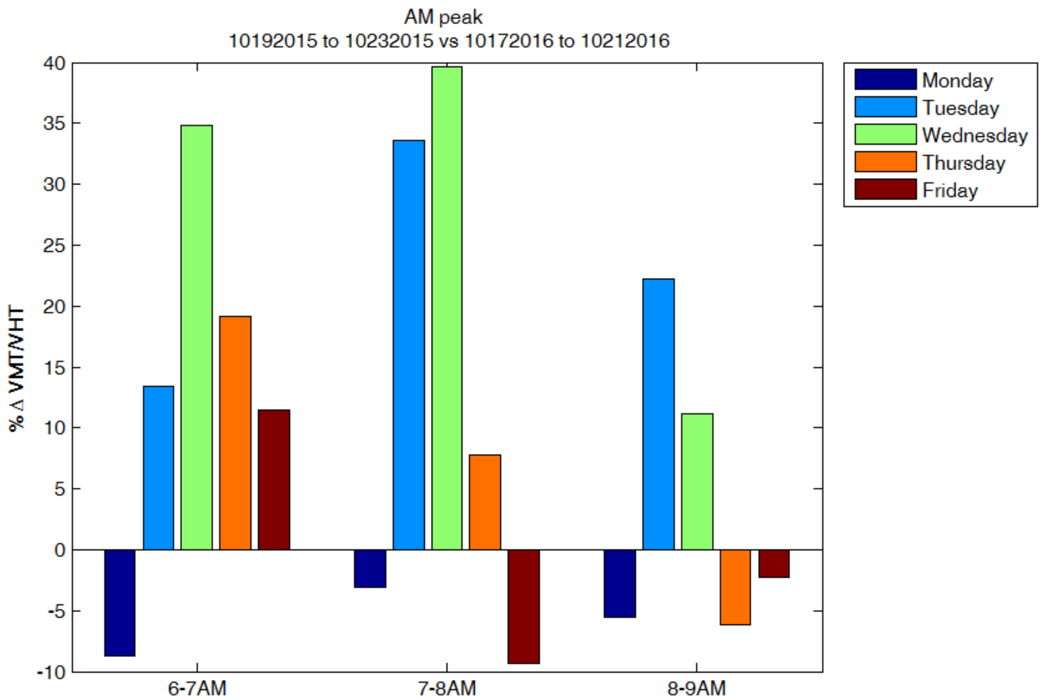

Figure A2-17. October Week3 AM %ΔQ

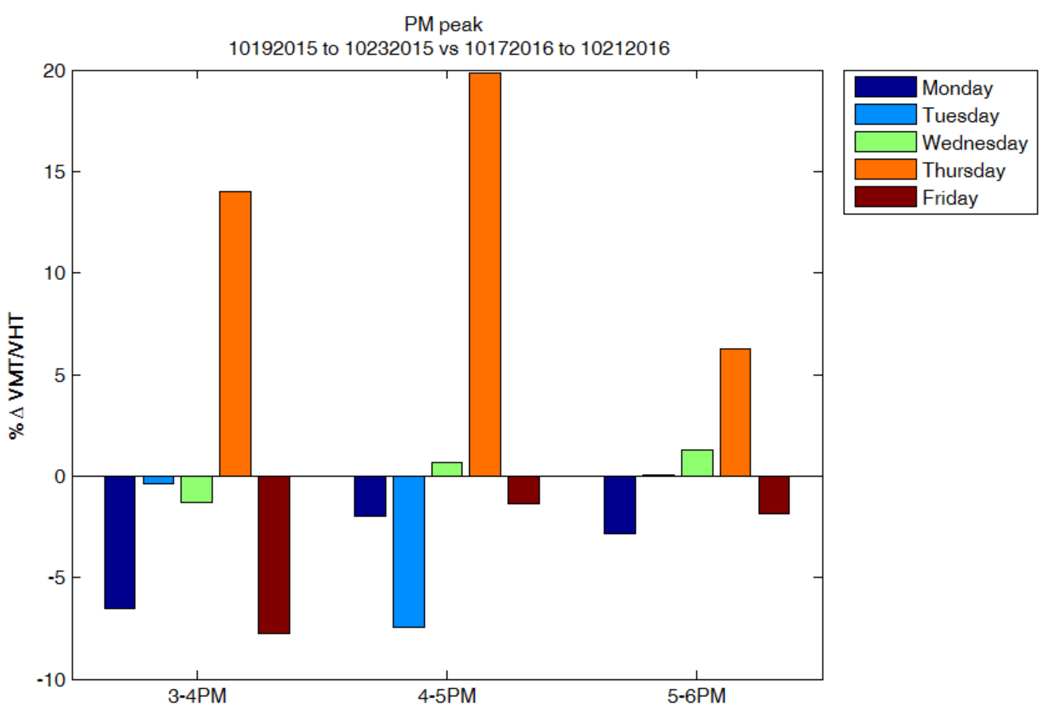

Figure A2-18. October Week3 AM %ΔQ

Table A2-3. October Week3 data

Monday

Table: 10192015 vs 10172016 AM peak performance comparison

Morning peak	6-7AM	7-8AM	8-9AM	Avg.	Std.
10192015 VMT	83354.20	76676.60	69561.70	76530.83	6897.41
10192015 VHT	2181.50	1743.30	1178.90	1701.23	502.62
10192015 VMT/VHT	38.21	43.98	59.01	47.07	10.74
10172016 VMT	78267.10	79835.10	72386.20	76829.47	3927.04
10172016 VHT	2243.00	1872.00	1298.30	1804.43	475.96
10172016 VMT/VHT	34.89	42.65	55.75	44.43	10.54
%improvement	-8.678%	-3.039%	-5.510%	-5.742%	2.826%

Table: 10192015 vs 10172016 PM peak performance comparison

Evening peak	3-4PM	4-5PM	5-6PM	Avg.	Std.
10192015 VMT	77568.90	75095.20	66008.50	72890.87	6087.28
10192015 VHT	1238.10	1219.30	1057.20	1171.53	99.46
10192015 VMT/VHT	62.65	61.59	62.44	62.23	0.56
10172016 VMT	76947.20	73343.60	66584.70	72291.83	5260.70
10172016 VHT	1313.80	1215.00	1097.90	1208.90	108.08
10172016 VMT/VHT	58.57	60.37	60.65	59.86	1.13
%improvement	-6.517%	-1.987%	-2.867%	-3.790%	2.402%

Tuesday

Table: 10202015 vs 10182016 AM peak performance comparison

Morning peak	6-7AM	7-8AM	8-9AM	Avg.	Std.
10202015 VMT	78602.20	69866.10	66184.60	71550.97	6377.95
10202015 VHT	2350.70	2291.00	1597.30	2079.67	418.81
10202015 VMT/VHT	33.44	30.50	41.44	35.12	5.66
10182016 VMT	84656.70	79159.60	72181.70	78666.00	6252.13
10182016 VHT	2232.50	1943.00	1425.80	1867.10	408.67
10182016 VMT/VHT	37.92	40.74	50.63	43.10	6.67
%improvement	13.405%	33.595%	22.179%	23.060%	10.124%

Table: 10202015 vs 10182016 PM peak performance comparison

Evening peak	3-4PM	4-5PM	5-6PM	Avg.	Std.
10202015 VMT	75678.40	71799.70	65290.30	70922.80	5249.27
10202015 VHT	1264.70	1175.40	1054.70	1164.93	105.39
10202015 VMT/VHT	59.84	61.09	61.90	60.94	1.04
10182016 VMT	83489.20	78278.00	73155.90	78307.70	5166.71
10182016 VHT	1400.40	1384.40	1180.90	1321.90	122.37
10182016 VMT/VHT	59.62	56.54	61.95	59.37	2.71
%improvement	-0.369%	-7.436%	0.073%	-2.577%	4.214%

Wednesday

Table: 10212015 vs 10192016 AM peak performance comparison

Morning peak	6-7AM	7-8AM	8-9AM	Avg.	Std.
10212015 VMT	63994.80	69252.30	72357.40	68534.83	4227.21
10212015 VHT	2643.90	2503.40	1521.30	2222.87	611.62
10212015 VMT/VHT	24.20	27.66	47.56	33.14	12.61
10192016 VMT	81971.10	82797.30	79263.20	81343.87	1848.66
10192016 VHT	2511.10	2143.80	1498.80	2051.23	512.46
10192016 VMT/VHT	32.64	38.62	52.88	41.38	10.40
%improvement	34.864%	39.614%	11.188%	28.555%	15.227%

Table: 10212015 vs 10192016 PM peak performance comparison

Evening peak	3-4PM	4-5PM	5-6PM	Avg.	Std.
10212015 VMT	76152.30	76242.10	69855.80	74083.40	3661.48
10212015 VHT	1290.70	1359.20	1137.60	1262.50	113.46
10212015 VMT/VHT	59.00	56.09	61.41	58.83	2.66
10192016 VMT	86573.40	82605.40	75040.20	81406.33	5859.35
10192016 VHT	1486.40	1463.20	1206.50	1385.37	155.34
10192016 VMT/VHT	58.24	56.46	62.20	58.97	2.94
%improvement	-1.283%	0.645%	1.287%	0.216%	1.338%

Thursday

Table: 10222015 vs 10202016 AM peak performance comparison

Morning peak	6-7AM	7-8AM	8-9AM	Avg.	Std.
10222015 VMT	78964.00	72975.60	70046.70	73995.43	4545.28
10222015 VHT	2335.70	1861.80	1209.10	1802.20	565.66
10222015 VMT/VHT	33.81	39.20	57.93	43.65	12.66
10202016 VMT	89650.00	82530.30	75634.00	82604.77	7008.30
10202016 VHT	2225.30	1954.00	1391.20	1856.83	425.45
10202016 VMT/VHT	40.29	42.24	54.37	45.63	7.63
%improvement	19.165%	7.757%	-6.157%	6.922%	12.682%

Table: 10222015 vs 10202016 PM peak performance comparison

Evening peak	3-4PM	4-5PM	5-6PM	Avg.	Std.
10222015 VMT	73939.20	72687.10	66708.80	71111.70	3864.08
10222015 VHT	1443.80	1428.80	1173.40	1348.67	151.97
10222015 VMT/VHT	51.21	50.87	56.85	52.98	3.36
10202016 VMT	81737.90	77437.50	73252.80	77476.07	4242.68
10202016 VHT	1399.90	1270.40	1212.90	1294.40	95.78
10202016 VMT/VHT	58.39	60.96	60.39	59.91	1.35
%improvement	14.014%	19.819%	6.234%	13.356%	6.816%

Friday

Table: 10232015 vs 10212016 AM peak performance comparison

Morning peak	6-7AM	7-8AM	8-9AM	Avg.	Std.
10232015 VMT	82599.80	74421.70	72999.50	76673.67	5181.21
10232015 VHT	2076.70	1502.70	1286.30	1621.90	408.46
10232015 VMT/VHT	39.77	49.53	56.75	48.68	8.52
10212016 VMT	88071.60	82265.80	78052.10	82796.50	5030.79
10212016 VHT	1986.30	1832.60	1407.70	1742.20	299.71
10212016 VMT/VHT	44.34	44.89	55.45	48.23	6.26
%improvement	11.477%	-9.359%	-2.299%	-0.060%	10.597%

Table: 10232015 vs 10212016 PM peak performance comparison

Evening peak	3-4PM	4-5PM	5-6PM	Avg.	Std.
10232015 VMT	84063.00	84309.40	85133.50	84501.97	560.63
10232015 VHT	1353.40	1354.40	1429.60	1379.13	43.71
10232015 VMT/VHT	62.11	62.25	59.55	61.30	1.52
10212016 VMT	82447.60	77497.40	86193.30	82046.10	4361.83
10212016 VHT	1439.40	1262.30	1474.60	1392.10	113.78
10212016 VMT/VHT	57.28	61.39	58.45	59.04	2.12
%improvement	-7.782%	-1.373%	-1.845%	-3.666%	3.572%

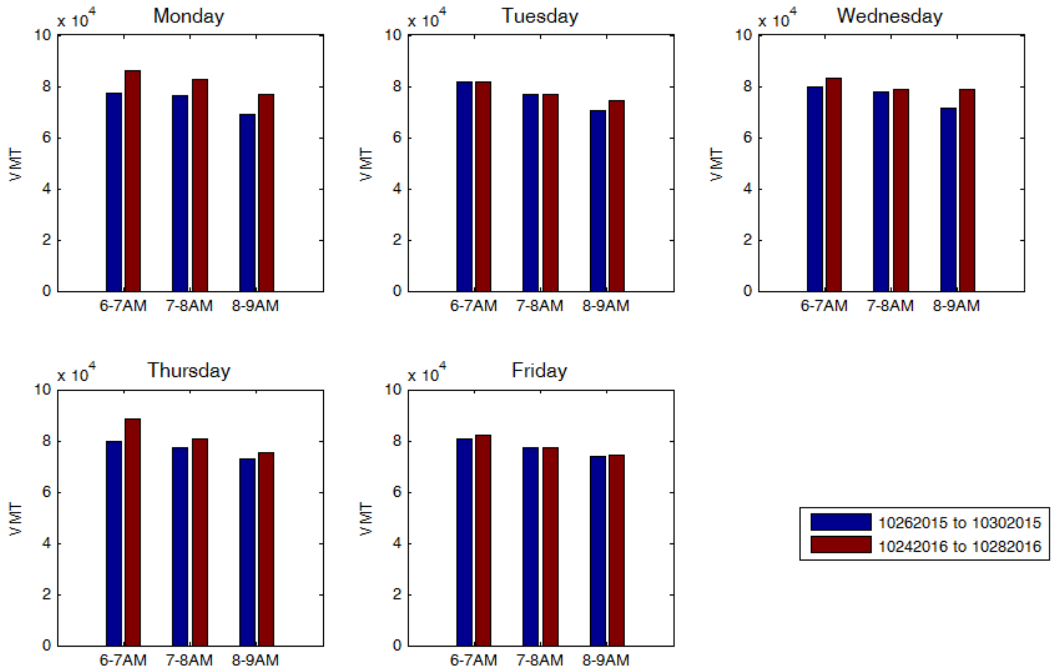

Figure A2-19. October Week4 AM VMT

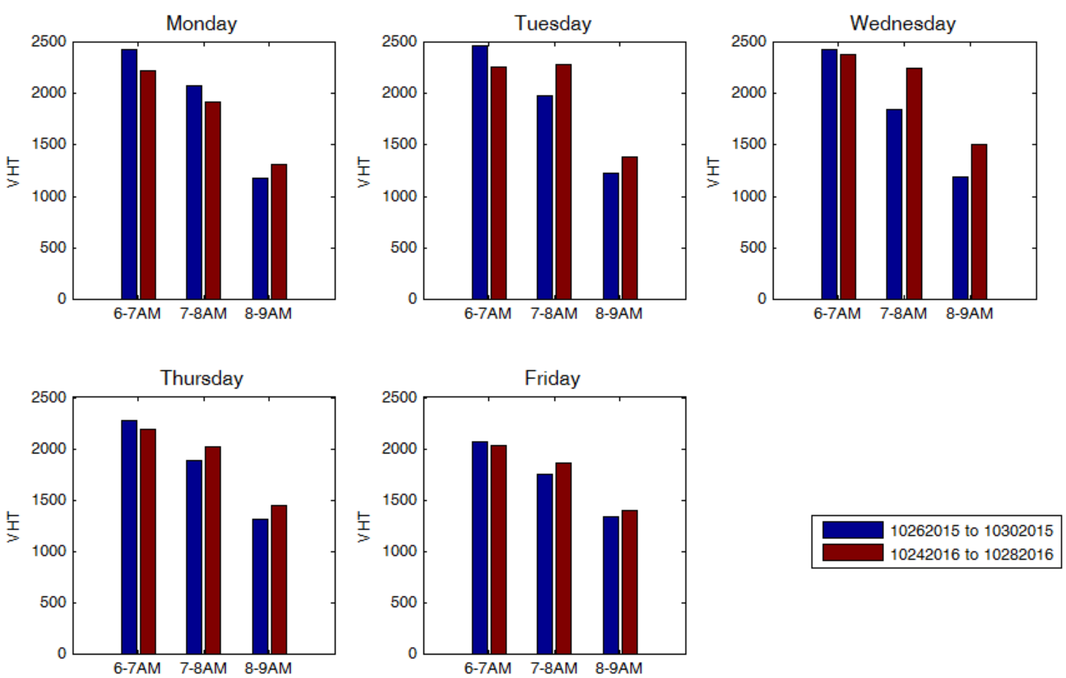

Figure A2-20. October Week4 AM VHT

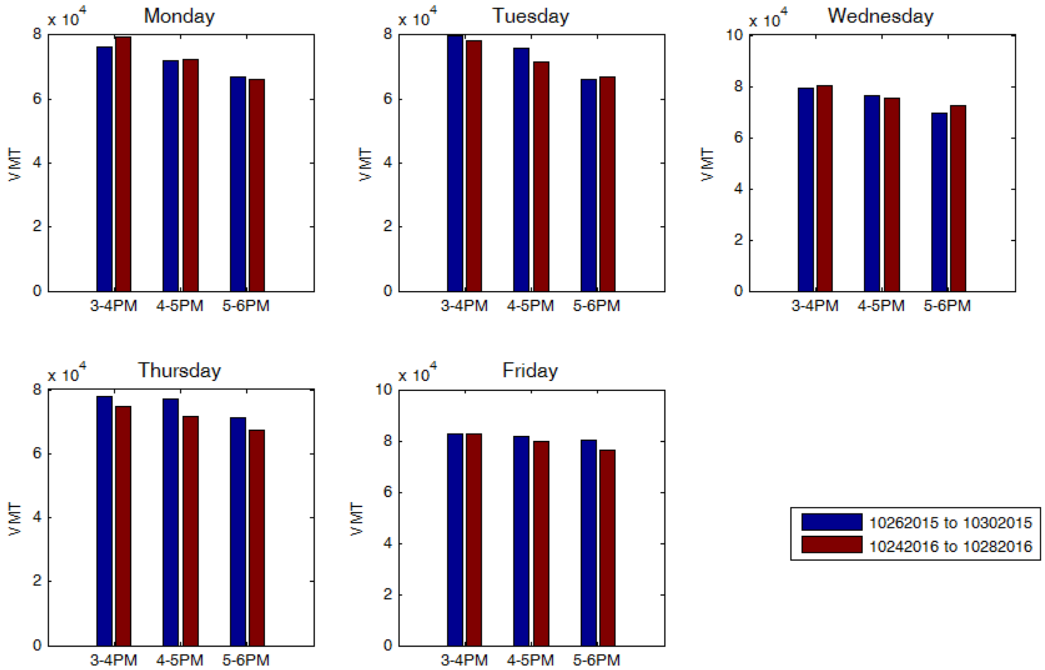

Figure A2-21. October Week4 PM VMT

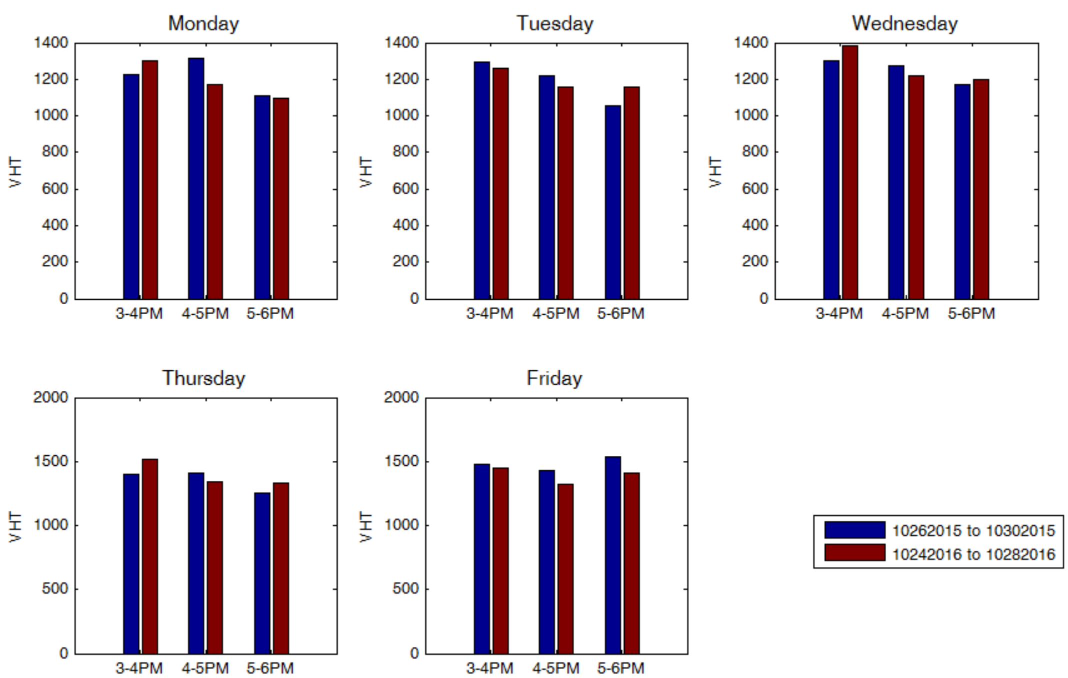

Figure A2-22. October Week4 PM VHT

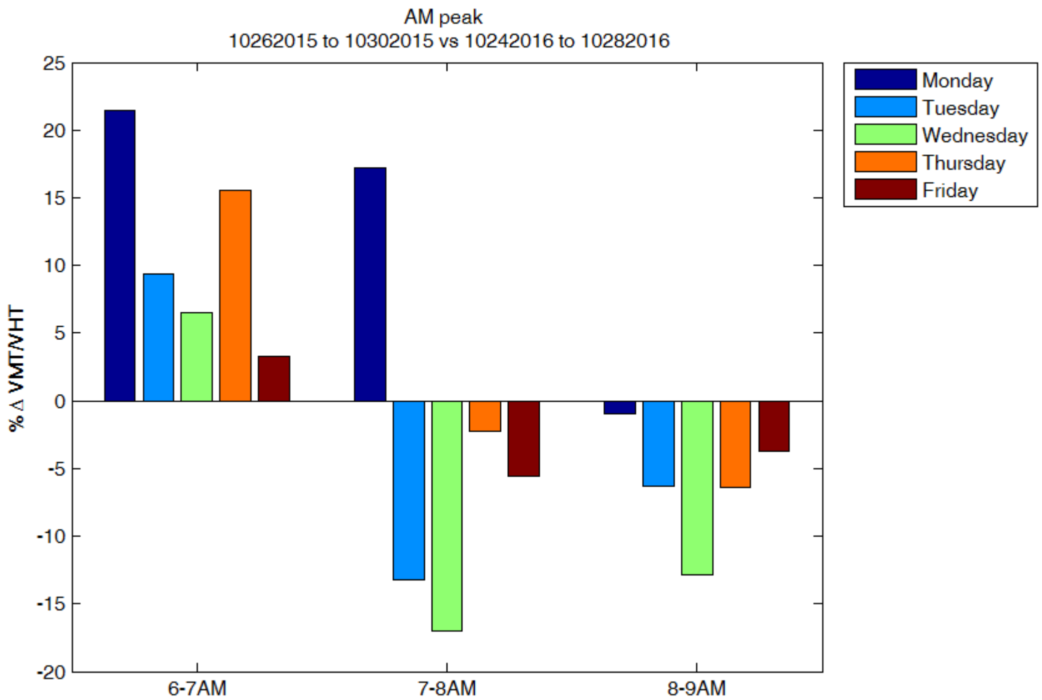

Figure A2-23. October Week4 AM %ΔQ

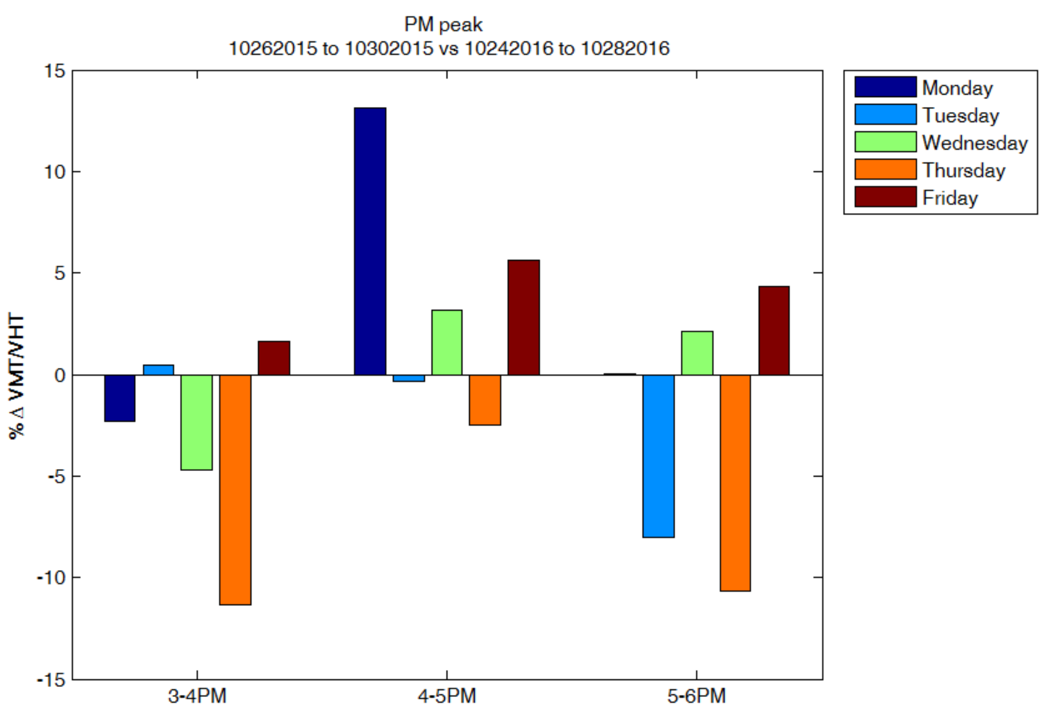

Figure A2-24. October Week4 PM %ΔQ

Table A2-4. October Week4 data

Monday

Table: 10262015 vs 10242016 AM peak performance comparison

Morning peak	6-7AM	7-8AM	8-9AM	Avg.	Std.
10262015 VMT	77418.50	76349.30	69030.20	74266.00	4565.74
10262015 VHT	2436.90	2081.30	1170.80	1896.33	653.00
10262015 VMT/VHT	31.77	36.68	58.96	42.47	14.49
10242016 VMT	85990.00	82840.00	76927.80	81919.27	4600.73
10242016 VHT	2228.50	1926.50	1317.00	1824.00	464.31
10242016 VMT/VHT	38.59	43.00	58.41	46.67	10.41
%improvement	21.459%	17.220%	-0.930%	12.583%	11.893%

Table: 10262015 vs 10242016 PM peak performance comparison

Evening peak	3-4PM	4-5PM	5-6PM	Avg.	Std.
10262015 VMT	76099.00	71989.30	67104.00	71730.77	4503.07
10262015 VHT	1222.90	1311.50	1112.20	1215.53	99.85
10262015 VMT/VHT	62.23	54.89	60.33	59.15	3.81
10242016 VMT	79232.40	72539.10	66083.90	72618.47	6574.61
10242016 VHT	1303.60	1168.00	1094.70	1188.77	105.99
10242016 VMT/VHT	60.78	62.11	60.37	61.08	0.91
%improvement	-2.328%	13.144%	0.054%	3.623%	8.330%

Tuesday

Table: 10272015 vs 10252016 AM peak performance comparison

Morning peak	6-7AM	7-8AM	8-9AM	Avg.	Std.
10272015 VMT	81540.40	76975.40	70466.20	76327.33	5565.47
10272015 VHT	2463.80	1986.60	1230.20	1893.53	622.04
10272015 VMT/VHT	33.10	38.75	57.28	43.04	12.65
10252016 VMT	81676.40	76928.40	74520.30	77708.37	3641.25
10252016 VHT	2256.60	2286.90	1388.20	1977.23	510.34
10252016 VMT/VHT	36.19	33.64	53.68	41.17	10.91
%improvement	9.364%	-13.184%	-6.283%	-3.368%	11.553%

Table: 10272015 vs 10252016 PM peak performance comparison

Evening peak	3-4PM	4-5PM	5-6PM	Avg.	Std.
10272015 VMT	79872.50	76094.60	66284.60	74083.90	7013.55
10272015 VHT	1295.50	1221.20	1052.80	1189.83	124.35
10272015 VMT/VHT	61.65	62.31	62.96	62.31	0.65
10252016 VMT	78218.00	71770.60	67004.70	72331.10	5627.62
10252016 VHT	1262.50	1155.80	1156.90	1191.73	61.29
10252016 VMT/VHT	61.95	62.10	57.92	60.66	2.37
%improvement	0.488%	-0.346%	-8.010%	-2.622%	4.684%

Wednesday

Table: 10282015 vs 10262016 AM peak performance comparison

Morning peak	6-7AM	7-8AM	8-9AM	Avg.	Std.
10282015 VMT	79871.40	77850.70	71442.60	76388.23	4400.60
10282015 VHT	2436.80	1845.80	1189.10	1823.90	624.14
10282015 VMT/VHT	32.78	42.18	60.08	45.01	13.87
10262016 VMT	83345.90	78847.30	78995.60	80396.27	2555.53
10262016 VHT	2386.40	2252.00	1509.50	2049.30	472.29
10262016 VMT/VHT	34.93	35.01	52.33	40.76	10.02
%improvement	6.554%	-16.988%	-12.897%	-7.777%	12.579%

Table: 10282015 vs 10262016 PM peak performance comparison

Evening peak	3-4PM	4-5PM	5-6PM	Avg.	Std.
10282015 VMT	79143.30	76337.90	69487.60	74989.60	4967.05
10282015 VHT	1299.30	1271.10	1173.80	1248.07	65.84
10282015 VMT/VHT	60.91	60.06	59.20	60.06	0.86
10262016 VMT	80477.00	75505.10	72457.70	76146.60	4047.95
10262016 VHT	1386.10	1218.80	1198.40	1267.77	102.99
10262016 VMT/VHT	58.06	61.95	60.46	60.16	1.96
%improvement	-4.683%	3.153%	2.134%	0.202%	4.260%

Thursday

Table: 10292015 vs 10272016 AM peak performance comparison

Morning peak	6-7AM	7-8AM	8-9AM	Avg.	Std.
10292015 VMT	79744.30	77212.40	73008.70	76655.13	3402.20
10292015 VHT	2277.40	1886.20	1311.00	1824.87	486.11
10292015 VMT/VHT	35.02	40.94	55.69	43.88	10.65
10272016 VMT	88697.90	80808.90	75592.20	81699.67	6598.10
10272016 VHT	2192.70	2019.70	1450.80	1887.73	388.16
10272016 VMT/VHT	40.45	40.01	52.10	44.19	6.86
%improvement	15.524%	-2.260%	-6.438%	2.275%	11.663%

Table: 10292015 vs 10272016 PM peak performance comparison

Evening peak	3-4PM	4-5PM	5-6PM	Avg.	Std.
10292015 VMT	77995.60	77243.70	71236.10	75491.80	3704.67
10292015 VHT	1404.00	1409.30	1254.90	1356.07	87.65
10292015 VMT/VHT	55.55	54.81	56.77	55.71	0.99
10272016 VMT	74874.40	71523.50	67353.00	71250.30	3768.14
10272016 VHT	1519.80	1338.30	1328.50	1395.53	107.73
10272016 VMT/VHT	49.27	53.44	50.70	51.14	2.12
%improvement	-11.316%	-2.493%	-10.689%	-8.166%	4.923%

Friday

Table: 10302015 vs 10282016 AM peak performance comparison

Morning peak	6-7AM	7-8AM	8-9AM	Avg.	Std.
10302015 VMT	80894.00	77422.40	73721.50	77345.97	3586.86
10302015 VHT	2071.20	1751.50	1330.50	1717.73	371.50
10302015 VMT/VHT	39.06	44.20	55.41	46.22	8.36
10282016 VMT	82106.00	77489.20	74277.90	77957.70	3935.02
10282016 VHT	2034.50	1855.70	1392.40	1760.87	331.39
10282016 VMT/VHT	40.36	41.76	53.35	45.15	7.13
%improvement	3.329%	-5.534%	-3.724%	-1.976%	4.683%

Table: 10302015 vs 10282016 PM peak performance comparison

Evening peak	3-4PM	4-5PM	5-6PM	Avg.	Std.
10302015 VMT	82762.10	81700.60	80115.70	81526.13	1331.80
10302015 VHT	1476.10	1426.40	1536.40	1479.63	55.09
10302015 VMT/VHT	56.07	57.28	52.15	55.16	2.68
10282016 VMT	82768.70	79826.70	76470.40	79688.60	3151.42
10282016 VHT	1452.60	1319.30	1405.60	1392.50	67.61
10282016 VMT/VHT	56.98	60.51	54.40	57.30	3.06
%improvement	1.626%	5.638%	4.332%	3.865%	2.046%

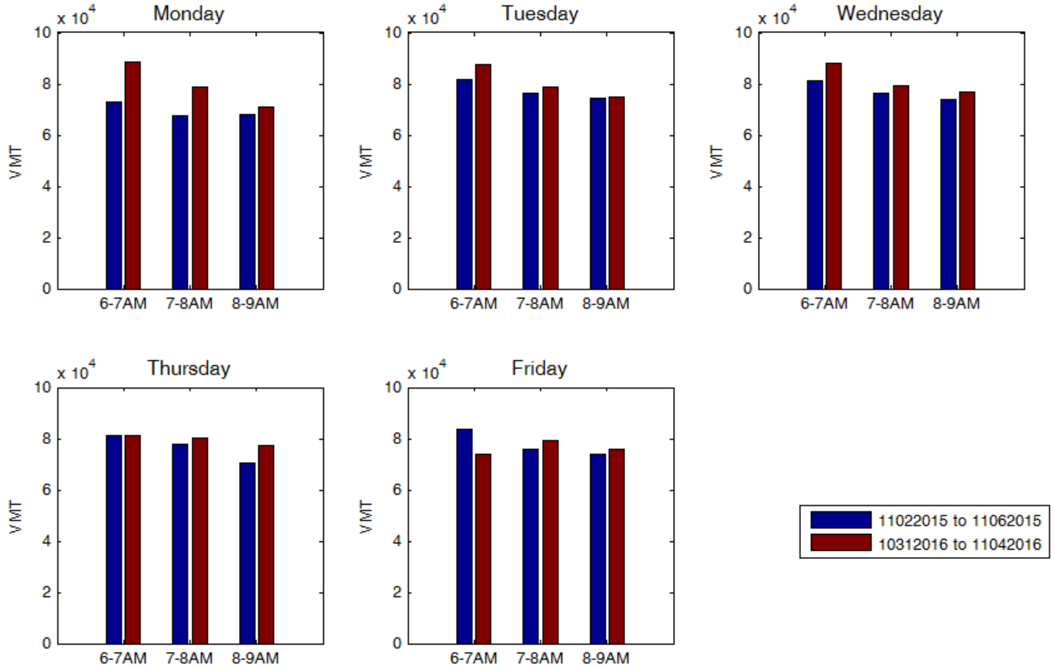

Figure A2-25. November Week1 AM VMT

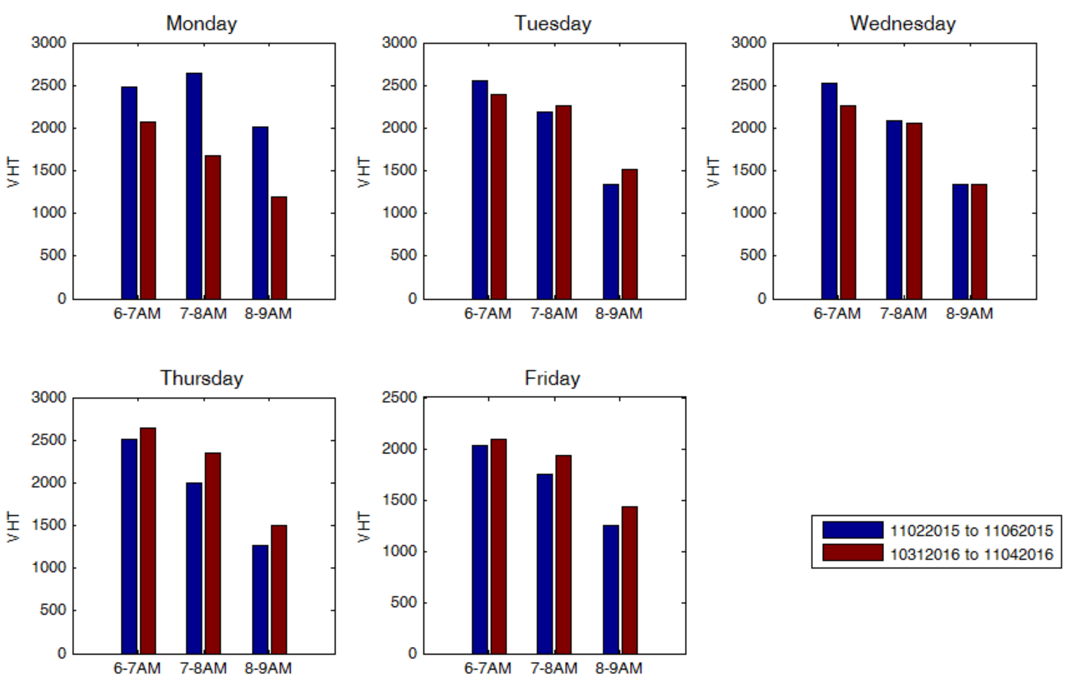

Figure A2-26. November Week1 AM VHT

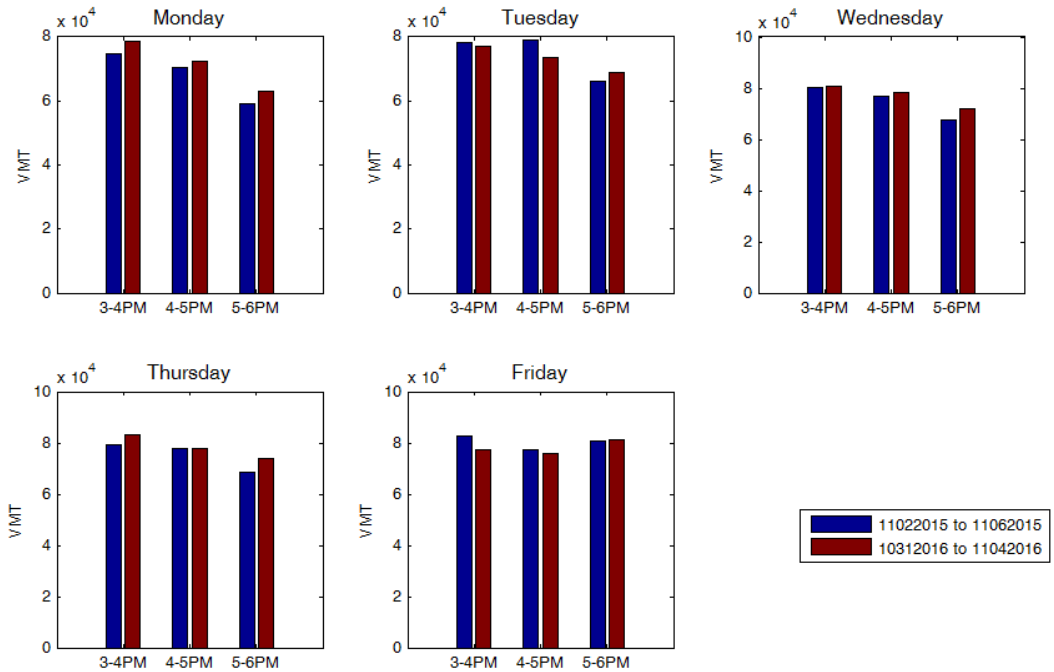

Figure A2-27. November Week1 PM VMT

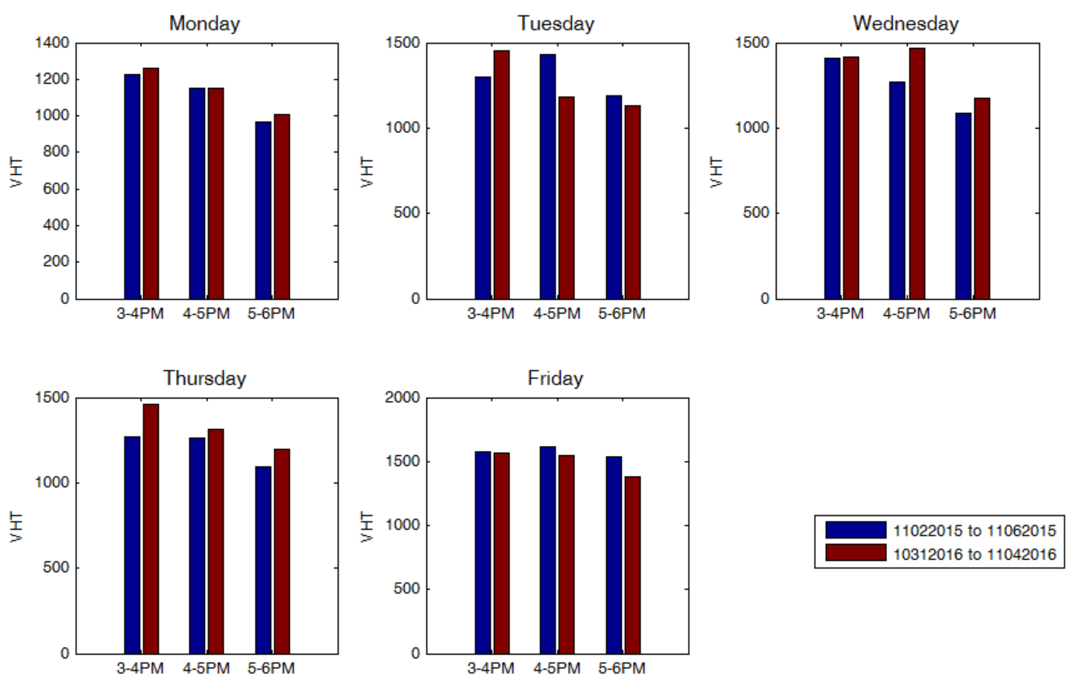

Figure A2-28. November Week1 PM VHT

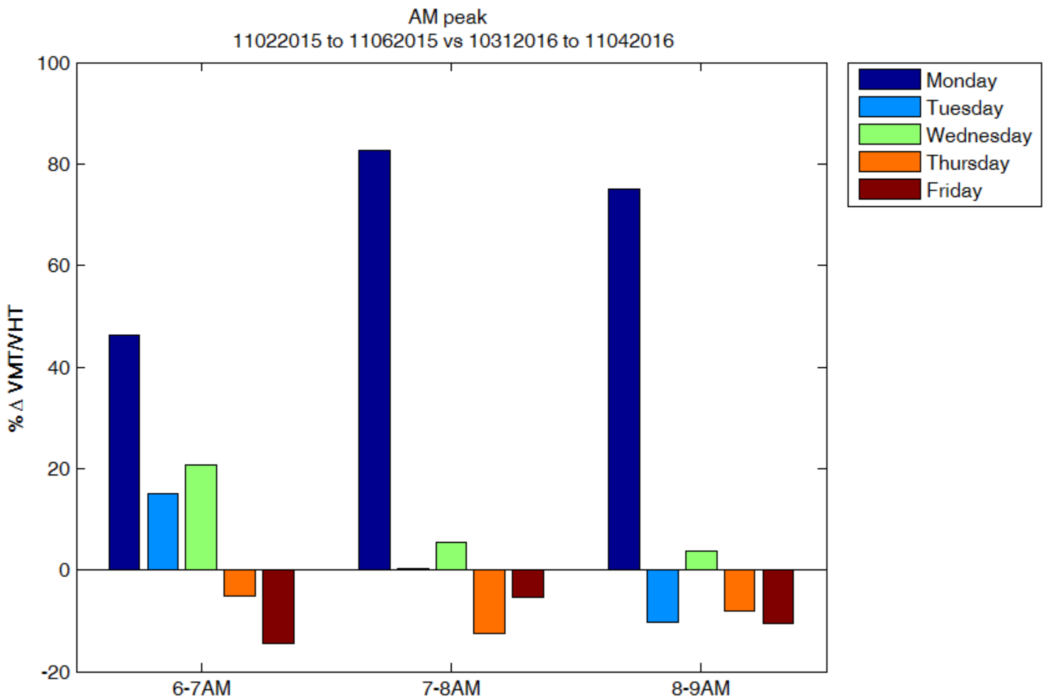

Figure A2-29. November Week1 AM %ΔQ

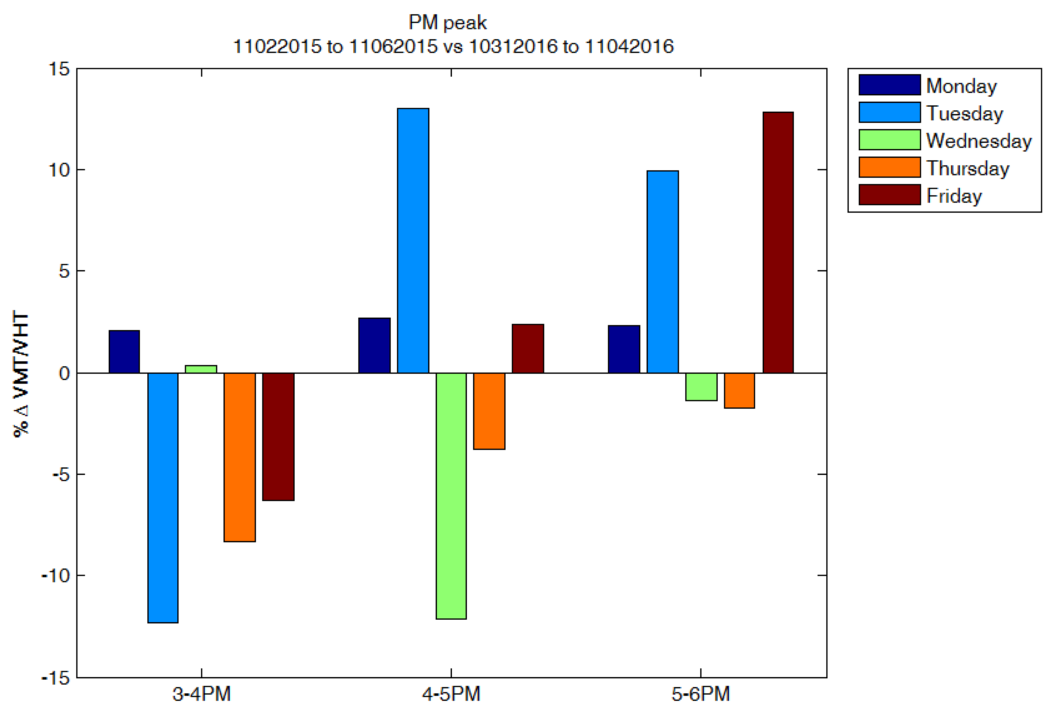

Figure A2-30. November Week1 PM %ΔQ

Table A2-5. November Week1 data

Monday

Table: 11022015 vs 10312016 AM peak performance comparison

Morning peak	6-7AM	7-8AM	8-9AM	Avg.	Std.
11022015 VMT	72780.80	67730.80	67924.60	69478.73	2861.31
11022015 VHT	2480.00	2639.30	2011.40	2376.90	326.40
11022015 VMT/VHT	29.35	25.66	33.77	29.59	4.06
10312016 VMT	88670.40	78798.60	70918.80	79462.60	8894.41
10312016 VHT	2064.30	1680.40	1199.00	1647.90	433.56
10312016 VMT/VHT	42.95	46.89	59.15	49.67	8.45
%improvement	46.366%	82.729%	75.151%	68.082%	19.185%

Table: 11022015 vs 10312016 PM peak performance comparison

Evening peak	3-4PM	4-5PM	5-6PM	Avg.	Std.
11022015 VMT	74737.30	70291.70	59258.50	68095.83	7969.61
11022015 VHT	1225.20	1153.10	967.60	1115.30	132.89
11022015 VMT/VHT	61.00	60.96	61.24	61.07	0.15
10312016 VMT	78525.70	72200.70	63029.30	71251.90	7791.65
10312016 VHT	1261.60	1153.60	1005.70	1140.30	128.47
10312016 VMT/VHT	62.24	62.59	62.67	62.50	0.23
%improvement	2.037%	2.671%	2.334%	2.348%	0.317%

Tuesday

Table: 11032015 vs 11012016 AM peak performance comparison

Morning peak	6-7AM	7-8AM	8-9AM	Avg.	Std.
11032015 VMT	81542.70	76460.70	74324.10	77442.50	3708.10
11032015 VHT	2557.90	2193.80	1345.50	2032.40	622.11
11032015 VMT/VHT	31.88	34.85	55.24	40.66	12.72
11012016 VMT	87505.20	79005.90	75101.80	80537.63	6341.98
11012016 VHT	2386.20	2260.00	1516.50	2054.23	469.95
11012016 VMT/VHT	36.67	34.96	49.52	40.38	7.96
%improvement	15.034%	0.302%	-10.348%	1.663%	12.745%

Table: 11032015 vs 11012016 PM peak performance comparison

Evening peak	3-4PM	4-5PM	5-6PM	Avg.	Std.
11032015 VMT	78228.90	78975.40	66074.40	74426.23	7242.52
11032015 VHT	1297.60	1431.80	1187.90	1305.77	122.15
11032015 VMT/VHT	60.29	55.16	55.62	57.02	2.84
11012016 VMT	76929.80	73610.80	68926.80	73155.80	4020.85
11012016 VHT	1455.70	1181.30	1127.10	1254.70	176.17
11012016 VMT/VHT	52.85	62.31	61.15	58.77	5.16
%improvement	-12.341%	12.972%	9.944%	3.525%	13.824%

Wednesday

Table: 11042015 vs 11022016 AM peak performance comparison

Morning peak	6-7AM	7-8AM	8-9AM	Avg.	Std.
11042015 VMT	80982.70	76138.10	73968.00	77029.60	3591.32
11042015 VHT	2521.40	2080.60	1344.60	1982.20	594.54
11042015 VMT/VHT	32.12	36.59	55.01	41.24	12.13
11022016 VMT	87925.90	79222.50	76736.90	81295.10	5875.39
11022016 VHT	2266.00	2051.20	1345.40	1887.53	481.63
11022016 VMT/VHT	38.80	38.62	57.04	44.82	10.58
%improvement	20.811%	5.542%	3.682%	10.012%	9.399%

Table: 11042015 vs 11022016 PM peak performance comparison

Evening peak	3-4PM	4-5PM	5-6PM	Avg.	Std.
11042015 VMT	80253.90	76748.50	67445.10	74815.83	6619.50
11042015 VHT	1408.30	1268.10	1090.10	1255.50	159.47
11042015 VMT/VHT	56.99	60.52	61.87	59.79	2.52
11022016 VMT	80885.10	78152.00	71791.20	76942.77	4665.99
11022016 VHT	1414.90	1469.50	1176.90	1353.77	155.58
11022016 VMT/VHT	57.17	53.18	61.00	57.12	3.91
%improvement	0.316%	-12.127%	-1.407%	-4.406%	6.742%

Thursday

Table: 11052015 vs 11032016 AM peak performance comparison

Morning peak	6-7AM	7-8AM	8-9AM	Avg.	Std.
11052015 VMT	81369.90	77629.80	70722.20	76573.97	5401.80
11052015 VHT	2509.30	1990.40	1259.40	1919.70	627.94
11052015 VMT/VHT	32.43	39.00	56.16	42.53	12.25
11032016 VMT	81020.80	80098.70	77486.10	79535.20	1833.49
11032016 VHT	2635.90	2349.30	1502.10	2162.43	589.55
11032016 VMT/VHT	30.74	34.09	51.59	38.81	11.19
%improvement	-5.211%	-12.582%	-8.139%	-8.644%	3.711%

Table: 11052015 vs 11032016 PM peak performance comparison

Evening peak	3-4PM	4-5PM	5-6PM	Avg.	Std.
11052015 VMT	79101.40	77911.40	68681.90	75231.57	5703.30
11052015 VHT	1272.50	1263.10	1095.60	1210.40	99.53
11052015 VMT/VHT	62.16	61.68	62.69	62.18	0.50
11032016 VMT	83221.10	77856.40	73854.50	78310.67	4699.79
11032016 VHT	1460.50	1312.20	1198.80	1323.83	131.24
11032016 VMT/VHT	56.98	59.33	61.61	59.31	2.31
%improvement	-8.335%	-3.810%	-1.726%	-4.623%	3.379%

Friday

Table: 11062015 vs 11042016 AM peak performance comparison

Morning peak	6-7AM	7-8AM	8-9AM	Avg.	Std.
11062015 VMT	83616.90	75907.00	74025.20	77849.70	5082.40
11062015 VHT	2029.20	1747.90	1253.80	1676.97	392.54
11062015 VMT/VHT	41.21	43.43	59.04	47.89	9.72
11042016 VMT	73868.70	79191.30	76041.90	76367.30	2676.18
11042016 VHT	2094.20	1927.70	1437.80	1819.90	341.22
11042016 VMT/VHT	35.27	41.08	52.89	43.08	8.98
%improvement	-14.400%	-5.404%	-10.422%	-10.075%	4.508%

Table: 11062015 vs 11042016 PM peak performance comparison

Evening peak	3-4PM	4-5PM	5-6PM	Avg.	Std.
11062015 VMT	82779.10	77360.70	80719.60	80286.47	2735.04
11062015 VHT	1572.00	1614.90	1540.90	1575.93	37.16
11062015 VMT/VHT	52.66	47.90	52.38	50.98	2.67
11042016 VMT	77359.60	76008.00	81346.50	78238.03	2775.54
11042016 VHT	1567.70	1549.90	1376.70	1498.10	105.51
11042016 VMT/VHT	49.35	49.04	59.09	52.49	5.71
%improvement	-6.291%	2.372%	12.796%	2.959%	9.557%

Appendix 3. Monitoring of Queue Length by Google Map

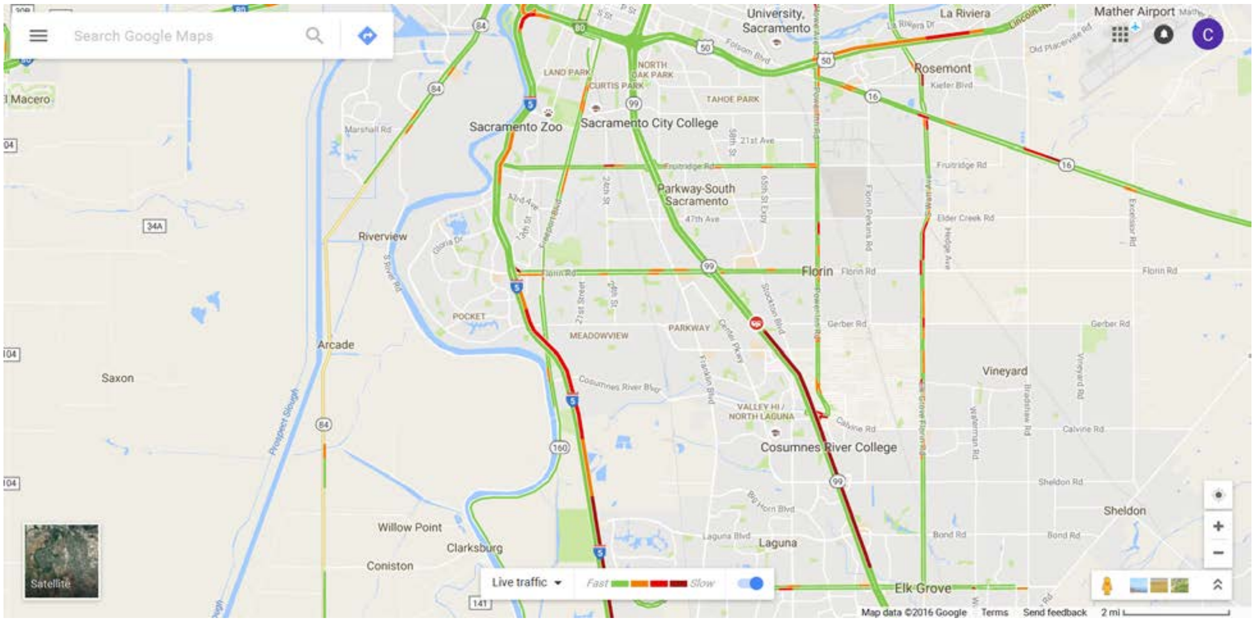

Figure A3-1. Test site overall at 7:24 AM on 10/19/2016

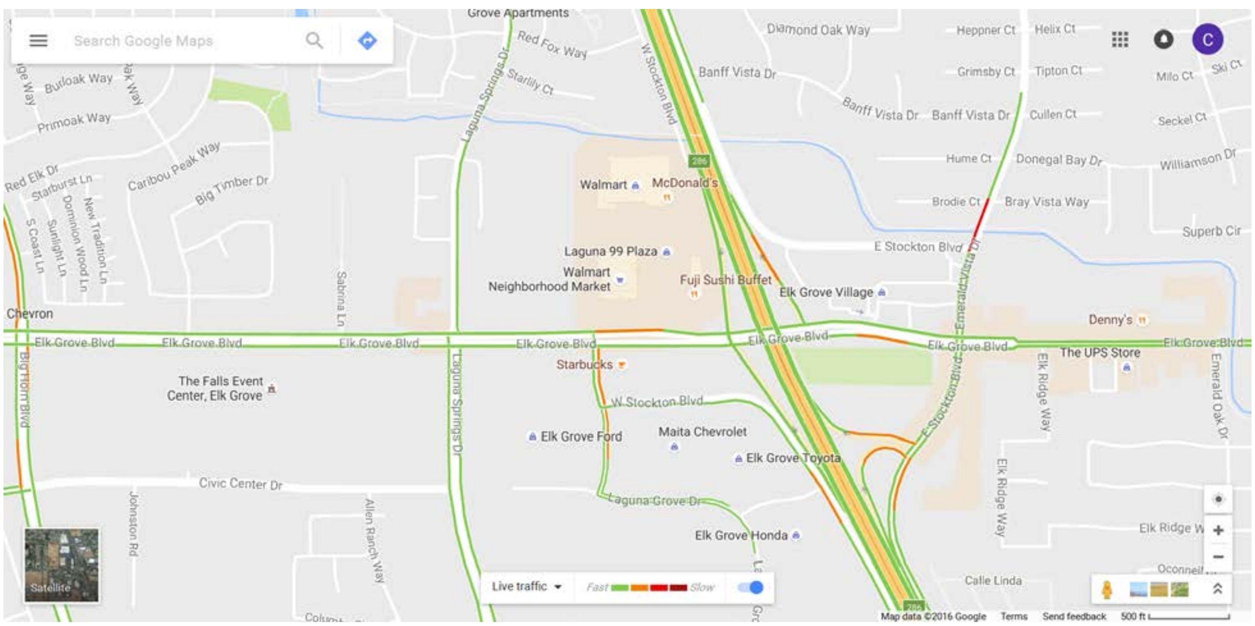

Figure A3-2. Elk Grove Blvd onramp at 7:24 AM on 10/19/2016

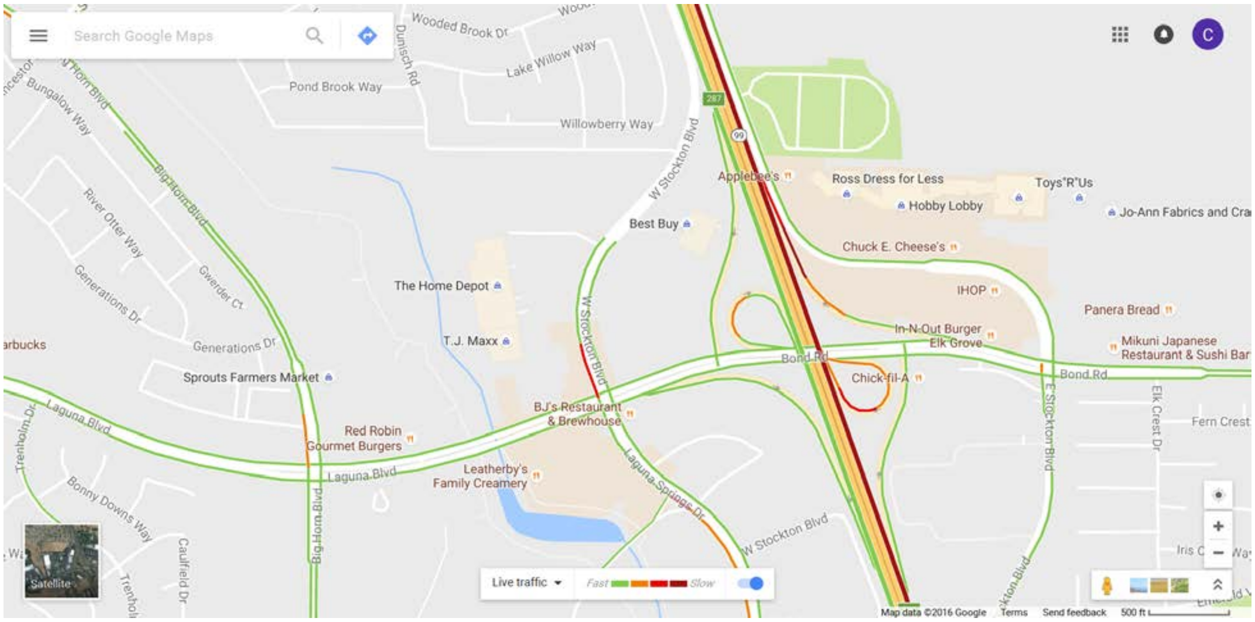

Figure A3-3. Laguna Blvd onramp at 7:24 AM on 10/19/2016

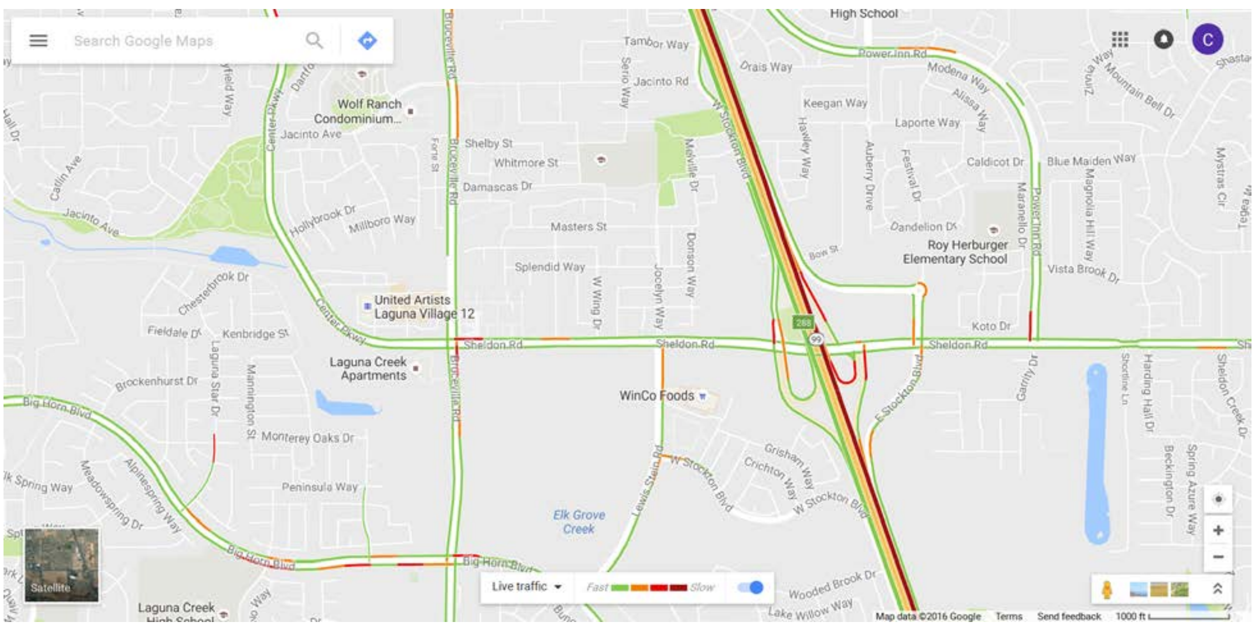

Figure A3-4. Sheldon Road onramp at 7:24 AM on 10/19/2016

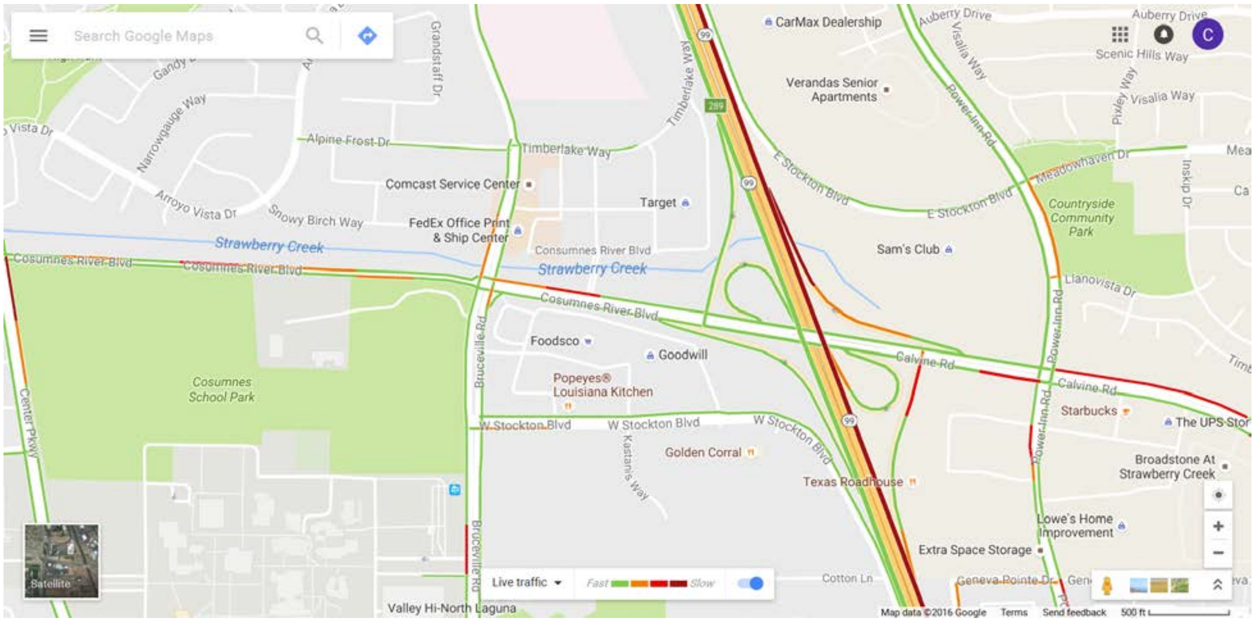

Figure A3-5. Calvine Blvd onramp at 7:24 AM on 10/19/2016

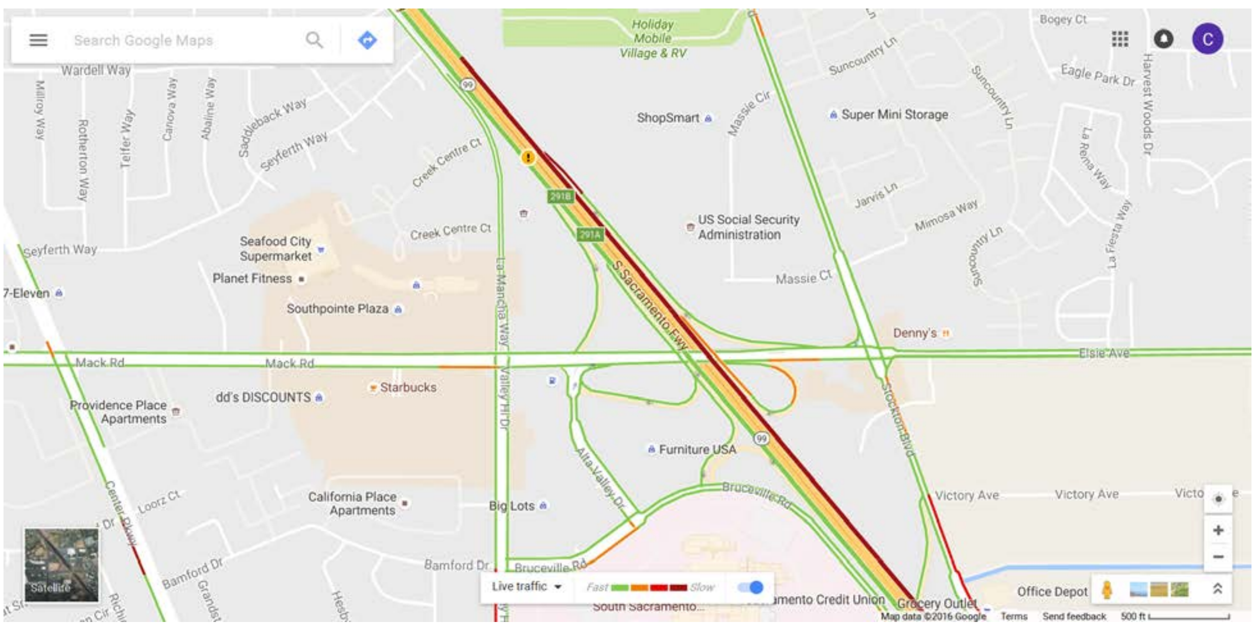

Figure A3-6. Mack Road onramp at 7:24 AM on 10/19/2016

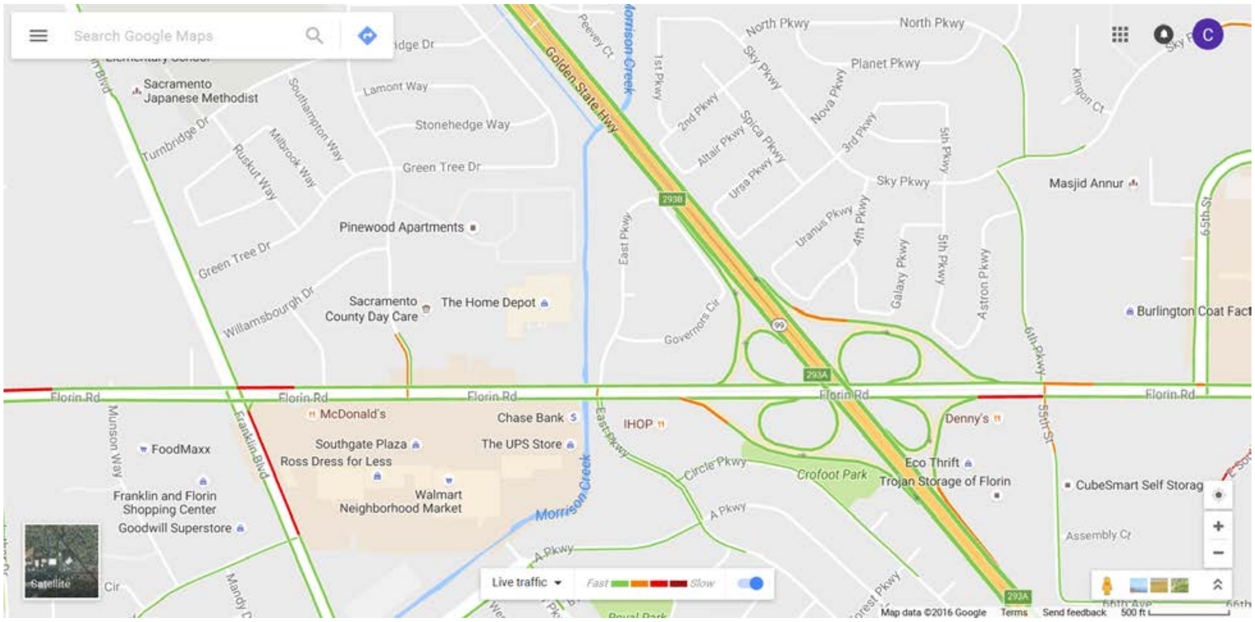

Figure A3-7. Florin Road onramp at 7:24 AM on 10/19/2016

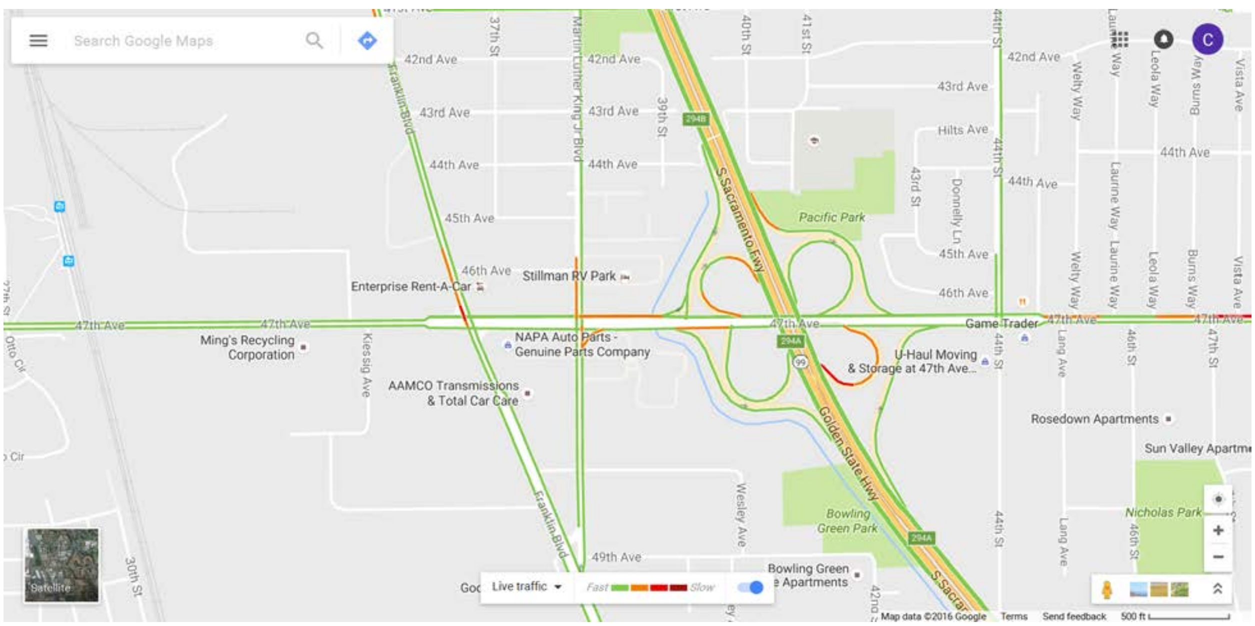

Figure A3-8. 47th Ave onramp at 7:24 AM on 10/19/2016

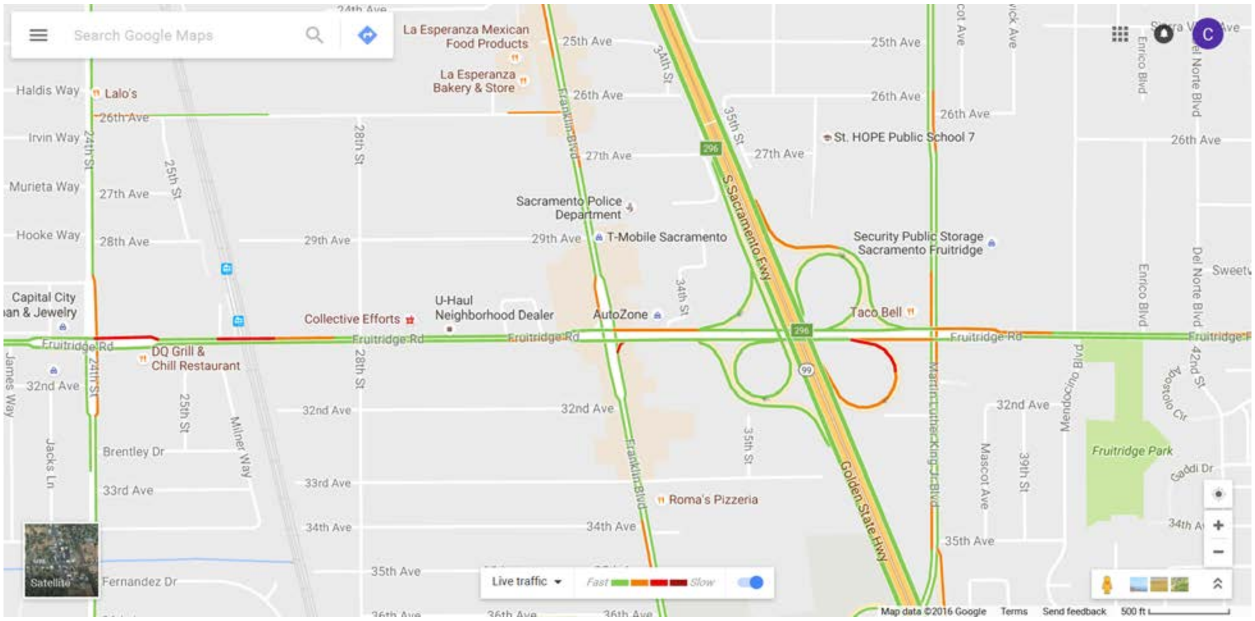

Figure A3-9. Fruitridge onramp at 7:24 AM on 10/19/2016

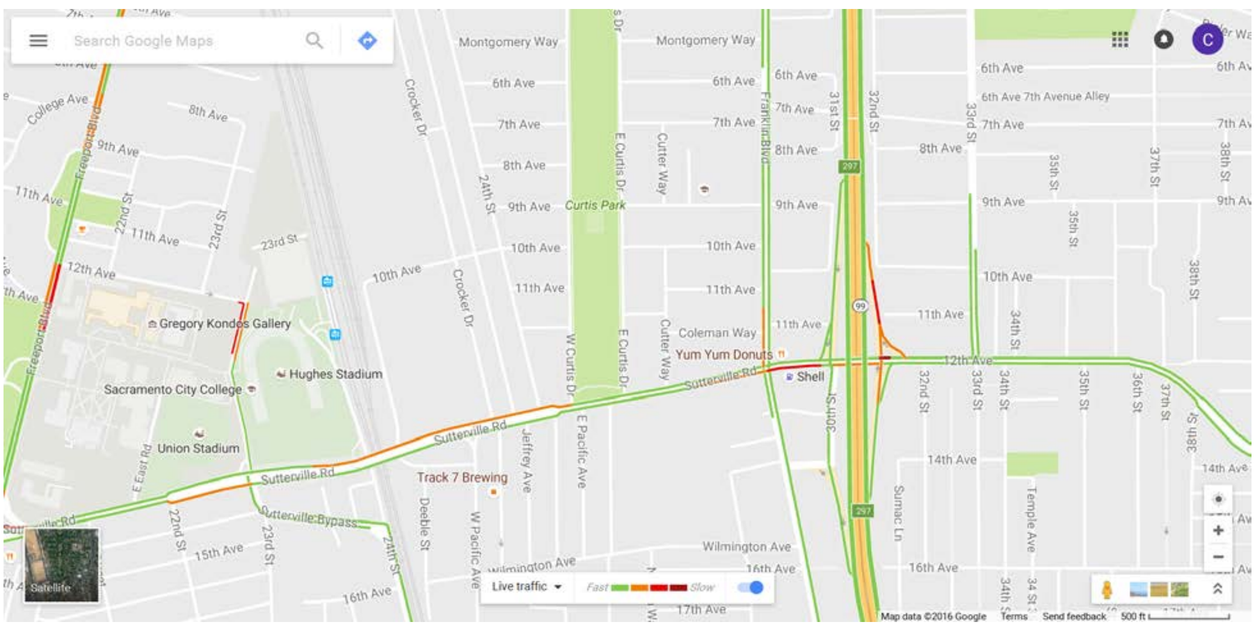

Figure A3-10. 12th Ave onramp at 7:24 AM on 10/19/2016

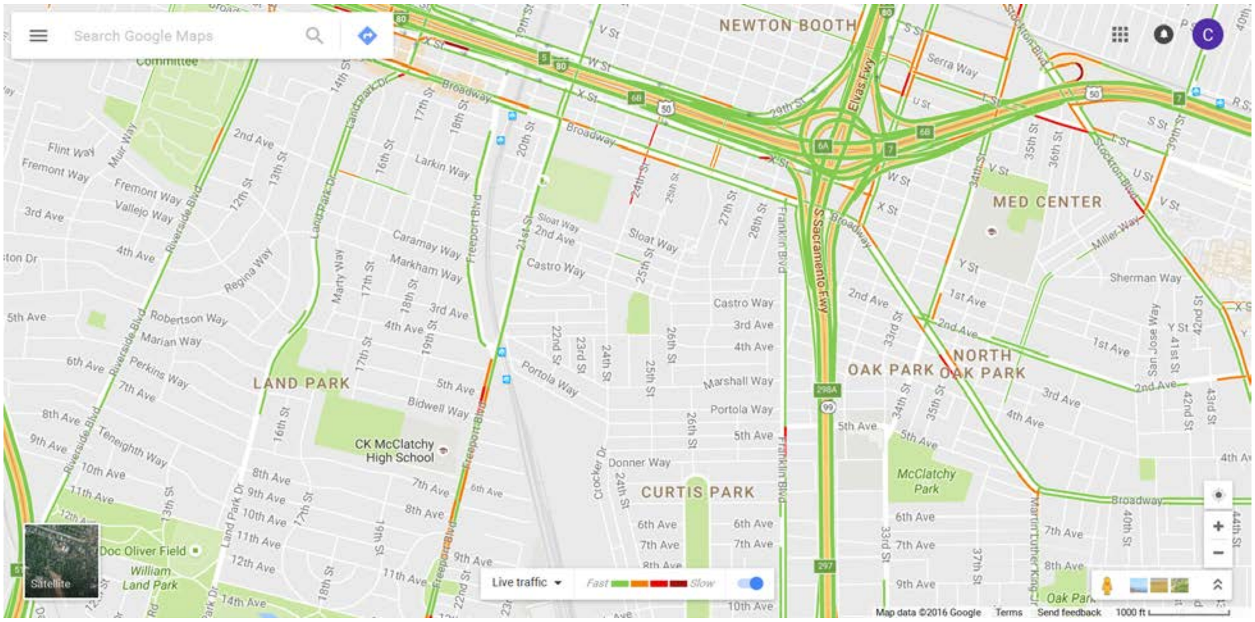

Figure A3-11. SR50 interchange at 7:24 AM on 10/19/2016