

Guidelines for Evaluation of Ramp Signaling Deployments in a Real-Time Operations Environment

FDOT Project BDV29-977-25

Final Report

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By

**Lehman Center of Transportation Research
Florida International University**



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DISCLAIMER

The opinions, findings, and conclusions expressed in this publication are those of the authors and not necessarily those of the State of Florida Department of Transportation.

METRIC CONVERSION CHART

APPROXIMATE CONVERSIONS TO SI UNITS

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
LENGTH				
in	inches	25.4	millimeters	mm
ft	feet	0.305	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km
AREA				
in²	square inches	645.2	square millimeters	mm ²
ft²	square feet	0.093	square meters	m ²
yd²	square yard	0.836	square meters	m ²
ac	acres	0.405	hectares	ha
mi²	square miles	2.59	square kilometers	km ²
VOLUME				
fl oz	fluid ounces	29.57	milliliters	mL
gal	gallons	3.785	liters	L
ft³	cubic feet	0.028	cubic meters	m ³
yd³	cubic yards	0.765	cubic meters	m ³
NOTE: volumes greater than 1000 L shall be shown in m ³				
MASS				
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams (or "metric ton")	Mg (or "t")
TEMPERATURE (exact degrees)				
°F	Fahrenheit	5 (F-32)/9 or (F-32)/1.8	Celsius	°C
ILLUMINATION				
fc	foot-candles	10.76	lux	lx
fl	foot-Lamberts	3.426	candela/m ²	cd/m ²
FORCE and PRESSURE or STRESS				
lbf	poundforce	4.45	newtons	N
lbf/in²	poundforce per square inch	6.89	kilopascals	kPa

*SI is the symbol for the International System of Units. Appropriate rounding should be made to comply with Section 4 of ASTM E380.

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16. Abstract State agencies have developed warrants and guidelines for the identification of on-ramps for metering. However, these warrants only consider recurrent traffic conditions in the vicinity of each on-ramp without considering the need to meter multiple ramps to mitigate the impacts of downstream system bottlenecks. The goal of this project was to develop a system-wide-based method in addition to existing local warrants for the offline identification of ramp metering installation locations and for online activation of ramp metering. A methodology was developed in this study to identify the ramps to meter based on system-wide recurrent and non-recurrent traffic conditions. This methodology incorporates the stochastic nature of the demand and capacity and the impacts of incidents and weather using Monte Carlo simulation and a ramp selection procedure based on a linear programming formulation. The variation in queue storage capacity between ramps, the available acceleration lanes, and the related impacts of trucks are also considered in the developed methodology. The output of this method can be used in benefit-cost analyses of ramp metering deployments and associated decisions such as which ramps to meter and when to activate in real-time.			
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EXECUTIVE SUMMARY

Proper ramp metering can improve safety and mobility as well as improve environmental conditions. State agencies have developed warrants and guidelines for the identification of on-ramps for metering. However, these warrants only consider the traffic conditions in the vicinity of each on-ramp without considering the need to meter multiple ramps to mitigate the impacts of downstream system bottlenecks. The warrants do not employ detailed analyses of traffic conditions, including both recurrent and non-recurrent incident and weather conditions, and do not take advantage of the increasing availability of detailed data from multiple sources. To address these gaps, the goal of this project was to develop a system-wide-based method to be used in conjunction with existing local warrants for the offline identification of ramp metering installation locations and a method for online activation of ramp metering. The objectives of this project were as follows:

- To develop system-based methods for ramp metering installation considering recurrent conditions in the planning stage
- To develop system-based methods for ramp metering installation considering recurrent conditions in the planning stage
- To extend the developed offline system-based ramp metering method for the real-time selection of the ramps for activation.

Below is a summary of the tasks conducted in this project to achieve the project goal and objectives.

E.1 Study Area and Data Collection

This study utilized a corridor of about 15 miles as a case study to demonstrate the application of the developed methodology. The corridor included ten on-ramps along Interstate I-95 in Broward County, FL. Traffic data including flow and speed and incident data for the northbound direction of the corridor were collected from the traffic management data warehouse of the Florida Department of Transportation (FDOT) for the morning and afternoon peaks (7:00-10:00 AM and 3:00-6:00 PM, respectively). Weather data were obtained from the National Climate Data Center (NCDC). Crash data were collected from both FDOT District 4 incident database and the Signal Four Analytics website developed by the University of Florida.

E.2 Assessment of the Need for System Consideration in Addition to Local Warrants for Ramp Metering Installation

Examining existing warrants used by states around the nation indicated that they were all based on local conditions in the vicinity of each examined ramp. This has created a disconnection

between existing agency metering warrants and the subsequent management and operations of the ramp meters since most meters use system-based metering algorithms. An initial exploration was done in this study to determine the impacts of considering system-wide conditions in offline selection of ramps for metering. A linear programming formulation combined with the consideration of the stochasticity of bottleneck capacity was used to select the ramps to be metered, based on the system bottleneck. The stochasticity of capacity at the bottleneck location was accounted for by identifying the historical real-world capacity distribution.

The study demonstrated that more ramps can be justified when using the linear programming formulation compared to the ramps identified utilizing the warrants based on existing conditions. The selection of additional ramps for metering based on system bottlenecks, in addition to those justified by local warrants, can delay the breakdown at the system bottleneck location and improve the performance of the freeway mainline. Another important benefit of selecting ramps for metering based on system operations is that it distributes the on-ramp delays and queues due to metering among more ramps. This leads to reducing the experienced delays on the ramps selected by utilizing the existing warrants that are based on local conditions.

E3. Offline Ramp Selection for Metering during Recurrent and Non-Recurrent Conditions

A methodology was then developed for offline selection of ramps for metering, based on system recurrent and non-recurrent conditions. This method is an extension of a linear programming formulation that has been utilized for a long time to estimate the ramp metering rate for time of day metering. According to this formulation, for each of the links between ramps, at a given period, there is demand, and there is capacity. For the ramp metering to be successful, the demand has to be kept lower than the capacity for each link. Thus, the ramps to meter and the metering rates are selected to keep the demands below capacity for each link. The demand for a link is calculated as the sum of all the inflows (from the upstream freeway mainline and upstream on-ramps) minus all out flows (to all upstream off-ramps). The rates, however, have to be also set to meet absolute minimum and maximum rates based on previous commonly used values. In addition, the rates have to be set such that the resultant on-ramp queues should not exceed the queueing storage constraints specified by the users. The proposed formulation was solved in this study by first metering only the first on-ramp upstream of a facility bottleneck location. If the constraints of the formulation were met, there was no need to meter another ramp, otherwise, the next upstream on-ramp was selected for metering and so on until the constraints of the formulation were met.

An important aspect of this study is that, rather than using averages, both demand and capacity are considered as stochastic variables, changing from day to day even for “normal days” (days with no events like incidents and rain). Although the Highway Capacity Manual (HCM) assumes deterministic capacity, there is an agreement that capacity is a stochastic variable that changes

from day to day depending on factors such as variations in truck proportion, driver aggressiveness, traffic composition, and so on. The stochastic distributions of demand and capacity are generated based on real-world point detector data for “normal days.” These distributions are then used in a Monte Carlo simulation to generate 1,000 combinations of capacity and demand values that represent the conditions in 1,000 normal days. Each of these combinations of capacity and demand values are used as inputs to one execution of the ramp selection algorithm described above and the ramps that need to be metered are selected based on each execution. This results in 1,000 sets of ramps that need to be metered, each representing the metering requirement for each day. The ramp selection formulation also provides estimates of the metering rate and queue length on each ramp that can be used in the planning analysis.

The procedure described above to determine the ramps to meter for “normal days” or “recurrent conditions” are also extended to determine the ramps that need to be metered during incident and weather events that produce “non-recurrent” congestion. The only difference is including the impacts of non-recurrent events (incidents and weather) on the capacity and possibly the demand when applying the method. The HCM recommends values for use to estimate the drops in capacity due to incidents with different lane blockages and rain with different intensities. These capacity drop values were used in this study. The diversion percentage during incidents were estimated using a model developed in a previous study. The study assumed no change in demand during rain, but the user can add the percentage change in demands, if there is evidence of such change.

The methodology described above was applied to select ramps within the study area for metering. The analysis results showed that in all the cases when the demand exceeded the capacity for the case study, metering on-ramps adjacent to the bottleneck location was not sufficient to prevent congestion, considering the constraints on the selection of the metering rates. About 13% and 30% of the conducted Monte Carlo experiments, representing days with different demand and capacity values during recurrent conditions, required the selection for metering of all 10 ramps of the case study corridor for the morning and afternoon peak periods. Moreover, the results showed that with lane blockage incidents and heavy rain during the peak hours, all 10 ramps would have to be activated in most cases. The benefits of system-wide metering were calculated in terms of delay savings, and this calculation can be used to support the decision to select a subset of the on-ramps for metering. The results were used to conduct benefit-cost analysis of the ramp metering deployment and to support associated decisions such as which ramp to meter and when to activate. Assuming the project life to be 7 years and the interest rate to be 6%, the analysis of the case study shows a benefit-cost ratio of 9.4 for ramp metering.

E4. Consideration of Acceleration Length and Queue Storage

This study extended the application of the methodology to address the variation in queue storage capacity between ramps, available acceleration length, and the related impacts of trucks. The acceleration capability of trucks is far less than that for passenger cars. This study described the use of alternative approaches to accommodate the acceleration requirements for trucks. A total of five alternatives were considered, including not metering the ramps with high truck volumes, providing the required acceleration length for a truck by compromising the queue storage length, truck preemption, providing truck by-pass lane, and diverting all truck volumes to on-ramps which can accommodate the required acceleration length for trucks, and not including the ramps that have acceleration constraints in the ramp selection procedure. The results showed that diverting the truck traffic to the ramps with sufficient acceleration lengths during metering period led to the best results in terms of the queue length and total number of the ramps to be metered. However, this option may not be feasible since it involves diverting traffic to other on-ramps and may impact adversely the operations of the arterial streets. The next a best alternative is the provision of an exclusive bypass lane, particularly if an additional lane can be added rather than taking a lane from the general traffic on the on-ramp.

E5. Real-Time Ramp Selection for Activation

Three approaches were compared for real-time activation of the ramps. The three methods used the probability of breakdown based on previous studies, which is only applicable to recurrent conditions, utilizing a “look-up table” based on the off-line Monte Carlo simulation described earlier, and using the real-world data traffic data as the inputs to the proposed linear programming formulation for execution in real-time. The three approaches resulted in similar starting time for activation. However, with regard to the number of the ramps to be metered, the results of the linear programming formulation utilization in real-time led to better adaptation of the number of the ramps to meter to the demand-to-capacity variation, compared to the look-up table. However, if an agency decides to use the look-up table method because of its simplicity of use, the results based on the 50th percentile demand-to-capacity values are recommended to be used since this produces results closer to the linear programming method.

E6. Safety Consideration in Selecting Ramps for Metering

An investigation was made in this study of models that can be used to determine the impact of ramp metering on the safety of the metered segment and potentially for use in the activation of ramp metering in real-time. A function to estimate a crash risk based on a previous study and a density and speed and ramp volume multiplication function were related to the crash rates for case study segments. Based on the results, it is recommended that ramp metering be activated when the $DS2*VR$ variable exceeds 800,000 and when the crash rate index becomes lower than -

18. This criterion for activation should be particularly considered for on-ramp merge locations with high crash rates. It is recommended by the Florida local warrants to install ramp metering at a location where the facility or roadway segment has a crash rate of over 80 crashes per hundred million vehicle miles (HMVM).

E.7. Recommendations for the Implementation of the Developed Method

As described in this executive summary, this study developed a method for ramp metering installation considering recurrent and non-recurrent conditions and also considering the truck acceleration requirements. In addition, the study recommended methods for real-time activation of ramps considering predicted mobility and safety performance. To apply these methods, transportation agencies need user-friendly tools that can be used to support the off-line and real-time installation and activation decisions, as outlined above. As part of this research, modules were developed and used in Excel and MATLAB to facilitate the various tasks of this research. These modules are not ready to use by FDOT staff and their consultants. Since the development of tools that can be used to support the off-line and real-time decisions investigated in this study is not within the scope of this project, recommendations are provided in this study for additional developments to produce the required tool.

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1. INTRODUCTION

Recurrent congestion in the peak hours and non-recurrent congestion due to incidents, work zones, weather, and/or special events significantly reduce the throughput and operating speed of the freeway. In order to better understand the impacts of congestion on traffic operation and how ramp metering can reduce these impacts, it is necessary to reference the traffic flow fundamental diagram that relates the flow, density, and speed of the uninterrupted flow on freeway facilities. As shown in Figure 1-1, when traffic density (k), defined as the number of vehicles occupying a certain space (vehicle/mile/lane), reaches the critical value k_c , the freeway flow reaches its maximum value of q_{max} , which is the segment capacity as defined in the Highway Capacity Manual (HCM). Beyond this point, increasing freeway density due to increased demands leads to a reduction in traffic flow (throughput). When the density reaches its maximum possible value, referred to as jam density (k_j), the traffic flow becomes zero and the traffic is actually stopped. Therefore, the fundamental diagram can be divided into two regions based on the value of density. If the density is lower than the critical value ($k < k_c$), the flow is stable (uncongested). When the density exceeds the critical value ($k > k_c$), the flow is unstable (congested), and capacity drops. Although q_{max} is commonly assumed as a deterministic value in traditional traffic analysis, it is really a stochastic variable, and this stochasticity is considered in this study.

Ramp metering can help to keep the density as close as possible to, but below, the critical density value, in order to prevent the reduction in flow (throughput) shown on the right side of the fundamental diagram in Figure 1-1, and thus maintain the full capacity of the freeway. It is very important to select a proper metering rate. If the metering rate is too restrictive, the ramp queue spills back to the surface street while part of freeway capacity is still unused and indeed wasted. On the other hand, if the metering rate is too permissive, the freeway congestion increases, and the capacity drops as the traffic operates in the “unstable region” (Hasan, 1999).

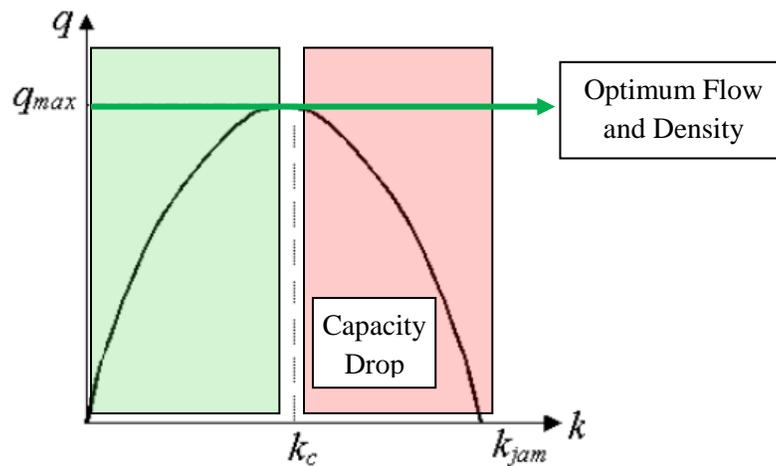


Figure 1-1 Fundamental Diagram of Flow-Density

The Federal Highway Administration (FHWA) defines ramp management as “the application of control devices such as traffic signals to regulate the number of vehicles entering or leaving the freeway, in order to achieve operational objectives” (FHWA, 2006a). Ramp metering is a ramp management strategy that regulates entering flow to freeway facilities, using traffic signals installed at on-ramps, as illustrated in Figure 1-2. Controlling the entering rate can help in smoothing the merging and thus the mainline flow and reduce congestion through maintaining the freeway capacity. Proper ramp metering can increase freeway throughput and travel speed which lead to reductions in fuel consumption and emissions. Freeway safety can also be improved by ramp metering by smoothing the traffic merge and reducing the stop and go patterns. Basically, ramp meters aim to control the entering flow to the freeway, reduce freeway demand, and breakup of the entering platoons (Balke et al., 2009). The first ramp metering in United States was implemented on the Eisenhower Expressway in Chicago in 1963. In Florida, the first ramp signal was installed on a I-95 section in Miami-Dade County in 2009 by Florida Department of Transportation, District 6 (Gan et al., 2011).

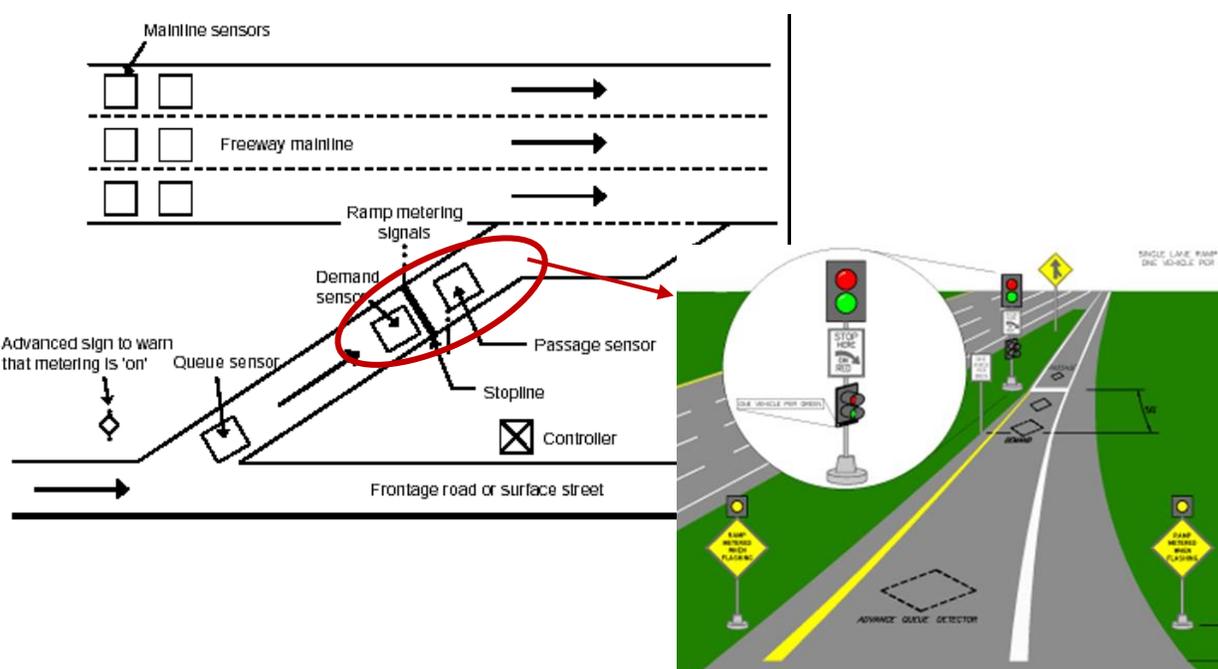


Figure 1-2 Illustration of Ramp Metering

(Source: <http://www.fhwa.dot.gov/publications/research/operations/its/06108/03.cfm>)

Many decisions that influence the performance of ramp metering need to be made when implementing ramp metering. These decisions include selecting the geographic extent of metering and the specific ramps to meter, ramp metering activation/deactivation operation strategies, metering strategy and algorithm, metering parameters, queue management, static and

dynamic signing, and potentially providing preferential treatments to traffic categories such as high occupancy vehicles and trucks. These aspects highly impact the effectiveness of this management strategy (FHWA, 2006a). Ramp metering has the most benefit where the mainline demand is not exceptionally higher than the freeway capacity and where there are several ramps that can be metered to reduce the congestion at a bottleneck (Aghdashi, 2013).

Proper ramp metering can improve safety and mobility as well as improving the environmental conditions. 5-37% reduction in crash potential has been reported as a result of ramp metering. Ramp metering also improves traffic operation by increasing travel speed and throughput, improving travel time reliability, and diverting local traffic to the arterials for short trips (Shea et al., 2015). Reducing congestion also leads to reduce fuel consumption and vehicle emissions. Overall, the benefits of ramp metering can be summarized as: traffic throughput improvement, travel time and travel time reliability improvements, safety benefits by reducing crash rates, environmental benefits by reduction in fuel consumption and vehicle emission (Gan et al., 2011).

Despite all the advantages of ramp metering, it may cause long queues on the ramps and spillbacks to upstream intersections. This needs to be addressed by metering rate adjustments. Moreover, the ramp metering may cause traffic diversion to parallel routes which can result in congestion in the network. However, a successful metering approach can overcome these disadvantages by better selection of the parameters of the implementation (Jacobs Engineering Group, Inc., 2013). It is also crucial to select metering locations to realize and maximize the benefits of ramp metering (Hasan, 1999).

To accomplish this, transportation agencies have developed guidelines and warrants to support the decisions to implement ramp metering during the planning stage and procedures to activate/deactivate the meters in real-time operations. The real-time activation decisions have been generally done based on time of day combined with decisions made by operators at the traffic management centers according to their observations of the traffic conditions. The existing warrants consider local conditions of the subject ramp and freeway mainline in the vicinity of the on-ramp merge area. However, the most advanced applications of ramp metering utilize system-based metering algorithms that involve metering a set of on-ramps to address a single system bottleneck location. These algorithms have been proven to perform better compared to local ramp metering algorithm. This has created a disconnection between existing agency metering warrants based on local conditions and the subsequent operations of the ramp meters based on system bottleneck consideration.

Moreover, transportation system management and operation agencies have realized the need to activate metering during non-recurrent events like incidents and adverse weather conditions. Some of these agencies have assigned operators to activate ramp metering during non-recurrent conditions based on their observations of traffic conditions using closed circuit television (CCTV)

cameras. Previous studies showed that non-recurrent conditions contribute significantly to the congestions and unreliability of the transportation system. However, the existing ramp metering warrants do not consider non-recurrent events as a contributing factor.

To address the above mentioned gaps, the goal of this project is to develop a system-wide-based method to be used, in conjunction with existing local warrants for the offline identification of ramp metering installation locations and a method for on-line activation of ramp metering. The objectives of this project are as follows:

- To develop system-based methods for ramp metering installation considering recurrent conditions in the planning stage.
- To develop system-based methods for ramp metering installation considering recurrent conditions in the planning stage.
- To extend the developed offline system-based ramp metering method for the real-time selection of the ramps for activation.

The remaining of the document is organized as follows.

- The information gathered from the interviews with FDOT District 4 and District 6 transportation system management and operations (TSM&O) and traffic management center staff regarding the ramp metering implementation and operations are discussed in Chapter 2.
- A review of literature on existing warrants, metering algorithms and methods, and other related topics is presented in Chapter 3.
- The details of the collected data from different sources and initial analysis are presented in Chapter 4.
- An assessment is presented in Chapter 5 to demonstrate the need for system-based method in addition to local warrants for ramp metering installation.
- The method developed in this study for offline selection of on-ramps for metering based on system-wide conditions of the freeway facility is described in Chapter 6.
- An extension of the developed formulation to consider the variation in queue storage capacity between ramps, available acceleration length, and the related impacts of trucks are presented in Chapter 7.
- Methods for on-line activation of ramp meters under recurrent and non-recurrent conditions are presented in Chapter 8.
- The consideration of safety impacts are discussed in Chapter 9.
- Recommendations for the implementation of the method developed in this study is presented in Chapter 10.

2. PROJECT NEEDS AND LESSONS LEARNED BASED ON MEETING WITH FDOT DISTRICTS

As part of this project, interviews were conducted with FDOT District 4 and District 6 TSM&O and traffic management center staff. Below is a summary of the discussion.

2.1 Meeting with FDOT District 4

Ramp meters are expected to be deployed by FDOT District 4 on over 60 I-95 ramps in Broward County and Palm Beach County, FL in the next 18-24 months. 2070 Lite controllers and loop detectors will be installed to support this ramp metering deployment with the detection technology being able to classify vehicles, in case this information is needed. As part of the planning and design of this ramp metering deployment, a FDOT consultant (AECOM) utilized the ramp metering warrants developed for Florida and discussed later in this document. The results show that some of the ramps do not meet the acceleration lane criteria that requires the distance between the stop bar and merge point (end point of the taper) to be higher than the safe merging distance. A decision was made to instrument all ramps with meters since the whole corridor is congested.

Based on the discussions with FDOT D4 personnel, the ramp metering implementation criteria that will be developed in this project should consider the dynamics of traffic flow based on detailed traffic and incident data. The newly developed criteria should not replace the existing simple criteria but should be an addendum or extension to it. The final method should be simple, easy to use, and easily understood by the user, although the analysis behind it can be at the level of complexity needed to produce effective warrants.

An important issue that needs to be addressed as part of this project is whether a single ramp or a group of ramps-based metering (that is, zone-based metering) is warranted under given conditions and should be implemented. In addition, if zone metering is warranted at a given location, a question that needs to be answered is whether the ramps should and can be assigned to the zones dynamically based on identified conditions. The interaction between managed lanes and ramp metering through the impacts of toll value and metering rate on traffic conditions may also be an issue. Another major concern is the acceleration of heavy vehicles from stop on ramps, particularly with ramps that have slopes and high percentages of heavy vehicles. This may warrant “bypass lanes for heavy vehicles” or “ramp signal preemption for heavy vehicles”. Safety may also be a consideration as part of the developed method.

2.2 Meeting with FDOT District 6

Currently, FDOT District 6 is operating ramp meters between Ives Dairy Road and NW 62th Street in Miami-Dade County, FL. The ramp meters were installed in the year 2000 but did not

start operation until 2009. District 6 has already scheduled a ramp metering deployment on SR 826 from SR 836 to NW 154th St. The district will also consider metering the ramps south of the current deployment on I-95 and on the SR 826 E-W segment. As with the deployment in District 4 described above, District 6 decided to install the ramp meters on all the ramps within the extent of a deployment. This does not mean that these meters will be necessary activated every day but it will give the FDOT the flexibility of activating the meters whenever they needed such as during incidents and special events.

The ramps are normally activated in groups and they are operated as coordinated since the downstream detectors at the bottleneck congestion are specified as detectors to be used in the fuzzy logic-based metering rate estimation. Normally, the intersections south of NW 95th Street along the I-95 northbound are activated around 2:45 PM and the ones north of it are activated at 3:30 PM based on analyzing historic data. These times have been selected when historical patterns indicate that the speed drops below 50 mph are sustained for a period of time. The operators normally also watch the traffic conditions utilizing closed circuit television (CCTV) cameras and make decisions to activate the ramps when conditions are bad such as during incidents. The activations under these conditions are based on judgment. The district feels that it will be very useful if a decision support system can be available to semi-automate the ramp metering activation process by alerting the operators of potential breakdown conditions. This is particularly important as more ramps are installed in a region making the operator task to manually making decisions more difficult.

FDOT District 6 has not observed any problems with the acceleration of vehicle from stop to merge with traffic. This possibly indicates that the related warrants are over conservative. FDOT District 6 agrees that it is possible to locate the stop lines for the ramp meters on the on-ramp some distance ahead of the gore to increase the acceleration distance. However, this should be done with the consideration of queue spillback due to shorten the queuing storage. Local agencies are usually very concerned about the impacts of backups from the on-ramps to upstream signals. In this regards, it is useful to coordinate the upstream signal timing with ramp metering to reduce the turns from the upstream signals to the on-ramps and thus to reduce the probability of queue backup. This is easier to implement with the 2070 controllers compared to the 170 controllers currently used in FDOT District 6.

Truck presence on ramps has not been observed to be a problem in District 6. FDOT District 6 feels that it may be difficult to implement “truck preemption” that deactivates the ramp meter until the passage of the truck. However, it may be useful to give a longer green time when a vehicle is classified as a truck by the detection system. It is also useful to extend the ramp signal's red time after a truck passing the signal to provide enough time for truck to clear the acceleration lane before another vehicle merge behind the truck.

Currently there is no coordination with managed lane operations when setting ramp metering. However, it may be useful to do this for example by having more restrictive metering when the managed lanes are congested to allow more vehicles to use the general purpose lanes.

There is a need for better documentation of ramp metering performance measures based on historical data. Currently, there is a limited documentation of the performance of the meters based on data. However, a dashboard is produced for each ramp and displayed on a web page at http://www.sunguide.info/sunguide/index.php/road_stats/ramp_signals. An example of these dashboards is shown in Figure 2-1. The current version of SunGuide software has a ramp metering module (<http://sunguidesoftware.com>). However, it does not archive the information such as metering rate. The new version of SunGuide will archive ramp metering parameters, specifically the ramp metering rates implemented by the controller. There is a need to archive other parameters such as spillback alerts and mainline conditions.

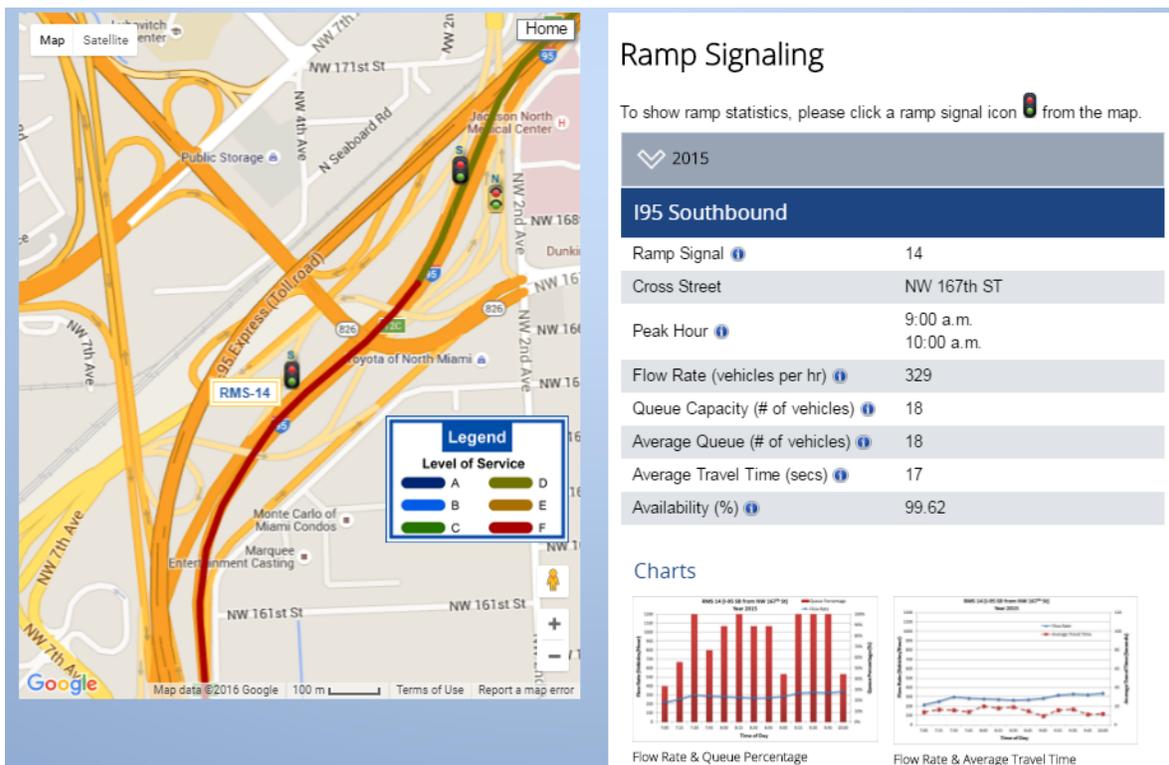


Figure 2-1 SunGuide Ramp Metering Dashboard

The parameters of the fuzzy logic algorithm were set by the ramp metering deployment consultants when the ramp meters were first set to operate. These parameters have not been changed by District 6. The parameters that are used to control the metering are the specified minimum and maximum rates. These rates are set by trials and errors to ensure no queue overflows on the ramps and good performance.

District 6 does not have a set of criteria for activating ramp meters during special events and incidents. The operators use their judgements to decide that an incident is causing congestion and usually activate at least three ramps in a group. It would be useful to have tools that supports proactive decisions for ramp activation such as predicting traffic breakdown and the impacts of weather and incidents on the traffic conditions.

District 6 emphasizes the importance of documenting the before conditions including the backups from the ramps to the arterials. This documentation is necessary for example to show whether the ramp metering is really the cause of the backups on these ramps or that the backup was there even before installing the meters.

3. LITERATURE REVIEW

3.1. RAMP METERING WARRANTS

As stated earlier, with all the advantages of ramp metering, not all freeway sections can benefit from this traffic management strategy. Thus, there is a need to determine the conditions, under which ramp metering strategy is suitable for improving traffic condition. Agencies have produced guidelines to support the decisions to implement ramp metering. In this chapter, warrants developed by different jurisdictions are introduced.

3.1.1. Manual on Uniform Traffic Control Devices (MUTCD)

The MUTCD 2003 recommended the consideration of ramp metering implementation, if at least one of the following conditions is met (FHWA, 2003):

“1. Congestion recurs on the freeway because traffic demand is in excess of the capacity, or a high frequency of crashes exists at the freeway entrance because of an inadequate ramp merging area. According to the MUTCD, a good indicator of recurring freeway congestion is a freeway operating speed less than 50 mph occurring regularly for at least a half-hour period. Freeway operating speeds less than 30 mph for a half-hour period or more would indicate severe congestion.

2. Controlling traffic entering a freeway assists in meeting local transportation system management objectives identified for freeway traffic flow, such as the following:

- A. Maintenance of a specific freeway level of service.
- B. Priority treatments with higher levels of service for mass transit and carpools.
- C. Redistribution of freeway access demand to other on-ramps.

3. Predictable, sporadic congestion occurs on isolated sections of freeway because of short period peak traffic loads from special events or from severe peak loads of recreational traffic”.

The MUTCD 2009 references the FHWA’s Ramp Management and Control Handbook as the guideline for ramp metering warrants and eliminates the above mentioned warrants (FHWA, 2009). However, the FHWA’s handbook does not specify any quantitative criteria for considering ramp metering and generally mentions safety, congestion, convenience, ramp capacity and queues, access and adjacent facility operations as the indicators for considering ramp management strategies (FHWA, 2006a).

3.1.2. Arizona

The Arizona Department of Transportation (ADOT) identified two ramp metering warrants. The installation of a ramp meter is warranted if both warrants are satisfied. In addition, the acceleration and ramp queue storage distance must be acceptable to recommend a ramp meter. The two warrants are:

1. Freeway right lane and entrance ramp flow rate: During a typical 15-minute period, the combined flow rate of the entrance ramp and the rightmost freeway lane is greater than 2,050 vehicles per hour, and during the same period, the entrance ramp flow rate is greater than 400 vehicles per hour.
2. During a typical 15-minute period, the speed of the freeway general purpose lanes (not including HOV, auxiliary, and entrance ramp lanes) is less than 50 mph due to recurring congestion adjacent to or within 2 miles downstream of the entrance ramp (Simpson and Yasmin, 2013).

3.1.3. California

The California Ramp Meter Design Manual suggests considering “customization” for each specific case when using any design advices or guidelines. Given this suggestion, the following warrants are recommended by this manual:

- A single lane ramp meter should be geometrically designed for volumes up to 900 vph. Where truck volumes (three axles or more) are 5% or greater on ascending entrance ramps to freeways with sustained upgrades exceeding 3% (at least throughout the merge area), a minimum 500-ft length of auxiliary lane should be provided beyond the ramp merge area. If the volume exceeds 900 vph and/or when a High Occupancy Vehicle (HOV) lane is determined to be necessary, a two- or three-lane ramp meter should be provided.
- Freeway-to-freeway connectors may also be metered when warranted. The need to meter a freeway-to-freeway connector should be determined on an individual basis.
- Storage length for ramp meters have practical lower and upper output limits of 240 and 900 vph per lane, respectively. Ramp meter signals set for flow rates outside this range tend to have high violation rates and cannot effectively control traffic. Therefore, on a ramp with peak-hour volume between 500 and 900 vph per lane, a two-lane ramp meter may be provided to double the vehicle capacity stored in the available storage area. A single-lane ramp meter should be used when rates are below 500 vph and no HOV preferential lane is provided.
- Ramp meter installations should operate in conjunction with, and complement, other transportation management system elements and transportation modes. As such, ramp

meter installations should include preferential treatment of carpools and transit riders. Specific treatment(s) must be tailored to the unique conditions at each ramp location; however, the standard or base treatment upon which other strategies are designed is the HOV preferential lane. An HOV preferential lane shall be provided at all ramp meter locations (Caltrans, 2000).

3.1.4. Colorado

The Colorado Department of Transportation (CDOT) considers three conditions for ramp metering to be warranted. The first condition is the total of upstream mainline volume and ramp volume exceeding defined thresholds as follows:

- 2,650 vph for two mainline lanes
- 4,250 vph for three mainline lanes
- 5,850 vph for four mainline lanes

The second condition considers ramp volumes of up to 900 vph for single lane ramps and above 900 vph for two lane ramps. The third condition is a descriptive warrant which recommends field observation and experience to justify ramp metering for the considered locations (Gaisser and DePinto, 2015).

3.1.5. Florida

Seven warrants were developed in Florida that consider traffic operation, geometric and safety criteria. According to these warrants, ramp metering is warranted if following conditions are met (Gan et al., 2011):

Warrant 1- Mainline Volume: The average mainline volume during the peak hour is higher than 1,200 vphpl.

Warrant 2- Mainline Speed: The average mainline speed during the peak hour is less than 50 mph.

Warrant 3- Ramp Volume: Considering the ramp volume criteria, ramp metering is warranted when:

- Peak hour on-ramp volume is between 240 and 1,200 vph for single lane ramps.
- Peak hour on-ramp volume is between 400 and 1,700 vph for ramps with multiple lanes.

Warrant 4- the Sum of Mainline and Ramp Volumes: Considering the mainline and ramp volumes, ramp metering is warranted when the total of the mainline and ramp volumes during the peak hour exceeds the following thresholds:

- For two mainline lanes, the total volume is greater than 2,650 vph.
- For three mainline lanes, the total volume is greater than 4,250 vph.
- For four mainline lanes, the total volume is greater than 5,850 vph.
- For five mainline lanes, the total volume is greater than 7,450 vph.
- For six mainline lanes, the total volume is greater than 9,050 vph.
- For more than six mainline lanes, the total volume is greater than 10,650 vph.

The number of lanes in the above is for one direction including the auxiliary lane(s) which continue for at least 1/3 mile downstream from the ramp gore.

- Ramp metering is also warranted when the peak hour volume of the rightmost lane exceeds 2,050 vph.

Warrant 5- Ramp Storage: Ramp metering may be warranted when the ramp storage length is greater than the queuing length. In order to estimate the queuing length, the following equation can be used:

$$L = 0.25 V - 0.00007422 V^2 \quad (3-1)$$

where:

L= required single lane storage length (m)

V= Peak hour ramp demand (vph)

Warrant 6- Acceleration Distance: Ramp metering may be warranted where the acceleration distance after the stop bar is longer than the required safe merging distance estimated by the following equation:

$$L = 0.14 V^2 + 3 V + 9.21 \quad (3-2)$$

where:

L= required minimum acceleration distance (ft)

V= freeway mainline prevailing speed (mph)

Warrant 7- Crash Rate: Unlike the previous six warrants which are applicable to individual ramp locations, the crash rate warrant can only be applied to a facility or roadway segment with multiple ramps. Ramp metering is warranted at a location where the facility or roadway segment has a crash rate of over 80 crashes per hundred million vehicle miles (HMVM). RHMVM is calculated using the following equation:

$$RHMVM = \frac{\text{Number of crashes per year} \times 100,000,000}{AADT \times 365 \times \text{Distance}} \quad (3-3)$$

where:

RHMVM= crash rate per hundred million vehicle-miles

AADT= Average Annual Daily Traffic on the facility (vpd)

Distance= length of roadway segment (mile)

3.1.6. Nevada

The Nevada Department of Transportation (NDOT) introduced nine warrants for ramp metering installation as follows (Jacobs Engineering Group, Inc., 2013):

Warrant 1- Ramp Volume: The minimum ramp volume during the critical peak hour is higher than 240 vphpl.

Warrant 2- Crash Rate: If the crash rate within 500 ft of the ramp gore point is higher than the mean crash rate for similar sections on freeways in the metropolitan area.

Warrant 3- Speed: If for 200 or more days per year, the freeway operates at speeds lower than 50 mph for at least 30 minutes.

Warrant 4- Level of Service: If the peak period level of service for the freeway is worse than level D.

Warrant 5- Mainline Volume and Ramp volume: If the total peak period mainline volume (excluding the managed lanes volume) downstream of the gore exceeds 2,650 vph for two mainline lanes, 4,250 vph for three mainline lanes, 5,850 vph for four mainline lanes, 7,450 vph for five mainline lanes, 9,050 vph for six mainline lanes and 10,650 vph for more than six mainline lanes in one direction.

Warrant 6- Mainline Right Lane Volume and Ramp Volume: If the ramp volume plus mainline right lane volume downstream of the gore is more than 2,100 vph during the peak hour.

Warrant 7- Platoons from Signalized Intersections: If the hourly volume entering from arterials, based on the highest 30-second volume readings (during the critical peak period) projected to hourly values, greater than 1,100 vph.

Warrant 8- Acceleration Length: If the available acceleration length after the stop bar is longer than the required acceleration length. The acceleration length must accommodate enough distance between the stop bar and the convergence point of ramp and freeway to

allow vehicles on the ramp to accelerate within 5 mph of the freeway's operating speed. Providing inadequate acceleration length is not allowed under any circumstance.

Warrant 9- Ramp Storage Length: If the available ramp storage length is greater than the estimated queuing length on the ramp. The queuing length is calculated using the following steps.

- Step 1- Obtain 20 year projected peak hour ramp demand volume.
- Step 2- Assume the peak hour factor of 0.8 and calculate 140 seconds arrival rate.
- Step 3- If the ramp peak hour volume is between 240-800 vph consider single lane ramp. If the volume is higher than 800 vph consider two lane ramp.
- Step 4- Assume discharge rate of 31 vehicles per 140 seconds for single lane ramp and 62 vehicles per 140 seconds for two lane ramps. Subtract discharge rate from the arrival rate calculated in step 2 to determine the excess number of vehicles per 140 seconds.
- Step 5- Calculate total queue length by multiplying the excess vehicles by a vehicle spacing of 30 feet.
- Step 6- Calculate queue length per lane by dividing the calculated total queue length by the number of lanes.
- Step 7- The required storage lane length is the calculated queue length per lane plus the minimum storage length. This minimum storage length accounts for platoons of vehicles arriving at the ramp meter. Required minimum storage lengths is 480 ft per lane for single and two lane ramps and 510 ft for three lane ramps.

The calculated storage length is to be rounded up to a multiple of 30. Additional storage must be provided if there is significant number of trucks, buses, or RVs using the ramp. The flowchart in Figure 3-1 is provided by NDOT for the application of the aforementioned warrants.

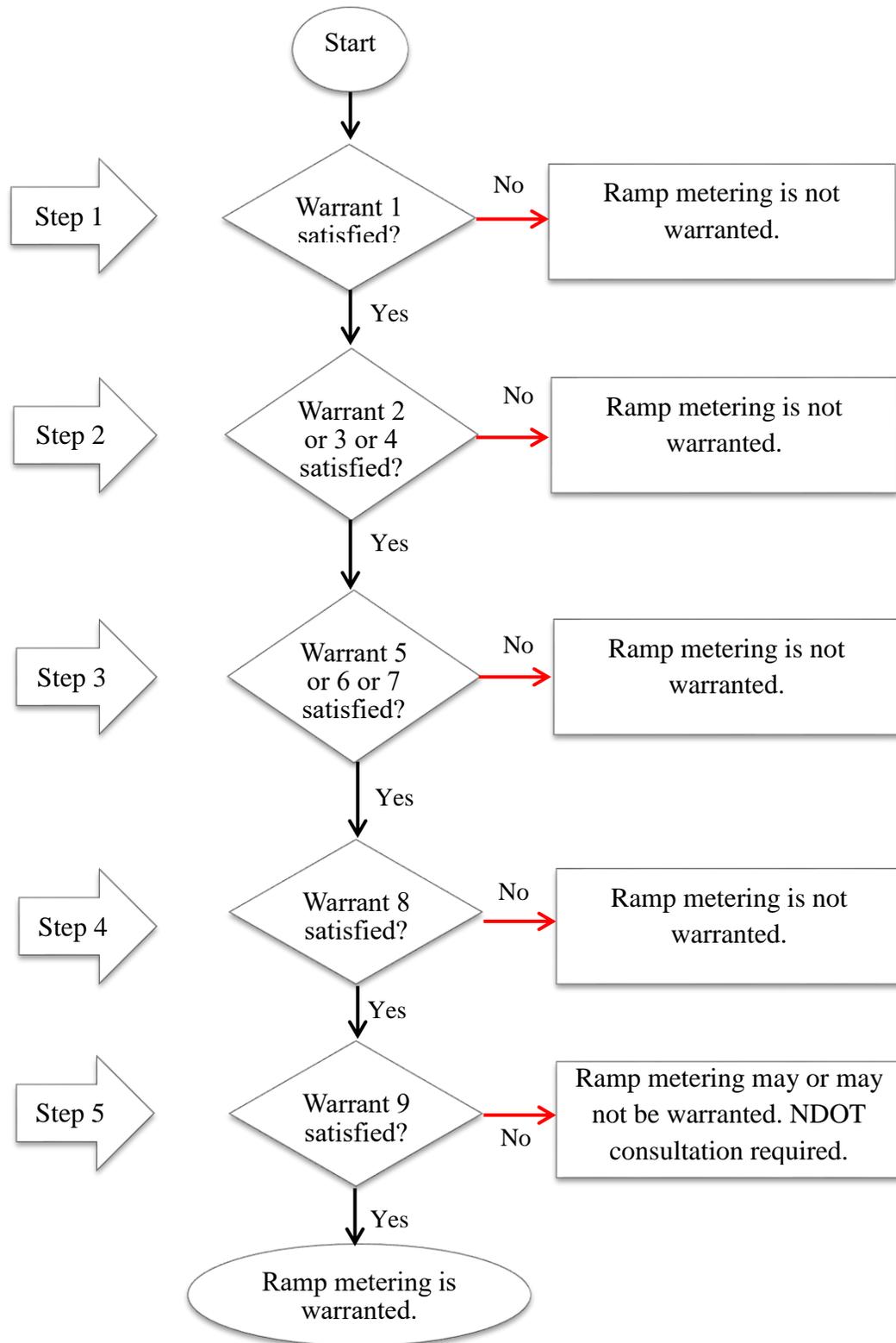


Figure 3-1 NDOT Ramp Metering Warrant Analysis (Jacobs Engineering Group, Inc., 2013)

3.1.7. New York

The New York State Highway Design Manual adapted the following ramp metering warrants from the National Cooperative Highway Research Program (NCHRP) Report 155:

- Freeway operating at a level of service worse than level D.
- Adequate parallel surface routes must be available for the traffic diverted from the ramps.
- Sufficient ramp storage capacity must be available to prevent queues of vehicles waiting to enter the freeway from blocking local street circulation.

The New York State Department of Transportation (NYSDOT) recommends the following value for the ramp metering rate:

- One lane ramp: metering rate range of 240-900 vph.
- Two lanes ramp: metering range from a low of 400 vph to a high of 1,500-1,800 vph (Gan et al., 2011).

3.1.8. Texas

The Texas Manual on Uniform Traffic Control Devices (TMUTCD) follows the ramp metering warrants of MUTCD 2003. However, the Texas Department of Transportation (TxDOT) introduced the following ramp metering warrants in 2009:

- The freeway regularly operates at speeds of less than 50 mph for at least a half-hour period during the day (presumably during the peak-period).
- The ramp sustains a minimum flow rate of at least 300 vph during the peak-periods.
- The average traffic flow rate of the two right-most lanes during the peak-periods exceeds 1,600 vphpl for entrance ramps that have acceleration lanes of 500 feet or less. This threshold level increases as the length of the acceleration lane on the ramp increases.
- The combined traffic flow rate in the rightmost freeway lane plus the flow rate on the entrance ramp during the peak-periods exceeds a minimum of 2,300 vphpl for entrance ramps with acceleration lanes of 500 feet or less. The threshold level increases as the length of the acceleration lane on the ramp increases.

From a safety perspective, TxDOT recommends that ramp metering application can be justified based on the following three criteria:

- The rate of crashes in the immediate vicinity of the ramp exceeds the mean crash rate for comparable freeway sections in the metropolitan area.
- The acceleration distance permits a vehicle starting from a stop at the signal to reach the prevailing speed of the freeway traffic in the merge area so as to prevent an unacceptable speed differential in the merge area. The interacting ramp and freeway traffic vehicles

must be able to maintain a desirable time to collision (TTC) after the merge. A TTC value lower than a specified threshold indicates an unsafe merge condition at the ramp meter. Sufficient storage length exists upstream of the ramp control signal to prevent queues from impeding operations on the frontage road or surface street intersection (Balke et al., 2009).

3.1.9. Utah

The Utah Department of Transportation (UDOT) specifies thresholds for the total of the mainline and ramp volumes and the ramp volumes by itself to justify the ramp metering. The total mainline and ramp volume threshold is dependent of number of lanes on the freeway. While, the ramp volume threshold depends on the number of lanes on the ramp and the percentage of High Occupancy Vehicles (HOV) (Gan et al., 2011). The thresholds are represented in Table 3-1 and 3-2.

Table 3-1 UDOT Total Mainline and Ramp Volume Thresholds (Gan et al., 2011)

Number of Lanes	Total Mainline and Ramp Volume
2	2,650
3	4,250
4	5,850
5	7,450
6	9,050
7	10,650

Table 3-2 UDOT Ramp Volume Thresholds (Gan et al., 2011)

Ramp Volumes	HOV (%)	Recommended Lane Configuration
<180	-	Signaling not recommended
180~600	-	Single lane metered ramp
600~900	<10%	Single lane metered ramp
600~900	>10%	Single lane metered ramp, or Two lane ramp with one lane metered and one HOV lane
900~1080	<10%	Two lane ramp with both lanes metered
900~1,080	>10%	Two lane ramp with both lanes metered, or Two lane ramp with one lane metered and one HOV lane
1,080~1,350	<10%	Two lane ramp with both lanes metered
1,080~1,350	>10%	Two lane ramp with both lanes metered, or Three lane ramp with two lanes metered and one HOV lane
1,350~1,720	<10%	Three lane ramp with all lanes metered
1,350~1,720	>10%	Three lane ramp with all lanes metered, or Three lane ramp with two lanes metered and one HOV lane
>1,720	-	Consider alternate metering strategies, or no metering

3.1.10. Virginia

The Virginia Transportation Research Council specifies the following warrants to justify ramp metering (Gan et al., 2011). :

- The freeway has poor traffic flow conditions in the peak-periods, such as speeds of less than 30 mph, low throughput per lane, levels of service of E or F, and stop-and-go traffic.
- There are numerous crashes on the freeway, especially in the on-ramp weaving areas.
- There are obvious merging problems occurring at freeway on-ramps.
- Heavy traffic volumes at closely spaced on-ramps.
- Feasible metering rate can accommodate the ramp demand volumes from both a maximum and minimum standpoint.
- There is adequate vehicle storage on the ramp.
- A freeway management system is being planned.

3.1.11. Wisconsin

The Wisconsin Department of Transportation (WisDOT) ramp metering warrants are (Wilbur Smith Associates, 2006):

- Mainline volume flow rates of at least 1,200 vphpl (approximately 20% to 30% occupancy) coupled with slow moving traffic along the freeway lanes.
- Ramp volume criteria — The ramp should have volumes of at least 240 vph (400 vph for two lanes).
- Speed criteria — 30 mph or less is the common minimum freeway speed to warrant ramp metering.
- Safety criteria — A reduction in crashes at the merge should be expected. Crash rates in the vicinity of the ramp of 80 or more per hundred million vehicle-miles of travel are recommended, as a starting point for further analysis.
- Ramp geometric criteria — These include queuing storage space, adequate acceleration distance and merge area beyond the meter, and sight distance.
- Funding criteria — An evaluation of potential funding sources should be completed to determine if there is sufficient support for the project.
- Alternate route criteria — The presence of an alternative route for motorists to avoid delays on entrance ramps created by a ramp meter may be required.
- Corridor criteria — In most implementations, ramp metering is addressed at the corridor level. It must be determined whether the section under consideration is part of a corridor.

3.1.12. Summary of Existing Warrants

Ramp metering installation warrants have been developed by a number of states, as reviewed in this report. As with the warrants developed in Florida, these warrants are generally simple and examines the conditions in the immediate vicinity of each ramp (local conditions) to determine if a ramp meter is warranted. Some states have fewer warrants like Arizona that has only two warrants: one based on mainline and ramp volumes and the other based on speed. Other states like Nevada and Florida have more warrants that also consider safety, acceleration lane, ramp queue storage, level of service, platoons from intersections, and the availability of diversion routes. The volume warrants vary significantly. For example, the Arizona warrants specify 2050 veh/mile/lane as a value to consider ramp metering, which appears to be a rough estimate of freeway lane capacity. However, other warrants specify much lower volumes that appear to reflect a rough estimation of the volumes that produce level of service D. In fact, some warrants specify level of service D as a criterion to install ramp metering. The Wisconsin warrants specify mainline volume flow rates of at least 1,200 vphpl with approximately 20% to 30% occupancy and slow moving traffic along the freeway lanes. This warrant is confusing in that the specified volume is much lower than capacity, while the specified occupancy is high. It is not clear if the specified volume is the demand or the capacity constrained volume that results in congestion.

The speed warrants in some states like Arizona and Nevada specify speed drops to less than 50 mph for half an hour without regard to the speed of the freeway under free flow conditions, possibly again reflecting level of service D. Again, some warrants specify level of service D as a criteria. On the other hands, other warrants like the Virginia warrants specify speed drops below 30 mph and level of service E and F for the consideration of ramp metering. For the safety warrants, quantitative values were included in the Wisconsin and Florida warrants (80 or more per hundred million vehicle-miles of travel). In addition, TTI recommended the use of a surrogate measure to safety, which is the desirable time to collision (TTC) after the merge. Table 3-3 summarizes the discussed warrants in this section.

Table 3-3 Summary of Ramp Metering Warrants

Warrant Category	Criteria	States
Traffic Criteria	Mainline volume	Minnesota, Texas, Washington, Wisconsin
	Ramp volume	California, Colorado, Nevada, New York, Oregon, Texas, Utah, Wisconsin
	Mainline and ramp volume	Arizona, Colorado, California, Texas, Utah
	Volume/capacity	California, Wisconsin
	Mainline speed	Arizona, California, Minnesota, Nevada, Texas, Virginia, Wisconsin
	Level of service	Nevada, New York, Virginia
	Occupancy/density	New York, Wisconsin
Geometric Criteria	Ramp storage	Nationwide
	Acceleration lane length	Nationwide
Safety Criteria	Crash rate around the merge point	Nationwide

3.2. Ramp Metering Strategies

Ramp metering strategies can be classified into different categories depending on geometric characteristics, activation and controlling algorithms as shown in Figure 3-2.

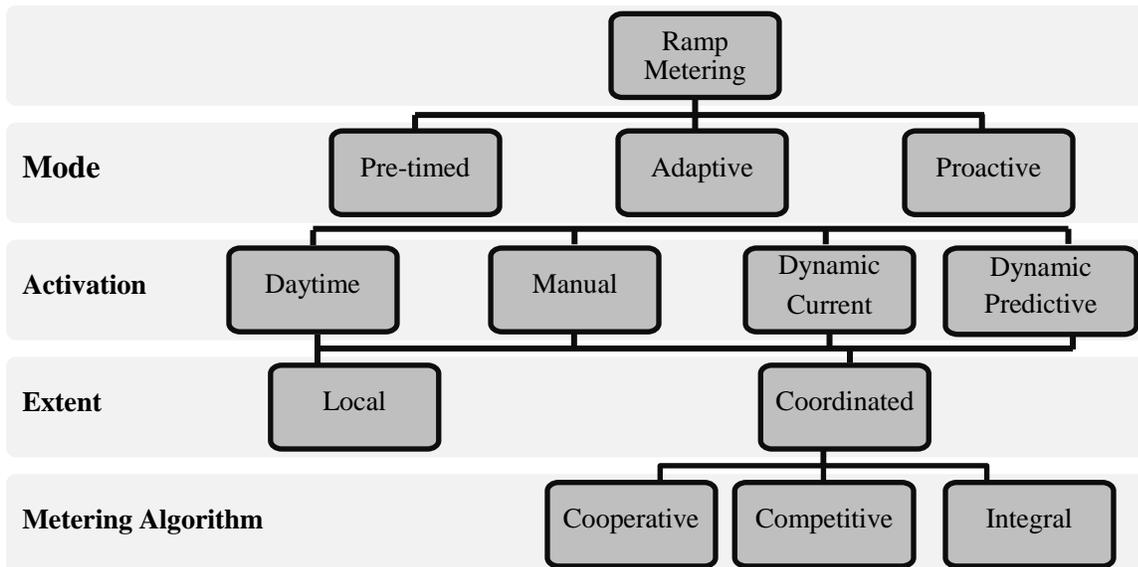


Figure 3-2 Ramp Metering Categories (Kristeleit, 2014)

3.2.1. Metering Rate Selection Mode

Metering rate calculation can be static, adaptive or pro-active. In static (time of day) control, the metering rate is calculated using historical data, assuming that the traffic always follows the average recurrent conditions. Adaptive (or traffic responsive) ramp control set the ramp metering rates utilizing the actual mainline and ramp traffic measurements to calculate the metering rate, which is compatible with current conditions. This mode of control can address non-recurrent congestion, as well as recurrent traffic congestion, if so set by traffic agencies. In the pro-active (predictive) mode of control, the operation parameters are predicted using real time data to prevent oversaturated conditions and traffic breakdown. Compared to static metering, both adaptive and pro-active metering strategies require the deployment of traffic detectors along the freeway mainline as well as on-ramps.

3.2.2. Metering Activation Strategies

Ramp metering can be activated either based on a daytime schedule, or manually, or dynamically according to current or predicted traffic conditions. The simplest is the daytime operation based on a fixed schedule (e.g., every day at 4:00 pm in the northbound direction). The manual strategy is for an operator to watch traffic conditions (using CCTV cameras) and make changes

in reaction to real time traffic condition. The dynamic metering strategy implements an automated method that utilizes current traffic measurements (or predicted traffic conditions) to prevent breakdown and congestion. Non-recurrent traffic conditions such as incidents can also trigger ramp metering activation. As the metering activation strategies become more advance and automated, the need for manual intervention is less; however, it requires more deployment of traffic field devices and more computational power.

3.2.3. Ramp Metering Extent and Associated Algorithms

Depending on the number of ramps considered in the ramp metering algorithm, metering strategies can be identified as a local algorithm that considers one ramp as an isolated element or a coordinated system (system-wide) formed by multiple ramps. The objective of system-wide (coordinated) metering is to optimize the metering rate of each ramp in order to improve the system-wide traffic conditions. The objective function may differ based on different policies of each agency. In both local and coordinated metering, measurements of macroscopic traffic parameters on the freeway mainlines (flow, speed, and occupancy) and on-ramp queue lengths are considered in the metering rate selection process to avoid freeway traffic breakdown and queue spillbacks to arterials. However, the system-wide (coordinated) metering requires detectors located on ramps and along the entire metering section, including the off-ramps and unmetered ramps as well as freeway mainline. While local metering algorithms, either fixed time or traffic-responsive only, require the detectors located at the vicinity of ramp area (the subject on ramp and freeway mainline). Moreover, local metering strategies do not require communication to TMC, while system-wide (coordinated) metering algorithms require this communication type. Failure in communication to TMC or central computer failure will affect the operation of system-wide metering. Coordinated metering is more effective since it can prevent or delay traffic breakdown at a location by metering multiple upstream ramps rather than relying on metering the ramp immediately upstream of the bottleneck, which may not be sufficient. This section discusses briefly traffic-responsive ramp metering algorithms that have been used to select ramp metering rates. It is important to point out that this review was done, not only to provide background materials for the activities of this project, but to identify concepts that could be adopted for use as part of the methodology for identifying the warrants in this project. The review below confirmed that some of the concepts in the reviewed algorithms could be used for the purpose of this study. Tables 3-4 and 3-5 present comparisons of ramp metering algorithms.

Table 3-4 Comparison of Ramp Metering Algorithms (Cambridge Systematics, Inc., 2001)

Capabilities & Requirements	Pre-timed		Traffic Responsive	
	Local	Coordinated	Local	Coordinated
Solve local or system wide issue	Local	System wide	Local	System wide (Most useful for corridor applications)
Field detection devices	No	No	Yes	Yes (Both upstream and downstream)
Communication devices	No	No	No	Yes
Maintenance	Periodic manual update		High capital and maintenance costs	Highest capital and maintenance costs; Requiring calibration and implementation by expertise
Effectiveness for recurrent condition and/or nonrecurrent conditions	Recurrent congestion	Recurrent congestion	Both recurrent and non-recurrent conditions	Both recurrent and non-recurrent conditions
Optimization of traffic conditions	Over-restrictive metering rates	Over-restrictive metering rates	Reactive improvement and not optimization of downstream traffic conditions	Optimal metering rate based on real-world conditions

Table 3-5 Comparison of Ramp Metering Algorithms (Atkins, 2013)

Ramp Meter Type	Description	Required resources	Advantages	Disadvantages
Fixed Time	<ul style="list-style-type: none"> Activated based on time of day, Time of day is based on historical or predicted volumes. 	<ul style="list-style-type: none"> Only vehicle detection on the ramp for queue management or to actuate and terminate the metering cycle. Traditionally, detection has been implemented in the form of induction loops. 	<ul style="list-style-type: none"> Simple Is able to operate even in temporary lack of communications, detectors malfunction or construction. 	<ul style="list-style-type: none"> Does not respond to real-time traffic condition. Does not respond to non-recurrent traffic conditions such as incidents and adverse weather. Does not consider the whole system.
Local Traffic responsive	<ul style="list-style-type: none"> Metering rate is adjusted based on current conditions of freeway condition at the vicinity of ramp area. 	<ul style="list-style-type: none"> Demand detectors: located just upstream from the stop bar, detect the presence of a vehicle at the ramp meter and initiate the ramp metering cycle. Passage Detectors: located just downstream from the stop bar to detect and count the number of vehicles entering the freeway (used to determine the duration of green time) Ramp queue detectors: located near the ramp intersection and the adjacent surface street. Mainline: located upstream of the entrance ramp gore point. 	<ul style="list-style-type: none"> Responses to real-time traffic conditions at the vicinity of the ramp area. Does not require communication to TMC. 	<ul style="list-style-type: none"> Does not respond to traffic conditions in the rest of system.
System-wide Traffic Responsive	<ul style="list-style-type: none"> The metering rate of each ramp is optimized in order to improve system-wide conditions. 	<ul style="list-style-type: none"> Demand detectors, Passage Detectors, Ramp queue detectors, (similar to local algorithms). Mainline: located upstream of the entrance ramp gore point. System-wide metering operations can use mainline detectors downstream of ramps. Off-ramp passage detectors Unmetered on-ramp Passage detectors 	<ul style="list-style-type: none"> Responses to real-time traffic conditions throughout the system. Is able to prevent bottleneck. Has the most potential benefits among all metering operations. 	<ul style="list-style-type: none"> Requires communication to TMC. Has potential to favor some ramps over others, creating inequity issues.

3.2.3.1. Local Ramp Metering Algorithms

The three widely referenced local metering algorithms are the demand-capacity, percent-occupancy, and the ALINEA algorithms.

Demand-Capacity Algorithm

The demand-capacity algorithm is one of the initial algorithms for traffic responsive ramp metering and can be considered as a fundamental basis for other metering algorithms. This algorithm utilizes real-time freeway flow or occupancy measured upstream and downstream of the ramp. The metering rate is calculated as the difference between upstream freeway flow and downstream capacity, or as a function of the difference between upstream occupancy and desired occupancy (Kristeleit, 2014).

The advantage of the demand-capacity algorithm is that it is simple and easy to understand concept. However, the level of congestion of the freeway may not be determined only using the upstream local freeway occupancy. Another version of this algorithm developed in the Netherlands, manages the metering rate utilizing flow and speed data to keep the actual flow below the critical flow.

In 1990, Papageorgiou et al. showed that this algorithm can be partially sensitive to non-measurable disturbances such as merging difficulties, shockwaves, and slow vehicles. The National Cooperative Highway Research Program (NCHRP) Report 3-87 suggested improving this algorithm by including the probability of breakdown for activation and/or metering rate calculation (Elefteriadou et al., 2009). This means that the ramp metering rate is set to keep the demand significantly lower than capacity, as is done with the simple demand-capacity algorithm.

Percent-Occupancy Algorithm

This algorithm depends only on the upstream occupancy measurements to determine the level of congestion and does not require the downstream capacity measurement. The algorithm utilizes a linear relationship between the metering rate and upstream occupancy. The algorithm is presented in Equation 3-4. In this equation, K_1 is the freeway capacity (K_1). K_2 is the slope of a straight line that relates occupancy to flow in the uncongested part of the fundamental diagram (K_2). The values of both the K_1 and K_2 are preset for each location (Hasan, 1999).

$$r(k) = K_1 - K_2 o_{in}(k - 1) \quad (3-4)$$

where:

$r(k)$ = metering rate at time interval k

K_1 = a constant value of freeway capacity (veh/hr)

$O_{in}(k-1)$ = the last measured upstream occupancy value (%)

ALINEA Algorithms

The Asservissement Linéaire d'Entrée Autoroutière (ALINEA) algorithm was developed by Papageorgiou in 1997 and was deployed in Paris, Amsterdam and Munich (Kristeleit, 2014). ALINEA is a local traffic responsive control algorithm with feedback regulator. The idea of this algorithm is to keep the occupancy under a static and predefined value of critical occupancy. The calculated metering rate is supposed to keep the traffic flow under the capacity level of the downstream segment, as indicated by occupancy measurements. The metering rate is calculated using the following equation:

$$r(k) = r(k - 1) + KR [\hat{o} - o_{out}(k - 1)] \quad (3-5)$$

where:

$r(k)$ = metering rate at time interval k

KR = a constant regulator parameter (veh/hr)

$o_{out}(k-1)$ = the last measured downstream occupancy value (%)

\hat{o} = the desired value for downstream occupancy (predefined)

The critical or desired occupancy can be set by determining the occupancy of the downstream segment when it reaches the near capacity condition.

The ALINEA algorithm is very frequently referenced and used in ramp metering studies. Through the years, multiple extensions of it have been developed to deal with different issues and challenges. FL-ALINEA, UF-ALINEA, UP-ALINEA, X-ALINEA/Q and MALINEA are extensions of the algorithm that are described below.

- ***FL-ALINEA***

The FL-ALINEA algorithm was developed to overcome possible difficulties in occupancy measurements and setting desired occupancy values. This extension modifies the original ALINEA equation by substituting occupancy by downstream flow measurements. It has been recommended to keep the critical flow at least 10% lower than the capacity.

- ***UF-ALINEA***

This is a modification to the FL-ALINEA algorithm described above which estimates the downstream flow instead of measuring it, simply by adding the on ramp flow to the upstream flow.

- *UP-ALINEA*

In some conditions only upstream occupancy is available and the ALINEA algorithm needs to be modified to calculate downstream occupancy based on the upstream measurements. This extension of the original algorithm is called the UP-ALINEA algorithm. To calculate downstream occupancy, additional measurements of the entering flow from the on-ramp to the freeway, as well as freeway upstream flow measurements are required.

Assuming $q_{out} = q_{in} + q_{ramp}$, the downstream occupancy is calculated using the following equation:

$$o_{out}(k) = o_{in}(k) \left[1 + \frac{q_{ramp}(k)}{q_{in}(k)} \right] \times \frac{\lambda_{in}}{\lambda_{out}} \quad (3-6)$$

where:

$o_{out}(k)$ = downstream occupancy at the time k

$o_{in}(k)$ = upstream occupancy at the time k

$q_{ramp}(k)$ = measured ramp volume (vph) at the time k

$q_{in}(k)$ = upstream measured freeway volume (vph) at the time k

λ_{in} , λ_{out} = the number of mainstream lanes for upstream and downstream segments, respectively.

- *X-ALINEA/Q*

Ramp metering may cause the formation of large queues on the ramp, which may affect the surface street. The X-ALINEA/Q algorithm was developed to account for ramp queue. X-ALINEA/Q requires measuring the ramp demands and queue lengths.

- *MALINEA*

The MALINEA algorithm was proposed by Oh and Sisiopiko in 2001 (Kristeleit, 2014). MALINEA measures both upstream occupancy and the time lag between upstream and downstream measurements to incorporate the upstream conditions in metering rate calculation. MALINEA uses the following equation for this purpose (Elefteriadou et al., 2009):

$$r(t+1) = [o_u(t+n+1) - o_u(t)] \times \frac{K}{A} + r(t-n) \quad (3-7)$$

where:

$r(t+1)$ = metering rate to be applied in the next time period

$o_u(t+n+1)$ = desirable upstream occupancy

$o_u(t)$ = measured occupancy at the upstream detector for the last time period

$r(t-n)$ = is the current metering rate

A , K = slope of occupancy upstream and downstream of the on-ramp, and the parameter used in ALINEA, respectively.

3.2.3.2. Coordinated Ramp Metering Algorithms

The coordinated ramp metering algorithms have been categorized as cooperative, competitive, or integral. For cooperative ramp metering, the calculated metering rates based on local conditions are adjusted by further considering the system wide conditions. For competitive ramp metering, both local and system wide metering rates are calculated and the most restrictive one is selected. For integral ramp metering, the local and system wide conditions are incorporated into the metering rate calculation simultaneously in order to determine the optimal metering rates (Bertini and Ahn, 2006).

Cooperative Algorithms

- *The Helper algorithm*

The Helper algorithm was developed in 1981 in Denver, Colorado and includes a local traffic responsive algorithm enhanced by a system override feature (Kristeleit, 2014). In this algorithm, the freeway corridor is divided into groups with each group containing one to seven metered ramps (Lipp et al., 1991). This algorithm initially determines the metering rate for each of the ramps using a local traffic responsive algorithm and simultaneously monitors the on-ramp queue using the occupancies of queue detectors. If the occupancy on a queue detector for a specific on-ramp exceeds the predefined threshold value, the subject ramp is specified as a “critical ramp”. Once a critical ramp is identified in a group, the system override feature of the Helper algorithm is activated. This feature increases the metering rate of the critical ramp and at the same time reduces the metering rate of the upstream ramps to mitigate the congestion around the critical ramp (Bertini and Ahn, 2006).

- *The Linked Ramp Algorithm*

The Linked Ramp algorithm was first deployed in San Diego, California in 1968 and is known as San Diego Ramp Metering System (SDRMS). This algorithm utilizes historical traffic flow data to calculate maximum and minimum metering rates at each ramp. The maximum metering rate is calculated based on the local capacity estimated based on historical data. The metering rate is simply set as the difference between the target traffic flow (considering capacity) and upstream traffic flow. As such, the algorithm operates as local. However, once the demands lead to calculating rates that are below the minimum rates, the cooperative feature of the Linked algorithm is activated resulting in metering upstream ramps (Kristeleit, 2014).

Competitive Algorithms

- *The FLOW Algorithm*

The FLOW algorithm is a bottleneck based algorithm developed by Jacobson et al. in 1989. This algorithm calculates both local and bottleneck metering rates and picks the more restrictive one. The calculations of these rates are described below.

The local metering rate is calculated using the percent-occupancy algorithm, described earlier. The metering rates are selected from a look up table which relates the upstream occupancy to the metering rate of the ramp. The look up table is produced based on historical volume-occupancy relationships (Jacobson et al., 1989).

The first step to calculate the bottleneck metering rate is determining the bottleneck locations on the freeway with an influence zone of at least one ramp. Then all of the metered on-ramps within the zone are weighted based on their distances from the bottleneck and historical ramp volume. This algorithm requires having loop detectors upstream and downstream of the influence zone as well as all metered and unmetered on and off-ramps. The bottleneck algorithm is activated once both of the following conditions are met. First, the downstream occupancy exceeds a threshold value. Second, the total entering volume (the sum of upstream freeway volume and entering volume from on-ramps) to the freeway section exceeds the total exiting volume (the sum of vehicle exiting the section and exiting off-ramps volume). The metering rates are calculated based on the difference of the total entering and exiting volumes and the weighting factors for the ramps, set as described earlier.

The influences area of different bottleneck locations may overlap. Therefore, each ramp may have more than one bottleneck metering rate. Also, the most restrictive value of the metering rates among the local and bottleneck metering rates is selected for the subject ramp. The FLOW algorithm also includes a queue control feature to prevent queue spillback to the surface street (Hasan, 1999).

- *Zone Algorithm*

The zone algorithm was initially implemented as a pre-timed metering system in the St. Paul metropolitan/Minneapolis. Later, the system was upgraded to a traffic responsive metering system. In this algorithm, as the name implies, the freeway is divided to multiple zones with low incidents upstream and a potential bottleneck downstream. Each zone includes freeway mainline, off-ramps, metered and unmetered on-ramps. The zone metering algorithm is designed to keep the level of traffic lower than the desired threshold by managing the entering flows to the zone. The downstream bottleneck capacity is the regulating factor for managing the entering and

leaving traffic over the zone. The basic equation describing the general concept of this algorithm for metering rate calculation is the following (Hourdakis and Michalopoulos, 2002):

$$M + A + U \leq B + X + S \quad (3-8)$$

where:

M = total volume entering the mainline from all entrance ramps in a given zone (also referred to as release rate),

A = arrival volume at the upstream boundary of a zone,

U = total volume entering a zone from un-metered entrance ramps,

B = capacity of downstream mainline boundary of a zone,

X = total exit volume from a zone,

S = available storage, or spare capacity in the beginning of each time interval.

The Minnesota Department of Transportation (MnDOT) evaluated this metering algorithm on Highway 169 and I-394 in the year 2000 and concluded the algorithm improved travel time by 6-16% and reduced the fuel consumption and emission by 2-47%.

The Stratified Zone Metering algorithm (SZM) was developed and implemented as an enhancement to the zone metering algorithm by adding the definition of overlapping zones. A zone can be defined between any two mainline detector stations and are typically set to be 0.5-3 mile long. Therefore, each primary zone in the conventional version is disintegrated to multiple layers in the SZM. Each layer is a set of successive detector stations. As illustrated in Figure 3-3, the higher level layers have longer zones and the zones at each level may overlap. The metering rates are calculated and reported for 30 seconds intervals. The ramp metering is calculated to avoid mainline congestion and on-ramp queue spillbacks to arterials.

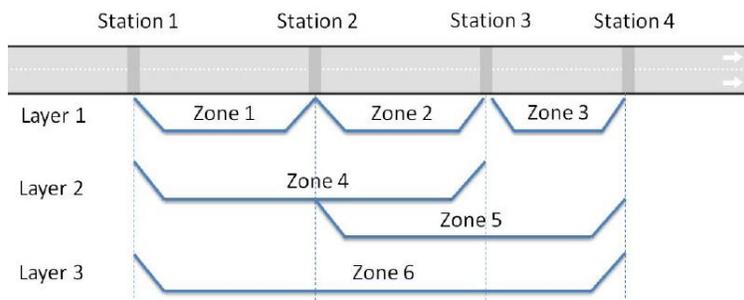


Figure 3-3 Zone and Layer Structure in SZM (Elefteriadou et al., 2009)

- *The SWARM Algorithm*

The System-Wide Area Ramp Metering (SWARM) algorithm divides the freeway to contiguous sections bounded with bottleneck locations identified by loop detector measurements. Each section may include multiple on and off-ramps. For each section, the SWARM 1 and SWRM 2 modes produce two metering rate values and the more restrictive one among the two is selected as the metering rate.

In the SWARM 1 mode (the system-wide level), the density at the bottleneck location is forecasted using a linear regression of the immediate past data and the application of Kalman filtering process. The idea of the algorithm is to keep the real-time density below a predetermined threshold value. The excess density is defined as the difference between the forecasted density and the threshold density. The needed volume reduction for the section can be calculated using the following equations.

$$\text{Required density} = \text{current density} - (\text{excess density} / T_{\text{crit}}) \quad (3-9)$$

$$\text{Volume reduction} = (\text{local density} - \text{required density}) \times (\text{No. of lanes}) \times (\text{distance to next station}) \quad (3-10)$$

where T_{crit} is the forecasting time span (minutes).

The volume reduction (ramp metering rate) is proportionally distributed to upstream on-ramps of the section depending on their demand and queue storage, which define the weighting factor for each of the ramps that is used in calculating the ramp metering rates.

In the SWARM 2 mode (local level), any traffic responsive local metering algorithm can be used to convert the measured headway upstream of the ramp to density estimated used in defining the metering rate. This mode also determines available storage zone and calculates the metering rate to maintain a level of service D with a maximum density of 35 pc/mi/ln (Bertini and Ahn, 2006).

SWARM is a predictive algorithm, therefore the performance is highly dependent on the accuracy of the prediction. This predictive feature enables the algorithm to prevent bottlenecks. However, inaccurate prediction may lead to poor results (Elefteriadou et al., 2009). Zhang et al. evaluated the performance of SWARM with five steps ahead prediction and reported very poor results compared to the ALINEA, zone and bottleneck algorithm (Zhang et al., 2001).

- *Seattle Bottleneck Algorithm*

The Seattle Bottleneck metering algorithm calculates both of the local metering rate and bottleneck metering rate using the upstream occupancy of each ramp obtained (for local

responsive detector data) and bottleneck information. Then, the lowest metering rate of the local metering rate and the bottleneck metering rate is assigned to each ramp.

The local metering rate is set as the difference between the real-time upstream volume and the estimated capacity. The capacity is estimated based on the volume-capacity relationship calculated using historical data upstream of the ramp. Jacobson et al. provided an example of a curve using historical data to determine the local metering rate depending on the mainline occupancy (Jacobson et al., 1989).

The bottleneck metering rate is calculated considering the system-wide conditions and capacity constraints. The calculations are based on real-time demand-capacity relationship. This algorithm is activated once the downstream occupancy exceeds the predefined threshold (usually 18%) and the entering volume to the section between two detectors exceeds the exiting volume. In this case, the upstream volume should be reduced by the number of the vehicles stored in the section. This volume reduction is distributed to all of the noncritical upstream ramps by weighting factors. The weighting factor depends on the ramp demand as well as the distance between the subject ramp and downstream bottleneck.

Once the metering rate is set as the lowest value of local and bottleneck metering rates, the rate needs to be adjusted considering the queues on the ramp, a ramp volume adjustment, and an advance queue override. The queue adjustment is applied to consider the presence of queue at the ramp queue detector. The ramp volume adjustment takes the driver violations and inattention in account by automatically correcting the metering rate while the number of vehicles entering the freeway from the ramp is less or more than the specified metering rate. The queue override adjustment increases the metering rate to a high value and is activated once the ramp queue reaches the worst acceptable queue detected by the ramp queue detector. The NCHRP report 3-87 suggests enhancing this algorithm by considering the probability of breakdown threshold for ramp metering activation and also local metering rate calculation (Elefteriadou et al., 2009).

Integral Algorithms

- ***METALINE***

METALINE is an extension of the ALINEA algorithm developed by Papageorgiou et al. in 1990 (Papageorgiou et al., 1990). In this algorithm, the fluctuation in the measured occupancy for each freeway segment and the difference between the critical occupancy and the measured occupancy determines the metering rate for each ramp. The metering rate, measured occupancy, and desired occupancy are presented in the form of a vector as illustrated in the following equation (Elefteriadou et al., 2009):

$$\vec{r}(k) = \vec{r}(k-1) - K_1[\vec{o}(k) - \vec{o}(k-1)] - K_2[\vec{O}(k) - \vec{O}^c] \quad (3-11)$$

where:

$\vec{r}(k)$ = The vector of metering rates for the m controlled ramps at time step k

$\vec{o}(k)$ = The vector of n measured occupancies within the directional freeway segment at time step k .

$\vec{O}(k), \vec{O}^c$ = The measured and desired occupancy (occupancy at capacity), respectively, downstream of controlled ramps.

K_1 and K_2 = Gain matrices.

- *Model Predictive Control*

The Model Predictive Algorithm (MPC) is an online predictive metering algorithm incorporating a cost criterion into the optimization process and constrained inputs and outputs (Camacho and Bordons, 1995; Maciejowski, 2002). MPC utilizes a receding horizon strategy to minimize the predefined objective function which results in optimization of traffic operation. This algorithm predicts the traffic parameters for a determined prediction horizon (N_p) based on a traffic model and in each sample step k determines the metering rate which minimizes the objective function for the time period $[k\Delta T_{ctrl}, (k+N_p)\Delta T_{ctrl}]$. ΔT_{ctrl} is the controller time step which defines the rate for updating the control signal and is usually set as one minute. In order to manage the computational efforts, a control horizon (N_c) smaller than N_p is defined. The metering rate is only allowed to change at this time period $[k\Delta T_{ctrl}, (k+N_c)\Delta T_{ctrl}]$ and remains unchanged after that. In this receding horizon framework, the metering rate is set as the first calculated rate and after implementation of this rate, the traffic measurements will be updated and fed to the next sample step ($k+1$). The whole process is iterated for each step.

It is very important to select proper value for N_p and N_c . Larger N_p enables the algorithm to predict further event but on the other hand increases the computational complexity. Smaller N_c increases the number of metering rates to be optimized and on the other hand increases the accuracy. Therefore, there should be a proper tradeoff between the accuracy and the computational of complexity of the algorithm (Bellemans et al., 2006).

- *Fuzzy Logic*

The Fuzzy logic algorithm was developed and implemented in the Washington State and has been adopted for use in Florida, with the first implementation is on I-95 in Miami-Dade County. The algorithm incorporates both objective and subjective knowledge to improve traffic operations. The subjective knowledge is represented in linguistic form. Unlike other ramp metering algorithms which try to fit the nonlinear nature of traffic to linear context, fuzzy logic can handle nonlinear systems with unknown models. The main advantage of the fuzzy logic algorithm is the ability to handle nonlinearity and uncertainties, ability to utilize incomplete and

inaccurate data, and balancing mainline congestion and ramp queues. The fuzzy logic controller does not require extensive system modeling and is easy to tune. This algorithm benefits from a set of rules, which is developed by human expertise. These rules are easy to modify, therefore the fuzzy controller can easily adapt to various situations and compensate for poor and inaccurate measurements. The successful implementation of a variety of fuzzy logic controllers in Amsterdam, Netherland, Miami and Seattle confirms the validity of this statement. Netherland deployed the fuzzy logic controller on nine ramp meters in 1995 and reported 13% improvement in travel time, as a result of this implementation. In 1999, Bogenberger and May proposed the adaptive fuzzy controller that incorporates genetic algorithms and neural networks (Bogenberger and May, 1999). In 2001, a new approach to fuzzy controller was proposed and referred to as Adaptive and Coordinated Control of Entrance Ramps with Fuzzy Logic (ACCEZZ) (Bogenberger, 2001).

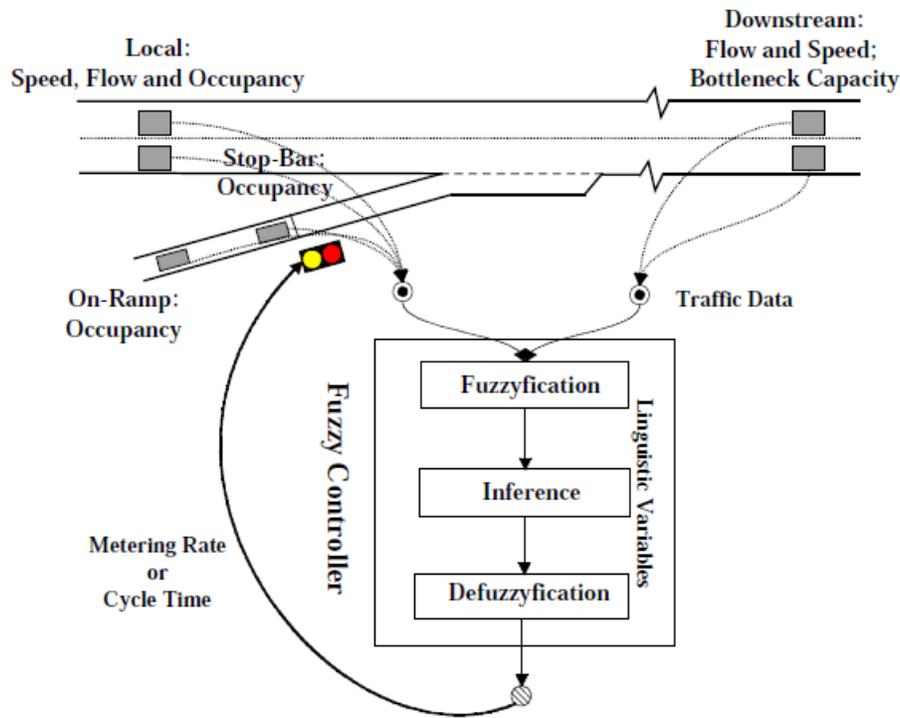


Figure 3-4 Fuzzy Logic Ramp Metering Structure (Bogenberger et al., 2001)

The fuzzy logic algorithm incorporates three main steps of fuzzification, rule activation and defuzzification. In the fuzzification step, each of the numerical inputs is classified into fuzzy classes and the degree of membership for each class is determined. The general fuzzy ramp controller used in the ACCEZZ models has a total of seven inputs: local flow, speed and occupancy on the mainline immediately upstream of the on ramp, downstream speed and v/c (volume/capacity) ratio, queue and check in occupancy. As shown in Figure 3-5, each value of the input corresponds to one or more of the predefined classes of the subject variable. The shape, distribution and dynamic range of these fuzzy classes are subject to a tuning process. In this case,

a degree of membership is assigned to each class based on the same figure. The degree of membership for each class indicates how true the input belongs to that class. The degree of membership is in scale 0 to 1. The degree of 1 indicates an input that belongs 100% to the subject class.

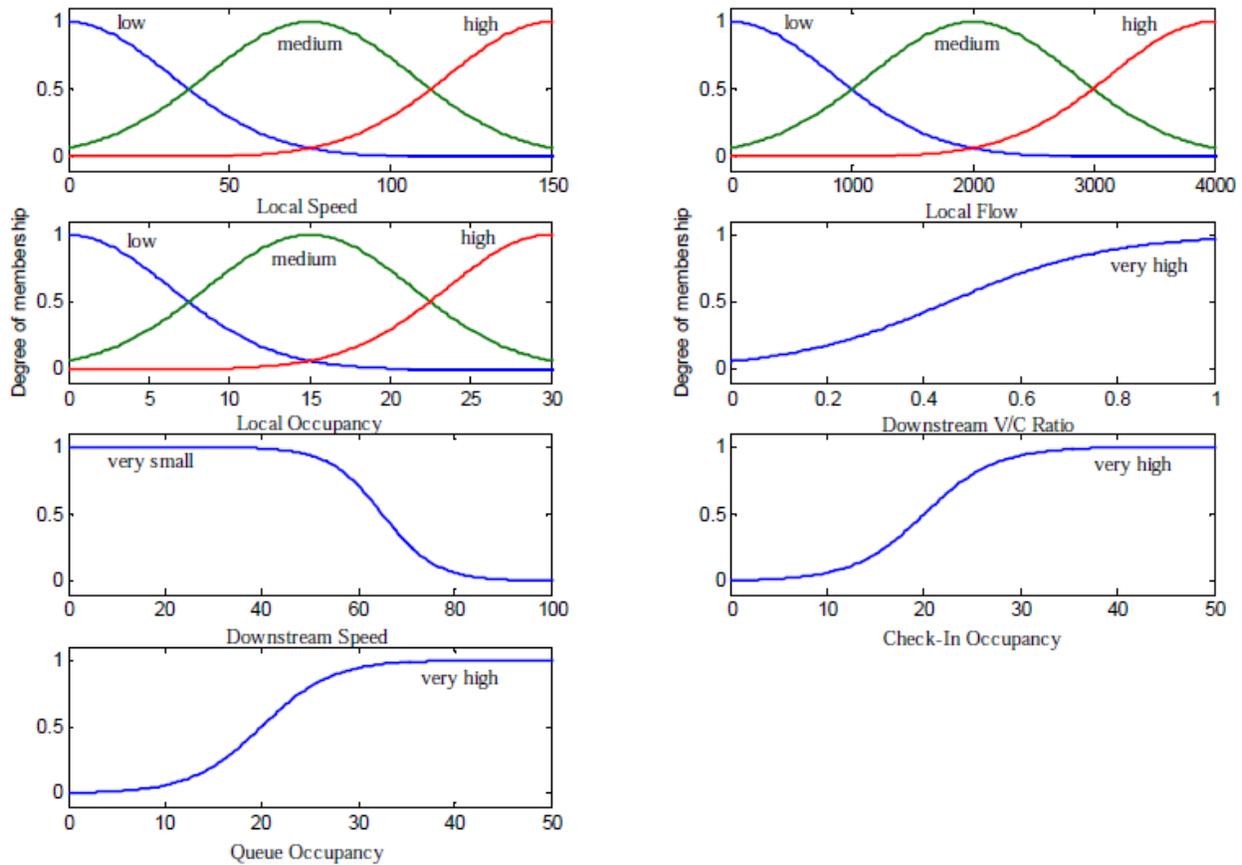


Figure 3-5 Fuzzy Logic Membership Function for Inputs (Bogenberger et al., 2001)

The second step is to use the fuzzy classes and their degree of membership from the previous step to find the appropriate rule to apply. Table 3-6 shows examples of these linguistic rules, their outcomes, and rule weights. The assigned weight for each rule indicates the relative importance of it compared to other rules.

Table 3-6 Linguistic Metering Rules (Bogenberger et al., 2001)

Rule #	Weight	Premise	Consequent
I	1.5	If local Occupancy is low	Then metering rate is high
II	1.5	If local occupancy is medium	Then metering rate is medium
III	2.0	If local occupancy is high	Then metering rate is low
IV	2.0	If local speed is low and local flow is high	Then metering rate is low
V	1.0	If local speed is medium and local occupancy is high	Then metering rate is medium
VI	1.0	If local speed is medium and local occupancy is low	Then metering rate is high
VII	1.0	If local speed is high and local flow is low	Then metering rate is high
VIII	3.0	If downstream speed is very low and downstream V/C ratio is very high	Then metering rate is low
IX	3.0	If check-in occupancy is very high or queue occupancy is very high	Then metering rate is high

The third step is to calculate the metering rate value based on the outcomes of previous step and report a single numeric value as the metering rate. This process of transformation from linguistic metering rules to reporting a quantitative value as the metering rate is called the defuzzification process. Diagrams are provided for the output variables and their degree of membership similar to the diagrams in Figure 3-5, which are provided for the inputs. The final metering rate is calculated as the weighted average of the metering rate and degree of membership.

The ACCEZZ models use the same fuzzy controller procedure and dynamically adjust the input and output fuzzy sets by redefining the linguistic variables. This adjustment can address inaccuracy of the data or change in traffic patterns and is automatically done by learning procedures such as the neural network algorithm. The ACCESS family models include a two-step process of bottleneck performance measurement and a dynamic freeway traffic model. The first step is to monitor the entire traffic system by incorporating traffic system and a queuing model. The second step is to capture the dynamic traffic state evolution and coordinate all the ramps to reach the network level optimum status.

The ACCEZZ family includes five algorithms: neuro fuzzy online, neuro fuzzy offline, genetic fuzzy online, genetic fuzzy offline, genetic fuzzy reality. The neuro fuzzy algorithms utilize neuro fuzzy architecture and a macroscopic traffic flow model. While, the genetic fuzzy algorithm incorporates a genetic algorithm. The online algorithms use the last 15 minutes traffic measurements as the best estimator of the next 15 minutes interval. However, the offline algorithm incorporates the estimated demand for the entire day. The genetic fuzzy reality algorithm does not include a traffic model and uses the measurements from the actual implementation (Bogenberger, 2001).

3.2.3.3 Comparison of Metering Algorithms

The metering algorithms described above are summarized and compared in the following table.

Table 3-7 Metering Algorithm Summary

Extent		Algorithm	Metering Rate Calculation Method
Local		Demand-Capacity Algorithm	<ul style="list-style-type: none"> Based on difference between upstream freeway flow and downstream capacity, or Based on difference between upstream occupancy and desired occupancy
		Percent-Occupancy Algorithm	<ul style="list-style-type: none"> Based on the difference between upstream freeway occupancy and occupancy at capacity, by conducting linear relationship between the upstream freeway occupancy and freeway flow
		ALINEA Algorithm	<ul style="list-style-type: none"> Based on the difference between downstream freeway occupancy and desired occupancy
Coordinated	Cooperative	Helper Algorithm	<ul style="list-style-type: none"> Based on upstream occupancy of critical ramp, one of seven predefined metering rate categories is selected. If the ramp queue from queue detector exceeds the threshold, the metering rate will be increased and the upstream ramp metering rate would decrease.
		Linked Ramp Algorithm	<ul style="list-style-type: none"> Based on the difference between upstream freeway flow and target flow. If the metering rate falls below a predefined threshold, the upstream ramp will have the same metering rate or less.
	Competitive	FLOW Algorithm	<ul style="list-style-type: none"> Calculates both local and bottleneck metering rates and picks the more restrictive one. Local metering rate is derived from percent-occupancy algorithm. Bottleneck metering rate is based on the difference of the total entering and exiting volumes and the weighting factors for the ramps (weights are based on their distances from the bottleneck and historical ramp volume)
		Zone Algorithm	<ul style="list-style-type: none"> The zone is defined as the area between boundaries of free flow in upstream and bottleneck in downstream. Metering rate is calculated based on the difference downstream bottleneck capacity and the upstream flow plus the on ramp volume (the difference of entering and exiting flow compared to the bottleneck capacity).
		SWARM Algorithm	<ul style="list-style-type: none"> Calculates both local and system-wide metering rates and picks the more restrictive one. Local metering rate based on upstream density. System-wide metering rate based on the difference between real-time density and predefined threshold.

Table 3-7 Metering Algorithm Summary (Con't)

Extent		Algorithm	Metering Rate Calculation Method
Coordinated	Competitive	Seattle Bottleneck Algorithm	<ul style="list-style-type: none"> • Calculates both local and system-wide metering rates and picks the more restrictive one. • Local metering rate based on upstream occupancy. • State-wide metering rate based on the difference between the downstream volume and bottleneck capacity.
	Integral	METALINE	<ul style="list-style-type: none"> • Based on the difference between the critical occupancy and the measured occupancy.
		Model Predictive Control	<ul style="list-style-type: none"> • Predicts traffic parameters for a predefined time horizon and calculates metering rate by optimization process based on the selected objective function.
		Fuzzy Logic	<ul style="list-style-type: none"> • Based on local speed, occupancy, flow, queue occupancy and downstream speed and predefined linguistic rules the metering rate is defined. • The rules compare the current condition to the desired condition and regulate to eliminate this difference and reach desired condition.

3.2.4. Useful Concepts from Metering Algorithms

Based on the review of the existing adaptive ramp metering algorithms, it was possible to identify concepts that can be adopted for use as part of the methodology for identifying the warrants in this project. For example, the Zone, FLOW, Seattle Bottleneck, and SWARM algorithms divide the freeway into multiple zones, each zone with a potential bottleneck downstream. The downstream bottleneck capacity is used as the regulating factor for managing the entering and leaving traffic over the zone in the Zone, FLOW, and Bottleneck algorithms and the density of the bottleneck is used as the controlling parameter in the SWARM algorithm to maintain level of service D. This can be adopted to identify what ramps to meter. The adopted method may utilize volume thresholds below capacity to constrain the incoming volume to a value that reduces the probability of breakdown.

The concept of multiple overlapping zones as used in the Stratified Zone algorithm and the FLOW algorithm can be helpful. The idea of weighting the ramps to determine their contributions to ramp metering is also interesting. The FLOW and Seattle bottleneck algorithms weight the ramps based on their distances from the bottleneck and ramp volumes. The SWARM algorithm weights the ramps based on their distances from the bottleneck and queuing storage.

3.3. Additional Considerations for Ramp Metering

There are several environmental and operational components which may impact traffic flow on a freeway or ramps. These components need to be considered in the ramp metering process as they can severely impact the performance of ramp metering and the transportation network as a whole. This section summarizes some of these components.

3.3.1. Incident

Traffic incidents may affect traffic operations by changing the roadway environment and driver behaviors. The roadway environment can be affected by capacity reductions due to lane or shoulder closure, changes to traffic control strategies including ramp metering, incident management and dynamic message sign activation. Driver behavior may be affected at the microscopic level (tactical and operational) such as lane changing, car following, speed, gap acceptance and accelerating behavior. Incident occurrence may also cause changes at the strategic level such as route shift, mode shift or change in the time of trip (Hadi et al., 2011). The occurrence of incident may require an early activation of ramp metering and more restrictive metering rate.

Incident impacts on capacity drop are well studied. Goolsby studied the capacity reduction due to incident in 1971 and concluded that an incident blocking one lane out of three lanes reduces capacity by about 50%. He also concluded that an incident blocking two lanes out of three lanes reduces the capacity by about 79% (Goolsby, 1971).

The HCM 2000 provides estimates of the expected reduction in capacity as a function of the number of blocked lanes (or shoulder) and the number of lanes of the highway section under consideration. In this study, the values recommended in the HCM 2000 are used as the average capacity reduction during incidents; for a three-lane freeway segment, these values are 17% for shoulder blockage incidents, 51% reduction in capacity for one-lane blockage incidents, and 83% capacity drop for two-lane blockage incidents. These values are 15%, 42%, and 75%, respectively, for a four-lane freeway segment (Transportation Research Board, 2000).

Qi and Smith suggested that the capacity reduction with one out of three lanes blocked can be modeled as a beta distribution with an average of 63% and a standard deviation of 14% (Qi and Smith, 2001). This average capacity reduction is higher than that reported in prior research, which showed about a 50% reduction. They also found that the capacity reduction due to incidents with two out of three lanes blocked can be modeled as a beta distribution with an average of 77% and a standard deviation of 12%. This average value is close to results (79%) reported earlier.

Knopp et al. found that in the case of a blocked driving lane, the queue discharge rate for each available lane is reduced by 50%. They also found that the queue discharge rate is reduced by 30% when the driving lanes are open but there is an incident on the shoulder or on the opposite direction of travel (Knoop et al., 2009).

Hadi et al. adjusted the parameters of three widely used microscopic simulation tools to determine their abilities to replicate the reported reductions in capacities due to traffic incidents. From the results, they concluded that it was possible to fine tune the parameters of the three simulation tools to simulate the drops in capacities due to incident lane blockage (Hadi et al., 2007).

Hadi et al. investigated capacity drop for different incidents using STEWARD data and concluded that just after incident occurrence with one lane closure, the capacity drops by 54-60%. During the lane blockage period another capacity drop happens due to truck arrival and starting to block other lanes. After incident clearance the queue starts to dissipate by queue discharge rate which is 19-39% lower than capacity before incident occurrence. This reduction is partly due to rubberneck effects (Hadi et al., 2011).

3.3.2. Adverse Weather

Adverse weather is one of seven sources of congestion, as identified by the FHWA. Adverse weather conditions such as rain, snow, fog, and high speed wind can affect driver behaviors, vehicle performance, and thus traffic flow characteristics including capacity, speed, travel time, and safety (FHWA, 2015a). Ramp metering activation and metering strategies may adapt to such changes due to adverse weather. Ibrahim and Hall investigated freeway speed reductions under adverse weather conditions and concluded that the speed is reduced by 3-5%, 14-15% and 30-40% for light precipitation (including both rain and snow), heavy rain, and heavy snow, respectively. The authors mentioned that these values could be different depending on the specific location characteristics and cannot be generalized for dissimilar regions (Ibrahim and Hall, 1994).

An empirical study was done by the Federal Highway Administration (FHWA) to examine the impact of adverse weather including precipitation and visibility on freeway free flow speed, speed at capacity, capacity, and jam density based on loop detector data from Baltimore, the Twin Cities, and Seattle. The results showed that the jam density is not impacted by weather conditions, while the free flow speed and speed at capacity decreased with increasing rain intensity. However the study found that the capacity reduction does not change with rain intensity and remains constant at a value of 10% to 11%. A 2% to 3.6% reduction in free flow speed and a 8-10% reduction of speed at capacity was reported for light rain conditions (less than

0.0039 in/hr). The values of 6-9% reduction in free flow speed and 8-14% reductions in speed at capacity were reported for heavy rain (0.63 in/hr) (FHWA, 2006b).

Stralen et al. investigated the impacts of adverse weather on the probability of traffic breakdown. They incorporated both the supply and demand aspects of adverse weather influence on traffic conditions using a panel mixed logit model. The average breakdown probability for dry weather was 50%, while the average breakdown probability for heavy rain condition was reported as 77.4% (Stralen et al., 2014).

Li et al. assessed travel time reliability under rainfall in Florida. They calibrated the rainfall intensity distribution on zip code basis and hourly precipitation and evaluated travel time reliability based on the rainfall probability. The final results showed 6% to 12% speed reduction for freeway and arterial facilities, depending on the rain intensity level (Li et al., 2014).

HCM 2010 presented capacity reduction percentages for freeway facilities under three levels of light, medium and heavy rain. These values represent 2% capacity reduction for light rain, 7.2% capacity reduction for medium rain and 14.1% capacity reduction for heavy rain (Transportation Research Board, 2010).

3.3.3. Diversion

Traffic diversion due to ramp metering was initially discussed to address the concern of traffic deterioration on the parallel streets due to additional demands on these streets. However, there are also conditions where diversion away from congested ramps and merge areas can cause improvements in traffic condition. When the traffic conditions on the freeway is improved because of proper ramp metering, drivers with longer distance trips tend to use the freeway instead of local streets. Therefore, the traffic diversion due to ramp metering does not always worsen the situation for parallel local streets. When the ramp metering increases the capacity of freeway bottlenecks, both freeway and local streets benefit from the additional capacity. The true impact of ramp metering on the network and different types of roads is complicated and requires thorough network analysis.

The diversion effect of ramp metering can be interpreted as negative or positive depending on the local streets traffic conditions. When the local streets are underutilized, the diversion from freeway to local streets will improve the network traffic conditions. However, when the parallel local streets are operating near capacity condition and the ramp metering causes diversion from freeway to these roads, this excess volume worsens the situation. Kang and Gillen reported no more than 5-10% diversion to the local streets (Kang and Gillen, 1999). Moreover, other studies in Los Angeles, Denver, Seattle and Detroit reported no significant diversion from freeway to local streets (Piotrowicz and Robinson, 1995).

Considering the advantages and disadvantages of diversion on network, ramp metering strategies can be classified as diversionary and non-diversionary. On the other hand, the traffic diversion resulted from ramp metering can be modal, temporal or spatial. The modal diversion refers to considering other modes of transportation and carpooling or transit. The temporal diversion represents changing the time of travel to avoid delay and long queues on the ramp. Temporal diversion helps to spread the peak flow in a longer period of time, which results in more effective utilization of the freeway capacity. The spatial diversion addresses the cases in which vehicles change their route of travel by either selecting a downstream unmeted ramp to enter the freeway or completely diverting to parallel local streets.

Haj-Salem and Papageorgiou conducted a study in Paris and reported that ramp metering caused about 20 % increase in travel time for parallel local streets which carry 5% of the network flow. However, considering the whole corridor, travel time was reduced by 7% (Haj-Salem and Papageorgiou, 1995).

A field study in the Twin Cities, Minnesota, reported contradictory results. A stated preference survey showed that about 75% of travelers mentioned that they are willing to use alternative routes to avoid delays on ramps. While, real-world data showed no significant diversion to alternative routes or other transportation modes (Wu, 2001).

The study on the northbound Dan Ryan Expressway in Chicago showed that having only four metered ramps did not cause enough diversion to avoid the freeway overload; however, the point of congestion propagation was shifted (Wu, 2001).

3.3.4. Vehicle Class

Truck acceleration capability affects the required acceleration length on ramps and therefore affects the metering performance. Yang et al. (2016) investigated the acceleration capability of three types of trucks using a video data collection. The results showed that existing acceleration lane length characteristics affected the acceleration behaviors. The observed speed profile illustrated a two-stage behavior. First, the acceleration rate decreases with increasing speed. Then as the truck reaches the merging point, the acceleration increases to catch up with the freeway stream. The results showed that the default values for acceleration rate in the ITE (Institute of Transportation Engineers) Traffic Engineering handbook and AASHTO (American Association of State Highway and Transportation Officials) green book need to be updated. This study suggested average acceleration rates of 2.82 ft/s² for light, 2.46 ft/s² for medium, and 1.96 ft/s² heavy trucks. Also it recommended that the 15th percentile acceleration performance data, which is approximately 30 percent lower than the measured average acceleration capability, should be considered when calculating the acceleration lane length (Yang et al., 2016).

The current practice is to provide acceleration lengths that meet the minimum acceleration lengths for the design speed of the on-ramps and the freeway mainline, which is significantly lower than that required to start from stops. The American Association of State Highway and Transportation Officials (AASHTO) Policy on Geometric Design of Highways and Streets, also known as AASHTO Green Book (AASHTO, 2011), provides minimum acceleration length requirement for different combination of beginning and ending vehicle speed of passenger cars (see Table 3-8). The FDOT Manual of Uniform Minimum Standards for Design, Construction and Maintenance for Streets and Highways (FDOT, 2016) mainly follows the acceleration lane length recommended by AASHTO (2011).

Tian et al. (2016) performed a detailed analysis of collected data from metered on ramps in California and recommended that the acceleration lane length design should be based on the 15th percentile acceleration rate, meaning that 85% of the drivers can safely achieve the required merging speed. Table 3-9 shows the minimum acceleration length from stop for the 85th percentile passenger cars and trucks.

Table 3-8 Minimum Acceleration Length for Entrance Terminals with Flat Grades of 2% or Less (AASHTO, 2011)

Acceleration Length, L (ft) for Entrance Curve Design Speed (mph)										
Highway Design Speed (mph)	Speed Reached (mph)	Stop Condition	15	20	25	30	35	40	45	50
		and initial speed								
		0	14	18	22	26	30	36	40	44
30	23	180	140	—	—	—	—	—	—	—
35	27	280	220	160	—	—	—	—	—	—
40	31	360	300	270	210	120	—	—	—	—
45	35	560	490	440	380	280	160	—	—	—
50	39	720	660	610	550	450	350	130	—	—
55	43	960	900	810	780	670	550	320	150	—
60	47	1,200	1,140	1,100	1,020	910	800	550	420	180
65	50	1,410	1,350	1,310	1,220	1,120	1,000	770	600	370
70	53	1,620	1,560	1,520	1,420	1,350	1,230	1,000	820	580
75	55	1,790	1,730	1,630	1,580	1,510	1,420	1,160	1,040	780

Table 3-9 Minimum Acceleration Length from Stop for the 85th Percentile Passenger Cars and Trucks (Tian et al. 2016)

Highway design speed (mph)	Speed reached (mph)	Stop condition	Stop condition
		Acceleration length (ft) requirement for 85th percentile passenger car ($y = 0.021x^{2.8228}$)	Acceleration length (ft) requirement 85th percentile Tractor -trailer trucks ($y = 0.3001x^{2.274}$)
30	23	147	375
35	27	231	540
40	31	340	739
45	35	480	975
50	39	651	1,245
55	43	857	1,555
60	47	1,102	1,904
65	50	1,312	2,190
70	53	1,547	2,502
75	55	1,718	2,720

Where y is defined as acceleration length in feet and x is the merging speed of vehicle in miles per hour.

Tian et al. (2016) further recommend two designs for acceleration lengths with ramp metering. The conservative design is recommended for ramps that have sufficient space, while the aggressive design is recommended for on-ramps that have insufficient distance to accommodate acceleration. According to Tian et al. (2016), the AASHTO Green Book design guidance could be reduced by 10% for the conservative design and by 35% for the aggressive design. However, the acceleration lengths for heavy trucks have to be set 60% higher compared to AASHTO standards for passenger cars.

The National Cooperative Highway Research Program (NCHRP) Report 505 (Harwood et al., 2003) provides the required acceleration lengths for 180 lb/hp trucks with different initial speeds, as shown in Table 3-10.

Table 3-10 Minimum Acceleration Length for a 180 lb/hp Truck (Harwood et al., 2003)

Highway design speed (mph)	Speed reached (mph)	Acceleration length (ft) for Entrance Curve Design Speed (mph)								
		Stop Condition	15	20	25	30	35	40	45	50
		and Initial Speed								
		0	14	18	22	26	30	36	40	44
30	23	275	160							
35	27	400	300	230						
40	31	590	475	400	310	170				
45	35	800	700	630	540	400	240			
50	39	1,100	1,020	950	850	720	560	200		
55	43	1,510	1,400	1,330	1,230	1,100	920	580	240	
60	47	2,000	1,900	1,830	1,740	1,600	1,430	1,070	760	330
65	50	2,490	2,380	2,280	2,230	2,090	1,920	1,560	1,220	800
70	53	3,060	2,960	2,900	2,800	2,670	2,510	2,140	1,810	1,260
75	55	3,520	3,430	3,360	3,260	3,130	2,960	2,590	2,290	1,850

3.4. Summary

Several states around the country have developed and used specific warrants for ramp metering installation. These warrants consider only the traffic, geometry, and safety conditions in the immediate vicinity of the subject on-ramp (local conditions). Ramp metering algorithms, as implemented during operations to reduce the impacts of traffic bottlenecks, can be categorized as local or system-wide algorithms. Local algorithms only target a single ramp, as an isolated element. On the other hand, system algorithms consider multiple ramps as a system and calculate the metering rates of each on-ramp to address the congestion and breakdown at the bottleneck locations to benefit the entire system. System-wide algorithms are proven to be more effective than local algorithms because they distribute the metering burden to several ramps, instead of relying on the metering of the single on-ramp immediately upstream of the bottleneck, which may not be sufficient and can cause long queues on the on-ramp.

Moreover, the existing local warrants only consider recurrent conditions to justify ramp metering installation with no consideration of the benefit of metering during non-recurrent events such as incidents and adverse weather. Non-recurrent conditions contribute significantly to the congestions and unreliability of the transportation system and need to be addressed by traffic management strategies.

The above discussion points out to a gap between existing ramp metering warrants and the ramp metering operation algorithms, which are used during operations. This project aims to bridge this gap by developing a methodology to address the system-wide traffic conditions in addition to taking non-recurrent conditions into consideration.

4. DATA COLLECTION AND PROCESSING

Data are needed in this project to estimate a number of parameters for use as inputs to potential warrants for ramp metering installation and real-time activation. Both recurrent and non-recurrent conditions are addressed in this study. Recurrent congestion condition refers to peak hour congestion which occurs on daily basis without the impact of events. Non-recurrent congestion conditions refer to congestion due to special events such as incidents, work zones, and/or adverse weather conditions. Thus, data items that allow the assessment of both recurrent and non-recurrent congestions were collected in this study. The collected data can be categorized into traffic and event data. Traffic data includes point traffic detector data archived in the Regional Integrated Transportation Information System (RITIS). The detector data includes volume, speed (detector data), and occupancy measurements. Event data was also initially obtained from the RITIS database. The data includes the description of the event in addition to the beginning and ending date and time. However, it was found that the RITIS incident data does not include important incident attributes. Thus, more detailed incident data was obtained from the FDOT SunGuide software database. The weather data was obtained from a national weather service. This chapter describes the data collection, preprocessing, and analysis processes conducted in this study. In this project, a corridor of about 15 miles including ten on-ramps from arterials, along I-95 in Broward County, Florida was used as a case study. Figure 4-1 illustrates the case study area.

4.1. Traffic Data

As aforementioned, traffic data including volume, occupancy, and speed for the northbound direction of the case study corridor during morning and afternoon peak hours were collected from RITIS, which is the traffic management data warehouse of the Florida Department of Transportation (FDOT). Figure 4-2 shows the interface of the RITIS web-based interface. The study uses data from weekdays of 2015 excluding holidays (total of 251 weekdays). The data was downloaded at 5 minute intervals and archived for further processing.

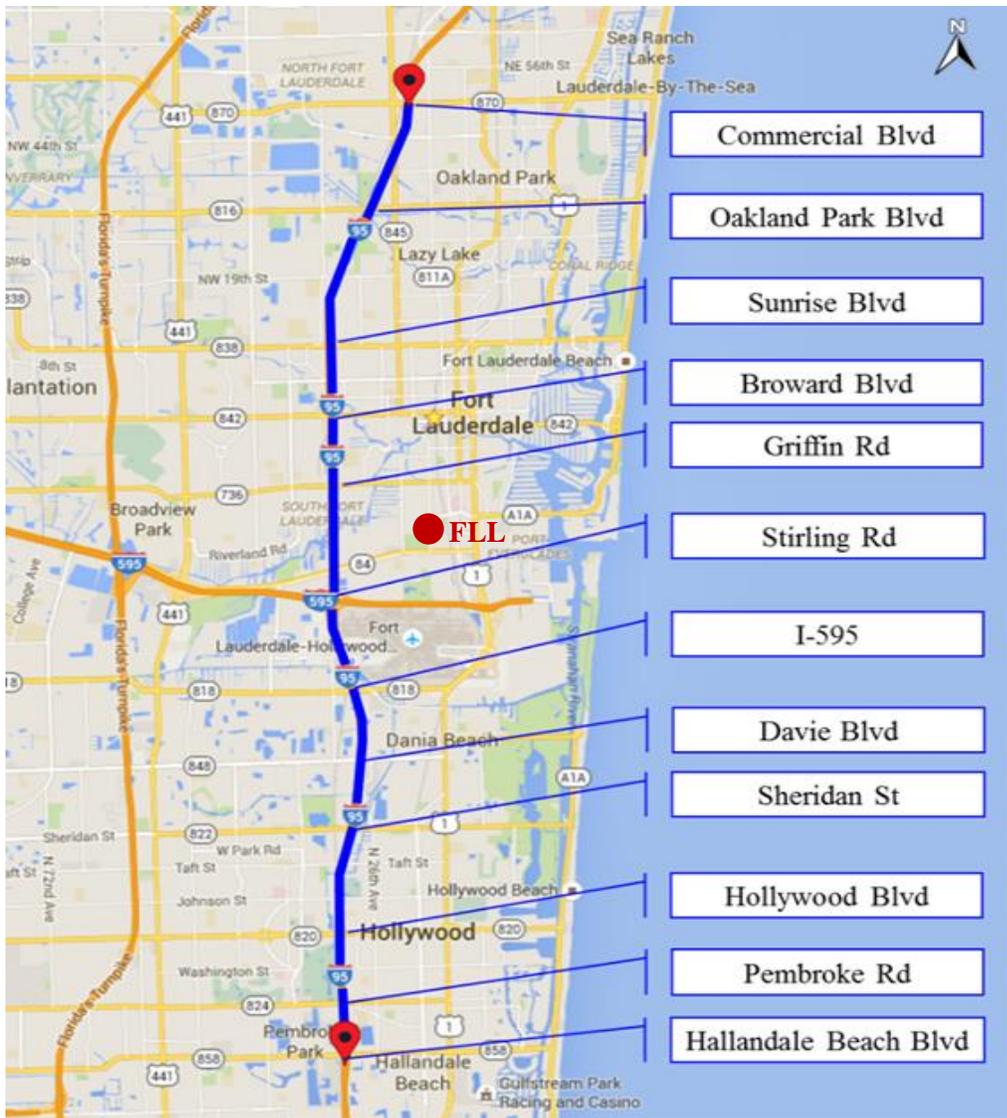


Figure 4-1 Study Area, I-95 Northbound in Broward County, Florida

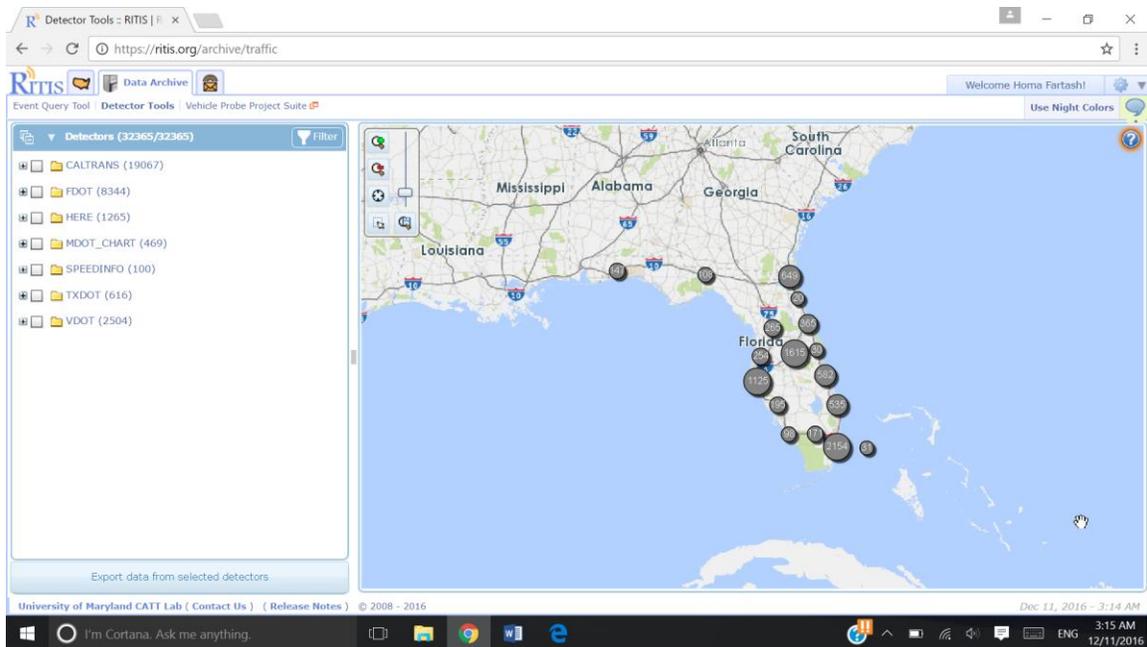
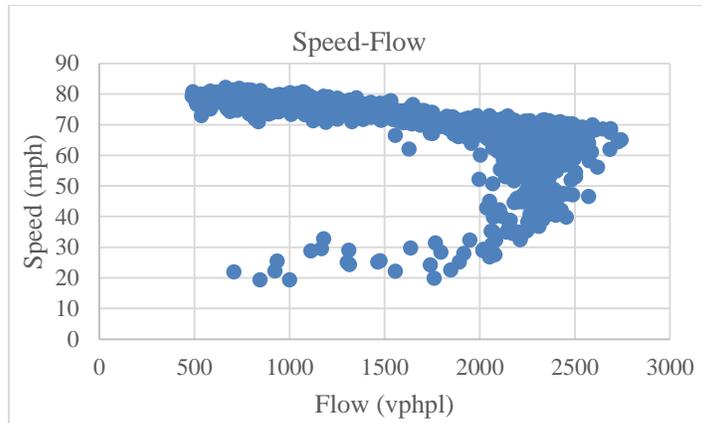


Figure 4-2 Interface of RITIS Database

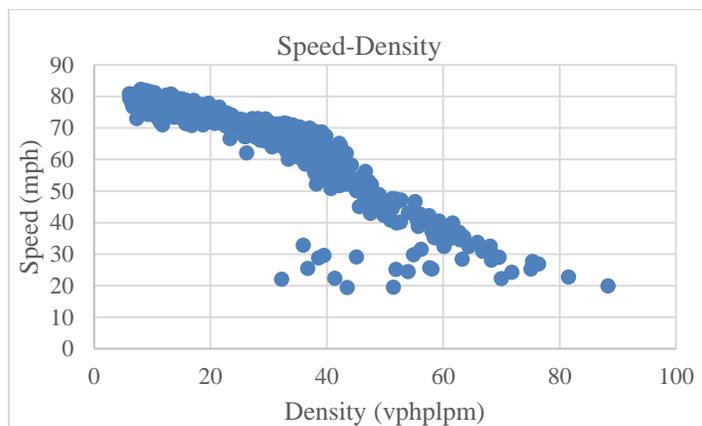
4.1.1. Exploration of the Traffic Data

The collected data needs to be assessed in terms of reasonability. In order to check the reasonableness of the data, fundamental diagrams are plotted using the speed and volume data for the event-free days (recurrent conditions) for both the 5 and 15 minute time interval aggregation levels at the bottleneck location of the northbound direction of the test corridor. The recurrent bottleneck location for both peaks is at the Commercial Boulevard exit.

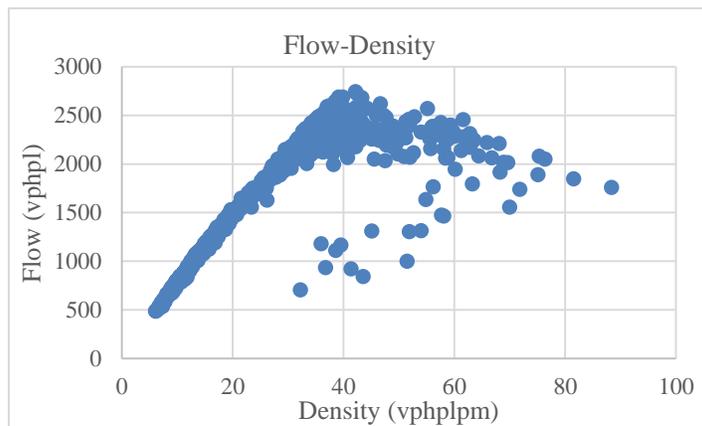
Figures 4-3 to 4-6 show the fundamental diagrams for the 5 minute aggregation level and for the 15 minutes time intervals in the morning and afternoon peaks, respectively. As shown in the figures, the data is reasonable and the relationship between macroscopic traffic parameters (flow, speed and density) are logical. The density is calculated as the quotient of the flow over the speed. At a first look, it appears that the capacity is around 2,100 to 2,500 veh/hour/lane (vphpl). The critical density at capacity appears to be 40 to 45 veh/mile/lane. The Highway Capacity Manual (HCM) is based on 15 minute analyses. The capacity and critical speed estimates are in line with the estimates based on the HCM analytical procedures.



(a) Speed-Flow Diagram for 5-Minute Interval

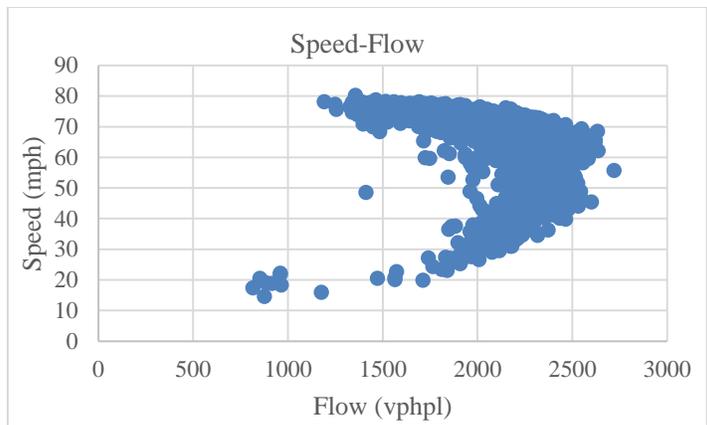


(b) Speed-Density Diagram for 5-Minute Interval

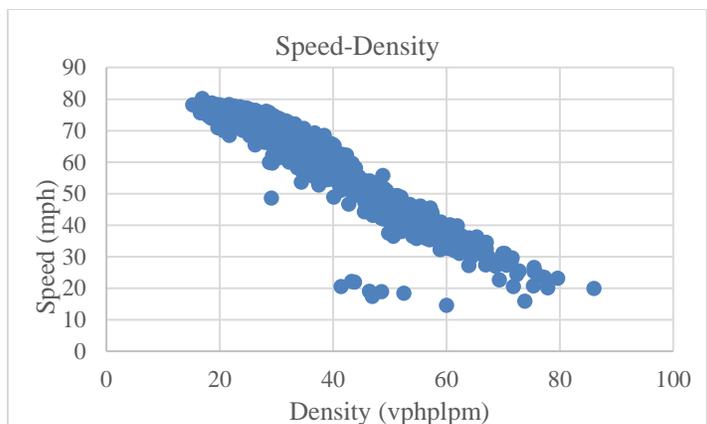


(c) Flow-Density Diagram for 5-Minute Interval

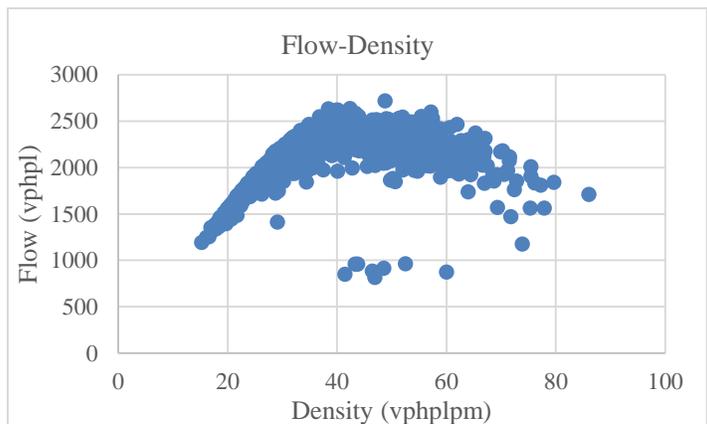
Figure 4-3 Fundamental Diagrams for Morning with 5-Minute Aggregation Level



(a) Speed-Flow Diagram for 5-Minute Interval

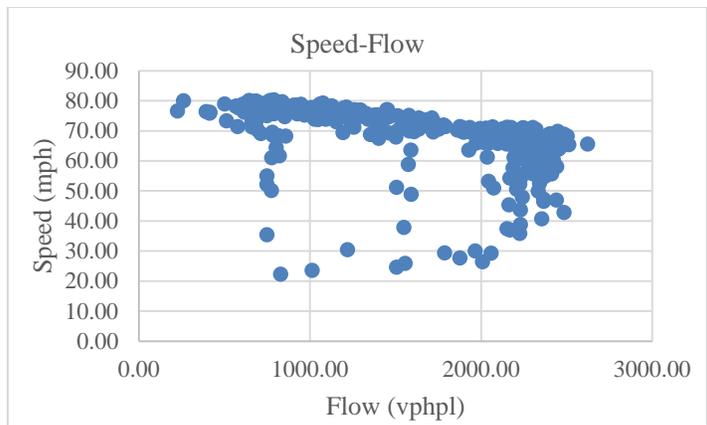


(b) Speed-Density Diagram for 5-Minute Interval

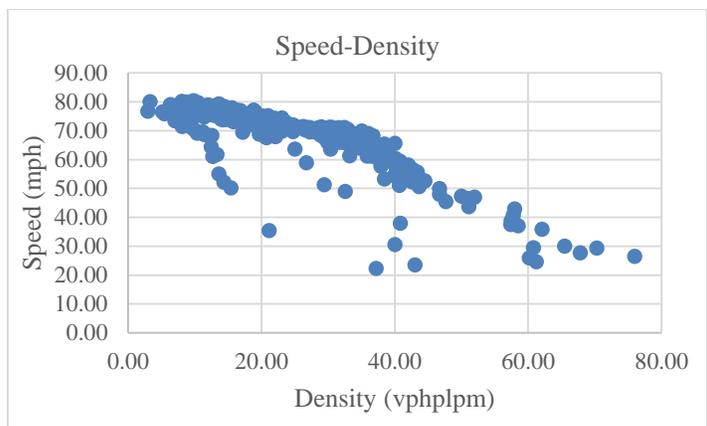


(c) Flow-Density Diagram for 5-Minute Interval

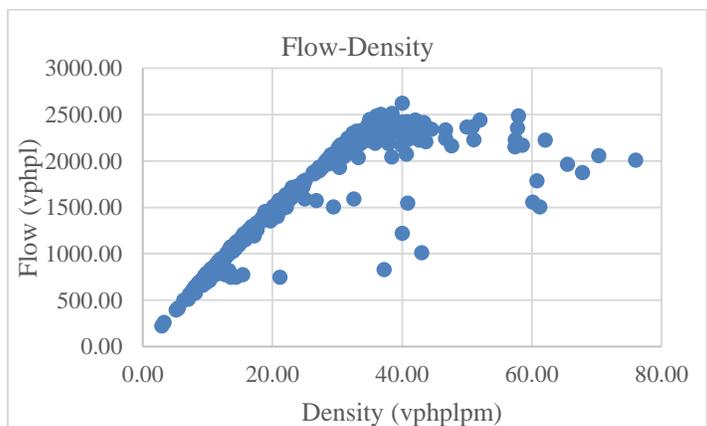
Figure 4-4 Fundamental Diagrams for Afternoon with 5-Minute Aggregation Level



(a) Speed-Flow Diagram for 15-Minute Interval

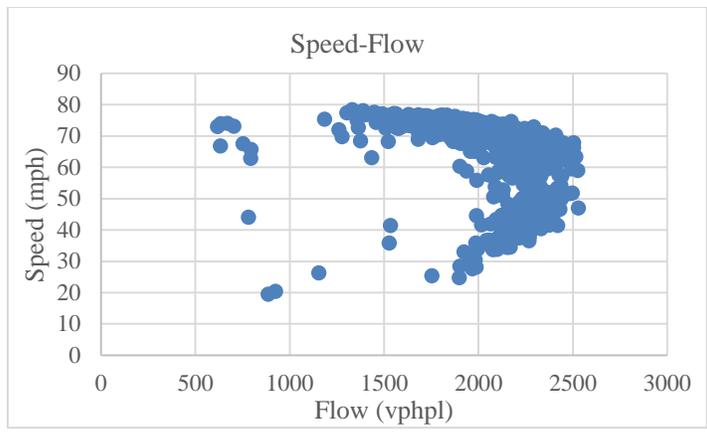


(b) Speed-Density Diagram for 15-Minute Interval

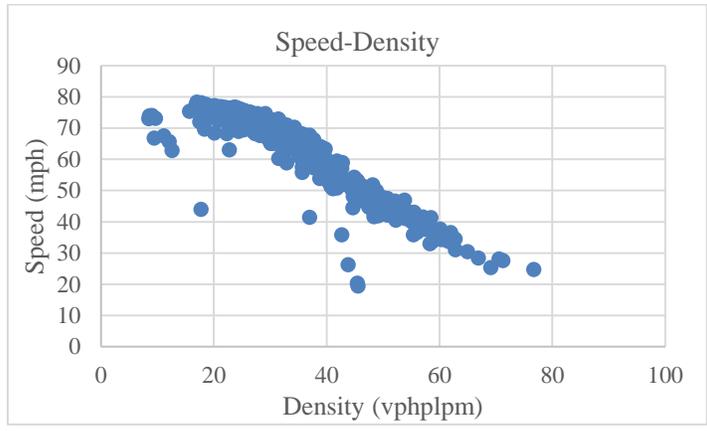


(c) Flow-Density Diagram for 15-Minute Interval

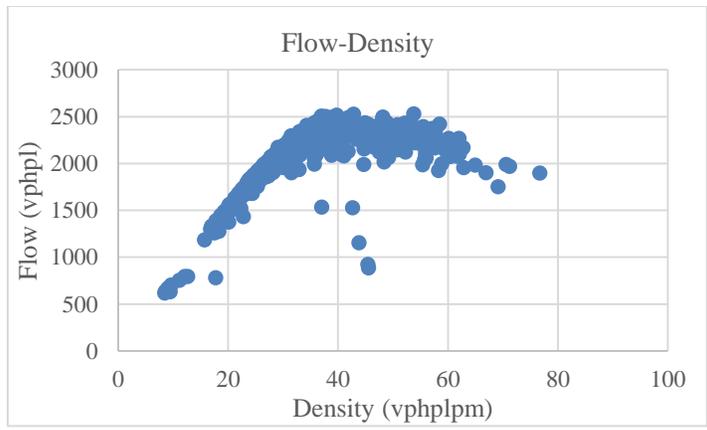
Figure 4-5 Fundamental Diagrams for Morning with 15-Minute Aggregation Level



(a) Speed-Flow Diagram for 15-Minute Interval



(b) Speed-Density Diagram for 15-Minute Interval



(c) Flow-Density Diagram for 15-Minute Interval

Figure 4-6 Fundamental Diagrams for Afternoon with 15-Minute Aggregation Level

4.1.2. Data Processing and Analysis

The first step in processing the data is preparation of the dataset to allow the development and testing of the warrants. Since the case study area is in the State of Florida, the ramp metering warrants in Florida are used as the base warrants for comparison as explained later in Chapter 5. The flowchart in Figure 4-7 represents the steps for applying the warrants (Gan et al., 2011). As shown in Figure 4-7, the application of the existing local warrants in Florida requires the following data:

- Mainline peak hour volumes
- Peak hour volumes on the rightmost lane
- Mainline peak hour speeds
- Number of the lanes on mainline and the on-ramps
- Ramp volumes
- Acceleration lengths
- Safety data

The same types of data, but at more detailed, are required for a potentially system-based method. Acceleration lengths and safety data collection will be addressed later in this chapter. This section focuses on traffic data.

The RITIS detector data can be downloaded in both zone- and lane-reading format. The zone-reading data files include the aggregated data over all lanes of the freeway mainline. However, the lane-reading data files provide detailed traffic data (volume, speed, and occupancy) for each lane on the freeway mainline. As each lane has a corresponding ID, the number of the lanes on freeway mainline is also obtained from these data files.

Since there is no detector installed for the on-ramps, ramp volumes are collected from the Florida Transportation Information (FTI) database over the years of 2013 and 2014. This data is less detailed than the SunGuide traffic detector data. They are collected for only three days of the year and are available from the FTI at the 15-minute aggregation level.

Recurrent and non-recurrent conditions are separated by filtering the eventful time intervals. The event data is also collected from RITIS database and the SunGuide database, as described later. Once the intervals are categorized based on the type of the event (event-free, incident, adverse weather, etc.), the analysis can be performed as required in this study.

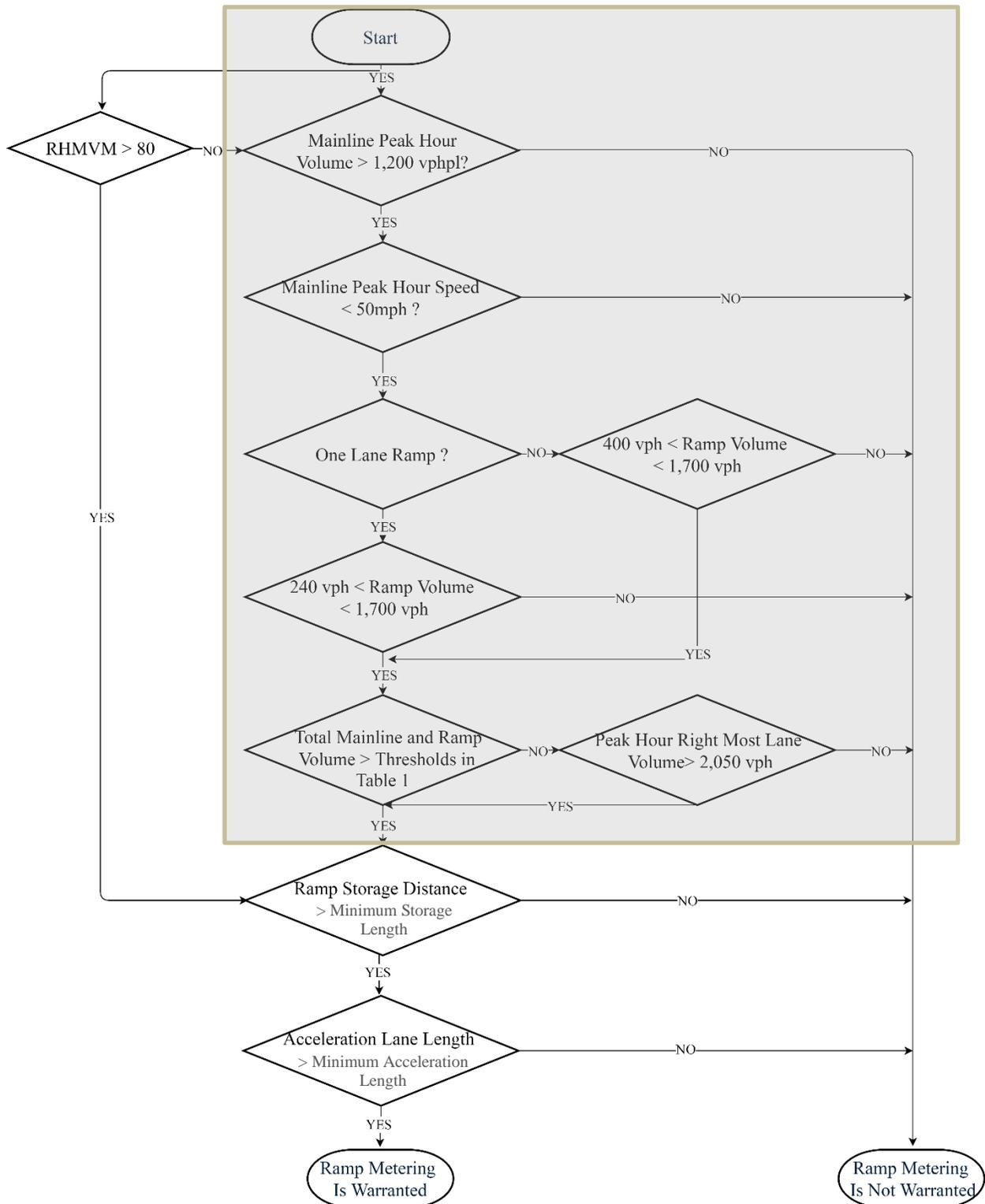


Figure 4-7 FDOT Ramp Metering Warrants Flowchart (Gan et al., 2011)

4.1.2.1. Stochastic Capacity Distribution

The capacity of the freeway is treated as a constant value in the current version of the Highway Capacity Manual (Transportation Research Record, 2010) and other traffic engineering guidelines. In 1996, Ponzlet showed that capacity can vary depending on environmental conditions such as light and pavement conditions as well as other operational conditions (Ponzlet, 1996). Other studies addressed the stochastic nature of the freeway capacity and showed that even under constant external conditions, various values of capacity can be observed (Elefteriadou et al., 1995; Lorenz and Elefteriadou, 2000; Okamura et al., 2000). These studies showed that there is a variation in the demand level, at which the flow of a freeway segment breaks down into stop-and-go operations.

To consider the bottleneck stochastic capacity in this research, first the recurrent bottleneck location is identified based on data of days with no events such as incidents, bad weather, and work zones. The bottlenecks are identified by detecting low speeds being propagated upstream, with free flowing (or near free-flowing) conditions occurring downstream. This identification is accomplished by examining the historical speed profiles at multiple locations upstream and downstream of the bottleneck (FHWA, 2015b). The FHWA recently introduced a tool for Congestion and Bottleneck Identification (CBI) which can identify bottleneck locations and their characteristics using numerical and graphical performance measures. Previous research suggested that a time interval of five minutes is the best compromise between the accuracy and simplicity of empirical analyses of breakdown at bottlenecks (Elefteriadou and Lertworawanich, 2003) and (Brilon et al., 2005). These studies utilize a speed value ranging between 45 mph and 55 mph to indicate the beginning of the breakdown. As described earlier, several state warrants including Florida use 50 mph as a criterion for local installation of ramp metering. Thus, this dissertation research uses the CBI tool methodology mentioned above with an average value of 50 mph as a threshold for bottleneck identification and five-minutes as the time interval of the analysis. However, other speed thresholds could be used by the analyst. As stated earlier, the main bottleneck location identified for the northbound direction of the study corridor is at the Commercial Boulevard exit, which involves a lane drop.

The next step of considering the stochasticity of freeway capacity at bottlenecks is to estimate the capacity distribution function using empirical analysis. This study utilizes the Product Limit Method (PLM) by Kaplan and Meier (1958), to estimate the distribution. PLM is used to estimate the probability of survival past given time points. In this case, the death refers to breakdown and the lifetime variable is the capacity (Brilon et al., 2005). Based on this method, the distribution function of the capacity could be derived as in Equation 4-1:

$$F_c(q) = 1 - \prod_{i:q_i < q} \frac{k_i - d_i}{k_i}; i \in B \quad (4-1)$$

where:

- $F_c(q)$ = Distribution function of capacity c , and
- q = Traffic volume (vph), and
- q_i = Traffic volume for interval i (vph), and
- k_i = Number of intervals with a traffic volume of $q \geq q_i$, and
- d_i = Number of breakdowns at a volume of q_i , and
- B = Set of breakdown intervals.

The set B includes traffic volumes, at which the average speed is higher than a threshold speed at time interval i and in the following time interval ($i+1$), the average speed drops below the threshold speed. Note that if the traffic is congested at the downstream cross section during the time interval of i or $i-1$, this case is excluded from set B , because interval i does not contain any information for the capacity assessment at the observation point (Elefteriadou and Lertworawanich, 2003).

In this study, the PLM method is applied to one year of archived speed and flow data. It should be mentioned that the PLM is a non-parametric method that does not require assumptions regarding the distribution function type. The results of the PLM method estimation and the fitted Weibull distribution based on the maximum likelihood method are shown in Figure 4-8. As illustrated in this figure; the 30th, 50th, 80th and 95th percentile of the bottleneck capacity equal to 2,242 vphpl, 2,278 vphpl, 2,382 vphpl, and 2,434 vphpl, respectively. It is interesting to note that the 50th percentile speed is 2,278 vphpl, which is close to the deterministic capacity estimated by the HCM 2016 (2,318 vphpl).

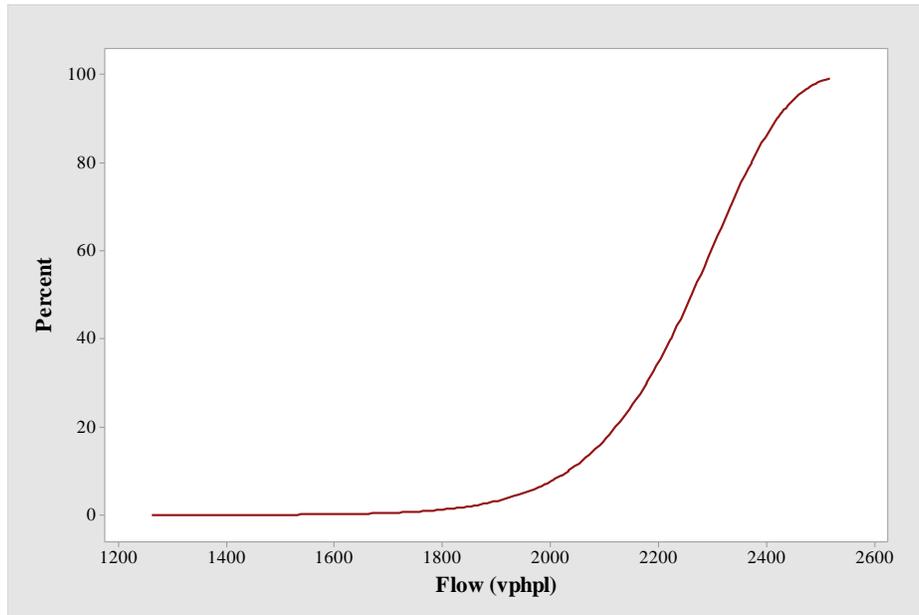


Figure 4-8 Stochastic Capacity Distribution for Bottleneck Location

Table 4-1 Percentile Values for Capacity at the Commercial Boulevard Bottleneck

Percentile	Capacity at Bottleneck Location (vphpl)
10	1,989
20	2,212
30	2,242
40	2,265
50	2,278
60	2,298
70	2,329
80	2,382
90	2,407
95	2,434

4.1.2.2. Stochastic Demand Distribution

Traffic demand is also a stochastic variable and this stochasticity is considered in this study. To derive the stochastic distributions of freeway demands, the historical detector volume data for the same period used in the capacity analysis, discussed in the previous section. The analyzed volume and speed data are for the event-free time intervals. Therefore, the first step is to filter the RITIS data to capture the event-free time intervals. All the time intervals with events such as incident, adverse weather, and work zone are excluded. This incident data includes the start and end time of each event as well as the event type. Knowing the start and end time of the event, all of the corridor data for the incident period is eliminated from the analysis.

The next step is analyzing the data. As 5 minute demand fluctuations is high, the data is aggregated to 15 minutes intervals. In order to aggregate the 5 minutes interval data to 15 minutes, the volume for each 15 minutes is simply calculated as the summation of the volumes for the three corresponding time intervals and the speed for each 15 minutes interval is the average of the three corresponding time intervals. If there is no queue at the bottleneck location, the demand is equal to the traffic volume measured at the bottleneck detection station. However, if there is a queue at the bottleneck location, the demand is calculated as the summation of the volume and the estimated additional number of the vehicles queued in the time interval, since the traffic volume by itself reflects capacity and not demand. If the speed for the considered time interval is lower than a speed threshold (45 mph) or higher than an occupancy threshold, a queue occurrence is identified at the detection station. The distance between each two detectors is divided into two equal lengths and each length is assigned to the nearest detector. If the aggregated speed over 15 minutes for a detector is less than the speed threshold, the assigned length to that detector would be added to the queue length. Therefore, the demand for each time interval is calculated using Equation 4-2:

$$D_i = F_i + Q_i - Q_{i-1} \quad (4-2)$$

where:

- D_i = Demand for time interval i
- F_i = Flow for time interval i
- Q_i = The entire queue length for time interval i
- Q_{i-1} = The entire queue length for time interval i-1

Once the demand for each time interval is calculated, the statistical distributions of the demands are plotted, as shown in Figures 4-9 and 4-10 for morning and afternoon peaks. The demand distribution for each 15 minute time interval is included in the appendix. Please, note that another potentially more accurate queue estimation method that could have been used is the cumulative arrival-departure volume-based method. However, that method requires the on-ramp and off-ramp volumes, in addition to mainline volumes. Unfortunately, the required ramp volumes were not available for this study since no detectors are installed on the ramps at this stage.

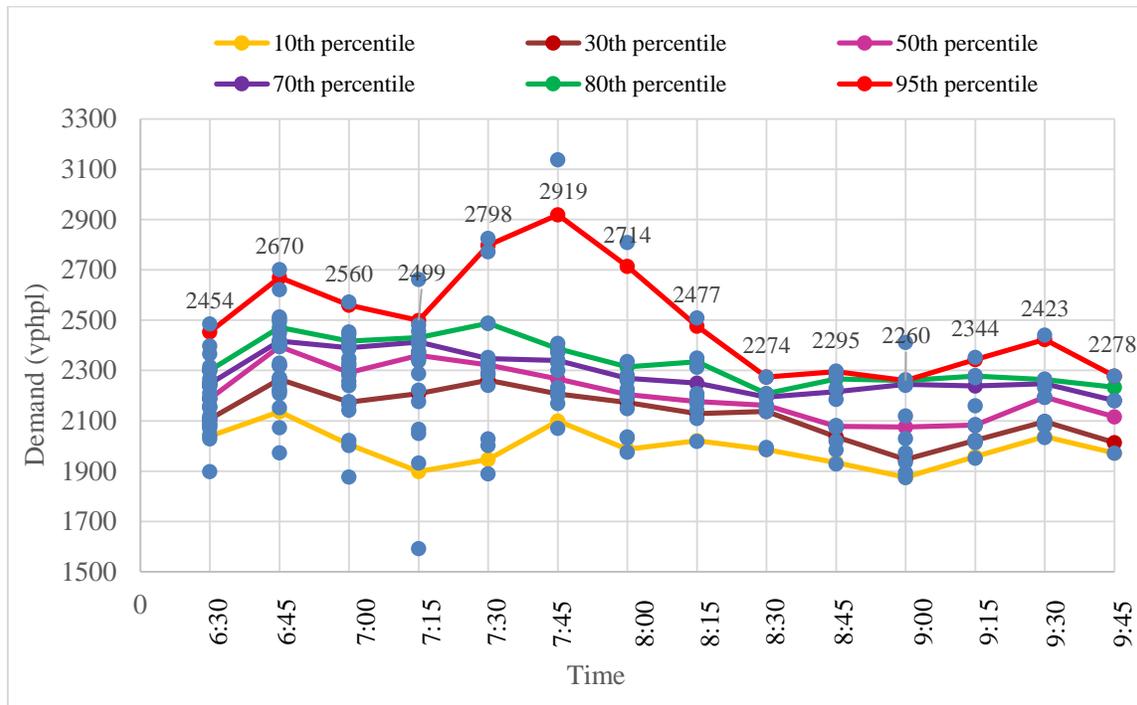


Figure 4-9 Historical Demand Distribution for the Morning Peak Period

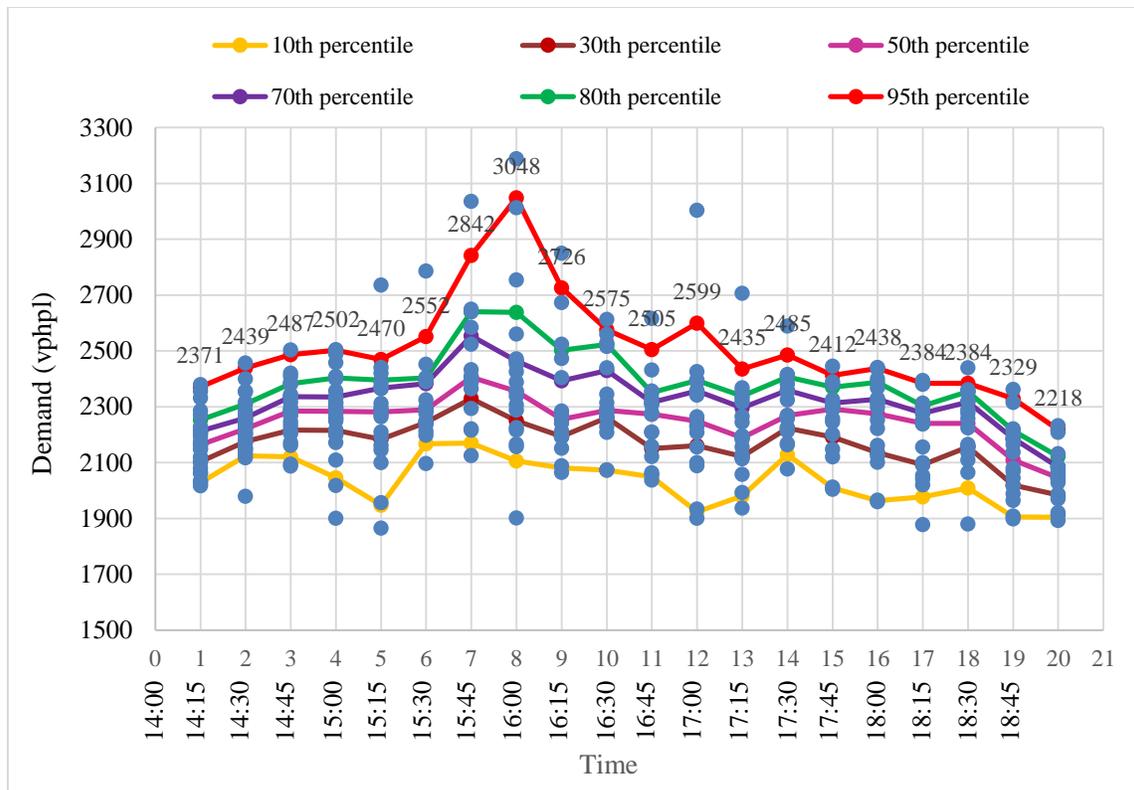


Figure 4-10 Historical Demand Distribution for the Afternoon Peak Period

4.1.2.3. Bottleneck Characteristics

Recurrent bottlenecks and their impacts on traffic operation is one of the main reasons to install ramp metering. Therefore, it is logical to use the characteristics of the bottlenecks in the decision-making process for installing ramp metering. In addition to the capacity and demand analysis, the bottleneck characteristics are included in warrant development processes. The dynamic variations in speed over time and space are analyzed to identify bottleneck characteristics.

Critical bottlenecks can be identified by low speeds being propagated upstream, with free flowing (or near free-flowing) conditions occurring downstream. This identification can be accomplished by examining historical speed profiles or occupancy at multiple locations upstream and downstream of the bottleneck. A bottleneck can be characterized by Duration, Intensity, Variability, and Extent (DIVE). Duration is the longest continuous time period during which the segment is congested. Intensity represents the percentage of an analysis box, which is congested, covering both time and space. Analysis box is a two-dimensional coordinate with location on the horizontal axis and time on the vertical axis. The analysis box is the area of influence of the bottleneck, considering multi-day operations. Extent is the longest continuous spatial length, during the congested time interval. Variability is the percentage difference between the intensity

of a specific day and the intensity of the average day (FHWA, 2015b). Table 4-2 and Table 4-3 represent the bottleneck characteristics in the morning and afternoon, respectively.

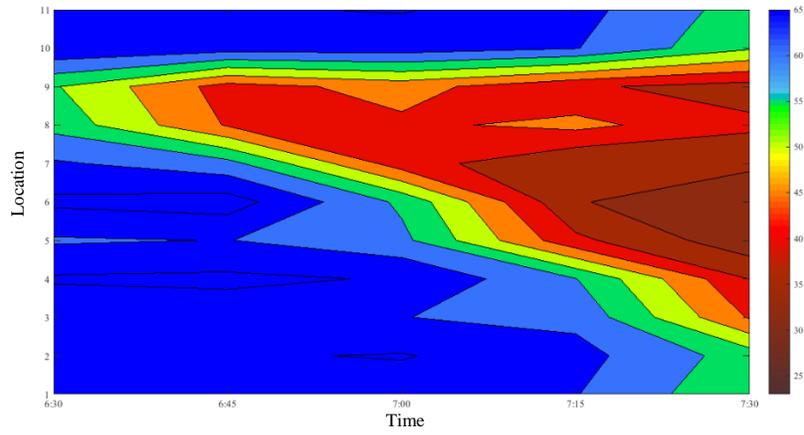
Table 4-2 Bottleneck Characteristics for the Morning

Date	Duration (min)	Extent (ft)	Intensity (%)	Variability (%)
3/31/2015	120	20,452	21.34	2.93
4/30/2015	60	15,075	13.49	10.79
7/1/2015	15	3,677	2.90	21.38
8/24/2015	30	11,769	35.25	10.97
9/3/2015	75	20,452	33.87	9.59
9/14/2015	15	20,452	17.55	6.73
12/9/2015	60	20,452	40.14	15.87
12/17/2015	60	17,782	24.28	0.00
12/21/2015	150	20,452	38.08	13.80

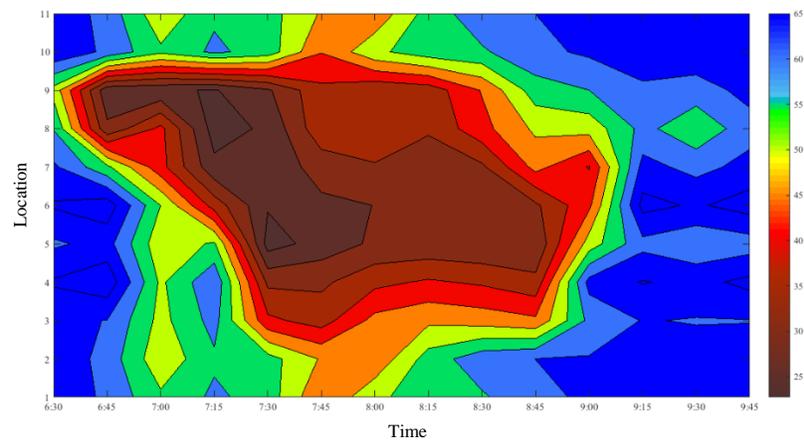
Table 4-3 Bottleneck Characteristics for the Afternoon

Date	Duration (min)	Extent (ft)	Intensity (%)	Variability (%)
3/31/2015	270	25,792	50.23	25.75
4/30/2015	90	15,075	8.72	15.76
9/30/2015	150	25,792	31.18	6.69
12/16/2015	225	23,122	35.86	11.38
12/17/2015	285	23,122	42.65	18.16
12/18/2015	210	15,075	24.86	0.37
12/21/2015	195	15,075	24.12	0.37
12/23/2015	195	17,782	13.22	11.26
12/24/2015	45	6,089	9.51	14.97
12/28/2015	90	15,075	9.14	15.35
12/29/2015	210	25,792	35.81	11.32
12/30/2015	90	11,769	12.55	11.94

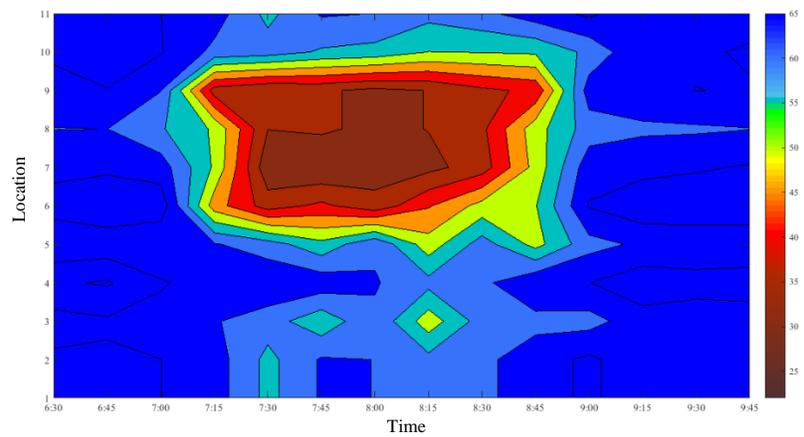
Figure 4-11 and 4-12 show the time-space-speed diagrams (also known as speed contour maps or speed heat maps) for the Morning and Afternoon peak periods, respectively. Figure 4-11 illustrates the speed contour maps for days with the 95th, 80th and 50th percentile of the intensity in the morning and Figure 4-12 illustrates the same for the afternoon period.



(a) Speed Contour Map for 95th Percentile Intensity

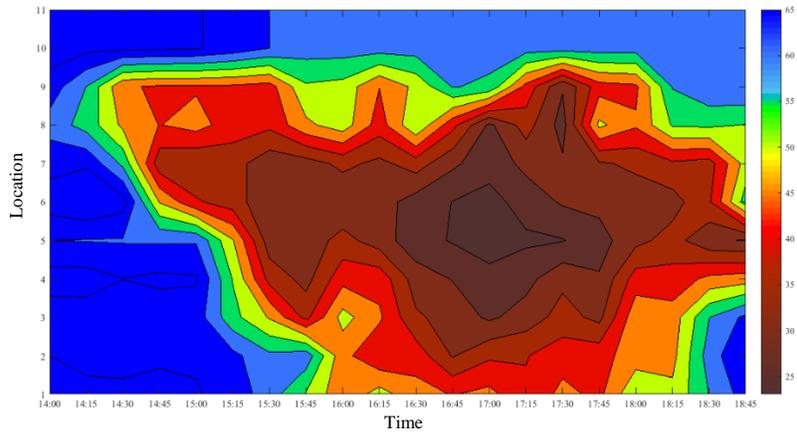


(b) Speed Contour Map for 80th Percentile Intensity

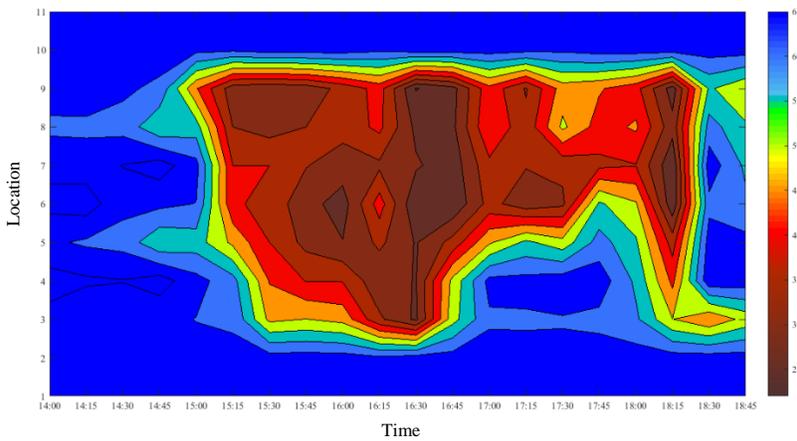


(c) Speed Contour Map for 50th Percentile Intensity

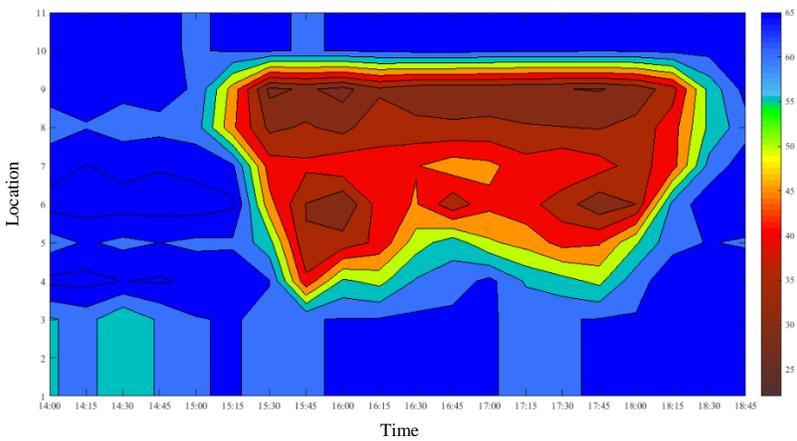
Figure 4-11 Speed Contour Maps for the Morning Peak



(a) Speed Contour Map for 95th Percentile Intensity



(b) Speed Contour Map for 80th Percentile Intensity



(c) Speed Contour Map for 50th Percentile Intensity

Figure 4-12 Speed Contour Maps for the Afternoon Peak

4.2. Acceleration Lane Limitations

One of the issues raised by the project stakeholders is the acceleration distance limitation as a decision criterion to install and operate ramp metering. The acceleration length is the distance which allows a vehicle starting from the stop bar at the ramp signal to reach the prevailing speed of the freeway traffic at the merge area. A minimum acceleration length is required for safe merging. The state warrants reviewed in Chapter 3 include specific criteria related to the acceleration lengths. However, these criteria may be difficult to meet in urban areas. It should be mentioned that the ramp metering stop bar is normally set some distance upstream of the gore area. However, this reduces the queue storage areas on the ramps. To support the examination of the application of the acceleration lane length for ramp metering warrants, existing acceleration lengths for on-ramps with ramp metering from multiple states of the United States were collected and compared. The acceleration lengths were measured from Google maps as shown in Figure 4-13. L_2 is defined as the distance from stop bar to the starting of gore area and L_3 is the distance from the starting of gore area to the end of the merging lane. The acceleration lane length is calculated as the summation of the L_2 and L_3 .

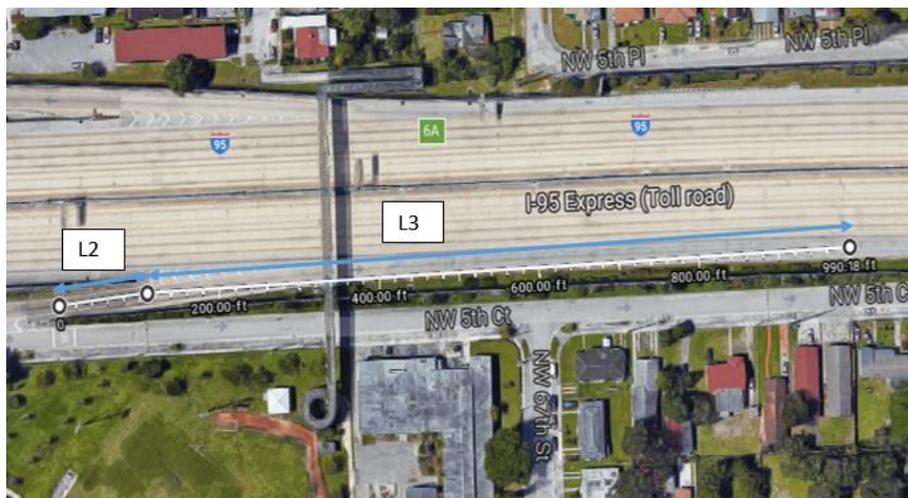


Figure 4-13 Acceleration Length Measurements

Four metered ramps with shorter acceleration lengths were selected for analysis from each of the following locations: Los Angeles, Denver, Minneapolis, Atlanta, Seattle, and Miami. The speed limits for all these facilities are 55 mph, except Seattle, where the corresponding speed limit is 60 mph. The minimum acceleration distances and maximum acceleration distances among the four selected ramps were identified and the corresponding state warrant for the minimum acceleration lengths are collected. Most of the states specify minimum acceleration lane lengths based on the AASHTO Policy on Geometric Design (AASHTO, 2011) considering the corresponding speed limit of the facilities and the starting from stop conditions of ramp metering.

Table 4-4 illustrates the minimum, average, and maximum acceleration lengths for the considered on-ramps, along with the minimum acceleration lane requirements for each city.

Table 4-4 Statistics of the Acceleration Lane Lengths

Cities	Minimum Acceleration Length (ft)	Average Acceleration Length (ft)	Maximum Acceleration Length (ft)	Minimum Acceleration length Required (ft)
Los Angeles	585	670	720	960
Denver	575	690	785	960
Minneapolis	425	527.5	635	1,000
Atlanta	500	581.25	725	960
Seattle	800	937.5	1,050	1,200
Miami	850	963.75	1,025	960

To better compare the required minimum acceleration length, Figure 4-14 plots the corresponding values for minimum, average and maximum lengths.

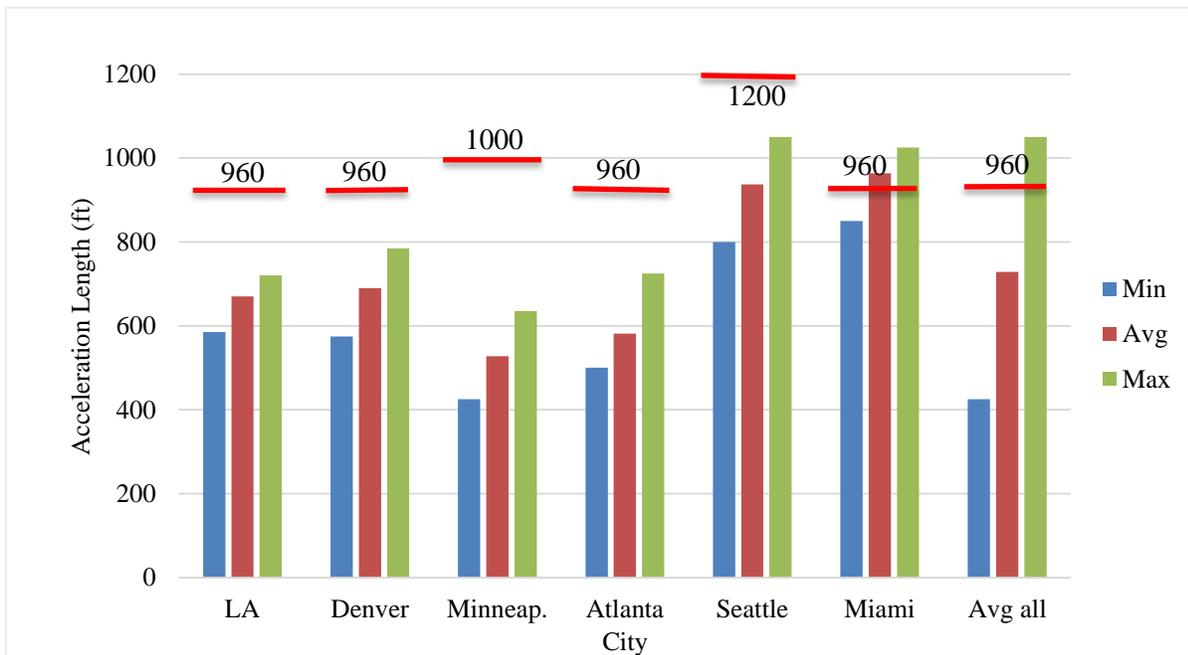


Figure 4-14 Statistics of the Acceleration Lane Lengths (Speed Limit of 55 and 60 mph)

As aforementioned, all the considered locations have a speed limit of 55 mph with the exception of Seattle. Therefore, to keep the consistency, Figure 4-15 includes data from all cities except Seattle to represent locations with 55 mph speed limit. From Figure 4-14 and Figure 4-15, it is clear that the minimum acceleration lengths found to be much lower than that of the requirements of the corresponding states.

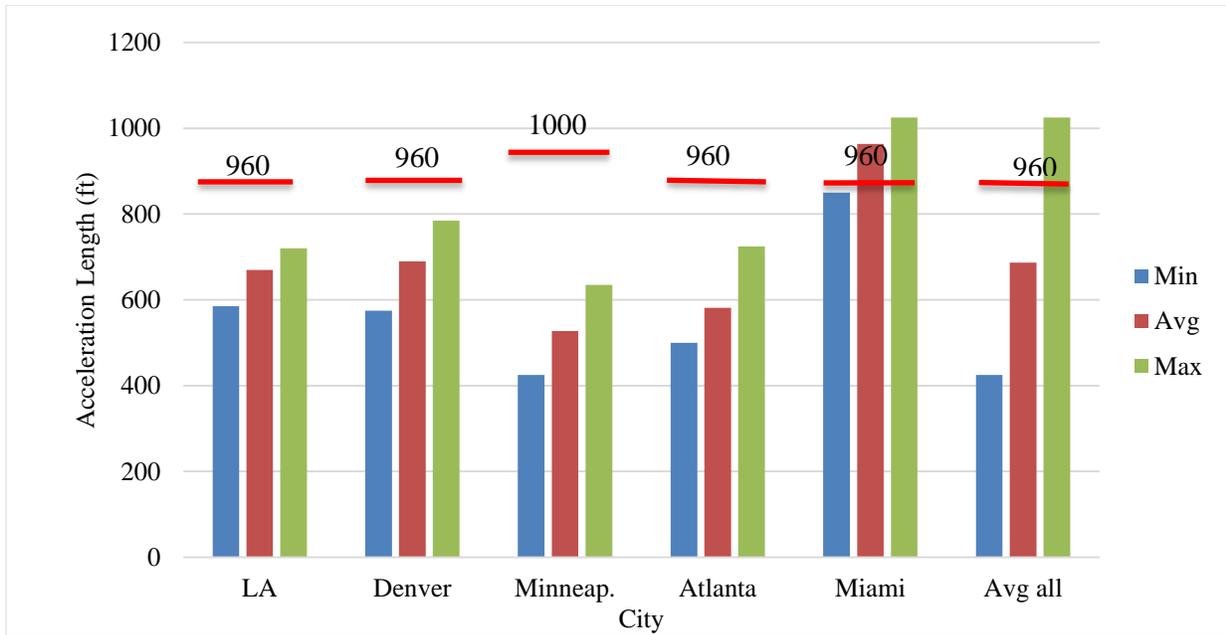


Figure 4-15 Statistics of the Acceleration Lane Lengths (Speed Limit of 55 mph)

Since the only investigated location with a speed limit of 60 mph is Seattle, more metered ramps were analyzed for this location. In addition, more data were also collected from the Miami deployment. A diagram for cumulative percentile of acceleration lengths for Miami, Seattle, and the other cities with 55 mph speed limit are plotted in Figure 4-16. Figure 4-16 shows that for the 50th percentile of acceleration lane length, the acceleration length is close to 650 ft for the 55 mph speed limit and 870ft for 60 mph speed limit. Similarly for 75th percentile, the minimum acceleration length is 700 ft for the 55 mph speed limit and 950 ft for 60 mph speed limit.

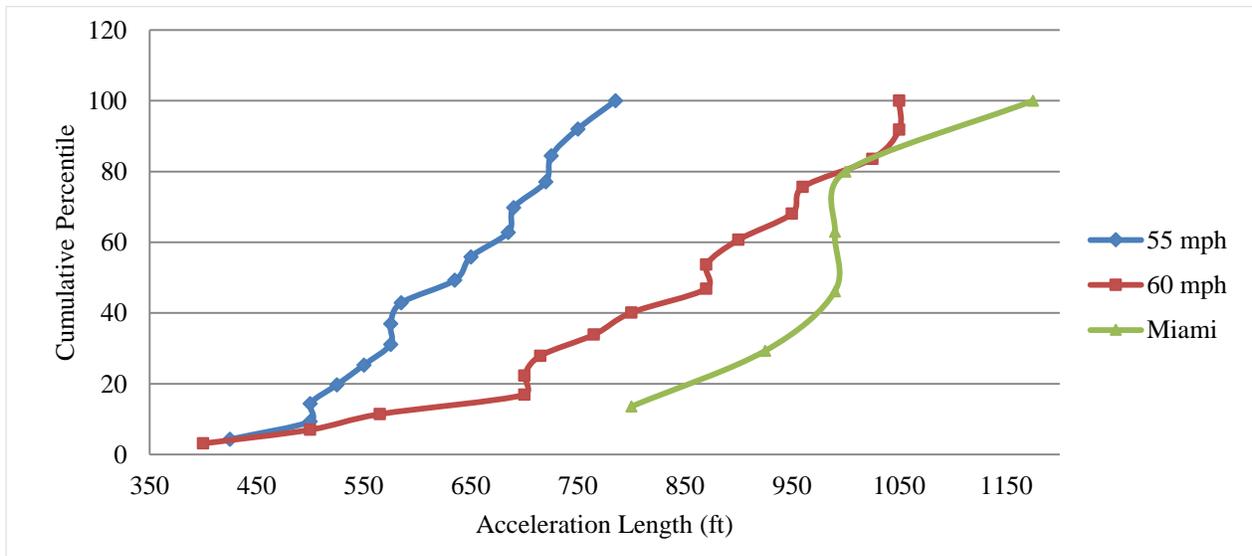


Figure 4-16 Cumulative Diagram of Acceleration Length for 55 mph, 60 mph, and Miami

4.3. Weather Data

As mentioned above, this study aims to differentiate between recurrent and non-recurrent conditions. This study specifically targets incident and adverse weather conditions which lead to non-recurrent congestions. As the study area is in the State of Florida and the most common adverse weather condition is rainfall, this study focuses on the rainfall event. To analyze the frequency and intensity of the rainfall, data was collected from the National Climate Data Center (NCDC) for the year 2015. The data was collected from two nearby weather stations at the Fort Lauderdale–Hollywood International Airport (FLL). This dataset includes the intensity of the rain in inches per hour for each 15 minutes interval. The resolution of the intensity is 0.01 inches per hour. Figure 4-17 and Figure 4-18 show the rainfall intensity distribution for the morning (6:30 am to 10:00 am) and the afternoon (2:00 pm -7:00 pm), respectively. Table 4-5 represents the percentile values.

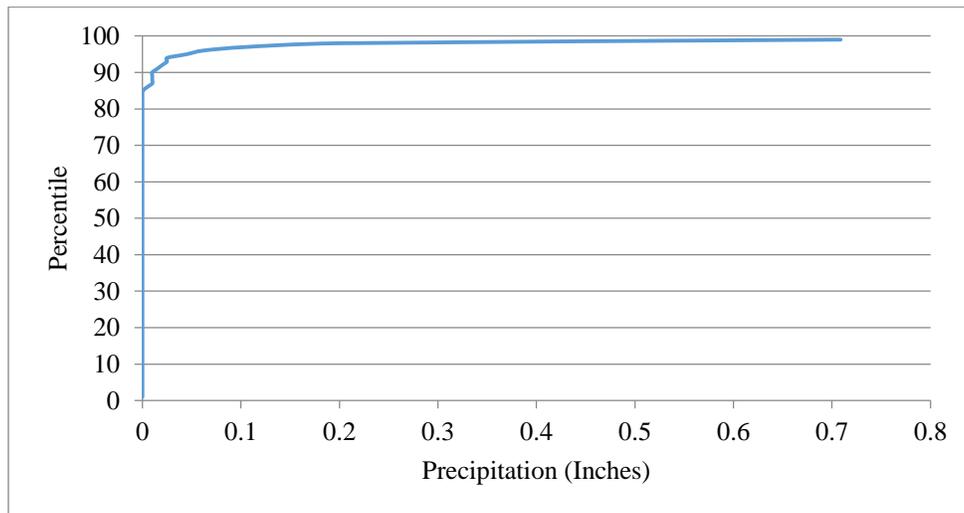


Figure 4-17 Rain Intensity Distribution in Morning

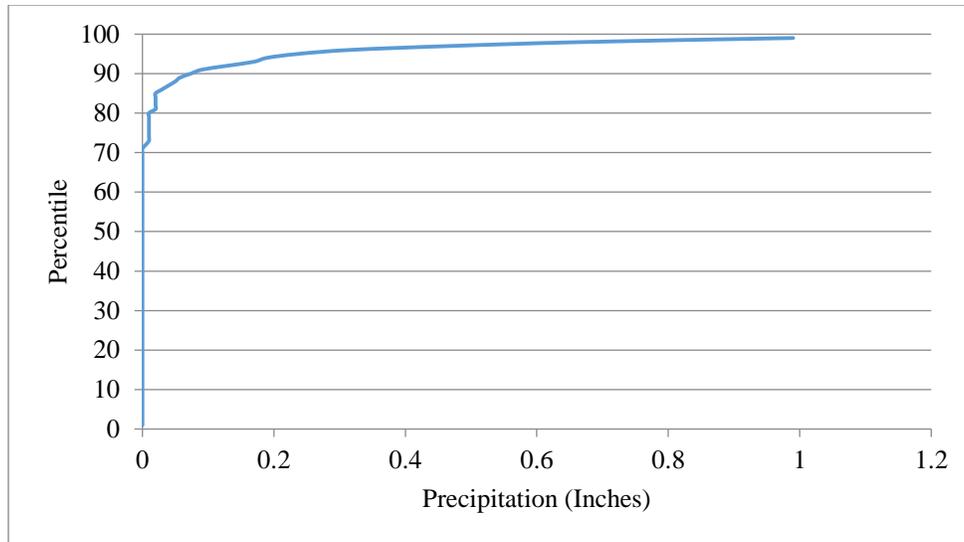


Figure 4-18 Rain Intensity Distribution in Afternoon

Table 4-5 Rain Intensity Percentiles

Percentile	Rain Intensity (in/h)	
	Morning	Afternoon
10	0.00	0.00
20	0.00	0.00
30	0.00	0.00
40	0.00	0.00
50	0.00	0.00
60	0.00	0.00
70	0.00	0.00
80	0.00	0.01
90	0.01	0.07
95	0.05	0.24

4.4. Incident Data

A detailed incident database obtained from FDOT District 4 traffic management center was analyzed in this study. The incidents were analyzed for the I-95 northbound direction from Hallandale Beach Blvd. ramp to the Commercial Blvd. ramp in 2015. The segments for the incident analysis were set from an on-ramp gore to the next on-ramp gore. The types of incidents included in the analysis are crashes, disabled vehicles, debris, emergency vehicles, interagency coordination, and vehicle Fire. Initially all incidents were included in the analysis. However, it was determined that better results could be obtained if incidents with durations exceeding 90 minutes were excluded from the analysis. Table 4-6 presents the results of the analysis for the whole segment (between Commercial Blvd. and Hallandale Beach Blvd.) without filtering out

the incidents that are longer than 90 minutes. Table 4-7 shows the results of the analyses with the 90 minute limit. Appendix B presents the incident statistics for each segment without isolating the long incident duration. Appendix C presents the incident statistics for each segment after filtering out the long incident duration.

Table 4-6 Incident Statistics from Hallandale Beach Blvd. to Commercial Blvd. without Filtering Out the Long Duration Incidents

Peak	Main Lanes Blocked	Number of Incidents	Incident Durations (Minutes)					
			Average	Min	Max	P95 th	P80 th	P50 th
AM	0 (Shoulder Only)	214	88	1	643	265.15	172.6	52
AM	1	101	93	2	346	215	153	86
AM	2	38	122	12	378	226.65	170.8	111
AM	3+	17	138	41	277	228.2	197.4	118
PM	0 (Shoulder Only)	251	90	0	574	318.5	181	37
PM	1	142	103	2	451	292.9	168.4	77
PM	2	59	127	4	462	284.6	198.2	116
PM	3+	32	142	42	341	312	207	127

Table 4-7 Incident Statistics from Hallandale Beach Blvd to Commercial Blvd after Filtering Out the Long Duration Incidents (More than 90 Minutes)

Peak	Main Lanes Blocked	Number of Incidents	Incident Durations (Minutes)					
			Average	Min	Max	P95 th	P80 th	P50 th
AM	0 (Shoulder Only)	133	28	1	90	79.4	48.6	19
AM	1	54	38	2	90	86.35	67.4	25.5
AM	2	15	59	12	90	83.7	78.2	65
AM	3+	4	69	41	90	89.1	86.4	73.5
PM	0 (Shoulder Only)	164	24	0	89	77.85	40	16
PM	1	79	39	2	90	85.3	65.8	33
PM	2	25	51	4	90	87.8	79.8	55
PM	3+	10	66	42	89	87.65	83.6	65.5

4.5. Crash Data

The source of the data used in the analysis of crash data was the FDOT District 4 incident database and crash data downloaded from the Signal Four Analytics website developed by the University of Florida.

5. ASSESSMENT OF THE NEED FOR SYSTEM-BASED WARRANTS

The review of the literature, presented in Chapter 3, clearly indicates that all existing metering warrants are based on local traffic conditions in the vicinity of the on-ramps. On the other hand, system-wide ramp metering algorithms consider also traffic conditions in a broader area around the subject ramp that includes downstream bottlenecks, as well as upstream and downstream ramps. This realization points to the need for developing additional warrants that consider system conditions to bridge the gap between the installation warrants at the planning stage and the actual operations of the meters.

An exploration was conducted in this study to assess the need for system-based warrants in addition to the existing local warrants. First, the existing local ramp metering warrants were applied to the ramps in the case study area in order to determine which ramps to meter according to these warrants. Second, a system-based method was used to select ramps for metering. Then, the study compared the results obtained when metering only the ramps justified according to the existing warrants (referred to as locally justified ramps) with the results obtained when combining the metering of these ramps with metering the ramps justified according to system-wide consideration (system justified ramps). The comparison was done using the VISSIM microscopic simulation modeling. The following subsections discuss these steps in details.

5.1. Applying Existing Ramp Metering Warrants

Since the case study area is in the state of Florida, the ramp metering warrants in Florida are used as the base warrants for comparison (Gan et al., 2011). The flowchart illustrated in Figure 4-7 represents the steps for applying the warrants. This flowchart is applied to each of the on-ramps in the study area to check which ramps are identified to be metered according to the warrants. Since the main purpose of this study is to examine the warrants based on their traffic operation impacts, only the traffic warrants, shaded in gray in Figure 4-7, are considered when selecting the “Locally Justified Ramps”.

5.2. Selecting Ramps Based on System Bottlenecks

This section describes the methodology used to select ramps for metering based on a system bottleneck capacity (System Justified Ramps). The methodology considers the stochastic nature of capacity of the freeway bottlenecks and involves two main steps. The first is to derive the stochastic distribution of capacity at the freeway bottlenecks. The second is to select the ramps for metering based on different capacity percentiles. The methodology to determine the stochastic distribution of bottleneck capacity and the corresponding analysis results have been discussed in Section 4.1.2.1. The following discussion explains the methodology used in the second step.

To select the ramps to be metered based on system bottleneck, all the ramps in the investigated segment of the freeway ahead of the bottleneck are included in an optimization process based on a linear programming formulation, adopted from the model used as part of the FREQ analysis model (Leiman et al., 1991). The linear programming formulation requires the bottleneck capacity as an input, in addition to the origin-destination demands between the mainline entry points, on-ramps, off-ramps, and mainline exit point within a considered system. The output of the model is the optimized metering rates for all the ramps in the subject system. The objective of the optimization is to maximize the vehicle throughput. The objective function used in this study is adopted from the one used in the FREQ model (May, 1976). However, the constraints are modified to fit the case study under consideration and data format. Additional constraints to account for other factors such as queuing capacity on the ramps, agency preferences, and so on could be added to the formulation as needed. The objective function and constraints of the linear programming are presented in Equation 5-1. Please note that the formulation of Equation 5-1 is extended later in Chapter 6 and used as part of the methodology developed in this project (see Section 6.1).

$$\text{Objective Function: } MAX \sum_{i=1}^n x_i \quad , i = 1, 2, \dots, n \quad (5-1)$$

Constraints;

$$\sum_{i=1}^n x_i + e - \sum_{j=1}^l y_j \leq B_k \quad , k = 1, 2, \dots, m, \quad j = 1, 2, \dots, l$$

$$x_{i,min} \leq x_i \leq V_i \quad , i = 1, 2, \dots, n$$

where,

x_i = Optimum ramp metering rate for on-ramp i (vph), and

n = Number of on-ramps (1, ..., i , ..., n), and

y_j = Ramp demand for off-ramp j (vph), and

e = Upstream mainline volume, and

B_k = Capacity of subsection k , and

m = Number of subsections, and

V_i = Demand rate at on-ramp i , and

$x_{i,min}$ = Minimum metering rate for on-ramp i .

The first constraint in the above equation limits the mainline volume at a subsection k to the total of entering flow from the on-ramp and upstream mainline minus the exiting flow from the off-ramps to be less than the capacity of subsection k . The value of the capacity can be selected from the capacity distribution as a desired percentile based on agency specifications. The second constraint requires the metering rate to be more than the minimum metering rate and less than the demand of the subject ramp. In this study, the minimum metering rate is calculated based on the estimated queue storage capacity of the subject on-ramp. The current Florida ramp metering warrants use Equation 5-2 to calculate the estimated queue length and minimum storage length

(L_s) (Gan et al., 2011). Thus, Equation 5-3 is used to represent the minimum metering rate $x_{i,min}$ for each on-ramp i .

$$L_s = 0.25 V - 0.00007422 V^2 \quad (5-2)$$

$$\begin{aligned} x_{i,min}(vph) &= V_i(vph) - L_s(meter) \times \frac{1 \text{ vehicle}}{6.1(meter)} \\ &= 0.9590164 V_i + 0.00001216 V_i^2 \end{aligned} \quad (5-3)$$

where,

L_s = Estimated queuing length on the ramp (meter) = minimum storage distance for one lane (meter), and

V_i = Peak hour ramp demand for ramp i (vph), and

$x_{i,min}$ = Minimum metering rate for on-ramp i .

If the resulting metering rate from applying the above methodology for a ramp is lower than the actual ramp volume, the ramp is considered for metering. Otherwise, the ramp is not justified to be metered.

5.3. Comparison Based on Simulation

The case study area was modeled in the VISSIM microscopic simulation to assess the need for selecting ramps to install metering equipment based on system-wide conditions. The VISSIM model was first calibrated in accordance with the real-world flow data for each capacity percentile at the bottleneck location to ensure that the simulation replicates the actual network and operation. As part of the calibration, the driving behavior parameters of VISSIM were fine-tuned and the resulting capacities were entirely compatible with the results of Figure 4-8. The methodology based on Equations 5-1 and 5-3, as described above, was applied to calculate the metering rates for the scenarios when selecting the ramps based on the existing local warrants and based on system bottleneck consideration. In order to assess the potential improvements from considering the system bottleneck, the simulation results were used to estimate performance measures including travel time on the on-ramps and travel time on the freeway mainline. The “average speed” on the freeway mainline for each 15-minute time interval was also used in the comparison.

The travel times on the freeway mainline and the on-ramps were used to calculate the benefits of the System-Justified approach compared to the Locally-Justified approach in terms of delay savings. The delay savings calculated as the difference between the travel times and installation and operation and maintenance costs of the two approaches, are used to calculate the benefit-cost ratio. The costs are estimated based on information obtained from FDOT District 4 and the present worth of the ramp metering deployment is calculated based on initial costs and the recurrent benefits and costs during the project life.

5.4. Results and Discussion

Table 5-1 compares the results of the selection of ramps for metering with those obtained using the existing Florida Warrants. As shown in the table, only 4 out of the 10 ramps are justified to be metered based on the existing Florida traffic warrants. When considering the system bottleneck, 8 out of the 10 ramps are warranted to be metered with the 95th and 80th percentile capacity and 9 are warranted with the 50th and 30th percentile capacity at the bottleneck.

Table 5-1 List of Warranted Ramp Metering Based on Existing Florida Warrants and System-wide Consideration

I-95 On-ramp to	Locally Justified Ramps	System Justified Ramps			
		95 th percentile Bottleneck Capacity	80 th percentile Bottleneck Capacity	50 th percentile Bottleneck Capacity	30 th percentile Bottleneck Capacity
Oakland Park Blvd	✓	✓	✓	✓	✓
Sunrise Blvd	✗	✓	✓	✓	✓
Broward Blvd	✗	✗	✗	✓	✓
Griffin Rd	✓	✓	✓	✓	✓
Stirling Rd	✗	✓	✓	✓	✓
Davie Blvd	✗	✗	✗	✗	✗
Sheridan St	✓	✓	✓	✓	✓
Hollywood Blvd	✗	✓	✓	✓	✓
Pembroke Rd	✗	✓	✓	✓	✓
Hallandale Beach Blvd	✓	✓	✓	✓	✓

The VISSIM results for average speed for each 15-minutes time interval along the freeway mainline is presented in speed contours for unmetered, locally justified metering and system justified metering scenarios. These contours illustrate the start time of traffic breakdown as well as the severity of the speed drop by color-coding. Figure 5-1 represents the speed contours for the 95th and 80th percentile capacity and Figure 5-2 shows the contours for the 50th and 30th percentile capacity of the bottleneck with the two metering installation scenarios.

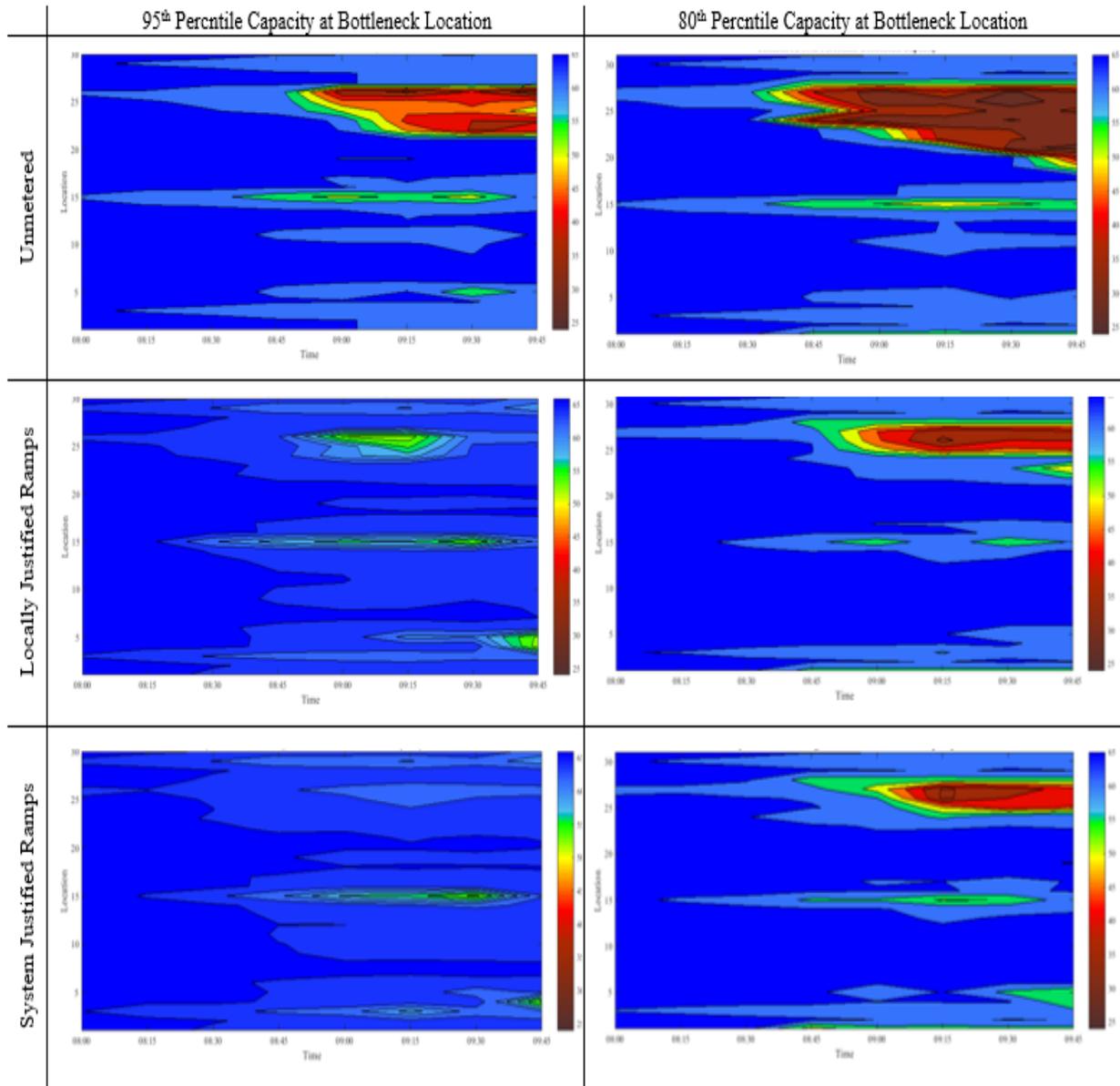


Figure 5-1 Impacts of Metering Strategies on Speed Contours for Freeway Mainline (95th and 80th Percentile Capacity at the Bottleneck)

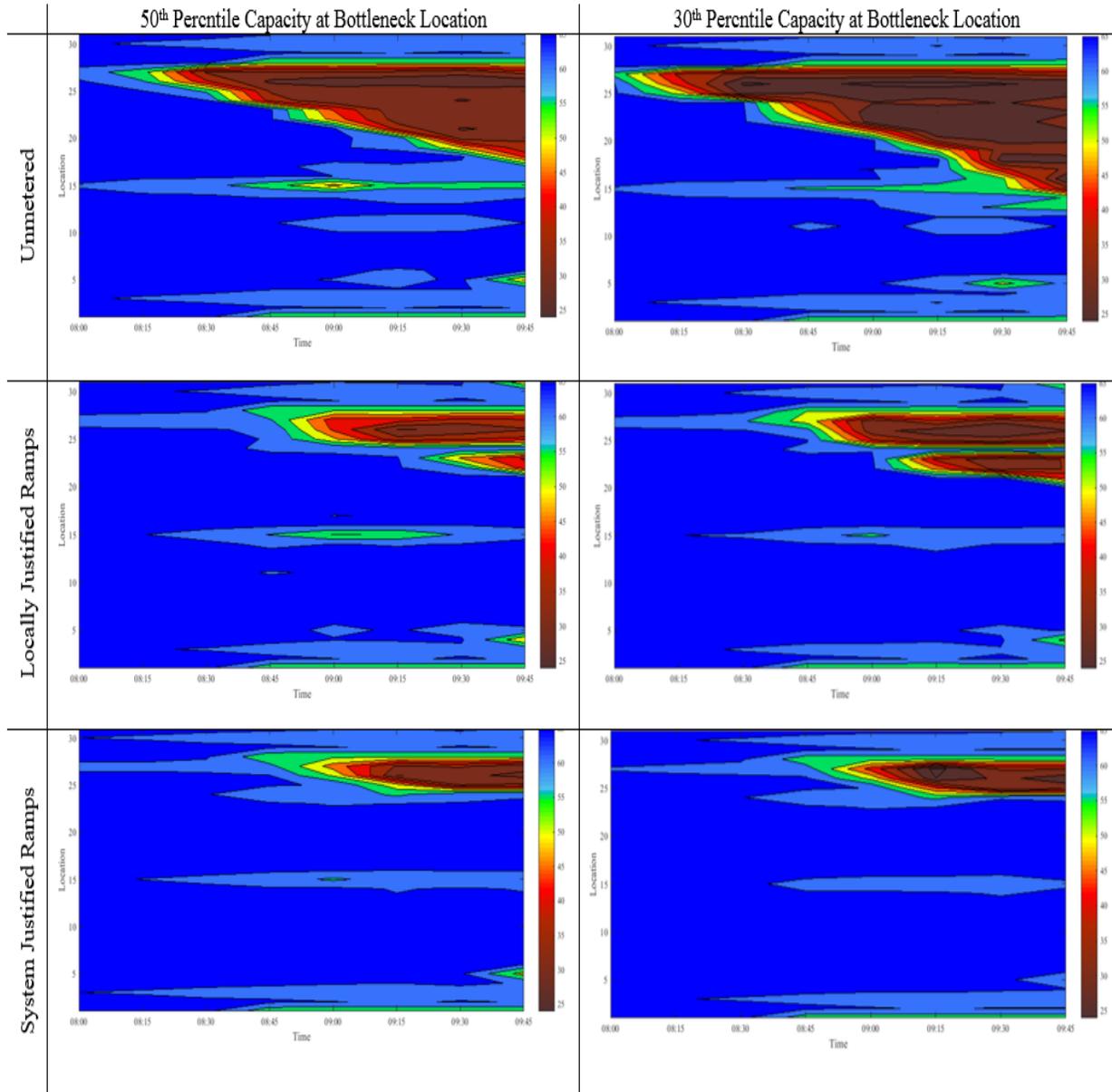


Figure 5-2 Impacts of Metering Strategies on Speed Contours for Freeway Mainline (50th and 30th Percentile Capacity at the Bottleneck)

As can be seen from Figure 5-1, in the case of the 95th percentile capacity, metering ramps based on existing warrants (locally-justified ramps) is able to prevent the breakdown but still there is an experienced speed drop at the bottleneck location. Metering the additional system-justified ramps was able to entirely prevent the speed drop at the bottleneck. In the case of the 80th percentile capacity at the bottleneck, Figure 5-2 shows that metering the locally-justified ramps postponed the breakdown for 15 minutes. The metering of the system-justified ramps postponed the breakdown by 30 minutes, resulting in a decrease in the spatial extent of the queue. In the case of the 50th and 30th percentile capacity at the bottleneck, shown in Figure 5-2, the lower capacity

resulted in an earlier and more severe breakdown. This makes it difficult to prevent breakdown with ramp metering, as tested in this research. However, in both cases, the locally-justified metering could postpone the breakdown at the bottleneck. Nonetheless, the system-justifiable metering could postpone the breakdown more and also is able to further limit the physical extent of the breakdown propagation. It should be mentioned that the comparison in this study utilized time-of-day metering to simplify the comparison. The implementation of adaptive metering may be able to prevent the breakdown or at least delay it more than what is identified in this evaluation, even with the 50th and 30th bottleneck capacities.

Figure 5-3 and Figure 5-4 illustrate the results for the travel time on freeway mainline and on-ramps, respectively. The mainline travel time is the average value for the travel time along the area, which is most affected by the bottleneck (end of the queue spillback and speed drop area due to the breakdown at the bottleneck).

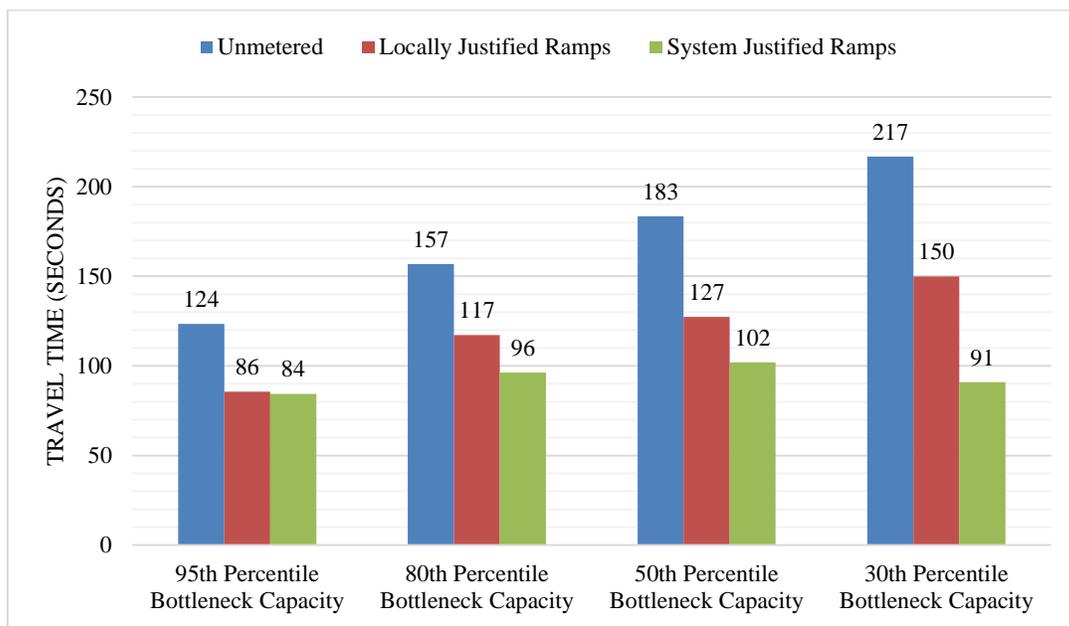


Figure 5-3 Travel Time on the Freeway Mainline with Different Percentiles of Bottleneck Capacity and Metering Scenarios

As shown in Figure 5-3, the application of ramp metering for locally justified ramps reduced the travel time by an average of 29.4% compared to unmetered condition. Whereas, metering the system justified ramps reduced the travel time by an average of 43.4% compared to unmetered conditions and 14% compared to just metering locally justified ramps. Matched paired t-tests were conducted to identify the statistical significance of the difference between mainline travel time for the three different scenarios. The results showed that the difference between the unmetered conditions and metering the locally justified ramps is statistically significant ($p\text{-value}=0.002 < 0.05$) at the 5% significance level. Also the difference between metering system-

justified and metering locally-justified ramps is statistically significant ($p\text{-value}=0.044<0.05$) at the 5% significance level.

The average travel time for all the on-ramps in the corridor are plotted in Figure 5-4. As shown in this figure, a main benefit of the system-justified metering is the distribution of the long ramp queues observed when using locally-justified metering on the additional system-justified ramps. As illustrated in Figure 5-4, metering only locally-justified metering leads to a significant increase in travel time for the four metered ramps (Oakland Park Blvd, Griffin Rd, Sheridan St and Hallandale Beach Blvd). In the case of system-justified metering, the burden is distributed between more ramps therefore the impact on the four aforementioned ramps are significantly decreased. Paired t-tests were conducted to identify the statistical significance of the difference between the overall travel times for all the on-ramps. The results showed that application of locally justified ramps increased the overall travel time on the on-ramps compared to unmetered condition by 118% and this difference is statistically significant ($p\text{-value}=0.006<0.05$) at the 5% significance level. Also, the decrease in the overall travel time on the on-ramps resulting from metering the system justified ramps (29.3% reduction compared to locally justified ramps) is statistically significant ($p\text{-value}=0.002<0.05$) at the 5% significance level.

The difference of the average travel times per vehicle on the freeway mainline and the on-ramps is the delay saving for the System-Justified approach compared to the Locally-Justified approach. This value is multiplied by the corresponding volumes to calculate the total delay savings in veh-hr. This involves multiplying the delay savings of the mainline traffic by the mainline traffic volume and the delay saving of each on-ramp by the on-ramp flow. The monetary value of the delay saving is calculated by multiplying the delay savings in veh-hr by the value of time assumed to be \$13.0 per passenger-hr and by a passenger occupancy of 1.2 passengers/vehicle based on previous studies in the region. The annual benefit of the system-wide metering in the case study area is calculated assuming 3 hours of ramp metering for 46 business days in the morning and afternoon (6 hours in total). According the data from the year of 2015, 46 days of the days experienced recurrent conditions without an incident or rainfall event. The project life is assumed to be 7 years and the interest rate is assumed to be 6%. Table 5-2 includes the parameters for benefit-cost analysis. The results show that benefit-cost ratio of the System-Justified metering compare to Locally-Justified metering is about 2.06.

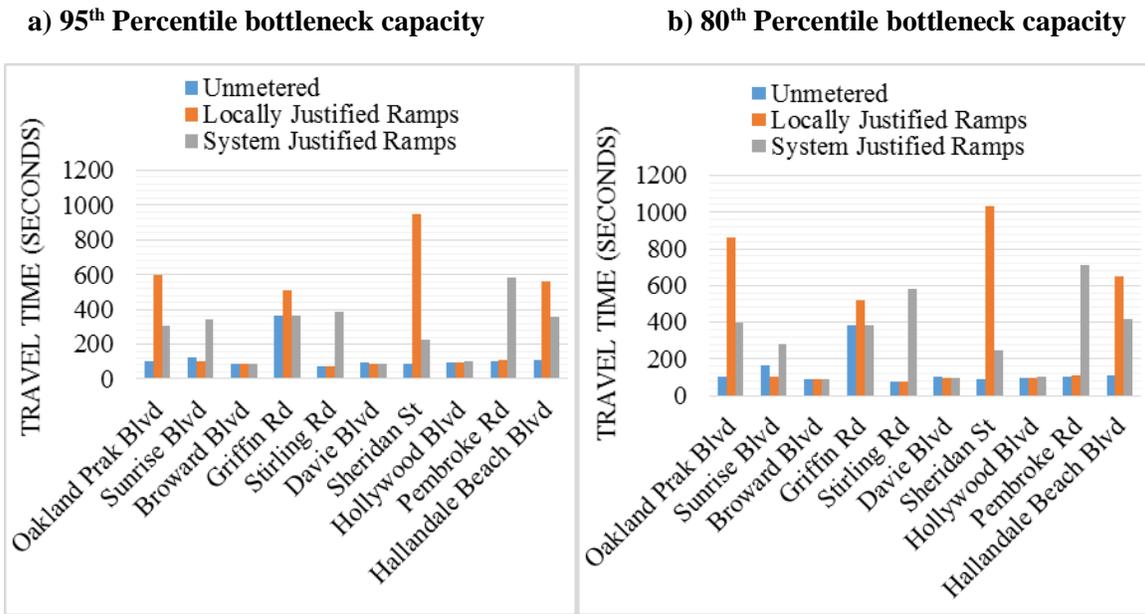
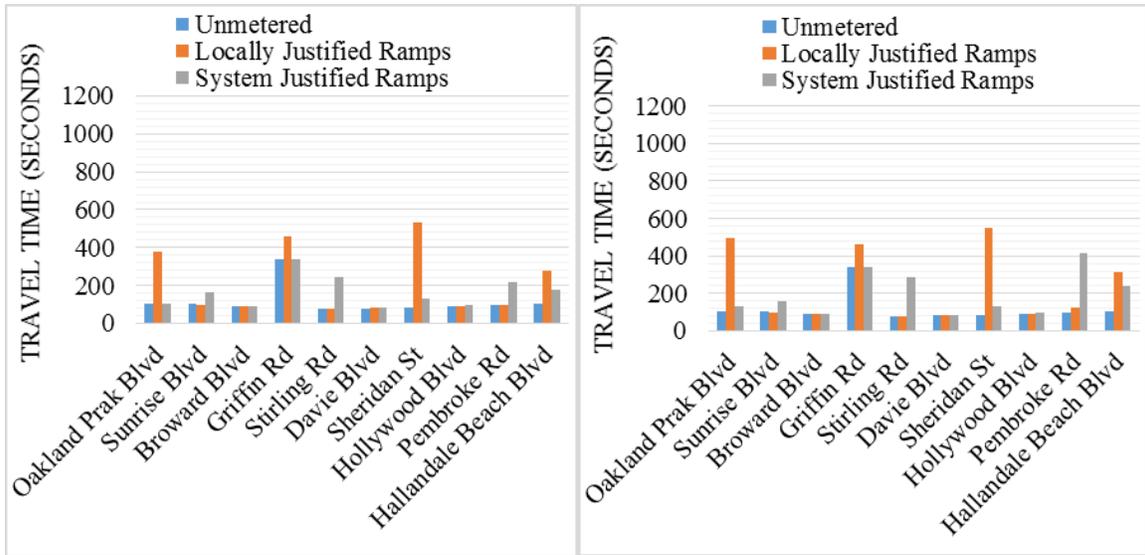


Figure 5-4 Travel Time on the On-ramps

Table 5-2 Delay Savings Due to System-Justified vs. Locally-Justified Ramp Metering and the Associated Benefit-Cost Analysis

Parameters		50th Percentile Capacity
Benefit	Total Delay Saving (veh-hr)	2,572
	Difference in Total Number of the Ramps (Local vs. System)	5
Cost	Difference in Capital Cost (\$)	1,812,000
	Difference in Maintenance Cost (\$)	66,240
Interest Parameters	Interest Rate	6%
	Project Life	7 years
Benefit-Cost Ratio		2.06

6. OFFLINE RAMP SELECTION FOR METERING DURING RECURRENT AND NON-RECURRENT CONDITIONS

This chapter describes a method developed in this study for offline selection of on-ramps for metering based on system-wide conditions of the freeway facility.

6.1. Formulation of the Identification of Ramps for Metering

The method developed in this study to select ramps for metering is based on a linear programming formulation that has been implemented in the past to estimate the ramp metering rate. The formulation was used as part of the FREQ freeway traffic analysis tool (Leiman et al. 1991; May, 1976). The method was extended in this study to consider the stochasticity of the demands and capacity and combined with derived models to estimate the impacts of ramp metering on bottleneck characteristics and performance measures. The linear programming formulation can be set to optimize the ramp metering operation to maximize the throughput of the system without violating constraints such as the minimum and maximum rates on each ramp and ramp queue lengths.

To select the ramps to be metered, all the ramps in a study area are included in the linear programming optimization. The linear programming formulation requires the bottleneck capacity as an input, in addition to the origin-destination demands between the mainline entry points, on-ramps, off-ramps, and mainline exit point of the considered system. Figure 6-1 illustrates a schematic representation of the variables of linear programming formulation.

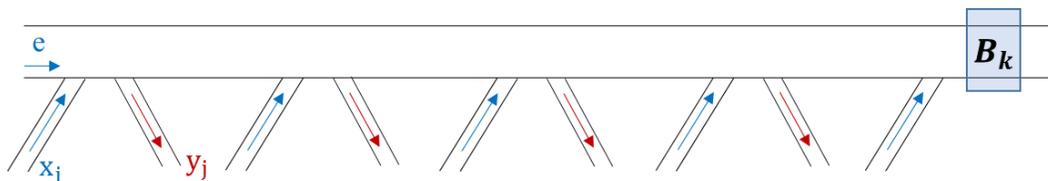


Figure 6-1 Schematic Demonstration for the Variables of Linear Programming Formulation

The output of the model is the optimized metering rates for all the ramps in the study area. The objective of the optimization is to maximize the vehicle throughput to keep the flow as close as possible but lower than the capacity to prevent the breakdown. The objective function used in this study is adopted from the one used in the FREQ model (May, 1976). However, the constraints are modified to fit the case study and data format. Additional constraints to account for other factors such as queuing capacity on the ramps, agency preferences, and so on could be added to the formulation, as needed. As discussed later in the document, safety and acceleration length consideration can be also considered as extensions of this formulation. The objective function and constraints of the linear programming are presented in Equation 6-1.

$$\text{Objective Function: } MAX \sum_{i=1}^n x_i \quad , i = 1, 2, \dots, n \quad (6-1)$$

Constraints;

$$\sum_{i=1}^n x_i + e - \sum_{j=1}^l y_j \leq B_k \quad , k = 1, 2, \dots, m, j = 1, 2, \dots, l$$

$$x_{i,min} \leq x_i \leq V_i \quad , i = 1, 2, \dots, n$$

$$x_{i,min} = V_i - \alpha S_i \quad , i = 1, 2, \dots, n$$

where:

x_i = Optimum ramp metering rate for on-ramp i (vph), and

n = Number of on-ramps (1, ..., i , ..., n), and

y_j = Ramp demand for off-ramp j (vph), and

e = Upstream mainline volume (vph), and

B_k = Capacity of subsection k (vph), and

m = Number of subsections, and

V_i = Demand rate at on-ramp i (vph), and

S_i = Available storage at on-ramp i (vph), and

$x_{i,min}$ = Minimum metering rate for on-ramp i (vph).

α = User specified portion of the maximum queue to be accommodate by the storage

The first constraint in Equation 6-1 requires that the mainline volume at a subsection k (the total of the entering flow from the on-ramps and upstream mainline minus the exiting flow from the off-ramps) to be less than the capacity of this subsection. In this study, the capacity and demand values utilized in the formulation are considered as stochastic variables and are generated from their distributions using Monte Carlo simulation, as discussed in details in the next section. The linear programming formulation was assessed with the values generated from each experiment of the Monte Carlo analysis. The second constraint in the formulation requires the metering rate to be more than the minimum metering rate and less than the demand of the subject ramp. The third constraint defines the minimum metering rate for each on-ramp as equal to the difference between the demand of the on-ramp and a multiplier (defining a portion) of the available queue storage (S_i), which is the distance from the metering stop line to a point on the ramp that is close to the upstream end of the ramp. The queue storage in feet is converted to vehicles by assuming that each queued vehicle and the associated distance headway occupy 25 feet and considering the ramp queue length and the number of lanes on the ramp. If the queue storage considering the distance between the metering stop line and the gore is known, then this storage should be used in the formulation. In reality, this storage requirement is a function of a number of factors including on-ramp demand, traffic flow arrival pattern, metering rate and algorithm, upstream signal control, and right turn-on-red vehicles (Tian et al. 2016). Based on the review of the literature presented in Chapter 3, one of the approaches for determining the minimum storage length on the on-ramp is to consider that the storage length should be able to accommodate at least 10 percent of the on-ramp demand (Gan et al., 2011). Tian et al. (2016) recommended that

queue storage length is designed as a certain percentage of the peak hour on-ramp demand with the maximum recommended percentage values are approximately 8 percent for urban arterial metered on-ramps. In this study, initially it is assumed that each on-ramp queue storage is set to accommodate at least 10 percent of the demand, therefore the minimum metering rate for each rate is calculated as 90 percent of the demand on the on-ramp. It should be mentioned that this value will be set as variant as a function of the actual ramp queuing storage and to put more constraints on queue lengths for specific ramps, as per agency requirements and identification of the queuing storage issues including preventing or reducing the spillback to upstream intersections on the arterial streets. This storage length is a function of the location of the metering stop line, which also affects the acceleration length of passenger cars and truck. This is discussed in Chapter 7.

An absolute minimum metering rate of 4 vehicles per minute (240 vehicles per hour) was also set for all ramps, as recommended in the literature. These percentages are user inputs and can change based on agencies policy. Further discussion is presented in the next section.

The above mentioned linear program provides the excessive entering flow from on-ramps which needs to be eliminated using ramp signals. This excessive volume equals to $\sum_{i=1}^n V_i - \sum_{i=1}^n x_i$. However, the regular linear programming distributes the excessive volume evenly between the on-ramps. This is not appropriate for the purpose of this study since the user may want to ensure that congestion can be addressed by metering the ramp(s) that are closer to the bottleneck before metering additional ramps. Thus, a modified programming formulation was deployed in this project to perform the selection of the ramps for metering starting from the ramp that is the closest to the system bottleneck location. If this ramp is unable to accommodate the excessive demand above capacity at the bottleneck, considering the constraints on the process listed in Equation 6-1, the metering is extended to the upstream ramp. This extension continues until the excessive volume is distributed between the ramps required to be meter. The metering rate is set to produce equal demand to capacity ratio on the ramps finally selected for metering.

6.2. Offline Ramp Selection for Recurrent Conditions

The developed methodology requires analyzing historical traffic data. An initial analysis was performed and reported in Chapter 4. This project considers the stochastic nature of the capacity and demand at the bottleneck location. Thus, the stochastic distributions of the capacity and demands at the bottleneck locations, together with the attributes of the bottleneck were analyzed and reported in Chapter 4. The analyzed attributes of the bottleneck included the duration, intensity, variability and extent (DIVE). Duration is the longest continuous time period during which the segment is congested. Intensity represents the percentage of an analysis box, which is congested, covering both time and space. The analysis box is a two-dimensional coordinate with location on the horizontal axis and time on the vertical axis that represents the area of influence

of the bottleneck, considering multi-day operations. Extent is the longest continuous spatial length, during the congested time interval. Variability is the percentage difference between the intensity of a specific day under consideration and the intensity of the average day (FHWA, 2015b). However, in this project, the overall day-to-day variability is also assessed as a measure of reliability, and defined as the mean absolute difference of the intensity of different days divided by the intensity of the median day. The DIVE measure has been recommended by the FHWA for use in assessing bottleneck operations.

Figure 6-2 represents a simplified summary of the main steps of the developed methodology to identify the locations that benefit from ramp metering. Step 1 involves collecting the input parameters for the analysis. These parameters are the demand distributions, capacity distributions, and bottleneck characteristics. Step 1 was conducted previously and the results are presented in Chapter 4. The following subsections discuss the remaining steps.

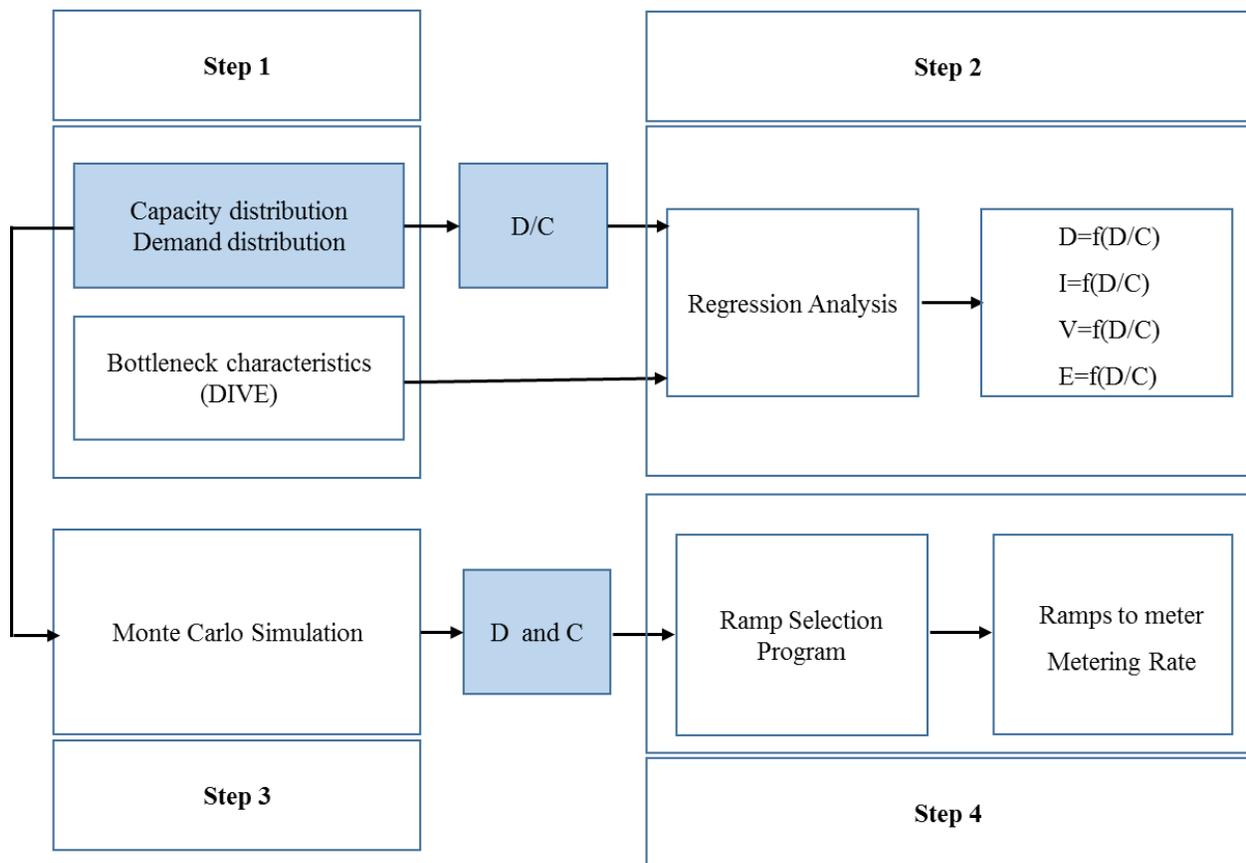


Figure 6-2 Main Steps of the Developed Methodology

6.2.1. Step 2: Regression Analysis to Estimate Bottleneck Characteristics

An aspect of the developed method is to estimate the impact of ramp metering on the traffic performance at the bottleneck locations. Thus, the second step is to relate the demand to capacity (D/C) ratio and the bottleneck characteristics (DIVE). To perform this step, the D/C for the congested event-free days and the associated bottleneck characteristics were fed into regression analyses to derive the relationships. Figures 6-3 to 6-6 represent the regression analysis results for the duration, intensity, variability and extent for the case study of the project (the I-95 northbound direction in Broward County, FL). Table 6-1 lists the estimated equations and their corresponding goodness-of-fit measure.

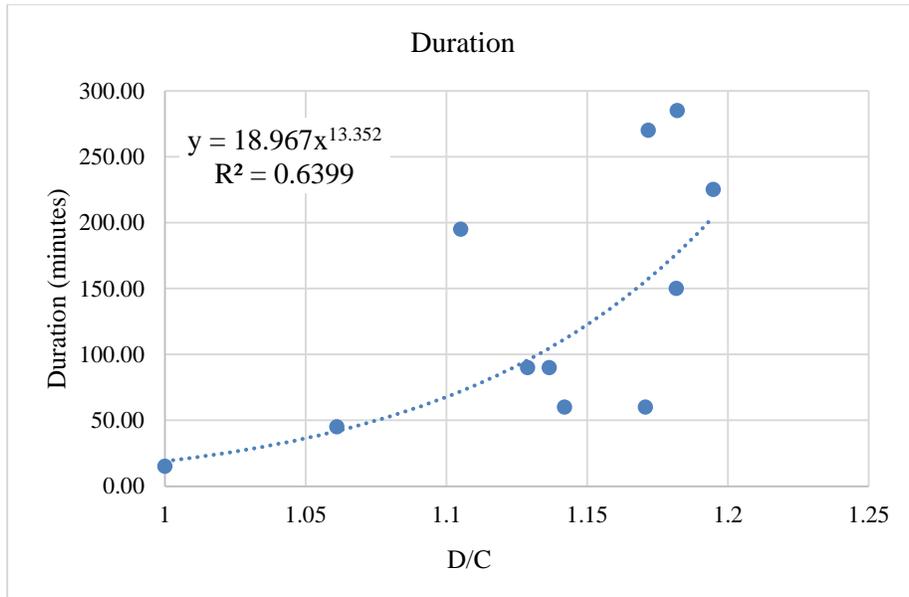


Figure 6-3 Regression Result for Duration

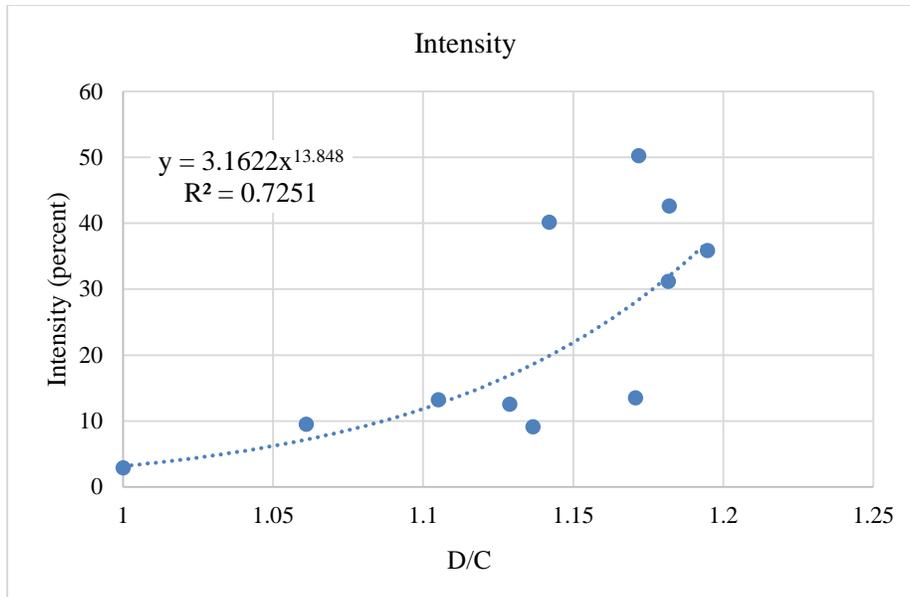


Figure 6-4 Regression Result for Intensity

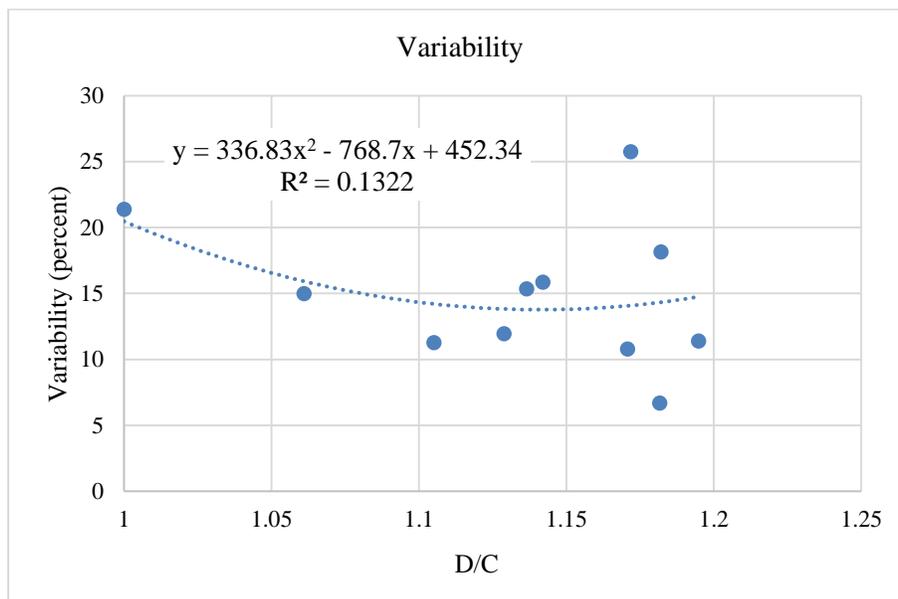


Figure 6-5 Regression Result for Variability

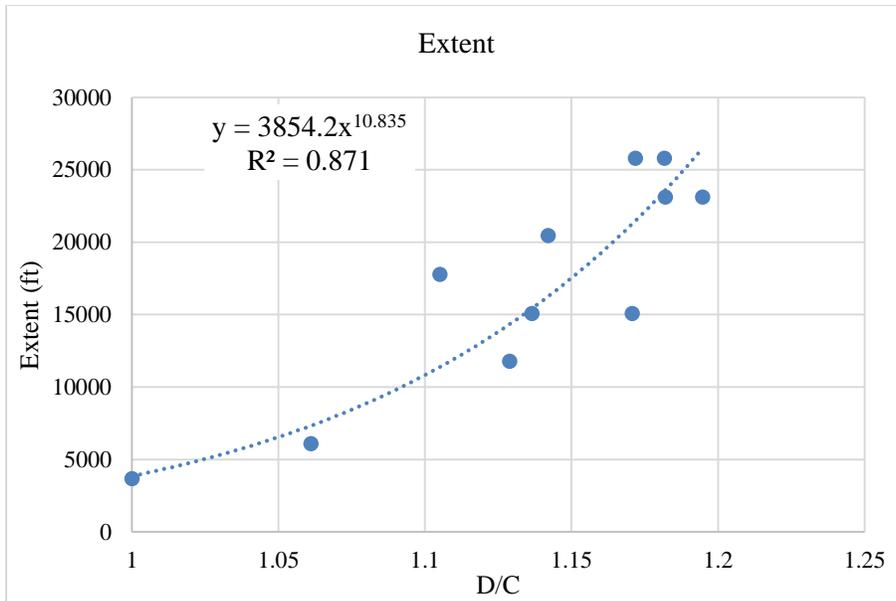


Figure 6-6 Regression Result for Extent

Table 6-1 Regression Analysis Results

Characteristic	Equation	a	b	R-Square (Goodness-of-Fit Measure)
Duration	$D_u = a \cdot (D/C)^b$	18.967	13.352	0.64
Intensity	$I = a \cdot (D/C)^b$	3.1622	13.848	0.77
Variability	---	---	---	---
Extent	$E = a \cdot (D/C)^b$	3,854.2	10.835	0.87

As shown in Table 6-1 and Figures 6-3 to 6-6, the regression analysis produced acceptable relationships between the D/C ratio and the duration, intensity and extent, as indicated by the R-Square values. Matched paired t-tests were also conducted to identify the statistical significance of the difference between the actual data and fitted model. The results showed that the p-values are 0.24, 0.26 and 0.41 for the duration, intensity and extent, respectively. Since these values are larger than 0.05, it can be concluded that the difference between observed and the estimated values using the fitted equation is not statistically significant at the 5% significance level.

With regards to the variability, no relationship was identified between the D/C ratio and variability. Variability, according to the FHWA bottleneck characteristics definition is the difference between the intensity for each day and the median intensity, as stated earlier. As D/C increases, the intensity of the median and the intensity of the considered day simultaneously increase, it was found that therefore the difference between these two variables does not follow a specific and meaningful pattern. As aforementioned, the overall day-to-day variability is also calculated as the mean absolute difference between the intensity of different days versus the

intensity of the median day. The overall variability is 10.23% for morning and %11.94 for the afternoon peak, respectively.

6.2.2. Step 3: Generating Analysis Scenarios Using Monte Carlo Simulation

In the real-world, different days involve different operation scenarios, as a result of the variations in demands and capacity, as described in Chapter 4. In this study, different analysis scenarios, representing different days, are generated using a Monte Carlo process to account for the stochastic nature of demand and capacity that result in variations in system performance between days even with the presence of no incidents, weather events, and/or construction activities. Monte Carlo is a popular statistical analysis method for considering the impacts of stochastic variables and is capable of dealing with various stochastic distribution types and large number of variables. As described in Chapter 4, statistical distributions were fitted to the historical traffic data for capacity and demands measurements for the year of 2015.

Monte Carlo simulation utilizes random sampling to conducts a large number of experiments. In each experiment, instances of input random variables (capacity and demand in our case) were generated based on their distributions to represent one realization of traffic conditions on the investigated facility. 1000 Monte Carlo experiments were performed in this study to obtain the distribution of the outcome.

6.2.3. Step 4: Application of Ramp Metering Selection Formulation

As described earlier, Monte Carlo simulation was used to produce different realizations of the facility operations by generating random demand and capacity values from distributions fitted to real-world data for use as inputs to the ramp metering selection formulation. Then, the linear programming formulation and its extension described earlier was applied for each realization and the outputs from all realizations were further analyzed to support the decision-making process. The results are the ramps that need to be metered, the associated metering rate for each on-ramp, and the queue length on each ramp. For a specific on-ramp, the queue length on the on-ramp is calculated as the difference between the calculated metering rate and the demand. Figure 6-7 illustrates this process. Performing this process for a large number of experiments generated a set of output variables that generate distributions of these variables that can be used in the analysis. Please, note that the ramp metering rates produced as part of the process are not expected to be used in the real-world applications in many cases since an adaptive ramp metering will most likely be used. However, the process generates the ramps to be metered and rates that can be used to have an initial assessment of ramp metering impacts. In addition, it allows the calculation of queue lengths on the on-ramps and provide inputs to the benefit-cost analysis of ramp metering, as discussed later in this document.

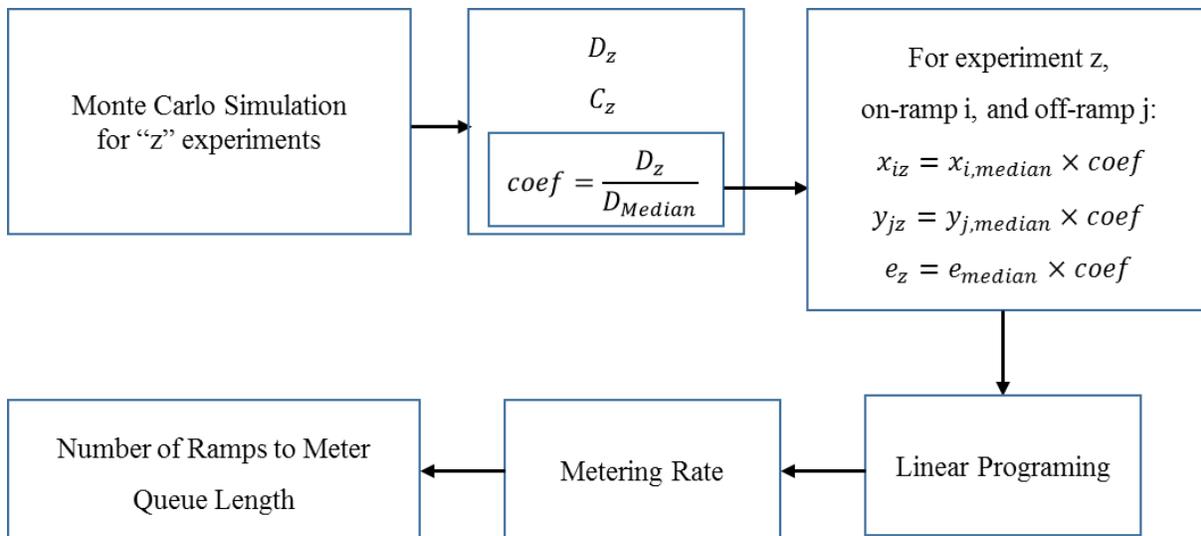


Figure 6-7 Inputs and Outputs of Linear Programming

6.2.4. Step 5: Estimation of Ramp Metering Return on Investment

This step involves estimating the anticipated benefits of ramp metering, in terms of delays on the mainline and ramps with and without ramp metering. Queuing theory equations are used in estimating the delays based on the outputs from the ramp metering formulation described earlier. The costs are estimated based on information obtained from FDOT District 4 and the present worth of the ramp metering deployment is calculated based on initial costs and the recurrent benefits and costs during the project life.

6.2.5. Analysis Results

This section presents the results from applying the discussed methodology to the I-95 corridor segment between Hallandale Beach Blvd and Commercial Blvd in Broward County, FL.

6.2.5.1. D/C Distributions Derived based on Monte Carlo Results

Figures 6-8 to 6-10 shows the fitted distributions to the capacity, AM peak demands, and PM peak demands at the bottleneck location at the Commercial Boulevard exit, which is the identified bottleneck location for both peaks on the subject corridor.

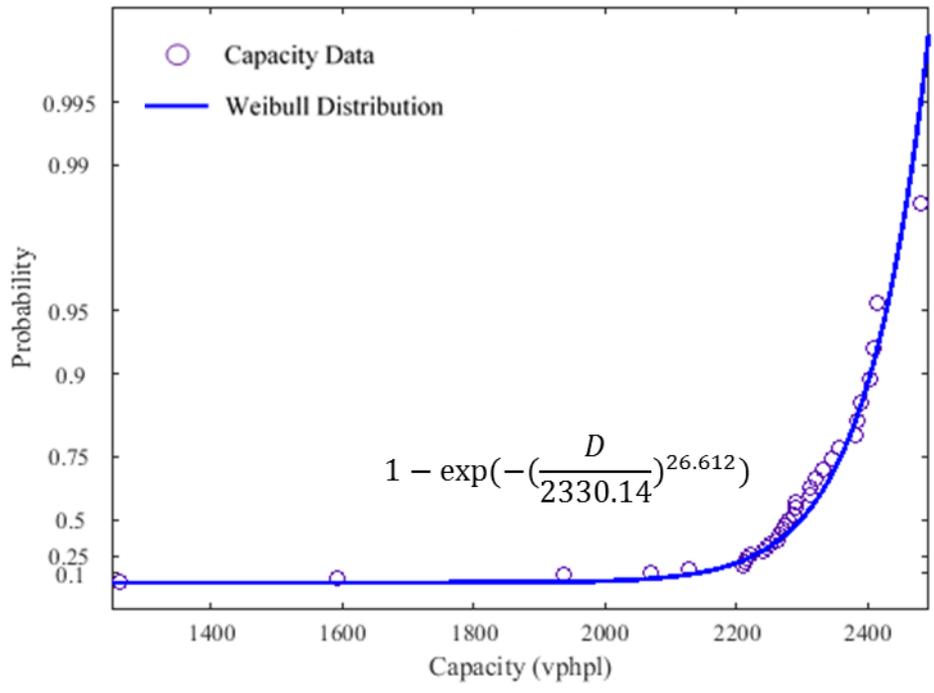


Figure 6-8 Capacity Distribution

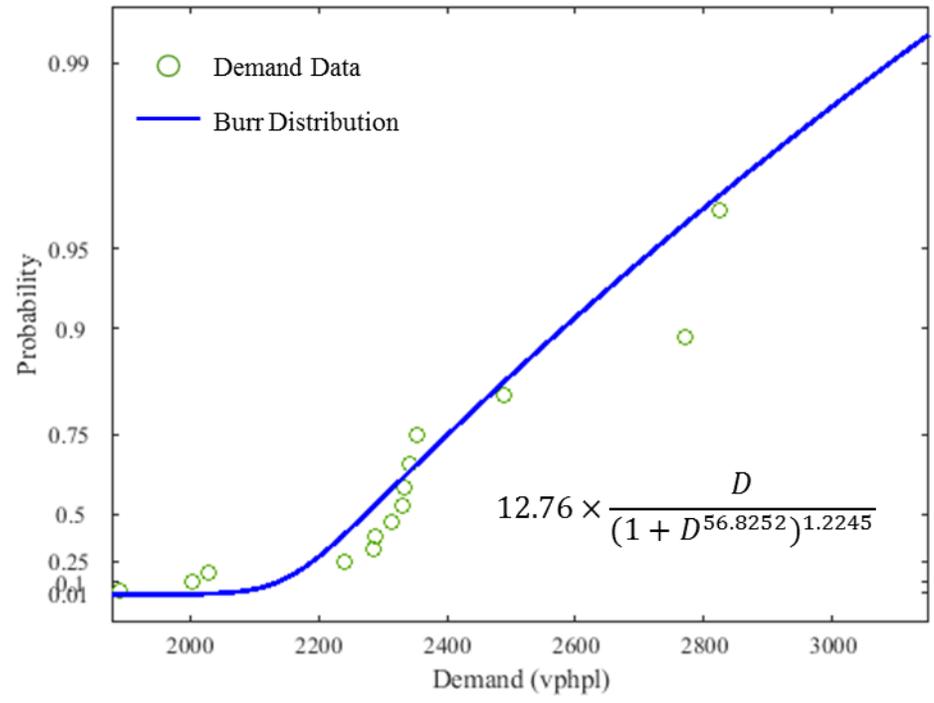


Figure 6-9 Demand Distribution for the Morning Peak

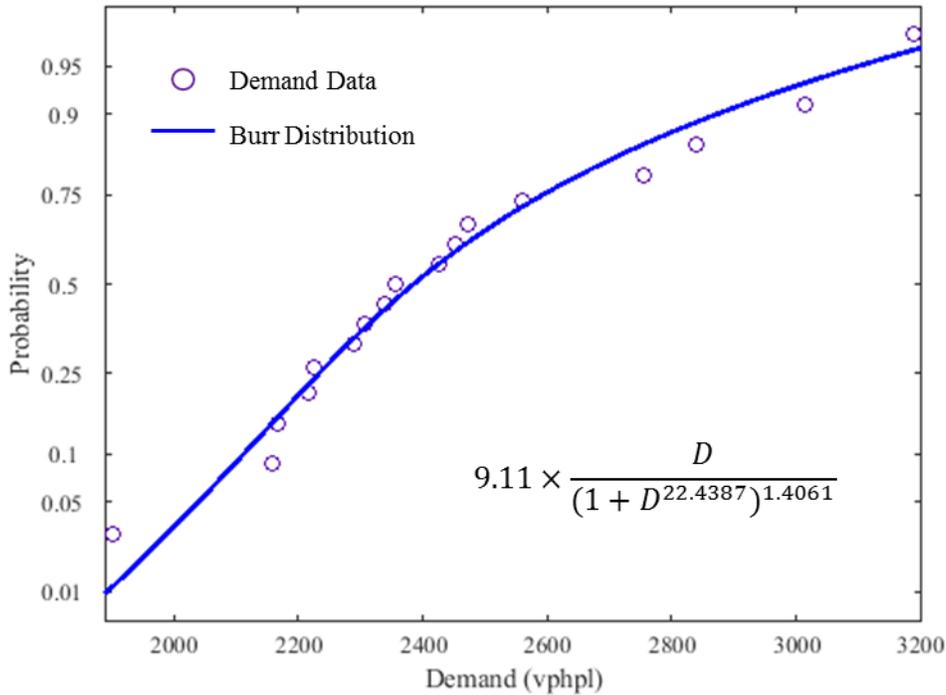


Figure 6-10 Demand Distribution for the Afternoon Peak

The capacity and demand values for each Monte Carlo experiment is generated from the three distributions in Figures 6-8 to 6-10. As stated earlier, a total of 1,000 independent experiments with different demand and capacity values were generated for the morning and afternoon peak and the D/C value was calculated for each experiment. Based on the calculated frequencies, the histograms for the morning and afternoon peaks were obtained, as shown in Figures 6-11 and 6-14, respectively. Figures 6-12 and 6-15 shows the Probability Density Functions (PDF) fitted to the D/C values for morning and afternoon, respectively. Figures 6-13 and 6-16 show that it is estimated that the median D/C ratio is about 0.995 in the morning peak and 1.039 in the afternoon for no incident conditions. The 85th D/C ratios are 1.108 and 1.184 for the two peaks, respectively.

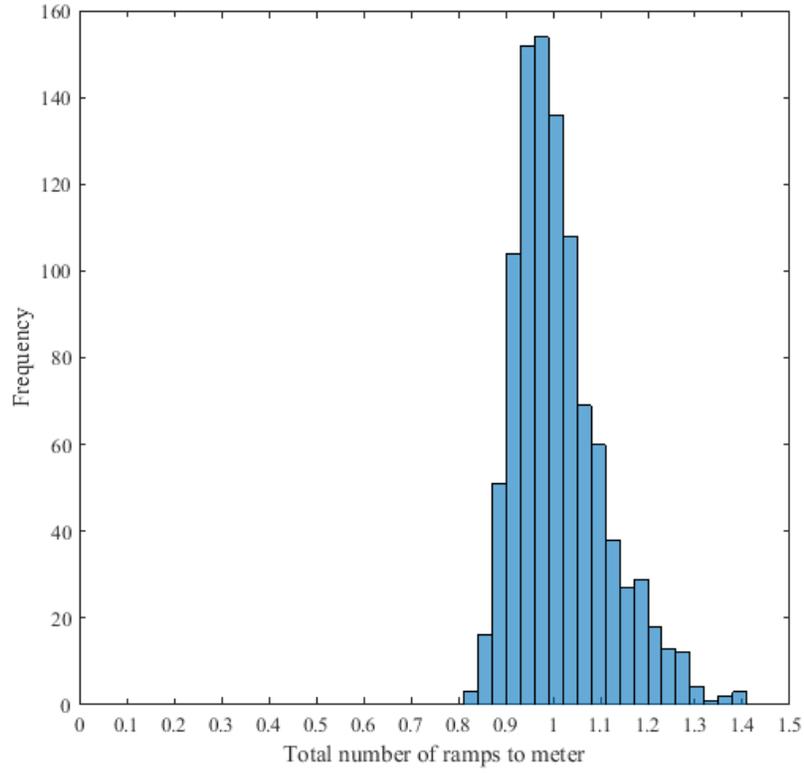


Figure 6-11 Simulated D/C Histogram for the Morning Peak with No Events

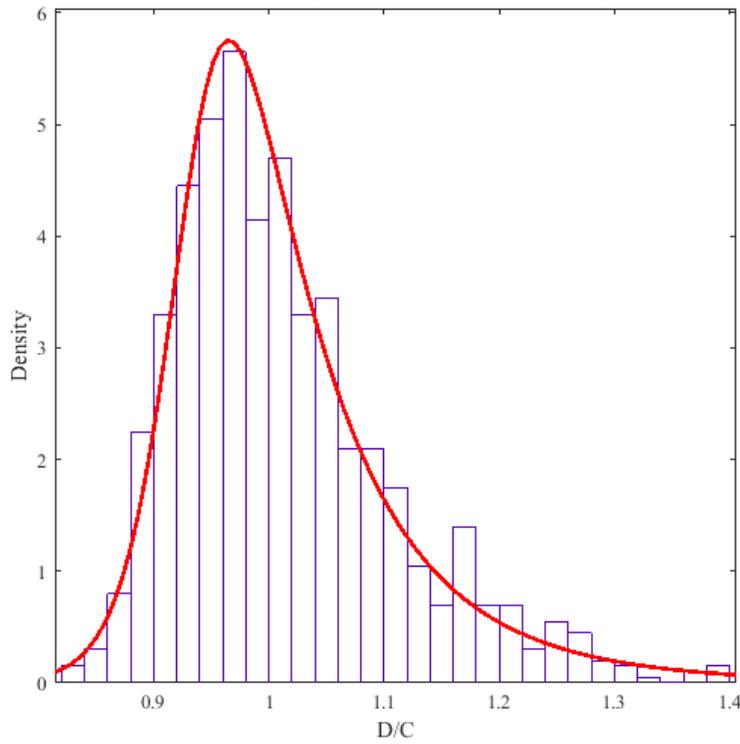


Figure 6-12 Fitted D/C PDF for the Morning Peak with No Events

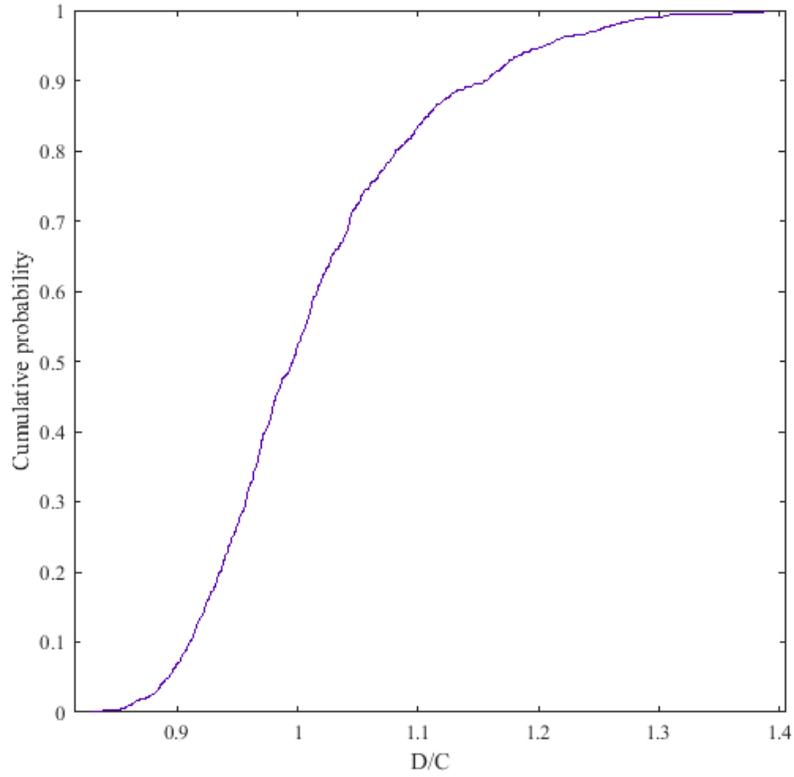


Figure 6-13 Cumulative Probability of D/C for the Morning Peak with No Events

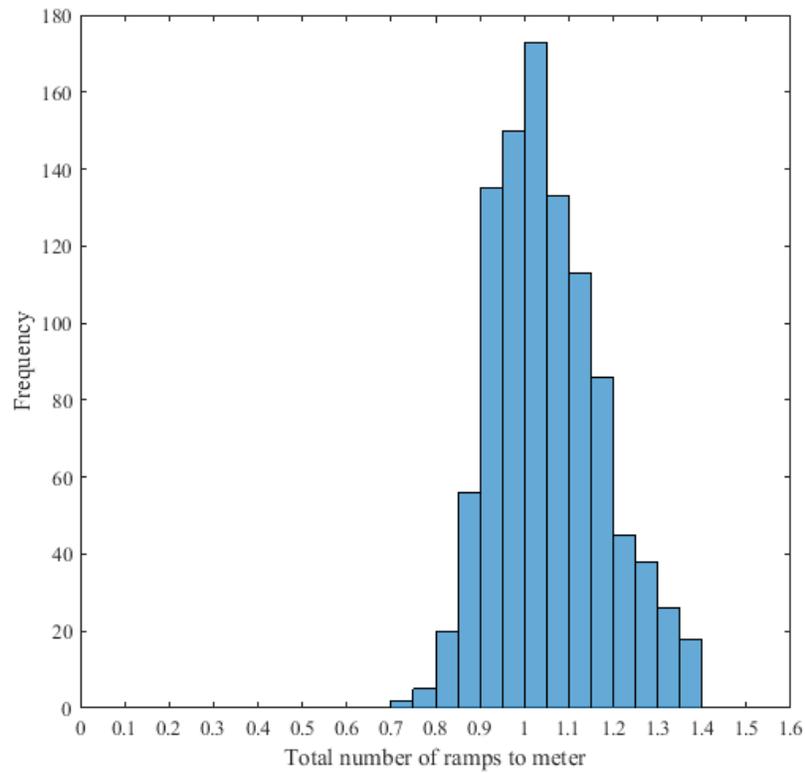


Figure 6-14 Simulated D/C Histogram for the Afternoon Peak with No Events

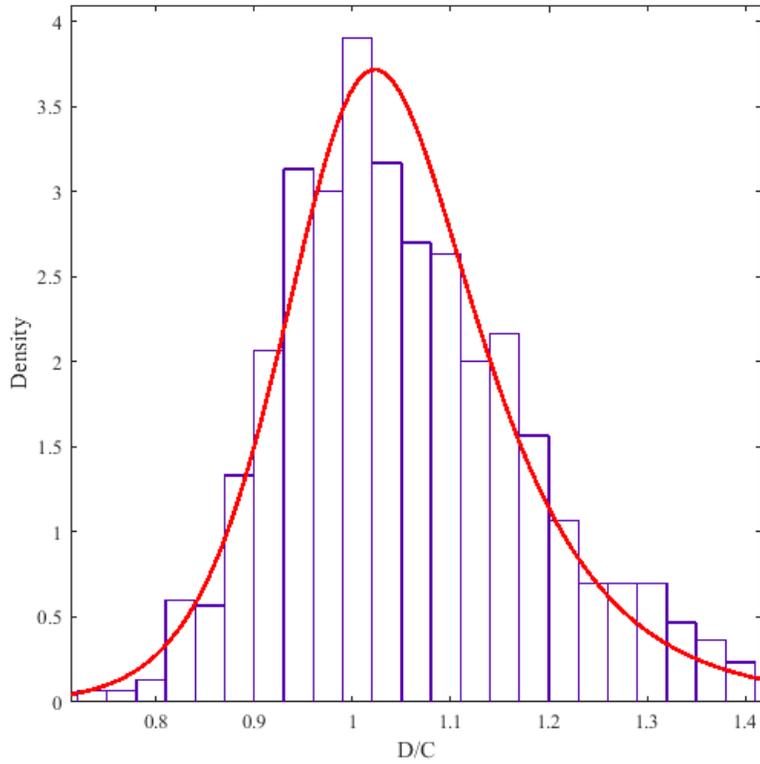


Figure 6-15 Fitted D/C PDF for the Afternoon Peak with No Events

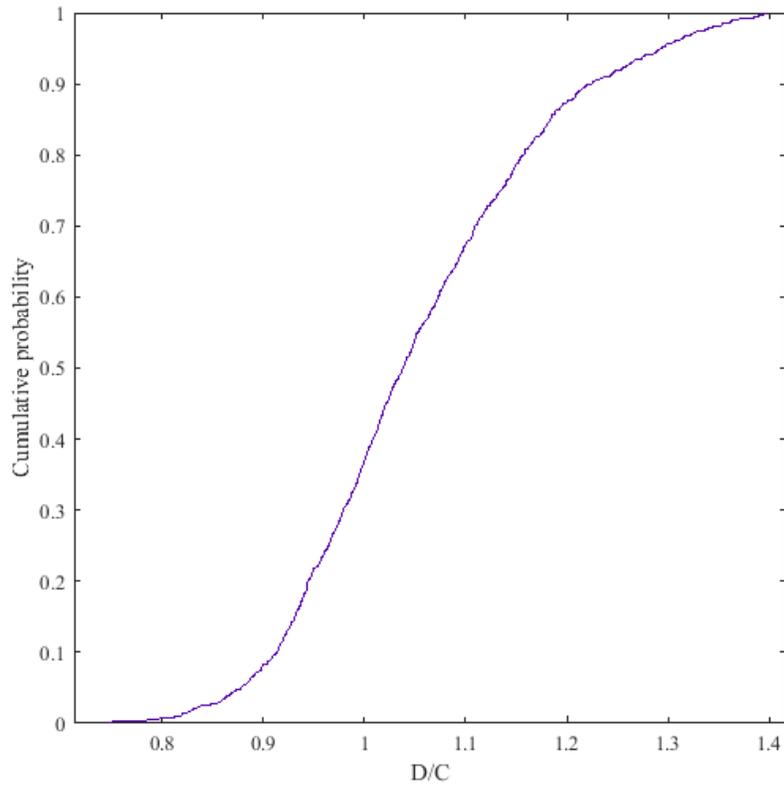


Figure 6-16 Cumulative Probability of D/C for the Afternoon Peak with No Events

6.2.5.2. Estimation of Bottleneck Characteristics with No Metering

As described in the methodology section, regression analysis was used to relate the D/C values to the bottleneck characteristics. Using the regression models presented in Figures 6-3 to 6-5 and Table 6-1, the duration, intensity, and extent of the bottleneck were calculated based on the simulated D/C values produced from the Monte Carlo runs. Figures 6-17 to 6-22 demonstrate statistical analysis of the calculated variables.

As stated earlier, duration is the longest continuous time period during which the segment is congested. Figures 6-17 and 6-18 show that it is estimated that the median congestion duration is about 40 minutes in the morning peak and 60 minutes in the afternoon. The 85th percentile congestion durations are 150 minutes and 180 minutes for the two peaks, respectively.

Extent is the longest continuous spatial length, during the congested time interval. Figures 6-19 and 20 show that it is estimated that the median extent is about 6,000 ft in the morning peak and 8,000 ft in the afternoon. The 85th percentile extent are 15,000 ft minutes and 17,500 ft in the two peaks, respectively.

Intensity represents the percentage of an analysis box, which is congested, covering both time and space. The AM peak time in the analysis is from 7:30 am to 10:00 am and the PM peak time is from 2:00 pm to 06:45 pm. The space is from Hallandale Beach Blvd to Commercial Blvd in Broward County, FL. Figures 6-21 and 6-22 show that it is estimated that the median intensity is about 7% in the morning peak and 9% in the afternoon. The 85th percentile intensity for these two peaks are 25% and 32%, respectively.

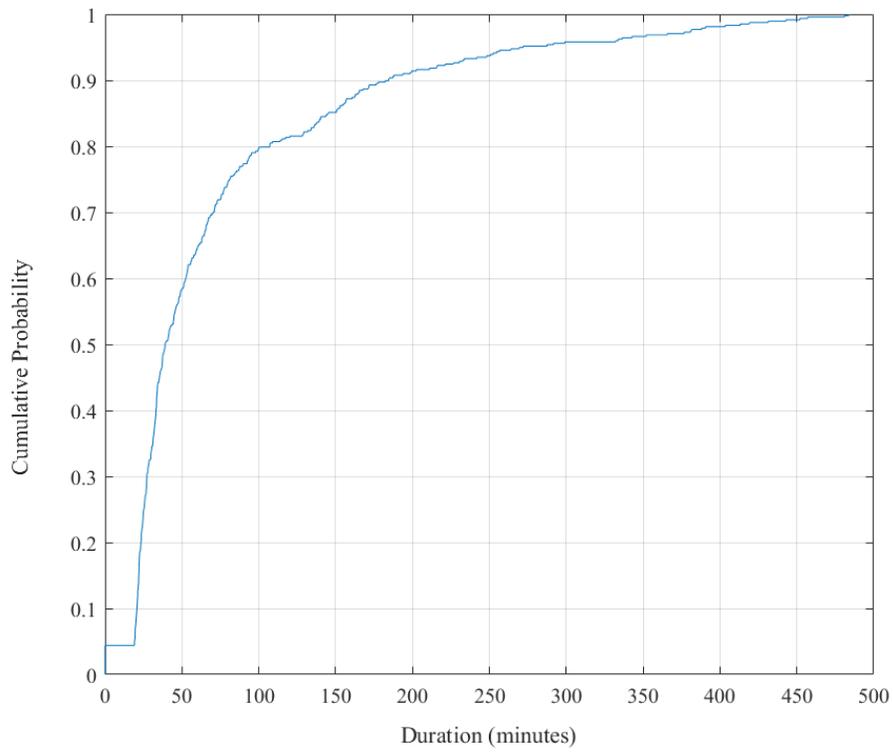


Figure 6-17 Cumulative Probability of the Simulated Duration for Morning

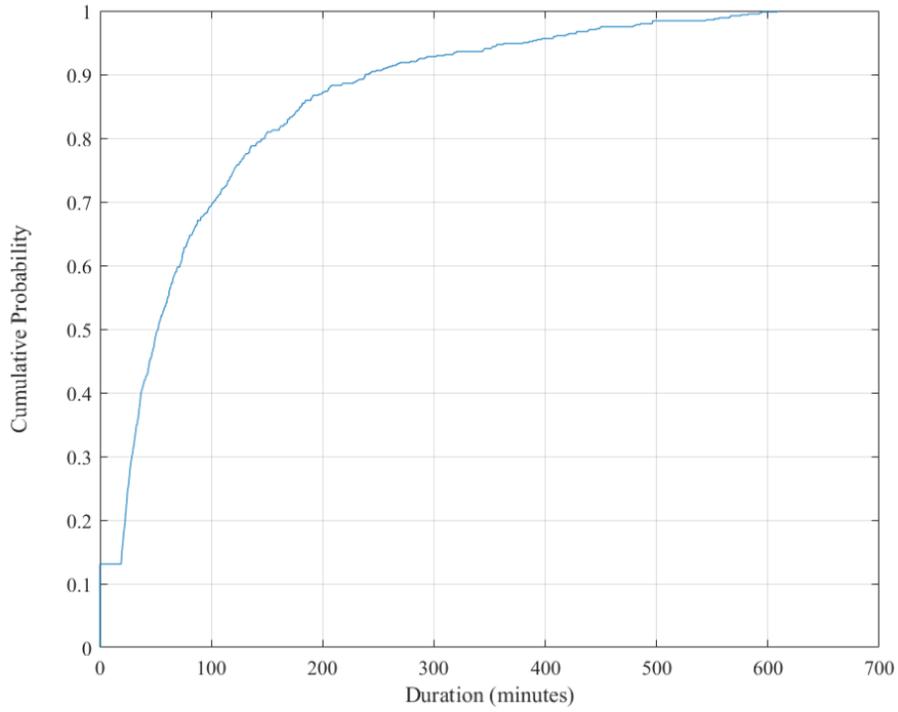


Figure 6-18 Cumulative Probability of the Simulated Duration for Afternoon

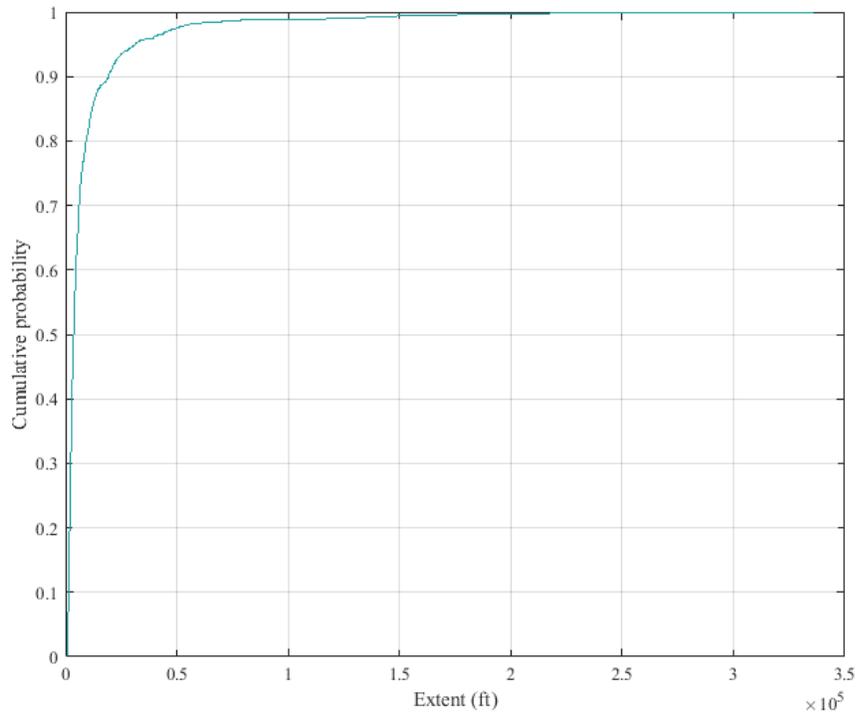


Figure 6-19 Cumulative Probability of the Simulated Extent for Morning

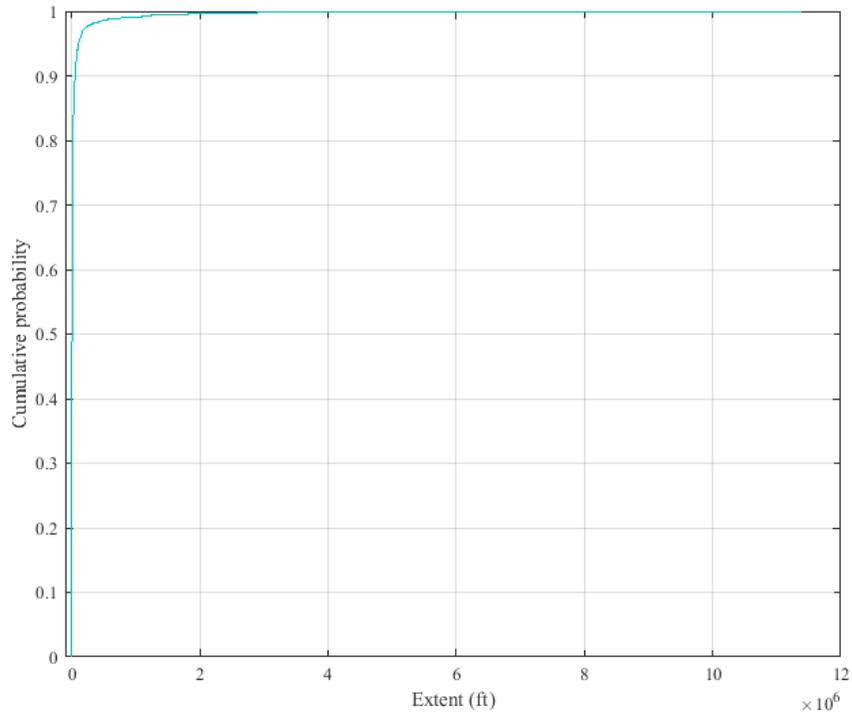


Figure 6-20 Cumulative Probability of the Simulated Extent for Afternoon

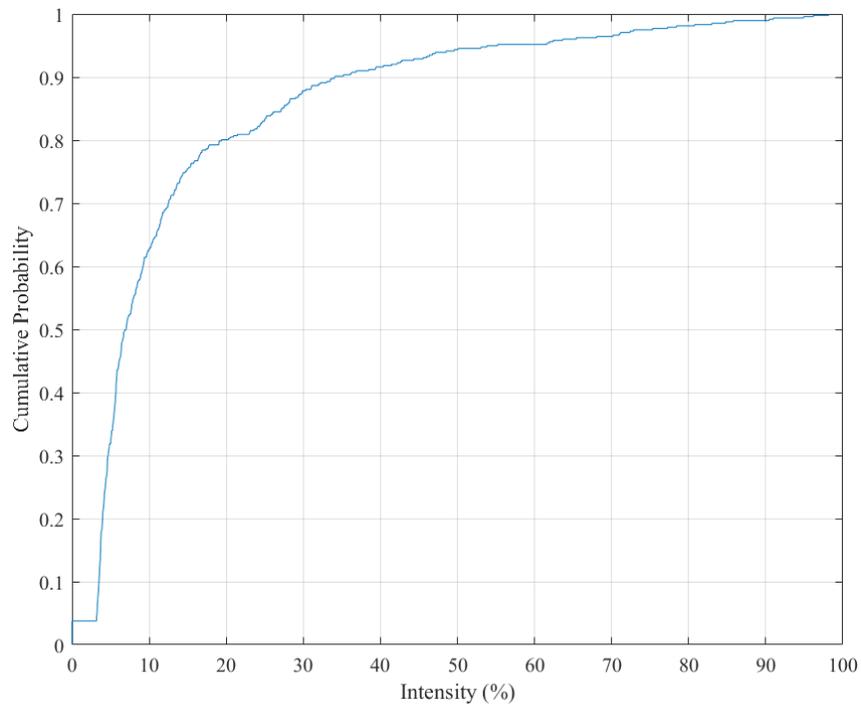


Figure 6-21 Cumulative Probability of the Simulated Intensity for the Morning Peak

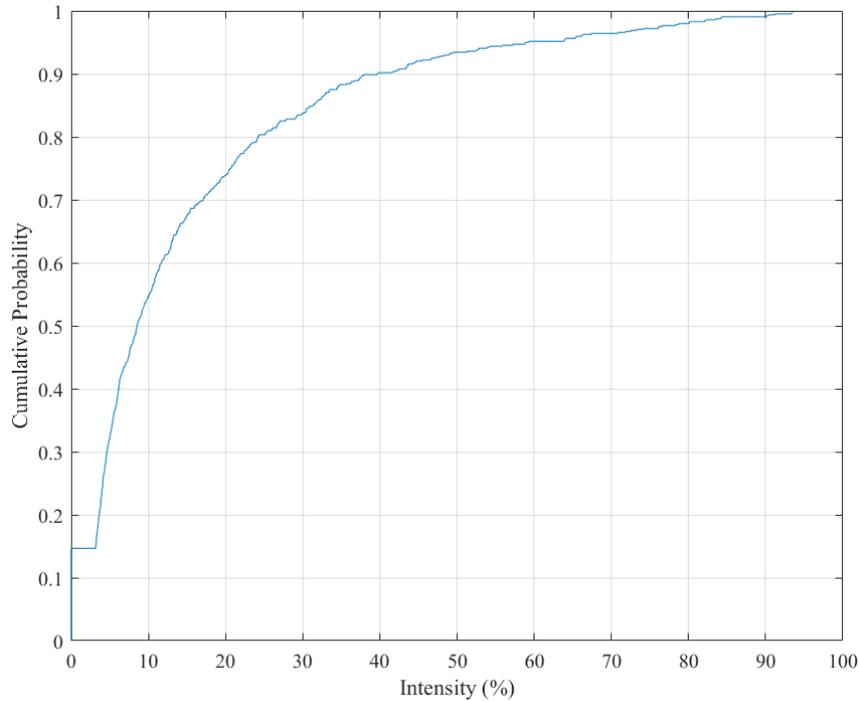


Figure 6-22 Cumulative Probability of the Simulated Intensity for the Afternoon Peak

6.2.5.3. Determining the Number of Ramps to Meter

The simulated capacity and demand values were used as inputs to the ramp metering selection formulation for 1,000 experiments for the morning period and also for the afternoon period. The outputs from the ramp identification formulation is the number of the ramps to be metered ahead of the bottleneck and also the queue length on each ramps, which can be calculated as a function of the estimate ramp metering rates. The statistical analysis of the total number of ramps to be metered are presented in Figures 6-23 and 6-24 for all of the D/C values produced by the Monte Carlo process for the morning and afternoon periods, respectively. As shown in these figures, the results from the simulation indicate that 129 of the 1,000 days with no incidents or weather events (each day represent one realization from the Monte Carlo process) will require the metering of all ten ramps between Hallandale Blvd. and Commercial Blvd. in the morning peak and 299 of the 1,000 days will require metering all ramps in the PM peak.

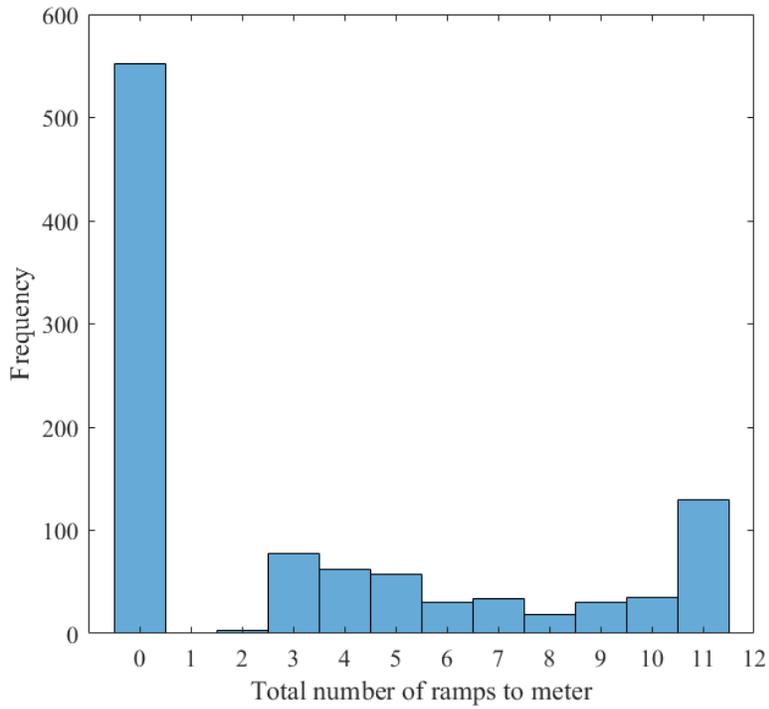


Figure 6-23 Histogram of Total Number of Ramps to Meter for the Morning Peak

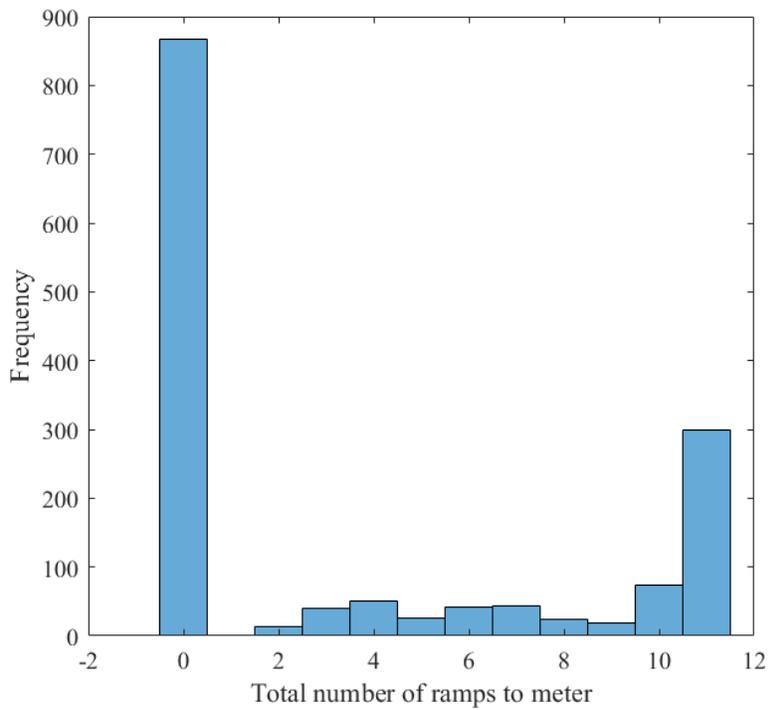


Figure 6-24 Histogram of Total Number of Ramps to Meter for the Afternoon Peak

In order to better explore the required metering for oversaturated conditions, the histograms for only D/C values that are higher than 1 are presented separately in Figures 6-25 and 6-26. The number of experiments (days) with D/C values higher than 1 is 477 for the morning and 641 for the afternoon out of 1,000 experiments. Figures 6-25 and 6-26 illustrate the histograms that represent the number of ramps to be metered only for the days with D/C values higher than 1 for the morning and afternoon, respectively. Table 6-2 summarizes the number of the ramps to meter in respect to the D/C values for morning and afternoon.

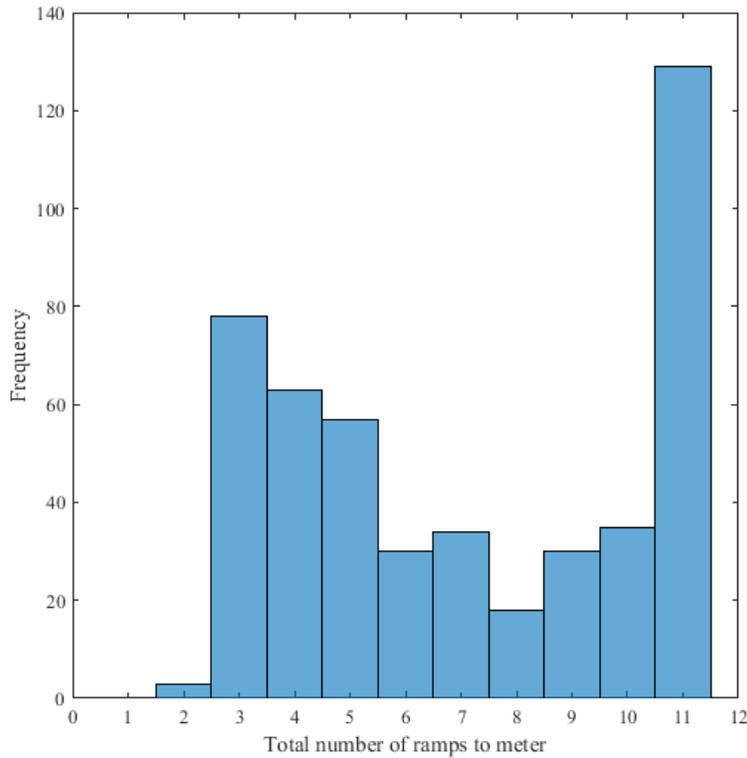


Figure 6-25 Histogram of Total Number of Ramps to Meter for D/C Values over One (Morning)

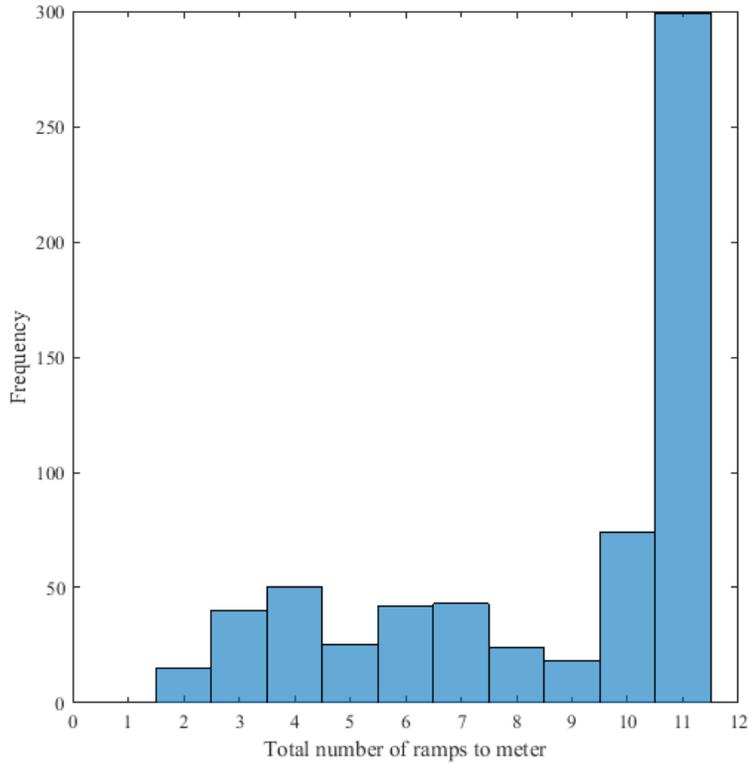


Figure 6-26 Histogram of Total Number of Ramps to Meter for D/C Values over One (Afternoon)

Table 6-2 Total Number of Ramps to Meter for Recurrent Conditions

Number of Metered Ramps	All D/C Values		D/C>1	
	Morning	Afternoon	Morning	Afternoon
0	523	370	0	0
1	0	0	0	0
2	3	15	3	15
3	78	40	78	40
4	63	50	63	50
5	57	25	57	25
6	30	42	30	42
7	34	43	34	43
8	18	24	18	24
9	30	18	30	18
10	35	74	35	74
11	129	299	129	299
Total	1,000	1,000	477	630

6.2.5.4. Determining the Ramp Queue Lengths

For a specific on-ramp, the maximum queue length on the on-ramp is calculated as the difference between the calculated metering rate and the demand. The average queue length is half of this value according to queuing equations. This value is converted to feet assuming that each queued vehicle and the associated distance headway occupies 25 feet and considering the number of lanes available for storage on the on-ramp. The analyst can examine the queue length results and if the queue length of specific ramps need to be decreased, the analyst can increase the constraint on the queue lengths of these specific ramps in the formulation presented in Equation 6-1. Tables 6-3 and 6-4 shows the cumulative probability of maximum queue length, in vehicles and feet, respectively on each on-ramp in morning. Tables 6-5 and 6-6 shows the corresponding values for the afternoon peak.

Table 6-3 Cumulative Probability of the Maximum Queue Length (Vehicles) on the On-Ramps in the Morning Peak

percentile	Oakland	Sunrise	Broward	Davie	I-595	Griffin	Stirling	Sheridan	Hollywood	Pembroke	Hallandale Beach
10	6	0	0	0	0	0	5	0	0	0	0
20	6	0	0	0	0	0	5	0	0	0	0
30	6	0	0	0	0	0	5	0	0	0	0
40	6	0	0	0	0	0	5	0	0	0	0
50	17	0	0	0	0	0	28	0	0	0	0
60	29	0	0	0	0	0	53	99	0	0	0
70	42	0	0	0	0	0	60	113	80	93	0
80	76	51	34	0	0	0	65	122	88	104	97
85	83	56	38	21	45	0	68	128	92	108	105
90	90	62	42	24	54	70	72	136	97	115	112
95	99	68	46	26	59	78	76	144	103	124	121
99	115	79	53	30	68	90	88	167	119	143	141

Table 6-4 Cumulative Probability of the Maximum Queue Length (feet) on the On-Ramps in the Morning Peak

percentile	Oakland	Sunrise	Broward	Davie	I-595	Griffin	Stirling	Sheridan	Hollywood	Pembroke	Hallandale Beach
10	150	0	0	0	0	0	125	0	0	0	0
20	150	0	0	0	0	0	125	0	0	0	0
30	150	0	0	0	0	0	125	0	0	0	0
40	150	0	0	0	0	0	125	0	0	0	0
50	425	0	0	0	0	0	700	0	0	0	0
60	725	0	0	0	0	0	1,325	2,475	0	0	0
70	1,050	0	0	0	0	0	1,500	2,825	2,000	2,325	0
80	1,900	1,275	850	0	0	0	1,625	3,050	2,200	2,600	2,425
85	2,075	1,400	950	525	1,125	0	1,700	3,200	2,300	2,700	2,625
90	2,250	1,550	1,050	600	1,350	1,750	1,800	3,400	2,425	2,875	2,800
95	2,475	1,700	1,150	650	1,475	1,950	1,900	3,600	2,575	3,100	3,025
99	2,875	1,975	1,325	750	1,700	2,250	2,200	4,175	2,975	3,575	3,525

Table 6-5 Cumulative Probability of the Maximum Queue Length (Vehicles) on the On-Ramps in the Afternoon Peak

percentile	Oakland	Sunrise	Broward	Davie	I-595	Griffin	Stirling	Sheridan	Hollywood	Pembroke	Hallandale Beach
10	6	0	0	0	0	0	4	0	0	0	0
20	6	0	0	0	0	0	5	0	0	0	0
30	6	0	0	0	0	0	5	0	0	0	0
40	25	0	0	0	0	0	58	56	0	0	0
50	39	0	0	0	0	0	68	70	66	62	0
60	57	63	31	0	0	0	75	78	74	70	91
70	67	74	40	15	46	0	81	85	81	79	107
80	74	83	45	17	51	80	88	93	88	85	117
85	77	86	46	18	53	83	91	95	91	88	120
90	80	90	48	19	56	86	95	99	95	91	125
95	83	93	50	20	58	89	98	103	98	95	130
99	91	102	55	21	63	98	107	113	108	104	142

Table 6-6 Cumulative Probability of the Maximum Queue Length (feet) on the On-Ramps in the Afternoon Peak

percentile	Oakland	Sunrise	Broward	Davie	I-595	Griffin	Stirling	Sheridan	Hollywood	Pembroke	Hallandale Beach
10	150	0	0	0	0	0	100	0	0	0	0
20	150	0	0	0	0	0	125	0	0	0	0
30	150	0	0	0	0	0	125	0	0	0	0
40	625	0	0	0	0	0	1,450	1,400	0	0	0
50	975	0	0	0	0	0	1,700	1,750	1,650	1,550	0
60	1,425	1,575	775	0	0	0	1,875	1,950	1,850	1,750	2,275
70	1,675	1,850	1,000	375	1,150	0	2,025	2,125	2,025	1,975	2,675
80	1,850	2,075	1,125	425	1,275	2,000	2,200	2,325	2,200	2,125	2,925
85	1,925	2,150	1,150	450	1,325	2,075	2,275	2,375	2,275	2,200	3,000
90	2,000	2,250	1,200	475	1,400	2,150	2,375	2,475	2,375	2,275	3,125
95	2,075	2,325	1,250	500	1,450	2,225	2,450	2,575	2,450	2,375	3,250
99	2,275	2,550	1,375	525	1,575	2,450	2,675	2,825	2,700	2,600	3,550

As seen in the tables, some on-ramps such as Oakland Blvd. and Griffin Road are expected to experience high queue lengths, as the ramp selection formulation allowed to have a queue length of up to 10 percent of the demand. This parameter can be changed as a user input, possibly just for the ramps with long queues. It should be noted that the adaptive metering algorithm will have a queue override feature in order to avoid long queues which may spill back to the arterial. This is expected to reduce the effectiveness of the ramp metering. On the other hand, it will possible to manage the demand on each on-ramp by proper signal timing on the arterials to manage the arrivals to the on-ramps. However, this may have an adverse impact on the arterial street operations.

6.3. Offline Ramp Selection for Non-Recurrent Conditions

This project aims to investigate the installation and activation of ramp metering considering both recurrent and non-recurrent conditions. Thus, this section summarizes the effort conducted to assess the ramps that need to be metered during incident and rainfall events. The process is basically similar to that used for recurrent conditions, described earlier. The only difference is including the impacts of non-recurrent events (incident and rainfall) on the capacity (and possibly demand) inputs to the ramp selection formulation. The following sections discuss the details of these impacts and the methodology to incorporate them in this study.

6.3.1. Incident Impact on Capacity

According to the latest version of the Highway Capacity Manual (HCM), once a lane is blocked due to an incident, the remaining lanes will not function at full capacity. Table 6-7 illustrates the remaining capacity of the unblocked lanes for different combinations of total number of lanes and the number of blocked lanes, as presented in the HCM. These values of capacity drop are used to modify the capacity distribution derived using data collected under normal (event-free) conditions. As an example, for an incident which results in one lane blockage out of four lanes, the capacity of the remaining three lanes drops to 77 percent of its original value. Therefore, the remaining capacity of the section will reduce to %57.75 of its original value ($3 \times 0.77 / 4 = 0.5775$).

Table 6-7 Remaining Capacity Ratio for Open Lanes after Closure Due to Incident Occurrence (Source: HCM (TRB, 2016))

Directional Lanes	No Incident	Shoulder Blocked	1 Lane Blocked	2 Lanes Blocked	3 Lanes Blocked	4 Lanes Blocked
2	1.00	0.81	0.70	N/A	N/A	N/A
3	1.00	0.83	0.74	0.51	N/A	N/A
4	1.00	0.85	0.77	0.50	0.52	N/A
5	1.00	0.87	0.81	0.67	0.50	0.50
6	1.00	0.89	0.85	0.75	0.52	0.52
7	1.00	0.91	0.88	0.80	0.63	0.63
8	1.00	0.93	0.89	0.84	0.66	0.66

6.3.2. Incident Impact on Demands

The impact of incidents on diversion has been investigated in previous studies. Hadi et al. (2013) performed an empirical analysis on a 7 miles segments in I-95 corridor in Miami, Florida between Golden Glades Interchange and SR-836 by analyzing 188 incident cases. The average diversion rates were estimated for various combinations of total number of lanes and the number of blocked lanes due to incident (Hadi et al., 2013). Table 6-8 shows the diversion rates for each case of lane blockages. These values are used in this study to calculate the remaining demand after incident occurrence. As an example, for an incident which results in one lane blockage out of four lanes, the adjusted demand for the whole segment is assumed to decrease by 11.07 percent. This diversion percentage can be varied by the analyst based on local conditions. In addition, it is possible to set as a variable in the Monte Carlo simulation base on variations in the diversion percentage as reported in the previous studies.

Table 6-8 Estimated Diversion Rates Due to Incident Occurrence (Hadi et al., 2013)

Directional Lanes	Number of Lanes Blocked	Average Diversion Rate (%)
3	1	14.81
3	2	10.68
3	3	30.27
4	1	11.07
4	2	16.88
4	3	24.61
4	4	34.83
5	1	8.60
5	2	9.87
5	3	17.30

To assess ramp metering requirements under incident conditions, different incident scenarios can be considered. For example, for a segment with four lanes, the incident scenarios may include shoulder only incidents, 1 lane, 2 lanes, and 3 lanes blocked incidents. The capacity of the mainline in the case of 3 lane blockage incident is expected to be 13 percent of the original capacity based on the information provided in Table 6-7. The demand drops by only about 24.61 percent, according to Table 6-8. Such scenarios with excessive D/C ratios are not expected to benefit from ramp metering and thus were excluded from further analysis. Table 6-9 summarizes the coefficients, which are used for each scenario application, based on the coefficients presented in Table 6-7.

Table 6-9 Diversion and Capacity Adjustment Coefficients for Incident Scenarios

Number of Blocked Lanes	Remaining Capacity for the Segment	Remaining Demand for the Segment
Shoulder Only	$0.85 \times (4 \text{ lanes}) \times \text{original capacity (vphpl)}$	original demand (vph)
1	$0.77 \times (3 \text{ lanes}) \times \text{original capacity (vphpl)}$	$(1 - 0.1107) \times \text{original demand (vph)}$
2	$0.50 \times (2 \text{ lanes}) \times \text{original capacity (vphpl)}$	$(1 - 0.1688) \times \text{original demand (vph)}$

6.3.3. Results of the Analysis of Ramp Metering Needs during Incidents

The results of the ramp selection formulation for incidents occurring at Commercial Blvd in the northbound direction in the AM and PM peak, in terms of the total number of ramps that need to be metered in 1,000 scenarios for each incident lane blockage type are illustrated in Table 6-10. It should be mentioned that a one hour incident duration is assumed in this analysis.

Table 6-10 The Frequency of the Number of Ramps to Meter for AM and PM Peak Period Incident Conditions out of a Total of 1,000 Scenarios of Each Lane Blockage Type at the Commercial Blvd. Bottleneck

Number of Metered Ramps	Shoulder Blocked		1 Lane Blocked		2 Lanes Blocked	
	Morning	Afternoon	Morning	Afternoon	Morning	Afternoon
1	0	0	0	0	0	0
2	19	18	0	3	0	0
3	22	10	1	4	0	0
4	47	15	1	3	0	0
5	54	7	4	4	0	0
6	46	24	3	12	0	0
7	63	27	4	2	0	0
8	43	18	17	15	0	0
9	98	12	26	7	0	0
10	68	33	46	33	0	0
11	540	827	898	908	1,000	1,000
Total	1,000	1,000	1,000	1,000	1,000	1,000

As aforementioned, the maximum queue length on each ramp is equal to the difference between the demand on the subject on-ramp and the optimized metering rate resulting from the ramp selection formulation. Tables 6-11 to 6-22 shows the cumulative probability of the maximum queue length of on ramps for each lane blockage type at the Commercial Blvd bottleneck locations during the two peak periods. As before, 25 feet per vehicle is assumed when calculating the queue length in feet based on the queue length in vehicles. The analyst can examine the queue length results and if the queue length of specific ramps need to be decreased, the analyst can increase the constraint on the queue lengths of these specific ramps in the formulation presented in Equation 6-1.

Table 6-11 Cumulative Probability of the On-Ramp Maximum Queue Length (Vehicles) for Shoulder Blocked Incident in the Morning Peak Period at the Commercial Blvd. Bottleneck

percentile	Oakland	Sunrise	Broward	Davie	I-595	Griffin	Stirling	Sheridan	Hollywood	Pembroke	Hallandale Beach
10	34	0	0	0	0	0	55	104	74	82	0
20	61	41	0	0	0	0	58	110	79	93	0
30	69	47	31	16	0	0	61	114	82	97	88
40	74	51	34	19	41	0	62	117	84	100	95
50	79	55	37	21	47	60	64	121	87	104	99
60	83	57	39	22	50	65	66	124	89	107	103
70	87	60	41	23	52	68	68	127	91	110	107
80	90	62	42	24	54	71	69	131	94	112	110
85	92	63	43	24	55	72	71	133	95	114	112
90	94	65	44	25	56	74	72	136	98	117	115
95	96	67	45	25	58	76	74	139	100	120	118
99	102	70	48	27	61	80	78	147	106	127	125

Table 6-12 Cumulative Probability of the On-Ramp Maximum Queue Length (Feet) for Shoulder Blocked Incident in the Morning Peak Period at the Commercial Blvd. Bottleneck

percentile	Oakland	Sunrise	Broward	Davie	I-595	Griffin	Stirling	Sheridan	Hollywood	Pembroke	Hallandale Beach
10	850	0	0	0	0	0	1,375	2,600	1,850	2,050	0
20	1,525	1,025	0	0	0	0	1,450	2,750	1,975	2,325	0
30	1,725	1,175	775	400	0	0	1,525	2,850	2,050	2,425	2,200
40	1,850	1,275	850	475	1,025	0	1,550	2,925	2,100	2,500	2,375
50	1,975	1,375	925	525	1,175	1,500	1,600	3,025	2,175	2,600	2,475
60	2,075	1,425	975	550	1,250	1,625	1,650	3,100	2,225	2,675	2,575
70	2,175	1,500	1,025	575	1,300	1,700	1,700	3,175	2,275	2,750	2,675
80	2,250	1,550	1,050	600	1,350	1,775	1,725	3,275	2,350	2,800	2,750
85	2,300	1,575	1,075	600	1,375	1,800	1,775	3,325	2,375	2,850	2,800
90	2,350	1,625	1,100	625	1,400	1,850	1,800	3,400	2,450	2,925	2,875
95	2,400	1,675	1,125	625	1,450	1,900	1,850	3,475	2,500	3,000	2,950
99	2,550	1,750	1,200	675	1,525	2,000	1,950	3,675	2,650	3,175	3,125

Table 6-13 Cumulative Probability of the On-Ramp Maximum Queue Length (Vehicles) for Shoulder Blocked Incident in the Afternoon Peak Period at the Commercial Blvd. Bottleneck

percentile	Oakland	Sunrise	Broward	Davie	I-595	Griffin	Stirling	Sheridan	Hollywood	Pembroke	Hallandale Beach
10	49	54	29	0	0	0	65	68	65	62	76
20	61	68	37	14	42	65	72	75	72	69	94
30	65	72	39	15	45	70	76	80	77	74	101
40	67	74	40	16	46	71	79	83	79	76	104
50	69	76	41	16	47	74	80	84	81	78	107
60	70	78	42	16	49	75	83	87	83	80	110
70	72	81	43	17	50	78	85	89	85	82	113
80	75	84	45	18	52	81	88	93	89	85	117
85	77	86	46	18	53	82	90	95	90	87	120
90	79	88	47	19	55	85	93	98	93	90	124
95	83	93	50	19	58	89	98	102	98	94	129
99	90	101	54	21	63	97	106	112	107	103	141

Table 6-14 Cumulative Probability of the On-Ramp Maximum Queue Length (Feet) for Shoulder Blocked Incident in the Afternoon Peak Period at the Commercial Blvd. Bottleneck

percentile	Oakland	Sunrise	Broward	Davie	I-595	Griffin	Stirling	Sheridan	Hollywood	Pembroke	Hallandale Beach
10	1,225	1,350	725	0	0	0	1,625	1,700	1,625	1,550	1,900
20	1,525	1,700	925	350	1,050	1,625	1,800	1,875	1,800	1,725	2,350
30	1,625	1,800	975	375	1,125	1,750	1,900	2,000	1,925	1,850	2,525
40	1,675	1,850	1,000	400	1,150	1,775	1,975	2,075	1,975	1,900	2,600
50	1,725	1,900	1,025	400	1,175	1,850	2,000	2,100	2,025	1,950	2,675
60	1,750	1,950	1,050	400	1,225	1,875	2,075	2,175	2,075	2,000	2,750
70	1,800	2,025	1,075	425	1,250	1,950	2,125	2,225	2,125	2,050	2,825
80	1,875	2,100	1,125	450	1,300	2,025	2,200	2,325	2,225	2,125	2,925
85	1,925	2,150	1,150	450	1,325	2,050	2,250	2,375	2,250	2,175	3,000
90	1,975	2,200	1,175	475	1,375	2,125	2,325	2,450	2,325	2,250	3,100
95	2,075	2,325	1,250	475	1,450	2,225	2,450	2,550	2,450	2,350	3,225
99	2,250	2,525	1,350	525	1,575	2,425	2,650	2,800	2,675	2,575	3,525

Table 6-15 Cumulative Probability of the On-Ramp Maximum Queue Length (Vehicles) for One-Lane Blockage Incident in the Morning Peak Period at the Commercial Blvd. Bottleneck

percentile	Oakland	Sunrise	Broward	Davie	I-595	Griffin	Stirling	Sheridan	Hollywood	Pembroke	Hallandale Beach
10	52	36	24	14	31	0	43	81	58	69	68
20	57	39	27	15	34	45	44	83	60	71	70
30	58	40	27	15	35	46	45	84	60	73	71
40	59	41	28	15	35	46	45	86	61	74	72
50	60	41	28	16	36	47	46	87	62	75	74
60	61	42	29	16	37	48	47	89	64	76	75
70	62	43	29	16	37	49	48	90	65	78	76
80	64	44	30	17	38	50	49	93	66	80	78
85	65	45	31	17	39	51	50	94	68	81	80
90	67	46	31	18	40	53	51	97	69	83	82
95	68	47	32	18	41	54	53	99	71	85	84
99	72	50	34	19	43	57	56	105	75	90	89

Table 6-16 Cumulative Probability of the On-Ramp Maximum Queue Length (Feet) for One-Lane Blockage Incident in the Morning Peak Period at the Commercial Blvd. Bottleneck

percentile	Oakland	Sunrise	Broward	Davie	I-595	Griffin	Stirling	Sheridan	Hollywood	Pembroke	Hallandale Beach
10	1,300	900	600	350	775	0	1,075	2,025	1,450	1,725	1,700
20	1,425	975	675	375	850	1,125	1,100	2,075	1,500	1,775	1,750
30	1,450	1,000	675	375	875	1,150	1,125	2,100	1,500	1,825	1,775
40	1,475	1,025	700	375	875	1,150	1,125	2,150	1,525	1,850	1,800
50	1,500	1,025	700	400	900	1,175	1,150	2,175	1,550	1,875	1,850
60	1,525	1,050	725	400	925	1,200	1,175	2,225	1,600	1,900	1,875
70	1,550	1,075	725	400	925	1,225	1,200	2,250	1,625	1,950	1,900
80	1,600	1,100	750	425	950	1,250	1,225	2,325	1,650	2,000	1,950
85	1,625	1,125	775	425	975	1,275	1,250	2,350	1,700	2,025	2,000
90	1,675	1,150	775	450	1,000	1,325	1,275	2,425	1,725	2,075	2,050
95	1,700	1,175	800	450	1,025	1,350	1,325	2,475	1,775	2,125	2,100
99	1,800	1,250	850	475	1,075	1,425	1,400	2,625	1,875	2,250	2,225

Table 6-17 Cumulative Probability of the On-Ramp Maximum Queue Length (Vehicles) for One-Lane Blockage Incident in the Afternoon Peak Period at the Commercial Blvd. Bottleneck

percentile	Oakland	Sunrise	Broward	Davie	I-595	Griffin	Stirling	Sheridan	Hollywood	Pembroke	Hallandale Beach
10	41	45	24	9	27	41	50	52	50	48	66
20	45	50	27	10	31	48	53	55	53	51	70
30	46	52	28	11	32	50	55	57	55	53	72
40	47	53	28	11	33	51	56	59	56	54	74
50	49	54	29	11	34	52	57	60	57	55	76
60	50	56	30	12	35	54	59	62	59	57	79
70	51	57	31	12	36	55	61	63	61	58	80
80	54	60	32	12	37	57	63	66	63	61	84
85	55	61	33	13	38	58	64	68	65	62	86
90	56	63	34	13	39	60	66	70	66	64	88
95	59	66	35	14	41	63	70	73	70	67	92
99	64	72	39	15	44	69	76	79	76	73	100

Table 6-18 Cumulative Probability of the On-Ramp Maximum Queue Length (Feet) for One-Lane Blockage Incident in the Afternoon Peak Period at the Commercial Blvd. Bottleneck

percentile	Oakland	Sunrise	Broward	Davie	I-595	Griffin	Stirling	Sheridan	Hollywood	Pembroke	Hallandale Beach
10	1,025	1,125	600	225	675	1,025	1,250	1,300	1,250	1,200	1,650
20	1,125	1,250	675	250	775	1,200	1,325	1,375	1,325	1,275	1,750
30	1,150	1,300	700	275	800	1,250	1,375	1,425	1,375	1,325	1,800
40	1,175	1,325	700	275	825	1,275	1,400	1,475	1,400	1,350	1,850
50	1,225	1,350	725	275	850	1,300	1,425	1,500	1,425	1,375	1,900
60	1,250	1,400	750	300	875	1,350	1,475	1,550	1,475	1,425	1,975
70	1,275	1,425	775	300	900	1,375	1,525	1,575	1,525	1,450	2,000
80	1,350	1,500	800	300	925	1,425	1,575	1,650	1,575	1,525	2,100
85	1,375	1,525	825	325	950	1,450	1,600	1,700	1,625	1,550	2,150
90	1,400	1,575	850	325	975	1,500	1,650	1,750	1,650	1,600	2,200
95	1,475	1,650	875	350	1,025	1,575	1,750	1,825	1,750	1,675	2,300
99	1,600	1,800	975	375	1,100	1,725	1,900	1,975	1,900	1,825	2,500

Table 6-19 Cumulative Probability of the On-Ramp Maximum Queue Length (Vehicles) for Two-Lane Blockage Incident in the Morning Peak Period at the Commercial Blvd. Bottleneck

percentile	Oakland	Sunrise	Broward	Davie	I-595	Griffin	Stirling	Sheridan	Hollywood	Pembroke	Hallandale Beach
10	49	34	23	13	29	39	38	71	51	62	60
20	50	35	24	13	30	40	39	73	52	63	61
30	51	35	24	13	30	40	39	74	53	63	62
40	51	35	24	13	31	40	40	75	54	64	63
50	52	36	24	14	31	41	40	76	54	65	64
60	53	37	25	14	32	42	41	77	55	66	65
70	54	37	25	14	32	43	42	79	56	68	66
80	56	38	26	15	33	44	43	81	58	69	68
85	57	39	26	15	34	45	44	82	59	71	69
90	58	40	27	15	35	46	45	84	61	73	71
95	60	42	28	16	36	47	46	87	63	75	74
99	64	44	30	17	38	50	49	93	67	80	78

Table 6-20 Cumulative Probability of the On-Ramp Maximum Queue Length (Feet) for Two-Lane Blockage Incident in the Morning Peak Period at the Commercial Blvd. Bottleneck

percentile	Oakland	Sunrise	Broward	Davie	I-595	Griffin	Stirling	Sheridan	Hollywood	Pembroke	Hallandale Beach
10	1,225	850	575	325	725	975	950	1,775	1,275	1,550	1,500
20	1,250	875	600	325	750	1,000	975	1,825	1,300	1,575	1,525
30	1,275	875	600	325	750	1,000	975	1,850	1,325	1,575	1,550
40	1,275	875	600	325	775	1,000	1,000	1,875	1,350	1,600	1,575
50	1,300	900	600	350	775	1,025	1,000	1,900	1,350	1,625	1,600
60	1,325	925	625	350	800	1,050	1,025	1,925	1,375	1,650	1,625
70	1,350	925	625	350	800	1,075	1,050	1,975	1,400	1,700	1,650
80	1,400	950	650	375	825	1,100	1,075	2,025	1,450	1,725	1,700
85	1,425	975	650	375	850	1,125	1,100	2,050	1,475	1,775	1,725
90	1,450	1,000	675	375	875	1,150	1,125	2,100	1,525	1,825	1,775
95	1,500	1,050	700	400	900	1,175	1,150	2,175	1,575	1,875	1,850
99	1,600	1,100	750	425	950	1,250	1,225	2,325	1,675	2,000	1,950

Table 6-21 Cumulative Probability of the On-Ramp Maximum Queue Length (Vehicles) for Two-Lane Blockage Incident in the Afternoon Peak Period at the Commercial Blvd. Bottleneck

percentile	Oakland	Sunrise	Broward	Davie	I-595	Griffin	Stirling	Sheridan	Hollywood	Pembroke	Hallandale Beach
10	34	38	20	8	23	36	40	42	40	38	53
20	35	39	21	8	24	38	41	43	41	40	55
30	36	40	22	8	25	39	43	45	43	41	56
40	37	41	22	8	26	40	44	46	44	42	58
50	38	42	23	9	26	41	45	47	45	43	59
60	39	43	23	9	27	42	46	48	46	44	61
70	40	45	24	9	28	43	47	50	47	46	63
80	42	47	25	10	29	45	49	52	49	48	65
85	43	48	26	10	30	47	51	53	51	49	68
90	44	49	26	10	31	48	52	55	52	51	69
95	46	52	28	11	32	50	55	57	55	53	72
99	50	56	30	12	35	54	59	62	60	57	79

Table 6-22 Cumulative Probability of the On-Ramp Maximum Queue Length (Feet) for Two-Lane Blockage Incident in the Afternoon Peak Period at the Commercial Blvd. Bottleneck

percentile	Oakland	Sunrise	Broward	Davie	I-595	Griffin	Stirling	Sheridan	Hollywood	Pembroke	Hallandale Beach
10	850	950	500	200	575	900	1,000	1,050	1,000	950	1,325
20	875	975	525	200	600	950	1,025	1,075	1,025	1,000	1,375
30	900	1,000	550	200	625	975	1,075	1,125	1,075	1,025	1,400
40	925	1,025	550	200	650	1,000	1,100	1,150	1,100	1,050	1,450
50	950	1,050	575	225	650	1,025	1,125	1,175	1,125	1,075	1,475
60	975	1,075	575	225	675	1,050	1,150	1,200	1,150	1,100	1,525
70	1,000	1,125	600	225	700	1,075	1,175	1,250	1,175	1,150	1,575
80	1,050	1,175	625	250	725	1,125	1,225	1,300	1,225	1,200	1,625
85	1,075	1,200	650	250	750	1,175	1,275	1,325	1,275	1,225	1,700
90	1,100	1,225	650	250	775	1,200	1,300	1,375	1,300	1,275	1,725
95	1,150	1,300	700	275	800	1,250	1,375	1,425	1,375	1,325	1,800
99	1,250	1,400	750	300	875	1,350	1,475	1,550	1,500	1,425	1,975

6.3.4. Results of the Analysis of Ramp Metering Needs during Rainfall

According to the HCM (TRB, 2016), rainfall reduces the capacity of the freeway and the intensity of the impact corresponds to the intensity of the rainfall in terms in inches per hour (in/hr). The 2016 version of the HCM presents the remaining capacity of the freeway segment for two categories of rain: medium and heavy, corresponding to the intensity of the rainfall. Table 6-23 illustrates the presented values for remaining capacity for medium and heavy rain according to the free flow speed of the freeway segment. The free flow speed of the case study area is 75 mph, therefore, the remaining capacity percentage is 0.90 and 0.82 for the medium and heavy rain category, respectively.

Table 6-23 Remaining Capacity Ratio for Rainfall (Source: HCM (TRB, 2016))

Rain Category	Rain Intensity	Free Flow Speed of the Freeway				
		55 mph	60 mph	65 mph	70 mph	75 mph
Medium Rain	> 0.10-0.25 in/hr	0.94	0.93	0.92	0.91	0.90
Heavy Rain	> 0.25 in/hr	0.89	0.88	0.86	0.84	0.82

To address rainfall conditions, the two scenarios of the medium and heavy rains are considered. The method used to estimate the required number of ramps, metering rate, and resulting queue lengths is similar to the method used for recurrent conditions and incident conditions. The only difference is that the capacity values for each scenario are adjusted by the remaining capacity ratios presented in Table 6-23. As presented earlier for the incident conditions, Table 6-24 shows the total ramps to meter for medium and heavy rain conditions. Note that 1000 experiments are done for each scenario. Tables 6-25 to 6-32 shows the cumulative probability of the maximum queue length of on ramps for different rain intensity during the two peak periods. The analyst can examine the queue length results and if the queue length of specific ramps need to be decreased, the analyst can increase the constraint on the queue lengths of these specific ramps in the formulation presented in Equation 6-1.

Table 6-24 Frequency of Total Number of Ramps to Meter during Rainfall Conditions

Number of Metered Ramps	Medium Rain		Heavy Rain	
	Morning	Afternoon	Morning	Afternoon
0	52	76	0	11
1	0	0	0	0
2	1	7	0	1
3	28	19	0	2
4	61	25	0	4
5	54	19	4	7
6	66	24	8	8
7	81	53	15	17
8	53	21	11	10
9	71	18	30	12
10	96	90	54	33
11	437	647	878	895
Total	948	924	1,000	989

Table 6-25 Cumulative Probability of Maximum Queue Length (Vehicles) for Medium Rainfall in the Morning Peak

percentile	Oakland	Sunrise	Broward	Davie	I-595	Griffin	Stirling	Sheridan	Hollywood	Pembroke	Hallandale Beach
10	29	0	0	0	0	0	52	97	67	0	0
20	50	0	0	0	0	0	56	105	75	88	0
30	65	44	28	0	0	0	58	109	78	93	82
40	72	50	33	16	0	0	60	113	81	97	93
50	77	53	36	20	42	0	62	117	84	100	97
60	81	56	38	21	48	62	64	121	87	103	101
70	85	59	40	22	51	67	66	125	90	107	105
80	89	62	42	24	53	70	69	129	93	111	109
85	91	63	43	24	55	72	70	132	95	113	112
90	94	65	44	25	56	74	72	136	97	117	115
95	98	68	46	26	58	77	75	141	101	122	120
99	106	74	50	28	64	84	82	154	111	133	131

Table 6-26 Cumulative Probability of Maximum Queue Length (Feet) for Medium Rainfall in the Morning Peak

percentile	Oakland	Sunrise	Broward	Davie	I-595	Griffin	Stirling	Sheridan	Hollywood	Pembroke	Hallandale Beach
10	725	0	0	0	0	0	1,300	2,425	1,675	0	0
20	1,250	0	0	0	0	0	1,400	2,625	1,875	2,200	0
30	1,625	1,100	700	0	0	0	1,450	2,725	1,950	2,325	2,050
40	1,800	1,250	825	400	0	0	1,500	2,825	2,025	2,425	2,325
50	1,925	1,325	900	500	1,050	0	1,550	2,925	2,100	2,500	2,425
60	2,025	1,400	950	525	1,200	1,550	1,600	3,025	2,175	2,575	2,525
70	2,125	1,475	1,000	550	1,275	1,675	1,650	3,125	2,250	2,675	2,625
80	2,225	1,550	1,050	600	1,325	1,750	1,725	3,225	2,325	2,775	2,725
85	2,275	1,575	1,075	600	1,375	1,800	1,750	3,300	2,375	2,825	2,800
90	2,350	1,625	1,100	625	1,400	1,850	1,800	3,400	2,425	2,925	2,875
95	2,450	1,700	1,150	650	1,450	1,925	1,875	3,525	2,525	3,050	3,000
99	2,650	1,850	1,250	700	1,600	2,100	2,050	3,850	2,775	3,325	3,275

Table 6-27 Cumulative Probability of Maximum Queue Length (Vehicle) for Medium Rainfall in the Afternoon Peak

percentile	Oakland	Sunrise	Broward	Davie	I-595	Griffin	Stirling	Sheridan	Hollywood	Pembroke	Hallandale Beach
10	27	0	0	0	0	0	56	57	0	0	0
20	45	50	0	0	0	0	63	66	63	60	80
30	58	65	34	13	38	0	71	75	72	69	93
40	65	72	39	15	45	69	78	81	78	75	103
50	68	76	41	16	47	73	80	84	81	78	107
60	70	78	42	16	49	75	83	86	83	80	109
70	73	81	44	17	50	78	85	90	86	83	113
80	76	85	45	18	53	81	89	94	90	86	119
85	79	88	47	18	54	84	93	97	93	90	123
90	82	91	49	19	57	88	96	101	97	93	128
95	85	95	51	20	59	91	100	105	100	97	133
99	91	101	54	21	63	97	107	112	107	103	141

Table 6-28 Cumulative Probability of Maximum Queue Length (Feet) for Medium Rainfall in the Afternoon Peak

percentile	Oakland	Sunrise	Broward	Davie	I-595	Griffin	Stirling	Sheridan	Hollywood	Pembroke	Hallandale Beach
10	675	0	0	0	0	0	1,400	1,425	0	0	0
20	1,125	1,250	0	0	0	0	1,575	1,650	1,575	1,500	2,000
30	1,450	1,625	850	325	950	0	1,775	1,875	1,800	1,725	2,325
40	1,625	1,800	975	375	1,125	1,725	1,950	2,025	1,950	1,875	2,575
50	1,700	1,900	1,025	400	1,175	1,825	2,000	2,100	2,025	1,950	2,675
60	1,750	1,950	1,050	400	1,225	1,875	2,075	2,150	2,075	2,000	2,725
70	1,825	2,025	1,100	425	1,250	1,950	2,125	2,250	2,150	2,075	2,825
80	1,900	2,125	1,125	450	1,325	2,025	2,225	2,350	2,250	2,150	2,975
85	1,975	2,200	1,175	450	1,350	2,100	2,325	2,425	2,325	2,250	3,075
90	2,050	2,275	1,225	475	1,425	2,200	2,400	2,525	2,425	2,325	3,200
95	2,125	2,375	1,275	500	1,475	2,275	2,500	2,625	2,500	2,425	3,325
99	2,275	2,525	1,350	525	1,575	2,425	2,675	2,800	2,675	2,575	3,525

Table 6-29 Cumulative Probability of Maximum Queue Length (Vehicle) for Heavy Rainfall in the Morning Peak

percentile	Oakland	Sunrise	Broward	Davie	I-595	Griffin	Stirling	Sheridan	Hollywood	Pembroke	Hallandale Beach
10	71	49	33	19	41	0	58	109	78	94	93
20	79	54	37	21	47	62	61	115	82	99	97
30	81	56	38	21	48	63	62	117	84	101	99
40	82	57	38	22	49	65	63	119	85	102	101
50	83	58	39	22	50	66	64	121	87	104	102
60	85	59	40	22	51	67	66	123	89	106	104
70	87	60	41	23	52	68	67	126	90	108	107
80	90	62	42	24	53	70	69	130	93	112	110
85	92	63	43	24	55	72	71	133	95	114	112
90	95	65	44	25	56	74	73	137	98	118	116
95	99	68	46	26	59	78	76	143	103	123	121
99	107	74	50	28	64	84	83	155	111	133	131

Table 6-30 Cumulative Probability of Maximum Queue Length (Feet) for Heavy Rainfall in the Morning Peak

percentile	Oakland	Sunrise	Broward	Davie	I-595	Griffin	Stirling	Sheridan	Hollywood	Pembroke	Hallandale Beach
10	1,775	1,225	825	475	1,025	0	1,450	2,725	1,950	2,350	2,325
20	1,975	1,350	925	525	1,175	1,550	1,525	2,875	2,050	2,475	2,425
30	2,025	1,400	950	525	1,200	1,575	1,550	2,925	2,100	2,525	2,475
40	2,050	1,425	950	550	1,225	1,625	1,575	2,975	2,125	2,550	2,525
50	2,075	1,450	975	550	1,250	1,650	1,600	3,025	2,175	2,600	2,550
60	2,125	1,475	1,000	550	1,275	1,675	1,650	3,075	2,225	2,650	2,600
70	2,175	1,500	1,025	575	1,300	1,700	1,675	3,150	2,250	2,700	2,675
80	2,250	1,550	1,050	600	1,325	1,750	1,725	3,250	2,325	2,800	2,750
85	2,300	1,575	1,075	600	1,375	1,800	1,775	3,325	2,375	2,850	2,800
90	2,375	1,625	1,100	625	1,400	1,850	1,825	3,425	2,450	2,950	2,900
95	2,475	1,700	1,150	650	1,475	1,950	1,900	3,575	2,575	3,075	3,025
99	2,675	1,850	1,250	700	1,600	2,100	2,075	3,875	2,775	3,325	3,275

Table 6-31 Cumulative Probability of Maximum Queue Length (Vehicle) for Heavy Rainfall in the Afternoon Peak

percentile	Oakland	Sunrise	Broward	Davie	I-595	Griffin	Stirling	Sheridan	Hollywood	Pembroke	Hallandale Beach
10	56	62	33	13	37	0	68	72	68	66	90
20	64	71	38	15	44	68	75	79	75	73	100
30	66	73	39	15	45	70	77	81	77	75	103
40	68	75	40	16	47	72	79	83	80	77	105
50	69	77	41	16	47	74	81	85	81	78	108
60	70	78	42	16	49	75	83	87	83	80	110
70	73	81	43	17	50	78	85	90	86	83	113
80	76	85	45	18	52	81	89	94	89	86	118
85	78	86	46	18	54	83	91	96	91	88	121
90	80	89	48	19	55	85	94	98	94	91	124
95	84	94	51	20	58	90	99	104	99	96	132
99	90	101	54	21	62	97	106	112	107	103	141

Table 6-32 Cumulative Probability of Maximum Queue Length (Feet) for Heavy Rainfall in the Afternoon Peak

percentile	Oakland	Sunrise	Broward	Davie	I-595	Griffin	Stirling	Sheridan	Hollywood	Pembroke	Hallandale Beach
10	1,400	1,550	825	325	925	0	1,700	1,800	1,700	1,650	2,250
20	1,600	1,775	950	375	1,100	1,700	1,875	1,975	1,875	1,825	2,500
30	1,650	1,825	975	375	1,125	1,750	1,925	2,025	1,925	1,875	2,575
40	1,700	1,875	1,000	400	1,175	1,800	1,975	2,075	2,000	1,925	2,625
50	1,725	1,925	1,025	400	1,175	1,850	2,025	2,125	2,025	1,950	2,700
60	1,750	1,950	1,050	400	1,225	1,875	2,075	2,175	2,075	2,000	2,750
70	1,825	2,025	1,075	425	1,250	1,950	2,125	2,250	2,150	2,075	2,825
80	1,900	2,125	1,125	450	1,300	2,025	2,225	2,350	2,225	2,150	2,950
85	1,950	2,150	1,150	450	1,350	2,075	2,275	2,400	2,275	2,200	3,025
90	2,000	2,225	1,200	475	1,375	2,125	2,350	2,450	2,350	2,275	3,100
95	2,100	2,350	1,275	500	1,450	2,250	2,475	2,600	2,475	2,400	3,300
99	2,250	2,525	1,350	525	1,550	2,425	2,650	2,800	2,675	2,575	3,525

6.4. Benefit-Cost Analysis of Ramp Metering Deployment

The benefits of the proposed methodology is evaluated in terms of delay savings. For this purpose, the queuing theory equations are used to calculate the delays for both freeway mainline and the on-ramps (May, 1990). The queue and associated delay on the mainline forms due to breakdown occurrence at the bottleneck locations. The queues due to metering are those that occur on the on-ramps due to ramp demands exceeding the ramp capacities with metering. The delay saving for each scenario or event type is calculated based on the difference between the delays in the absence of ramp metering and with the ramp metering application. Based on previous analysis of incident data from the case study corridor, the average incident durations utilized in the analysis are 40 minutes for shoulder incidents and 50 minutes for incidents with lane blockages (Hadi et al., 2007). The total delay on the freeway mainline and each on-ramp (TD) is calculated using Equation 6-2 based on queuing theory analysis (May, 1990).

$$TD = t_R t_Q (\lambda - \mu_R) / 2 \quad (6-2)$$

$$t_Q = t_R (\mu - \mu_R) / (\mu - \lambda) \quad (6-3)$$

where:

λ = Arrival rate (demand) (vph), and

μ = Capacity rate under recurrent condition (vph), and

μ_R = Reduced capacity rate under metering for ramps or under non-recurrent conditions for the mainline (vph), and

t_R = Event duration (hr), and
 t_Q = Time in queue duration (hr).

The delay savings can be converted to monetary values to be incorporated into benefit-cost analyses.

The utilized methodology described in the previous sections can be used to conduct benefit-cost analysis of the ramp metering deployment and associated decisions such as which ramp to meter and when to activate. A summary of calculated delay saving of ramp metering is presented in Table 6-33 for recurrent and non-recurrent conditions. As aforementioned, the delays are calculated for the two peak periods. The duration of the rainfall is assumed to be one hour and the duration of the incident to be 40 minutes for shoulder incidents and 50 minutes for lane blockage incidents. Table 6-33 indicates that ramp metering can provide significant benefits in terms of mobility and reliability. During recurrent conditions, the median delays decreased by 859 veh-hrs (about 7.8 minutes/veh) and 1,174 veh-hrs (about 10.3 minute/vehicles) during the AM and PM peak hours, respectively. The delay savings are even more during shoulder and one-lane blockage incidents and rain events that occur during the peak hours, as shown in Table 6-33. In case of two-lane blockage incidents, ramp metering is not capable of preventing the breakdown; therefore, the queue forms on the mainline and reduces the delay savings.

The following discussion illustrates the use of the delay saving results in benefit-cost analysis of the ramp metering deployment. The monetary value of the delay savings is calculated by multiplying the median delay savings in veh-hr by the value of time assumed to be \$13.0 per passenger-hr and passenger occupancy of 1.2 passengers/vehicle based on previous studies in the region. The annual benefit of the system-wide metering in the case study area is calculated assuming 48% and 63% of the days with no events having recurrent congestion in the morning and afternoon peaks, respectively, as obtained from the Monte Carlo analysis discussed earlier. 12% and 5% of the weekdays were assumed to have additional delays due to incident and rain events, respectively. The cost values were obtained from FDOT District 4. The project life is assumed to be 7 years and the interest rate is assumed to be 6%. The results show that benefit-cost ratio of the analysis is about 9.4.

The results in Table 6-33 illustrate the use of the methodology of this study to estimate the impacts of ramp metering on system performance and the return on investment of the deployment. The method can be used also to determine the performance and return on investment of alternative ramp selection decisions. Such analyses can be combined with other criteria such as the maximum queue length on each ramp and the available budget for deployment to support the decision to select ramps for metering.

Table 6-33 Delay Savings Due to Ramp Metering with Recurrent and Non-Recurrent Conditions and the Associated Benefit-Cost Analysis

Event	Time	percentile	Total Delay with Metering (veh-hr)	Total Delay without Metering (veh-hr)	Total Delay Saving (veh-hr)	Benefit (\$)
Recurrent Condition	AM	50	21	880	859	13,400
		85	372	1,248	876	13,666
	PM	50	166	1,340	1,174	18,314
		85	615	2,148	1,533	23,915
Medium Rain	AM	50	446	1,144	698	10,889
		85	622	1,532	910	14,196
	PM	50	485	1,604	1,119	17,456
		85	602	2,432	1,830	28,548
Heavy Rain	AM	50	512	1,638	1,126	17,566
		85	647	1,818	1,171	18,268
	PM	50	499	2,311	1,812	28,267
		85	587	3,193	2,606	40,654
Shoulder Blocked Incident	AM	50	1,152	3,033	1,880	29,328
		85	1,316	3,168	1,853	28,907
	PM	50	1,093	4,264	3,171	49,468
		85	1,298	4,403	3,105	48,438
One Lane Blocked Incident	AM	50	3,447	4,813	1,365	21,294
		85	3,783	5,183	1,400	21,840
	PM	50	3,444	6,454	3,010	46,956
		85	3,796	6,841	3,045	47,502
Two Lanes Blocked Incident	AM	50	5,717	5,901	183	2,855
		85	6,205	6,355	149	2,324
	PM	50	5,691	5,906	215	3,354
		85	6,264	6,437	173	2,699
Benefit-Cost Analysis Results						
Benefit			Annual benefit for Total of 11 Ramps			\$ 6,419,910
Cost			Capital Cost for 11 Ramps			\$ 3,320,000
			Annual Operation and Maintenance Cost for 11 ramps			\$121,451
Interest Parameters			Interest Rate			6%
			Project Life			7 years
Benefit-Cost Ratio						9.4

7. CONSIDERATION OF ACCELERATION LENGTH AND QUEUE STORAGE

The application of the ramp selection formulation presented in Chapter 6 addressed the offline selection of ramps to meter for the mitigation of recurrent and non-recurrent congestion. This section extends the formulation application to consider the variation in queue storage capacity between ramps, the available acceleration lanes, and the related impacts of trucks. The ramp metering stop line can be located on the ramp some distance upstream of the gore area to increase the acceleration distance, as shown in Figure 7-1. However, this will reduce the available storage capacity. Thus, it is important to reach an adequate balance between these two geometric design elements.

Figure 7-1 shows that there is a balance between the available queue storage capacity and the acceleration length. Any increase in the queue storage will reduce the available acceleration length. In most cases, the total available distance is limited and there is a need to provide the required queue storage length without sacrificing safety by using inadequate acceleration length, considering that the required acceleration length from stop is significantly higher than that required from traveling at the operating speed of the on-ramps. This is complicated by the fact that trucks require significantly longer acceleration lengths than passenger cars and they should be treated differently when examining these issues.

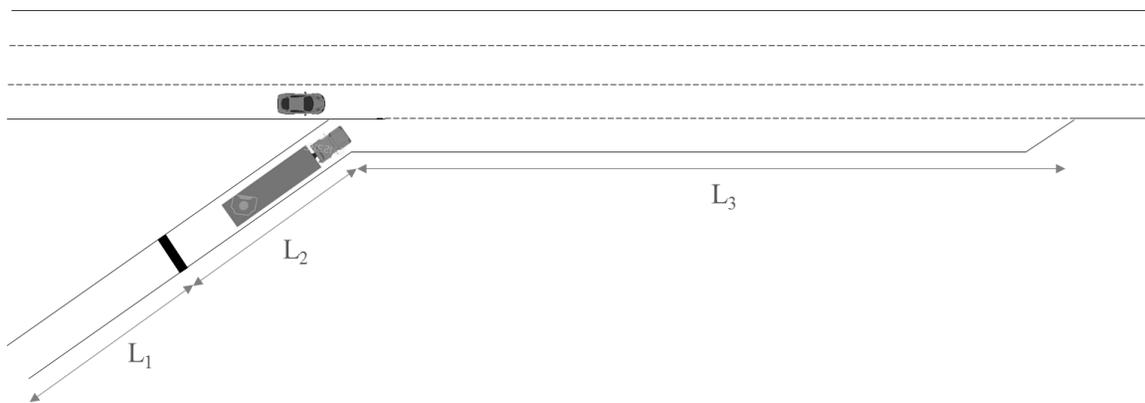


Figure 7-1 Illustration of the Balance between Queue Storage Capacity and Available Acceleration Length

7.1. Warrants on Acceleration Length and Queue Length for Ramp Metering

Ramp metering installation warrants, considering acceleration lanes and queue lengths, have been developed by a number of states, as described in Chapter 3. However, these warrants are generally simple and examines the conditions in the immediate vicinity of each ramp (local conditions) to determine if a ramp meter is warranted. Table 7-1 provides a summary of these warrants. The warrants aim to provide sufficient queue storage capacity to reduce queue

spillover to the upstream arterial streets and sufficient acceleration lengths to allow vehicles to reach sufficient speed from zero initial speed at the metering stop line, in order to merge safely with the mainline traffic flow.

Table 7-1 Summary of Ramp Length and Queue Storage Warrants Developed by a Number of States

States	Acceleration Lane Length Warrants	Queue Storage Length Warrants						
Arizona	<p>For Grade (G) less than 1% and less than 3% Trucks (T):</p> <table border="0"> <tr> <td>Freeway Speed</td> <td>Acceleration length</td> </tr> <tr> <td>55 mph</td> <td>600 ft.</td> </tr> <tr> <td>65 mph</td> <td>800 ft.</td> </tr> </table> <p>Increase in Acceleration length = $G \times 65$, when $G > 1\%$ and $(T - 3) \times 100$ When $T > 3\%$</p>	Freeway Speed	Acceleration length	55 mph	600 ft.	65 mph	800 ft.	<p>Queue Storage Distance =</p> $\frac{(\text{Rate}_{\text{ramp}} - \text{Rate}_{\text{meter}}) * \text{Time} * (L_{\text{car}}(1 - \frac{T}{100}) + L_{\text{Truck}}(\frac{T}{100}))}{\text{Lane}}$ <p>Minimum Queue Storage Length= 400 feet $\text{Rate}_{\text{ramp}}$= Entrance ramp design flow rate (vph) $\text{Rate}_{\text{meter}}$= Design metering rate (vph) L_{car}= Average car plus gap length (ft/veh) L_{Truck}= Average truck plus gap length (ft/veh) T= Percentage of trucks in entrance ramp traffic (percent)</p>
Freeway Speed	Acceleration length							
55 mph	600 ft.							
65 mph	800 ft.							
California	<table border="1"> <tr> <td>Minimum auxiliary lane length</td> <td>Condition:</td> </tr> <tr> <td>150 m, beyond ramp convergence point</td> <td>Truck volume > 5% and Upgrade > 3%</td> </tr> <tr> <td>300 m, beyond ramp convergence point</td> <td>Truck Volume > 5%, Upgrade > 3% and Volume_{ramp} > 1500 vph</td> </tr> </table>	Minimum auxiliary lane length	Condition:	150 m, beyond ramp convergence point	Truck volume > 5% and Upgrade > 3%	300 m, beyond ramp convergence point	Truck Volume > 5%, Upgrade > 3% and Volume _{ramp} > 1500 vph	<p>A ramp with peak hour volume between 500 and 900, a two-lane ramp meter may be provided to double the vehicles stored within the available storage area. A single-lane ramp meter should be used when rates are below 500 vph and no HOV preferential lane is provided.</p>
Minimum auxiliary lane length	Condition:							
150 m, beyond ramp convergence point	Truck volume > 5% and Upgrade > 3%							
300 m, beyond ramp convergence point	Truck Volume > 5%, Upgrade > 3% and Volume _{ramp} > 1500 vph							
Florida	<p>Acceleration distance is greater than the minimum requirement determined by the freeway mainline prevailing speed. Acceleration lane length equation as follows: $L = 0.3987V^2 - 26.62V + 267$. Where, L = required minimum acceleration distance (feet), and V = freeway mainline prevailing speed (mph).</p>	<p>Ramp storage distance is greater than the minimum requirement determined by the peak hour ramp volume. In order to estimate the queuing length, the following equation can be used: $L = 0.25 V - 0.00007422 V^2$ Where: L= required single lane storage length (m) V= Peak hour ramp demand (vph)</p>						
Nevada	<p>The minimum acceleration length must conform to the most recent edition of the AASHTO Green Book. If the available acceleration length after the stop bar is longer than the required acceleration length allows vehicles on the ramp to accelerate within 5 mph of the freeway's operating speed. Providing inadequate acceleration length is not allowed under any circumstance</p>	<p>If the available ramp storage length is greater than the estimated queuing length on the ramp.</p>						

Table 7-1 Summary of Ramp Length and Queue Storage Warrants Developed by a Number of States (Con't)

States	Acceleration Lane Length Warrants	Queue Storage Length Warrants
New York	Adequate parallel surface routes must be available for the traffic diverted from the ramps.	Ramp metering is considered feasible when, <ul style="list-style-type: none"> ➤ Available ramp storage exceeds 10% of the pre-metered peak-hour volume. ➤ Ramp storage for 5% to 10% of the peak-volume with additional analysis and possible mitigating measures (e.g., additional ramp lane, queue detection, etc.) ➤ Ramp metering is not considered feasible if the storage is less than 5% of the pre-metered peak-hour volume.
Texas	Average traffic flow rate of the two right-most lanes and combined traffic flow rate in the rightmost freeway lane plus the flow rate on the entrance ramp during peak-periods exceeds a minimum of 1,600 vehicles per hour per lane (vphpl) and 2,300 vphpl respectively, with acceleration lanes of 500 feet or less, and this threshold level increases as the length of the acceleration lane on the ramp increases.	-

7.2. Acceleration Length Requirements

Acceleration length requirement for starting from stop can be a major challenge when designing ramp metering, particularly when considering trucks, which require longer distances to accelerate compared to passenger cars. As acceleration capability of trucks is approximately 60 percent of that for passenger cars, acceleration length requirement for trucks is higher comparing to passenger car (Tian et al., 2016).

Table 7-2 summaries the conservative and aggressive ramp metering requirements for passenger cars based on the literature reviewed in Section 3.3.4. Table 7-3 presents the values obtained according to the aggressive and conservative designs for truck acceleration length based on the recommendations of Tian et al. (2016) and NCHRP Report 505 (Harwood et al., 2003). The values in these tables were obtained based on the values recommended in the existing sources. Since these values are recommended for design purposes in these sources, they are expected to provide adequate acceleration for 85% to 90% of the traffic, as is normally expected with the values recommended in design standards and guidelines.

Tian et al. (2016) recommended two designs for acceleration lengths with ramp metering. The conservative design was recommended for ramps that that have sufficient space, while the aggressive design was recommended for on-ramps that have insufficient distance to accommodate acceleration. According to Tian et al. (2016), the AASHTO Green Book design guidance could be reduced by 10% for the conservative design and by 35% for the aggressive design.

Table 7-2 Recommended Minimum Acceleration Distance for Passenger Cars at 70 mph Freeway Design Speeds According to Different Sources

Source	Acceleration Length, L (ft.) for Entrance Curve Design Speed (mph)										
	Highway Design Speed (mph)	Speed Reached (mph)	Stop Condition	15	20	25	30	35	40	45	50
			and Initial Speed								
			0	14	18	22	26	30	36	40	44
AASHTO Green Book (AASHTO, 2011)	70	53	1,620	1,560	1,520	1,420	1,350	1,230	1,000	820	580
Aggressive Design (Tian et al., 2016)	70	53	1,053	1,014	988	923	878	800	650	533	377
Conservative Design (Tian et al., 2016)	70	53	1,458	1,404	1,368	1,278	1,215	1,107	900	738	522
Average (Tian et al., 2016)	70	53	1,256	1,209	1,178	1,101	1,046	953	775	636	450

Table 7-3 Recommended Minimum Acceleration Distance for Trucks and 70 mph Freeway Design Speed According to Different Sources

Source	Highway design speed (mph)	Speed reached (mph)	Acceleration length (ft) for Entrance Curve Design Speed (mph)								
			Stop Condition	15	20	25	30	35	40	45	50
			and Initial Speed								
			0	14	18	22	26	30	36	40	44
NCHRP Report 505 (Harwood et al., 2003)	70	53	3,060	2,960	2,900	2,800	2,670	2,510	2,140	1,810	1,260
Aggressive Design (Tian et al., 2016)	70	53	1,685	1,622	1,581	1,477	1,404	1,279	1,040	853	603
Conservative Design (Tian et al., 2016)	70	53	2,333	2,246	2,189	2,045	1,944	1,771	1,440	1,181	835
Average (Tian et al., 2016)	70	53	2,009	1,934	1,885	1,761	1,674	1,525	1,240	1,017	719

Figures 7-2 and 7-3 shows the acceleration length requirement pattern according to the AASHTO Green Book (AASHTO, 2011) for passenger cars and NCHRP Report 505 (Harwood et al., 2003) for trucks, along with the the conservative and aggressive designs and their averages according to Tian et al. (2016). The recommended values are plotted as a function of the initial

speed with the requirement to achieve a merge speed of 53 mph with mainline traffics. It can be seen that variation in length requirements decreases with increasing initial speed for both passenger cars and trucks.

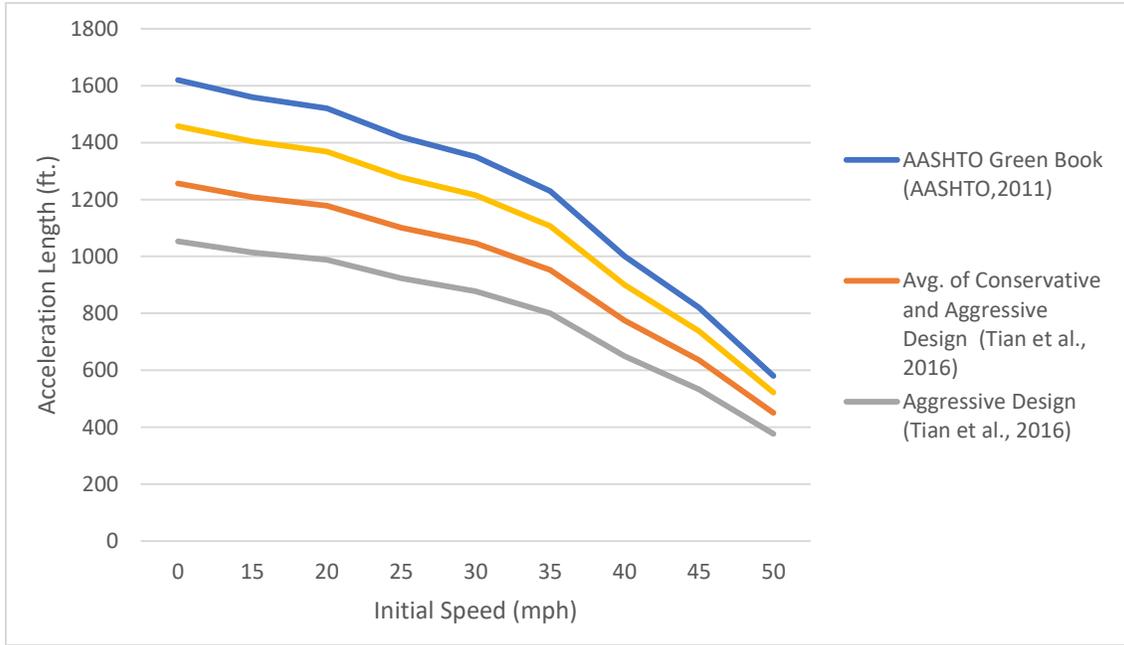


Figure 7-2 Comparison of Passenger Car Acceleration Length Requirements According to Different Sources

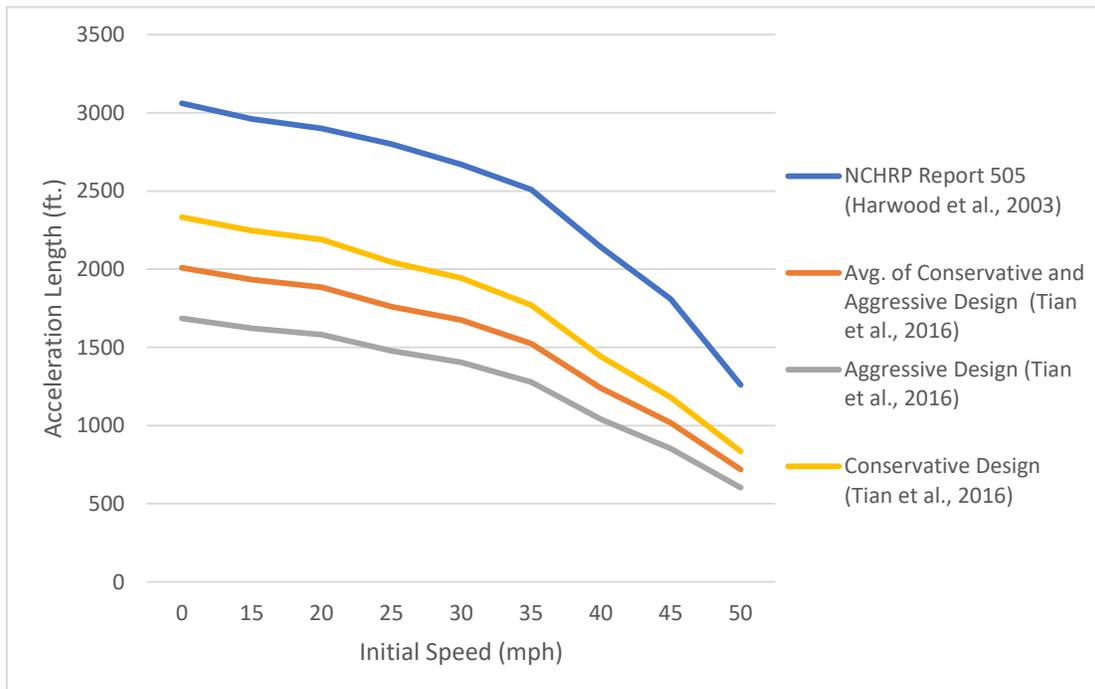


Figure 7-3 Comparison of Truck Acceleration Length Requirements According to Different Sources

7.3. Statistics of Existing Acceleration Lane Lengths around the Country

As mentioned in Section 4.2, metered ramps with shorter acceleration lengths were selected for analysis from Los Angeles, Denver, Minneapolis, Atlanta, Seattle, and Miami. The design speed for all these facilities are 60 mph, except Seattle, where the corresponding design speed is 65 mph (design speed is assumed to equal the speed limit plus 5 mph according to HCM 2016). The minimum acceleration distances and maximum acceleration distances among the selected ramps were identified and the corresponding state warrant for the minimum acceleration lengths are collected. It should be noted that for design speed 60 mph and 65 mph, the vehicles will have to reach 47 and 50 mph merging speeds, respectively (AASHTO, 2011).

Table 7-4 illustrates the minimum, average, and maximum acceleration lengths for the considered on-ramps, along with the minimum acceleration lane requirements according to AASHTO (2011), NCHRP Report 505 (Harwood et al., 2003), and Tian et al. (2016).

Table 7-4 Statistics of the Acceleration Lane Lengths for Selected Ramps with Short Acceleration Lengths

Cities	Acceleration Length (ft.) in Existing Ramps			Minimum Acceleration Length Required for Passenger Car (ft.) for Metered Ramp			Minimum Acceleration Length Required for Trucks (ft.) for Metered Ramp		
	Min.	Avg.	Max.	AASHTO Green Book (AASHTO, 2011)	Conservative Design by Tian et al. (2016)	Aggressive Design by Tian et al. (2016)	NCHRP Report 505 (Harwood et al., 2003)	Conservative Design by Tian et al. (2016)	Aggressive Design by Tian et al. (2016)
Los Angeles	585	670	720	1,200	1,080	780	2,000	1,728	1,248
Denver	575	690	785	1,200	1,080	780	2,000	1,728	1,248
Minneapolis	425	527.5	635	1,200	1,080	780	2,000	1,728	1,248
Atlanta	500	581.25	725	1,200	1,080	780	2,000	1,728	1,248
Seattle	800	937.5	1,050	1,410	1,269	917	2,490	2,030	1,466
Miami	850	963.75	1,025	1,200	1,080	780	2,000	2,030	1,248

According to Table 7-4, the average acceleration length in Miami and Seattle satisfies the aggressive design threshold recommended by Tian et al (2016) for passenger cars but it is not sufficient for trucks to accelerate from a stop condition to desired merging speed.

7.4. Queue Storage Requirements

The existing signal warrants require that the ramp queue storage capacities are able to accommodate 90% of the demands. The available queue storage should be calculated

considering the distance required from the gore back to on-ramp stop line to provide sufficient distance for acceleration (L_2 as shown in Figure 7-1). The queue storage in feet is converted to vehicles by assuming that each queued vehicle and the associated distance headway is 25 ft using the following equation:

$$\text{Queue Storage (Veh.)} = \frac{\text{Ramp Length (ft.)} - L_2 \text{ (ft.)}}{25 \text{ ft./veh.}} \times \text{No. of Lanes in Ramp} \quad (7-1)$$

This storage is converted to a proportion of the ramp volume for use in the ramp identification formulation of this study.

7.5. Alternative Approaches to Accommodate Acceleration Requirements

Five alternatives were identified to consider the ramps that do not satisfy the acceleration length requirements. The following are these five possible alternatives with a description of how each alternative is considered in the formulation presented in Chapter 6 for off-line selection of the ramps. The results from applying the formulation will be compared for the project case study to identify the impacts of the five alternatives.

- **Alternative 1:** Not to include the on-ramp in the metering

This alternative involves not metering the on-ramps that do not meet the required acceleration lengths for the trucks when having high truck traffic at a given period of the day. These ramps are excluded from the optimization in the linear programming formulation utilized in this study. In the formulation, the metering rate on such a ramp is set to be equal to the traffic demand on the ramp. This demand is deducted from the bottleneck capacity available to other traffic to account for the traffic coming from the unmetered ramps.

- **Alternative 2:** Pushing the metering stop line back on the on-ramp to increase the acceleration distance but decreasing the queuing storage capacity

The available storage length on each on-ramp determines the maximum allowable queue and eventually the assigned metering rate. Therefore, reducing the queuing storage capacity results in increasing the minimum metering rate and changing this parameter in the input values of the linear programming formulations will reflect this strategy.

- **Alternative 3:** Providing truck preemption on the on-ramp to turn the ramp metering signal to green once a truck is detected at a certain distance from the stop line to allow the trucks to accelerate to an acceptable speed when merging

Based on this alternative, once the truck is detected, the ramp signal will discharge all the vehicles ahead of the truck on the on-ramp to the freeway mainline. To account for this in the formulation, the number of these vehicles is calculated based on the truck percentage in the on-ramp traffic and the distance between the detection point and the stop line. The number of vehicles is deducted from the capacity of the mainline bottleneck since these vehicles are not metered.

- **Alternative 4:** Providing a lane for truck by-pass at a distance that allows the truck to accelerate to an acceptable speed when merging

This strategy directly impacts the number of vehicles, which can be stored in the storage area and eventually impacts the minimum metering rate. In addition, this strategy is only applicable if the number of the lanes on the on-ramp is more than one lane. In the formulation, the equivalent passenger car volume of the truck will be deducted from the bottleneck capacity. If a by-pass lane is actually added to the existing geometry, then this will not impact the storage capacity but will just results in not metering the truck traffic. This alternative with adding a lane is referred to as alternative 4-b.

- **Alternative 5:** Diverting truck traffic to other ramps with sufficient acceleration lengths during the ramp metering periods

This strategy diverts trucks from on-ramps with acceleration length constraints. To consider this strategy in the formulation, the trucks on the on-ramps from which the trucks are to be diverted, are deducted from the volume of those ramps and added to the volumes of adjacent ramps which can accommodate the required acceleration lengths of the trucks.

7.6. Application to Project Case Study

The case study of this project is the northbound segment of I-95 between Hallandale Blvd and Commercial Blvd in Broward County. All on-ramps in this segment have auxiliary lanes and enough acceleration length. Thus, to address the issue of insufficient acceleration length for trucks, hypothetical lane drops were assumed on three of the on-ramps that exhibit higher percentages of truck traffic compared to other on-ramps based on FDOT statistics office data. The assumed available acceleration length for the three ramps is 1,301 ft. The assumed available distance 1,301 ft. acceleration length satisfies the acceleration length value of the average of conservative and aggressive acceleration from stop design for passenger cars, according to Tian et al. (2016). However, it does not satisfy the requirement for truck acceleration. Thus, the alternative approaches to accommodate truck acceleration length requirements discussed earlier,

have to be investigated. The distance from the ramp stop line to the gore point (L_2) is set to 340 ft, which is required for passenger cars to reach 31 mph from stop condition at design speed 40 mph, according to Green Book requirements (AASHTO, 2011). Table 7-5 presents geometric configuration and truck percentage for each of the three ramps selected for the acceleration length investigation of this study.

Table 7-5 Geometric Configurations and Truck Percentages of the Ramps Selected for Acceleration Length Investigation

Ramp Location	Ramp Length (ft.)	No. of Lanes on Ramp	No. of Lanes at Gore to Merge point	Percentage of Truck
Griffin	1,210	2	1	5.6
Sunrise	1,700	2	1	4.9
Davie	1,100	3	2	4

Table 7-6 represents the acceleration length requirement for the ramps from Griffin, Sunrise and Davie road, the length that the stop line should be pushed back from the gore to accommodate the required acceleration lengths, and the remaining queue storage capacity on the three ramps for different alternatives. Figure 7-4 displays the remaining queue storage calculated in the last column of Table 7-6 utilizing the average values of aggressive and conservative designs. As can be seen, even with the aggressive design, if the adopted alternative is to push the stop line back to accommodate the truck acceleration, the queue storage reduces to 38, 78, and 45 vehicles for Griffin Road, Sunrise Blvd., and Davie Road, respectively. With the conservative designs, the corresponding values are -13, 26, and -21 vehicles, with the negative values indicating excessive queue that spills back to the upstream streets. With the preemption or the by-pass lane alternatives, the truck is allowed to accelerate either on a by-pass lane or by turning the signal green when the truck is at a designated point on the ramp that allows the truck to merge at a safe speed on the freeway. The queue storage capacities with these two alternatives, according to the aggressive design, increases to 71, 110, and 94 vehicles. Diverting trucks to other ramps, if this alternative is feasible and does not impact adversely the arterial street operations and the operations at other ramps, will increase the queue storage further to 90, 130, and 123 vehicles, assuming the aggressive design. There is no queuing consideration with the fifth alternative.

Table 7-6 represents the acceleration length requirement for the ramps from Griffin, Sunrise and Davie road, the length that the stop line should be pushed back from the gore to accommodate the required acceleration lengths, and the remaining queue storage capacity on the three ramps for different alternatives. Figure 7-4 displays the remaining queue storage calculated in the last column of Table 7-6 utilizing the average values of aggressive and conservative designs. As can be seen, even with the aggressive design, if the adopted alternative is to push the stop line back to accommodate the truck acceleration, the queue storage reduces to 38, 78, and 45 vehicles for

Griffin Road, Sunrise Blvd., and Davie Road, respectively. With the conservative designs, the corresponding values are -13, 26, and -21 vehicles, with the negative values indicating that the full length of the ramp is not adequate for acceleration of trucks to an acceptable speed. With the preemption or the by-pass lane alternatives, the truck is allowed to accelerate either on a by-pass lane or by turning the ramp signal green when the truck is at a designated point on the ramp that allows the truck to merge at a safe speed on the freeway. The queue storage capacities with these two alternatives, according to the aggressive design, increases to 71, 110, and 94 vehicles, respectively for the three ramps. Diverting trucks to other ramps, if this alternative is feasible and does not impact adversely the arterial street operations and the operations at other ramps, will increase the queue storage further to 90, 130, and 123 vehicles, respectively, with the aggressive design.

The results from including the proposed alternatives to accommodate truck acceleration in the formulation of this study are presented below. Table 7-7 and Table 7-8 present the percentage of the number of ramps to be metered for each alternative under recurrent conditions for aggressive and conservative designs, respectively. The percentage of a number of ramps to meter “i” is calculated as the number of experiments which requires “i” number of ramps to meter divided by total number of experiments which resulted in one or more ramps to be metered. The utilized data is for the average morning and afternoon peak period traffic. As shown in Table 2-5 and Table 2-6, Alternatives 1 and 5 results in fewer number of ramps to be metered for the morning and afternoon peaks for both aggressive and conservative designs.

Table 7-6 Queue Storage Capacity Calculation with Different Alternatives to Accommodate Truck Acceleration Requirements

Aggressive Design									
Alternatives	Initial Speed (mph)	Design Speed (mph)	Final Speed (mph)	Acceleration Length Required for PC/Trucks (ft)	Required L₂ for Passenger Cars (ft)	Required L₂ for Trucks (ft)	Ramp	Queue Storage Length (ft)	Queue Storage (veh)
Alternative 2	0	70	53	1,053/1,685	74	724	Griffin	486	38
							Sunrise	976	78
							Davie	376	45
Alternative 3 and 4	35	70	53	1,035/1,279	74	318	Griffin	892	71
							Sunrise	1,382	110
							Davie	782	94
Alternative 5	0	70	53	1,035	74	-	Griffin	1,136	90
							Sunrise	1,626	130
							Davie	1,026	123
Conservative Design									
Alternatives	Initial Speed (mph)	Design Speed (mph)	Final Speed (mph)	Acceleration Length Required for PC/Trucks (ft)	Required L₂ for Passenger Cars (ft)	Required L₂ for Trucks (ft)	Ramp	Queue Storage Length (ft)	Queue Storage (veh)
Alternative 2	0	70	53	1,458/2,333	497	1,372	Griffin	-162	-13
							Sunrise	328	26
							Davie	-272	-21
Alternative 3 and 4	35	70	53	1,458/1,771	497	810	Griffin	400	32
							Sunrise	890	71
							Davie	290	35
Alternative 5	0	70	53	1,458	497	-	Griffin	713	57
							Sunrise	1,203	96
							Davie	603	72

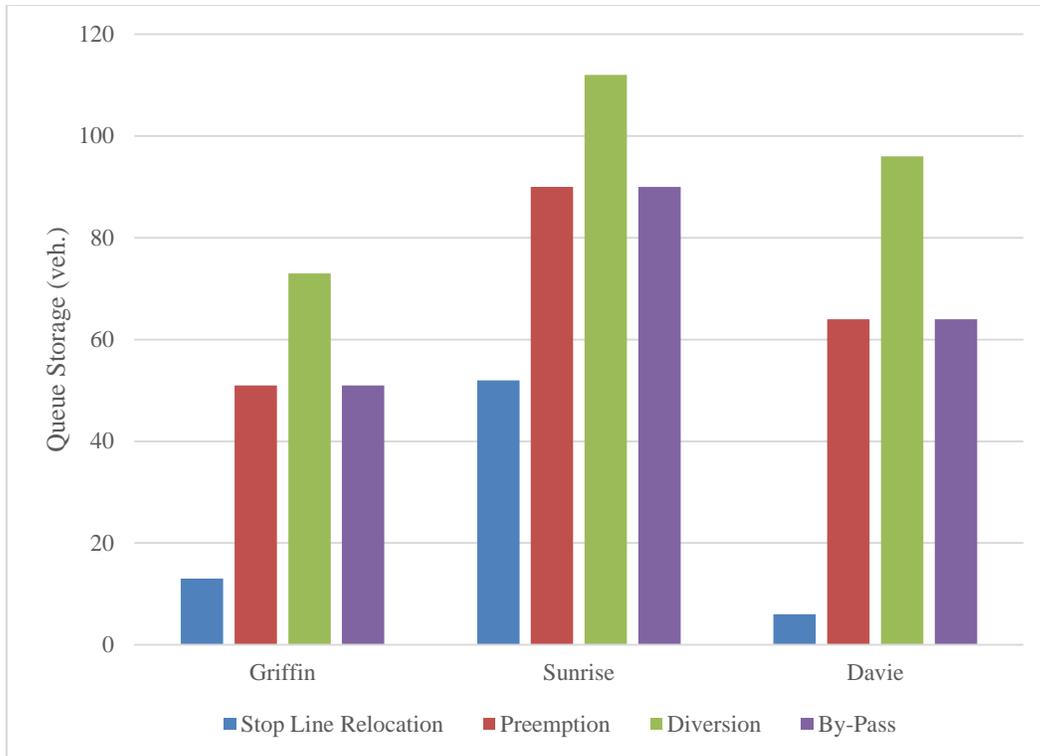


Figure 7-4 Remaining Queue Storage for Different Alternatives to Accommodate Truck Acceleration Length Requirements Utilizing the Average Values of Aggressive and Conservative Designs

Table 7-7 Percentage of Days with Specific Number of Ramps to Meter for Aggressive Design

Number of Metered Ramps	Alternative 1		Alternative 2		Alternative 3		Alternative 4-a		Alternative 4-b		Alternative 5	
	Morning	Afternoon	Morning	Afternoon	Morning	Afternoon	Morning	Afternoon	Morning	Afternoon	Morning	Afternoon
1	19.2	7.1	17.1	10.0	16.4	10.0	15.6	14.1	15.6	14.1	17.1	10.0
2	21.2	6.9	31.4	16.7	8.8	16.7	8.4	9.6	13.0	15.6	31.4	17.8
3	13.5	9.9	14.3	8.9	8.8	8.9	5.8	8.9	5.2	7.4	22.9	16.7
4	4.8	2.5	8.6	8.9	5.0	8.9	3.9	4.4	13.0	8.9	11.4	16.7
5	5.8	3.8	11.4	16.7	12.3	16.7	20.1	14.1	14.3	12.6	2.9	15.6
6	6.7	5.9	2.9	11.2	7.9	11.2	7.1	6.6	9.1	10.4	0.0	8.9
7	28.8	63.8	0.0	4.4	2.9	4.4	5.2	4.4	3.2	5.2	2.9	3.3
8	0.0	0.0	0.0	7.8	2.6	7.8	1.9	4.4	3.9	3.0	2.9	1.1
9	0.0	0.0	0.0	1.1	2.6	1.1	4.5	5.2	1.3	5.2	8.6	10.0
10	0.0	0.0	14.3	14.4	32.6	14.4	27.3	28.1	21.4	17.8	0.0	0.0

Table 7-8 Percentage of Days with Specific Number of Ramps to Meter for Conservative Design

Number of Metered Ramps	Alternative 1		Alternative 2		Alternative 3		Alternative 4-a		Alternative 4-b		Alternative 5	
	Morning	Afternoon	Morning	Afternoon	Morning	Afternoon	Morning	Afternoon	Morning	Afternoon	Morning	Afternoon
1	3.8	4.0	3.8	4.0	7.0	6.7	9.7	8.1	9.7	8.1	11.4	6.7
2	4.7	5.0	8.0	6.1	12.9	14.4	14.3	14.8	14.3	14.8	17.1	6.7
3	14.1	5.0	5.6	4.3	5.9	6.7	4.5	6.7	4.5	6.7	20.0	14.4
4	2.7	9.9	5.0	4.0	4.1	7.8	3.2	5.2	3.2	5.2	22.9	17.8
5	3.3	3.4	10.8	8.6	12.9	20.0	15.6	12.6	15.6	12.6	2.9	7.8
6	4.8	4.4	4.7	4.9	5.8	11.1	11.7	11.7	11.7	6.7	8.6	7.7
7	66.6	2.5	2.7	2.5	3.5	3.3	1.9	3.7	1.9	3.7	2.9	5.6
8	0.0	1.5	4.8	1.5	2.9	3.3	2.6	3.0	2.6	3.0	0.0	3.3
9	0.0	64.3	2.9	1.9	2.9	4.4	3.9	4.4	3.9	4.4	14.3	30.0
10	0.0	0.0	51.8	62.3	41.9	22.2	32.5	34.8	32.5	34.8	0.0	0.0

Tables 7-9 and 7-10 present the results of maximum queue length for each alternative corresponding to the 50th, 85th and 95th percentiles of the queue lengths. As expected, the conservative design leads to longer queues on the ramps and also results in formation of the queue on a higher number of ramps. For instance, in the case of Alternative 2, the aggressive design results in queue formation on four ramps for the PM period, when considering the 95th percentile condition. The conservative design results in queue formation on all considered 10 ramps for the same time period. Moreover, the length of the queue on each ramp is higher when the conservative design is applied. This is simply because the conservative design requires longer acceleration lengths which lead to shorter queue storage lengths and more number of ramps to be metered. As shown in Tables 7-9 and 7-10, Alternative 5 leads to smaller queue lengths for both aggressive and conservative designs. However, this option may not be feasible since it may

impact adversely the operations on the arterial streets, which has not been assessed in this study. The next a best alternative is the provision of an exclusive bypass lane, particularly if an additional lane can be added rather than taking a lane from the general traffic on the on-ramp.

Table 7-9 Cumulative Probability of Maximum Queue Length (Feet) for Aggressive Design

Event	Time	percentile	Oakland	Sunrise	Broward	Davie	Griffin	Stirling	Sheridan	Hollywood	Pembroke	Hallandale Beach
Alternative 1	AM	50	0	0	0	0	0	0	0	0	0	0
		85	0	0	0	0	0	0	0	0	0	0
		95	2,275	0	1,100	0	0	0	0	0	0	0
	PM	50	1,900	0	1,100	0	0	650	0	0	0	0
		85	2,275	0	1,100	0	0	1,100	1,275	1,250	1,200	1,000
		95	2,275	0	1,100	0	0	1,100	1,275	1,250	1,200	1,000
Alternative 2	AM	50	0	0	0	0	0	0	0	0	0	0
		85	0	0	0	0	0	0	0	0	0	0
		95	0	0	0	0	0	0	0	0	0	0
	PM	50	0	0	0	0	0	0	0	0	0	0
		85	0	0	0	0	0	0	0	0	0	0
		95	1,725	1,925	1,000	375	0	0	0	0	0	0
Alternative 3	AM	50	0	0	0	0	0	0	0	0	0	0
		85	2,275	1,100	1,100	375	825	400	0	0	0	0
		95	2,275	1,500	1,100	375	950	1,100	1,275	1,250	1,200	1,000
	PM	50	0	0	0	0	0	0	0	0	0	0
		85	0	0	0	0	0	0	0	0	0	0
		95	2,050	1,500	1,100	250	0	0	0	0	0	0
Alternative 4-a	AM	50	0	0	0	0	0	0	0	0	0	0
		85	500	0	0	0	0	0	0	0	0	0
		95	2,200	1,375	1,000	500	900	725	1,275	825	0	0
	PM	50	0	0	0	0	0	0	0	0	0	0
		85	0	0	0	0	0	0	0	0	0	0
		95	1,975	1,375	1,100	500	900	875	925	875	0	0
Alternative 4-b	AM	50	0	0	0	0	0	0	0	0	0	0
		85	500	0	0	0	0	0	0	0	0	0
		95	2,275	1,600	1,075	775	1,625	375	0	0	0	0
	PM	50	0	0	0	0	0	0	0	0	0	0
		85	0	0	0	0	0	0	0	0	0	0
		95	2,050	2,125	1,100	775	1,775	0	0	0	0	0
Alternative 5	AM	50	0	0	0	0	0	0	0	0	0	0
		85	0	0	0	0	0	0	0	0	0	0
		95	0	0	0	0	0	0	0	0	0	0
	PM	50	0	0	0	0	0	0	0	0	0	0
		85	0	0	0	0	0	0	0	0	0	0
		95	2,050	0	1,100	475	0	0	0	0	0	0

Table 7-10 Cumulative Probability of Maximum Queue Length (Feet) for Conservative Design

Event	Time	percentile	Oakland	Sunrise	Broward	Davie	Griffin	Stirling	Sheridan	Hollywood	Pembroke	Hallandale Beach	
Alternative 1	AM	50	1,275	0	700	0	0	525	875	700	800	600	
		85	1,275	0	700	0	0	700	875	850	800	600	
		95	1,275	0	700	0	0	700	875	850	800	600	
	PM	50	1,275	0	700	0	0	0	0	0	0	0	0
		85	1,275	0	700	0	0	700	875	850	800	600	600
		95	1,275	0	700	0	0	700	875	850	800	600	600
Alternative 2	AM	50	1,275	950	700	375	925	0	0	0	0	0	
		85	1,275	1,175	700	375	950	700	875	850	800	600	
		95	1,275	1,175	700	375	950	700	875	850	800	600	
	PM	50	1,075	1,200	700	375	0	0	0	0	0	0	0
		85	1,275	1,675	700	375	950	700	875	850	800	600	600
		95	1,275	1,675	700	375	950	700	875	850	800	600	600
Alternative 3	AM	50	0	0	0	0	0	0	0	0	0	0	
		85	1,275	1,025	700	275	800	500	875	625	725	0	
		95	1,275	1,125	700	275	800	700	875	850	800	600	
	PM	50	0	0	0	0	0	0	0	0	0	0	0
		85	0	0	0	0	0	0	0	0	0	0	0
		95	1,275	1,300	700	225	0	0	0	0	0	0	0
Alternative 4-a	AM	50	0	0	0	0	0	0	0	0	0	0	
		85	500	0	0	0	0	0	0	0	0	0	
		95	1,275	1,375	700	525	1,275	525	875	725	800	550	
	PM	50	0	0	0	0	0	0	0	0	0	0	0
		85	0	0	0	0	0	0	0	0	0	0	0
		95	1,275	1,925	700	525	1,275	700	775	750	700	0	0
Alternative 4-b	AM	50	0	0	0	0	0	0	0	0	0	0	
		85	500	0	0	0	0	0	0	0	0	0	
		95	1,275	1,375	700	525	1,275	525	875	725	800	550	
	PM	50	0	0	0	0	0	0	0	0	0	0	0
		85	0	0	0	0	0	0	0	0	0	0	0
		95	1,275	1,925	700	525	1,275	700	775	750	700	0	0
Alternative 5	AM	50	0	0	0	0	0	0	0	0	0	0	
		85	0	0	0	0	0	0	0	0	0	0	
		95	0	0	0	0	0	0	0	0	0	0	
	PM	50	0	0	0	0	0	0	0	0	0	0	0
		85	0	0	0	0	0	0	0	0	0	0	0
		95	1,275	0	700	375	0	0	0	0	0	0	0

8. REAL-TIME RAMP SELECTION FOR ACTIVATION

8.1. Methodology for Real-Time Selection of Ramps for Activation

The NCHRP Report 3-87 recommends considering the probability of breakdown as a measure to activate the ramp meters or be incorporated into the metering algorithms to calculate the metering rate (Elefteriadou et al., 2009). The first step is to develop a breakdown probability model. This model is supposed to estimate the probability of breakdown occurrence for any specific traffic flow (either the total freeway mainline flow or occupancy downstream of the merge or the main line flow upstream of the merge and the on-ramp, separately). The model is developed based on an analysis approach referred to as lifetime data analysis statistics. This approach is usually used to estimate the time until failure for mechanical parts. In traffic studies, it can provide the probability that a particular flow will not lead to breakdown. The Product Limit Method (PLM) was used to achieve the distribution of traffic flow that causes breakdown. Figure 8-1 shows an example of the probability of breakdown derived in the NCHRP Report 3-87 for a four lane bottleneck at the I-95 NB ramp from NW 103rd St. in Miami-Dade County. The PLM method is the same approach used in this study to estimate the stochastic distribution of capacity, as described in Chapter 4

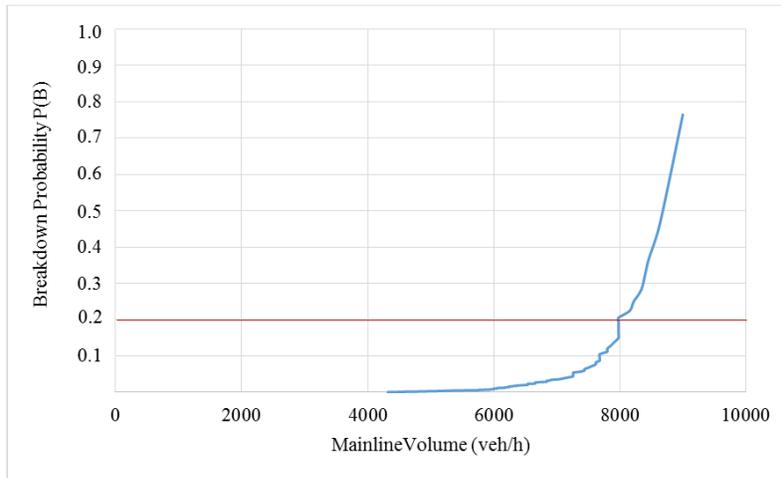


Figure 8-1 Breakdown Probability Model for I-95 NB Ramp from NW 103rd St. (Elefteriadou et al., 2009)

The NCHRP Report 3-87 proposed 20% probability of breakdown as the threshold, i.e. once the upstream flow reaches a value which corresponds to 20% or more probability of breakdown, the ramp metering is recommended to be activated. This method is quite simple and only requires the immediate upstream and on-ramp volumes or immediate downstream volumes or occupancy as the inputs. However, it does not allow the determination of how many ramps and which ramps need to be metered and only determines when the ramps need to be activated. Moreover, this

method is only applicable to recurrent traffic conditions and does not support the activation for non-recurrent conditions.

The PLM method referenced in the above discussion was used in this project to derive the capacity distribution, as mentioned above. The maximum flow before breakdown occurrence was interpreted as the capacity. Therefore the probability of breakdown for the bottleneck can be estimated based on the stochastic distribution of capacity considering the estimated demands.

The proposed methodology for off-line selection of the ramps to meter in Chapter 6 can be extended to real-time activation of ramp signals. In addition to the stochastic capacity distribution, there is a need to predict the demand. For this purpose, a simplified prediction model is used by deriving factors derived based on the historical demand data for 15-minutes intervals for the prediction of the demands on freeway mainline for the next 15-minutes. For the purpose of this research, the prediction is based on the ratio of the demand in the next 15-minutes over the instantaneous demand. This ratio is derived from historical demand distributions for the morning and afternoon peak periods. The 80th percentile demand for each 15-minutes is used for the analysis. More complex travel demand prediction methods can also be used to provide the input to the proposed ramp selection methodology.

The ramp selection formulation described in Chapter 6 can be run in real-time to identify the needs for activation and the ramps to activate using the stochastic capacity and the predicted demands. The constraints of the formulation may be updated based on the preference of the operating agency on the storage utilizations. This method can provide the ramp metering activation for the ramps in addition to the total number of the ramps to be metered. The capacity input to the formulation is assumed to be the 20th percentile capacity. Since, real-time activation is based on the forecasted demand value for each 15-minutes, Monte Carlo simulation is not required.

Another approach to use the outcome of the proposed formulation of this research is to provide a Look-Up Table derived off-line that relates the number of the ramps to be metered to the predicted D/C values at the activation decision stage, rather than running the ramp metering formulation in real time. Therefore, by predicting the demand value for the next 15 minutes and calculating the forecasted D/C ratio value for the next 15 minutes, the specific ramps to be metered can be determined, as well as the time of their activation.

The developed distributions for demand prediction and Look-Up Table as well as the comparison results of these three approaches (probability of breakdown, using the formulation, and using the Look-Up Table) are presented below.

8.2. Application of Real-Time Ramp Selection

The historical demand distributions for the morning and afternoon peak periods are illustrated previously in Figure 4-9 and Figure 4-10, respectively. The demand values are presented for different percentiles for each 15-minute interval. These distributions were used to derive factors used as a part of a simple prediction of the next 15-minutes demands. As discussed above, the factors are derived based on the 80th percentile demands and used as the basis for the next 15-minutes demand prediction in this research.

Using the historical demand distributions, three methods for selection of the ramps for real-time application are applied to randomly selected days with different conditions. For each condition (recurrent and non-recurrent), two random days are selected from the year 2015 and real-world data for these days are used to apply the aforementioned methods. The probability of breakdown method is only applicable to the recurrent conditions and event-free days. Using the capacity distribution derived earlier, the probability of breakdown of 20% and more corresponds to the flow of 2,203 vphpl and more. Therefore, the breakdown method corresponds to the method developed in this study when the activation is done based on predicting demands higher than this value. However, the breakdown method does not allow determining the number of ramps, which are required to be metered, in order to prevent the breakdown.

In order to extend the proposed methodology for real-time activation, the 20th percentile capacity is used as the capacity value in order to be comparable to the probability of breakdown method. Table 8-1 shows the Look-Up Table derived from the analysis of the results of the off-line selection of ramps to meter for different D/C ratios. This table provides a correspondence between each D/C ratio for each condition (recurrent, incident with different lane blockages, and weather with different rain intensities) to the 50th and 80th percentiles of the total number of ramps to meter. The D/C values for the non-recurrent conditions are the values before the event occurrence. For instance, if the predicted D/C value for the next 15 minutes is 0.93 according to recurrent conditions, none of the ramps will be required for the metering. However, once a shoulder incident is reported with the same predicted D/C value (0.93), 7 to 10 ramps are required to be metered depending on the peak period and agency willingness to take risk (e.g., the 80% percentile eliminates breakdown for 80% of the cases for the specific scenario under consideration).

Table 8-1 Look-Up Table for Real-Time Activation of the Ramps

Condition	Time	Percentile	D/C Values				
			0.7-0.8	0.8-0.9	0.9-1.0	1.0-1.1	> 1.1
Recurrent Condition	AM	50	0	0	0	5	10
		80	0	0	0	7	10
	PM	50	0	0	0	5	10
		80	0	0	0	7	10
Medium Rain	AM	50	0	2	7	10	10
		80	0	2	9	10	10
	PM	50	0	2	7	10	10
		80	0	2	10	10	10
Heavy Rain	AM	50	6	8	10	10	10
		80	8	9	10	10	10
	PM	50	6	8	10	10	10
		80	8	9	10	10	10
Shoulder Blocked Incident	AM	50	0	2	7	10	10
		80	0	3	9	10	10
	PM	50	0	2	8	10	10
		80	0	3	10	10	10
One Lane Blocked Incident	AM	50	9	10	10	10	10
		80	10	10	10	10	10
	PM	50	6	10	10	10	10
		80	8	10	10	10	10
Two Lanes Blocked Incident	AM	50	10	10	10	10	10
		80	10	10	10	10	10
	PM	50	10	10	10	10	10

Tables 8-2 and 8-3 present the results for the real-time selection of the ramps to meter for the morning and afternoon periods, respectively, for the days selected as case studies. The shaded cells represent the intervals with an event. For instance, in Table 8-2, the medium rain for Day 1 occurred from 7:30 AM to 8:15 AM. In Table 8-2 and 8-3, the “P(B)” represents the Probability of Breakdown method, the “50th Percentile” and “80th Percentile” refer to the Look-Up Table method with these two percentiles, as shown in Table 8-1. “Programming” refers to the extension of methodology of off-line selection of the ramps to meter, proposed in this research, to real-time activation.

Table 8-2 Real-Time Activation Results for the Morning

Condition		Ramp Selection Method	6:30	6:45	7:00	7:15	7:30	7:45	8:00	8:15	8:30	8:45	9:00	9:15	9:30	9:45		
			✓	✓	✓	✓	✓	✓	✓	✓	✓	0	0	0	0	0	0	0
Recurrent Condition	Day 1	P(B)	✓	✓	✓	✓	✓	✓	✓	✓	0	0	0	0	0	0	0	
		50 th percentile	5	5	5	5	5	5	5	5	5	0	0	0	0	0	0	0
		80 th percentile	7	7	7	7	7	7	7	7	7	0	0	0	0	0	0	0
		Programming	3	9	8	8	9	6	4	5	0	1	1	2	1	0	0	0
	Day2	P(B)	0	✓	✓	✓	✓	0	0	0	0	0	0	0	0	0	0	0
		50 th percentile	0	5	5	5	5	0	0	0	0	0	0	0	0	0	0	0
		80 th percentile	0	7	7	7	7	0	0	0	0	0	0	0	0	0	0	0
		Programming	0	4	2	2	5	0	0	0	0	0	0	0	0	0	0	0
Medium Rain	Day 1	50 th percentile	0	0	0	0	10	7	7	7	0	0	0	0	0	0	0	
		80 th percentile	0	0	0	0	10	9	9	9	0	0	0	0	0	0	0	
		Programming	0	0	0	0	10	8	5	5	0	0	0	0	0	0	0	
	Day2	50 th percentile	0	5	7	7	10	7	7	0	0	0	0	0	0	0	0	
		80 th percentile		7	9	9	10	9	9	0	0	0	0	0	0	0	0	
		Programming	0	2	9	9	10	8	5	0	0	0	0	0	0	0	0	
Heavy Rain	Day 1	50 th percentile	0	0	0	10	10	10	10	10	8	10	0	0	0	0	0	
		80 th percentile	0	0	0	10	10	10	10	10	9	10	0	0	0	0	0	
		Programming	0	0	0	10	10	10	10	10	7	9	0	0	0	0	0	
	Day2	50 th percentile	0	5	0	0	5	0	0	10	8	10	11	0	0	0	0	
		80 th percentile	0	7	0	0	7	0	0	10	9	10	10	0	0	0	0	
		Programming	0	1	0	0	2	0	0	10	8	9	9	0	0	0	0	
Shoulder Blocked Incident	Day 1	50 th percentile	0	0	0	7	7	2	2	0	0	0	0	0	0	0		
		80 th percentile	0	0	0	9	9	3	3	0	0	0	0	0	0	0		
		Programming	0	0	0	6	8	4	1	0	0	0	0	0	0	0		
	Day2	50 th percentile	0	0	0	7	10	7	0	0	0	0	0	0	0	0		
		80 th percentile	0	0	0	9	10	9	0	0	0	0	0	0	0	0		
		Programming	0	0	0	10	10	9	0	0	0	0	0	0	0	0		
One Lane Blocked Incident	Day 1	50 th percentile	0	0	0	10	10	10	10	10	10	5	5	0	0	0		
		80 th percentile	0	0	0	10	10	10	10	10	10	7	7	0	0	0		
		Programming	0	0	0	10	10	10	10	10	10	3	3	0	0	0		
	Day2	50 th percentile	0	0	0	0	0	0	0	0	10	10	10	10	10	10		
		80 th percentile	0	0	0	0	0	0	0	0	10	10	10	10	10	10		
		Programming	0	0	0	0	0	0	0	0	10	10	10	10	10	10		

Note: the “P(B)” represents the Probability of Breakdown method, the “50th Percentile” and “80th Percentile” refer to the Look-Up Table method with these two percentiles, as shown in Table 8-1. “Programming” refers to the extension of methodology of offline selection of the ramps to meter, proposed in this research, to real-time activation.

Table 8-3 Real-Time Activation Results for the Afternoon

Condition	Ramp Selection Method	14:00	14:15	14:30	14:45	15:00	15:15	15:30	15:45	16:00	16:15	16:30	16:45	17:00	17:15	17:30	17:45	18:00	18:15	18:30	18:45		
Recurrent Condition	Day 1	P(B)	0	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	0	0	
		50 th percentile	0	5	5	5	5	5	10	10	10	5	5	5	5	5	5	5	5	5	5	0	0
		80 th percentile	0	7	7	7	7	7	10	10	10	7	7	7	7	7	7	7	7	7	7	0	0
	Day 2	Programming	0	2	5	6	6	6	10	10	9	10	4	5	3	6	5	5	2	4	0	0	
		P(B)	0	0	✓	✓	✓	✓	✓	✓	✓	✓	0	✓	0	✓	0	0	0	0	0	0	0
		50 th percentile	0	0	5	5	5	10	10	5	5	5	0	5	0	5	0	0	0	0	0	0	0
	Medium Rain	Day 1	80 th percentile	0	0	0	7	7	7	10	10	10	10	10	10	0	0	0	0	0	0	0	0
			Programming	0	0	0	1	0	1	9	9	10	10	8	9	0	0	0	0	0	0	0	0
			50 th percentile	0	0	0	0	0	0	5	5	10	10	7	7	0	0	0	0	0	0	0	0
Day 2		80 th percentile	0	0	0	0	0	0	7	7	10	10	10	10	0	0	0	0	0	0	0	0	
		Programming	0	0	0	0	0	0	6	6	10	10	5	7	0	0	0	0	0	0	0	0	
		50 th percentile	0	0	10	10	10	10	10	10	5	0	0	0	0	0	0	0	0	0	0	0	
Heavy Rain		Day 1	80 th percentile	0	0	10	10	10	10	10	10	7	0	0	0	0	0	0	0	0	0	0	0
			Programming	0	0	10	10	10	10	10	10	3	0	0	0	0	0	0	0	0	0	0	0
			50 th percentile	0	0	0	0	0	0	5	5	5	5	0	0	0	0	0	10	10	10	10	5
	Day 2	80 th percentile	0	0	0	0	0	0	7	7	7	7	0	0	0	0	0	10	10	10	10	7	
		Programming	0	0	0	0	0	0	5	5	1	1	0	0	0	0	0	10	9	10	9	2	
		50 th percentile	0	0	5	5	5	5	10	10	10	10	5	5	5	5	5	5	0	0	0	0	
	Shoulder Blocked Incident	Day 1	80 th percentile	0	0	7	7	7	7	10	10	10	10	5	7	7	7	7	7	0	0	0	0
			Programming	0	0	1	1	1	1	9	10	10	10	5	5	5	3	3	3	0	0	0	0
			50 th percentile	0	5	5	5	5	5	10	10	10	10	5	5	5	5	5	5	10	10	10	10
Day 2		80 th percentile	0	7	7	7	7	7	10	10	10	10	7	7	7	7	7	7	10	10	10	10	
		Programming	0	1	4	5	4	5	10	10	8	8	2	4	1	4	3	3	10	10	10	10	
		50 th percentile	0	0	0	0	0	0	0	0	0	0	0	0	10	10	10	10	10	10	5	0	
One Lane Blocked Incident		Day 1	80 th percentile	0	0	0	0	0	0	0	0	0	0	0	10	10	10	10	10	10	10	7	0
			Programming	0	0	0	0	0	0	0	0	0	0	0	0	10	10	10	10	10	9	5	0
			50 th percentile	0	10	10	10	10	10	10	5	5	5	0	0	0	0	0	0	0	0	0	0
	Day 2	80 th percentile	0	10	10	10	10	10	10	7	7	7	0	0	0	0	0	0	0	0	0	0	
		Programming	0	10	10	10	10	10	9	5	5	3	0	0	0	0	0	0	0	0	0	0	
		50 th percentile	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	

Note: the “P(B)” represents the Probability of Breakdown method, the “50th Percentile” and “80th Percentile” refer to the Look-Up Table method with these two percentiles, as shown in Table 8-1. “Programming” refers to the extension of methodology of offline selection of the ramps to meter, proposed in this research, to real-time activation.

According to the results shown in Tables 8-2 and 8-3, the three methodologies lead to the same activation time. Therefore, the activation time results of the proposed methodology is compatible with the results of the probability of breakdown method. However, the probability of breakdown method does not deal with non-recurrent conditions and the number of ramps to activate.

As seen in these two tables, the results of the Programming formulation utilization in real-time (the extension of the off-line selection of the ramps) led to better adaption of the number of the ramps to meter to the D/C value of each case, compared to the Look-Up Table. This is due to the

fact that in the case of the Look-Up Table method, the D/C values are categorized in groups, with each group covering a range of D/C values, while the Programming Formulation utilizes the exact value of the D/C value for the selection of the ramps. Therefore, it is logical that the Programming Formulation is more adaptive in its selection of the ramps to meter based on the ramps to meter. It is interesting to see that the results of the Programming Formulation are closer to the results of using the 50th percentile in the Look-Up Table compared to using the 80th percentile in the Look-Up Table. The 80th percentile results of the Look-Up Table are more conservative compared to the 50th percentile and programming formulation which is reasonable. Thus, if an agency decides to use the Look-Up Table method because of the simplicity of utilizing the Look-Up Table, the 50th percentile values should be used.

9. SAFETY CONSIDERATION IN SELECTING RAMPS FOR METERING

9.1. Introduction

This section describes an investigation made in this study of models that can be used to determine the impact of ramp metering on the safety of the metered segment and potentially for use in the activation of ramp metering. As reviewed in Chapter 3, warrants have been developed by transportation agencies to support the decision to implement ramp metering including local safety warrants that consider the safety in the merge area of a ramp that is being considered for metering. Models are identified in this section to potentially allow the estimation of increased safety due to ramp metering.

9.2. Existing Crash Prediction Models

Increasing traffic demand and congestion is expected to have a significant impact on safety. Safety can be improved by reducing traffic congestion and providing smooth traffic flow. Chang et al. (2000) conducted regression analysis to relate hourly crash rate to hourly volume/capacity (v/c) ratio for different types of freeway sections including the basic freeway, tunnel section, and toll gate section. Figure 9-1 reveals a bell shape variation of crash rate with respect to the v/c ratio for basic freeway segments based on the Chang et al. (2000). The accident rate reaches the minimum when v/c ratio is 0.78.

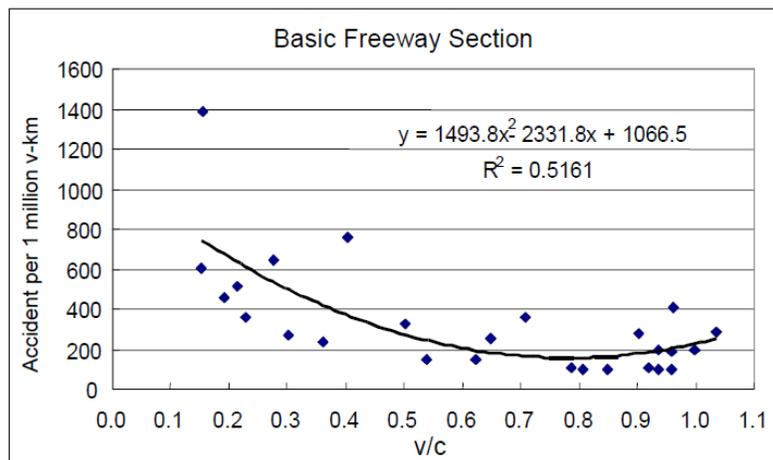


Figure 9-1 Relationship between Hourly V/C Ratio and Crash Rate for Basic Freeway Section (Chang et al., 2000)

Martin (2002) shows that there is a similar trend of crash frequency with traffic hourly volume. Variation of the crash and hourly volumes was found to be extremely close to each other, as shown in Figure 9-2. However, it should be mentioned that Figure 9-2 shows the crash frequency (number of crashes) rather than showing the crash rate top normalize for the volume.

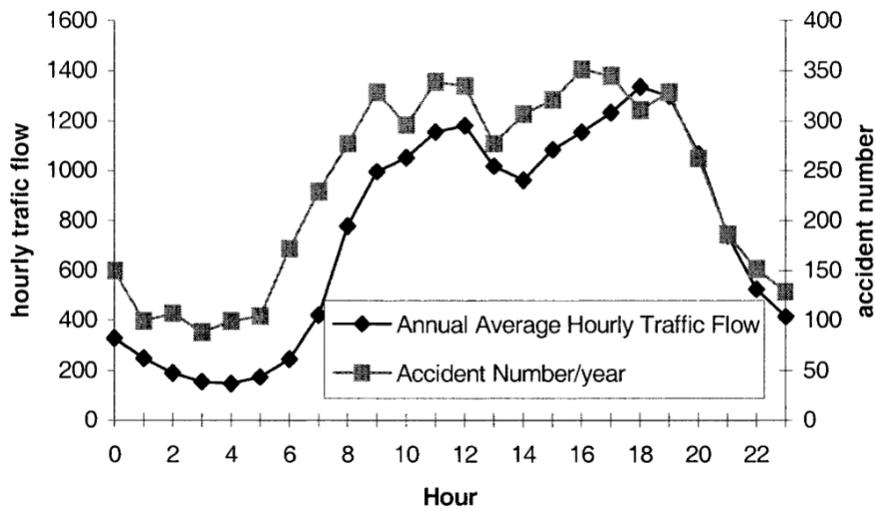


Figure 9-2 Relationship between Hourly Volume and Crash frequency (Martin, 2002)

A more recent study by Kononov et al. (2012) show that an increases in flow and density without a reduction in speed have a significant influence on safety. Kononov et al. (2012) pointed out a critical threshold for combination of density and speed parameters for abrupt change in crash rate.

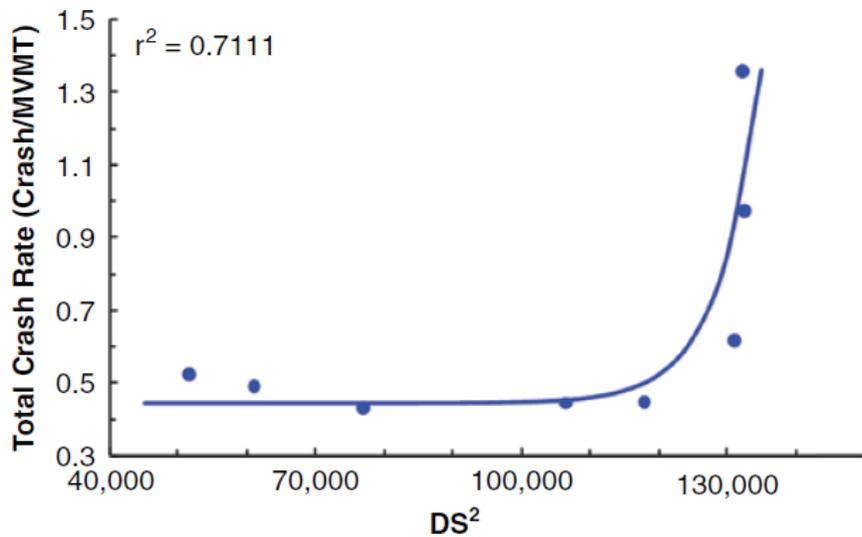


Figure 9-3 DS² vs. Total Crash Rate for Four-Lane Corridor-470 (Kononov et al., 2012) (D: Density, S: Speed)

According to Abdel-Aty et al. (2014), the variation of the percentage of crashes with the 5-minute average vehicular speed shows two mound-shaped distributions as illustrated in Figure 9-4. Based on this, two separate models for crash risk prediction, as shown in Figure 9-5 were developed, the first is for low speed condition and the second for high speed conditions. Note that the symbol Log in Figure 9-5 refers to the logarithm, CVS represents the coefficient of

variation in speed, AO is the average occupancy AV is the average volume, and SV is the standard deviation of volume. For each crash, traffic data from seven stations and six 5-minute intervals were used as potential variables in the developed Abdel-Aty et al. study (2014). The stations were identified as stations B to H with B referring to the farthest upstream station, F referring to the station closest to the crash location, and G and H referred to the downstream stations of the crash. The time intervals were identified from 1 to 6 with 1 representing the time interval between the time crash occurs and 5 minutes before the occurrence of the crash, 2 referring to the time interval between 5 minutes and 10 minutes before the occurrence of the crash, and so on. These station and time interval identifications were used in naming the variables, as shown in Figure 9-2. For example, the variable LogCVSF2 represents the log coefficient of the variation in speed at the detector station that is closest to the crash location in the 5 to 10 minute interval before the occurrence of the crash.

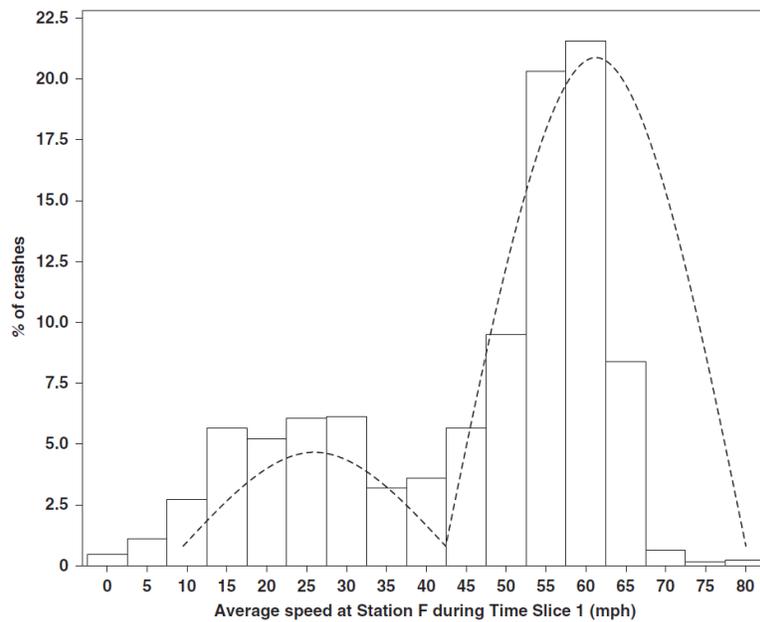


Figure 9-4 5-Minute Average Speed vs. Crash Frequency (Abdel-Aty et al., 2014)

TABLE 1 Final Models Developed by Using Stepwise Selection Procedures: Low-Speed Regime

Variable	DF	Parameter Estimate*	Standard Error	Chi Square	Pr > Chi Square	Hazard Ratio
LogCVSF2	1	2.64827	0.49216	28.9539	<.0001	14.130
LogCVSF3	1	0.88842	0.47859	3.4460	0.0634	2.431
LogAOE2	1	1.33966	0.46218	8.4018	0.0037	3.818
LogAOH3	1	0.97766	0.55210	3.1357	0.0766	2.658
SVF2	1	-0.43603	0.12289	12.5883	0.0004	0.647

*Parameters from Stations E through H examined.

TABLE 2 Final Models Developed by Using Stepwise Selection Procedures: High-Speed Regime

Variable	DF	Parameter Estimate*	Standard Error	Chi Square	Pr > Chi Square	Hazard Ratio
LogAOF2	1	-0.93423	0.39970	5.4632	0.0194	0.393
LogAOH3	1	1.14584	0.34156	11.2541	0.0008	3.145
SVH2	1	-0.22878	0.07181	10.1494	0.0014	0.796
AVG2	1	-0.10055	0.03376	8.8723	0.0029	0.904
AVE3	1	0.05932	0.03419	3.0100	0.0828	1.061

*Parameters from Stations D through H examined.

Figure 9-5 The Crash Prediction Models Developed for Low Speed and High Speed Conditions (Abdel-Aty et al., 2014)

Lee et al. (2016) conducted a safety analysis for three expressways in Central Florida for three levels of data aggregation: daily, hourly, and 5-minute crash and traffic data. Based on the analysis, regression models were developed for each data aggregation level to associate crash measures with contributing factors including traffic and geometry parameters. At the daily level, a Bayesian Poisson-lognormal model was developed to predict the total number of crashes based on annual average daily traffic (AADT). At the hourly level, a Bayesian multilevel Poisson-lognormal model was used to estimate crash frequency based on the average hourly traffic (AHT) and other factors. A Bayesian multilevel logistic regression model was developed for the 5-minute level to estimate the likelihood of crash. Lee et al. (2016) recommended that the AHT-based model is used for long-term traffic safety analysis and the 5-minute-based model for the implementation of active traffic management. The results of Lee et al. (2016) in Figure 9-6 show that the temporal variation of crash frequency is consistent with the traffic volume variation and the crash frequency has two peaks during the peak hours. However, it should be mentioned that the models were developed for two and three lane freeway segments. The case study in this project (I-95 in Broward County) mostly consists of four and five lane segments.

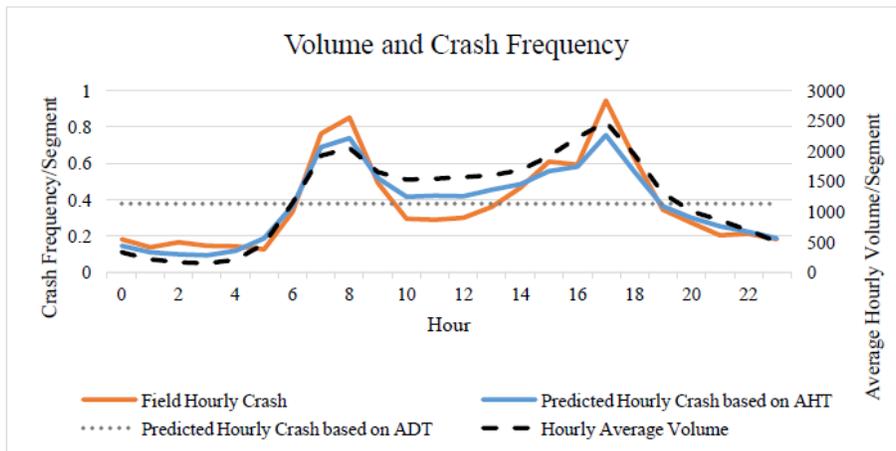


Figure 9-6 Relationship between Hourly Volume/Segment and Crash Frequency/Segment (Lee et al., 2016)

The above literature review reveals that there is correlation between traffic flow parameters and crash frequencies. The two parameters identified for further examination in this study based on the above discussion are the Crash Risk calculated based on the Abdel-Aty et al. (2014) model presented in Figure 9-6 and a multiplication function of density and speed, based on the discussion presented in Kononov et al. (2012).

9.3. Investigation of the Safety Prediction Models

An analysis was conducted in this study to assess the changes in safety resulting from the changes in traffic flow parameters. The purpose is to identify safe and unsafe conditions by using traffic flow parameters as input. As stated in Section 9.2, the investigated parameters include Crash Risk calculated based on the Abdel-Aty et al. (2014) model presented in Figure 9-6 and a multiplication function of density and speed, based on the discussion presented in Kononov et al. (2012).

This analysis was conducted for four segments along the I-95 in Broward County, FL, as shown in Figure 9-7. The merge area of four on-ramps were included in these study segments including the on-ramps from Hallandale Beach Blvd., Hollywood Blvd., Griffin Road and W. Sunrise Blvd. as highlighted in Figure 9-7. Three-year detector data and crash data between January 1, 2013 and December 31, 2015 for the study segments were collected from the RITIS website and Signal Four Analytics website, respectively. It should be noted that both lane by lane and station-level detector data were retrieved at 5-minute intervals. The collected detector data were then sorted and averaged for non-holiday days in the MATLAB software. The crash data were filtered for the ramps related crashes along the I-95 northbound using the ArcGIS geographic information system (GIS) software. Based on the literature review presented in Section 9.2.1 (Kononov et al. 2012), traffic flow parameters including DS^2 (where S is speed), D^2S , and the multiplications of these parameters by ramp volume (VR) were calculated and related to the crash rate using the cleaned data.

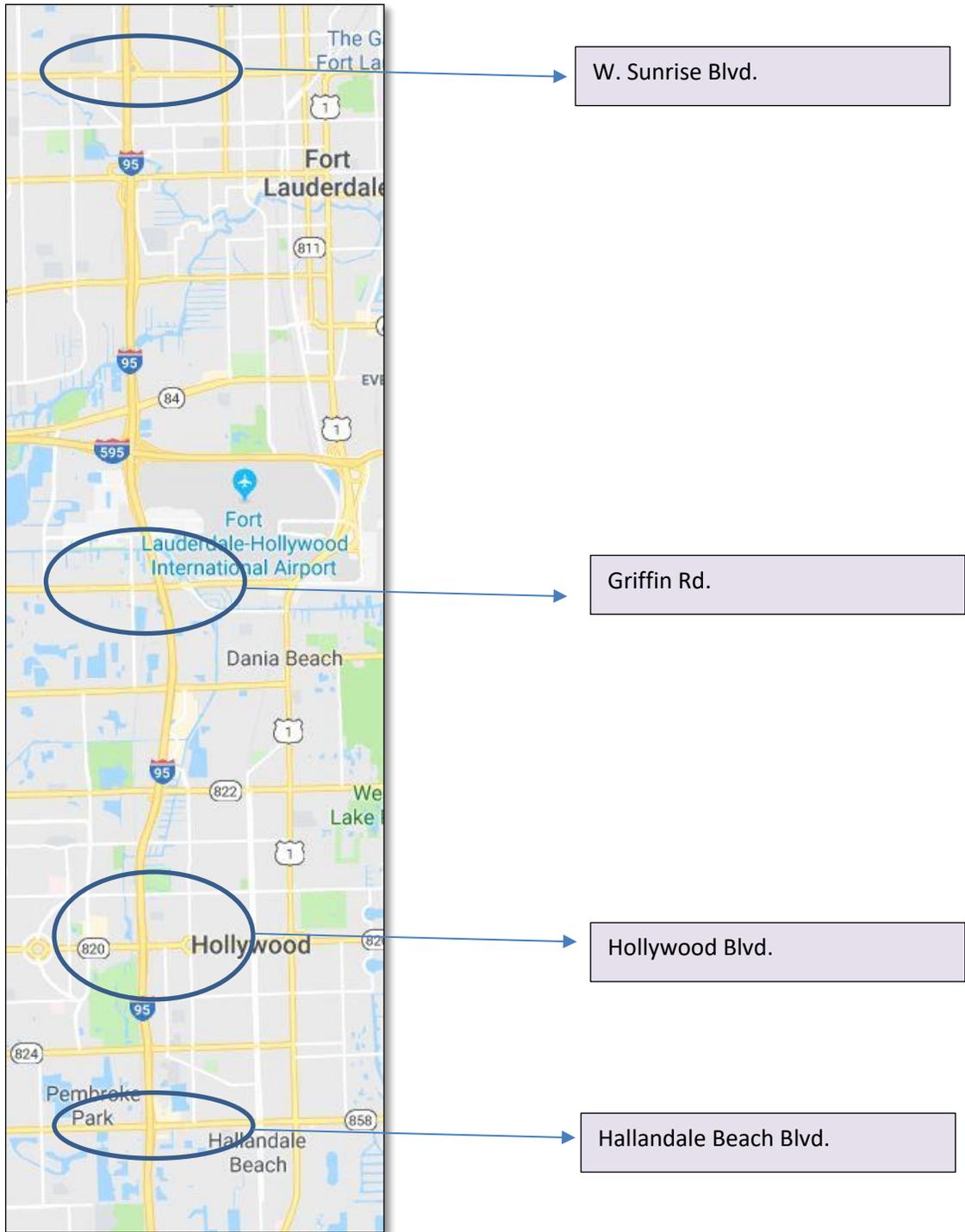


Figure 9-7 Safety Investigation Segments along the I-95

Table 9-1 presents a comparison of the geometry of the four studied on-ramps as roadway condition has a major impact on safety. As shown in this table, the merging areas at Hollywood Blvd. and Hallandale Blvd. have four general-purpose lanes, two managed lanes, and one on-ramp lane, while the merging area at Griffin Rd. and W. Sunrise Blvd. have five general-purpose lanes, one managed lane, and two on-ramp lanes that are merged into one.

Table 9-1 Geometry Comparison of Four Studied On-Ramps

Geometric features	Hollywood	Hallandale	Griffin	Sunrise
Number of GP Lanes	4	4	5	5
Number of Managed lanes	2	2	1	1
Number of On-ramp lanes	1	1	2 merged to 1	2 merged to 1
Average Width of Lane	10.5 ft.	10.5 ft.	11 ft.	11.5 ft.
Shoulder	Right side of GP lane and Left side of Managed Lane	Right side of GP lane and Left side of Managed Lane	Right side of GP lane and Left side of Managed Lane	Right side of GP lane and Left side of Managed Lane
Shoulder Width	4.5 ft.	4.5 ft.	5 ft.	6.6 ft.
Ramp Type	Arterial On-Ramp	Arterial On-Ramp	Arterial On-Ramp	Arterial On-Ramp
Merging Type	Continuous Auxiliary Lane	Continuous Auxiliary Lane	Continuous Auxiliary Lane	Continuous Auxiliary Lane
Grade	Flat Grade	Flat Grade	Flat Grade	Flat Grade
Acceleration Lane Drop	No Drop	No Drop	No Drop	No Drop

Figure 9-8 shows the total crash rate in terms of number of crashes per vehicle mile traveled between 6:00 am and 8:00 pm for the four study segments. The results in this figure show higher number of crash rates at Hallandale Blvd. and Hollywood Blvd. segments than Griffin Rd. and W. Sunrise Blvd. segments. This may be related in part to the smaller number of lanes and narrower lane widths on Hallandale Blvd. and Hollywood Blvd. segments. The two on-ramps at these segments also have higher on-ramp volumes than the Griffin Road and Sunrise Blvd on-ramps, particularly in the morning peak, as discussed in the next paragraph.

Figure 9-9 presents the temporal variation of the crash rate along the four study segments. It is seen for all the study locations that the crash rates during the AM and PM peak periods are high, and that the crash rate is particularly high during the AM peak. There is also a peak of crash rate during the Midday for Hallandale Blvd. and Hollywood Blvd. study segments. Figure 9-10 shows the corresponding variation in the ramp volumes (VR). The two on-ramps from Hallandale Blvd. and Hollywood Blvd also have higher high on-ramp volumes than the Griffin Road on-ramp, particularly in the A M peak, as shown in Figure 9-10.

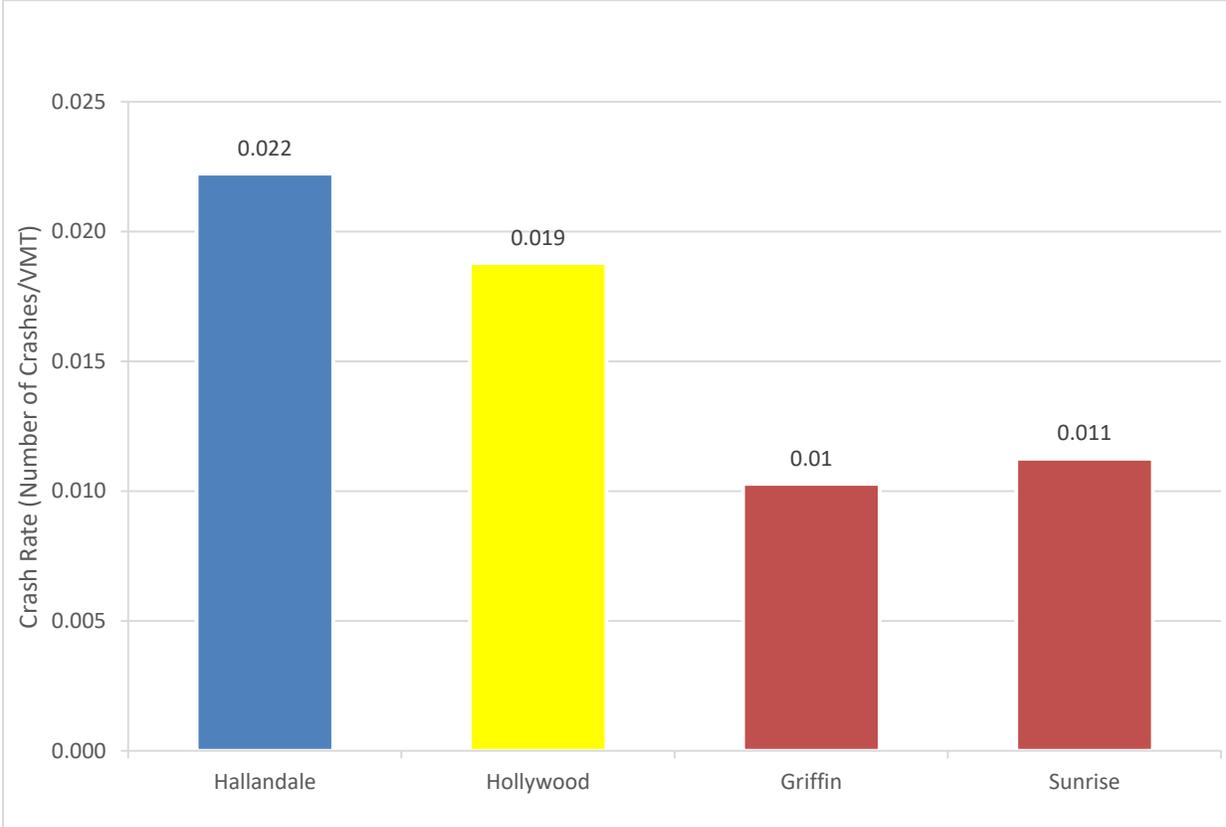


Figure 9-8 Total Crash in Four Study Segments

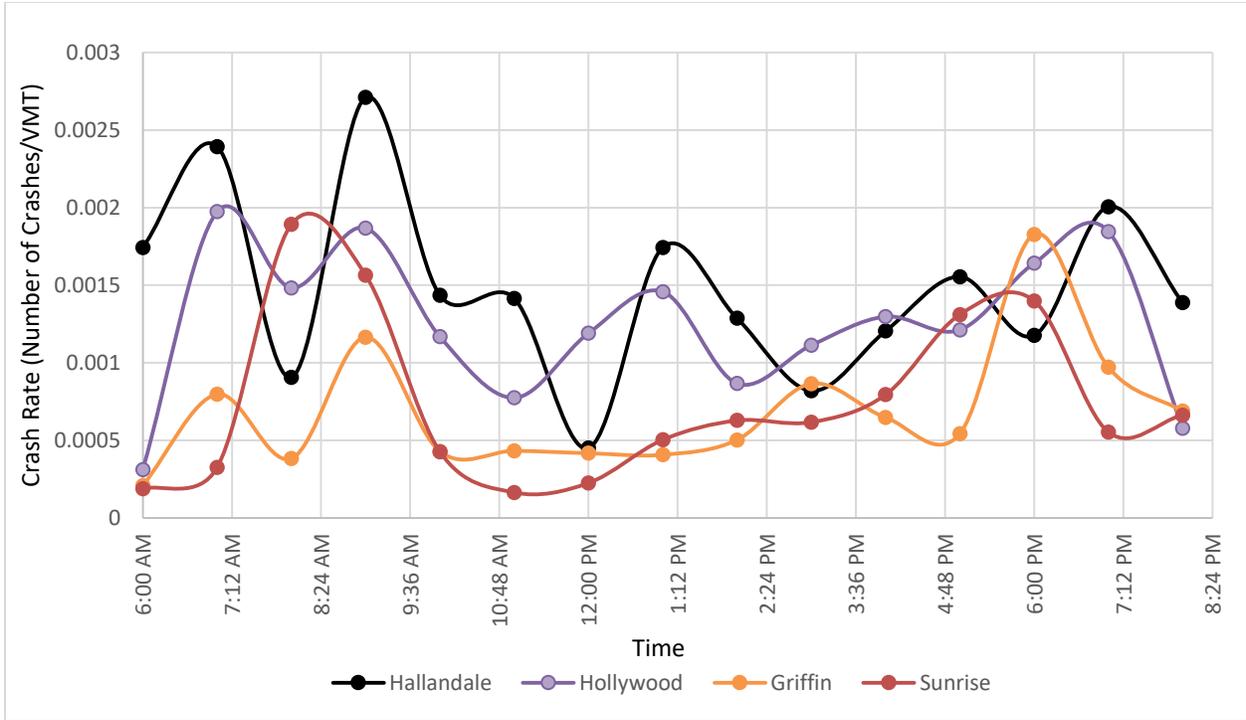


Figure 9-9 Temporal Variation of Crash Rate in All the Study Segments

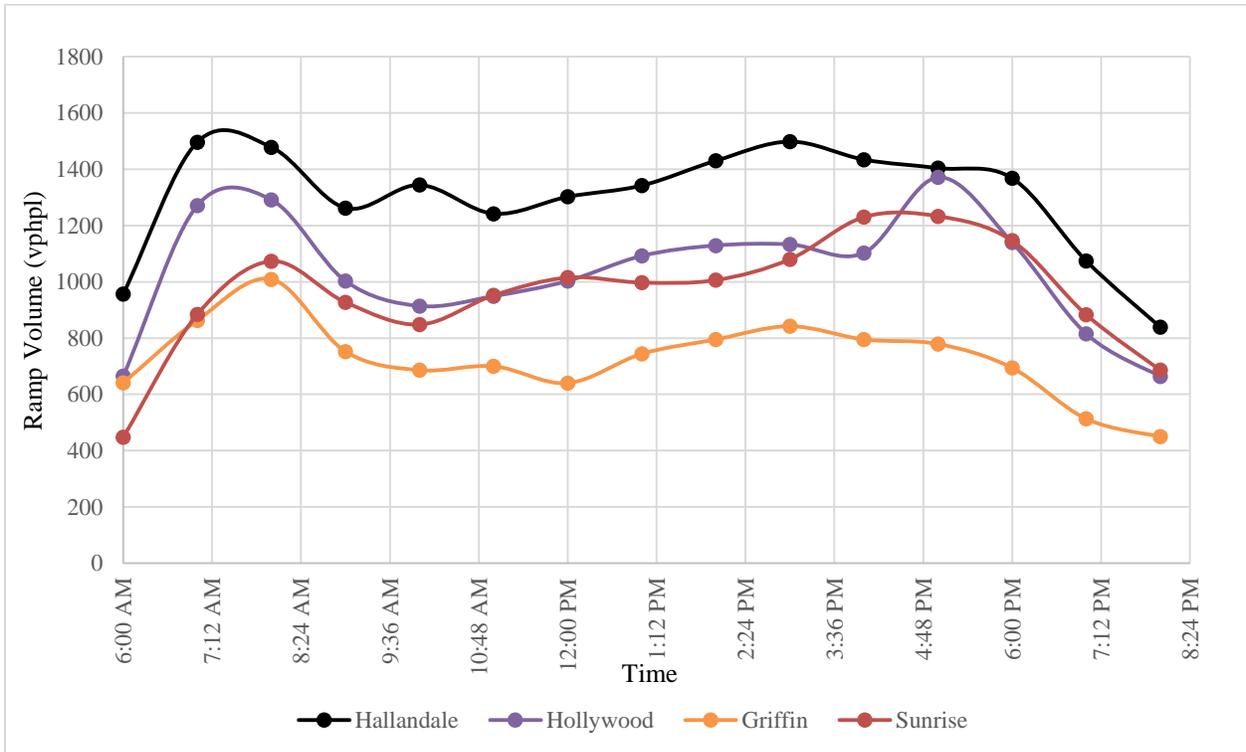


Figure 9-10 Temporal Variation of Ramp Volume in All the Study Segments

Density and speed multiplication parameters were related to the crash rates at the two high crash locations (Hollywood Blvd. and Hallandale Blvd). These parameters include the 90th percentile values of DS^2 , Lane 1- DS^2 (that is, the DS^2 for the rightmost lane), DS^2*VR , D^2S . The parameters and crash rates at 5-minute intervals were aggregated into three-hour value and plotted in Figures 9-11 and 9-12 for these two segments. Note that the value of the DS^2*VR was scaled down by 2000 in order to be shown in the same figure as the other parameters. The DS^2*VR variable shows the most corresponding to the crash rate peaking. Figure 9-13 indicates that the crash risk index for the four segments, calculated based on the models developed by Abdel-Aty et al. (2014). As shown in this figure, the crash risk index peaks at the AM peak period, with particularly high values for the two segments with the high crash rates. Based on the results, it is recommended that the ramp metering is activated when the DS^2*VR variable exceeds 800,000 and when the crash rate index becomes lower than -16. This criteria for activation should be particularly considered for on-ramp merge locations with high crash rates. The Florida local warrants recommend installing ramp metering is warranted at a location where the facility or roadway segment has a crash rate of over 80 crashes per hundred million vehicle miles (HMVM).

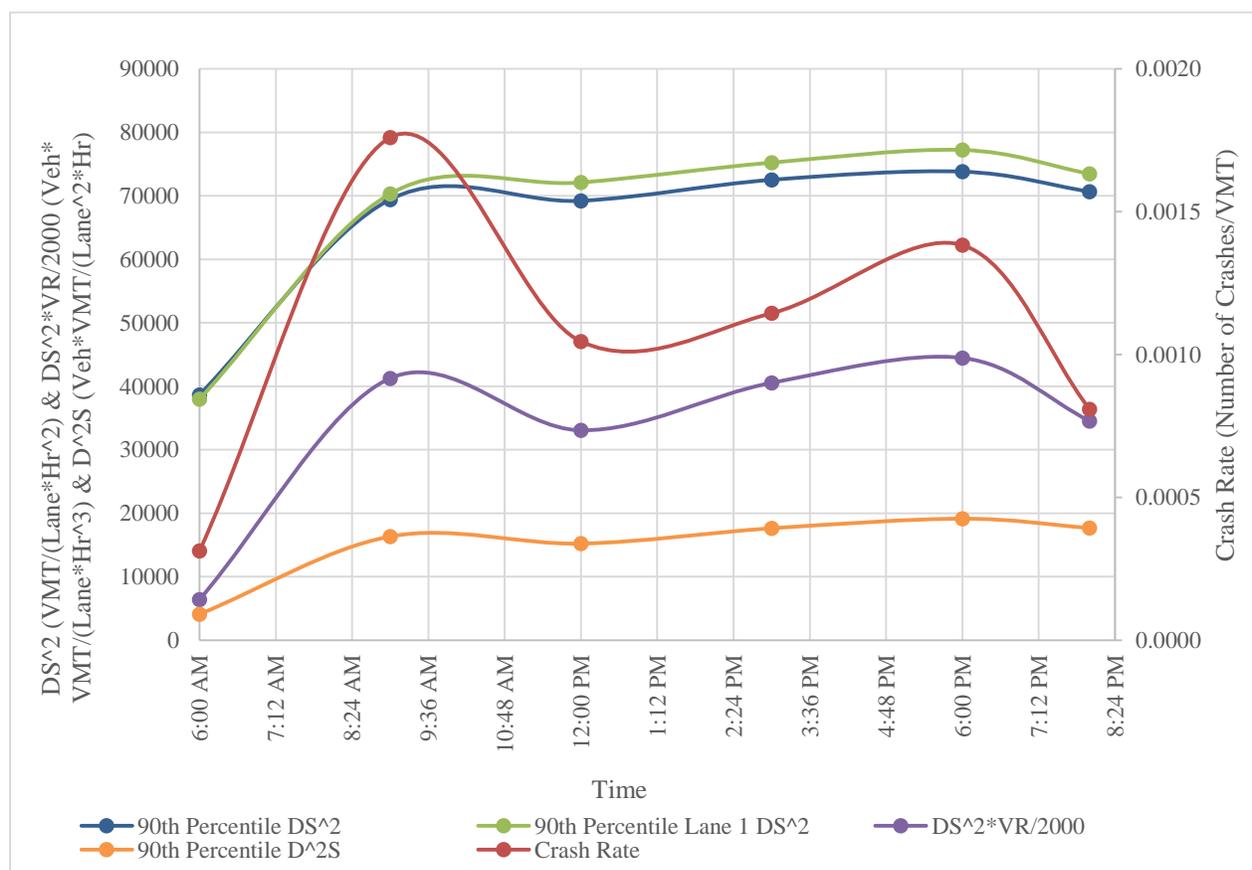


Figure 9-11 Temporal Variation of Density and Speed Multiplication Parameters and Crash Rate at the Hollywood Blvd. Ramp to Next Exit

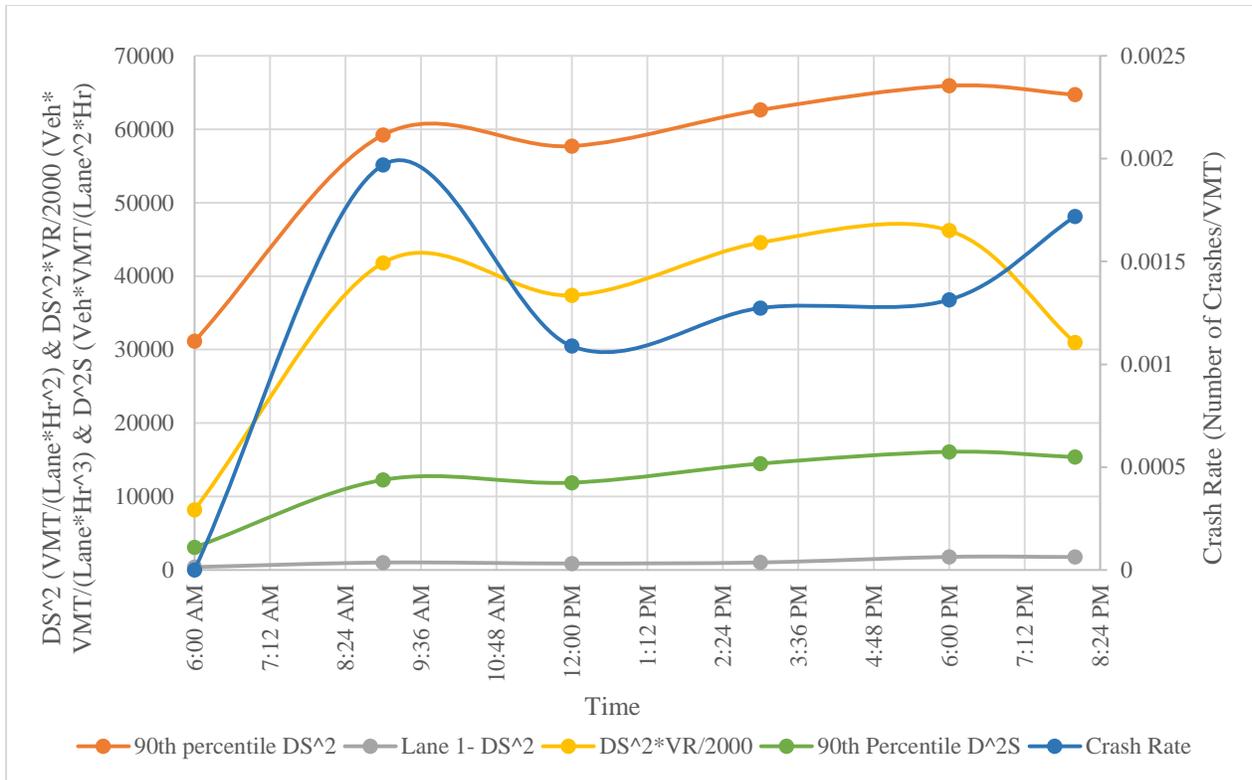


Figure 9-12 Temporal Variation of Density and Speed Multiplication Parameters and Crash Rate at the Hallandale Blvd. Ramp to Next Exit

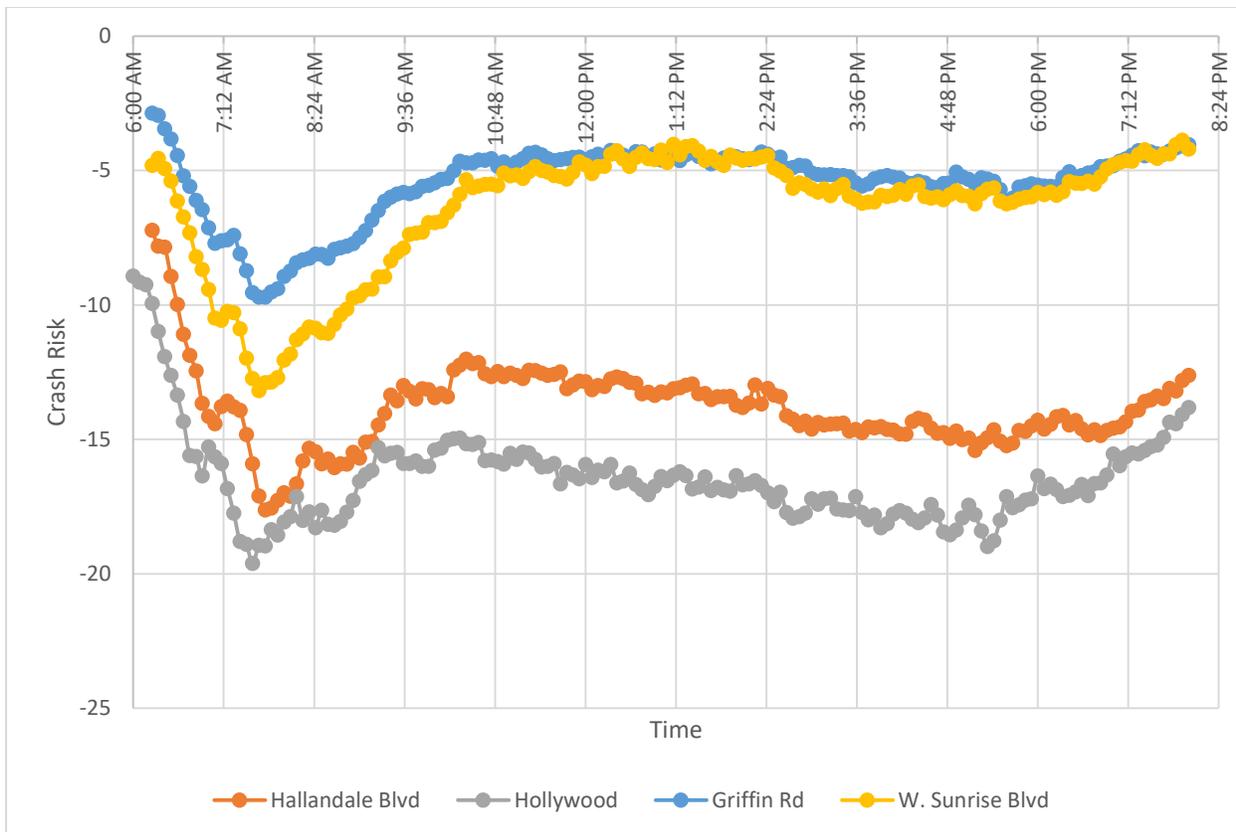


Figure 9-13 Temporal Variation of Crash Risk Based on the Study of Abdel-Aty et al. (2014)

10. RECOMMENDATIONS FOR THE IMPLEMENTATION OF THE DEVELOPED METHOD

10.1 Introduction

As described in the previous chapters, this study developed a method for ramp metering installation considering recurrent and non-recurrent conditions and also considering the truck acceleration requirements. In addition, the study recommended methods for real-time activation of ramps considering predicted mobility and safety performance. To apply these methods, transportation agencies need user friendly tools that can be used to support the off-line and real-time installation and activation decisions, as outlined above. As part of this research, modules were developed and used in Excel and MATLAB to facilitate the various tasks of this research. These modules are not ready to use by FDOT staff and their consultants. Development of tools that can be used to support the off-line and real-time decisions investigated in this study is not within the scope of this project. It is recommended that additional developments are conducted for this purpose as follows.

10.2. Development of a Tool to Support Off-Line Decisions

It is recommended that a tool is developed to support the off-line decisions addressed in this study. This tool should allow the collecting roadway geometry and traffic and incident data, as described in Chapter 4, to allow the estimation of the distributions of demand and capacity. The tool should implement the ramp metering selection formulation and the associated Monte Carlo simulation, developed in Chapter 6 of this study, to identify the ramps that need to be metered under recurrent conditions. In addition, the tool should be able to predict the impacts of incidents and weather conditions on capacity and identify the ramps to be metered under these conditions, as described also in Chapter 6. In addition, the tool should be able to estimate the requirements, assess deficiencies, and assess solutions to address truck acceleration requirements associated with ramp metering, as presented in Chapter 7. The tool should also be able to calculate the benefits and costs of ramp metering, as described in Chapters 5 and 6.

10.3. Application of the Real-Time Metering Activation Method in SunGuideSM

The central software used by the FDOT TMCs is the SunGuide software, The SunGuide software is an integration of a set of modules that allows the control of roadway devices as well as information exchange across a variety of transportation agencies and is deployed throughout the State of Florida. Figure 3-1 provides a graphical view of the SunGuide software architecture. The managed ITS devices by the SunGuide software includes traffic detection devices, cameras and associated encoders and decoders, video walls, dynamic message signs, highway advisory radios, road weather information systems, connected vehicle basic probe data, reversible lane

systems, vehicle safety barriers, ramp metering, variable speed limits, wrong way driving, and express lanes. The software has automated incident and adverse weather conditions detection. In addition to collecting data from variety of point detection devices and automatic vehicle re-identification technologies, data are collected from third-party data feeds including HERE and WAZE. The traffic information is displayed on the operator map interface for use by the operator and used as an input to various SunGuide software modules such as automatic incident detection, travel time calculation, setting variable speed limits, and express lanes pricing. In addition, traffic and incident data are shared through center-to-center facilities with external systems including the Florida's advanced traveler information system, referred to as FL511. The central software also directly interacts with mobile devices utilized by the service patrols, referred to as Road Rangers in Florida. The software recommends response plans, stores event details, and produces customizable reports in graphical or tabular formats. The software also share data with the statewide data archived hosted in the Regional Integrated Transportation Information System (RITIS).

It is recommended that the SunGuide software is modified to implement the ramp metering activation methodology during recurrent and non-recurrent conditions, based on the methods developed in this study, as discussed in Chapter 8. These methods include the Table Lookup method and the adaption of the ramp metering selection formulation for real-time execution methods. In addition, it is recommended that the tool is able to activate metering based on the safety measures identified in Chapter 9. An evaluation should be made by the FDOT of the effectiveness of the developed method after its application.

REFERENCES

Abdel-Aty, M., Uddin, N., and Pande, A. (2014). "Split Models for Predicting Multivehicle Crashes during High-Speed and Low-Speed Operating Conditions on Freeways." *Transportation Research Record: Journal of the Transportation Research Board*, Vol.1908, pp. 51-58.

Aghdashi, S. B. (2013). *Traffic Flow Optimization in a Freeway with Stochastic Segments Capacity*. North Carolina State University, Raleigh, North Carolina.

American Association of State Highway and Transportation Officials (AASHTO). (2011). *A Policy on Geometric Design of Highways and Streets*. AASHTO, Washington, D.C.

Atkins. (2013). *Ramp Metering Feasibility Study for Durham and Wake Counties*, North Carolina Department of Transportation.

Balke, K., Chaudhary, P., Songchitruksa, P., & Pesti, G. (2009). *Development of Criteria and Guidelines for Installing, Operating, and Removing TxDOT Ramp Control Signals*. Texas Department of Transportation, Austin, Texas.

Bellemans, T., De Schutter, B., and De Moor, B. (2006). "Model Predictive Control for Ramp Metering of Motorway Traffic: A Case Study." *Control Engineering Practice*, Vol. 14. No. 7, pp. 757-767.

Bertini, S., and Ahn, S. (2006). *Using archived ITS Data to Measure the Operational Benefits of A System-Wide Adaptive Ramp Metering System*. Oregon Department of Transportation.

Bogenberger, K. (2001). *Adaptive Fuzzy Systems for Traffic Responsive and Coordinated Ramp Metering*. Munich, Germany: Fachgebiet Verkehrstechnik und Verkehrsplanung.

Bogenberger, K., Keller, H., and Ritchie, S. G. (2001). *Adaptive Fuzzy Systems for Traffic Responsive and Coordinated Ramp Metering*. Irvine State: Institute of Transportation Studies, University of California.

Bogenberger, K., and May, A. D. (1999). *Advanced Coordinated Traffic Responsive Ramp Metering Strategies* (California PATH Working Paper, UCB-ITS-PWP-99-1). Berkeley, CA: Institute of Transportation Studies, University of California.

Brilon, W., Geistefeldt, J., and Regler, M. (2005). "Reliability of Freeway Traffic Flow: A Stochastic Concept of Capacity." in H. S. Mahmassani (ed.), *Transportation and Traffic Theory : Flow, Dynamics and Human Interaction: Proceedings of the 16th International Symposium on*

Transportation and Traffic Theory, University of Maryland, College Park, Maryland, 19-21 July 2005. Washington, D.C.: Transportation Research Board. pp. 125 – 144.

Camacho, E., and Bordons, C. (1995). *Model Predictive Control in the Process Industry*. Berlin, Germany: Springer-Verlag.

California Department of Transportation (Caltrans). (2000). *California Ramp Meter Design Manual*. California Department of Transportation: Division of Traffic Operations.

Cambridge Systematics, Inc. (2001). *Twin Cities Ramp Meter Evaluation*. Minnesota Department of Transportation, Oakland, California.

Chang, J., Oh, C., and Chang, M. (2000). “Effects of Traffic Condition (v/c) on Safety at Freeway Facility Sections.” in *Proceedings of the Transportation Research, E-Circular: 4th International Symposium on Highway Capacity, Maui, Hawaii*. Washington, D.C.: Transportation Research Board.

Elefteriadou, L., Kondyli, A., Brilon, W., Jacobson, L., Hall, F., and Persaud, B. (2009). *Proactive Ramp Management under the Threat of Freeway-Flow Breakdown*, National Cooperative Highway Research Program.

Elefteriadou, L., and Lertworawanich, P. (2003). *Defining, Measuring and Estimating Freeway Capacity*, Proceedings of the 82nd Annual Meeting of Transportation Research Board, Washington D.C.

Elefteriadou, L., Roess, R.P., and McShane, W.R. (1995). “Probabilistic Nature of Breakdown at Freeway Merge Junctions.” *Transportation Research Record: Journal of the Transportation Research Board*, Vol. 1484, pp. 80-89.

Federal Highway Administration (FHWA). (2015a). *How Do Weather Events Impact Roads?* Retrieved from Road weather Management Program: http://ops.fhwa.dot.gov/Weather/q1_roadimpact.htm, Accessed April 9, 2015.

Federal Highway Administration (FHWA). (2015b). *Traffic Congestion and Bottlenecks; Identification, Diagnosis, Solutions*. U.S. Department of Transportation.

Federal Highway Administration (FHWA). (2009). *Manual on Uniform Traffic Control Devices (MUTCD)*.

Federal Highway Administration (FHWA). (2006a). *Ramp Management and Control Handbook*. U.S. Department of Transportation, Federal Highway Administration.

Federal Highway Administration (FHWA). (2006b). *Empirical Studies on Traffic Flow in Inclement Weather*, U.S. Department of Transportation, Federal Highway Administration.

Federal Highway Administration (FHWA). (2003). *Manual on Uniform Traffic Control Devices (MUTCD)*.

Florida Department of Transportation (FDOT). (2016). *Manual of Uniform Minimum Standards for Design, Construction and Maintenance for Streets and Highways*. Florida.

Gaisser, T., and DePinto, K. (2015). *I-70 East Ramp Metering Assessment*. Colorado Department of Transportation.

Gan, A., Liu, K., Alluri, P., & Zhu, X. (2011). *Integrated Database and Analysis System for the Evaluation of Freeway Corridors for Potential Ramp Signaling*. Florida Department of Transportation.

Goolsby, M. E. (1971). "Influence of Incidents on Freeway Quality of Service." *Transportation Research Record: Journal of the Transportation Research Board*. Vol. 349, pp. 41-46.

Hadi, M., Sinha, P., and Wang, A. (2007). "Modeling Reduction in Freeway Capacity due to Incidents in Microscopic Simulation Models." *Transportation Research Record: Journal of the Transportation Research Board*. Vol. 1999, pp. 62-68.

Hadi, M., Xiao, Y., Ozen, H., and Alvarez, P. (2008). *Evaluation Tools to Support ITS Planning Process: Development of a Sketch Planning Tool in FSUTMS/Cube Environment*. Florida Department of Transportation.

Hadi, M., Xiao, Y., and Rojas, M. L. (2013). "Estimation of Diversion Rate during Incidents on Basis of Main-Line Detector Data." *Transportation Research Record: Journal of the Transportation Research Board*, Vol. 2396, pp. 54-60.

Hadi, M., Zhan, C., and Alvarez, P. (2011). *Traffic Management Simulation Development*. Florida Department of Transportation.

Haj-Salem, H. and Papageorgiou, M. (1995). "Ramp Metering Impact On Urban Corridor Traffic: Field Results." *Transportation Research Part A: Policy and Practice*. Vol. 29, pp. 303-319.

- Harwood, D. W., Torbic, D. J., Richard, K. R., Glauz, W. D., and Elefteriadou, L. (2003). *Review of Truck Characteristics as Factors in Highway Design*. NCHRP Report 505, Transportation Research Board of the National Academies, Washington, D.C.
- Hasan, M. (1999). *Evaluation of Ramp Control Algorithms Using a Microscopic Traffic Simulation Laboratory, MITSIM*. Massachusetts Institute of Technology.
- Hourdakis, J., and Michalopoulos, P. G. (2002). *Evaluation of Ramp Control Effectiveness in Two Twin Cities Freeways*. Proceedings of the 81st Annual Meeting of Transportation Research Board, Washington D.C.
- Ibrahim, A. T., and Hall, F. L. (1994). Effect of Adverse Weather Conditions on Speed- Flow Occupancy Relationships. *Transportation Research Record: Journal of the Transportation Research Board*, Vol. 1457, pp. 184-191.
- Jacobs Engineering Group, Inc. (2013). *Managed Lanes and Ramp Metering Manual, Part 2: Implementation Plan*. Nevada Department of Transportation.
- Jacobson, L., Henry, K., and Mahyar, O. (1989). "Real-time Metering Algorithm for Centralized Control." *Transportation Research Record: Journal of the Transportation Research Board*. Vol. 1232.
- Kang, S., and Gillen, D. (1999). *Assessing the Benefits and Costs Of Intelligent Transportation Systems: Ramp Meters*. California Partners For Advanced Transit And Highways Research Report.
- Kaplan, E. L., and Meier, P. (1958). "Nonparametric estimation from incomplete observations." *Journal of the American Statistical Association*, Vol. 53, pp. 457-481.
- Knoop, V. L., Hoogendoorn, S. P., and Adams, K. (2009). "Capacity Reductions at Incidents Sites on Motorways." *European Journal of Transport and Infrastructure Research*, Vo. 9, No. 4.
- Kononov, J., Durso, C., Reeves, D., and Allery, B. K. (2012). "Relationship between Traffic Density, Speed, and Safety and Its Implications for Setting Variable Speed Limits on Freeways." *Transportation Research Record: Journal of the Transportation Research Board*, Vol. 2280, pp. 1-9.
- Kristeleit, T. P. (2014). *Analysis and Evaluation of Ramp Metering Algorithms*. Munich: University of the Federal Armed Forces.

Lee, J., Abdel-Aty, M., and Wang, L. (2016). *Utilizing Micro Simulation to Evaluate the Safety and Efficiency of the Expressway System*. University of Central Florida.

Leiman, L., Bouaunia, M., and May, A.D. (1991). "Integrated System of Freeway Corridor Simulation Models." *Transportation Research Record: Journal of the Transportation Research Board*. Vol. 1320, pp. 177-189.

Li, Z., Elefteriadou, L., and Kondyli, A. (2014). Modeling Weather Impacts on Traffic Operations: Implementation into Florida's Travel Time Reliability Model. Proceedings of the 93rd Annual Meeting of the Transportation Research Board, Washington D.C.

Lipp, L., Corcoran, L., and Hickman, G. (1991). *Benefits of Central Computer Control for the Denver Ramp Metering System*, Proceedings of the 70th Annual Meeting of Transportation Research Board, Washington D.C.

Maciejowski, J. (2002). *Predictive Control with Constraints*. Prentice Hal. Harlow.

Martin, J. L. (2002). "Relationship between Crash Rate and Hourly Traffic Flow on Interurban Motorways." *Accident Analysis & Prevention*, Vol. 34, No. 5, pp. 619-629.

May, A. D. (1976). *Application of Simulation Models in Traffic Management Operations*, University of California, Berkely.

May, A. D. (1990). *Traffic Flow Fundamentals*. Englewood Cliffs, New Jersey.

Okamura, H., Watanabe, S., and Watanabe, T. (2000). *An Empirical Study on the Capacity of Bottlenecks on the Basic Suburban Expressway Sections in Japan*. Proceedings of the 4th International Symposium on Highway Capacity, TRB Circular E-C018, pp. 120-129.

Papageorgiou, M., Blosseville, J. M., and Hadj-Salem, H. (1990). "Modeling and Real-Time Control of Traffic Flow on the Southern Part of Boulevard Peripherique in Parris, Part II: Coordinated On-Ramp Metering." *Transportation Research*, Vol. 24A.

Persaud, B., and Dzbik, L. "Accident Prediction Models for Freeways." *Transportation Research Record: Journal of the Transportation Research Board*, Vol. 1401, pp.55-60.

Piotrowicz, G., and Robinson, J. (1995). *Ramp Metering Status in North America-1995 Update*. Federal Highway Administration, U.S. Department of Transportation.

Ponzlet, M. (1996). Dynamik der Leistungsfähigkeiten von Autobahnen (Dynamics of Freeway Capacity), *Schriftenreihe des Lehrstuhls fuer Verkehrswesen der Ruhr Universitaet Bochum*, No. 16. Bochum.

Qi, L., and Smith, B. (2001). *Characterization of Accident Capacity Reduction*. Center for Transportation Studies, University of Virginia.

Shea, M. S., Le, T., & Porter, R. (2015). "Combined Crash Frequency-Crash Severity Evaluation of Geometric Design Decisions: Entrance-Exit Ramp Spacing and Auxiliary Lane Presence." *Transportation Research Record: Journal of the Transportation Research Board*. Vol. 2521, pp. 54-63.

Simpson, S., and Yasmin, F. (2013). *Ramp Metering Design Guide*. Arizona Department of Transportation.

Stralen, V., Calvert, S. C., and Molin, E. J. (2014). The Influence of Adverse Weather Conditions on the Probability of Congestion on Dutch Motorways. 93th Annual Meeting of the Transportation Research Board. Washington D.C.

Tian, Z., Xu, H., Yang, G., Khan, A., and Zhao, Y. (2016). *Queue Storage and Acceleration Lane Length Design at Metered On-ramps in California*. Final Report for California Department of Transportation, Research Project # 65A0486.

Transportation Research Board. (2016). *Highway Capacity Manual (HCM) 2016*. Washington, D.C.

Transportation Research Board. (2010). *Highway Capacity Manual (HCM) 2010*. Washington, D.C.

Transportation Research Board. (2000). *Highway Capacity Manual (HCM) 2000*. Washington, D.C.

Wilbur Smith Associates. (2006). *Wisconsin Statewide Ramp Control Plan*. Wisconsin Department of Transportation.

Wu, J. (2001). *Traffic Diversion Resulting from Ramp Metering*. The University of Wisconsin, Milwaukee.

Yang, G., Xu, H., Tian, Z., and Wang, Z. (2016). Investigating Truck Acceleration Capability at Metered On-Ramps. *Proceedings of the 95th Annual Meeting of the Transportation Research Board*. Washington D.C.

Zhang , M., Kim, T., Nie, X., Jin, W., Chu, L., and Recker, W. (2001). *Evaluation of On-Ramp Control Algorithms*. California PATH Research Report UCB-ITS-PRR-2001-36.

Zhou, M., and Sisiopiku, V. (1997). “Relationship between Volume-to-Capacity Ratios and Accident Rates.” *Transportation Research Record: Journal of the Transportation Research Board*, Vol. 1581, pp.47-52.

APPENDIX A DEMAND DISTRIBUTIONS FOR 15-MINUTE INTERVALS

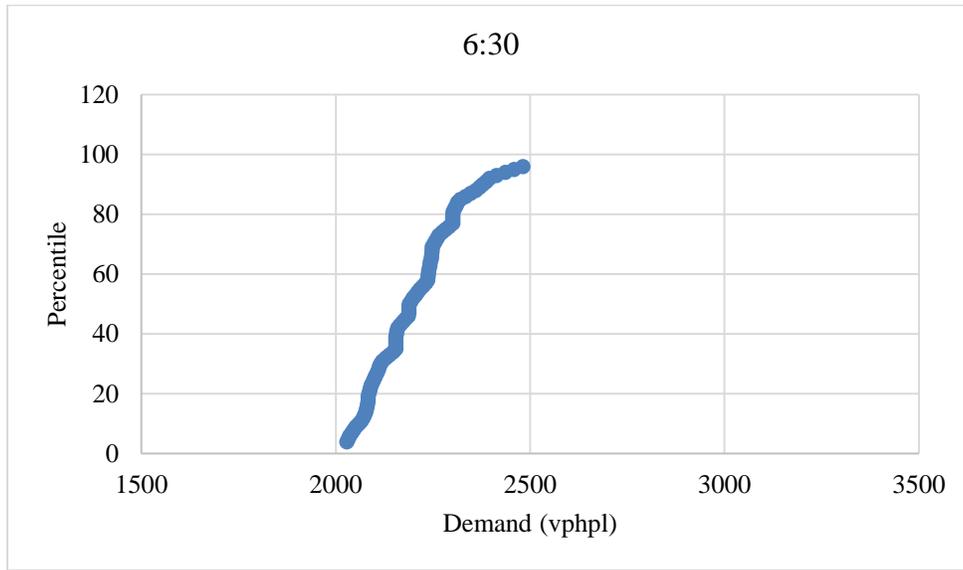


Figure A-1 Demand Distribution at 6:30 am

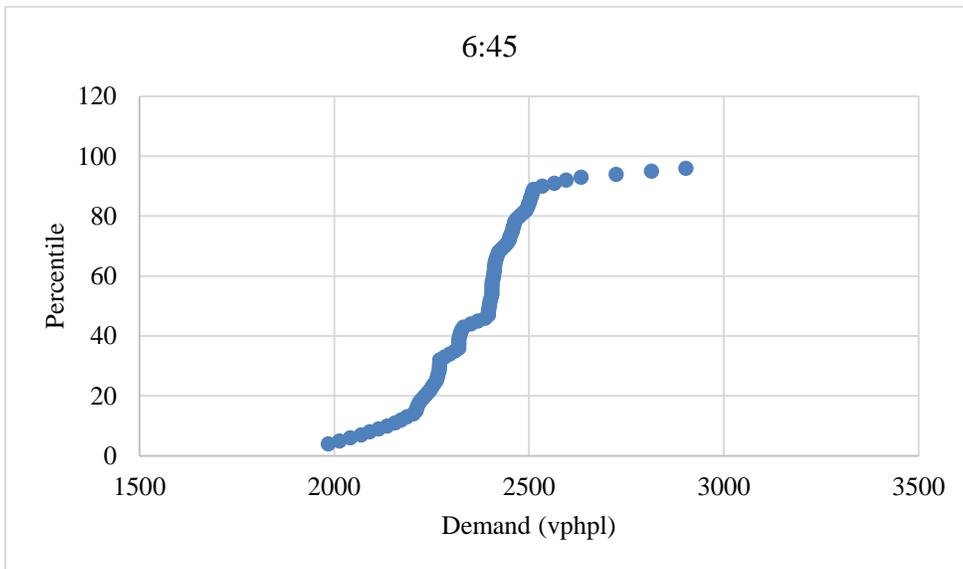


Figure A-2 Demand Distribution at 6:45 am

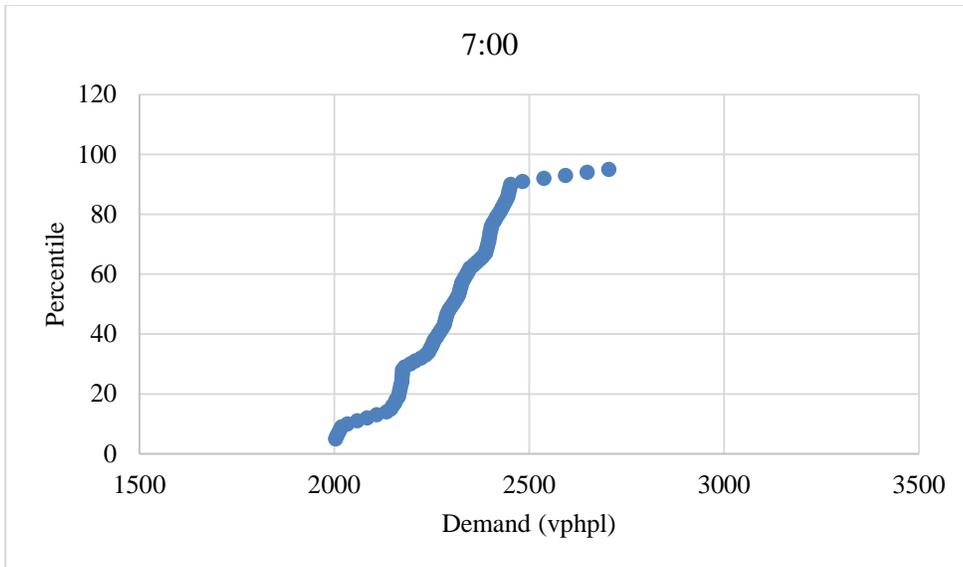


Figure A-3 Demand Distribution at 7:00 am

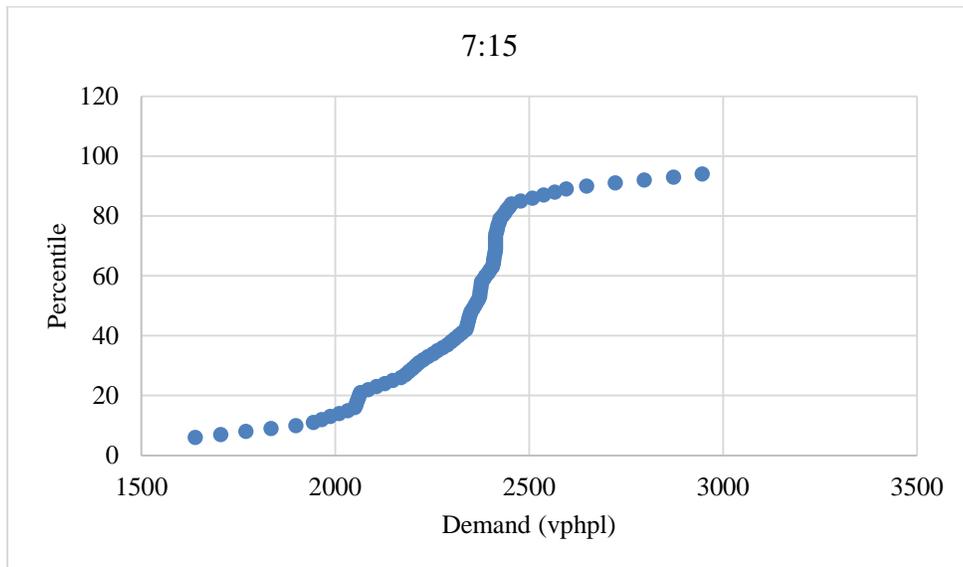


Figure A-4 Demand Distribution at 7:15 am

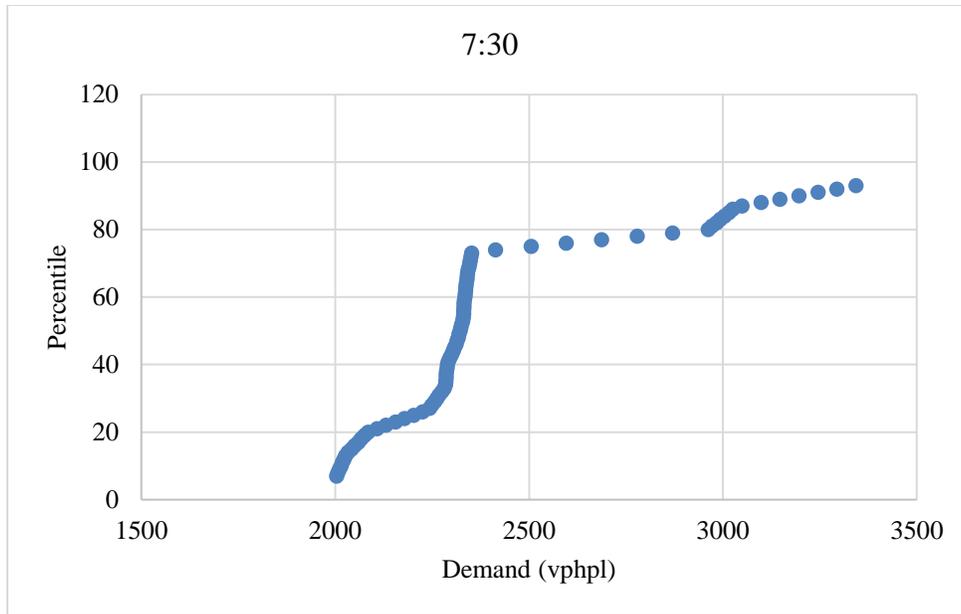


Figure A-5 Demand Distribution at 7:30 am

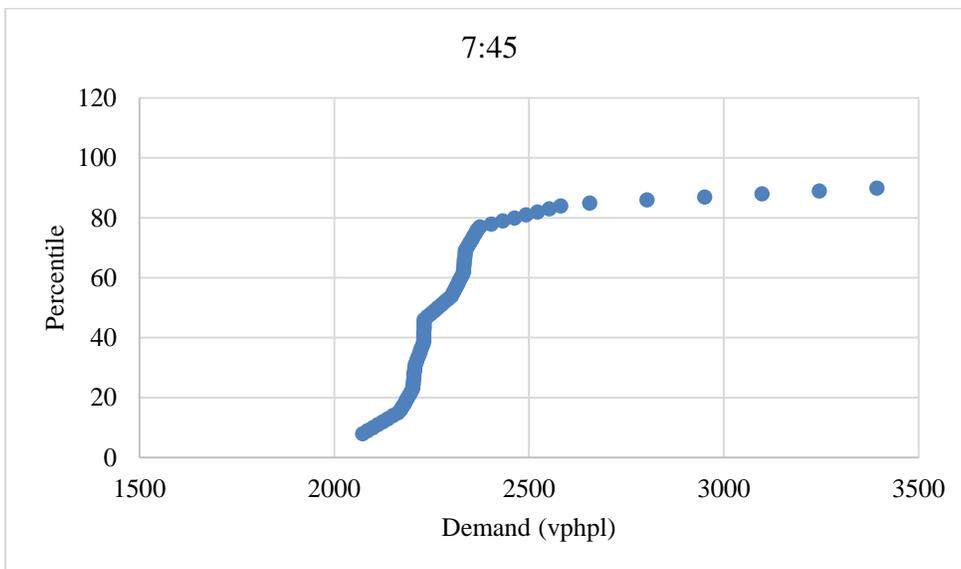


Figure A-6 Demand Distribution at 7:45 am

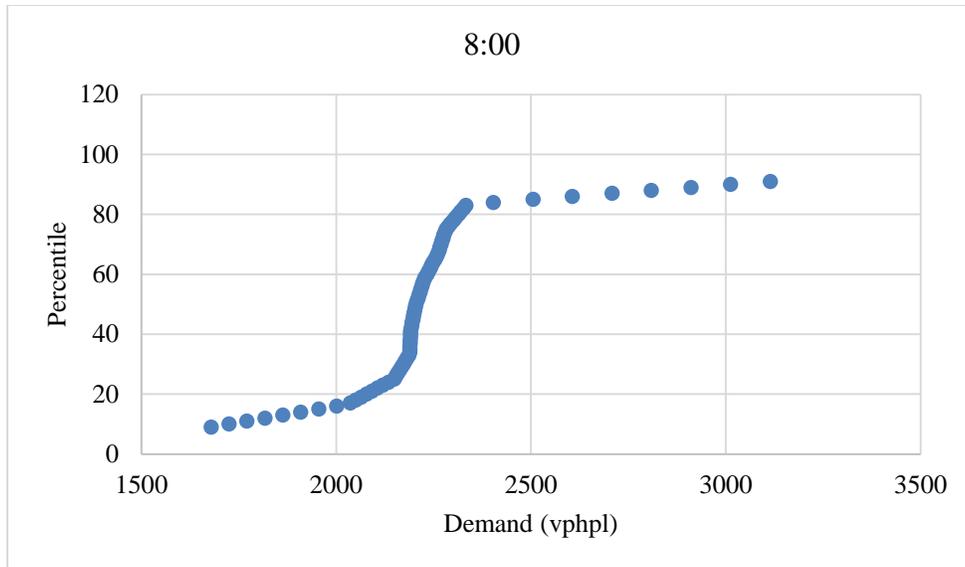


Figure A-7 Demand Distribution at 8:00 am

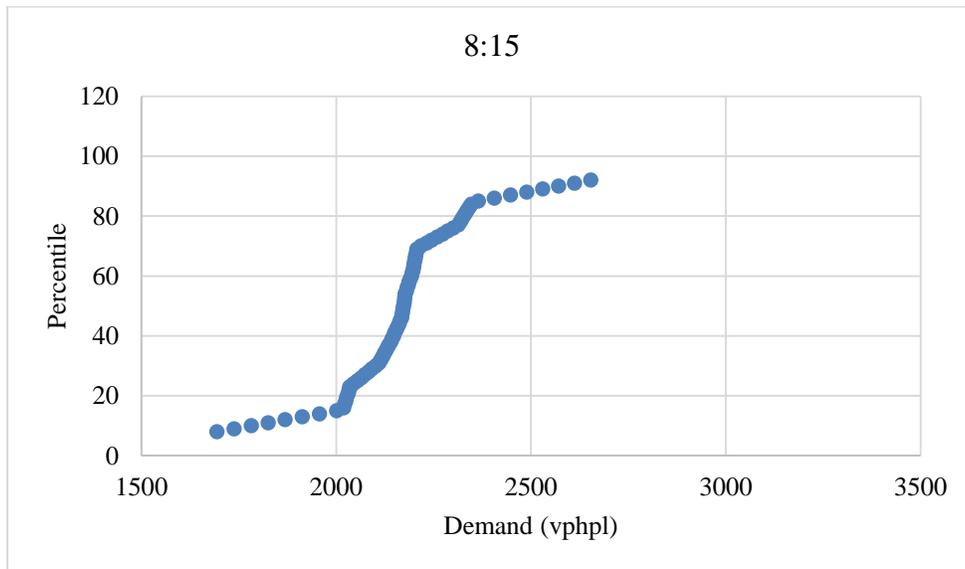


Figure A-8 Demand Distribution at 8:15 am

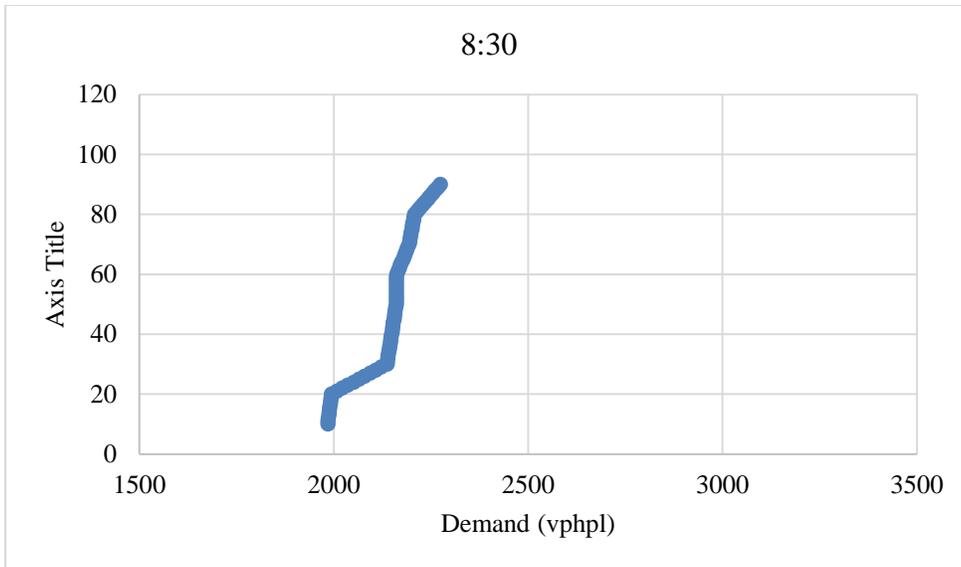


Figure A-9 Demand Distribution at 8:30 am

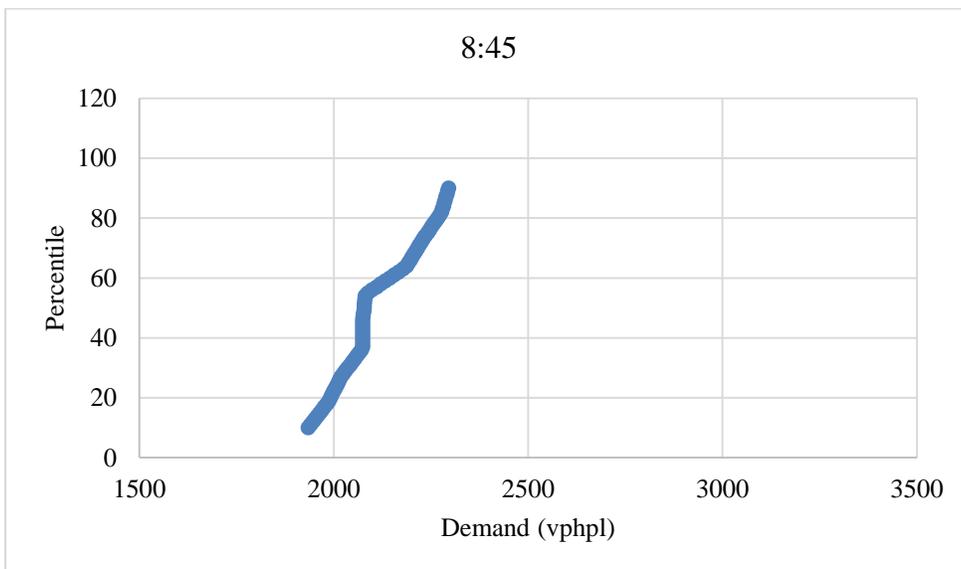


Figure A-10 Demand Distribution at 8:45 am

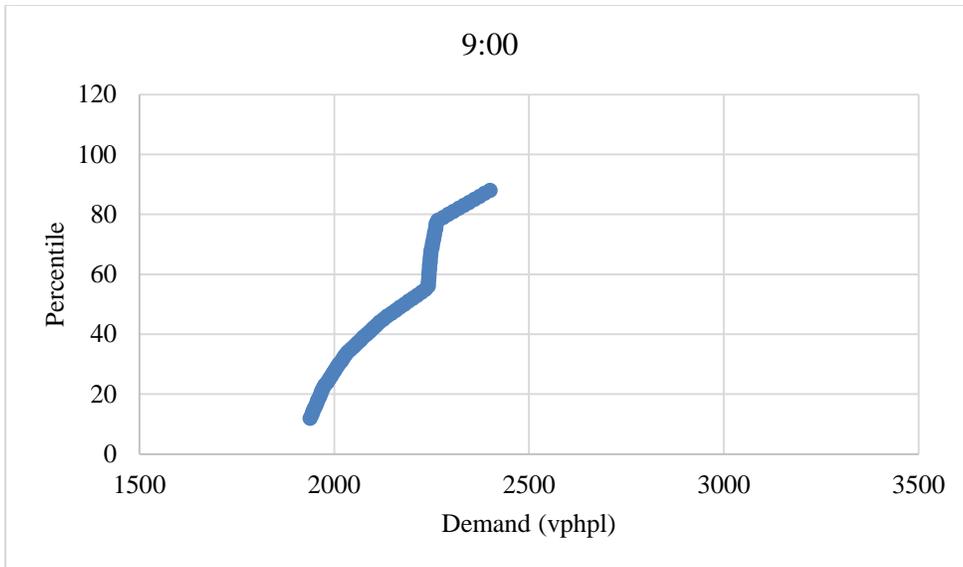


Figure A-11 Demand Distribution at 9:00 am

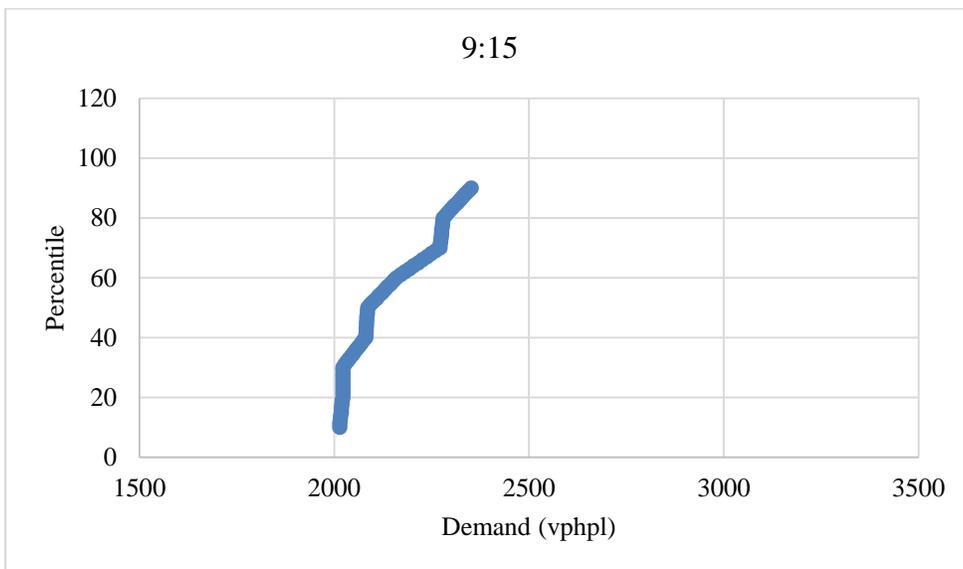


Figure A-12 Demand Distribution at 9:15 am

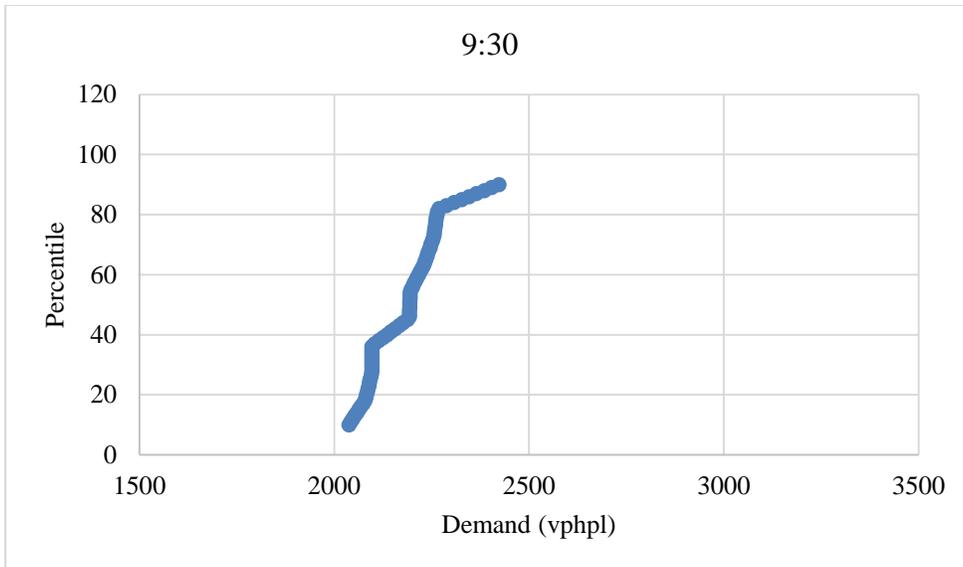


Figure A-13 Demand Distribution at 9:30 am

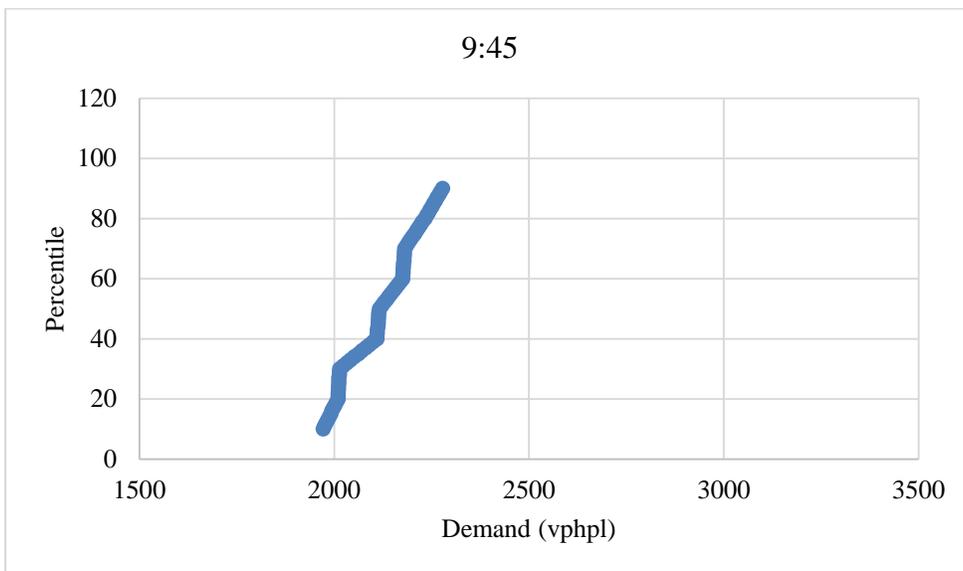


Figure A-14 Demand Distribution at 9:45 am

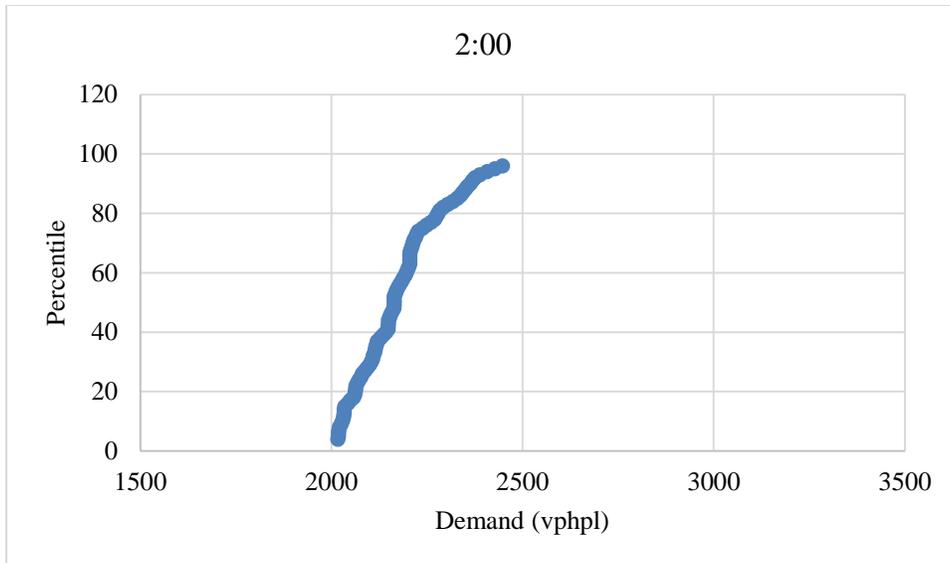


Figure A-15 Demand Distribution at 2:00 pm

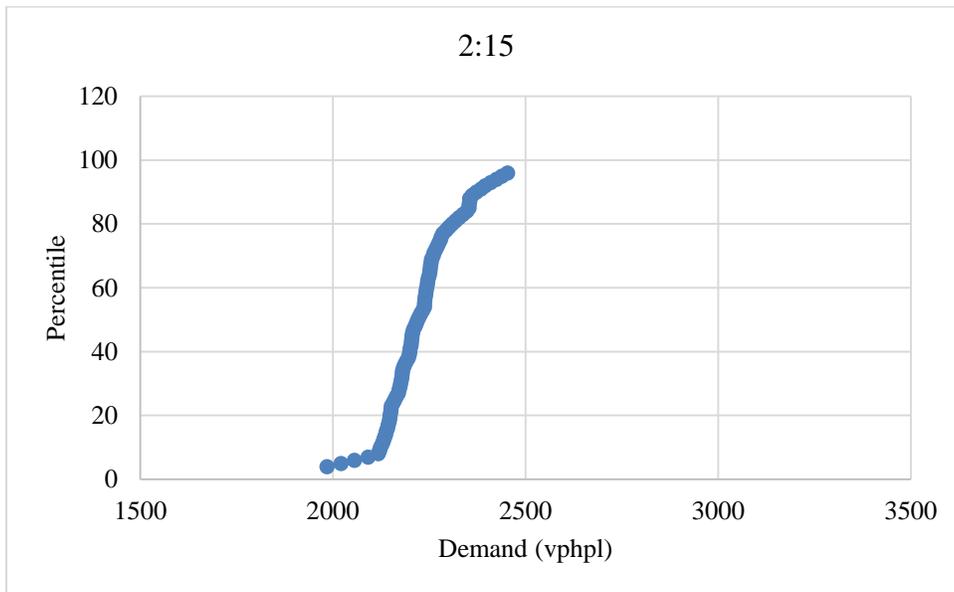


Figure A-16 Demand Distribution at 2:15 pm

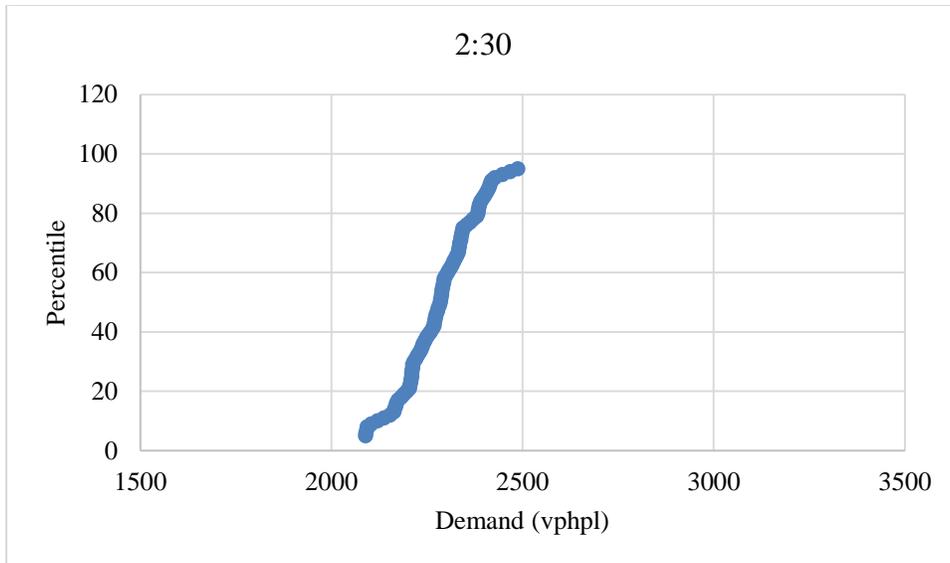


Figure A-17 Demand Distribution at 2:30 pm

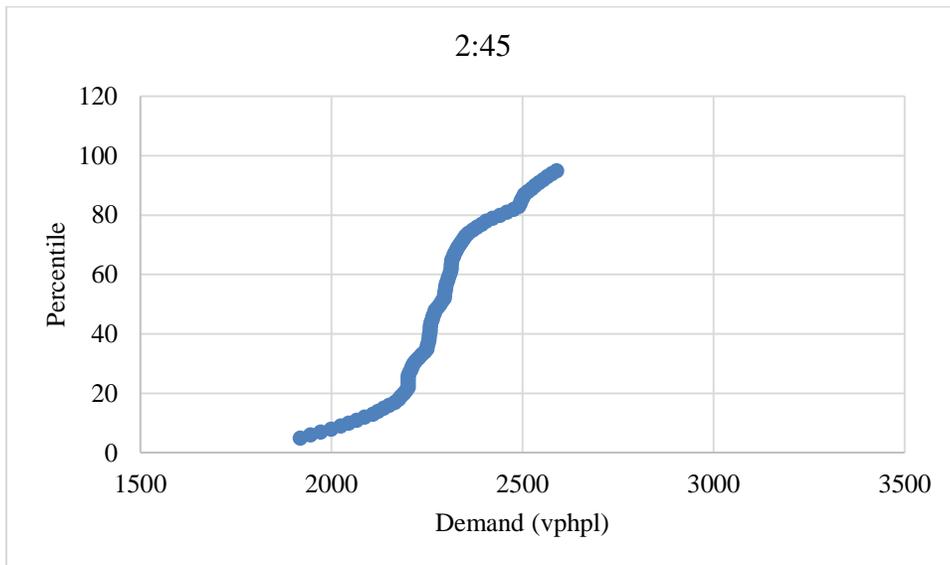


Figure A-18 Demand Distribution at 2:45 pm

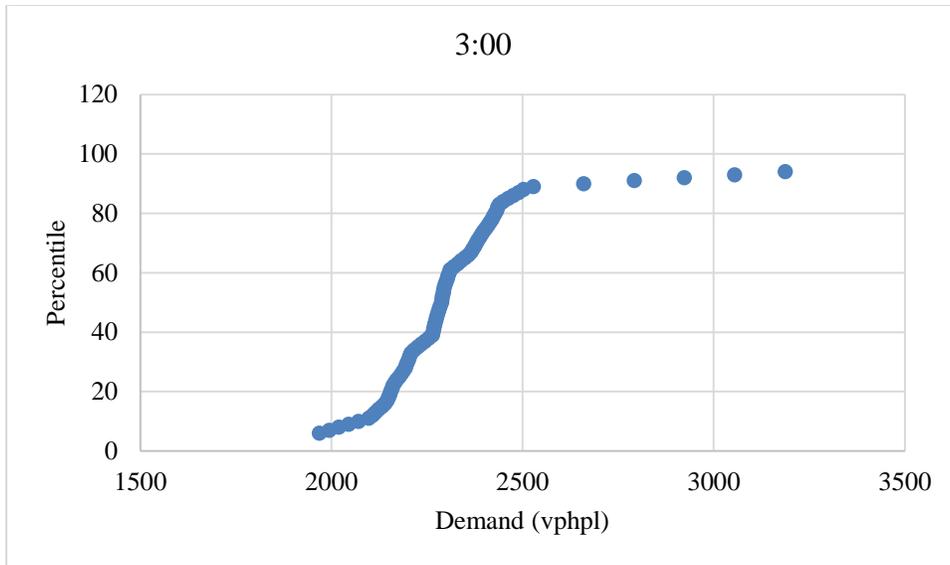


Figure A-19 Demand Distribution at 3:00 pm

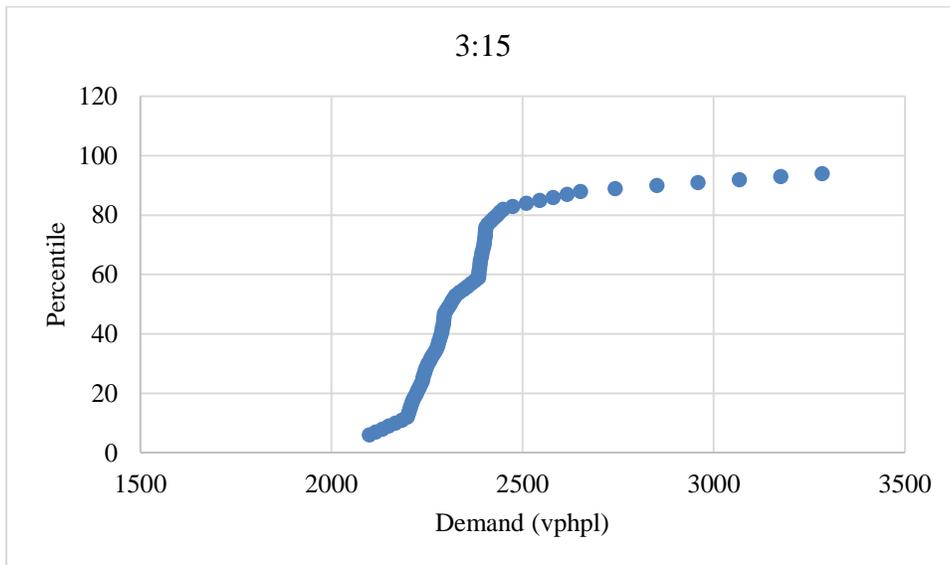


Figure A-20 Demand Distribution at 3:15 pm

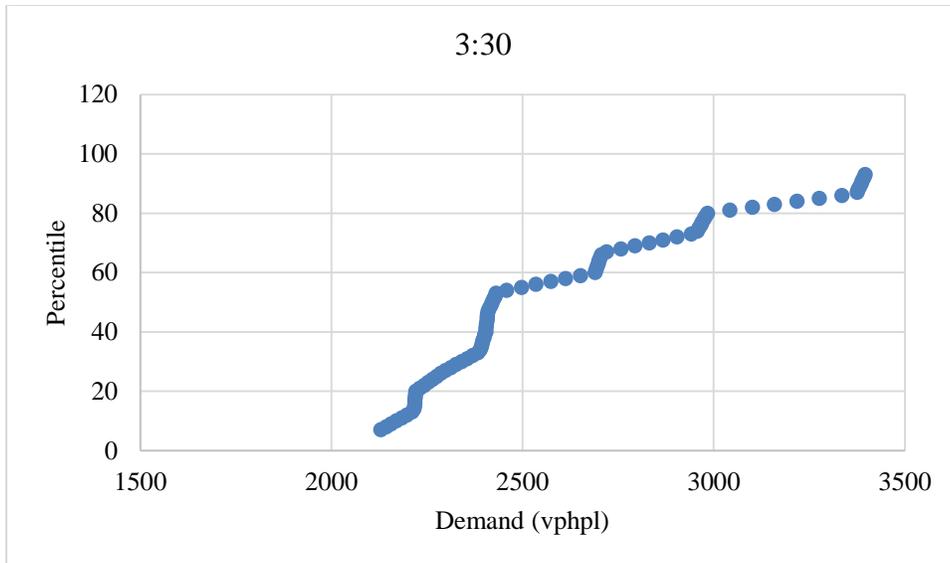


Figure A-21 Demand Distribution at 3:30 pm

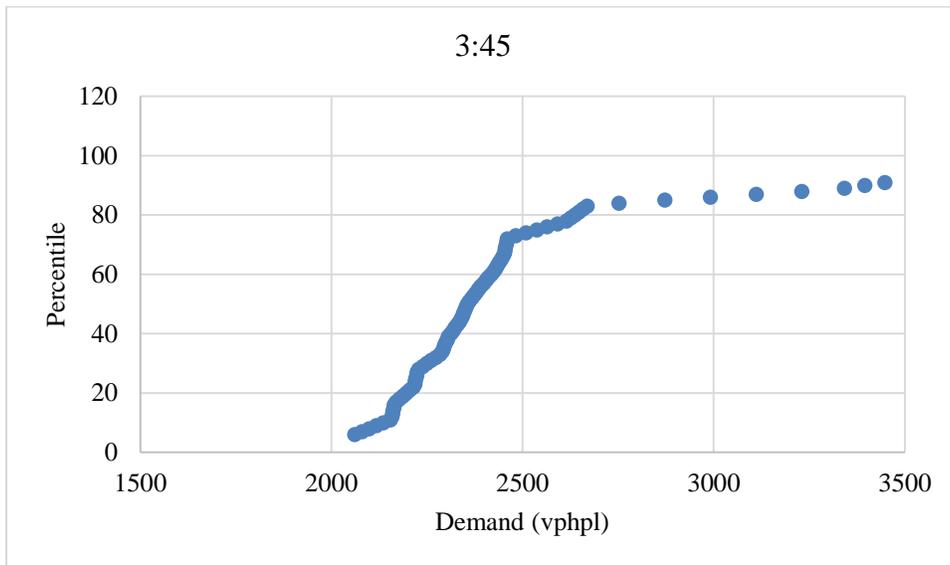


Figure A-22 Demand Distribution at 3:45 pm

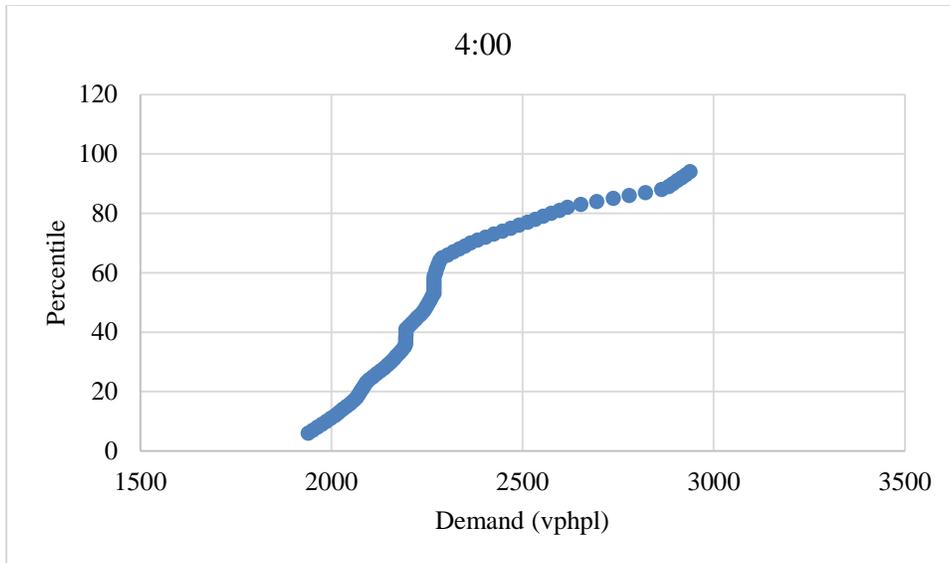


Figure A-23 Demand Distribution at 4:00 pm

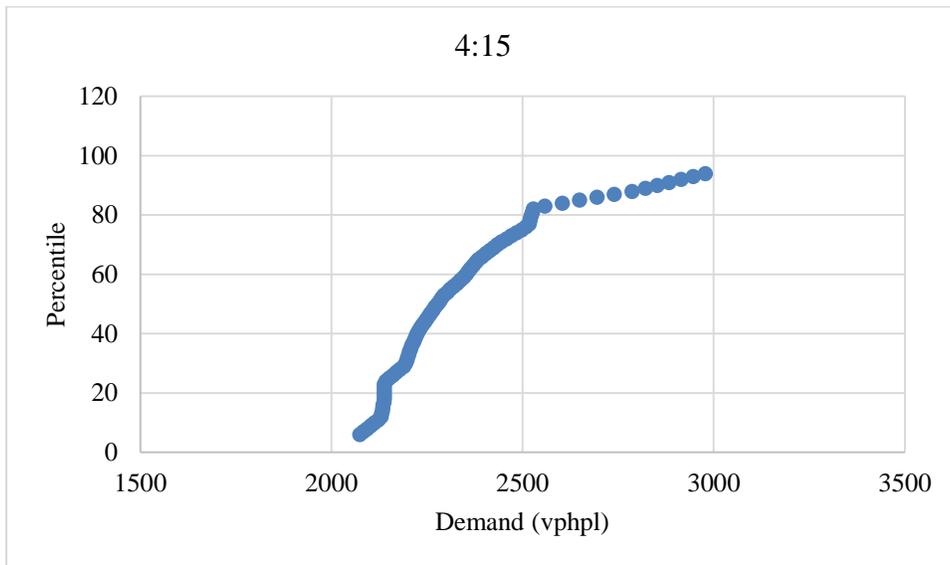


Figure A-24 Demand Distribution at 4:15 pm

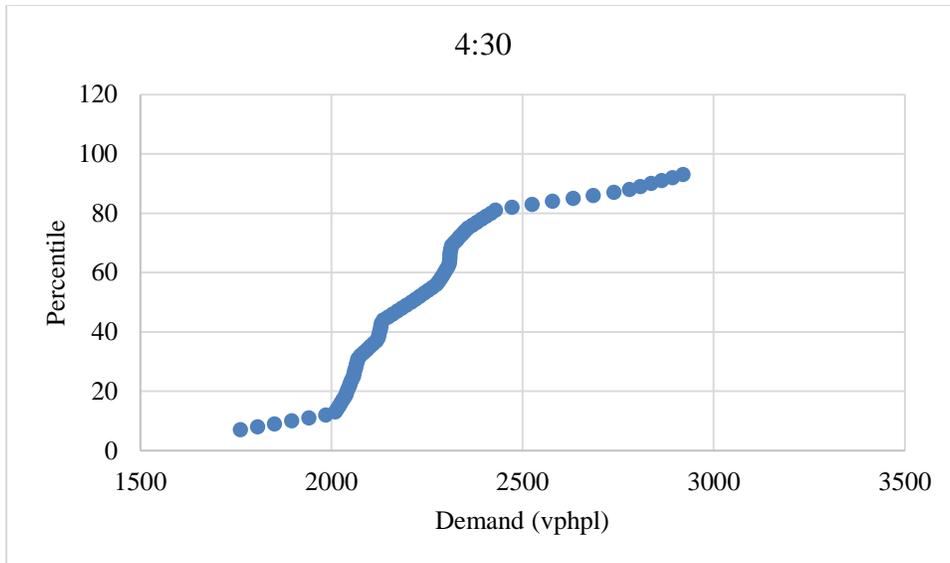


Figure A-25 Demand Distribution at 4:30 pm

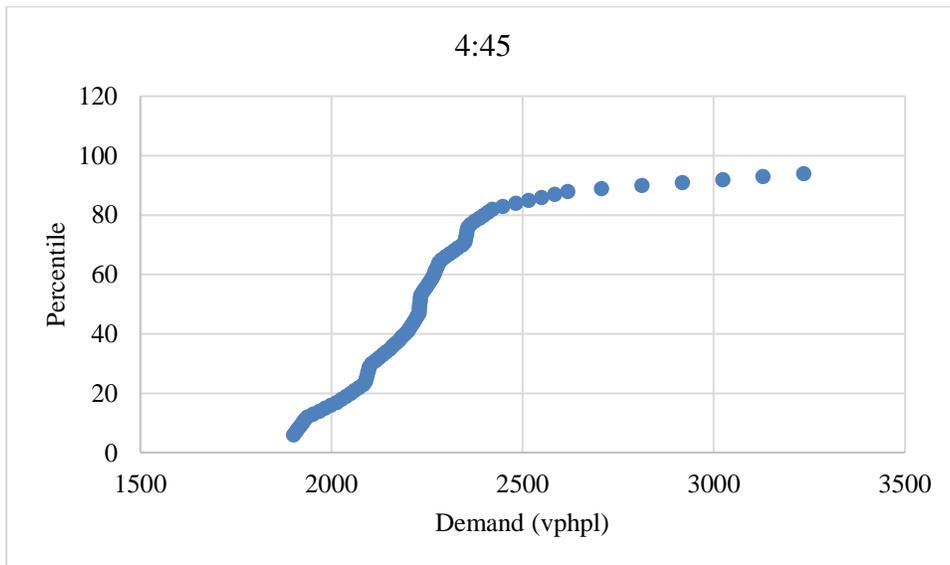


Figure A-26 Demand Distribution at 4:45 pm

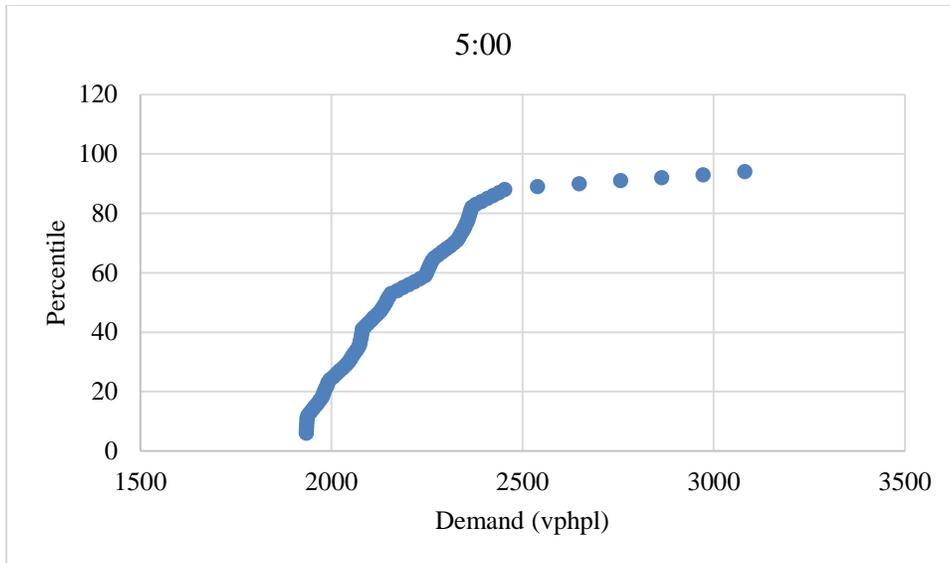


Figure A-27 Demand Distribution at 5:00 pm

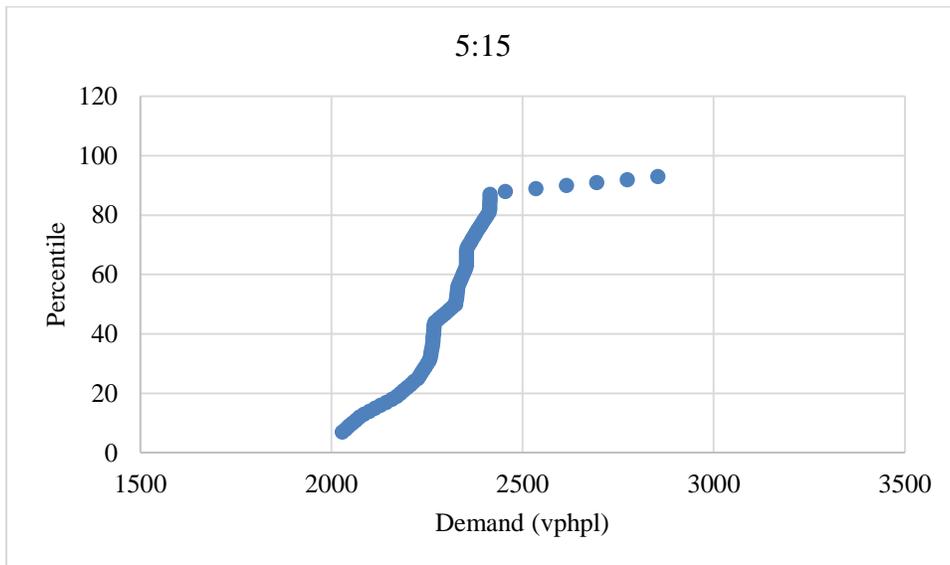


Figure A-28 Demand Distribution at 5:15 pm

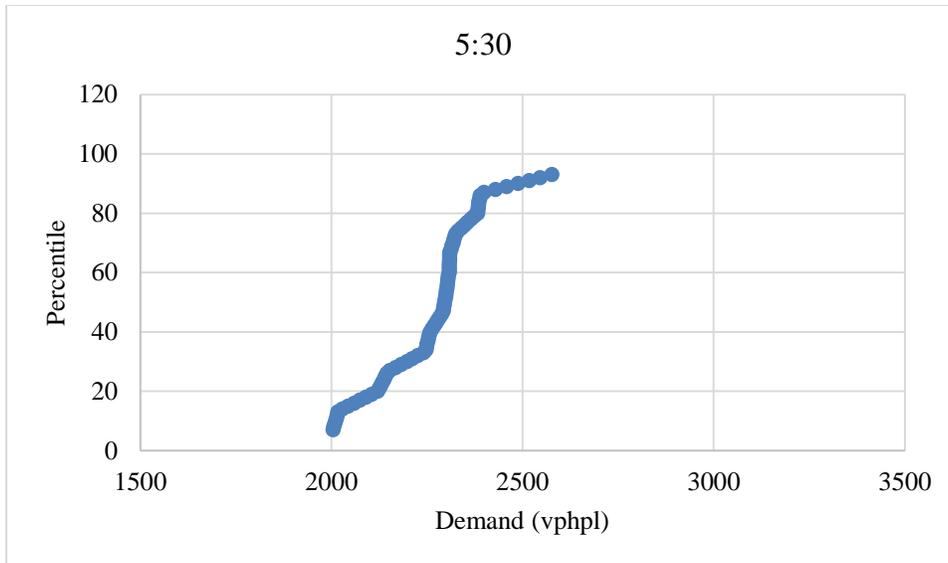


Figure A-29 Demand Distribution at 5:30 pm

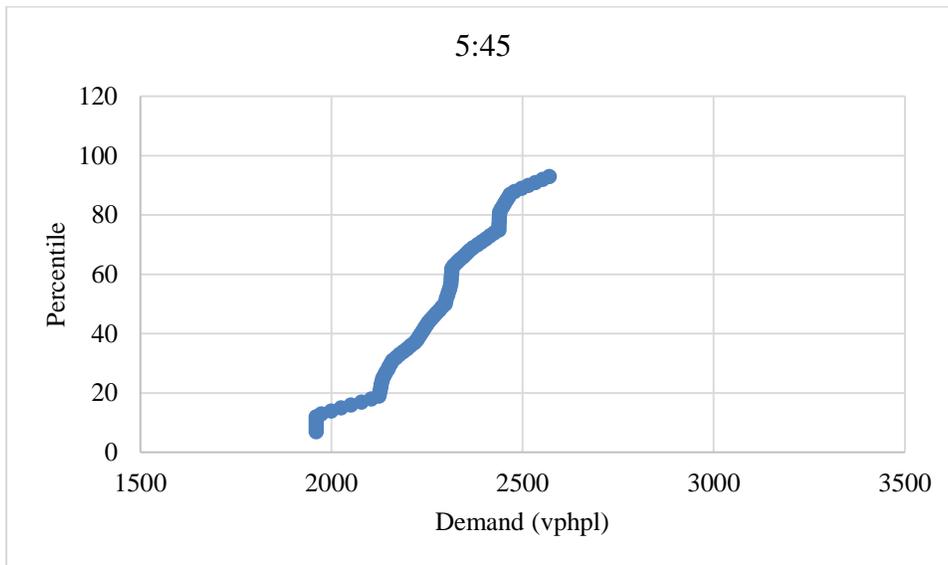


Figure A-30 Demand Distribution at 5:45 pm

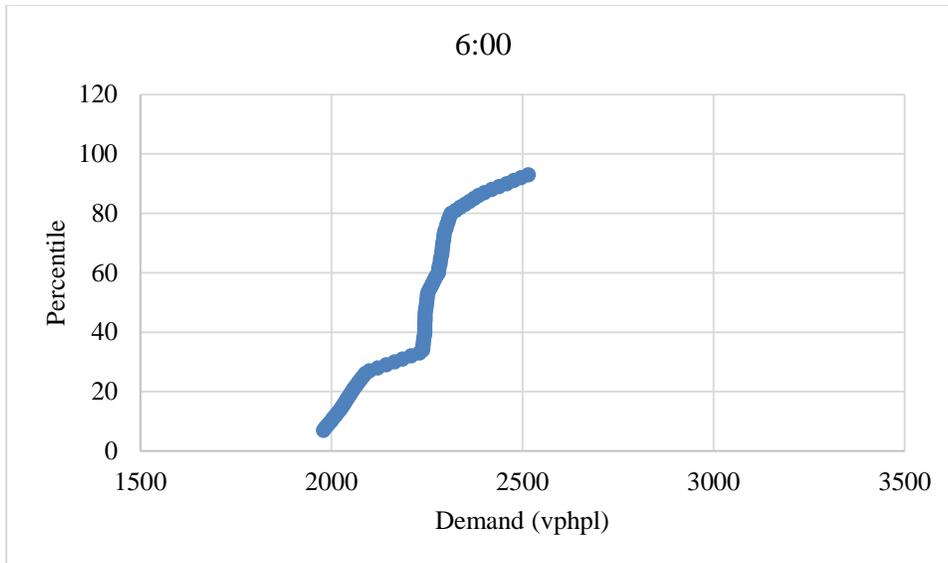


Figure A-31 Demand Distribution at 6:00 pm

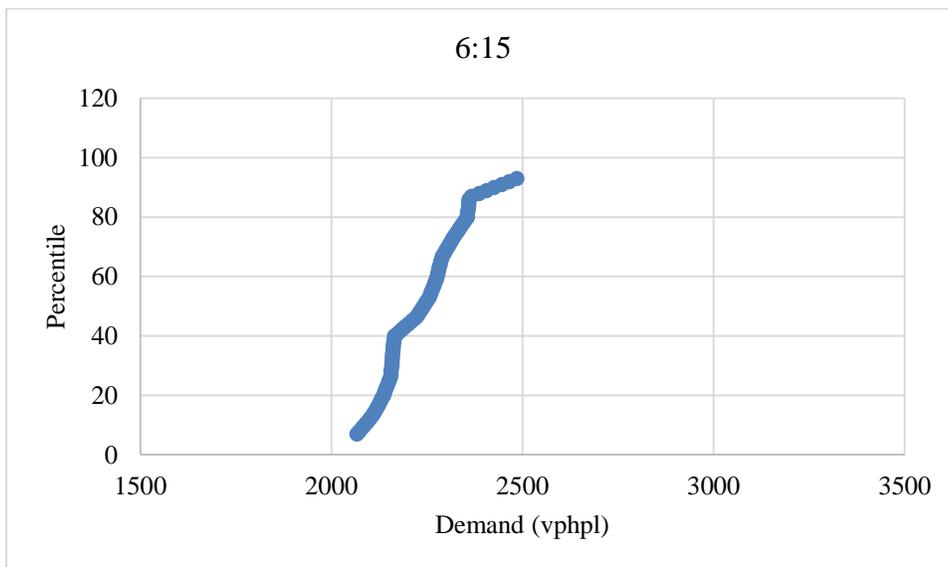


Figure A-32 Demand Distribution at 6:15 pm

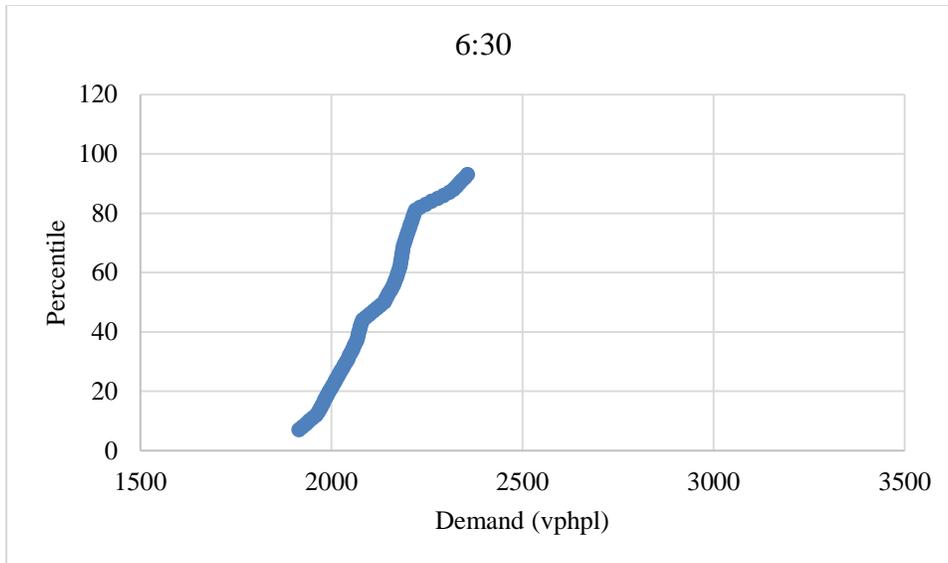


Figure A-33 Demand Distribution at 6:30 pm

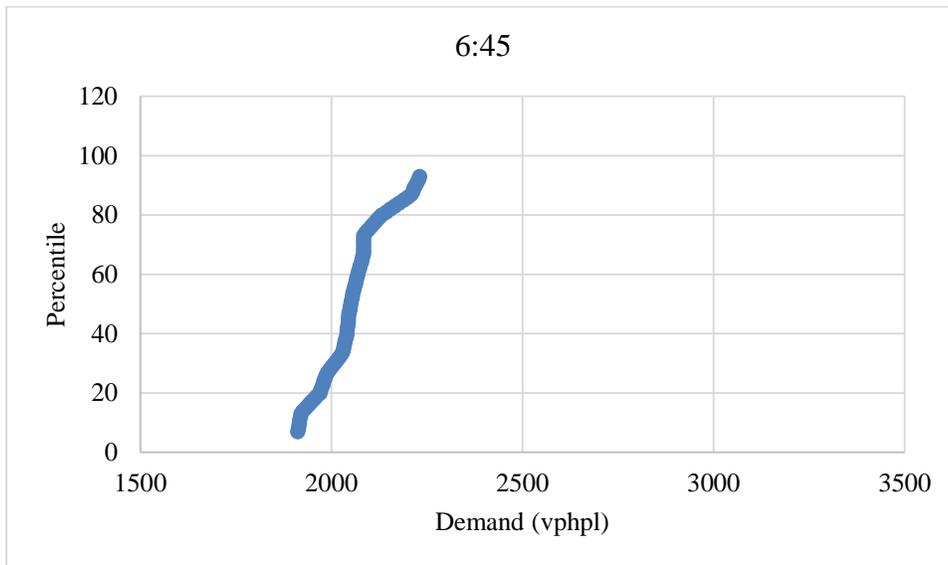


Figure A-34 Demand Distribution at 6:45 pm

APPENDIX B: INCIDENT STATISTICS FOR EACH SEGMENT WITHOUT ISOLATING THE LONG INCIDENT DURATION

Table B-1 Incident Durations from Hallandale Beach Blvd to Commercial Blvd

Peak	Main Lanes Blocked	Number of Incidents	Incident Durations (Minutes)					
			Average	Min	Max	P95 th	P80 th	P50 th
AM	0 (Shoulder Only)	133	28	1	90	79.4	48.6	19
AM	1	54	38	2	90	86.35	67.4	25.5
AM	2	15	59	12	90	83.7	78.2	65
AM	3+	4	69	41	90	89.1	86.4	73.5
PM	0 (Shoulder Only)	164	24	0	89	77.85	40	16
PM	1	79	39	2	90	85.3	65.8	33
PM	2	25	51	4	90	87.8	79.8	55
PM	3+	10	66	42	89	87.65	83.6	65.5

Table B-2 Incident Durations from Hallandale Beach Blvd to Pembroke Rd

Peak	Main Lanes Blocked	Number of Incidents	Incident Durations (Minutes)					
			Average	Min	Max	P95 th	P80 th	P50 th
AM	0 (Shoulder Only)	10	20	4	58	53.05	38.2	11.5
AM	1	1	65	65	65	65	65	65
AM	2	1	39	39	39	39	39	39
AM	3+	1	63	63	63	63	63	63
PM	0 (Shoulder Only)	7	26	10	59	52.4	34.6	24
PM	1	8	23	3	63	54.25	36.8	20.5

Table B-3 Incident Durations from Pembroke Rd to Hollywood Blvd

Peak	Main Lanes Blocked	Number of Incidents	Incident Durations (Minutes)					
			Average	Min	Max	P95 th	P80 th	P50 th
AM	0 (Shoulder Only)	4	21	6	56	50	32	12.5
AM	1	8	37	9	89	86.55	73.2	18.5
AM	3+	1	84	84	84	84	84	84
PM	0 (Shoulder Only)	11	23	2	77	64.5	32	13
PM	1	12	27	10	65	58.4	37.6	22.5
PM	2	1	55	55	55	55	55	55
PM	3+	3	72	50	86	85.4	83.6	80

Table B-4 Incident Durations from Hollywood Blvd to Sheridan St

Peak	Main Lanes Blocked	Number of Incidents	Incident Durations (Minutes)					
			Average	Min	Max	P95 th	P80 th	P50 th
AM	0 (Shoulder Only)	12	35	1	90	86.7	73.6	24.5
AM	1	10	30	5	71	68.3	49	20.5
AM	2	7	63	12	90	87.3	80.6	69
AM	3+	1	41	41	41	41	41	41
PM	0 (Shoulder Only)	15	18	3	60	41.1	27.2	16
PM	1	9	36	10	60	59.2	50.2	39
PM	2	2	81	73	90	89.15	86.6	81.5

Table B-5 Incident Durations from Sheridan St to Stirling Rd

Peak	Main Lanes Blocked	Number of Incidents	Incident Durations (Minutes)					
			Average	Min	Max	P95 th	P80 th	P50 th
AM	0 (Shoulder Only)	7	26	3	64	56.2	35.4	23
AM	1	8	35	16	82	74.65	48.2	25.5
AM	2	1	72	72	72	72	72	72
PM	0 (Shoulder Only)	19	27	2	88	79	47	21
PM	1	5	42	5	81	74.8	56.2	46
PM	2	2	13	4	22	21.1	18.4	13
PM	3+	2	48	42	54	53.4	51.6	48

Table B-6 Incident Durations from Stirling Rd to Griffin Rd

Peak	Main Lanes Blocked	Number of Incidents	Incident Durations (Minutes)					
			Average	Min	Max	P95 th	P80 th	P50 th
AM	0 (Shoulder Only)	12	22	4	41	41	37.4	20.5
AM	1	2	18	16	20	19.8	19.2	18
AM	2	2	31	27	35	34.6	33.4	31
AM	3+	1	90	90	90	90	90	90
PM	0 (Shoulder Only)	10	37	5	86	79.7	67.2	31
PM	1	12	40	10	78	75.25	68.2	31.5
PM	2	4	45	23	72	69.75	63	43.5
PM	3+	1	63	63	63	63	63	63

Table B-7 Incident Durations from Griffin Rd to Marina Mile Blvd

Peak	Main Lanes Blocked	Number of Incidents	Incident Durations (Minutes)					
			Average	Min	Max	P95 th	P80 th	P50 th
AM	0 (Shoulder Only)	17	27	3	70	66.8	45	23
AM	1	1	50	50	50	50	50	50
PM	0 (Shoulder Only)	25	19	1	55	43.2	30.8	13
PM	1	4	31	2	85	76	49	20
PM	2	4	69	48	88	86.05	80.2	71

Table B-8 Incident Durations from Marina Mile Blvd to Davie Blvd

Peak	Main Lanes Blocked	Number of Incidents	Incident Durations (Minutes)					
			Average	Min	Max	P95 th	P80 th	P50 th
AM	0 (Shoulder Only)	11	31	8	72	66.5	49	24
AM	1	3	31	2	87	78.9	54.6	6
AM	2	2	64	62	66	65.8	65.2	64
PM	0 (Shoulder Only)	17	38	0	86	84.4	78.4	37
PM	1	3	34	12	67	62.6	49.4	23
PM	2	1	43	43	43	43	43	43

Table B-9 Incident Durations from Davie Blvd to W Broward Blvd

Peak	Main Lanes Blocked	Number of Incidents	Incident Durations (Minutes)					
			Average	Min	Max	P95 th	P80 th	P50 th
AM	0 (Shoulder Only)	7	16	2	45	39	23.4	11
AM	1	4	70	23	90	89.4	87.6	83.5
PM	0 (Shoulder Only)	9	20	5	69	52.6	23.8	12
PM	1	1	61	61	61	61	61	61
PM	3+	1	83	83	83	83	83	83

Table B-10 Incident Durations from W Broward Blvd to W Sunrise Blvd

Peak	Main Lanes Blocked	Number of Incidents	Incident Durations (Minutes)					
			Average	Min	Max	P95 th	P80 th	P50 th
AM	0 (Shoulder Only)	19	29	7	77	66.2	57	18
AM	1	6	30	6	78	69.5	44	23
PM	0 (Shoulder Only)	7	19	1	42	39.3	30.8	19
PM	1	7	50	3	88	87.1	81.8	59
PM	2	3	40	13	57	56.3	54.2	50
PM	3+	1	89	89	89	89	89	89

Table B-11 Incident Durations from W Sunrise Blvd to W Oakland Park Blvd

Peak	Main Lanes Blocked	Number of Incidents	Incident Durations (Minutes)					
			Average	Min	Max	P95 th	P80 th	P50 th
AM	0 (Shoulder Only)	25	30	2	88	82.2	59.4	16
AM	1	7	32	2	63	60.3	53.2	23
AM	2	2	71	65	78	77.35	75.4	71.5
PM	0 (Shoulder Only)	23	22	1	86	63.8	43.6	13
PM	1	12	48	8	90	90	75.8	48.5
PM	2	5	62	14	87	86.4	84.6	83
PM	3+	2	60	53	68	67.25	65	60.5

Table B-12 Incident Durations from W Oakland Park Blvd to Commercial Blvd

Peak	Main Lanes Blocked	Number of Incidents	Incident Durations (Minutes)					
			Average	Min	Max	P95 th	P80 th	P50 th
AM	0 (Shoulder Only)	9	41	8	86	85.6	67	34
AM	1	4	68	44	86	85.1	82.4	71
PM	0 (Shoulder Only)	21	23	2	89	63	35	13
PM	1	6	60	36	89	84.75	72	58.5
PM	2	3	34	10	79	72.6	53.4	15

APPENDIX C: INCIDENT STATISTICS FOR EACH SEGMENT AFTER FILTERING OUT THE LONG INCIDENT DURATION

Table C-1 Incident Durations from Hallandale Beach Blvd to Commercial Blvd

Peak	Main Lanes Blocked	Number of Incidents	Incident Durations (Minutes)					
			Average	Min	Max	P95 th	P80 th	P50 th
AM	0 (Shoulder Only)	133	28	1	90	79.4	48.6	19
AM	1	54	38	2	90	86.35	67.4	25.5
AM	2	15	59	12	90	83.7	78.2	65
AM	3+	4	69	41	90	89.1	86.4	73.5
PM	0 (Shoulder Only)	164	24	0	89	77.85	40	16
PM	1	79	39	2	90	85.3	65.8	33
PM	2	25	51	4	90	87.8	79.8	55
PM	3+	10	66	42	89	87.65	83.6	65.5

Table C-2 Incident Durations from Hallandale Beach Blvd to Pembroke Rd

Peak	Main Lanes Blocked	Number of Incidents	Incident Durations (Minutes)					
			Average	Min	Max	P95 th	P80 th	P50 th
AM	0 (Shoulder Only)	10	20	4	58	53.05	38.2	11.5
AM	1	1	65	65	65	65	65	65
AM	2	1	39	39	39	39	39	39
AM	3+	1	63	63	63	63	63	63
PM	0 (Shoulder Only)	7	26	10	59	52.4	34.6	24
PM	1	8	23	3	63	54.25	36.8	20.5

Table C-3 Incident Durations from Pembroke Rd to Hollywood Blvd

Peak	Main Lanes Blocked	Number of Incidents	Incident Durations (Minutes)					
			Average	Min	Max	P95 th	P80 th	P50 th
AM	0 (Shoulder Only)	4	21	6	56	50	32	12.5
AM	1	8	37	9	89	86.55	73.2	18.5
AM	3+	1	84	84	84	84	84	84
PM	0 (Shoulder Only)	11	23	2	77	64.5	32	13
PM	1	12	27	10	65	58.4	37.6	22.5
PM	2	1	55	55	55	55	55	55
PM	3+	3	72	50	86	85.4	83.6	80

Table C-4 Incident Durations from Hollywood Blvd to Sheridan St

Peak	Main Lanes Blocked	Number of Incidents	Incident Durations (Minutes)					
			Average	Min	Max	P95 th	P80 th	P50 th
AM	0 (Shoulder Only)	12	35	1	90	86.7	73.6	24.5
AM	1	10	30	5	71	68.3	49	20.5
AM	2	7	63	12	90	87.3	80.6	69
AM	3+	1	41	41	41	41	41	41
PM	0 (Shoulder Only)	15	18	3	60	41.1	27.2	16
PM	1	9	36	10	60	59.2	50.2	39
PM	2	2	81	73	90	89.15	86.6	81.5

Table C-5 Incident Durations from Sheridan St to Stirling Rd

Peak	Main Lanes Blocked	Number of Incidents	Incident Durations (Minutes)					
			Average	Min	Max	P95 th	P80 th	P50 th
AM	0 (Shoulder Only)	7	26	3	64	56.2	35.4	23
AM	1	8	35	16	82	74.65	48.2	25.5
AM	2	1	72	72	72	72	72	72
PM	0 (Shoulder Only)	19	27	2	88	79	47	21
PM	1	5	42	5	81	74.8	56.2	46
PM	2	2	13	4	22	21.1	18.4	13
PM	3+	2	48	42	54	53.4	51.6	48

Table C-6 Incident Durations from Stirling Rd to Griffin Rd

Peak	Main Lanes Blocked	Number of Incidents	Incident Durations (Minutes)					
			Average	Min	Max	P95 th	P80 th	P50 th
AM	0 (Shoulder Only)	12	22	4	41	41	37.4	20.5
AM	1	2	18	16	20	19.8	19.2	18
AM	2	2	31	27	35	34.6	33.4	31
AM	3+	1	90	90	90	90	90	90
PM	0 (Shoulder Only)	10	37	5	86	79.7	67.2	31
PM	1	12	40	10	78	75.25	68.2	31.5
PM	2	4	45	23	72	69.75	63	43.5
PM	3+	1	63	63	63	63	63	63

Table C-7 Incident Durations from Griffin Rd to Marina Mile Blvd

Peak	Main Lanes Blocked	Number of Incidents	Incident Durations (Minutes)					
			Average	Min	Max	P95 th	P80 th	P50 th
AM	0 (Shoulder Only)	17	27	3	70	66.8	45	23
AM	1	1	50	50	50	50	50	50
PM	0 (Shoulder Only)	25	19	1	55	43.2	30.8	13
PM	1	4	31	2	85	76	49	20
PM	2	4	69	48	88	86.05	80.2	71

Table C-8 Incident Durations from Marina Mile Blvd to Davie Blvd

Peak	Main Lanes Blocked	Number of Incidents	Incident Durations (Minutes)					
			Average	Min	Max	P95 th	P80 th	P50 th
AM	0 (Shoulder Only)	11	31	8	72	66.5	49	24
AM	1	3	31	2	87	78.9	54.6	6
AM	2	2	64	62	66	65.8	65.2	64
PM	0 (Shoulder Only)	17	38	0	86	84.4	78.4	37
PM	1	3	34	12	67	62.6	49.4	23
PM	2	1	43	43	43	43	43	43

Table C-9 Incident Durations from Davie Blvd to W Broward Blvd

Peak	Main Lanes Blocked	Number of Incidents	Incident Durations (Minutes)					
			Average	Min	Max	P95 th	P80 th	P50 th
AM	0 (Shoulder Only)	7	16	2	45	39	23.4	11
AM	1	4	70	23	90	89.4	87.6	83.5
PM	0 (Shoulder Only)	9	20	5	69	52.6	23.8	12
PM	1	1	61	61	61	61	61	61
PM	3+	1	83	83	83	83	83	83

Table C-10 Incident Durations from W Broward Blvd to W Sunrise Blvd

Peak	Main Lanes Blocked	Number of Incidents	Incident Durations (Minutes)					
			Average	Min	Max	P95 th	P80 th	P50 th
AM	0 (Shoulder Only)	19	29	7	77	66.2	57	18
AM	1	6	30	6	78	69.5	44	23
PM	0 (Shoulder Only)	7	19	1	42	39.3	30.8	19
PM	1	7	50	3	88	87.1	81.8	59
PM	2	3	40	13	57	56.3	54.2	50
PM	3+	1	89	89	89	89	89	89

Table C-11 Incident Durations from W Sunrise Blvd to W Oakland Park Blvd

Peak	Main Lanes Blocked	Number of Incidents	Incident Durations (Minutes)					
			Average	Min	Max	P95 th	P80 th	P50 th
AM	0 (Shoulder Only)	25	30	2	88	82.2	59.4	16
AM	1	7	32	2	63	60.3	53.2	23
AM	2	2	71	65	78	77.35	75.4	71.5
PM	0 (Shoulder Only)	23	22	1	86	63.8	43.6	13
PM	1	12	48	8	90	90	75.8	48.5
PM	2	5	62	14	87	86.4	84.6	83
PM	3+	2	60	53	68	67.25	65	60.5

Table C-12 Incident Durations from W Oakland Park Blvd to Commercial Blvd

Peak	Main Lanes Blocked	Number of Incidents	Incident Durations (Minutes)					
			Average	Min	Max	P95 th	P80 th	P50 th
AM	0 (Shoulder Only)	9	41	8	86	85.6	67	34
AM	1	4	68	44	86	85.1	82.4	71
PM	0 (Shoulder Only)	21	23	2	89	63	35	13
PM	1	6	60	36	89	84.75	72	58.5
PM	2	3	34	10	79	72.6	53.4	15