STATE OF CALIFORNIA • DEPARTMENT OF TRANSPORTATION
TECHNICAL REPORT DOCUMENTATION PAGE
TR0003 (REV 10/98)

1. REPORT NUMBER	2. GOVERNMENT ASSOCIATION NUMBER	3. RECIPIENT'S CATALOG NUMBER
CA16-2449		

4. TITLE AND SUBTITLE	5. REPORT DATE
Queue Storage and Acceleration Lane Length Design at Metered On-ramps in California	05/03/2016
	6. PERFORMING ORGANIZATION CODE
7. AUTHOR	8. PERFORMING ORGANIZATION REPORT NO.
Zong Tian, Hao Xu, Guangchuan Yang, Arafat Khan, and Yue Zhao	
9. PERFORMING ORGANIZATION NAME AND ADDRESS	10. WORK UNIT NUMBER
Center for Advanced Transportation Education and Research (CATER) University of Nevada, Reno 1664 N. Virginia Street Reno, NV 89557	11. CONTRACT OR GRANT NUMBER 65A0486
12. SPONSORING AGENCY AND ADDRESS	13. TYPE OF REPORT AND PERIOD COVERED
California Department of Transportation Division of Research, Innovation and System Information P.O. Box 942873, MS-83 Sacramento, CA 94273-0001	Final Report 04/03/2013-10/31/2015
	14. SPONSORING AGENCY CODE

15. SUPPLEMENTARY NOTES

16. ABSTRACT

Across California, most of the freeway on-ramps at urban interchanges are either currently being metered, or proposed to be metered in the near future. The current California Department of Transportation (Caltrans) Highway Design Manual (HDM) does not contain specific standards on queue storage design for metered entrance ramps and the standards prescribed in the manual for acceleration lane length design are found to be insufficient at times since at metered on-ramps, approaching vehicles have to stop before picking up speeds in order to merge with mainline traffic.

Under this task, data was collected at several metered on-ramps in California and a simulation model was used to analyze the data and develop comprehensive design guidance for metered on-ramps, in the form of a combination of charts and tables, that will help Caltrans designers and operators with ramp metering applications. The standards developed under this research have been incorporated into the latest update to the Caltrans Highway Design Manual and Ramp Metering Design Manual.

Additional research is recommended to further investigate the relationship between metering rate and freeway volume to model a traffic responsive metering strategy, to investigate the influence of grade on acceleration performance data to provide adjustment factors for the recommended acceleration lengths, and to figure out what is the required auxiliary lane length for various geometric and traffic conditions.

17. KEY WORDS	18. DISTRIBUTION STATEMENT
Queue length, acceleration length, freeway, arterial, flow, merging, simulation model, pilot study, car, truck, vehicle, tractor-trailers, major findings, recommendations, arterial on-ramp, freeway connector	No restrictions. This document is available to the public through the National Technical Information Service, Springfield, VA 22161.

19. SECURITY CLASSIFICATION *(of this report)*	20. NUMBER OF PAGES	21. COST OF REPORT CHARGED
Unclassified	139	

DISCLAIMER STATEMENT

QUEUE STORAGE AND ACCELERATION LANE LENGTH DESIGN AT METERED ON-RAMPS IN CALIFORNIA

Final Report

May 2016

Prepared for the California Department of Transportation

Prepared by

Center for Advanced Transportation Education and Research (CATER)
University of Nevada, Reno

Queue Storage and Acceleration Lane Length Design at Metered On-ramps in California

Final Report

Research Project # 65A0486

Prepare for

California Department of Transportation

Prepare by

Zong Tian, Ph.D., P.E. (PI)

Hao Xu, Ph.D., P.E. (Co-PI)

Guangchuan Yang

Arafat Khan

Yue Zhao

Center for Advanced Transportation Education and Research (CATER)

University of Nevada, Reno

1664 N. Virginia Street, MS258, Reno, NV89557

May 3, 2016

EXECUTIVE SUMMARY

Project Purposes

The California Department of Transportation (Caltrans) is committed to using ramp metering as an effective traffic management strategy for freeway operations. One of the major challenges currently facing Caltrans in designing and operating ramp metering is related to queue storage and acceleration length design. The location of the metering signal is the governing factor since it affects both queue storage and acceleration. When queue storage space is insufficient, vehicle queues may spillover to the upstream arterial signals to impact the normal operations of surface street networks. On the other hand, as on-ramp vehicles have to accelerate from zero to a safe merging speed to merge onto freeway mainline, insufficient acceleration distance may not allow on-ramp vehicles to safely and adequately attain desired merging speeds. Merging problems and potential crashes may lead to increased congestion, diminishing the major purpose of ramp metering.

The metered on-ramp design should therefore fully consider a balance between queue storage space and acceleration distance. The two major relevant design documents which Caltrans currently maintains, the *"Highway Design Manual"* (HDM) and the *"Ramp Metering Design Manual"* (RMDM), do not contain specific standards regarding metered on-ramp design. The primary purpose of this research project is to fill this gap, providing new recommendations to be accommodated into the above two documents. The key objectives of the project are to:

- investigate and identify factors affecting queue storage and acceleration length needs at metered on-ramps;
- develop models and tools for estimating queue length considering different types of arrival, metering methods, and demand levels;
- develop a methodology for determining acceleration lane length;
- develop standards for queue storage length, acceleration length, and to produce technical documents that are readily adopted into existing Caltrans design manuals.

Report Overview

This research is for studying queue storage and acceleration lane design at metered on-ramps in California. The document synthesizes current practices regarding metered on-ramp design, presents research methodologies, and makes recommendations to assist Caltrans planning and design of safer metered on-ramps. The major content of this report involves background introduction of ramp metering operations and design, literature review of metered on-ramp queue storage and acceleration design, summary of lessons learned from the preliminary pilot study, development of queue length simulation model and acceleration length prediction method, development of metered on-ramp design standards, and summary of major findings. The core methodology used for queue storage design is a mesoscopic simulation model based on the input-output method; for acceleration length design, the regression technique was used based on field measured speed profile data to predict the required acceleration length at given merge speeds. The deployable products are comprehensive design guidance for metered on-ramps, in the form of a combination of charts and tables that should help both designers and operators with ramp metering applications. Ultimately, the findings and conclusions will be of critical importance for updating Caltrans Highway Design Manual and Ramp Metering Design Manual.

Major Findings

Queue Length Modeling

- Ramp queue is mainly affected by the on-ramp demand, trafficl flow arrival pattern, and metering rate. Other influencing factors include: upstream signal timing, lane usage, right-turn-on-red vehicles, and potential violation of ramp metering rules.
- For under-saturated conditions, ramp queue is mainly caused by the vehicle platoons released from the upstream intersection.
- For over-saturated conditions where ramp demand exceeds metering capacity, the queue length becomes unstable and is more difficult to accurately predict.
- The on-ramp flow arrival pattern plays a critical role in ramp queue length; an accurate description of the on-ramp flow arrival profile would help to capture the real-time queueing process and thus improve queue length modeling results.

Acceleration characteristics

- Acceleration rate at metered on-ramps is not constant; drivers tend to accelerate at a higher acceleration rate when speed is lower and vice-versa.
- Existing acceleration length is the primary factor affecting drivers' acceleration behavior and consequently, the required acceleration lengths.
- The acceleration profile for trucks has shown a similar trend as that of passenger cars; in general, the acceleration capability for heavy trucks is approximately 60 percent of that for passenger cars.

Recommendations

Queue Storage Design for Metered On-ramps

- The 95th percentile queue length is recommended for queue storage design.

- Queue storage length is recommended to be designed as a certain percentage of peak hour on-ramp demand; the percentage number varies with different ramp configurations and on-ramp flow arrival patterns.
- The maximum recommended percentage values are approximately 8 percent for urban arterial metered on-ramps and 4.3 percent for freeway-to-freeway connectors. The percentages are much lower than these numbers when the demand-to-capacity ratio is below 0.6.

Acceleration Length Design for Metered On-ramps

- Acceleration lane length design should be based on the 15th percentile acceleration rate (i.e., 85 percent of the drivers can safely achieve the required merging speed) so as to accommodate the majority of vehicles.
- A dual-level acceleration length design is recommended to accommodate the unique requirements of metered on-ramps: the conservative design is recommended for ramps that have sufficient space (both existing and proposed metered on-ramps); while the aggressive design recommendation could be used for existing metered on-ramps that have insufficient ramp space or recurrent ramp queue spillovers.
- The AASHTO Green Book design guidance could be reduced by 10 percent (conservative design recommendations) to 35 percent (aggressive design recommendations) for passenger cars.
- Acceleration lengths for heavy trucks are approximately 1.6 times of AASHTO standard for passenger cars.

The details of the design recommendations are included in Chapter 6.

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

DISCLAIMER

The contents of this report reflect the views of the authors who are responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official views or policies of the California Department of Transportation. This report does not constitute a standard, specification or regulation.

ACKNOWLEDGEMENTS

This report is prepared in cooperation with the California Department of Transportation (Caltrans). Special thanks to Dr. Zhongren Wang from Caltrans Head Quarter for his discussions and constructive suggestions throughout this research. The authors thank Larry Hall and Leo Anselmo from Caltrans District 3, Afsaneh Razavi from Caltrans District 7, Jose Perez from Caltrans Division of Research, Innovation, and System Information (DRISI), Dian Mao, Andrew Jayankura, Daniel Rodriguez from University of Nevada, Reno (UNR) for their help with the site selection, device testing, and field data collection; Daniel Rodriguez and Rui Yue from UNR for their help with queue length data extraction; Dr. Daobin Wang from UNR for help with coding the freeway connector simulation model; Anabel Hernandez and Erika Hutton from UNR for help with editing the report.

Project Panel List

Name	Organization/Branch	Phone	Email
Jose Perez	Caltrans/DRISI	916-654-9390	jose.d.perez@dot.ca.gov
Zhongren Wang	Caltrans/HQ	916-654-6133	zhongren_wang@dot.ca.gov
Larry Hall	Caltrans/D03	510-622-8718	larry.hall@dot.ca.gov
Leo Anselmo	Caltrans/D03	916-859-7954	leo.anselmo@dot.ca.gov
Lester Lee	Caltrans/D04/Traffic Sys.	510-286-4528	lester.s.lee@dot.ca.gov
Afsaneh Razavi	Caltrans/D07	323-259-1841	afsaneh.razavi@dot.ca.gov

Zong Tian	UNR CATER	775-784-1232	zongt@unr.edu
Hao Xu	UNR CATER	775-784-6909	haox@unr.edu
Guangchuan Yang	UNR CATER	775-784-6906	gyang@unr.edu
Arafat Khan	UNR CATER	775-338-9105	sagar_buet@yahoo.com
Yue Zhao	UNR CATER	775-997-6815	yuezhao118@gmail.com

1. INTRODUCTION

1.1 Research Background

Ramp metering was first implemented in 1963 on the Eisenhower Expressway (Interstate 290) in Chicago, Illinois. Since then it has been systematically deployed in major urban areas, including many locations in California *(1,2)*. As a traffic control device installed at on-ramps, ramp metering regulates and controls the traffic demand entering a freeway. It temporarily stores the entering traffic on the ramps to alleviate congestion on freeways using the "access rate reduction technique". This "access rate reduction" technique ensures total traffic demand not to exceed the freeway capacity. Another objective of ramp metering is to break up vehicle platoons entering the freeways to promote a smoother and safer merging maneuver. A study conducted in Minnesota pointed out that when ramp meters were turned off, freeway throughput decreased by 9 percent, travel time increased by 22 percent, speeds dropped by 7 percent, and crashes increased by 26 percent *(3)*. Likewise, studies conducted in the San Francisco Bay Area indicated that ramp metering systems reduced travel time by 30 percent along an 18-mile long stretch of Route 580 *(4)*.

In California, the California Department of Transportation (Caltrans) is committed to using ramp metering as an effective traffic management strategy to maintain efficient freeway operations by operating the freeways at or near capacity. By December 2013, Caltrans manages 2,802 ramp meters that account for more than 60 percent of all existing ramp meters in the U.S, and 1,642 ramp meters are going to be planned throughout the state *(5)*. Effective ramp metering strategies are of significant importance for Caltrans particularly because of the level of congestion on California freeways, HOV lanes, HOT lanes, and an expanded arterial network that is an integral part for providing mobility to people and businesses in California. Although ramp metering has proven to be an effective freeway management strategy, there are several challenges involved in designing and operating ramp metering. These challenges mostly stem from the queue storage length issue. The majority of the ramp-metering locations in the U.S. are retrofitted to existing ramps where sufficient queue storage length is not available. Most ramp metering locations are in the vicinity of surface street signals where platoon arrivals from the upstream signals exacerbate the queue storage and spillback issues. Local agencies that manage surface street systems generally do not wish queue spillover from ramp metering. A typical practice is to apply a queue override strategy when spillover occurs *(6,7)*. Nevertheless, such a queue override strategy can diminish the primary purpose of ramp metering to improve freeway operations. On the other hand, when an on-ramp is metered, approaching vehicles have to stop at the ramp meter signal before accelerating again and merging to the freeway mainline traffic flow. Insufficient acceleration lane length could have significant highway performance and safety implications. Vehicles unable to accelerate to freeway mainline speeds will cause delays at the interchange as well as increase the potential for collisions. Therefore, an accurate prediction of required queue storage length and the corresponding acceleration length is necessary for optimally designing and operating ramp meters to achieve the goals of improving safety, reducing congestion, and reducing vehicle emissions.

1.2 Problem Statement

Although ramp metering has proven to be an effective freeway management strategy, there are several challenges involved in designing and operating ramp metering. A metered on-ramp consists of two parts, the upstream queue storage part and the downstream acceleration part. Under normal conditions, the approaching vehicles have to stop at the metering signal before the metering signal turns green. After passing the signal, vehicles accelerate to a desired merging speed to join the mainline traffic. Such a traffic flow process poses unique challenges for the design and operations of ramp metering. The two major issues are related to queue storage length and acceleration lane length design.

Across the State of California, most of the freeway on-ramps at urban interchanges are either currently being metered or are proposed to be metered in the near future *(5,8)*. Successful operations at metered on-ramps call for both sufficient queue storage to avoid on-ramp queue overspill and sufficient acceleration length to reach desired safe merging speeds. However, the current *"Caltrans Highway Design Manual"* (HDM) *(9)* and *"Ramp Meter Design Manual"* (RMDM) *(10)* do not contain specific standards concerning queue storage length design for metered ramps. Abundant information is available for dealing with freeway and ramp operation and coordination; but there is little pertaining to the design of metered on-ramps. Consequently, engineers have to design the queue storage length based on similar existing on-ramps or using Arrival-Discharge Charts. Some guidelines are available to approximate queue storage length based on the 7 percent rule, i.e., the number of vehicles in queue at a metered ramp is about 7 percent of the peak hour volume *(11,12)*. Such simple guideline recommendations do not take into consideration the unique and varying traffic patterns and metering methods, particularly with different vehicle arrivals due to an upstream signal, and fixed versus responsive metering.

Additionally, approaching vehicles have to stop at the ramp meter before speeding to merge with mainline traffic. The current standards prescribed in the Caltrans HDM *(9)* and AASHTO (American Association of State Highway and Transportation Officials) Green Book, *"A Policy on Geometric Design of Highways and Streets" (13)* for acceleration lane length design is found to be insufficient at times. Vehicles, especially buses and large trucks departing the limit line may not have sufficient acceleration length to reach safe merging speeds. These challenges can jeopardize the acceptance and implementation of ramp metering. The Department may also be exposed to potential tort liabilities. It is necessary to study the existing acceleration lane length design guidelines, acceleration manners of different types of vehicles (e.g., passenger car, single-unit truck, semi-tractor trailer trucks and buses), and merging behaviors around the freeway merging areas.

In short, an ample storage capacity key to a successful ramp metering program. Most state DOTs would like to have longer and wider ramps to prevent queues from extending beyond the ramps onto the arterials. If long queues with backups onto the arterials occur on a consistent basis, implementation of queue detection systems and adoption of a more conservative strategy may be necessary. However, for retrofitted ramps, longer queue storage space implies shorter acceleration lane lengths. To further exacerbate the situation, previous research has found that the current design provided by the AASHTO Green Book which is executed by most states in the U.S. is insufficient for trucks *(14,15,16)*. More efforts need to be made to improve the estimation of required queue storage space and acceleration lane length design at metered on-ramps.

1.3 Research Objectives

This research addressed the problems described above. The main objectives are to develop methodologies for on-ramp queue storage length estimation at new or reconstructed interchanges, and to investigate vehicles' actual acceleration capability at metered on-ramps. Standards on queue storage length design and acceleration length design at metered on-ramps will then be developed. The deployable product is a comprehensive design guidance for metered on-ramps that helps both designers and operators with ramp metering applications. The guideline will also help alleviate the potential tort liability the Department might be exposed to in terms of queue overspill and insufficient acceleration length at metered on-ramps. Ultimately, the conclusions and findings from this research are of critical importance for the update of the Caltrans Ramp Metering Design Manual and Highway Design Manual.

A series of research activities were carried out to achieve the research goals including:

- Review existing methodologies and practices pertaining to queue length estimation and acceleration length design.
- Investigate and identify factors affecting queue storage and acceleration length needs at metered on-ramps.
- Collect data at representative locations considering (i) diverse metering methods, e.g. fixed metering rates versus responsive metering; (ii) different vehicle arrival patterns, e.g. random versus platoon; (iii) different demand levels, e.g. under-saturated versus over-saturated.
- Develop models and tools for estimating ramp queue length considering different types of arrival, metering methods, and demand levels.
- Develop a methodology for estimating required acceleration length.
- Provide recommendations for queue storage length and acceleration length.
- Produce technical documents that are readily adopted into existing Caltrans design manuals.

2. LITERATURE REVIEW

This chapter presents a comprehensive review of literature related to studies about metered on-ramp design. The review will cover the following four aspects of metered on-ramp design: (i) the state-of-the-art practice and issues with queue length estimation; (ii) the existing methodologies for estimation of required queue storage; (iii) the studies regarding the acceleration characteristics; and (iv) existing methodologies and practices for design of acceleration lane length.

2.1 Queue Length Estimation Methodologies

A ramp meter is a standard queuing system. A vehicle queue is formed behind a ramp metering signal when the vehicle arrival rate exceeds the ramp metering rate. The queue may exist temporally due to a short-term surge of traffic arrival (e.g., a platoon arrival) or it can be prolonged due to over-saturation over an extended period of time. Various methodologies have been developed for estimating on-ramp queue length and hence the storage requirement at metered ramps, as demonstrated in Wang's study *(12)*. Most of these methodologies are based on the input-output (also known as cumulative arrival and departure) method. The arrival rate is the on-ramp demand and the departure rate is the metering rate and thereby the queue length is the accumulated difference between the arrival and departure rate over time. The commonly used methodologies in practice are described below.

In the early 1990s, Rodney Oto, an engineer from Caltrans proposed a storage design method for metered on-ramps *(12)*. This method assumed that the maximum tolerable individual delay was already obtained from either field observation or other resources. The fundamental input-output diagram was the basis of his method. According to Oto's observations in the San Francisco Bay Area, the maximum individual delay was between 8-10 minutes for ramps with a metering rate of 300 vph. It can be seen that a 10-minute delay is rather long while the metering rate is rather low. This methodology was apparently based on the assumption of a known platoon rate and duration. Currently, the Roads Corporative of Victoria in Australia *(17)* is executing this method where 4 minutes maximum delay and an average arrival rate instead of metering rate are used to calculate the storage needs. The advantage of this method is that there is no need to obtain the actual on-ramp demand; however the model cannot be applied universally since the maximum delay threshold is related to the platoon size and rate which vary by location.

Another widely used approach for estimating queue storage length is to assess some similar existing on-ramps. In the current Caltrans Ramp Metering Design Manual *(10)* and the Arizona's Ramp Meter Design, Operations, and Maintenance Guidelines *(18)*, the queue storage length is determined using the arrival-discharge chart method. The arrival-discharge counts at short intervals, usually 5-6 minutes, were conducted to estimate the queue. Essentially, when the arrival curve (arrival counts or on-ramp demand) falls above the discharge curve (discharge counts or metering rate), queue forms and accumulates. As a result, the maximum queue length, total delay, total number of vehicles delayed, and the average delay can be obtained. Basically, this method is similar to the input-output method with consideration of the short-term over-saturation. Nevertheless, this method is not very practical because it simply measures the queue during over-saturation intervals. In reality, during under-saturation situation, queue may also be formed due to the vehicle platoons released from the upstream signalized intersection. Additionally, there is no consideration of the situation when over-saturation lasts for an extended time interval. Therefore the residual queues are not carried over to the next interval which indicates that this method will recommend insufficient queue storage length for design purposes.

Besides the applications using the queuing theory, the Texas Transportation Institute (TTI) developed mathematical models to predict on-ramp queue storage length. An early study conducted in 1994 *(19)* presented a mathematical methodology for determining the distance requirements for metered on-ramps. This study took into account light trucks at metered ramps by assuming the required space of 25 feet per truck. The proposed queue length estimation model is depicted as follows:

$$\boldsymbol{L_Q = \frac{0.122(\alpha V T)}{1+\frac{T}{D}}}$$

Where, L_Q is the length of the queue in meters, V is the vehicle arrival rate in vehicles per hour (vph), T is the analysis time period under consideration in minutes, D is the acceptable ramp delay in minutes and α is a constant corresponding to 95 percent Poisson arrivals.

Later on, the update of the Texas DOT's Roadway Design Manual *(20)* adds detailed discussions of criteria for ramp design with explicit consideration of ramp metering. Regression models were developed to predict on-ramp single-lane storage length as (i) Storage Length in feet: $L_Q = 0.25V - 0.00007422V^2$ $(V \leq 1600vph)$; where V is the peak hour volume and (ii) Storage (number of vehicles in queue): $S_Q = \left((3.28 \times 10^{-2} - 9.74 \times 10^{-6}V) \times 100\%\right)V$. It can be seen that merely peak hour volume (on-ramp demand) is considered as an independent variable in the model.

In reality, real-time on-ramp queue lengths are dynamically related to on-ramp demand, arrival pattern and metering rate. Abundant information is available for dealing with real-time queue length estimation at metered on-ramps. In Wu et al.'s studies *(21,22)*, three types of methods to estimate the real time on-ramp queue lengths were discussed including Kalman filter, linear occupancy and HCM back of queue. The authors compared the estimated queue using three approaches with field observed queue lengths. They concluded that Kalman filter and linear occupancy are usable for real world operations but both approaches have certain limitations. The HCM method on the other hand does not produce reliable estimates. In addition, Sun and Horowitz *(23)* designed an on-ramp vehicle queue length regulator to prevent queue spillback to surface streets from ramp metering. The underlying theory was based on the Kinematic theory.

The queue data detected from the loop detectors was used to estimate the on-ramp queue length. The authors pointed out that since a majority of on-ramp queue detectors are single inductive loop detectors, it is quite hard to directly measure vehicle speeds in the queue. The single loop detector speed data is usually estimated based on fundamental traffic flow theory that involves variables of loop length, average vehicle length, occupancy and traffic volumes. Similarly, Vigos et al. *(24,15)* developed an improved algorithm to estimate the number of on-ramp queued vehicles by employing a Kalman Filter and using data from queue loop detectors. Time occupancy data collected by loop detectors is translated into space occupancy data, which is directly related to the number of on-ramp vehicles. Sanchez et al. *(26)* discussed the potential of using magnetic sensors for queue length estimation. Results showed that estimated queue using vehicle real-time re-identification method could properly correlate with field observed data when an on-ramp was uncongested, but it underperformed during saturated on-ramp conditions. This is mainly because the testing algorithm did not take into account on-ramp specific factors such as ramp curvature, slope, vehicle headway, sensor location. In summary, these studies found that the Kalman-Filter and vehicle magnetic detection approaches produced some useful results regarding real-time queue length estimation. Nevertheless, the purposes of these studies were mostly focusing on real time control rather than infrastructure design. The queue length estimation methodologies and strategies developed in these studies will thus not solve the design issues and challenges which this research focuses on.

The commonly used methodologies to estimate required queue length at metered on-ramps in practice are summarized in Table 2-1.

Table 2-1 Summary of Queue Length Estimation Methodologies

Methodology	Assumptions	Equations and Concepts	Applications	Advantages	Disadvantages
Maximum Individual Delay Based Estimation	Maximum tolerable individual delay (d_{max}) is known.	$Queue = \frac{d_{max}}{60} \times \mu$ where, μ is the metering rate.	The Roads Corporative of Victoria in Australia uses 4 minutes maximum delay to calculate the storage needs.	No need to obtain the actual on-ramp demand.	The model cannot be applied universally since the maximum delay threshold is related to the platoon size and rate which vary by location.
Average Delay Based Estimation	Average time a vehicle spends at on-ramps ($\bar{d}$) is known.	$Queue = \left[\frac{\bar{d}}{60}\right] \times \lambda$ where, λ is the average arrival rate.	Minnesota; Wisconsin.	Queue is estimated as a certain percentage of on-ramp demand.	Using the average queue as design criterion is insufficient.
Arrival-Discharge Chart	For retrofitted ramps or new construction, the chart can be used at existing on-ramps to mimic the design situation.	Count vehicle arrive and discharge in short intervals (e.g., 5, 6 or 15 minutes) to determine the queue.	California; Arizona.	Similar to the input-output method with consideration of short-term over-saturation.	The 5, 6, or 15 minutes intervals are way too long to capture the maximum queue which should be used for the storage design.
Mimic Signalized or Unsignalized Intersections	The storage length design at signalized intersections has the same pattern as metered on-ramps.	$Queue = \left(\frac{C}{360}\right) \times \left(\lambda \times \frac{D}{PHF}\right)$ where, C is the signal cycle length, λ is the average hourly arrival rate, D is the lane distribution factor and PHF is the peak hour factor.	Nevada	Easy to collect the data.	Cannot reflect the actual freeway on-ramp operation and design.
Mathematical Modeling	The arrival rates were assumed to be range from 200 vph to 800 vph. The acceptable delay ranged from 1 minute to 5 minutes. The analysis time periods of 2 minutes and 4 minutes were used.	$L_Q = 0.122(\alpha VT)/(1 + T/D)$ where, V is the vehicle arrival rate; T is the analysis time period; D is the acceptable ramp delay and α is a constant corresponding to 95% Poisson arrivals.	Texas	Queue is estimated as a function of on-ramp demand.	The regression model was based on Texas data and only on-ramp demand is considered as an independent variable in the model.
	Arrival rate less than 1,600 vph.	$L_Q = 0.25V - 0.00007422V^2$ where, V is the arrival rate.	Texas		
Real Time Queue Length Estimation	--	Estimate real time queue length to adjust the metering rate. For instance, metering rate will increases when on-ramp queue becomes longer than a certain threshold.	Not applicable for design.	Regulate the queue length to prevent queue spillback.	This is real-time control instead of infrastructure design. Accurate real time queue length is very difficult to achieve.

2.2 Guidelines for Queue Storage Design

Currently, only a handful of states in the U.S. have specific guidelines readily available for use in designing and operating ramp-metering systems. In the State of California, the basic references for ramp metering design include the Caltrans *Highway Design Manual* and *Ramp Meter Design Manual*. In terms of queue storage length, both manuals emphasized that every effort should be made to meet the recommended storage length calculated based on the Arrival-Discharge Chart to minimize the impact on local street operations. Wherever feasible, ramp metering storage should be contained on the ramp by either widening or lengthening it. Moreover, an extensive ramp metering evaluation study was conducted in Los Angeles along the westbound (WB) I-10 between Route 110 and Overland Ave in 1978 *(11)*. This study tended to find a certain percentage of peak hour demand that can be used universally for queue storage design. Eventually 7 percent was found to be widely accepted in practice based on the data analysis.

The State of Minnesota and the State of Wisconsin determine queue length as a certain percentage of the arrival flow rate. In its Traffic Engineering Manual, the Minnesota Department of Transportation (MnDOT) *(27, 28)* recommends that a minimum of 300 feet between the ramp control signal and the nose (end of physical curb separation between ramp and freeway) be provided for metered freeway entrance ramps acceleration lane length design. The minimum storage length of 25 feet per vehicle for a six-minute metered volume between the cross street and the ramp control signal is recommended for queue storage length design. The manual further recommends that two-lane ramps with a single-lane entrance should be provided for all ramps with projected volumes of 500 vph or greater. The Arizona Department of Transportation (AZDOT) also points out that a two-lane storage area should be considered for ramps having a peak hour volume between 500 and 900 vph, and a two-lane storage area should be provided for all ramps with peak hour volumes greater than 900 vph *(18)*. Similarly, the Wisconsin Department of Transportation (WisDOT) *(29)* uses a minimum of 10 percent of the current peak hour volume as a criterion to decide the storage needs. For reconstruction, an on-ramp should accommodate a minimum of 10 percent of the design year projected peak hour volume. For ramp meters retrofitted onto existing infrastructure, a minimum storage of 5 percent of the current peak hour volume may be used with additional approval. The drawback of this method is that it merely uses the average queue as design criterion. The average queue is insufficient to some extent, because the actual maximum queue may be much higher than the average queue with typical platoon arrivals. Apparently this method does not take into consideration such critical conditions.

The Texas Roadway Design Manual provides design criteria including regression equations for queue storage length prediction *(20)*; detailed equations were discussed in the previous section. Additionally, TTI recommended providing a minimum stopping distance of 250 feet from the center of upstream signals to the back of the expected queue storage area, and an additional minimum storage length of 450 feet along the ramp to the meter *(30)*.

In most of the methods and guidelines reviewed, the on-ramp storage length was recommended to be designed as a certain percentage of peak hour demand which ranged from 2 percent to 10 percent. This percentage number varied by jurisdictions and the methods applied as summarized in Table 2-2. However, no universally accepted methodology or standard can be simply adopted. Each methodology has its own limitations and assumptions.

Table 2-2 Summary of Queue Storage Design Guidance

Jurisdiction		Methodology	Design Guideline
United States	California	Arrival-Discharge Chart	7%
	Minnesota	Average Delay-Based Estimation	10%
	Texas	Mathematical Modeling	1.72%-3.3%
	Wisconsin	Average Delay-Based Estimation	5% (retrofitted) 10% (new construction)
	Ohio	Recommended in the state design manual	10%
	Nevada	Mimic Signalized Intersection	3.3%
	New York	Recommended in the state design manual	5%
	Connecticut	Recommended in the state design manual	5%
Overseas	Australia	Maximum Individual Delay-Based Estimation	6.67%

2.3 Research of Vehicle Acceleration Characteristics

Acceleration characteristics of different vehicle types vary and are usually influenced by prevailing traffic conditions and road geometric features. Several studies have been performed to describe the speed and acceleration profiles of different vehicles at various road facilities. In general, these studies can be divided into two categories: models based on vehicle dynamic, and models based on kinematic theory.

Kinematic Theory Model

The kinematic theory based model, which takes into account the mathematical relationships between speed, acceleration, and distance (or time) of moving objects, is the most commonly used acceleration measurement model. Previous kinematic based acceleration behavior studies proposed four different acceleration models: (a) constant model, (b) dual-regime model, (c) linear decreasing model, and (d) polynomial model. Figure 2-1 presents a qualitative illustration of typical speed versus time and associated acceleration versus time profiles in accordance to the above mentioned acceleration models.

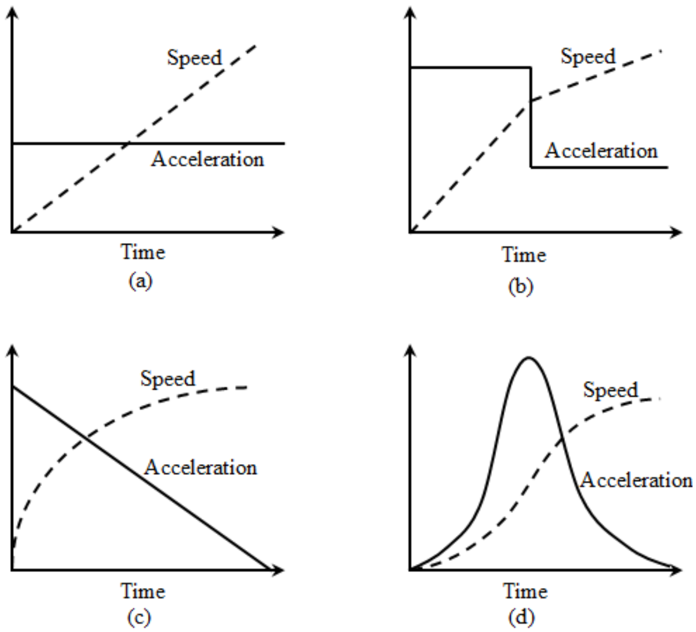


Figure 2-1 Four Common Acceleration Models.

The constant acceleration model (Figure 2-1,a) assumes acceleration rate of a vehicle is constant for the entire accelerating process; accordingly, acceleration lengths could be calculated based on the given initial speed and reached speed. A study by Firzpatrick and Zimmerman *(31)* made a recommendation that a constant acceleration rate of 2.5 ft/s^2 should be used to generate potential acceleration lengths of freeway on-ramps.

In reality, vehicles usually accelerate at a higher acceleration rate when the speed is lower, and vice versa. On consideration of this, Bham *(32)* proposed a dual-regime acceleration model (Figure 2-1,b). This model divided the entire acceleration period into two parts, one for lower speeds and another for higher speeds; each part is exactly a constant acceleration model. Their field data indicated that the breaking point of the two parts is at around 13 m/sec (29 mph).

To better describe the actual acceleration characteristics, linear decreasing models (Figure 2-1,c) were developed. These models also assume that acceleration rates vary inversely with the speed. Vehicles will attain maximum acceleration rate at speed zero, and the acceleration rate decreases linearly to zero when the vehicle reaches the maximum speed. Long *(33)* summarized a list of typical parameters included in the linear decreasing models and concluded that such model performs successfully for both maximum vehicle acceleration and normal motorist-chosen acceleration. However, he also pointed out that the model may not be completely accurate for the starting up stage, since there are relatively few data points collected for this stage.

Several polynomial acceleration models (Figure 2-1,d) were proposed in attempts of capturing the realistic condition more accurately. Ackelik *(34)* pointed out that the linear decreasing models assume high initial acceleration value which is usually unrealistic. To satisfy the realistic condition, in his study the following statement for a realistic acceleration model was described: speed-time profile should indicate an *S* shape; zero jerk and zero acceleration rate at the start and end of the acceleration. Accordingly, a polynomial acceleration model was proposed. Evaluation results show that it is more accurate in comparison with the constant and linear decreasing models when acceleration time is known. Similarly, a recent study made by Bokare *(35)* indicated that the polynomial acceleration model is suitable for lower speeds while the dual regime linear acceleration model explains the acceleration behavior of vehicles at higher speed. Wang *(36)* also indicated that a quadratic relationship between acceleration and speed would better fit the real world condition; his model indicates that drivers normally accelerate at the speed with a polynomial decreasing mode.

For acceleration studies at metered ramps, an experimental acceleration performance testing conducted by Caltrans in 1988 *(37)* indicated that an average acceleration rate for a passenger car at metered on-ramp was about 5.47 ft/s^2, and acceleration rate from 100 to 400 feet is approximately one-third of the rate during the first 100 feet. Seven different probe vehicles were used and each vehicle made three runs; Figure 2-2 shows the distance versus speed profile of a typical probe vehicle.

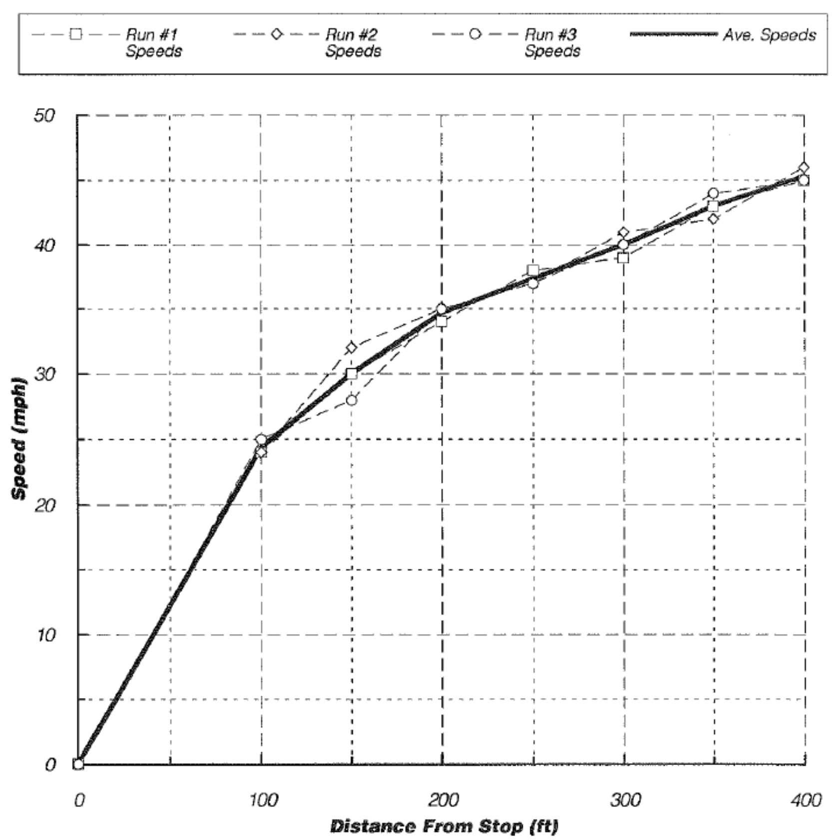


Figure 2-2 A Typical Distance-Speed Profile of Caltrans's Experimental Acceleration Performance Testing

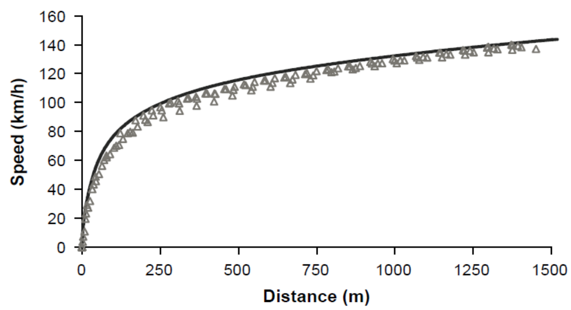
A summary of average acceleration rates reported in previous studies is presented in Table 2-3.

Table 2-3 Acceleration Rates Documented by Various Literatures

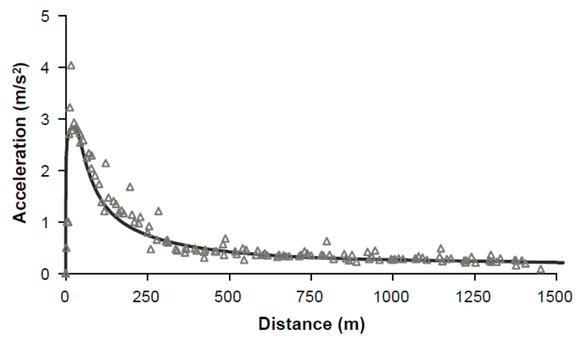
Author	Year	Acceleration Rates (ft/s^2)		Note
		Passenger Car	Truck	
Loutzenheiser et al.	1938	1.90 - 3.00	N/A	
Deen	1957	2.06 – 2.81	0.95 - 1.80	
Oto	1988	5.47	N/A	Average acceleration rate at metered on-ramp
Long	2000	4.72	N/A	Average acceleration over 0-25 mph
Bham et al.	2001	4.69 and then 2.82	N/A	Dual regime model
Hunter et al.	2001	0 - 2.92 (good geometrics) 0 - 5.83 (bad geometrics)	N/A	
Harwood et al.	2003	N/A	0.93 - 2.08	
Haas et al.	2004	4.5	N/A	Average acceleration rate leaving a stop sign
AASHTO	2004	1.83 - 3.18	N/A	
Fitzpatrick et al.	2007	2.5 (95th percentile) 3.0 (85th percentile)	N/A	Constant acceleration rate
Gattis et al.	2008	N/A	0.98 - 1.18	
Kraft et al.	2009	1.91 – 4.84	N/A	Average acceleration rate

Vehicle Dynamics Model

Vehicle dynamics models aim to describe the actual acceleration pattern of the vehicle through experimental testing. In comparison with kinematic models, vehicle dynamics models have the capability of modeling a vehicle's actual physics of motion and predict the theoretical acceleration values under various road geometric features. Rakha et al. *(38,39,40)* proposed and applied a series of vehicle dynamics models for acceleration behavior modeling of various trucks and passenger cars. Acceleration profiles of the testing vehicles were presented in three domains: acceleration versus speed, acceleration versus time, and acceleration versus distance. Modeling results show that the acceleration rate first increases from zero to the maximum value and then decrease exponentially to zero, as illustrated in Figure 2-3. However, since vehicle dynamics modeling relies greatly on testing facilities, very limited acceleration models have been developed that incorporate vehicle dynamics. Also, in reality the acceleration capability of different vehicle types varies. Acceleration capability is usually influenced by prevailing traffic conditions and road geometric features. Therefore, vehicles dynamics modeling results usually cannot fully represent the real-world conditions and also not suitable for acceleration lane length design purpose.



(a) Speed versus distance profile



(b) Acceleration versus distance profile

Figure 2-3 An Example of Rakha's Vehicle Dynamic Model to Typical Passenger Car

2.4 Guidelines for Acceleration Lane Length Design

Currently, several state DOTs, including California, Arizona, Florida, Texas, Minnesota etc., require the acceleration distances to be consistent with the general ramp guidelines that appear in the AASHTO Green Book *(13)*. The recommendations given by the green book are the foundation for the design of acceleration lane length at existing on-ramps in the U.S. Table 10-3(reproduced in Table 2-4) and Table 10-4 (reproduced in Table 2-5) in the 2011 AASHTO Green Book present the minimum acceleration lane lengths and adjustment factors for various combinations of beginning and ending vehicle speeds. As shown in Table 2-4, the desired distance increases with the increasing of the merging speed and ramp grade. From the literature review, it was found that the 2011 Green Book acceleration lengths differ slightly from the 1965 Blue Book *(41)*. The Blue Book states that the required distances were generated using three factors, the speed at which drivers enter the acceleration lane, the speed at which drivers merge into the mainline traffic, and the manner of acceleration *(41)*.The underlying assumptions are that drivers would enter the acceleration lane at an average running speed that was determined from the design speed of the ramp's controlling curve, and that drivers would join the freeway mainline traffic at a speed that is equal to the mainline average running speed minus 5 mph. When the average grade of the acceleration lane is 3% or greater, the minimum acceleration lengths should be adjusted in accordance with the values presented in Table 2-5.

Table 2-4 Minimum Acceleration Lengths for Entrance Terminals with Flat Grade of 2% or Less

Speed Reached, V_m (mph)	Stop Condition	15	20	25	30	35	40	45	50
	Initial speed, V_0 (mph)								
	0	14	18	22	26	30	36	40	44
23	180	140	-	-	-	-	-	-	-
27	280	220	160	-	-	-	-	-	-
31	360	300	270	210	120	-	-	-	-
35	560	490	440	380	280	160	-	-	-
39	720	660	610	550	450	350	130	-	-
43	960	900	810	780	670	550	320	150	-
47	1200	1140	1100	1020	910	800	550	420	180
50	1410	1350	1310	1220	1120	1000	770	600	370
53	1620	1560	1520	1420	1350	1230	1000	820	580
55	1790	1730	1630	1580	1510	1420	1160	1040	780

Table 2-5 Speed Change Lane Adjustment Factors as a Function of Grade

Design speed of highway (mph)	Acceleration lanes Ratio of length on grade to length on level for design speed of turning curve (mph)[a]				
	20	30	40	50	All speeds
	3 to 4% upgrade				3 to 4% downgrade
40	1.3	1.3	-	-	0.7
45	1.3	1.35	-	-	0.675
50	1.3	1.4	1.4	-	0.65
55	1.35	1.45	1.45	-	0.625
60	1.4	1.5	1.5	1.6	0.6
65	1.45	1.55	1.6	1.7	0.6
70	1.5	1.6	1.7	1.8	0.6
	5 to 6% upgrade				5 to 6% downgrade
40	1.5	1.5	-	-	0.6
45	1.5	1.6	-	-	0.575
50	1.5	1.7	1.9	-	0.55
55	1.6	1.8	2.05	-	0.525
60	1.7	1.9	2.2	2.5	0.5
65	1.85	2.05	2.4	2.75	0.5
70	2.0	2.2	2.6	3.0	0.5

a: Ratio from this table multiplied by the length in Exhibit 10-70 gives length of speed change lane on grade.

Arizona DOT recommends that a minimum of 500 ft. of additional acceleration length be provided beyond the ramp convergence point where truck volumes of three or more axle trucks exceed 5% and with sustained grades exceeding 3% *(18)*. Nevada DOT also suggests that a minimum of 300 feet should be provided from the stop bar to the end of the physical separation between the metered ramps and the mainline *(42)*. Ohio DOT suggests that for new construction, acceleration lengths will be determined using 10 mph below the mainline design speed; the minimum tolerable speed can be the design speed minus 15 mph *(43)*.

Another major research was conducted by Fitzpatrick and Zimmerman from the Texas Transportation Institute (TTI) to update the Green Book acceleration lane length design *(31)*. Their focus was also based on passenger cars. They further extended the acceleration lane values to accommodate design speeds greater than 80 mph and a few other methods of calculating acceleration lane length. The authors pointed out that the assumptions in the AASHTO Green Book are out of date and need to be updated using findings from more recent research. By examining the existing acceleration rate and lane lengths documented in the AASHTO Green Book and the NCHRP Report, the authors made a recommendation that an average acceleration rate of 2.5 ft/s^2 be used to calculate potential acceleration distances. Their results also recommended longer acceleration lengths than the Green Book.

One issue with the acceleration lane lengths provided by the Green Book is that they were designed to accommodate passenger cars; therefore, they might be too short for large and heavy vehicles. Several studies have been conducted previously concerning acceleration length design on the basis of the AASHTO Green Book. NCHRP Report 505 *(15)* discussed the role of truck characteristics in roadway design. Based on a 180 lb/hp truck and similar conditions used in the Green Book but with 0% grade, they found that the minimum acceleration lane lengths were about 1.8 times greater than the minimum acceleration lane lengths provided in the Green Book.

A similar research by Gattis et al. made recommendations regarding the length of acceleration lane needed for heavy vehicles to accelerate to speeds close to the freeway mainline speeds *(16)*. The authors examined the acceleration behaviors of tractor-trailer trucks and developed mathematical models to predict the average and 10^{th} percentile speeds. The 10^{th} percentile speed means 90 percent drivers can reach the assumed speed in the given distance. The acceleration lane lengths design recommendations from previous studies are summarized in Table 2-6.

Table 2-6 Acceleration Lane Lengths from Previous Research

	AASHTO Green Book 2011 Edition	NCHRP Report 505, 2003	Fitzpatrick and Zimmerman, 2007	Gattis, et al., 2008
Model Design Vehicle Type	Passenger Car on 0%~2% Grade	180 lb/hp Tractor-Trailer Truck on 0% Grade	Passenger Car	Tractor-Trailer Truck on Level Grade
Assumed Initial Speed (mph)	22	22	20	17
Speed Reached (mph)	Distance to Reach Speed (ft.)			
39	550	850	---	---
40	---	---	908	1203
50	1020	2230	1383	2119
55	1580	3260	1653	2731
60	---	---	1945	3655

2.5 Summary of Literature Review

To conclude, the design of metered on-ramps must strike a balance between available queue storage space and acceleration distance needs. Currently, the guidelines pertaining to queue storage length and acceleration lane length design are not complete or comprehensive for the geometric design of ramp meters. Most of the states that do not possess specific design guidance tend to follow the basic AASHTO guidelines. They provide minimum AASHTO acceleration lengths for posted speeds and maximum vehicle storage concerning queue overspill problems. Additionally, the queue storage and acceleration length requirements for ramp metering should accommodate a wide range of traffic volume and freeway geometric conditions.

3. DATA COLLECTION

3.1 Pilot Study

A pilot study was conducted at the early stage of the project to gain additional insights into the issues and factors affecting queue storage length and acceleration lane length. The pilot study also helped to verify the effect of proposed data collection and processing procedures. Several representative ramp metering sites were identified and video data of acceleration and traffic queue were collected at the sites. Findings from the pilot study were used to develop the site selection criteria for the full data collection.

3.1.1 Queue Length Study

The major purposes of the queue length pilot study are to identify: 1). accuracy of the input-output method for queue length modeling; 2). factors that affect ramp queue length; 3). Typical interchange types and ramp control strategies in California.

During this stage, the research team developed a preliminary mesoscopic queue length estimation model based on the input-output approach. The model was coded in Microsoft Office Excel to simulate the ramp queue length with different combinations of arrival types, metering methods, and traffic demand levels. Queue length data was collected at Center St. to EB I-80 metered on-ramp in Reno, Nevada to verify the data collection approach and data analysis methodologies. Comparison between field observed queue lengths and modeling results is illustrated in Figure 3-1.

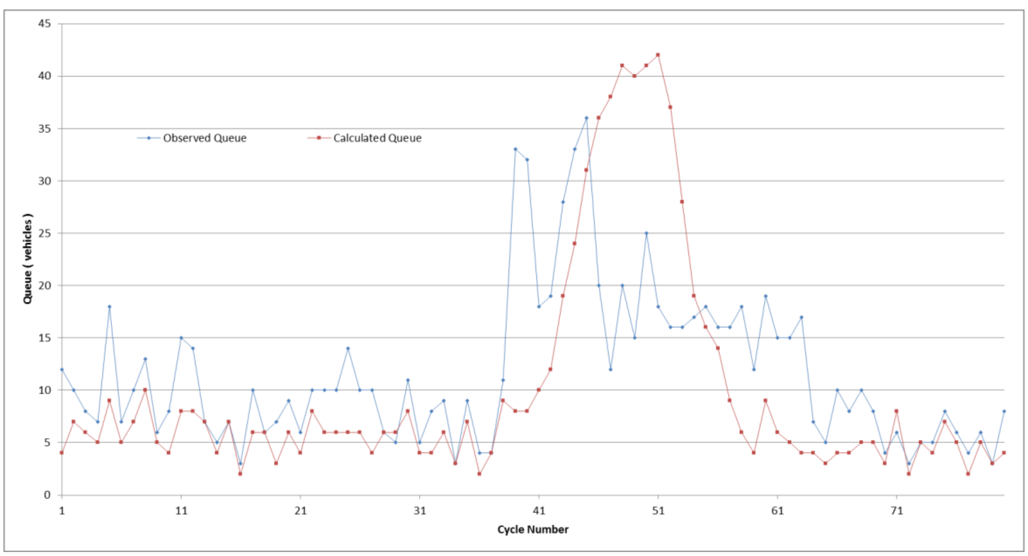


Figure 3-1 Comparison of Field Observed Queue Lengths and Modeling Results

Based on both field investigation and modeling, it was found that the maximum queue is more uncertain than the 95th percentile queue. Also, the design recommendation aims to accommodate the majority drivers rather than all the drivers. For design purposes, instead of using the maximum queue length, it is recommended that the 95th percentile queue length be used to avoid recommendations are too conservative. In general, the modeling results can capture the realistic queue profile. The deviations were mostly stemmed from the random nature of the parameters involved in the modeling, which are needed to be built into the revised queue length simulation model. The deterministic and stochastic factors that affect ramp queue length are summarized as follows.

Deterministic Factors

Deterministic factors are primary parameters that contribute to the queue in a queuing system; for metered on-ramps, the deterministic factors are the *On-Ramp Demand* and the *Metering Rate (i.e., capacity)*. A queue will be formed when on-ramp demand is greater than the metering rate. Usually on-ramp demand indicates a proportional relationship with ramp queue length, while metering rate is inversely proportional related to ramp queue length.

Stochastic Factors

The stochastic factors are the random parameters that would affect the queue length. In real world conditions, traffic flow and signal control strategies varies cycle by cycle. Hence queue length is affected by the dynamic nature of potential parameters involved in the queuing system, including:

On-Ramp Arrival Flow Profile

Based on the stochastic queuing theory, when on-ramp demand is less than capacity, the ramp queue is largely contributed by short-term over-saturation of upstream arrival flow, thus accurately modeling the on-ramp arrival flow profile is critical for capturing the queuing process.

Upstream Signal Timing

Due to the unique geometry and traffic characteristics at metered on-ramps, upstream signal timing directly affects the on-ramp arrival flow profile. This is because each on-ramp feeding movement controlled by the upstream signal will arrive at the ramp meter with two flow regimes: the saturated queue discharge regime (platoon arrival) and the uniform arrival regime (non-platoon arrival). Therefore, the upstream signal timing will be used to determine the time period of each regime.

Lane Imbalance

Field investigation shows that the actual ramp metering rate of a ramp with two or more lanes is affected by lane imbalance phenomenon. Therefore, the lane imbalance is also an important factor influencing the queue length.

Right Turn on Red

It was found that drivers usually turn right on a red signal whenever there is a safe gap. With right-turn-on-red, the on-ramp arrival flow profile of each feeding movement is changed. Field observations also revealed that right-turn-on-red tends to exacerbate the maximum queue length in a cycle by adding additional on-ramp demands to one particular feeding movement.

The pilot study also found that ramps with actuated upstream signal timing and traffic responsive metering are very common in California. Traffic flow diverging at a ramp entrance is also typical in California, as illustrated in Figure 3-2, so that on-ramp demands do not always equal to upstream intersection departures. All of these control strategies and geometric factors should be taken into consideration during data extraction and modeling.

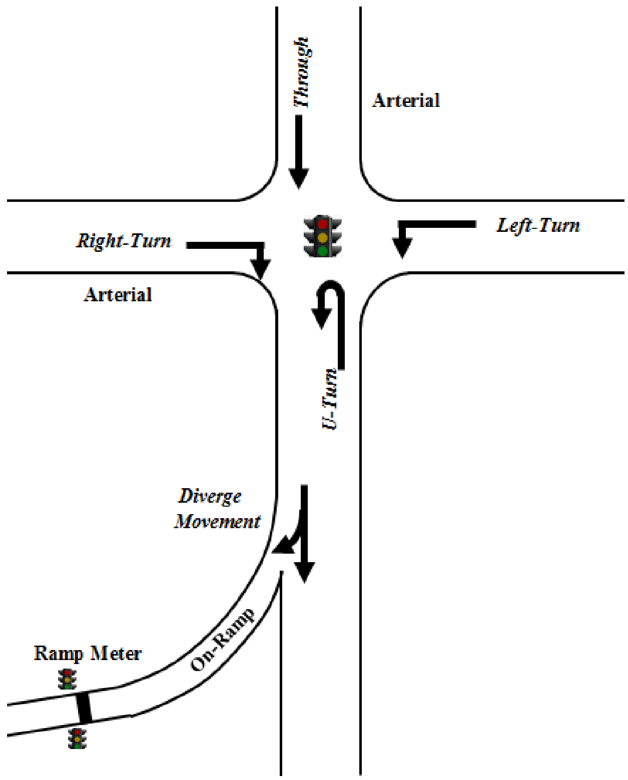


Figure 3-2 Illustration of Ramp-Metering Site with Diverge Movement

3.1.2 Acceleration Study

The pilot study revealed that acceleration data cannot usually be measured directly from the field. It was found that a probe vehicle equipped an accelerometer seems to be the only available method that can directly provide acceleration data; on the other hand, however, the probe vehicle method has many limitations such as high data collection costs, limited sample size, influences of drivers on acceleration performance, etc. Bias may exist in the analysis results of probe vehicle data. In this research, acceleration data were derived from second-by-second speed data.

The required data are spot speeds of each individual vehicle at the pre-determined locations of the on-ramp acceleration lane, as illustrated in Figure 3-3 below.

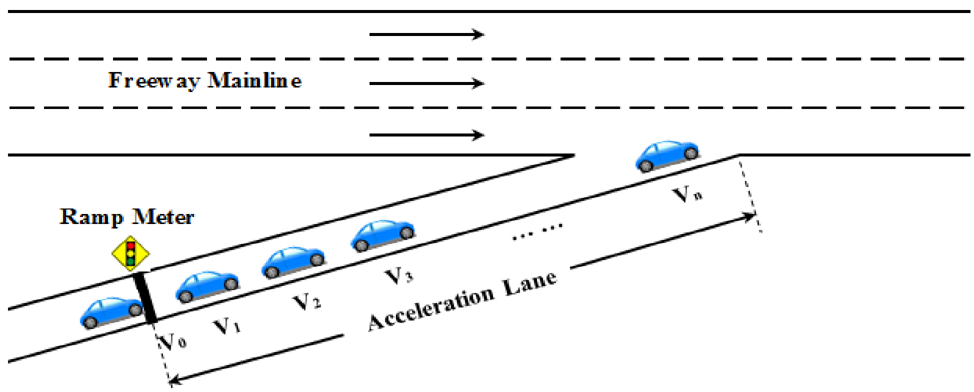


Figure 3-3 Illustration of the Required Speed Data for This Study

To obtain the required speed data, several data collection methods were reviewed or tested in the pilot study. The methods include probe vehicle, automatic video processing, videotaping with manual data extraction, radar detectors, pneumatic tube counter, and magnetic traffic counters. The selection of data collection approach considered the output data type, cost for temporary data collection, device installment, accuracy of the data, etc. The automatic video processing system needs great effort for calibration for each site under different environmental conditions. A radar gun can be used to collect real-time speed information; however, it is difficult to know whether the radar gun is tracking the right object in a carpool, and it's also difficult to identify the accurate location when a speed number is obtained. A magnetic sensor has the ability of obtaining vehicle spot speed at a designated location; however, it can only obtain one minute group average speed rather than speed of an individual sample. The data from magnetic sensors showed high error with low traffic speed. With consideration of the issues listed above, video based data collection and manual data extraction was used in this project. The time and location information were extracted manually from cameras deployed along the selected metered ramps; then spot speed at designated locations was calculated assuming uniform acceleration over a short time or space interval (between two adjacent cameras). Although the video based method is time-consuming, it can provide better accuracy and the flexibility to retrieve detailed trajectories of each sample. Table 3-1 compares the strengths and weaknesses of each data collection technology that were reviewed and tested in this study.

Table 3-1 Comparisons of Candidate Speed Data Collection Technologies

Technologies	Strength	Weakness
Probe Vehicle	Direct measurement of speed and acceleration; Accurate location and time information.	Data only available from probe vehicles; Limited data samples; High costs Data are likely to be influenced by different drivers.
Video Image Processors	Direct measurement of spot speed.	Speed data has low accuracy and precision; Device installation requires 50 to 60ft camera mounting height for optimum speed measurement.
Radar Gun	Direct measurement of spot speed.	Speed data has low accuracy and precision; Difficult to track the entire moving trajectory of an individual vehicle.
Pneumatic Tube	Direct measurement of spot speed.	Cannot collect speed information of an individual vehicle; Speed data has low accuracy and precision; Installation requires lane closure.
Magnetic Counter	Direct measurement of spot speed; Quick installation for temporary data collection.	Installation requires working within the traffic lane; Cannot collect speed information of an individual vehicle; Inaccurate counting of trucks; Higher error when speed is lower.
Parallel Cameras with Manual Extraction	Selection of individual samples; Accurate and precise speed data. Able to obtain entire time-location information	Time and labor consuming.

3.2 Site Selection for the Full Data Collection

3.2.1 Site Selection Criteria

Since modeling queue length and studying acceleration characteristics at metered ramps need to consider a variety of factors, sites must be carefully selected to cover a wide range of ramp geometry, traffic flow, and adjacent traffic network conditions. In this study, field collected data at the selected representative sites were used for: 1). developing and validating the queue length simulation model; 2). Studying acceleration characteristics and developing acceleration length design recommendations. Figure 3-4 illustrates the factors considered in identification of full data collection sites.

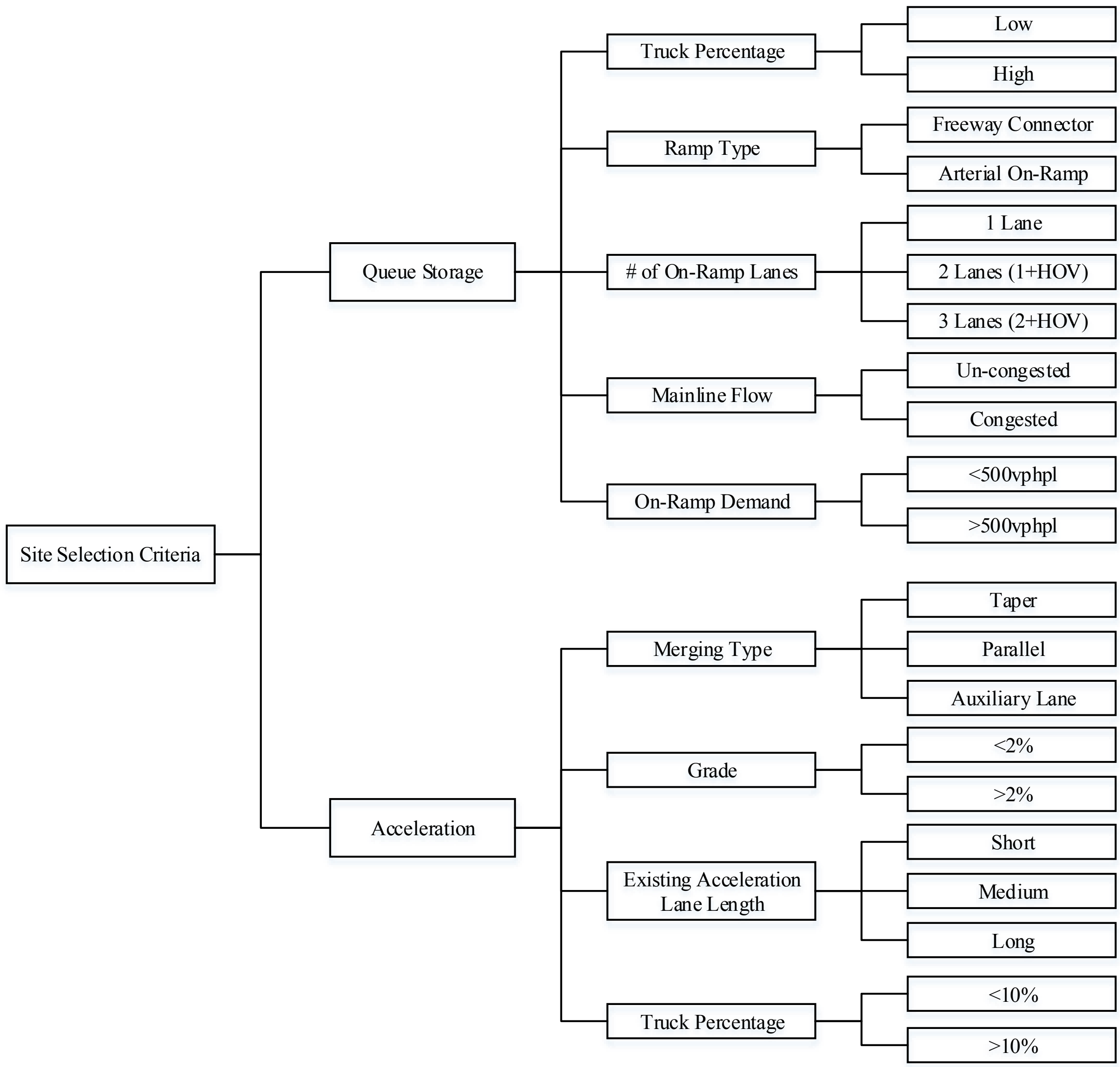


Figure 3-4 Site Selection Criteria for Field Data Collection

The project panel provided candidate sites for data collection with consideration of the factors in Table 3-1. Table 3-2 shows the candidate sites for queue storage study and Table 3-3 listed the sites for acceleration study.

Table 3-2 Candidate Sites for Queue Storage Study

Ramp Location	Caltrans District	Ramp Type	Number of Onramp Lane	Freeway Flow	On-ramp Demand
12th Ave. to NB 99	District 3	Slip (Diamond)	1	Congested	>500 vphpl
E St. to NB 99	District 3	Slip (Diamond)	2	Congested	>500 vphpl
SB Hazel Ave. to WB 50	District 3	Direct Diagonal	2+M HOV	Congested	>500 vphpl
SB Bradshaw Rd. to WB 50	District 3	Direct Diagonal	1	Congested	< 500vphpl
Marina Blvd. to NB880	District 4	Outer Diagonal	1+M HOV	Un-Cong	< 500vphpl
SB Rte 262 to WB 880	District 4	Connector	2+M HOV	Congested	< 500vphpl
Woodman Ave. to NB101	District 7	Slip (Diamond)	1+ Bulk	Congested	> 500vphpl
Torrance Blvd. to NB 110	District 7	Hook	1	Congested	< 500vphpl
Bundy Dr. to EB 10	District 7	Slip (Diamond)	1+HOV	Congested	> 500vphpl
Balboa Blvd. to WB 118	District 7	Slip (Diamond)	2	Congested	> 500vphpl
Tampa Ave. to EB 118	District 7	Slip (Diamond)	2	Un-Cong	< 500vphpl

Note: Ave. = Avenue; St. = Street; Rd. = Road; Blvd. = Boulevard; Rte = Route; Dr. = Drive; NB = Northbound; SB = Southbound; WB = Westbound; EB = Eastbound; HOV=High Occupancy Vehicle.

Table 3-3 Candidate Sites for Acceleration Study

Ramp Location	Ramp Type	Merge Type	Existing Acceleration Length (ft.)	On-ramp Lane	Grade	Freeway Flow	On-ramp Demand
EB Mowry Ave. to NB 880	Loop	Taper	765	1+HOV	Flat	Un-Cong	< 500vphpl
WB Alvarado-Niles Rd. to SB 880	Loop	Taper	660	1+HOV	Flat	Un-Cong	> 500vphpl
Artesia Blvd. to NB 405	Hook	Taper	475	1	Flat	Un-Cong	< 500vphpl
SB Douglas Blvd. to WB 80	Slip	Taper	820	1	Flat	Un-Cong	> 500vphpl
Fruitridge Rd. to NB 99	Slip	Auxiliary	310	1	Flat	Un-Cong	> 500vphpl
Industrial Pkwy. to NB 880	Slip	Auxiliary	395	1+HOV	Flat	Un-Cong	> 500vphpl
WB Rosecrans Ave. to NB 710	Hook	Taper	4,450	1+HOV	Flat	Un-Cong	< 500vphpl

Note: Ave. = Avenue; Rd. = Road; Blvd. = Boulevard; Pkwy. = Parkway; NB = Northbound; SB = Southbound; WB = Westbound; EB = Eastbound; HOV=High Occupancy Vehicle; Un-Cong = Un-congestion. * for taper and parallel ramps, existing acceleration length is the distance from the stop bar to the end of the dash line (as descripted in AASHTO Green Book *(10)*); **for auxiliary lane ramps, existing acceleration length is the distance from the stop bar to the gore; after the gore is the auxiliary lane.

3.2.2 Illustrations of Representative Sites

In this section, three typical interchange types with metered on-ramps in California are illustrated. Figure 3-5 shows a conventional diamond interchange with three movements feeding the metered on ramp, which was defined as ***Category 1***. The frontage road northbound through movement, the arterial left turn and right turn movements are controlled by the upstream signal, usually under three phases.

Figure 3-5 Diamond Ramp with Three Feeding Movements (E St. to NB 99, Caltrans District 3)

Figure 3-6 illustrates another common ramp configuration in California: a diamond ramp with two feeding movements (in some cases, the off-ramp vehicles can re-enter the freeway; however, field observation showed volume of this movement is very limited, usually less than one percent of the total on-ramp feeding volume, thus was ignored in this study), defined as ***Category 2***. In this case, the arterial left turn and right turn on-ramp feeding movements contribute to the ramp queue. For this ramp configuration, usually the upstream signal has three phases, while only two phases are on-ramp feeding phases. Ramps with similar on-ramp feeding movement pattern and upstream signal control including: hook type ramp, as illustrated in Figure 3-7, and outer diagonal type ramp, as illustrated in Figure 3-8.

Figure 3-6 Diamond Ramp with Two Feeding Movements (Woodman Ave to NB 101, Caltrans District 7)

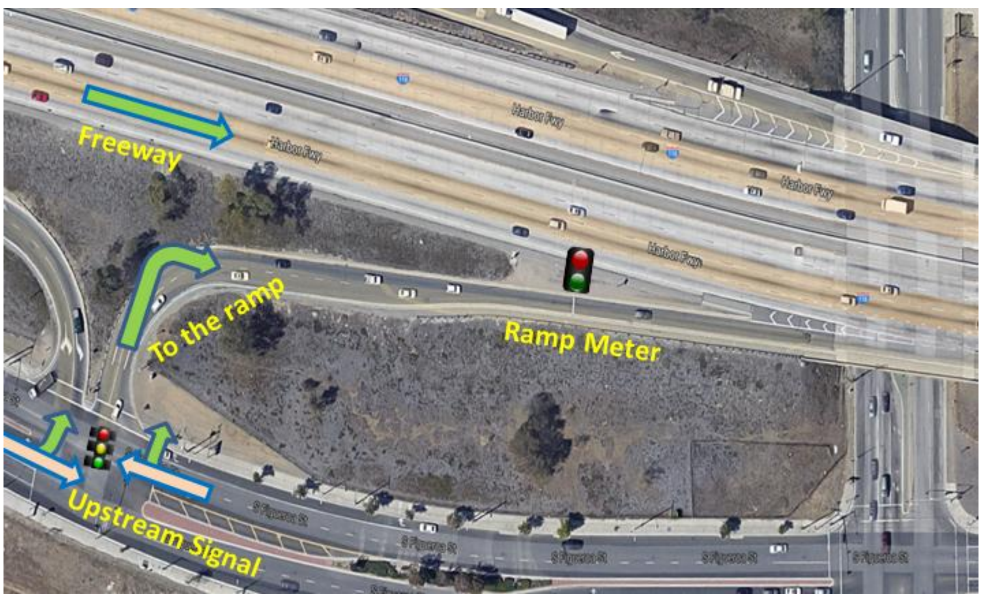


Figure 3-7 Hook Ramp with Two Feeding Movements (Torrance Blvd to NB 110, Caltrans District 7)

Figure 3-8 Outer Diagonal Ramp with Two Feeding Movements (Marina Blvd to NB 880, Caltrans District 4)

Figure 3-9 illustrates the third common ramp configuration in California, which was defined as ***Category 3***: slip ramp with diverging movement. The ramp arrival flow pattern of this ramp type differs from the aforementioned ramps. Typically there are three movements feeding to the ramp with a four-phase upstream signal control strategy (four feedings can exist when there are off-ramp vehicles make a U-turn to re-enter the on-ramp). Traffic flow diverges at the ramp entrance: a part of the traffic from the upstream intersection enters the metered ramp and the other part stays on the arterial road. The key differences between Category 3 and Category 1 include: 1). the upstream intersection departures will affect the upstream signal timing and thus result in different on-ramp arrival flow patterns, which is particularly significant for actuated signals; 2). Since a portion of upstream departure flow diverges to the arterial, the platoon from the upstream intersection is partially dispersed. The queue length estimation model should therefore take into account the difference of on-ramp platoons.

Figure 3-9 Slip Ramp with Diverging Movement (Bradshaw Road to WB 50, Caltrans District 3)

3.3 Full Data Collection

3.3.1 Queue Storage Data Collection Method

In this study, the queue length related data were collected using multiple video cameras. A minimum of three video cameras were typically placed at different locations of the ramp and upstream intersection to capture the key queue length modeling parameters: cycle-by-cycle onramp demands, real-time queue lengths, and metering rate. Figure 3-10 illustrates the camera layout at the E St. to NB 99 metered on-ramp in Sacramento, Caltrans District 3. Camera 1 was placed at the upstream intersection, which aims to capture the upstream signal timing and also the upstream vehicle movements. Camera 2 was placed at the upstream intersection toward the ramp meter signal to capture the ramp queue length. Camera 3 was placed in front of the ramp meter (upstream) to capture the metering rate and also the mainline volume. For curvy ramps where the whole queue length cannot be captured by one camera, backup camera(s) were added.

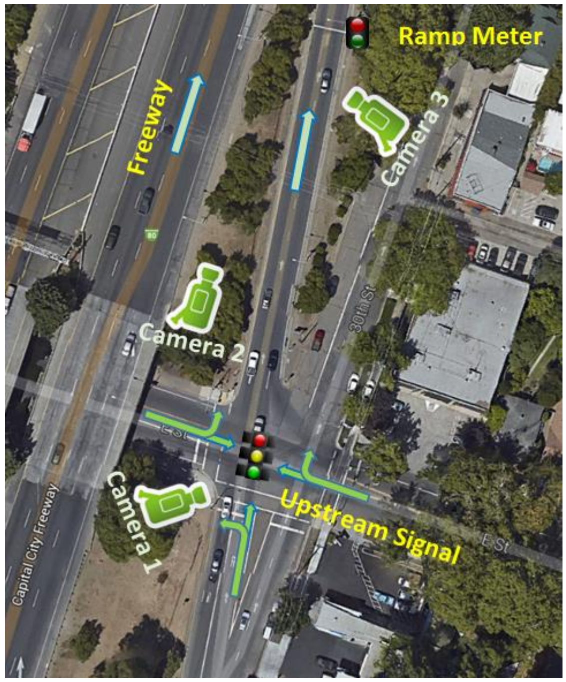


Figure 3-10 Typical Camera Layout for Queue Storage Data Collection

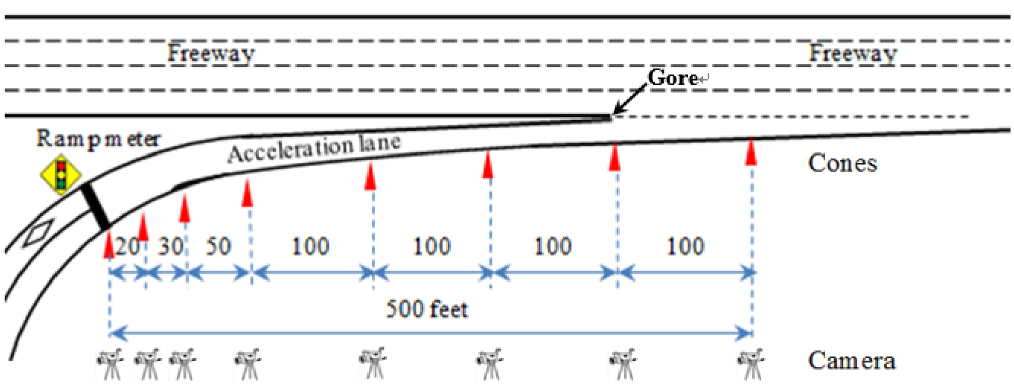
Detailed queue storage data collection information is summarized in Table 3-4 below:

Table 3-4 Queue Storage Data Collection Information

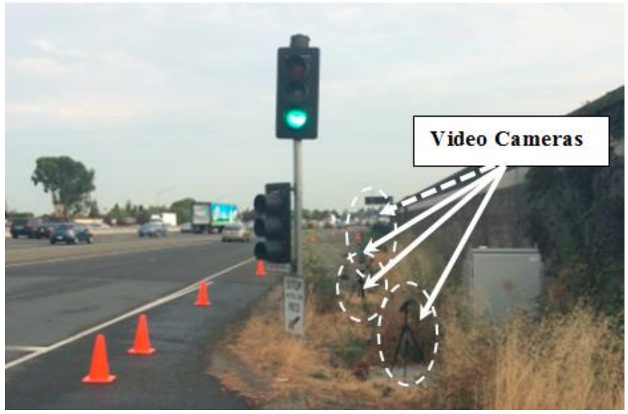
Ramp Location	Caltrans District	Data Collection Date	Data Collection Period	Data Collection Time Interval (minutes)
12th Ave. to NB 99	District 3	03/17/2014	AM Peak	90
E St. to NB 99	District 3	03/17/2014	PM Peak	90
SB Hazel Ave. to WB 50	District 3	03/18/2014	AM Peak	90
SB Bradshaw Rd. to WB 50	District 3	03/21/2014	AM Peak	90
Marina Blvd. to NB880	District 4	08/04/2014	AM Peak	90
SB Rte 262 to WB 880	District 4	08/05/2014	PM Peak	90
Balboa Blvd. to WB 118	District 7	03/16/2015	PM Peak	75
Tampa Ave. to EB 118	District 7	03/16/2015	PM Peak	75
Torrance Blvd. to NB 110	District 7	03/17/2015	AM Peak	75
Woodman Ave. to NB101	District 7	03/17/2015	Mid-Day	75
Bundy Dr. to EB 10	District 7	03/17/2015	PM Peak	75

3.3.2 Acceleration Data Collection Method

Based on the pilot study findings, the video-based method was selected for acceleration data collection. Traffic sign cones were placed along a metered ramp as reference points with known distance from the ramp meter stop bar. Based on a series of pilot data collection and theoretical analysis, it was determined that cones should be closely spaced at the beginning segment of the acceleration lane and spaced further apart the closer one gets to the freeway merge. A video camera was placed at each reference point to record the time stamp of each vehicle passing this designated reference point. In an attempt to minimize the impact of traffic cones on driver behavior, the cones were placed a certain distance from the acceleration lane. The layout of reference cones and cameras are demonstrated in Figure 3-11(a) and 3-11(b). Pilot investigation and speed data analysis found that most of the approaching vehicles did not come to a complete stop at the stop bar, particularly at fast metering rates. Previous acceleration studies on metered on-ramps *(37)*, however, ignored this phenomenon and assumed speed at the stop bar is zero, which tended to overestimate acceleration rates. Therefore, in our speed data collection, a cone was placed at the stop bar location and another cone was placed before the stop bar, which could provide the initial speed of approaching vehicles and lead to more accurate measurements. For the data collection sites, eight cameras were placed along the acceleration lane of the study ramp metering sites and covered a total distance of 500 feet. Since the time points of a vehicle were captured by different cameras, the time points needed to be synchronized. By presenting a high-accuracy stopwatch in front of each camera at the beginning of video recording (as illustrated in Figure 3-11(c)), the time offset between cameras were recorded. The camera time was synchronized with the recorded offsets in the procedure of acceleration data processing.



(a) Cone and Camera Layout at EB Mowry Ave/NB 880 Ramp Metering Site



(b) Field Picture of Data Collection

(c) Time Synchronization

Figure 3-11 Illustration of Video-based Speed Data Collection Methods

Detailed acceleration data collection information is summarized in Table 3-5 below:

Table 3-5 Acceleration Data Collection Information

Ramp Location	Caltrans District	Data Collection Date	Data Collection Period	Data Collection Time Interval (minutes)
Fruitridge Rd. to NB 99	District 3	03/19/2014	AM Peak	45
SB Douglas Blvd. to WB 80	District 3	03/19/2014	PM Peak	40
Industrial Pkwy. to NB 880	District 4	08/06/2014	AM Peak	90
WB Alvarado-Niles Rd. to SB 880	District 4	11/18/2014	PM Peak	50
EB Mowry Ave. to NB 880	District 4	11/19/2014 & 11/21/2014	AM Peak	120
WB Rosecrans Ave. to NB 710	District 7	03/16/2015	AM Peak	75
Artesia Blvd. to NB 405	District 7	03/17/2015	PM Peak	60

4. QUEUE LENGTH MODELING

4.1 Queue Length Data Processing

4.1.1 Video Merging

The queue data processing starts with merging all video clips. The purpose of video merging is to combine the synchronized video clips onto one screen, which would facilitate the data extraction. For each site, videos were recorded at different locations to capture the upstream signal timing, ramp feeding traffic flow, ramp metering rate and ramp queue length. In order to identify the different phase movements' influence on the queue length, the feeding traffic from the upstream intersections needed to be related to the queue length when they arrived at the ramp meter. It is time and labor consuming to manually synchronize each group of feeding traffic and related queue length, so a semiautomatic software tool was developed to synchronize the videos. Software users need to input the time offsets between videos, which were calculated based on the time of an identified vehicle appearing at the different locations, as shown in Figure 4-1. Then the tool syncs all the videos and generates an integrated video clip, as depicted in Figure 4-2.

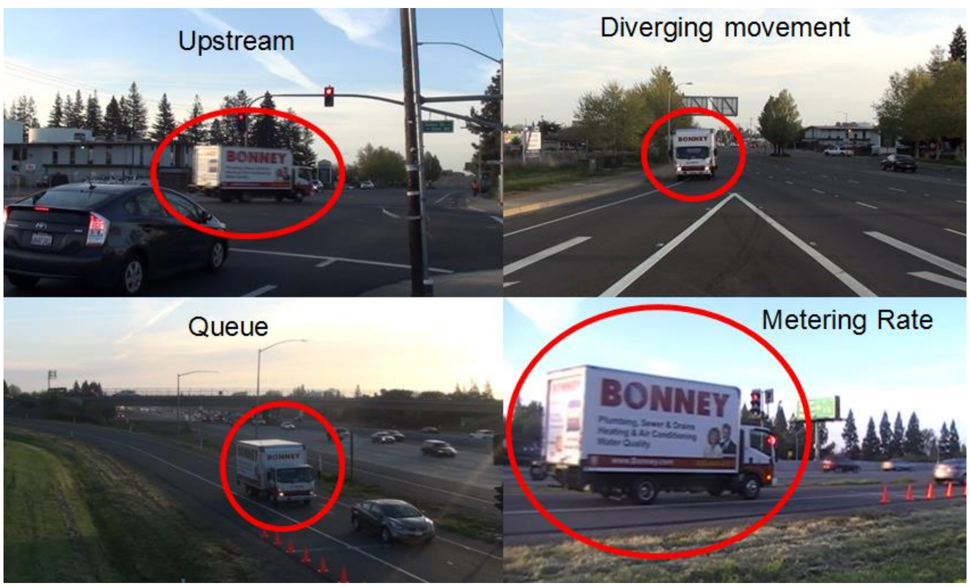


Figure 4-1 Illustration of Video Synchronization

The video merging tool was developed based on the ASP.NET platform, the Open Source Computer Vision Library, and the C++ programming language.

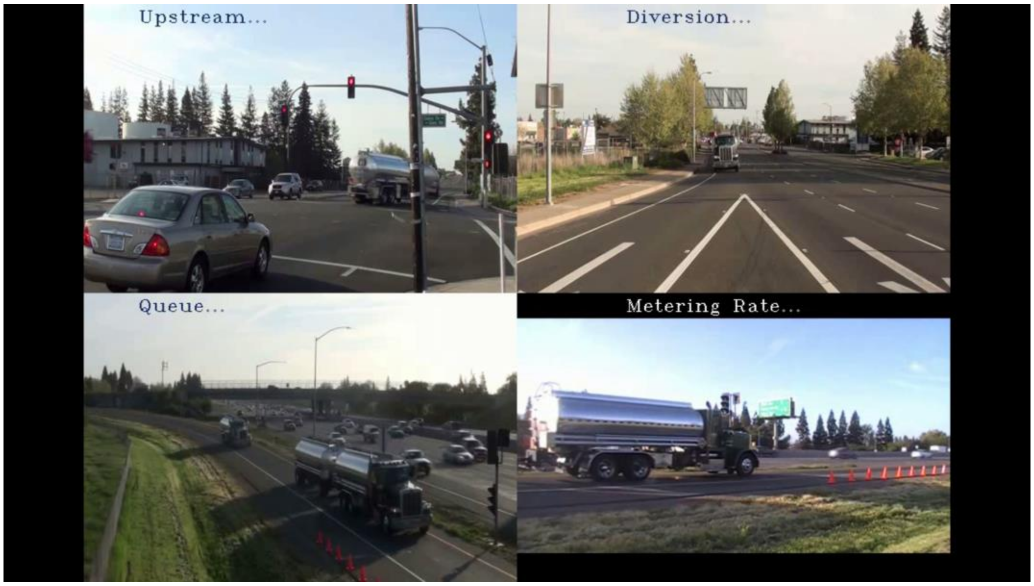


Figure 4-2 Merged Video after Time Synchronization

4.1.2 Data Extraction

In this study the data of upstream intersection signal, traffic demand, ramp metering rate and queue length were manually extracted from the video clips. The details of the data extraction method are presented below:

Upstream signal timing information

For upstream intersections controlled by actuated traffic signals, the cycle lengths vary cycle by cycle. It is therefore necessary to document the starting and ending time of each phase of the upstream signals throughout the entire data extraction period. Then, the cycle length can be calculated as the absolute time difference between two identical phases.

Traffic volume of the upstream intersection

The purposes of recording traffic volumes at the upstream intersections include: 1). investigating the relationship between traffic volume and actuated signal control strategy; 2). estimating the on-ramp feeding volumes from each intersection movement

Average metering rate

In accordance with the extracted upstream signal cycle lengths, the average metering rate during each cycle was calculated with the green time of ramp metering signal over the cycle length.

Real-time ramp queue length

In accordance with the extracted upstream signal cycle lengths, the maximum ramp queue length of each cycle was counted and recorded as the vehicle number in the ramp queue.

Traffic diverging percentage

This parameter is required only for ramps where traffic flow diverges at the ramp entrance (i.e., Category 3 ramps). An example is the SB Bradshaw Rd. to WB 50 metered on-ramp, as illustrated in Figure 3-8. The on-ramp vehicles and the arterial pass-by vehicles were counted. The percentage of on-ramp vehicles over the total vehicles was calculated as the traffic diverging percentage.

Freeway mainline volume

In accordance with the extracted upstream signal cycle lengths, the freeway mainline volume and the right-most lane volume during each cycle were manually counted from the videos. Freeway mainline volumes were used for investigating the relationship between real-time metering rate and freeway mainline flow rate.

4.1.3 Summary of Observed Queue Length

By the end of the data processing, the field observed 95th percentile queue lengths were summarized as illustrated in Table 4-1.

Table 4-1 Summary of the Observed Queue Length

Site Location	Ramp Type	# of Lanes	On-Ramp Demand (vph)	Average Metering Rate (vph)	D/C Ratio	Observed 95th Percentile Queue	
						Queue Length (veh)	Percentage of On-ramp Demand
Bradshaw Rd.	Direct Diagonal	1	350	600	0.58	6	1.8
E St	Diamond	2	700	850	0.82	16	2.3
12th Ave	Diamond	1	750	850	0.88	18	3.9
Hazel Ave.	Direct Diagonal	2 + M HOV	1250	1285	0.97	66	5.3
Marina Blvd.	Outer Diagonal	1 + M HOV	675	1120	0.6	7	1
Torrance Blvd.	Hook	1 + Bulk	770	900	0.86	26	3.4
Bundy Dr.	Slip	1 + NM HOV	1321*	700	N/A	12	N/A
Balboa Blvd.	Diamond	2	700	1200	0.58	5	1.3
Tampa Ave.	Diamond	2	1300	1420	0.92	9	1.5
Woodman Ave.	Diamond	1 + Bulk	760	900	0.84	13	1.7
Route 262	Connector	2 + M HOV	926	873	1.06	60	6.9

Note: M HOV = metered high occupancy vehicle lane; NM HOV = non-metered high occupancy vehicle lane. * Need to determine the proportion of HOV vehicles

4.2 Queue Length Modeling at Metered On-Ramps

4.2.1 The Input-Output Method

The input-output approach, which is also known as the cumulative arrival and departure method *(44)*, has been widely used in modeling queue and delay of different traffic facilities *(45, 46, 47)*. For metered on-ramps, the arrival rate is the on-ramp demand and the maximum departure rate is the metering rate and thereby the queue length is the accumulated difference between the arrival and departure rate over time *(47)*. Equations 1 through 3 provide a generalized description of the traditional input-output method. Given the traffic demand, *V(t)*, and the capacity of the facility, *c*, the cumulative arrival function, *A(t)*, and the departure rate, *D(t)*, can be determined. Subsequently, the performance measure of queue length, *q(t)* can be obtained. For metered ramps, the capacity is the metering rate that is also the maximum departure rate.

$$\frac{dA}{dt} = V(t) \tag{4-1}$$

$$\frac{dD}{dt} = \begin{cases} c, & if\ A(t) > D(t) \\ V(t), & otherwise \end{cases} \tag{4-2}$$

$$q(t) = A(t) - D(t) \tag{4-3}$$

The traditional input-output method is also illustrated in Figure 4-3. The basic principle is to plot the cumulative vehicle arrival and departure curves. The total area bound by the two curves represents the total vehicle delays in vehicle-hour, and the vertical offset represents the queue length in terms of the number of vehicles at an instant *t*. As depicted in Figure 4-3, queue forms when the arrival rate is larger than the departure rate. The queue is cleared when cumulative arrivals are equal to cumulative departures. For any time point t_1, queue length $Q(t_1)$ could be calculated as $A(t_1) - D(t_1)$.

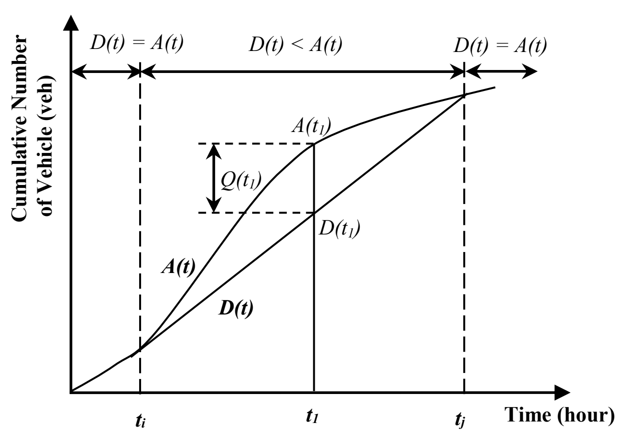


Figure 4-3 Accumulative Arrival Departure Curve for Queue Length Estimation

4.2.2 Analysis of On-ramp Flow Arrival Profiles

The input-output method has been widely used for queue length estimation at intersections and freeway bottlenecks. The traffic arrival and departure patterns at metered ramps are different. The arrival traffic flow is decided by the control type and signal timing of the upstream intersection; and is impacted by the diverging percentage if traffic diverging exists. The departure flow is mainly decided by the metering signal, such as one vehicle/two vehicle per green, and the real time metering rate decided by the freeway mainline traffic condition.

Figure 4-4 shows a typical metered on-ramp with three feeding movements at the upstream intersection. Traffic from each feeding movement arrives at the ramp meter in two flow regimes: the regime with saturated upstream-intersection discharging flow (platoon arrival) and the regime with traffic flow equal to the arrival flow of the upstream intersection (non-platoon arrival). For example, during the frontage road through movement *(TH)*, or phase $\phi 4$, the first portion of flow occurs when the frontage road through movement discharges at its saturation flow rate S_{TH}. After the queue of the through movement is cleared, the flow rate reduces to its average arrival rate A_{TH}.

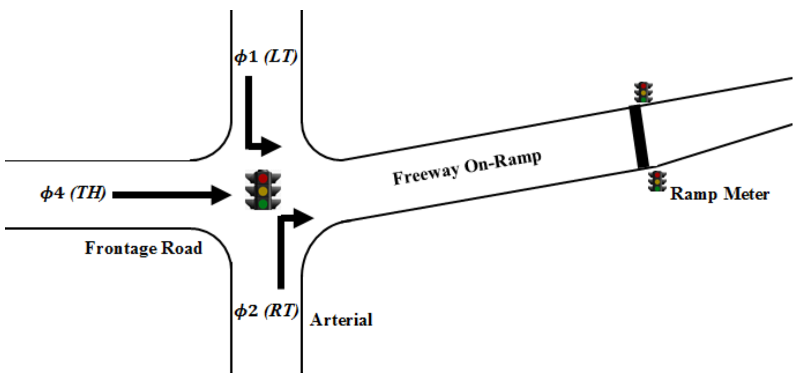


Figure 4-4 A Typical Metered On-Ramp with Three Feeding Movements (Category 1)

By the assumption that arrival traffic uniformly arrives at the upstream intersection, the time periods of the two regimes can be estimated by applying the queuing theory. Figure 4-5 shows the queuing diagram of a feeding movement at the upstream intersection. The queue clearance time G_0, which is also the time interval of the platoon arrival regime of the ramp, could be estimated as:

$$G_0^i = \frac{A_i \times (C - G_i)}{S_i - A_i} \quad (4\text{-}4)$$

Where,
G_0^i: queue clearance time for the i^{th} feeding movement, $G_0^i \leq G_i$;
C: cycle length of upstream signal;
G_i: effective green time of the i^{th} feeding movement;
A_i: uniform arrival flow rate of the i^{th} feeding movement; and
S_i: saturation flow rate of the i^{th} feeding movement.

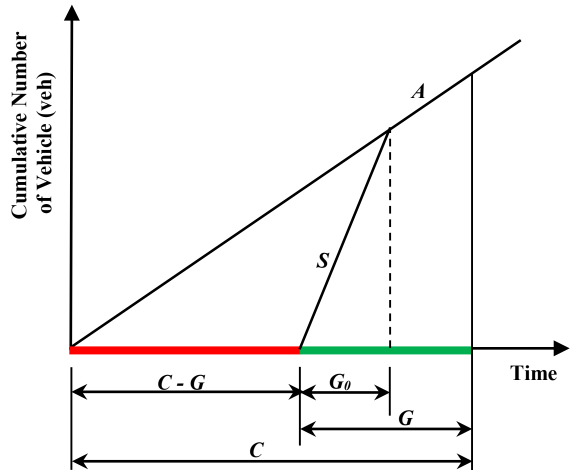


Figure 4-5 Queuing Diagram of an Upstream Intersection Movement

Accurate modeling of on-ramp queue lengths relies on accurate modeling of the ramp arrival profiles. With knowing the phasing sequence of upstream intersection (e.g., $\phi 4$, $\phi 2$, and $\phi 1$), the saturation flow rate S_i and the average arrival rate A_i of each phase, the on-ramp arrival flow pattern of each cycle can be depicted as illustrated in Figure 4-6.

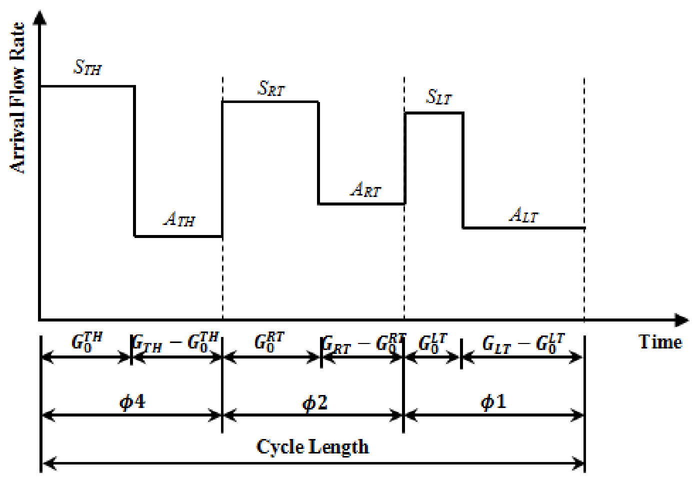
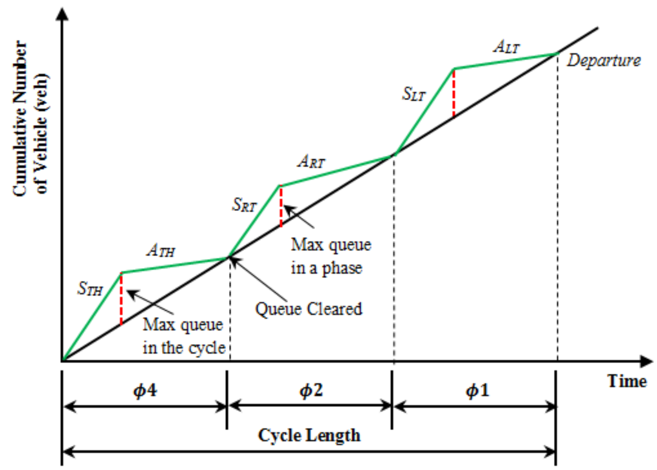


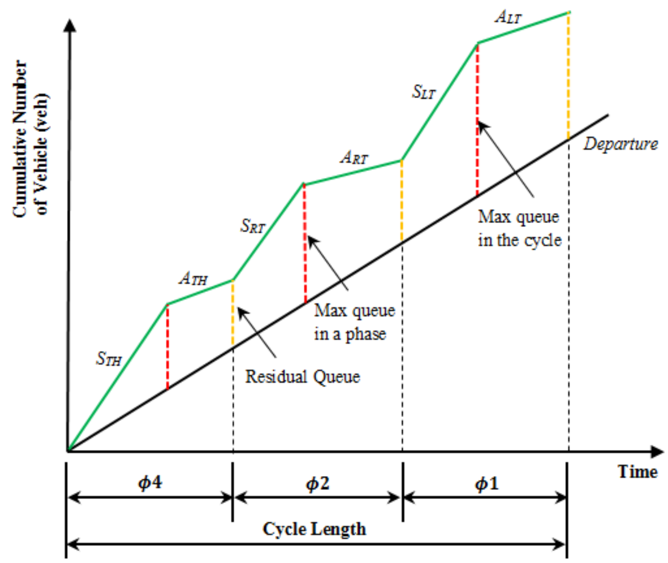
Figure 4-6 Ramp Arrival Flow Profile without RTOR

4.2.3 Queue Formed at Metered On-Ramps

A metered on-ramp is a standard queuing system: a traffic queue is formed at the ramp metering signal when the vehicle arrival rate exceeds the ramp metering rate. The queue may exist temporarily due to a short-term surge of traffic arrival (e.g., a platoon arrival) as illustrated in Figure 4-7(a) or it can be prolonged due to over-saturation over an extended period of time, as Figure 4-7(b). Queue lengths under both scenarios can be estimated by the queuing theory with certain assumptions. The average metering rate was assumed and used as the on-ramp capacity which is also the maximum departure rate. In consistency with Figure 4-6, two arrival traffic regimes exist in each phase.



(a) Queue Formed by Under-Saturated Arrival Traffic



(b) Queue Formed by Oversaturated Arrival Traffic

Figure 4-7 Queue Generation Profiles at Metered Entrance Ramps

4.3 Development of Queue Length Simulation Model for Metered Ramps

4.3.1 Mesoscopic Simulation

While the macroscopic models have the ability to simulate network operations throughout a long time period, they usually cannot provide detailed traffic performance data such as cycle-by-cycle queue lengths, since they generally use aggregate level input data. Microscopic models, on the other hand, have the ability of modeling an individual sample's performance in some detail. However, microscopic models generally call for careful coding of the network details, since these models are sensitive to errors in the setting of the simulation parameters. Therefore, the mesoscopic traffic simulation models are gaining popularity as mesoscopic models fill the gap between the aggregate level of macroscopic models and the individual interactions of the microscopic ones. Mesoscopic models normally describe the traffic entities at a high level of detail, but their behavior and interactions are described at a lower level of detail. This makes mesoscopic models ideal for prediction applications.

The developed mesoscopic queue length simulation model aims to provide sufficient modeling details while providing simple information for use by transportation engineers. Using this model, regression equations can be developed based on a large number of simulation runs to cover a broad range of on-ramp demand and metering rate scenarios. In turn, summary tables and charts are expected to be generated for quick estimation during design stages.

4.3.2 Queue Length Simulation Model

Based on the aforementioned arterial metered on-ramp queue length modeling procedure, the mesoscopic simulation model was implemented using the Microsoft Visual Basic programming language and was visualized in a Microsoft Excel spreadsheet. The developed simulation model contains three modules: the on-ramp demand modeling module, the metering rate modeling module, and the real-time queue length modeling module. The simulation flow chart as presented in Figure 4-8 illustrates the queue length simulation process.

Before running the simulation, users need to provide input of the upstream signal control type (actuated signal timing or fixed time signal timing) and metering strategy (traffic responsive metering or fixed time metering). Then, users need to input the general simulation parameters including: average hourly on-ramp demand of each feeding movement (for Category 3 type ramp, it is required to input the percentage of upstream departure volume that feed to the ramp), average hourly metering rate, saturation flow rate of each on-ramp feeding movement departing from the upstream signal, peak hour factor, number of ramp lanes, upstream signal timing information, and metering rate. For fixed-time upstream signal, the required signal timing information are cycle length and phase split; while for actuated upstream signal, it is required to input the maximum and minimum green time of each on-ramp feeding phase. Also, for the traffic responsive metering scenario, it is required to input the maximum and minimum metering rate before running the simulation.

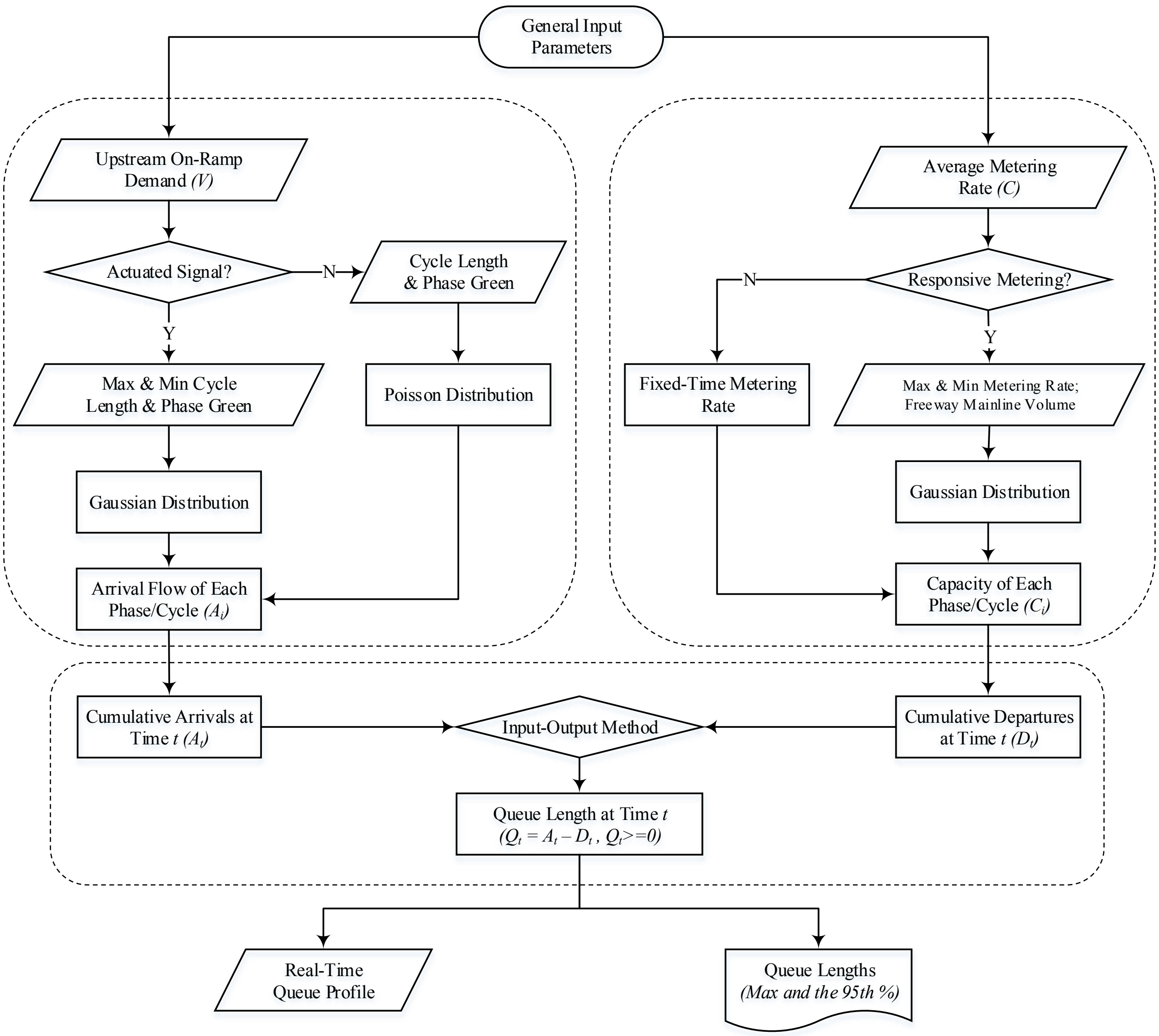


Figure 4-8 Arterial On-Ramp Mesoscopic Queue Length Simulation Flow Chart

The simulation model first allows a user to select the simulation scenario from the potential four candidate scenarios: 1). actuated upstream signal and traffic responsive metering; 2). actuated upstream signal and fixed-time metering; 3). fixed-time upstream signal and traffic responsive metering; 4). fixed-time upstream signal and fixed-time metering. The on-ramp demand modeling module reads the average hourly on-ramp demand input and distribute the total demand to each cycle and phase randomly. Meanwhile, the metering rate modeling module reads the metering rate input and equally (fixed-time metering scenario) or randomly (traffic responsive metering scenario) assigns the cycle length of the metering signal.

The queue length modeling module then simulates the length of the queue at the metering signal. The default simulation time is one hour. The cumulative arrivals and departures at time t could be determined; accordingly the queue length at time t could be calculated through the input-output method. Finally, the simulation model can generate the queue versus time profile and output the maximum and the 95th percentile queue for each simulation. The user interface of the developed mesoscopic simulation model is shown in Figure 4-9.

	A	B	C	D	E
1	**General Input Parameters**				
2	**Average Metering Rate (vphpl)**	600	Simulate		
3	**Maximum Metering Rate (vphpl)**	900			
4	**Minimum Metering Rate (vphpl)**	300			
5	**# of Ramp Lanes**	2			
6					
7	**Input Parameters**				
8	**Ramp Feeding Movements**	Movement 1	Movement 2	Movement 3	Movement 4
9	**Is Protected? (TRUE/FALSE)**	TRUE	TRUE	TRUE	FALSE
10	**Is Ramp Feeding? (TRUE/FALSE)**	TRUE	TRUE	TRUE	FALSE
11	**Volume (vph)**	435	253	229	0
12	**Percentage Feeding to the Ramp**	100	100	3	0
13	**Saturation Flow Rate (vph)**	3800	3800	1900	0
14	**Heavy Vehicle Percentage**	0	0	0	0
15	**Average Green Time (sec)**	53	18	34	0
16	**Max Green Time (sec)**	79	38	49	0
17	**Minimum Green Time (sec)**	8	8	8	0
18	**All Red (sec)**	1	1	1	0
19	**Yellow (sec)**	3	3	3	0
20	**Peak Hour Factor**	0.9	0.9	0.9	0
21					
22	**General Output**				
23	**Upstream Volume (vph)**	919			
24	**Ramp Feed (vph)**	696			
25	**Average Metering Rate (vph)**	1200			
26	**95th Percentile Queue (veh)**	8			
27	**Maximum Queue (veh)**	12			
28					

(a) Actuated Upstream Signal Control Scenario

	A	B	C	D	E
1	**General Input Parameters**				
2	**Cycle Length (sec)**	100			
3	**Average Metering Rate (vphpl)**	837	Simulate		
4	**Maximum Metering Rate (vphpl)**	976			
5	**Minimum Metering Rate (vphpl)**	710			
6	**# of Ramp Lanes**	1			
7					
8	**Input Parameters**				
9	**Ramp Feeding Movements**	Movement 1	Movement 2	Movement 3	Movement 4
10	**Is Protected? (TRUE/FALSE)**	TRUE	TRUE	TRUE	FALSE
11	**Is Ramp Feeding? (TRUE/FALSE)**	TRUE	FALSE	TRUE	TRUE
12	**Volume (vph)**	354	429	240	240
13	**Percentage Feeding to the Ramp**	100	0	2	100
14	**Saturation Flow Rate (vph)**	1900	1900	1900	1900
15	**Heavy Vehicle Percentage**	0	0	0	0
16	**Green Time (sec)**	53	21	14	0
17	**All Red (sec)**	1	1	1	0
18	**Yellow (sec)**	3	3	3	0
19	**Peak Hour Factor**	0.98	0.98	0.98	0.98
20					
21					
22	**General Output**				
23	**Upstream Volume (vph)**	1265			
24	**Ramp Feed (vph)**	599			
25	**Average Metering Rate (vph)**	837			
26	**95th Percentile Queue (veh)**	9			
27	**Maximum Queue (veh)**	17			
28					

(b) Fixed-time Upstream Signal Control Scenario

Figure 4-9 User Interface of the Developed Arterial On-Ramp Queue Length Simulation Model

4.3.3 Model Validation

To validate the accuracy of the developed mesoscopic simulation model, the phase by phase arrival and departure traffic data were collected at metering ramps and the upstream intersections. The traffic phase-by-phase data were imported into the simulation model and the ramp queue was estimated. The modeling results were compared to the field observed queue lengths to verify the derivations between the two. Figure 4-10 through Figure 4-14 demonstrate the observed and modeled queue length profiles at the aforementioned five representative metered on-ramps.

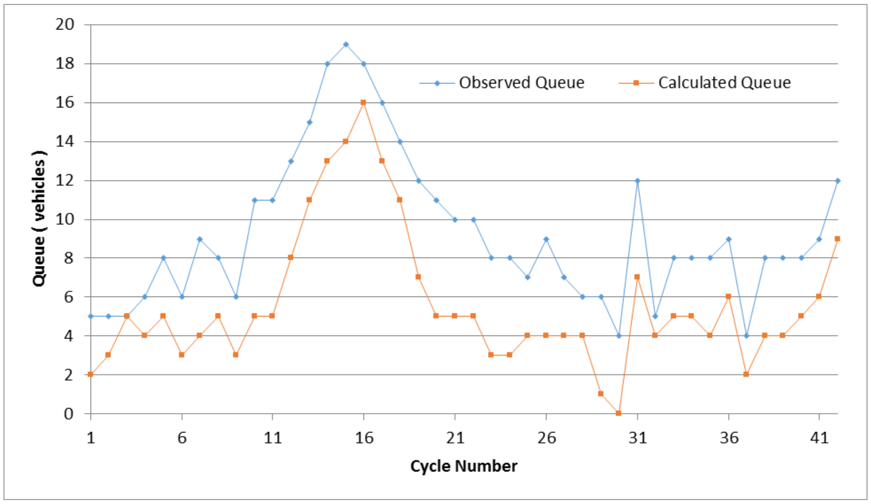


Figure 4-10 Field Observed Queue versus Modeling Result - E St. to NB 99

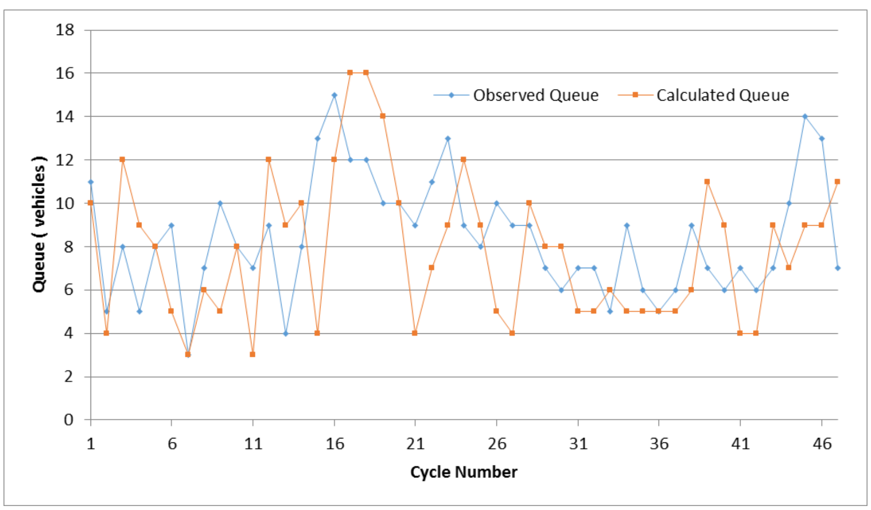


Figure 4-11 Field Observed Queue versus Modeling Result – Woodman Ave. to NB 101

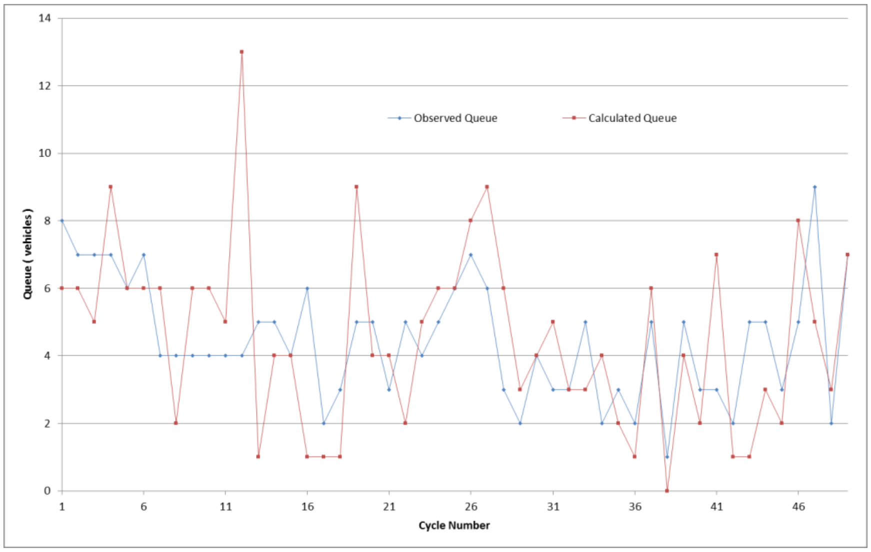


Figure 4-12 Field Observed Queue versus Modeling Result – Marian Blvd. to NB 880

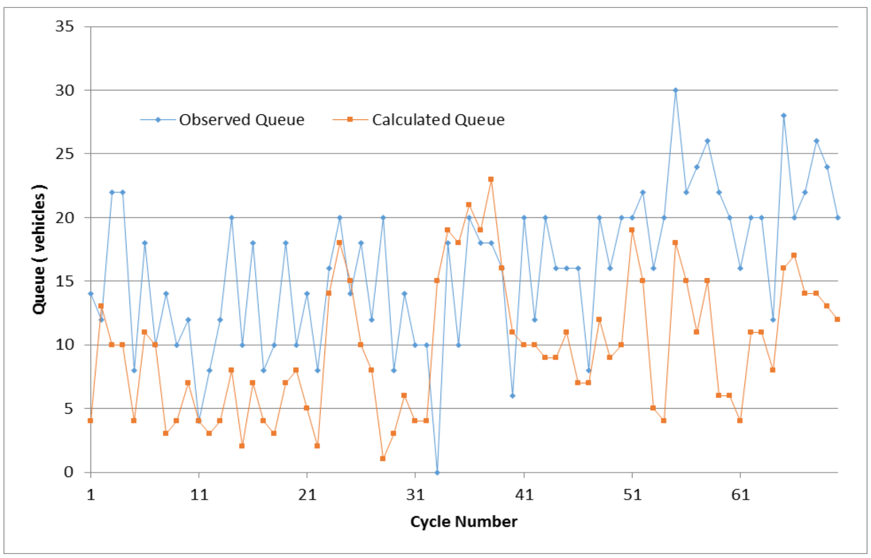


Figure 4-13 Field Observed Queue versus Modeling Result – Torrance Blvd. to NB 110

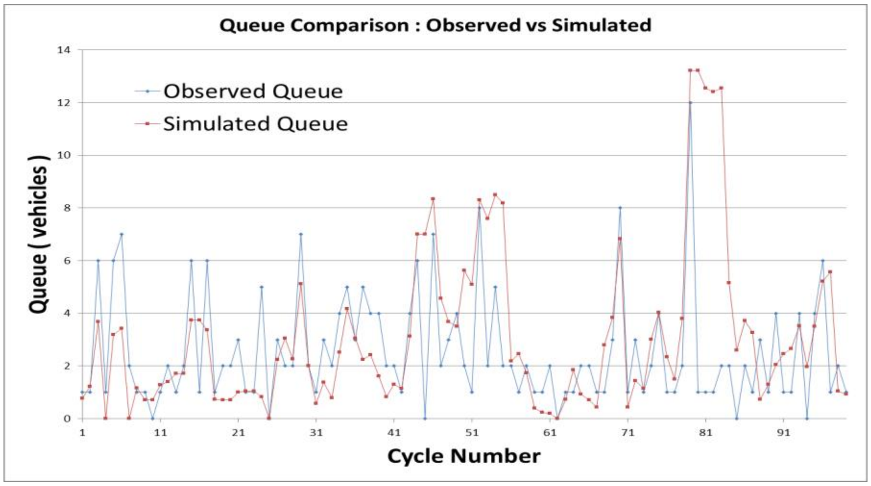


Figure 4-14 Field Observed Queue versus Modeling Result - SB Bradshaw Rd. to WB 50

Comparison of maximum and the 95th percentile queue lengths from field data and simulation is presented in Table 4-2. During the pilot study, it was found that the maximum queue length is affected by various factors, while the 95th percentile would be more suitable for queue storage design purpose and thus was used by this study.

Table 4-2 Summary of the Observed Queue and Modeling Result for Each Site

Site Location	Observed Queue (veh)		Modeling Result (veh)	
	Maximum	95th Percentile	Maximum	95th Percentile
12th Ave.	20	18	22	15
E St.	19	16	16	11
Hazel Ave.	74	66	76	72
Bradshaw Rd.	11	6	13	6
Marina Blvd.	9	7	12	6
Woodman Dr.	15	13	16	12
Torrance Blvd.	30	24	23	18
Bundy Dr.	13	12	11	8
Balboa Blvd	8	5	9	7
Tampa Ave.	10	10	12	10

The comparison shows that the queue length estimation model can accurately capture the observed queue profile, and the estimated 95th percentile queue lengths since results were close to the field observation. The developed queue length simulation model for ramp meter was used to estimate queue lengths under various on-ramp demands and metering rates, as introduced in Section 4.5.

4.4 Development of Queue Length Simulation Model for Freeway Connectors

4.4.1 Mesoscopic Simulation Flow Chart and User Interface

Similar to the arterial metered on-ramp queue length simulation model, the proposed freeway connector queue length simulation model has three modules: the on-ramp demand modeling module, the metering rate modeling module, and the real-time queue length modeling module. The simulation model will generate random on-ramp flows to capture the randomness of ramp arrival flow, and then simulate queue lengths based on the input-output method. The input parameters for queue length simulation includes: ramp demand and metering rate in vehicle-per-hour, and queue length analysis interval, which was predetermined as 15, 30, or 60 seconds. The output results are the maximum and the 95th percentile queue length in number of vehicles and the queue length versus time profile of the entire analysis period. The mesoscopic queue length simulation flow chart is illustrated in Figure 4-15. In this study, the metering strategy was assumed to be fixed-time.

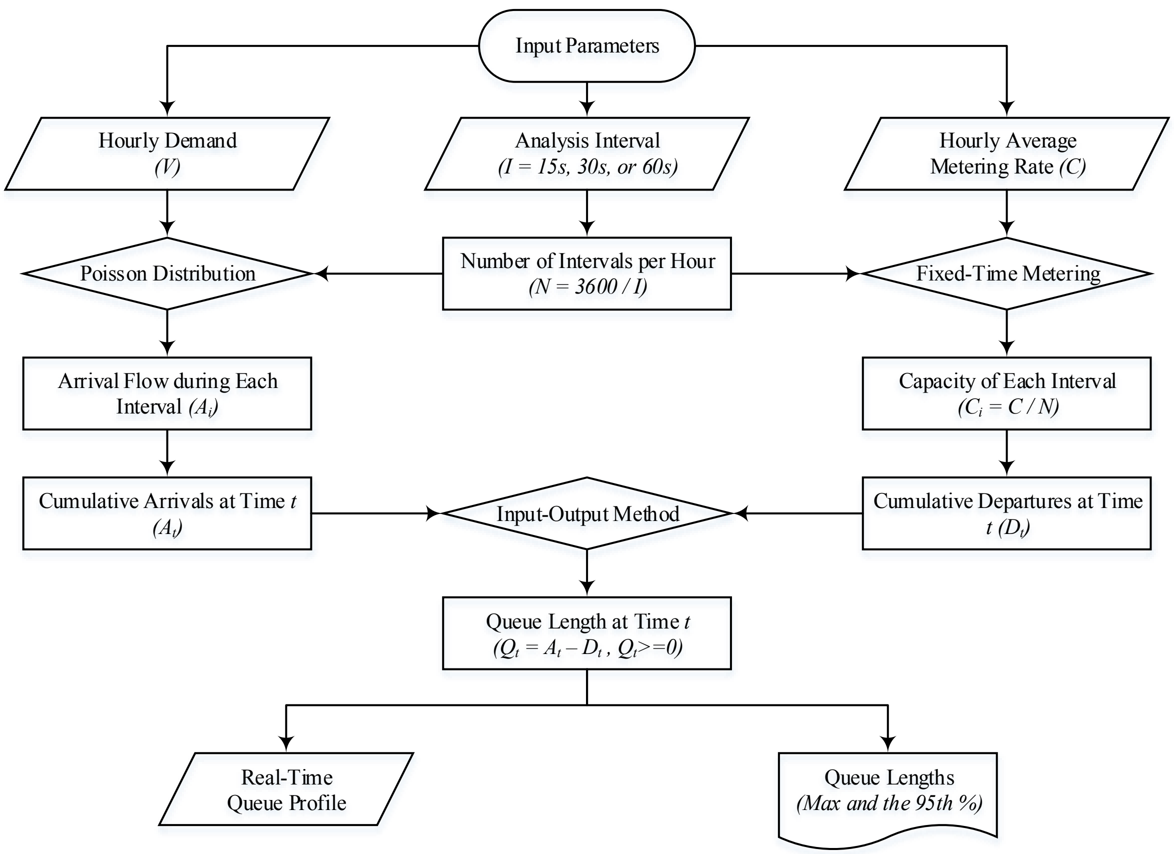


Figure 4-15 Freeway Connector Mesoscopic Queue Length Simulation Flow Chart

The mesoscopic simulation model was developed using the C# programming language. The entire simulation period is assumed to be one hour. By reading a selected analysis interval, the simulation period is equally divided into the corresponding number of intervals (e.g., 240 intervals if choosing 15 sec as analysis interval). Then, the on-ramp demand modeling module will read the input upstream demand and randomly divide the total demand into each interval. In this study the ramp arrival flow of each interval is assumed to follow the Poisson Distribution. Based on the field collected traffic arrival data at the study metered freeway connector, Based on the collected traffic arrival data at the study metered freeway-to-freeway connector, the generated ramp arrival flow at an interval was assumed to range between zero and two times the average arrivals per interval (summation of all the arrivals equal to the total on-ramp demand). Meanwhile, the metering rate modeling module reads the input metering rate and equally divides it into each interval, since a fixed-time metering strategy was assumed. The real-time queue length modeling module will then build a time series from time point 0 to the 3600th second; the time step could be 15 sec, 30 sec or 60 sec, depending on the selected analysis interval. By adding all the arrivals and departures from time 0 to time *t*, the cumulative arrivals and departures at time *t* could be determined; accordingly the real-time queue length at time *t* could be calculated through the input-output method. Finally, the model will generate the queue versus time profile and output the maximum and the 95th percentile queue for each simulation. The user interface of the developed mesoscopic simulation model is demonstrated in Figure 4-16.

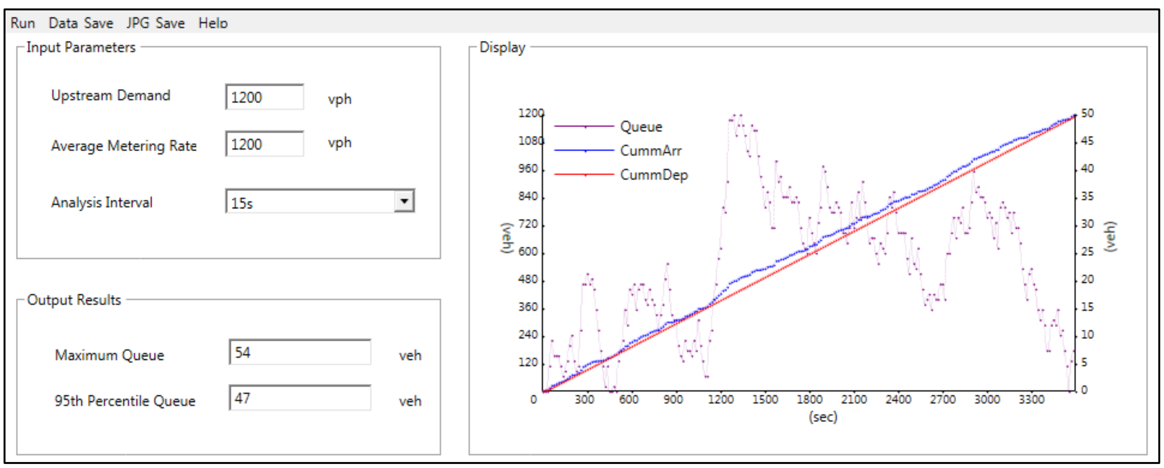
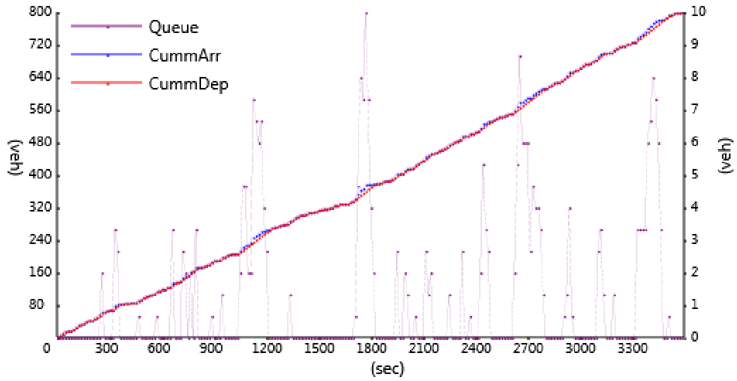
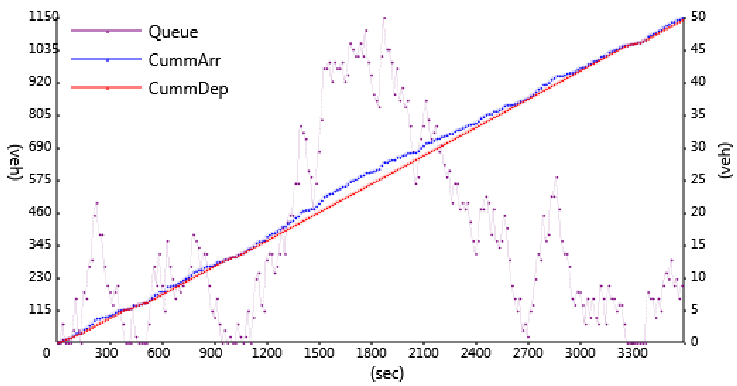


Figure 4-16 User Interface of the Developed Freeway Connector Queue Length Simulation Model

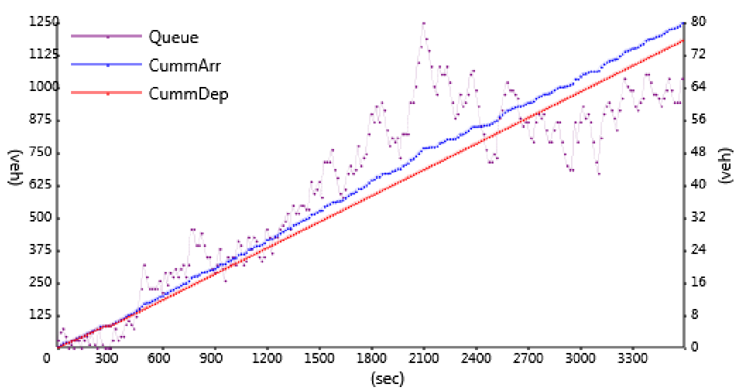
Figures 4-17(a) through 4-17(c) demonstrate simulated real-time queue profiles under three demand-to-capacity ratio scenarios: under-saturated scenario (D/C=0.67), quasi-saturated scenario (D/C=0.95) and over-saturated scenario (D/C=1.05). Based on the simulation, it was found that for the under-saturated scenario, the queue lengths are pretty random and will disperse quickly. For quasi-saturated scenario, it is more likely to have short term cumulative vehicle queue, but eventually the queue will be cleared at the end of analysis period. While the over-saturated scenario resulted in the formation of continuous queue when the on-ramp demand was greater than the capacity.



(a) Under-Saturation Scenario



(b) Quasi-Saturation Scenario



(c) Over-Saturation Scenario

Figure 4-17 Demonstration of Simulated Real-Time Queue Profiles

4.4.2 Model Validation

To validate the accuracy of the proposed mesoscopic simulation model, traffic performance data including the 15 second based on-ramp arrivals, departures, and the actual queue lengths were collected at the EB Route 262 to NB 880 metered freeway-to-freeway connector in Caltrans District 4. The detailed arrival and departure traffic data were imported into the developed freeway connector queue length simulation model to duplicate real world conditions; then, the simulation results were compared to the field observed queue lengths to verify the derivations, as illustrated in Figure 4-18. In general, the modeling results can accurately capture the realistic queue profile and the estimated queue lengths are close to the field observed queue for most of the time.

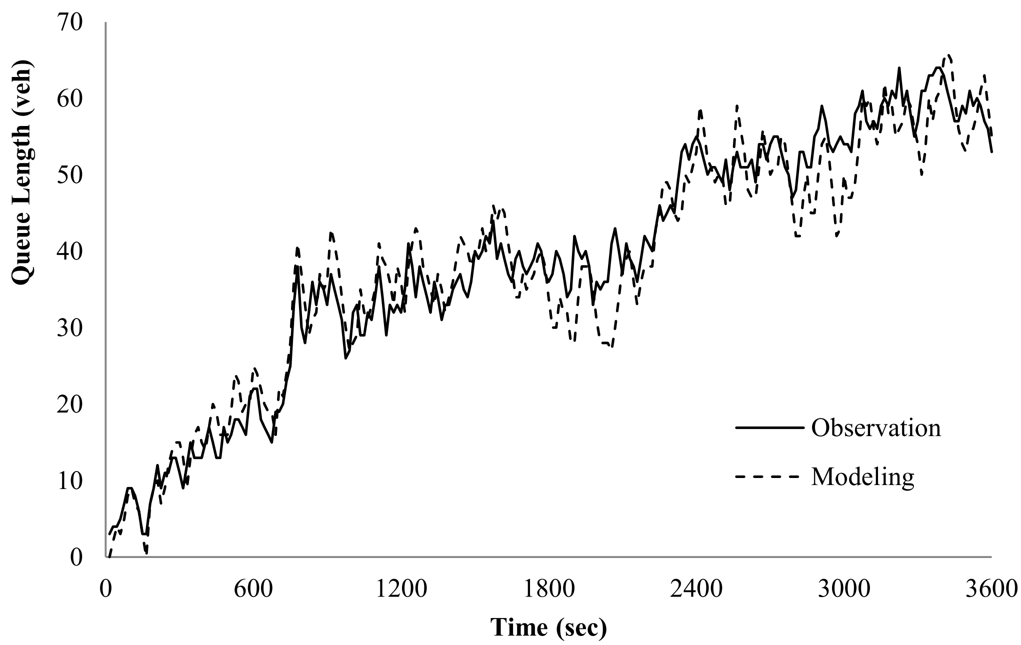


Figure 4-18 Simulated Queue Profile Compared to Field Observation

4.5 Simulated Queue Lengths

To study the queue length trend with the developed queue length estimation models for metered on-ramps and for metered freeway connectors, various scenarios were created for different combinations of on-ramp demands and metering rates. For each scenario, five simulations were performed for the mean of the simulated 95th percentile queue lengths. This section presents the simulated queue lengths for each ramp category and summarizes the relationship between the on-ramp demand and queue length. Detailed simulation results are documented in Appendix A.

4.5.1 Arterial On-Ramp

As mentioned in Section 3.2.2, in this study the arterial on-ramps were classified into three categories on the basis of on-ramp feeding flow arrival pattern. Based on the observed metering operation strategies, it was found that most metered on-ramps in California are dual-lane ramps and the average metering rate ranged from 300 vphpl to 900 vphpl. Scenarios with different combinations of demand and metering rates were designed to cover a wide range of demand-to-capacity ratios from approximate 0.3 to 1.0. The simulated 95th percentile queue lengths for Category 1, Category 2 and Category 3 type ramps are summarized in Table 4-3 through Table 4-5 which can be used for quick estimation of queue length (in number of vehicles) under a given demand and metering rate scenario.

Table 4-3 Summary of the 95th Percentile Queue Lengths - Category 1 Type Ramp

Average Metering Rate (vph)	On-Ramp Demand (vph)								
	300	400	500	600	700	800	900	1200	1500
	95th Percentile Queue Length (veh)								
600	7	19	66	*203*	*n/a*	*n/a*	*n/a*	*n/a*	*n/a*
800	6	11	20	45	90	*240*	*n/a*	*n/a*	*n/a*
1000	4	7	13	20	37	67	137	*n/a*	*n/a*
1200	4	6	9	14	23	36	59	*n/a*	*n/a*
1400	3	5	8	12	18	25	37	133	*n/a*
1600	3	4	6	10	14	19	28	64	207
1800	2	4	6	9	11	16	21	44	108

Note: Queue lengths for D/C >=1 scenario are shown in red and italic.

Table 4-4 Summary of the 95th Percentile Queue Lengths - Category 2 Type Ramp

Average Metering Rate (vph)	On-Ramp Demand (vph)								
	300	400	500	600	700	800	900	1200	1500
	95th Percentile Queue Length (veh)								
600	7	18	60	*n/a*	*n/a*	*n/a*	*n/a*	*n/a*	*n/a*
800	5	10	19	38	86	*n/a*	*n/a*	*n/a*	*n/a*
1000	4	7	12	18	33	46	79	*n/a*	*n/a*
1200	3	6	8	14	20	24	39	64	*n/a*
1400	3	4	7	11	15	20	26	36	*n/a*
1600	3	4	5	9	12	15	18	28	29
1800	2	3	5	7	9	13	16	20	24

Note: Queue lengths for D/C >=1 scenario are shown in red and italic.

Table 4-5 Summary of the 95th Percentile Queue Lengths – Category 3 Type Ramp

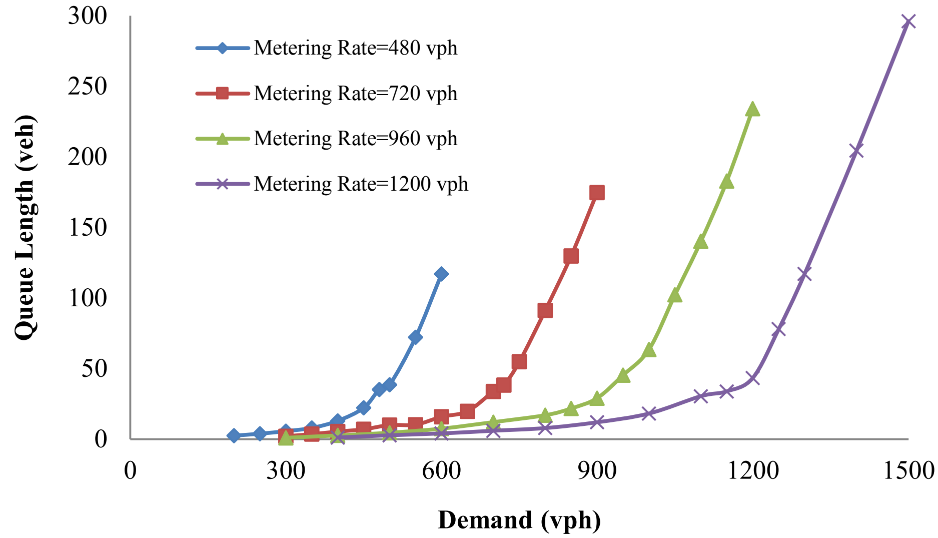
Average Metering Rate (vph)	On-Ramp Demand (vph)								
	300	400	500	600	700	800	900	1200	1500
	95th Percentile Queue Length (veh)								
600	9	22	65	*202*	*n/a*	*n/a*	*n/a*	*n/a*	*n/a*
800	7	11	23	60	115	*243*	*n/a*	*n/a*	*n/a*
1000	5	9	14	24	39	82	147	*n/a*	*n/a*
1200	4	7	10	15	27	43	69	*306*	*n/a*
1400	4	6	9	13	18	27	43	142	*n/a*
1600	4	5	8	12	15	21	32	81	194
1800	3	5	8	11	14	17	24	54	104

Note: Queue lengths for D/C >=1 scenario are shown in red and italic.

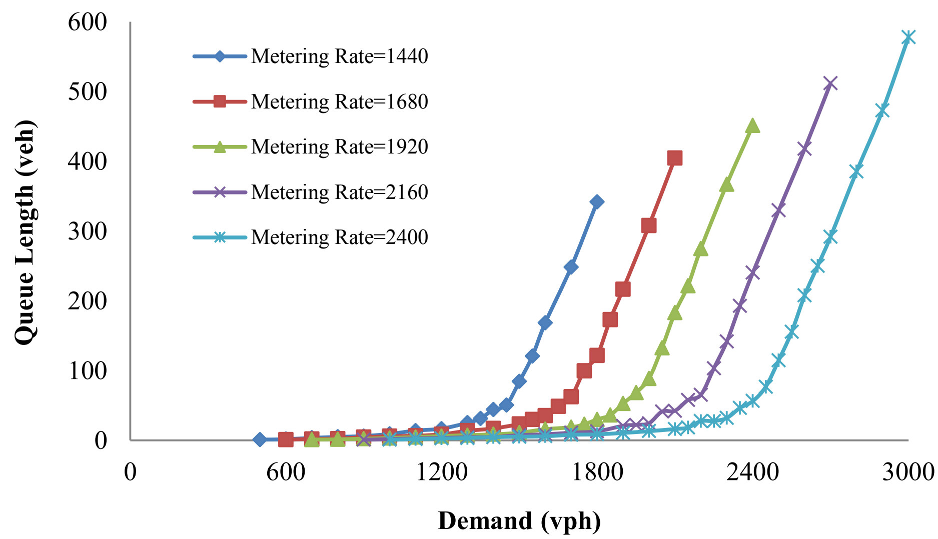
4.5.2 Freeway Connector

A freeway-to-freeway connector has different on-ramp flow arrival pattern in comparison with arterial on-ramp. Since there is no upstream signal at freeway connectors, on-ramp vehicles will arrive more uniformly. Also, it was found through field observations that traffic volume entering a freeway connector is usually much higher than those found at typical arterial on-ramps. For metered freeway connectors, traffic engineers can usually only control the capacity (i.e., metering rate); unlike arterial on-ramps, it is not possible to regular on-ramp demands by adjusting the upstream signals. With consideration of the unique on-ramp flow pattern of freeway connectors, the final simulation scenarios were classified into two categories: low metering rate conditions (average metering rate less than 1200 vph) and high metering rate conditions (average metering rate between 1200 vph and 2400 vph).

Based on the developed freeway connector queue length simulation model, different combinations of demand and metering rate scenarios were designed to cover a wide range of D/C ratios from approximate 0.4 to 1.25. For each demand versus metering rate scenario, ten simulation runs was performed to obtain the mean of the simulated 95th percentile queue length. Simulated queue lengths under various on-ramp demand and metering rate scenarios are illustrated in Figure 4-19 below.



(a) Low Metering Rate Conditions



(b) High Metering Rate Conditions

Figure 4-19 Simulated Queue Length under Various On-Ramp Demand and Metering Rate Scenarios

The simulated 95th percentile queue lengths are listed in Table 4-6 and Table 4-7 for quick estimation of queue length in number of vehicles under a given demand and metering rate scenario.

Table 4-6 Summary of the 95th Percentile Queue Lengths – Low Metering Rate Scenarios

Average Metering Rate (vph)	On-Ramp Demand (vph)												
	300	400	500	600	700	800	900	1000	1100	1200	1300	1400	1500
	95th Percentile Queue Length (veh)												
480	6	13	38	*117*	*n/a*	*n/a*	*n/a*	*n/a*	*n/a*	*n/a*	*n/a*	*n/a*	*n/a*
720	2	5	10	16	34	*91*	*175*	*n/a*	*n/a*	*n/a*	*n/a*	*n/a*	*n/a*
960	1	3	5	7	12	17	29	*64*	*140*	*234*	*n/a*	*n/a*	*n/a*
1200	n/a	1	3	4	6	8	12	18	30	43	*117*	*204*	*296*

Note: Queue lengths for D/C >=1 scenario are shown in red and italic.

Table 4-7 Summary of the 95th Percentile Queue Lengths – High Metering Rate Scenarios

Average Metering Rate (vph)	On-Ramp Demand (vph)												
	600	800	1000	1200	1400	1600	1800	2000	2200	2400	2600	2800	3000
	95th Percentile Queue Length (veh)												
1440	2	5	9	17	44	*169*	*342*	*n/a*	*n/a*	*n/a*	*n/a*	*n/a*	*n/a*
1680	1	3	6	9	17	35	*122*	*308*	*n/a*	*n/a*	*n/a*	*n/a*	*n/a*
1920	n/a	2	4	6	9	16	30	*89*	*275*	*451*	*n/a*	*n/a*	*n/a*
2160	n/a	n/a	2	4	5	9	13	25	*66*	*240*	*418*	*n/a*	*n/a*
2400	n/a	n/a	1	3	5	6	9	13	28	56	*208*	*385*	*578*

Note: Queue lengths for D/C >=1 scenario are shown in red and italic.

5. ACCELERATION CHARACTERISTICS AND ACCELERATION LENGTH

5.1 Acceleration Data Processing

5.1.1 Piecewise Constant Acceleration Model

Previous studies have demonstrated that an acceleration rate of the entire accelerating period is not a constant; however, it is reasonable to assume vehicles uniformly accelerate within a short time (or space) interval. Accordingly, a piecewise-constant acceleration model is used to calculate vehicle spot speeds at the pre-determined locations along the ramp.

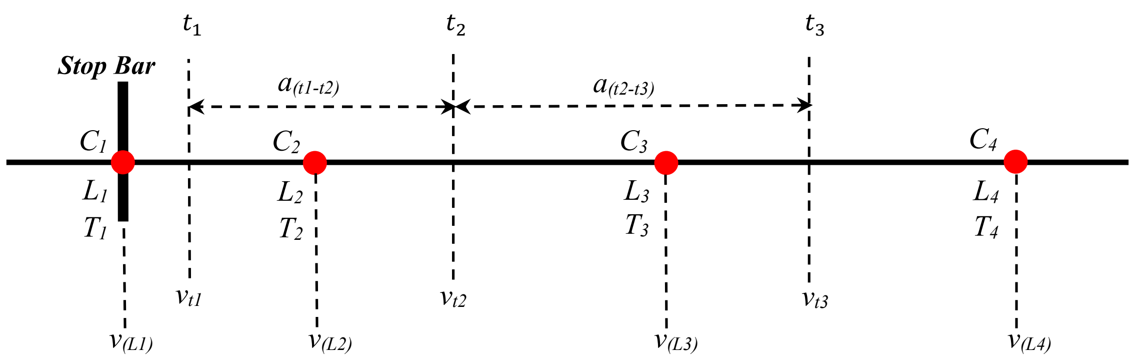


Figure 5-1 Spot Speed Data Extraction Procedure

With the given location information (distance downstream from the stop bar) and extracted time stamps, vehicle speed can be calculated by applying the kinematic theory. The calculation assumed that the vehicle has a fixed acceleration rate on a short segment. In Figure 5-1, C_i means the i^{th} reference cone and L_i is the location of the i^{th} reference cone from the ramp meter stop bar, which is determined prior to placing the cone. T_i is the time point of a vehicle passing the reference cone C_i, which is extracted from the video camera. So the average speed between adjacent cones of C_i and C_{i+1} is calculated by the following equation:

$$V_{i\sim i+1} = \frac{L_{i+1}-L_i}{T_{i+1}-T_i} \quad (5\text{-}1)$$

Based on the assumption that a vehicle has a fixed acceleration rate within a short time interval and according to the kinematic theory, a vehicle's average speed in a short time interval occurs at the middle-time point t_i of the i^{th} segment.

Where, $t_i = T_i + \frac{T_{i+1}-T_i}{2}$ (5-2)

Therefore, the real-time speed v_{ti} at the middle-time point of each segment t_i can be estimated using the average speed, i.e., $v_{ti} = V_{i\sim i+1}$.

For time interval $(t_1{\sim}t_2)$, it can also be written as: $(T_3 - T_1)/2$; during this time interval, speed increases from v_{t1} to v_{t2}; accordingly, the average acceleration rate of this period could be calculated as:

$$a_{t_1{\sim}t_2} = (v_2 - v_1)/(t_2 - t_1) \tag{5-3}$$

Knowing $V_{i{\sim}i+1}$, $a_{t_i{\sim}t_{i+1}}$ and T_i, then the spot speed at each cone location could be calculated using the following equation:

$$v_{i+1} = v_{ti} + a_{i{\sim}i+1} \times \frac{(T_{i+1} - T_i)}{2} \tag{5-4}$$

i.e., $v_{L_2} = v_{t_1} + a_{t_1{\sim}t_2} \times \frac{T_2 - T_1}{2}$; $v_{L_2} = V_2 + a_{t_2{\sim}t_3} \times \frac{T_3 - T_2}{2}$; $\cdots$

(note: $v_{L1} = v_{t_1} - a_{t_1{\sim}t_2} \times \frac{T_2 - T_1}{2}$)

5.1.2 Data Processing

With the video clips captured by cameras along a metered ramp, the time stamps when each vehicle passing reference cones were recorded; then data processing is conducted to extract speed information for each individual vehicle. The data extraction starts with time synchronization of videos recorded by different cameras. Each camera recorded the stopwatch time with an accuracy level of a hundredth second. The time offsets between the stopwatch time and video time of two consecutive cameras are calculated and then the relative offsets are calculated and used for the extraction of travel times between cones. An example of time synchronization and synchronized time series is demonstrated in Table 5-1(a) through Table 5-1(c). Properties of each captured vehicle were documented, including vehicle color, type, and model; therefore, the records of the same vehicle in different cameras can be easily identified so that the entire trajectories of a vehicle, including time and location information, along the acceleration lane are depicted. Table 5-1(d) through Table 5-1(g) illustrate how the spot speeds of each individual vehicle were calculated based on the aforementioned piecewise constant acceleration model.

Table 5-1 Example of Spot Speed Data Processing Procedure – EB Mowry Ave. to NB 880 Entrance Ramp

(a) Time Synchronization

Camera ID	A	B	C	D	E	F	G	H
Stopwatch Time	00.05.22	00.39.60	00.57.02	01.16.66	01.52.54	02.22.77	02.52.37	03.19.05
Video Time	00.03.14	00.03.26	00.02.10	00.01.32	00.01.26	00.02.65	00.02.18	00.02.49
Offset of Stopwatch (sec)	34.38	17.42	19.64	15.03	30.23	29.6	26.68	
Offset of Video (sec)	0.12	-1.16	-0.78	-0.23	1.39	-0.47	0.31	
Relative Offset (sec)	34.26	18.58	20.42	15.26	28.84	30.07	26.37	

(b) Original Data Extracted from Video Clips

				Time Point Passing Each Cone Location (Actual time point in video clips)							
Vehicle ID	Color	Type	Model	0 ft.	20 ft.	50 ft.	100 ft.	200 ft.	300 ft.	400 ft.	500 ft.
1	Black	Sedan	Dodge	03.48.43	03.15.31	02.58.04	02.39.17	02.05.56	01.38.70	01.10.32	00.45.45
2	White	Truck	Volvo	03.57.61	03.24.88	03.08.22	02.50.11	02.17.61	01.51.70	01.23.93	00.59.70
3	Silver	SUV	Honda	04.22.60	03.49.50	03.32.19	03.13.32	02.39.84	02.13.18	01.44.96	01.20.27

(c) Synchronized Time and Location Information

				Time Point Passing Each Cone Location (Synchronized time series)							
Vehicle ID	Color	Type	Model	0 ft.	20 ft.	50 ft.	100 ft.	200 ft.	300 ft.	400 ft.	500 ft.
1	Black	Sedan	Dodge	00:00.00	00:01.14	00:02.45	00:04.00	00:06.33	00:08.31	00:10.00	00:11.15
2	White	Truck	Volvo	00:00.00	00:01.53	00:03.45	00:05.76	00:09.20	00:12.13	00:14.43	00:16.57
3	Silver	SUV	Honda	00:00.00	00:01.16	00:02.43	00:03.98	00:06.44	00:08.62	00:10.47	00:12.15

(d) Travel Time between Adjacent Cones

				Travel Time (sec)						
Vehicle ID	Color	Type	Model	$T_{A\sim B}$	$T_{B\sim C}$	$T_{C\sim D}$	$T_{D\sim E}$	$T_{E\sim F}$	$T_{F\sim G}$	$T_{G\sim H}$
1	Black	Sedan	Dodge	1.14	1.31	1.55	2.33	1.98	1.69	1.5
2	White	Truck	Volvo	1.53	1.92	2.31	3.44	2.93	2.3	2.14
3	Silver	SUV	Honda	1.16	1.27	1.55	2.46	2.18	1.85	1.68

(e) Average Speeds between Adjacent Cones

				Average Speed (ft/s)						
Vehicle ID	Color	Type	Model	$V_{A\sim B}$	$V_{B\sim C}$	$V_{C\sim D}$	$V_{D\sim E}$	$V_{E\sim F}$	$V_{F\sim G}$	$V_{G\sim H}$
1	Black	Sedan	Dodge	17.54	22.90	32.26	42.92	50.51	59.17	66.67
2	White	Truck	Volvo	13.07	15.63	21.65	29.07	34.13	43.48	46.73
3	Silver	SUV	Honda	17.24	23.62	32.26	40.65	45.87	54.05	59.52

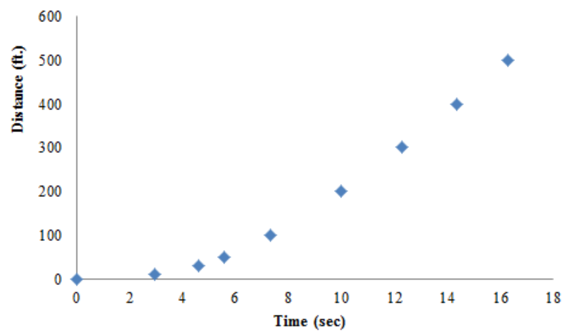
(f) Average Acceleration Rates between Two Middle Time Points

				Average Acceleration Rate (ft/s²)					
Vehicle ID	Color	Type	Model	$a_{t_{AB}} \sim a_{t_{BC}}$	$a_{t_{BC}} \sim a_{t_{CD}}$	$a_{t_{CD}} \sim a_{t_{DE}}$	$a_{t_{DE}} \sim a_{t_{EF}}$	$a_{t_{EF}} \sim a_{t_{FG}}$	$a_{t_{FG}} \sim a_{t_{GH}}$
1	Black	Sedan	Dodge	4.376	6.545	5.495	3.522	4.719	4.702
2	White	Truck	Volvo	1.484	2.846	2.581	1.589	3.576	1.464
3	Silver	SUV	Honda	5.251	6.128	4.185	2.250	4.060	3.099

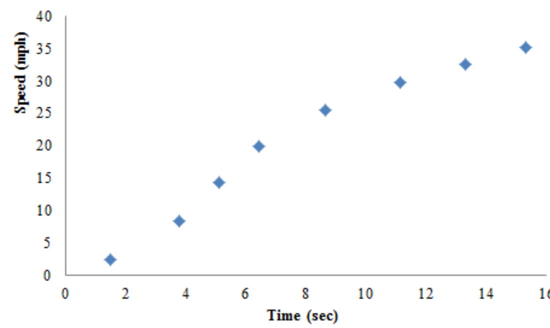
(g) Spot Speeds at Designated Locations

				Spot Speed (ft/s)							
Vehicle ID	Color	Type	Model	V_0	V_{20}	V_{50}	V_{100}	V_{200}	V_{300}	V_{400}	V_{500}
1	Black	Sedan	Dodge	15.05	20.04	27.19	36.35	48.05	55.18	63.14	70.19
2	White	Truck	Volvo	11.94	14.20	18.36	24.55	32.52	39.37	45.16	48.30
3	Silver	SUV	Honda	14.20	20.29	27.51	35.46	44.30	50.30	56.92	62.13

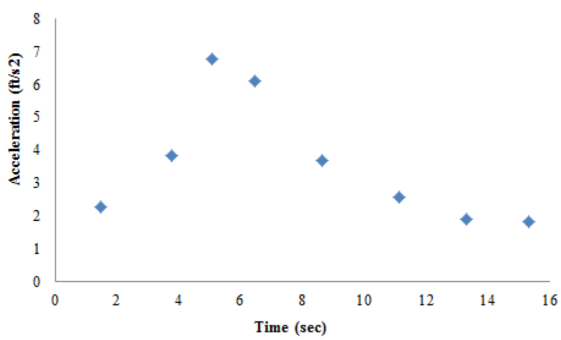
One of the advantages of the video-based method is its capability of tracking and retrieving each individual vehicle. Therefore, a vehicle that made a complete stop at the stop bar was selected and presented in Figure 5-2 to illustrate the data analysis results of the proposed procedure. Five curves are generated to show the typical profiles of different parameters: time-distance profile, time-speed profile, time-acceleration profile, distance-speed profile, and distance-acceleration profile.



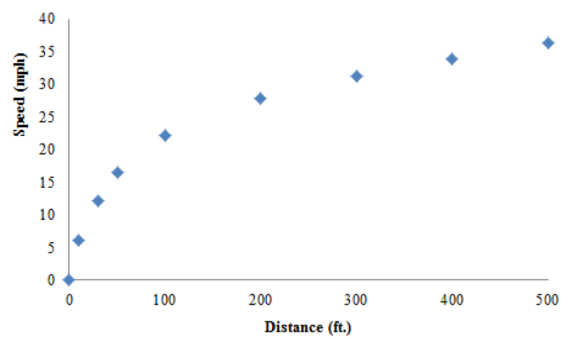
(a) Time versus Distance



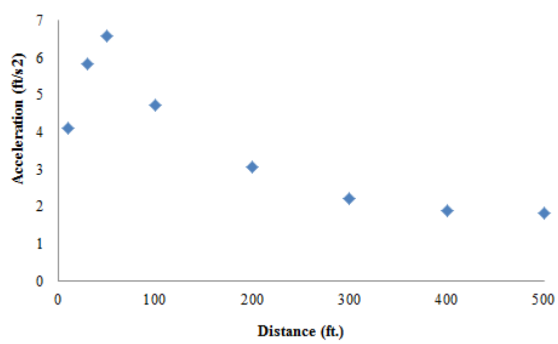
(b) Time versus Speed



(c) Time versus Acceleration



(d) Distance versus Speed



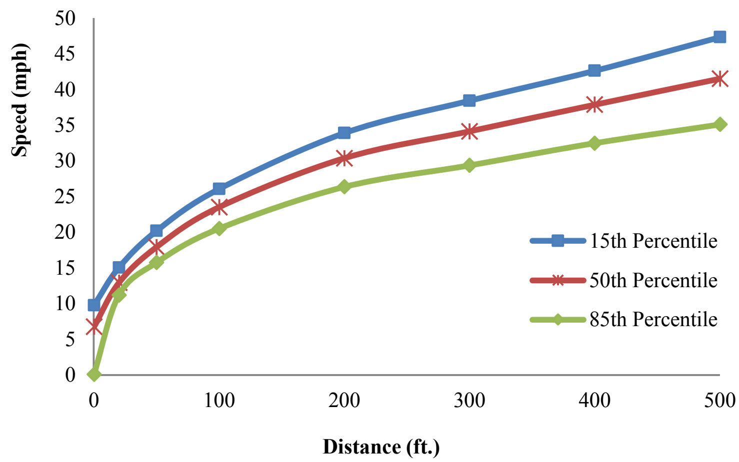
(e) Distance versus Acceleration

Figure 5-2 An Example of the Proposed Data Extraction Method

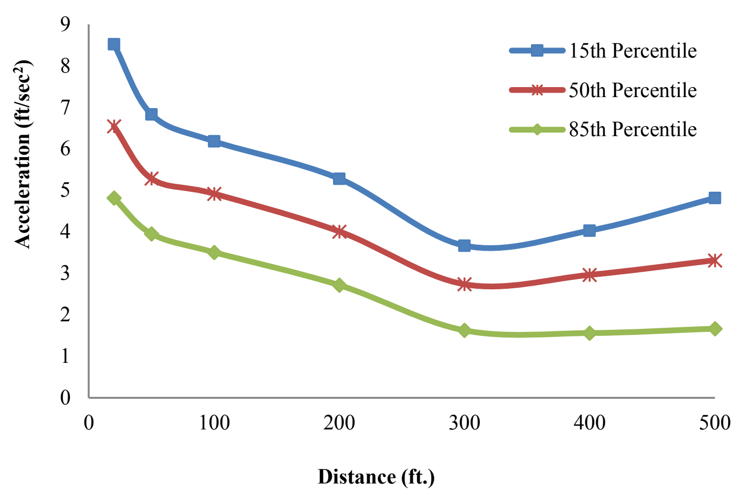
5.2 Acceleration Characteristics Study

5.2.1 Percentile Speed versus Distance Profiles

The video based method has the capability of identifying the percentile speed and acceleration rate at different locations of the acceleration lane. For each data collection site, the 15th percentile, 50th percentile, and 85th percentile spot speeds at each cone location were identified. The 15th percentile speed means that 15 percent of speeds are lower than this speed and the 85th percentile speed means 85 percent of speeds are lower than this speed. Percentile speed and acceleration values could be used for determining the upper or lower boundaries of required acceleration length. Figure 5-3 illustrates the profiles of 15th, 50th, and 85th percentile speed versus distance and the profiles of acceleration versus distance, which were plotted with the data collected on the Industrial Pkwy to NB 880 metered on-ramp.



(a) Speed versus Distance Profiles

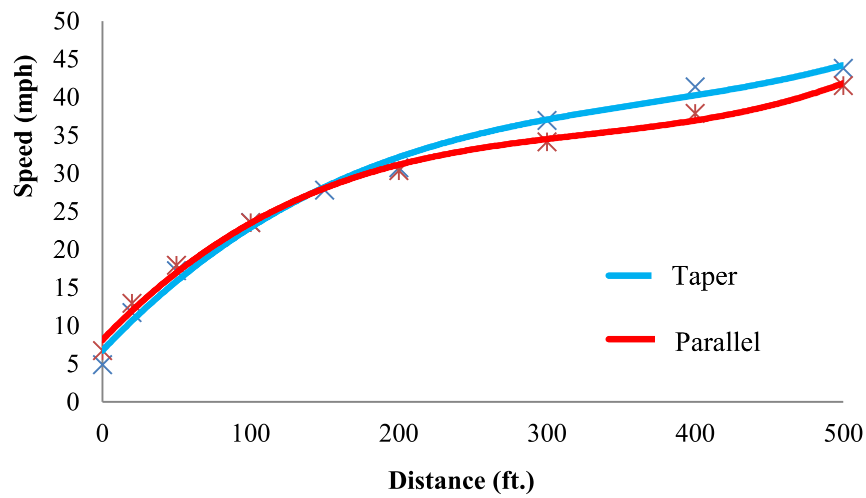


(b) Acceleration versus Distance Profiles

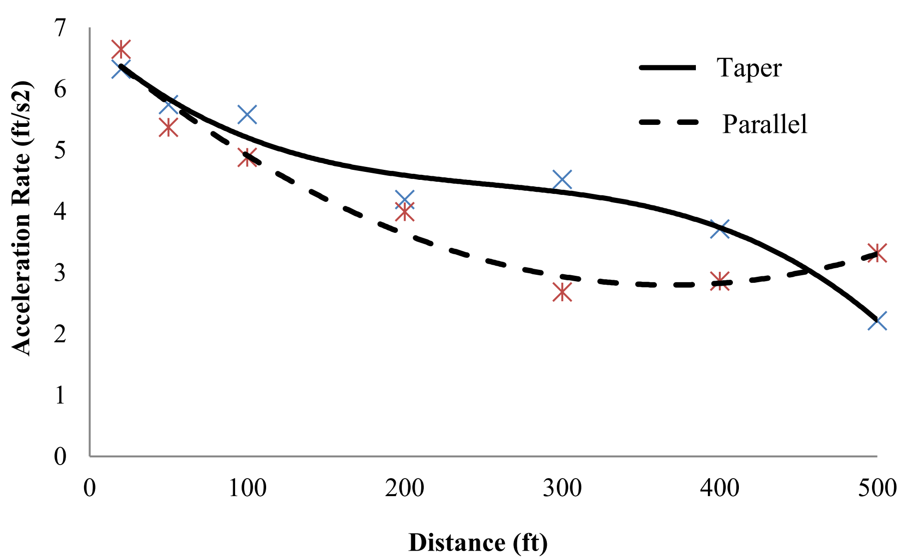
Figure 5-3 Percentile Speed and Acceleration Profiles of a Passenger Car at Industrial Pkwy to NB 880 Metered Ramp

5.2.2 Taper vs. Auxiliary Lane Ramp

Speed and acceleration patterns are influenced by the geometric designs of metered on ramps. Speed-distance and acceleration-distance profiles based on data at two entrance ramps with different geometrics are compared here. The taper type ramp provides a direct entry onto the freeway at a flat angle while the auxiliary lane ramp provides an extension of the acceleration lane to meet the freeway mainline running speed. Acceleration characteristics of the two ramp types may therefore be different. Ramp metering sites EB Mowry Ave to NB 880 and Industrial Pkwy to NB 880 have similar existing acceleration lengths (i.e., from stop bar to the gore) and freeway mainline traffic flow conditions. Consequently they were chosen for the comparison study. Figure 5-4(a) and Figure 5-4(b) present the difference of the speed-distance and acceleration-distance profiles at the two metered on-ramps, respectively.



(a) Speed Profile



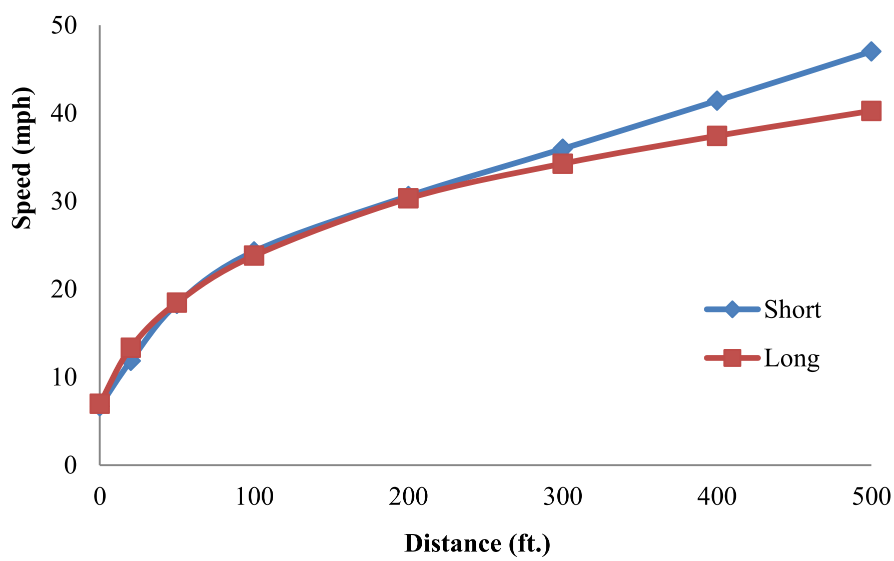
(b) Acceleration Profile

Figure 5-4 Taper Type Ramp versus Auxiliary Lane Type Ramp

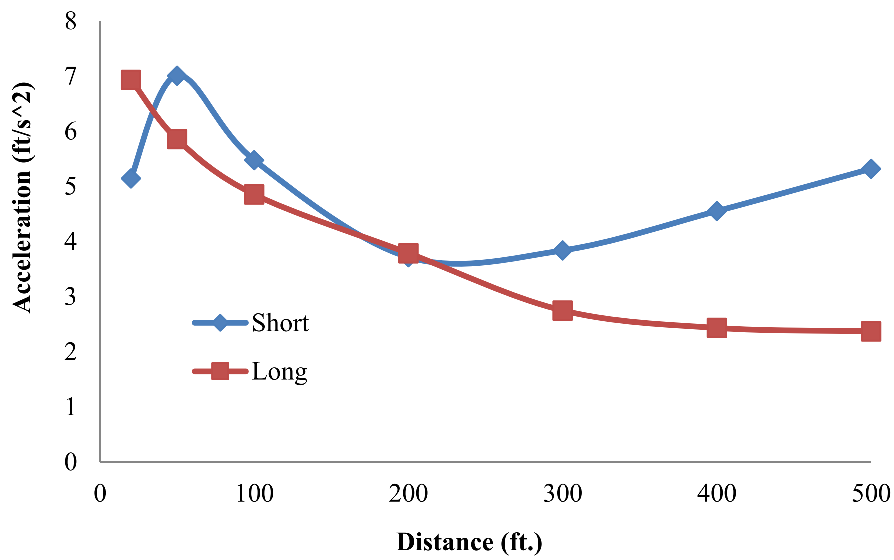
Results show that accelerations on the two ramps are similar at the beginning stage. Acceleration on the ramp with an auxiliary lane decreases sharper than the taper type ramp. In general, higher accelerations were observed at the taper type ramp than the ramp with an auxiliary lane. Vehicles on ramps with auxiliary lanes tend to have a longer acceleration length. Their acceleration-distance profiles indicate that the acceleration behavior is to have a high acceleration rate in the beginning, then decrease the acceleration rate in the middles stage, and increase acceleration again as drivers approach the merging area. In comparison, vehicles at taper type ramps tend to stay at a higher acceleration rate.

5.2.3 Short vs. Long Existing Acceleration Length

In reality, the existing acceleration length also influences drivers' acceleration behavior. Comparison was made for two taper type ramps that have similar traffic conditions; The Artesia Blvd metered on-ramp has a total available acceleration length (i.e., including both acceleration and merge) of 475 feet, and the total available acceleration length of Rosecrans Blvd. metered on-ramp is 4450 feet. Acceleration versus distance profiles of the two ramps are illustrated in Figure 5-5.



(a) Speed Profile



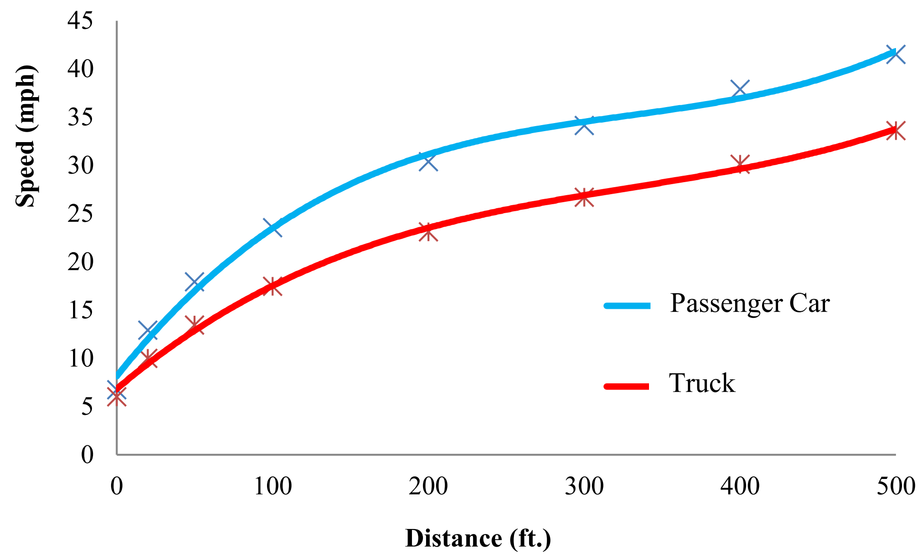
(b) Acceleration Profile

Figure 5-5 Short Existing Acceleration Lane Ramp versus Long Existing Acceleration Lane Ramp

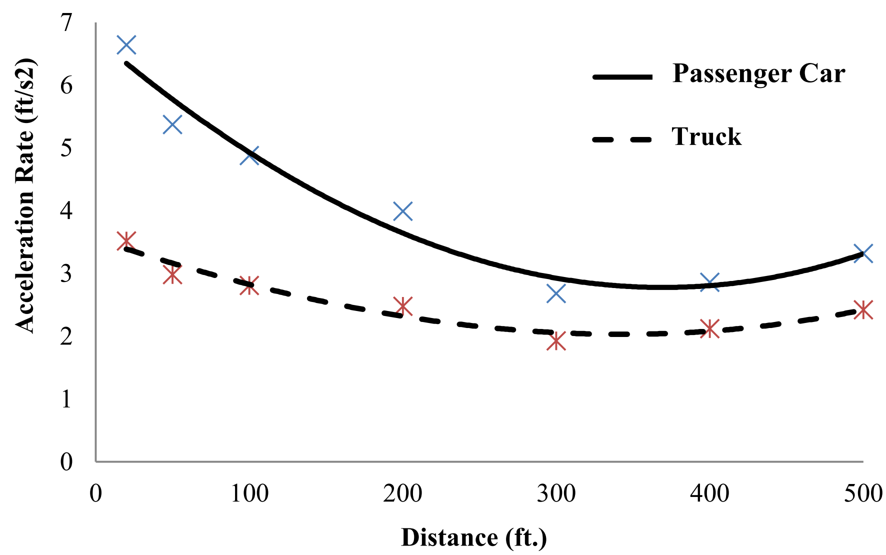
Results also show that the acceleration rates of the two ramps are similar in the beginning. With speed increasing, the existing acceleration length would certainly influence drivers' acceleration behavior. In general, drivers tend to accelerate slower when given a longer acceleration distance; and drivers accelerate at a higher rate when shorter acceleration length is given. Also, for the on-ramp with a longer existing acceleration length (Rosecrans Blvd. metered on-ramp), the acceleration versus distance profile indicates an exponential decreasing trend. In comparison, for the on-ramp with a shorter existing acceleration length (Artesia Blvd metered on-ramp), the general trend is that the acceleration rates decrease with the speed increasing; when vehicles are approaching the merging area, drivers are more likely to accelerate at higher acceleration rates so as to catch up with the freeway mainline speed and merge into the freeway.

5.2.4 Passenger Car vs. Truck

Acceleration capabilities of different vehicle types vary and trucks usually have lower acceleration capabilities than passenger cars. For ramp metering sites with high truck volume (e.g., truck volume higher than 5 percent), it is necessary to provide a longer acceleration length or an auxiliary lane to accommodate trucks accelerating to the desired merge speed. Truck acceleration performance data were collected at the Industrial Pkwy to NB 880 metered on-ramp and the acceleration versus distance profile is compared with that of passenger car, as illustrated in Figure 5-6.



(a) Speed Profile



(b) Acceleration Profile

Figure 5-6 Passenger Car versus Truck

Results show that the acceleration versus distance profile of trucks indicates a similar trend as passenger cars. As expected, field collected data confirm that passenger cars have a higher acceleration rate than trucks. The acceleration capability of a truck is approximately 60 percent of a passenger car, which can be used as a rule of thumb when truck acceleration performance data are not available.

5.2.5 Major Findings from Acceleration Characteristics Study

This section presents a qualitative analysis of the acceleration profile at metered on-ramps. Major findings of this acceleration characteristics study are presented as follows:

- Acceleration rate at metered on-ramps is not constant. Drivers tend to accelerate at a higher acceleration rate when speed is lower and vice-versa.
- Acceleration versus distance profiles of various ramp geometric configurations differ from one another. Acceleration data analysis results show that polynomial acceleration models would better capture the realistic acceleration behavior.
- For ramp metering sites that have similar existing acceleration lane length, higher acceleration rates were observed at on-ramps with taper merging type than auxiliary lane merging type. Similarly, ramps with a shorter existing acceleration lane tend to produce higher acceleration rates.
- In general, the acceleration profile of taper merging ramps indicates a decreasing trend with speed increase. For ramps with an auxiliary lane that have sufficient acceleration distance, the entire accelerating process could be divided into two stages: in the first stage, acceleration rates decrease with the speed increasing; and then, when vehicles are approaching the merging area, drivers are more likely to accelerate at higher acceleration rates so as to catch up with the freeway mainline speed and merge into the freeway.
- Field data shows that the existing acceleration length influences acceleration behavior. For taper merging ramps with a long existing acceleration length, the acceleration profile indicates an exponential decreasing trend. For ramps with a shorter existing acceleration length, the acceleration-decreasing trend is smoother during the accelerating process, and an S shape acceleration-distance profile was observed.

5.3 Acceleration Length Estimation

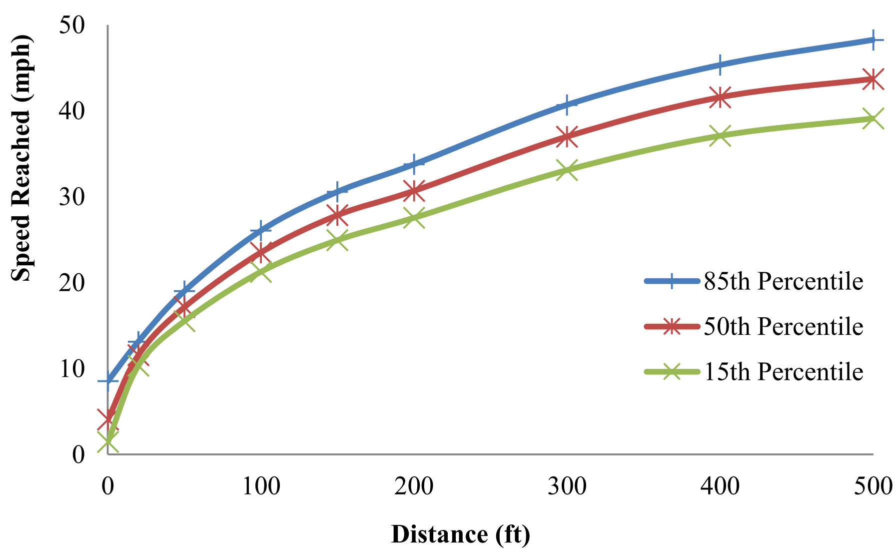
5.3.1 Distance-Speed Regression Model

Spot speeds of each individual vehicle at the pre-determined locations were extracted from the field videos. For each data collection site, the 15th percentile, 50th percentile, and 85th percentile spot speeds at each cone location were identified. Figure 5-7(a) demonstrates the field observed percentile speed versus distance profiles at the EB Mowry Ave. to NB 880 ramp-metering site, which is based on 395 sample passenger cars captured by cameras. The speed profiles show that the 15th percentile, 50th percentile and 85th percentile spot speeds are respectively 39.1 mph, 43.7 mph and 48.3 mph at the 500-ft point of this ramp. It indicates for the Mowry Ave. ramp-metering site that on average passenger car drivers can accelerate from 0 mph to approximately 44 mph in 500 feet, and about 15 percent of drivers will reach a speed higher than 48 mph while another 15 percent of drivers are not able to accelerate to 39 mph in 500 feet.

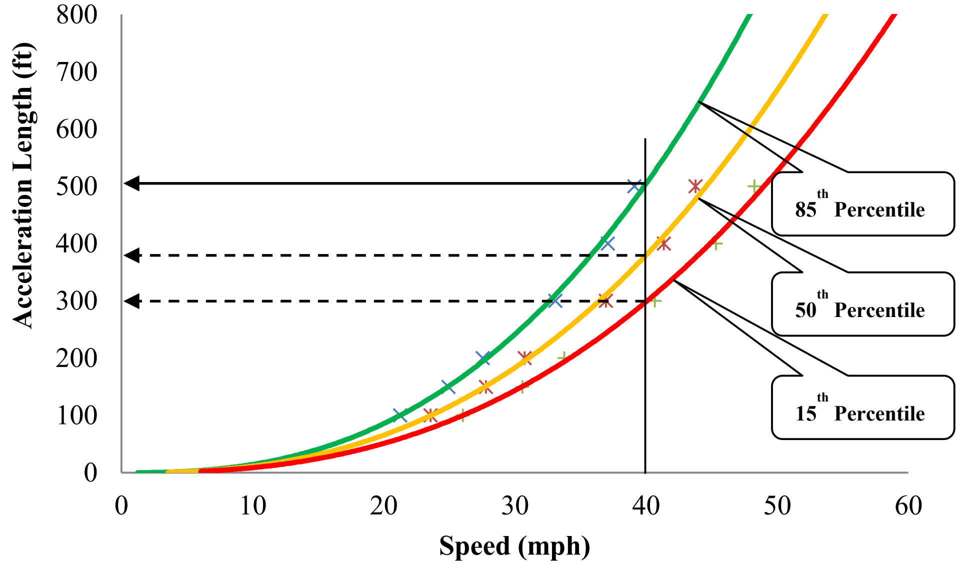
Based on the field observed speed versus distance profiles of Figure 5-7(a), the authors employed the regression analysis method to generate the distance versus speed equations, since such equations could better describe the required acceleration lengths for a given speed. It was found that the power function model would best capture the realistic distance versus speed profile. The 85^{th} percentile, 50^{th} percentile and 15^{th} percentile distance versus speed regression models for this particular entrance ramp can be described by the following power functions and also demonstrated in Figure 5-7(b).

$$\begin{cases} L_{85^{th}\ Percentile} = 0.0402 \times v^{2.5580}, & R^2 = 0.9968 \\ L_{50^{th}\ Percentile} = 0.0334 \times v^{2.5312}, & R^2 = 0.9972 \\ L_{15^{th}\ Percentile} = 0.0249 \times v^{2.5453}, & R^2 = 0.9976 \end{cases}$$

The generated speed-distance relationships can be used as a recommendation of acceleration length design for ramp metering sites with similar geometric and traffic conditions. For example, if another site has similar geometric and traffic conditions as the EB Mowry Ave. to NB 880 metered on-ramp, and knowing the merging speed is 40 mph, then the estimated medium acceleration length would be 370 feet and the allowable acceleration length can be in the range of 300 feet and 485 feet. The 85^{th} percentile acceleration length means 85 percent of the drivers need a shorter distance to accelerate to the given speed. Therefore, to accommodate the majority of drivers to accelerate to a safe merging speed, the authors recommend using the 85^{th} percentile distance as the minimum acceleration lane length design value.



(a) Field Observed Percentile Speed versus Distance Profiles



(b) The Distance versus Speed Regression Model for Acceleration Length Prediction

Figure 5-7 An Example of Speed Profile Model for Acceleration Length Predication at EB Mowry Ave. to NB 880 Metered Ramp

A probe vehicle equipped with an iPhone-based GPS trajectory recorder was employed to collect speed data for accuracy testing of the acceleration length prediction method. The original output data were second-by-second speed information; detailed speed and time information was exported to a spreadsheet, from which the time point that the probe vehicle passing the stop bar could be easily identified. Probe vehicle GPS trajectory data within the first 500 feet from the stop bar were selected for regression analysis to predict required acceleration lengths at higher speeds. Three tests were made in this study; acceleration lengths were rounded to the nearest 5 ft. The predicted lengths were compared with GPS trajectory data, as illustrated in Figure 5-8. Also, mean percentage errors (MPE) of prediction were identified. MPEs were calculated through equation (5) and are listed in Table 5-2.

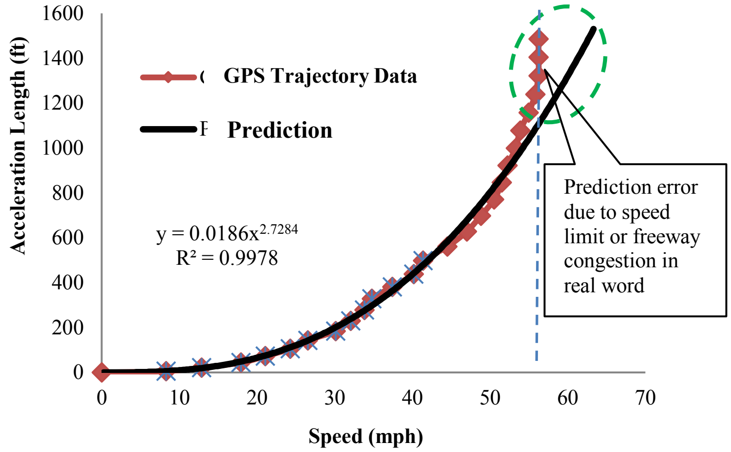
$$\mathrm{MPE} = \frac{100\%}{n}\sum_{i=1}^{n}\frac{A_i - F_i}{A_i} \qquad (5)$$

Where,

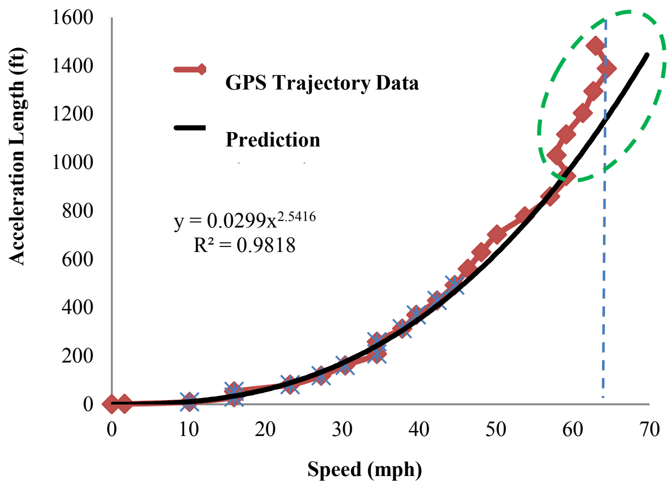
A_i is the actual value of the quantity being forecast;

F_i is the forecast; and

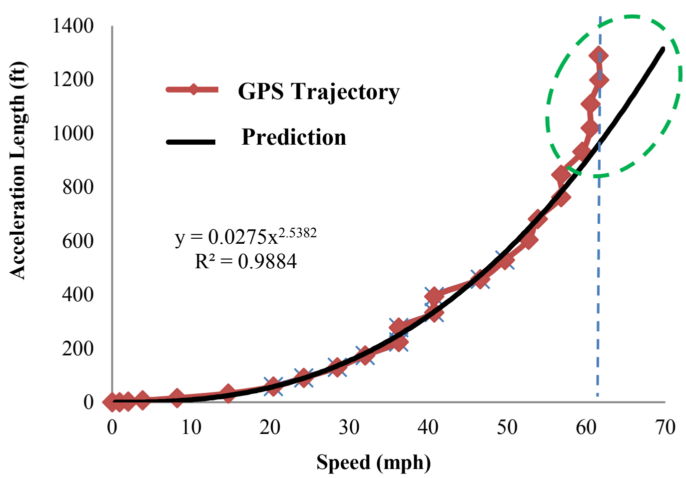
n is the number of different times for which the variable is forecast.



(a) Test 1 - Freeway Running Speed $V_f \approx 55$mph



(b) Test 2 - Freeway Running Speed $V_f \approx 65$mph



(c) Test 3 - Freeway Running Speed $V_f \approx 61$mph

Figure 5-8 GPS Trajectory Data for Model Validation

Table 5-2 Mean Percentage Error of Distance versus Speed Regression Models

		Speed reached (mph)								
		20	25	30	35	40	45	50	55	60
		Acceleration length (ft.) to reach this speed								
Test 1	GPS Trajectory Data	70	120	185	330	435	570	770	1,160	n/a
	Prediction	65	120	200	305	435	600	800	1,045	1,320
	MPE ($n=1$)	-7.1%	0.0%	8.1%	-7.6%	0.0%	5.3%	3.9%	-9.9%	n/a
Test 2	GPS Trajectory Data	65	100	155	265	370	495	675	800	1,100
	Prediction	60	105	170	250	355	475	620	795	990
	MPE ($n=1$)	-7.7%	5.0%	9.7%	-5.7%	-4.1%	-4.0%	-8.2%	-0.6%	-10.0%
Test 3	GPS Trajectory Data	55	100	150	220	330	450	550	750	950
	Prediction	55	95	155	225	320	430	565	720	895
	MPE ($n=1$)	0.0%	-5.0%	3.3%	2.3%	-3.0%	-4.4%	2.7%	-4.0%	-5.8%

Note: For each test, there is a unique GPS trajectory data and prediction. Even if all the tests were performed by the same driver using the same vehicle and at the same ramp, there would not be two identical trajectories. Therefore, for each test the value of n can only be "1", and it is incorrect to calculate MPEs by averaging the three tests.

Results indicated the predicted acceleration lengths were consistent with GPS trajectory data. Generally, the MPEs are lower than 10 percent. It can be seen that for low merge speed conditions (i.e., lower than 50 mph), the maximum absolute error between prediction and GPS trajectory data is 30 ft. (except Test 2 at 50 mph). It is necessary to point out that the proposed model tends to underestimate acceleration lengths for high merge speeds when it is close to the speed limit (e.g., 5 mph lower than freeway running speed or posted speed limit). This is mainly caused by the monotonous increasing nature of a power function, since in real world condition speed will not increase after reaching the freeway running speed or speed limit.

5.3.2 Acceleration Length Recommendation

With the aforementioned method, regression analyses were applied to the distance versus speed profiles at each entrance ramp. Accordingly, a summary of the 85th percentile and 50th percentile predicted acceleration lengths for various merging speeds are listed in Table 5-3. Detailed acceleration length predictions for each site are presented in Appendix B.

Table 5-3 Predicted 85th Percentile and 50th Percentile Acceleration Lengths

Ramp Location	Sample Size	Distance versus Speed Regression Model	R^2	Merge Speed (mph) 30	35	40	45	50	55	60
				Predicted Acceleration Length (ft.) to reach this speed						
EB Mowry Rd. to NB 880	395	$L_{85^{th}\%} = 0.0277v^{2.6688}$	0.9968	240	360	505	680	890	1,140	1,420
		$L_{50^{th}\%} = 0.0334v^{2.5312}$	0.9972	185	270	380	510	665	850	1,060
WB Alvarado Rd. to SB 880	156	$L_{85^{th}\%} = 0.0277v^{2.6688}$	0.9906	240	365	520	715	950	1,220	1,540
		$L_{50^{th}\%} = 0.0215v^{2.6489}$	0.9951	175	265	375	515	680	875	1,100
Artesia Blvd. to NB 405	70	$L_{85^{th}\%} = 0.0558v^{2.4816}$	0.9954	260	380	530	705	920	1165	1,445
		$L_{50^{th}\%} = 0.0486v^{2.4230}$	0.9961	185	265	370	490	635	800	990
SB Douglas Blvd. to WB 80	223	$L_{85^{th}\%} = 0.0033v^{3.2052}$	0.9842	180	295	450	655	920	1,250	1,650
		$L_{50^{th}\%} = 0.0081v^{2.8856}$	0.9911	150	230	340	480	650	850	1,095
Fruitridge Rd. to NB 99	100	$L_{85^{th}\%} = 0.0121v^{2.9331}$	0.9943	255	410	605	855	1,165	1,540	1,990
		$L_{50^{th}\%} = 0.0112v^{2.8550}$	0.9992	185	285	420	585	795	1,040	1,335
Industrial Pkwy. to NB 880	626	$L_{85^{th}\%} = 0.0210v^{2.8228}$	0.9984	310	480	700	975	1,310	1,720	2,195
		$L_{50^{th}\%} = 0.0166v^{2.7676}$	0.9992	205	310	450	625	835	1,090	1,385
WB Rosecrans Ave. to NB 710	88	$L_{85^{th}\%} = 0.0203v^{2.8256}$	0.9992	305	470	685	950	1285	1,680	2,145
		$L_{50^{th}\%} = 0.0092v^{2.9440}$	0.9986	205	325	480	675	925	1,225	1,580

Note: $L_{85^{th}\%}$ and $L_{50^{th}\%}$ represent for the 85th percentile and 50th percentile acceleration lengths of entrance ramps; Sample size for passenger vehicle only.

Modeling results revealed that the existing acceleration length is the primary factor impacting drivers' acceleration behavior and consequently, the required acceleration lengths. Therefore, in this study, predicted acceleration lengths were categorized into two groups; group one: ramps with short existing acceleration length (mean 85th percentile predicted acceleration lengths for the four typical taper ramps), and group two: ramps with long existing acceleration length (mean 85th percentile predicted acceleration lengths for the two auxiliary lane ramps and the Rosecrans Avenue metered on-ramp). The mean values could be used as the minimum acceleration lane design lengths for metered ramps. Two design standards are recommended. Predicted lengths from group one is recommended as the aggressive design standard, which would be used for metered ramps with restricted geometric conditions or insufficient queue storage space. For ramps that have adequate space, the conservative design standard, which is generated from group two, is recommended so that drivers are able to accelerate at a more comfortable manner.

5.3.3 Major Findings from Acceleration Length Prediction

Since the acceleration rate at metered entrance ramps is not constant, kinematic equations cannot be directly used to calculate acceleration distance. In comparison, distance versus speed regression models could better capture the realistic accelerating behavior and thus are more applicable than kinematic model for acceleration lane length design. Based on the field collected acceleration performance data, it was found that existing acceleration length is the primary factor affecting drivers' acceleration behavior and consequently, the required acceleration lengths. For ramps with a long acceleration length (e.g., ramps with an auxiliary lane or a long acceleration lane), drivers tend to accelerate at a lower and more comfortable acceleration rate compared to at ramps that have shorter acceleration length.

5.4 Truck Acceleration Capability Study

The impacts of truck acceleration capability on freeway on-ramp acceleration lane length design have aroused transportation engineers' concern since the 1950s *(14)*. A number of studies have been made in attempts of investigating the truck speed and acceleration profiles *(39,40,48)* and updating freeway on-ramp acceleration length design for trucks *(15,16)*. However, so far to date there is no specific document that provides truck acceleration performance data for acceleration length design at metered on-ramp.

The ITE's (Institute of Transportation Engineers) *"Traffic Engineering Handbook" (49)* provides tables and charts that describe the speed versus distance relationships during the maximum acceleration rate for tractor-semitrailer trucks with various weight-to-power ratios, as reproduced in Table 5-4. However, the maximum accelerations are not adequate to determine the proper acceleration lengths required on the freeway because the acceleration behavior of vehicles depends not only on vehicle capabilities but also on driver preferences. In reality, drivers usually accelerate at a normal acceleration rate, which is lower than the maximum capability. With consideration of the potential impacts of ramp metering on drivers' acceleration behavior, existing truck acceleration studies may not be applicable for metered on-ramps acceleration length design. Therefore, it is of significant importance to investigate the actual truck acceleration capability at existing metered on-ramps to determine the sufficient acceleration length that could accommodate truck drivers to accelerate to a desired merge speed.

Table 5-4 Typical Maximum Acceleration Rates for Tractor-Semitrailer Combination Trucks from ITE Traffic Engineering Handbook

(a) Maximum Acceleration from Standing Start

Vehicle Type	Weight-to-Power Ratio (lb/hp)	Typical Maximum Acceleration Rate on Level Road (ft/s^2)				
		0 to 10 mph	0 to 20 mph	0 to 30 mph	0 to 40 mph	0 to 50 mph
Tractor-Semitrailer	100	2.9	2.3	2.2	2.0	1.6
	200	1.8	1.6	1.5	1.2	1.0
	300	1.3	1.3	1.2	1.1	0.6
	400	1.3	1.2	1.1	0.7	---

(b) Maximum Acceleration for 10 mph Increments

Vehicle Type	Weight-to-Power Ratio (lb/hp)	Typical Maximum Acceleration Rate on Level Road (ft/s^2)			
		20 to 30 mph	30 to 40 mph	40 to 50 mph	50 to 60 mph
Tractor-Semitrailer	100	2.1	1.5	1.0	0.6
	200	1.3	0.8	0.5	0.4
	300	1.0	0.6	0.3	---
	400	0.9	0.4	---	---

Truck acceleration performance data under actual conditions were collected at two existing metered on-ramps in the San Francisco Bay Area, California. The Industrial Pkwy to NB 880 ramp-metering site has an auxiliary lane; therefore, truck drivers are provided sufficient space for acceleration. In comparison, the Mowry Ave to NB 880 ramp-metering site is a taper type on-ramp with limited existing acceleration length. Geometric and traffic features of the two candidate sites are listed in Table 5-5.

Table 5-5 Geometric Features and Traffic Conditions of Data Collection Sites

Criteria		Industrial Pkwy to NB 880	EB Mowry Ave. to NB 880
Geometric Features	Merging Type	Auxiliary lane	Taper
	Existing Length (ft.)*	395	390
	On-ramp Lane	1+HOV	1+HOV
	Grade	Flat	Flat
Traffic Conditions	Freeway Flow	Uncongested	Uncongested
	On-ramp Demand	Medium	Low
	Sample Size	174	55

Note: *Existing length is the distance from the stop bar to the gore; location of the gore was illustrated in Figure 3-11(a).

5.4.1 Vehicle Classification

Based on the field observation in California sites, in this paper, trucks are categorized into three types: light, medium, and heavy duty trucks. Description and graphic example of each truck type are listed in Table 5-6.

Table 5-6 Truck Type Defined in This Study

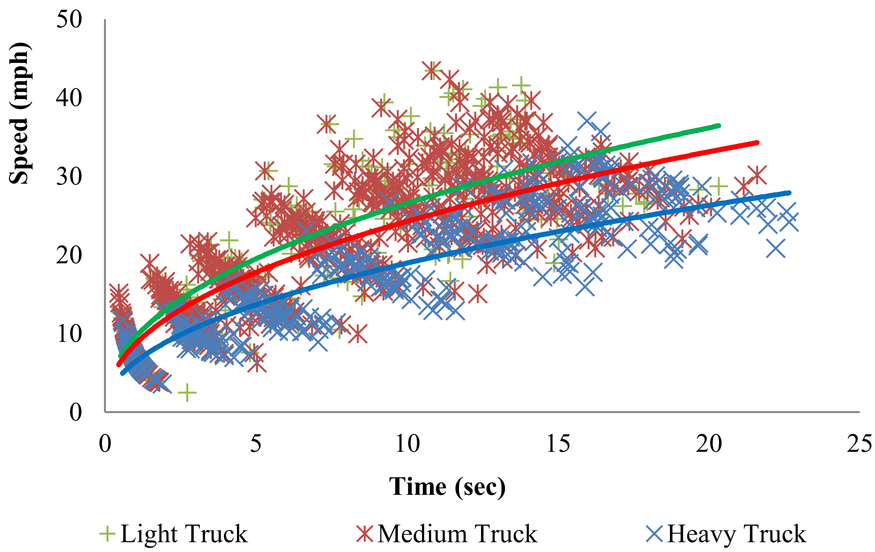
Truck Type Defined in This Study	Vehicle Description	Typical Model
Light Duty Truck	Single unit 2-axle trucks	
Medium Duty Truck	Single unit, 3 or more axles trucks	
Heavy Duty Truck	Single trailer, 3, 4, 5 axles trucks	

5.4.2 Acceleration Profiles of Different Truck Classes

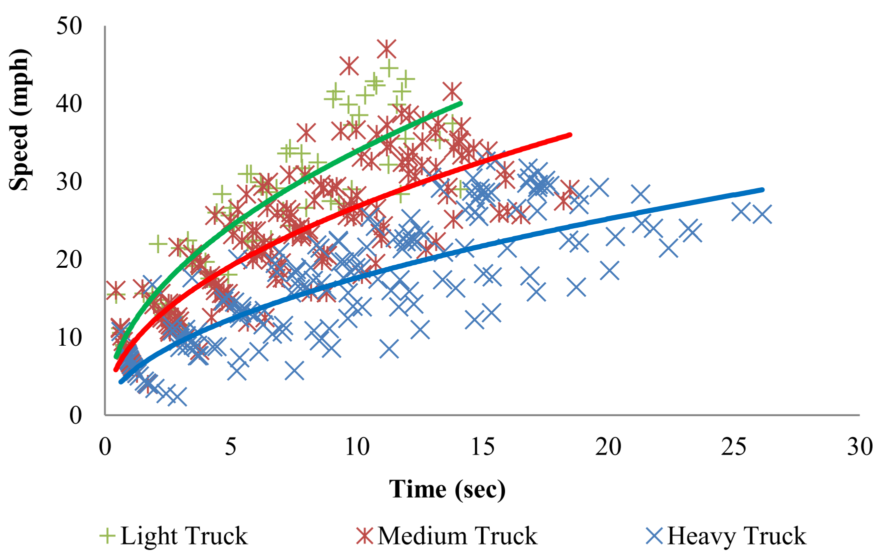
One of the advantages of the video-based method is its capability of selecting an individual sample, so that samples pertaining to each vehicle type would be identified manually. Although this method calls for extensive labor, it has the ability of providing more accurate vehicle classification and speed measurement.

Speed versus Time Profile

A total number of 229 truck trajectories were captured by the cameras; based on the proposed piecewise constant acceleration model, spot speeds were calculated and the speed versus time profile of each truck type was generated, as illustrated in Figure 5-9. As expected, light trucks can accelerate to a higher speed in a given time frame. On average, light truck drivers can accelerate from the stop condition to approximately 37 mph in 500 feet, which was based on 43 sample vehicles. In comparison, medium (115 sample vehicles) and heavy (71 sample vehicles) truck drivers can accelerate to approximately 34 mph and 31 mph in 500 feet, respectively.



(a) Industrial Pkwy to NB 880 Auxiliary Lane Type Ramp

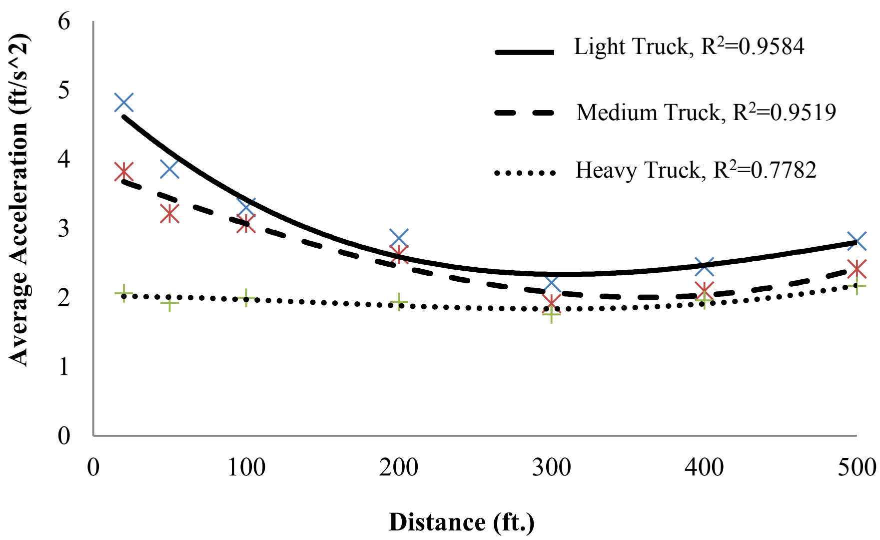


(b) EB Mowry Ave to NB 880 Taper Type Ramp

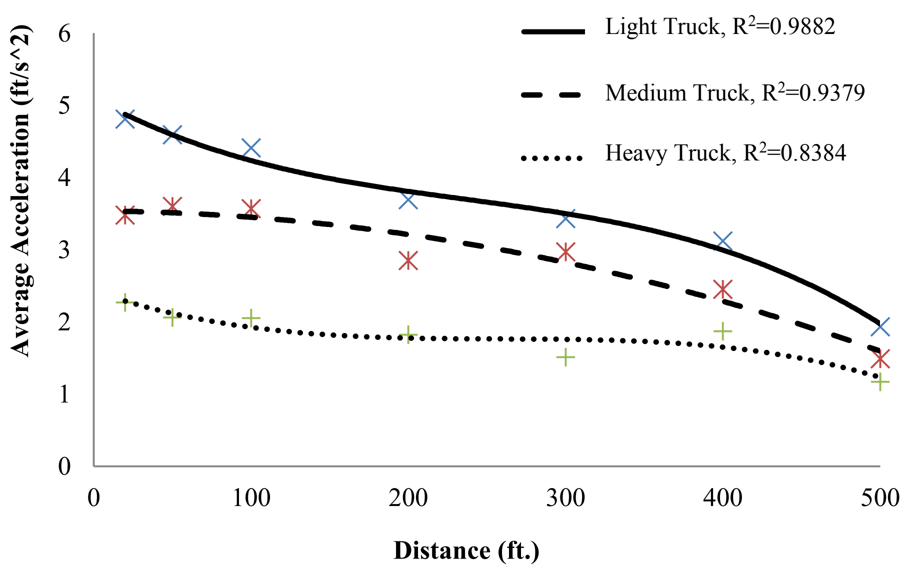
Figure 5-9 Speed versus Time Scatter Plots and Profiles of Three Truck Types

Acceleration Profiles of Different Truck Types

For each individual sample, the extracted location versus time information was eventually used to generate acceleration versus location (or time) profile. For demonstration purposes, the average acceleration values of each truck type were used. Polynomial regression analysis of the field collected acceleration data was employed to generate acceleration versus distance profiles at two typical ramps: taper type ramp and auxiliary lane type ramp, as illustrated in Figure 5-10(a) and 5-10(b), respectively.



(a) Industrial Pkwy to NB 880 Auxiliary Lane Type Ramp



(b) EB Mowry Ave to NB 880 Taper Type Ramp

Figure 5-10 Average Acceleration versus Distance Profiles of Three Truck Types

Results show that, for an auxiliary lane type ramp that has a longer potential acceleration length, the acceleration versus distance profiles indicate that the acceleration behavior is to have a high acceleration rate in the beginning, then the acceleration rate decreases as the speed increases, and acceleration rate increases again as drivers approach the merging area. In comparison, acceleration behavior at a taper type ramp indicates an exponential decreasing trend with speed increase.

Truck Acceleration Performance Data

Table 5-7 documents the piecewise-constant average acceleration rates of the three truck types at the two metered on-ramps. Also, to better illustrate the actual acceleration capability of different trucks, the average acceleration rate from the stop bar to 500 feet downstream (i.e., assume a constant acceleration rate during the entire accelerating period) was calculated, including the mean, the 15th percentile and the 85th percentile acceleration performance data, as listed in Table 5-7.

Table 5-7 Truck Acceleration Performance Data at Two Metered On-Ramp

Ramp Type	Truck Type	Sample Size	Piecewise-constant Average Acceleration Rates (ft/s^2)							0-500 ft. Average Acceleration Rate (ft/s^2)			
			$a_{0\text{-}20}$	$a_{20\text{-}50}$	$a_{50\text{-}100}$	$a_{100\text{-}200}$	$a_{200\text{-}300}$	$a_{300\text{-}400}$	$a_{400\text{-}500}$	Mean	S.D.	15th %	85th %
Auxiliary Lane	Light	34	4.82	3.86	3.30	2.86	2.21	2.45	2.82	2.82	0.79	1.91	3.74
	Medium	91	3.82	3.22	3.07	2.62	1.91	2.09	2.41	2.46	0.66	1.80	3.23
	Heavy	49	2.06	1.92	2.00	1.94	1.75	1.96	2.17	1.96	0.45	1.57	2.27
Taper	Light	9	4.81	4.59	4.41	3.69	3.43	3.12	1.93	3.35	0.88	---	---
	Medium	24	3.48	3.60	3.57	2.85	2.97	2.45	1.49	2.66	0.74	1.95	3.19
	Heavy	22	2.27	2.06	2.05	1.82	1.51	1.87	1.17	1.85	0.34	1.33	2.20

Note: S.D. is the standard deviation of the mean acceleration rate of each group; 15th % and 85th % represent for the 15th percentile and 85th percentile acceleration rate of each group; --- means sample size is too limited to generate a percentile data.

The ITE Traffic Engineering Handbook *(49)* documented the maximum acceleration rate of tractor-semitrailer trucks with various weight-to-power ratios. Based on the similar reached speed (0 to 30 mph), the estimated heavy truck acceleration performance data were compared to the ITE values. In this study, the 85th percentile average acceleration rate of heavy trucks is 2.27 ft/s^2 at the auxiliary lane type ramp and 2.2 ft/s^2 at the taper type ramp. The ITE Traffic Engineering Handbook recommended the maximum acceleration rate for tractor-semitrailer trucks with 100, 200, 300, and 400 lb/hp weight-to-power ratios are 2.2, 1.5, 1.2, and 1.1 ft/s^2, respectively. It can be seen that field collected acceleration performance data are much higher than that documented in the ITE Traffic Engineering Handbook. However, the ITE values were based on a truck acceleration study performed in 1970, which is out-of-date for modern trucks.

5.4.3 Acceleration Lengths for Tractor-Trailer Trucks

Tractor-trailer trucks usually have lower acceleration capability and require greater lengths to accelerate to a desired merging speed than passenger cars. A total number of 49 tractor-trailer truck samples were collected from Industrial Pkwy to NB 880 entrance ramp. Figure 5-11 illustrates the 85th percentile distance versus speed regression model. Based on the generated regression model, minimum acceleration lengths for tractor-trailer trucks at various merging speeds were predicted. Comparisons between the predicted lengths, Deen's study *(14)*, Harwood's study*(15)*, and Gattis's study *(16)* are presented in Table 5-8.

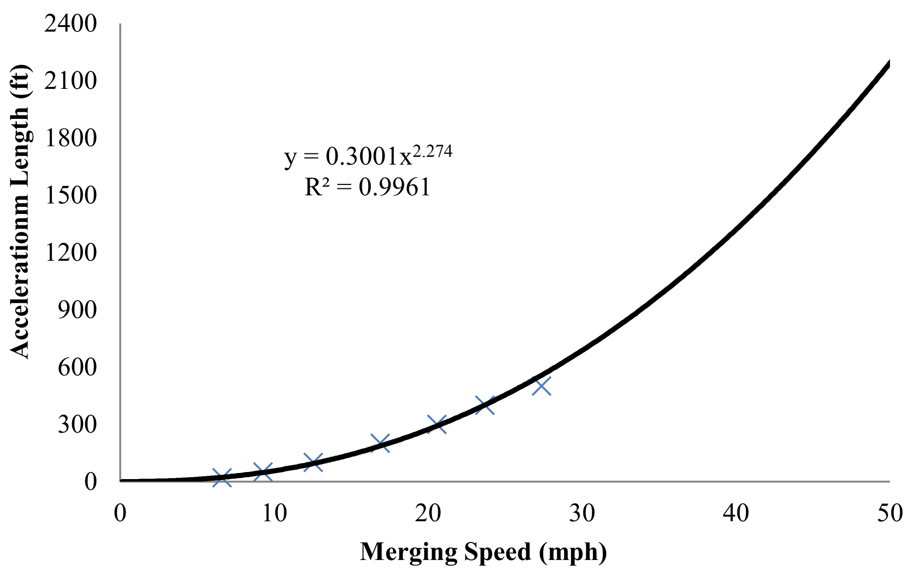


Figure 5-11 The 85th Percentile Speed Profile Model for Tractor-Trailer Truck Acceleration Length Prediction at Industrial Pkwy to NB88 Metered Ramp

Table 5-8 Comparison of Acceleration Lengths between Prediction and Previous Studies

		Merge Speed (mph)							
		30	35	39	40	45	50	55	60
Author	Initial Speed (mph)	Predicted Acceleration Length (ft.) to reach this speed							
Deen, 1957	22	---	1,240	---	1,820	---	---	---	---
Harwood, 2003	22	---	---	850	---	---	2,230	3,260	---
Gattis, 2008	17	---	---	---	1,203	---	2,219	2,731	3,655
This Study	0	685	975	1,245	1,320	1,725	2,190	2,720	3,315

Note:
Deen's study was for semi-trailer trucks;
Harwood's acceleration distances are for a 180 lb/hp truck on a 0% grade;
Gattis's acceleration performance data were collected for tractor-trailer trucks exit weight stations.
Predicted acceleration distances in this study are 85th percentile data for tractor-trailer trucks at metered on-ramp with 0% grade.

Results show that the AASHTO acceleration length design standard is insufficient for heavy trucks; in general, the predicted 85th percentile acceleration lengths for tractor-trailer trucks are approximately 1.6 times of the AASHTO standard. Also, it was found that modeling results are close to Gattis's acceleration length study for tractor-trailer trucks. Since the initial speed of Gattis's study is 17 mph, which is greater than this study, it further proved ramp metering does influence drivers' acceleration behavior.

The acceleration lengths recommended by Deen, Harwood, and Gattis are substantially longer than those proposed by this study. Deen's study was performed during the 1950s; the result seems to be out-of-date. Harwood's study was based on a 180 lb/hp truck, and Gattis's study was for tractor-trailer trucks (i.e., heavy truck) in real world condition. Acceleration lengths from these studies are longer than the estimated acceleration lengths for heavy trucks. However, it is necessary to point out that Gattis's results aimed to accommodate the 10th percentile vehicle drivers, while acceleration lengths presented in this study were based on the 50th percentile data. In this study, field collected data show that the 15th percentile acceleration rate is approximately 30 percent lower than the average acceleration capability.

5.4.4 Major Findings from Truck Acceleration Capability Study

Ramp metering has more significant influences to trucks than passenger cars since trucks usually have poorer acceleration capability, hence call for a longer acceleration distance to accelerate to the desired merge speed. In reality, the acceleration capability of different vehicle types varies and is usually influenced by prevailing traffic and road geometric features. Therefore, acceleration performance data should be based on large field data collected at ramp-metering sites with different geometric configurations. Major findings of this are listed as follows:

- Field data show that on average, light, medium and heavy truck drivers can accelerate from speed zero to approximately 37 mph, 34 mph and 31 mph in 500 feet, respectively. The average acceleration rates of light, medium, and heavy trucks at typical existing metered on-ramp are approximately 2.82 ft/s^2, 2.46ft/s^2, and 1.96 ft/s^2, respectively.
- The Green Book acceleration length design standard can only accommodate the 50th percentile of light truck drivers to accelerate to the desired speed. This study found that heavy truck needs approximately 3,315 feet to accelerate from stop condition to 60 mph. Therefore, for metered on-ramps where heavy truck demand is higher than 5 percent, a longer acceleration lane, or better, an auxiliary lane, should be provided to accommodate heavy truck drivers to accelerate to a safe merge speed.
- Acceleration lane length design should be based on the 15th percentile acceleration rate (i.e. 15 percent sample lower than this value) so as to accommodate the majority of vehicles. Statistical results show that the 15th percentile truck acceleration rate is approximately 30 percent lower than the average acceleration.

6. RECOMMENDATIONS FOR METERED ON-RAMP DESIGN

Based on the results of the developed models, recommendations concerning queue storage and acceleration lane length design were developed. The recommendations are based on selected performance criteria. For example, queue storage requirement was based on the 95th percentile queue length; the acceleration lane length was based on a 5 mph speed differential from the mainline design speed. The recommendations include a combination of charts and tables for estimating queue length and acceleration length at metered on-ramps.

6.1 Queue Storage Design Recommendations

The developed mesoscopic queue length simulation model is able to produce accurate queue length estimations given the detailed site-specific input information. Based on the large number of simulation runs, macroscopic level regression equations are developed to figure out what are the suitable percentage numbers to design queue storage length as percentage of on-ramp demand, under various demand-to-capacity ratio scenarios. In turn, summary tables and charts are generated for quick estimation during design stages.

Since the on-ramp flow arrival pattern of arterial on-ramps significantly differ from that of freeway-to-freeway connectors, the queue storage design standards for arterial on-ramps and freeway connectors are presented separately.

6.1.1 Arterial On-Ramp

The scatter plots of queue length as a percentage of demand under various demand-to-capacity ratios is presented for the pre-determined three ramp categories, as illustrated in Figure 6-1 through Figure 6-3. From the simulation results, it was found that an exponential function could best capture the queue length profile, and was finally recommended by this study.

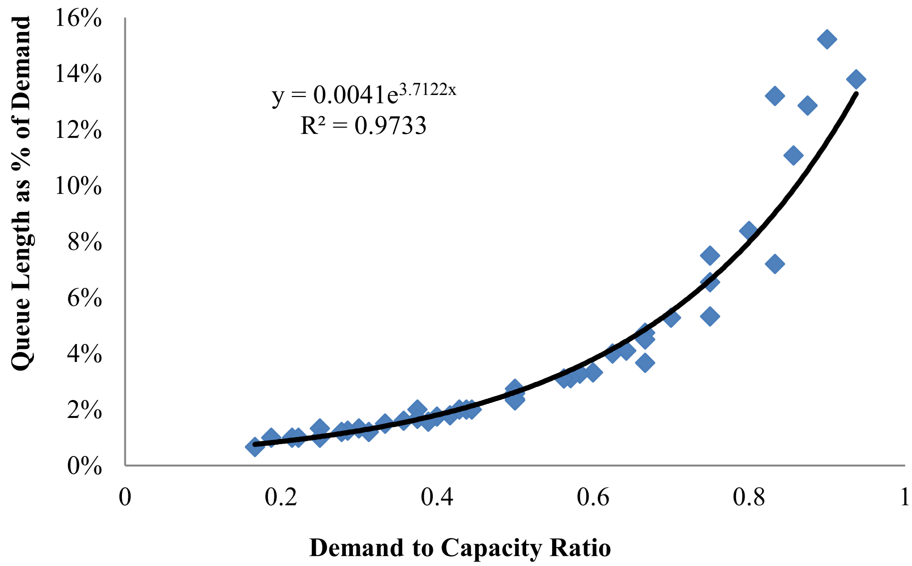


Figure 6-1 Queue Length as Percentage of Ramp Demand – Category 1 Type Ramp Configuration

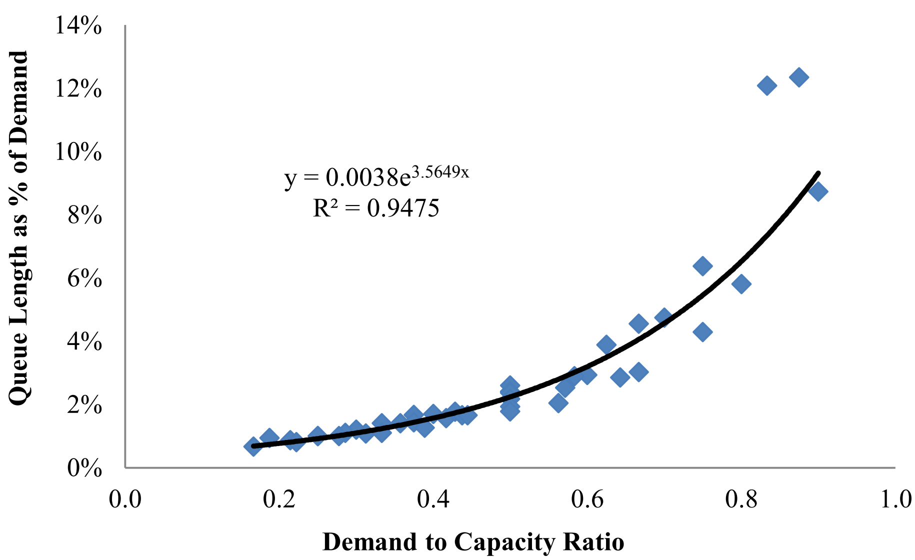


Figure 6-2 Queue Length as Percentage of Ramp Demand – Category 2 Type Ramp Configuration

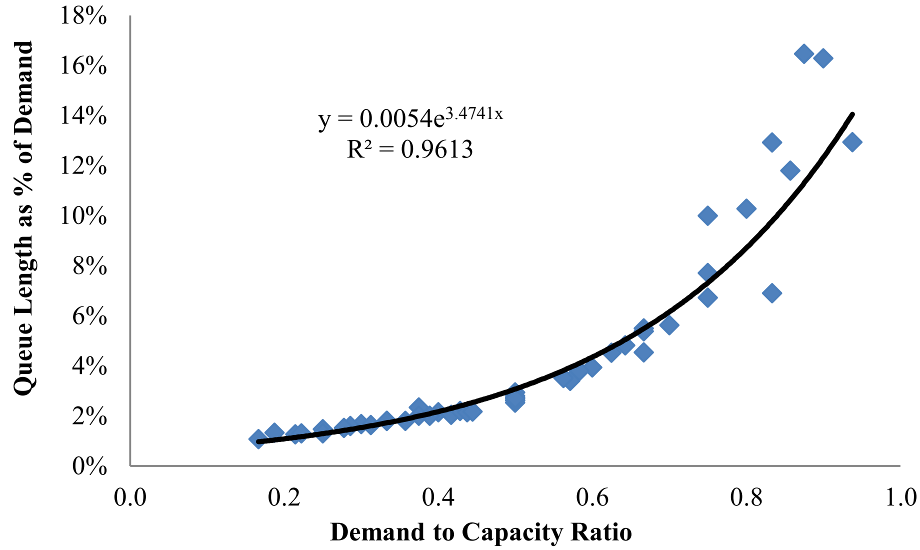


Figure 6-3 Queue Length as Percentage of Ramp Demand – Category 3 Type Ramp Configuration

Based on the generated regression equations, the percentage numbers were calculated for each demand-to-capacity scenario. The upper boundary of each scenario was recommended for metered arterial on-ramp queue storage length design, as listed in Table 6-1.

Table 6-1 Queue Length as Percentage of Ramp Demand Recommendations for Arterial Metered On-Ramps

Demand to Capacity Ratio	Queue Length as percentage of Ramp Demand		
	Category 1	Category 2	Category 3
<0.3	1.3%	1.1%	1.5%
0.4	1.8%	1.6%	2.2%
0.5	2.6%	2.3%	3.1%
0.6	3.8%	3.2%	4.3%
0.7	5.5%	4.6%	6.2%
0.8	8.0%	6.6%	8.7%
0.9	11.6%	9.4%	12.3%
1.0	16.8%	13.4%	17.4%

6.1.2 Freeway Connector

The scatter plots of queue length as a percentage of demand under various demand-to-capacity ratios is presented for the two metering rate conditions, as illustrated in Figure 6-4 and Figure 6-5. From the simulation results, it was found that for under-saturated scenarios, ramp queue was mainly caused by the random short-term surge of traffic arrival, and the exponential function could best capture the queue length profile; while for over-saturated scenarios, the simulated queue length tends to increase linearly with demand-to-capacity ratio.

Low metering rates conditions

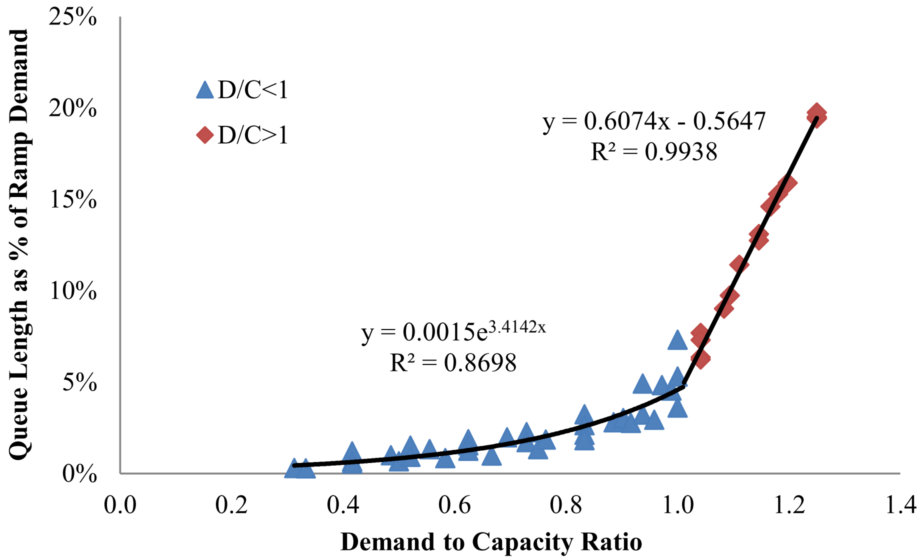


Figure 6-4 Queue Length as Percentage of Ramp Demand – Freeway Connector Low Metering Rate Conditions

High metering rates conditions

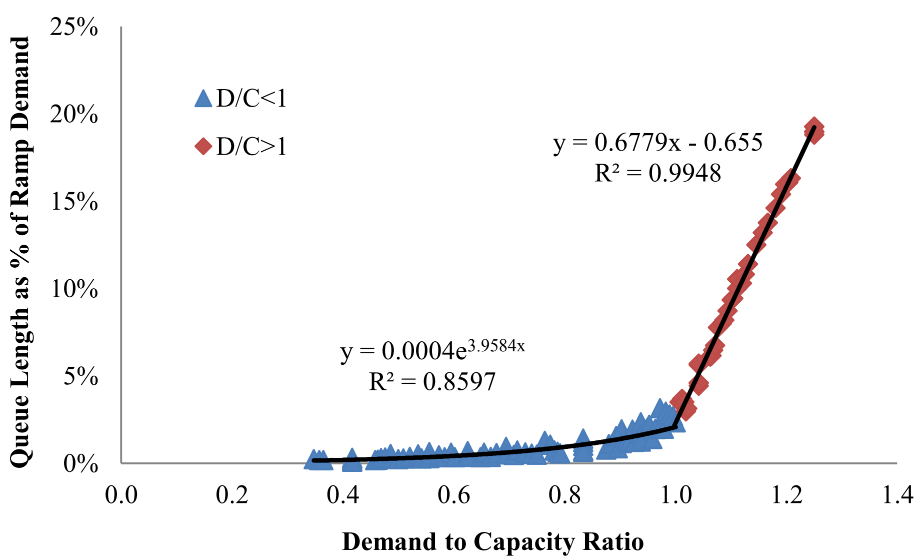


Figure 6-5 Queue Length as Percentage of Ramp Demand – Freeway Connector High Metering Rate Conditions

Based on the generated regression equations, the percentage numbers were calculated for each demand-to-capacity scenario. The upper boundary of each scenario was recommended for metered freeway-to-freeway connector queue storage length design, as listed in Table 6-2.

Table 6-2 Queue Length as Percentage of Ramp Demand Recommendations for Metered Freeway Connectors

Demand to Capacity Ratio	Queue Length as percentage of Ramp Demand	
	Low Metering Rates	High Metering Rates
<0.4	0.6%	0.2%
0.5	0.8%	0.3%
0.6	1.2%	0.4%
0.7	1.6%	0.6%
0.8	2.3%	0.9%
0.9	3.2%	1.4%
1.0	4.3%	2.3%

6.2 Acceleration Length Design Recommendations

Acceleration study results revealed that the existing acceleration length is the primary factor impacting drivers' acceleration behavior and consequently, the required acceleration lengths. Therefore, two design standards are recommended in this study: the aggressive design standard and the conservative design standard. Predicted acceleration lengths were categorized into two groups; group one: ramps with short existing acceleration length (mean 85th percentile predicted acceleration lengths for the four typical taper ramps), and group two: ramps with long existing acceleration length (mean 85th percentile predicted acceleration lengths for the two auxiliary lane ramps and the Rosecrans Ave metered on-ramp). The mean values could be used as the minimum acceleration lane design lengths for metered ramps, as listed in Table 6-3 and Table 6-4.

Table 6-3 Predicted Acceleration Lengths for Ramps with Short Existing Acceleration Length

Site	Merge Speed (mph)						
	30	35	40	45	50	55	60
	85th Percentile Predicted Acceleration Length (ft.)						
Mowry	240	360	505	680	890	1,140	1,420
Alvarado	240	365	520	715	950	1,220	1,540
Artesia	260	380	530	705	920	1165	1,445
Douglas	180	295	450	655	920	1,250	1,650
Average	230	350	501	689	920	1,194	1,514

Table 6-4 Predicted Acceleration Lengths for Ramps with Long Existing Acceleration Length

Site	Merge Speed (mph)						
	30	35	40	45	50	55	60
	85th Percentile Predicted Acceleration Length (ft.)						
Fruitridge	255	410	605	855	1,165	1,540	1,990
Industrial	310	480	700	975	1,310	1,720	2,195
Rosecrans	305	470	685	950	1285	1,680	2,145
Average	290	453	663	927	1,253	1,647	2,110

Note: Rosecrans Ave entrance ramp is taper merge type; however, due to it has very long existing speed change lane which has similar function as an auxiliary lane, here it was categorized into auxiliary lane type.

Predicted lengths from group one is recommended as the aggressive design standard, which would be used for metered ramps with restricted geometric conditions or insufficient queue storage space. For ramps that have adequate space, the conservative design standard, which is generated from group two, is recommended so that drivers are able to accelerate at a more comfortable manner. Minimum acceleration lengths for two design standards at metered on-ramps are listed in Table 6-5. Comparison between the predictions and the Green Book acceleration length design standard is illustrated in Figure 6-6.

Table 6-5 Minimum Acceleration Lengths for Two Design Standards at Metered Ramps with Flat Grade

	Merge speed (mph)						
	30	35	40	45	50	55	60
Design Standard	Acceleration length (ft.) to reach the speed						
Aggressive Design	230	350	500	690	920	1,195	1,515
Conservative Design	290	455	665	925	1,255	1,645	2,110

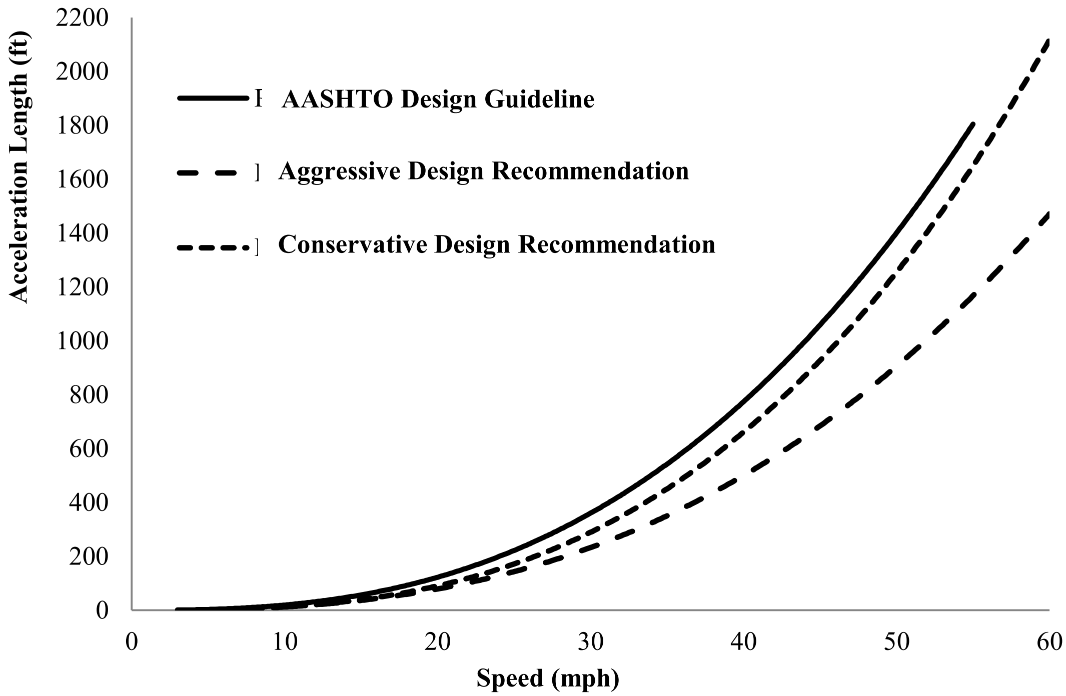


Figure 6-6 Comparison of the Two Design Recommendations with AASHTO Acceleration Length Design Guideline

Results indicated that the Green Book acceleration length design standard seems to be conservative for metered entrance ramps. It could be reduced by approximately 35 percent when using the aggressive design recommendation, or 10 percent when using the conservative design recommendation.

7. CONCLUDING REMARKS AND DISCUSSIONS

7.1 Concluding Remarks

7.1.1 Queue Storage Design

Field observations revealed that in real world conditions, the feeding traffic flow varies cycle by cycle; hence, simply using an average peak hour demand cannot provide accurate queue length estimation. From this study, it was found that the on-ramp queue length is dynamically related to the on-ramp demand, arrival flow arrival pattern, and metering rate. Additionally, on-ramp flow arrival pattern plays a critical role to queue length; an accurate description of real-time on-ramp flow arrival profile would help to capture the real-time queuing process and thus improve queue length modeling results.

This report proposed an improved approach for queue length modeling at metered arterial on-ramps; a key methodological contribution of this approach is that it can address the challenge of platoon onramp arrivals released from upstream-signalized intersections on queue generation, which was usually ignored by traditional analytical methods. Mesoscopic simulation models were developed for both arterial on-ramps and freeway-to-freeway connectors; queue lengths under various demand-to-capacity scenarios, including both the absolute queue length in number of vehicles and queue length as a percentage of on-ramp demand, was presented in forms of charts and tables.

Simulations results indicate that, for a metered arterial on-ramp, queue storage length as 8 percent of on-ramp demand could satisfy the majority of situations; the percentage number for a metered freeway connector is 4.3 percent for low metering rate scenarios (average metering rate lower than 1200 vph) and 2.3 percent for high metering rate scenarios (average metering rate greater than 1200 vph).

7.1.2 Acceleration Lane Length Design

This study revealed that the acceleration rate at metered entrance ramps is not constant; consequently kinematic equations cannot be directly used to calculate acceleration distance. Also, with consideration of the potential impacts of ramp metering on drivers' acceleration behavior, previous acceleration length design guidelines that were based on un-metered ramps may not be applicable for metered on-ramp acceleration length design. In this study, it was found that the distance versus speed regression method could better capture the realistic accelerating behavior and was recommended for acceleration lane length design.

The primary finding from the acceleration characteristic study is that existing acceleration length is the key factor affecting drivers' acceleration behavior and consequently, the required acceleration lengths. Accordingly, a dual-level acceleration lane length design standard was recommended to accommodate the unique operational features of a metered on-ramp. The conservative design is recommended for ramps that have sufficient space (both existing and proposed metered on-ramps); while the aggressive design recommendation could be used for existing metered on-ramps that have insufficient ramp space or recurrent ramp queue spillovers. The recommended acceleration lengths were compared to the AASHTO Green Book acceleration length design guidance; results indicated that the Green Book design guidance seems to be conservative for passenger vehicles at metered on-ramps. It was found that the aggressive design standard is approximately 35 percent shorter than the Green Book guideline, and the conservative design standard is approximately 10 percent shorter than the Green Book guideline. Nevertheless, the Green Book design guidance is insufficient for heavy trucks; the recommended acceleration lengths for tractor-trailer trucks are approximately 60 percent greater than the Green Book design guidance.

7.2 Discussions and Future Works

7.2.1 Queue Length Modeling

In real world conditions, it was found that field observed queue length may not represent the true queue length when queue overspills to the upstream intersection. Also, under this situation it is difficult to measure the true on-ramp demand. The simulation method can well address the queue overspill issue, since it was assumed to have sufficient queue storage space in the simulation model. Based on the simulation tool, it is possible to simulate various combinations of demand versus capacity scenarios to determine what is an adequate queue storage length for a given ramp-metering site. However the queue storage length recommendations rely on the accuracy of queue length modeling methodology and the performance simulation model. Though the proposed queue length modeling method can capture the real world condition to some extent, model validation results show that it still cannot exactly match the actual queue profile. Several ideas for future research to extend this research project are presented:

- Field observations showed that the actual ramp metering rate is affected by lane imbalance when multiple on-ramp lanes exist. Also, right-turn-on-red will influence the real-time onramp demand of each cycle. It is recommended that multiple factors should be jointly considered in modeling the real-time ramp queue length.
- Seek improved data collection methods that can measure the true demand and queue length.
- This study found that field observed metering rates did not coincide with the predetermined Caltrans rule of metering rate. A primary reason is that the actual metering rate is not only decided by mainline flow, but also the queue length. The potential queue flush or queue override strategies could affect the realistic departure rate and thus disrupt the corresponding between metering rate and freeway volume. Future studies need to further investigate the relationship between metering rate and freeway volume to mode a traffic responsive metering strategy.

7.2.2 Acceleration Data Collection and Processing

Vehicle acceleration performance data is considered to be the determining parameter in updating the acceleration lane length design guidance for metered on-ramps. In reality, the acceleration capability of different vehicle types varies and is usually influenced by prevailing traffic conditions and road geometric features. Therefore, acceleration lane length design should be based on large acceleration performance data collected at sites with different geometric configurations. However, measuring the actual acceleration characteristics from the field is sometimes a complex procedure, which is costly and time consuming. With consideration of the data requirements, the output data types, data verification, device installment, and cost for temporary data collection, the video based method was finally selected for this study. Also, a simple procedure for measuring traffic flow parameters in the real traffic conditions with a satisfactory level of accuracy was proposed. In addition, field investigations show metered vehicles may or may not came to a complete stop at the stop bar. When the metering rate is fast, vehicles are less likely to come to a complete stop. This indicates that initial speed at the stop bar location is not always equal to zero. One of the solutions for this problem is selecting samples that made a complete stop at the stop bar. On consideration of this, the video-based method, even though it calls for extensive labor, will still perform an irreplaceable role in the field data collection.

The regression analysis method could provide a more accurate description of realistic distance versus speed profiles; nevertheless, it is necessary to point out that a high goodness-of-fit value (R square) does not mean a regression model will perfectly match the realistic distance versus speed profile. Model validation results show a power function tends to underestimate acceleration lengths when the speed is close to the speed limit. In current practice, it is necessary to turn on the ramp meter when freeway-running speeds reach a certain threshold (e.g., lower than 50 mph in Los Angeles, California). Under such conditions, freeway-running speed is lower than the speed limit and so is the merge speed. It seems to be plausible that the predictions are not likely to be significantly influenced by the aforementioned error. However, metered on-ramp acceleration length design should also be based on free-flow conditions to accommodate the most challenging condition. Therefore, future studies should investigate a suitable correction of coefficients to eliminate the prediction error caused by the monotonous increasing nature of a power function.

Several areas are identified for further research:

- This research was limited to the first 500 feet downstream of the ramp meter stop bar. This is because usually there is limited right of way at existing ramps to place more cameras. To more accurately capture the realistic distance versus speed profiles, future studies should cover longer distances where the majority of vehicles have merged into the freeway mainline. This is especially critical for acceleration length design for heavy trucks, since the average speed of tractor-trailer trucks at 500 feet downstream the stop bar was observed to be around 30 mph, which is far from the freeway mainline speed. For future truck acceleration characteristics study, LiDAR speed guns may be used to collect spot speed measurements over longer distances.

- The design recommendations were developed based on ideal geometric (flat ramps with good sight distance) and traffic components (low truck percentage). Future studies should investigate the influence of grade on acceleration performance data to provide adjustment factors for the recommended acceleration lengths.
- The design recommendations did not take into account the safety impact of metered on-ramp design, such as the relationship between acceleration length and crash rate. Future studies could further evaluate the potential impacts of acceleration length on vehicular crash rate at the freeway merging area.
- For truck acceleration length design, it is recommended to further investigate what is the threshold (i.e., percentage of truck) for installation of auxiliary lanes at metered on-ramps. Currently, it is recommended by the Caltrans Highway Design Manual to install a minimum 500-foot length of auxiliary lane for a single-lane metered entrance ramp (a minimum 1,000-foot length of auxiliary lane for metered multilane ramps) when truck volume (three-axle or more) exceed 5 percent or greater on ascending entrance ramps with sustained upgrades exceeding 3 percent. Future works should also figure out what is the required auxiliary lane length for various geometric and traffic conditions.

REFERENCES

1. Papageorgiou, M., Kotsialos, A. (2002) Freeway Ramp Metering: An Overview. *IEEE Transactions on Intelligent Transportation Systems*, 3(4), 271-281.
2. Jacobson, L., J. Stribiak, L. Nelson, and D. Sallman. *Ramp Management and Control Handbook*. FHWA-HOP-06-001. FHWA, U.S. Department of Transportation, 2006.
3. *Twin Cities Ramp Meter Evaluation: Executive Summary*. Cambridge Systematics, Inc., February, 2001. Available from: http://ntl.bts.gov/lib/jpodocs/repts_te/13424.pdf
4. *I-580 Ramp Metering "Before" and "After" Studies, Final Report*. Kimley-Horn Associates Inc., August, 2008. Available from: http://www.mtc.ca.gov/services/arterial_operations/downloads/I580_Ramp_Meter.pdf.
5. Caltrans (2013). *Ramp Metering Development Plan*. California Department of Transportation, Sacramento. Available: http://www.dot.ca.gov/hq/traffops/trafmgmt/ramp_meter/RMDP.pdf
6. Gordon, R. L. (1996). Algorithm for Controlling Spillback from Ramp Meters. *Transportation Research Record*, No.1554, pp.162-171.
7. Spiliopoulou, A.D., Manolis, D., Papamichail, I., and Papageorgiou, M. (2010). Queue management techniques for metered freeway on-ramps. *Transportation Research Record*. No.2178, 40-48.
8. Wang, Z. Ramp Metering Status in California. *International Journal of Transportation Science*, 2013, Vol.2, No.4, pp.37-50.
9. Caltrans (2012). *California Highway Design Manual*. California Department of Transportation, Sacramento. Available: http://www.dot.ca.gov/hq/oppd/hdm/pdf/chp0500.pdf
10. Caltrans (2000). *Ramp Metering Design Manual*. California Department of Transportation, Sacramento. Available: http://www.dot.ca.gov/hq/traffops/systemops/ramp_meter/RMDM.pdf
11. Chow, P., A.M., Dunnet. *Evaluation of Ramp Control on the Santa Monica Freeway in Los Angeles*. Freeway Operations Branch, California Department of Transportation District 7, June 1978.
12. Wang, Z.R. Queue Storage Design for Metered On-Ramps. *International Journal of Transportation Science and Technology,* Vol.2, No.1, 2013, pp. 47-64.
13. AASHTO. *A Policy on Geometric Design of Highways and Streets*. American Association of State Highway and Transportation Officials, Washington, D.C., 2011.
14. Deen, T. B. Acceleration Lane Lengths for Heavy Commercial Vehicles. *Traffic Engineering*, Vol. 27, No. 5, 1957, pp. 212–217, 236.
15. Harwood, D. W., D. J. Torbic, K. R. Richard et al. *Review of Truck Characteristics as Factors in Highway Design*. NCHRP Report 505, Transportation Research Board of the National Academies, Washington, D.C., 2003.
16. Gattis, J., M. Bryant, and L. Duncan. *Acceleration Lane Design for Higher Truck Volumes*. Mack-Blackwell Transportation Center, Fayetteville, AR. 2008.
17. Burley, M. and Gaffney, J. Freeway Ramp Signals Handbook. VicRoads, Victoria, Australia, November, 2010. Available: http://www.vicroads.vic.gov.au/designstandards
18. ADOT. *Ramp Meter Design, Operations, and Maintenance Guidelines*. Arizona Department of

Transportation, August 2003. Available: http://www.azdot.gov/highways/ttg/pdf/rampmeter-designguide-0803.pdf

19. Texas Transportation Institute (TTI). Distance Requirements for Ramp Metering. 1994.

20. Chaudhary, N.A., and Messer, C. J. *Design Criteria for Ramp Metering: Appendix to TxDOT Roadway Design Manual*. Texas Transportation Institute Report 2121-3. Texas Transportation Institute, 2000.

21. Wu, J., X. Jin, A. J. Horawitz. Methodologies for Estimating Vehicle Queue Length at Metered On-Ramps. *Transportation Research Record: Journal of the Transportation Research Board,* No. 2047, Transportation Research Board of the National Academies, Washington, D.C., 2008, pp. 75-82.

22. Wu, J., X. Jin, A. J. Horawitz, and D. Gong. Experiment to Improve Estimation of Vehicle Queue Length at Metered On-Ramps. *Transportation Research Record: Journal of the Transportation Research Board,* No. 2099, Transportation Research Board of the National Academies, Washington, D.C., 2009, pp. 30-38.

23. Sun, X. and Horowitz, R. *A Set of New Traffic-Responsive Ramp-Metering Algorithms and Microscopic Simulation Results*. In the 85th Annual Meeting of the Transportation Research Board. CD-ROM. TRB, National Research Council, Washington, D.C., January 2006.

24. Vigos, G., M. Papageorgiou, and Q. Yibing. A Ramp Queue Length Estimation Algorithm. IEEE Intelligent Transportation Systems Proceedings, Toronto, Canada, Oct. 2006, pp. 418–425.

25. Vigos, G., M. Papageorgiou, and Y. Wang. Real-Time Estimation of Vehicle-Count within Signalized Links. *Transportation Research Part C,* Vol. 16, 2008, pp.18-35.

26. Sanchez R. O., R. Horawitz, and P. Varaiya. Analysis of Queue Estimation Methods Using Wireless Magnetic Sensors. *Transportation Research Record: Journal of the Transportation Research Board,* No. 2229, Transportation Research Board of the National Academies, Washington, D.C., 2011, pp. 34-45.

27. *Chapter 3. Freeway Corridor Traffic Management, Traffic Engineering Manual*. Minnesota Department of Transportation, August 2007.

28. Lau R. *Stratified Metering Algorithm-Internal Report*. Minnesota Department of Transportation, 2001.

29. Intelligent Transportation System (ITS) Design Manual. Wisconsin Department of Transportation, December 2000.

30. Chaudhary, N.A., C.J. Messer. *Ramp Metering Design and Operations Guidelines for Texas*. Texas Transportation Institute Report 2121-5. Texas Transportation Institute, January 2001.

31. Fitzpatrick, K., and K. Zimmerman. Potential Updates to 2004 Green Book's Acceleration Lengths for Entrance Terminals. *Transportation Research Record: Journal of the Transportation Research Board*, No. 2023, Transportation Research Board of the National Academies, Washington, D.C., 2007, pp. 130–139.

32. Bham, G. H., and R. F. Benekohal. *Acceleration Behavior of Drivers in a Platoon*. Proc., First International Driving Symposium on Human Factors in Driver Assessment, Training and Vehicle Design, University of Iowa Public Policy Center, Iowa City, 2001, pp. 280–285.

33. Long, G. Acceleration characteristics of starting vehicles. In *Transportation Research Record: Journal of the Transportation Research Board, No. 1737*, Transportation Research Board of the National Academic, Washington, D.C., 2000, pp. 58–70.

34. Akçelik, R., and D. C. Biggs. Acceleration Profile Models for Vehicles in Road Traffic. *Transportation Science*, Vol. 21, 1987, pp. 36–54.

35. Bokare, P. S., and A. K. Maurya. *Acceleration modeling of vehicles in developing countries*. Proceedings of the 2nd International Conference on Models and Technologies for Intelligent Transportation System, 22-24 June, 2011, Leuven, Belgium.

36. Wang, J., K. Dixon, H. Li, J. Ogle. Normal acceleration behavior of passenger vehicles starting from rest at all way stop controlled intersections. In *Transportation Research Record: Journal of the Transportation Research Board, No. 1883*, Transportation Research Board of the National Academic, Washington, D.C., 2004, pp. 158–166.

37. Oto, R. *Vehicle acceleration tests*. California Department of Transportation, Oakland, California, 1988.

38. Rakha, H., M. Snare, and F. Dion. Vehicle Dynamic Model for Estimating Maximum Light-Duty Vehicle Acceleration Levels. In *Transportation Research Record: Journal of the Transportation Research Board, No. 1883*, Transportation Research Board of the National Academic, Washington, D.C., 2004, pp. 40–49.

39. Rakha, H., I. Lucic, S. H. Demarchi, J. R. Setti, and M. Van Aerde. (2001). Vehicle Dynamics Model for Predicting Maximum Truck Acceleration Levels. *Journal of Transportation Engineering*, 127(5), 418–425.

40. Rakha, H., and I. Lucic. Variable Power Vehicle Dynamics Model for Estimating Maximum Truck Acceleration Levels. *Journal of Transportation Engineering*, Vol. 128, 2002, pp. 412–419.

41. AASHTO. *A Policy on Geometric Design of Rural Highways*. American Association of State Highway and Transportation Officials, Washington, D.C., 1965.

42. NDOT. *Managed Lanes and Ramp Metering Design Manual, Part 3: Design Manual*. Nevada Department of Transportation, March 2006. Available: http://www.nevadadot.com/uploadedFiles/NDOT/About_NDOT/NDOT_Divisions/Planning/Safety_Engineering/Design%20Manual.pdf

43. *Ramp Metering in Ohio, Planning, Design of New Construction and Retrofit*. Ohio Department of Transportation, 2004.

44. Newell, G. F. *Applications of Queuing Theory*, 2nd ed. Chapman and Hall, New York, 1982.

45. Lawson, T. W., D. J. Lovell, and C. F. Daganzo. Using the Input-Output Diagram to Determine the Spatial and Temporal Extents of a Queue Upstream of a Bottleneck. *Transportation Research Record: Journal of the Transportation Research Board,* No. 1572, Transportation Research Board of the National Academies, Washington, D.C., 1997, pp. 140-147.

46. Sharma, A., D. M. Bullock, and J. A. Bonneson. Input–Output and Hybrid Techniques for Real-Time Prediction of Delay and Maximum Queue Length at Signalized Intersections. *Transportation Research Record: Journal of the Transportation Research Board,* No. 2035, Transportation Research

Board of the National Academies, Washington, D.C., 2007, pp. 69-80.

47. Tian, Z. *Development and Evaluation of Operational Strategies for Providing an Integrated Diamond Interchange Ramp-Metering Control System*. PhD dissertation. Texas A&M University, College Station, May 2004.

48. Gattis, J. L., M. A. Bryant, and L. K. Duncan. Truck Acceleration Speeds and Distance at Weight Stations. *Transportation Research Record: Journal of the Transportation Research Board*, No. 2195, Transportation Research Board of the National Academies, Washington, DC, 2010, pp. 20-26.

49. Kraft, W. H., (ed.), *Traffic Engineering Handbook - 6th Edition*. Institute of Transportation Engineers, Washington, DC: ITE, 2009.

APPENDIX

Appendix A: Simulated Queue Lengths

A1: Arterial On-Ramps

Category 1

(Representative Metered On-Ramp Site: E St. to NB 99 Diamond On-ramp, Caltrans District 3)

Signal timing information used for simulation: C = **90**, G_{TH}=**45**; G_{RT}=**30**; G_{LT}=**15**

Saturate Flow Rate used for simulation: TH = **3600** vph; RT = **2300** vph; LT = **1600** vph; # of On-Ramp Lane = **2**; PHF = **0.9**

Upstream demand: TH: **50%**; LT: **40%**; RT:**10%**

Demand Scenario: **300** vph (EBT: 150vph; NBR:135vph; SBL:15vph)

Metering Rate (vphpl)	Simulated Queue Length (veh)										Average Queue Length (veh)		D/C	Q/D
	1st Run		2nd Run		3rd Run		4th Run		5th Run					
	95th %	Max	95th %	Max	95th %	Max	95th %	Max	95th %	Max	95th %	Max		
300	7	13	8	19	7	16	6	12	8	16	**7**	15	0.5	**2.3%**
400	6	14	5	14	6	16	6	14	7	17	**6**	15	0.375	**2.0%**
500	5	13	5	12	4	14	4	13	4	12	**4**	13	0.3	**1.3%**
600	4	13	4	14	4	13	3	12	4	13	**4**	13	0.25	**1.3%**
700	3	11	3	10	3	10	3	10	3	13	**3**	11	0.21	**1.0%**
800	3	13	3	10	3	13	3	11	3	14	**3**	12	0.19	**1.0%**
900	2	11	2	9	3	9	2	8	2	10	**2**	9	0.17	**0.7%**

Demand Scenario: **400** vph (EBT: 200vph; NBR:180vph; SBL:20vph)

Metering Rate (vphpl)	Simulated Queue Length (veh)										Average Queue Length (veh)		D/C	Q/D
	1st Run		2nd Run		3rd Run		4th Run		5th Run					
	95th %	Max	95th %	Max	95th %	Max	95th %	Max	95th %	Max	95th %	Max		
300	20	30	17	28	20	32	17	28	20	30	**19**	30	0.67	**4.8%**
400	12	20	11	24	14	26	11	22	8	18	**11**	22	0.5	**2.8%**
500	8	20	7	15	8	20	8	21	6	16	**7**	18	0.4	**1.8%**
600	6	18	6	19	6	18	6	17	6	17	**6**	18	0.33	**1.5%**
700	6	15	6	21	4	13	4	14	5	15	**5**	16	0.29	**1.3%**
800	5	17	4	16	5	15	4	16	4	15	**4**	16	0.25	**1.0%**
900	3	15	3	13	4	17	4	16	4	15	**4**	15	0.22	**1.0%**

Demand Scenario: **500** vph (EBT: 250vph; NBR:225vph; SBL:25vph)

Metering Rate (vphpl)	Simulated Queue Length (veh)										Average Queue Length (veh)		D/C	Q/D
	1st Run		2nd Run		3rd Run		4th Run		5th Run					
	95th %	Max	95th %	Max	95th %	Max	95th %	Max	95th %	Max	95th %	Max		
300	71	86	69	82	59	74	69	81	63	79	**66**	80	0.83	**13.2%**
400	19	32	22	32	17	29	22	40	21	34	**20**	33	0.63	**4.0%**
500	14	27	14	29	13	24	12	21	10	21	**13**	24	0.5	**2.6%**
600	9	20	9	19	10	19	9	24	10	23	**9**	21	0.42	**1.8%**
700	8	19	8	23	9	19	9	22	8	21	**8**	21	0.36	**1.6%**
800	6	19	8	20	6	17	5	16	7	19	**6**	18	0.31	**1.2%**
900	5	18	5	20	6	20	6	19	7	22	**6**	20	0.28	**1.2%**

Demand Scenario: **600** vph (EBT: 300vph; NBR:270vph; SBL:30vph)

Metering Rate (vphpl)	Simulated Queue Length (veh)										Average Queue Length (veh)		D/C	Q/D
	1st Run		2nd Run		3rd Run		4th Run		5th Run					
	95th %	Max	95th %	Max	95th %	Max	95th %	Max	95th %	Max	95th %	Max		
300	224	253	217	249	189	215	208	228	199	225	**203**	234	1	**33.8%**
400	48	58	50	66	42	54	45	61	39	54	**45**	59	0.75	**7.5%**
500	21	38	20	39	23	40	21	38	17	30	**20**	37	0.6	**3.3%**
600	13	25	13	25	13	25	15	23	14	31	**14**	26	0.5	**2.3%**
700	11	28	13	25	12	28	13	26	13	27	**12**	27	0.43	**2.0%**
800	8	22	11	24	10	22	9	20	11	23	**10**	22	0.375	**1.7%**
900	8	23	8	28	9	24	8	19	11	29	**9**	25	0.33	**1.5%**

Demand Scenario: **700** vph (EBT: 350vph; NBR:315vph; SBL:35vph)

Metering Rate (vphpl)	Simulated Queue Length (veh)										Average Queue Length (veh)		D/C	Q/D
	1st Run		2nd Run		3rd Run		4th Run		5th Run					
	95th %	Max	95th %	Max	95th %	Max	95th %	Max	95th %	Max	95th %	Max		
300	454	486	423	488	455	487	437	476	423	473	**438**	482	1.17	**62.6%**
400	80	101	79	101	113	136	91	109	88	113	**90**	112	0.88	**12.9%**
500	35	51	39	54	36	58	41	63	35	52	**37**	56	0.7	**5.3%**
600	22	38	22	39	23	39	22	35	26	37	**23**	38	0.58	**3.3%**
700	20	37	19	36	16	35	19	37	15	30	**18**	35	0.5	**2.6%**
800	14	30	16	34	15	34	14	27	13	26	**14**	30	0.44	**2.0%**
900	9	26	12	25	11	24	12	24	10	30	**11**	26	0.39	**1.6%**

Demand Scenario: 8**00** vph (EBT: 400vph; NBR:360vph; SBL:40vph)

Metering Rate (vphpl)	Simulated Queue Length (veh)										Average Queue Length (veh)		D/C	Q/D
	1st Run		2nd Run		3rd Run		4th Run		5th Run					
	95th %	Max	95th %	Max	95th %	Max	95th %	Max	95th %	Max	95th %	Max		
300	681	742	684	744	683	744	670	737	699	746	**683**	743	1.33	**85.4%**
400	231	255	231	258	255	286	242	260	241	277	**240**	267	1	**30.0%**
500	71	89	65	85	70	99	51	70	76	93	**67**	87	0.8	**8.4%**
600	34	53	42	66	43	61	36	58	26	39	**36**	55	0.67	**4.5%**
700	23	37	26	45	30	49	17	32	28	47	**25**	42	0.57	**3.1%**
800	21	45	19	42	16	38	19	44	21	43	**19**	42	0.5	**2.4%**
900	17	32	18	43	13	30	13	27	18	38	**16**	34	0.44	**2.0%**

Demand Scenario: **900** vph (EBT: 450vph; NBR:405vph; SBL:45vph)

Metering Rate (vphpl)	Simulated Queue Length (veh)										Average Queue Length (veh)		D/C	Q/D
	1st Run		2nd Run		3rd Run		4th Run		5th Run					
	95th %	Max	95th %	Max	95th %	Max	95th %	Max	95th %	Max	95th %	Max		
300	956	1015	942	1020	931	1014	952	1021	937	1026	**944**	1019	1.5	**104.9%**
400	487	537	484	516	475	517	504	542	461	530	**482**	528	1.1	**53.6%**
500	153	186	172	217	123	151	118	143	118	144	**137**	168	0.9	**15.2%**
600	59	83	64	85	54	74	57	73	62	82	**59**	79	0.75	**6.6%**
700	48	76	33	56	39	62	42	66	24	41	**37**	60	0.64	**4.1%**
800	29	47	30	59	33	58	29	47	20	39	**28**	50	0.56	**3.1%**
900	22	37	20	41	20	40	20	39	22	38	**21**	39	0.5	**2.3%**

Demand Scenario: **1200** vph (EBT: 600vph; NBR:540vph; SBL:60vph)

Metering Rate (vphpl)	**Simulated Queue Length (veh)**										**Average Queue Length (veh)**		**D/C**	**Q/D**
	1st Run		**2nd Run**		**3rd Run**		**4th Run**		**5th Run**					
	95th %	**Max**	**95th %**	**Max**	**95th %**	**Max**	**95th %**	**Max**	**95th %**	**Max**	**95th %**	**Max**		
300	1670	1767	1644	1773	1626	1764	1692	1785	1678	1791	**1662**	1776	2	**138.5%**
400	1210	1287	1168	1273	1182	1278	1174	1260	1153	1283	**1177**	1276	1.5	**98.1%**
500	719	796	736	801	750	816	722	819	724	798	**730**	806	1.2	**60.8%**
600	340	376	301	341	357	384	330	366	354	396	**336**	373	1	**28.0%**
700	133	163	150	173	102	131	150	176	132	156	**133**	160	0.86	**11.1%**
800	62	86	65	88	61	90	59	89	74	99	**64**	90	0.75	**5.3%**
900	44	67	43	62	55	83	38	59	40	62	**44**	67	0.67	**3.7%**

Demand Scenario: **1500** vph (EBT: 750vph; NBR:675vph; SBL:75vph)

Metering Rate (vphpl)	**Simulated Queue Length (veh)**										**Average Queue Length (veh)**		**D/C**	**Q/D**
	1st Run		**2nd Run**		**3rd Run**		**4th Run**		**5th Run**					
	95th %	**Max**	**95th %**	**Max**	**95th %**	**Max**	**95th %**	**Max**	**95th %**	**Max**	**95th %**	**Max**		
300														
400														
500	1317	1434	1392	1481	1365	1453					**1358**	1456	1.5	**90.5%**
600	910	1006	944	1016	873	968	917	969	919	1002	**913**	992	1.25	**60.9%**
700	456	503	452	495	483	527	485	531	477	570	**471**	525	1.1	**31.4%**
800	202	236	242	290	222	248	162	188	206	248	**207**	242	0.94	**13.8%**
900	122	157	133	158	102	137	70	91	115	137	**108**	136	0.83	**7.2%**

Category 2

(Representative Metered On-Ramp Site: Woodman Ave. to NB 101 Diamond On-ramp, Caltrans District 7)

Signal timing information used for simulation: C = **90**, G_{TH}=**35**; G_{RT}=**30**; G_{LT}=**25**

Saturate Flow Rate used for simulation: RT = **1800** vph; LT = **1800** vph; # of On-Ramp Lane = **2**; PHF = **0.9**

Upstream demand: LT: **40%**; RT:**60%**

Demand Scenario: **300** vph (L: 120vph; R:180vph)

Metering Rate (vphpl)	Simulated Queue Length (veh)										Average Queue Length (veh)		D/C	Q/D
	1st Run		2nd Run		3rd Run		4th Run		5th Run					
	95th %	Max	95th %	Max	95th %	Max	95th %	Max	95th %	Max	95th %	Max		
600	7	15	7	15	8	18	8	16	6	12	**7**	15	0.50	**2.4%**
800	5	14	4	13	5	15	5	14	6	16	**5**	14	0.38	**1.7%**
1000	4	12	3	13	4	12	3	13	4	11	**4**	12	0.30	**1.2%**
1200	3	13	3	13	3	12	3	10	3	11	**3**	12	0.25	**1.0%**
1400	2	8	3	10	2	10	3	10	3	11	**3**	10	0.21	**0.9%**
1600	2	11	3	9	3	11	3	11	3	11	**3**	11	0.19	**0.9%**
1800	2	11	2	9	2	10	2	8	2	10	**2**	10	0.17	**0.7%**

Demand Scenario: **400** vph (L: 160vph; R:240vph)

Metering Rate (vphpl)	Simulated Queue Length (veh)										Average Queue Length (veh)		D/C	Q/D
	1st Run		2nd Run		3rd Run		4th Run		5th Run					
	95th %	Max	95th %	Max	95th %	Max	95th %	Max	95th %	Max	95th %	Max		
600	19	30	17	28	19	31	16	28	20	30	**18**	29	0.67	**4.6%**
800	11	21	11	23	12	25	10	25	8	18	**10**	22	0.50	**2.6%**
1000	8	20	6	16	7	18	8	18	5	16	**7**	18	0.40	**1.7%**
1200	6	18	5	18	7	22	5	18	5	20	**6**	19	0.33	**1.4%**
1400	4	14	5	15	5	20	4	12	4	13	**4**	15	0.29	**1.1%**
1600	4	16	4	15	4	14	4	14	4	15	**4**	15	0.25	**1.0%**
1800	3	12	3	14	3	14	4	16	3	15	**3**	14	0.22	**0.8%**

Demand Scenario: **500** vph (L: 200vph; R:300vph)

Metering Rate (vphpl)	Simulated Queue Length (veh)										Average Queue Length (veh)		D/C	Q/D
	1st Run		2nd Run		3rd Run		4th Run		5th Run					
	95th %	Max	95th %	Max	95th %	Max	95th %	Max	95th %	Max	95th %	Max		
600	52	68	65	80	63	76	58	73	64	77	**60**	75	0.83	**12.1%**
800	20	31	19	32	21	34	16	30	21	38	**19**	33	0.63	**3.9%**
1000	12	26	13	28	11	26	13	24	11	21	**12**	25	0.50	**2.4%**
1200	7	19	8	18	8	21	8	18	8	22	**8**	20	0.42	**1.6%**
1400	7	22	7	19	7	21	7	19	7	21	**7**	20	0.36	**1.4%**
1600	6	19	5	15	6	18	5	15	5	14	**5**	16	0.31	**1.1%**
1800	5	16	5	15	5	19	5	19	5	15	**5**	17	0.28	**1.0%**

Demand Scenario: **600** vph (L: 240vph; WBR:360vph)

Metering Rate (vphpl)	Simulated Queue Length (veh)										Average Queue Length (veh)		D/C	Q/D
	1st Run		2nd Run		3rd Run		4th Run		5th Run					
	95th %	Max	95th %	Max	95th %	Max	95th %	Max	95th %	Max	95th %	Max		
600	197	213	175	194	175	196	197	222	182	196	**185**	204	1.00	**30.9%**
800	42	55	37	53	35	52	31	48	46	58	**38**	53	0.75	**6.4%**
1000	20	34	16	27	19	32	16	30	17	27	**18**	30	0.60	**2.9%**
1200	15	34	13	29	15	31	14	29	14	26	**14**	30	0.50	**2.4%**
1400	13	28	12	28	8	26	11	27	9	23	**11**	26	0.43	**1.8%**
1600	9	23	7	23	8	23	9	25	10	29	**9**	25	0.38	**1.4%**
1800	6	21	6	22	8	21	5	21	8	25	**7**	22	0.33	**1.1%**

Demand Scenario: **700** vph (L: 280vph; R:420vph)

Metering Rate (vphpl)	Simulated Queue Length (veh)										Average Queue Length (veh)		D/C	Q/D
	1st Run		2nd Run		3rd Run		4th Run		5th Run					
	95th %	Max	95th %	Max	95th %	Max	95th %	Max	95th %	Max	95th %	Max		
600	388	432	388	432	408	440	390	417			**394**	430	1.17	**56.2%**
800	66	81	90	107	85	98	84	96	107	119	**86**	100	0.88	**12.3%**
1000	23	37	41	59	30	50	39	55	33	48	**33**	50	0.70	**4.7%**
1200	15	28	23	37	23	38	21	38	19	33	**20**	35	0.58	**2.9%**
1400	15	37	17	38	15	30	16	33	13	27	**15**	33	0.50	**2.2%**
1600	9	23	12	27	13	28	11	31	13	30	**12**	28	0.44	**1.7%**
1800	9	25	9	26	9	25	7	25	10	28	**9**	26	0.39	**1.3%**

Demand Scenario: **800** vph (EBL: 320vph; WBR:480vph)

Metering Rate (vphpl)	Simulated Queue Length (veh)										Average Queue Length (veh)		D/C	Q/D
	1st Run		2nd Run		3rd Run		4th Run		5th Run					
	95th %	Max	95th %	Max	95th %	Max	95th %	Max	95th %	Max	95th %	Max		
600	578	621	621	667	589	625					**596**	638	1.33	**74.5%**
800	193	212	167	188	175	200	198	227	166	184	**180**	202	1.00	**22.5%**
1000	48	64	48	63	44	67	42	61	50	64	**46**	64	0.80	**5.8%**
1200	26	43	22	35	25	38	25	40	23	33	**24**	38	0.67	**3.0%**
1400	21	39	19	33	21	40	21	42	19	22	**20**	35	0.57	**2.5%**
1600	18	30	17	39	12	30	18	36	12	28	**15**	33	0.50	**1.9%**
1800	12	27	12	26	12	28	14	32	16	31	**13**	29	0.44	**1.7%**

Demand Scenario: **900** vph (EBL: 360vph; WBR:540vph)

Metering Rate (vphpl)	Simulated Queue Length (veh)										Average Queue Length (veh)		D/C	Q/D
	1st Run		2nd Run		3rd Run		4th Run		5th Run					
	95th %	Max	95th %	Max	95th %	Max	95th %	Max	95th %	Max	95th %	Max		
600	776	839	790	844							**783**	842	1.50	**87.0%**
800	335	375	334	371	328	356					**332**	367	1.13	**36.9%**
1000	97	112	85	101	62	78	69	84	80	98	**79**	95	0.90	**8.7%**
1200	47	69	38	54	27	44	40	63	41	55	**39**	57	0.75	**4.3%**
1400	26	49	24	35	27	46	28	53	23	43	**26**	45	0.64	**2.8%**
1600	19	37	18	36	16	29	19	37	20	37	**18**	35	0.56	**2.0%**
1800	16	40	19	39	15	32	15	36	15	30	**16**	35	0.50	**1.8%**

Demand Scenario: **1200** vph (EBL: 480vph; WBR:720vph)

Metering Rate (vphpl)	Simulated Queue Length (veh)										Average Queue Length (veh)		D/C	Q/D
	1st Run		2nd Run		3rd Run		4th Run		5th Run					
	95th %	Max	95th %	Max	95th %	Max	95th %	Max	95th %	Max	95th %	Max		
600														
800														
1000	249	285	271	300	279	297					**266**	294	1.20	**22.2%**
1200	61	84	70	88	59	75	61	78	70	89	**64**	83	1.00	**5.4%**
1400	42	59	31	45	37	58	33	48	39	61	**36**	54	0.86	**3.0%**
1600	28	48	30	49	29	52	29	49	22	39	**28**	47	0.75	**2.3%**
1800	18	41	21	40	20	35	21	36	19	37	**20**	38	0.67	**1.7%**

Demand Scenario: **1500** vph (EBL: 600vph; WBR:900vph)

Metering Rate (vphpl)	Simulated Queue Length (veh)										Average Queue Length (veh)		D/C	Q/D
	1st Run		2nd Run		3rd Run		4th Run		5th Run					
	95th %	Max	95th %	Max	95th %	Max	95th %	Max	95th %	Max	95th %	Max		
600														
800														
1000														
1200	85	105	87	111	83	104								
1400	49	75	48	66	46	68	50	77	44	64				
1600	29	49	28	44	32	44	28	43	26	46	**29**	45	0.94	**1.9%**
1800	25	41	24	45	23	39	22	39	24	43	**24**	41	0.83	**1.6%**

Category 3

(Representative Metered On-Ramp Site: SB Bradshaw Rd. to WB 50 Slip On-ramp, Caltrans District 3)

Signal timing information used for simulation: C = **120**, G_{TH}=**48**; G_{UT}=**24**; G_{RT}=**24**; G_{LT}=**24**

Saturate Flow Rate used for simulation: TH = **3600** vph; UT = **1500** vph; RT = **1800** vph; LT = **1800** vph; # of On-Ramp Lane = **2**; PHF = **0.9**

Upstream demand: TH: **60%**; U-Turn: **3%**; LT: **17%**; RT:**20%**

Demand Scenario: **300** vph (T: 180vph; U: 9vph; L: 51vph; R:60vph)

Metering Rate (vphpl)	Simulated Queue Length (veh)										Average Queue Length (veh)		D/C	Q/D
	1st Run		2nd Run		3rd Run		4th Run		5th Run					
	95th %	Max	95th %	Max	95th %	Max	95th %	Max	95th %	Max	95th %	Max		
600	9	17	9	19	8	15	9	19	9	19	**9**	18	0.50	**2.9%**
800	6	14	7	16	6	15	8	16	8	16	**7**	15	0.38	**2.3%**
1000	5	13	5	15	5	13	5	15	5	14	**5**	14	0.30	**1.7%**
1200	5	14	4	13	5	15	4	12	4	11	**4**	13	0.25	**1.5%**
1400	4	10	4	12	4	10	4	12	3	14	**4**	12	0.21	**1.3%**
1600	4	14	4	12	4	13	4	12	4	14	**4**	13	0.19	**1.3%**
1800	4	11	3	10	3	10	3	10	3	14	**3**	11	0.17	**1.1%**

Demand Scenario: **400** vph (T: 240vph; U: 12vph; L: 68vph; R:80vph)

Metering Rate (vphpl)	Simulated Queue Length (veh)										Average Queue Length (veh)		D/C	Q/D
	1st Run		2nd Run		3rd Run		4th Run		5th Run					
	95th %	Max	95th %	Max	95th %	Max	95th %	Max	95th %	Max	95th %	Max		
600	20	29	24	36	28	40	23	37	15	23	**22**	33	0.67	**5.5%**
800	12	22	11	23	10	21	11	20	12	23	**11**	22	0.50	**2.8%**
1000	9	12	8	16	10	21	9	20	7	15	**9**	17	0.40	**2.2%**
1200	7	14	8	20	8	20	7	20	6	18	**7**	18	0.33	**1.8%**
1400	6	14	8	21	5	14	6	14	7	18	**6**	16	0.29	**1.6%**
1600	5	18	5	15	6	18	5	17	5	15	**5**	17	0.25	**1.3%**
1800	5	17	5	15	6	16	5	14	5	16	**5**	16	0.22	**1.3%**

Demand Scenario: **500** vph (T: 300vph; U: 15vph; L: 85vph; R:100vph)

Metering Rate (vphpl)	Simulated Queue Length (veh)										Average Queue Length (veh)		D/C	Q/D
	1st Run		2nd Run		3rd Run		4th Run		5th Run					
	95th %	Max	95th %	Max	95th %	Max	95th %	Max	95th %	Max	95th %	Max		
600	62	78	47	63	74	92	70	82	70	82	**65**	79	0.83	**12.9%**
800	27	40	21	32	19	31	22	36	24	38	**23**	35	0.63	**4.5%**
1000	13	28	14	28	16	27	12	24	13	22	**14**	26	0.50	**2.7%**
1200	9	22	11	21	11	27	9	24	11	26	**10**	24	0.42	**2.0%**
1400	10	23	10	21	9	22	8	20	8	17	**9**	21	0.36	**1.8%**
1600	8	21	8	21	8	22	9	27	8	21	**8**	22	0.31	**1.6%**
1800	7	24	7	20	8	24	8	21	8	21	**8**	22	0.28	**1.5%**

Demand Scenario: **600** vph (T: 360vph; U: 18vph; L: 102vph; R:120vph)

Metering Rate (vphpl)	Simulated Queue Length (veh)										Average Queue Length (veh)		D/C	Q/D
	1st Run		2nd Run		3rd Run		4th Run		5th Run					
	95th %	Max	95th %	Max	95th %	Max	95th %	Max	95th %	Max	95th %	Max		
600	196	224	208	230	184	205	217	240	203	217	**202**	223	1.00	**33.6%**
800	60	79	56	71	57	69	66	86	61	76	**60**	76	0.75	**10.0%**
1000	20	36	22	45	26	43	27	43	23	39	**24**	41	0.60	**3.9%**
1200	14	26	13	26	17	32	17	31	15	30	**15**	29	0.50	**2.5%**
1400	14	28	13	28	14	33	12	28	13	26	**13**	29	0.43	**2.2%**
1600	11	27	14	31	11	24	12	31	12	25	**12**	28	0.38	**2.0%**
1800	12	31	13	29	10	25	9	27	10	27	**11**	28	0.33	**1.8%**

Demand Scenario: **700** vph (T: 420vph; U: 21vph; L: 119vph; R:140vph)

Metering Rate (vphpl)	Simulated Queue Length (veh)										Average Queue Length (veh)		D/C	Q/D
	1st Run		2nd Run		3rd Run		4th Run		5th Run					
	95th %	Max	95th %	Max	95th %	Max	95th %	Max	95th %	Max	95th %	Max		
600														
800	117	139	121	134	118	144	97	115	123	144	**115**	135	0.88	**16.5%**
1000	35	53	46	61	42	60	41	60	33	51	**39**	57	0.70	**5.6%**
1200	26	47	28	44	21	37	25	41	33	51	**27**	44	0.58	**3.8%**
1400	20	34	19	37	16	33	18	34	19	41	**18**	36	0.50	**2.6%**
1600	17	32	15	27	14	31	16	32	13	29	**15**	30	0.44	**2.1%**
1800	12	28	15	31	13	25	15	34	15	31	**14**	30	0.39	**2.0%**

Demand Scenario: **800** vph (T: 480vph; U: 24vph; L: 136vph; R:160vph)

Metering Rate (vphpl)	**Simulated Queue Length (veh)**										**Average Queue Length (veh)**		**D/C**	**Q/D**
	1st Run		**2nd Run**		**3rd Run**		**4th Run**		**5th Run**					
	95th %	**Max**	**95th %**	**Max**	**95th %**	**Max**	**95th %**	**Max**	**95th %**	**Max**	**95th %**	**Max**		
600														
800	285	307	234	266	235	264	236	256	227	270	**243**	273	1.00	**30.4%**
1000	103	140	71	89	88	108	73	93	76	97	**82**	105	0.80	**10.3%**
1200	42	65	40	57	43	62	43	59	47	67	**43**	62	0.67	**5.4%**
1400	24	44	25	53	29	49	30	55	28	49	**27**	50	0.57	**3.4%**
1600	21	45	21	38	20	38	21	48	22	44	**21**	43	0.50	**2.6%**
1800	19	41	16	35	15	27	19	39	18	36	**17**	36	0.44	**2.2%**

Demand Scenario: **900** vph (T: 540vph; U: 27vph; L: 153vph; R:180vph)

Metering Rate (vphpl)	**Simulated Queue Length (veh)**										**Average Queue Length (veh)**		**D/C**	**Q/D**
	1st Run		**2nd Run**		**3rd Run**		**4th Run**		**5th Run**					
	95th %	**Max**	**95th %**	**Max**	**95th %**	**Max**	**95th %**	**Max**	**95th %**	**Max**	**95th %**	**Max**		
600														
800														
1000	137	159	141	162	172	197	140	165	143	172	**147**	171	0.90	**16.3%**
1200	66	96	52	72	83	104	61	81	85	109	**69**	92	0.75	**7.7%**
1400	57	82	46	72	30	46	44	64	40	60	**43**	65	0.64	**4.8%**
1600	30	50	37	62	21	38	37	62	33	56	**32**	54	0.56	**3.5%**
1800	29	54	22	45	22	49	26	52	23	43	**24**	49	0.50	**2.7%**

Demand Scenario: **1200** vph (T: 720vph; U: 36vph; L: 204vph; R:240vph)

Metering Rate (vphpl)	Simulated Queue Length (veh)										Average Queue Length (veh)		D/C	Q/D
	1st Run		2nd Run		3rd Run		4th Run		5th Run					
	95th %	Max	95th %	Max	95th %	Max	95th %	Max	95th %	Max	95th %	Max		
600														
800														
1000														
1200	276	310	324	372	299	339	332	358	297	331	**306**	342	1.00	**25.5%**
1400	124	150	154	182	146	167	143	172	141	178	**142**	170	0.86	**11.8%**
1600	72	99	77	102	92	116	82	101	80	109	**81**	105	0.75	**6.7%**
1800	51	76	65	88	43	64	59	90	54	81	**54**	80	0.67	**4.5%**

Demand Scenario: **1500** vph (T: 900vph; U: 45vph; L: 255vph; R:300vph)

Metering Rate (vphpl)	Simulated Queue Length (veh)										Average Queue Length (veh)		D/C	Q/D
	1st Run		2nd Run		3rd Run		4th Run		5th Run					
	95th %	Max	95th %	Max	95th %	Max	95th %	Max	95th %	Max	95th %	Max		
600														
800														
1000														
1200														
1400														
1600	207	240	199	234	207	246	209	248	148	184	**194**	230	0.94	**12.9%**
1800	114	153	101	142	99	124	104	130	100	129	**104**	136	0.83	**6.9%**

A2: Freeway-to-Freeway Connector

Average Metering Rate: **480** vph

Demand (vph)	Queue Scenario	Simulated Queue Length (veh)										Mean	S.D.	D/C	Q/D
		1st Run	2nd Run	3rd Run	4th Run	5th Run	6th Run	7th Run	8th Run	9th Run	10th Run				
200	Max	3	5	9	6	9	4	6	4	5	5	6	2.0	0.42	2.8%
	95th %	2	2	4	3	3	2	2	2	2	2	2	0.7	0.42	1.2%
250	Max	6	7	9	6	7	8	8	6	10	7	7	1.3	0.52	3.0%
	95th %	4	3	6	4	5	3	2	3	4	4	4	1.1	0.52	1.5%
300	Max	11	11	8	8	13	8	13	11	8	10	10	2.0	0.63	3.4%
	95th %	5	7	4	5	7	4	7	7	4	6	6	1.3	0.63	1.9%
350	Max	14	13	16	10	11	11	12	13	11	11	12	1.8	0.73	3.5%
	95th %	8	7	7	7	7	7	11	10	8	7	8	1.4	0.73	2.3%
400	Max	21	20	14	11	20	16	16	22	21	19	18	3.6	0.83	4.5%
	95th %	16	16	9	8	12	9	11	20	16	12	13	3.9	0.83	3.2%
450	Max	25	31	28	31	30	24	22	16	34	25	27	5.3	0.94	5.9%
	95th %	21	28	21	27	26	20	17	13	27	21	22	4.9	0.94	4.9%
480	Max	39	31	42	51	38	45	52	36	28	40	40	7.7	1.00	8.4%
	95th %	33	27	37	43	32	38	50	33	24	34	35	7.5	1.00	7.3%
500	Max	32	46	53	41	40	49	38	49	47	44	44	6.2	1.04	8.8%
	95th %	29	39	48	35	35	44	31	41	43	39	38	5.9	1.04	7.7%
550	Max	82	73	72	87	80	73	72	83	77	78	78	5.3	1.15	14.1%
	95th %	79	65	64	79	77	67	66	78	71	74	72	6.1	1.15	13.1%
600	Max	125	129	123	120	123	131	135	129	123	126	126	4.6	1.25	21.1%
	95th %	118	124	114	107	117	116	129	108	115	121	117	6.7	1.25	19.5%

Average Metering Rate: **720** vph

Demand (vph)	Queue Scenario	Simulated Queue Length (veh)										Mean	S.D.	D/C	Q/D
		1st Run	2nd Run	3rd Run	4th Run	5th Run	6th Run	7th Run	8th Run	9th Run	10th Run				
300	Max	7	6	6	6	6	9	7	5	9	5	7	1.4	0.42	2.2%
	95th %	2	2	2	2	1	3	2	2	2	2	2	0.5	0.42	0.7%
350	Max	7	6	7	7	6	7	10	8	8	9	8	1.3	0.49	2.1%
	95th %	3	3	4	4	3	2	4	5	3	4	4	0.8	0.49	1.0%
400	Max	10	9	7	9	10	14	9	12	7	9	10	2.1	0.56	2.4%
	95th %	6	5	5	4	6	8	4	6	4	5	5	1.3	0.56	1.3%
450	Max	11	13	17	11	12	9	14	11	9	9	12	2.5	0.63	2.6%
	95th %	6	7	10	7	6	5	11	6	6	5	7	2.0	0.63	1.5%
500	Max	9	19	12	22	23	15	16	17	12	11	16	4.7	0.69	3.1%
	95th %	6	12	7	17	12	9	8	11	8	9	10	3.2	0.69	2.0%
550	Max	21	16	12	15	11	18	8	17	14	19	15	4.0	0.76	2.7%
	95th %	13	9	8	10	8	13	6	12	10	13	10	2.5	0.76	1.9%
600	Max	27	15	19	30	25	21	16	15	26	20	21	5.4	0.83	3.6%
	95th %	17	12	16	23	20	13	12	11	18	16	16	3.9	0.83	2.6%
650	Max	27	19	38	29	36	26	16	22	24	26	26	6.8	0.90	4.0%
	95th %	21	15	32	23	27	13	12	15	20	19	20	6.4	0.90	3.0%
700	Max	55	44	34	45	33	41	37	28	44	42	40	7.6	0.97	5.8%
	95th %	48	36	29	38	28	35	29	23	39	33	34	7.1	0.97	4.8%
720	Max	43	34	71	30	52	44	41	53	51	36	46	11.9	1.00	6.3%
	95th %	37	29	62	26	39	41	35	43	42	28	38	10.3	1.00	5.3%
750	Max	63	49	55	48	78	52	56	82	71	58	61	12.0	1.04	8.2%
	95th %	58	43	47	41	75	50	53	73	60	47	55	11.8	1.04	7.3%
800	Max	116	89	105	91	96	98	106	98	95	100	99	7.9	1.11	12.4%
	95th %	112	83	93	84	77	94	97	93	81	98	91	10.3	1.11	11.4%
850	Max	134	166	140	130	145	139	147	138	153	138	143	10.4	1.18	16.8%
	95th %	120	159	116	117	136	125	136	123	149	117	130	14.8	1.18	15.3%
900	Max	184	182	187	186	186	183	189	196	180	193	187	4.9	1.25	20.7%
	95th %	174	169	178	165	174	166	183	183	168	187	175	7.8	1.25	19.4%

Average Metering Rate: **960** vph

Demand (vph)	Queue Scenario	Simulated Queue Length (veh)										Mean	S.D.	D/C	Q/D
		1st Run	2nd Run	3rd Run	4th Run	5th Run	6th Run	7th Run	8th Run	9th Run	10th Run				
300	Max	9	5	6	6	4	5	5	7	8	4	6	1.7	0.31	2.0%
	95th %	1	1	1	1	0	1	2	0	1	1	1	0.6	0.31	0.3%
400	Max	8	6	7	6	6	6	6	6	5	6	6	0.8	0.42	1.6%
	95th %	2	2	3	4	2	3	2	4	2	2	3	0.8	0.42	0.7%
500	Max	7	9	12	11	8	10	8	10	7	12	9	1.9	0.52	1.9%
	95th %	4	4	5	5	5	5	4	4	4	5	5	0.5	0.52	0.9%
600	Max	11	15	14	10	14	11	10	21	13	14	13	3.3	0.63	2.2%
	95th %	7	10	9	6	7	6	6	9	6	8	7	1.5	0.63	1.2%
700	Max	19	16	17	13	16	29	12	24	24	20	19	5.4	0.73	2.7%
	95th %	11	9	12	9	11	13	7	17	16	15	12	3.3	0.73	1.7%
800	Max	37	23	28	21	23	27	20	22	23	22	25	5.0	0.83	3.1%
	95th %	22	17	21	12	15	18	12	19	16	17	17	3.3	0.83	2.1%
850	Max	23	30	19	23	43	31	27	35	36	30	30	7.1	0.89	3.5%
	95th %	18	20	14	19	34	20	19	30	23	20	22	5.9	0.89	2.6%
900	Max	37	22	28	38	48	32	41	34	40	35	36	7.2	0.94	3.9%
	95th %	27	17	25	34	40	27	35	27	32	26	29	6.4	0.94	3.2%
950	Max	61	61	73	67	43	55	44	45	39	37	53	12.6	0.99	5.5%
	95th %	48	53	64	62	35	50	38	40	30	33	45	12.0	0.99	4.8%
1000	Max	70	59	73	69	76	66	62	93	67	82	72	10.0	1.04	7.2%
	95th %	64	52	66	62	66	58	55	86	59	67	64	9.4	1.04	6.4%
1050	Max	102	106	116	93	120	136	97	114	137	109	113	14.9	1.09	10.8%
	95th %	75	99	105	79	111	124	93	104	129	103	102	17.2	1.09	9.7%
1100	Max	153	140	140	160	160	151	143	159	146	163	152	8.8	1.15	13.8%
	95th %	135	124	132	152	144	144	126	146	142	156	140	10.6	1.15	12.7%
1150	Max	194	196	196	205	200	194	197	190	193	196	196	4.1	1.20	17.1%
	95th %	188	191	178	200	182	170	178	177	182	182	183	8.4	1.20	15.9%
1200	Max	256	252	251	241	246	247	241	240	245	245	246	5.3	1.25	20.5%
	95th %	235	242	238	229	232	240	232	219	235	238	234	6.6	1.25	19.5%

Average Metering Rate: **1200** vph

Demand (vph)	Queue Scenario	Simulated Queue Length (veh)										Mean	S.D.	D/C	Q/D
		1st Run	2nd Run	3rd Run	4th Run	5th Run	6th Run	7th Run	8th Run	9th Run	10th Run				
400	Max	4	5	4	6	6	6	5	7	4	5	5	1.0	0.33	1.3%
	95th %	1	2	1	1	1	0	1	2	1	1	1	0.6	0.33	0.3%
500	Max	7	10	7	5	7	5	7	8	7	10	7	1.7	0.42	1.5%
	95th %	3	4	3	2	2	2	2	3	3	3	3	0.7	0.42	0.5%
600	Max	7	14	13	7	8	6	7	8	10	8	9	2.7	0.50	1.5%
	95th %	5	5	5	3	4	3	3	4	4	4	4	0.8	0.50	0.7%
700	Max	15	8	10	9	12	8	12	13	17	12	12	3.0	0.58	1.7%
	95th %	7	6	5	4	8	5	7	5	7	5	6	1.3	0.58	0.8%
800	Max	14	20	14	12	14	10	21	9	12	13	14	3.9	0.67	1.7%
	95th %	6	14	5	8	7	7	10	6	8	7	8	2.6	0.67	1.0%
900	Max	26	27	16	18	33	11	18	16	12	20	20	7.0	0.75	2.2%
	95th %	13	19	11	14	15	7	11	11	8	10	12	3.5	0.75	1.3%
1000	Max	35	35	27	28	30	19	21	25	23	27	27	5.4	0.83	2.7%
	95th %	21	26	16	20	21	14	13	14	16	20	18	4.1	0.83	1.8%
1100	Max	52	62	61	19	39	34	21	41	23	51	40	16.0	0.92	3.7%
	95th %	46	51	48	14	23	27	16	33	16	30	30	13.9	0.92	2.8%
1150	Max	41	36	48	43	48	32	73	30	30	35	42	12.9	0.96	3.6%
	95th %	34	28	40	35	36	24	62	23	26	30	34	11.4	0.96	2.9%
1200	Max	52	43	68	39	58	48	55	42	70	39	51	11.3	1.00	4.3%
	95th %	42	34	47	35	51	41	49	38	65	31	43	10.1	1.00	3.6%
1250	Max	68	101	90	80	95	90	74	88	78	88	85	10.0	1.04	6.8%
	95th %	61	96	77	72	90	83	67	80	69	84	78	10.9	1.04	6.2%
1300	Max	117	150	133	129	130	120	132	119	137	110	128	11.5	1.08	9.8%
	95th %	113	142	125	108	127	103	120	110	126	97	117	13.4	1.08	9.0%
1400	Max	212	201	200	231	211	228	242	200	210	215	215	14.3	1.17	15.4%
	95th %	206	190	192	224	189	220	225	184	202	211	204	15.3	1.17	14.6%
1500	Max	306	319	312	306	305	315	318	306	305	312	310	5.5	1.25	20.7%
	95th %	285	308	301	291	289	303	303	286	295	300	296	8.0	1.25	19.7%

Average Metering Rate: **1440** vph

Demand (vph)	Queue Scenario	Simulated Queue Length (veh)										Mean	S.D.	D/C	Q/D
		1st Run	2nd Run	3rd Run	4th Run	5th Run	6th Run	7th Run	8th Run	9th Run	10th Run				
500	Max	6	7	5	7	6	5	7	8	6	11	7	1.8	0.35	1.4%
	95th %	1	1	2	1	1	1	1	1	1	2	1	0.4	0.35	0.2%
600	Max	5	5	4	7	7	5	6	8	9	6	6	1.5	0.42	1.0%
	95th %	1	2	2	1	2	2	2	2	2	2	2	0.4	0.42	0.3%
700	Max	20	8	8	10	6	8	12	11	11	8	10	3.9	0.49	1.5%
	95th %	5	3	4	3	3	4	5	4	3	3	4	0.8	0.49	0.5%
800	Max	11	12	10	14	12	10	11	13	10	10	11	1.4	0.56	1.4%
	95th %	5	6	4	5	5	5	7	4	4	6	5	1.0	0.56	0.6%
900	Max	17	14	15	9	18	12	14	11	10	15	14	3.0	0.63	1.5%
	95th %	9	7	5	6	7	5	7	5	6	6	6	1.3	0.63	0.7%
1000	Max	13	23	12	22	12	16	11	17	17	21	16	4.4	0.69	1.6%
	95th %	9	12	7	12	8	7	6	8	11	13	9	2.5	0.69	0.9%
1100	Max	34	21	16	19	27	17	19	20	25	21	22	5.4	0.76	2.0%
	95th %	25	12	12	13	16	11	9	14	15	14	14	4.3	0.76	1.3%
1200	Max	23	29	18	19	27	23	22	28	28	28	25	4.0	0.83	2.0%
	95th %	15	18	15	13	20	15	14	17	20	22	17	3.0	0.83	1.4%
1300	Max	33	35	27	30	32	37	41	30	23	37	33	5.3	0.90	2.5%
	95th %	27	27	21	17	25	26	37	24	18	31	25	5.9	0.90	1.9%
1350	Max	43	46	38	46	46	38	27	38	49	41	41	6.4	0.94	3.1%
	95th %	34	34	28	34	31	31	21	30	39	34	32	4.8	0.94	2.3%
1400	Max	77	54	62	84	45	36	37	47	41	37	52	17.2	0.97	3.7%
	95th %	62	45	56	76	36	30	31	37	38	30	44	15.6	0.97	3.2%
1450	Max	57	47	51	50	74	70	64	64	59	57	59	8.8	1.01	4.1%
	95th %	49	37	45	43	67	62	53	56	51	45	51	9.1	1.01	3.5%
1500	Max	89	119	111	92	101	92	75	102	72	92	95	14.6	1.04	6.3%
	95th %	81	107	102	75	95	85	62	89	63	85	84	15.0	1.04	5.6%
1550	Max	118	126	147	137	114	122	116	156	148	126	131	15.0	1.08	8.5%
	95th %	112	117	136	128	102	100	109	142	141	119	121	15.5	1.08	7.8%

Average Metering Rate: **1440** vph (Continue)

1600	Max	192	189	194	172	170	171	173	179	183	184	181	9.0	1.11	11.3%
	95^{th} %	181	173	183	165	142	162	161	173	174	173	169	11.9	1.11	10.5%
1700	Max	263	265	294	263	265	278	271	260	260	260	268	10.8	1.18	15.8%
	95^{th} %	247	236	277	249	255	259	262	235	239	226	249	15.2	1.18	14.6%
1800	Max	368	362	362	363	365	378	371	362	360	362	365	5.6	1.25	20.3%
	95^{th} %	356	332	358	334	337	339	354	340	322	347	342	11.7	1.25	19.0%

Average Metering Rate: **1680** vph

Demand (vph)	Queue Scenario	Simulated Queue Length (veh)										Mean	S.D.	D/C	Q/D
		1st Run	2nd Run	3rd Run	4th Run	5th Run	6th Run	7th Run	8th Run	9th Run	10th Run				
600	Max	9	8	5	7	7	8	6	3	3	8	6	2.1	0.36	1.1%
	95th %	1	0	1	2	2	2	1	1	0	1	1	0.7	0.36	0.2%
700	Max	10	9	12	4	6	5	7	8	6	8	8	2.4	0.42	1.1%
	95th %	2	2	3	1	2	2	2	3	2	3	2	0.6	0.42	0.3%
800	Max	4	12	6	16	9	5	6	11	9	9	9	3.7	0.48	1.1%
	95th %	2	3	3	4	2	2	3	3	3	2	3	0.7	0.48	0.3%
900	Max	9	8	13	9	10	10	8	8	7	13	10	2.1	0.54	1.1%
	95th %	5	4	5	5	6	4	5	3	4	5	5	0.8	0.54	0.5%
1000	Max	10	13	17	12	10	8	14	17	11	14	13	3.0	0.60	1.3%
	95th %	4	6	9	6	5	4	7	6	7	6	6	1.5	0.60	0.6%
1100	Max	11	15	11	12	18	13	15	14	13	18	14	2.5	0.65	1.3%
	95th %	7	7	5	6	9	6	7	6	6	8	7	1.2	0.65	0.6%
1200	Max	18	20	12	18	10	21	11	12	19	15	16	4.1	0.71	1.3%
	95th %	9	12	7	9	7	8	5	9	11	11	9	2.1	0.71	0.7%
1300	Max	16	15	36	20	21	25	12	23	26	21	22	6.8	0.77	1.7%
	95th %	13	10	21	13	11	16	8	13	20	13	14	4.1	0.77	1.1%
1400	Max	24	21	29	21	34	15	20	33	44	26	27	8.5	0.83	1.9%
	95th %	14	12	22	12	22	10	14	19	31	16	17	6.4	0.83	1.2%
1500	Max	29	25	26	36	38	30	26	35	45	26	32	6.7	0.89	2.1%
	95th %	19	21	23	26	30	22	18	28	28	20	24	4.2	0.89	1.6%
1550	Max	33	36	37	38	44	36	45	31	43	41	38	4.7	0.92	2.5%
	95th %	21	28	26	28	36	30	40	22	35	32	30	6.1	0.92	1.9%
1600	Max	53	35	38	32	51	45	37	47	36	55	43	8.3	0.95	2.7%
	95th %	42	26	35	22	44	37	26	43	31	47	35	8.7	0.95	2.2%
1650	Max	41	75	37	54	67	75	63	55	46	54	57	13.3	0.98	3.4%
	95th %	35	65	28	49	58	59	56	51	39	47	49	11.7	0.98	3.0%
1700	Max	81	46	74	87	94	56	69	71	85	71	73	14.5	1.01	4.3%
	95th %	72	38	64	73	83	40	53	63	79	62	63	15.2	1.01	3.7%

Average Metering Rate: **1680** vph (Continue)

1750	Max	89	104	129	135	107	113	124	101	89	99	109	16.0	1.04	6.2%
	95th %	83	100	116	120	90	107	117	92	84	90	100	14.2	1.04	5.7%
1800	Max	155	122	151	138	126	133	128	131	121	141	135	11.6	1.07	7.5%
	95th %	143	95	141	134	122	124	104	109	108	135	122	16.8	1.07	6.8%
1850	Max	181	210	181	190	178	184	206	184	179	171	186	12.4	1.10	10.1%
	95th %	172	205	142	182	166	175	199	175	172	142	173	20.4	1.10	9.4%
1900	Max	231	236	234	222	228	233	233	221	227	234	230	5.2	1.13	12.1%
	95th %	211	230	226	210	212	214	222	200	220	223	217	9.0	1.13	11.4%
2000	Max	329	321	320	320	320	326	330	325	320	354	327	10.4	1.19	16.3%
	95th %	316	304	301	298	294	300	312	308	303	344	308	14.2	1.19	15.4%
2100	Max	420	438	432	420	420	421	433	425	420	422	425	6.7	1.25	20.2%
	95th %	378	426	400	392	409	399	420	408	403	411	405	13.7	1.25	19.3%

Average Metering Rate: **1920** vph

Demand (vph)	Queue Scenario	Simulated Queue Length (veh)										Mean	S.D.	D/C	Q/D
		1st Run	2nd Run	3rd Run	4th Run	5th Run	6th Run	7th Run	8th Run	9th Run	10th Run				
600	Max	5	3	3	4	4	6	3	5	4	4	4	1.0	0.31	0.7%
	95th %	0	0	0	0	0	0	0	0	0	0	0	0.0	0.31	0.0%
700	Max	6	3	5	4	6	6	5	8	4	3	5	1.6	0.36	0.7%
	95th %	2	1	1	1	1	1	1	2	2	1	1	0.5	0.36	0.2%
800	Max	7	9	6	9	6	8	10	8	4	7	7	1.8	0.42	0.9%
	95th %	2	2	2	2	2	1	1	1	2	1	2	0.5	0.42	0.2%
900	Max	8	6	8	5	8	11	11	6	11	9	8	2.2	0.47	0.9%
	95th %	2	3	3	3	2	2	3	2	3	3	3	0.5	0.47	0.3%
1000	Max	6	9	11	9	6	11	7	5	6	10	8	2.3	0.52	0.8%
	95th %	3	4	4	5	3	4	4	2	3	4	4	0.8	0.52	0.4%
1100	Max	9	11	16	11	15	10	13	9	14	8	12	2.8	0.57	1.1%
	95th %	4	6	5	4	7	5	6	4	5	4	5	1.1	0.57	0.5%
1200	Max	13	12	11	12	17	11	11	9	10	11	12	2.2	0.63	1.0%
	95th %	7	5	4	7	8	6	7	7	6	6	6	1.2	0.63	0.5%
1300	Max	11	21	16	11	16	22	16	13	12	12	15	4.0	0.68	1.2%
	95th %	7	9	8	6	9	11	7	6	8	6	8	1.6	0.68	0.6%
1400	Max	17	30	16	21	14	19	14	15	17	15	18	4.8	0.73	1.3%
	95th %	9	11	8	9	10	8	8	9	7	8	9	1.2	0.73	0.6%
1500	Max	20	14	25	15	23	20	16	17	14	19	18	3.8	0.78	1.2%
	95th %	11	10	13	10	14	10	10	11	10	12	11	1.4	0.78	0.7%
1600	Max	29	20	29	32	17	27	28	33	18	12	25	7.2	0.83	1.5%
	95th %	11	11	20	21	11	19	18	27	13	10	16	5.7	0.83	1.0%
1700	Max	23	29	32	42	28	26	21	22	21	27	27	6.4	0.89	1.6%
	95th %	15	18	22	32	17	21	14	17	15	19	19	5.2	0.89	1.1%
1750	Max	25	31	36	45	34	32	31	28	24	24	31	6.4	0.91	1.8%
	95th %	17	19	26	40	23	26	23	20	18	19	23	6.7	0.91	1.3%
1800	Max	35	42	60	29	48	27	33	52	45	49	42	10.8	0.94	2.3%
	95th %	26	25	43	17	33	22	25	39	35	37	30	8.4	0.94	1.7%

Average Metering Rate: **1920** vph (Continue)

1850	Max	47	44	43	28	39	47	57	27	28	63	42	12.2	0.96	2.3%
	95th %	36	40	38	23	35	40	51	23	24	53	36	10.7	0.96	2.0%
1900	Max	45	61	83	64	61	66	52	76	51	64	62	11.4	0.99	3.3%
	95th %	39	56	75	56	49	52	42	62	44	54	53	10.5	0.99	2.8%
1950	Max	68	93	78	74	71	71	106	85	67	82	80	12.4	1.02	4.1%
	95th %	60	84	69	64	54	61	95	73	49	75	68	13.9	1.02	3.5%
2000	Max	90	80	112	98	85	105	114	99	101	109	99	11.4	1.04	5.0%
	95th %	82	75	100	86	78	98	101	84	93	92	89	9.3	1.04	4.4%
2050	Max	145	150	189	130	144	177	142	132	160	148	152	18.8	1.07	7.4%
	95th %	136	142	173	92	115	160	122	104	148	136	133	25.0	1.07	6.5%
2100	Max	221	206	195	181	187	213	196	193	203	195	199	12.0	1.09	9.5%
	95th %	213	189	188	158	155	203	170	183	189	184	183	18.2	1.09	8.7%
2150	Max	236	243	242	234	234	242	235	249	234	232	238	5.5	1.12	11.1%
	95th %	217	238	230	210	226	232	208	234	203	218	222	12.1	1.12	10.3%
2200	Max	296	290	312	288	287	296	280	283	286	292	291	9.0	1.15	13.2%
	95th %	288	274	307	283	270	286	257	263	251	272	275	16.5	1.15	12.5%
2300	Max	391	386	386	386	380	401	408	380	380	389	389	9.3	1.20	16.9%
	95th %	366	349	360	370	363	389	384	371	348	373	367	13.3	1.20	16.0%
2400	Max	480	480	484	489	484	484	480	489	480	480	483	3.7	1.25	20.1%
	95th %	462	435	458	471	467	425	442	441	455	457	451	14.9	1.25	18.8%

Average Metering Rate: **2160** vph

Demand (vph)	Queue Scenario	Simulated Queue Length (veh)										Mean	S.D.	D/C	Q/D
		1st Run	2nd Run	3rd Run	4th Run	5th Run	6th Run	7th Run	8th Run	9th Run	10th Run				
700	Max	6	5	3	5	2						4	1.6	0.32	0.6%
	95th %	0	0	0	0	0						0	0.0	0.32	0.0%
800	Max	8	4	3	5	2						4	2.3	0.37	0.6%
	95th %	0	0	0	1	0						0	0.4	0.37	0.0%
900	Max	5	5	5	5	6	4	5	5	7	7	5	1.0	0.42	0.6%
	95th %	1	0	1	2	1	1	0	1	2	1	1	0.7	0.42	0.1%
1000	Max	7	11	3	15	5	6	6	4	6	6	7	3.5	0.46	0.7%
	95th %	2	2	2	3	2	1	2	2	2	2	2	0.5	0.46	0.2%
1100	Max	8	7	10	7	10	6	9	7	5	9	8	1.7	0.51	0.7%
	95th %	2	3	4	3	3	4	2	3	3	3	3	0.7	0.51	0.3%
1200	Max	8	12	9	9	15	8	9	11	10	9	10	2.2	0.56	0.8%
	95th %	2	5	4	2	4	3	4	4	3	5	4	1.1	0.56	0.3%
1300	Max	16	12	9	10	8	14	14	9	12	7	11	3.0	0.60	0.9%
	95th %	5	5	4	5	4	6	4	5	4	4	5	0.7	0.60	0.4%
1400	Max	11	10	12	12	9	10	11	13	11	10	11	1.2	0.65	0.8%
	95th %	6	5	5	5	5	4	5	6	4	4	5	0.7	0.65	0.4%
1500	Max	11	10	17	18	12	11	14	13	11	15	13	2.7	0.69	0.9%
	95th %	7	7	10	8	6	7	7	9	6	9	8	1.3	0.69	0.5%
1600	Max	19	19	20	16	10	13	20	13	17	21	17	3.7	0.74	1.1%
	95th %	11	10	9	9	6	7	9	8	9	11	9	1.6	0.74	0.6%
1700	Max	15	18	20	20	23	21	20	15	17	19	19	2.6	0.79	1.1%
	95th %	9	10	10	11	15	17	10	10	10	9	11	2.7	0.79	0.7%
1800	Max	17	31	30	16	16	18	19	17	23	20	21	5.6	0.83	1.2%
	95th %	13	17	20	12	11	12	12	11	13	10	13	3.1	0.83	0.7%
1900	Max	26	26	28	23	38	27	36	28	23	31	29	5.0	0.88	1.5%
	95th %	19	20	18	15	31	16	28	20	17	26	21	5.4	0.88	1.1%
1950	Max	40	48	31	30	27	23	28	35	32	22	32	7.8	0.90	1.6%
	95th %	37	37	19	22	18	16	18	21	21	15	22	8.0	0.90	1.1%

Average Metering Rate: **2160** vph (Continue)

2000	Max	22	30	28	27	35	43	43	33	33	34	33	6.6	0.93	1.6%
	95th %	16	22	22	21	30	36	28	26	25	21	25	5.6	0.93	1.2%
2050	Max	44	54	42	51	40	41	66	67	57	37	50	10.9	0.95	2.4%
	95th %	37	42	30	42	34	35	55	59	51	29	41	10.5	0.95	2.0%
2100	Max	59	72	41	42	56	50	62	39	51	73	55	12.2	0.97	2.6%
	95th %	50	57	29	34	47	43	46	30	39	51	43	9.4	0.97	2.0%
2150	Max	59	79	67	73	67	64	71	67	73	76	70	5.9	1.00	3.2%
	95th %	45	61	61	63	59	48	65	45	65	67	58	8.6	1.00	2.7%
2200	Max	65	84	91	107	63	77	68	71	57	68	75	15.1	1.02	3.4%
	95th %	49	73	80	99	54	68	63	63	43	63	66	16.1	1.02	3.0%
2250	Max	94	140	108	113	143	107	129	99	116	93	114	17.9	1.04	5.1%
	95th %	84	125	102	98	127	93	119	90	110	86	103	16.0	1.04	4.6%
2300	Max	142	147	163	163	145	154	164	140	171	152	154	10.7	1.06	6.7%
	95th %	136	126	143	152	124	147	155	129	163	144	142	13.0	1.06	6.2%
2350	Max	214	198	206	212	193	198	196	217	197	213	204	9.0	1.09	8.7%
	95th %	209	177	198	202	182	193	176	206	187	198	193	11.8	1.09	8.2%
2400	Max	243	256	273	248	245	249	249	247	266	268	254	10.8	1.11	10.6%
	95th %	230	247	259	223	234	234	244	233	238	261	240	12.4	1.11	10.0%
2500	Max	341	359	357	351	343	351	343	341	343	355	348	7.0	1.16	13.9%
	95th %	318	346	337	345	310	342	336	305	316	343	330	15.8	1.16	13.2%
2600	Max	440	459	440	443	440	456	440	443	440	446	445	7.1	1.20	17.1%
	95th %	399	432	391	413	415	440	409	423	419	439	418	16.2	1.20	16.1%
2700	Max	540	540	540	544	542	550	546	543	540	554	544	4.8	1.25	20.1%
	95th %	511	500	513	512	512	522	517	512	494	527	512	9.5	1.25	19.0%

Average Metering Rate: **2400** vph

Demand (vph)	Queue Scenario	Simulated Queue Length (veh)										Mean	S.D.	D/C	Q/D
		1st Run	2nd Run	3rd Run	4th Run	5th Run	6th Run	7th Run	8th Run	9th Run	10th Run				
800	Max	3	3	2	6	3						3	1.5	0.33	0.4%
	95th %	0	0	0	0	0						0	0.0	0.33	0.0%
900	Max	4	3	3	5	5	3	3	5	3	4	4	0.9	0.38	0.4%
	95th %	0	0	0	0	2	0	0	0	1	0	0	0.7	0.38	0.0%
1000	Max	7	5	4	3	5	4	9	8	5	9	6	2.2	0.42	0.6%
	95th %	1	1	1	0	1	0	1	1	1	2	1	0.6	0.42	0.1%
1100	Max	10	3	6	5	8	4	6	6	10	7	7	2.3	0.46	0.6%
	95th %	2	1	1	2	3	1	2	2	4	2	2	0.9	0.46	0.2%
1200	Max	6	10	7	6	7	13	12	6	4	8	8	2.9	0.50	0.7%
	95th %	2	3	3	3	3	3	3	3	2	3	3	0.4	0.50	0.2%
1300	Max	4	8	7	11	8	6	14	9	6	10	8	2.9	0.54	0.6%
	95th %	2	3	3	5	3	3	5	5	3	4	4	1.1	0.54	0.3%
1400	Max	11	8	8	19	13	7	20	12	8	9	12	4.6	0.58	0.8%
	95th %	5	4	5	5	4	3	8	6	3	5	5	1.5	0.58	0.3%
1500	Max	11	16	8	11	19	11	12	20	9	13	13	4.1	0.63	0.9%
	95th %	5	6	5	6	6	4	6	6	5	5	5	0.7	0.63	0.4%
1600	Max	8	11	13	8	15	10	14	22	8	10	12	4.4	0.67	0.7%
	95th %	5	5	5	5	9	5	7	9	5	5	6	1.7	0.67	0.4%
1700	Max	11	12	20	14	19	10	14	18	17	18	15	3.6	0.71	0.9%
	95th %	6	7	9	10	8	6	7	7	13	8	8	2.1	0.71	0.5%
1800	Max	18	12	16	11	14	12	13	16	25	14	15	4.1	0.75	0.8%
	95th %	13	7	9	6	10	9	8	8	10	8	9	1.9	0.75	0.5%
1900	Max	22	24	16	15	16	21	28	21	15	19	20	4.3	0.79	1.0%
	95th %	19	12	9	8	9	11	12	8	10	10	11	3.2	0.79	0.6%
2000	Max	19	19	15	22	24	29	24	19	22	19	21	3.9	0.83	1.1%
	95th %	12	15	11	11	11	19	14	12	15	13	13	2.5	0.83	0.7%
2100	Max	19	23	29	33	21	24	28	27	27	23	25	4.2	0.88	1.2%
	95th %	13	18	14	19	15	17	14	16	21	17	16	2.5	0.88	0.8%

Average Metering Rate: **2400** vph (Continue)

2150	Max	19	25	27	34	33	24	21	22	29	43	28	7.3	0.90	1.3%
	95th %	15	16	23	20	19	19	18	17	17	21	19	2.4	0.90	0.9%
2200	Max	59	40	35	24	51	23	41	40	27	3	34	15.9	0.92	1.6%
	95th %	47	32	25	20	27	17	22	34	21	32	28	8.9	0.92	1.3%
2250	Max	34	53	30	23	37	39	43	36	29	46	37	8.8	0.94	1.6%
	95th %	21	40	22	17	27	32	26	31	23	40	28	7.8	0.94	1.2%
2300	Max	35	50	30	39	52	30	64	47	29	41	42	11.5	0.96	1.8%
	95th %	29	33	24	27	38	27	55	36	22	31	32	9.5	0.96	1.4%
2350	Max	43	43	61	52	51	49	75	78	65	58	58	12.3	0.98	2.4%
	95th %	34	30	56	44	41	35	57	69	55	44	47	12.4	0.98	2.0%
2400	Max	50	99	65	72	52	76	57	84	65	54	67	15.7	1.00	2.8%
	95th %	43	80	56	65	43	71	48	65	51	42	56	13.3	1.00	2.4%
2450	Max	91	119	72	84	120	99	69	87	65	72	88	19.8	1.02	3.6%
	95th %	79	112	52	74	111	84	64	77	55	62	77	20.9	1.02	3.1%
2500	Max	108	120	173	139	131	120	135	104	126	119	128	19.4	1.04	5.1%
	95th %	81	111	159	130	121	111	124	90	111	112	115	21.4	1.04	4.6%
2550	Max	184	178	150	159	193	179	153	163	169	165	169	13.9	1.06	6.6%
	95th %	171	158	136	151	166	174	144	154	145	158	156	12.2	1.06	6.1%
2600	Max	200	208	212	225	214	272	251	220	235	201	224	23.0	1.08	8.6%
	95th %	173	198	201	201	202	259	238	198	227	183	208	25.9	1.08	8.0%
2650	Max	267	273	278	263	263	255	260	253	260	287	266	10.6	1.10	10.0%
	95th %	243	258	243	257	256	245	233	233	253	279	250	13.7	1.10	9.4%
2700	Max	300	317	307	302	305	305	314	315	310	301	308	6.1	1.13	11.4%
	95th %	291	305	283	288	298	277	302	305	301	271	292	12.1	1.13	10.8%
2800	Max	409	400	406	405	411	400	405	411	401	400	405	4.5	1.17	14.5%
	95th %	382	388	400	381	370	381	374	404	389	385	385	10.5	1.17	13.8%
2900	Max	510	504	503	507	501	500	500	500	502	517	504	5.5	1.21	17.4%
	95th %	495	471	449	470	479	465	466	483	478	475	473	12.3	1.21	16.3%
3000	Max	600	619	601	602	604	605	611	600	603	600	605	6.1	1.25	20.2%
	95th %	579	603	581	563	587	573	584	554	588	568	578	14.1	1.25	19.3%

Appendix B: Acceleration Predictions for Each Data Collection Site

B1: EB Mowry Ave to NB 880

Table B1. Field observed 85th, 50th, and 15th percentile spot speed at the predetermined locations

%	Percentile Spot Speed (mph)								
	0	**20**	**50**	**100**	**150**	**200**	**300**	**400**	**500**
85%	1.49	10.35	15.53	21.26	24.94	27.54	33.10	37.09	39.11
50%	4.06	11.57	17.15	23.51	27.85	30.68	37.00	41.58	43.72
15%	8.54	13.15	19.01	26.05	30.57	33.77	40.69	45.33	48.26

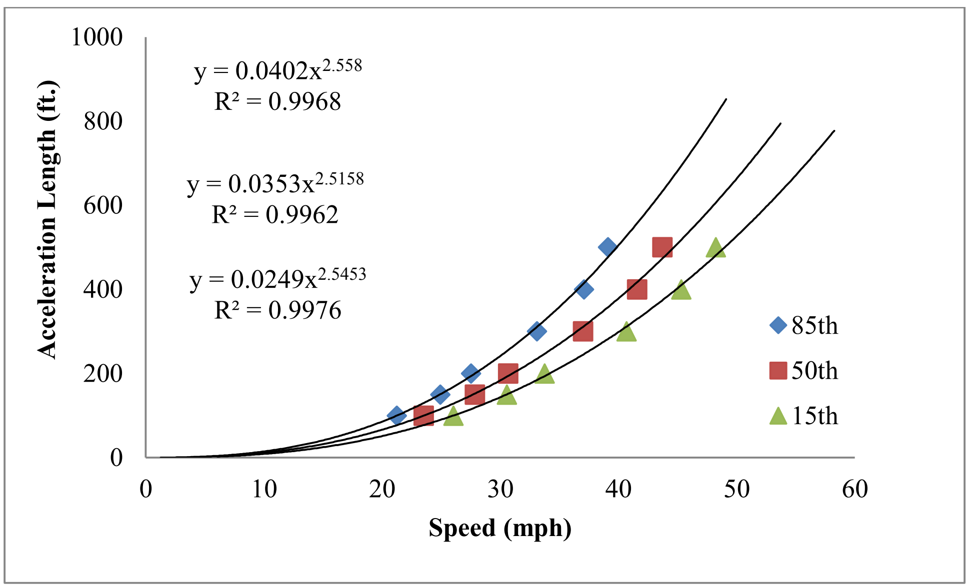


Figure B1. Predicted 85th, 50th, and 15th percentile acceleration length versus speed profiles

B2: WB Alvarado Rd. to SB 880

Table B2. Field observed 85th, 50th, and 15th percentile spot speed at the predetermined locations

%	Spot Speed at Designated Locations (mph)								
	V0	**V20**	**V50**	**V100**	**V150**	**V200**	**V300**	**V400**	**V500**
85%	0.00	11.80	16.08	21.32	26.50	29.32	32.81	35.89	36.85
50%	7.76	13.38	18.05	23.70	29.41	32.54	36.94	40.71	42.94
15%	10.25	15.06	20.09	26.44	32.84	36.26	41.15	45.22	47.55

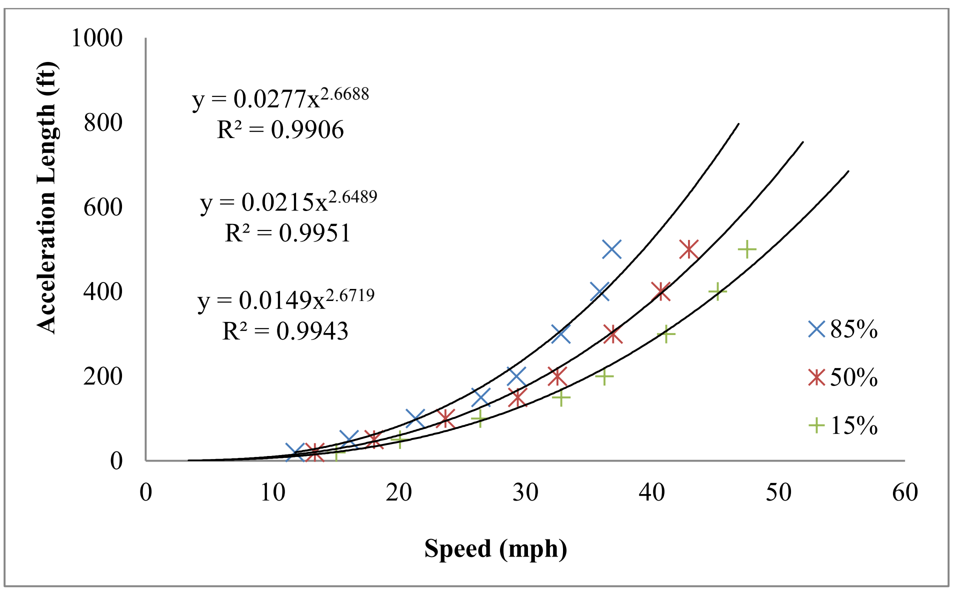


Figure B2. Predicted 85th, 50th, and 15th percentile acceleration length versus speed profiles

B3: Artesia Blvd to NB 405

Table B3. Field observed 85th, 50th, and 15th percentile spot speed at the predetermined locations

%	Percentile Spot Speed (mph)								
	0	**20**	**50**	**100**	**150**	**200**	**300**	**400**	**500**
85%	3.47	10.39	16.04	21.33	26.53	30.78	35.16	40.05	3.47
50%	7.29	11.67	18.23	24.09	30.11	35.63	41.08	46.14	7.29
15%	10.04	13.38	20.61	26.72	33.43	39.24	45.84	52.28	10.04

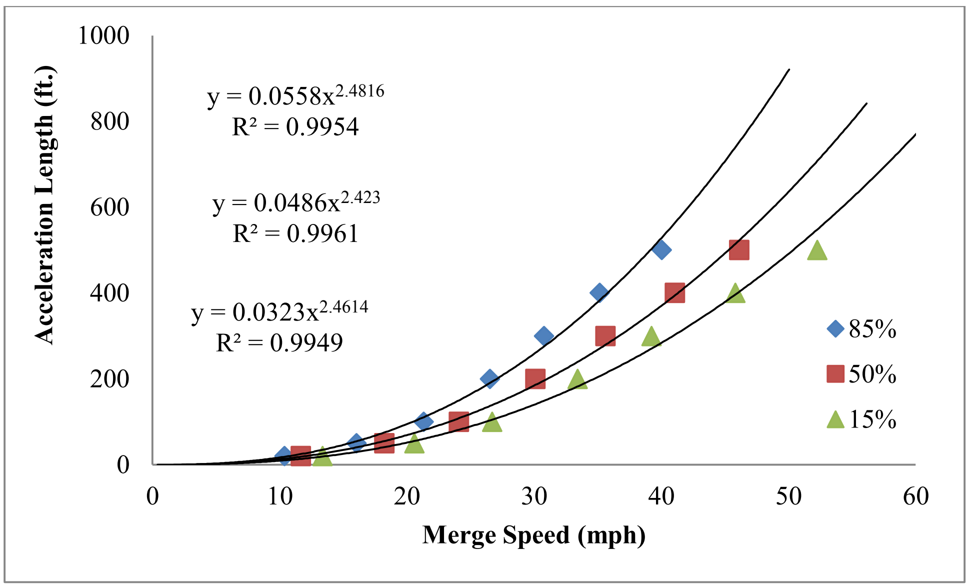


Figure B3. Predicted 85th, 50th, and 15th percentile acceleration length versus speed profiles

B4: SB Douglas Blvd to WB 80

Table B4. Field observed 85th, 50th, and 15th percentile spot speed at the predetermined locations

%	Spot Speed at Designated Locations (mph)											
	0	**10**	**20**	**40**	**60**	**80**	**130**	**180**	**280**	**380**	**480**	**580**
85th	5.03	11.42	13.88	18.02	20.98	22.45	28.33	30.38	35.37	37.29	40.43	42.40
50th	9.15	12.12	14.44	18.30	21.42	25.78	30.52	33.74	38.12	39.90	43.02	47.55
15th	12.67	14.36	16.20	20.01	23.59	27.60	32.67	35.67	40.67	42.67	46.40	53.20

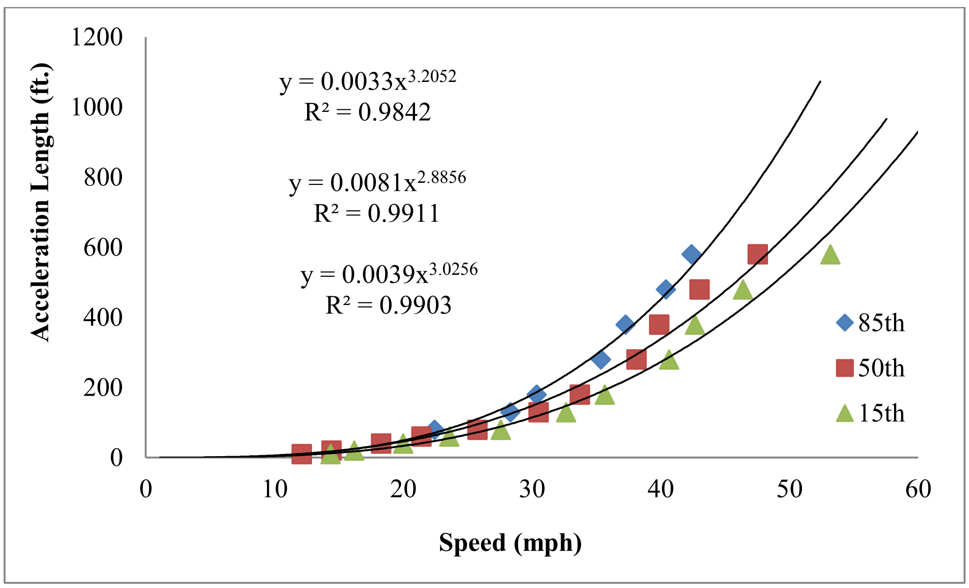


Figure B4. Predicted 85th, 50th, and 15th percentile acceleration length versus speed profiles

B5: Fruitridge Rd. to NB 99

Table B5. Field observed 85th, 50th, and 15th percentile spot speed at the predetermined locations

%	Spot Speed at Designated Locations (mph)								
	V0	V10	V30	V50	V100	V200	V300	V400	V500
85th	0.184	7.026	12.215	16.026	21.379	27.970	31.872	34.794	36.770
50th	4.469	8.182	12.979	17.544	23.811	30.888	35.342	38.957	42.309
15th	7.386	10.258	15.088	19.412	27.228	34.648	40.094	43.500	46.665

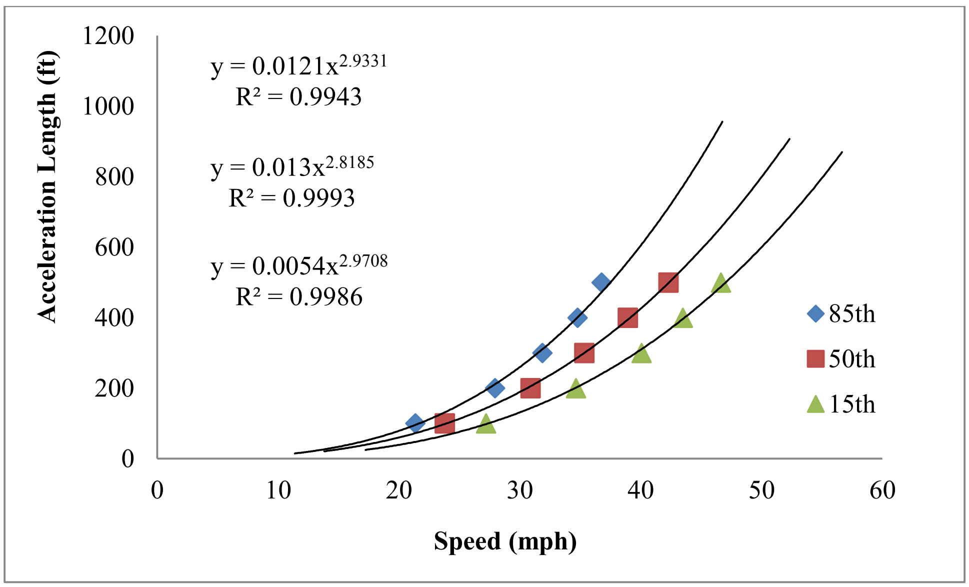


Figure B5. Predicted 85th, 50th, and 15th percentile acceleration length versus speed profiles

B6: Industrial Pkwy to NB 880

Table B6. Field observed 85th, 50th, and 15th percentile spot speed at the predetermined locations

%	Spot Speed at Designated Locations (mph)							
	0	20	50	100	200	300	400	500
85th	0.00	11.19	15.76	20.52	26.36	29.37	32.48	35.10
50th	6.75	12.93	17.92	23.52	30.37	34.14	37.87	41.51
15th	9.77	15.06	20.19	26.07	33.89	38.42	42.61	47.32

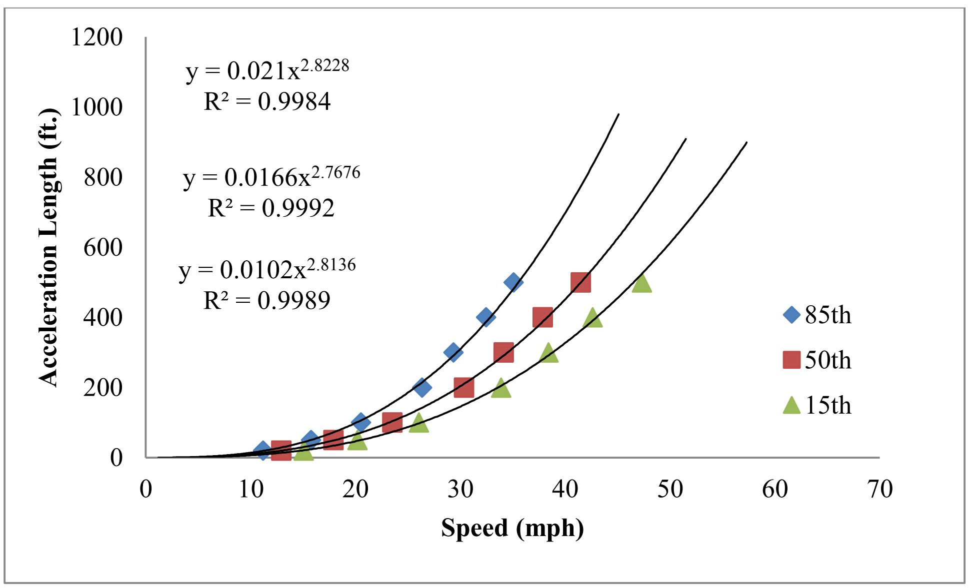


Figure B6. Predicted 85th, 50th, and 15th percentile acceleration length versus speed profiles

B7: WB Rosecrans Blvd to NB 710

Table B7. Field observed 85th, 50th, and 15th percentile spot speed at the predetermined locations

%	Spot Speed at Designated Locations (mph)								
	0	20	50	100	200	300	400	500	600
85%	3.87	11.29	16.05	20.35	26.47	29.79	32.86	35.83	37.78
50%	6.96	13.31	18.77	24.05	30.19	34.26	37.18	40.28	42.78
15%	9.07	14.70	20.09	26.14	33.68	37.90	42.09	44.93	47.61

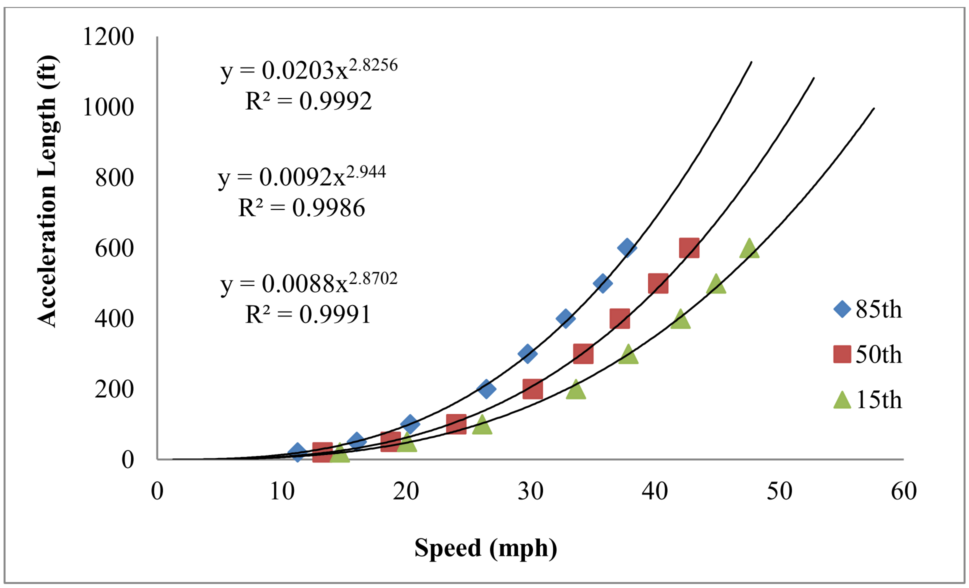


Figure B7. Predicted 85th, 50th, and 15th percentile acceleration length versus speed profiles