Evaluation of Ramp Meter Effectiveness for Wisconsin Freeways, A Milwaukee Case Study

Part 1: Diversion and Simulation
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Part 2: Ramp Metering Effect on Traffic Operations and Crashes
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16. Abstract

The purpose of the research is to determine the benefits of ramp meters in the Milwaukee area freeway system, to determine underlying relationships that permit evaluation of new ramp meters or ramp meter systems elsewhere, and to develop a coherent framework for performing evaluation of ramp meter effectiveness on a whole system. Part 1 concentrates on diversion and traffic simulation. In order to assure that the conclusions are relevant to Wisconsin drivers and conditions on Wisconsin freeways, the research focused on data collected from the US 45 corridor, mostly in Milwaukee County, from before and after the deployment of seven new ramp meters in the southbound direction in early March 2000. This corridor spanned about 15 miles and included the freeway itself and two parallel arterials. Statistically significant diversions away from US 45 occurred at times and places where traffic volumes were heaviest and ramp queues were longest. The data also revealed that there was diversion between on-ramps along US 45 in response to queuing at ramps. The data did not reveal any temporal diversions due to the meters. This study investigated three types of simulation software packages: microscopic (Paramics), mesoscopic (Dynasmart-P) and macroscopic (QRS II). US 45 was simulated with two packages (Paramics and QRS II) within which a reasonable representation of a meter could be achieved. The most convincing simulations of US 45 (without parallel arterials) was created with Paramics. There were two simulations, one for the "before" period and one for the "after" period, both of which matched mainline speeds closely and behaved realistically at the meters.

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16. Abstract

The purpose of the research is to determine the benefits of ramp meters in the Milwaukee area freeway system, to determine underlying relationships that permit evaluation of new ramp meters or ramp meter systems elsewhere, and to develop a coherent framework for performing evaluation of ramp meter effectiveness on a whole system. Part 2 concentrates on the traffic operations affect six new ramp meters had on the 14-mile long corridor where six ramp meters were already operational. A crash rate comparison was performed between the periods the corridor operated without and with the six new ramp meters. Metered on-ramp queue length and delay information is presented in Appendix A; details of the operation of a metered on-ramp as well as mainline speed occupancy and volume information in the vicinity of the ramp are presented in Appendix B.

Average corridor speeds improved when the new ramp meters were operational. Vehicle-hours of travel were lower during the more congested afternoon peak period. It is suggested that fine-tuning of ramp metering parameters is very likely to result in additional benefits for the corridor.

Crash rates during ramp metering hours were lower by 13% with the new ramp meters operational.

17. Key Words Ramp meter, freeway, measures of effectiveness, speed, vehicle hours of travel, vehicle miles of travel, average corridor speed, crash rates, ramp delay, mainline delay, ramp queue length.	18. Distribution Statement No restriction. This document is available to the public through the National Technical Information Service 5285 Port Royal Road Springfield VA 22161

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Evaluation of Ramp Meter Effectiveness for Wisconsin Freeways, A Milwaukee Case Study: Part 1, Diversion and Simulation

Combined Executive Summary for Part 1 and Part 2

Overview

This report is one of two companion reports that identify methods for evaluating the effectiveness of ramp meters on Wisconsin freeways. The other report, "Evaluation of Ramp Meter Effectiveness for Wisconsin Freeways, A Milwaukee Case Study: Part 2, Travel Times, Speeds and Collisions" has been prepared by Marquette University. This Executive Summary is a description of the procedures, conclusions and recommendations from both reports.

The purpose of the research is to determine the benefits of ramp meters in the Milwaukee area freeway system, to determine underlying relationships that permit evaluation of new ramp meters or ramp meter systems elsewhere, and to develop a coherent framework for performing evaluation of ramp meter effectiveness on a whole system.

A better understanding of ramp meter operation can lead to more optimal placement of new ramp meters and optimal design of each new ramp meter (including its geometry, HOV lane, storage capacity, upstream signalization strategies, timing plan, and ITS strategies), and optimized coordination between ramp meters. Toward this end, the project also identified and tested simulation software packages for modeling ramp meter operations at a system level.

The project focused on the three most critical areas of ramp meter system effectiveness: collision reduction, congestion reduction, and traffic diversion behavior and related effects. A systems viewpoint is critical. The presence of the meter at a freeway on-ramp has implications for this particular ramp, any upstream traffic controls, the downstream freeway mainline and the parallel arterials. The geometry and operational characteristics of the meter influence delay (as one measure of congestion), collisions, and other measures of effectiveness (MOEs) for the system as a whole. This project also recognizes that a single ramp meter influences and is influenced by traffic conditions considerably upstream and downstream of its own location and that coordination of the meter with other meters needs to be investigated. Traffic diversion can only be analyzed in the context of a system, as this behavior depends upon the operation of the meter and the freeway, the conditions on parallel arterials, driver experience, and information provided to the driver just ahead of the diversion decision.

Case Study: US 45 in Milwaukee

In order to assure that the conclusions are relevant to Wisconsin drivers and conditions on Wisconsin freeways, the research focused on data collected from the US 45 corridor, mostly in Milwaukee County, from before and after the deployment of seven new ramp meters in the southbound direction in early March 2000. This corridor spanned about 15 miles and included the freeway itself and two parallel arterials, Highway 100 and 124th Street. There were 14 on-ramps to US 45 (southbound) within the corridor, of which 13 were eventually metered.

A comprehensive data collection, coordinating the efforts of Marquette University, the University of Wisconsin – Milwaukee and the Wisconsin Department of Transportation, resulted in precise snap-shots of the traffic conditions before and after deployment. Data related to traffic flow were collected on six weekdays (Tuesdays, Wednesdays and Thursdays) before deployment and six weekdays after deployment for 1 ½ hours in each of the morning and evening peaks. Data covered these items.

- *Travel Times.* Floating car runs were made continuously during the 1 ½ hour peak periods on southbound US 45, Highway 100 and 124th Street. Travel times were recorded for segments along US 45 of about 1 mile in length every 15 minutes. Segment lengths on Highway 100 and 124th Street average a little more than ½ mile, also at 15 minute intervals.
- *Detector and Tube Counts.* Count data from all loop detectors on US 45 were assembled. These loop detectors included mainline detectors for each metered ramp and other mainline detectors between ramps. In addition, counts were obtained from detectors located on the ramp (both at the top of the ramp, the "queue" detector, and at the stopline, the "passage" detector). Road tubes were deployed at all locations that could not be adequately counted by loop detectors: the southbound lanes of Highway 100 and 124th Street; all off-ramps; and all on-ramps without meters in the southbound direction of US 45.
- *Origin-Destination Tables.* Two tables giving the split of vehicle to each off-ramp from each on-ramp were obtained by video logging of license plates. The data for these tables were collected in 1999 and 2001, well before and well after the ramp meter deployment.
- *Detector Speeds.* Mainline detectors were arrayed in pairs so that speeds could be obtained continuously at many points along US 45. Because of the possibility of detector error, these speeds were verified against the floating car runs.
- *Queue Length Counts.* Queue lengths were hand counted at all metered on-ramps in the southbound direction of US 45.
- *Collision History.* Collision records for US 45 were obtained from the Milwaukee County Sheriff's office for 3 years before and 2 years after the ramp meter deployment.

In addition, a survey of Wisconsin drivers administered by the University of Wisconsin—Madison provided insights into drivers' route choices in reaction to ramp meter deployment.

Findings

Diversion

The consensus of the literature is that the ability to divert selected traffic from the freeway mainline is essential in achieving positive benefits of ramp meters. Thus, it is important to understand the diversion propensity of Wisconsin drivers when faced with the need to wait for vehicles ahead of them at a ramp meter. There are three common forms of diversion: spatial, temporal or modal. Modal diversion (such as shifts to carpools or transit) was not analyzed and there was no evidence in the collected data that temporal diversion (sometimes referred to as "peak spreading") occurred in the study corridor. Spatial diversion was ascertained by three different methods from the before and after data.

- *Origin Destination Tables.* The average trip length from the origin-destination tables increased from the before to the after periods. There was a 7% increase in the morning and a 4% increase in the afternoon. A contributing factor to this increase was a reduction in very short trips.
- *Traffic Counts.* Mainline and arterial traffic counts were compiled for each 15 minutes along eight east-west cutlines across US 45, Highway 100 and 124th Street. Traffic counts indicated that diversion occurred between the freeway and parallel arterials, although not all times and not all cutlines were impacted the same. Statistically significant diversions away from US 45 occurred at times and places where traffic volumes were heaviest and ramp queues were longest. The data also revealed that there was diversion between on-ramps along US 45 in response to queuing at ramps.
- *Questionnaire.* From the questionnaire responses from those Wisconsin drivers who said that they regularly encountered ramp meters, it was found that 72% of drivers are aware of alternate routes and 65% have a good idea of the travel time the alternate route would take. Only 24% of drivers said they would divert if the ramp was half full, but 62% said they would divert if the ramp was nearly full and 82% said they would divert if the ramp was overflowing.

Speeds and Travel Times

During the period with new ramp meters in operation the most congested south part of the analysis corridor experienced an improvement in traffic operations measures of effectiveness, during the most critical (most congested) afternoon peak period.

During the afternoon peak period, a substantial reduction in vehicle-hours of travel due to increases in travel speeds, under minimal volume changes (a zero to two percent increase) was documented between Capitol Drive and Greenfield Avenue. Speeds increased by 13% in the segment between Capitol Drive and Burleigh Street, by 10% between North Avenue and Wisconsin Avenue, and by 6% between Bluemound Road and Greenfield Avenue.

Corridor average speed increased by only four percent during the afternoon peak, because no speed changes were effected on the north part of the corridor where near-free-flow speeds existed at all times. Although mainline vehicle hours of travel decreased by five percent, when ramp delay was also taken into account, total vehicle hours of travel decreased by two percent. There was an overall increase of two percent in corridor vehicle miles of travel.

Crashes

The crash rate was 298 crashes per 100 MVM of travel "Without," and 260 crashes per 100 MVM of travel "With" the new ramp meters. Operation of the new ramp meters in conjunction with improved ramp merging geometrics and mainline pavement resurfacing resulted in an overall 13% crash rate reduction (16% reduction in the number of crashes) during ramp metering hours.

Simulation

The traffic impact of ramp meters can be simulated using a variety of methods; the choice of method would depend upon needs of the analysis. This study investigated three types of simulation software packages: microscopic (Paramics), mesoscopic (Dynasmart-P) and macroscopic (QRS II). US 45 was simulated with two packages (Paramics and QRS II) within which a reasonable representation of a meter could be achieved. A simulation has the advantage of isolating only those effects that need to be analyzed. Thus, the impact of a new ramp meter could be investigated without worrying about weather, incidents, fluctuations in traffic volumes or any other confounding influences.

The most convincing simulations of US 45 (without parallel arterials) was created with Paramics. There were two simulations, one for the "before" period and one for the "after" period, both of which matched mainline speeds closely and behaved realistically at the meters. These models were given a minimum of calibration to obtain a relatively "hands-free" evaluation of the performance of the software. Model parameters were adopted verbatim from a study of I-5 in Irvine, California. The largest impediments to implementing Paramics on US 45 were the complexity of the software, the inadequate documentation and the need to adapt standard features of the software to match the peculiarities of US 45. For example, the project team encountered problems getting Paramics to represent alternative release of vehicles from two-lane approaches to meters and to represent the split of traffic to HOV lanes.

A comparison of the two simulations for US 45 indicated that traffic flowed better after the new meters were added. Both delays at meters and along the mainline were considered in this assessment. However, these simulations did not contain creation of platoons at upstream signalized intersections, so the benefits associated with platoon dispersion by meters could not be assessed. The traffic impacts on adjacent arterials were not considered.

Conclusions

The analysis and simulation of US 45 lead to the following major conclusions.

- Drivers react to recurrent delays at ramp meters and along freeway mainlines when choosing between alternate routes. When faced with a long queue at an on-ramp, some drivers divert to another on-ramp while some others avoid the freeway entirely. The US 45 experience suggests that average trip length on the freeway increases when meters are deployed, thereby resulting in less entering or exiting for a given level of traffic on the mainline.
- During the period with new ramp meters in operation the most congested south part of the analysis corridor experienced an improvement in traffic operations measures of effectiveness, during the most critical (most congested) afternoon peak period: a substantial reduction in vehicle-hours of travel and increases in travel speeds, under minimal volume changes (a zero to two percent increase) was documented between Capitol Drive and Greenfield Avenue. Speeds increased by 13% in the segment between Capitol Drive and Burleigh Street, by 10% between North Avenue and Wisconsin Avenue, and by 6% between Bluemound Road and Greenfield Avenue. Corridor average speed increased by four percent during the afternoon peak.

- New ramp meter operation, in conjunction with geometric improvements in ramp merging areas and mainline resurfacing resulted in a 21% crash rate reduction for the analyzed corridor during ramp metering hours.
- It is possible to develop high quality mathematical models for assessing of the impact of ramp meters with a suitable microscopic traffic simulation software package. The software package currently used by WisDOT to simulate the Milwaukee freeway system, Paramics, is a good choice.
- The simulations of before and after conditions on US 45 indicated that traffic on the freeway flowed better after the meters were deployed.

Recommendations

Because an unwarranted ramp meter can cause delay on the ramp without achieving sufficient offsetting travel time savings on the mainline, the deployment of ramp meters should proceed cautiously. The deployment decision should be based on a careful engineering study that includes collection and analysis of speed and volume data and an assessment of impact on mainline speeds, arterial speeds, entrance and exit volumes and ramp queuing. A simulation model, such as the ones developed in this study with Paramics, should be used to assist this evaluation.

Ramp meters can have significant effects on traffic flow within a freeway mainline. Any simulation model of a freeway in Wisconsin should include ramp meters, if present. Any future forecasts of traffic volumes on freeways should consider the possible diversion effects of ramp meters.

The current ramp meter timing algorithm used in Milwaukee has not had a thorough review. WisDOT should undertake such a review to determine whether it still fully satisfies the objectives of freeway operation. Other algorithms should be considered that have the potential to more effectively optimize traffic flow downstream from the meter. The current algorithm deals with each ramp meter in isolation from other ramp meters. WisDOT should consider means by which several adjacent ramp meters could be jointly timed to provide better traffic flow. It is important that any software for ramp meter evaluation be capable of correctly representing the timing algorithm.

The Paramics models for US 45, both before and after ramp metering, that were developed for this study should be used to investigate the effects of freeway operational policies that are not location specific. The models should be upgraded to increase the realism of the simulations. These Paramics models did not invoke Paramics' ability to divert traffic to alternate routes; such capability should be added. In addition, the Paramics models did not contain traffic signals upstream from the ramps that would create platoons of vehicles, which would have a serious impact on mainline traffic flow, if not metered.

Fine-tuning of ramp metering parameters during the morning peak period in order to reduce ramp delays is very likely to produce a reduction in total freeway veh-hr of travel. Further reductions in total freeway veh-hr of travel during the afternoon peak may also be possible by reducing ramp delay on the existing Good Hope Road loop ramp where the mainline is not very congested; the current high level of ramp delay on the new Burleigh Street ramp could probably also be reduced. County Line Road and Pilgrim Road ramp metering probably contributes rather small mainline benefits at the present time, given the lower traffic volumes and substantial distance from the currently congested part of the corridor. Minimizing delays on these ramps would, in all likelihood decrease corridor delays.

Any changes in ramp metering parameters aiming to reduce ramp delays, should be carefully balanced against possible increases in mainline travel times.

Evaluation of Ramp Meter Effectiveness for Wisconsin Freeways, A Milwaukee Case Study: Part 1, Diversion and Simulation

Project identification number 0092-45-17

Final Report

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Chapter 1
Ramp Metering and Diversion:
A Review of Literature

Introduction

Ramp metering is the most widely used method of freeway traffic control. Ramp meters are traffic signals on freeway entrance ramps that limit the rate at which vehicles can enter the freeway so that demand does not exceed capacity.

There are three types of application of ramp metering: freeway entrance ramp metering, freeway-to-freeway connector metering, and freeway mainline metering. The simplest form of control is a fixed-time operation. The next level of control is a traffic responsive operation, which establishes metering rates based on actual freeway conditions. The third level is system-wide control, which is also a form of traffic responsive control but operates on the basis of conditions on the whole freeway system.

Piotrowicz and Robinson (1995) provide a general update on the status of ramp metering in the US. Ramp metering has been applied since the 1960's in the Chicago, Detroit, and Los Angeles areas. The success of these early applications contributed to the expansion of ramp metering systems to 22 additional metropolitan areas in the US by the early 1990's.

However, inappropriate use of ramp meters can produce negative benefits. A major issue that is raised in connection with ramp metering is the potential diversion of freeway trips to adjacent surface streets to avoid queues at the meters. Studies of the impact of ramp metering on parallel arterials have been conducted in Los Angeles, Denver, Seattle, Detroit and other cities. No significant diversion from the freeway to parallel arterials occurred in any of these locations (Piotrowicz and Robinson, 1995).

Ramp Metering Diversion Effects

Significant diversion from metered ramps is required in order to improve the overall network performance by ramp metering. Diversion is essential to achieve positive benefits. While diversion is an important metering concern, empirical results suggest no more than 5-10% of vehicles will be diverted when ramp meters are turned on (Kang and Gillen, 1999).

Field Study in Twin Cities, Minnesota

The Minnesota Department of Transportation (MN/DOT, 2001) conducted a study of the effectiveness of ramp meters in selected corridor segments in the fall of 2000 by turning off all ramp meters in the Twin Cities metropolitan area. It was found that with the meters off, the peak period traffic volume was reduced by about nine percent for all study corridors, or approximately 1,200 vehicles per corridor. An average *decrease* of 56 vehicles per studied parallel arterial was observed with the meters off. In the absence of metering, there was an increase of freeway point-to-point travel time of 22 percent during the peak period on the tested corridor segments, which averaged about nine miles in length and about 12 minutes of travel time.

The study concluded that there was some diversion to other time periods (Figure 1-1) or different ramp entrances, and no significant diversion to different routes or other transportation modes. However, Figure 1-1 appears inconclusive as to whether there was peak spreading or simply a suppression of the peak between 3 PM and 6 PM

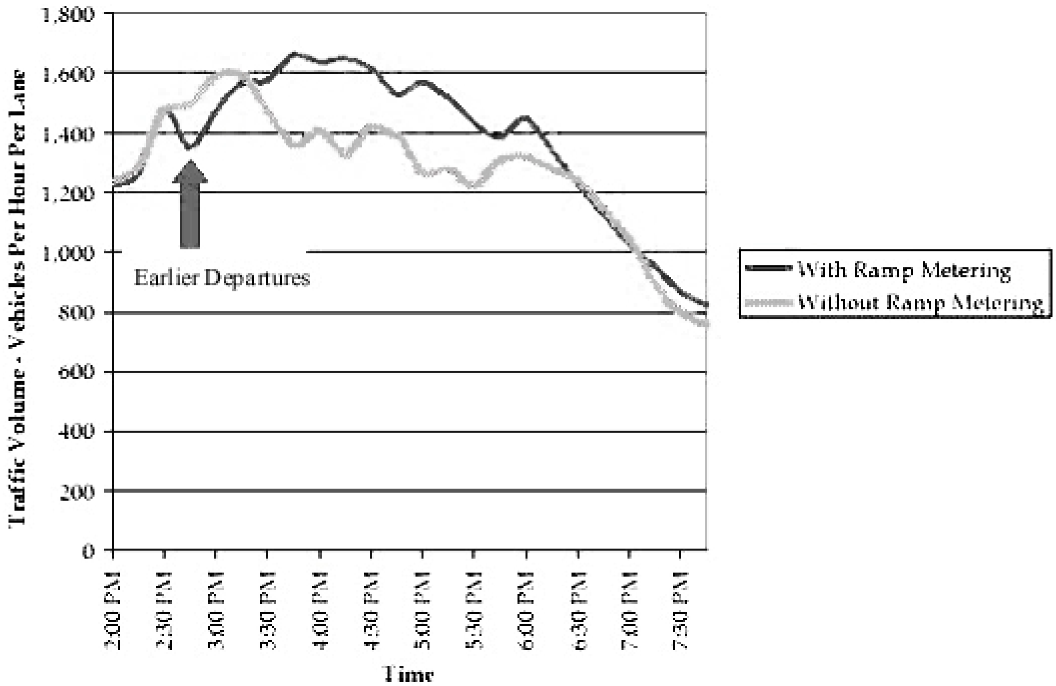


Figure 1-1. I-94 EB Afternoon Volume Spread (MN/DOT, 2001)

Through random and corridor surveys when the meters were on, it was found that about 70 percent of travelers would use alternate routes to avoid waiting at ramp meters, more than 75 percent of travelers would leave at a different time of day to avoid congestion, and over 75 percent travelers would use a different ramp entrance to avoid back-ups (Table 1-1 and Table 1-2). These percentages suggest that diversion should have been a very significant effect of turning off the meters. However, significant diversion cannot be concluded from the traffic data available within this study. MN/DOT did not monitor all possible diversion routes and therefore could have missed a large amount of diverted traffic.

Table 1-1. Diversion Patterns in the "With Ramp Meters" Surveys (MN/DOT, 2001)

	Random Sample	I-494 Corridor	I-35E Corridor	I-35W Corridor	I-94 Corridor
Route Diversion					
Sometimes use alternate routes to avoid waiting at ramp meters	68.8%	71.4%	72.0%	72.0%	71.0%
No	31.2%	28.6%	28.0%	28.0%	29.0%
Time-of-Day Diversion					
Sometimes leave earlier or later to avoid traffic congestion	78.7%	75.4%	78.4%	85.6%	74.8%
No	21.3%	24.6%	21.6%	14.4%	25.2%
Ramp Diversion					
Sometimes avoid a ramp that is backed up with traffic and use a different ramp to enter a freeway	75.1%	77.0%	76.0%	80.0%	79.4%
No	24.9%	23.0%	24.0%	20.0%	20.6%

Table 1-2. Diversion by Frequent Freeway Users in the "Without Ramp Meters" Surveys (MN/DOT, 2001)

	Random Sample	I-494 Corridor	I-35E Corridor	I-35W Corridor	I-94 Corridor
Route Diversion					
Tried other routes since the ramp meter shutdown	23.3%	45.3%	36.0%	35.7%	41.9%
Always used the same route since the ramp meter shutdown	76.7%	54.7%	64.0%	64.3%	58.1%
Time-of-Day Diversion					
Sometimes leave earlier or later to avoid traffic congestion	25.6%	40.2%	33.9%	41.7%	33.1%
Did not leave earlier or later to avoid congestion	74.4%	59.8%	66.1%	58.3%	66.9%

Field Study in Paris

Haj-Salem and Papageorgiou (1995) conducted a field study of the corridor traffic pattern and the impact of ramp metering in the southern part of the Corridor Périphérique in Paris. The Corridor Périphérique consisted of two parallel rings around the city of Paris. The two rings were connected by a number of radial roads with corresponding on-ramps and off-ramps. The impacts of application of ramp metering were the ameliorations by 8.1 percent and 6.9 percent in total travel time for the two parallel rings including the ramps. There was an increase by 20 percent in total travel time for the radial roads, which addressed only 5 percent of the overall system travel demand. Overall system travel time was reduced by 6.1 percent. The benefits of ramp metering were even higher under nonrecurrent congestion. The total travel times were reduced by 10.8 percent, 11.6 percent, and 10 percent for the system and the two parallel rings, respectively. This study is the most comprehensive analysis of the timesaving effects of ramp metering in an actual system. The travel time reductions are likely the most accurate values available.

The ramp metering strategy applied for the field test is ALINEA (Papageorgiou *et al*., 1991). ALINEA is a local ramp control algorithm that is based on a feedback principle. The basic idea is to maintain an optimal occupancy on the mainline that will maximize the throughput. The control law of ALINEA can be stated as:

$$r_k = r_{k-1} + K(O^* - O_k)$$

where r_k and r_{k-1} are on-ramp volumes at discrete time periods k and $k-1$, respectively, O_k is the measured downstream occupancy at discrete time k, O^* is a pre-set desired occupancy value (typically O^* is set equal to the critical occupancy) and K is a regulation parameter. If the measured occupancy O_k at cycle k is found to be lower than the desired occupancy O^*, the second term of the right hand side of the equation becomes positive and the ordered on-ramp volume r_k is increased as compared to its last value r_{k-1}.

ALINEA may be considered a highly efficient local ramp metering strategy according to the reported field results (Papageorgiou *et al*., 1998). The main distinguishing features of ALINEA are the following:

- Simplicity. ALINEA consists of a single equation without any switching, threshold values, and so forth.
- Transferability.
- Low implementation cost. ALINEA requires only one mainstream measurement, downstream of the ramp.
- Efficiency. ALINEA was found not to be inferior to coordinated ramp metering (METALINE) in the absence of incidents.
- Flexibility.

Field Study in Chicago

The Chicago Area Expressway Surveillance Project (Fonda, 1976) undertook a ramp metering study on the northbound Dan Ryan Expressway. It was found that the severity of congestion was reduced such that individual motorists saved up to five minutes in traversing the 3.6-mile study section. A daily average of 627 vehicle hours of expressway travel time was saved during control, while the peak-period vehicle-miles of expressway travel increased by 5 percent. Ramp metering at just four ramps did not produce enough diversion to downstream ramps and/or surface street routes to completely prevent expressway overloading from occurring in the study section, but shifted the point of initial overloading upstream. It is unclear from the report whether parallel arterials were affected.

Simulation Studies

Based on their simulation of a freeway section including two ramp junctions and a parallel arterial, Hellinga and Aerde (1995) concluded that a reduction of 12.17 percent in system travel time could be obtained under a user-optimal diversion condition and a reduction of 14.21 percent reduction under a system-optimal diversion condition. However, in the absence of the diversion of vehicles, ramp metering was shown to be an inefficient means of reducing total travel time, which was only a 0.39 percent reduction in system travel time. In their analysis, a fixed-rate time-of-day metering control was assumed. The INTEGRATION simulation model was used. In their study, a number of factors were examined to show their impacts on the benefits of ramp metering. These factors included diversion strategies, O-D demands, metering rates, initiation time of metering, and capacity drop.

It should be noted that a user-optimal traffic assignment (including diversion effects) is considered the most reasonable representation of actual traffic patterns in a congested traffic system. The results of this simulation study are consistent with the results from Haj-Salem and Papageorgiou (1995) described earlier, indicating that simulation can be a valid approach to measuring benefits of ramp metering.

Using the INTRAS simulation model, Nsour *et al*. (1992) evaluated the effects of ramp metering and traffic diversion on a system's performance for a seven-mile long corridor comprising a freeway, two parallel arterials and seven connecting arterials. It was concluded that significant diversions from metered ramps were required in order to improve the overall network performance by ramp metering. There was a 10.5 percent reduction in system delay and a 4.1 percent increase in average speed under ideal metering and diversion conditions. While a more restrictive ramp metering strategy significantly improved freeway flow, it adversely affected the overall system performance because overflowing queues behind meters blocked street traffic, creating a severe disturbance on feeder streets. A less restrictive ramp metering strategy was reported to be insufficient to bring the congested freeway to its normal condition during the simulated time period.

Three levels of ramp metering were designed for simulation (Nsour *et al*., 1992):

- *Meter Level I: restrictive metering: The restrictive ramp metering plan was designed to reduce the demand at the incident site to the observed capacity at the incident site.*
- *Meter Level II: more restrictive metering: The total hourly demand on the incident link would be further reduced by 400 vehicles compared with the demand in the Level I restrictive metering. It is the best strategy for alleviating the effects of the incident.*
- *Meter Level III: less restrictive metering: This strategy was based on metering rates that did not result in any overflow queues from the ramp meters. It is the best strategy considering metering only without diversion. It is the least effective for alleviating congestion on the freeway.*

The findings of this study are highly consistent with the previously mentioned study (Hellinga and Aide, 1995), in spite of the differences in simulation methodology. Both studies cap the travel times saving at about 10 percent when diversion is allowed and the metering is optimized for the traffic situation.

A series of simulations, using KRONOS and INTEGRATION, and a two-week experiment of ramp closure experiment were conducted for Honolulu's H-1 freeway, which was one of the busiest and most congested 6-lane freeways in the nation (Prevedouros, 1998). KRONOS was used for freeway simulation and INTEGRATION was applied for detailed network assessment. Simulations showed that diversions eliminated some merging activity and was very beneficial to the city streets that fed the Lunalilo on-ramp. The arterial streets were wide and offered reentry onto the freeway. In the actual ramp closure experiment, it was found that travel times on routes feeding the Lunalilo on-ramp decreased by about 2 minutes. Travel times for paths requiring reentry to the freeway increased by 2 to 4 minutes, depending on the specific destination and the reentry ramp chosen.

Another simulation study (Hasan, 1999) evaluated two ramp control algorithms: a local control algorithm (ALINEA, described earlier) and a coordinated algorithm (FLOW) using the MITSIM microscopic traffic simulator on a network including a part of the Central Artery/Tunnel (CA/T) Project in Boston. It should be noted that ALINEA was the method of control in the Paris study (Haj-Salem and Papageorgiou, 1995). The system travel time was significantly ***increased*** by ramp metering for a large number of scenarios, especially at low demand levels. The performance of ALINEA was satisfactory when there was not a bottleneck downstream of the metered ramps. FLOW outperformed ALINEA under a downstream bottleneck scenario. The improvements of total travel time in FLOW were higher than those in ALINEA when demand was high (110% and 120% of the base demand). The study concluded that for ramp queues occupying 75% of the physical length of the on-ramp, both algorithms' performance was better than that for ramp queues occupying the entire length of the on-ramp.

This study indicated the superiority of system-wide optimization of ramp meter control. It also illustrated the possibility of negative benefits of ramp metering if not applied appropriately.

A Methodology Study

Stephanedes *et al*. (1989) proposed a utility-based approach for the dynamic diversion problem, which, when combined with an appropriate filter, will more realistically model the commuter diversion process for simulation, control, and guidance-navigation in congested freeway corridors. It was assumed for this particular study that diversion occurs at two points: the trip origin and the entrance to the freeway ramp.

From a survey of approximately 1,100 commuters in the south I-35W corridor of the Twin Cities metropolitan area, two logit specifications of the diversion choice were estimated. The data indicated that diversion at the origin is a function of trip time, route length, and the number of intersections along the trip. However, trip time is the dominant determining factor and can be employed to estimate the decision in the absence of additional information. Diversion at freeway entrance ramps depends on the perceived trip time on the freeway and arterial and the perceived waiting time at the ramp queue. Further, the data confirmed that socioeconomic indicators do not play a role in the diversion decision. It was also determined that for commuter trips shorter than one hour, freeway drivers consider only one diversion alternative, a preferred arterial, and do not divert to downstream ramps.

According to Stephanedes and Kwon (1993), drivers approaching the freeway have two major opportunities for diversion before entering the freeway through the ramp. The first diversion opportunity occurs at the intersection directly upstream from the ramp. The second diversion opportunity occurs at a frontage road, which might be used by drivers when the entrance-ramp traffic conditions deteriorate. Diversion is primarily influenced by the perceived traffic condition of the ramp, which includes two major factors: the size of the queue on the ramp and the speed of queue reduction.

This study developed behavioral demand-diversion models and an extended Kalman filter to predict (over a very short-term) the ramp approaching flow at the intersection upstream from the ramp, and the ramp entering proportions in the flow approaching the ramp from the intersection. The proposed method was tested and validated in the I-35W freeway corridor in Minneapolis, Minnesota.

Results from three typical ramp areas along the I-35W corridor indicated mean absolute error (MAE) of the freeway- and ramp-approaching proportions between 3% and 7%. This research is not useful in predicting behavior if real-time data are unavailable, and therefore not useful for benefits analysis.

Zhang and Recker (1998) analyzed the state and control relationships to arrive at general analytical results regarding optimal metering policies. Their traffic dynamics of a simple corridor consisted of three parts — freeway traffic dynamics, freeway alternative traffic dynamics, and queue dynamics on the ramp.

Driver's diversion behavior and queuing are two major factors influencing ramp traffic dynamics. The authors assumed that without real-time traffic information, a driver's propensity to divert from a metered ramp was a function of the driver's estimate of the waiting time due to queuing on the ramp and an expectation of the travel time on the alternate route. It was also assumed that there would be no diversion in the absence of any queue on the ramp and any traffic would always divert if the queue on the ramp reached its maximum storage capacity. There are variations of diversion between these two extremes.

Zhang and Recker concluded that the optimal control strategy for metering depended on whether or not travel time saved on the freeway could offset travel time delayed at controlled ramps. Optimal ramp control policies depended not only on traffic diversion but also on differentials between freeway and parallel arterial traffic conditions. Unless drivers had at least some propensity to divert from entering the freeway based on the queue at the entry ramp, it would never be beneficial to the total system performance to meter the entry ramps. Even in the limiting case in which drivers were extremely sensitive to the presence of ramp queues, metering a congested freeway might not be beneficial to overall system performance.

Chen, Hotz and Ben-Akiva (1997) developed a dynamic ramp metering control system for real-time freeway operations. They proposed a hierarchical control system that manages traffic at both local and area-wide levels. It included the following four modules: state estimation, OD prediction, local control, and area-wide control. The dynamic area-wide metering control model captures the mainline traffic and the ramp queuing dynamics by modeling the traffic flows according to their origin-destination pairs. The model uses a generic nonlinear speed-density function that is valid under most traffic conditions.

The proposed dynamic traffic control system and the algorithms were tested in the MITSIM microscopic traffic simulator. The selected network consisted of a 2.7-mile freeway with four to five lanes, six on-ramps and five off-ramps. All of the MOEs in this simulation study demonstrated that the local linear-quadratic (LQ) feedback control algorithm and the area dynamic optimal control algorithm show very close results. However, the bilevel algorithm that combines the local and the area algorithms in a hierarchical structure shows a substantial improvement over any of the other strategies.

In another methodological study, Lovell and Daganzo (1999) explored control strategies for metered networks with unique O-D paths, with particular emphasis on those situations where some time-dependent O-D information was known in advance. The application in mind was a small freeway network with signalized meters, such as circumferential ring freeways or congested weaving sections where upstream access could be controlled.

For networks with a single origin or a single bottleneck a myopic strategy, which required the solution of a sequence of simple linear programs, was optimal. For networks with a single destination, the nonlinearities disappeared and the problem became a large-scale linear program. This was also true for general networks if the fractional distribution of flow across destinations for every origin is independent of time.

Reluctance and Equity of Diversion

Ullman, Dudek and Balke (1994) conducted two short telephone surveys of a group of 44 subjects known to travel to and from work on the North Central Expressway in Dallas, Texas. Subjects were read a series of eight traffic messages in random order and were asked to envision themselves receiving these messages over the radio as a traffic advisory broadcast. The subjects were asked how much time they would need to save to cause them to consider diverting. The resulting average time saved thresholds ranged from a low of 10.2 min to a high of 17.6 min. The 50th percentile subject in this study required a nearly 15-minute timesavings before he or she would consider diverting. This study illustrates the reluctance to divert for preplanned individual trips, but does not give an indication of behavior over larger time frames.

Because ramp metering favors through traffic, metering benefits longer trips at the expense of "local" motorists. Trips may be diverted to local surface streets, and residents close to the CBD may be deprived of access given to suburban dwellers. In Milwaukee, for example, where equity proved to be a delicate subject, metering rates were adjusted so that delay to the average motorist was the same on close-in ramps and on outlying ramps (Kang and Gillen, 1999).

Simulation Models for Determining Ramp Meter Benefits

Traffic simulation models are becoming an increasingly important tool for traffic control. Simulations are needed, not only to assess the benefits of ITS in a planning mode, but also to generate scenarios, optimize control, and predict network behavior at the operational level.

Traffic simulation models can be classified as either microscopic, mesoscopic, or macroscopic (Boxill and Yu, 2000). Microscopic models continuously or discretely predict the state of individual vehicles. Microscopic models measure individual vehicle speeds and locations. Macroscopic models aggregate the description of traffic flow. Macroscopic measures of effectiveness are speed, flow, and density. Mesoscopic models are models that have aspects of both macro and microscopic models. In addition, simulation models can be classified by functionality, i.e. signal, freeway, or integrated.

Limitations and Strengths of Simulation Modeling

May (1990) points out that it is important to keep simulation modeling in its context and view simulation modeling as one of several analytical techniques available to the traffic and transportation analyst. May emphasizes these potential pitfalls to simulation modeling.

1. There may be easier ways to solve the problem.
2. Simulation can be time-consuming.
3. Simulation models require considerable input characteristics and data, which may be difficult or impossible to obtain.
4. Simulation models require verification, calibration and validation that if overlooked renders the model useless.
5. A simulation model may be difficult for persons other than the developer to use because of lack of documentation.
6. Simulation is not possible unless the modeler fully understands the system.
7. Some users may treat simulation models as black boxes, while not understanding what they represent.
8. Some users may not know or appreciate model limitations and assumptions.

May also points out the following strengths of simulation modeling.

1. Other analytical approaches may not be appropriate.
2. Models allow for experiments off-line without using an on-line trial and error approach.
3. Models allow for experiments with new situations that do not exist today.
4. Models can yield insights into which variables are important and how they interrelate.
5. Models give time and space sequence information in addition to mean and variances.
6. Systems can be studied in real time, compressed time, or expanded time.
7. Potentially unsafe experiments can be conducted without risk to system users.
8. Models can replicate base conditions for equitable comparison of improvement alternatives.
9. An individual can study the effects of changes on the operation of a system: "What if…happens?"
10. Models can handle interacting queuing processes.
11. Models can transfer unserved queue traffic from one time period to the next.
12. Demand can be varied over time and space.
13. Unusual arrival and service patterns, which do not follow more traditional mathematical distributions, can be modeled.

With regard to traffic simulation within an ITS framework, some limitations have also been identified by Smartest (1997) as follows.

Modeling congestion. Most simulation models use simple car following and lane changing algorithms to determine vehicle movements. During congested conditions these do not realistically reflect driver behavior. The way congestion is modeled is often critical to the results obtained.

Environmental modeling. Considerable effort is being directed at producing emission models for incorporation into simulation models. For some emissions this is straightforward but for others complex chemical reactions are taking place within car exhausts making predictions difficult. It is also proving difficult to get reliable emission data for a reasonable mix of traffic.

Integrated environments and common data. Simulation models are often used with other models such as assignment models. There are common inputs required by all these models, such as origin-destination data, network topology, and bus route definitions. However, each model often requires the data in a different format so effort is wasted in re-entering data or writing conversion programs.

Safety evaluation. Safety is a very complex issue. Most safety prediction models are very crude, being based on vehicle flows on given roadways or on lane changes in mean vehicle speeds. Simulation models completely ignore vulnerable road users such as cyclists or pedestrians.

Standard procedures and indicators for evaluation. The traffic simulation must produce outputs, which will rank the alternatives realistically. Alternative rankings are a function of the chosen performance indicators and the weights used. Standard sets of performance indicators and procedures need to be produced.

Selection of Simulation Models

Elefteriadou *et al*. (1999) presented a framework for selecting simulation models that are applicable to the problem at hand, including ramp metering. The authors presented a series of steps to follow when selecting and applying a simulation model.

1. Project Scoping: The first step is to identify the problem and the purpose of the study.
2. HCM Assessment: The next step is to consider the available Highway Capacity Manual procedures, and determine if any of them can be applied to the issues identified in project scoping. Limitations of the HCM procedures, with respect to the problem statement and issues from step 1 should be identified. If the limitations cannot be overcome with HCM procedures, simulation may be a viable alternative. The HCM does not have delay relations for ramp meters.
3. As the authors note, every simulation model has its strengths and weaknesses. It is important for the analyst to understand model limitations and deficiencies, relate the limitations to the needs of the project, and select the model that best satisfies the specified needs. Model capabilities, data requirements and availability, ease of use, staff expertise, technical support, and past model application and experience should all be taken into consideration.
4. Data Assembly: The analyst should identify the existing data and develop a comprehensive plan for collecting those data that are missing.
5. Data Input: This step creates the input files according to the input format required by the selected model.
6. Model Calibration and Validation: This step refers to the process by which the analyst confirms that the model provides a reasonable approximation of reality.
7. Output Analysis: The analyst can conduct a statistical analysis of the simulation results for the baseline case with calibrated parameters.
8. Alternatives Analysis: The last step is to prepare data sets for alternative cases by varying geometry, controls, traffic demand, or all of these parameters.

Evaluation of Simulation Models

Boxill and Yu (2000) conducted a two-step evaluation study of simulation models: initial screening and in-depth evaluation. Criteria for initial screening were developed in order to eliminate models with no potential for use with ITS applications. In-depth evaluation attempts to identify more specific features and limitations of models selected from the initial screening process.

It is found that CORSIM appears to be the leading model for testing most of the scenarios involving alternative geometric configurations (weaving, merging, diverging), incident and work zone impacts, and various ramp metering options. It also appears to be the leading model for testing scenarios involving intersection design, signal coordination options, and transit modeling for exclusive lanes or mixed traffic. CORSIM can assess advanced traffic control scenarios in which the route is fixed (such as adaptive traffic signal control on arterials and traffic responsive ramp metering without diversion).

INTEGRATION appears to be the leading model for evaluating ITS scenarios along corridors that involve effects of real time route guidance systems, or changes in traffic patterns as a result of freeway ramp metering options.

Table 1-3 summarizes the in-depth evaluation. In the table the rows most pertinent to the evaluation of benefits of ramp meters have been shaded.

Table 1-3. Summary of Models Based of In-Depth Criteria (Boxill and Yu, 2000)

ITS Features Modeled	**AIMSUN 2**	**CONTRAM**	**CORFLO**	**CORSIM**	**FLEXYT II**	**HUTSIM**	**INTEGRATION**	**PARAMICS**	**VISSIM**
Traffic devices	X						X	X	
Traffic device functions	X						X	X	
Traffic calming					X	X	X	X	X
Driver behavior	X			X	X		X	X	
Vehicle interaction	X			X	X		X	X	
Congestion pricing						X		X	
Incidents	X		X	X	X	X	X	X	X
Queue spillback	X			X	X	X	X	X	X
Ramp metering	X			X	X	X	X	X	X
Coordinated traffic signals	X	X		X	X	X	X	X	X
Adaptive traffic signals	X	X		X	X	X	X	X	X
Interface w/other ITS algorithms	X								
Network conditions	X					X		X	
Network flow pattern predictions					X	X	X	X	X
Route guidance									
Integrated simulation	X	X		X		X	X	X	X
Other properties									
Runs on a PC	X	X		X	X	X	X	X	X
Graphical network builder	X	X			X	X			X
Graphical presentation of results	X	X		X	X	X	X	X	X
Well documented	X	X	X	X	X	X	X	X	X

Table 1-4 is the summary of application areas of selected models.

Table 1-4. Summary of Application Areas of Selected Models (Boxill and Yu, 2000)

AIMSUN 2	Traffic control systems Route guidance, VMS Evaluation of roadway design alternatives
CONTRAM	Time varying traffic demands Prediction of variation of traffic through time of the resulting routes, queues, and delays Design of urban traffic management options
CORSIM	Assessment of advanced traffic control scenarios • Adaptive traffic signal control on arterials • Traffic responsive ramp metering without diversion
FLEXYTT II	Effects of control strategies of comparison of different strategies of control
HUTSIM	Evaluation and testing of signal control strategies and different traffic arrangements Development of new control systems Evaluation of ITS applications
INTEGRATION	Corridor improvement strategies (HOV) Assessment of real time route guidance and information benefits
PARAMICS	Simulation of • Impact of traffic signals • Ramp meters • Loop detectors linked to variable speed signs • VMS and CMS signing strategies • In-vehicle network state display devices • In-vehicle route guidance
VISSIM	Transit signal priority studies Intersection/interchange design and operations

Two well-known models are the INTEGRATION and the CORSIM microscopic traffic simulation models. CORSIM consists of two sub-models: NETSIM and FRESIM. NETSIM is used to simulate urban surface street conditions while FRESIM is used to simulate freeway conditions. INTEGRATION can be used for network-wide ramp metering impacts, particularly for traffic diversions. CORISM can be used in the assessment of traffic control strategies in which route selection is fixed.

According to Crowther (2001), when various models were applied to an urban arterial, they were consistent in estimates of delay time and travel time, and inconsistent in estimates of vehicle stops, stopped delay, fuel consumption, and emissions. Specifically, it was observed that NETSIM underestimates the number of vehicle stops in comparison with INTEGRATION. It was also observed that NETSIM's vehicle speed and acceleration profiles can be characterized by more abrupt accelerations than observed in INTEGRATION. It was observed that the abrupt accelerations and decelerations of the FRESIM vehicles significantly impacted estimates of stopped delay and vehicle emissions. The differences in emissions estimates were also attributed to differences in the embedded fuel consumption and emissions rate tables of each model.

When the models were applied to a freeway environment with under-saturated conditions, INTEGRATION returned higher values of travel time and delay time, and lower values of average speed than FRESIM. For saturated conditions, FRESIM vehicles were observed to undergo substantial decelerations upon entering the reduced-capacity link. These decelerations, along with the resultant queues and lower average speeds, resulted in longer travel times and delay estimates with FRESIM than with INTEGRATION.

In over-saturated conditions, both models showed consistency in the estimates of travel time, delay time and average speed. However, longer queue lengths were observed in FRESIM than in INTEGRATION, resulting in slightly higher travel and delay estimates in the FRESIM model. For random arrivals at volume-to-capacity ratios ranging from 0.5 to 1.5, INTEGRATION consistently estimated higher travel times than FRESIM.

Traffic Assignment and Prediction of Diversions

The success of ramp meter deployment depends on the availability of advanced traffic analysis tools to predict network conditions and to analyze network performance in the planning and operational stages. Ramp meter operation is heavily dependent on the availability of timely and accurate wide-area estimates of prevailing and emerging traffic conditions. FHWA is sponsoring the development of a Traffic Estimation and Prediction System (TrEPS) to meet these information requirements and to aid in the evaluation of traffic management and information strategies. Traffic assignment is one of the most important parts of TrEPS. Traffic assignment techniques include minimum path (all-or-nothing) assignment, equilibrium assignment, stochastic assignment, and dynamic assignment.

Minimum Path (All-or-Nothing) Traffic Assignment

The all-or-nothing technique simply assumes that all of the traffic between a particular origin and destination (O-D) will take the shortest path with respect to time. A given route between a given O-D pair receives either all of the traffic or none of the traffic.

Advantages of this approach are that it is simple and inexpensive to use; it depicts the routes most travelers would be expected to use in the absence of capacity and/or congestion effects; and the results are easy to understand and interpret. The major disadvantage of the approach is that it clearly generates unrealistic flow patterns in situations where capacity constraints and congestion effects exist (Meyer and Miller, 2001).

Equilibrium Traffic Assignment

Equilibrium assignment techniques consider the existence of capacity constraints and congestion. The more congested a highway link is, the more cost will be associated with travel along it. The fundamental paradigm of congested network equilibrium is the user-optimal principle, attributed originally to Wardrop (1952). He mentions that under network equilibrium conditions each individual chooses the route perceived as being the best. Each individual minimizes or optimizes travel time or cost. No driver can achieve a lower travel time or cost by switching to another route.

In contrast, the system-optimal principle requires that the total cost of travel (e.g. total travel time) in the network be minimized. That means the system average cost of travel is minimum. All unused routes, therefore, must have travel cost greater than that of the used routes. The user-optimal principle will not always yield the lowest possible average system cost. The user-optimal principle is more approximate to the real situation and involves less complicated techniques for solution than system-optimal principle.

Stochastic Traffic Assignment

Equilibrium assignment methods assume that all users in the system have perfect information about the travel times on alternative paths within the network and that they can make perfectly correct route choice decisions base on this information. However, in practice, users generally do not have perfect information about the travel times. Thus, various stochastic traffic assignment approaches have been proposed and sometimes used. These procedures recognize that several routes between an origin and a destination might be perceived to have equal travel times or otherwise be equally attractive to a traveler. As a result, these routes would be equally likely to be used by that traveler. These procedures treat link costs as random variables that can vary among individuals given their individual preferences, experiences, and perceptions (Meyer and Miller, 2001). However, stochastic assignment approaches have seen more limited applications than equilibrium approaches, due both to a relative lack of commercially available software and to their considerably greater theoretical and computational complexity.

Dynamic Traffic Assignment

Both equilibrium approaches and stochastic approaches generally are developed in a ***static*** way. That is, these procedures assume that each vehicle is simultaneously located on every link on its chosen path and therefore cannot capture detailed temporal and spatial dynamics. However, these dynamics are important for capturing congestion patterns within small intervals of time.

A dynamic representation of route choice behavior and resulting network performance is required in which the movements of vehicles along their chosen paths is explicitly simulated through time. At each point in simulated time t, a given vehicle i will have a computed location x_{it}, a speed v_{it}, etc. Dynamic assignment models may be either probabilistic in terms of the simulation of users' route choices and/or determination of vehicles' travel time along given links, or they can be deterministic. They may solve for an equilibrium traffic pattern or simply generate a "single outcome" from a distribution of possible flow patterns, and they can be developed at various levels of spatial and temporal aggregation (Meyer and Miller, 2001).

Several approaches to the dynamic network flow problem have emerged, including:

- Simulation-based approaches;
- Optimal control theory;
- Variational inequality;
- Dynamic systems approaches;
- Mathematical optimization.

Simulation-based dynamic network flow modeling uses numerical methods to predict dynamic flow and network performance, often within a traditional iterative flow assignment procedure such as incremental assignment. The objective of conventional optimal control theory is to determine control strategies that cause a process to satisfy the physical constraints while at the same time minimize or maximize performance criterion. An alternative but related approach is to describe the behavior of a traffic network as a variational inequality problem with exact flow propagation constraints. The dynamic system approach generally takes the form of a system of differential equations that describe a trajectory of disequilibrium states tending towards equilibrium. In a mathematical optimization approach, time-dependent network flows are defined by a set of link performance functions and an equilibrium condition that extends the Wardrop's user-optimal principle (Wu *et al*., 2001)

Application to Freeway Corridors and Prediction of Diversions

Romph *et al*. (1994) conducted a study using the dynamic assignment model 3DAS as a planning tool. The 3DAS model is based on the work carried out by Hamerslag and Opstal (1987) and Hamerslag (1989). The model determines the flow distribution in the network with an iterative process. In each iteration the shortest paths in the network are calculated for all O-D pairs and for every departure period. The link parameters are defined separately for each period. The properties of the network and travel demand are presumed to be given. The model was applied to the eastern part (Virginia part) of the Capital Beltway around Washington, D.C. The major Interstates were I-95, I-66, and I-495; a larger part of the arterial network was also included. Three different scenarios were calculated: a morning peak hour scenario, a scenario with several ramp metering installations, and a scenario with an incident.

The results show that more detailed information about the occurrences of traffic jams, and the location or cause of congestion can be identified more precisely than using static assignment approaches. Dynamic assignment has the advantage that all kinds of temporary disturbances, such as accidents or roadwork, can be simulated and the duration of delays can be derived. However, data requirements are much more stringent, and the calculation time required is longer than that required for static assignment approaches. To alleviate congestion ramp metering can be simulated, and all kinds of evaluations are possible, such as the influence on travel time and jam length and the effects of ramp metering and rerouting.

Robles and Janson (1995) applied a dynamic traffic assignment model (DYMOD) to predict time-varying traffic conditions on a moderate-sized urban network in the southeastearn Denver metropolitan area during incidents and congested periods. With speed-volume functions to predict travel times and a set of zone-to-zone trip tables containing the number of vehicle trips departing from each zone and headed towards each zone in successive time intervals, DYMOD finds the volume of vehicles on each link within a network in each time interval that satisfies dynamic user-optimal equilibrium conditions.

Peak-hour counts for about 20 percent of the network links were used to estimate a morning peak-period trip matrix between 110 zones covering this area. Volume counts of 5 minutes collected from loop detectors at the on-ramps to I-25 and I-225 were used to estimate the departure times of these trips from each zone.

The results indicate that DYMOD can be used off-line to develop proactive response plans for accidents at critical network locations, work-zone traffic control and detour routing plans, or traffic impact predictions for a major spectator event or storms. Dynamic traffic modeling yields much closer estimates of traffic conditions than conventional transportation planning models when applied to urban area networks during congested periods. The key to successful dynamic traffic modeling is the care with which the supply and demand databases are developed. Much more detail is needed than for a typical static model. Wider regional coverage of traffic detection must be a priority to support the successful development and operation of dynamic traffic modeling and route guidance from a traffic management center. O-D and departure time estimation is operationally the weakest link in dynamic travel modeling because of such limited count coverage in most urban areas.

Eventually, dynamic traffic models will be integrated with traffic control centers that respond directly to real-time conditions through adjustments of arterial signals, ramp meters, and variable message signs (VMS).

An integrated system-optimum control model for commuting traffic corridors is described by Chang, Ho and Wei (1993). The proposed model has five principal components:

1. Dynamic Traffic Forecasting Module (DTFM): With this module, one can predict the time-dependent arriving flows to each ramp and surface street segment based on both historical data and on-line traffic information. It also yields a time-dependent, O-D matrix to indicate the fraction of traffic from a given ramp to various destinations.
2. Dynamic Ramp Assignment Model (DRAM)
3. Diversion Flow Prediction Model (DFPM): DFPM is used to estimate ramp flow diversion since commuters may choose to use different ramps after perceiving unexpected queues. The underlying assumption is that a commuter's choice between the freeway and the surface street is based mainly on the *difference in the expected travel time*. Such a pretrip decision may be revised if unexpected long ramp queues have been observed. The diversion traffic is a function of the ramp metering rate and queue length, which along with the original surface street and off-ramp flows constitute the principal elements for estimating both the surface street and intersection delays. The resulting delay in the surface street sections, and the waiting time at ramps as well as intersections, are then used to determine the optimal metering rates for all ramps during the subsequent interval.
4. System–Optimum Control Model (SOCM): SOCM is employed to determine the optimal ramp metering rates and intersection signal settings.
5. System Monitoring and Feedback Component (SMFC): This component serves to monitor the performance of ATMS based on the predicted demands and actual flows detected by the traffic surveillance system.

An integrated model and a heuristic algorithm for optimal diversion control in commuting corridors are presented by Wu and Chang (1999). The proposed optimal control integrates ramp metering, intersection signal timing, and off-ramp diversion in a real-time environment that can minimize the total travel time of time-varying demand in nonrecurrent congestion. In order to reliably capture the flow interactions during freeway incidents, the proposed control model embodies the following three unique features:

- Modeling traffic state evolution on surface streets with flow conservation within sections, flow transition between sections, and flow discharge at downstream intersections;
- Estimating time-dependent model parameters adaptively with real-time traffic measurements; and
- Having an efficient solution algorithm.

FHWA's Traffic Estimation and Prediction System (TrEPS) includes dynamic traffic assignment as one means of ascertaining traffic demands. TrEPS in intended for:

- *Estimating and predicting (short-term) traffic OD demand for traffic control and management*
- *Estimating and predicting traffic network conditions*
- *Providing travel mode, departure time, route, and other traffic information and advisory to travelers through ATIS for meeting various traffic management and control objectives*
- *Interacting with other ITS sub-systems or, in the interim, interfacing with other ATMS support systems within the TMCs and with ATIS* (FHWA, 20001)

Two different simulation models are being evaluated for inclusion in TrEPS: DynaMIT and DYNASMART-X. Both packages use real-time as well as historical data in their forecasts. DYNASMART-X, in particular, has a dynamic traffic assignment module.

Conclusions

Simulation studies and field measurement place the VHT reductions owing to ramp metering at about 10%, at most. Inappropriate use of ramp meters can produce negative benefits. Diversion is essential to achieve positive benefits.

Drivers are reluctant to divert from preplanned routes in reaction to traffic congestion.

Long-term route selection is based primarily on travel time. Equilibrium concepts are valid for determining long-term effects of ramp meters.

Both microscopic and macroscopic simulation models have been effective at modeling ramp meters in freeway corridors. Selection of the appropriate model depends on several criteria and no single model stands out as best.

The Minnesota study provided contradictory results. Diversions that would have been predicted from a stated choice survey were not measured in field data. The study saw a reduction in freeway volumes when meters were turned off, but the reasons for the reduction are unknown.

System-wide optimization schemes using real-time data appear to perform better than localized methods of ramp meter control.

Fairness is a criterion when establishing ramp metering, but strict rules of fairness may undermine time-savings benefits.

References

Boxill, S. A. and L. Yu, "An Evaluation of Traffic Simulation Models for Supporting ITS Development", Center for Transportation Training and Research, Texas Southern University, October 2000.

Chang, G., P. Ho, and C. Wei, "A Dynamic System-Optimum Control Model for Commuting Traffic Corridors", *Transportation Research*, C Vol. 1C, No. 1, pp. 3-22, 1993.

Chen, O. J., F. H. Anthony, and M. E. Ben-Akiva, "Development and Evaluation of a Dynamic Ramp Metering Model", *Proceedings of the 8th International Federation of Automatic Control (IFAC) Symposium on Transportation System*. Chania, Greece, June 16-18, 1997, pp. 1162-1168.

Crowther, B. C., "A Comparison of Corsim and Integration for The Modeling Of Stationary Bottlenecks", Virginia Polytechnic Institute and State University, May 9, 2001.

Elefteriadou, L., J. D. Leonard II, G. List, H. Lieu, M. Thomas, R. Giguere, G. Johnson, and R. Brewish, "Beyond the Highway Capacity Manual. Framework for Selecting Simulation Models in Traffic Operational Analyses", *Transportation Research Record #1678*, 1999, pp. 96-106.

Federal Highway Administration, TrEPS Web Site, http://www.dynamictrafficassignment.org, 2001.

Fonda, R. D., "An Analysis of Short-Term Implementation of Ramp Control on the Dan Ryan Expressway", *Transportation Research Record #1603*, 1976, pp. 8-9.

Haj-Salem, H., and M. Papageorgiou, "Ramp Metering Impact On Urban Corridor Traffic: Field Results", *Transportation Research A*, Vol. 29A, No. 4, 1995, pp. 303-319.

Hamerslag, R., and P. C. H. Opstal", "A Three-Dimensional Assignment Method in Timespace", Report 87-94, Faculty of Mathematics and Informatics, Delft University of Technology, Delft, The Netherlands, 1987.

Hamerslag, R., "Dynamic Assignment in the Three Dimensional Timespace", *Transportation Research Record #1220*, 1989, pp. 28-32.

Hasan, M., "Evaluation of Ramp Control Algorithms Using A Microscopic Traffic Simulation Laboratory, MITSIM", Massachusetts Institute Of Technology, February 1999.

Hellinga, B., and M. V. Aerde, "Examining The Potential of Using Ramp Metering as a Component of an ATMS", *Transportation Research Record #1494*, 1995, pp. 75-83.

Kang, S., and D. Gillen, "Assessing the Benefits and Costs of Intelligent Transportation Systems: Ramp Meters", California Partners for Advanced Transit and Highways Research Report, UCB-ITS-PRR-99-19, July 1999.

Lovell, D. J., and C. F. Daganzo, "Access Control on Networks with Unique Origin-Destination Paths", *California Partners For Advanced Transit and Highways*, April 19, 1999.

May, A.D., *Traffic Flow Fundamentals*, Prentice Hall, Englewood Cliffs, New Jersey, 1990.

Meyer, M. D., and E. J. Miller, *Urban Transportation Planning (2nd Edition)*, McGraw Hill, New York, 2001.

The Minnesota Department of Transportation, "Twin Cities Ramp Meter Evaluation Final Report", Prepared by Cambridge Systematics, Inc. with SRF Consulting Group, Inc. N.K. Friedrichs Consulting, Inc, February 1, 2001.

Nsour, S. A., S. L. Cohen, J. E. Clark, and A. J. Santiago, "Investigation of the Impacts of Ramp Metering on Traffic Flow with and without Diversion", *Transportation Research Record #1365*, 1992, pp. 116-124.

Papageorgiou, M., H. Hadj Salem, and J. M. Blosseville, "ALINEA: A Local Feedback Control Law for On-Ramp Metering", *Transportation Research Record #1320*, 1991, pp. 58-64.

Papageorgiou, M., H. Haj-Salem, and F. Middelham, "ALINEA Local Ramp Metering: Summary of Field Results", *Transportation Research Record #1603,* 1998, pp. 90-98.

Piotrowicz, G., and J. Robinson, "Ramp Metering Status in North America — 1995 Update", Report No. DOT-T-95-17, Federal Highway Administration, U.S. Department of Transportation, Washington, D.C., June 1995.

Prevedouros, P. D. "H-1 Freeway Ramp Closure: Simulation and Real-World Experiment", *Transportation Research Board,* 78th Annual Meeting, January 10-15, 1999, Washington, D.C.

Robles, J., and B. N. Janson, "Dynamic Traffic Modeling of the I-25/HOV Corridor Southeast of Denver", *Transportation Research Record #1516*, 1995, pp. 48-60.

Romph, E, H. J. M. Van Grol, and R. Hamerslag, "Application of Dynamic Assignment in Washington, D.C., Metropolitan Area", *Transportation Research Record #1443*, 1994, pp. 100-109.

The Smartest Project, "Review of Micro-Simulation Models Appendix D: Analysis of Tools", Leeds University, 1997.

Stephanedes, Y. J., E. Kwon, and P. G. Michalopoulos, "Demand Diversion for Vehicle Guidance, Simulation, and Control in Freeway Corridors", *Transportation Research Record #1220*, 1989, pp. 12-20.

Stephanedes, Y. J., and E. Kwon, "Adaptive Demand-Diversion Prediction for Integrated Control of Freeway Corridors", *Transportation Research C*, Vol 1C, No. 1, pp. 23-42, 1993.

Ullman, G. L., C. L. Dudek, and K. N. Balke, "Effect of Freeway Corridor Attributes on Motorist Diversion Responses to Travel Time Information", *Transportation Research Record #1464*, 1994, pp. 19-27.

Wardrop, J. G., **"**Some Theoretical Aspects of Road Traffic Research, *Proceedings of the Institution of Civil Engineers*, Part II, 1, pp.325-378, 1952.

Wu, J., and G. Chang, **"**Heuristic Method for Optimal Diversion Control in Freeway Corridors", *Transportation Research Record #1667*, 1999, pp. 8-15.

Wu, Y, H. J. Miller, and M Hung, "A GIS-Based Decision Support System for Analysis of Route Choice in Congested Urban Road Networks", *Journal of Geographical Systems*, Vol. 3, Issue 1, 2001, pp. 3-24.

Zhang, H. M., and W. W. Recker, "On Optimal Freeway Ramp Control Policies for Congested Traffic Corridors", *Transportation Research B*, Vol. 33B, 1999, pp. 417-436.

Chapter 2
Diversion of Traffic from Ramp Meters on US 45

Evaluation Framework

Diverting traffic from the freeway to alternate surface routes and from high volume, substandard, or other problem ramps to more desirable entrance ramps would be one of the positive benefits of ramp metering. Such an operation policy would require a thorough evaluation of the alternate routes, the entrance ramps, and the impacts of diversion on those routes and ramps. This evaluation study consists of three stages:

- Field data collection;
- Data preparation and calibration; and
- Analysis and evaluation.

Evaluation Background

The first metered ramp in the United States was installed in Chicago on the Eisenhower Expressway in 1963. In the Milwaukee metropolitan area, the first ramp meters began to operate in 1969. There were three ramp meters installed along I-94 westbound at 17th Street, 28th Street and Hawley Road. Since 1969, more ramp meters, VMS and CCTV cameras have been installed, which have become today's MONITOR. MONITOR is the Milwaukee metropolitan area computerized freeway management system consisting of electronic detectors, ramp meters, VMS, and CCTV. There are 3000 detectors along the system with 117 ramp meters, 51 of which offer HOV lanes. The Wisconsin Department of Transportation (WisDOT) uses ramp meters to manage freeway access on approximately 110 miles of freeways in the Milwaukee metropolitan area.

WisDOT began operation of seven new ramp meters on USH 45 southbound from the week of February 21, 2000, which are intended to improve traffic flow during weekday congestion and other incident related activities. One such incident would be the USH 45 southbound resurfacing project starting in late March 2000. Fifteen miles of southbound USH 45 from County Q (Washington/Waukesha County Line) to Lincoln Avenue in West Allis were resurfaced from March through September in 2000.

After the new ramp meters began operation, travelers would make corresponding adjustments to choose the route perceived as being the best. The probable new traffic patterns, including diversion, can then either be accommodated in the design and operation of the system, or become part of a decision that metering is not feasible. The objective of this study is to evaluate the impacts of traffic diversion resulting from ramp metering. Thus, improvements could be made to utilize the network capacity more efficiently.

The studied roadway network is a USH 45 section located in Milwaukee metropolitan area and two parallel arterial streets, STH 100 and 124th Street. The study was conducted in February 2000 as a "without ramp metering" period and in March 2000 as a "with ramp metering" period.

Site Description

The USH 45 corridor is located in the western part of the Milwaukee metropolitan area (Figure 2-1). The studied southbound USH 45 section is from County Q (Washington/Waukesha County Line) to Lincoln Avenue, which is about fifteen miles long and has three lanes throughout. There are 34 ramps in the studied section (Table 2-1). Among them, six ramps are freeway-to-freeway connectors, which are not metered. Fourteen ramps are exit ramps, which also do not have ramp meters. The remaining fourteen ramps are entrance ramps, thirteen of which are being metered. The only entrance ramp without a ramp meter is the on-ramp from Silver Spring Drive

There are two surface arterial streets, STH 100 and 124th Street, paralleling to USH 45. The studied section of STH 100 is from Silver Spring Drive to Lincoln Avenue, which is about 8 miles long and mostly has three lanes in both directions. The studied segment of 124th Street is from Silver Spring Drive to a location 0.5 mile south to North Avenue, which is about 5 miles long and mostly has two lanes in both directions. There are 13 major signalized intersections within the studied area.

The USH 45 corridor is one of the Milwaukee metropolitan area’s busiest north-south freeway corridors. The studied corridor carries various types of traffic, including traffic from outside the Milwaukee metropolitan area and commuter traffic between the residential area north of the corridor and employment destinations to the south. It also serves the traffic induced by baseball games and holidays. Recurrent traffic congestion occurs regularly both in the AM peak period and in the PM peak period. When it is a holiday or a baseball game is held at Miller Park, traffic congestions can extend for several hours and for several miles.

There are two alternate routes for this corridor, STH 100 and 124th street. Drivers traveling southbound within the network may choose USH 45 or a parallel alternate route. When traffic congestion begins on USH 45, the two parallel arterial streets are operating under light traffic conditions. When ramp meters are operating in the peak period, traffic may be diverted from the congested freeway to the more attractive and possibly more efficient alternate routes.

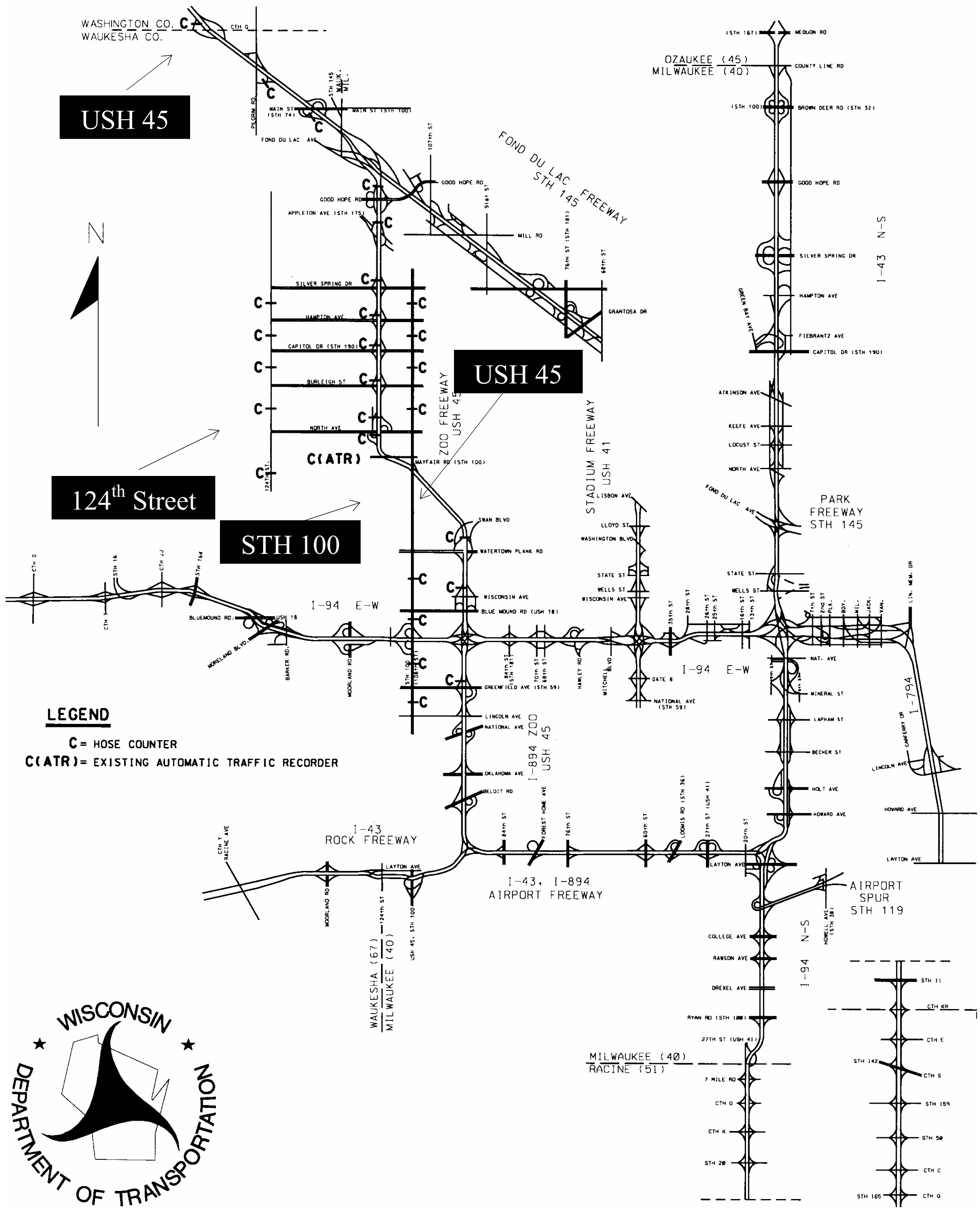


Figure 2-1. USH 45 Corridor

Table 2-1. List of Ramps in Studied Corridor

	Ramp Location	Ramp Type	Metering	Controller ID
1	Off to County Q	Exit	No	N/A
2	On from County Q	Entrance	Yes	RM 91
3	Off to Pilgrim Road	Exit	No	N/A
4	On from Pilgrim Road	Entrance	Yes	RM 92
5	Off to Main Street	Exit	No	N/A
6	On from Main Street	Entrance	Yes	RM 93
7	Off to Fond du Lac Freeway	Freeway Connector	No	N/A
8	On from Fond du Lac Freeway	Freeway Connector	No	N/A
9	Off to Good Hope Road	Exit	No	N/A
10	On from Good Hope Road WB	Entrance (Loop)	Yes	RM 89
11	On from Good Hope Road EB	Entrance (Slip)	Yes	RM 90
12	Off to Appleton Avenue	Exit	No	N/A
13	On from Appleton Avenue	Entrance	Yes	RM 96
14	Off to Silver Spring Drive	Exit	No	N/A
15	On from Silver Spring Drive	Entrance	No	N/A
16	Off to Hampton Avenue	Exit	No	N/A
17	On from Hampton Avenue	Entrance	Yes	RM 100
18	Off to Capitol Drive	Exit	No	N/A
19	On from Capitol Drive	Entrance	Yes	RM 102
20	Off to Burleigh Street	Exit	No	N/A
21	On from Burleigh Street	Entrance	Yes	RM 103
22	Off to North Avenue	Exit (WB)	No	N/A
23	Off to North Avenue	Exit (EB)	No	N/A
24	On from North Avenue	Entrance	Yes	RM 26
25	Off to Watertown Plank Road	Exit	No	N/A
26	On from Watertown Plank Road	Entrance	Yes	RM 25
27	Off to Wisconsin Avenue	Exit	No	N/A
28	On from Wisconsin Avenue	Entrance	Yes	RM 24
29	Off to I-94 WB	Freeway Connector	No	N/A
30	Off to I-94 EB	Freeway Connector	No	N/A
31	On from I-94 EB	Freeway Connector	No	N/A
32	On from I-94 WB	Freeway Connector	No	N/A
33	Off to Greenfield Avenue	Exit	No	N/A
34	On from Greenfield Avenue	Entrance	Yes	RM 41

Field Data Collection

Data related to the measures of effectiveness (MOE) were collected in February 2000 as a "without ramp metering" period and March 2000 as a "with ramp metering" period.

Data collected during the first period were used to assess the baseline or "without ramp metering" scenario for the purpose of identifying the traffic diversion occurring in the "with ramp metering" scenario. In this scenario, the ramp meters were operating only at six entrance ramps (Table 2-2).

Table 2-2. Ramp Meters Operating in the "Without Ramp Metering" Period

	Ramp Location	Ramp Type	Metered	Controller ID
1	On from Good Hope Road WB	Entrance (Loop)	Yes	RM 89
2	On from Good Hope Road EB	Entrance (Slip)	Yes	RM 90
3	On from North Avenue	Entrance	Yes	RM 26
4	On from Watertown Plank Road	Entrance	Yes	RM 25
5	On from Wisconsin Avenue	Entrance	Yes	RM 24
6	On from Greenfield Avenue	Entrance	Yes	RM 41

Table 2-3. Ramp Meters Operating in the "With Ramp Metering" Period

	Ramp Location	Ramp Type	Metered	Controller ID
1	On from County Q	Entrance	Yes	RM 91
2	On from Pilgrim Road	Entrance	Yes	RM 92
3	On from Main Street	Entrance	Yes	RM 93
4	On from Good Hope Road WB	Entrance (Loop)	Yes	RM 89
5	On from Good Hope Road EB	Entrance (Slip)	Yes	RM 90
6	On from Appleton Avenue	Entrance	Yes	RM 96
7	On from Silver Spring Drive	Entrance	No	N/A
8	On from Hampton Avenue	Entrance	Yes	RM 100
9	On from Capitol Drive	Entrance	Yes	RM 102
10	On from Burleigh Street	Entrance	Yes	RM 103
11	On from North Avenue	Entrance	Yes	RM 26
12	On from Watertown Plank Road	Entrance	Yes	RM 25
13	On from Wisconsin Avenue	Entrance	Yes	RM 24
14	On from Greenfield Avenue	Entrance	Yes	RM 41

Data collected during the second period were used to evaluate the “with ramp metering” scenario. In this scenario, thirteen ramp meters were operating (Table 2-3), including the seven new ramp meters beginning operation in the week of February 21, 2000.

Data collection occurred over a two-week period during both the “without ramp metering” and “with ramp metering” scenarios. “Without ramp metering” data collection occurred between February 1 and February 3, 2000, and between February 8 and February 10, 2000. “With ramp metering” data collection occurred between March 14 and March 16, 2000, and between March 21 and March 23, 2000. Message display signs were placed at each ramp a week before the seven new ramp meters began operation to advise motorists.

The premise of the field data collection was to measure the traffic diversion impacts of the ramp metering system in the USH 45 corridor. This task involved an extensive “without ramp metering” and “with ramp metering” traffic data collection program to address the impacts on traffic diversion. Traffic data were collected at specific ramps, along USH 45 mainline and along the two arterial streets (Figure 2-1) over two weeks for both the “without” and “with” ramp metering evaluation scenario. Data collection occurred during the morning and afternoon peak periods for approximately 1.5 hours per peak period each day within the evaluation timeframe.

Traffic volume data were collected to examine the traffic diversion impacts of the ramp metering system. These data include traffic volume data from freeway mainline, freeway ramps and the two alternate routes. Two different data collection methods were used including existing freeway loop detectors and portable counting devices (road tubes).

Freeway mainline and entrance ramp traffic volume data were obtained from loop detectors (Figure 2-2). There are three main types of vehicle detectors used in modern traffic control systems: inductive loop detectors, magnetic detectors, and magnetometers. By far, the inductive loop detector system is the most widely used method of vehicle detection. As shown in Figure 2-3, the system consists of three parts: a detector oscillator, a lead-in cable, and a loop embedded in the pavement consisting of one or more turns of wire. The oscillator serves as source of energy for the loop. When a vehicle passes over the loop or is stopped within the loop area, it reduces the loop inductance, causing an increase in the oscillator frequency. The change in inductance or frequency activates a relay or circuit which sends an electrical output to the controller signifying that it has detected the presence of a vehicle (ITE, 1990).

As many as three important types of traffic data are collected by the loop detector system every twenty seconds. They are volume, occupancy and speed. Volume is the flow rate of traffic derived from the number of vehicles that pass over a loop detector in a twenty second period. Occupancy is the percentage of time that a loop detector is occupied by a vehicle during the surveillance period. Speed can only be obtained directly when there is a pair of loop detectors in a lane or through a calibrated empirical formula.

In this study, traffic volume data are twenty seconds per lane, 24 hours per day. Freeway mainline traffic volume data were obtained by retrieving data from the primary mainline loop detectors of System Detector Stations (SDS) and ramp meters. Entrance ramp traffic volume data were obtained by retrieving data from the passage loop detectors of ramp meters.

Road tubes were used to collect traffic volume data along the two arterial streets and the freeway exit ramps under evaluation. In this case, traffic volume data are fifteen minutes for all lanes during the AM and PM peak periods.

Ramp queue length data at the metered entrance ramps were also collected. One observer was positioned at each metered entrance ramp. Throughout the AM and PM peak periods, observations of the number of the vehicles in the on-ramp queue were made approximately every twenty seconds.

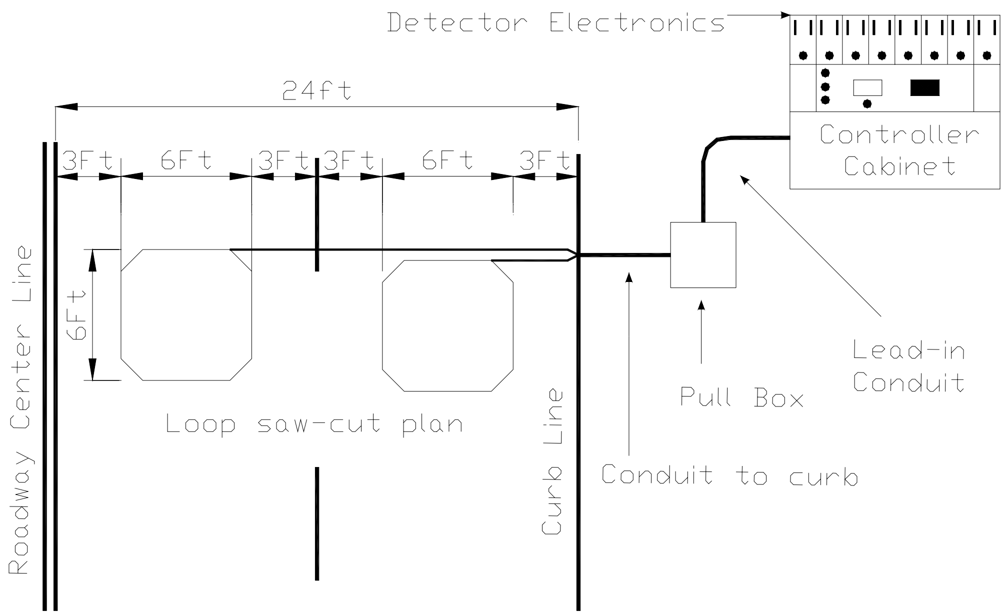


Figure 2-3. Loop Detector System

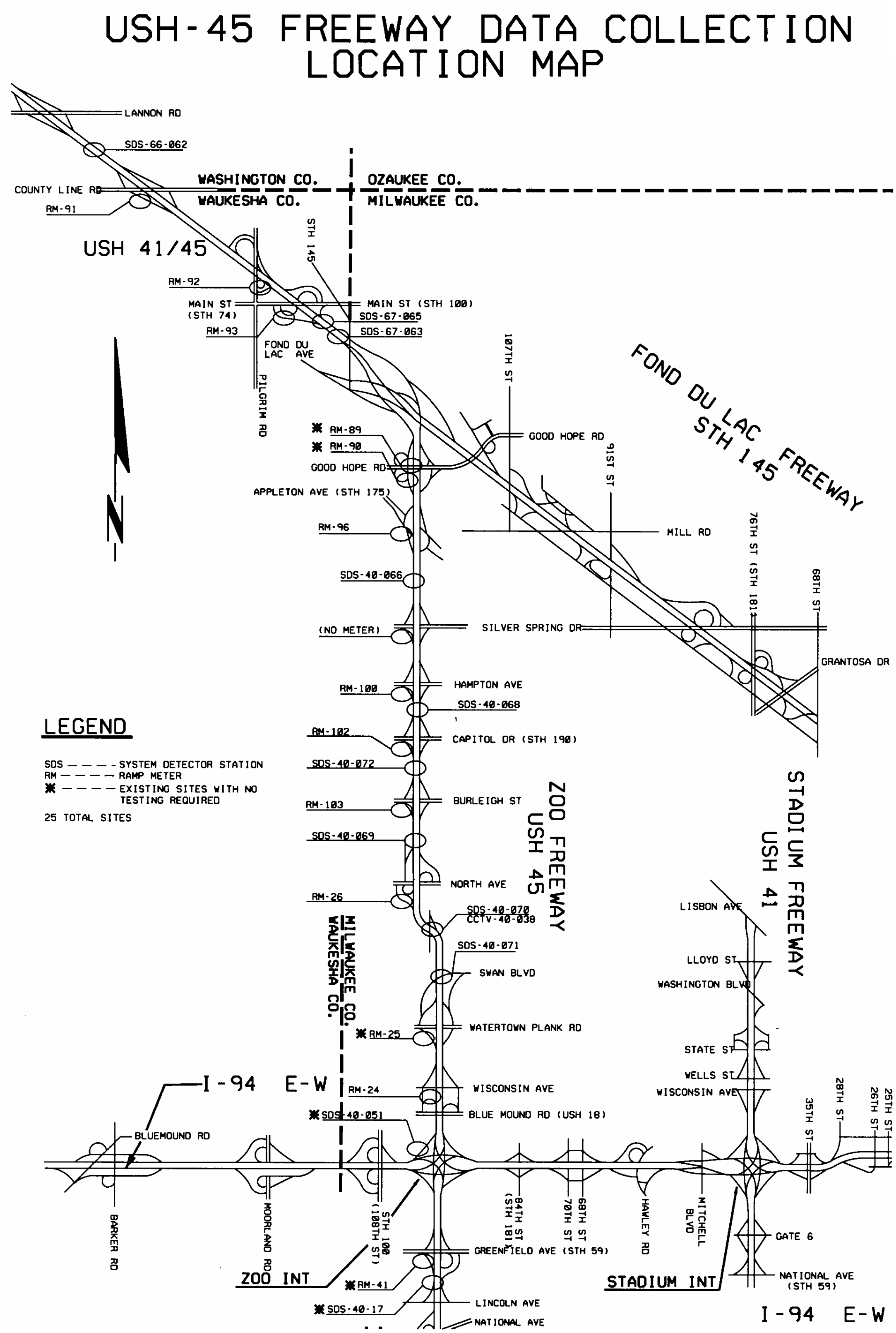


Figure 2-2. Freeway Mainline and Entrance Ramp Data Collection Location

Data Preparation and Calibration

Spreadsheets and databases are used to process the data. After all of the data are input into the spreadsheets and databases, the first and the most important step is the screening process of the collected data. First, the data of all of the time periods that included major detector and road tube failures are discarded. Second, data from all time periods with atypical traffic patterns are discarded. Finally, data from all time periods that included significant crashes or other incidents are discarded. This screening process eventually delivered three PM peak periods in the "without ramp metering" period and three PM peak periods in the "with ramp metering" period.

The results reported are, in fact, based on this final set of data. Although the total number of days available for the evaluation is not very high, traffic conditions appearing in the corresponding data are judged to be quite representative.

After the screening process, both traffic volume data and ramp queue length data are aggregated to 15 -minute intervals during the PM peak periods. Data are also aggregated to hourly totals to allow analysis of traffic diversion. Average on-ramp delay and queue length are calculated for the same purpose.

It is necessary to mention that in order to collect the data, two different data collection methods were used, including existing freeway loop detectors and portable counting devices (road tubes). These two methods may cause inconsistent traffic counts.

The tubes count number of axles instead of the number of vehicles. If a large number of commercial vehicles and trucks pass by, road tubes can cause overcounting. The loop detector will give only a single output for most vehicles, regardless of the number of axles. However, the loop detector tends to miscount vehicles. If a second vehicle moves over part of a loop before the first vehicle has left its part of the loop, only one continuous output will be registered. When counts are made in multiple lanes and vehicles change lanes without a good lane discipline, vehicles can pass between loops and can also straddle two loops. As the tubes are laid over the roadway, they are especially vulnerable to the wear and tear of passing traffic. The tube tends to distort counts prior to failure. However, if the buried loops experience a failure, all counting will stop.

Since the objective of this study is to evaluate the impacts of traffic diversion resulting from ramp metering, only the data relevant to conduct the evaluation were collected. Because the freeway mainline and most of the entrance ramps have loop detectors, most of the road tubes were placed on the exit ramps and the two arterial streets. As a result, was not possible for this study to calibrate the traffic volume data between loop detectors data and road tubes data. However, the special character of this study is that it compares the traffic volume counts at the same location between the "without ramp metering" period and the "with ramp metering" period. The same location uses the same method to collect data. Therefore, the difference caused by different data collection methods should be negligible.

It is also necessary to note that because of the failure of freeway loop detectors and road tubes, traffic volume data at some locations were not collected successfully. In order to conduct the evaluation, it is necessary to have these locations filled with approximate traffic volume data. The theory used in this process is that the downstream freeway mainline volume is equal to the upstream freeway mainline volume plus the on-ramp volume or the upstream freeway mainline volume less any off-ramp volume. As a result of calculation, each location with traffic volume data missing can have two traffic volume count numbers, calculated from upstream and downstream, respectively. The final data used in the evaluation is an average of these two numbers. This process may introduce a data inconsistency caused by mixing loop detectors data and road tubes data. However, because of the reasons mentioned above, it is not possible for this study to calibrate the traffic volume data between loop detectors data and road tubes data. It is assumed that this data inconsistency would not affect the results of the evaluation. The analysis is based mainly on the locations where the data were collected successfully.

Table 2-4 shows an example of the traffic volume counts in the second 15 minute period, which are averages of three PM peak periods in both the “without” and the “with” periods and are in vehicles per 15 minutes for all of the lanes.

Table 2-4. Average Traffic Volume Counts

	Second 15 Minutes	
Location	**February**	**March**
124th Street Between Hampton & Capitol	128	106
124th Street Between Burleigh & North	202	229
124th Street South to North Avenue	140	149
USH45 SB Between Hampton & Capitol	1275	1296
USH45 SB Between Burleigh & North	1413	1321
USH45 SB South to North Avenue	1422	1457
USH45 SB North to Watertown Plank Road	1391	1485
USH45 SB Between Watertown & Wisconsin	1602	1577
USH45 SB Between Blue Mound & I-94	1645	1614
USH45 SB Between I-94 & Greenfield	1840	1832
USH45 SB Between Greenfield & Lincoln	1764	1748
STH 100 Between Hampton & Capitol	156	173
STH 100 Between Burleigh & North	381	407
STH 100 South to North Avenue	367	406
STH 100 North to Watertown Plank Road	370	405
STH 100 Between Watertown & Wisconsin	372	404
STH 100 Between Blue Mound & I-94	429	453
STH 100 Between I-94 & Greenfield	365	441
STH 100 Between Greenfield & Lincoln	379	494

Evaluation Results

The first section analyzes the traffic diversion from USH 45 to the alternate arterial streets. Then the traffic diversion between different entrance ramps is evaluated. The last section provides the analysis of temporal diversion.

Diversion from Freeway to Arterial Streets

Traffic volume counts were collected along USH 45 mainline and the two arterial streets during the morning and afternoon peak periods for approximately 1.5 hours per peak period each day within the evaluation timeframe. After the screening process, three PM peak periods in both the "without" and the "with" periods were remained.

Cut Lines

Depending on the data availability and the potential traffic diversion pattern, eight cut lines are defined within the study corridor (Figure 2-4 and Table 2-5). The cut lines are numbered 1 through 8 from the north to the south. The analysis of the traffic diversion from USH 45 to the alternate arterial streets is then based on these eight cut lines. Table 2-6 summarizes the traffic volume counts passing these cut lines, which are averages of three PM peak periods in both the "without" and the "with" periods and are in vehicles per 15 minutes for all of the lanes.

Table 2-5. Cut Line Distribution

Cut Line No.	124th Street SB	USH 45 SB	STH 100 SB
1	Between Hampton & Capitol	Between Hampton & Capitol	Between Hampton & Capitol
2	Between Burleigh & North	Between Burleigh & North	Between Burleigh & North
3	South to North Avenue	South to North Avenue	South to North Avenue
4	—	North to Watertown Plank Road	North to Watertown Plank Road
5	—	Between Watertown & Wisconsin	Between Watertown & Wisconsin
6	—	Between Blue Mound & I-94	Between Blue Mound & I-94
7	—	Between I-94 & Greenfield	Between I-94 & Greenfield
8	—	Between Greenfield & Lincoln	Between Greenfield & Lincoln

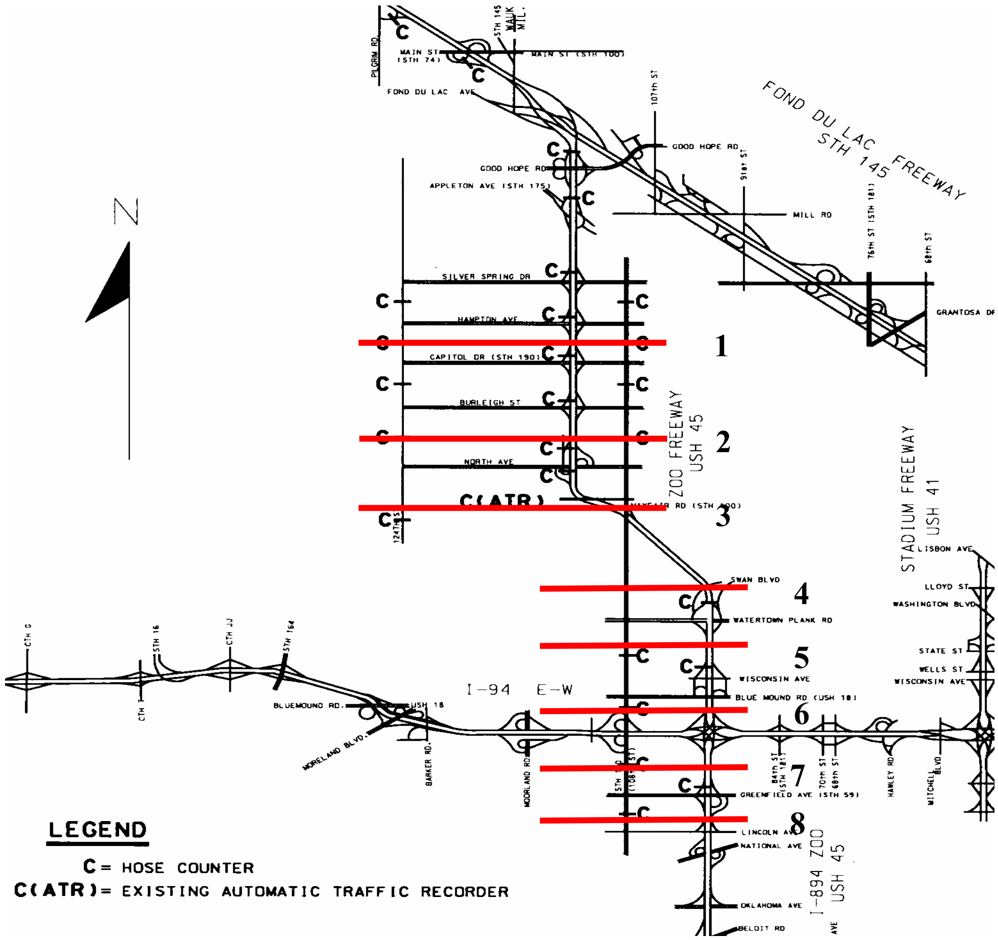


Figure 2-4. Cut Line Distribution

Table 2-6. Average Traffic Volume Counts — Vehicles Per 15 Minutes For All Lanes

Cut Line	Location	1st 15 min.		2nd 15 min.		3rd 15 min.		4th 15 min.		5th 15 min.		6th 15 min.	
		Feb.	Mar.	Feb.	Mar.	Feb.	Mar.	Feb.	Mar.	Feb.	Mar.	Feb.	Mar.
1	124th Street SB Between Hampton & Capitol	135	126	128	106	155	151	130	120	159	151	116	123
2	124th Street SB Between Burleigh & North	232	230	202	229	250	287	223	261	283	292	250	249
3	124th Street SB South to North Avenue	151	155	140	149	188	185	154	170	175	179	180	171
1	USH45 SB Between Hampton & Capitol	1271	1279	1275	1296	1407	1435	1360	1342	1355	1336	1143	1063
2	USH45 SB Between Burleigh & North	1394	1346	1413	1321	1417	1414	1423	1400	1352	1350	1192	1027
3	USH45 SB South to North Avenue	1477	1465	1422	1457	1512	1453	1485	1517	1408	1461	1363	1343
4	USH45 SB North to Watertown Plank Road	1469	1515	1391	1485	1495	1462	1413	1484	1381	1460	1153	1300
5	USH45 SB Between Watertown & Wisconsin	1680	1657	1602	1577	1628	1545	1600	1620	1583	1599	1500	1489
6	USH45 SB Between Blue Mound & I-94	1774	1710	1645	1614	1721	1645	1677	1676	1662	1679	1530	1510
7	USH45 SB Between I-94 & Greenfield	1805	1785	1840	1832	1761	1761	1796	1790	1758	1755	1543	1702
8	USH45 SB Between Greenfield & Lincoln	1773	1752	1764	1748	1717	1734	1772	1737	1746	1777	1344	1385
1	STH 100 SB Between Hampton & Capitol	153	168	156	173	187	215	208	207	206	225	209	205
2	STH 100 SB Between Burleigh & North	369	407	381	407	412	424	404	412	454	519	391	409
3	STH 100 SB South to North Avenue	387	402	367	406	433	441	412	431	444	500	408	436
4	STH 100 SB North to Watertown Plank Road	406	398	370	405	450	458	424	450	457	481	438	463
5	STH 100 SB Between Watertown & Wisconsin	425	393	372	404	465	474	435	469	469	463	468	490
6	STH 100 SB Between Blue Mound & I-94	476	449	429	453	507	542	474	518	573	566	530	533
7	STH 100 SB Between I-94 & Greenfield	398	404	365	441	407	459	405	468	452	478	456	506
8	STH 100 SB Between Greenfield & Lincoln	389	443	379	494	416	497	408	542	433	527	447	557

Diverted Traffic from Freeway to Arterial Streets

Drivers traveling southbound within the study corridor may choose USH 45 or a parallel alternate route. When traffic congestion starts on USH 45, the two parallel arterial streets are operating under nearly free flow conditions. When ramp meters are operating in the peak period, traffic may be diverted from the congested freeway to the attractive alternate routes. The traffic volume counts of three PM peak periods in both the "without" and the "with" periods are aggregated into six 15 minute periods, which are from 4 PM to 5:30 PM

Traffic diversion impacts can be obtained by comparing the traffic volume counts on the freeway and the arterial streets between the "without" and the "with" periods. In this case, the "without ramp metering" period is February 2000 and the "with ramp metering" period is March 2000. Table 2-7 through Table 2-12 illustrate the diverted traffic from USH 45 to the arterial streets on each cut line for each 15 minute period.

Table 2-13 presents the total traffic counts in the 1.5 peak hour periods. Based on the "without ramp metering" period, a positive diverted volume shows an increase of traffic in the "with ramp metering" period, and a negative number shows a decrease. The percentage of diverted volume is also calculated based on the "without" period.

Table 2-7. Diversion from USH 45 to Arterial Streets in First Fifteen Minutes

Cut Line	124th Street				USH 45				STH 100			
	Feb.	Mar.	Diversion	%	Feb.	Mar.	Diversion	%	Feb.	Mar.	Diversion	%
1	135	126	-9	-6.67%	1271	1279	8	0.63%	153	168	15	9.80%
2	232	230	-2	-0.86%	1394	1346	-48	-3.44%	369	407	38	10.30%
3	151	155	4	2.65%	1477	1465	-12	-0.81%	387	402	15	3.88%
4					1469	1515	46	3.13%	406	398	-8	-1.97%
5					1680	1657	-23	-1.37%	425	393	-32	-7.53%
6					1774	1710	-64	-3.61%	476	449	-27	-5.67%
7					1805	1785	-20	-1.11%	398	404	6	1.51%
8					1773	1752	-21	-1.18%	389	443	54	13.88%

Table 2-8. Diversion from USH 45 to Arterial Streets in Second Fifteen Minutes

Cut Line	124th Street				USH 45				STH 100			
	Feb.	Mar.	Diversion	%	Feb.	Mar.	Diversion	%	Feb.	Mar.	Diversion	%
1	128	106	-22	-17.19%	1275	1296	21	1.65%	156	173	17	10.90%
2	202	229	27	13.37%	1413	1321	-92	-6.51%	381	407	26	6.82%
3	140	149	9	6.43%	1422	1457	35	2.46%	367	406	39	10.63%
4					1391	1485	94	6.76%	370	405	35	9.46%
5					1602	1577	-25	-1.56%	372	404	32	8.60%
6					1645	1614	-31	-1.88%	429	453	24	5.59%
7					1840	1832	-8	-0.43%	365	441	76	20.82%
8					1764	1748	-16	-0.91%	379	494	115	30.34%

Table 2-9. Diversion from USH 45 to Arterial Streets in Third Fifteen Minutes

Cut Line	124th Street				USH 45				STH 100			
	Feb.	Mar.	Diversion	%	Feb.	Mar.	Diversion.	%	Feb.	Mar.	Diversion	%
1	155	151	-4	-2.58%	1407	1435	28	1.99%	187	215	28	14.97%
2	250	287	37	14.80%	1417	1414	-3	-0.21%	412	424	12	2.91%
3	188	185	-3	-1.60%	1512	1453	-59	-3.90%	433	441	8	1.85%
4					1495	1462	-33	-2.21%	450	458	8	1.78%
5					1628	1545	-83	-5.10%	465	474	9	1.94%
6					1721	1645	-76	-4.42%	507	542	35	6.90%
7					1761	1761	0	0.00%	407	459	52	12.78%
8					1717	1734	17	0.99%	416	497	81	19.47%

Table 2-10. Diversion from USH 45 to Arterial Streets in Fourth Fifteen Minutes

Cut Line	124th Street				USH 45				STH 100			
	Feb.	Mar.	Diversion	%	Feb.	Mar.	Diversion	%	Feb.	Mar.	Diversion	%
1	130	120	-10	-7.69%	1360	1342	-18	-1.32%	208	207	-1	-0.48%
2	223	261	38	17.04%	1423	1400	-23	-1.62%	404	412	8	1.98%
3	154	170	16	10.39%	1485	1517	32	2.15%	412	431	19	4.61%
4					1413	1484	71	5.02%	424	450	26	6.13%
5					1600	1620	20	1.25%	435	469	34	7.82%
6					1677	1676	-1	-0.06%	474	518	44	9.28%
7					1796	1790	-6	-0.33%	405	468	63	15.56%
8					1772	1737	-35	-1.98%	408	542	134	32.84%

Table 2-11. Diversion from USH 45 to Arterial Streets in Fifth Fifteen Minutes

Cut Line	124th Street				USH 45				STH 100			
	Feb.	Mar.	Diversion	%	Feb.	Mar.	Diversion	%	Feb.	Mar.	Diversion	%
1	159	151	-8	-5.03%	1355	1336	-19	-1.40%	206	225	19	9.22%
2	283	292	9	3.18%	1352	1350	-2	-0.15%	454	519	65	14.32%
3	175	179	4	2.29%	1408	1461	53	3.76%	444	500	56	12.61%
4					1381	1460	79	5.72%	457	481	24	5.25%
5					1583	1599	16	1.01%	469	463	-6	-1.28%
6					1662	1679	17	1.02%	573	566	-7	-1.22%
7					1758	1755	-3	-0.17%	452	478	26	5.75%
8					1746	1777	31	1.78%	433	527	94	21.71%

Table 2-12. Diversion from USH 45 to Arterial Streets in Sixth Fifteen Minutes

Cut Line	124th Street				USH 45				STH 100			
	Feb.	Mar.	Diversion	%	Feb.	Mar.	Diversion	%	Feb.	Mar.	Diversion	%
1	116	123	7	6.03%	1143	1063	-80	-7.00%	209	205	-4	-1.91%
2	250	249	-1	-0.40%	1192	1027	-165	-13.84%	391	409	18	4.60%
3	180	171	-9	-5.00%	1363	1343	-20	-1.47%	408	436	28	6.86%
4					1153	1300	147	12.75%	438	463	25	5.71%
5					1500	1489	-11	-0.73%	468	490	22	4.70%
6					1530	1510	-20	-1.31%	530	533	3	0.57%
7					1543	1702	159	10.30%	456	506	50	10.96%
8					1344	1385	41	3.05%	447	557	110	24.61%

Table 2-13. Diversion from USH 45 to Arterial Streets in 1.5 Hours

Cut Line	124th Street				USH 45				STH 100			
	Feb.	Mar.	Diversion	%	Feb.	Mar.	Diversion	%	Feb.	Mar.	Diversion	%
1	823	777	-46	-5.59%	7811	7751	-60	-0.77%	1119	1193	74	6.61%
2	1440	1548	108	7.50%	8191	7858	-333	-4.07%	2411	2578	167	6.93%
3	988	1009	21	2.13%	8667	8696	29	0.33%	2451	2616	165	6.73%
4					8302	8706	404	4.87%	2545	2655	110	4.32%
5					9593	9487	-106	-1.10%	2634	2693	59	2.24%
6					10009	9834	-175	-1.75%	2989	3061	72	2.41%
7					10503	10625	122	1.16%	2483	2756	273	10.99%
8					10116	10133	17	0.17%	2472	3060	588	23.79%

By checking the traffic volumes in Tables 2-7 through 2-13, it can be concluded that traffic diversion does take place between 124th street, USH 45 and STH 100 when the seven new ramp meters are operating, although the diversion is not large. The largest amount of diverted traffic from USH 45 in a 15 minute period is 165 vehicles. The largest total traffic diverted in the 1.5 peak hours is 333 vehicles. For STH 100, the largest volume increase in a fifteen-minute period is 134 vehicles, and the largest total increase in 1.5 hours is 588 vehicles. The result of Mn/DOT's study (Mn/DOT, 2001) shows an average decrease of 56 vehicles per studied parallel arterial with the meters off.

Not only can traffic be diverted from the freeway, traffic can also be attracted onto USH 45. In the "with ramp metering" period, the largest increase of volume in 1.5 hours is 404 vehicles. These results can be seen graphically in Appendix II.

Tables 2-7 through 2-13 show that traffic diverted from USH 45 is almost always less than 10%. The results from Kang and Gillen's study (1999) are that no more than 5-10% of vehicles will be diverted when ramp meters are turned on. It can also be found that with the same amount change of traffic volume, the percentage of change on the freeway is much lower than that on the arterial street, which is reasonable because the freeway has a much higher base volume.

Traffic diversion resulting from ramp metering is very uncertain and not consistent between cut lines. Even for one cut line, traffic diversion is not always consistent between time periods. A good example is Cut Line 8, which is located between Greenfield Avenue and Lincoln Avenue Traffic on STH 100 passing this cut line consistently increases more than 10% with meters on for all six fifteen-minute periods. However, traffic on USH 45 passing this cut line only diverts in three fifteen-minute periods and in the remaining three fifteen-minute periods traffic is attracted to the freeway.

Traffic diversion resulting from ramp metering is reasonably consistent with the theory underlying dynamic traffic assignment models, which can capture detailed spatial and temporal dynamics as reviewed in literature review. It should be noticed that fifteen minutes is an appropriate time period for the study of traffic diversion resulting from ramp metering, although shorter time periods could not be investigated. A period longer than fifteen minutes may tend to reduce the apparent impacts of traffic diversion.

Generally, the amount of traffic diverted from USH 45 is not equal to the total traffic increase on 124th street and STH 100. Moreover, the traffic volume increase on USH45 is not always equal to the total traffic decrease on 124th street and STH 100. It suggests that the alternate routes of the freeway are not always parallel to and close to the freeway. The alternate arterial streets can be far away from the freeway. Thus, a thorough study of the diversion impacts of ramp metering should be based on the whole roadway network system.

Considering that the actual traffic volume change in the network during the "without" and "with" periods may affect the results of the analysis, a percentage distribution of volume between 124th street, USH 45 and STH 100 on each cut line is calculated (Table 2-14). Table 2-14 shows a more consistent pattern of traffic diversion between 124th street, USH 45 and STH 100.

Table 2-14a. Percentage Distribution of Volume on Each Cut Line

	First 15 Minutes							
Cut Line	**124th Street**		**USH 45**		**STH 100**		**Total**	
	Feb.	**Mar.**	**Feb.**	**Mar.**	**Feb.**	**Mar.**	**Feb.**	**Mar.**
1	8.66%	8.01%	81.53%	81.31%	9.81%	10.68%	100.00%	100.00%
2	11.63%	11.60%	69.87%	67.88%	18.50%	20.52%	100.00%	100.00%
3	7.49%	7.67%	73.30%	72.45%	19.21%	19.88%	100.00%	100.00%
4			78.35%	79.19%	21.65%	20.81%	100.00%	100.00%
5			79.81%	80.83%	20.19%	19.17%	100.00%	100.00%
6			78.84%	79.20%	21.16%	20.80%	100.00%	100.00%
7			81.93%	81.54%	18.07%	18.46%	100.00%	100.00%
8			82.01%	79.82%	17.99%	20.18%	100.00%	100.00%

Table 2-14b. Percentage Distribution of Volume on Each Cut Line

	Second 15 Minutes							
Cut Line	124th Street		USH 45		STH 100		Total	
	Feb.	Mar.	Feb.	Mar.	Feb.	Mar.	Feb.	Mar.
1	8.21%	6.73%	81.78%	82.29%	10.01%	10.98%	100.00%	100.00%
2	10.12%	11.70%	70.79%	67.50%	19.09%	20.80%	100.00%	100.00%
3	7.26%	7.41%	73.72%	72.42%	19.03%	20.18%	100.00%	100.00%
4			78.99%	78.57%	21.01%	21.43%	100.00%	100.00%
5			81.16%	79.61%	18.84%	20.39%	100.00%	100.00%
6			79.32%	78.08%	20.68%	21.92%	100.00%	100.00%
7			83.45%	80.60%	16.55%	19.40%	100.00%	100.00%
8			82.31%	77.97%	17.69%	22.03%	100.00%	100.00%

	Third 15 Minutes							
Cut Line	124th Street		USH 45		STH 100		Total	
	Feb.	Mar.	Feb.	Mar.	Feb.	Mar.	Feb.	Mar.
1	8.86%	8.38%	80.45%	79.68%	10.69%	11.94%	100.00%	100.00%
2	12.03%	13.51%	68.16%	66.54%	19.82%	19.95%	100.00%	100.00%
3	8.81%	8.90%	70.89%	69.89%	20.30%	21.21%	100.00%	100.00%
4			76.86%	76.15%	23.14%	23.85%	100.00%	100.00%
5			77.78%	76.52%	22.22%	23.48%	100.00%	100.00%
6			77.24%	75.22%	22.76%	24.78%	100.00%	100.00%
7			81.23%	79.32%	18.77%	20.68%	100.00%	100.00%
8			80.50%	77.72%	19.50%	22.28%	100.00%	100.00%

	Fourth 15 Minutes							
Cut Line	124th Street		USH 45		STH 100		Total	
	Feb.	Mar.	Feb.	Mar.	Feb.	Mar.	Feb.	Mar.
1	7.66%	7.19%	80.09%	80.41%	12.25%	12.40%	100.00%	100.00%
2	10.88%	12.59%	69.41%	67.53%	19.71%	19.87%	100.00%	100.00%
3	7.51%	8.03%	72.40%	71.62%	20.09%	20.35%	100.00%	100.00%
4			76.92%	76.73%	23.08%	23.27%	100.00%	100.00%
5			78.62%	77.55%	21.38%	22.45%	100.00%	100.00%
6			77.96%	76.39%	22.04%	23.61%	100.00%	100.00%
7			81.60%	79.27%	18.40%	20.73%	100.00%	100.00%
8			81.28%	76.22%	18.72%	23.78%	100.00%	100.00%

Table 2-14c. Percentage Distribution of Volume on Each Cut Line

	Fifth 15 Minutes							
Cut Line	124th Street		USH 45		STH 100		Total	
	Feb.	Mar.	Feb.	Mar.	Feb.	Mar.	Feb.	Mar.
1	9.24%	8.82%	78.78%	78.04%	11.98%	13.14%	100.00%	100.00%
2	13.55%	13.51%	64.72%	62.47%	21.73%	24.02%	100.00%	100.00%
3	8.63%	8.36%	69.46%	68.27%	21.90%	23.36%	100.00%	100.00%
4			75.14%	75.22%	24.86%	24.78%	100.00%	100.00%
5			77.14%	77.55%	22.86%	22.45%	100.00%	100.00%
6			74.36%	74.79%	25.64%	25.21%	100.00%	100.00%
7			79.55%	78.59%	20.45%	21.41%	100.00%	100.00%
8			80.13%	77.13%	19.87%	22.87%	100.00%	100.00%

	Sixth 15 Minutes							
Cut Line	124th Street		USH 45		STH 100		Total	
	Feb.	Mar.	Feb.	Mar.	Feb.	Mar.	Feb.	Mar.
1	7.90%	8.84%	77.86%	76.42%	14.24%	14.74%	100.00%	100.00%
2	13.64%	14.78%	65.03%	60.95%	21.33%	24.27%	100.00%	100.00%
3	9.23%	8.77%	69.86%	68.87%	20.91%	22.36%	100.00%	100.00%
4			72.47%	73.74%	27.53%	26.26%	100.00%	100.00%
5			76.22%	75.24%	23.78%	24.76%	100.00%	100.00%
6			74.27%	73.91%	25.73%	26.09%	100.00%	100.00%
7			77.19%	77.08%	22.81%	22.92%	100.00%	100.00%
8			75.04%	71.32%	24.96%	28.68%	100.00%	100.00%

Statistical Significance Tests

In order to identify the statistical significance level of traffic volume differences observed on USH 45 and the arterial streets in the "without" and "with" study periods, chi-square tests are conducted. Table 2-15 shows the results. Forty-eight chi-square tests are conducted based on the total traffic volume counts of three PM peak periods on each of the eight cut lines in each of the six 15 minute periods. For nineteen of them, traffic diversion resulting from ramp metering is statistically significant with a 95% confidence interval.

Table 2-15. Results of Chi-Square Tests

Cut Line	First 15 Minutes		Second 15 Minutes		Third 15 Minutes		Fourth 15 Minutes		Fifth 15 Minutes		Sixth 15 Minutes	
	χ^2	*df*	χ^2	*df*	χ^2	*df*	χ^2	*df*	χ^2	*df*	χ^2	*df*
1	2.92	2	9.11	2	4.55	2	0.82	2	3.48	2	3.22	2
2	8.02	2	15.78	2	6.67	2	9.36	2	9.80	2	19.38	2
3	1.12	2	2.78	2	1.74	2	1.45	2	3.85	2	3.92	2
4	1.22	1	0.29	1	0.83	1	0.06	1	0.01	1	2.05	1
5	2.05	1	4.51	1	2.78	1	2.09	1	0.28	1	1.55	1
6	0.26	1	2.81	1	7.51	1	4.58	1	0.32	1	0.21	1
7	0.34	1	18.46	1	7.52	1	11.49	1	1.83	1	0.02	1
8	10.14	1	38.98	1	15.22	1	51.18	1	17.99	1	19.71	1

Shaded areas are statistically significant with a 95% confidence interval.
χ^2 — Chi-Square; df — Degree of Freedom

It can be concluded that traffic diversion does take place between 124th Street, USH 45 and STH 100 when the seven new ramp meters are operating, although the diversion is not large. Traffic diverted from the freeway is almost always less than 10%.

Diversion between Entrance Ramps

The spatial traffic diversion of ramp metering involves the diversion of trips from the freeway to alternate surface network routes. Traffic diverted from one specific ramp may come back by entering the freeway through different downstream ramps or keep staying on the arterial streets never coming back to the freeway. This section analyzes the distribution of the traffic entering USH 45 between different entrance ramps in both the “without” and “with” period.

Entering Traffic Distribution Pattern between On-Ramps

Depending on the data availability and the potential traffic distribution pattern between on-ramps, eight entrance ramps are identified within the study corridor. Table 2-16 illustrates the average traffic diverted on these entrance ramps in each 15 minute period for both the “without” and the “with” peak periods. Based on the “without ramp metering” period, a positive diverted volume shows an increase of entering traffic in the “with ramp metering” period, and a negative number shows a decrease. The percentage of diverted volume is also calculated based on the “without” period. Considering that the actual traffic volume change in the network during the “without” and “with” periods may affect the results of the analysis, a percentage distribution of volume between these entrance ramps is calculated (Table 2-17).

Table 2-16. Average On-Ramp Volume — Vehicles Per 15 Minutes For All Lanes

Ramp Location	First 15 Minutes				Second 15 Minutes			
	Feb.	Mar.	Diversion	%	Feb.	Mar.	Diversion	%
On from County Q	116	109	-7	-6.03%	95	95	0	0.00%
On from Pilgrim Road	108	111	3	2.78%	93	92	-1	-1.08%
On from Main Street	145	150	5	3.45%	127	137	10	7.87%
On from Silver Spring Drive	281	294	13	4.63%	247	289	42	17.00%
On from North Avenue	164	160	-4	-2.44%	153	148	-5	-3.27%
On from Watertown Plank Road	154	167	13	8.44%	142	149	7	4.93%
On from Wisconsin Avenue	148	146	-2	-1.35%	112	105	-7	-6.25%
On from Greenfield Avenue	135	139	4	2.96%	113	132	19	16.81%
Ramp Location	**Third 15 Minutes**				**Fourth 15 Minutes**			
	Feb.	**Mar.**	**Diversion**	**%**	**Feb.**	**Mar.**	**Diversion**	**%**
On from County Q	107	105	-2	-1.87%	106	107	1	0.94%
On from Pilgrim Road	120	108	-12	-10.00%	83	83	0	0.00%
On from Main Street	212	201	-11	-5.19%	128	140	12	9.38%
On from Silver Spring Drive	324	349	25	7.72%	241	280	39	16.18%
On from North Avenue	188	163	-25	-13.30%	138	136	-2	-1.45%
On from Watertown Plank Road	146	150	4	2.74%	138	148	10	7.25%
On from Wisconsin Avenue	163	162	-1	-0.61%	152	153	1	0.66%
On from Greenfield Avenue	154	157	3	1.95%	143	150	7	4.90%
Ramp Location	**Fifth 15 Minutes**				**Sixth 15 Minutes**			
	Feb.	**Mar.**	**Diversion**	**%**	**Feb.**	**Mar.**	**Diversion**	**%**
On from County Q	122	132	10	8.20%	109	114	5	4.59%
On from Pilgrim Road	95	110	15	15.79%	71	80	9	12.68%
On from Main Street	132	152	20	15.15%	109	119	10	9.17%
On from Silver Spring Drive	274	317	43	15.69%	230	275	45	19.57%
On from North Avenue	165	159	-6	-3.64%	167	109	-58	-34.73%
On from Watertown Plank Road	153	122	-31	-20.26%	120	113	-7	-5.83%
On from Wisconsin Avenue	147	159	12	8.16%	113	91	-22	-19.47%
On from Greenfield Avenue	157	157	0	0.00%	132	131	-1	-0.76%

Shaded areas are entrance ramps without a meters

Table 2-17. Percentage Distribution of Volume on Each Entrance Ramp

Ramp Location	First 15 Minutes		Second 15 Minutes		Third 15 Minutes	
	Feb.	Mar.	Feb.	Mar.	Feb.	Mar.
On from County Q	9.26%	8.52%	8.78%	8.31%	7.55%	7.55%
On from Pilgrim Road	8.65%	8.67%	8.57%	8.02%	8.49%	7.74%
On from Main Street	11.61%	11.78%	11.77%	11.91%	15.02%	14.43%
On from Silver Spring Drive	22.47%	23.01%	22.81%	25.19%	22.90%	25.01%
On from North Avenue	13.10%	12.54%	14.10%	12.93%	13.28%	11.66%
On from Watertown Plank Road	12.30%	13.11%	13.13%	12.99%	10.33%	10.75%
On from Wisconsin Avenue	11.82%	11.47%	10.36%	9.12%	11.53%	11.59%
On from Greenfield Avenue	10.78%	10.89%	10.48%	11.53%	10.90%	11.28%
Total	100.00%	100.00%	100.00%	100.00%	100.00%	100.00%
Ramp Location	**Fourth 15 Minutes**		**Fifth 15 Minutes**		**Sixth 15 Minutes**	
	Feb.	**Mar.**	**Feb.**	**Mar.**	**Feb.**	**Mar.**
On from County Q	9.36%	8.92%	9.82%	10.07%	10.33%	11.02%
On from Pilgrim Road	7.38%	6.94%	7.65%	8.41%	6.78%	7.75%
On from Main Street	11.37%	11.73%	10.57%	11.62%	10.33%	11.53%
On from Silver Spring Drive	21.32%	23.40%	22.02%	24.26%	21.89%	26.66%
On from North Avenue	12.25%	11.37%	13.27%	12.13%	15.91%	10.57%
On from Watertown Plank Road	12.19%	12.34%	12.28%	9.33%	11.41%	10.95%
On from Wisconsin Avenue	13.43%	12.79%	11.80%	12.16%	10.77%	8.82%
On from Greenfield Avenue	12.70%	12.51%	12.60%	12.03%	12.58%	12.70%
Total	100.00%	100.00%	100.00%	100.00%	100.00%	100.00%

Shaded areas are entrance ramps without a meters

By checking the traffic volumes in Table 2-16 and the percentages in Table 2-17, it can be concluded that traffic diversion does take place between different entrance ramps when the seven new ramp meters are operating, and the diversion is not small. It is important to observe that the only consistent increase of entering traffic in the "with ramp metering" periods is at the on-ramp from Silver Spring Drive, which is the only entrance ramp without a ramp meter among the fourteen on-ramps in the studied southbound USH 45 section. Furthermore, the entering traffic at the on-ramp from North Avenue, which is an on-ramp south of the on-ramp from Silver Spring Drive, decreases consistently. These results can be seen graphically in Appendix III.

These results suggest that traffic entering the freeway could be diverted from high volume, substandard, or other problem ramps to a more desirable entrance. More than 10% of vehicles can be diverted between different entrance ramps. Figures 2-5 through 2-10 present the average percentage distribution of on-ramp volume in each of the six 15 minute periods for both the "without" and the "with" peak periods. Obviously, a consistent traffic diversion pattern is presented.

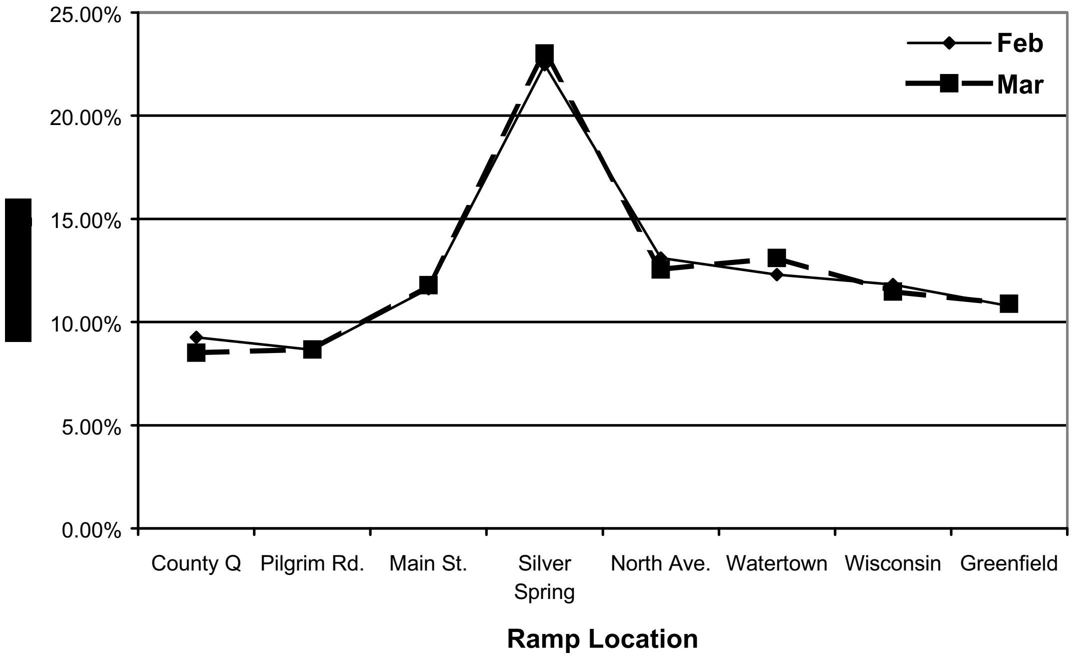


Figure 2-5. Percentage Distribution of On-Ramp Volume in First 15 Minutes

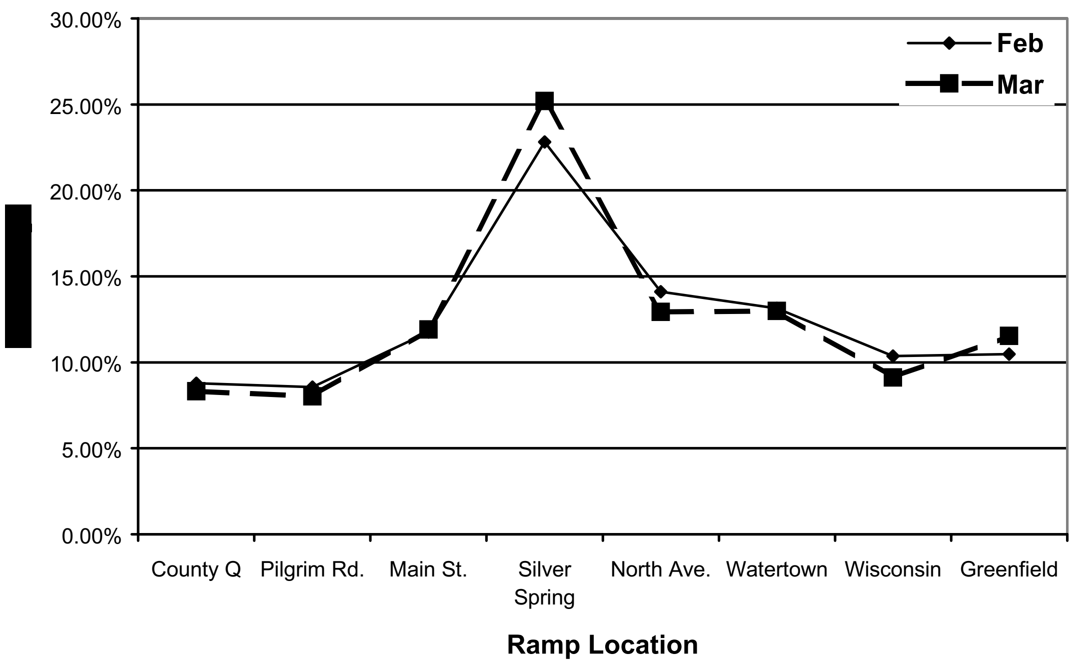


Figure 2-6. Percentage Distribution of On-Ramp Volume in Second 15 Minutes

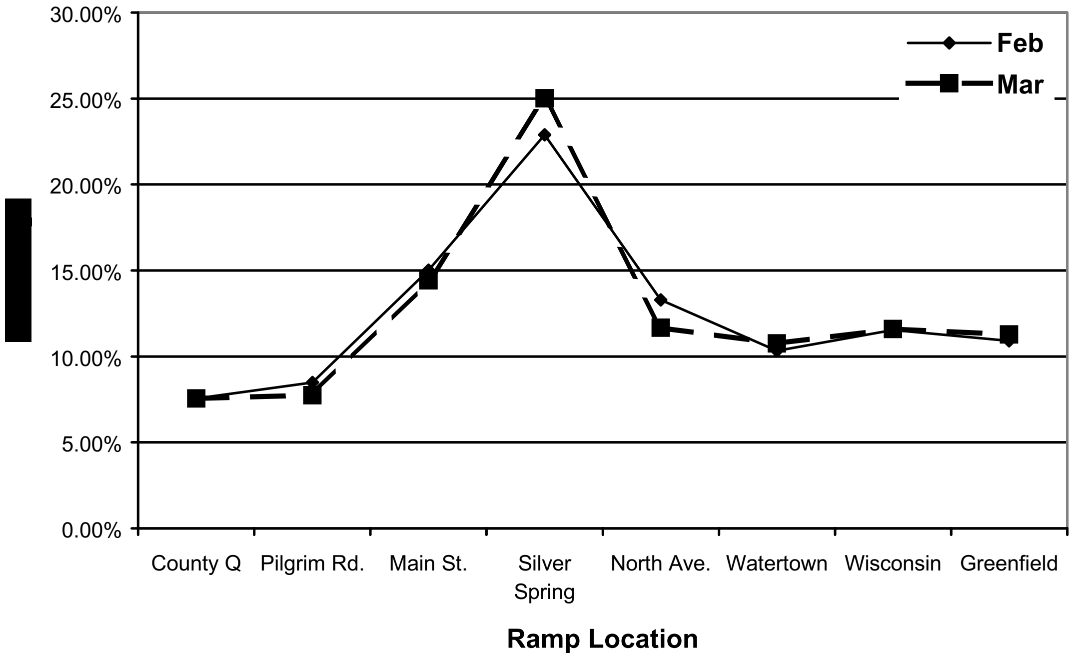


Figure 2-7. Percentage Distribution of On-Ramp Volume in Third 15 Minutes

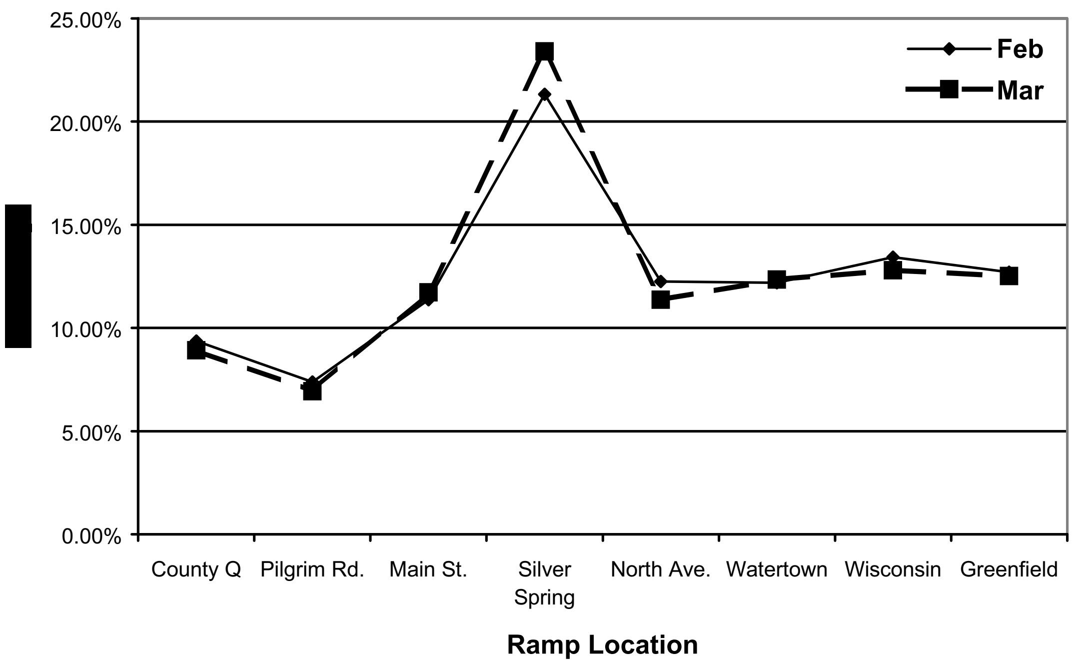


Figure 2-8. Percentage Distribution of On-Ramp Volume in Fourth 15 Minutes

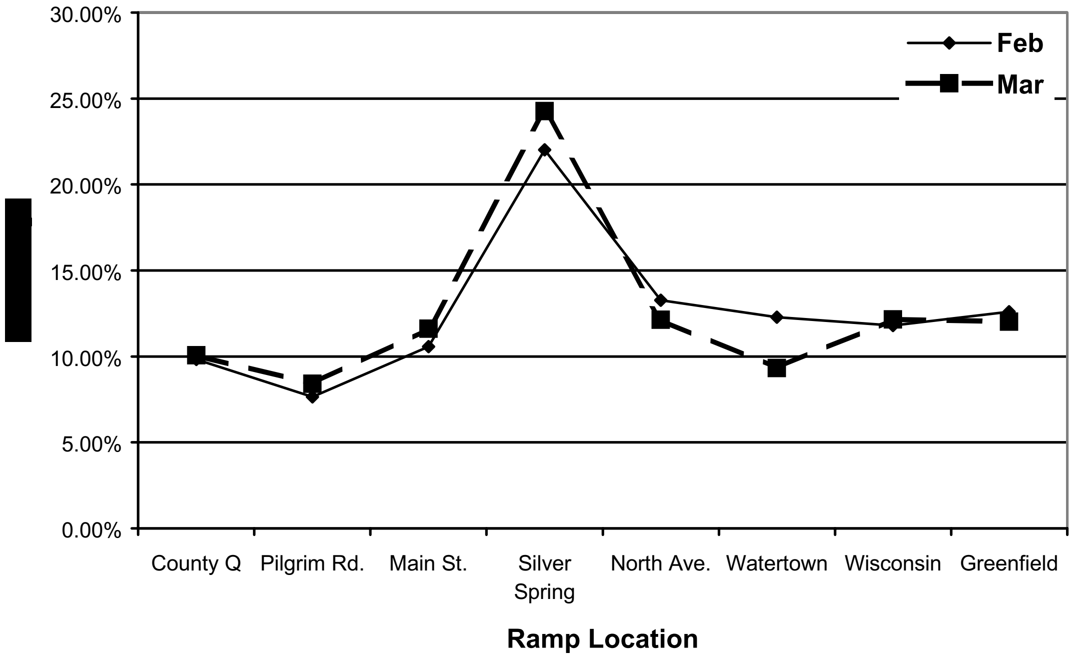


Figure 2-9. Percentage Distribution of On-Ramp Volume in Fifth 15 Minutes

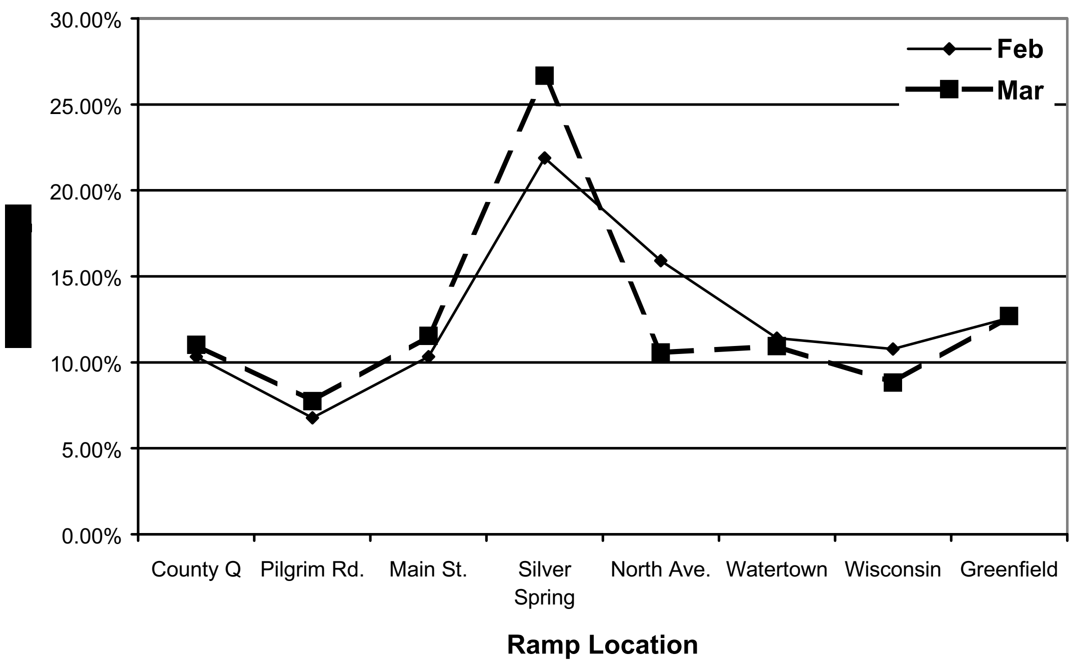


Figure 2-10. Percentage Distribution of On-Ramp Volume in Sixth 15 Minutes

Statistical Significance Tests

In order to identify the statistical significance level of traffic diversion observed at entrance ramps in the studied southbound USH 45 section in the "without" and "with" study periods, chi-square tests were conducted. Table 2-18 shows the calculation results. Six chi-square tests were based on the total traffic volume counts of three PM peak periods on each of the eight entrance ramps in each of the six 15 minute periods. For two (or one third) of them, traffic diversion between different entrance ramps was statistically significant with a 95% confidence interval.

Table 2-18. Results of Chi-Square Tests, Diversion between Entrance Ramps

15 Minute Periods	χ^2	df
1	3.11	7
2	11.20	7
3	10.83	7
4	6.03	7
5	25.67	7
6	59.10	7

Shaded areas are statistically significant with a 95% confidence interval.
χ^2 — Chi-Square; df — Degree of Freedom

In order to identify the statistical significance level of the relationship between on-ramps at Silver Spring Drive and North Avenue, six more chi-square tests are conducted. In fact, there are three entrance ramps between on-ramps at Silver Spring Drive and North Avenue. They are on-ramps at Hampton Avenue, Capitol Drive, and Burleigh Street. Because of the reasons mentioned above, the data obtained at these locations are not appropriate to be used to conduct the analysis. Therefore, these locations are excluded from the evaluation. However, the results of the evaluation should be the same and the traffic diversion pattern may only be a little slighter, because the on-ramp at North Avenue is farther from the on-ramp at Silver Spring Drive than those three on-ramps. As a result, the on-ramp at Silver Spring Drive should have less effect on the on-ramp at North Avenue.

Table 2-19 shows the results of calculation. For four of six 15 minute periods, traffic diversion from the North Avenue on-ramp to the Silver Spring Drive on-ramp is statistically significant with a 95% confidence interval. It indicates that traffic traveling to the south on the arterial streets will tend to enter the freeway earlier in order to avoid the on-ramp delay at the entrance ramps in the south side.

Table 2-19. Results of Chi-Square Tests, North Avenue to Silver Spring Drive

15 Minute Periods	χ^2	df
1	0.76	1
2	5.23	1
3	8.13	1
4	3.64	1
5	5.24	1
6	47.96	1

Shaded areas are statistically significant with a 95% confidence interval.
χ^2 — Chi-Square; df — Degree of Freedom

Temporal Diversion

Ramp metering can also result in temporal diversion, where drivers shift ramp arrival time, earlier or later. Thus, flow peaks can be spread out over a longer period resulting in better freeway capacity utilization.

Because on-ramp traffic volume data were only collected during the peak periods, USH 45 mainline traffic volume data are used to conduct the temporal diversion analysis. Two cut lines were placed according to the data availability. One cut line was located between Hampton Avenue and Capitol Drive, and the other one was located on Watertown Plank Road Table 2-20 summarizes the traffic volume counts passing the two cut lines.

Figure 2-11 and Figure 2-12 present the temporal traffic diversion at each of the two locations. The traffic volume are averages of three PM periods in both the “without” and the “with” periods and are in vehicles per hour per lane. Slightly significant temporal variation is indicated, but the results were difficult to interpret as temporal diversion.

Statistical Significance Tests

In order to identify the statistical significance level of temporal variation observed in the studied southbound USH 45 section in the “without” and “with” study periods, chi-square tests were conducted with SPSS (Table 2-21). Two chi-square tests were conducted based on the total traffic volume counts of three PM periods in both the “without” and the “with” periods. For both of them, temporal variation was statistically significant with a 95% confidence interval.

Table 2-20. Average Traffic Volume Counts —Vehicles Per Hour Per Lane

Time	Hampton & Capitol		Watertown Plank	
	Feb. PM	Mar. PM	Feb. PM	Mar. PM
14:00	1332	1357	1703	1647
14:15	1396	1338	1687	1753
14:30	1646	1615	1838	1962
14:45	1561	1510	1958	2018
15:00	1700	1661	2004	1922
15:15	1741	1716	2093	2142
15:30	1913	1819	2027	1975
15:45	1852	1851	2084	2070
16:00	1649	1664	1945	1951
16:15	1754	1762	1941	1974
16:30	1855	1880	1946	1872
16:45	1849	1822	1971	1977
17:00	1774	1749	1845	1895
17:15	1785	1860	1805	1835
17:30	1623	1664	1589	1838
17:45	1515	1513	1582	1710
18:00	1385	1447	1601	1637
18:15	1305	1391	1583	1609
18:30	1142	1181	1392	1508
18:45	1015	1061	1304	1353
19:00	855	925	1188	1186
19:15	784	845	1144	1157
19:30	670	707	927	986
19:45	590	632	855	901

Table 2-21. Results of Chi-Square Tests

Cut Lines	χ^2	df
Between Hampton Ave. and Capitol Dr.	60.98	23
At Watertown Plank Road	107.43	23

Shaded areas are statistically significant with a 95% confidence interval.

χ^2 — Chi-Square; df — Degree of Freedom

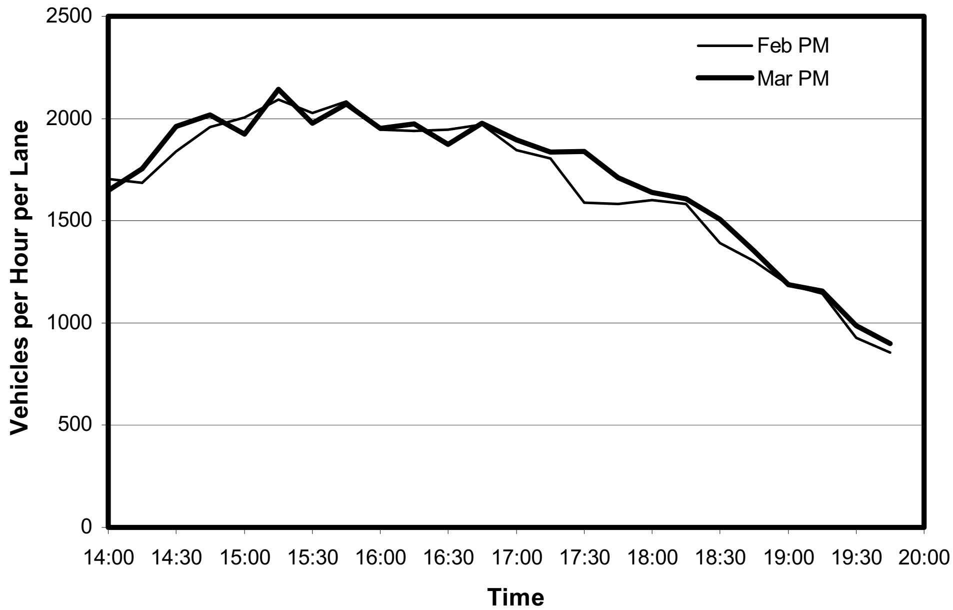


Figure 2-11. Temporal Diversion on USH 45 at Watertown Plank Road

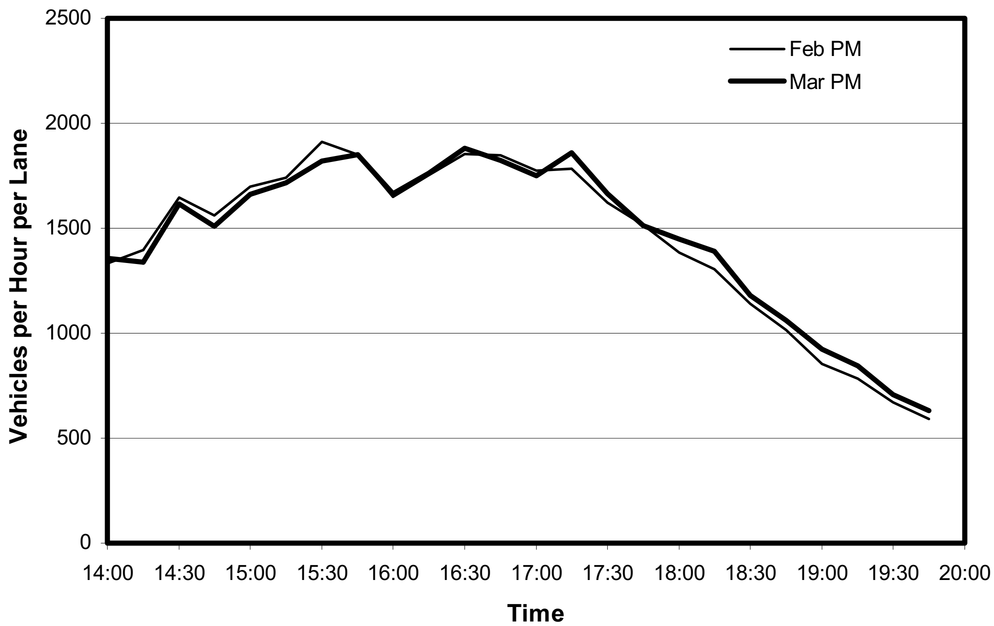


Figure 2-12. Temporal Diversion on USH 45 between Hampton Avenue and Capitol Drive

Temporal Diversion, Whole Cutline

Figures 2-13 and 2-14 show the sum of all 15-minute counts for streets at the first and eighth cut lines. The other cut lines are displayed in the appendix. These graphs do not extend outside the peak, so it is more difficult to use them to confirm peak spreading than Figures 2-10 and 2-11. In Figure 2-13, the before and after volumes are nearly identical, suggesting that no peak spreading is occurring. In Figure 2-14, the before and after curves are almost parallel, again suggesting that peak spreading is absent. The other cutlines (see appendix) show a greater random variation, but the conclusion about peak spreading is the same.

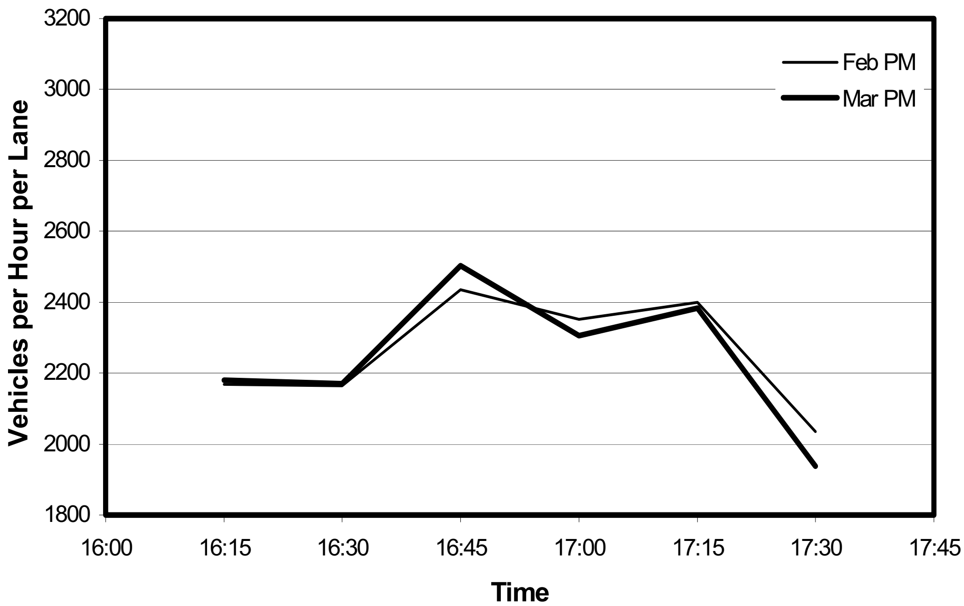


Figure 2-13. Temporal Distribution of Counts at Cutline 1

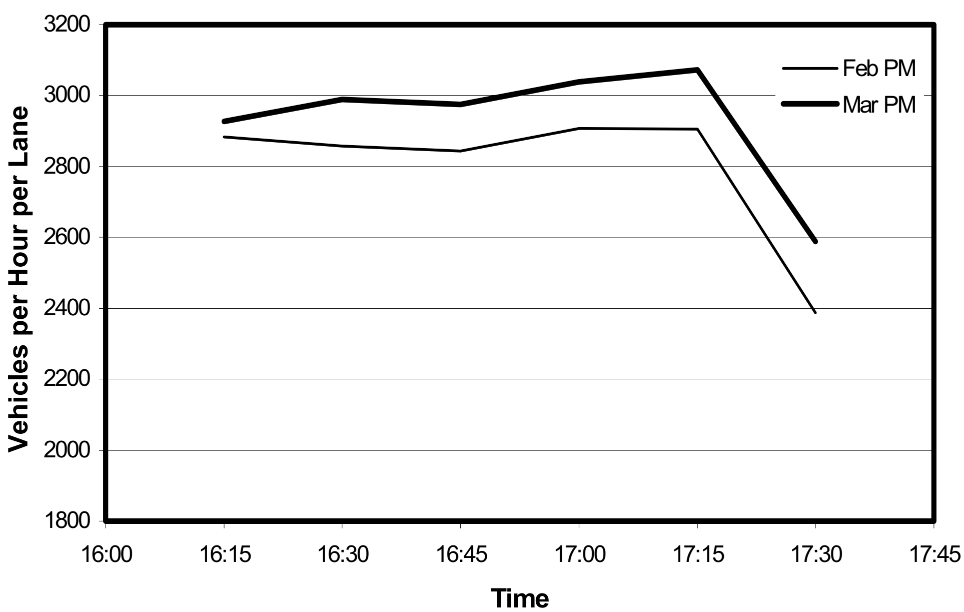


Figure 2-14. Temporal Distribution of Counts at Cutline 8

Combination of Temporal and Spatial Diversion

Two analyses of variance (ANOVA) were performed on the traffic volumes to determine whether the interpretations from the chi square tests remained valid when combinations of factors are considered together. The first analysis of variance deals with volumes at all cutlines during the 6 15 minute intervals. These four factors were defined:

Metered: before period; after period (2 levels)
Cutline: #1, #2, #3, #4, #5, #6, #7, #8 (8 levels)
Time: 1st 15 minute, 2nd 15 minute, 3rd 15 minute, 4th 15 minute, 5th 15 minute, 6th 15 minute (6 levels)
Street: US 45, STH 100, 124th St (3 levels)

Generally, there were three replications per cell; however, cells for 124th street at cutlines 4-8 were empty for all periods and intervals. Table 2-22 summarizes the results for first order effects and second order interactions. Of greatest interest to this study are the effects and interactions that involve the "metered" factor. "Metered" is significant by itself and with "cutline" and "street". It is not significant with "time". Third and fourth order interactions were not interesting and have been omitted from this report. This table is further evidence that spatial and temporal diversions are occurring because of ramp meters and is entirely consistent with the chi square tests.

Table 2-22. Analysis of Variance of Cutlines

Tests of Between-Subjects Effects

Dependent Variable: VOLUME

Source	Type III Sum of Squares	df	Mean Square	F	Sig.
Corrected Model	246017206[a]	83	2964062.719	962.351	.000
Intercept	312883609	1	312883608.8	101584.8	.000
STREET	209684346	2	104842172.9	34039.407	.000
CUTLINE	7397175.528	7	1056739.361	343.095	.000
TIME	296992.095	5	59398.419	19.285	.000
METERED	29373.936	1	29373.936	9.537	.002
STREET * CUTLINE	2532796.779	9	281421.864	91.370	.000
STREET * TIME	1350198.158	10	135019.816	43.837	.000
STREET * METERED	40510.207	2	20255.103	6.576	.001
CUTLINE * TIME	251899.682	35	7197.134	2.337	.000
CUTLINE * METERED	73128.537	7	10446.934	3.392	.001
TIME * METERED	17804.766	5	3560.953	1.156	.330
Error	1848014.073	600	3080.023		
Total	735893348	684			
Corrected Total	247865220	683			

a. R Squared = .993 (Adjusted R Squared = .992)

The second analysis of variance concerns the data used to draw Figures 5 and 10 and involves three factors. "Metered" and "time" are the same as before. The third factor, "location" refers to on-ramps and is defined as follows:

Location: County Q, Pilgrim, Main, Silver Spring, North, Watertown, Wisconsin, Greenfield (8 levels)

The analysis of variance statistics, Table 2-22, show that "metered" was barely significant, but the interaction between "metered" and "location" is much stronger. Again, this result is consistent with the chi-square tests.

Table 2-22. Analysis of Variance of On-Ramps

Tests of Between-Subjects Effects

Dependent Variable: VOLUME

Source	Type III Sum of Squares	df	Mean Square	F	Sig.
Corrected Model	940137.667[a]	60	15668.961	36.048	.000
Intercept	6549485.281	1	6549485.281	15067.724	.000
LOCATION	814048.302	7	116292.615	267.542	.000
TIME	67360.615	5	13472.123	30.994	.000
METERED	1686.837	1	1686.837	3.881	.050
LOCATION * TIME	43728.635	35	1249.390	2.874	.000
LOCATION * METERED	10703.302	7	1529.043	3.518	.001
TIME * METERED	2609.976	5	521.995	1.201	.310
Error	98670.052	227	434.670		
Total	7588293.000	288			
Corrected Total	1038807.719	287			

a. R Squared = .905 (Adjusted R Squared = .880)

Conclusion

Summary

Ramp meters are traffic signals on freeway entrance ramps used to control the rate of vehicles entering the freeway so that demand stays below capacity. Implementation of ramp metering to control freeway traffic can bring both positive and negative impacts. A major and controversial issue that is raised in connection with ramp metering is the potential diversion of freeway trips to adjacent arterial streets. Depending on traffic conditions on arterial streets, the diversion can be a positive benefit or be a negative impact.

Ramp metering can be operated under a diversionary strategy or a nondiversionary strategy. The objective of metering with a diversionary strategy is to cause freeway traffic to divert. The traffic diversion resulting from ramp metering can be divided into three types: spatial diversion, temporal diversion, and modal diversion. Spatial diversion involves the diversion of trips from the freeway to alternate surface network routes and from high volume, substandard, or other problem ramps to a more desirable entrance.

A review of literature in Chapter 1 shows that simulation studies and field measurements place the vehicle hours of travel reductions owing to ramp metering at about 10%, at most. Diversion is essential to achieve positive benefits. Drivers are reluctant to divert from preplanned routes, in reaction to traffic congestion. Long-term route selection is based primarily on travel time. Fairness is a criterion when establishing ramp metering, which may undermine time-savings benefits.

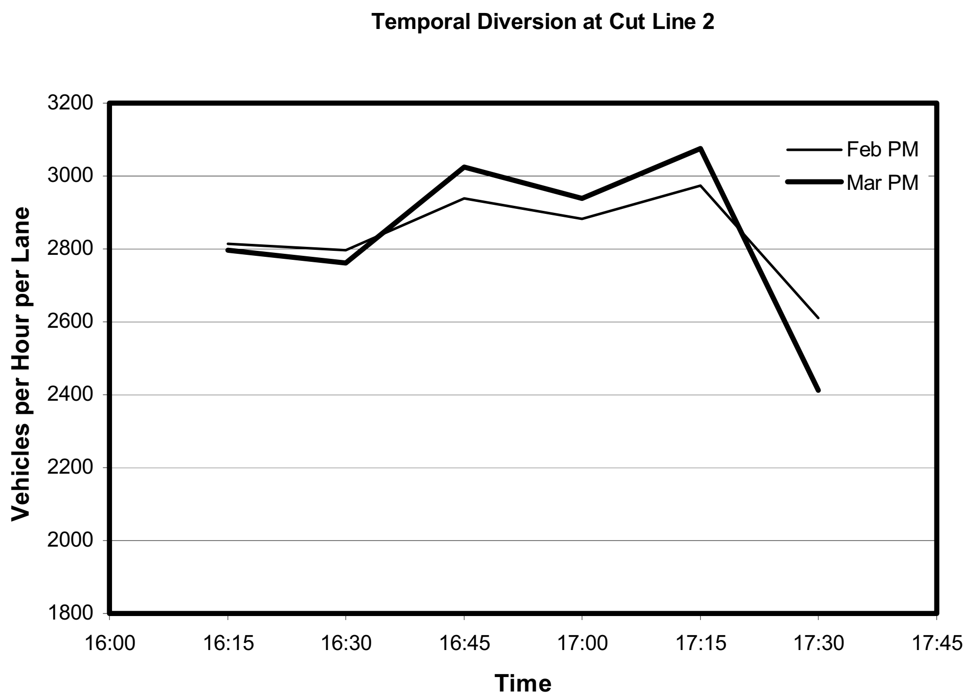
Findings

Diverting traffic should be an objective of metering where it is feasible. Such an action requires a thorough analysis of the alternate routes and the impacts of diversion on those routes, and improvements on the alternate routes when and where they are needed.

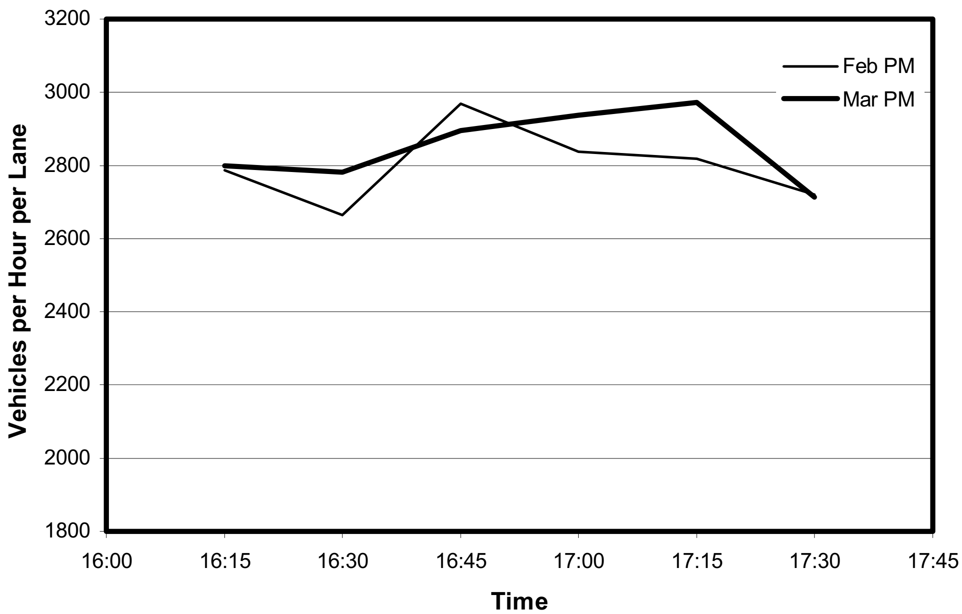
This study presents a field evaluation study of the traffic diversion resulting from ramp metering. The studied roadway network is the southbound USH45 section from County Q (Washington/Waukesha County Line) to Lincoln Avenue, and two parallel arterial streets, STH 100 and 124th street. The study occurred in February 2000 as a "without ramp metering" period and March 2000 as a "with ramp metering" period. Findings from this study can be summarized as follows:

- Spatial diversion from the freeway to the alternate routes and from substandard ramps to a more desirable entrance took place within the studied roadway network. Slightly significant temporal variation was found, but the results were difficult to interpret as temporal diversion.
- When attractive and efficient alternate routes are available, traffic will be diverted from the freeway to the alternate routes by ramp metering. Traffic will also be diverted to an entrance ramp where the delay is obviously less than other entrances. In this study, traffic was diverted from metered on-ramps to an entrance ramp without ramp metering.
- The largest amount of diverted traffic from USH 45 in a fifteen-minute period is 165 vehicles. The largest amount total trips diverted in the 1.5 peak hours is 333 vehicles. Trips diverted from USH 45 are almost always less than 10%.
- Not only can traffic be diverted from the freeway, traffic can also be attracted onto USH 45. In the "with ramp metering" period, the largest increase of volume in 1.5 hours is 404 vehicles.
- With the same amount change of traffic volume, the percentage of change on the freeway was much lower than that on the arterial street, which is reasonable because the freeway has a much higher base volume.
- Traffic diversion resulting from ramp metering was reasonably consistent with the theory underlying dynamic traffic assignment models.
- Fifteen minutes seems to be an appropriate time period for the study of traffic diversion resulting from ramp metering. A period longer than fifteen minutes may tend to reduce the apparent impacts of traffic diversion.

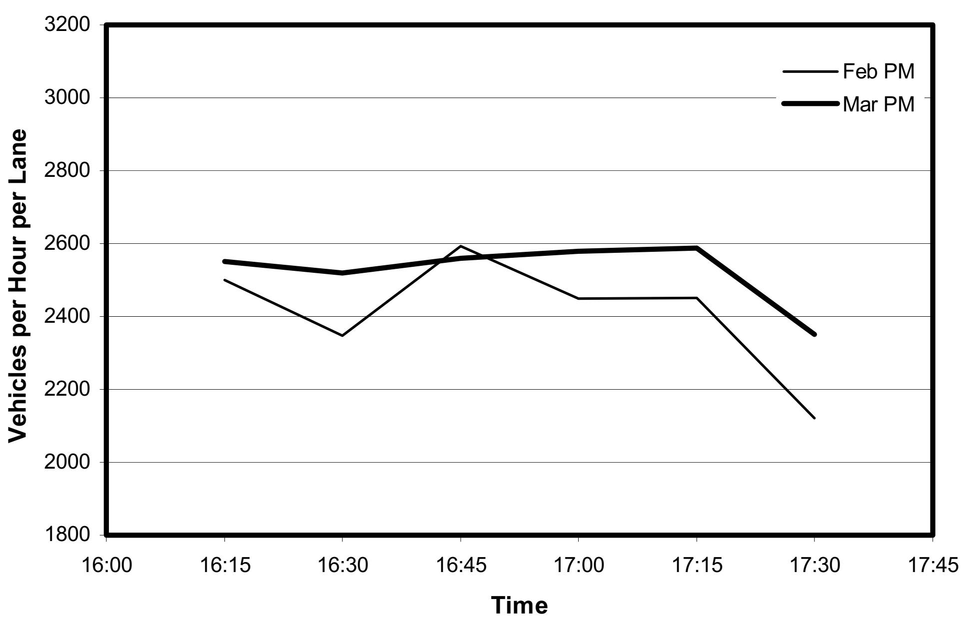
Appendix 2-1: Temporal Distribution of Traffic at Cutlines 2 to 7

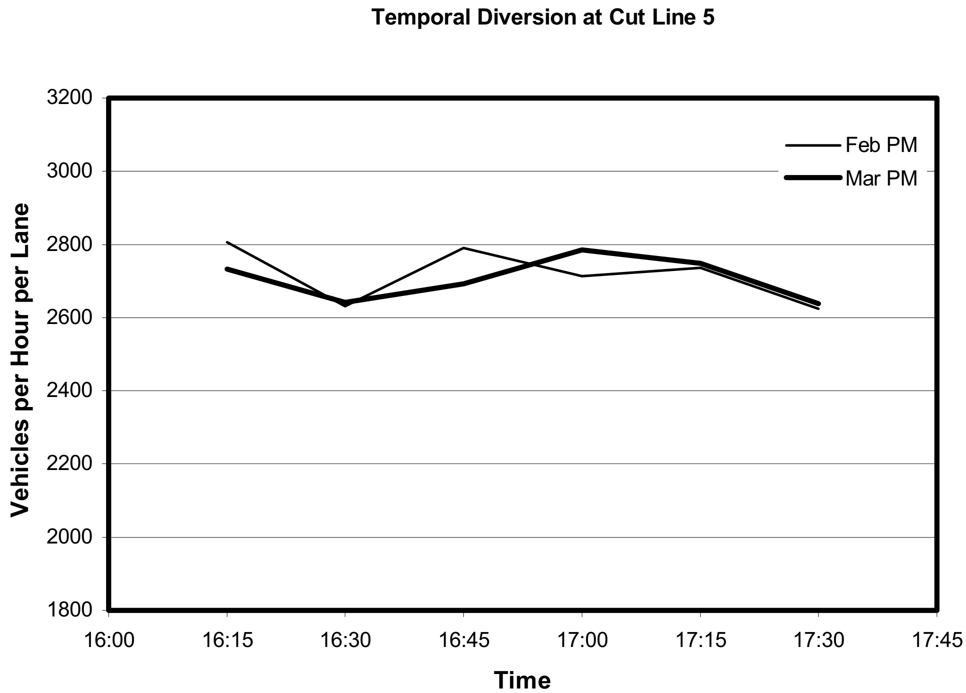


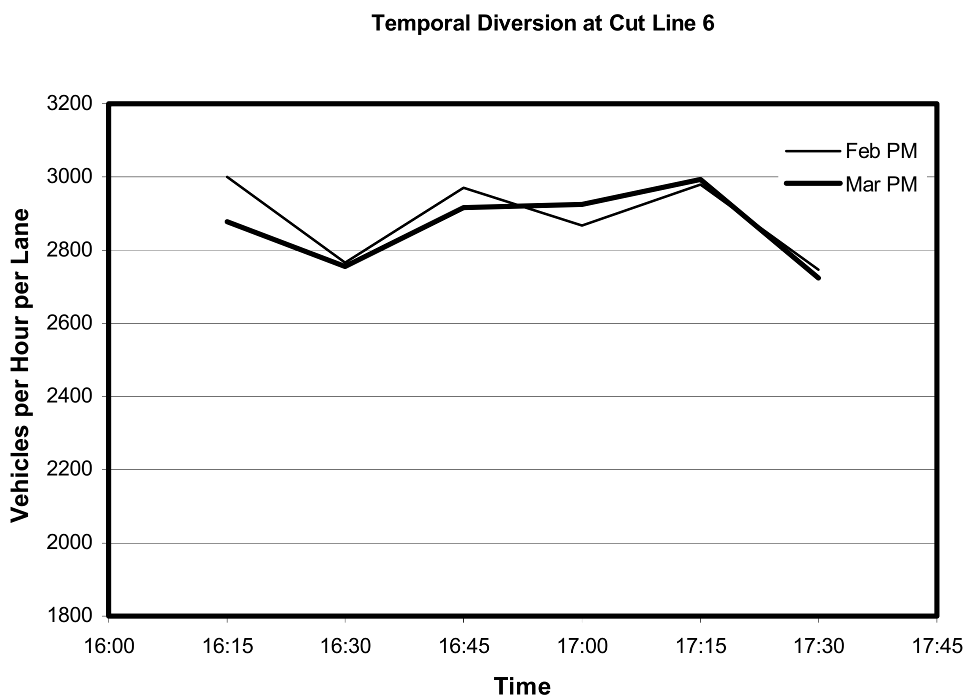
Temporal Diversion at Cut Line 3

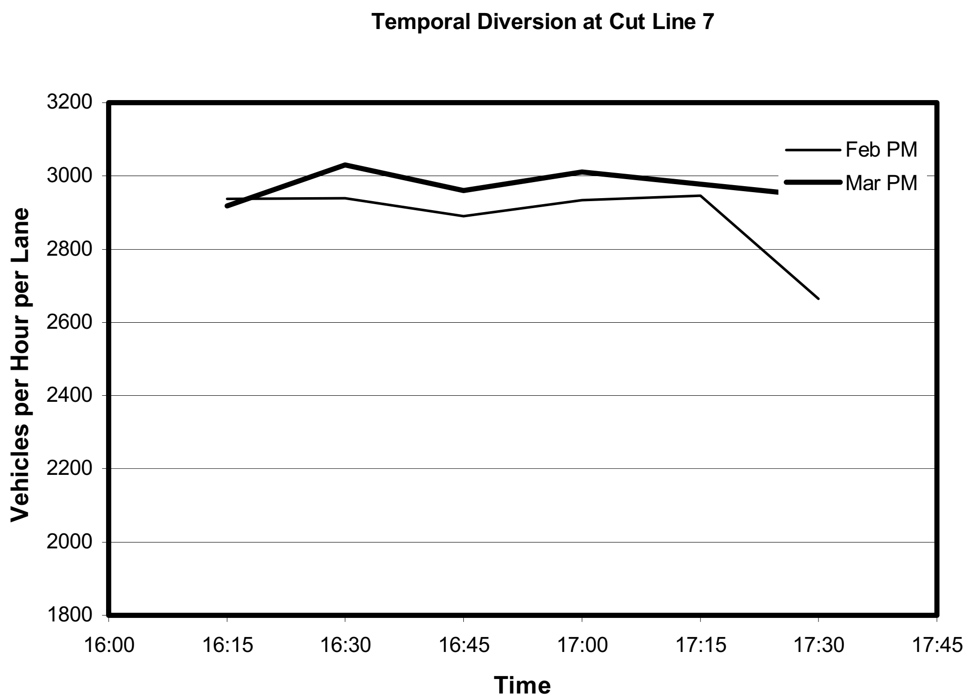


Temporal Diversion at Cut Line 4



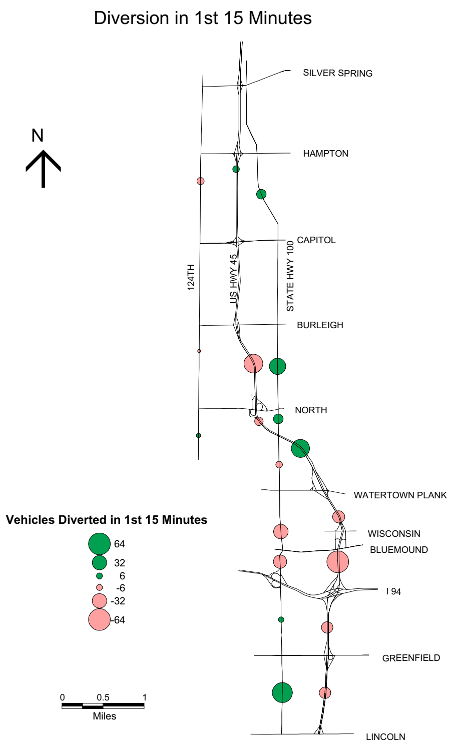

Temporal Diversion at Cut Line 5
Vehicles per Hour per Lane
3200
3000
2800
2600
2400
2200
2000
1800
16:00
16:15
16:30
16:45
17:00
17:15
17:30
17:45
Time
Feb PM
Mar PM

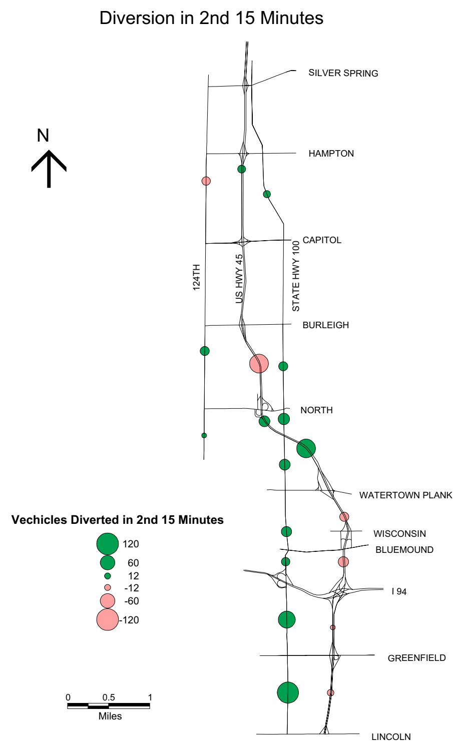

Temporal Diversion at Cut Line 6
Vehicles per Hour per Lane
3200
3000
2800
2600
2400
2200
2000
1800
16:00
16:15
16:30
16:45
17:00
17:15
17:30
17:45
Time
Feb PM
Mar PM


Temporal Diversion at Cut Line 7
3200
3000
2800
2600
2400
2200
2000
1800
Vehicles per Hour per Lane
Feb PM
Mar PM
16:00
16:15
16:30
16:45
17:00
17:15
17:30
17:45
Time

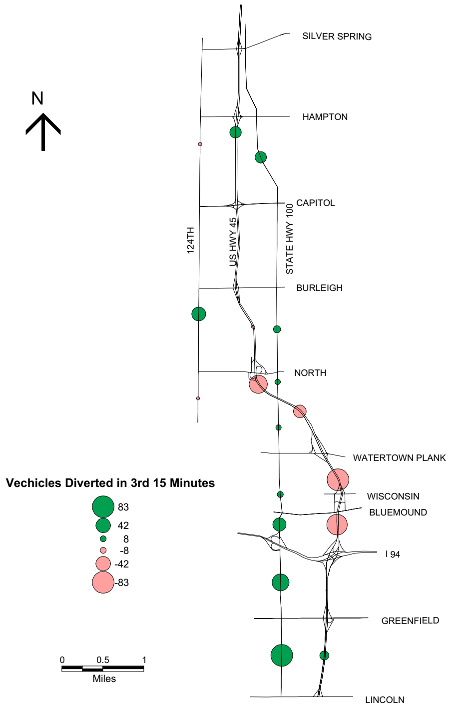
Appendix 2-2: Volume Changes on US 45 and Parallel Arterials

Increases in volumes between the before and after periods on the segments are shown in dark gray. Decreases are shown in light gray.

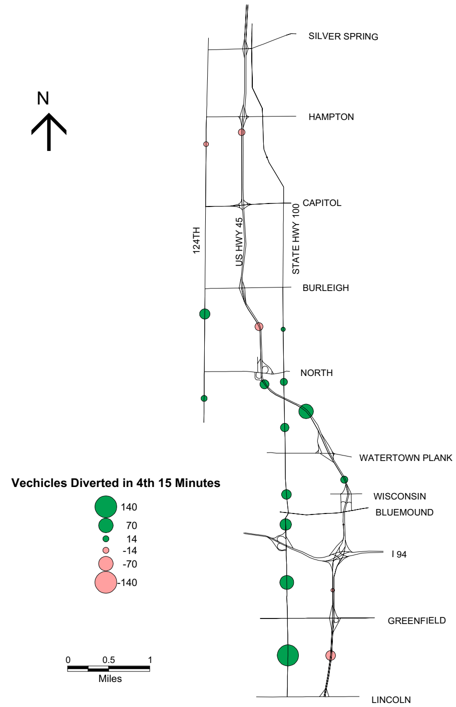




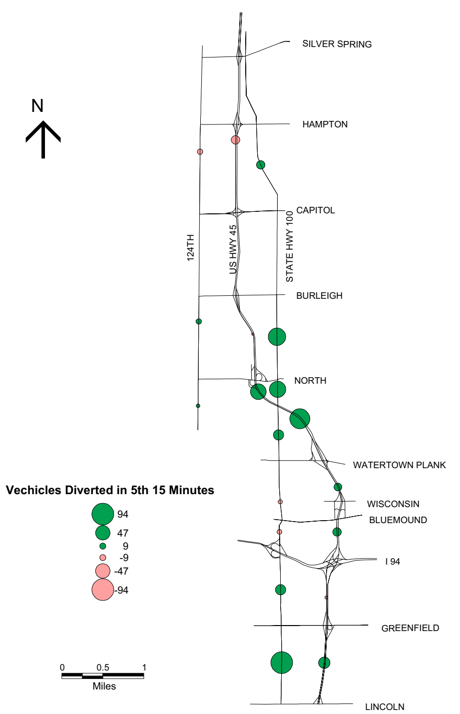
Diversion in 3rd 15 Minutes



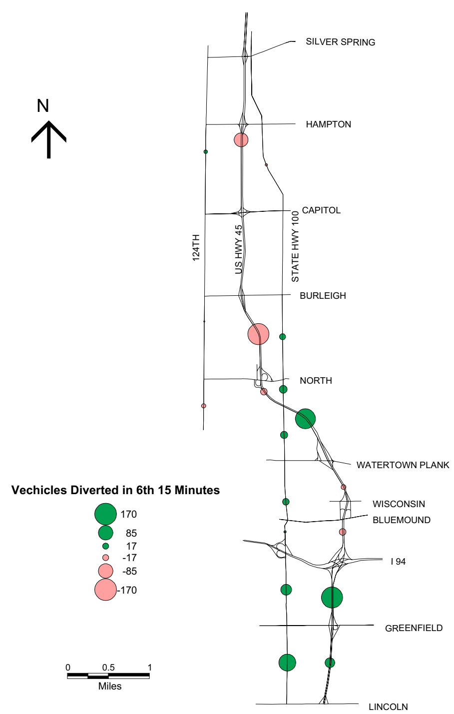
Diversion in 4th 15 Minutes



Diversion in 5th 15 Minutes

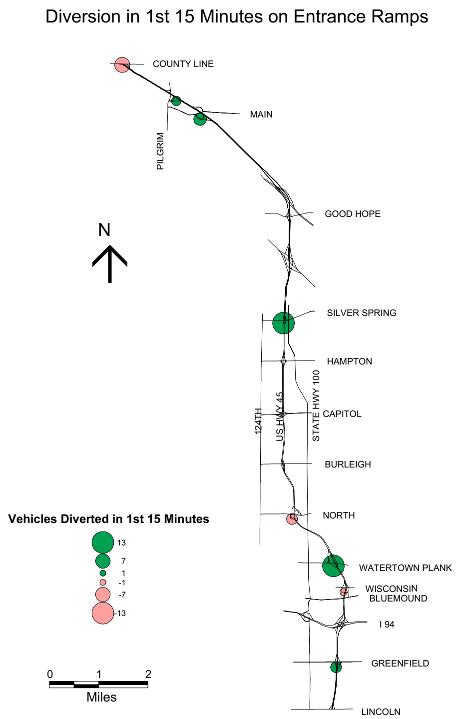


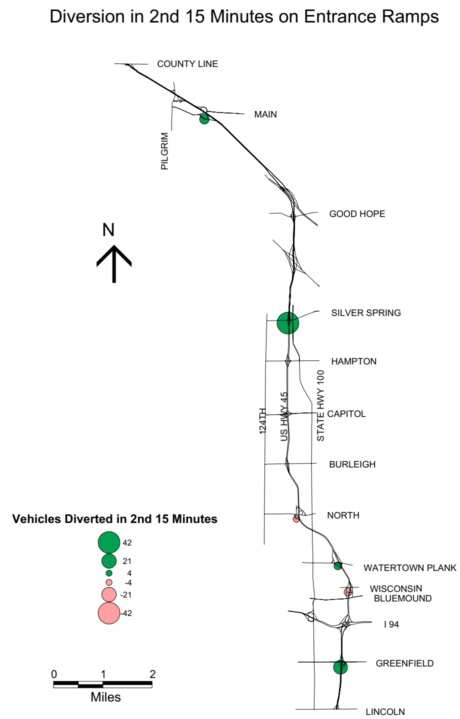
Diversion in 6th 15 Minutes



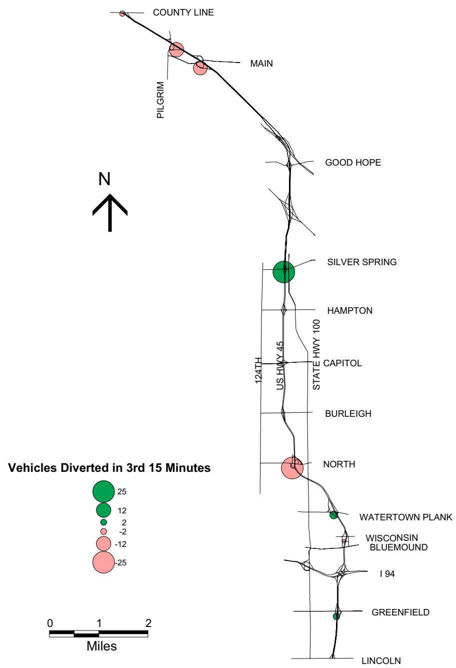
Appendix 2-3: Volume Changes on Selected US 45 Ramps

Increases in volumes between the before and after periods on the ramps are shown in dark gray. Decreases are shown in light gray.

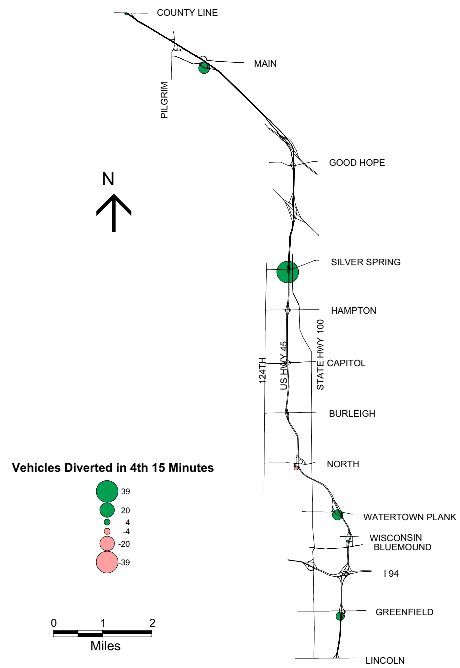




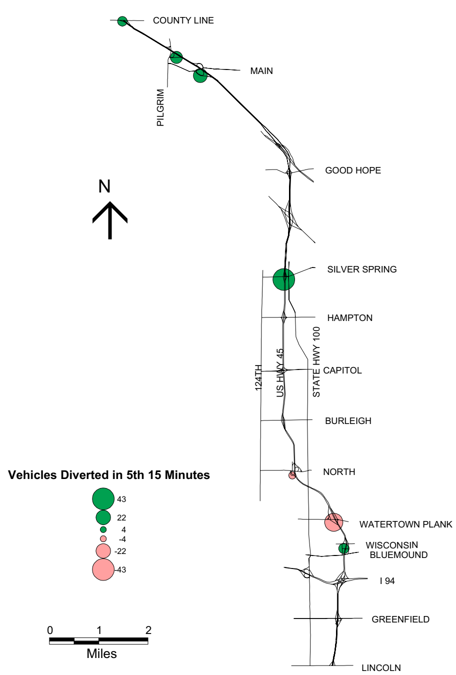
Diversion in 3rd 15 Minutes on Entrance Ramps



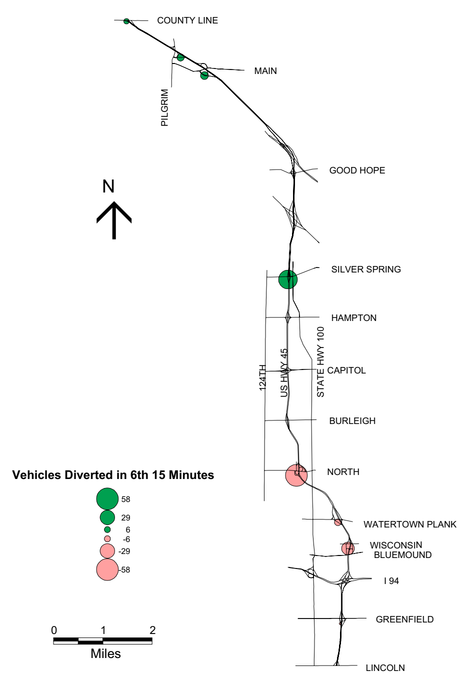
Diversion in 4th 15 Minutes on Entrance Ramps



Diversion in 5th 15 Minutes on Entrance Ramps



Diversion in 6th 15 Minutes on Entrance Ramps



Chapter 3
Ramp Meters Relation to Freeway Segment Trip Length

Diversion Propensity of Wisconsin Drivers

A series of questions related to diversion from ramp meters was administered to a random sample of Wisconsin drivers as part of a larger study of variable message signs. A total of 221 valid questionnaires were returned. The portion of the questionnaire that contains the diversion questions is shown in Figure 3-1.

The next 5 questions refer to your knowledge and perceptions of ramp meters. Please answer the questions by either providing the information or by circling the response that matches your perceptions.

Ramp meters are traffic signals located at the entrance ramps of freeways. Meters separate groups of vehicles entering the freeway, allowing safer and smoother merging.

18. Are there ramp meters on any of the routes you travel most frequently?

    a. No
    b. Yes

If you have answered no to Question 18, skip to Question 23

19. About how long (in minutes) was the last trip you took involving a ramp meter?
    ______ minutes

20. For the trip in Question 18, are you aware of alternate routes that allow you to avoid this particular ramp meter?

    a. No
    b. Yes

21. Do you have a good idea of how long the trip would take on any of the alternate routes of Question 20?

    a. No
    b. Yes

22. Would you take one of these alternate routes in Question 20 if:

a.	The ramp was empty of waiting vehicles?	Yes	No
b.	The ramp was about half full of waiting vehicles?	Yes	No
c.	The ramp was nearly completely full of waiting vehicles?	Yes	No
d.	The ramp was overflowing with waiting vehicles	Yes	No

Figure 3-1. Questions Related to Ramp Meters from Questionnaire Administered to Wisconsin Drivers

The analysis of diversion required that Question 18 (“Are there ramp meters on any of the routes you travel most frequently?”) be answered yes. After selecting those respondents and eliminating any respondents who failed to answer any part of question 22 (Would you take one of these alternative routes?), there were 91 remaining questionnaires.

Table 3-1 contains collective responses to each of the questions yes/no questions, except question 18.

Table 3-1. Summary of Responses from Yes/No Questions

Question (Shortened)	Percent indicating “Yes”
20. Aware of alternate routes	72%
21. Idea of how long the trip	65%
22a. Divert if ramp empty	15%
22b. Divert if ramp half full	24%
22c. Divert if ramp nearly full	62%
22d. Divert if ramp overflowing	82%

The average trip length (Question 19) across all valid respondents is 25.5 minutes. It is interesting that most drivers who encounter a ramp meter on a frequent trip are aware of an alternative route and have a good idea of how long that alternative route will take.

Of greatest interest are the responses to question 22, as this question establishes a relationship between a driver’s propensity to divert and the length of queue at a ramp meter. Question 22 asks drivers for their stated preference and does not necessarily predict actual behavior. Table 3-1 suggests that drivers are quite sensitive to the length of the queue at the ramp meter, with only a few drivers stating they would divert even if there is no queue. Many of these drivers are probably already taking the alternative route.

Table 3-2 compares average trip length between drivers who divert and drivers who do not divert for each of the four queue lengths of Question 22.

Table 3-2. Average Trip Lengths for Diverted and Not Diverted Trips from All Respondents

Queue Length Question (Shortened)	Average Trip Length for Drivers Who Divert (Minutes)	Average Trip Length for Drivers Who Do Not Divert (Minutes)
22a. Divert if ramp empty	16.5	27.1
22b. Divert if ramp half full	21.5	26.7
22c. Divert if ramp nearly full	26.9	23.2
22d. Divert if ramp overflowing	25.9	23.8

For each part of question 22, the two means are not significantly different within a 95% confidence using the t-test. Nonetheless, there is a pattern in the data that suggests that drivers are more likely to divert when their trips are short. A larger sample size would be needed to validate this hypothesis.

Table 3-3 shows the responses to question 22 for only those 60 drivers who answered "yes" to question 21, saying they are knowledgeable about an alternative route and its length.

Table 3-3. Willingness to Divert for Knowledgeable Drivers

Question (Shortened)	Percent indicating "Yes"
22a. Divert if ramp empty	8%
22b. Divert if ramp half full	23%
22c. Divert if ramp nearly full	70%
22d. Divert if ramp overflowing	87%

The data in Table 3-3 appear fairly similar to the data in Table 3-1. However, knowledgeable drivers seem more willing to divert when the queue is long and less willing, almost unwilling, to divert when the queue is short. The data suggests that knowledgeable drivers are showing more strength (less neutral) in their route choice decisions.

The results of this survey, although based on a small sample, are entirely consistent with our existing theories of path choice. Drivers indicate that they have good knowledge of alternative routes are willing to divert to an alternative route to time waiting within a queue at a ramp meter.

Trip Lengths from Origin-Destination Observations

WisDOT conduct two origin-destination counts along US 45, once before the ramp metering change in 1999 and again in 2001. Both morning (AM) and afternoon (PM) counts were available. The method of counting was matching license plates by video logging. The data supplied to this project consisted of a percentage distribution of traffic from each on-ramp to all off-ramps. The percentages were multiplied by the February and March on-ramp, peak-hour volumes to obtain estimates of numbers of vehicle trips between on-ramps and off-ramps both before and after metering. On-ramp to off-ramp distances were calculated from a scaled network of US 45. The data spanned US 45 from Pilgrim Road on the north to the I-94 interchange on the south.

Table 3-4 summarizes the average trip lengths between the before and after periods. In both the morning and after time periods, the average trip length increases. The increase (of 0.266 miles) was more pronounced in the morning.

Table 3-4. Average Trip Lengths (Miles) for All Trips on Southbound US 45

	Morning (AM)	Evening (PM)
Before	3.861	3.980
After	4.127	4.131

Figures 2 and 3 show the distribution of trip lengths on this section of freeway.

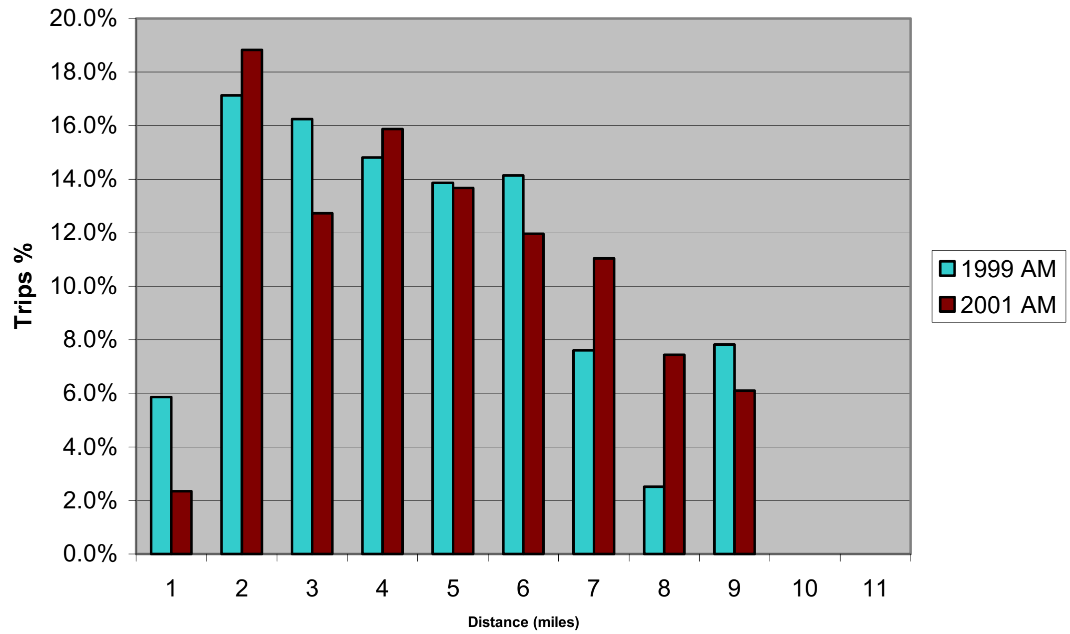


Figure 3-2. Trip Length Distribution on US 45, both Before and After Ramp Metering, Morning

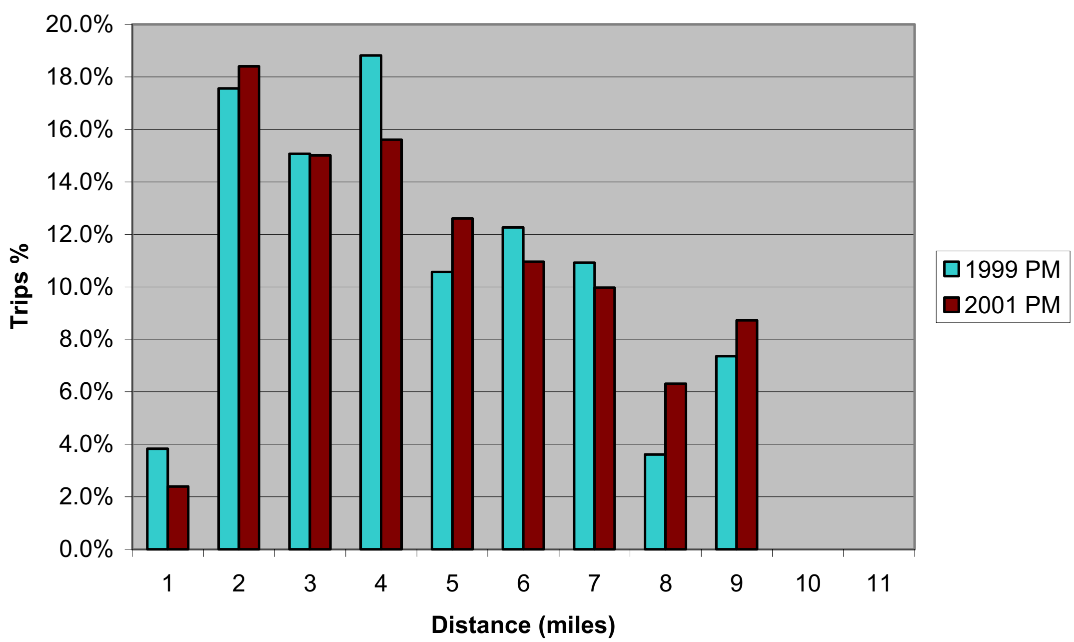


Figure 3-3. Trip Length Distribution on US 45, both Before and After Ramp Metering, Afternoon

These figures indicate that there is a reduced number of very short trips, both during the morning and afternoon periods. Otherwise, there are no obvious patterns in the data.

Chapter 4
Methods for Evaluating Ramp Meters in Milwaukee: Case Studies of Microscopic and Macroscopic Models

Introduction

A simulation model allows traffic engineers to create and evaluate scenarios and alternatives during the planning phase of a traffic system modification in order to optimize traffic controls. This study presents an assessment of two existing traffic simulation software packages, in order to identify which class of model can potentially be most useful in ramp metering simulation analysis.

Problem Summary

During the past three decades the Milwaukee area freeway system has been enhanced with the deployment of ramp meters on most on-ramps. The deployment of more meters along US Highway 45 in March of 2000 provided an opportunity to study the impacts before and after the implementation. There were only five meters working along US 45 in February of 2000 in the following on-ramps: Good Hope loop; Good Hope slip; North Avenue; Watertown Road; Wisconsin Avenue; and Greenfield Avenue. In March of the same year, WisDOT deployed seven new meters along all but one of the remaining on-ramps of this freeway: County Q; Pilgrim Road; Main Street; Appleton Avenue; Hampton Avenue; Capitol Drive; and Burleigh Street. As expected, there were some flow rate variations on the freeway and on the arterials next to it. An earlier study recorded traffic counts of the whole corridor from the before (February 2000) and after (March 2000) periods. Detector speeds and floating car speeds were also obtained. Queue lengths were recorded at all of the ramp meters. Additionally, there were two origin-destination tables obtained by video logging for periods about one year before and one year after the deployment.

The data obtained makes it possible to evaluate ramp meter's benefits directly or by using simulation models. Currently, WisDOT is in the early stages of developing a model of the whole Milwaukee area freeway system with a microscopic simulation program called Paramics. This model, at present, does not contain any ramp meters. Paramics could be used for ramp metering evaluation, but it first needs validation for this application. A comparison of a model's calculations with data other than those used in estimating the model is required in order to reliably use this simulation model to forecast future traffic behavior.

Types of Ramp Metering Systems

There are three major types of ramp meters.

- *Fixed time* operation that has the basic function to break down platoons and smooth out fluctuations in entering traffic volumes.
- *Traffic responsive* control where timing is based on the actual freeway conditions in the vicinity of the ramp, with the idea of storing vehicles on the ramp to provide more efficient downstream traffic flow.

- *System-wide* control where timing is traffic responsive to total freeway conditions and optimizes traffic flow across the whole system.

Milwaukee implements the second type of ramp meter control. The Milwaukee ramp meter algorithm will be fully explained later.

Under all three forms of ramp meter control maximum discharge rate of a single metered lane is about 900 vehicles per hour (vph). This capacity is based on a minimum reasonable cycle length of 4 seconds: 2.5 seconds of red and 1.5 seconds of green. By increasing the cycle to 6 seconds or 6.5 seconds two vehicles can be served from adjacent lanes, thereby increasing the discharge rate to 1100 or 1200 vph. A maximum discharge per lane of 1800 vph can be produced when the meter permits one or two vehicles per green and the ramp has two or more lanes at the meter. Drivers will not wait more than 15 seconds, so the most restrictive rate is about 240 vph (Piotrowicz and Robinson, 1995).

Scope of Chapter

This chapter reviews three major software products for simulating freeway systems with ramp meters: Paramics, QRS II and Dynasmart-P. Two of these products (Paramics and QRS II) are used to build models of US 45, both before and after the deployment of ramp meters. These models are tested against ground data with a minimum of calibration to similar data in order to obtain a relatively hands-free validation. Once built, the models can show the travel time savings benefits of ramp meter deployment and provide other measures of effectiveness.

Microscopic, Mesoscopic and Macroscopic Models for Freeway Ramp Meter Operation

This section describes the three models considered for evaluating the benefits of ramp meters. In addition, a short review of origin destination trip (OD) matrix estimation is provided, because of the need to develop a good time-dependent OD table for each of the three models.

Traffic simulation models can be classified as either microscopic, macroscopic or mesoscopic (Boxill and Yu, 2000). Microscopic models predict and analyze the path of individual vehicles throughout the system; their results are based on the average of all vehicles modeled. These models require an extreme amount of input data, consequently these models are most often used in small areas. Macroscopic models forecast traffic flows; they concentrate on groups of vehicles that are more general and whose results are group-wide or area-wide. Macroscopic models are mostly used in large area networks. Mesoscopic models have aspects of both microscopic and macroscopic models.

In this study three different software packages are reviewed. Paramics (microscopic), Dyanasmart-P (mesoscopic) and Quick Response System II, QRS II (macroscopic). These models are each respected representatives of their class.

Paramics: Microsimulation Software

Paramics is an example of a traffic microsimulation software package, of which there are many available. This software package was selected for this study because WisDOT has adopted it for its current evaluation of the Milwaukee area freeway system.

Traffic demands are given to Paramics in the form of a zone-to-zone OD (origin-destination) trip matrix. The software simulates the movements and behavior of individual vehicles on a given traffic network. Paramics consists of three parts: a Modeller, a Processor and an Analyzer. The Modeller uses a GUI (graphical user interface) to visualize the simulation of the traffic flow on the network. The Processor simulates the traffic situation without the GUI, thereby increasing simulation speed when numerous tests are required. The Analyzer reads and analyzes the simulation outputs, providing graphics to compare the results.

To perform the traffic simulation, Paramics requires a time-dependent OD trip matrix. The trips are represented from zone to zone and are separated into vehicle types and into time slices as small as 5 minutes. Vehicle trips are not loaded at centroids but directly to links. The number of trips between any origin and any destination is used to create a probability that a trip is made during a time slice, so that vehicle trips can be randomly created. Parking lots can be used as origin or destination points.

A traffic assignment is applied to all vehicles except fixed route vehicles, such as buses. Route choice in Paramics depends upon network coding, link cost factors, sign posted routes, lane and turn restrictions, model parameters, generalized cost coefficients, percent of familiar drivers and default parking origin/destination assumptions.

A major feature of Paramics is its application programmer's interface (API) that allows customization of the simulation to local traffic controls, conditions and future ITS options.

Paramics uses one of three assignment methods: stochastic, dynamic feedback or all-or-nothing.

- All-or-nothing assignment: This assignment method assumes that drivers traveling from zone A to zone B choose the same shortest path using a cost function based on free flow speed, distance and tolls. Path choice is also influenced by assumptions about driver familiarity with the route.
- Stochastic: This assignment method chooses paths randomly for each vehicle based on assumptions about driver's perceptions of the shortest path and variability in travel costs. Path choice is re-evaluated at every node encountered along the trip.
- Dynamic Feedback: At each time slice the amount of congestion is estimated. The algorithm re-routes the remaining parts of a trip for a familiar driver, given assumptions about the driver's perceptions of the new traffic situation.

Driver Behavior: Paramics documentation states, "The movement of individual vehicles is governed by three interacting models representing vehicle following, gap acceptance and lane changing. Vehicle dynamics are relatively simple, combining a mixture of driver behavior and some limitations based on vehicles' physical type and kinematics (e.g., size, acceleration/deceleration). These models are applied simultaneously at the level of individual vehicles." Consequently, each simulation is unique; i.e., the same inputs can have different results.

Geometry and Controls: Paramics allows a wide variety of road geometries, vehicle restrictions and intersection controls to be placed on the network, including detector locations. Paramics does not provide for a large array of ITS elements. However, Paramics can create random incidents and evaluate the performance of the traffic systems with such incidents. Paramics also has the capacity to simulate the effects of variable message signs, provided that the user specifies a set of compliance rules. Unlike some other microsimulation packages, Paramics can handle very large arterial networks.

Paramics sets the timing for actuated traffic signals by programming language code, written by the user, that controls temporary and permanent changes to timings. For each particular signal, the user must design a "plan". The plan includes a set of loop detectors and an optional set of parameters. Each plan is given the phases that would be controlled by that plan. The programming language is C-like. The "if-then-else" statements are used in this language.

The following program is an example of fixed-time ramp metering taken from the Paramics Modeller user guide.

```
Plan count 1
Plan 1 definition
Loops 1
Parameters 1
If (init) { variable;
}
If (occupied[1])
{
          red3       =
parameter[1];
}
else
```

This plan assumes a two-phase signal, with one phase all green (red time is zero) and the other phase all red (green time is zero). This plan is applied to the all red phase. A fixed-time ramp metering policy is implemented if a vehicle is detected on a loop. If no vehicle is present the signal is on green permanently, which is equivalent to "meter-off" conditions.

Link speed is calculated according to mean speed. It represents the average speed of all vehicles traversing the link in the current time step. Particularly, the speed calculation is as follows.

link speed = link length / actual time

Link delay is calculated according to absolute delay. The units for delay are seconds, and delay is calculated as the actual time taken by vehicles to traverse the link minus their free flow time. Specifically:

min speed= the smaller of link and vehicles maximum speed
free flow time = link length / min speed
link delay = actual time – free flow time

Travel Forecasting with Integrated Macroscopic Traffic Simulation: QRS II

The Quick Response System II (QRS II) is a travel forecasting software package, which performs macroscopic traffic simulation and uses the four-step travel demand process (trip generation, trip distribution, mode split and traffic assignment). Several minor steps are included in the model such as activity allocation, auto occupancy and time of day simulation. QRS II uses the traffic analysis procedures from the Highway Capacity Software (HCS) and other sources.

Refer to Figure 4-1 for a flow diagram of conventional travel forecasting models, including QRS II.

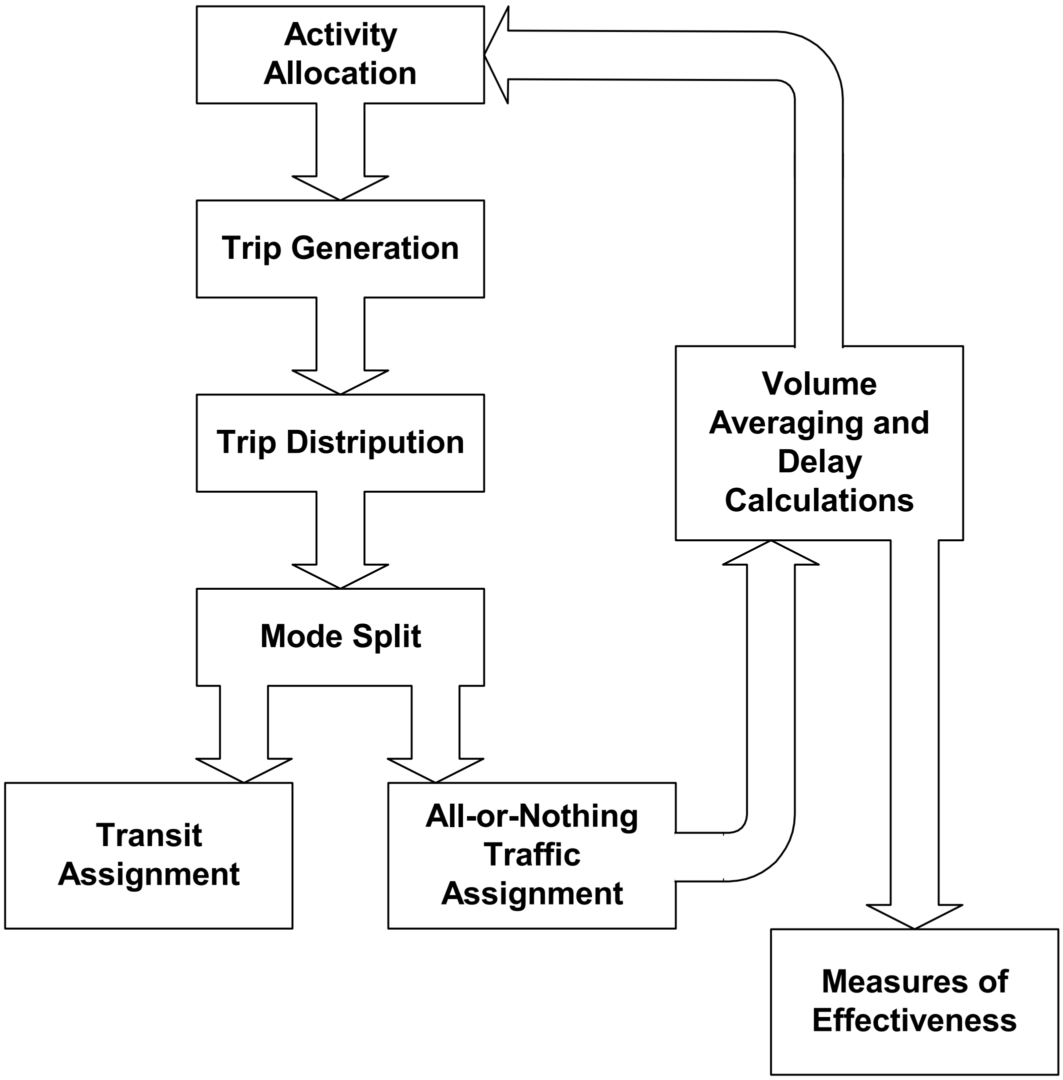


Figure 4-1. Flow Diagram of a Typical Travel Forecasting Model

QRS II has the ability to work with any trip generation model because it is possible for the user to supply the mathematical expressions for it. Also QRS II has default the trip generation rates and procedures from the National Cooperative Highway Research Program (NCHRP) Reports #187 and #365.

Trip distribution primarily works with the gravity model, which is a method analogous to Newton's law of gravity. The gravity model is used to allocate trips between a pair of zones depending on the proximity between them and the overall levels of trip making activity in each zone.

The multinomial logit model is used in the mode split step. Based on trip utility, the proportion of trips choosing a particular mode is assigned to that mode. The modes can be automobile, generalized transit and a user-defined third mode.

Automobile occupancy is a minor step done by QRS II in which factors are applied to person trips that can change by time of day. Also, a long range forecast simulation could be performed by QRS II by using the optional land-use forecasting module.

The traffic assignment step uses a static user-optimal equilibrium traffic assignment. The specific algorithm recommend for the software is the method of successive averages MSA. In this study, an experimental version of QRS II is used. This version performs dynamic traffic assignment. The software is able to assign traffic in time slices depending on the demand percentage applied for each time slice.

QRS II uses the Bureau of Public Roads (BPR) curve to obtain travel times on uninterrupted facilities, such as freeways and uncontrolled long portions of the arterial streets.

QRS II comes with its own network editor. The General Network Editor (GNE) is used in order to represent the network characteristics.

Ramp meters are represented in QRS II as one-way streets. The lane geometry and sign code must be set to "M" for a two-lane on-ramp, and an "m" for a one lane on-ramp.

In 1994, John A. Biebetitz developed a macroscopic ramp metering delay and queuing model in a master's thesis called "Area wide Impacts of Ramp Metering". This study determined the area wide impacts of different ramp metering plans. A new delay model was developed at the same time because of the lack of a macroscopic ramp metering delay/queuing model. The ramp metering delay/queuing model was included in QRS II giving satisfactory results after testing different ramp meters schemes with it.

In QRS II, delay calculations are made depending upon volume/capacity (V/C) ratio. For high V/C (>>1) storage will occur over an extended period of time, up to one full hour. For low V/C (<<1) queues occasionally form due to the randomness of vehicles arrivals. And for a V/C near 1, QRS II interpolates between both of the above.

The storage delay model in QRS II is based on a cumulative flow diagram to handle stored vehicles. The delay on the ramp due to queuing during saturated conditions corresponds to the shaded area in Figure 4-2. To calculate delay it is required that the arrival and departure lines meet at the end of the period of analysis (T). The arrival line represents the demand and the departure line represents the metering rate. This type of delay occurs only when the volume/capacity ratio is greater than 1.0 (Bieberitz, 1994). For delay when the volume to capacity ratio is less than 1, QRS II uses an M/G/1 queuing model with constant service times.

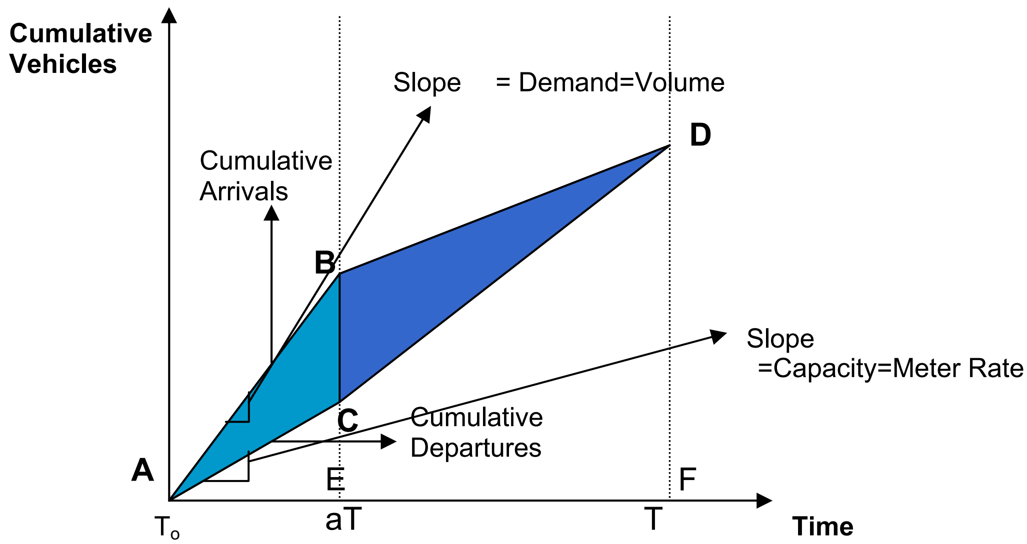


Figure 4-2. Graphical Representation of On-Ramp Queuing and Delay Used by QRS II

These delays are included in the link travel times in addition to the acceleration delay, red-time delay, and travel time on the link where the ramp meter is placed.

QRS II requires the user to set two important parameters for the ramp meter representation, the metered period fraction and the meter volume fraction. The metered period fraction is an approximation of peaking characteristics. It is the fraction of the time period, typically 1 hour, that is metered at the rate given in the capacity attribute. A higher meter rate is used to discharge a queue. The meter volume fraction is the fraction of the time period's volume that occurs during the metered period. The critical attribute of each ramp meter is the meter rate. QRS II does not represent a traffic responsive ramp meter. It is necessary to manually set the meter rate, which is the same as the capacity of the ramp meter.

QRS II generates a file where the measures of effectiveness (MOEs) are shown. This file provides the general results of speeds, vehicle-hours-traveled, vehicle-miles-traveled and vehicle emissions for the whole system.

It is possible for QRS II to be run as a subroutine of another computer program, such as one that is intended to optimize traffic control strategies.

Mesoscopic Traffic Simulator with Dynamic Traffic Assignment: Dynasmart-P

Dynasmart-P is an example of a mesoscopic traffic simulation with dynamic traffic assignment. Dynasmart-P is one of two, essentially duplicative, software products being developed by the TrEPS (Traffic Estimation and Prediction System) project sponsored by the Federal Highway Administration. Its forerunner, Dynasmart, has been used for many years at the University of Texas at Austin and elsewhere for research purposes.

Dynasmart-P is a software package that allows users to simulate and evaluate the design and planning of traffic in intelligent transportation networks. The software is able to simulate most driver behavior within a network, such as variations of traffic flow patterns over time and drivers' routing decisions. Dynasmart-P allows calculating for each of many short time slices the speeds, densities, queues, vehicles trajectories and different characteristics of every link in the network. The software has capabilities to evaluate an array of operational strategies, including ATMS (Advanced Transportation Management System) strategies, HOV facilities, ramp meters and special use lanes. Dynasmart was developed for short-term forecasting of traffic in real-time; Dynasmart-P is a version of Dynasmart for the purposes of traffic planning.

Dynasmart-P simulates intelligent transportation networks using two different methods of vehicle generation. The first method requires the user to specify origin and destination trip matrices, aggregated to zones, for different demand time slices. The second requires the user to specify the characteristics of vehicles, their stops and their corresponding travel times.

The vehicles generated can be assigned in two different ways. The first way is "one step simulation assignment", in which all vehicles are individually assigned to its currently best path, a random path among a limited set of shortest paths or any predetermined path. The second way is "iterative simulation assignment" in which Dynasmart-P applies dynamic user-optimal equilibrium assignment using MSA. MSA is applied to time slices as short as 1 minute, with different demands and different network characteristics applicable to each slice.

Dynasmart-P is able to evaluate traffic management strategies such as ramp metering, variable message signs (VMS) and path and corridor coordination. Dynasmart-P can also calculate congestion pricing for regular links, HOV links and HOT (high occupancy toll) links. It can simulate incidents, including their starting time, ending time, location and severity.

Dynasmart-P also has the ability to find system optimal traffic assignments, which can serve as upper bounds on the benefits that might be achieved by rerouting traffic to avoid incidents or other random traffic events.

Dynasmart-P assumes that ramp metering follows the ALINEA procedure (Papageorgiou, Hadj-Salem and Middelham, 1997). ALINEA is a traffic responsive feedback process where the metering rate is set according to the occupancy at a point downstream on the mainline. To evaluate ramp metering in a network it is necessary to create a text file in which the following data items must be included: the number of ramps, the frequency in which the ramp metering is checked by the simulator, and the location of two detectors downstream from the meter. Dynasmart-P also asks for two constants required by ALINEA for each ramp that are used to determine the ramp's meter rate. The first constant is the target occupancy (defaulted to 0.2); ALINEA will adjust the metering rate in an attempt to achieve this occupancy value. The second constant relates to how much the metering rate changes for a given difference between the actual occupancy and the target (defaulted to 0.32 vehicles per minute per lane per difference in occupancy from target). Also required are the time slices in which the ramp metering is effective and the ramp saturation flow rate. The ALINEA procedure calculates, at each period T=1,2,3…, the rate as follows:

$$Rate(T+1) = Rate(T) + Cons1[Cons2 - Occ]$$

Where Cons1= 0.32 or as externally specified, Cons2=0.2 or externally specified, Occ is the upstream occupancy, and Rate is the measurable on ramp traffic volume (5 veh/min-lane < Rate (t) < 25-35 veh/min-lane).

Dynasmart-P is not being used in this study because of the following reasons.

- Dynasmart-P uses a traffic responsive algorithm called ALINEA to determine ramp meter rates. This software cannot replicate the WisDOT meter rates because its algorithm uses different thresholds and different loop detector locations for the meter rate calculation.
- Dynasmart-P uses the same dynamic traffic assignment procedure found in QRS II. Thus, having more knowledge about the use of QRS II, the results obtained would be more reliable.
- The Dynasmart-P software requires text files for all its data inputs. Convenient network or data editors are not available at this time, although it is possible to edit many text files within Dynasmart-P's graphical user interface. The time and resources necessary to code a network from scratch and provide all the necessary input data are substantial. Network data can be converted from other sources, such as a GISs (geographic information systems), CORSIM (a microscopic simulation model) and several planning models.
- To effectively use the dynamic aspects of the software, it is necessary to supply complete OD trip tables for very small time slices. Dynasmart-P eases this data requirement somewhat by allowing the user to factor a static OD trip table into time slices, but it does not contain algorithms for creating a static OD trip table from planning data.

OD Matrix Estimation

All three models must be provided with a trip table in order to represent trips between zones during the simulation. When the only reliable data available is traffic counts, it is possible to obtain an OD trip matrix using the generalized least squares (GLS) technique. In summary, the GLS technique finds a final OD trip matrix obtained from a target OD trip matrix that is assumed to contain probabilistic error. The objective is to obtain the OD trip matrix most matching the link counts and closest to the target OD trip matrix.

It is possible that the obtained OD trip matrix does not precisely reproduce all traffic counts. This is caused by irregularities in traffic counts caused during the traffic flow data collection at different times or from an aggregated transportation network representation (Abrahamsson, 1998).

Model Framework

Evaluation Background

Ground data for this study was collected for two different time periods. Data from February of 2000 are the "before" period which represents "without ramp metering", and data from March of 2000 represent the "after" period or "with ramp metering".

The corridor of study is US Highway 45. This urban freeway is located at the west side of the Milwaukee metropolitan area. The study section is the southbound lanes of US Highway 45 from Good Hope Road to a point just past Wisconsin Avenue. The section of freeway north of Good Hope was removed from the study because ground data revealed little congestion and this section had been omitted from WisDOT's Paramics model.

The ramp meters working in the before period (February) were located at the following on-ramps:

- Good Hope Road (loop);
- Good Hope Road (slip);
- North Avenue;
- Watertown Road; and
- Wisconsin Avenue.

In March, more meters were added at the following on-ramps:

- Appleton Avenue;
- Hampton Avenue;
- Capitol Drive; and
- Burleigh Street.

Silver Spring Drive was the only on-ramp that did not have a meter in either the before or after period.

Data Preparation and Calibration

It is necessary to build an OD (origin-destination) trip matrix that replicates the traffic counts recorded by WisDOT. There are two trip matrices, one for the before period (February 2000), and the other for the after period (March 2000). The generalized least squares (GLS) technique is used for this estimation.

The data sample of OD trips that was obtained from the video logging gave a distribution percentage table that did not match well with the traffic counts for the PM peak hour. Video logging misses a large percentage of vehicles, either entering or leaving the freeway. In addition, the detectors and road tubes used to get traffic counts have their own errors.

Given that these tables are the only distribution of vehicles to off-ramps available, it was necessary to adjust them using the GLS technique to better match ground data. Each crossing street that has access to US Highway 45 is assumed to be a zone. The on-ramp volumes are the number of trips, whose origin is the corresponding zone and the off-ramp volumes are the number of trips, whose destination is to that zone.

The following problems were found at the beginning of this process.

- Some data was missing, such as entering volumes and mainline volumes.
- The mainline volumes did not agree with the entering and exiting volumes.

Table 4-1 shows this percentage distribution table for the before period. Table 4-2 shows the new distribution percentages matrix where the last three columns are aggregated to just one downstream zone. An OD trip table is obtained by multiplying those percentages by the upstream zone volumes and by the on-ramps volumes.

Table 4-1. Distribution Percentages from Zone to Zone (Raw)

	Upstream Zone	Good Hope Road	Appleton Avenue	Silver Spring Drive	Hampton Avenue	Capitol Drive	Burleigh Street	North Avenue	Watertown Plank Rd	Wisconsin Avenue	I94 West	I94 East	Greenfield	Downstream Zone
Upstream Zone	0	4.49	1.86	3.36	5.59	12.68	11.93	5.92	5.97	5.59	13.79	22.75	5.97	42.51
Good Hope Road		0	1.95	3.52	5.86	13.28	12.5	6.2	6.25	5.86	14.45	23.83	6.25	44.53
Appleton Avenue			0	8.16	0	15.31	10.2	11.2	4.08	3.06	12.24	29.59	6.12	47.95
Silver Spring Dr.				0	0.67	8.3	6.95	5.6	2.91	1.79	19.06	49.78	4.93	73.77
Hampton Avenue					0	5.6	8	4.5	3.73	1.87	25.33	45.87	5.07	76.27
Capitol Drive						0	4.69	3.5	2.65	1.84	26.53	57.35	3.47	87.35
Burleigh Street							0	3.7	2.63	0.79	29.21	57.11	6.58	92.90
North Avenue								0	0.27	1.37	34.34	54.67	9.34	98.35
Watertown Plank Rd									0	1.49	28.33	59.4	10.78	98.51
Wisconsin										0	4.3	69.34	26.37	100.01
I94 west											0			
I94 East												0		
Greenfield													0	
Downstream Zone														0

Table 4-2. Distribution Percentages from Zone to Zone (Aggregated)

	Upstream Zone	Good Hope Road	Appleton Avenue	Silver Spring Drive	Hampton Avenue	Capitol Drive	Burleigh Street	North Avenue	Watertown Plank Rd	Wisconsin Avenue	Downstream Zone
Upstream Zone	0	4.49	1.86	3.36	5.59	12.68	11.93	5.92	5.97	5.59	42.51
Good Hope Road		0	1.95	3.52	5.86	13.28	12.5	6.2	6.25	5.86	44.53
Appleton Avenue			0	8.16	0	15.31	10.2	11.2	4.08	3.06	47.95
Silver Spring Dr.				0	0.67	8.3	6.95	5.6	2.91	1.79	73.77
Hampton Avenue					0	5.6	8	4.5	3.73	1.87	76.27
Capitol Drive						0	4.69	3.5	2.65	1.84	87.35
Burleigh Street							0	3.7	2.63	0.79	92.90
North Avenue								0	0.27	1.37	98.35
Watertown Plank Rd									0	1.49	98.51
Wisconsin										0	100.01
Downstream Zone											0

The OD table estimation process minimizes this function.

$$\min\left\{z\left[\sum_i\sum_j T_{ij}^{\,2}(1-Y_jX_i)^2\right]+\left[\sum_i\left(\sum_j(Y_jX_iT_{ij})-O_i\right)^2\right]+\left[\sum_j\left(\sum_i(Y_jX_iT_{ij})-D_j\right)^2\right]+\left[\sum_{LinksVol}(Vest-Vact)^2\right]\right\}$$

where,

z is a weight factor (=1 in this case)
T_{ij} is the original trip table,
Y_j is the adjustment factor applied to destinations,
X_i is the adjustment factor applied to origins,
O_i are the origins,
D_j are the destinations,
Vest is the estimated volume on the mainline,
Vact is the actual or real volume on the mainline.

Good Hope Road has two on-ramps, a loop ramp for traffic coming from the east, and a slip ramp for traffic coming from the west. Most of the trips coming from Good Hope use the loop ramp; thus, one more zone was added to the system on the east side of US 45. Figure 4-3 illustrates all the zones assumed for this study.

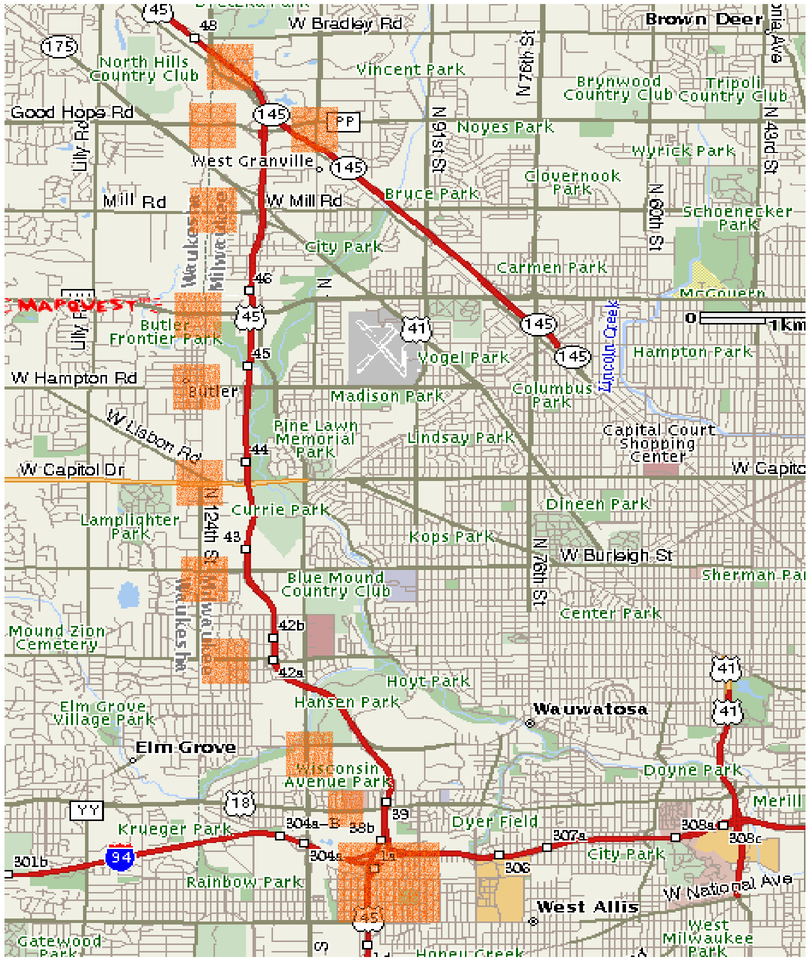


Figure 4-3. Study Location and Zone System

The trips exiting Good Hope Road are assigned to the Good Hope Road slip zone. The final OD trip matrices for the February and March periods are shown in Tables 4-3 and 4-4, respectively.

Table 4-3. February OD Matrix

	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]
Upstream Zone	0	189	301	416	339	494	261	216	128	143	1681
Good Hope Road (Slip)		0	17	24	20	29	15	13	7	8	97
Good Hope Road (loop)		0	98	135	110	160	85	70	41	46	545
Appleton Avenue			0	49	0	29	11	20	4	4	93
Silver Spring Dr.				0	9	73	34	46	14	10	659
Hampton Avenue					0	13	11	10	5	3	184
Capitol Drive						0	14	18	8	7	488
Burleigh Street							0	27	11	4	739
North Avenue								0	1	7	794
Watertown Plank Rd									0	5	526
Wisconsin Av										0	578
Downstream Zone											0

Table 4-4. March OD Matrix

	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]
Upstream Zone	0	294	201	275	264	500	257	199	143	223	1352
Good Hope Road (slip)		0	10	14	14	26	13	10	7	12	71
Good Hope Road (Loop)		0	59	80	77	146	75	58	42	65	395
Appleton Avenue			0	52	0	47	17	30	8	10	120
Silver Spring Dr.				0	10	108	49	62	23	24	774
Hampton Avenue					0	24	18	16	9	8	258
Capitol Drive						0	22	26	14	16	619
Burleigh Street							0	28	14	7	666
North Avenue								0	1	11	616
Watertown Plank Rd									0	10	548
Wisconsin Av										0	489
Downstream Zone											0

General Issues in Modeling the Network

The simulations have two principal outputs: speeds and OD travel times. These two measures of effectiveness (MOEs) are compared between both periods in order to determine the effect caused by the meters. For validation purposes, it is necessary to compare the speeds given by the models with ground data.

Both models require the percentage of vehicles released during each 15-minute period. From the traffic counts, the percentages during the peak hour were set as follows: 25%, 26%, 25%, and 24%.

Paramics Simulation

WisDOT sketched the US 45 corridor in Paramics. They coded the mainline of the freeway and the crossing arterials, including the off and on-ramps to the freeway. The northbound and southbound lanes of the freeway are coded, but only the southbound lanes are analyzed. This original network had to be changed substantially in order to represent the ramp meters.

The following data files in Paramics must to be added for ramp meters.

- *Plans File:* In order to set the timing for actuated signals, the user must define a "plan" for each signal by using a programming language. The plan includes a set of loop detectors and parameters. Each plan is given the phases that would be controlled by that plan. Different plans can be contained in this file. An example is shown later.
- *Phases File:* This file is used for actuated signals. It contains the node number where the signal is located, the plan number to be applied at this node, the name of each loop used by the plan and the value of each parameter.
- *Profile File:* .This file defines the percentages of demand applied in each time slice during a period of time.

Ramp meters in Paramics are represented by using a "plan file" defined by the user for a standard traffic signal. This plan depends on the geometric characteristics of the ramp, such as the number of lanes in the mainline before the merging area and the number of lanes before the meter. It is possible to have different plans for different geometries. A plan is applied at the node where the meter is located; therefore, a node needed to be added to each on-ramp. Additionally, WisDOT sets different thresholds for the PM and AM peak hours so that meters will be responsive to the actual traffic conditions and modify red and green times accordingly. Four different flow characteristics have these thresholds, which are the following:

- Volume in the mainline;
- Occupancy in the mainline;
- Vehicle speed in the mainline; and
- Occupancy in the queue at a point on the ramp farthest upstream from the meter.

Different thresholds were set for each on-ramp in the freeway. Refer to Table 4-5 for the values.

Table 4-5. Timing Strategies for US Highway 45 Southbound

Good Hope Loop
PM Interval Timing

	1	2	3	4	5	6
Green	2	2	2	2	2	2
Yellow	0	0	0	0	0	0
Red	2	2	3	4	5	6
Red Extension	0	0	0	0	0	0

PM Thresholds

	1	2	3	4	5	6
Volume	1600	1650	1725	1800	2050	2300
Occupancy	23	25	27	30	33	36
Vehicle Speed	55	50	45	40	33	25
Queue Occupancy	40	35	33	30	25	20

GoodHope Slip
PM Interval Timing

	1	2	3	4	5	6
Green	2	2	2	2	2	2
Yellow	0	0	0	0	0	0
Red	3	4	6	8	9	10
Red Extension	0	0	0	0	0	0

PM Thresholds

	1	2	3	4	5	6
Volume	592	646	1034	1149	1264	1418
Occupancy	5	6	9	10	11	36
Vehicle Speed	60	59	58	57	56	55
Queue Occupancy	60	55	50	40	30	20

Appleton Ave
PM Interval Timing

	1	2	3	4	5	6
Green	2	2	2	2	2	2
Yellow	0	0	0	0	0	0
Red	2	3	4	5	7	8
Red Extension	0	0	0	0	0	0

PM Thresholds

	1	2	3	4	5	6
Volume	730	930	1191	1249	1507	1710
Occupancy	5	6	7	9	11	13
Vehicle Speed	60	59	58	57	56	55
Queue Occupancy	40	35	33	30	25	20

Silver Spring Drive
PM Interval Timing

	1	2	3	4	5	6
Green	2	2	2	2	2	2
Yellow	0	0	0	0	0	0
Red	2	3	5	7	9	10
Red Extension	0	0	0	0	0	0

PM Thresholds

	1	2	3	4	5	6
Volume	714	900	1183	1254	1325	1478
Occupancy	5	7	8	9	10	11
Vehicle Speed	60	59	58	57	56	55
Queue Occupancy	50	45	40	35	30	20

Hampton Avenue
PM Interval Timing

	1	2	3	4	5	6
Green	2	2	2	2	2	2
Yellow	0	0	0	0	0	0
Red	2	3	5	7	8	10
Red Extension	0	0	0	0	0	0

PM Thresholds

	1	2	3	4	5	6
Volume	818	822	1419	1521	1622	1761
Occupancy	6	8	10	11	12	13
Vehicle Speed	60	59	58	57	56	55
Queue Occupancy	40	35	30	25	20	15

Capitol Drive
PM Interval Timing

	1	2	3	4	5	6
Green	2	2	2	2	2	2
Yellow	0	0	0	0	0	0
Red	2	3	4	6	8	10
Red Extension	0	0	0	0	0	0

PM Thresholds

	1	2	3	4	5	6
Volume	966	1064	1161	1356	1735	1789
Occupancy	6	7	11	14	28	40
Vehicle Speed	58	50	42	34	26	17
Queue Occupancy	40	35	30	25	20	10

Burleigh Street
PM Interval Timing

	1	2	3	4	5	6
Green	2	2	2	2	2	2
Yellow	0	0	0	0	0	0
Red	2	3	4	6	8	10
Red Extension	0	0	0	0	0	0

PM Thresholds

	1	2	3	4	5	6
Volume	900	1100	1300	1500	1700	1850
Occupancy	6	13	20	27	34	41
Vehicle Speed	60	50	40	30	20	10
Queue Occupancy	35	30	25	20	15	10

North Avenue
PM Interval Timing

	1	2	3	4	5	6
Green	2	2	2	2	2	2
Yellow	0	0	0	0	0	0
Red	2	3	5	7	9	10
Red Extension	0	0	0	0	0	0

PM Thresholds

	1	2	3	4	5	6
Volume	1050	1250	1450	1650	1850	2050
Occupancy	6	13	21	28	35	41
Vehicle Speed	60	50	40	30	20	15
Queue Occupancy	40	35	30	25	22	20

Watertown Plank Road
PM Interval Timing

	1	2	3	4	5	6
Green	2	2	2	2	2	2
Yellow	0	0	0	0	0	0
Red	2	2	3	3	4	6
Red Extension	0	0	0	0	0	0

PM Thresholds

	1	2	3	4	5	6
Volume	1150	1390	1550	1750	1950	2150
Occupancy	7	11	15	19	24	29
Vehicle Speed	60	55	50	40	30	25
Queue Occupancy	34	30	26	22	18	14

Wisconsin Avenue
PM Interval Timing

	1	2	3	4	5	6
Green	2	2	2	2	2	2
Yellow	0	0	0	0	0	0
Red	2	3	5	7	9	10
Red Extension	0	0	0	0	0	0

PM Thresholds

	1	2	3	4	5	6
Volume	976	1031	1451	1556	1662	1780
Occupancy	7	10	13	15	17	22
Vehicle Speed	60	57	55	48	43	30
Queue Occupancy	50	40	35	30	25	20

The following is an example of how to use the thresholds for the ramp meters and at the same time to see how the "plan file" and "phases file" work together.

Assume the following flow characteristics for the vicinity of the Good Hope loop ramp meter: Mainline volume = 2000 vph, mainline speed = 47 mph, mainline occupancy = 18%, and on-ramp occupancy = 10 %. Each of these values is compared with the corresponding threshold in the table for this on-ramp (the plan file makes the comparison and it makes reference to the parameters contained in the phases file).

As is shown in the corresponding table for the Good Hope loop, the volume corresponds to column 4 in the table because 2000 vph is greater than 1800 vph (column 4) and is lower than 2050 vph (column 5). The speed of 47 mph is between 50 mph (column 2) and 45 mph (column 3), corresponding to column 2. With 18% of occupancy in the mainline, the corresponding column would be number 1 because it is less than 23%. As is shown, the most critical value is the volume because it needs a lower meter rate. But, the corresponding timing for that column is set only to the extent that the queue occupancy allows it. Queue occupancy must be less than the threshold for it in column 4. In this case 10% is less than 33%. Therefore, the red time is equal to 4 seconds and the green time is equal to 2 seconds (corresponding to timings for column 4).

Here is one more example for the same ramp:

Volume = 1750 vph, corresponding column 3;
Occupancy = 35%, corresponding column 5;
Speed = 43 mph, corresponding column 3; and
Queue occupancy = 41%, corresponding column 1.

As is shown, the most critical characteristic is the mainline occupancy that corresponds to column 5. However, the queue occupancy is very high; so it needs a higher meter rate than column 1 (Red 2 seconds, green 2 seconds).

In addition to the plan file, a "phases file" is required to create a place where all these thresholds can be referenced. The following is an example of the plan and phases files for the Good Hope loop ramp. This on-ramp is a two-lane ramp; the mainline before the merging area consists of two lanes. Depending on the occupancy, speed and volume of the mainline, and the occupancy of the queue, the cycle length is set. The phases file for the Good Hope loop is shown in Figure 4-4.

```
use plan 4
     on node 3264 phase 2
     with loops
          D5GH lane 1
          D5GH lane 2
          D4GH lane 1
          D4GH lane 2
     with parameters
     2300
     36
     25
     20
     2050
     33
     33
     25
     1800
     30
     40
     30
     1725
     27
     45
     33
     1650
     25
     50
     35
     1600
     23
     55
     40
     6
     5
     4
     3
     2
     2
     2
```

Figure 4-4. Example of Phases File used by Paramics

Plan 4 applies to this meter, and it is shown below. Node 3264 is the location of the meter, and loops D4GH and D5GH are detectors located on the mainline before the merging area. Figure 4-5 is an illustration of the vicinity of the on-ramp at Good Hope Road.

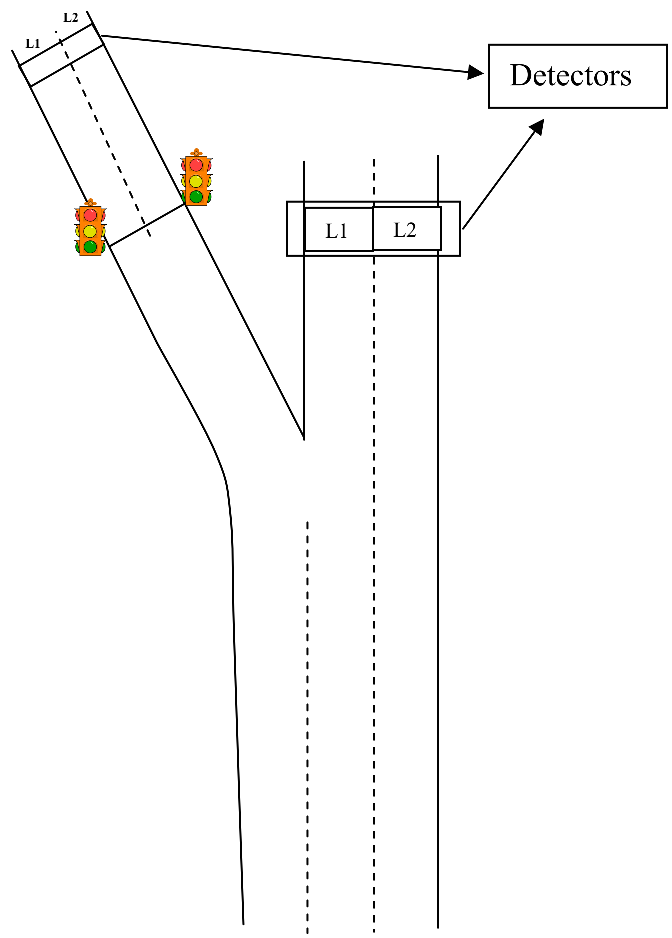


Figure 4-5. Illustration of On-ramp

Figure 4-5 shows the required locations of the detectors. Data from them is collected lane by lane. It is necessary to make reference in the plan file to each of the thresholds per lane, and then add them together and divide them by the number of lanes. The following is the plan used for the Good Hope on-ramp loop.

```
plan 4 definition
loops 4
parameters 31

If (init) { variable; }

If ((((flow [1] + flow [2]) / 2) > Parameter[1]) || (((((occupancy [1]  / (gap [1]  + occupancy [1]
)) + (occupancy [2]  /(gap [2]  + occupancy [2]  ))) / 2)*100) > Parameter[2]) || (((speed [1] +
speed [2]) / 2) < Parameter[3]))
{
        If ((((((occupancy [3]  /(gap [3]  + occupancy [3]  )) + (occupancy [4]  / (gap [4]  +
occupancy [4]  ))) / 2)*100) < parameter[4]))
        {
        green2 [2] = Parameter [31];
        green2 [1] = Parameter [25];
        }
        else

        If ((((flow [1] + flow [2]) / 2) > Parameter[5]) || (((((occupancy [1]  / (gap [1]  +
occupancy [1]  )) + (occupancy [2]  /(gap [2]  + occupancy [2]  ))) / 2)*100) > Parameter[6]) ||
(((speed [1] + speed [2]) / 2) < Parameter[7]))
        {
                If ((((((occupancy [3]  /(gap [3]  + occupancy [3]  )) + (occupancy [4]  / (gap [4]
+ occupancy [4]  ))) / 2)*100) < parameter[8]))
                {
                green2 [2] = Parameter [31];
                green2 [1] = Parameter [26];
                }
                else

                If ((((flow [1] + flow [2]) / 2) > Parameter[9]) || (((((occupancy [1]  / (gap [1]  +
occupancy [1]  )) + (occupancy [2]  /(gap [2]  + occupancy [2]  ))) / 2)*100) > Parameter[10]) ||
(((speed [1] + speed [2]) / 2) < Parameter[11]))
                {
                        If ((((((occupancy [3]  /(gap [3]  + occupancy [3]  )) + (occupancy [4]  /
(gap [4]  + occupancy [4]  ))) / 2)*100) < parameter[12]))
                        {
                        green2 [2] = Parameter [31];
                        green2 [1] = Parameter [27];
                        }
                        else
```

```
			If ((((flow [1] + flow [2]) / 2) > Parameter[13]) || (((((occupancy [1]  / (gap [1]  + occupancy [1]  )) + (occupancy [2]  /(gap [2]  + occupancy [2]  ))) / 2)*100) > Parameter[14]) || (((speed [1] + speed [2]) / 2) < Parameter[15]))

			{
				If ((((((occupancy [3]  /(gap [3]  + occupancy [3]  )) + (occupancy [4]  / (gap [4]  + occupancy [4]  ))) / 2)*100) < parameter[16]))
				{
				green2 [2] = Parameter [31];
				green2 [1] = Parameter [28];
				}
				else

				If ((((flow [1] + flow [2]) / 2) > Parameter[17]) || (((((occupancy [1]  / (gap [1]  + occupancy [1]  )) + (occupancy [2]  /(gap [2]  + occupancy [2]  ))) / 2)*100) > Parameter[18]) || (((speed [1] + speed [2]) / 2) < Parameter[19]))
				{
					If ((((((occupancy [3]  /(gap [3]  + occupancy [3]  )) + (occupancy [4]  / (gap [4]  + occupancy [4]  ))) / 2)*100) < parameter[20]))
					{
					green2 [2] = Parameter [31];
					green2 [1] = Parameter [29];
					}
					else

					If ((((flow [1] + flow [2]) / 2) > Parameter[21]) || (((((occupancy [1]  / (gap [1]  + occupancy [1]  )) + (occupancy [2]  /(gap [2]  + occupancy [2]  ))) / 2)*100) > Parameter[22]) || (((speed [1] + speed [2]) / 2) < Parameter[23]))
					{
						If ((((((occupancy [3]  /(gap [3]  + occupancy [3]  )) + (occupancy [4]  / (gap [4]  + occupancy [4]  ))) / 2)*100) < parameter[24]))
						{
						green2 [2] = Parameter [31];
						green2 [1] = Parameter [30];
						}
						else

					{
						green2 [2] = parameter [31];
						green2 [1] =  parameter [30];
					}
			}
}
}
}
```

}
}

This plan consists of four different comparisons, one for each threshold. There are 4 loops, one for each lane and there are 31 parameters, one for each condition or threshold and one for each red and green time. As mentioned before, green times for all phases were set to 2 seconds because it is not possible in Paramics to use fractions of seconds. It is also important to know that the "occupancy" in Paramics is the actual time that the loop is occupied and not the percentage of time in which the loop has been occupied by a vehicle. For that reason, the occupancy in Paramics is divided by the sum of the gap plus the occupancy for the current time slice, and then multiplied by 100.

The simulation is started 30 minutes early in order to have a stable flow of vehicles on the network before the peak hour demand. Paramics requires a "profile file" in order to distribute the demand in the time slices. Figure 4-6 shows the "profile file" used for the Paramics runs.

```
demand profile
period count 1
divisor 1
interval 5

period 1  start 16:30:00
5  5  5  5  6  6  5  6  6  6  6  6  6  6  6  5  5  5
```

Figure 4-6. Profile file Used in Paramics

The demand was distributed for one hour and 30 minutes, where the last hour represents the peak hour of the freeway. Each number corresponds to the percentage of total demand released during a time slice of 5 minutes.

During trial simulations it was observed that the Paramics network required many fixes. On-ramp links were replaced with simple links because of they malfunctioned in the merging areas. Other problems and details were fixed in order to better accommodate congestion caused along the mainline.

For the best hands-free calibration it is important to use past efforts elsewhere at calibrating a Paramics model. In a previous study done by the University of California at Berkley it was found that increasing or decreasing the mean target headway and the mean reaction time could change the overall behavior of the Paramics model. The default values for these parameters are both one, which represent United Kingdom (UK) traffic conditions. A value of 0.615 seconds for mean target headway and 0.415 seconds for the mean reaction time are recommended in the Berkeley study for US applications. Other factors were also calibrated, such as the curve speed factor from 1 to 5, the time step detail from 2 to 5 and the speed memory from 3 to 8 (Gardes, May, Dahlgren and Skabardonis, 2001).

The default truck percentage in Paramics is very high (12%). From the Quick Response Freight Manual (1996) new truck percentages were estimated for the area in study. It is important to recognize that those truck percentages are applied by Paramics uniformly to the entire network. The "signposting" default values were also changed in order to give all vehicles simulated a larger section of decision space before any particular hazard.

Figure 4-7 shows Paramics during a simulation.

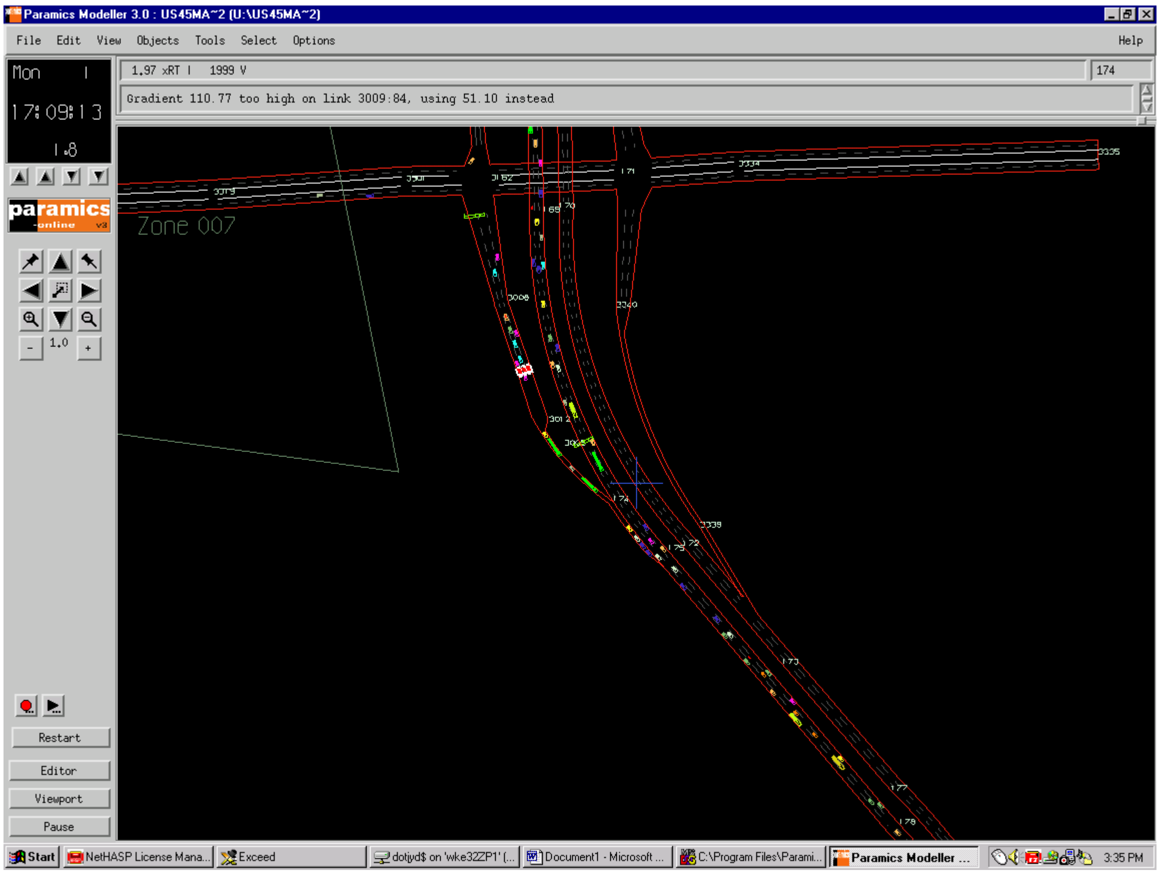


Figure 4-7. Screen Image of a Paramics Simulation

QRS II Simulation

In order to develop a macroscopic representation of the network in study, the General Network Editor (GNE) was used to code a network, and QRS II was used to analyze it. An experimental version of QRS II was used in this study because it is able to perform dynamic traffic assignment (DTA).

The network was coded using the QRSDetailed.dta application schema in GNE to obtain a better representation of reality. A correctly-scaled map of the western part of the Milwaukee metropolitan area was used as a background graphic to code the network.

The freeway section is the southbound lanes of US 45, including the principal arterials with access to the freeway. Also, 124th Street and Highway 100 are in the network but those roads are not used in this study.

There are 12 zones representing centroids along the freeway: a centroid for each on-ramp, the upstream zone and the downstream zone. There are no intersections with delay in the section in use. The meters are represented as intersections without delay, and the software represents them by using approach codes: "m" for a ramp with a meter of one lane and "M" for a ramp with a meter of two lanes.

Two-way links are used for the arterials, and one-way links are used for the freeway mainlines. The default value of speed in QRS II is for a level of service (LOS) C. The speed in the mainline is set to 55 mph. Depending on the number of lanes, the capacity attribute is set as follows:

1 lane at 1800 vph;
2 lanes at 3600 vph;
3 lanes at 5400 vph; and
4 lanes at 7200 vph.

The capacities were not calibrated to existing traffic counts, as is traditionally done in travel forecasting models.

The speed for links upstream of the meter was 25 mph. The capacity corresponds to the meter's rate, which was initially set to 600 vph for one lane links, and 900 vph for two-lane links (these values were later adjusted in order to replicate real meter rates and delays). The speed for on-ramps downstream from meters was 55 mph. The speed for on-ramps without meters was set to 45 mph.

Each period, February and March, required a separate network. The differences between them were the new meters implemented in March.

The delay caused by the meters was calculated using the same network as a reference but without the meters. Thus, the difference between the travel time on the link with a meter and the travel time on the link without the meter would be the delay caused by the meter implementation.

Figure 4-8 shows the network coded for this simulation.

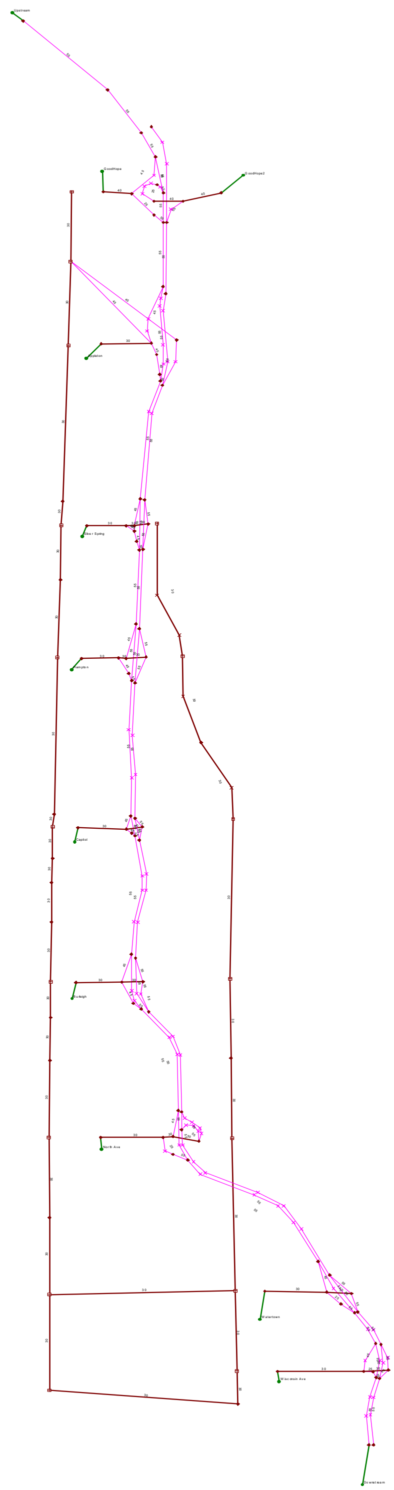


Figure 4-8. QRS II Network of US 45

Evaluation and Analysis of the Simulation Results

Ground Data

Traffic counts, speeds, and ramp queue length caused by ramp meters had been collected along US Highway 45 mainline and the on-ramps during the before and after periods. The data was collected for one and a half hours during the PM peak. The speeds and travel times are used for comparison purposes only.

Queue length information had been gathered by personnel in the field during an earlier WisDOT project. This data consists of the queue length of the on-ramps throughout the PM peak period. The queue length was reported every 20 seconds during that period. Tables 6 and 7 show the total delay and average delay caused by the ramp meters in each period:

Table 4-6. February Ground On-Ramp Delays

	Total Delay	Delay/Vehicle
On Ramp	**Sec**	**Sec**
GoodHope Loop	70460	54.6202
GoodHope Slip	1320	5.7391
North Ave	67760	84.3836
Watertown	90980	171.3371
Wisconsin	20880	36.1246

Table 4-7. March Ground On-Ramp Delays

	Total Delay	Delay/Vehicle
On Ramp	**Sec**	**Sec**
GoodHope Loop	53160	53.2665
GoodHope Slip	1140	6.4045
Appleton	4960	17.5265
Silver Spring	0	0
Hampton	21120	57.7049
Capitol	15360	22.0057
Burleigh	77940	109.0070
North Ave	51480	81.9745
Watertown	93840	168.1720
Wisconsin	22260	45.5215

Figure 4-9 compares the delay caused by the meters in February and March. Notice that the delay is very similar for the ramps with meters in both periods.

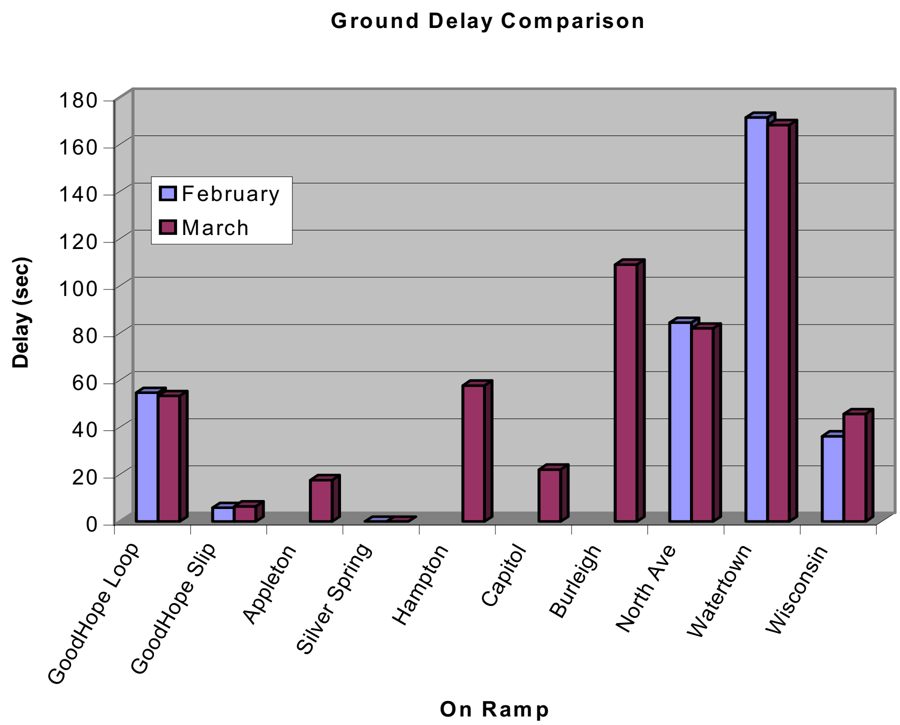


Figure 4-9. Ground Delay Comparison

Speeds were reported by detectors every 20 seconds and these speeds were validated against floating car runs. Tables 8 and 9 summarize the average speeds for each 15 minutes at several points along the freeway.

Table 4-8. February Ground Speeds (Speed Summary and Speed Profile for USH 45 Feb. 1 to Feb. 10, 2000 - Southbound PM Period (4:00 to 5:30) Source: Marquette University, Ramp meter project)

Checkpoint	4:00 to 4:15	4:15 to 4:30	4:30 to 4:45	4:45 to 5:00	5:00 to 5:15	5:15 to 5:30	Average Speed
Ped. (OH)							
	67.36	66.51	64.68	65.09	68.08	67.07	66.46
Pilgrim Road							
	68.69	70.34	68.32	67.92	68.50	67.04	68.47
Main Street							
	67.46	67.22	67.89	68.53	66.45	69.40	67.82
N. 124th Street							
	61.86	62.94	58.09	62.96	61.18	59.94	61.16
Good Hope							
	58.76	60.69	61.78	61.81	60.01	60.62	60.61
Silver Spring							
	62.39	63.66	64.61	61.17	62.52	65.27	63.27
Hampton							
	64.27	66.74	63.28	60.32	66.44	65.33	64.40
STH 190							
	57.05	61.71	60.34	49.21	58.12	54.44	56.81
Burleigh							
	57.75	58.53	52.11	46.31	39.51	52.02	51.04
North							
	45.38	57.88	53.88	32.09	28.66	45.02	43.82
STH 100							
	56.46	54.67	52.03	35.37	43.25	47.16	48.16
Watertown Plank							
	48.36	50.33	49.69	32.68	36.94	47.30	44.22
Bluemound							
	42.74	55.66	37.56	32.81	35.59	34.90	39.88
Schlinger							
	51.36	52.27	50.74	39.93	33.61	51.10	46.50
Belton (RR)							
Total	57.73	57.87	55.55	48.35	48.92	55.44	53.97

Table 4-9. March Ground Speeds (Speed Summary and Speed Profile for USH 45, Mar. 14 to Mar. 23, 2000 - Southbound PM Period (4:00 to 5:30) Source: Marquette University, Ramp meter project)

Checkpoint	4:00 to 4:15	4:15 to 4:30	4:30 to 4:45	4:45 to 5:00	5:00 to 5:15	5:15 to 5:30	Average Speed
Ped. (OH)							
	68.74	57.49	65.25	65.64	67.70	60.47	64.21
Pilgrim Road							
	69.74	79.08	66.89	61.93	69.72	57.81	67.53
Main Street							
	63.79	61.51	68.50	66.42	66.37	65.20	65.30
N. 124th Street							
	59.77	74.93	65.58	59.45	60.89	47.00	61.27
Good Hope							
	64.39	55.14	61.36	57.38	58.72	48.76	57.63
Silver Spring							
	61.55	62.67	63.00	58.87	56.73	59.64	60.41
Hampton							
	66.98	58.73	64.18	62.32	64.98	57.66	62.48
STH 190							
	62.18	61.91	56.91	46.16	57.66	69.10	58.99
Burleigh							
	59.15	49.72	49.19	43.58	61.01	67.42	55.01
North							
	55.35	45.00	25.34	44.39	50.34	23.42	40.64
STH 100							
	57.29	54.42	51.33	42.53	40.37	52.07	49.67
Watertown Plank							
	53.50	48.15	65.11	34.13	27.16	36.72	44.13
Bluemound							
	52.77	42.81	48.40	42.61	25.25	25.83	39.61
Schlinger							
	56.82	64.20	51.06	41.39	38.69	33.66	47.64
Belton (RR)							
Total	60.82	55.71	53.40	49.54	49.56	48.07	52.85

Unlike ramp meter delay, speeds vary from February to March. Figure 4-10 compares February and March average speeds along southbound US 45 in order to see the impact of the ramp metering.

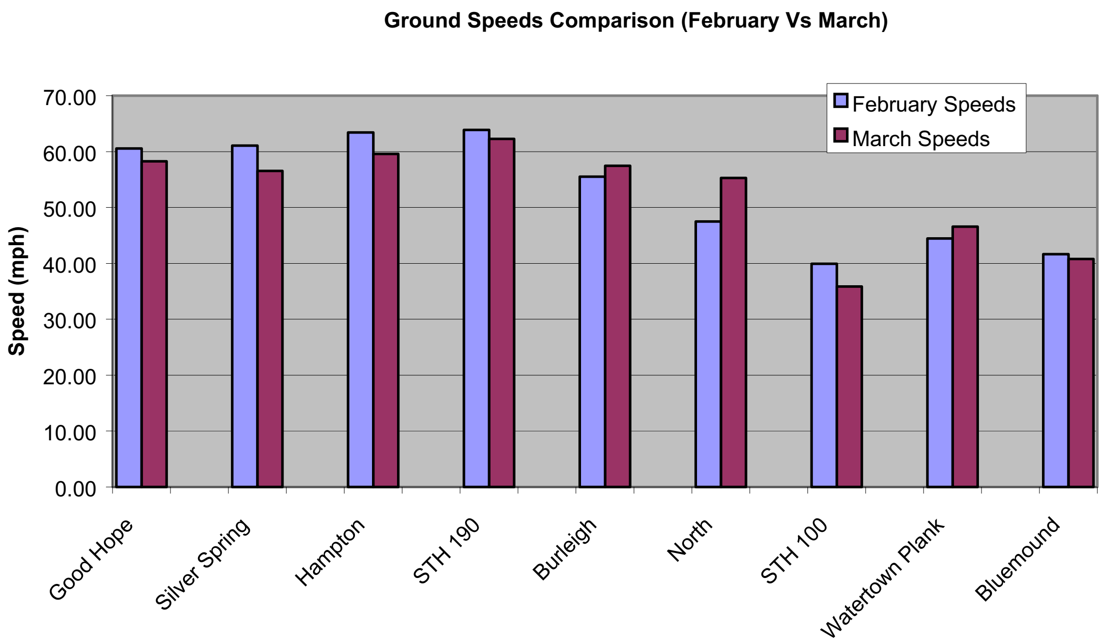


Figure 4-10. Total Average Ground Speeds Comparison

The speeds and delays showed in Figure 4-10 are compared below with the results obtained by modeling the network with both Paramics and QRS II.

Analysis of Paramics Results

The simulation covered a period of 1 hour and 30 minutes in order to generate a spin-up of vehicles into the network. Speed comparisons are for the last hour of the simulation, only. Tables 10 and 11 show the speeds obtained at specific Paramics detector locations.

Table 4-10. February Paramics Speeds

	4:30 to 4:45	4:45 to 5:00	5:00 to 5:15	5:15 to 6:00
Good Hope Loop	58.8	61.1	55.1	53.35
Good Hope Slip	65.5	57.6	62.6	36.8
Appleton	63.2	55.1	59.7	65.6
Silver Spring	63.3	63.1	57.2	59.2
Hampton	66.3	63.3	66.0	58.2
Capitol	66.0	63.8	64.0	54.5
Burleigh	53.9	61.7	19.1	37.5
North	41.3	26.3	28.6	41.8
Water Town	50.5	19.9	5.3	55.8
Wisconsin	49.1	23.9	25.8	14.8

Table 4-11 March Paramics Speeds

	4:30 to 4:45	4:45 to 5:00	5:00 to 5:15	5:15 to 6:00
Good Hope Loop	62.9	67.5	59.2	63.3
Good Hope Slip	61.8	56.7	60.7	59.9
Appleton	63.0	57.9	62.9	58.9
Silver Spring	62.2	58.7	66.2	61.9
Hampton	51.6	54.5	63.6	51.1
Capitol	39.9	63.8	64.4	0.0
Burleigh	57.2	57.8	51.5	64.0
North	50.2	48.0	38.6	48.6
Water Town	60.8	38.2	60.4	62.2
Wisconsin	56.9	36.6	29.7	12.9

The variations of the speeds for each 15-minute interval depend on the percentage of demand assigned to that time slice. It is important to note that Paramics did not record the speeds obtained for some 15-minute slices at some lanes. The value of the speed equal to zero at Capitol Drive is an example of a non-recorded value in any of the lanes. February and March speeds are compared in Figure 4-11. Total average speeds simulated by Paramics increased by about 10% from February to March.

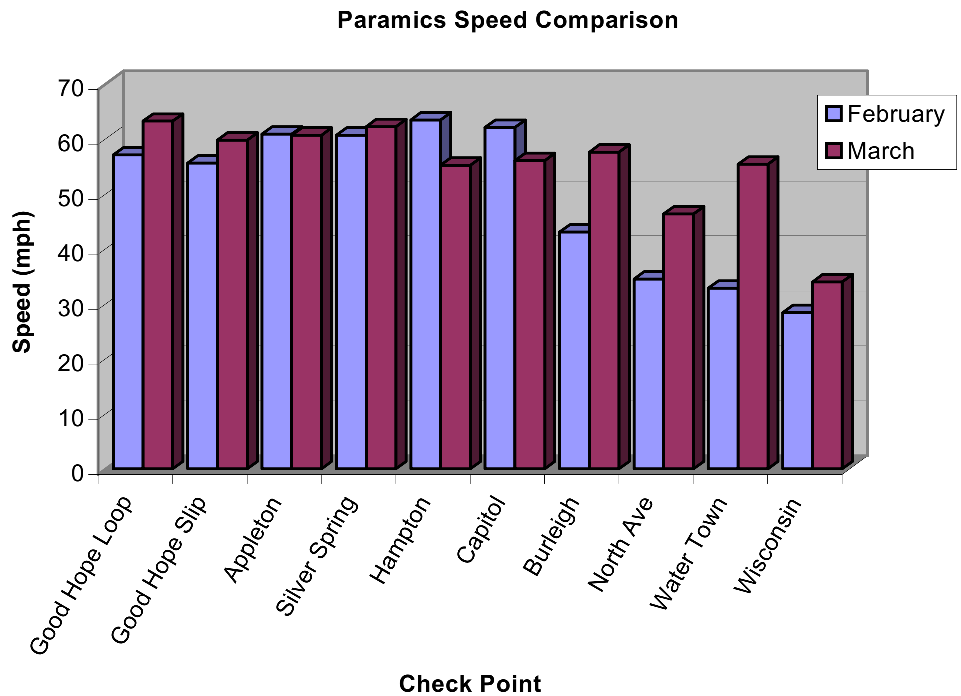


Figure 4-11. Paramics Speed Comparison

In addition to speeds, travel times from zone to zone were extracted from Paramics. Tables 12 and 13 present travel times found by Paramics from zone to zone.

Table 4-12. February Zone to Zone Travel Times

	February Travel Times	1	2	12	3	4	5	6	7	8	9	10	11
1	Upstream Zone	0.0	1.4	0.0	2.8	3.3	4.0	4.7	6.4	8.0	8.9	10.3	9.7
2	Good Hope Road (Slip)		0.0	0.0	1.8	2.6	3.6	4.0	5.2	6.7	9.8	10.4	10.4
12	Good Hope Road (loop)			0.0	3.3	4.3	4.9	5.1	7.3	8.5	10.3	10.3	11.5
3	Appleton Avenue				0.0	1.7	0.0	3.7	4.8	6.3	3.6	9.7	8.9
4	Silver Spring Drive					0.0	0.6	2.4	3.5	4.5	7.4	9.1	8.7
5	Hampton Avenue						0.0	1.2	2.9	6.3	4.8	0.0	7.9
6	Capitol Drive							0.0	4.1	4.3	7.0	7.3	8.7
7	Burleigh Street								0.0	3.2	4.3	2.2	6.8
8	North Avenue									0.0	0.0	5.3	7.2
9	Watertown Plank Road										0.0	1.5	3.3
10	Wisconsin Avenue											0.0	3.1
11	Downstream												0.0

Table 4-13. March Zone to Zone Travel Times

	March Travel Times	1	2	12	3	4	5	6	7	8	9	10	11
1	Upstream Zone	0.0	1.4	0.0	2.8	3.5	4.1	4.6	6.9	8.1	7.6	8.7	8.9
2	Good Hope Road (Slip)		0.0	0.0	1.9	3.0	3.9	4.4	5.8	6.4	5.7	8.0	10.7
12	Good Hope Road (loop)			0.0	3.7	4.2	5.2	5.9	7.7	7.0	9.8	9.0	10.9
3	Appleton Avenue				0.0	2.3	0.0	4.2	6.2	6.3	7.6	8.2	8.9
4	Silver Spring Drive					0.0	1.4	2.2	4.2	5.1	6.3	6.2	7.9
5	Hampton Avenue						0.0	2.0	3.1	4.6	5.9	4.1	7.3
6	Capitol Drive							0.0	3.3	3.4	4.6	5.2	6.7
7	Burleigh Street								0.0	3.8	5.4	6.7	6.4
8	North Avenue									0.0	0.0	4.9	6.7
9	Watertown Plank Road										0.0	1.5	2.9
10	Wisconsin Avenue											0.0	2.6
11	Downstream												0.0

Note that in Tables 12 and 13 there are travel times equal to 0. Paramics did not calculate those values because there were no trips between those two zones.

There are also longer travel times for closer zones (for example, notice that sometimes it is faster to travel from zone 1 to zone 9 than to travel from zone 1 to zone 8). This could be happening because the types of vehicles traveling between two zones are different for each pair; the vehicles traveling longer distances use the left most lanes, which usually allows greater speeds; or there is stochastic variation in the model. For trips between a different pair of zones these travel times include the delay caused by the ramp meter at the on-ramp of the origin zone.

After simulating US 45 with Paramics the following observations and comments about its limitations are noted.

- Paramics is sensitive to the percentage of trucks, but the vehicle mix is constant for the network; it cannot be varied zone to zone.
- Driver behavior cannot be varied from zone to zone.
- Paramics has traffic signals, but it does not have true ramp meters. The following ramp meter characteristics cannot be represented:
    - Alternate release of vehicles for on-ramps of more than one lane;
    - Provision of HOV lanes at meters;
    - Identification of HOV's at any location; and
    - Fractional seconds of cycle lengths and green times.
- It is not easy to compare Paramics queue lengths at meters with field data because:
    - Paramics reported queue length at the start of each green phase, which occurs at variable times depending on the traffic flow characteristics;
    - Queues of zero length were not reported;
    - Paramics default definition of a queue was a group of vehicles moving at less than 4.5 mph, which is too low for the definition of "queue" at some on-ramps.
- The way that Paramics splits trips to all links in a zone is not behaviorally plausible.

- Some peculiarities of geometry cannot be accommodated except speed, width, type and curve speed reduction,
- The default parameters are erroneous. Parameter adjustment is required for US applications.

Consequently, these issues limit how well a simulation can approximate reality.

Analysis of QRS II Results

As mentioned previously, an experimental version of QRS II was used for this analysis. This version performs dynamic traffic assignment (DTA) using time slices that were set at 15 minutes for the study. A percentage of demand is assigned to each time slice in order to simulate real traffic conditions.

QRS II provides a summary report where all the network characteristics are presented. The measures of effectiveness (MOE's) given by QRS II show that vehicle distance traveled (VDT) decreased from 50,713 miles in February to 47,473 miles in March, or 6% less. This occurred because the March flow rates are less than the February flow rates. The vehicles hours traveled (VHT) also decreases from 1,717 in February to 1,380 in March, or 20% less.

The average speed increased from February to March. In QRS II uninterrupted speeds are calculated using the BPR (Bureau of Public Roads) curve, which depends on the volume/capacity (V/C) ratio. Thus, having lower flow rates causes the speed to increase. Tables 4.10 and 4.11 present the speeds obtained by QRS II for each period.

Table 4-14. February QRS II Speeds (mph)

	4:30 to 4:45	4:45 to 5:00	5:00 to 5:15	5:15 to 6:00
Good Hope Loop	25.93	22.7	25.93	29.42
Good Hope Slip	36.66	33.29	36.66	40.05
Appleton	39.62	36.35	39.62	42.84
Silver Spring	46.04	43.21	46.04	48.7
Hampton	40.37	37.14	40.37	43.54
Capitol	48.95	46.42	48.95	51.28
Burleigh	47.5	44.82	47.5	50.01
North Ave	42.07	38.93	42.07	45.1
Water Town	32.77	29.86	32.77	36.28
Wisconsin	28.28	24.96	28.28	31.81

Table 4-15. March QRS II Speeds (mph)

	4:30 to 4:45	4:45 to 5:00	5:00 to 5:15	5:15 to 6:00
Good Hope Loop	39.04	35.75	39.04	42.3
Good Hope Slip	49.69	47.25	49.69	51.93
Appleton	50.87	48.59	50.87	52.96
Silver Spring	52.56	50.5	52.56	54.41
Hampton	43.07	39.99	43.07	46.02
Capitol	50.5	48.17	50.5	52.63
Burleigh	47.15	44.42	47.15	49.69
North Avenue	42.86	39.78	42.86	45.84
Water Town	37.02	33.66	37.02	40.38
Wisconsin	34.26	30.86	34.26	37.73

The speeds for each 15-minute interval depend on the percentage of demand assigned to that time slice. Notice that the speeds obtained for the first and third 15-minute slices are the same. This occurs because QRS II calculates the travel times depending on V/C ratios; the volumes assigned for these two time slices (15 and 45 minutes) are equal and vehicles in these time slices are able to finish their trip within the 15-minute length of the time slice.

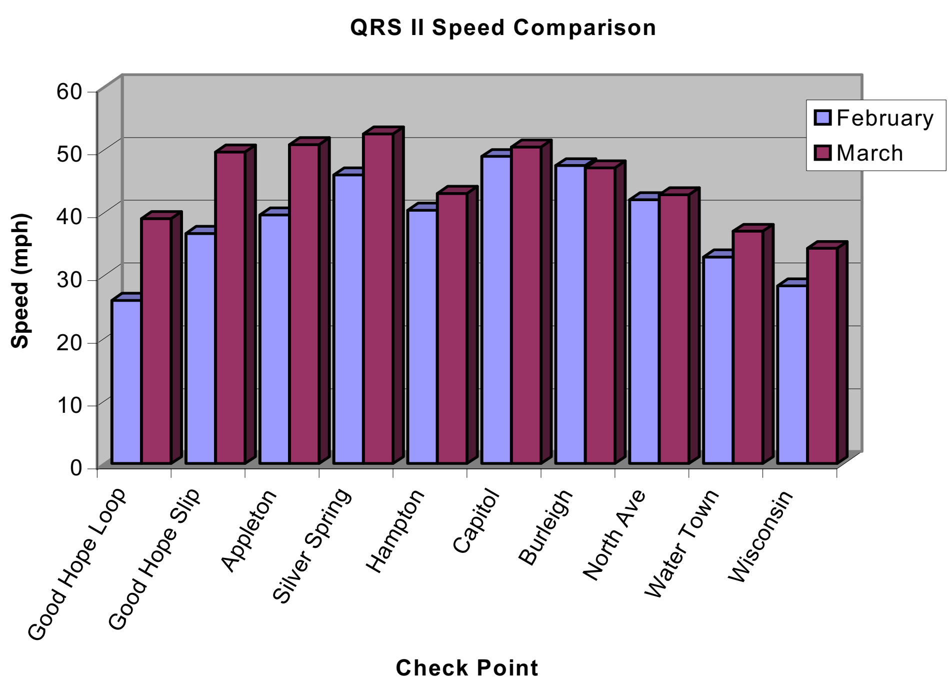


Figure 4-12. QRS II Speeds Comparison

Using QRS II total average speeds increased in by 15.1% between February and March. Figure 4-12 compares the February and March average speeds along US Highway 45 obtained by QRS II.

Ramp meter rates in QRS II are represented as single "capacity" of the ramp. As a starting point meter rates of 900 vph and 600 vph were assigned to two-lane and one-lane on-ramps, respectively. Meter rates needed to be adjusted to obtain a better approximation of the model to rates actually experienced. Tables 16 and 17 show the adjusted meter rates and the new delays obtained by running QRS II for each period:

Table 4-16. February Meter Rates and Delay Adjustment

	Meter Rate	Travel Time	Travel Time No Meter	Delay (Seconds)
GH Loop	1200	4.741	0.246	269.706
GH Slip	850	0.497	0.202	17.718
North	1000	1.577	0.154	85.410
Watertown	600	3.142	0.121	181.266
Wisconsin	800	0.752	0.063	41.358

Table 4-17. March Meter Rates and Delay Adjustment

	Meter Rate	Travel Time	Travel Time No Meter	Delay (Seconds)
GH Loop	1200	1.938	0.237	102.048
GH Slip	850	0.491	0.201	17.400
Appleton	650	0.431	0.140	17.472
Silver Spring	-	0.077	0.077	0.000
Hampton	470	0.984	0.126	51.486
Capitol	1000	0.520	0.043	28.674
Burleigh	840	2.106	0.167	116.370
North	790	1.587	0.152	86.124
Watertown	620	2.896	0.121	166.500
Wisconsin	660	0.954	0.061	53.538

Figures 13 and 14 show the comparison between QRS II delays and ground delays for the on-ramps for each period.

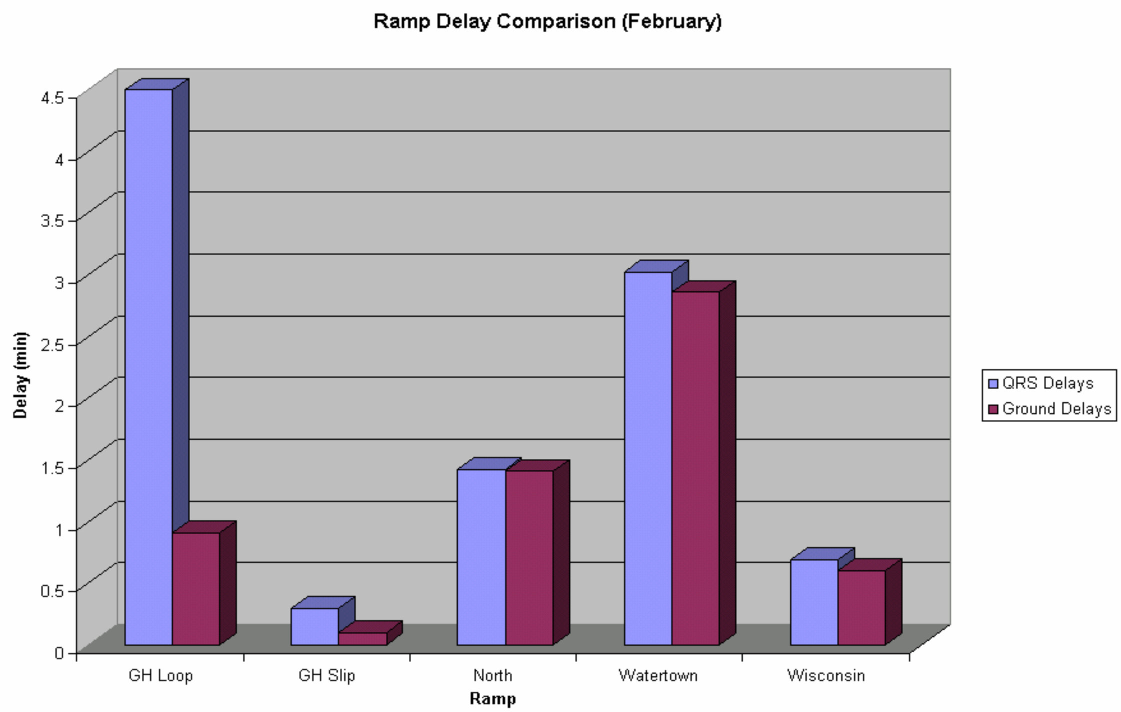


Figure 4-13. February Ramp Delay Comparison

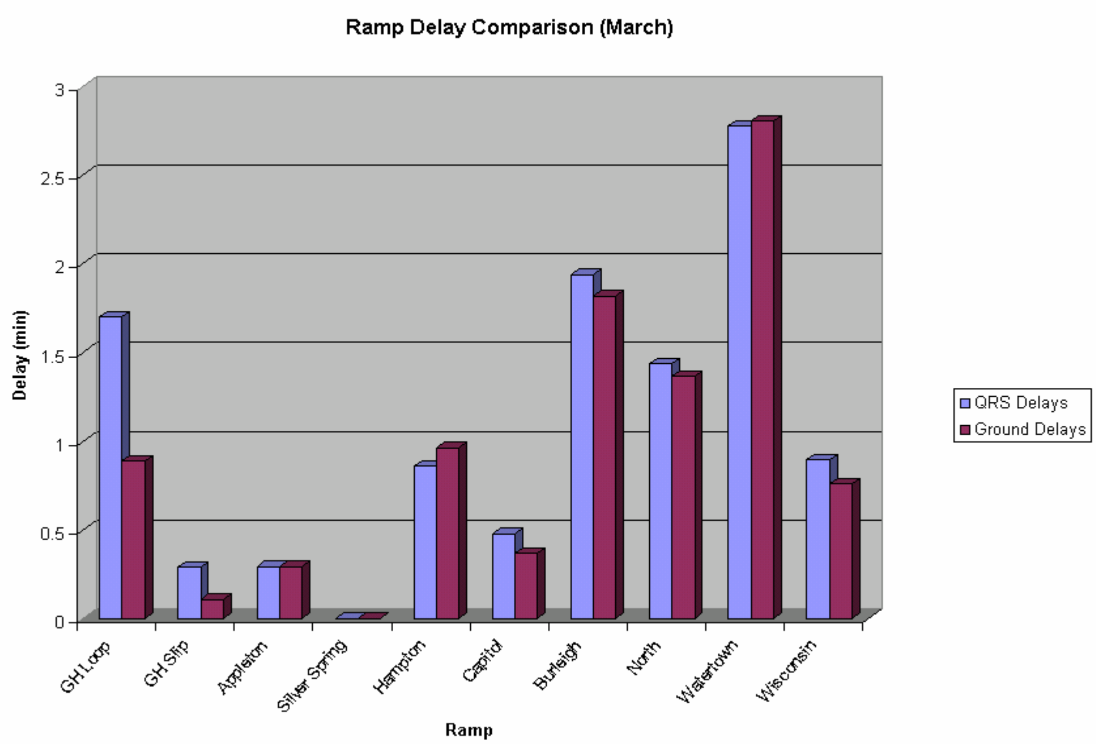


Figure 4-14. March Ramp Delay Comparison

On most ramp meters, QRS II delays are very similar to ground delays. The Good Hope ramp presents the only large difference between ground and QRS II delays. The meter rate used for both periods is 1200 vehicles per hour (vph), which corresponds to the minimum reasonable cycle length of 4 seconds (2.5 seconds of red and 1.5 of green). Higher values of meter rates would be similar to the no-meter condition.

Tables 18 and 19 present the travel times obtained by QRS II from zone to zone.

Table 4-18 February QRS II Travel Times

	QRS Travel Times	1	2	12	3	4	5	6	7	8	9	10	11
1	Upstream	0.0	2.3	0.0	3.6	4.6	5.4	6.6	7.6	8.9	10.5	11.0	11.1
2	GoodHope Slip		0.0	0.0	2.8	3.8	4.7	5.8	6.8	8.1	9.7	10.3	10.3
12	GoodHope Loop			0.0	3.5	4.5	5.4	6.5	7.5	8.8	10.4	10.9	11.0
3	Appleton				0.0	2.9	3.8	4.9	5.9	7.2	8.8	9.3	9.4
4	Silver Spring					0.0	2.3	3.4	4.5	5.7	7.3	7.9	7.9
5	Hampton						0.0	2.6	3.6	4.9	6.5	7.0	7.1
6	Capitol							0.0	2.6	3.9	5.5	6.0	6.1
7	Burleigh								0.0	2.9	4.5	5.1	5.1
8	North Ave									0.0	3.7	4.3	4.3
9	Watertown										0.0	3.1	3.1
10	Wisconsin Ave											0.0	2.4
11	Downstream												0.0

Table 4-19 March QRS II Travel Times

	QRS Travel Times	1	2	12	3	4	5	6	7	8	9	10	11
1	Upstream	0.0	2.3	0.0	3.6	4.6	5.4	6.6	7.6	8.9	10.5	11.0	11.1
2	GoodHope Slip		0.0	0.0	2.8	3.8	4.7	5.8	6.8	8.1	9.7	10.3	10.3
12	GoodHope Loop			0.0	3.5	4.5	5.4	6.5	7.5	8.8	10.4	10.9	11.0
3	Appleton				0.0	3.0	3.9	5.0	6.0	7.3	8.9	9.5	9.5
4	Silver Spring					0.0	2.3	3.4	4.5	5.7	7.3	7.9	7.9
5	Hampton						0.0	2.7	3.7	5.0	6.6	7.2	7.2
6	Capitol							0.0	2.7	3.9	5.5	6.1	6.1
7	Burleigh								0.0	3.1	4.7	5.2	5.3
8	North Ave									0.0	3.7	4.3	4.3
9	Watertown										0.0	3.1	3.1
10	Wisconsin Ave											0.0	2.4
11	Downstream												0.0

It is important to note that Paramics travel distances are somewhat longer than those obtained by QRS II. The vehicles travel until a link end in Paramics, which is farther away than the trip-ending centroid in QRS II.

Comparing Speeds between Paramics, QRS II and Ground Data

The following several figures (15 to 28) show the speed comparison between ground, Paramics and QRS II for each segment. Those speeds were measured or obtained at the mainline link before each on-ramp (at the detectors' location).

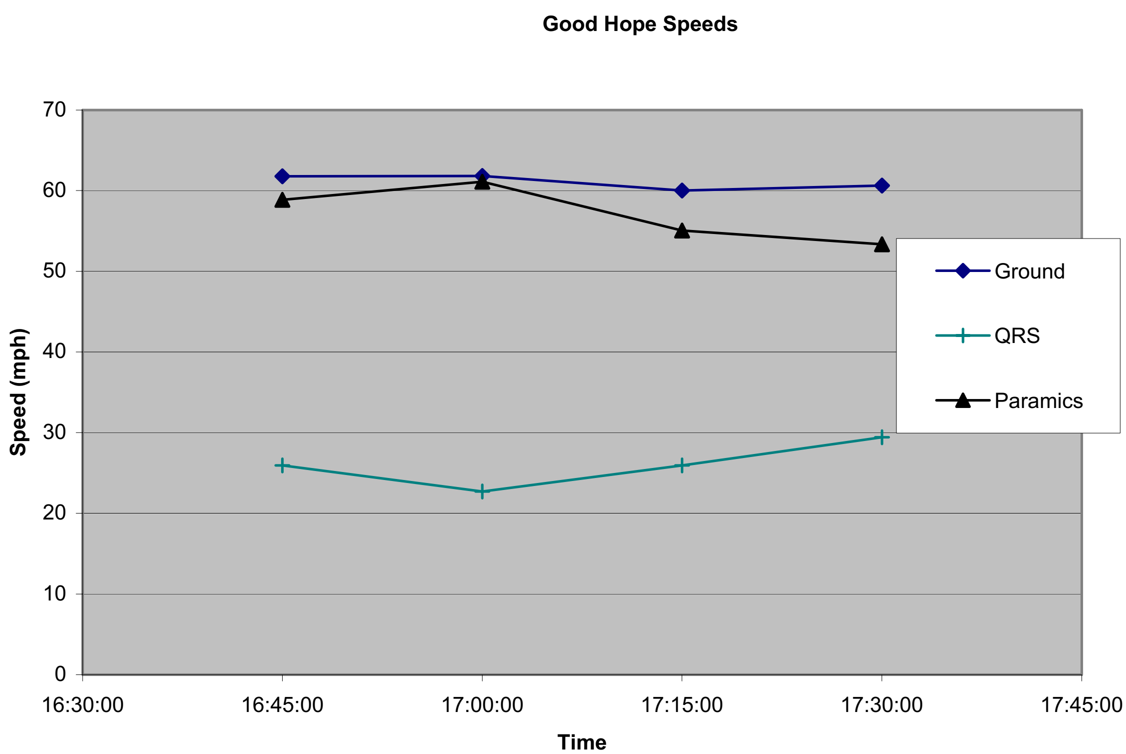


Figure 4-15. February Good Hope Speed Comparison

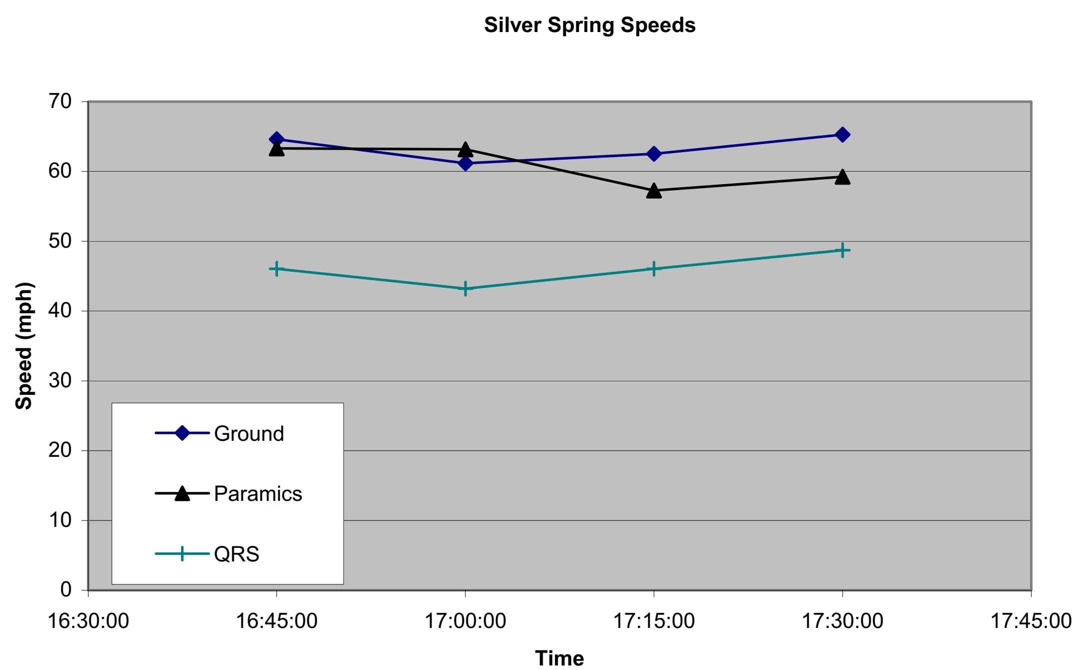


Figure 4-16. February Silver Spring Drive Speed Comparison

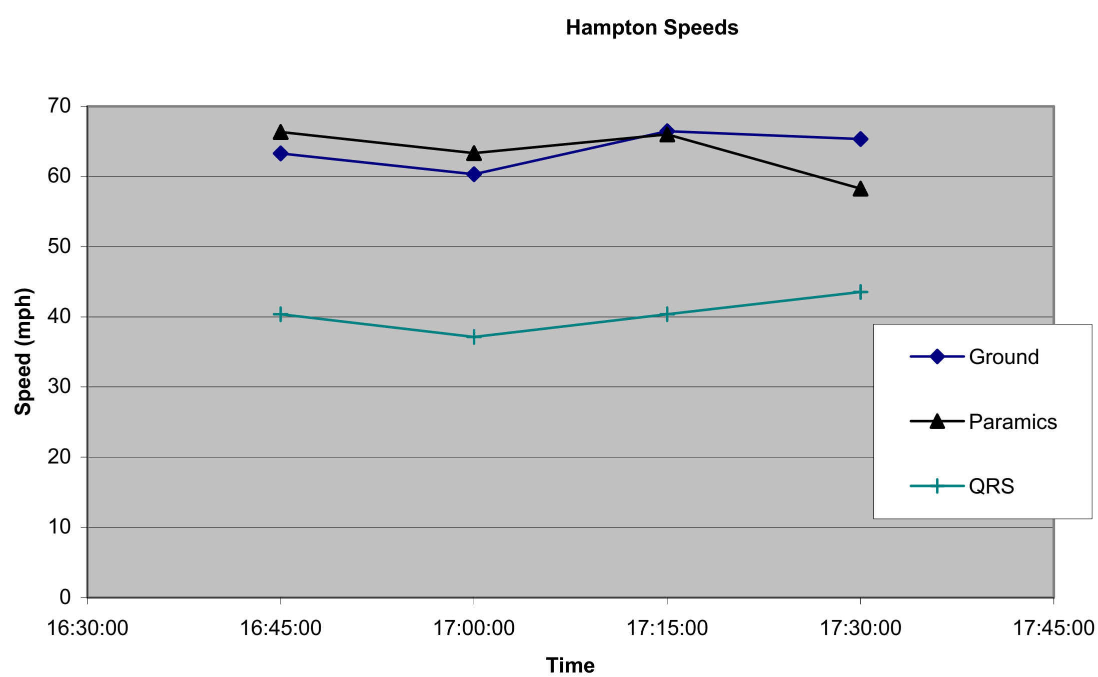


Figure 4-17. February Hampton Avenue Speed Comparison

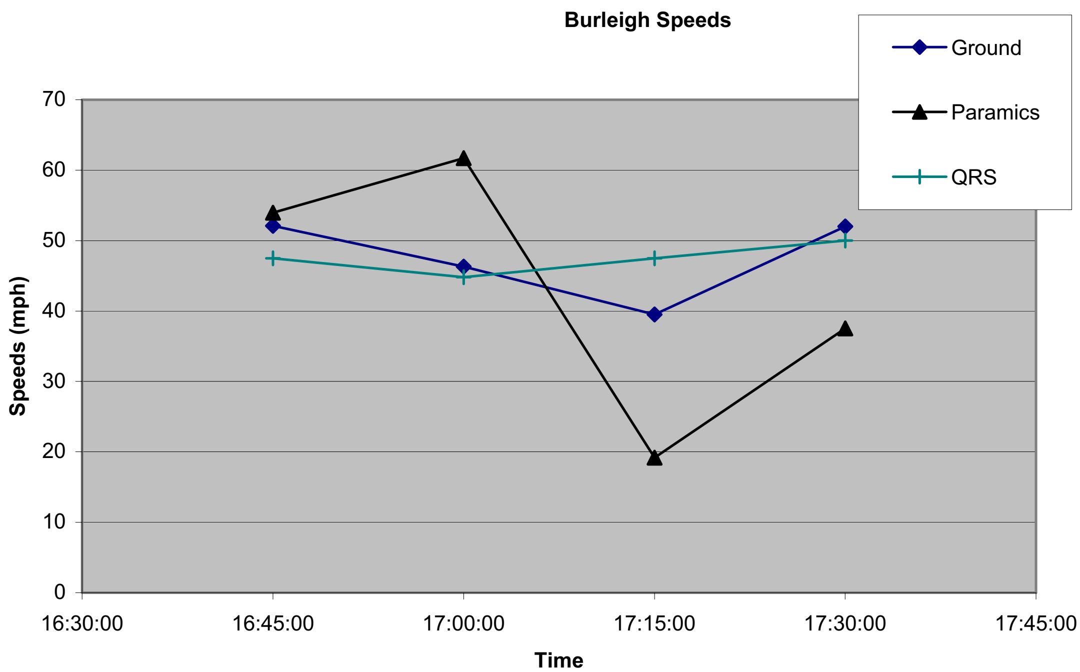


Figure 4-18. February Burleigh Street Speed Comparison

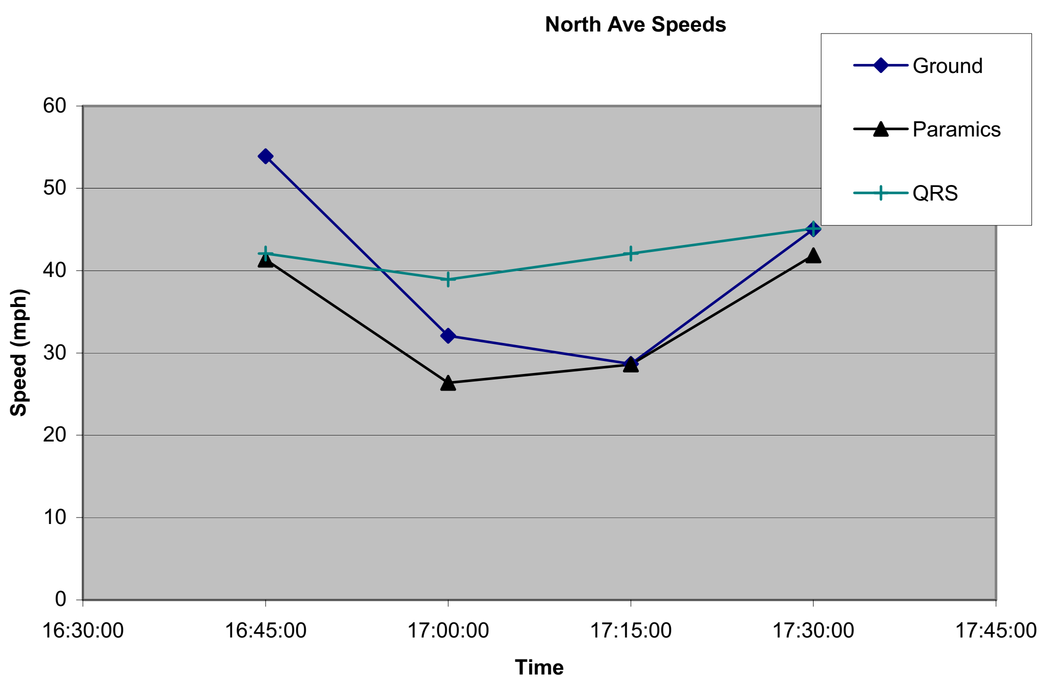


Figure 4-19. February North Avenue Speed Comparison

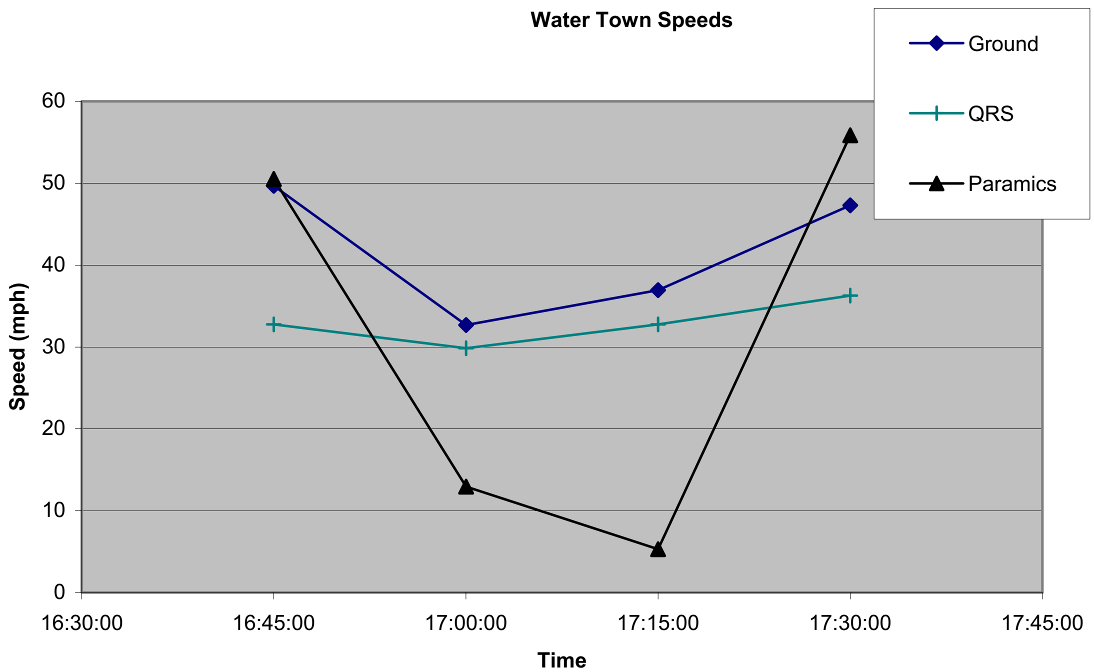


Figure 4-20. February Watertown Speed Comparison

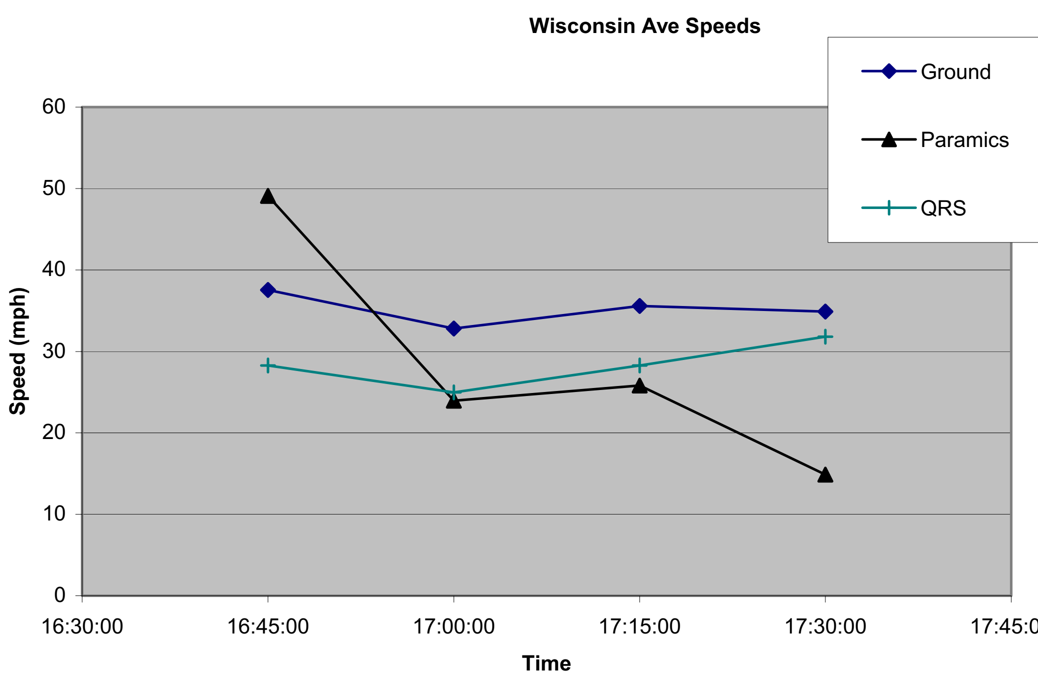


Figure 4-21. February Wisconsin Avenue Speed Comparison

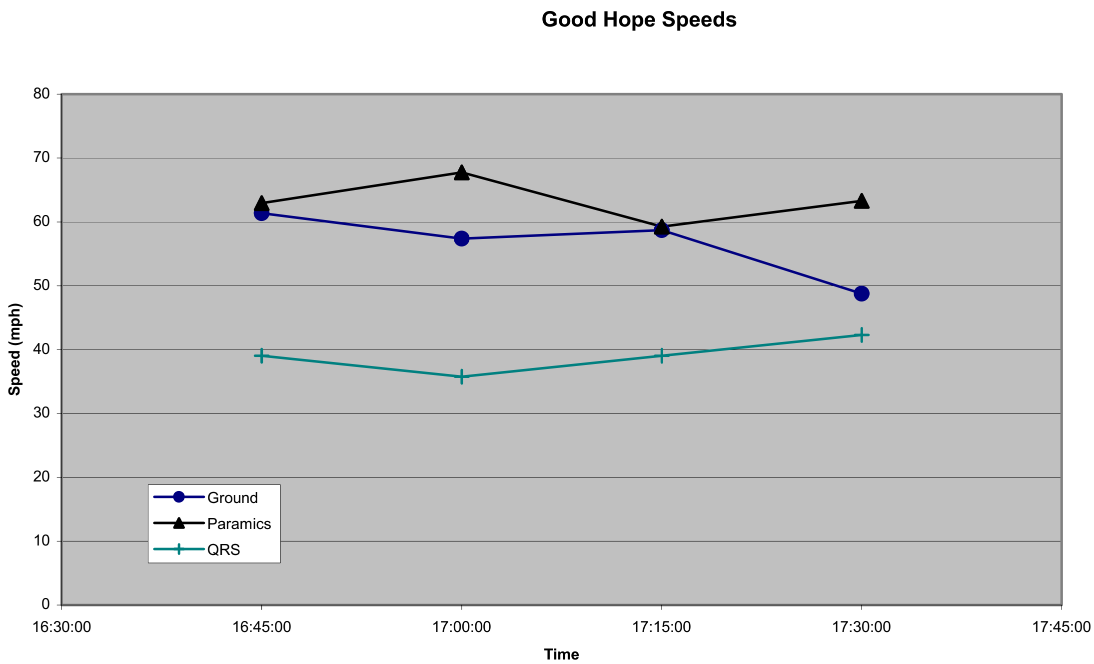


Figure 4-22. March Good Hope Speed Comparison

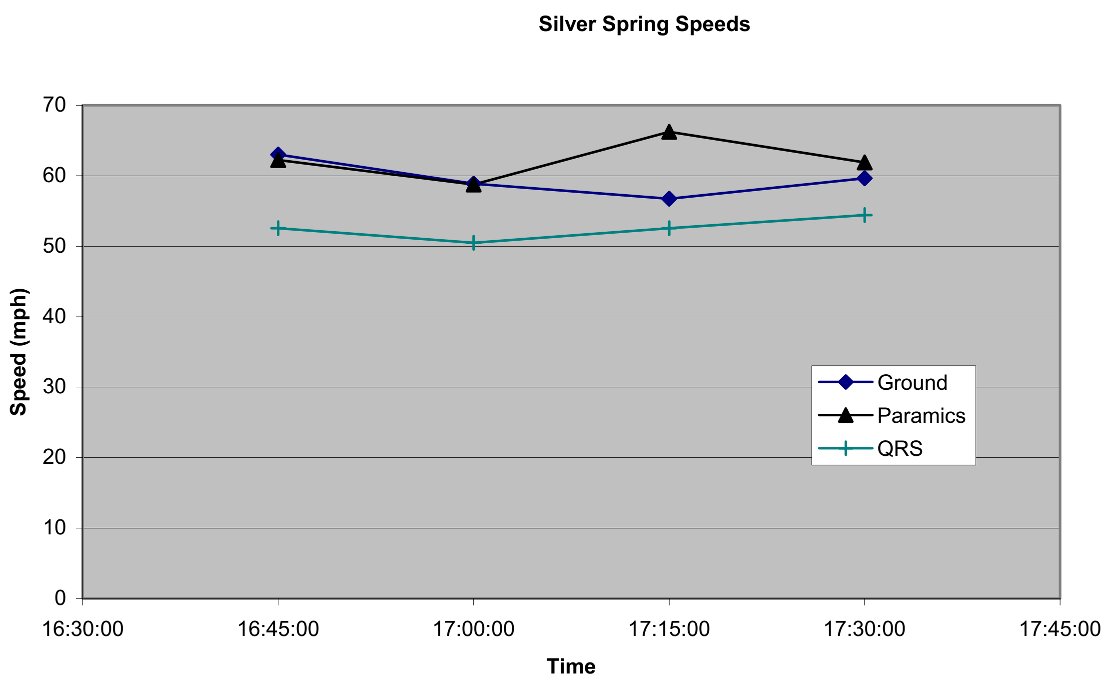


Figure 4-23. March Silver Spring Drive Speed Comparison

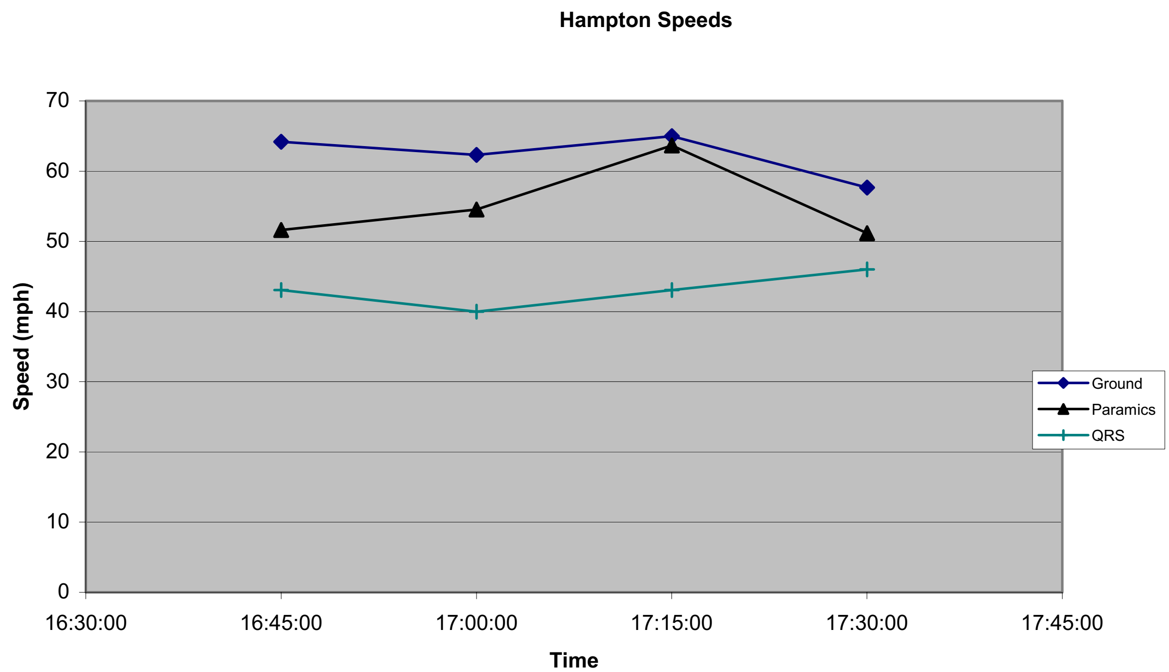


Figure 4-24. March Hampton Avenue Speed Comparison

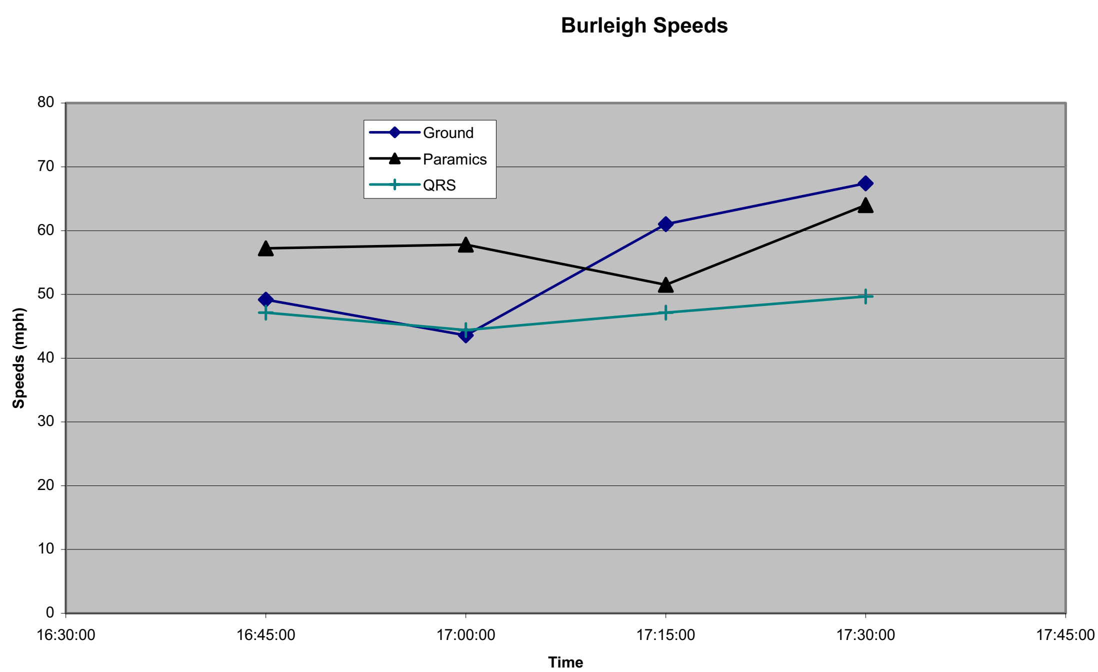


Figure 4-25. March Burleigh Street Speed Comparison

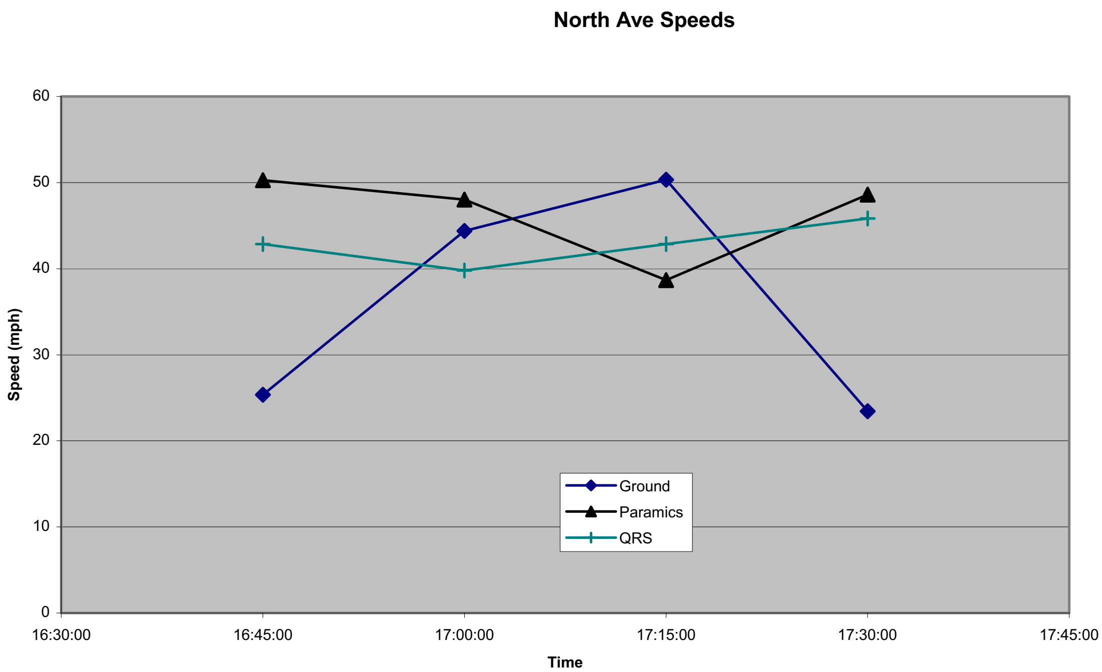


Figure 4-26. March North Avenue Speed Comparison

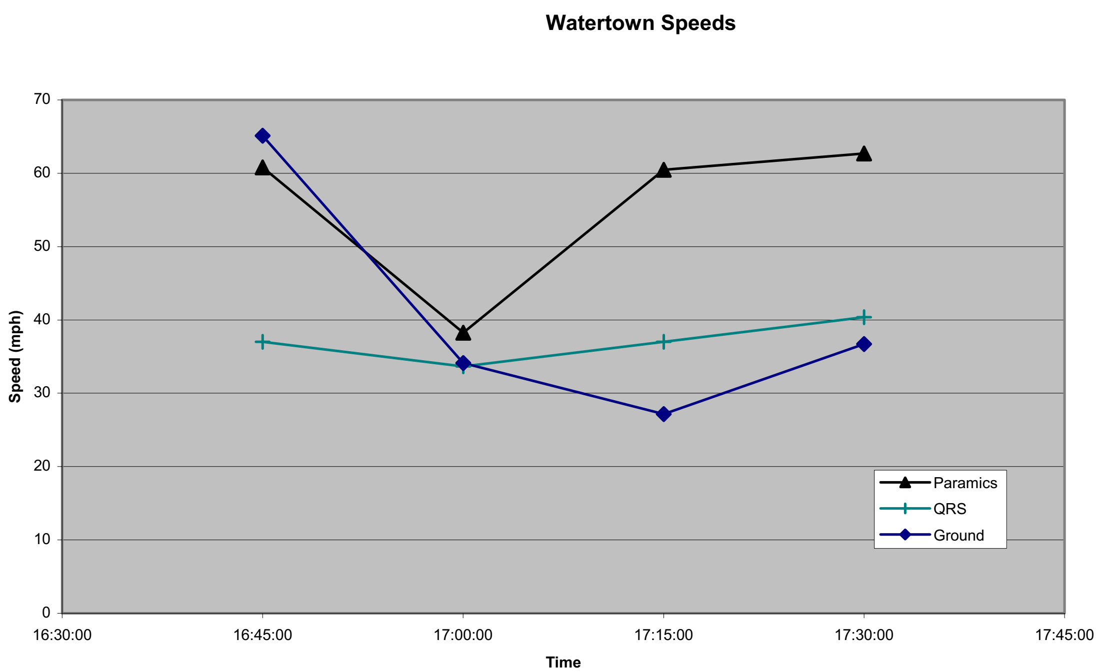


Figure 4-27. March Watertown Speed Comparison

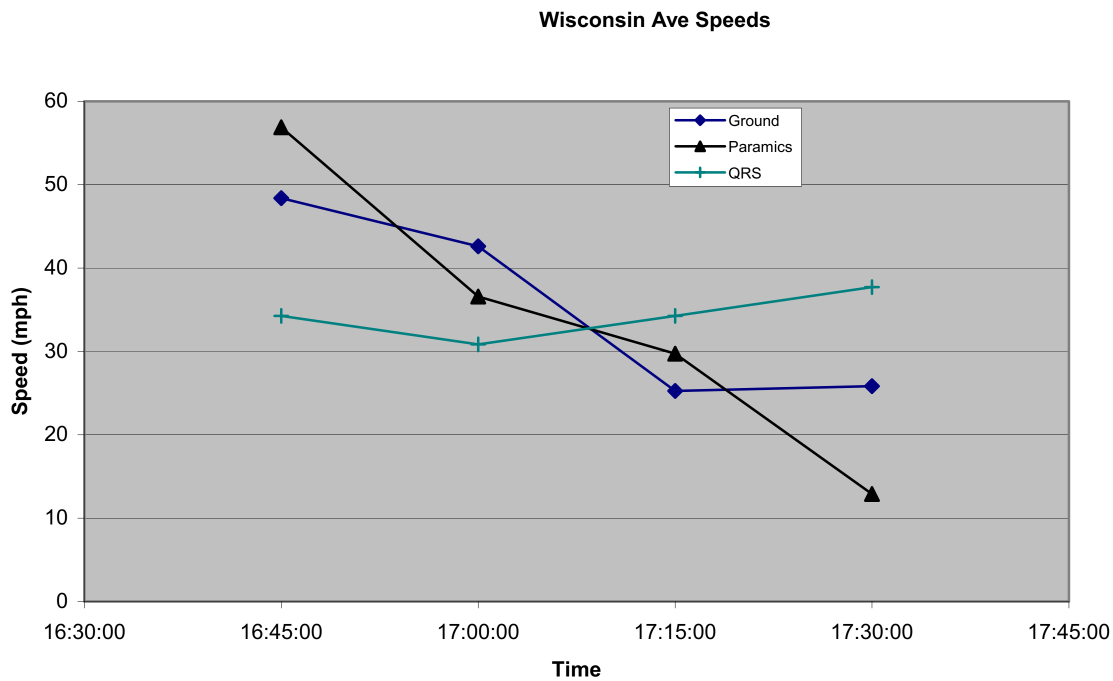


Figure 4-28. March Wisconsin Avenue Speed Comparison

Paramics gives speeds estimates that are similar to actual speeds. On the other hand, the QRS II speeds are generally lower than Paramics speeds. Ground data shows that speeds are not increasing as was expected to occur after the ramp meter implementation (March period). Speeds resulting from the simulation of both models increased.

Overall Validation Quality

A root mean square (RMS) deviation analysis is conducted in order to establish the quality of results obtained after simulating using both models. The speeds from each model are compared in order to identify which model provides the more accurate results. The following is the formula used in this analysis:

$$RMS = \sqrt{\frac{1}{N}\sum(\bar{X} - X)^2}$$

where $\bar{X}$ is the average mean speed taken from the traffic counts for each of N points along the freeway, and X is the average mean speed obtained by the model at the same point. Table 4-20 shows the RMS errors.

Table 4-20. Root Mean Square Deviation from Ground Speeds

Root Mean Square	QRS II	Paramics
February	17.8	5.3
March	11.1	7.9

The RMS deviations show that Paramics gives a more accurate speed results than QRS II, given the relatively hands-free calibrations of the models. QRS II's consistent underestimation could be improved by changing the lane capacity assumption.

Criteria for Models Evaluation

In order to identify different capabilities and limitations of the models in this evaluation, it was helpful to establish the following list of criteria in which they are compared.

- General: A general comparison was made in order to compare the models' capabilities such as:
    - How much data does the model require?
    - Does it have a network editor?
    - What is its complexity?
    - What is its learning time (comparing the models to each other)?
    - What is its ease of use?
    - What units can be used (SI and British)?
    - What is the computation time (minutes)?
    - Is it well documented?
- Ramp Metering: This criterion compares how well the model represents ramp meters. The most important point is if the model is capable of replicating WisDOT meters rates.
    - Strategies: fixed time, local traffic responsive, system wide traffic responsive.
    - Devices: Meters and detectors
    - Capability of replicating WisDOT ramp meter rates
- Simulation outputs: It is necessary that the outputs of the model are useful, readable and reported at the required period of time.
- Summarized: Is there a general report of the measures of effectiveness?
    - Is it exportable to other formats?
    - Usefulness
    - Does it have speeds?
    - Does it have queue length?
    - Does it have travel times?
    - Are results reported at the required time slice?
- Accuracy: The most comparable output with ground data is the speed; its accuracy needs to be calculated.
    - February speeds: % of the ground speed
    - March speeds: % of the ground speed
    - Root mean square error

The Table 4-21 shows the comparison between Paramics and QRS II using these criteria.

Table 4-21. Software Comparison

Criteria	Microscopic Paramics	Macroscopic QRS II
General		
Data Required	more	less
Network editor	yes	yes
Complexity	high	low
Learning time	moderate	moderate
Ease of use	moderate	easy
Units	SI and British	SI and British
OD Trip Tables	User supplied	Created or user supplied
Computation Time	20 min	1 min
Well documented	no	yes
Ramp Metering Strategies		
Fixed Time	yes	yes
Local Traffic Response	yes	no
System-wide T.R.	maybe	maybe
Devices		
Meters	yes	yes
Detectors	yes	no
WisDOT Meter rates	possible	not possible
Simulation outputs		
Summarized	some	some
Exportable to other formats	yes	yes
Usefulness	high	high
Type of output		
Speeds	yes	yes
Queue Length	some	no
Travel Times	yes	yes
At required time slice	yes	yes
Accuracy (Compared v. Ground Data)		
February Speeds	91%	75%
March Speeds	108%	86%
Root Mean Square (February)	5.3	17.8
Root Mean Square (March)	7.9	11.1

Both models have pros and cons for traffic simulation. While QRS II is friendlier to the user and requires less data, Paramics is able to represent many more ramp meter strategies and can be used to directly assess ramp meter benefits in Wisconsin. Neither model was tested with system-wide ramp meter control, but both models should be capable of modeling the traffic impacts of such control given a considerable amount of additional computer programming. Because of QRS II's speed advantage, it would be better for finding optimal control strategies.

A critical MOE is the average trip duration of all trips made from every zone. Before (without meters) and after (with meters) periods are compared by using Paramics results, in order to see whether ramp meters are improving traffic conditions.

In order to get the total travel time of all vehicles from one zone to other, the number of trips between an OD pair is multiplied by the average travel time for that trip during the peak hour. Then, the average trip duration is obtained by dividing the sum of all average travel times by the total number of trips in each period (before and after). The results of this comparison is shown on Table 4-22.

Table 4-22. Ramp Meter's Average Trip Duration Improvement

	Average Trip Duration (min)	**Improvement**
February	6.76	
March	6.24	7.6%

The results obtained show a reduction in average trip duration from February to March by 7.6%, though the fraction of very short trips is smaller in March than in February.

Conclusions and Recommendations

Conclusions

The use of a simulation model enables planners and engineers to model actual traffic situations and estimate impacts caused by any improvement to a traffic network. A simulation model also allows engineers to create and evaluate scenarios and alternatives during the planning phase to obtain better results in optimizing traffic control.

This study evaluates two different software packages for simulating the traffic conditions of US Highway 45 before and after new ramp meters are deployed. The study section includes the southbound lanes from upstream at Good Hope Road to downstream at Wisconsin Avenue. Ground data was collected in February 2000 as a "without ramp metering" period and March 2000 as a "with ramp metering" period by UWM, Marquette University and WisDOT.

Prior to both simulations, it was necessary to estimate the OD trip table. The generalized least squares (GLS) technique worked well.

Testing Paramics and QRS II gives an idea of how different types of traffic simulation models (microscopic and macroscopic, respectively) represent actual conditions. Both models were tested with a minimumal amount of calibration to ground data, so that the validation would not be compromised.

During Paramics network augmentation, it was found that this model is able to simulate more realistic traffic situations such as driver behavior, different vehicle types, and advanced signal control. The large amount of data needed by Paramics and the time required to code the network could be detrimental. Also, programming is required in order to represent an actuated traffic signal, making it even more time-consuming. QRS II uses less data for the same representation and requires far less time for coding and computation.

By using traffic signals Paramics represents ramp meters closely to how they are operated in the field. Each meter rate is responsive to the actual traffic conditions in the vicinity of the respective on-ramp. Ramp metering can be simulated for a diversity of plans. Each ramp meter has its own plan to be implemented, depending on the network characteristics.

However, traffic responsive meters cannot be represented by QRS II. Thus, QRS II requires adjusting the meter rates (“capacity” attribute) in order to approximate the meter rate during a peak period. QRS II’s results, such as speeds, are somewhat lower than ground speeds.

The simulation presented by Paramics was closer to actual traffic conditions. Paramics is a very useful traffic simulation tool that can be adapted for ramp meter representation. However, some ramp meter’s characteristics such as HOV lanes, alternate release and fractional timings cannot be represented by this software package.

After executing both simulation models, it was found that in order to obtain the most accurate results, the modeler must be completely familiar with the network system being simulated. It is often necessary to know what the software can do and what assumptions should be made in order to avoid possible problems. This is particularly true using Paramics, where many of its limitations were unknown before its initial execution. It is likely that even better results are obtainable with a more intensive calibration than the “hands-free” calibration used in this study.

Neither model was tested with algorithms to optimize ramp meters system-wide, so their capabilities for this purpose remain unclear.

Recommendations

More experimentation with Paramics is needed because it seems to be a very powerful software package that has potential in any application when its limitations are known and where its calibration is complete before being implemented. The US Highway 45 model should be used in order to improve actual traffic conditions by simulating new and different strategies. In addition, the model should be used for testing diversion, recognizing that it would need a much larger network with many more zones.

A complete evaluation of benefits requires simulations of every time period in which the meters are in operation, not just the PM peak hour as performed in this study.

A software package selected for ramp meter simulation should be able to represent every type of strategy (fixed time operation, local traffic responsive, system-wide traffic responsive). Ramp meter representation made by QRS II is limited, out-of-the-box, to a fixed time operation strategy. A traffic responsive strategy in ramp meter representation should be included in this software to increase its ability to model different strategies.

Both packages should be tested for their abilities to optimize ramp meters system-wide. Because QRS II can be embedded within another computer program as a subroutine and because of its superior computation speed, it should be able to be used to optimize the operation of a variety of traffic controls. Paramics may be able to simulate an already optimized ramp meter control strategy, but this simulation would require features of Paramics's API (application programmers interface) that go well beyond the actuated signals "plan" that was used for the US 45 tests.

References

T. Abrahamsson, “Estimation of Origin-Destination Matrices Using Traffic Counts -- A Literature Survey”, International Institute for Applied Systems Analysis, 1998.

J. A. Bieberitz, “Area Wide Impacts of Ramp Metering”, MS Thesis, University of Wisconsin Milwaukee, December 1994.

S. A. Boxill, and L. Yu, “An Evaluation of Traffic Simulation Models for Supporting ITS Development”, Center for Transportation Training and Research, Texas Southern University, October 2000.

Center for Transportation Research, “Dynasmart-P Intelligent Transportation Network Planning Tool, Version 0.926, User’s Guide”, University of Texas at Austin, 2001.

Y. Gardes, A.D. May, J. Dahlgren, A. Skabardonis, “Freeway Calibration and Application of the Paramics Model”, Transportation Research Board 81st Annual Meeting CD ROM, 2001.

A. J. Horowitz, “Reference Manual Quick Response System II”, Version 6, Center for Urban Transportation Studies, University of Wisconsin – Milwaukee, 2001.

A. J. Horowitz, “Reference Manual Advanced General Network Editor for Windows” Version 7, Center for Urban Transportation Studies, University of Wisconsin – Milwaukee, 2002.

Papageorgiou M., H. Hadj-Salem, and F. Middelham, “ALINEA Local Ramp Metering, Summary of Field Results”, *Transportation Research Record #1603*, pp. 90-98, 1997.

G. Piotrowicz and J. Robinson, “Ramp Metering Status in North America. 1995 Update”, Federal Highway Administration, DOT-T-95-17, June 1995

Quadstone, Paramics-Online Modeller V3.0, User Guide

Quadstone, Paramics-Online Modeller V3.0, Reference Manual

Quadstone, Paramics-Online Processor V3.0, Reference Manual

Quadstone, Paramics-Online Analyser V3.0, Reference Manual

“Quick Response Freight Manual” Final Report. Prepared by Cambridge Systematics, Inc., COMSIS Corporation and University of Wisconsin-Milwaukee, DOT-T-97-10, September 1996.

Evaluation of Ramp Meter Effectiveness for Wisconsin Freeways, A Milwaukee Case Study: Part 2, Ramp Metering Effect on Traffic Operations and Crashes

Project identification number 0092-45-17

Final Report

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Submitted to the Wisconsin Department of Transportation October 2004

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Appendix B Wisconsin Avenue Ramp Meter Operation, Afternoon Peak Period (4:00 pm to 5:30 pm) Wednesday, February 9, 2000.

Chapter 5
Ramp Metering Effect on Traffic Operations and Crashes

Introduction

The objective of the present chapter is to evaluate the incremental traffic operations impact of newly introduced ramp metering on six ramps in the southbound direction of USH 45. The evaluated corridor extended from the Waukesha – Washington County line on the North to just South of the Greenfield Avenue (a length of 14 miles). Ramp metering was already present on six ramps; four of these ramps were located at the south end of the corridor, which carried the heaviest traffic volumes.

The chapter addresses ramp and mainline freeway traffic operations Measures of Effectiveness (MOE) separately; overall MOE are also provided.

Analysis Corridor

The analysis corridor consisted of the southbound U.S. 45 direction, starting at the Washington/Waukesha County line on the North, crossing into Milwaukee County and extending through the interchange with I-94 (Zoo interchange) and continuing on to I-894 (the extension of U.S. 45) to a point just South of the Greenfield Avenue on-ramp (**Figure 5-1**).

Ramp metering was operational on six on-ramps along the analysis corridor when the study was initiated. Six additional on-ramps began to be metered as part of the WisDOT ramp metering program (see **Figure 5-1**). It was desired to evaluate the impact that these additional ramp meters would have on traffic operations in the analysis corridor.

Analysis Methodology

A "Without" and "With" new ramp meters comparison evaluation was chosen as an appropriate way to measure the impact of the newly installed ramp meters on freeway operations. The "Without" period represented freeway conditions when only the six existing ramp meters were operational. The "With" period represented freeway conditions when the additional six ramp meters were also operational.[1]

Detailed information on ramp delay and queue length patterns during the evaluation period is provided in **Appendix A**. Ramp metering settings and details of the ramp meter operation during the afternoon peak period of February 9 of 2000 are presented in **Appendix B** for Wisconsin Avenue, one of the most congested parts of the analyzed corridor. This information allows a detailed insight into metered ramp queue patterns and the effect of the chosen ramp metering

[1] The Main Street ramp meter was installed but not turned on during the evaluation period, thus only six new ramp meters were operating during the "With" period.

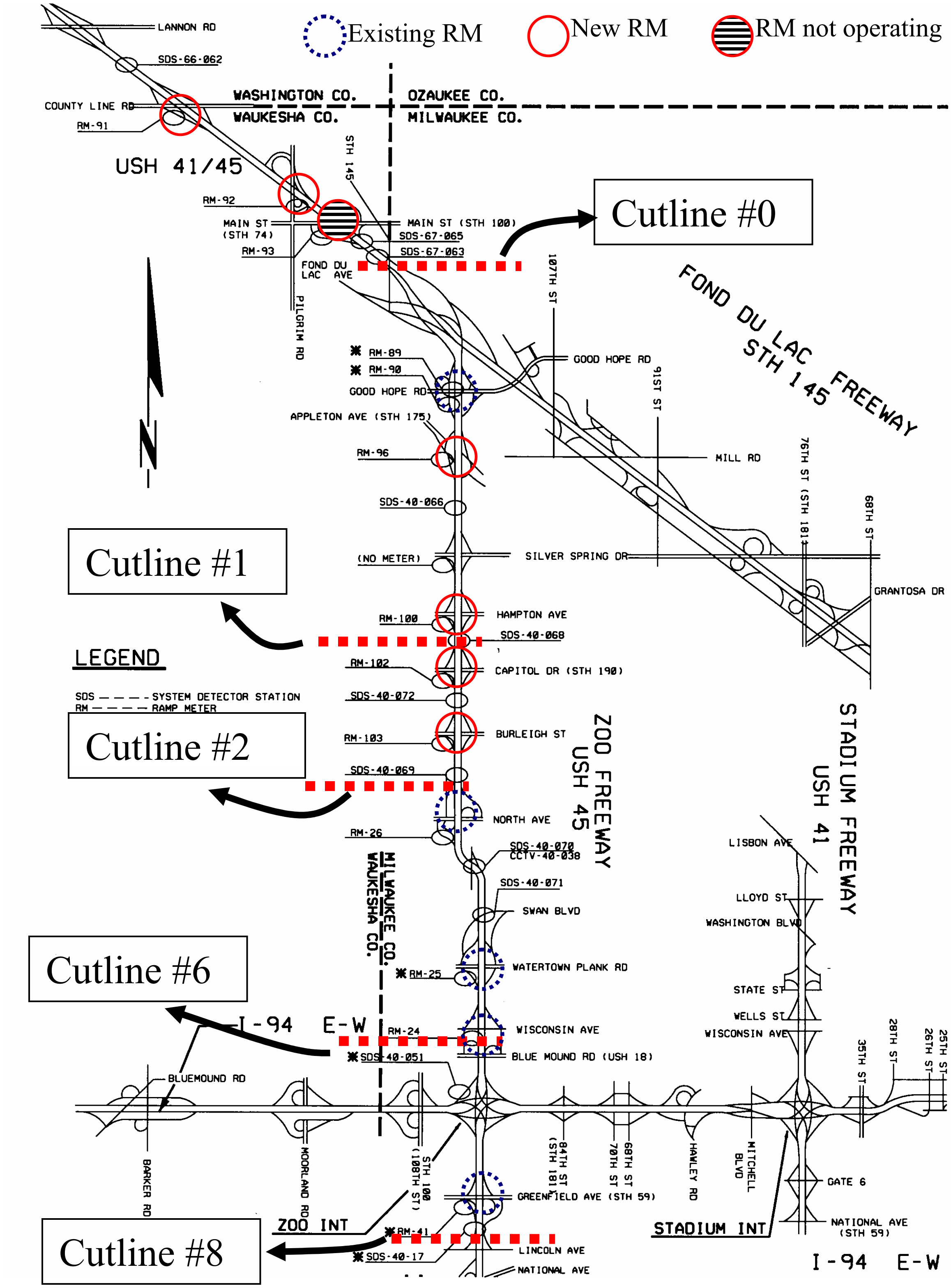

Figure 5-1. Cutlines Used for Traffic Operations MOE Evaluation & Ramp Meter Locations

parameters on metered ramp operation. Information on mainline traffic operations parameters at the same location is also presented in detail.

Database

Traffic data was gathered on Tuesdays, Wednesdays and Thursdays during consecutive weeks, in order to capture travel patterns that were most representative of weekday commuter traffic. "Without" period data were gathered on February 1, 2, and 3 (week 1), February 8, 9, and 10 (week 2) of 2000. "With" period data were gathered on March 14, 15, and 16 (week 3), March 21, 22, and 23 (week 4), starting on the 33rd day after the end of the "Without" period. The time period separating the Without and With periods was intended to allow drivers to become accustomed to the presence of the new ramp meters. Data was collected during the morning and the afternoon peak periods (7:00 am to 8:30 am and 4:00 pm to 5:30 pm, respectively).

Gathered data consisted of:

1. **Travel time runs** performed every 15 minutes during the peak periods.
2. **Traffic volume and speed**, collected through mainline and ramp pavement-embedded detectors, **every 20 seconds**.
3. **Fifteen-minute traffic volume counts** were collected through specially-installed on-ramp counters (on-ramps not equipped with pavement-embedded detectors).
4. **On-ramp queue lengths** recorded every 20 seconds (videotaped or observer-recorded in the field).

Travel Time Runs

Vehicles were dispatched every fifteen minutes during the analyzed peak periods and a crew recorded travel times between fixed landmarks along the analysis corridor. Thus, no more than six travel time runs were performed during any given one and one-half hour peak period. Travel time data were scheduled to be collected on the dates indicated above. However, no data were collected during certain dates as shown in **Table 5-1** below, due to certain circumstances (e.g., predicted adverse weather, traffic incidents, etc.)

Table 5-1. Number of Travel Time Runs Performed on US 45.

Day/Date	Without		Day/Date	With	
	AM Peak Period	PM Peak Period		AM Peak Period	PM Peak Period
Tue 2/1/00	0	6	Tue 3/14/00	0	0
Wed 2/2/00	6	0	Wed 3/15/00	6	0
Thu 2/3/00	0	0	Thu 3/16/00	6	0
Tue 2/8/00	6	6	Tue 3/21/00	4	4
Wed 2/9/00	4	6	Wed 3/22/00	4	6
Thu 2/10/00	4	0	Thu 3/23/00	6	4
Total	20	18		26	14

Volume and speed data from pavement-embedded detectors.

Volume and speed data were collected through 13 controllers (see "RM" listings in column "Controller ID," **Table 2-1**). However, due to various equipment problems, uninterrupted information for the Without and the With periods was available only for the five controllers that collected information corresponding to cutlines#1 (Congress Str.), #2 (Center Str.), #6 (S. of Wisconsin Ave.) and #8 (Lapham Str.) [2] An additional cutline (cutline #0) was established for Part 2 of the report at the Waukesha/Milwaukee County line (**Figure 5-1**). Each controller provided 20-second summary information for an on-ramp and each of the three mainline lanes.

On-Ramp Queue Length

Table 5-2 summarizes available on-ramp data availability for each peak period (morning and afternoon) and each analysis period (Without and With ramp metering). A detailed inventory of ramp queue length and delay information is presented in **Appendix A**.

Table 5-2. Ramp Delay-Available Data.

		On-Ramp Delay Data Inventory			
Location	Ramp meter	AM Peak Without	AM Peak With	PM Peak Without	PM Peak With
County Line Rd.[1]	New		√		
Pilgrim Rd.[1]	New		√		
Main Str.[2]	New				
Good Hope Rd. Loop Ramp	Existing	√	√	√	√
Good Hope Rd. Slip Ramp	Existing	√	√	√	√
Appleton Ave.	New		√		√
Hampton Ave.	New		√		√
Capitol Dr.	New		√		√
Burleigh St.	New		√		√
North Ave.	Existing	√	√	√	√
Watertown Plank Rd.	Existing	√	√	√	√
Wisconsin Ave.	Existing	√	√	√	√
Greenfield Ave.	Existing		√	√	√

Check marks indicate that data was available.

[1] The County Line Rd. and Pilgrim Rd. ramp meters did not operate during the PM peak in the With period.

[2]The Main Str. Ramp meter was installed but did not operate during the evaluation period.

[2] These cutline numbers are shown in **Figure 5-1**, and **Table 2-5** page 32 and **Figure 2-4** page 33 in Part 1 of the present report.

Freeway Operation Measures of Effectiveness

Loop detector data, collected in 20-second intervals were converted to equivalent hourly volumes and cumulative statistics were compiled for the morning and afternoon one and one-half hour peak periods. Thus, 270 values were used as inputs for volumes and an identical number for speeds for each lane at each cut line during each analyzed peak period of any given day.

Travel times compiled based on travel time runs were compared to travel times based on loop detector data in order to verify the validity of loop detector information. The two data sources were found to be in close agreement. It was decided to use loop detector data in lieu of travel time run data, because they provided travel time information compiled every 20 seconds (270 values per peak period) compared to six travel time runs—at most—during any given peak period (see **Table 5-1** for available number of travel time runs).

Data collected at the cut lines were aggregated into one and one-half hour average values for each peak period and each analysis day. The tables presented below show overall averages for all "Without" days and all "With" days at each cut line. The freeway lengths on which cut line statistics are applied is provided in **Table 5-3**. Cumulative statistics for the entire analyzed corridor are provided in each table.

Mainline Traffic Volumes: **Table 5-3** indicates small traffic volume increases along the corridor. A two-to-three percent increase was experienced at the south, most congested, end of the analyzed corridor during the morning peak; the same area experienced a zero-to-two percent increase during the afternoon peak, when the largest increase, percentage-wise (4%) was evidenced at the north end of the corridor, which had lighter traffic volumes.

Freeway Vehicle-Miles of Travel: **Table 5-3** presents the changes in Vehicle-Miles of Travel (VMT) that occurred between the Without and With periods for each of the daily peak traffic periods. There was an overall VMT increase of one percent during the morning peak; the increase was two percent for the afternoon peak.

Freeway-Vehicle Hours of Travel: Mainline freeway hours of travel decreased by 2% during the morning peak and by 5% during the afternoon peak period (see **Table 5-4**). However, total freeway vehicle hours *increased* by 4% (69.32 veh-hr) during the morning peak and decreased by 2% (36.32 veh-hr) during the afternoon peak.

Ramp Delay: This discrepancy between mainline and total vehicle hours of travel is explained by ramp delay statistics (see **Table 5-5**): ramp delay increased 64% (106.17 veh-hr) during the morning peak and 34% (54.14 veh-hr) during the afternoon peak. Minor overall delay increases were evident on existing ramps (15.14 veh-hr during the am peak, 6.24 veh-hr during the pm peak). The operation of new ramp meters introduced 91.03 veh-hr of delay during the morning peak and 47.91 veh-hr of delay during the afternoon peak. Thus the new ramp meters played a pivotal role in overall veh-hr statistics.

Ramp delay was 3.2% of total freeway veh-hr without the new ramp meters and 8.6% with the new ramp meters during the morning peak period. For the afternoon peak period, the corresponding percentages were 4.9% without and 7.6% with the new ramp meters in operation.

Freeway Speeds: Corridor speeds increased during both peaks when the new ramp meters were operational (**Table 5-6**). The increase was 1.83 mph (3%) during the morning peak, and 2.35 mph (4%) during the afternoon peak.

On-Ramp Queue Lengths: **Appendix A** presents all collected queue length and delay information. The longest queues occurred on the existing Good Hope loop ramp where maximum queue lengths averaged 60 vehicles during the morning and 50 vehicles during the afternoon peak period (pp. 8-15, **Appendix A**). Although queue lengths did not change substantially when the new ramp meters were operational, ramp delays increased.

Table 5-3. Freeway Vehicle-Miles of Travel: Without-and-With New Ramp Meters.
AM peak period (7:00 am to 8:30 am) PM peak period (4:00 pm to 5:30 pm)

	Mainline Volume Per Peak Period (vehicles)						
Cut Line	Miles	AM Peak Without	AM Peak With	% Change	PM Peak Without	PM Peak With	% Change
	3.2						
#0 Waukesha Co. Line		6476	6491	0	4044	4209	4
	4.5						
#1 Congress Str.		8485	8411	-1	7881	8009	2
	2.0						
#2 Center Str.		8418	8677	3	8006	8112	1
	2.4						
#6 Wisconsin Ave.		8380	8550	2	9829	9827	0
	1.9						
#8 Belton RR		7027	7243	3	10174	10434	2
Total Freeway VMT		109208	110254	1	107338	109144	2

Table 5-4. Freeway Vehicle-Hours of Travel: Without-and-With New Ramp Meters. AM peak period (7:00 am to 8:30 am) PM peak period (4:00 pm to 5:30 pm)

Cut Line	Mainline Vehicle-Hours of Travel Per Peak Period					
	AM Peak Without	AM Peak With	% Change	PM Peak Without	PM Peak With	% Change
#0 Waukesha Co. Line	297.64	300.96	1	188.11	196.49	4
#1 Congress Str.	618.42	570.11	-8	524.29	532.87	2
#2 Center Str.	294.96	294.64	0	331.92	298.24	-11
#6 Wisconsin Ave.	353.43	357.68	1	547.23	489.94	-12
#8 Belton RR	223.69	227.92	2	416.95	400.51	-4
Freeway VHT	1788.15	1751.30	-2	2008.51	1918.05	-5
Ramp VH Delay	58.88	165.05	64	103.63	157.77	34
Total Freeway VH	1847.03	1916.35	4	2112.14	2075.82	-2

Table 5-5. Ramp Delay: Without-and-With New Ramp Meters.
AM peak period (7:00 am to 8:30 am) PM peak period (4:00 pm to 5:30 pm)

Location	Ramp meter	Ramp Metering Delay (veh-hr)			
		AM Peak Without	AM Peak With	PM Peak Without	PM Peak With
County Line Rd.	New		15.49		
Pilgrim Rd.	New		11.96		
Main Str.	New				
Good Hope Rd. Loop Ramp	Existing	26.48	38.55	13.74	21.80
Good Hope Rd. Slip Ramp	Existing	0.28	0.80	0.43	0.30
Appleton Ave.	New		15.91		2.22
Hampton Ave.	New		11.75		7.21
Capitol Dr.	New		20.56		7.84
Burleigh St.	New		15.36		30.64
North Ave.	Existing	16.10	13.82	28.81	28.37
Watertown Plank Rd.	Existing	14.29	18.60	41.98	40.40
Wisconsin Ave.	Existing	1.72	1.53	4.78	9.34
Greenfield Ave.	Existing		0.73	13.88	9.65
New ramp meters		not installed	91.03	not installed	47.91
Existing ramp meters		58.88	74.02	103.63	109.87
Total		58.88	165.05	103.63	157.77

Notes:
Main Str. ramp metering was installed, but not turned on during the evaluation period.
Greenfield Ave. existing ramp metering was not turned on during the AM peak in the Without period.
County Line Rd. and Pilgrim Rd. ramp metering was not turned on during the afternoon peak in the With period.

Table 5-6. Freeway Speeds: Without-and-With New Ramp Meters.
AM peak period (7:00 am to 8:30 am) PM peak period (4:00 pm to 5:30 pm)

Cut Line	Freeway Speeds (MPH)					
	AM Peak Without	AM Peak With	% Change	PM Peak Without	PM Peak With	% Change
#0 Waukesha Co. Line	69.62	69.01	-1	68.81	68.55	0
#1 Congress Str.	61.94	66.39	7	67.64	67.63	0
#2 Center Str.	57.09	58.90	3	48.29	54.42	13
#6 Wisconsin Ave.	56.91	57.39	1	44.09	48.28	10
#8 Belton RR	59.69	60.38	1	46.78	49.53	6
Corridor Average Speed	61.45	63.28	3	55.96	58.31	4

Table 5-5 indicates that the highest ramp delays (42 and 40 veh-hr during the afternoon per peak period without and with the new meters, respectively) occurred on the existing Watertown Plank Road on-ramp. These delays corresponded to queues with average maximum lengths of 47 and 40 vehicles Without and With the new ramp meters operational, respectively (pp. 83-100, **Appendix A**). Maximum queue length for the HOV lane was one vehicle.

Maximum queue length on the new Burleigh Street ramp meter was about 30 vehicles during the morning peak and 45 vehicles during the afternoon peak, when ramp delay averaged 30.6 veh-hr. The High Occupancy Vehicle ramp was seldom utilized; queue length did not exceed 2-3 vehicles.

Maximum queue lengths on the existing North Avenue ramp meter averaged 32 vehicles during the afternoon peak, with ramp delays of approximately 28 veh-hr throughout the evaluation period.

High Occupancy Vehicle ramps were seldom utilized; queue lengths rarely exceeded one or two vehicles at a time.

Traffic Flow Characteristics-Discussion

Ramp meters were already installed in the southern, most congested part of the analyzed corridor, where ramp metering would be anticipated to have the greatest impact in terms of facilitating merging into the mainline and potentially diverting traffic to alternate routes during peak periods. Smoother merging into the mainline was expected to lead to increased capacity and decreased mainline travel times by minimizing the potential of shock wave formation at merge areas. The six new ramp meters were installed north of the most congested part of the corridor, thus they were expected to smooth traffic feeding into this most congested part of the corridor. The strongest smoothing effects were expected to be from the new ramp meters installed immediately upstream of the existing ramp metering installations, at Burleigh Street, Capitol Drive, and Hampton Avenue.

Because two of the remaining three new ramp meters were installed in the northern-most, less traveled part of the corridor (County Line Road and Pilgrim Road), their incremental impact on freeway operations MOE would not be expected to result in a net benefit for the north end of the corridor in terms of speeds and travel times:

- Speeds at cutline #0 were at- or near-free-flow levels before the new meters became operational and could not be expected to increase significantly. (Speeds were somewhat lower at cutline #1, allowing some room for a moderate speed increase.)
- Ramp delays (not present in this part of the corridor before the new ramp meters were operational) would thus mainly increase travel times, because drivers would not be able to make up for ramp delay by traveling much faster on the mainline.

Given the traffic flow conditions at the north end of the analysis corridor before the new ramp meters became operational, it is mainly traffic volumes that could experience an increase among the reported MOE: the highest per lane volume was 1,885 vehicles per hour (at cutline #1), allowing room for a substantial increase. These two ramps were more than four miles away from cutline #2 where the first significant speed reductions were present, thus their impact on mainline operations south of cutline #2 would be minimal.

Moderate congestion existed between cutlines #1 and #2, where the new Capitol Drive and Burleigh Street ramp meters were installed, with maximum per lane volumes of 1,870 vehicles per hour at cutline #2.

On-ramps at the south end of the corridor, represented by statistics at cutlines #6 and #8, were metered during both analysis periods (Without and With the new ramp meters). This was the most congested part of the corridor (with maximum per lane volumes of 2,260 vehicles during the afternoon peak period) operating at speeds significantly lower than the north end of the corridor. Thus there was substantially more room for speed improvement in this part of the corridor than at the north end, where speeds were near free-flow levels.

Speed increases were evident for the corridor (**Table 5-6**) with the most encouraging findings being speed increases observed at the south end of the corridor at cutlines #2, #6 and #8, during the most heavily-traveled afternoon peak period. Speeds increased by

13%, 10% and 6% at these cutlines, respectively, resulting in an overall corridor five percent reduction in mainline vehicle-hours of travel. Vehicle-hours of travel were two percent lower during the morning peak period.

An added benefit was that the above-mentioned speed increases occurred in the presence of small mainline traffic volume increases (0 - 2% during the afternoon peak and 2 - 3% during the morning peak) at the south end of the corridor. Corridor vehicle-miles of travel increased by two percent during the afternoon peak and by one percent during the morning peak.

The following discussion is based on information presented in **Table 5-7**, which is compiled from **Tables 5-4** and **5-5**.

Table 5-7. Corridor Vehicle-Hours of Travel.

	Vehicle-Hours of Travel (veh-hr)					
	AM Peak Without	AM Peak With	Change (veh-hr)	PM Peak Without	PM Peak With	Change (veh-hr)
Freeway VHT	1788.15	1751.30	-36.85	2008.51	1918.05	-90.46
New ramp meters	*not installed*	*91.03*	*91.03*	*not installed*	*47.91*	*47.91*
Existing ramp meters	*58.88*	*74.02*	*15.14*	*103.63*	*109.87*	*6.24*
Total Ramp VH Delay	58.88	165.05	106.17	103.63	157.77	54.14
Total Freeway VH	1847.03	1916.35	69.32	2112.14	2075.82	-36.32

Ramp delay was a higher percentage of total freeway vehicle hours of travel when the new meters were operational. Ramp delay at 3.2% of total freeway vehicle hours of travel in the morning peak, increased to 8.6%; for the afternoon peak the increase was from 4.9% to 7.6%.

Ramp delay increases were mostly due to the new ramp meters. New ramp meters added 91.03 vehicle-hours of delay to the morning peak (the total increase was 106.17 veh-hr of delay) and 47.91 veh-hr of delay to the afternoon peak (total increase was 54.14 veh-hr of delay).

Ramp delays were a small percentage of total veh-hr of travel, however, increases in ramp delays when the new ramp meters were operational, had a drastic impact on overall vehicle-hours of travel. Despite a decrease of 36.85 veh-hr of travel on the mainline during the ***morning peak***, an increase of 106.17 veh-hr of ramp delay resulted in an overall increase of 69.32 corridor veh-hr of travel (a 4% increase).

The impact of increased ramp meter delays was of a smaller magnitude during the ***afternoon peak***. Due to the smaller magnitude of the ramp delay, and the larger magnitude of the mainline veh-hr of travel during this peak, ramp delay had a much smaller impact on overall freeway veh-hr of travel. Despite the increased ramp delay, overall veh-hr of travel decreased by two percent when the new ramp meters were operational.

Crashes

New ramp metering equipment was installed in 1999 and was activated on February 15, 2000. Crash statistics presented herein are based on a six-month period that the corridor operated without the new ramp meters (from August 10, 1999 to February 10, 2000) and a six-month period that the corridor operated with the new ramp meters (from August 10, 2000, to February 10, 2001). The analysis included all I-94 Southbound crashes between the Waukesha County/Washington County line, and the Zoo interchange, as well as all I-894 southbound crashes between the Zoo interchange and Lincoln Avenue.

Crash statistics changes along the corridor were due to the ramp meters installed in addition to those already in operation at Good Hope Rd., North Ave., Watertown Plank Rd., Wisconsin Ave. and Greenfield Ave., as well as geometric improvements to ramps and pavement resurfacing that took place during the new ramp meter installation project.

During ramp metering hours of operation[3] a total of 152 crashes occurred along the analysis corridor in the period when the freeway operated without the new ramp meters, and 128 crashes occurred in the period that the freeway operated with the new ramp meters in place. The crash rate was 298 crashes per 100 MVM of travel "Without," and 260 crashes per 100 MVM of travel "With" the new ramp meters.

Operation of the new ramp meters in conjunction with improved ramp merging geometrics and mainline pavement resurfacing resulted in an overall 16% reduction in the number of crashes (a 13% crash rate reduction) during ramp metering hours.

Conclusions

During the period with new ramp meters in operation the most congested south part of the analysis corridor experienced an improvement in traffic operations measures of effectiveness, during the most critical (most congested) afternoon peak period.

During the afternoon peak period, a substantial reduction in vehicle-hours of travel due to increases in travel speeds, under minimal volume changes (a zero to two percent increase) was documented between Capitol Drive and Greenfield Avenue. Speeds increased by 13% in the segment between Capitol Drive and Burleigh Street, by 10% between North Avenue and Wisconsin Avenue, and by 6% between Bluemound Road and Greenfield Avenue.

However, corridor average speed increased by only four percent during the afternoon peak, because no speed changes were effected on the north part of the corridor where near-free-flow speeds existed at all times. Although mainline vehicle hours of travel

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[3] Assumed to be 6:00 am to 9:00 am (morning peak period) and 2:00 pm to 7:00 pm (afternoon peak period), Monday through Friday for the crash analysis.

decreased by five percent, when ramp delay was also taken into account, total vehicle hours of travel decreased by two percent. There was an overall increase of two percent in corridor vehicle miles of travel.

It is interesting to note that morning peak period ramp delays without the new ramp meters were approximately half the ramp delays of the afternoon peak period ramp delays. Ramp delays with the new ramp meters were approximately equal during both peak periods. Given that traffic volumes were lighter during the morning peak period, it is quite likely that ramp metering rates were more restrictive than their optimal values during this period.

The operation of new ramp meters, in conjunction with geometric improvements in ramp merging areas and mainline resurfacing resulted in a 13% crash rate reduction for the analyzed corridor during ramp metering hours.

Appendix B information indicates that ramp metering rate override due to high ramp occupancy occurs rather frequently and over a large portion of peak periods. When queue override occurs, ramp queues are very likely to be discharged at the highest metering rate when heavier mainline volumes demand more restrictive metering rates. This situation moderates potential ramp metering benefits.

Recommendations

Ramp delay played a critical role in the balance of overall corridor veh-hr of delay: although mainline veh-hr of travel decreased when the new ramp meters were operational, overall veh-hr of travel increased during the morning peak due to ramp delays. Travel time reduction benefits in the most congested part of the corridor during the afternoon peak were tempered due to additional ramp delays. Fine-tuning of ramp metering parameters during the morning peak period in order to reduce ramp delays is very likely to produce a reduction in total freeway veh-hr of travel.

Further reductions in total freeway veh-hr of travel during the afternoon peak may also be possible by reducing ramp delay on the existing Good Hope Road loop ramp where the mainline is not very congested; the current high level of ramp delay on the new Burleigh Street ramp could probably also be reduced. County Line Road and Pilgrim Road ramp metering probably contributes rather small mainline benefits at the present time, given the lower traffic volumes and substantial distance from the currently congested part of the corridor. Minimizing delays on these ramps would, in all likelihood decrease corridor delays.

Any changes in ramp metering parameters aiming to reduce ramp delays, should be carefully balanced against possible increases in mainline travel times.

Appendices A and **B** provide detailed information that can serve as the decision-making foundation for desired ramp metering parameter changes.

Appendix A

Inventory of
Ramp Delay and Queue Length Information

Introduction

The present appendix contains all collected ramp delay and queue length information. Information is presented in spatial order, from the North to the South end of the analyzed corridor. Data for each ramp are presented in a temporal sequence; High-Occupancy-Vehicle (HOV) ramp data are presented, wherever available, following Single-Occupancy-Vehicle (SOV) ramp data. Where no HOV ramp was present, the term SOV was used, although high-occupancy vehicles would also use the same ramp.

The index in pages A - ii through A - v provides Appendix page numbers where information about a specific location can be found for a specific peak period and a specific ramp. Shaded cells indicate that ramp metering was not operational during this time. Blank cells indicate that, although the ramp was operational, information for a specific period was not available. HOV cells left blank for all four weeks indicate locations that did not have an HOV ramp.

Weeks 1 and 2 (February 1-3 and 8-10) correspond to freeway operation without the new ramp meters; weeks 3 and 4 (March 14-16 and 21-23) correspond to freeway operation with the new ramp meters on-line.

Graphs contained in this Appendix provide a visual representation of queue length (used as the y-axis) and delay (the shaded area in each graph) during any instant (the x-axis represents time) of a reported peak period. Heavily shaded graphs represent peak periods with more significant ramp delay.

Certain ramps present an appearance of frequent narrow “spikes,” indicating an increased arrival rate (the left side of the spike, leading to the peak), followed by vehicles being released from the stop line, leading to shorter queues or completely dissipated queues (the right side of the spike). When ramp occupancies reached a predetermined level, “queue override” took over and set the fastest ramp metering rate, until ramp occupancy was at a predetermined low level. Such occurrences would be indicated by a faster queue length dissipation.

The operation of the Wisconsin Avenue ramp meter during the afternoon peak period of February 9, 2000, is examined in **Appendix B**, where the factors determining ramp metering rates are analyzed in detail.

RAMP DELAY DATA AVAILABILITY

February-March 2000

Table entries indicate appendix page number

Week 1

	2/1				2/2				2/3			
	AM		PM		AM		PM		AM		PM	
	SOV	HOV	SOV	HOV	SOV	HOV	SOV	HOV	SOV	HOV	SOV	HOV
County Line Rd. (North end)												
Pilgrim Rd.												
Good Hope Rd. Loop Ramp	8										9	
Good Hope Rd. Slip Ramp	16	16	17	17	18	18	19	19	20	20	21	21
Appleton Ave.												
Hampton Ave.												
Capitol Dr.												
Burleigh St.												
North Ave.	72		73		73		74		74		75	
Watertown Plank Rd.	83	83	84		84	85	85	86	86	87	87	
Wisconsin Ave.			100		101				101			
Greenfield Ave. (South end)			110				111					

NOTES:

SOV = Single-Occupancy-Vehicle lane

HOV = High-Occupancy-Vehicle lane

Shaded areas: ramp meters operational only during the after period

RAMP DELAY DATA AVAILABILITY

February-March 2000

Table entries indicate appendix page number

Week 2

	2/8				2/9				2/10			
	AM		PM		AM		PM		AM		PM	
	SOV	HOV	SOV	HOV	SOV	HOV	SOV	HOV	SOV	HOV	SOV	HOV
County Line Rd. (North end)												
Pilgrim Rd.												
Good Hope Rd. Loop Ramp			9		10		10		11		11	
Good Hope Rd. Slip Ramp	22	22	23	23	24	24	25	25	26	26	27	27
Appleton Ave.												
Hampton Ave.												
Capitol Dr.												
Burleigh St.												
North Ave.	75		76		76		77		77		78	
Watertown Plank Rd.	88	88	89		89	90	90	91	91	92	92	
Wisconsin Ave.	102				102		103		103		104	
Greenfield Ave. (South end)			111				112				112	

NOTES:

SOV = Single-Occupancy-Vehicle lane

HOV = High-Occupancy-Vehicle lane

Shaded areas: ramp meters operational only du

RAMP DELAY DATA AVAILABILITY

February-March 2000

Table entries indicate appendix page number

Week 3

	3/14				3/15				3/16			
	AM		PM		AM		PM		AM		PM	
	SOV	HOV	SOV	HOV	SOV	HOV	SOV	HOV	SOV	HOV	SOV	HOV
County Line Rd. (North end)	1	1			2	2			3	3		
Pilgrim Rd.	6								6			
Good Hope Rd. Loop Ramp	12				12		13		13		14	
Good Hope Rd. Slip Ramp	28	28	29	29			30	30	31	31		32
Appleton Ave.	35		36		36		37		37		38	
Hampton Ave.	40	41			41	42			42	43	43	44
Capitol Dr.	49	50	50	51	51	52	52	53			53	54
Burleigh St.	60	61	61	62	62	63	63	64	64	65	65	66
North Ave.	78		79				79		80		80	
Watertown Plank Rd.	93	93	94		94	95	95	96	96	97	97	
Wisconsin Ave.	104		105		105		106		106		107	
Greenfield Ave. (South end)			113		113	114	114					

NOTES:

SOV = Single-Occupancy-Vehicle lane

HOV = High-Occupancy-Vehicle lane

Shaded areas: ramp meters operational only du

RAMP DELAY DATA AVAILABILITY

February-March 2000

Table entries indicate appendix page number

Week 4

	3/21				3/22				3/23			
	AM		PM		AM		PM		AM		PM	
	SOV	HOV	SOV	HOV	SOV	HOV	SOV	HOV	SOV	HOV	SOV	HOV
County Line Rd. (North end)	4	4							5	5		
Pilgrim Rd.	7				7				8			
Good Hope Rd. Loop Ramp			14		15						15	
Good Hope Rd. Slip Ramp			32	33	33	34					34	35
Appleton Ave.	38		39		39				40			
Hampton Ave.	44	45	45	46			46	47	47	48	48	49
Capitol Dr.	54	55	55	56	56	57	57	58	58	59	59	60
Burleigh St.	66	67	67	68	68	69	69	70	70	71	71	72
North Ave.			81		81		82				82	
Watertown Plank Rd.	98		98		99		99				100	
Wisconsin Ave.	107		108		108		109		109		110	
Greenfield Ave. (South end)											115	

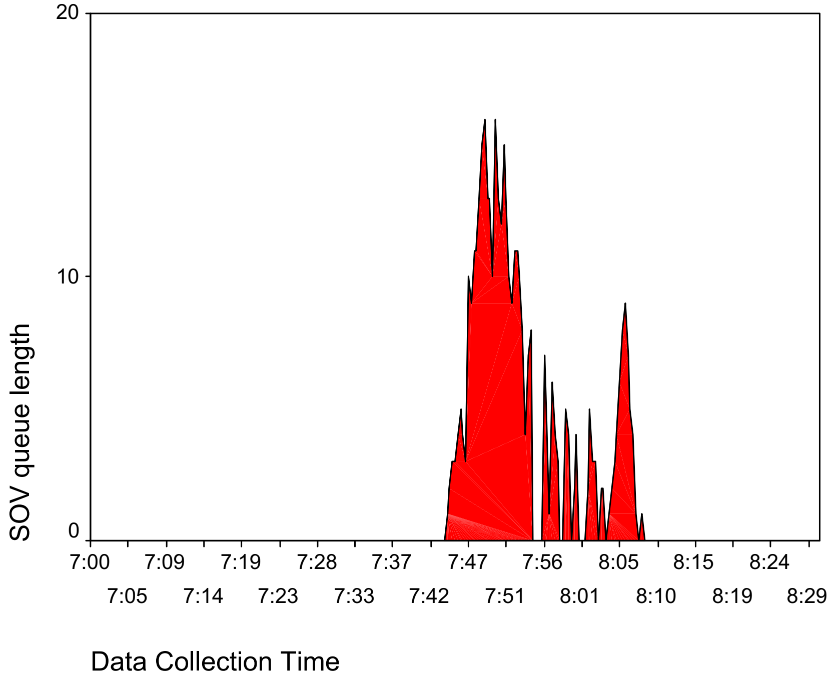
NOTES:

SOV = Single-Occupancy-Vehicle lane

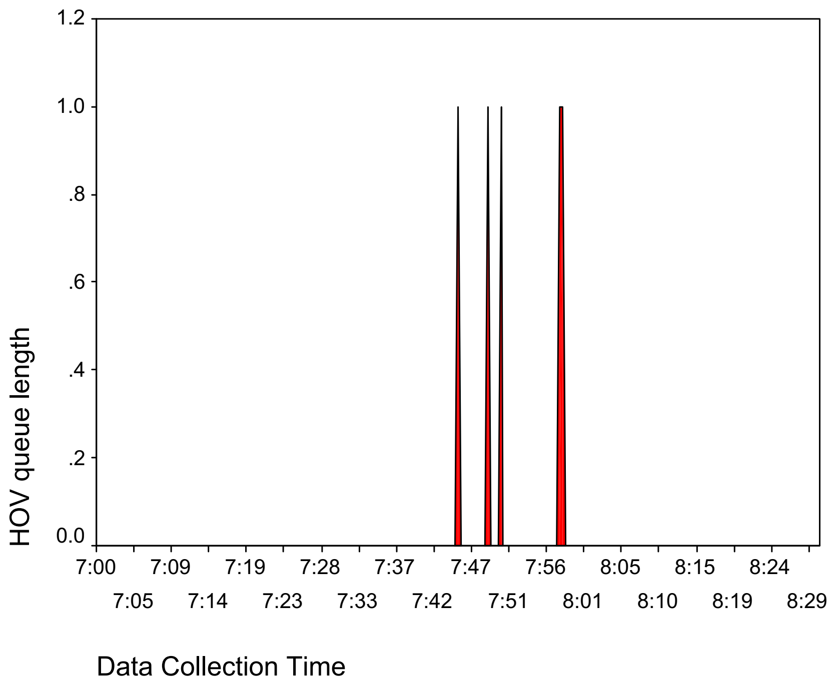
HOV = High-Occupancy-Vehicle lane

Shaded areas: ramp meters operational only du

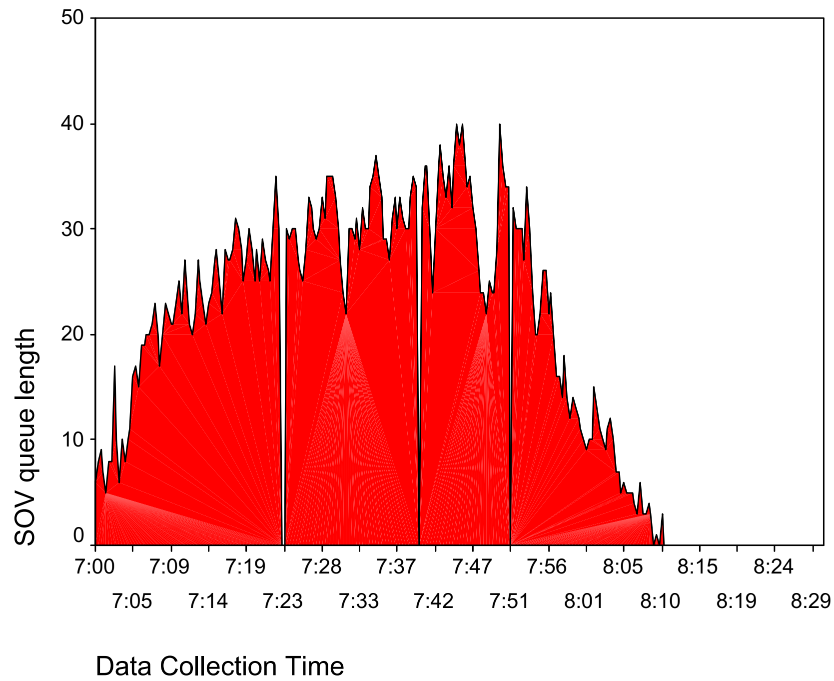
County Line Road 3/14/2000 AM peak



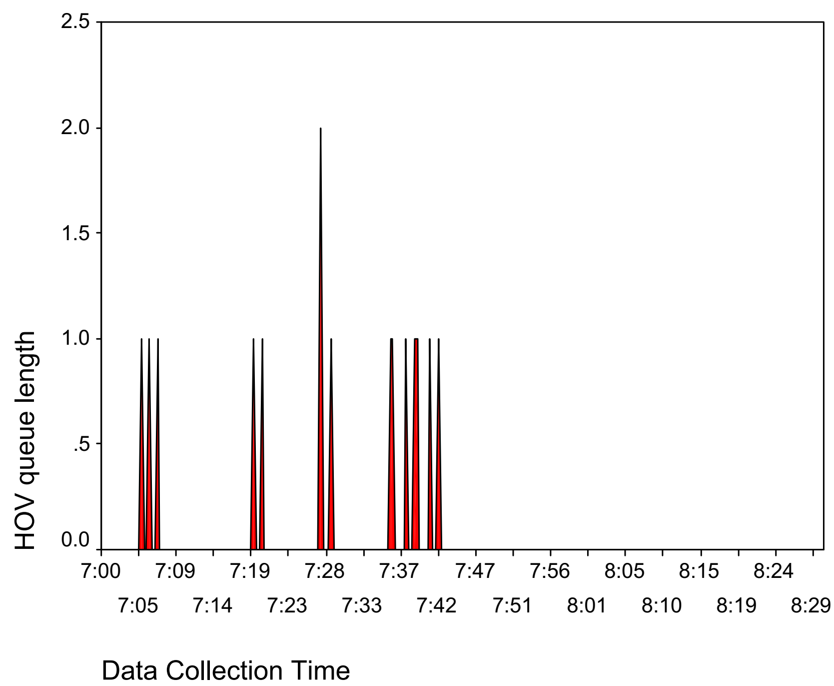
County Line Road 3/14/2000 AM peak



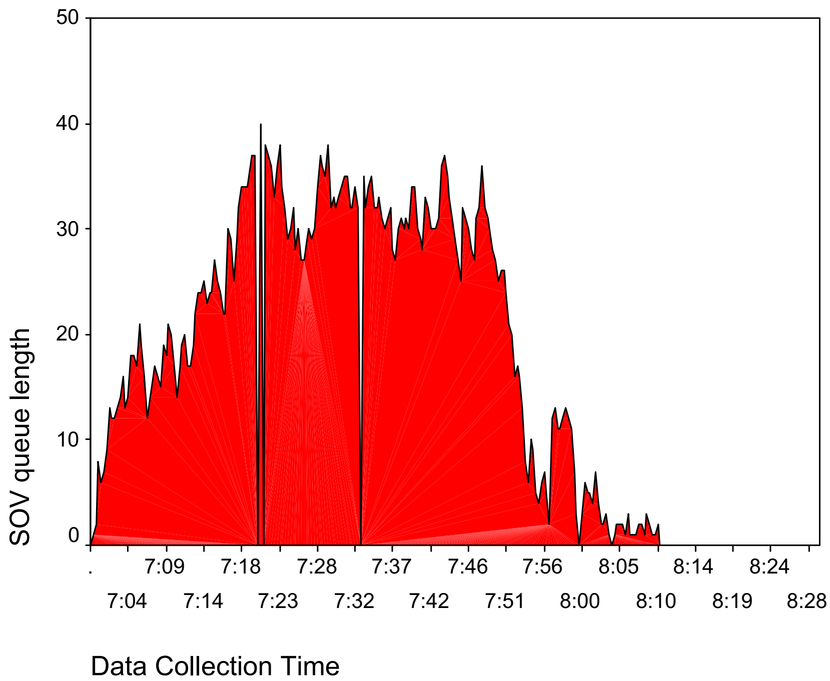
County Line Road 3/15/2000 AM peak



County Line Road 3/15/2000 AM peak



County Line Road 3/16/2000 AM peak



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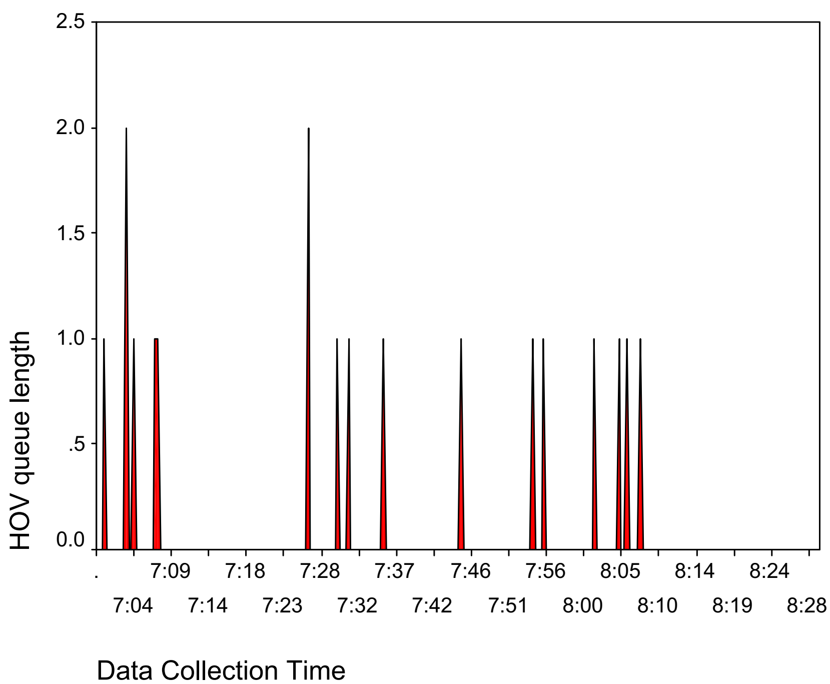
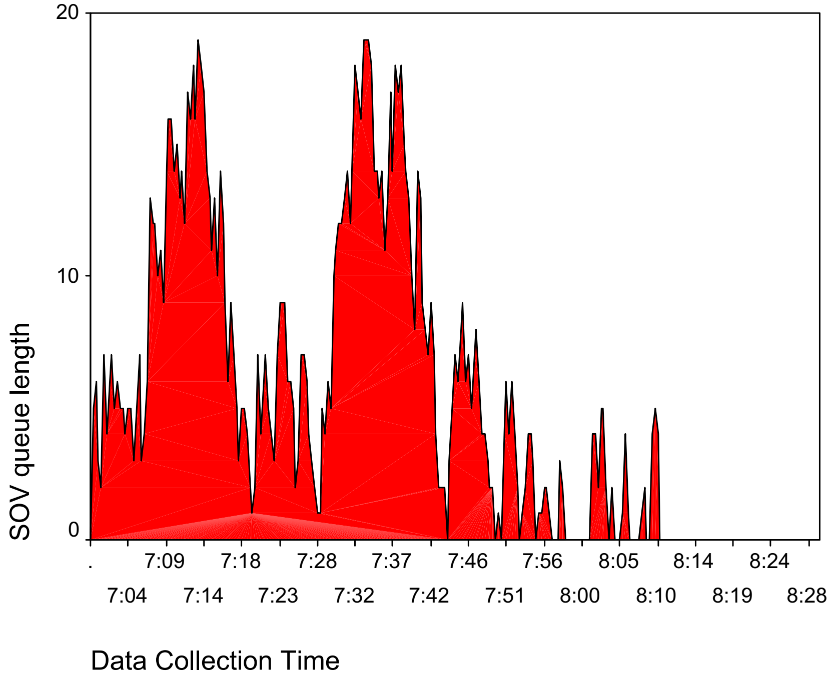


County Line Road 3/21/2000 AM peak



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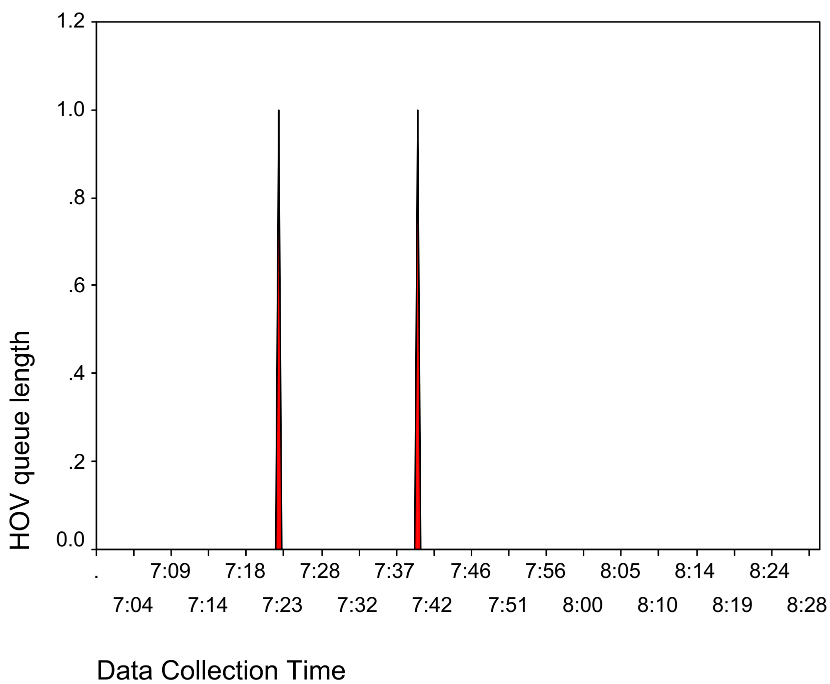

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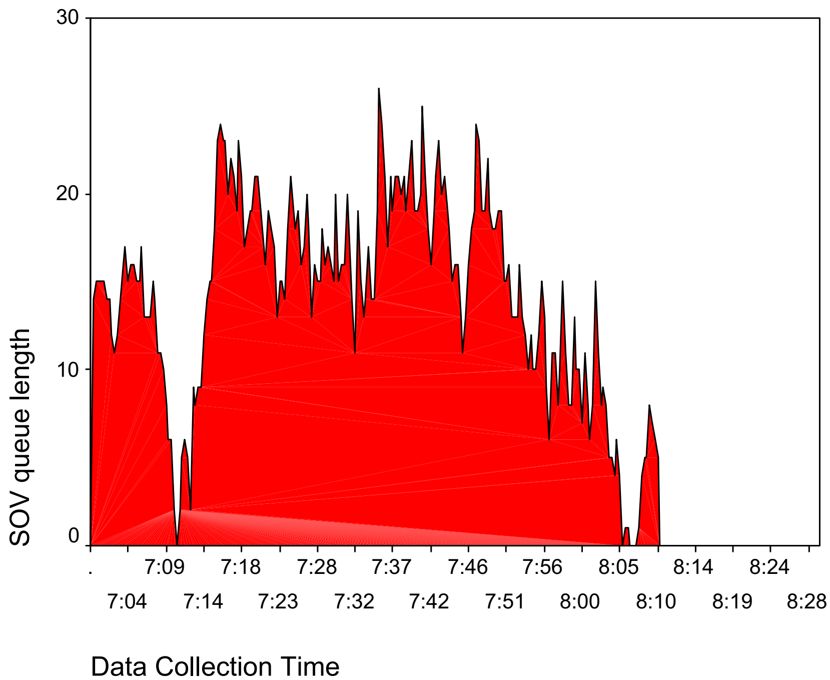

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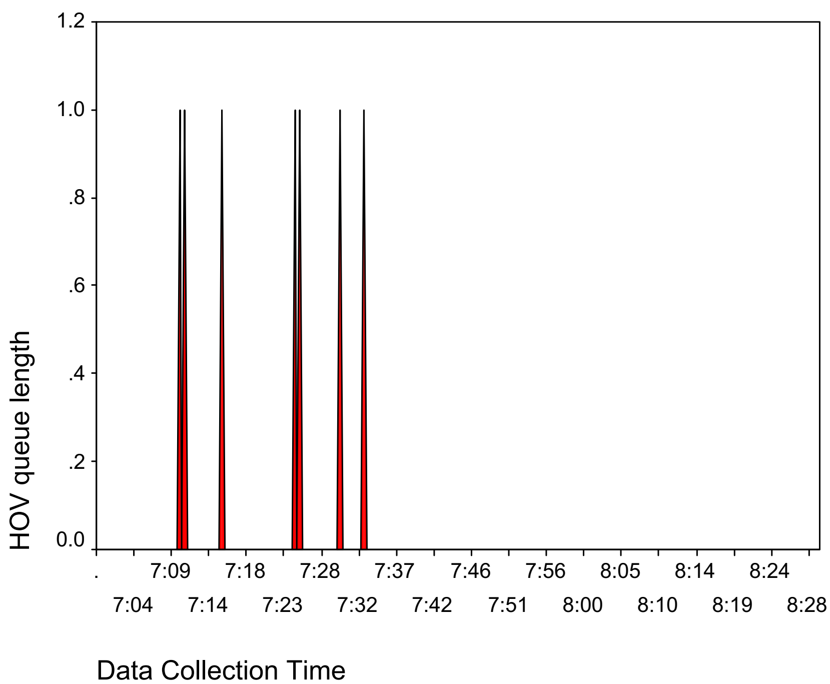

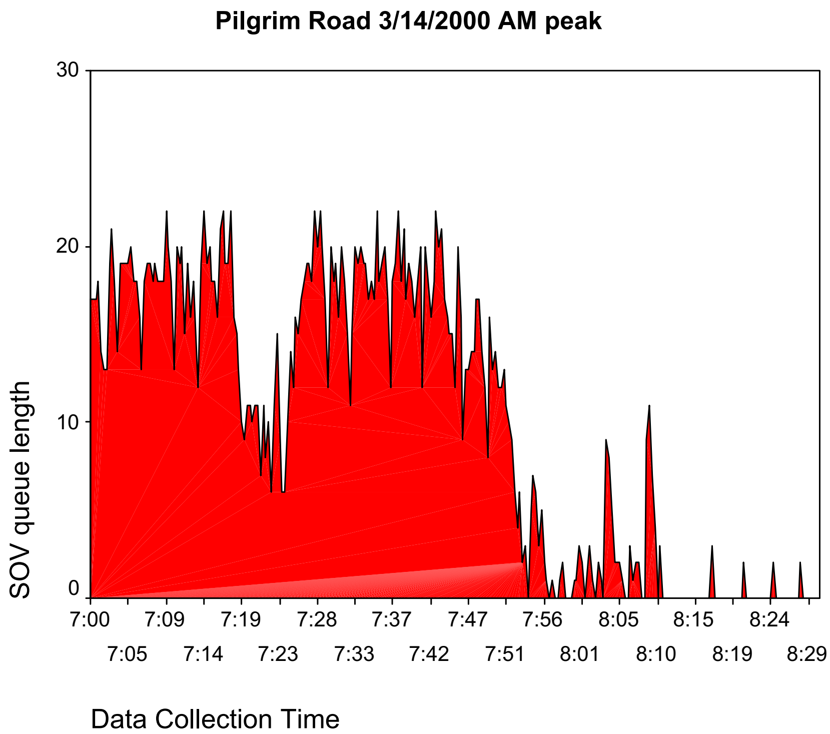
Pilgrim Road 3/14/2000 AM peak
SOV queue length
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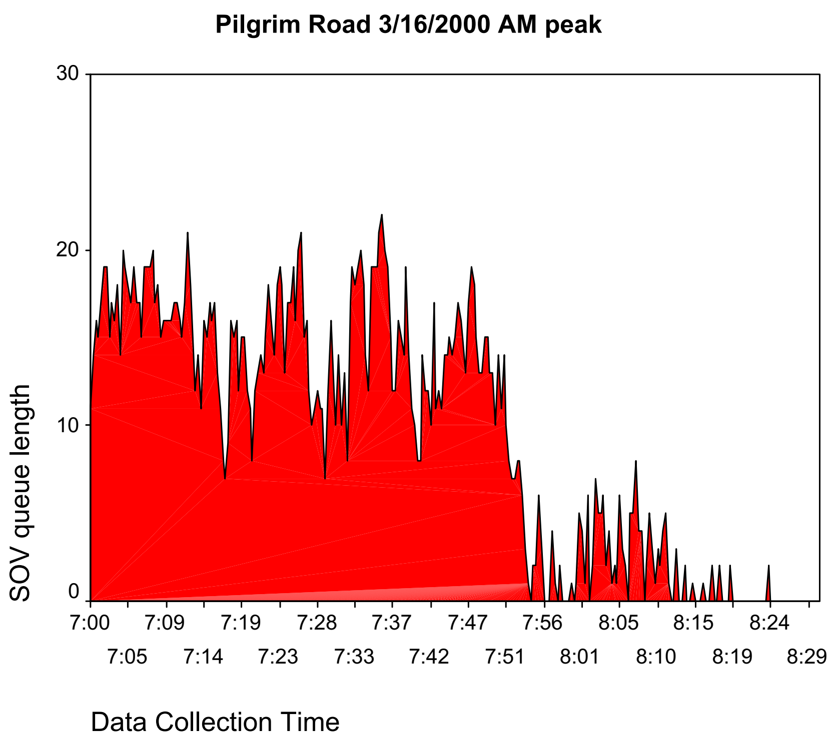
20
10
0
7:00
7:05
7:09
7:14
7:19
7:23
7:28
7:33
7:37
7:42
7:47
7:51
7:56
8:01
8:05
8:10
8:15
8:19
8:24
8:29
Data Collection Time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10
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7:00
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7:37
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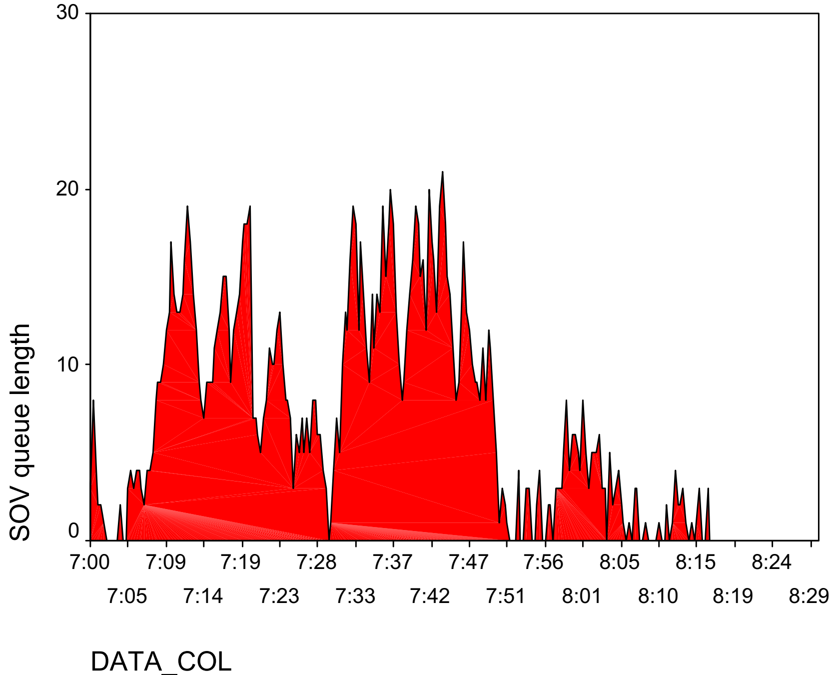
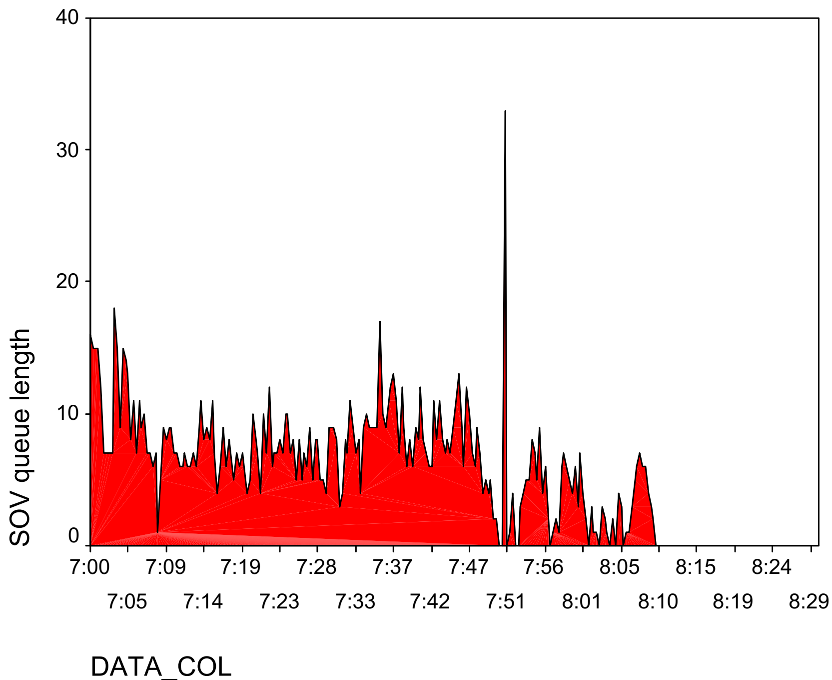
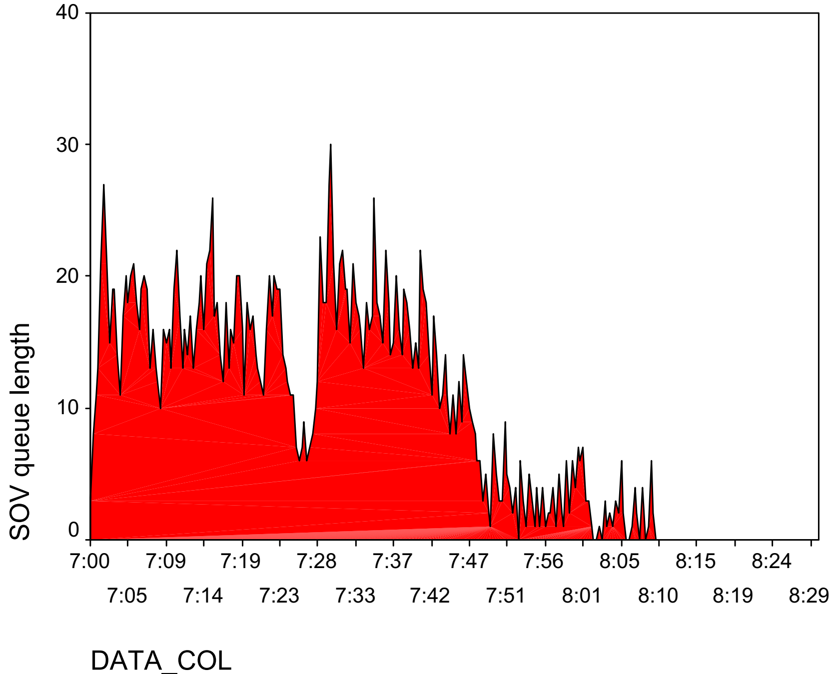
Pilgrim Road 3/21/2000 AM peak



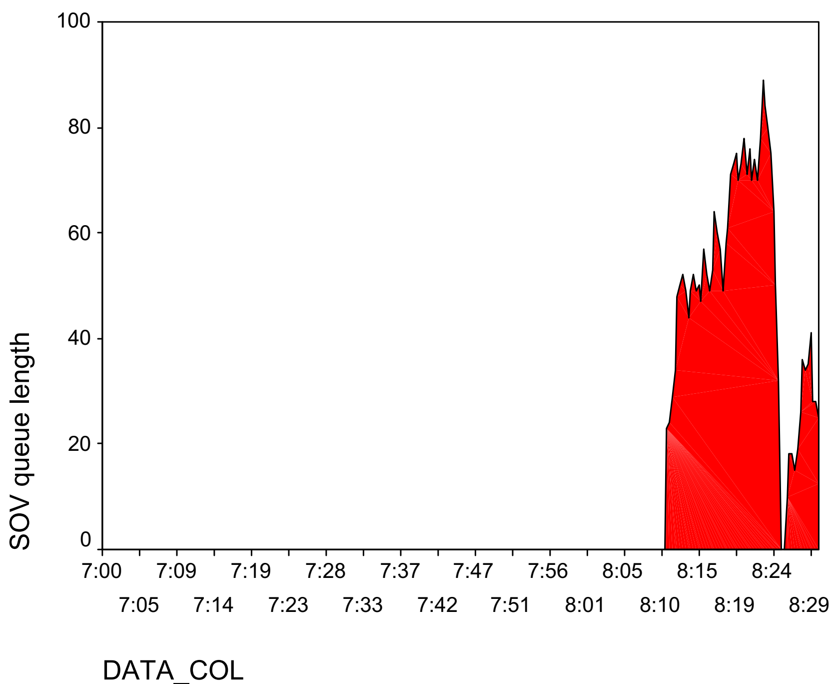
Pilgrim Road 3/22/2000 AM peak



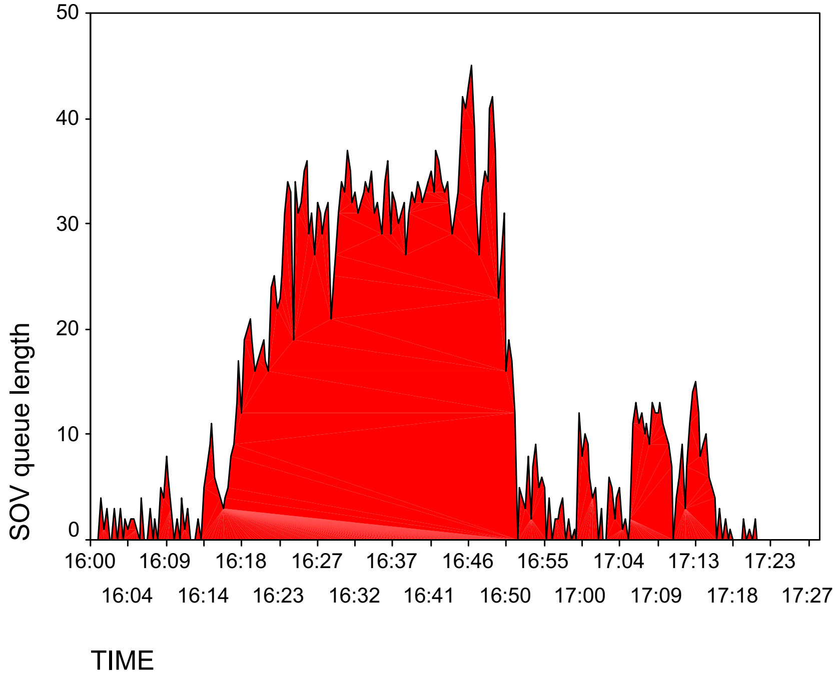
Pilgrim Road 3/23/2000 AM peak



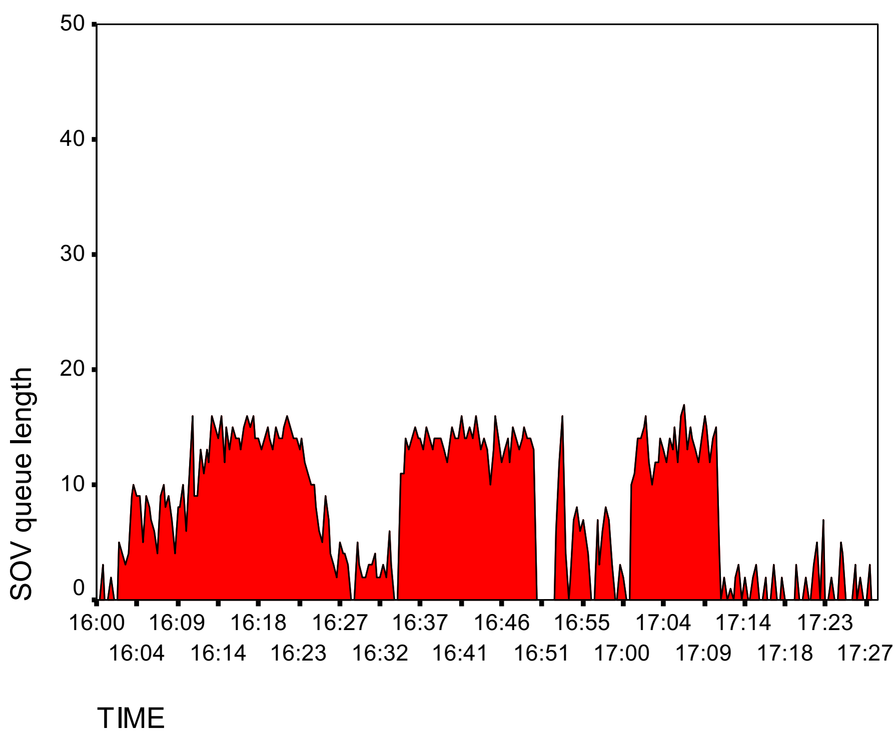
Good Hope Road Loop Ramp 2/1/2000 AM peak



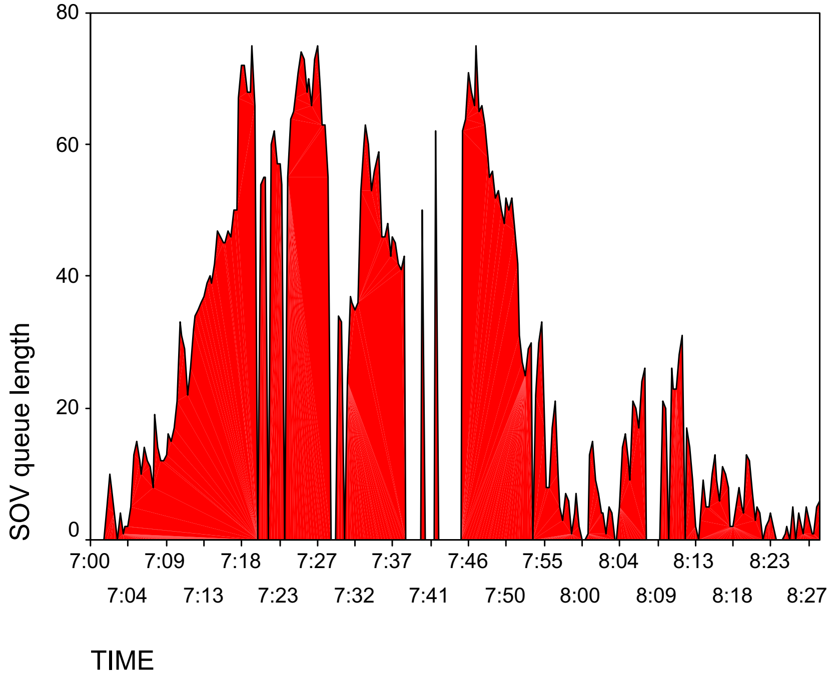
Good Hope Loop Ramp 2/3/2000 PM peak



Good Hope Loop Ramp 2/8/2000 PM peak



Good Hope Loop Ramp 2/9/2000 AM peak



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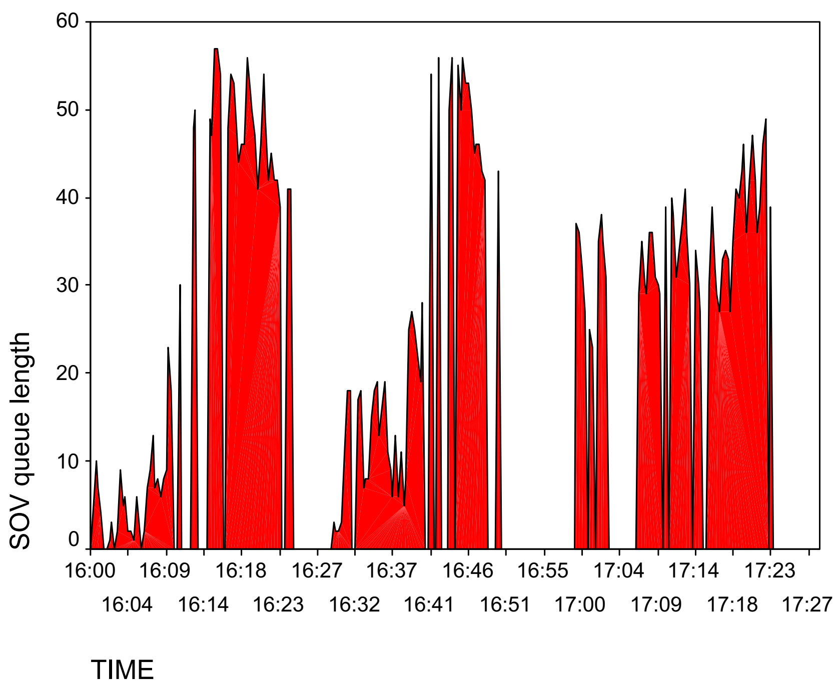

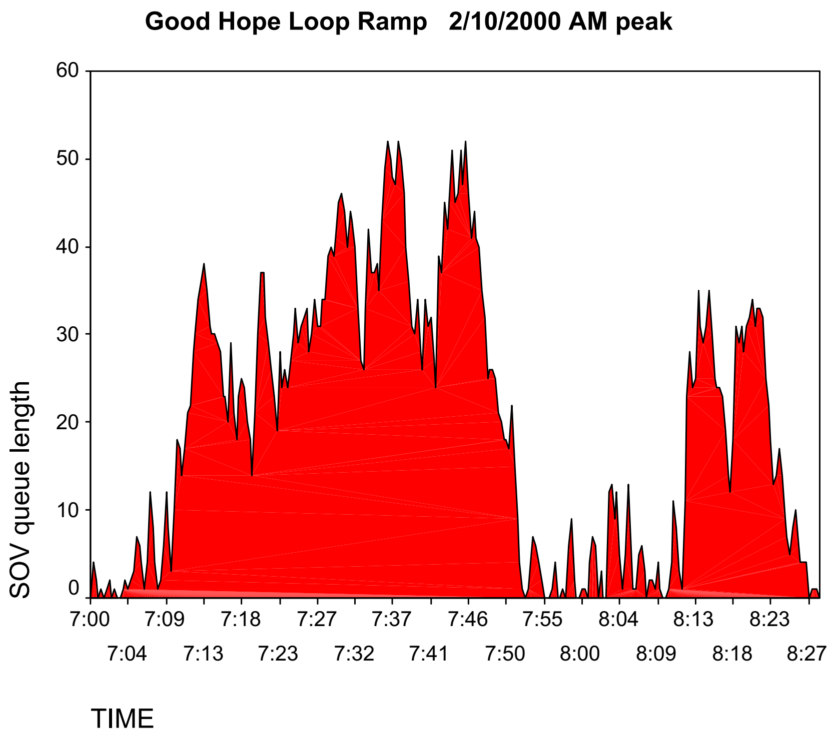
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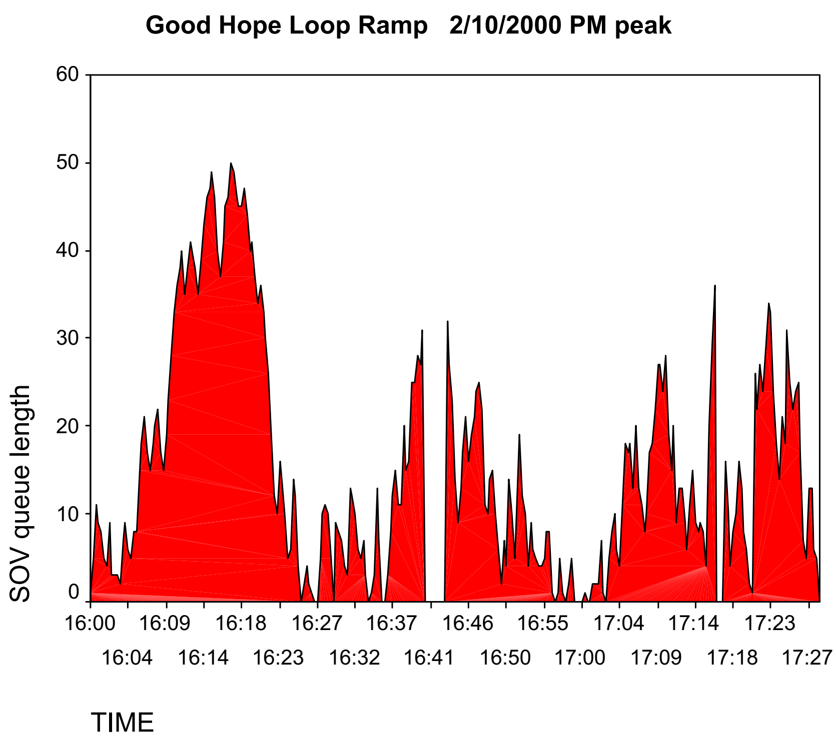
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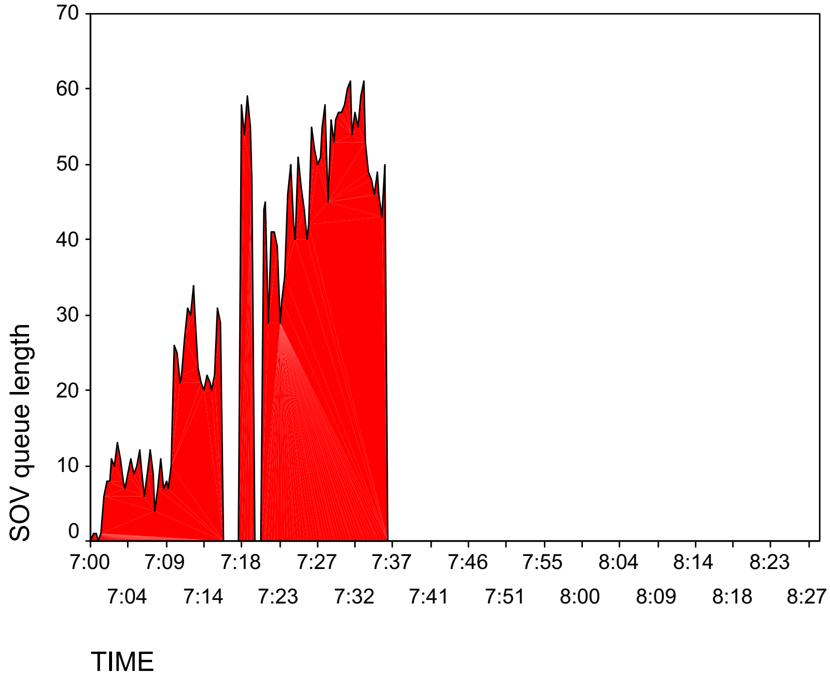
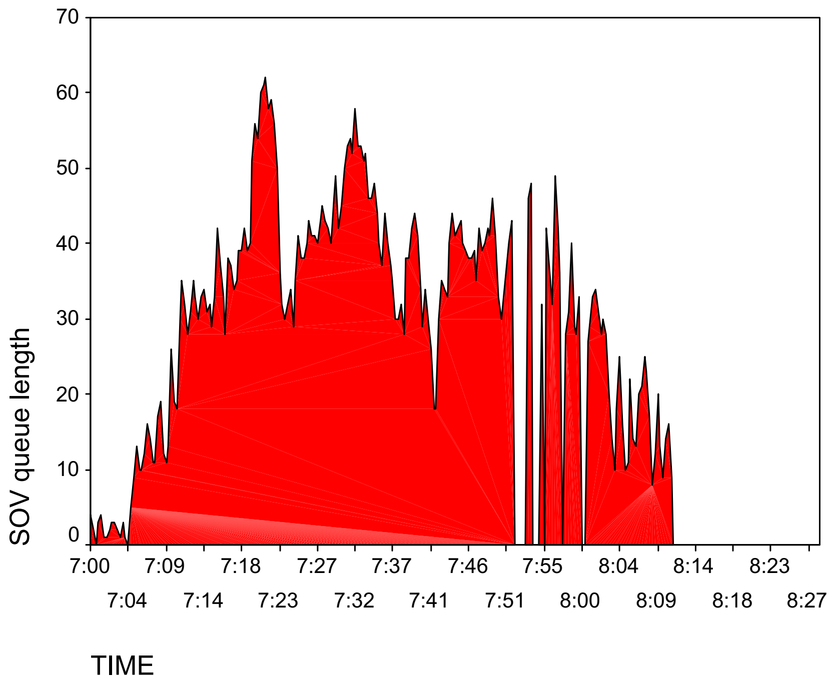
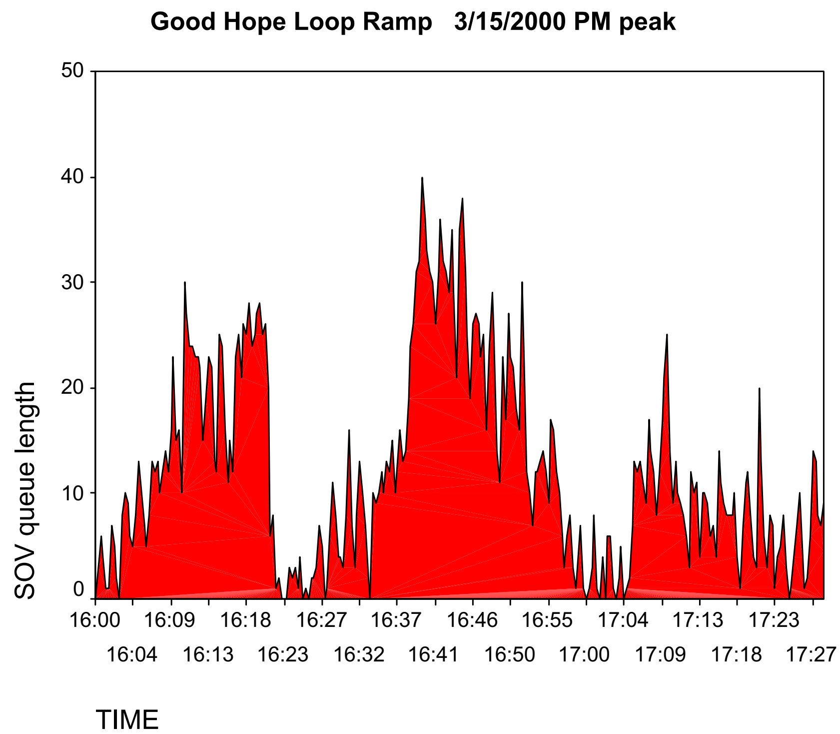
TIME

Good Hope Loop Ramp   3/14/2000 AM peak



Good Hope Loop Ramp   3/15/2000 AM peak

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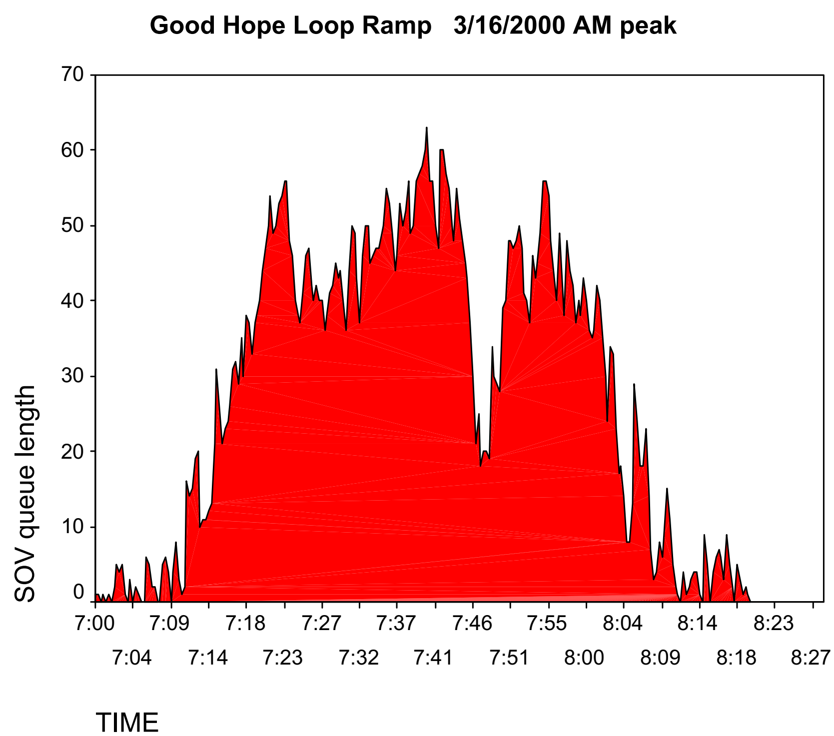
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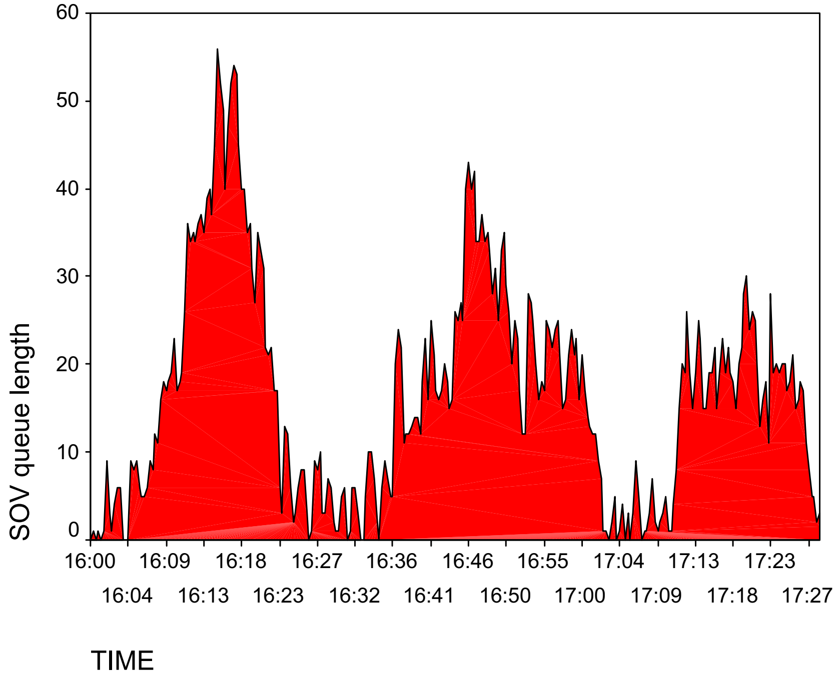
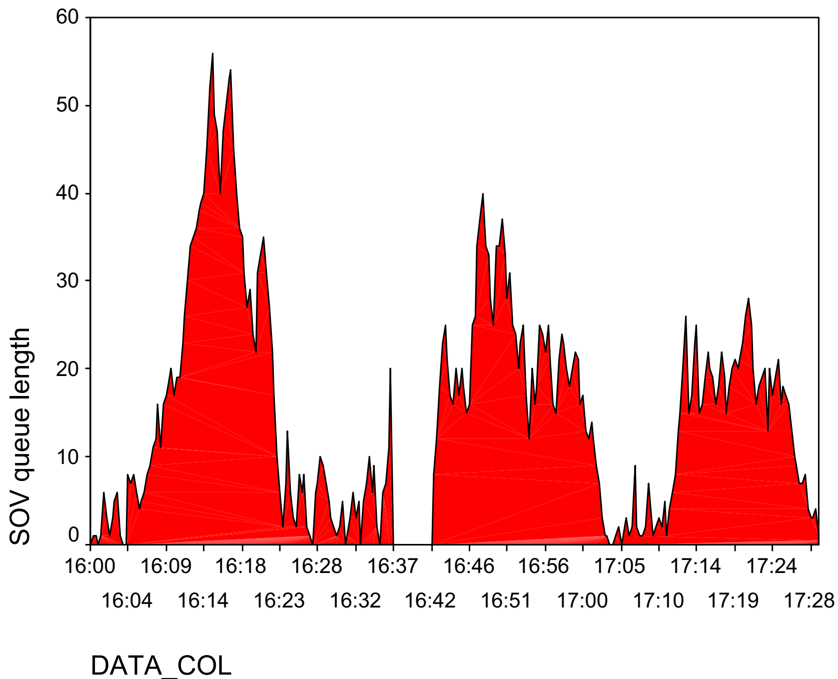
TIME

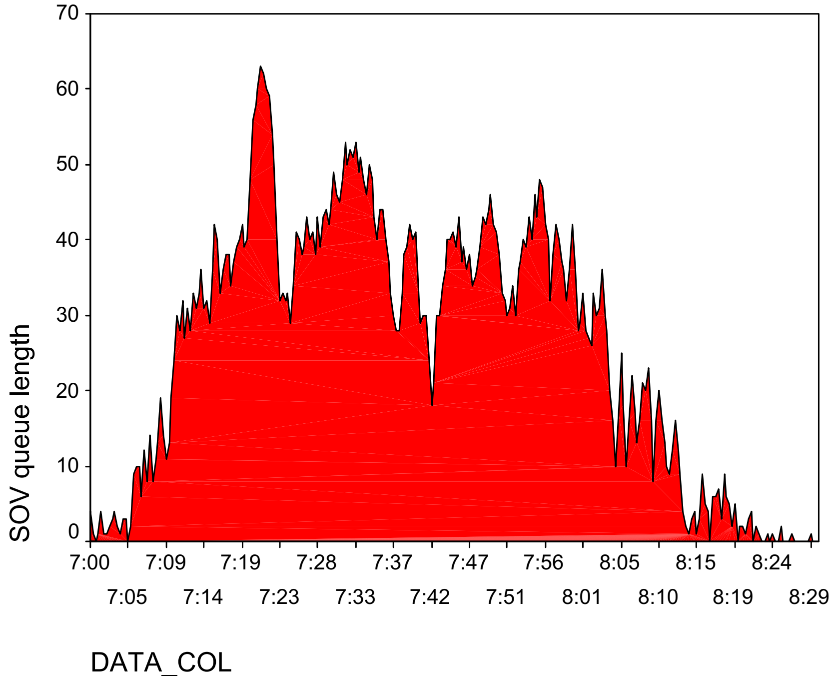
Good Hope Loop Ramp   3/16/2000 PM peak



Good Hope Road Loop Ramp 3/21/2000 PM peak



Good Hope Road Loop Ramp 3/22/2000 AM peak



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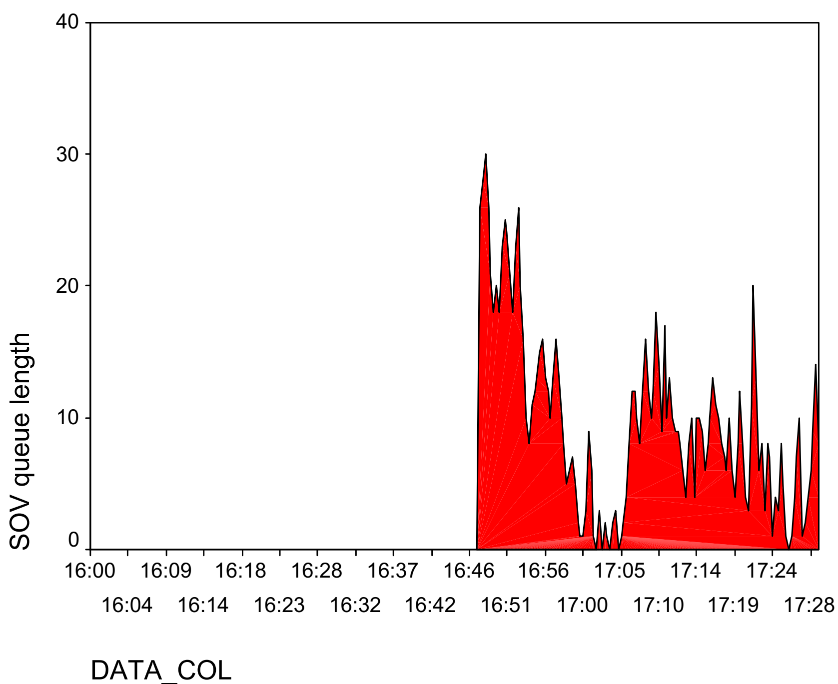
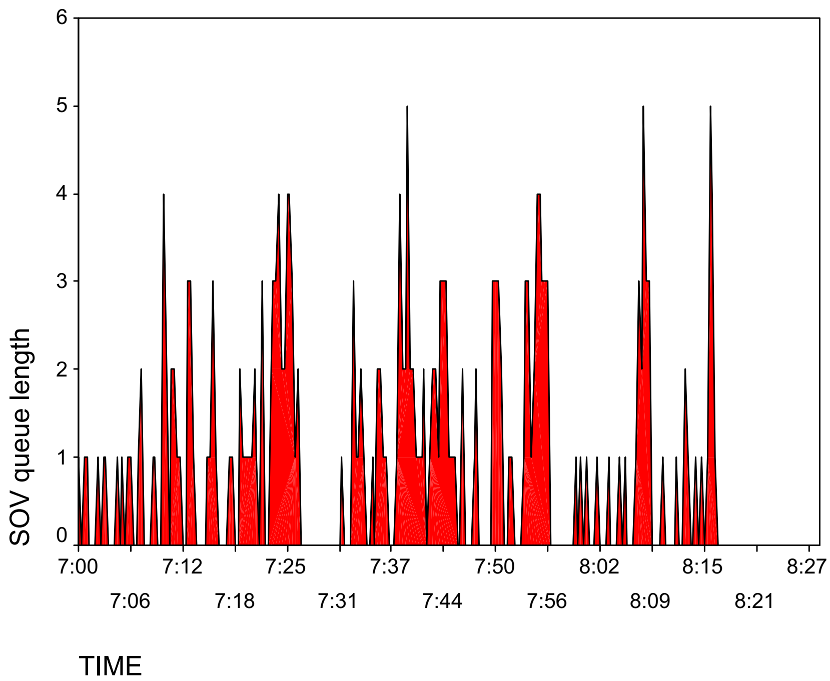


Good Hope Road Slip Ramp 2/1/2000 AM peak



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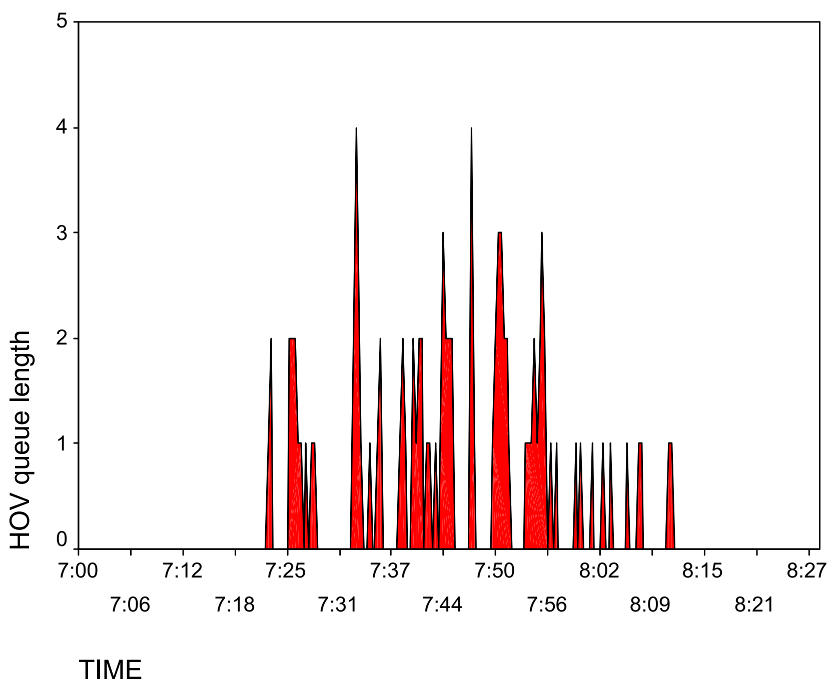
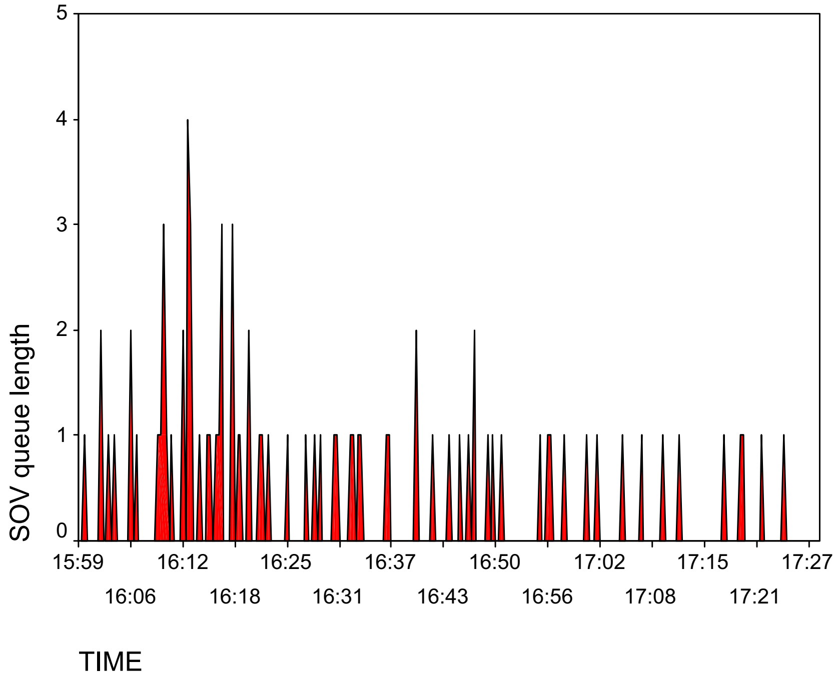
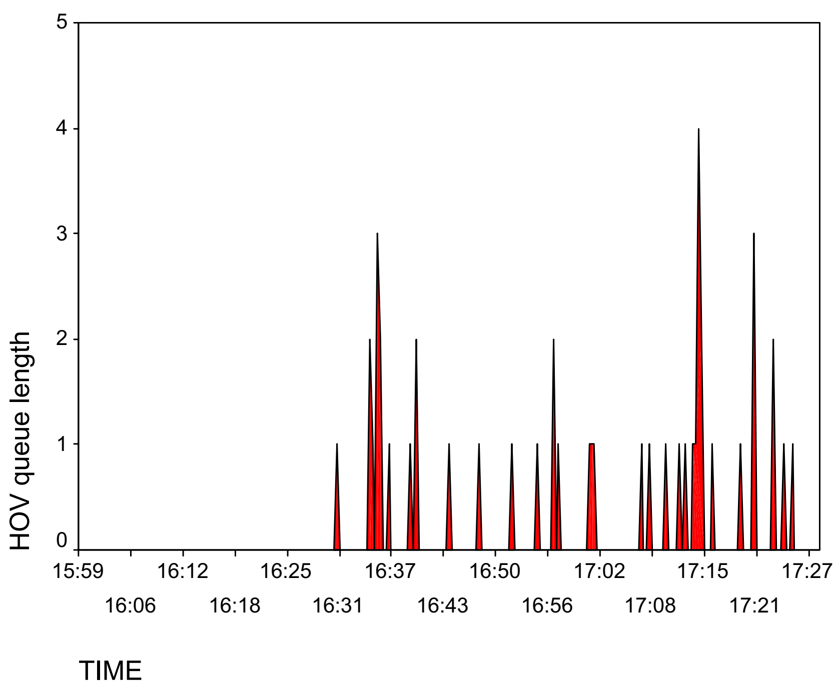


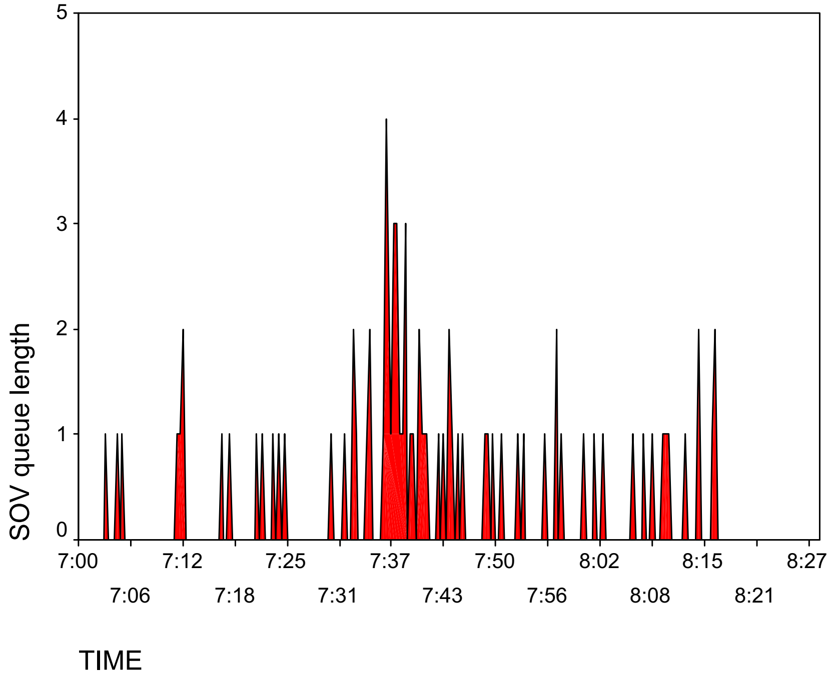
Good Hope Road Slip Ramp 2/1/2000 PM peak



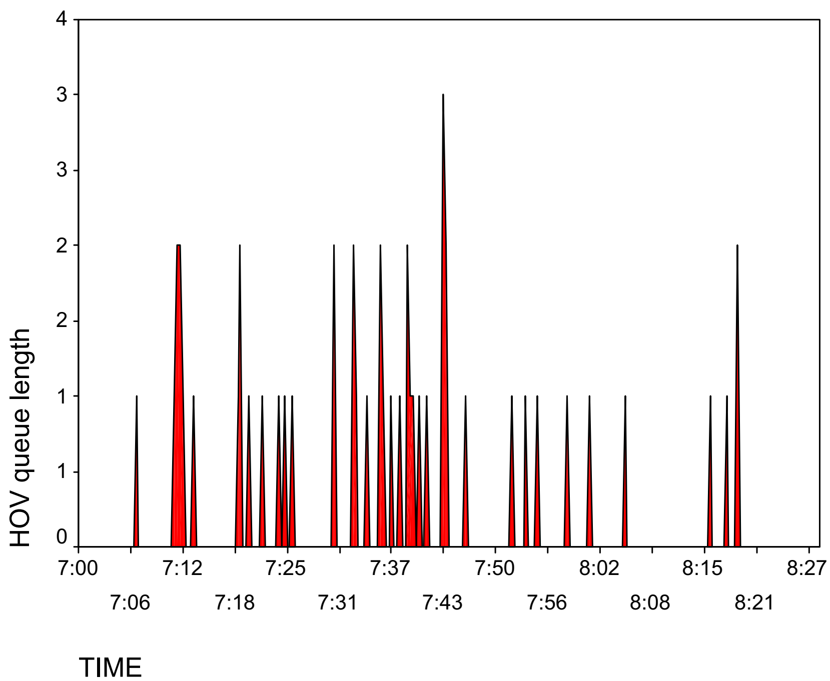
Good Hope Road Slip Ramp 2/1/2000 PM peak



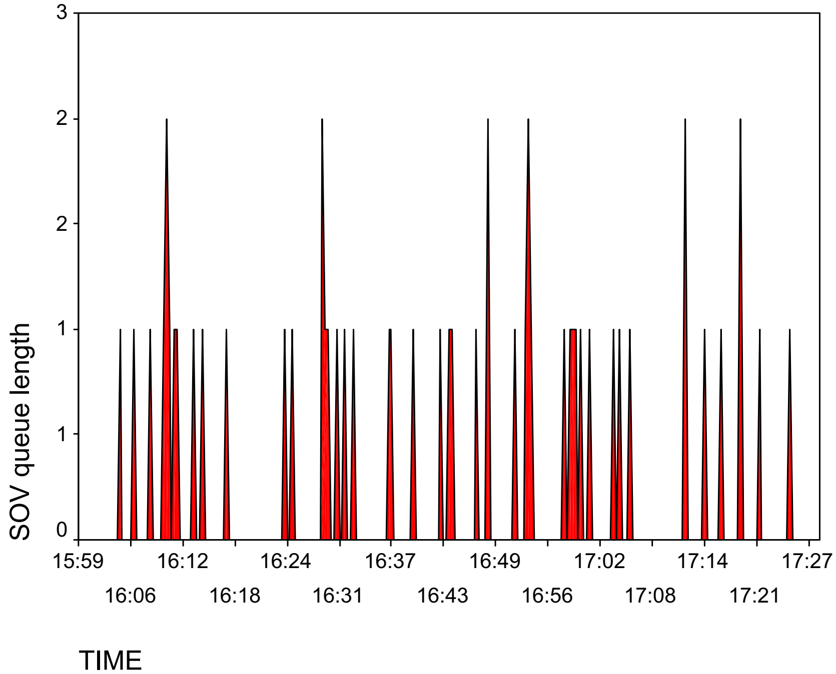
Good Hope Road Slip Ramp 2/2/2000 AM peak



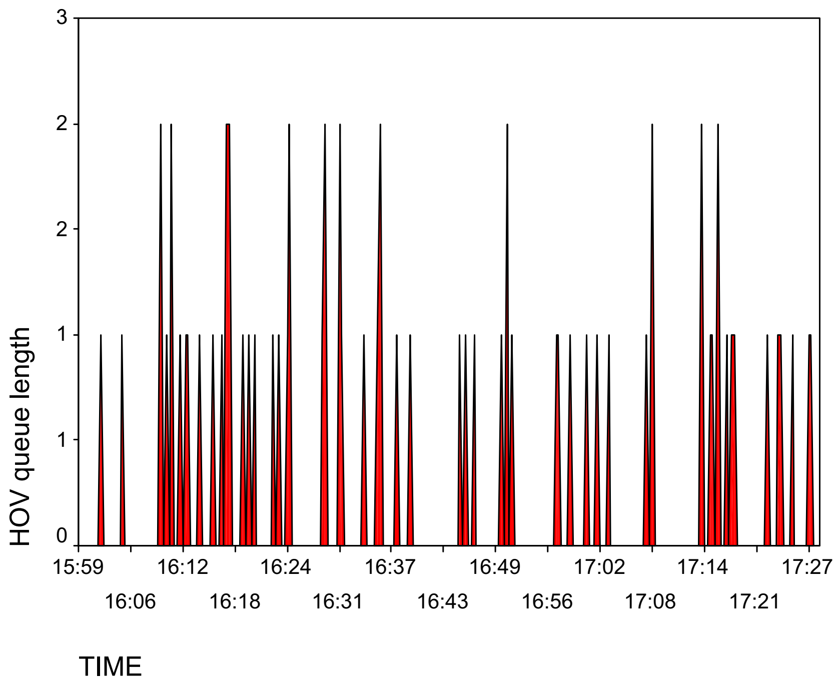
Good Hope Road Slip Ramp 2/2/2000 AM peak

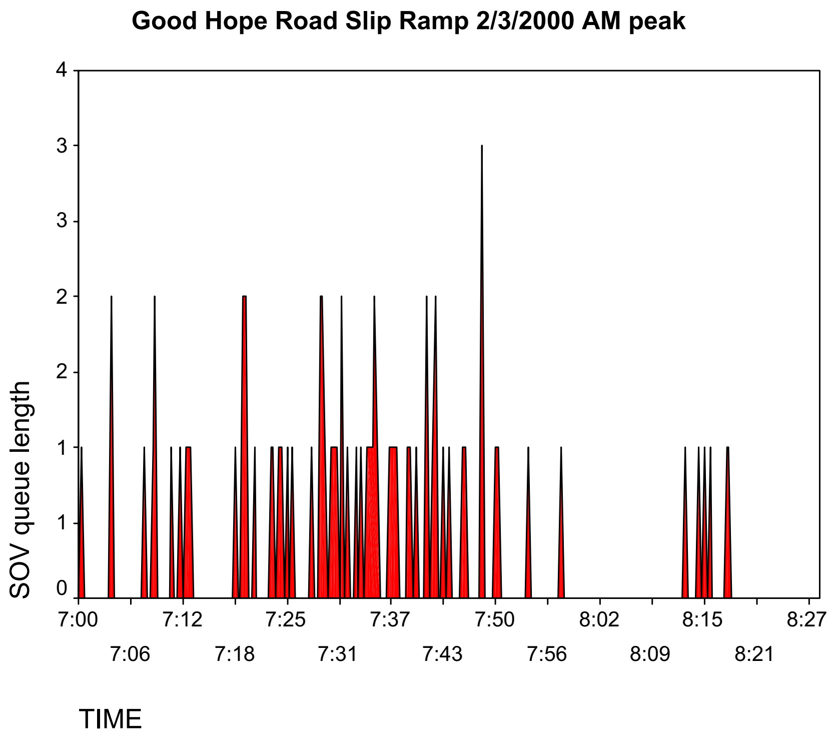


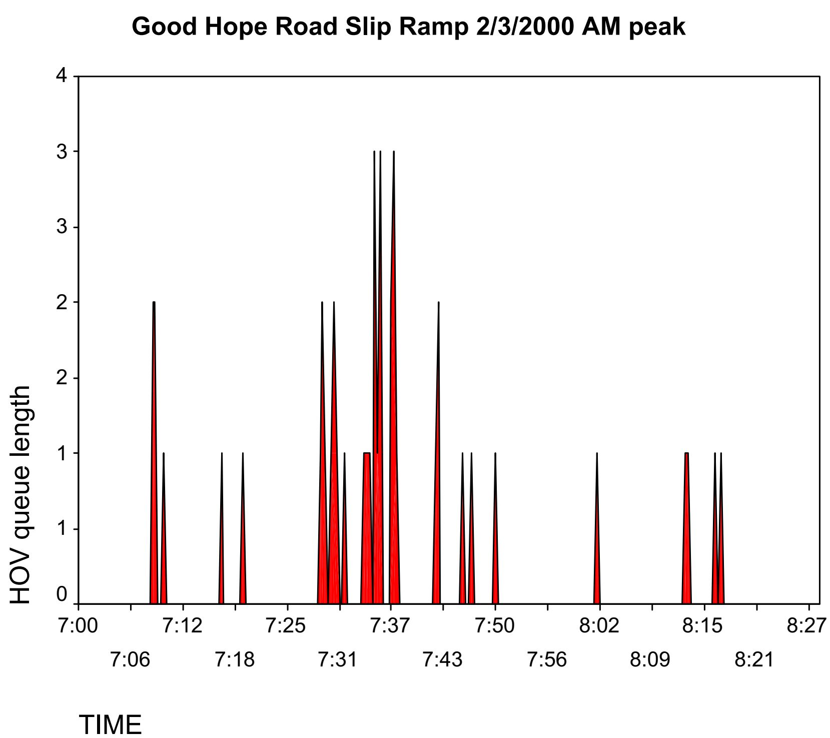
Good Hope Road Slip Ramp 2/2/2000 PM peak



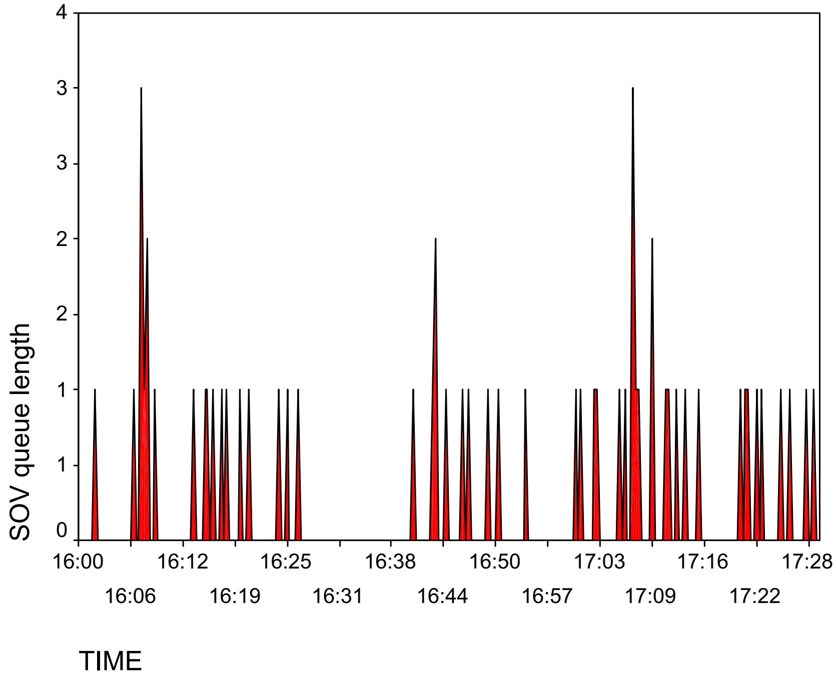
Good Hope Road Slip Ramp 2/2/2000 PM peak



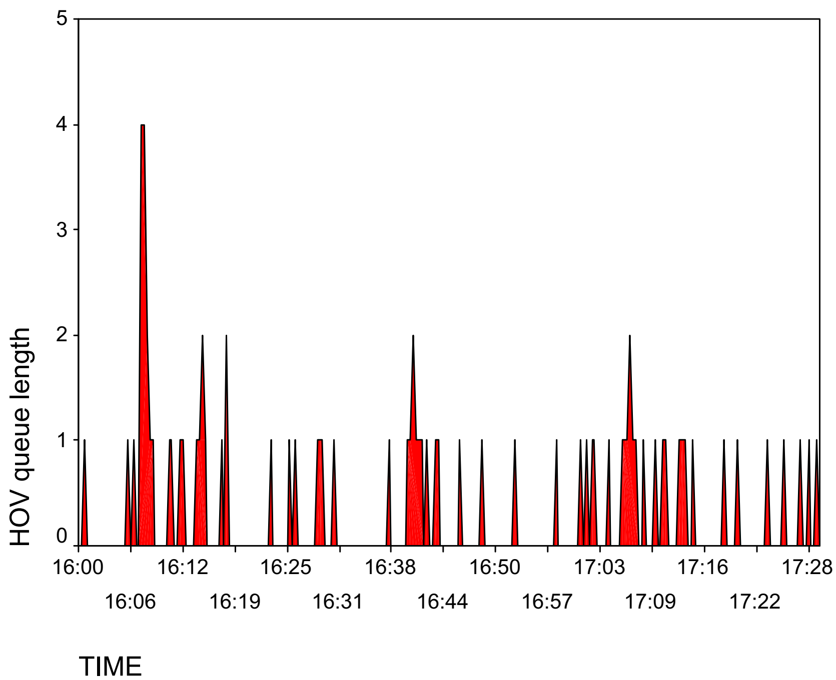

Good Hope Road Slip Ramp 2/3/2000 AM peak
SOV queue length
TIME
0
1
1
2
2
3
3
4
7:00
7:06
7:12
7:18
7:25
7:31
7:37
7:43
7:50
7:56
8:02
8:09
8:15
8:21
8:27

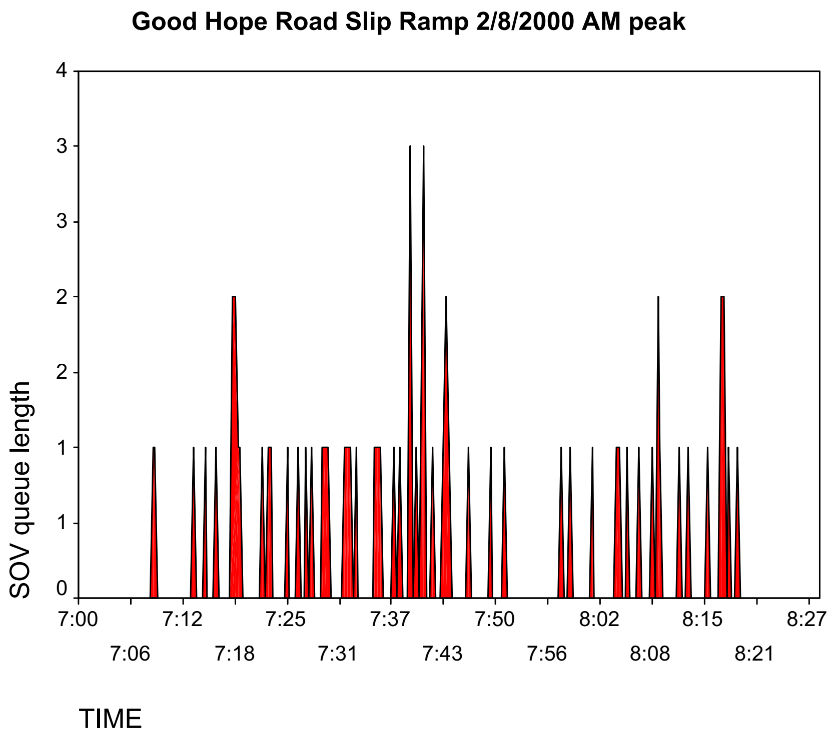

Good Hope Road Slip Ramp 2/3/2000 AM peak
HOV queue length
TIME
0
1
1
2
2
3
3
4
7:00
7:06
7:12
7:18
7:25
7:31
7:37
7:43
7:50
7:56
8:02
8:09
8:15
8:21
8:27

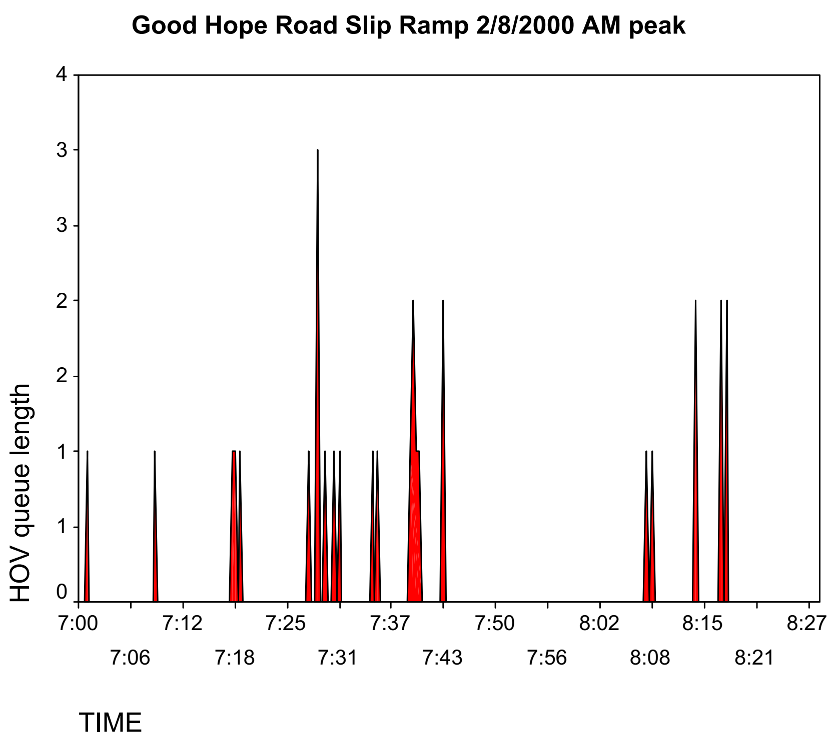
Good Hope Road Slip Ramp 2/3/2000 PM peak



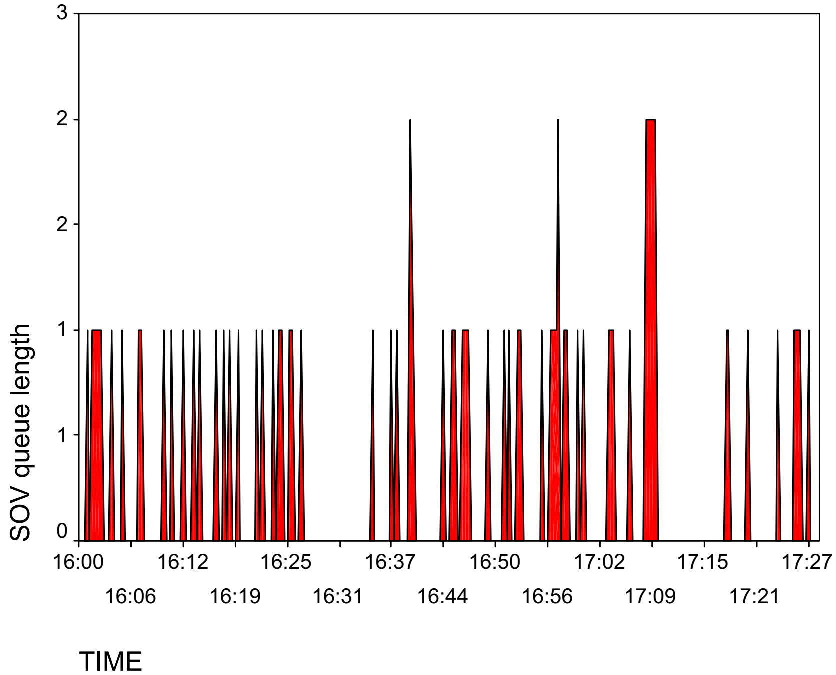
Good Hope Road Slip Ramp 2/3/2000 PM peak



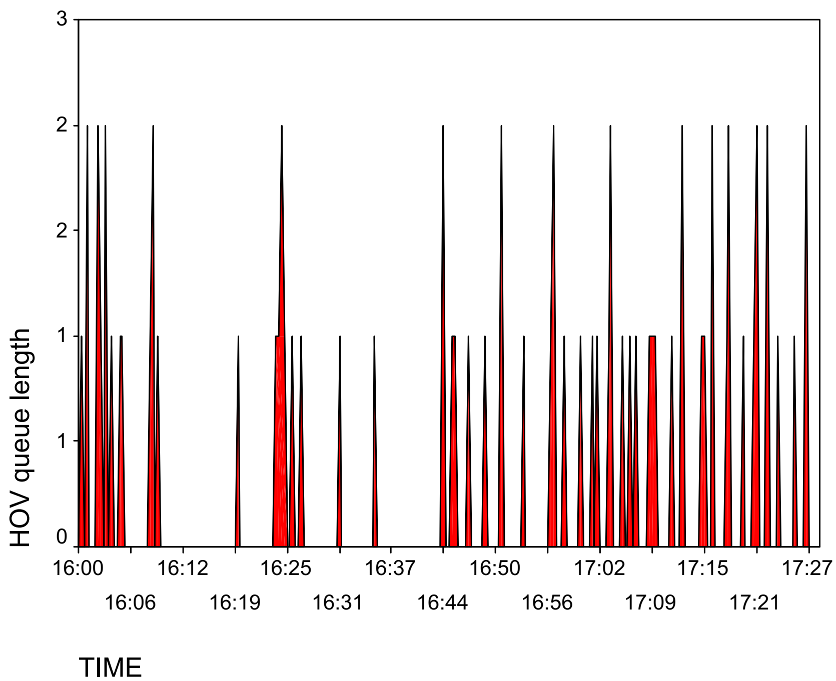

Good Hope Road Slip Ramp 2/8/2000 AM peak
SOV queue length
0
1
1
2
2
3
3
4
7:00
7:06
7:12
7:18
7:25
7:31
7:37
7:43
7:50
7:56
8:02
8:08
8:15
8:21
8:27
TIME

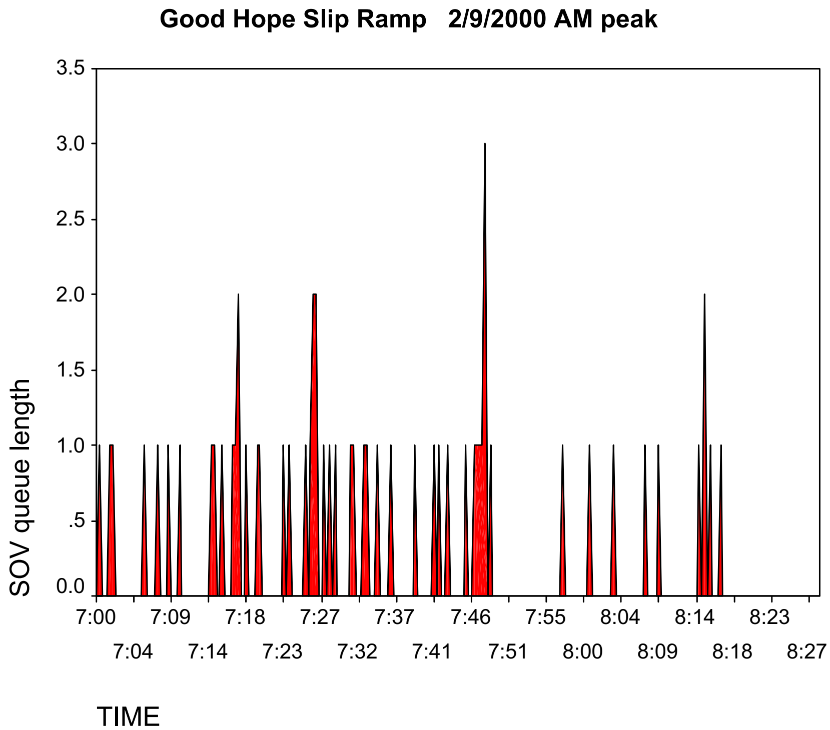

Good Hope Road Slip Ramp 2/8/2000 AM peak
HOV queue length
0
1
1
2
2
3
3
4
7:00
7:06
7:12
7:18
7:25
7:31
7:37
7:43
7:50
7:56
8:02
8:08
8:15
8:21
8:27
TIME

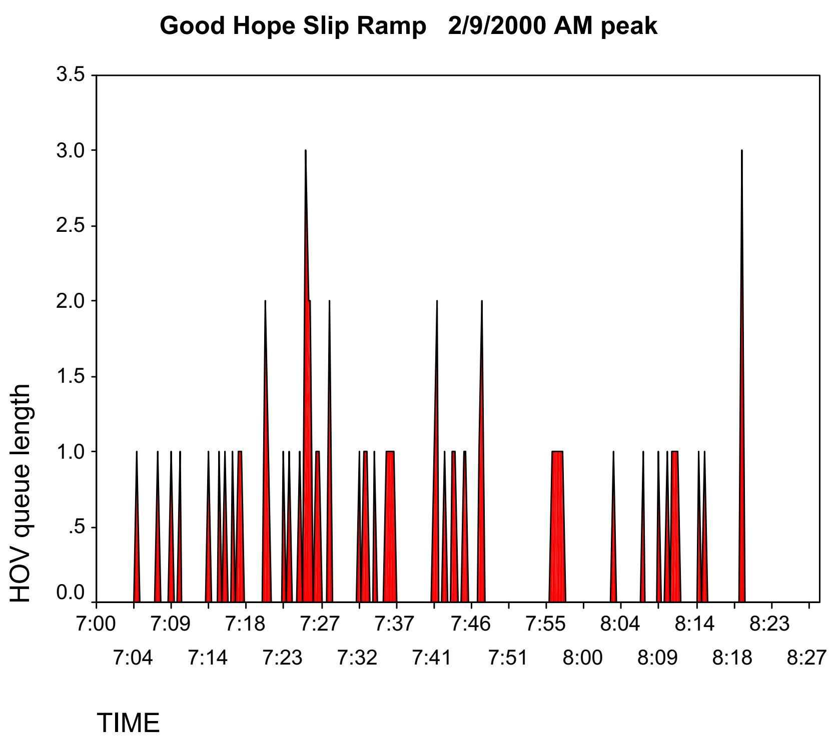
Good Hope Road Slip Ramp 2/8/2000 PM peak



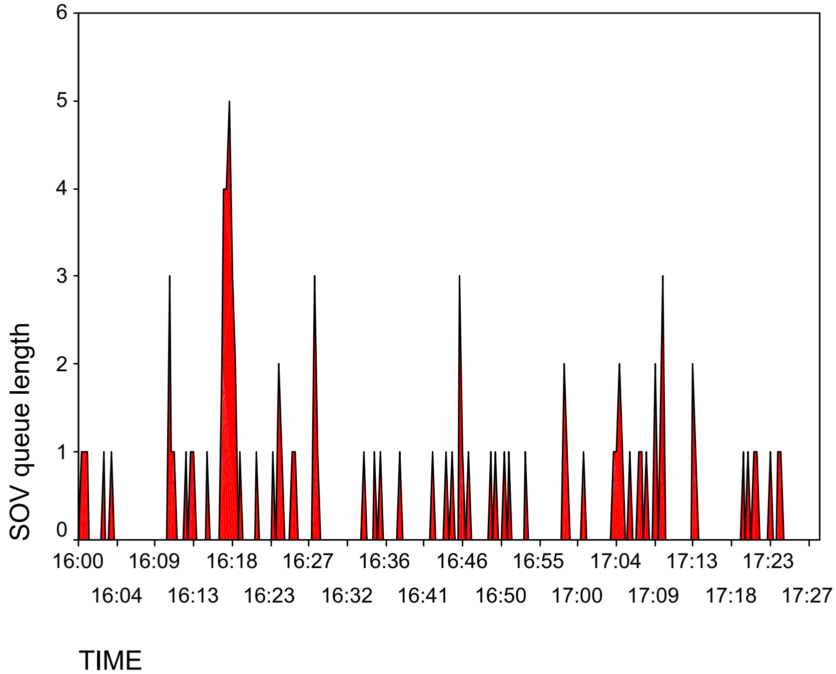
Good Hope Road Slip Ramp 2/8/2000 PM peak



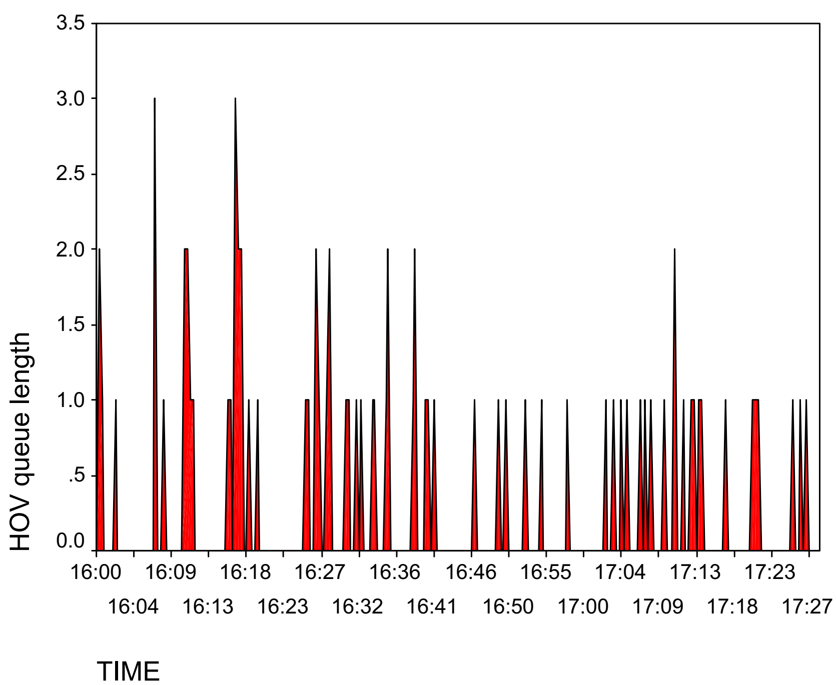

Good Hope Slip Ramp 2/9/2000 AM peak
SOV queue length
3.5
3.0
2.5
2.0
1.5
1.0
.5
0.0
7:00
7:04
7:09
7:14
7:18
7:23
7:27
7:32
7:37
7:41
7:46
7:51
7:55
8:00
8:04
8:09
8:14
8:18
8:23
8:27
TIME


Good Hope Slip Ramp 2/9/2000 AM peak
HOV queue length
3.5
3.0
2.5
2.0
1.5
1.0
.5
0.0
7:00
7:04
7:09
7:14
7:18
7:23
7:27
7:32
7:37
7:41
7:46
7:51
7:55
8:00
8:04
8:09
8:14
8:18
8:23
8:27
TIME

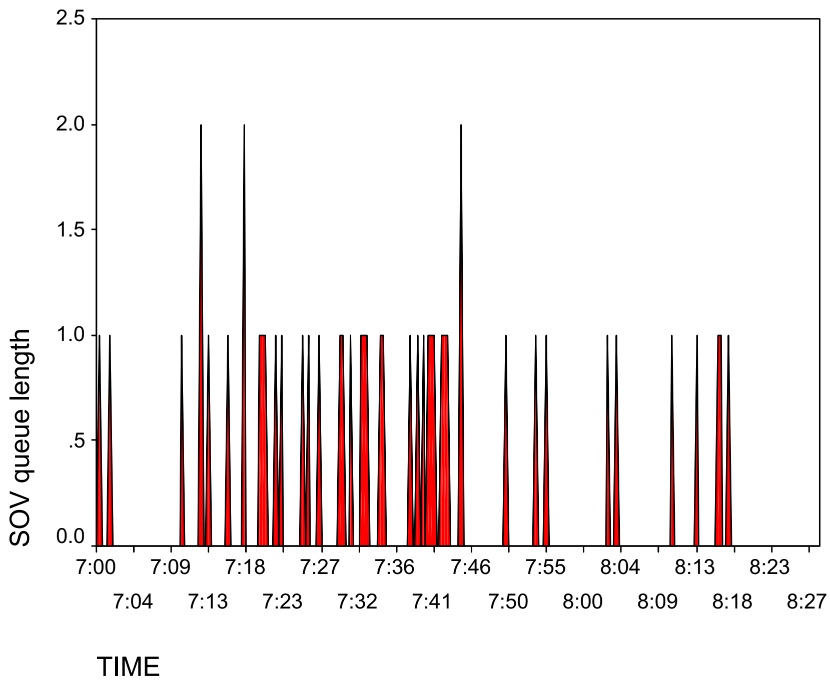
Good Hope Slip Ramp   2/9/2000 PM peak



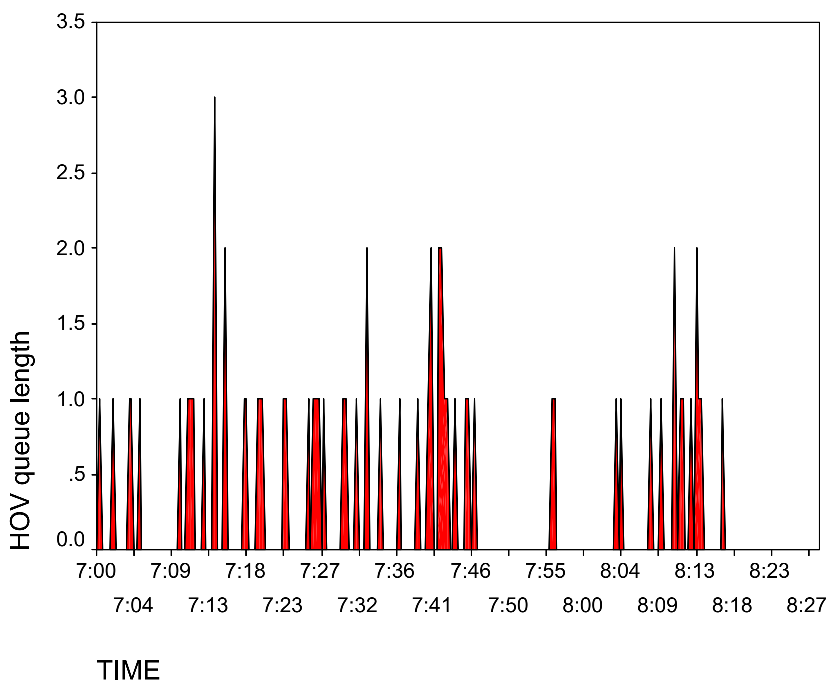
Good Hope Slip Ramp   2/9/2000 PM peak

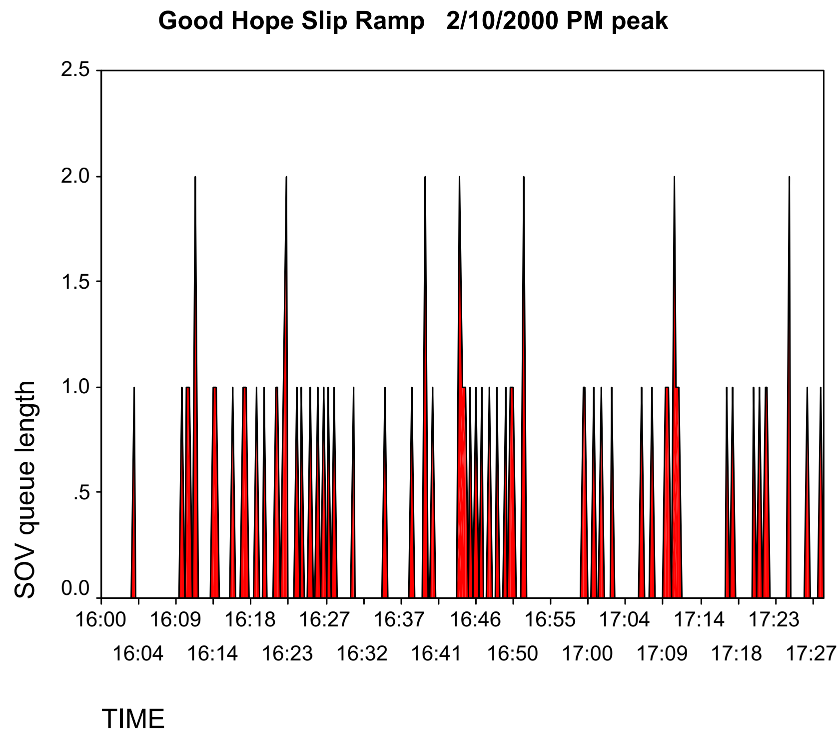


Good Hope Slip Ramp   2/10/2000 AM peak



Good Hope Slip Ramp   2/10/2000 AM peak




Good Hope Slip Ramp 2/10/2000 PM peak
SOV queue length
0.0
.5
1.0
1.5
2.0
2.5
16:00
16:04
16:09
16:14
16:18
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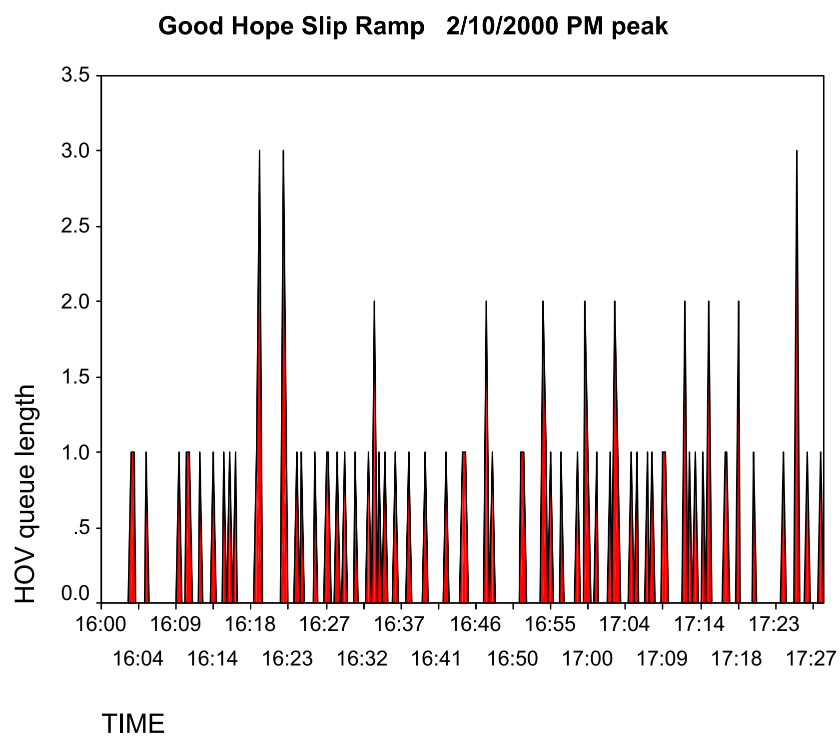
16:32
16:37
16:41
16:46
16:50
16:55
17:00
17:04
17:09
17:14
17:18
17:23
17:27
TIME


Good Hope Slip Ramp 2/10/2000 PM peak
HOV queue length
0.0
.5
1.0
1.5
2.0
2.5
3.0
3.5
16:00
16:04
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16:18
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16:32
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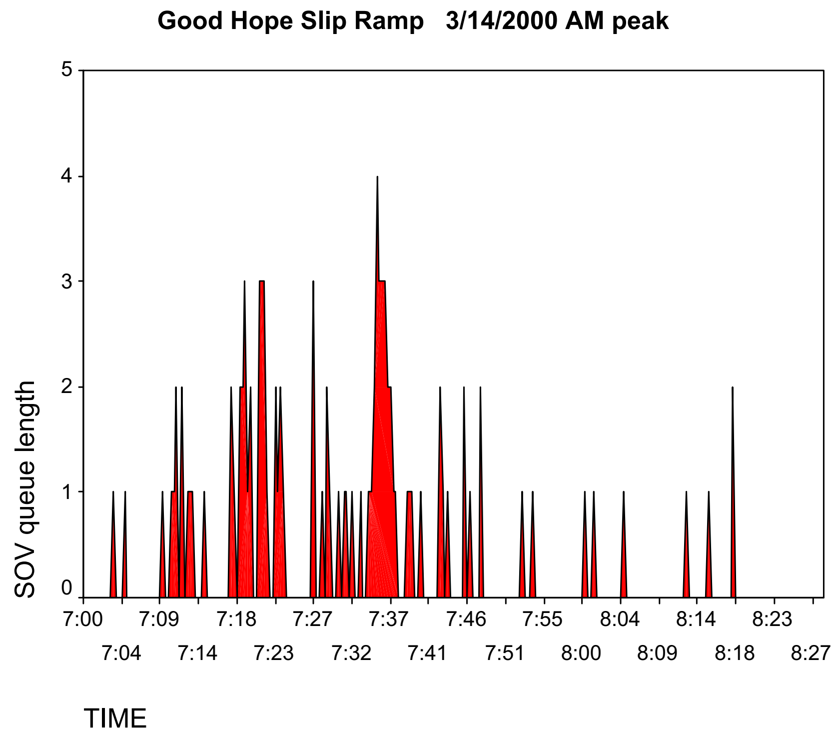
16:50
16:55
17:00
17:04
17:09
17:14
17:18
17:23
17:27
TIME


Good Hope Slip Ramp 3/14/2000 AM peak
SOV queue length
0
1
2
3
4
5
7:00
7:04
7:09
7:14
7:18
7:23
7:27
7:32
7:37
7:41
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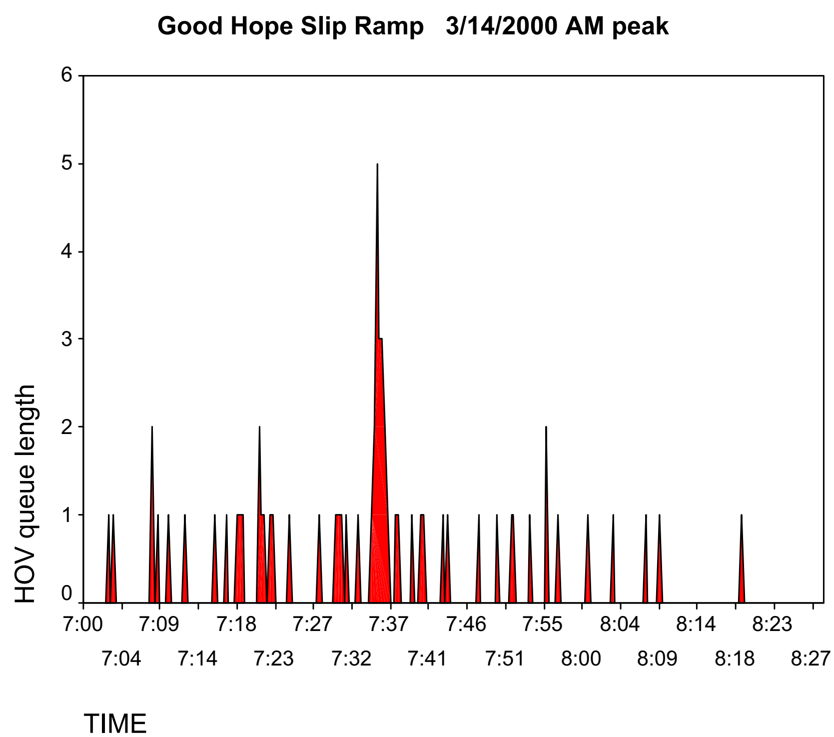
7:55
8:00
8:04
8:09
8:14
8:18
8:23
8:27
TIME


Good Hope Slip Ramp 3/14/2000 AM peak
HOV queue length
0
1
2
3
4
5
6
7:00
7:04
7:09
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7:27
7:32
7:37
7:41
7:46
7:51
7:55
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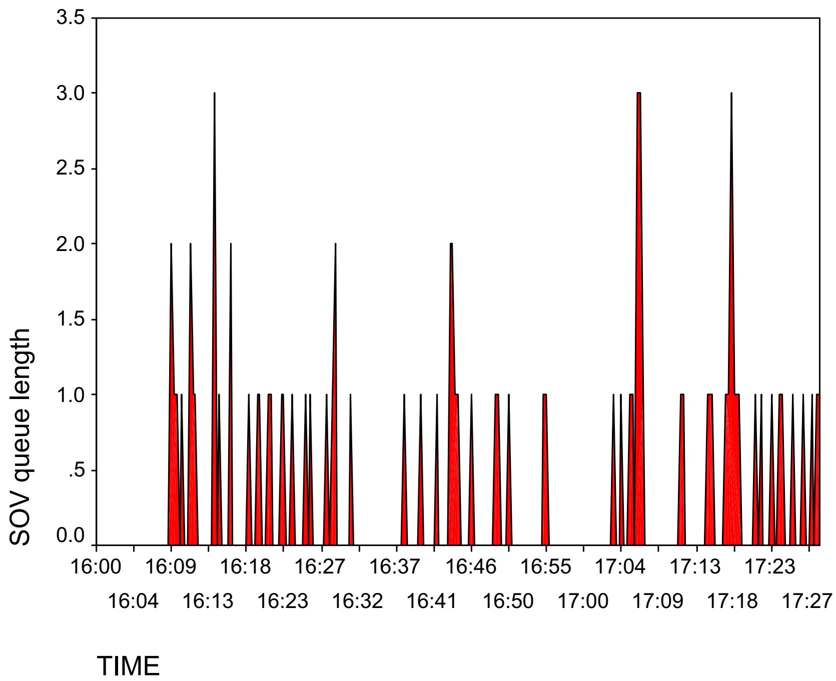
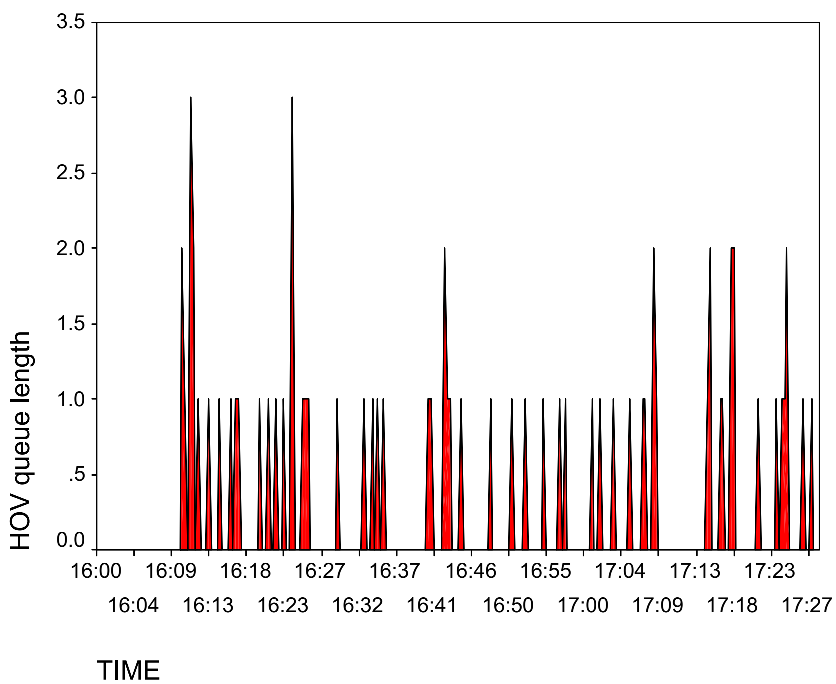
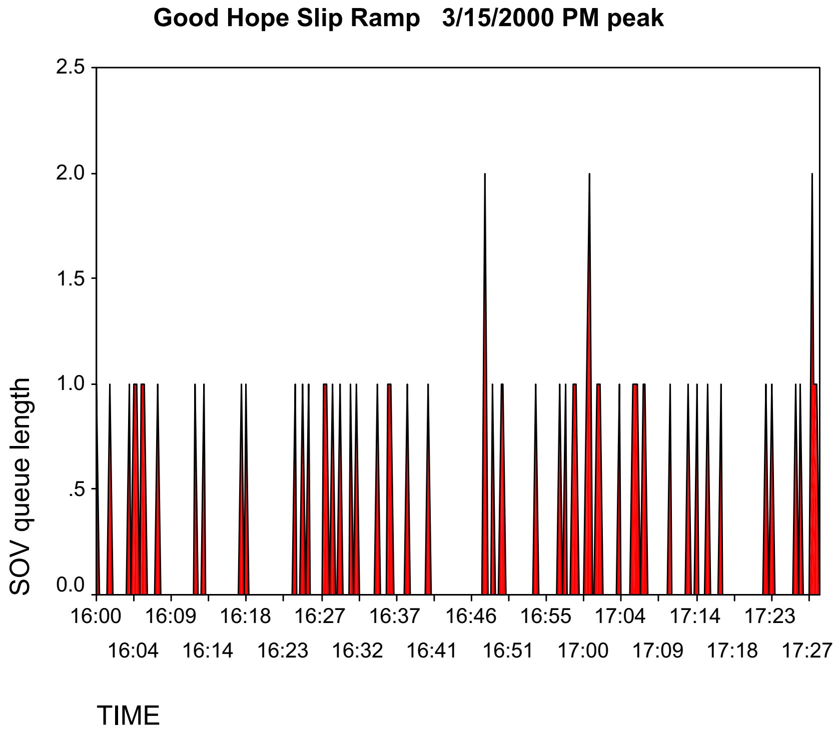
8:04
8:09
8:14
8:18
8:23
8:27
TIME

Good Hope Slip Ramp 3/14/2000 PM peak



Good Hope Slip Ramp 3/14/2000 PM peak




Good Hope Slip Ramp 3/15/2000 PM peak
SOV queue length
2.5
2.0
1.5
1.0
.5
0.0
16:00
16:04
16:09
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16:18
16:23
16:27
16:32
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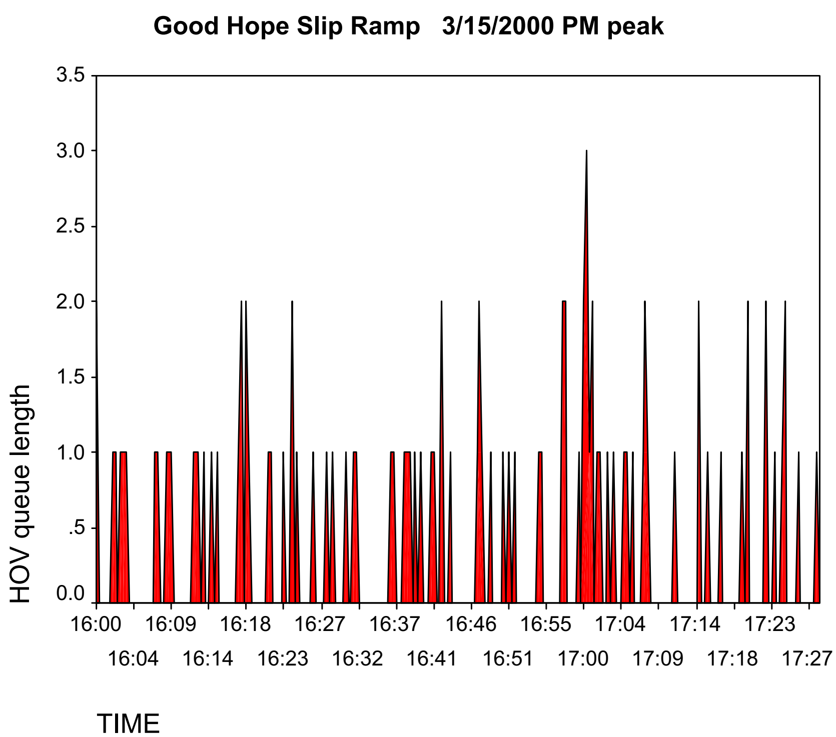
16:41
16:46
16:51
16:55
17:00
17:04
17:09
17:14
17:18
17:23
17:27
TIME


Good Hope Slip Ramp 3/15/2000 PM peak
HOV queue length
3.5
3.0
2.5
2.0
1.5
1.0
.5
0.0
16:00
16:04
16:09
16:14
16:18
16:23
16:27
16:32
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16:41
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17:04
17:09
17:14
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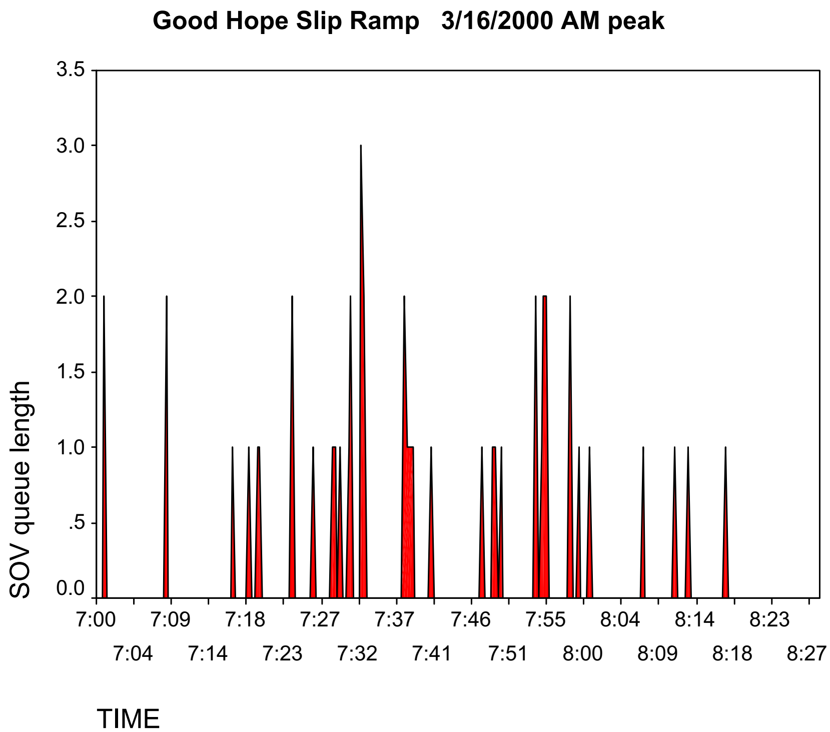
17:23
17:27
TIME


Good Hope Slip Ramp 3/16/2000 AM peak
SOV queue length
3.5
3.0
2.5
2.0
1.5
1.0
.5
0.0
7:00
7:09
7:18
7:27
7:37
7:46
7:55
8:04
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8:23
7:04
7:14
7:23
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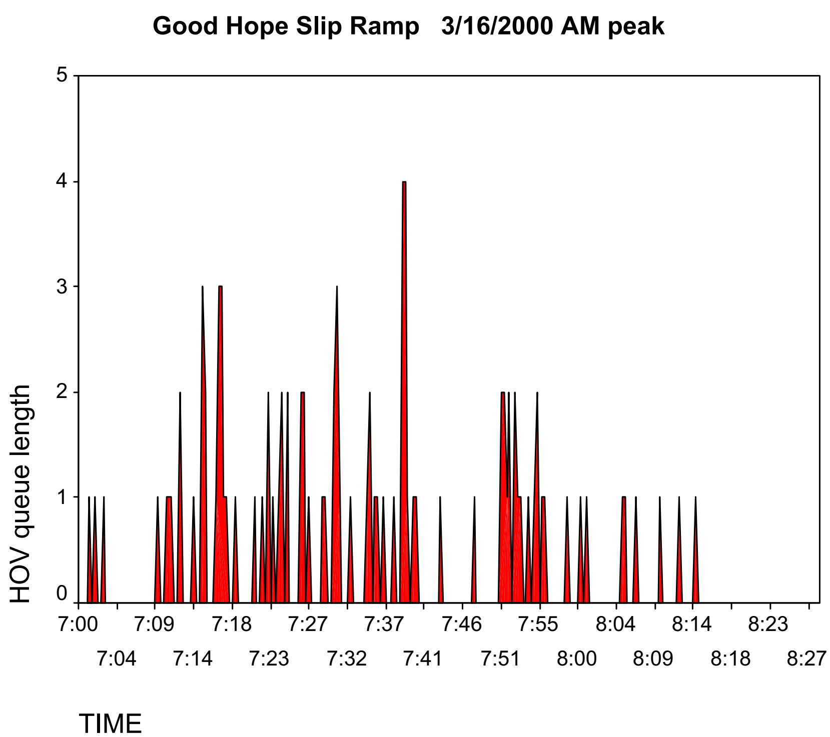
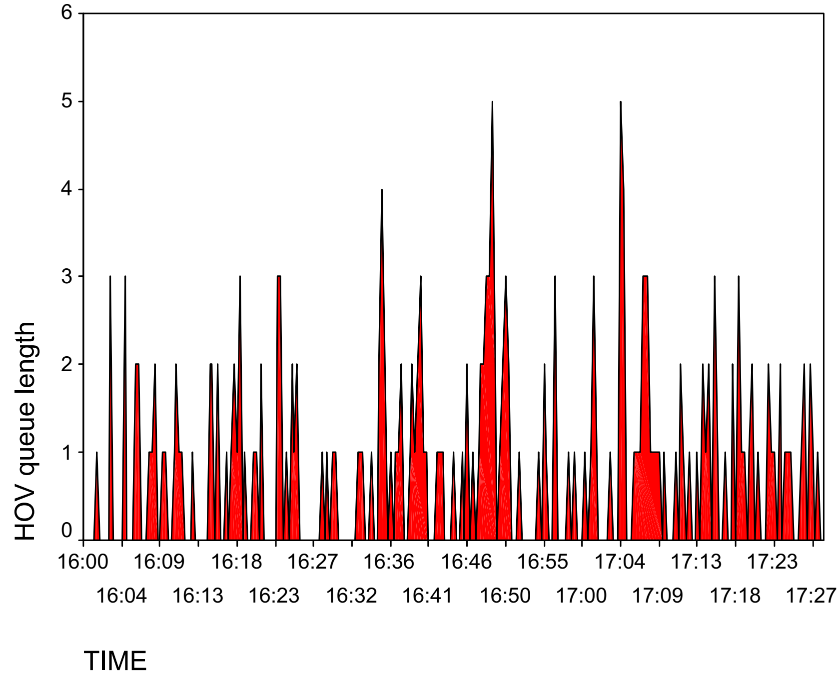
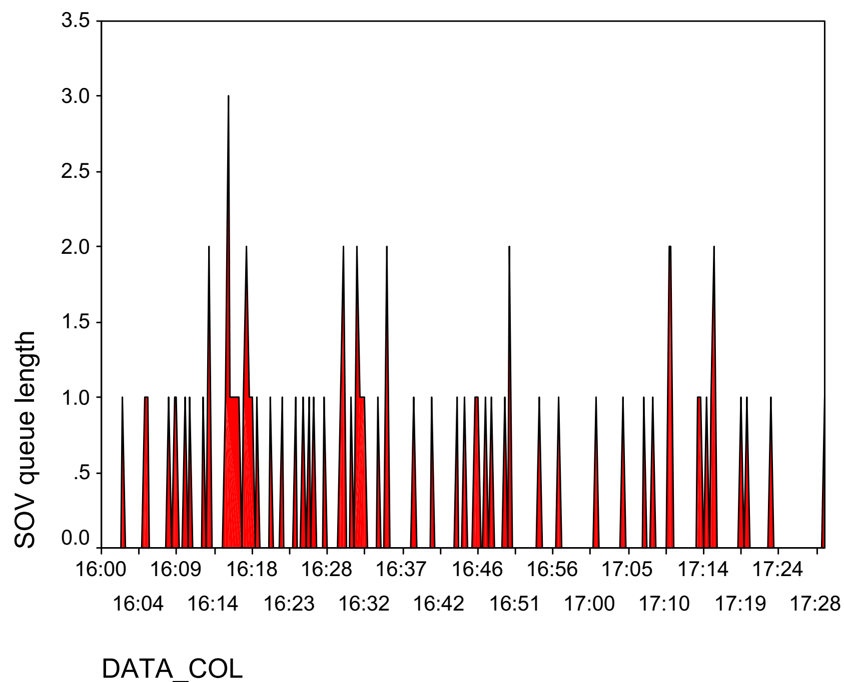
8:00
8:09
8:18
8:27
TIME


Good Hope Slip Ramp 3/16/2000 AM peak
HOV queue length
5
4
3
2
1
0
7:00
7:09
7:18
7:27
7:37
7:46
7:55
8:04
8:14
8:23
7:04
7:14
7:23
7:32
7:41
7:51
8:00
8:09
8:18
8:27
TIME

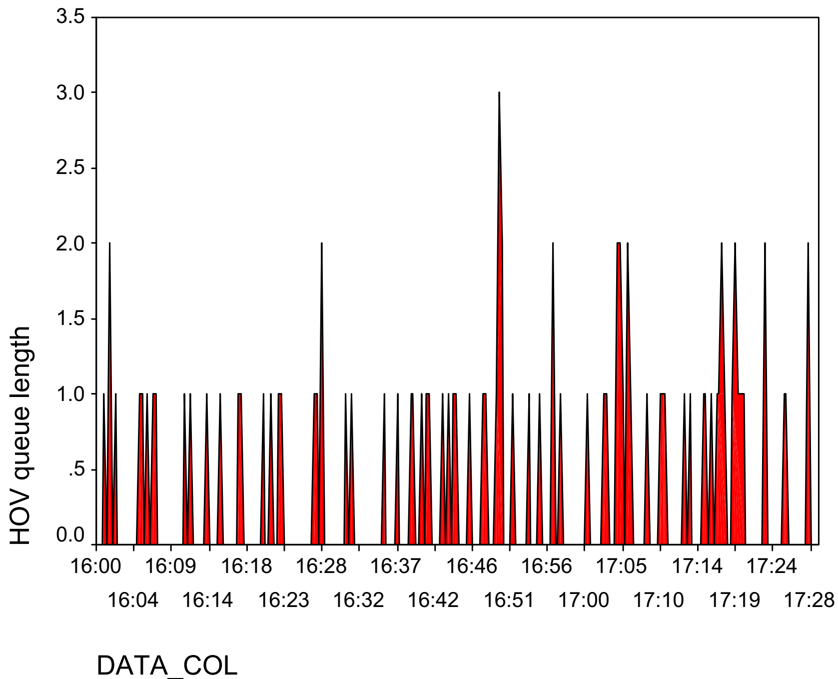
Good Hope Slip Ramp 3/16/2000 PM peak



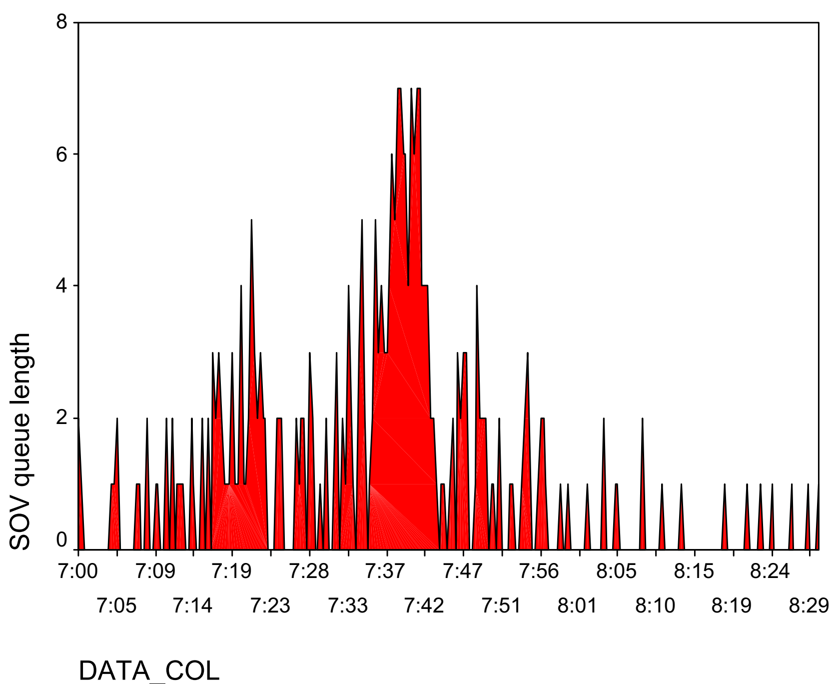
Good Hope Road Slip Ramp 3/21/2000 PM peak

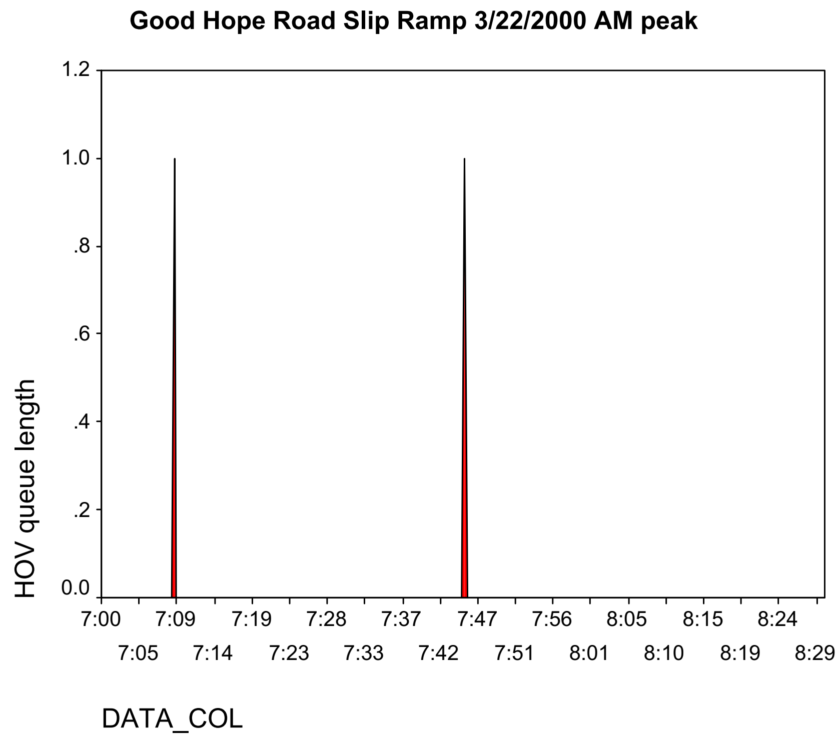


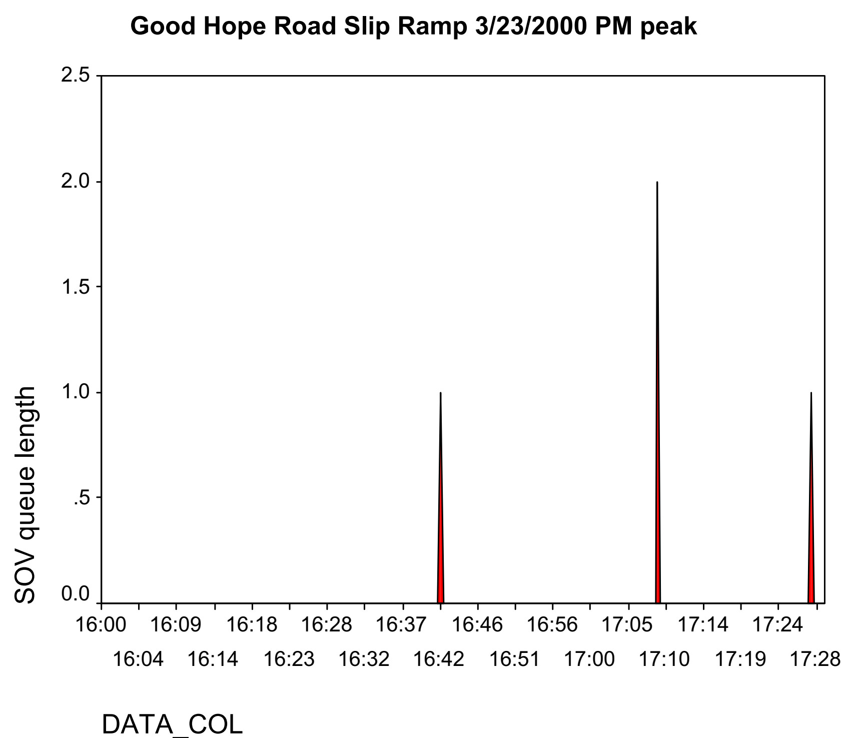
Good Hope Road Slip Ramp 3/21/2000 PM peak



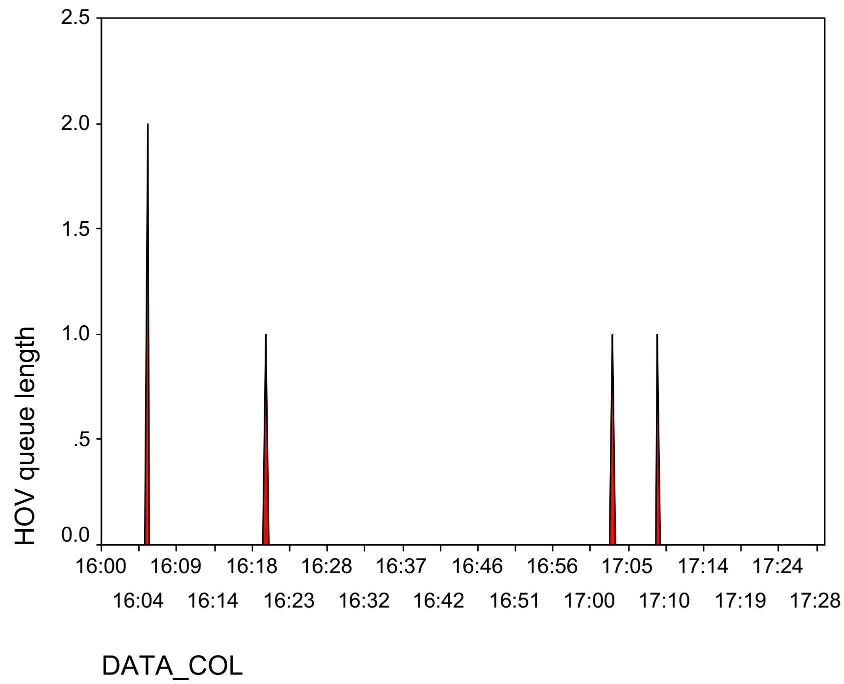
Good Hope Road Slip Ramp 3/22/2000 AM peak



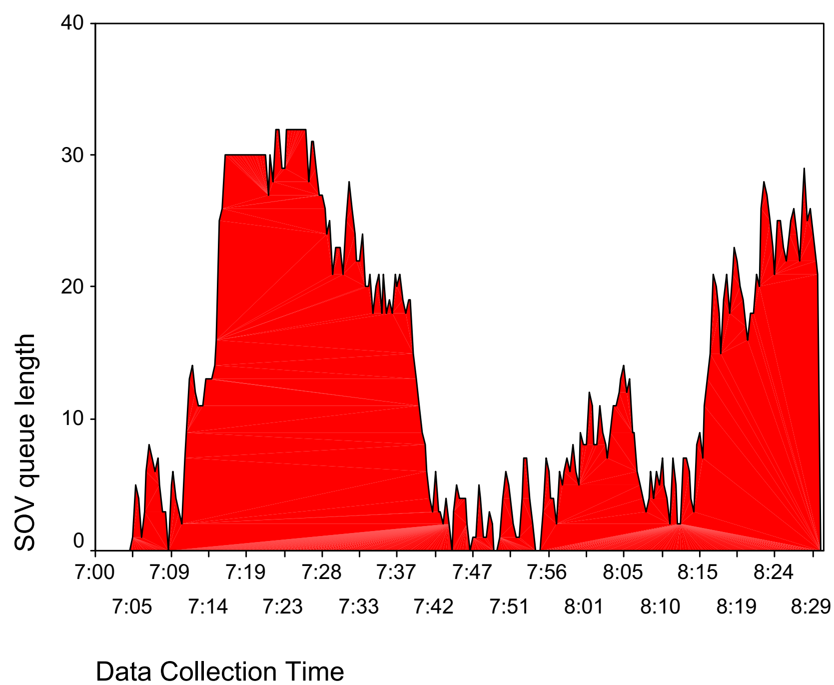

Good Hope Road Slip Ramp 3/22/2000 AM peak
HOV queue length
1.2
1.0
.8
.6
.4
.2
0.0
7:00
7:05
7:09
7:14
7:19
7:23
7:28
7:33
7:37
7:42
7:47
7:51
7:56
8:01
8:05
8:10
8:15
8:19
8:24
8:29
DATA_COL

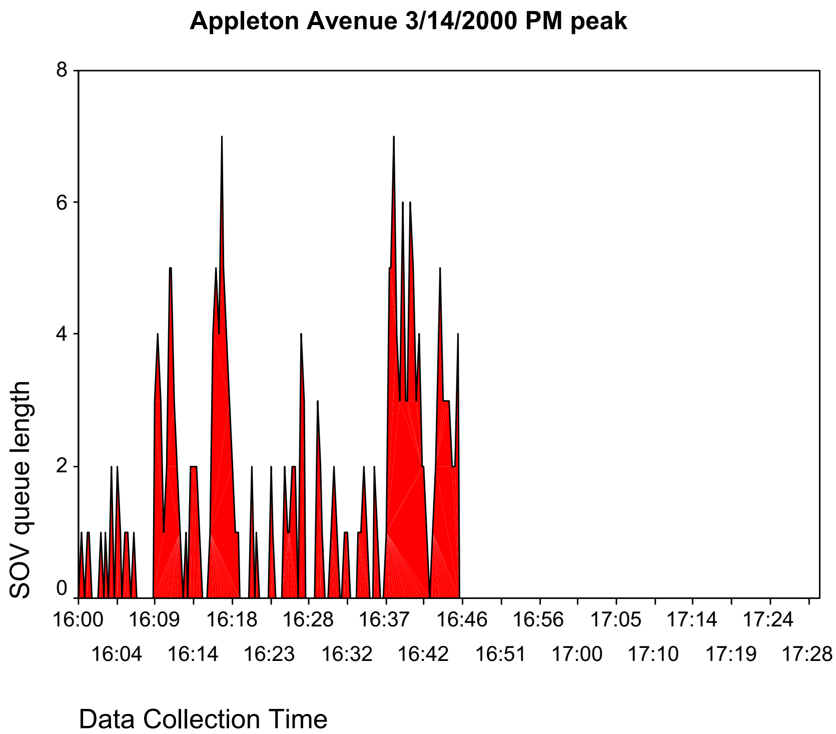

Good Hope Road Slip Ramp 3/23/2000 PM peak
SOV queue length
2.5
2.0
1.5
1.0
.5
0.0
16:00
16:04
16:09
16:14
16:18
16:23
16:28
16:32
16:37
16:42
16:46
16:51
16:56
17:00
17:05
17:10
17:14
17:19
17:24
17:28
DATA_COL

Good Hope Road Slip Ramp 3/23/2000 PM peak



Appleton Avenue Street 3/14/2000 AM peak




Appleton Avenue 3/14/2000 PM peak
SOV queue length
0
2
4
6
8
16:00
16:04
16:09
16:14
16:18
16:23
16:28
16:32
16:37
16:42
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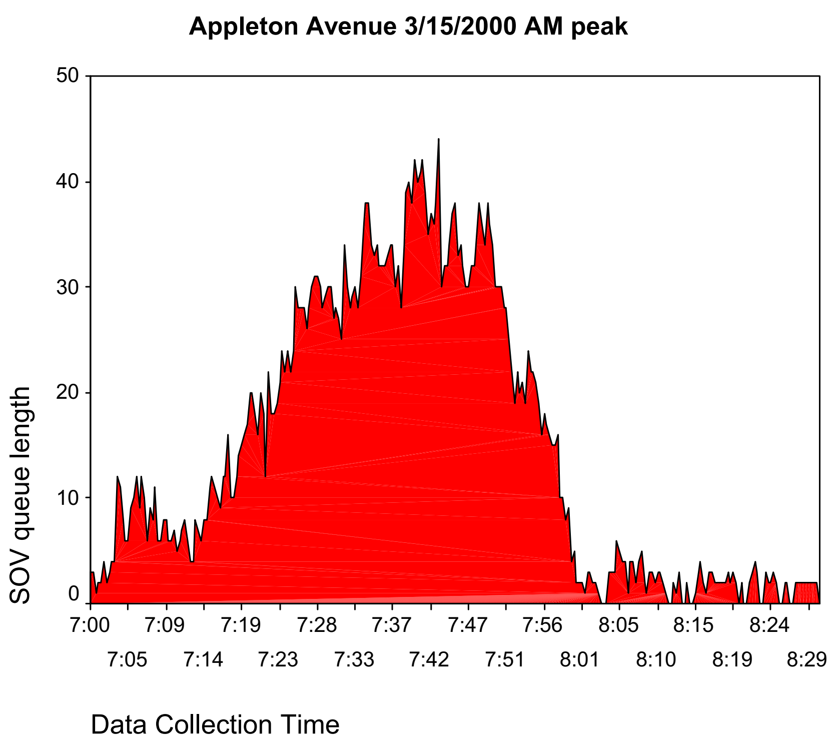
16:56
17:00
17:05
17:10
17:14
17:19
17:24
17:28
Data Collection Time


Appleton Avenue 3/15/2000 AM peak
SOV queue length
0
10
20
30
40
50
7:00
7:05
7:09
7:14
7:19
7:23
7:28
7:33
7:37
7:42
7:47
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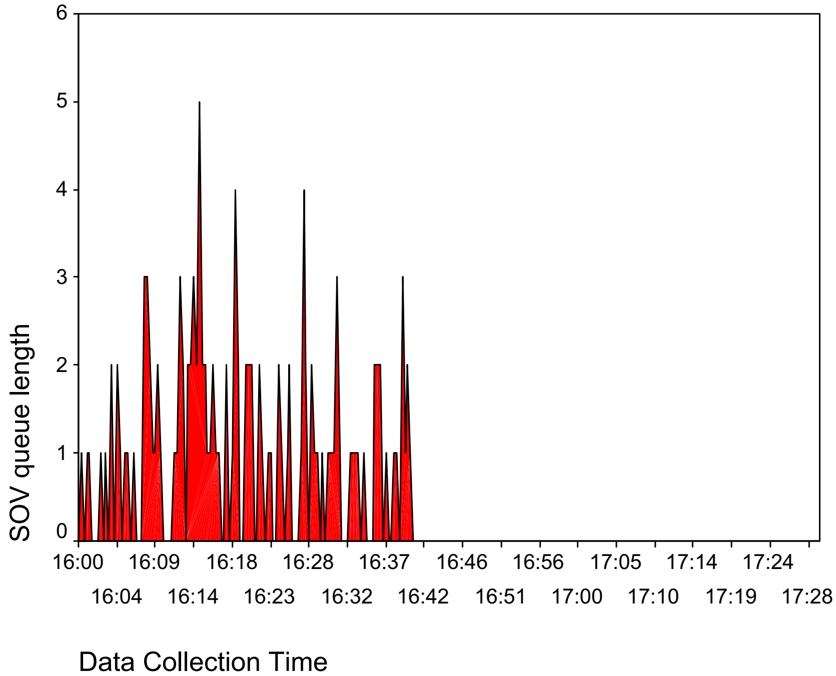
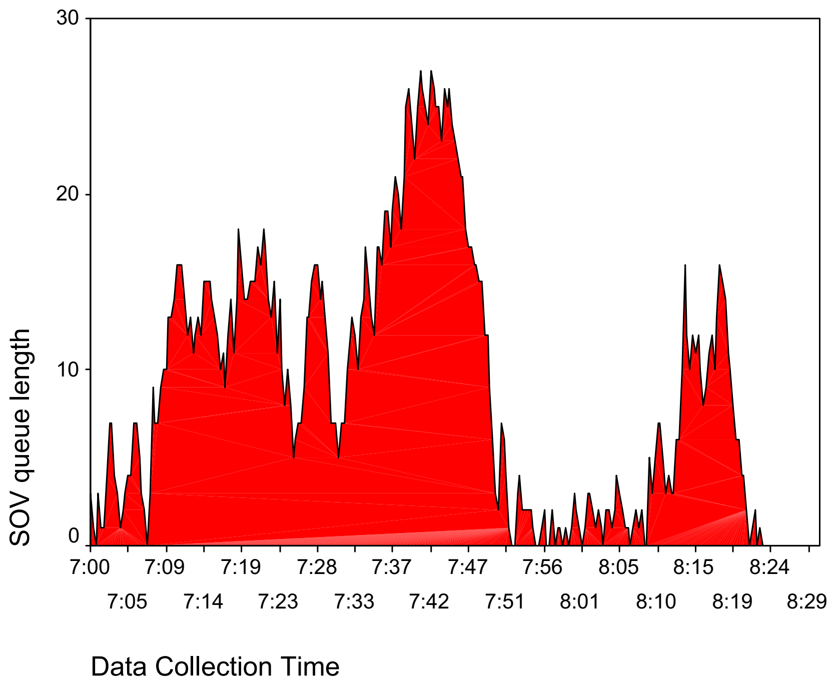
8:05
8:10
8:15
8:19
8:24
8:29
Data Collection Time

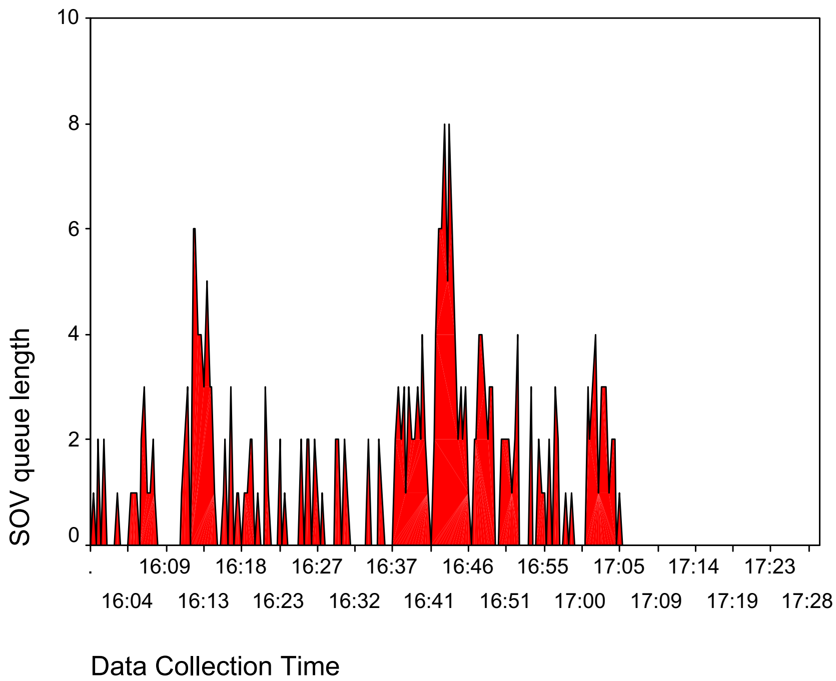
Appleton Avenue 3/15/2000 PM peak



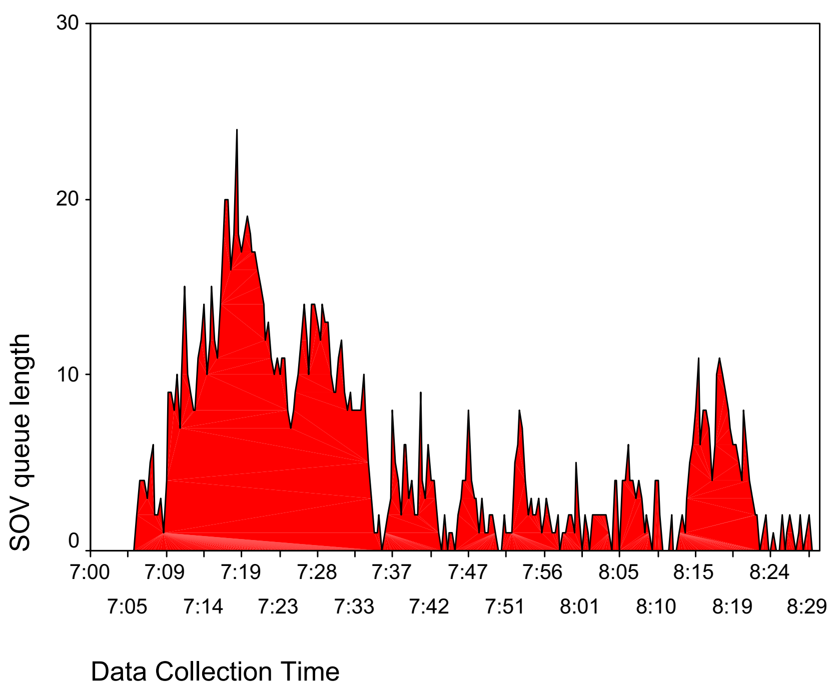
Appleton Avenue 3/16/2000 AM peak



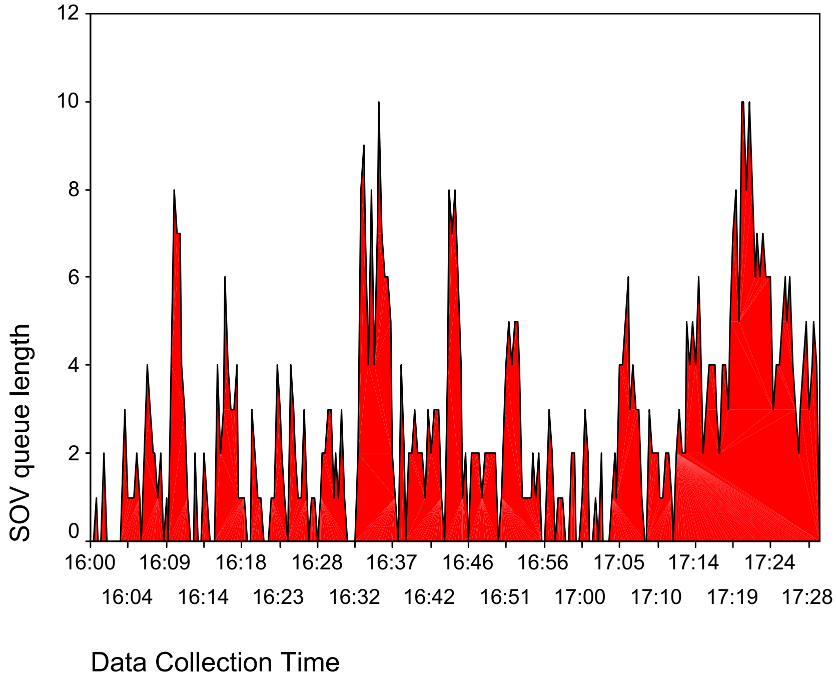
Appleton Avenue 3/16/2000 PM peak



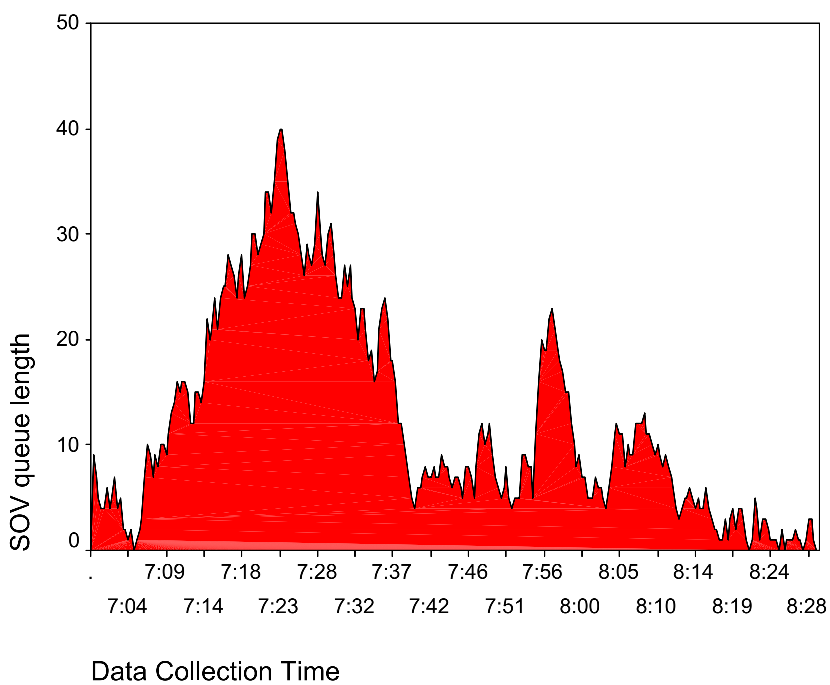
Appleton Avenue 3/21/2000 AM peak



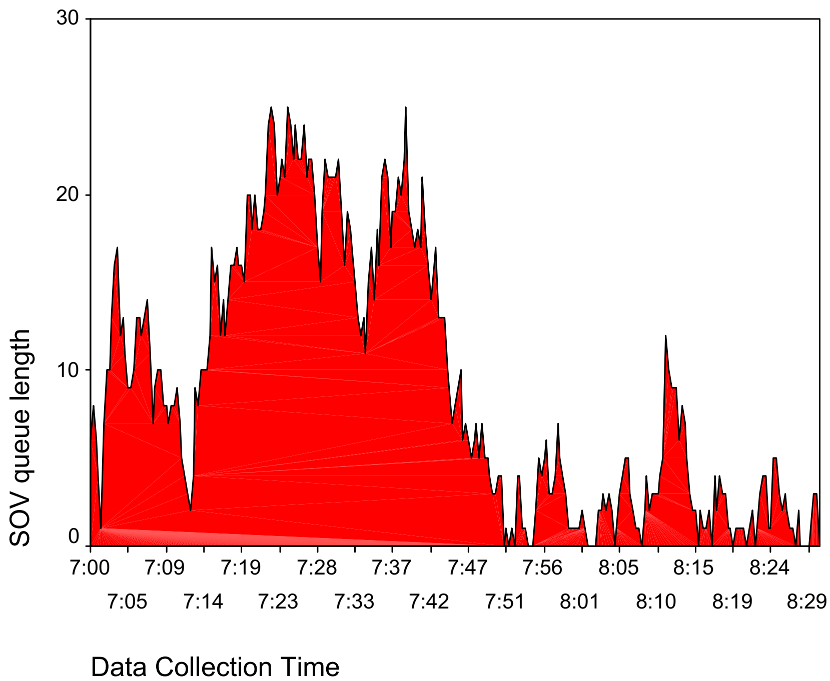
Appleton Avenue 3/21/2000 PM peak



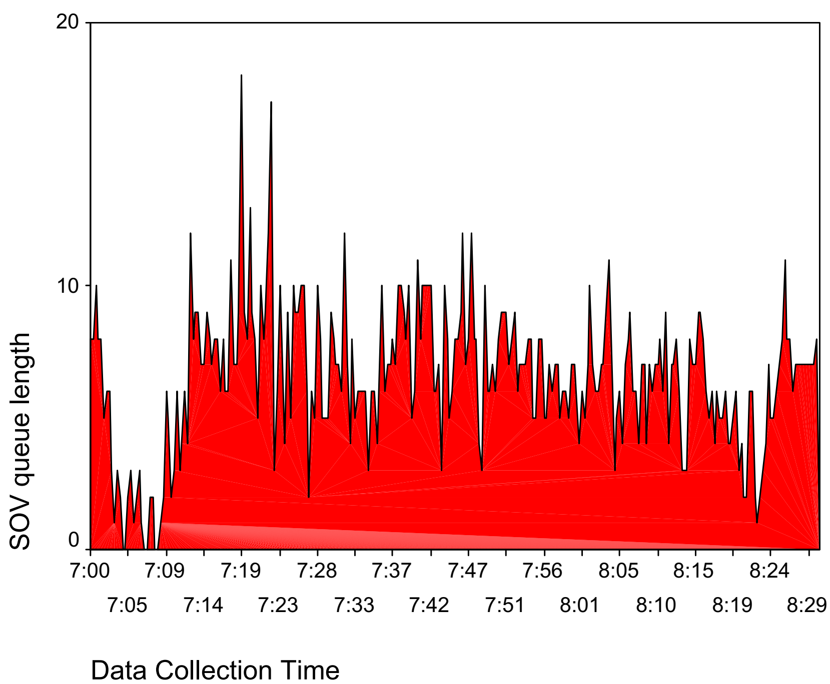
Appleton Avenue 3/22/2000 AM peak



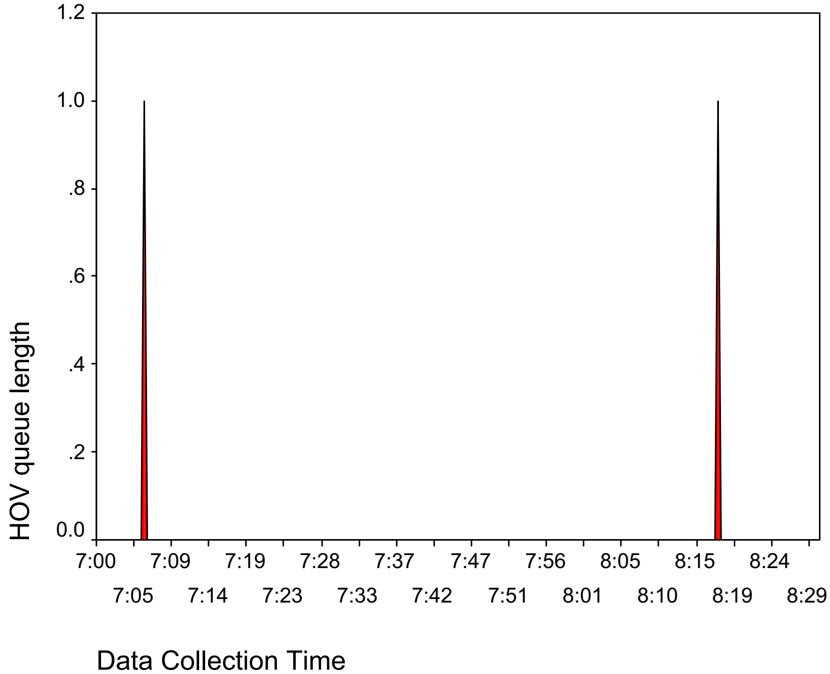
Appleton Avenue 3/23/2000 AM peak



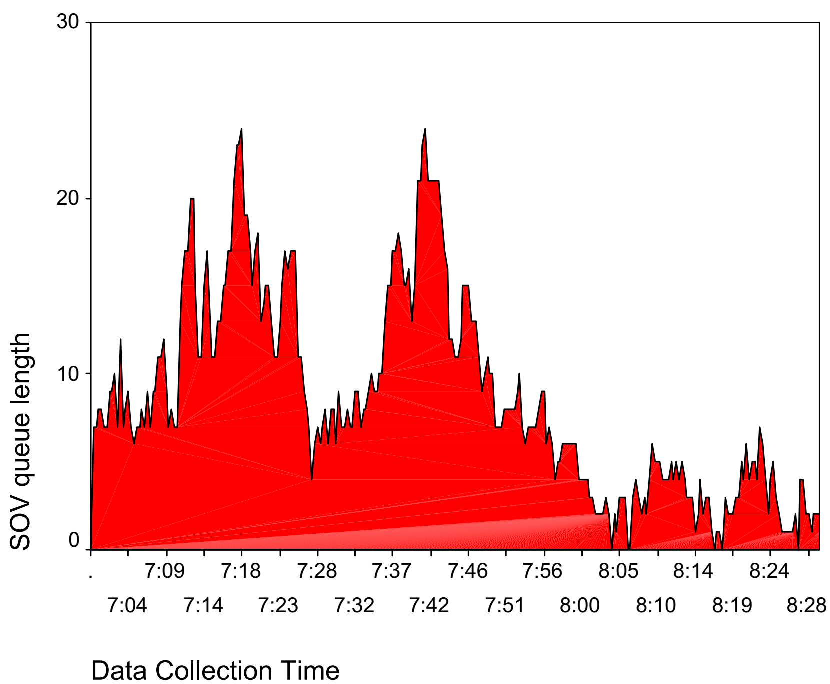
Hampton Avenue 3/14/2000 AM peak



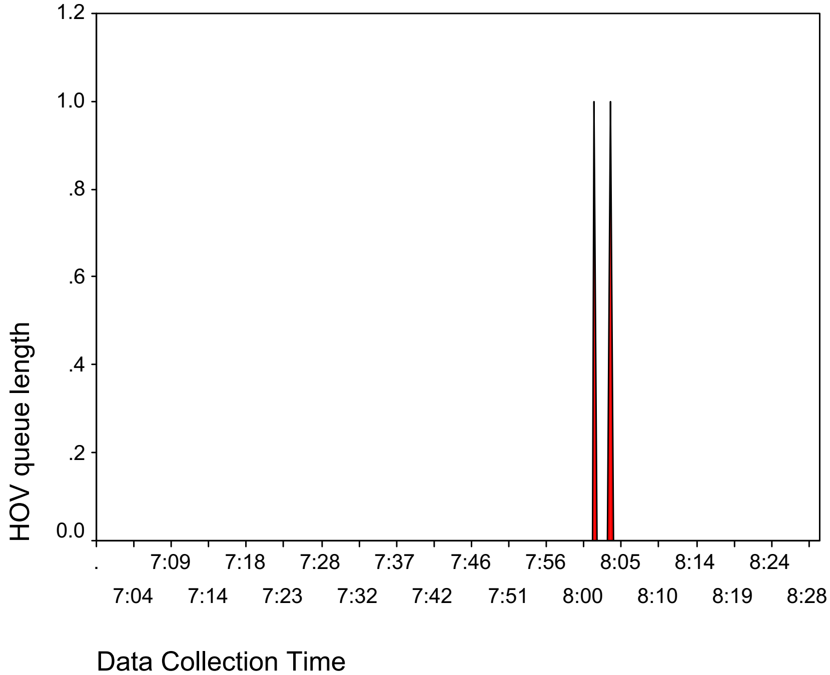
Hampton Avenue 3/14/2000 AM peak



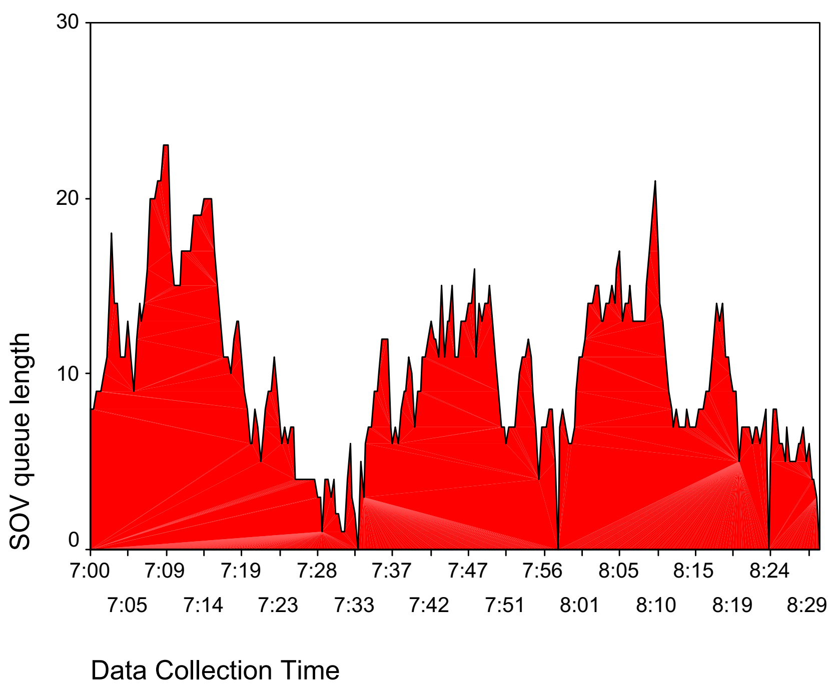
Hampton Avenue 3/15/2000 AM peak

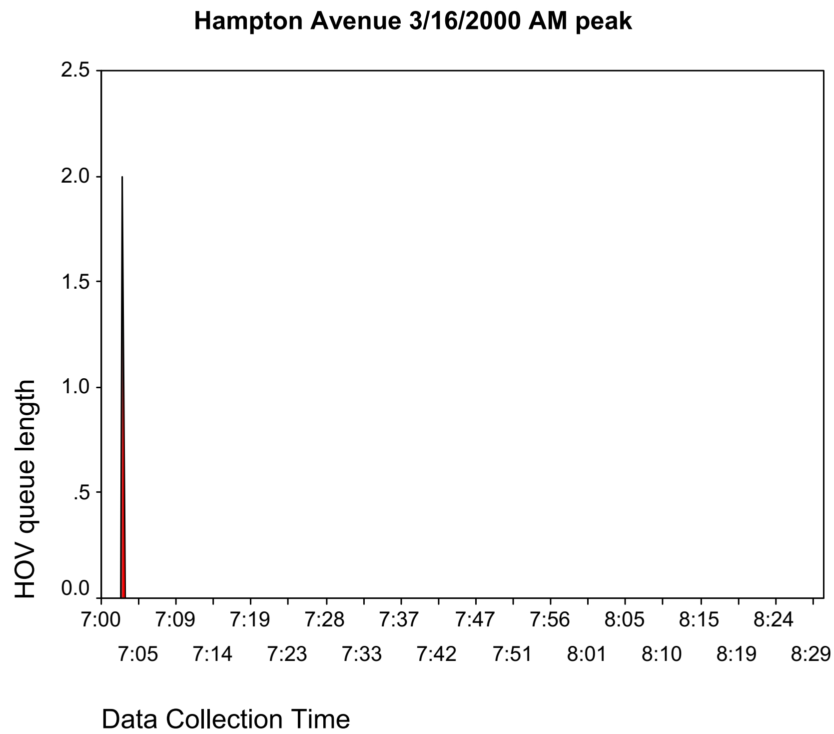


Hampton Avenue 3/15/2000 AM peak



Hampton Avenue 3/16/2000 AM peak




Hampton Avenue 3/16/2000 AM peak
HOV queue length
2.5
2.0
1.5
1.0
.5
0.0
7:00
7:09
7:19
7:28
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8:01
8:10
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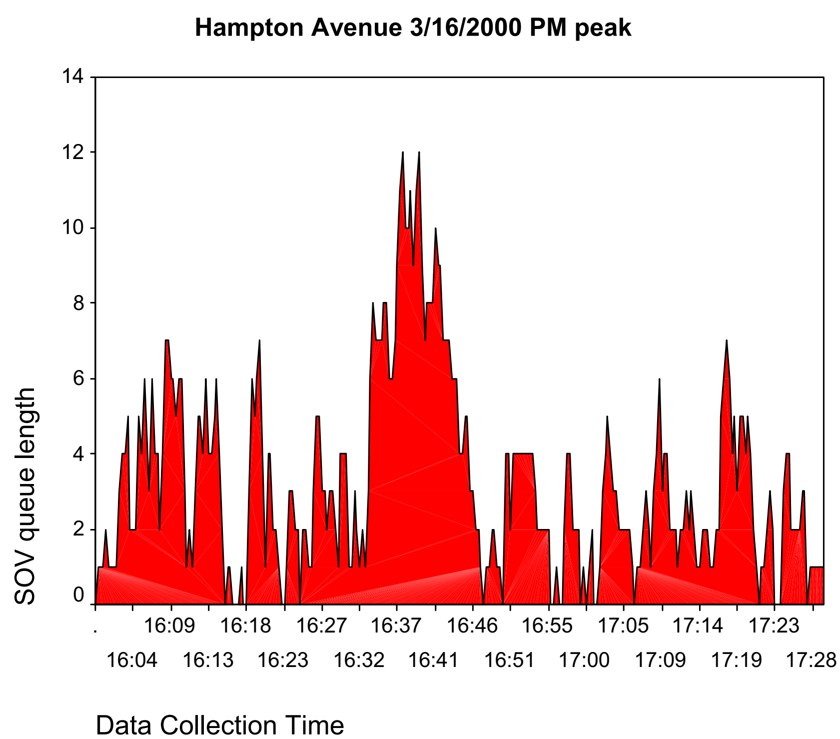
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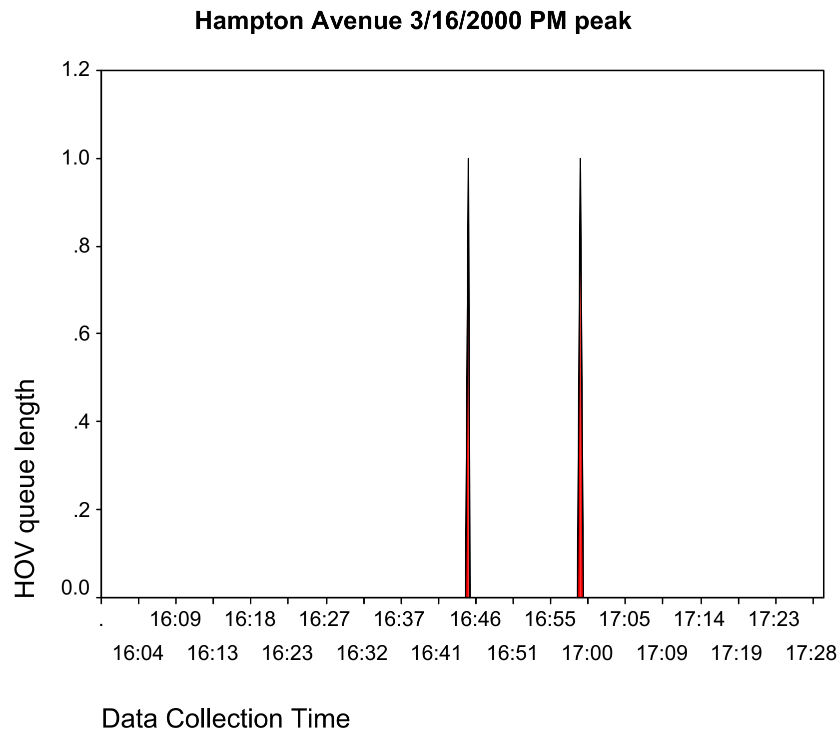
Data Collection Time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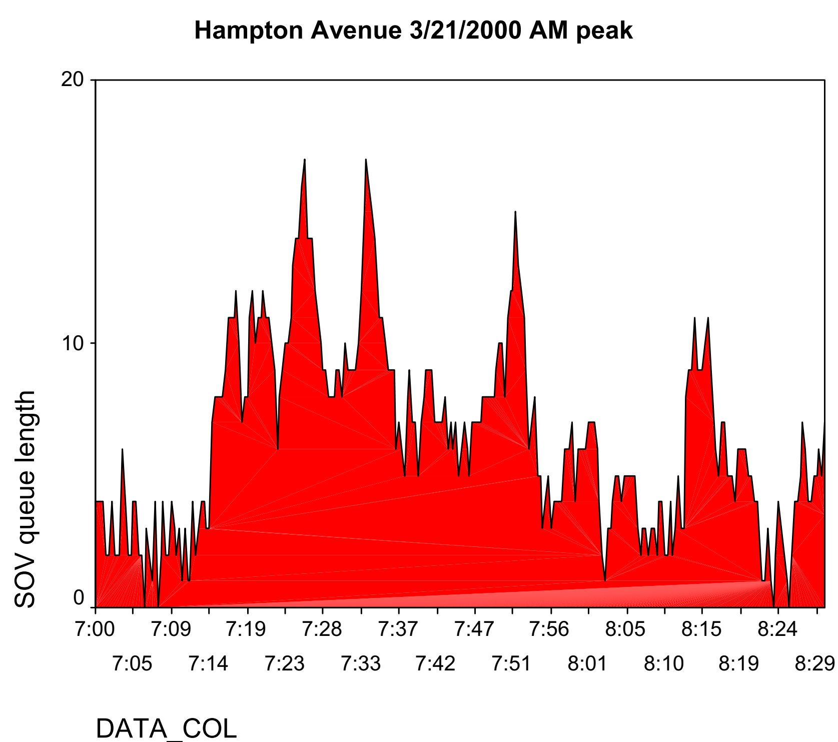
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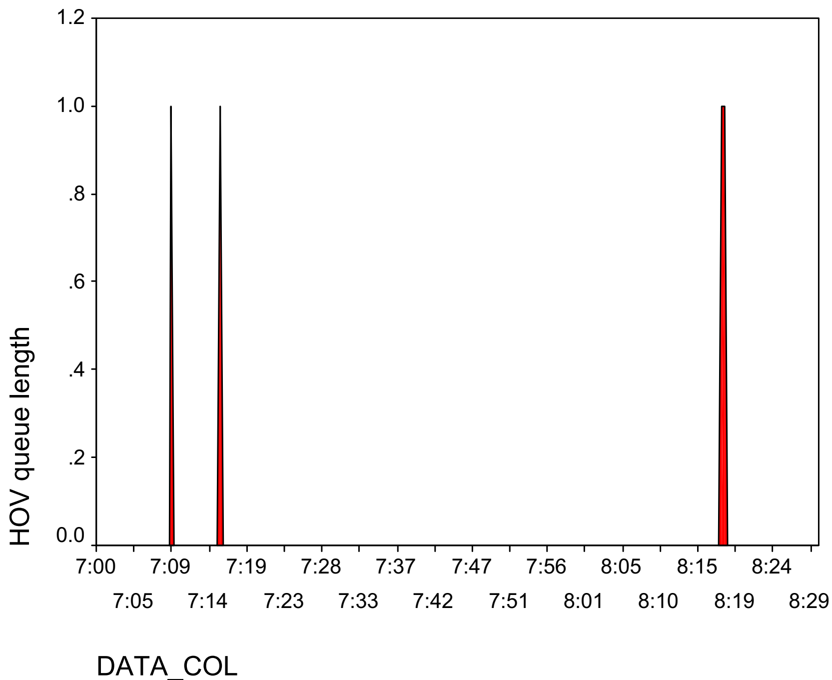
DATA_COL

Hampton Avenue 3/21/2000 AM peak



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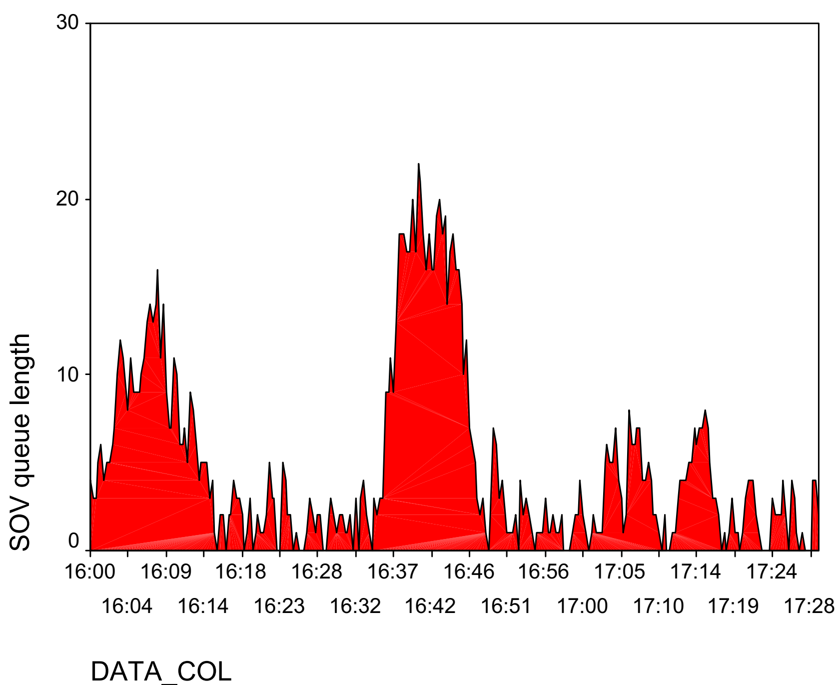

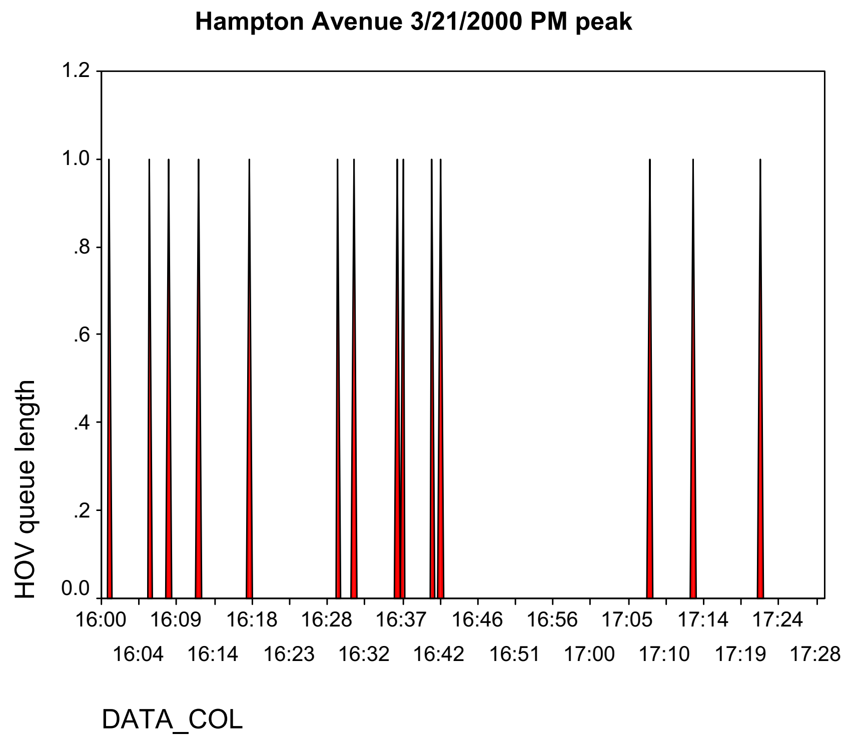
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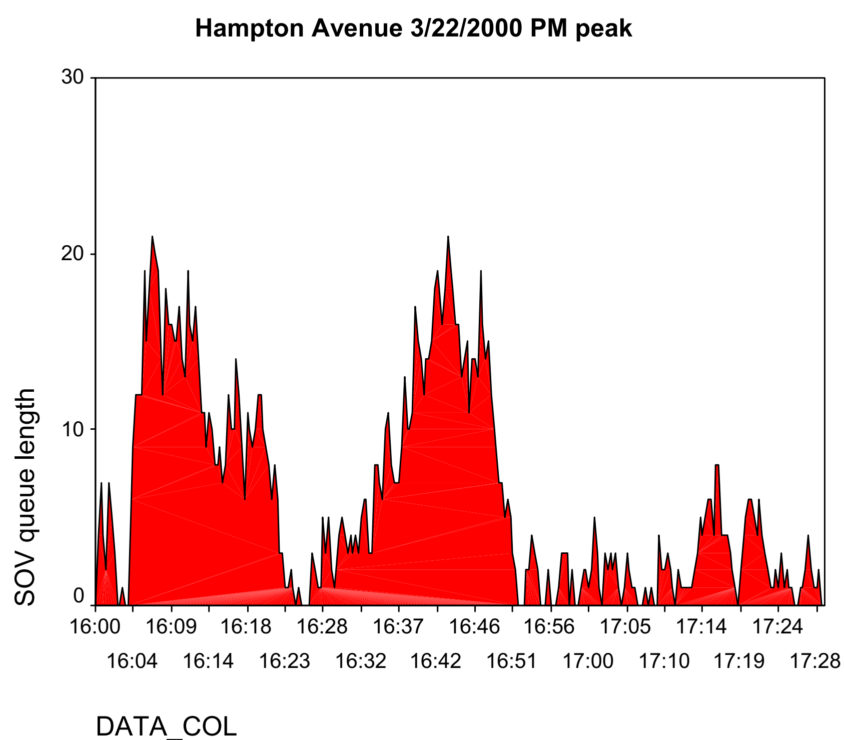
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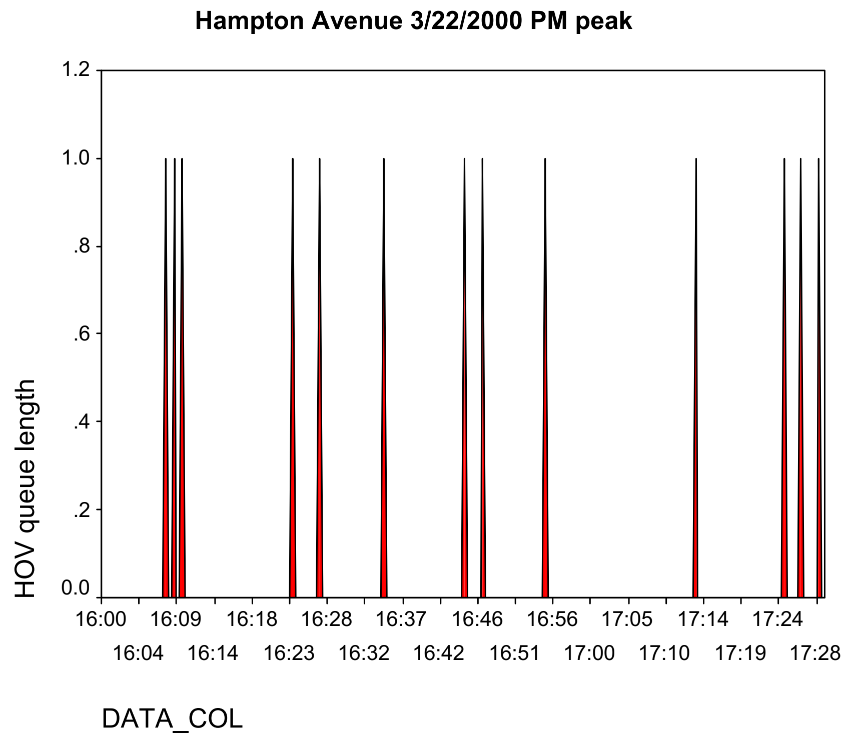
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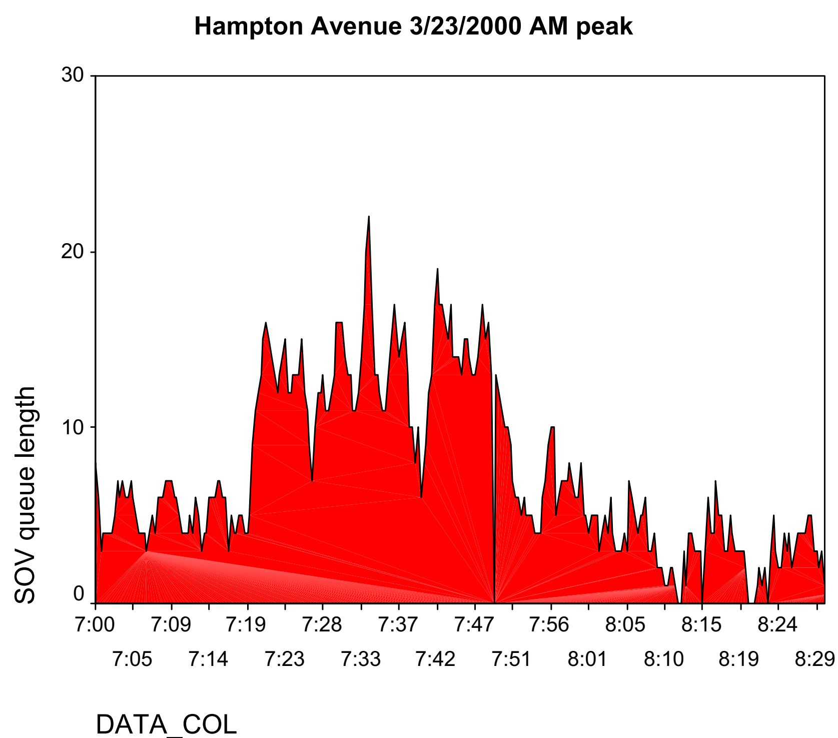
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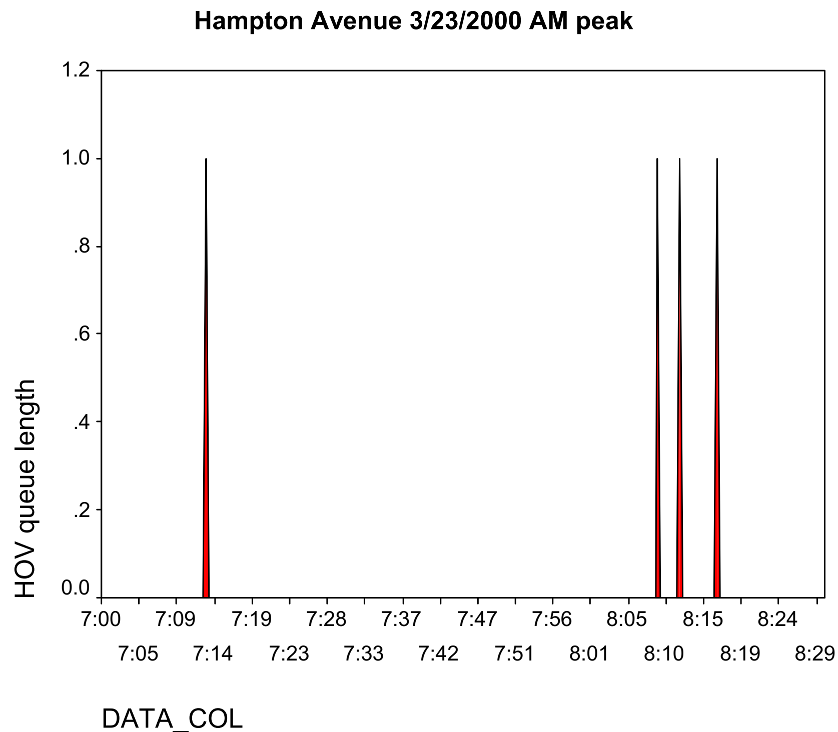
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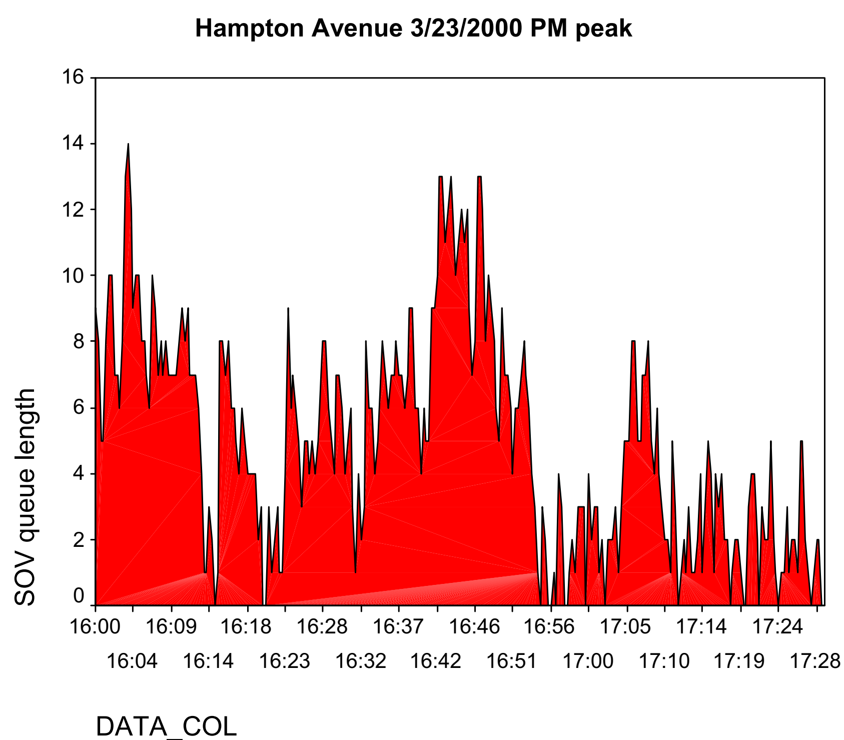
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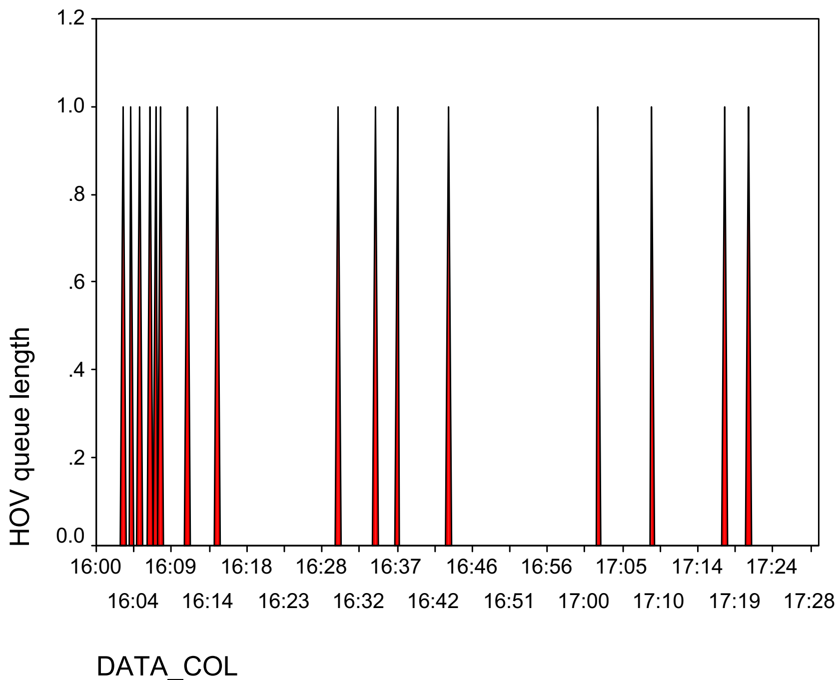
Hampton Avenue 3/23/2000 PM peak



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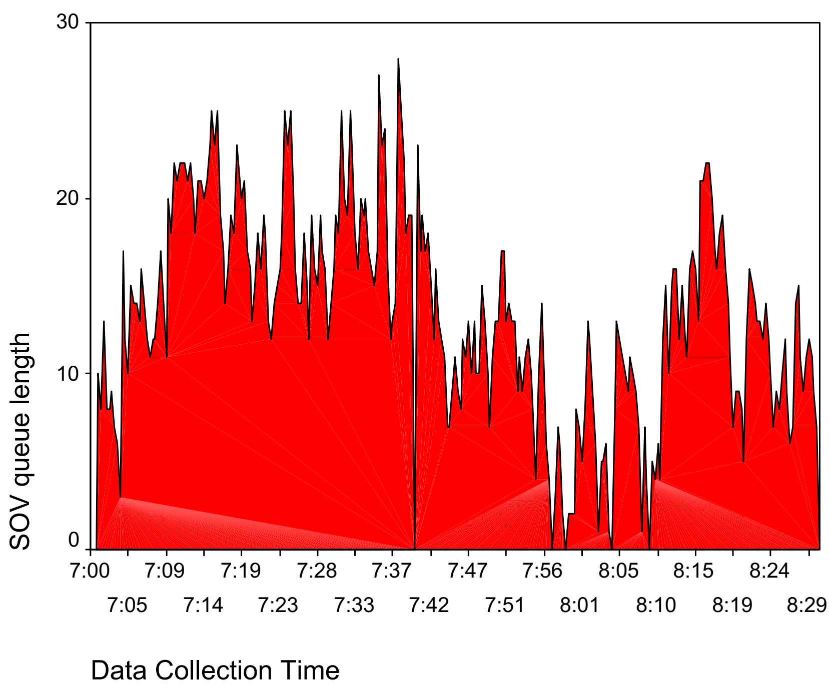

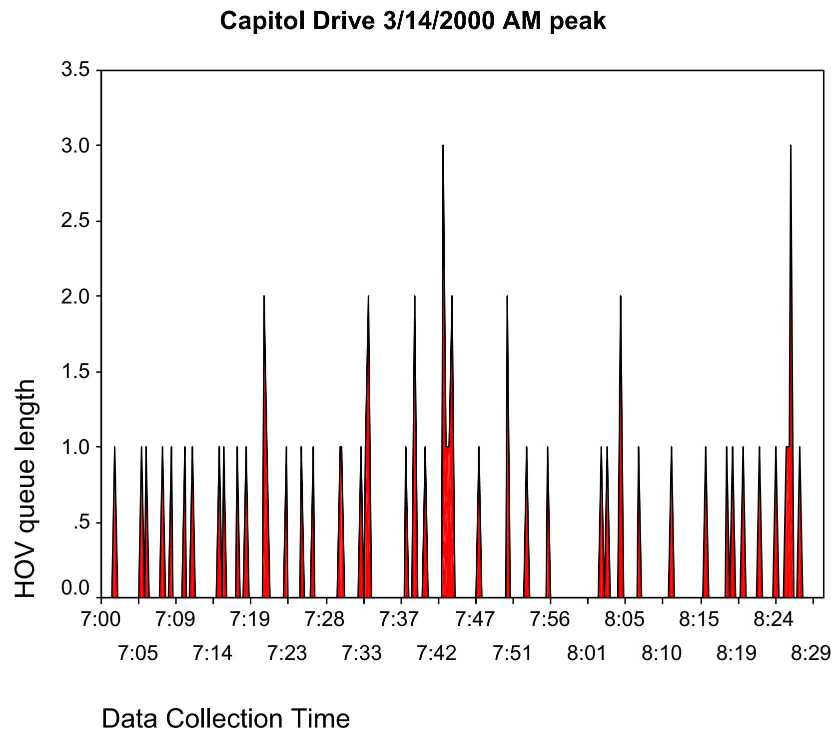
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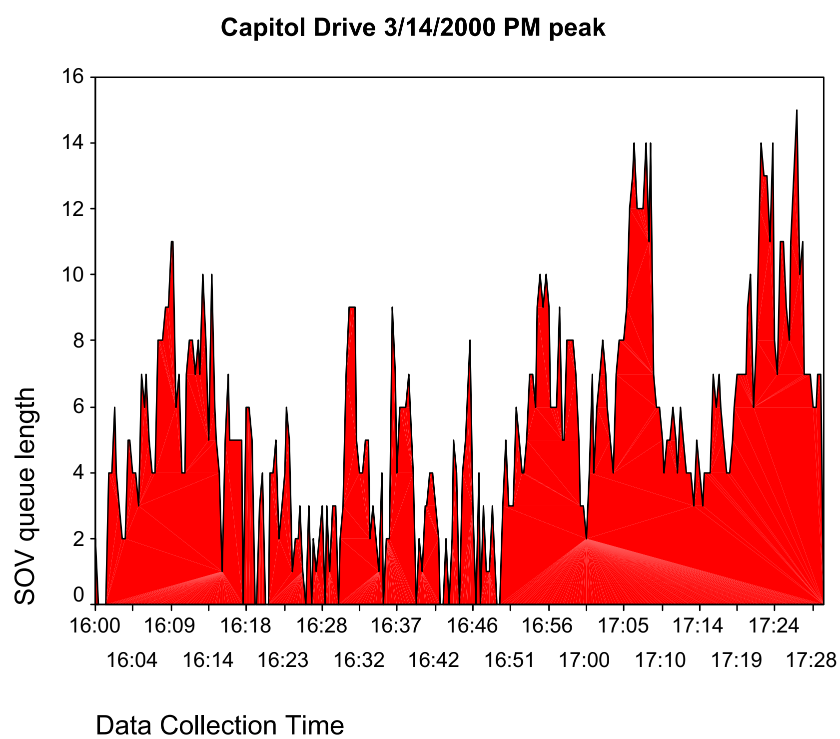
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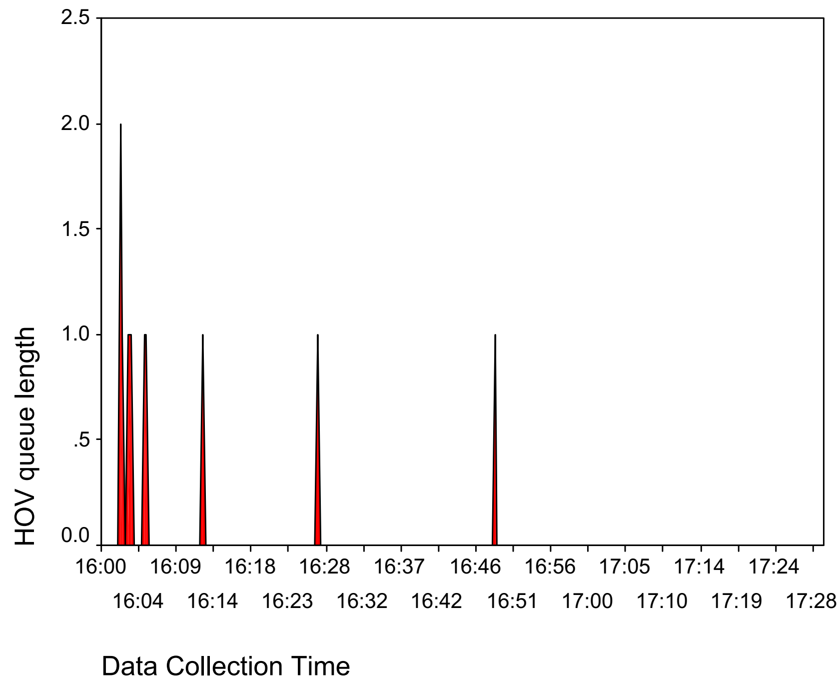
Data Collection Time

Capitol Drive 3/14/2000 PM peak



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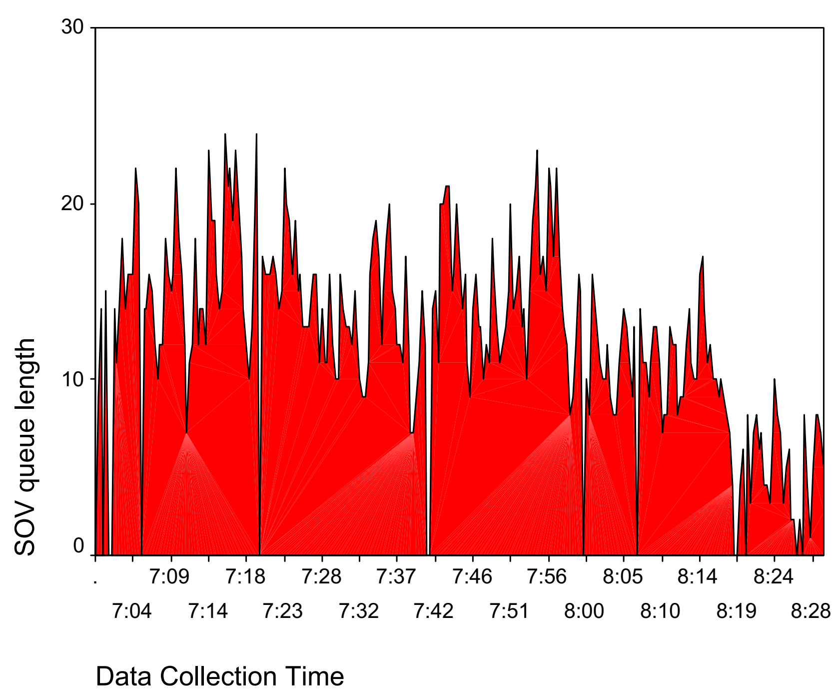

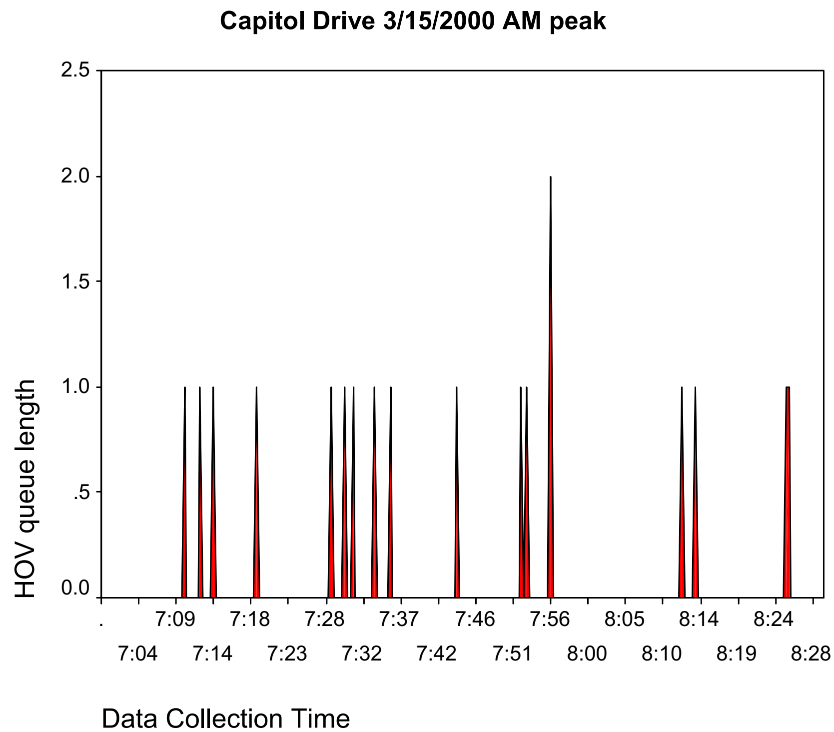
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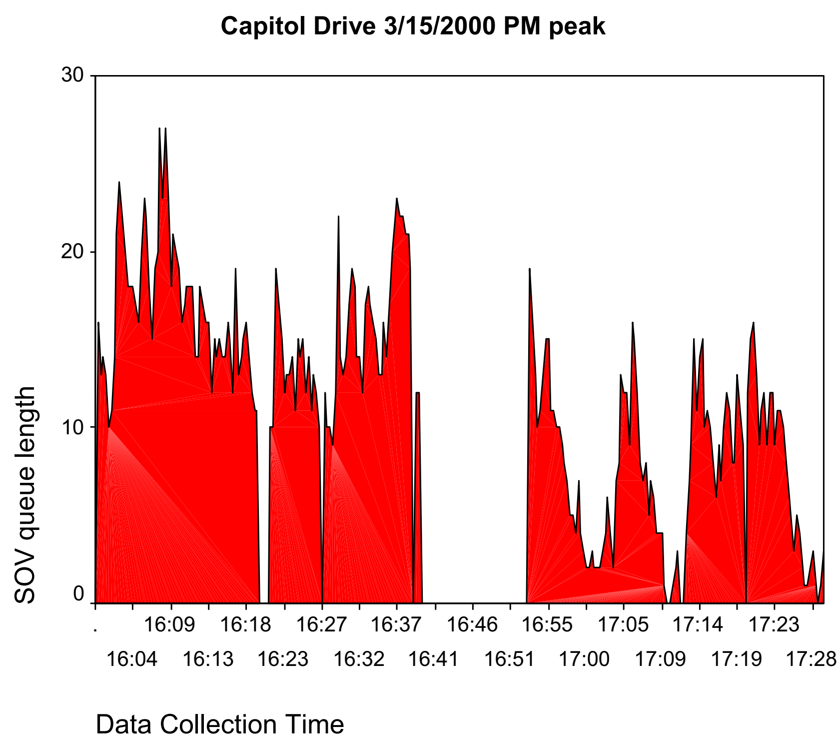
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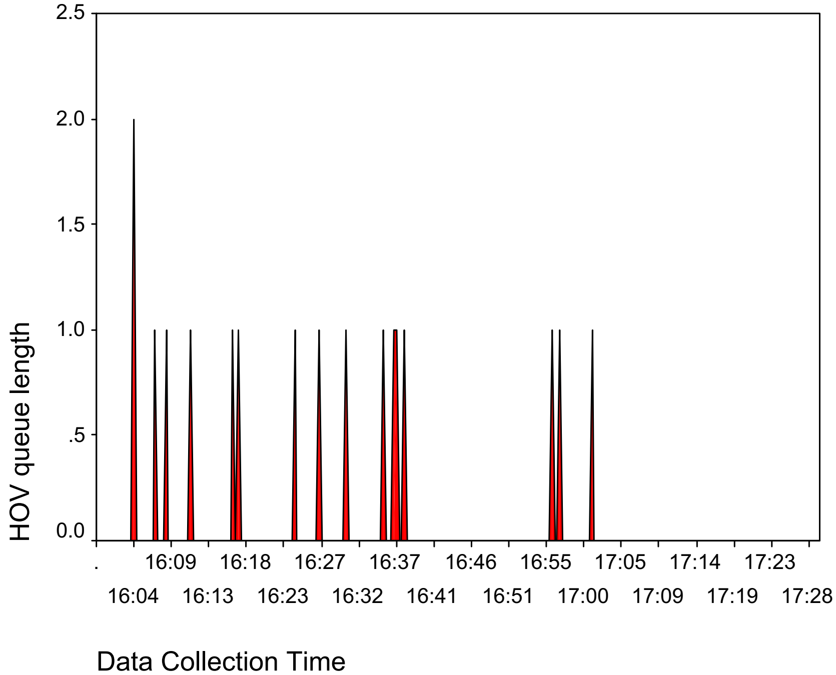

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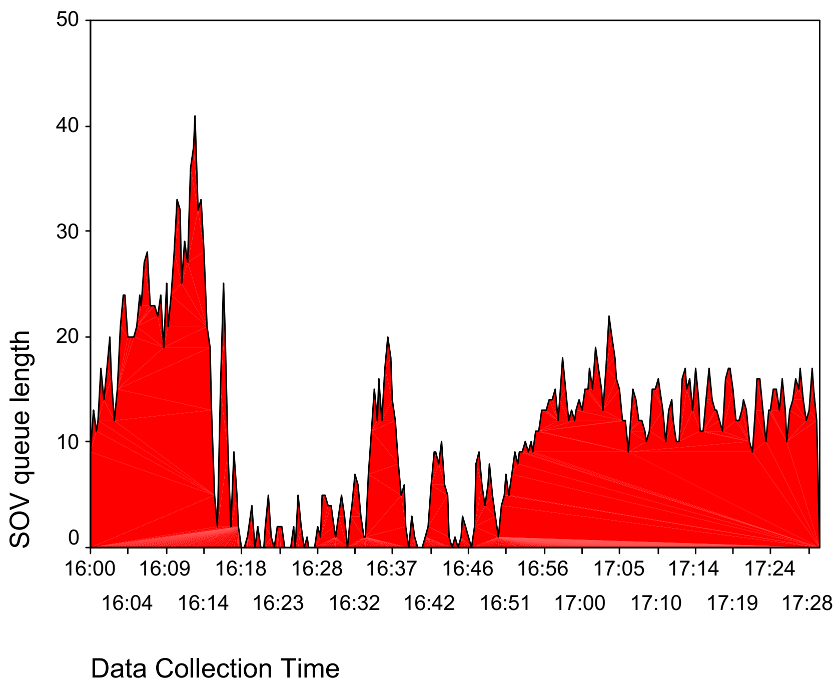

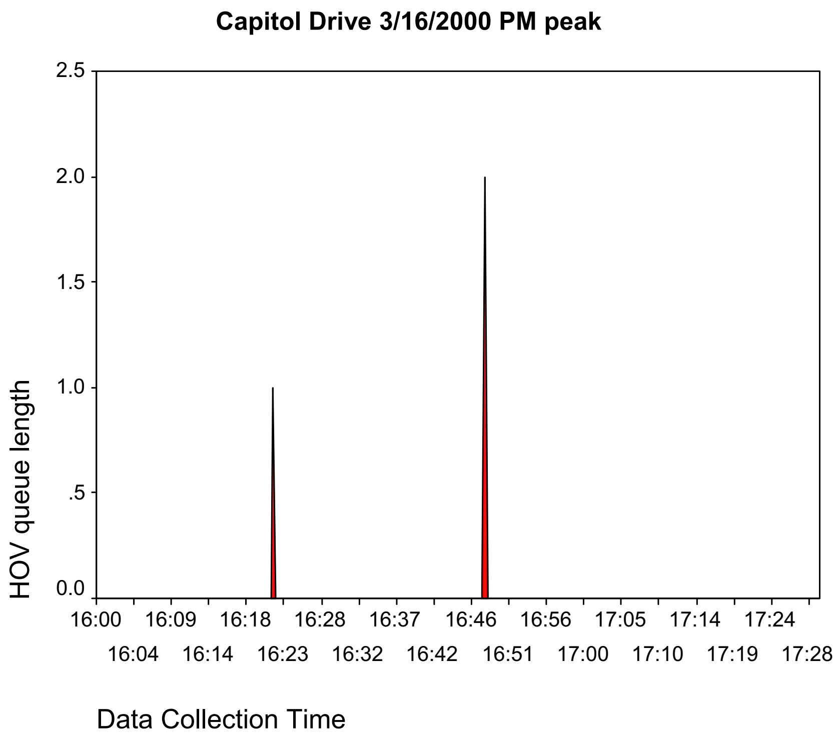
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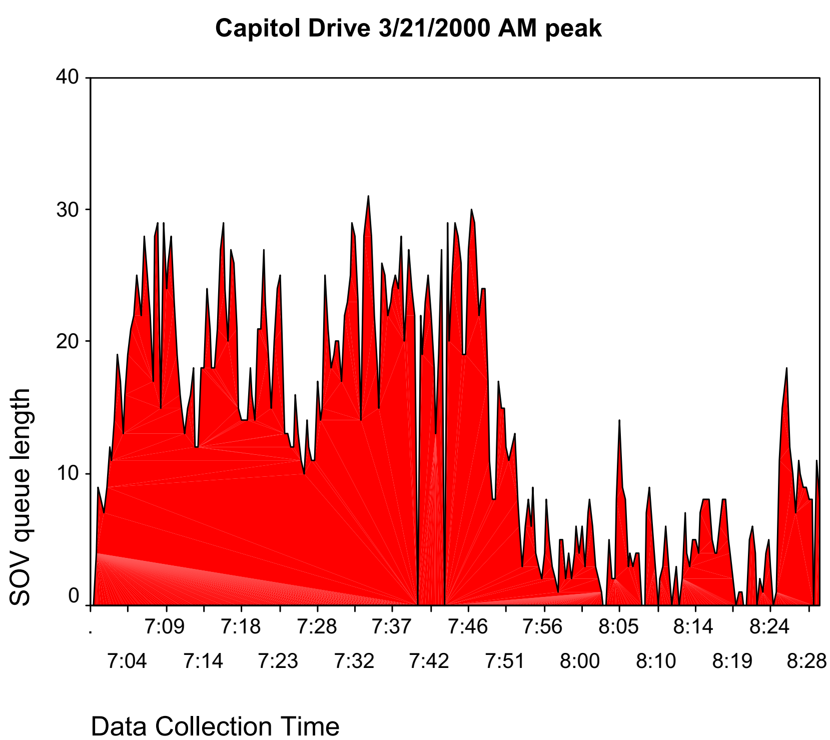
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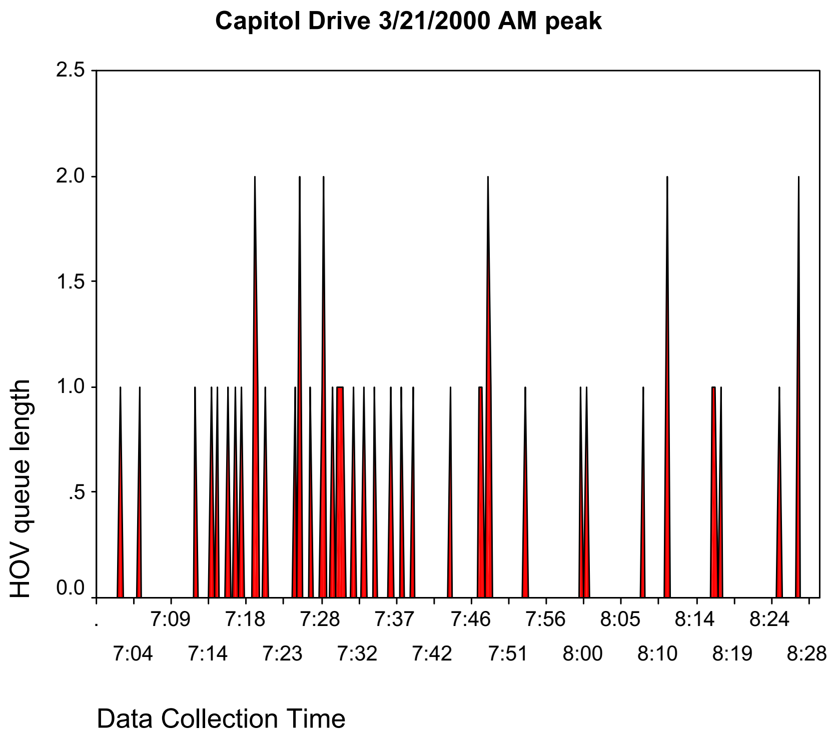
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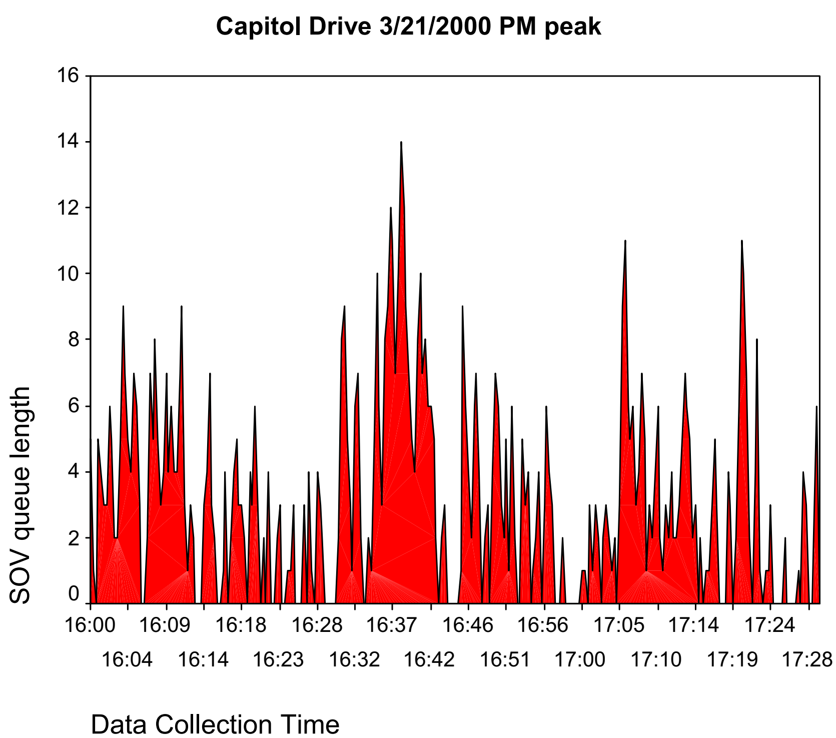
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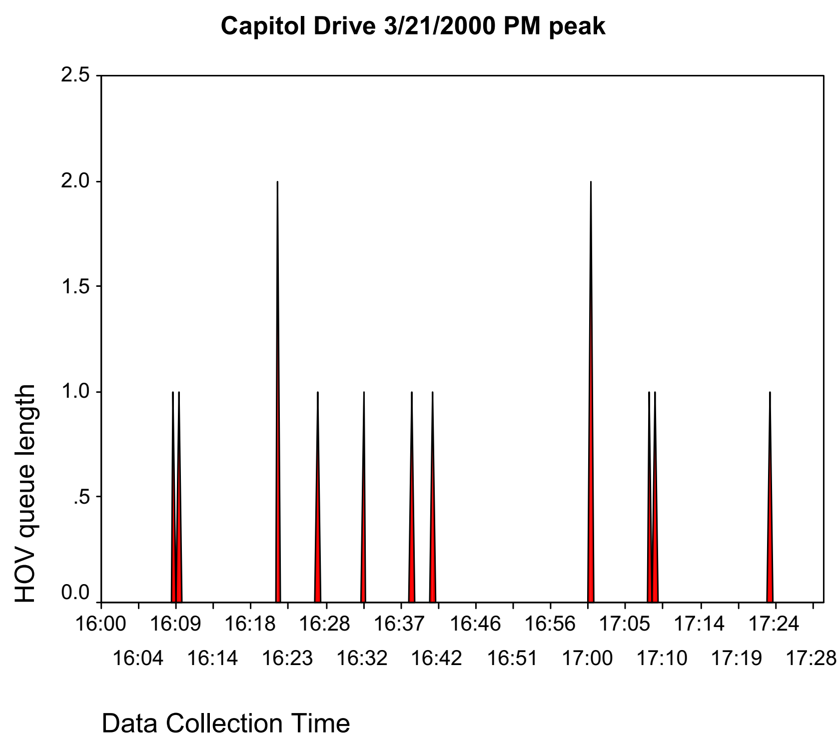
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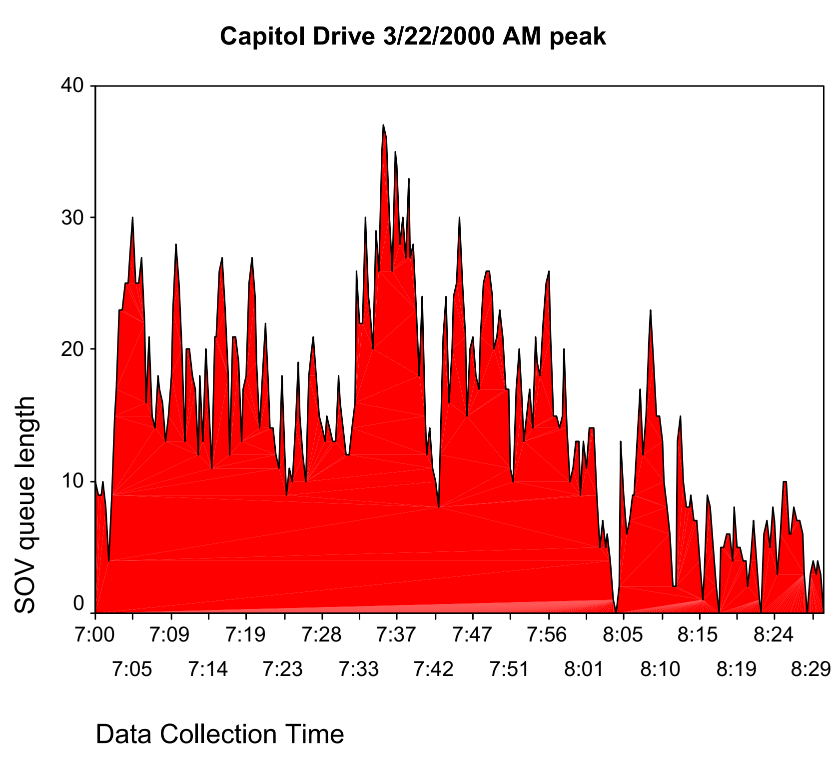
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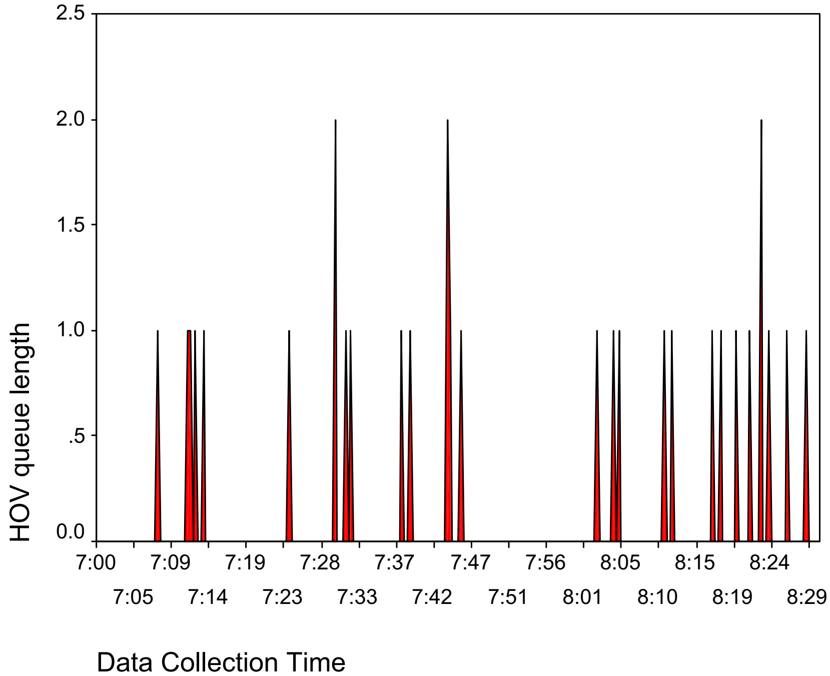

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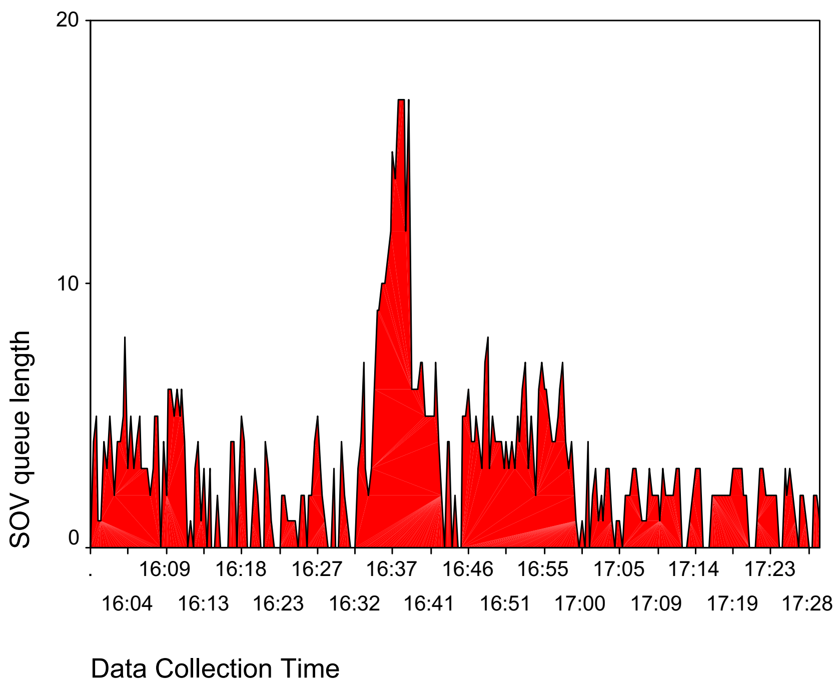

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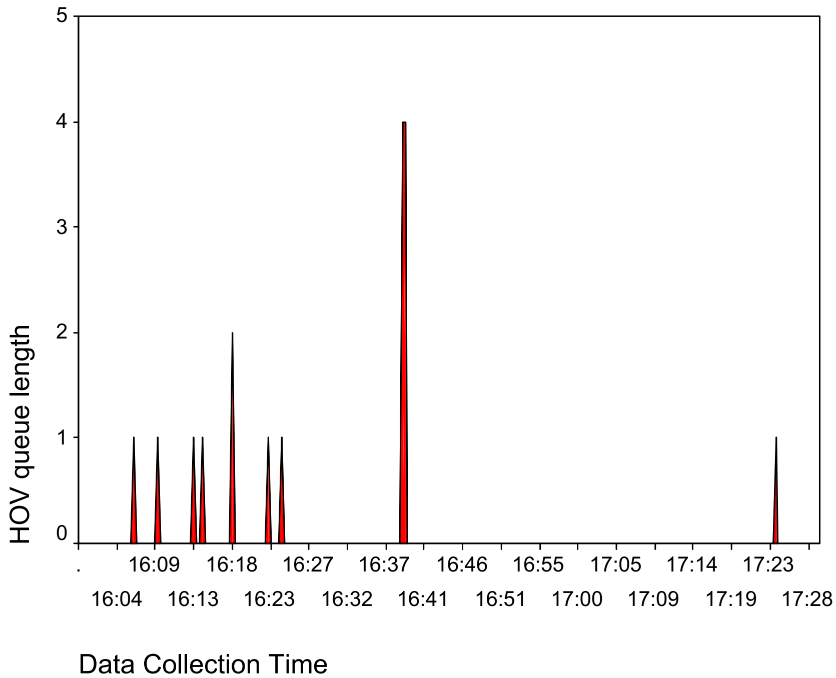

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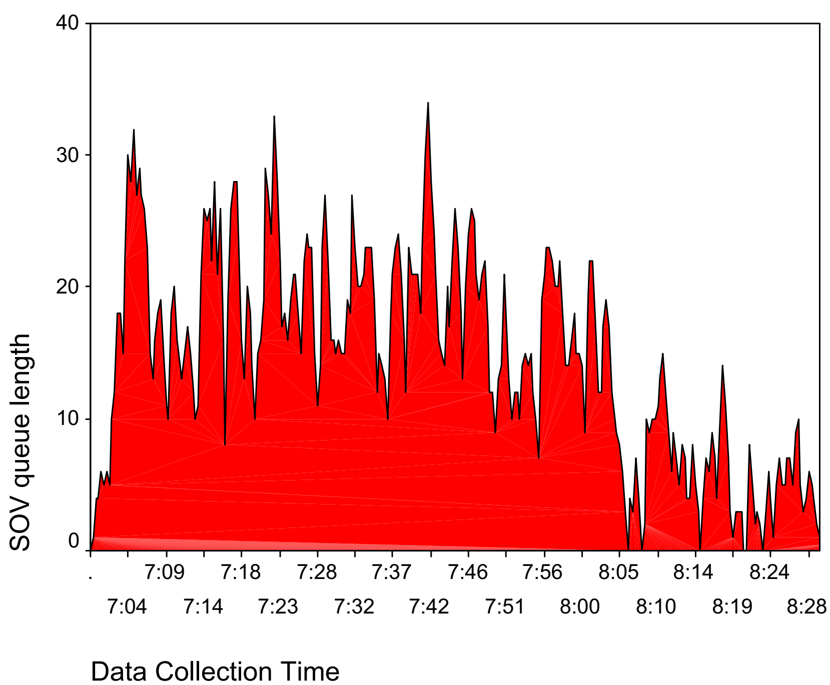

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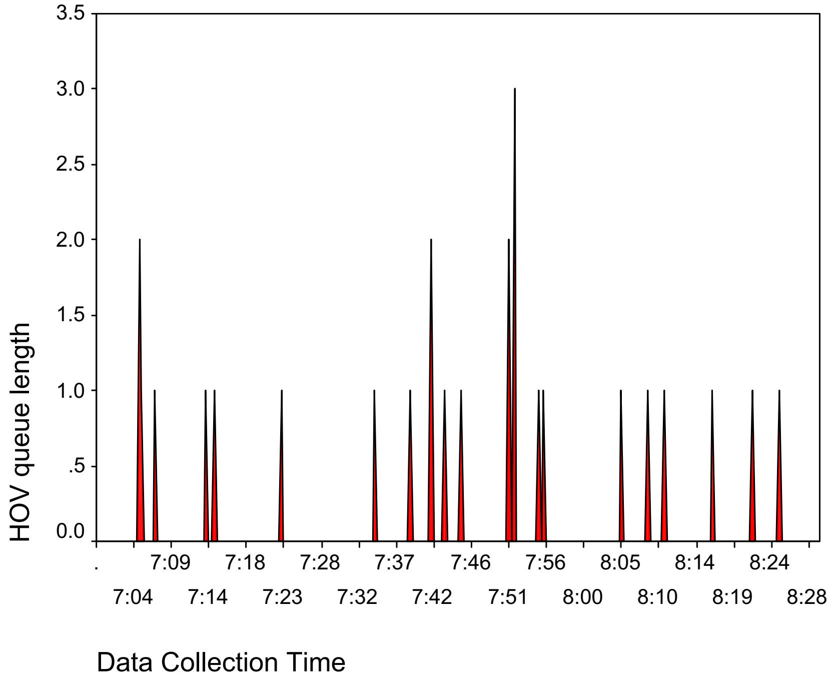

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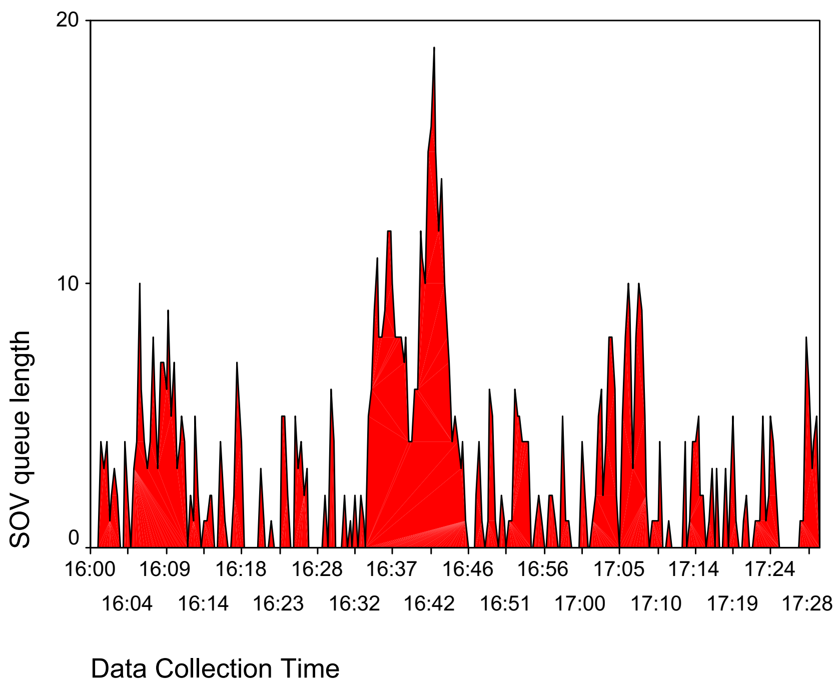

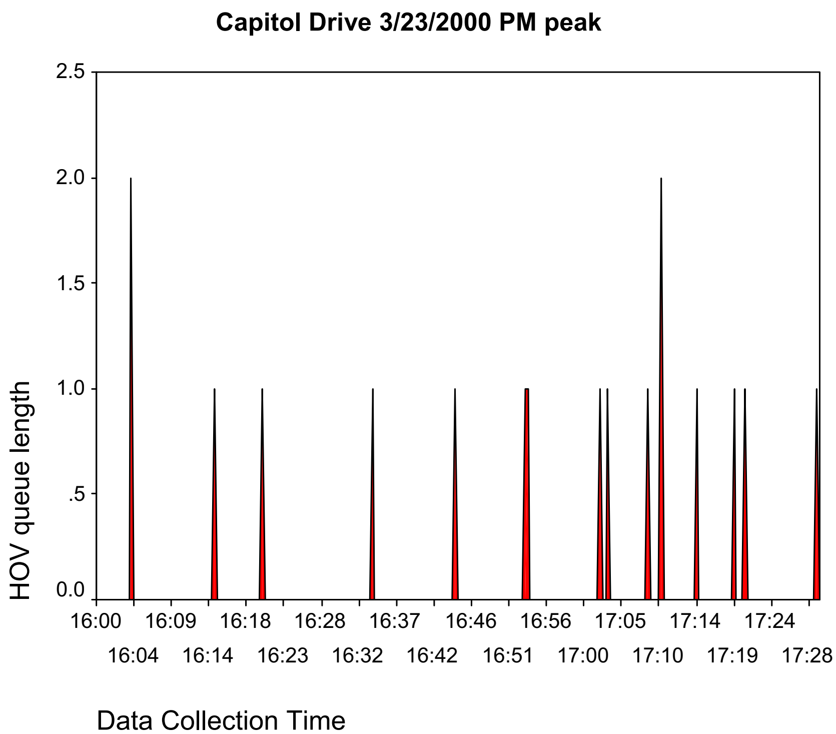
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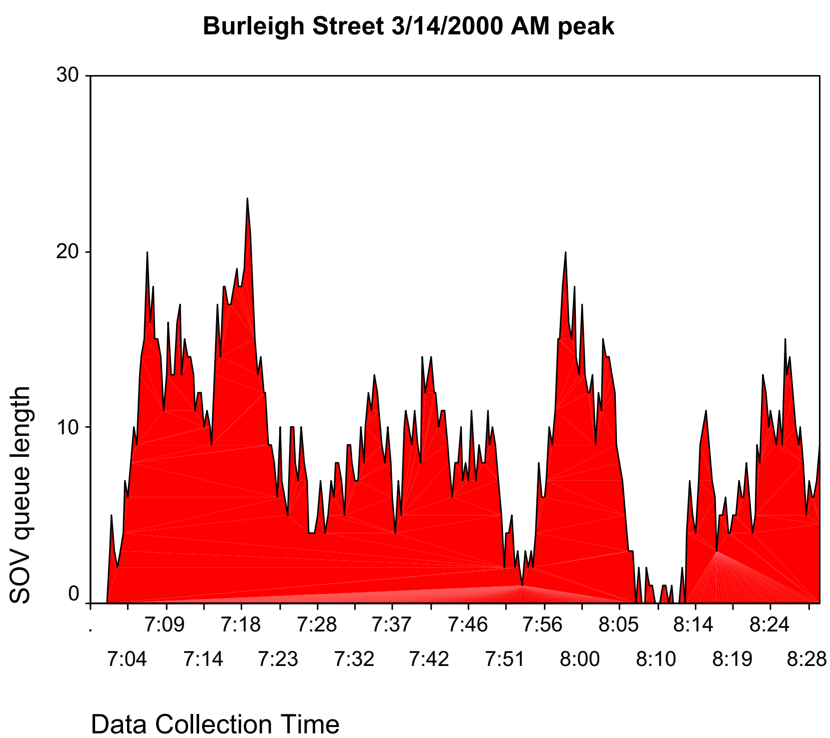
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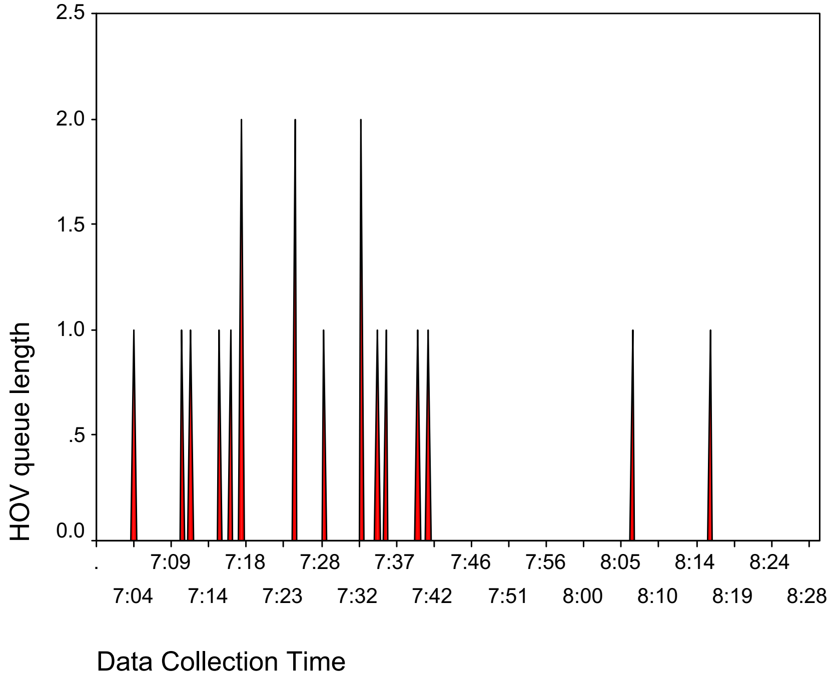

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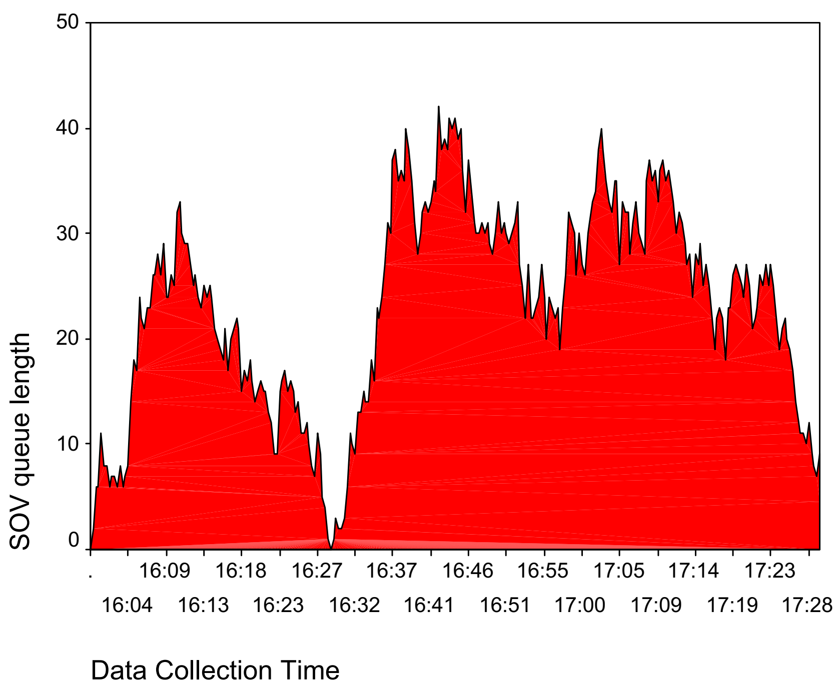

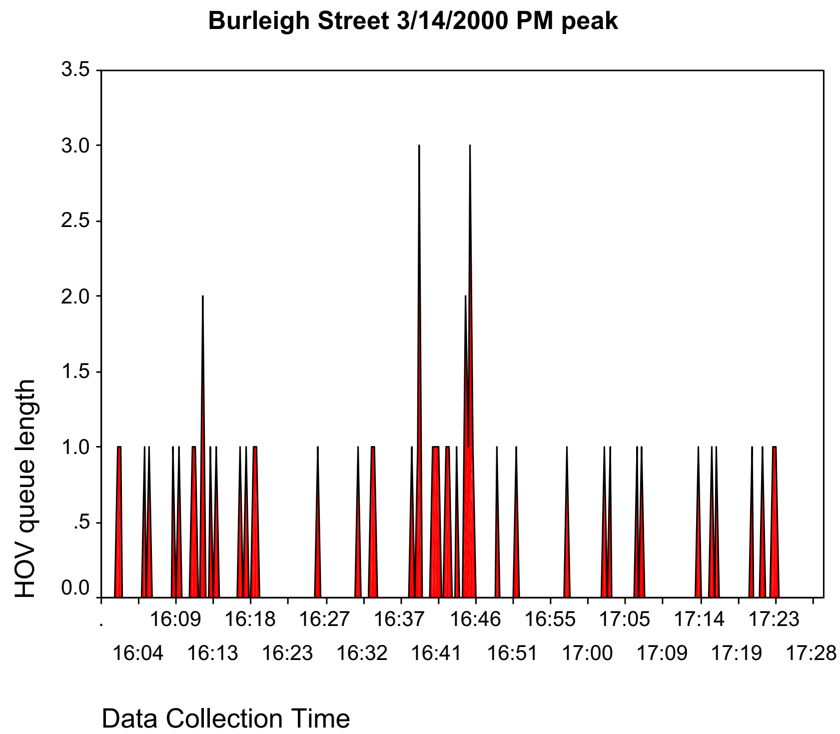
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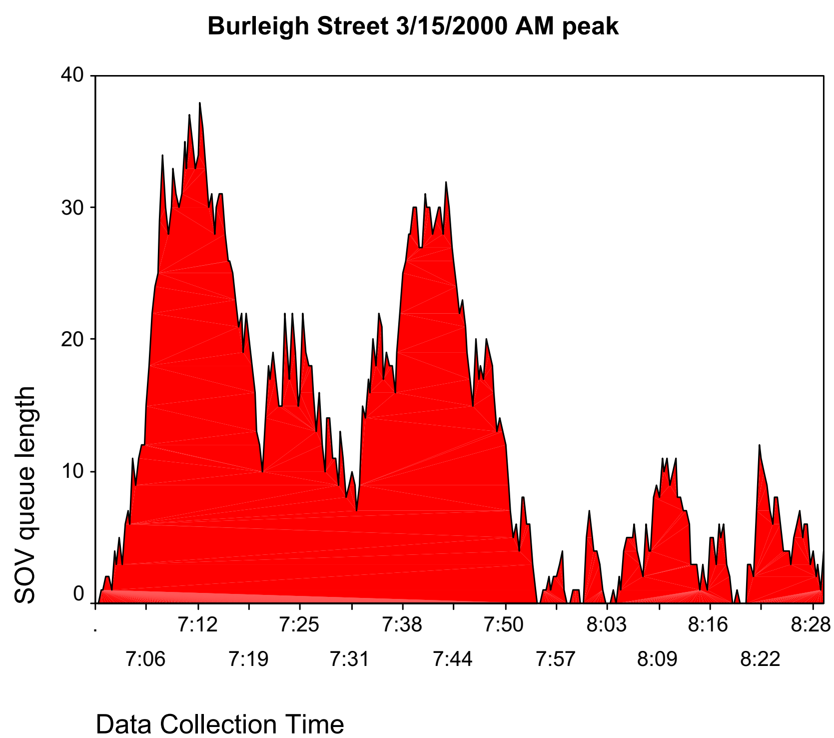
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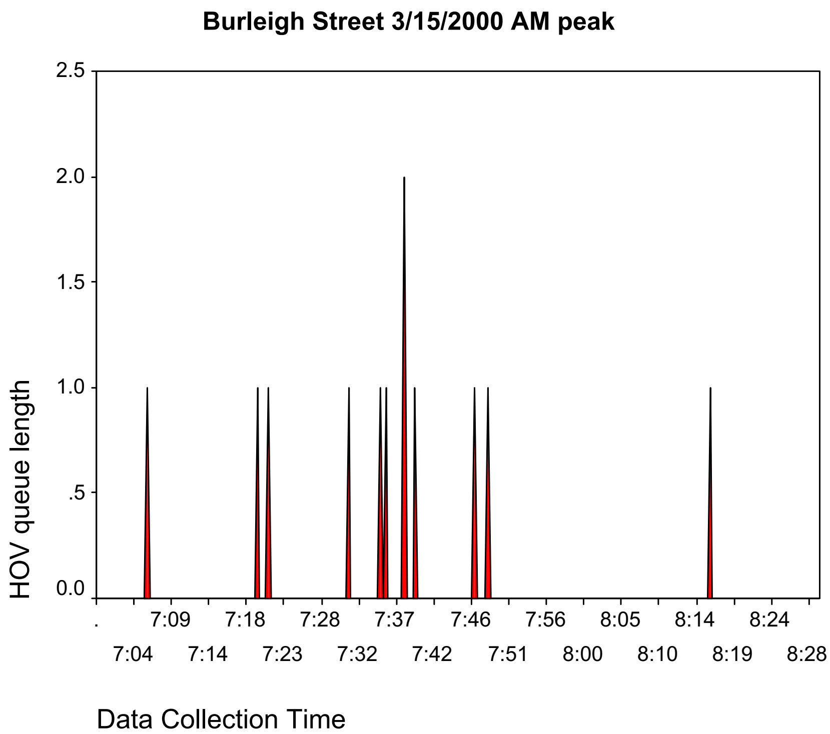
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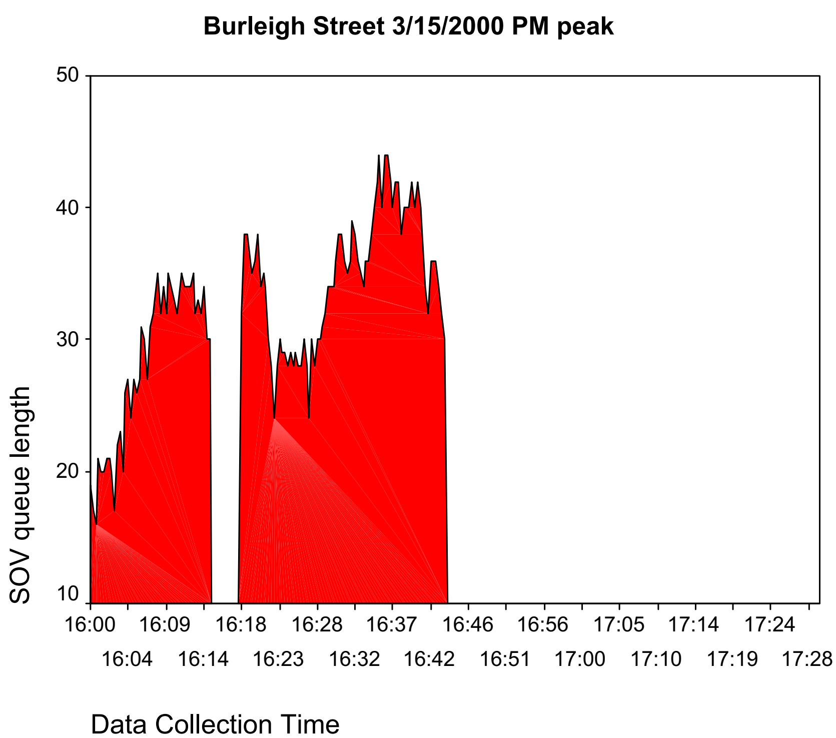
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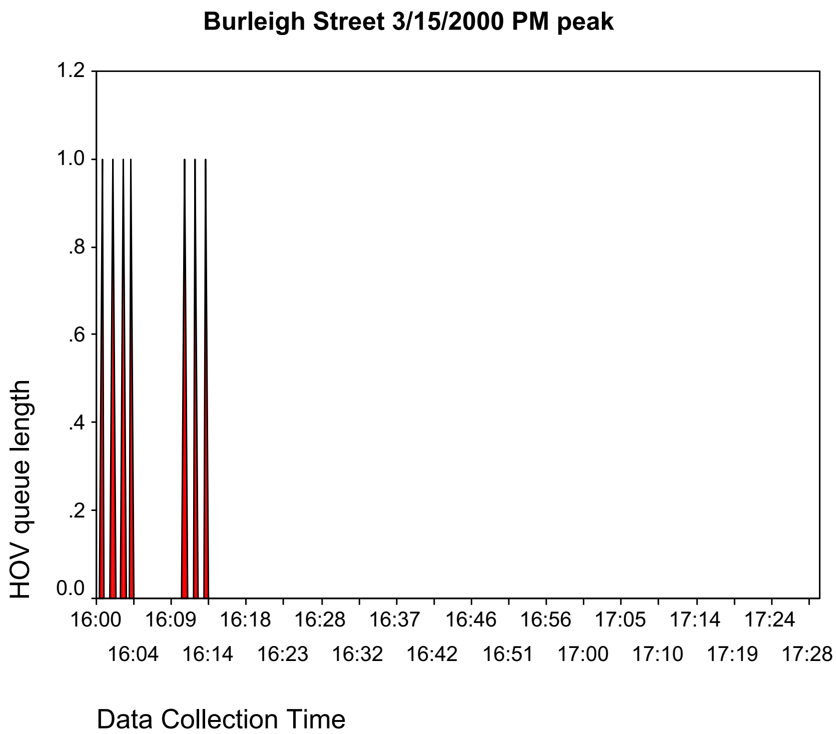
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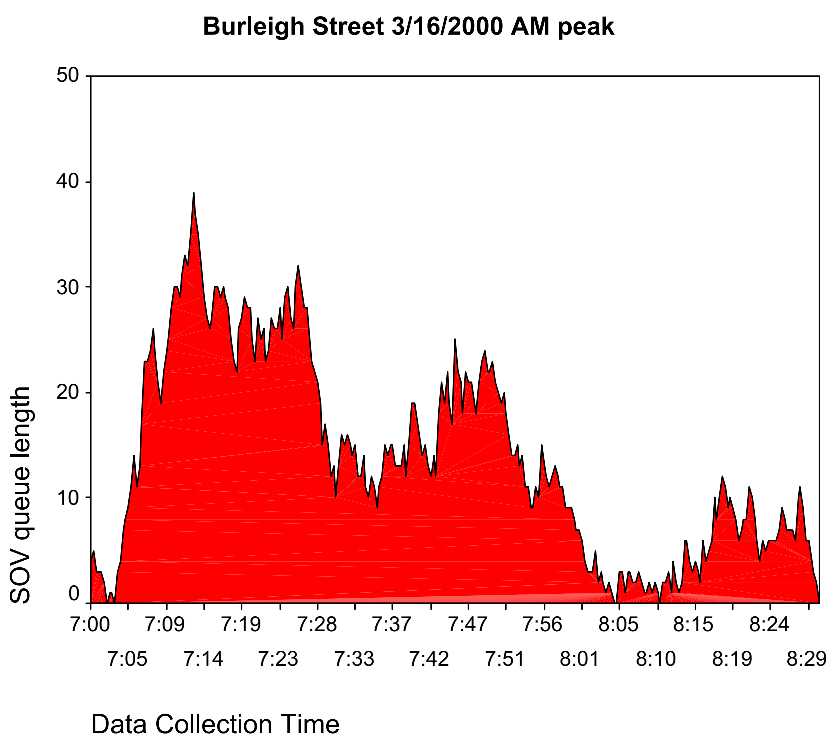
17:10
17:19
17:28
Data Collection Time


Burleigh Street 3/16/2000 AM peak
SOV queue length
50
40
30
20
10
0
7:00
7:09
7:19
7:28
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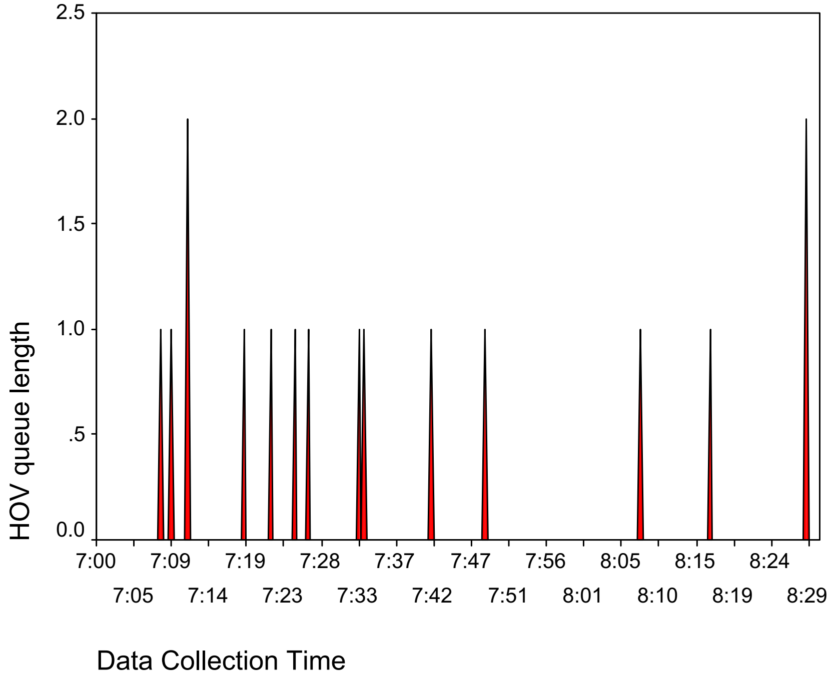
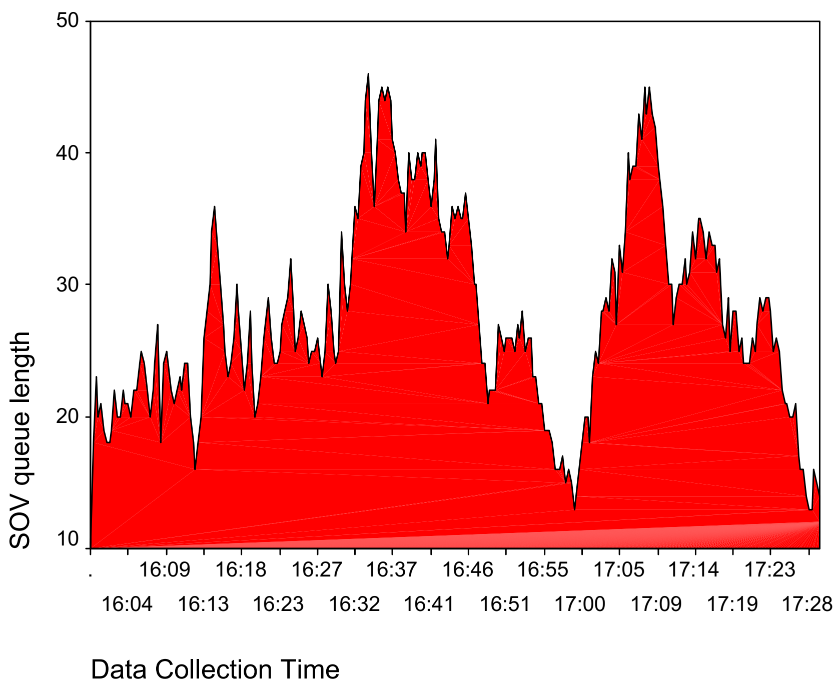
7:47
7:56
8:05
8:15
8:24
7:05
7:14
7:23
7:33
7:42
7:51
8:01
8:10
8:19
8:29
Data Collection Time

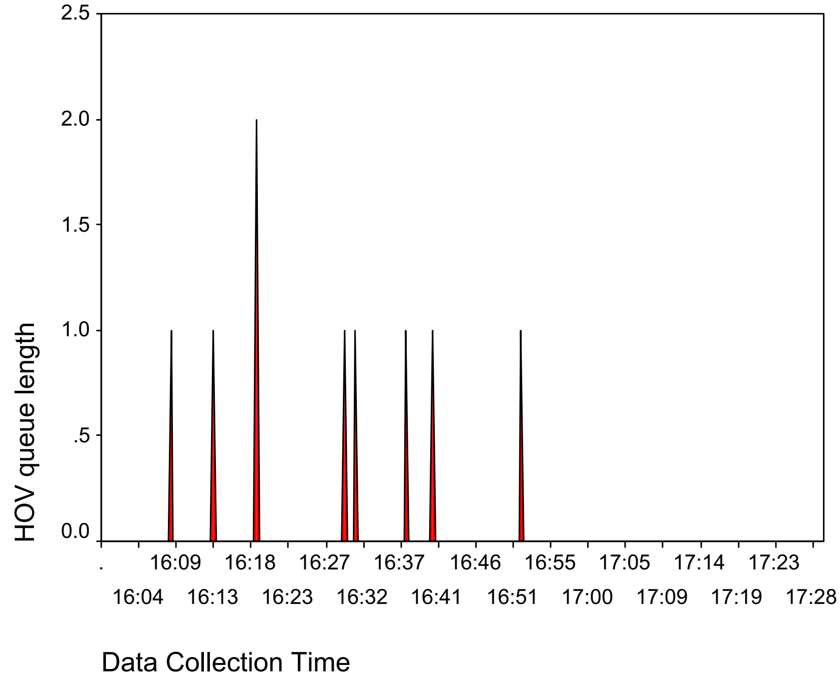
Burleigh Street 3/16/2000 AM peak



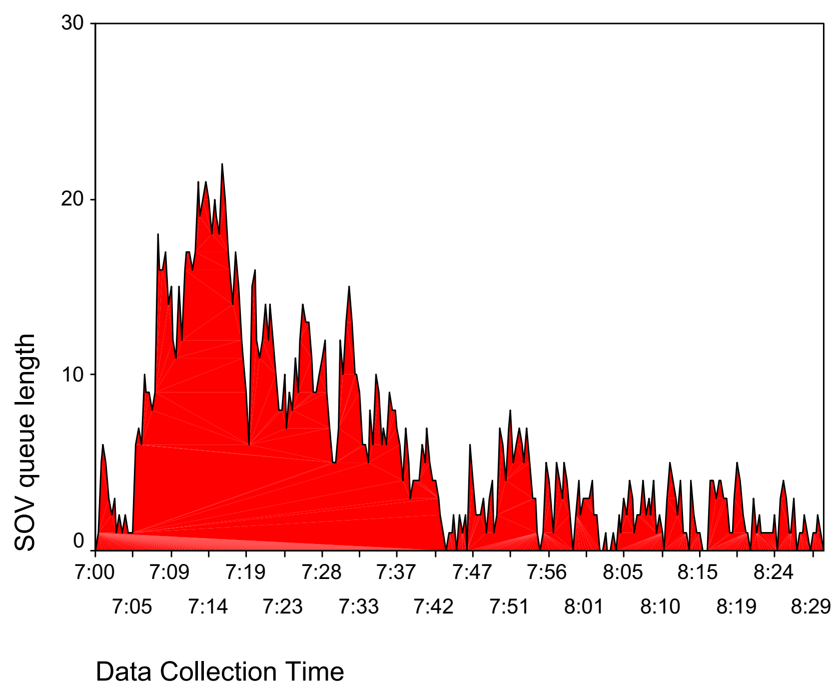
Burleigh Street 3/16/2000 PM peak

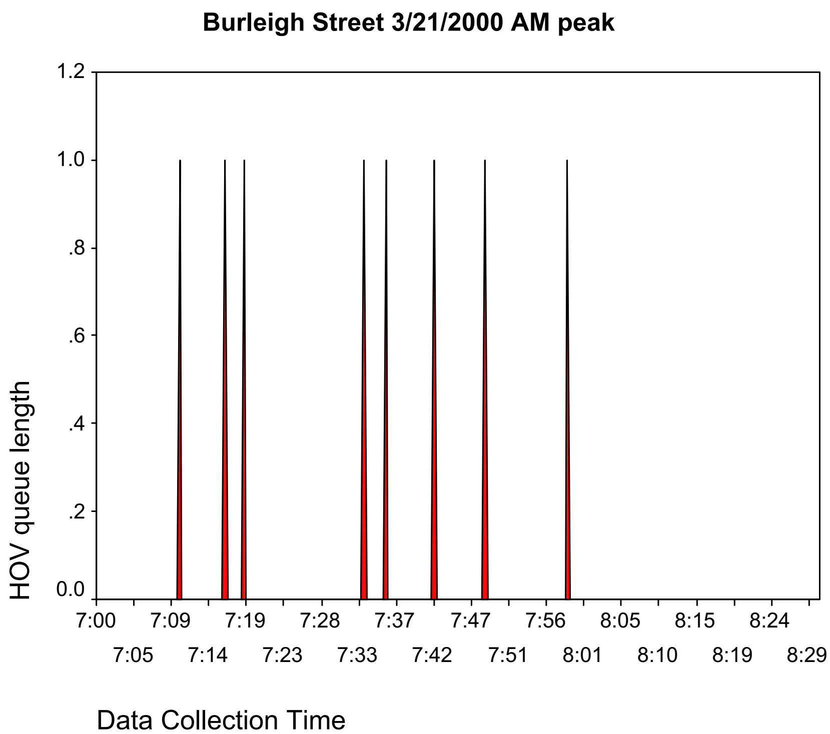


Burleigh Street 3/16/2000 PM peak



Burleigh Street 3/21/2000 AM peak




Burleigh Street 3/21/2000 AM peak
HOV queue length
1.2
1.0
.8
.6
.4
.2
0.0
7:00
7:05
7:09
7:14
7:19
7:23
7:28
7:33
7:37
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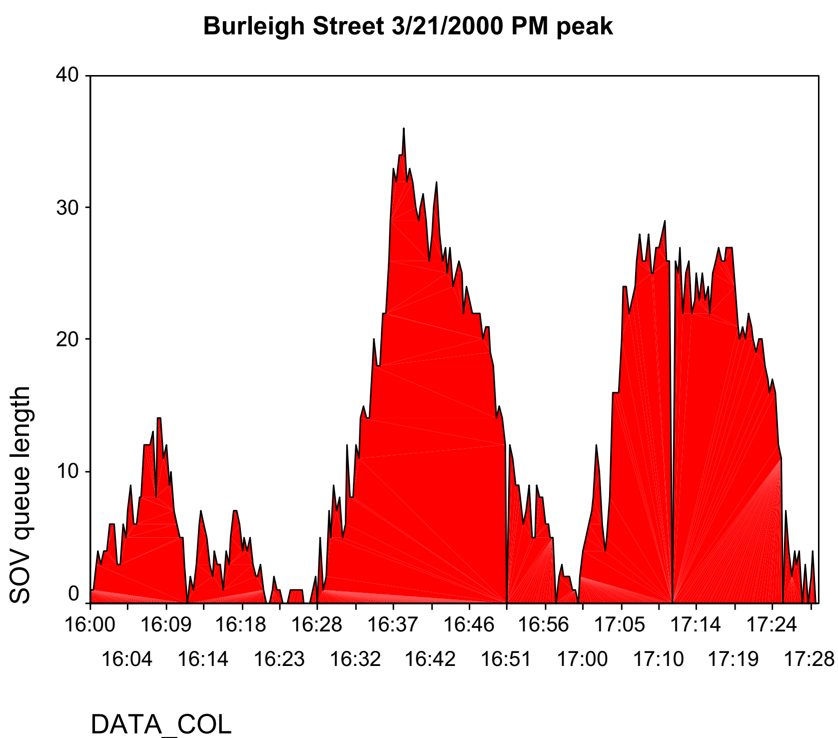
7:56
8:01
8:05
8:10
8:15
8:19
8:24
8:29
Data Collection Time


Burleigh Street 3/21/2000 PM peak
SOV queue length
40
30
20
10
0
16:00
16:04
16:09
16:14
16:18
16:23
16:28
16:32
16:37
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16:56
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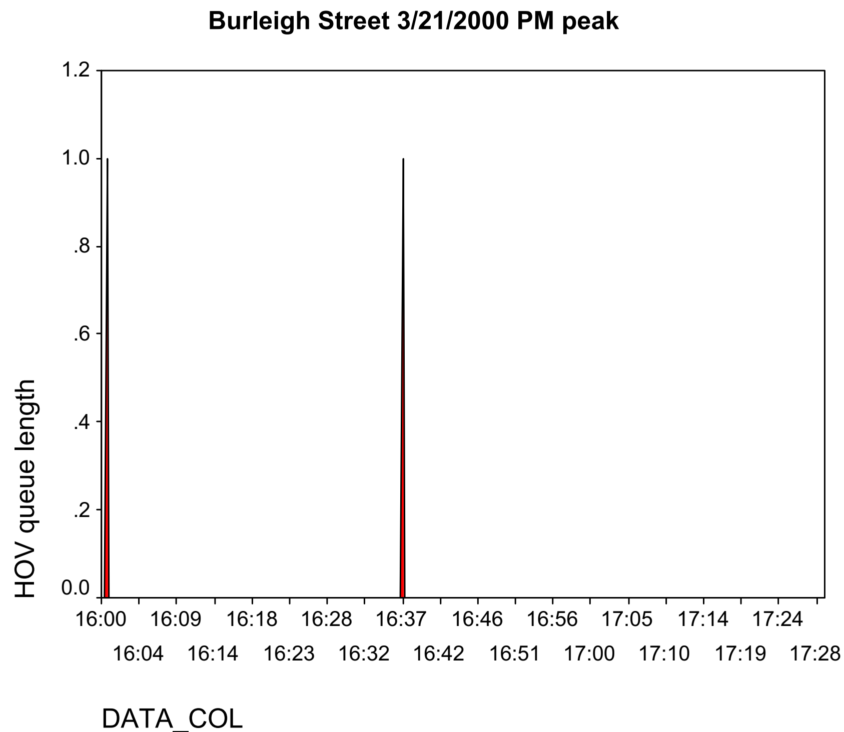
17:24
17:28
DATA_COL


Burleigh Street 3/21/2000 PM peak
HOV queue length
1.2
1.0
.8
.6
.4
.2
0.0
16:00
16:04
16:09
16:14
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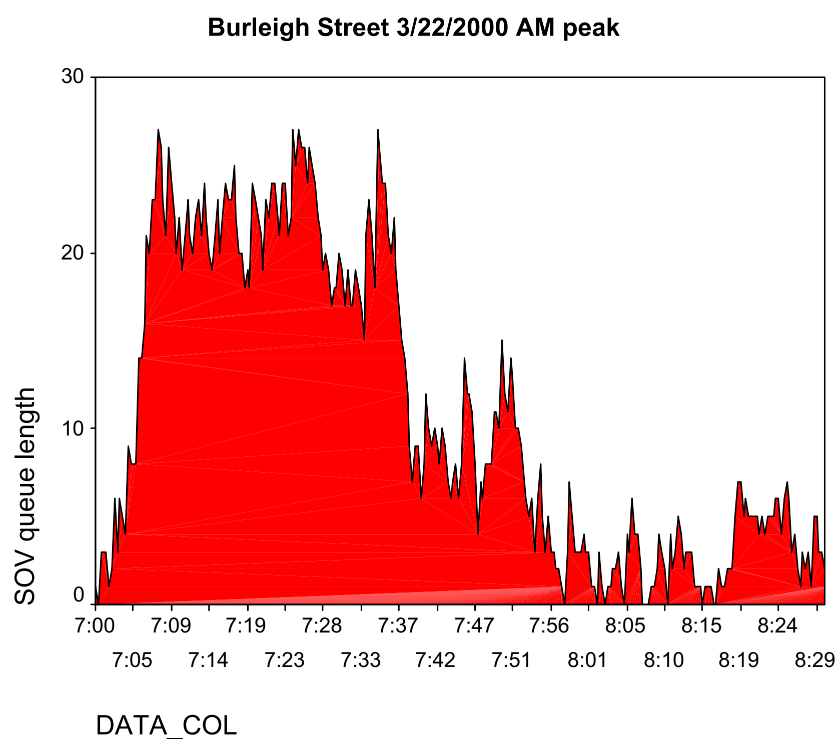
DATA_COL


Burleigh Street 3/22/2000 AM peak
SOV queue length
30
20
10
0
7:00
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7:14
7:19
7:23
7:28
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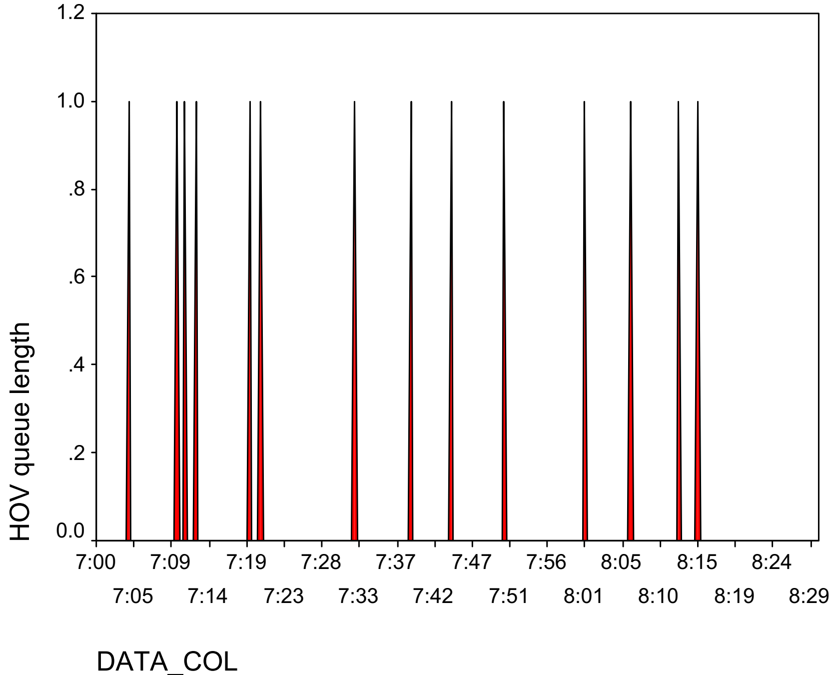
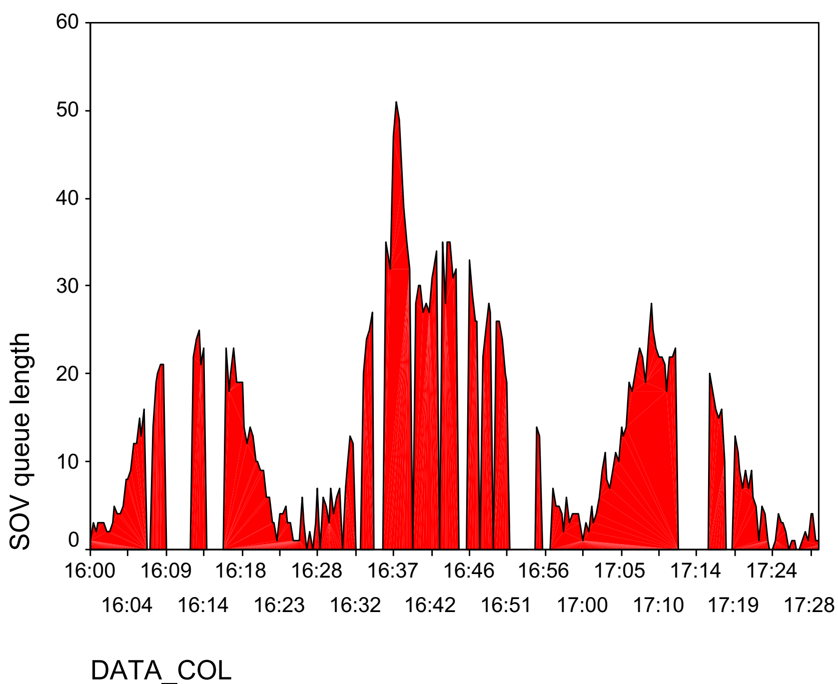
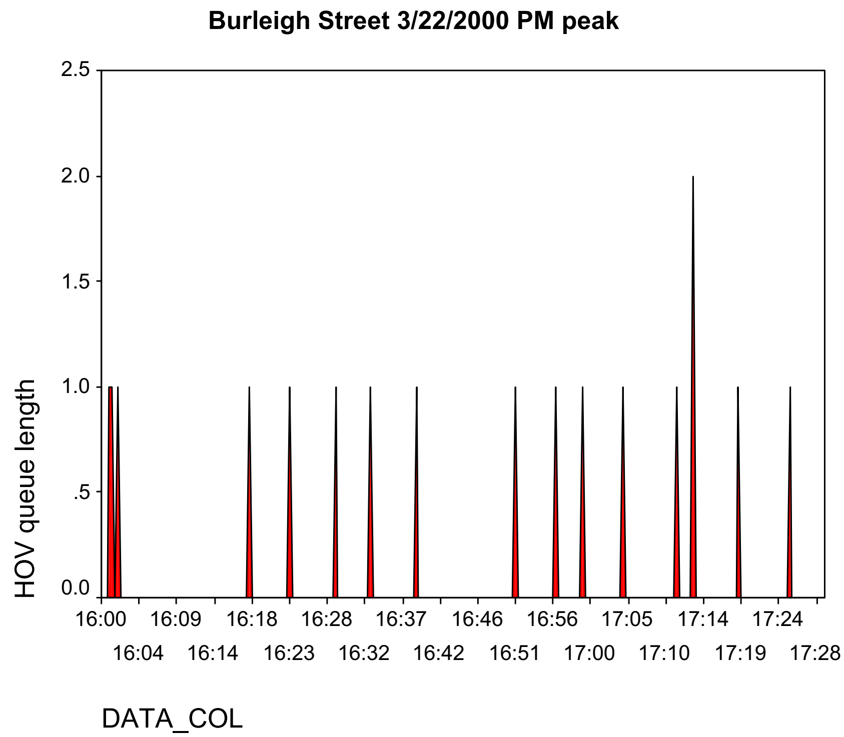
DATA_COL

Burleigh Street 3/22/2000 AM peak



Burleigh Street 3/22/2000 PM peak




Burleigh Street 3/22/2000 PM peak
HOV queue length
2.5
2.0
1.5
1.0
.5
0.0
16:00
16:09
16:18
16:28
16:37
16:46
16:56
17:05
17:14
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16:23
16:32
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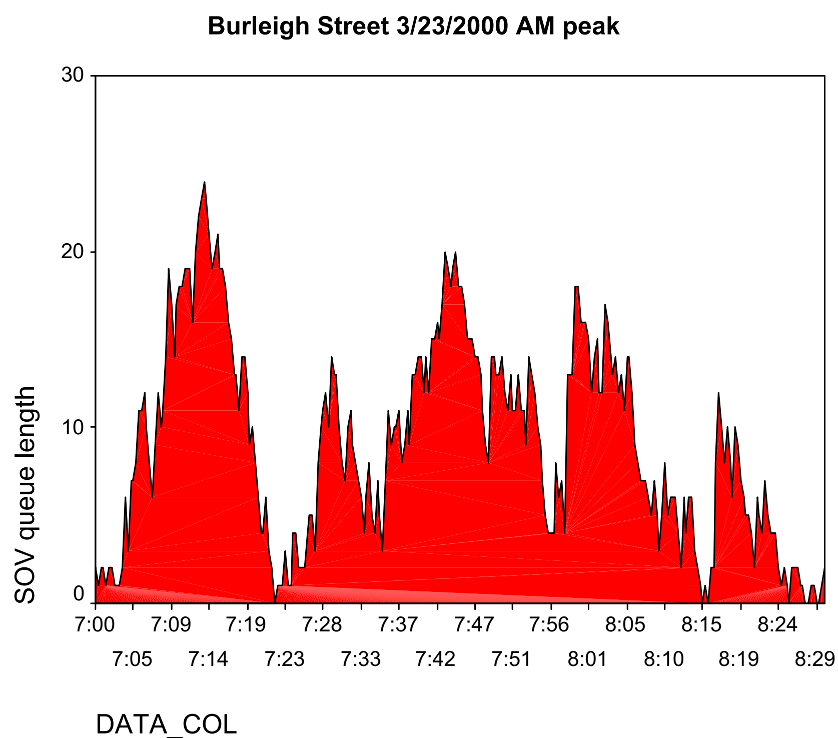
17:00
17:10
17:19
17:28
DATA_COL


Burleigh Street 3/23/2000 AM peak
SOV queue length
30
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10
0
7:00
7:09
7:19
7:28
7:37
7:47
7:56
8:05
8:15
8:24
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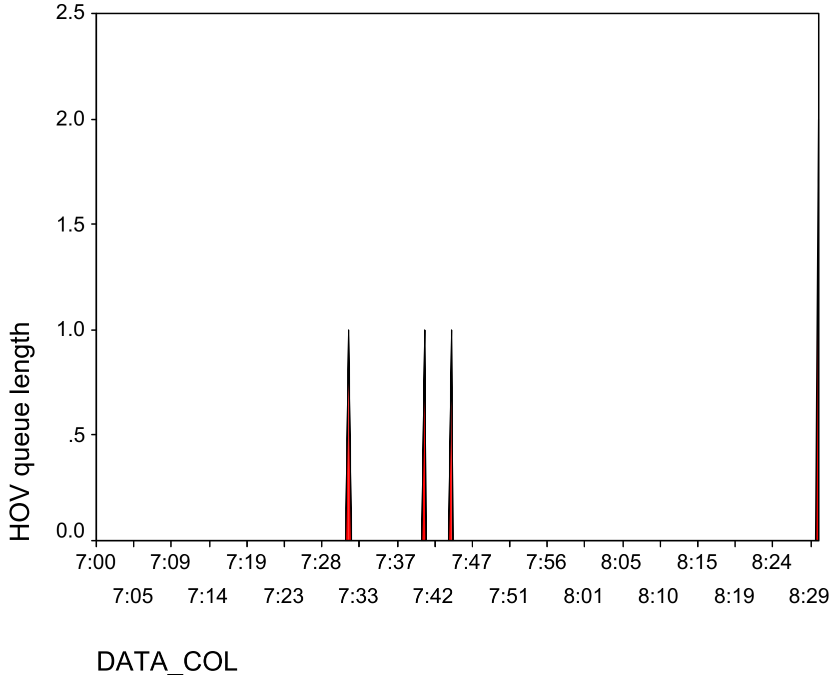
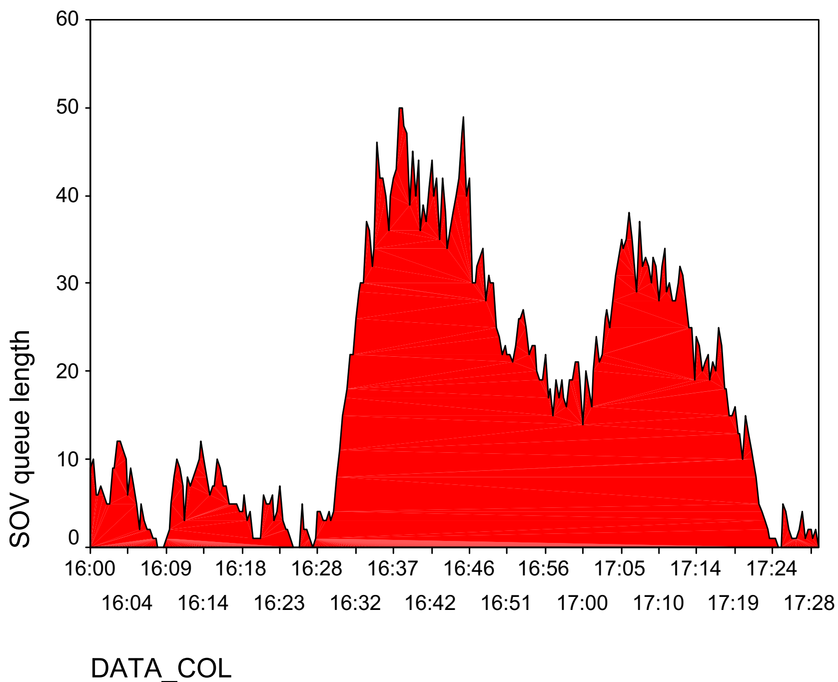
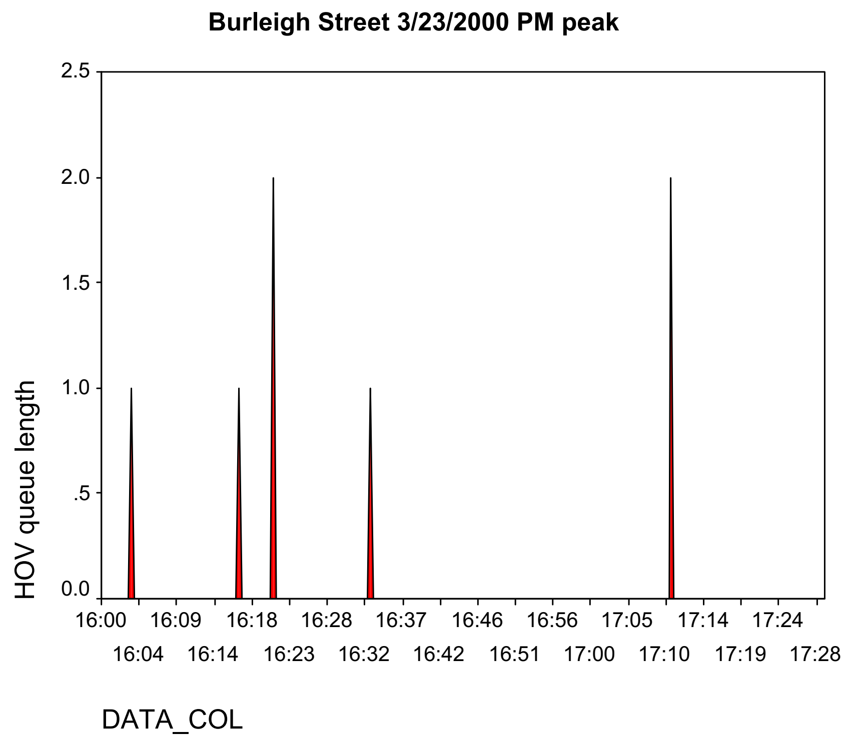
8:01
8:10
8:19
8:29
DATA_COL

Burleigh Street 3/23/2000 AM peak



Burleigh Street 3/23/2000 PM peak




Burleigh Street 3/23/2000 PM peak
2.5
2.0
1.5
1.0
.5
0.0
HOV queue length
16:00
16:09
16:18
16:28
16:37
16:46
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17:05
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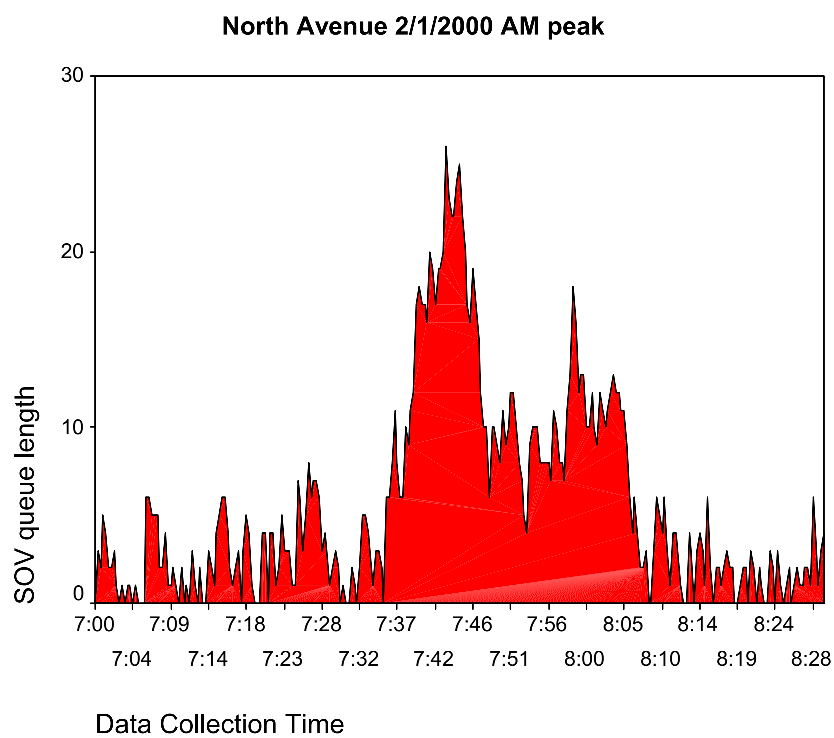
17:28
DATA_COL


North Avenue 2/1/2000 AM peak
30
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SOV queue length
7:00
7:09
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8:05
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8:24
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7:32
7:42
7:51
8:00
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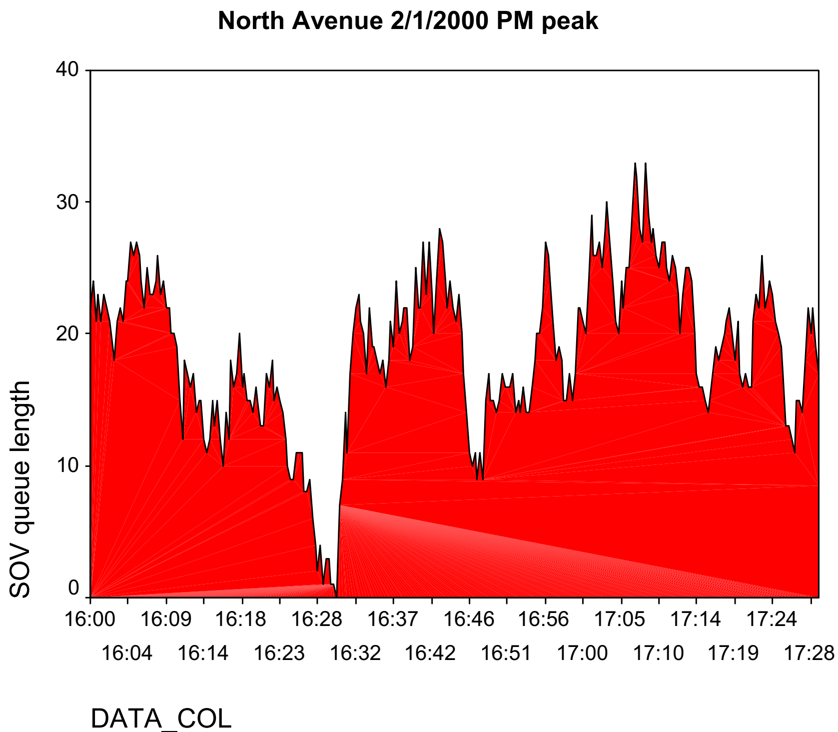
8:19
8:28
Data Collection Time


North Avenue 2/1/2000 PM peak
SOV queue length
0
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40
16:00
16:04
16:09
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16:23
16:28
16:32
16:37
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17:14
17:19
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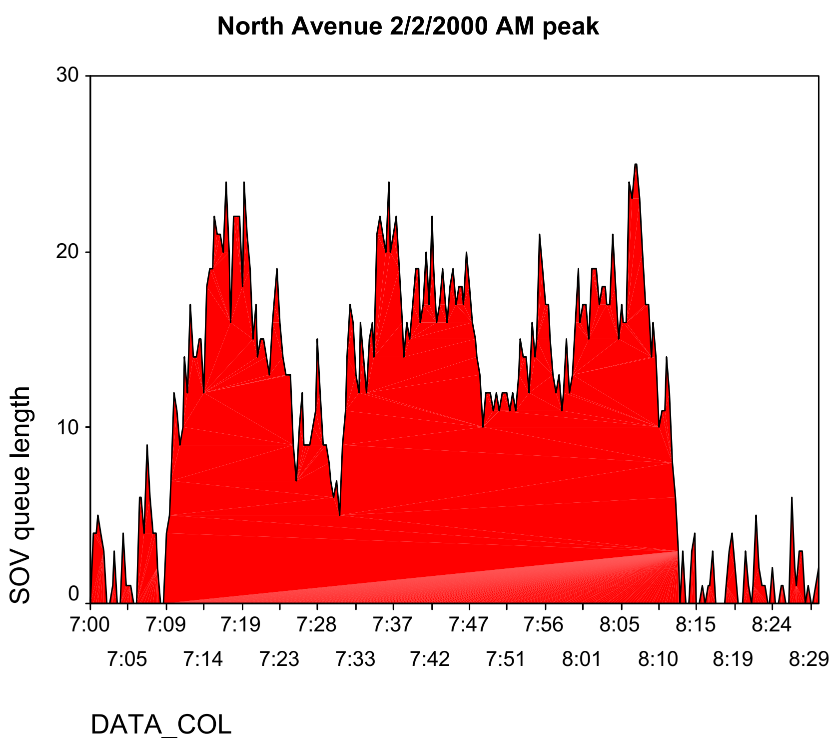
17:28
DATA_COL


North Avenue 2/2/2000 AM peak
SOV queue length
0
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20
30
7:00
7:05
7:09
7:14
7:19
7:23
7:28
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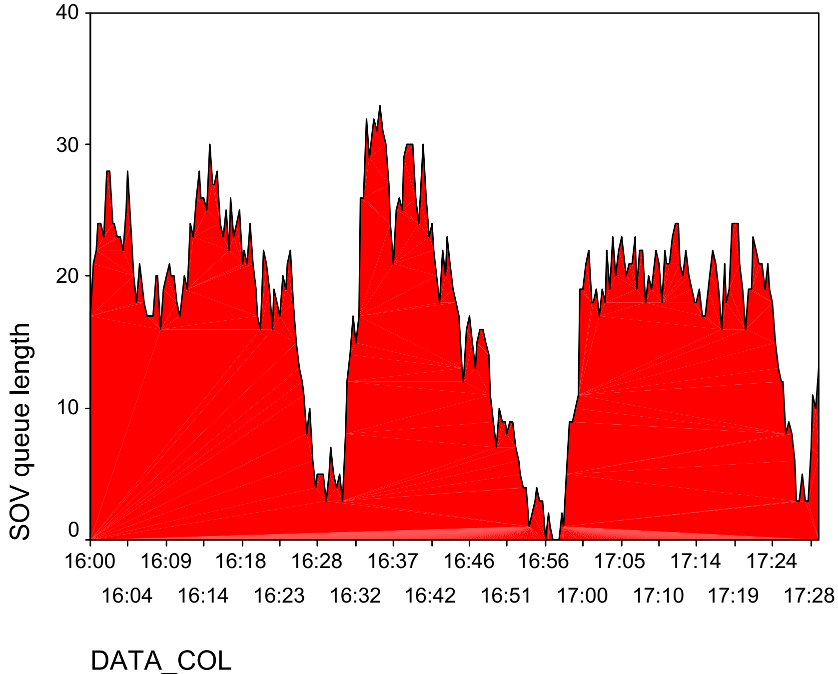
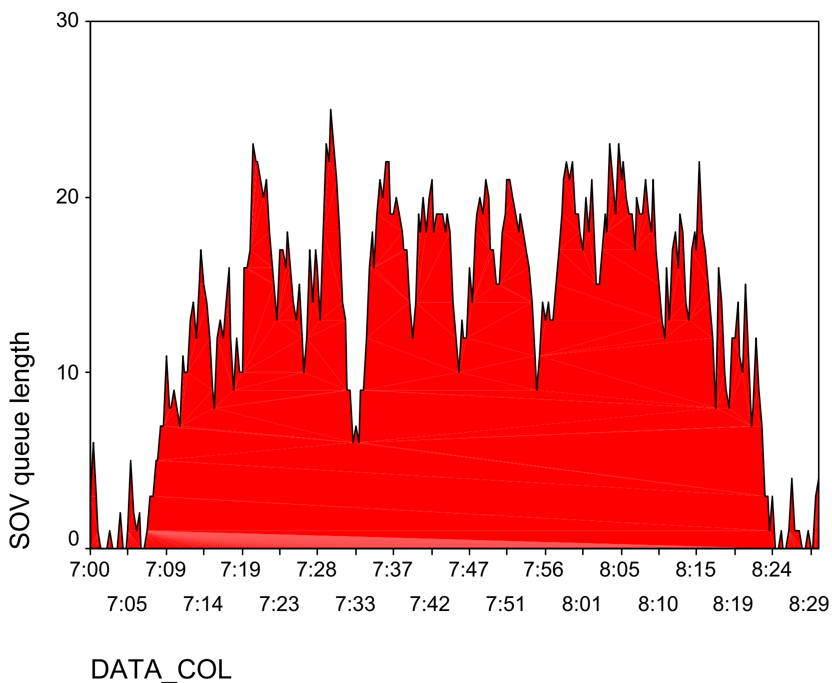
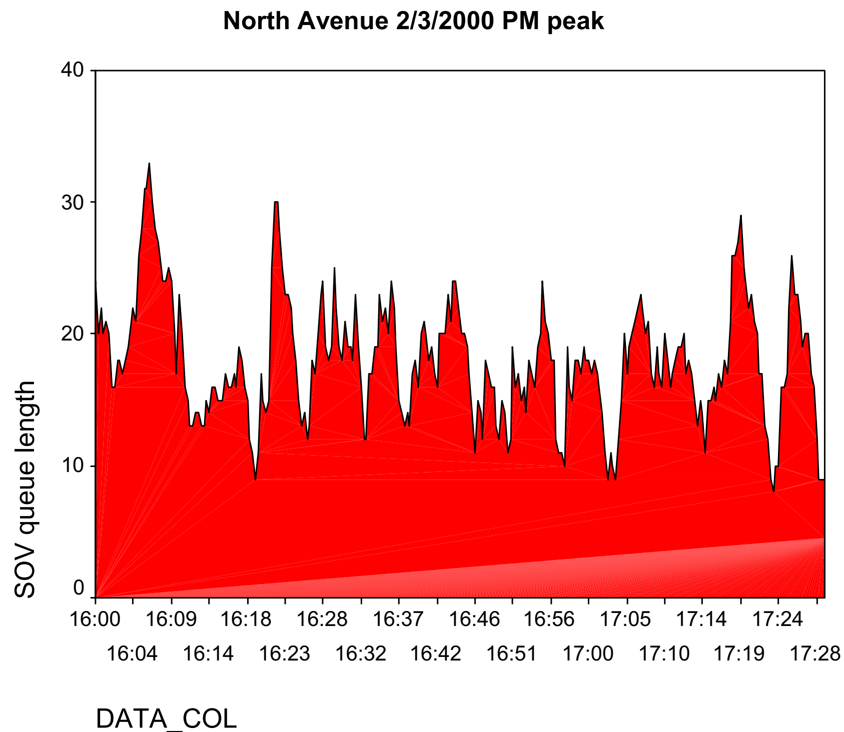
8:29
DATA_COL

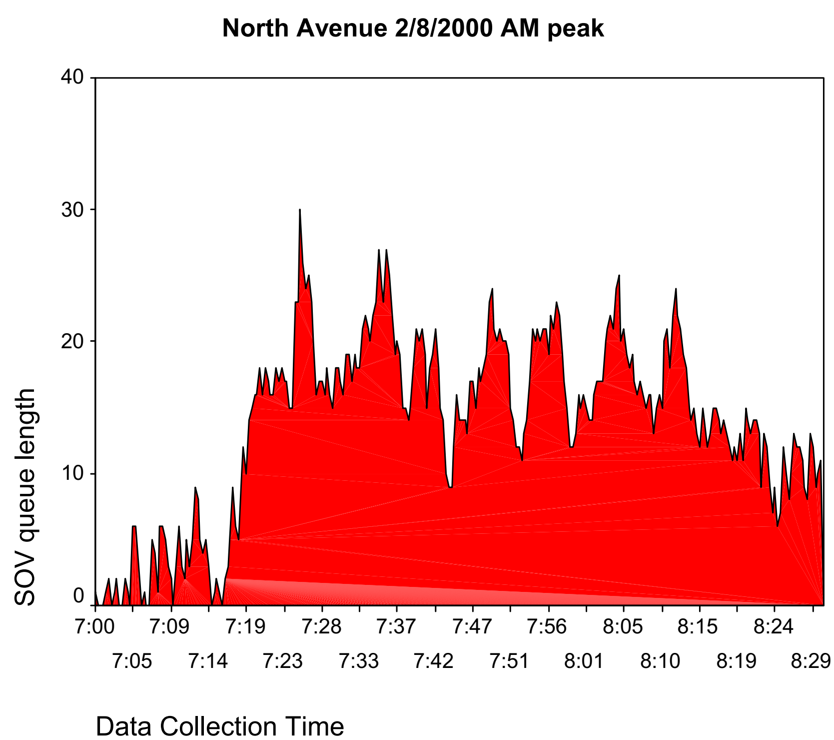
North Avenue 2/2/2000 PM peak



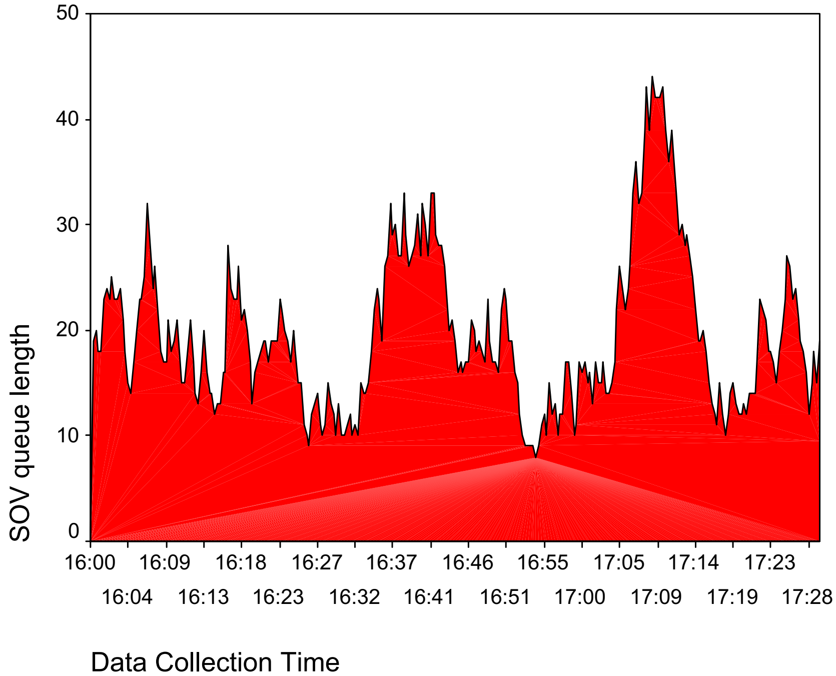
North Avenue 2/3/2000 AM peak



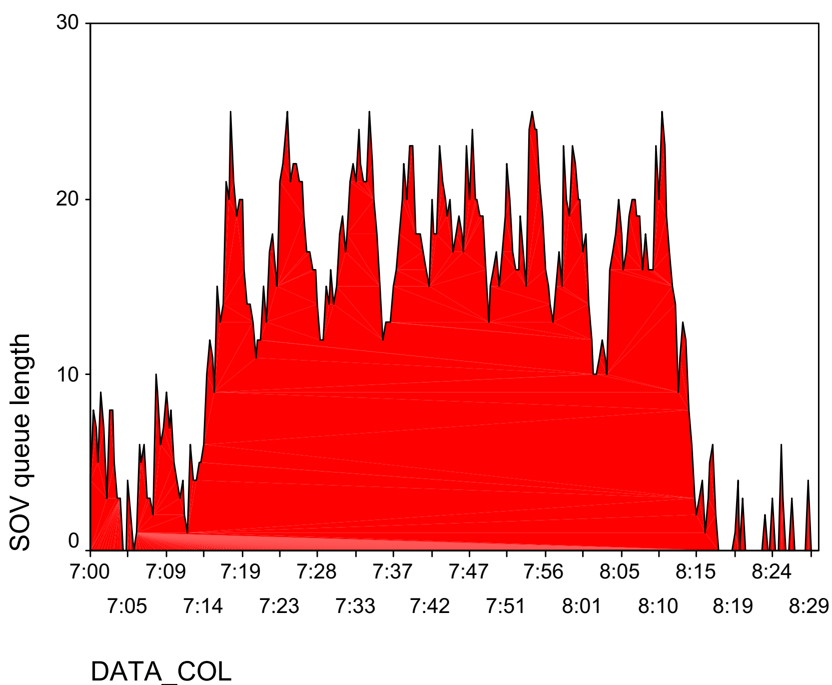

North Avenue 2/3/2000 PM peak
40
30
20
10
0
SOV queue length
16:00 16:09 16:18 16:28 16:37 16:46 16:56 17:05 17:14 17:24
16:04 16:14 16:23 16:32 16:42 16:51 17:00 17:10 17:19 17:28
DATA_COL


North Avenue 2/8/2000 AM peak
40
30
20
10
0
SOV queue length
7:00 7:09 7:19 7:28 7:37 7:47 7:56 8:05 8:15 8:24
7:05 7:14 7:23 7:33 7:42 7:51 8:01 8:10 8:19 8:29
Data Collection Time

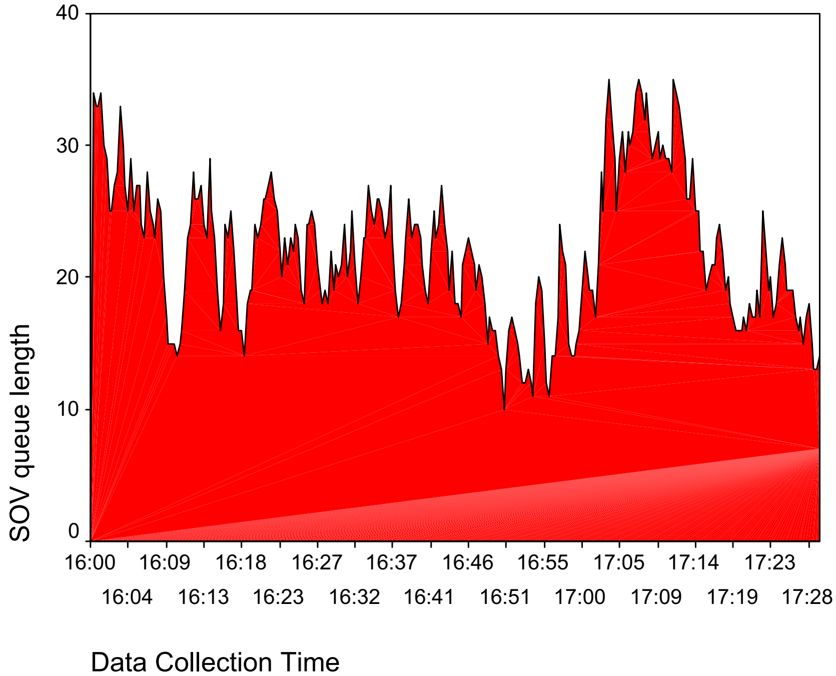
North Avenue 2/8/2000 PM peak



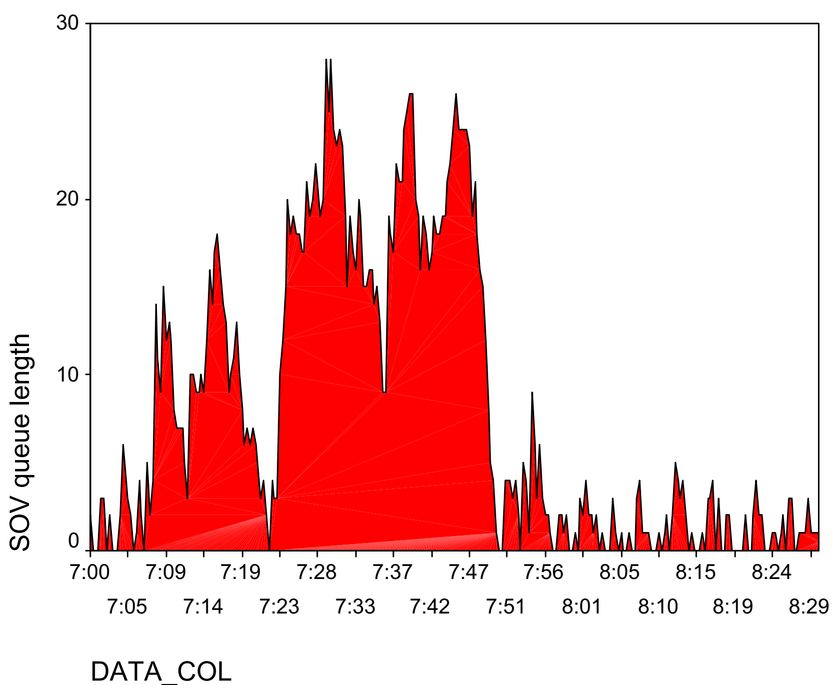
North Avenue 2/9/2000 AM peak

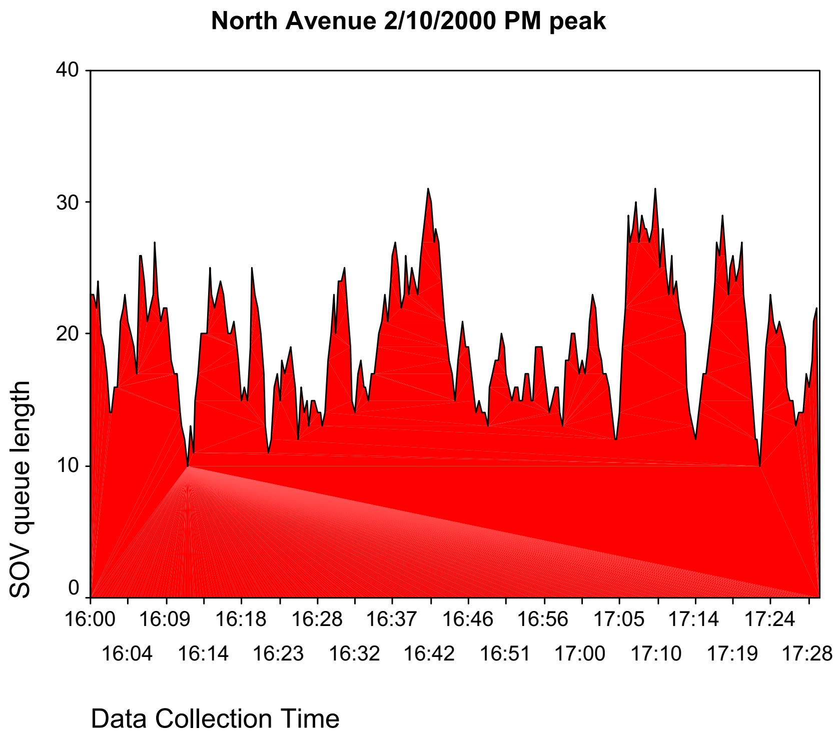


North Avenue 2/9/2000 PM peak



North Avenue 2/10/2000 AM peak




North Avenue 2/10/2000 PM peak
SOV queue length
0
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16:04
16:09
16:14
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16:23
16:28
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16:56
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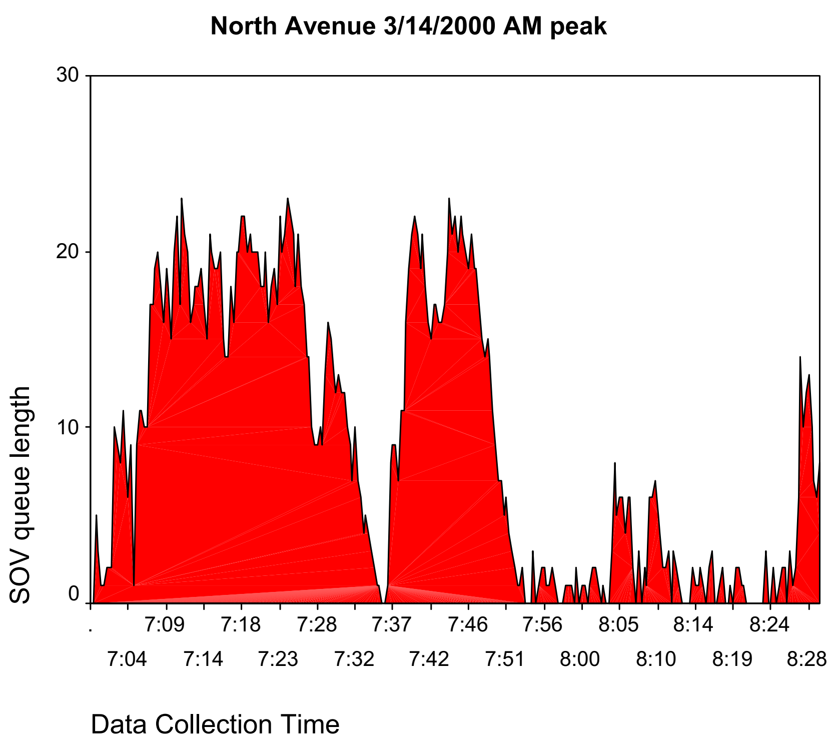
17:14
17:19
17:24
17:28
Data Collection Time


North Avenue 3/14/2000 AM peak
SOV queue length
0
10
20
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7:04
7:09
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8:00
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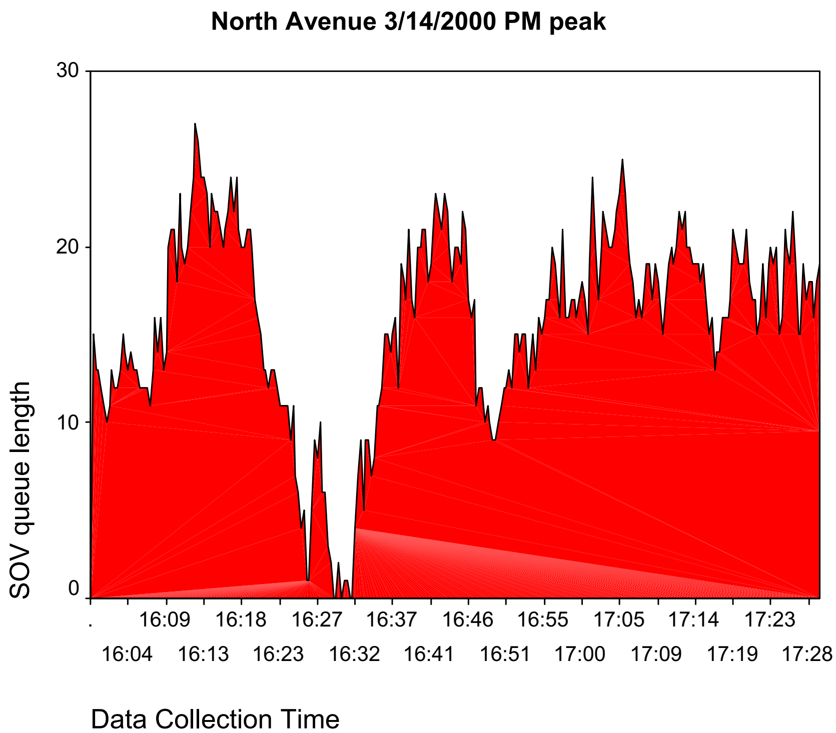
8:19
8:24
8:28
Data Collection Time


North Avenue 3/14/2000 PM peak
30
20
10
0
SOV queue length
16:04
16:09
16:13
16:18
16:23
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16:51
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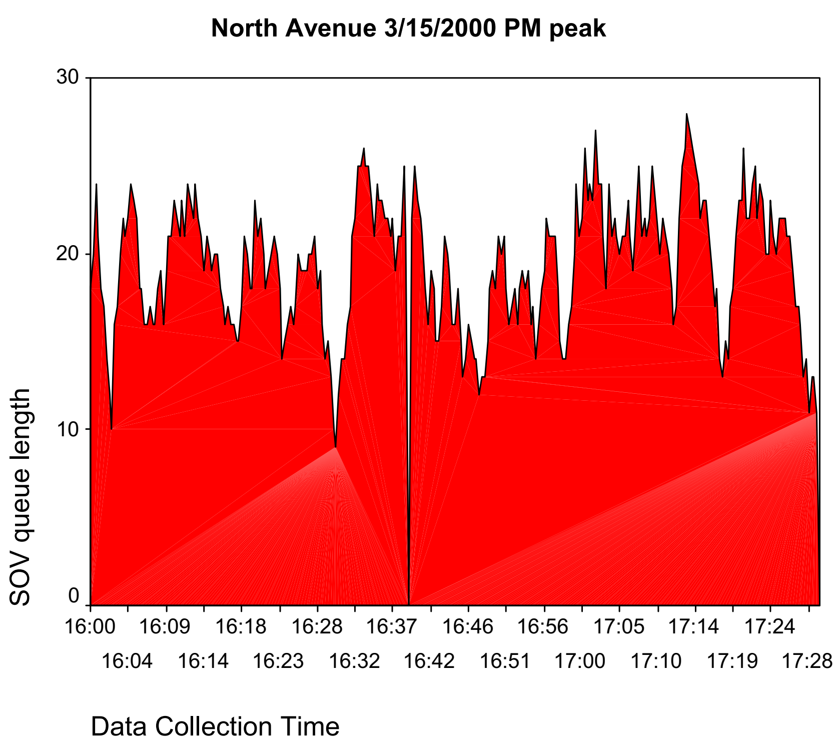
17:14
17:19
17:23
17:28
Data Collection Time


North Avenue 3/15/2000 PM peak
30
20
10
0
SOV queue length
16:00
16:04
16:09
16:14
16:18
16:23
16:28
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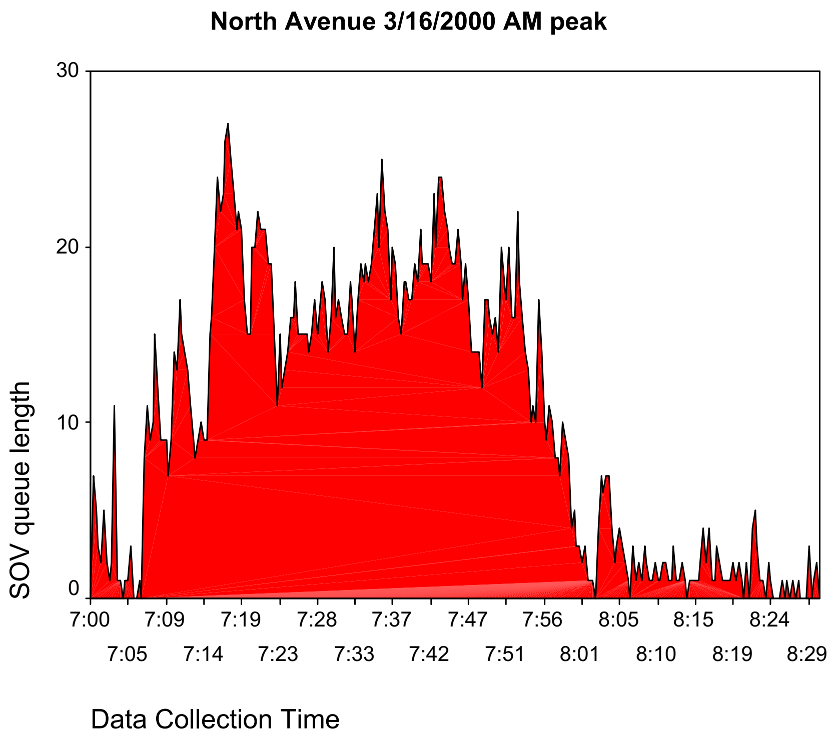
Data Collection Time


North Avenue 3/16/2000 AM peak
SOV queue length
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7:00
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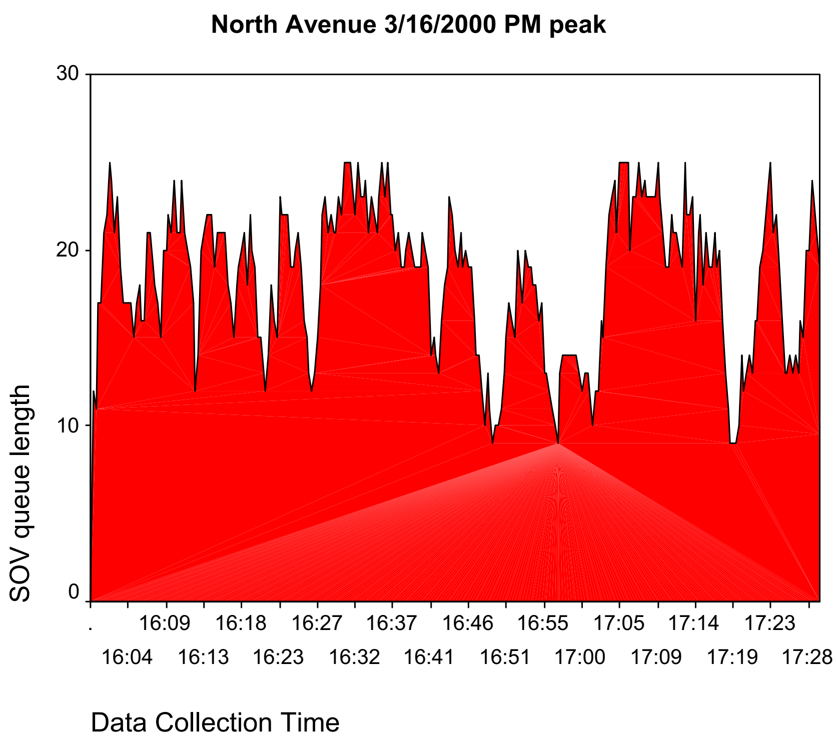
8:29
Data Collection Time


North Avenue 3/16/2000 PM peak
SOV queue length
0
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16:18
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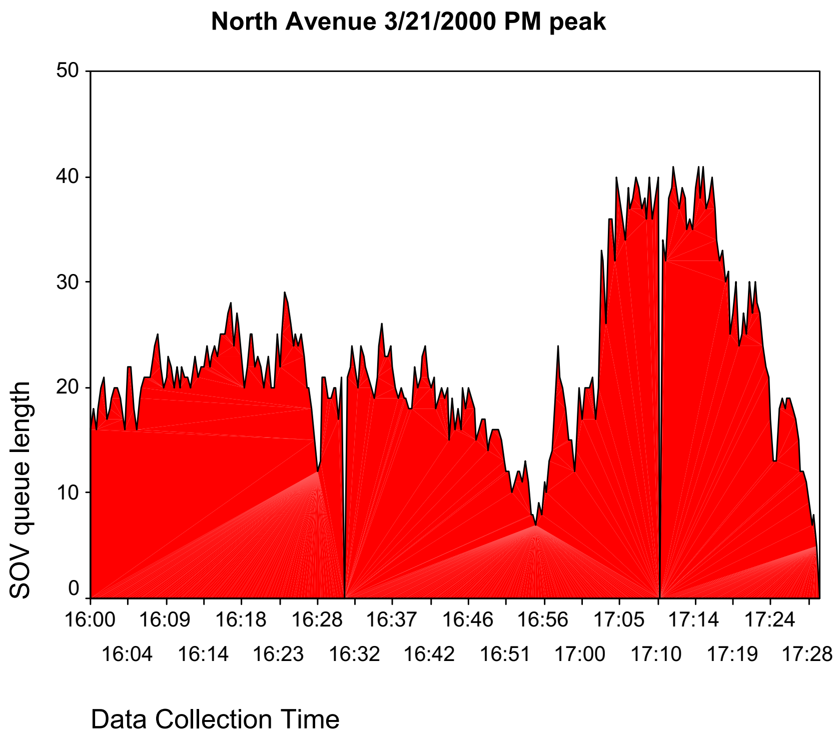
Data Collection Time


North Avenue 3/21/2000 PM peak
SOV queue length
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0
16:00
16:09
16:18
16:28
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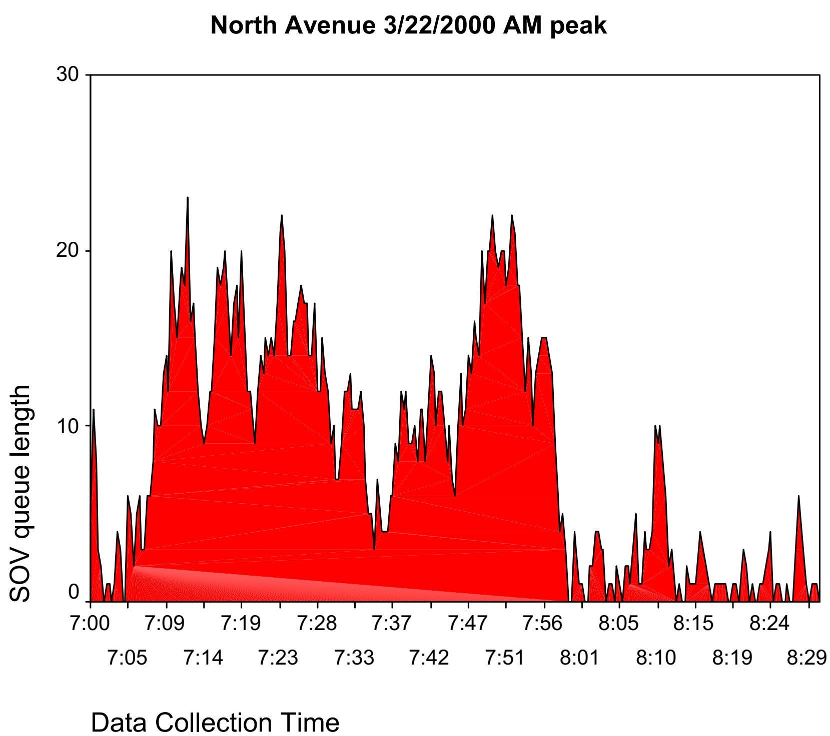
Data Collection Time


North Avenue 3/22/2000 AM peak
SOV queue length
30
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7:00
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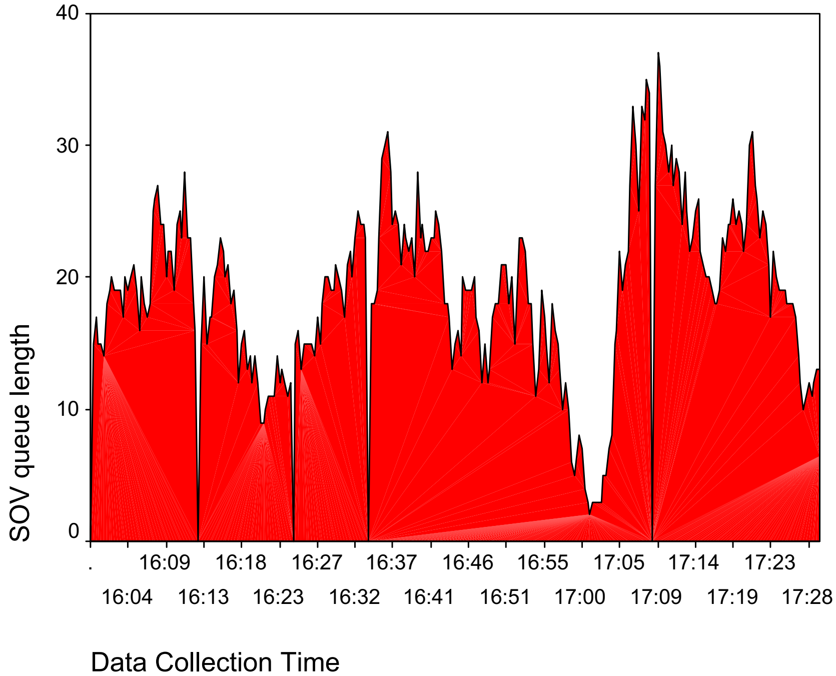
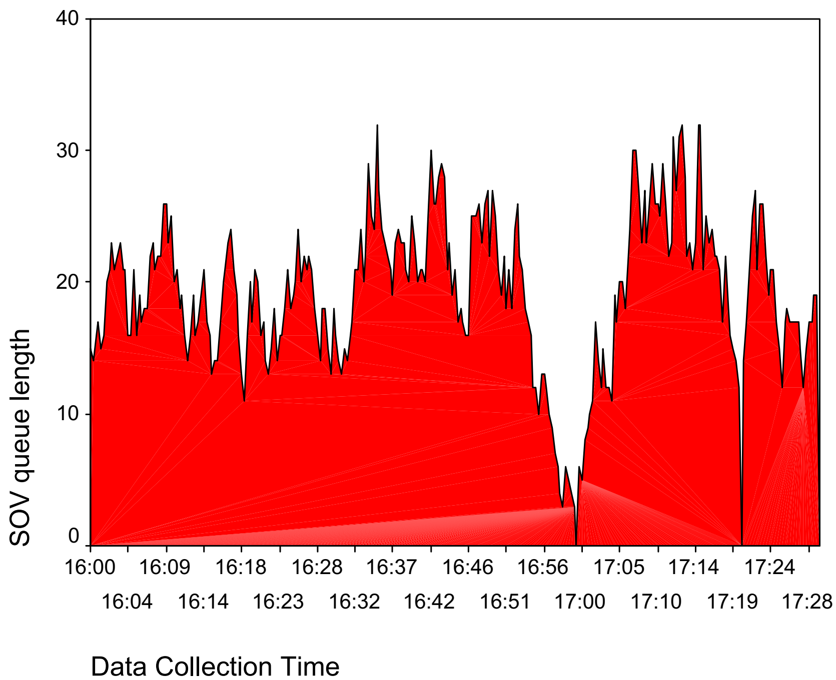
8:29
Data Collection Time

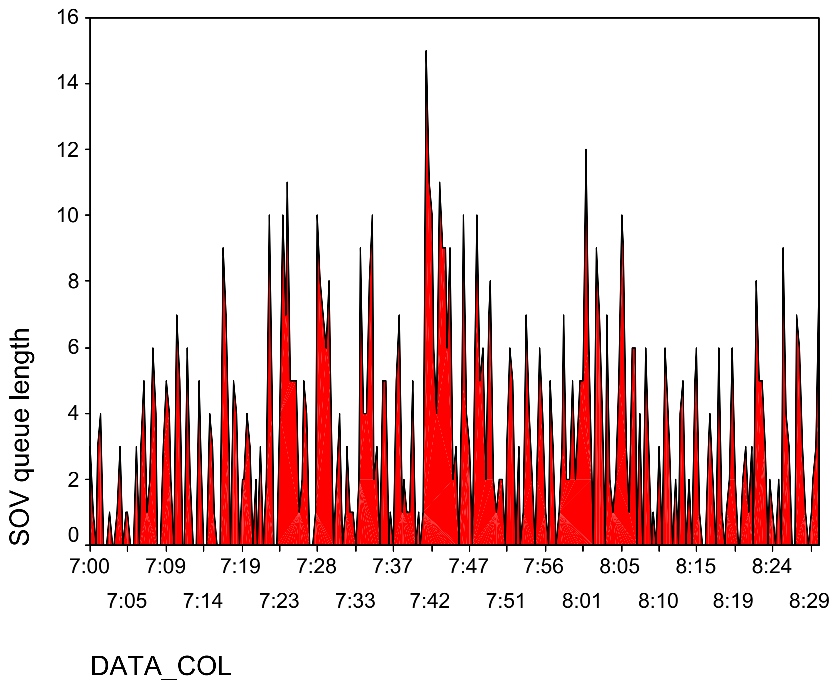
North Avenue 3/22/2000 PM peak



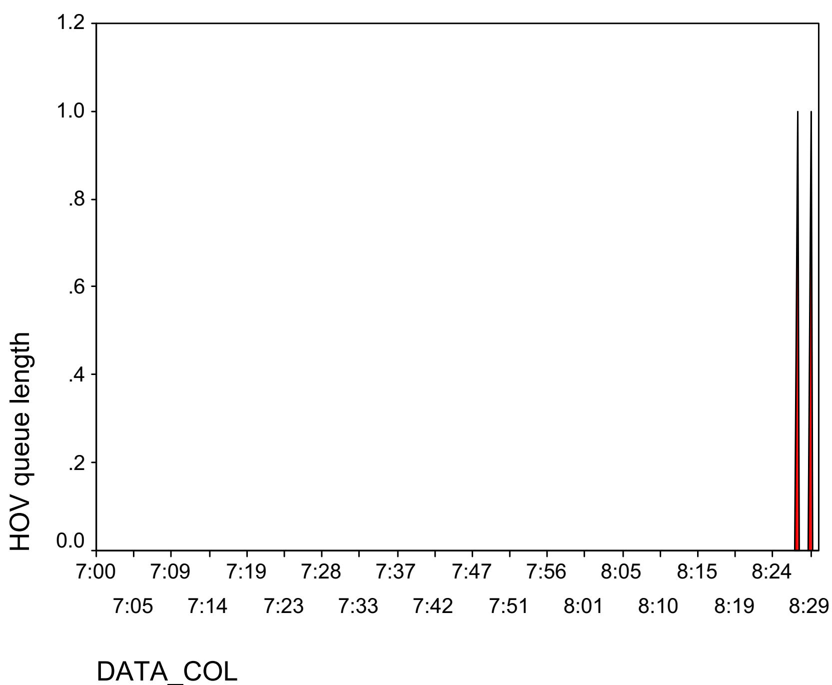
North Avenue 3/23/2000 PM peak



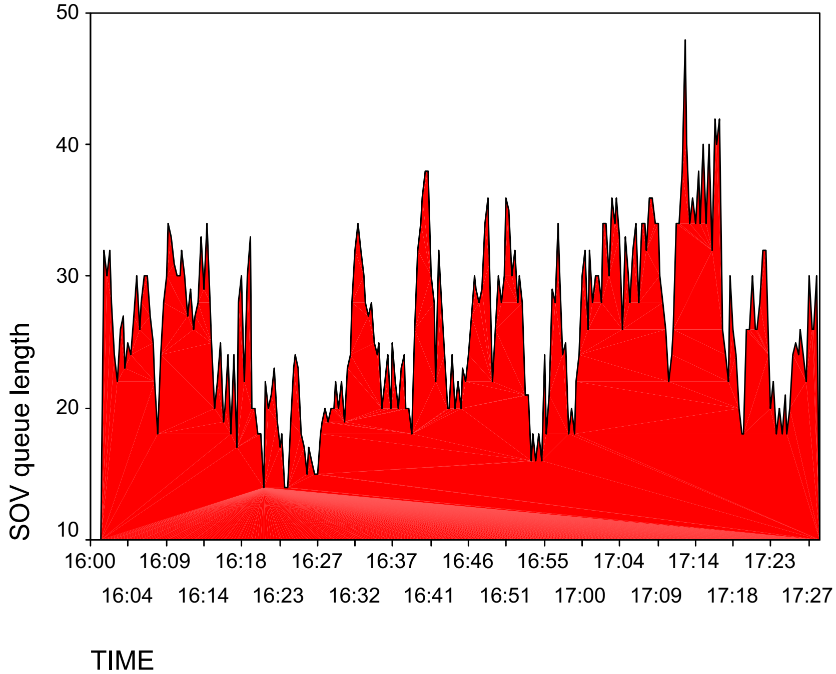
Watertown Plank Road 2/1/2000 AM peak



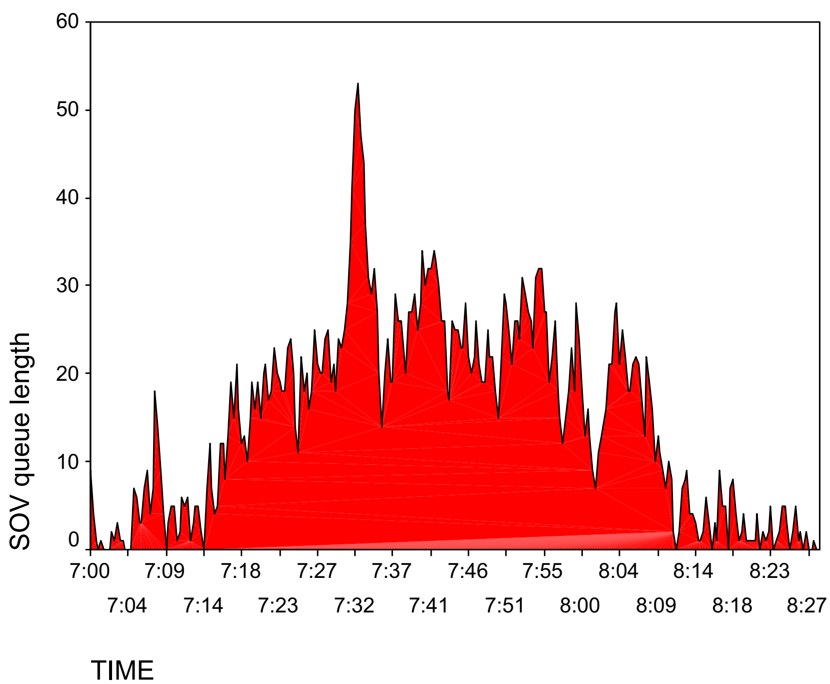
Watertown Plank Road 2/1/2000 AM peak



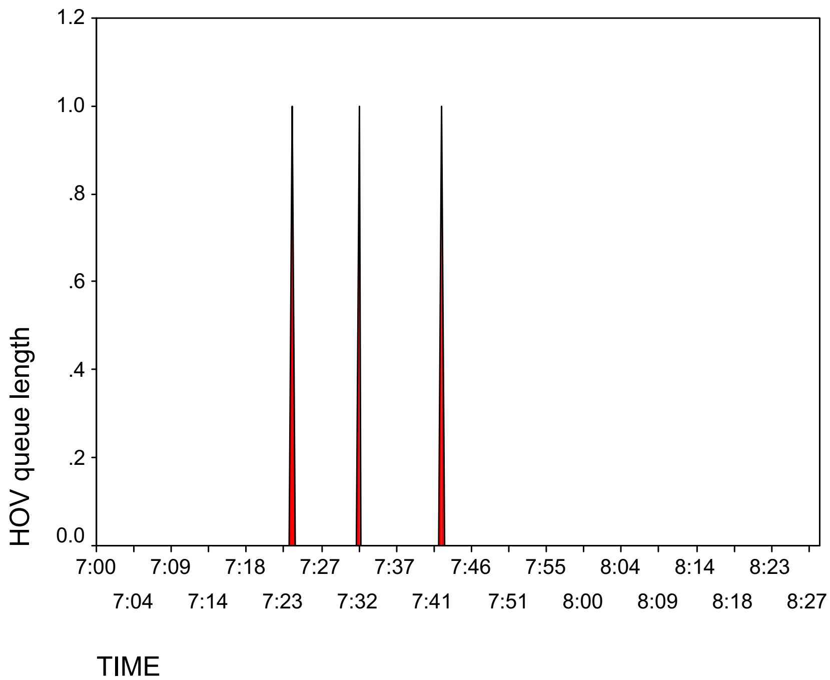
Watertown Plank Road 2/1/2000 PM peak



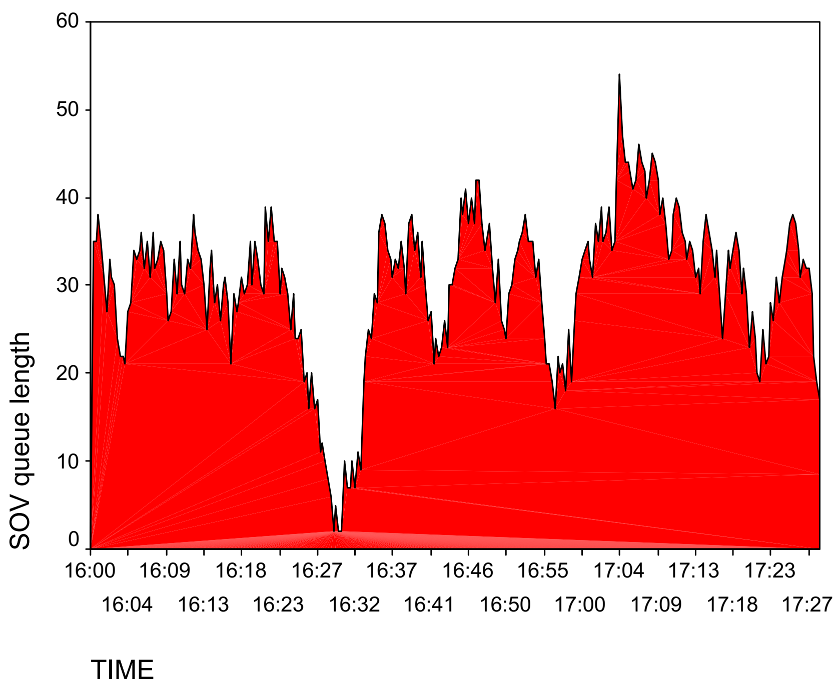
Watertown Plank Road 2/2/2000 AM peak



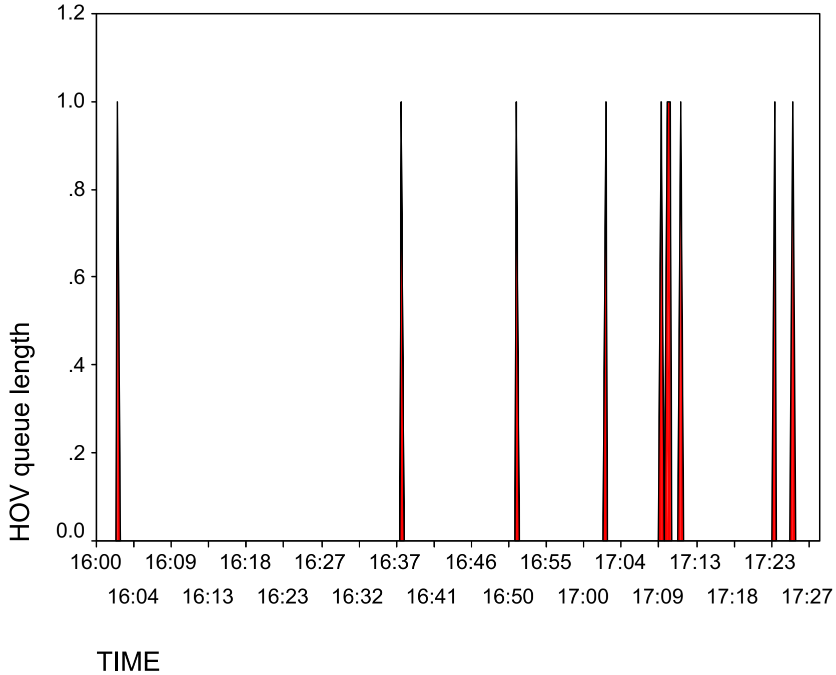
Watertown Plank Road 2/2/2000 AM peak



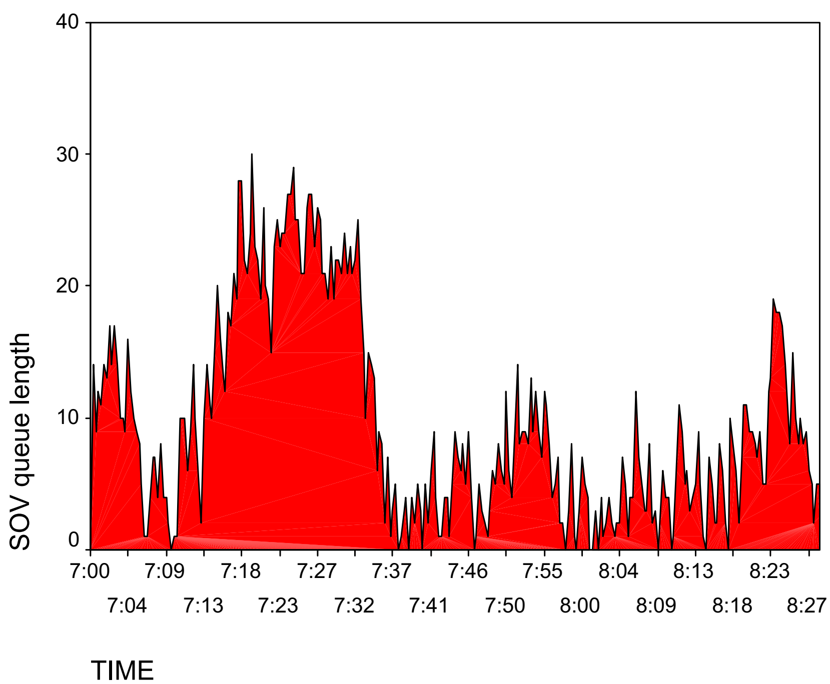
Watertown Plank Road 2/2/2000 PM peak



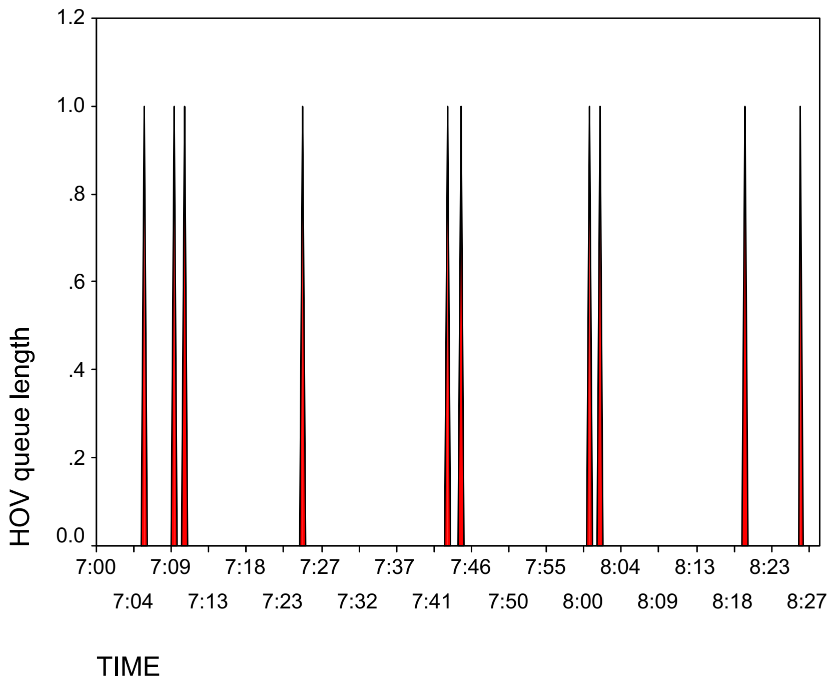
Watertown Plank Road 2/2/2000 PM peak



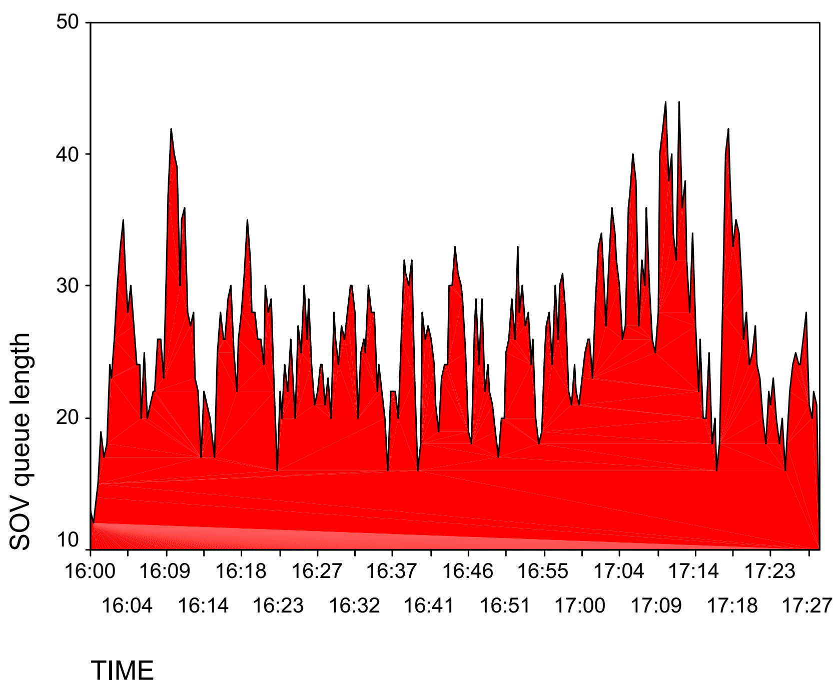
Watertown Plank Road 2/3/2000 AM peak

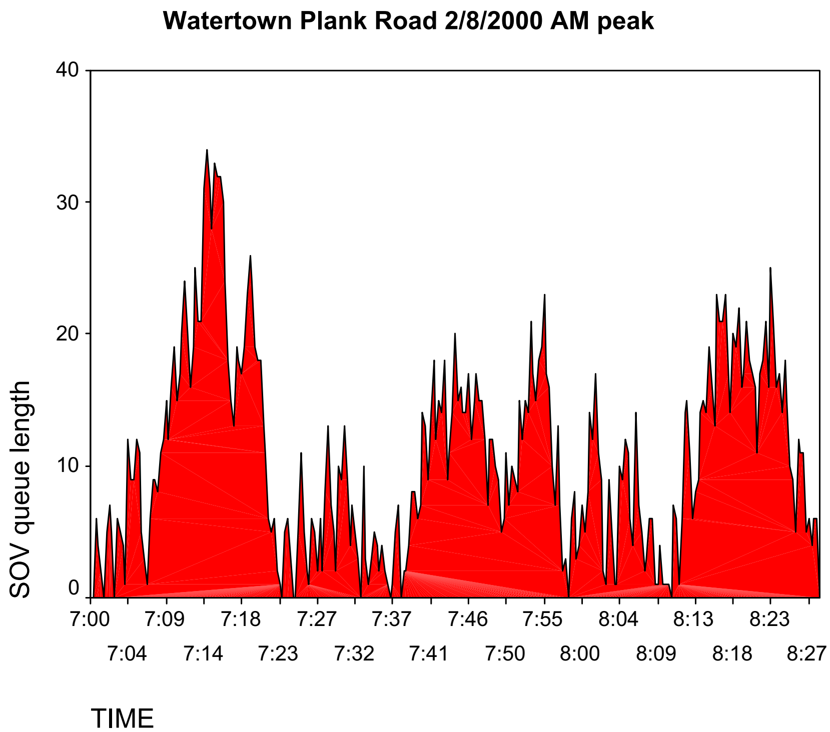


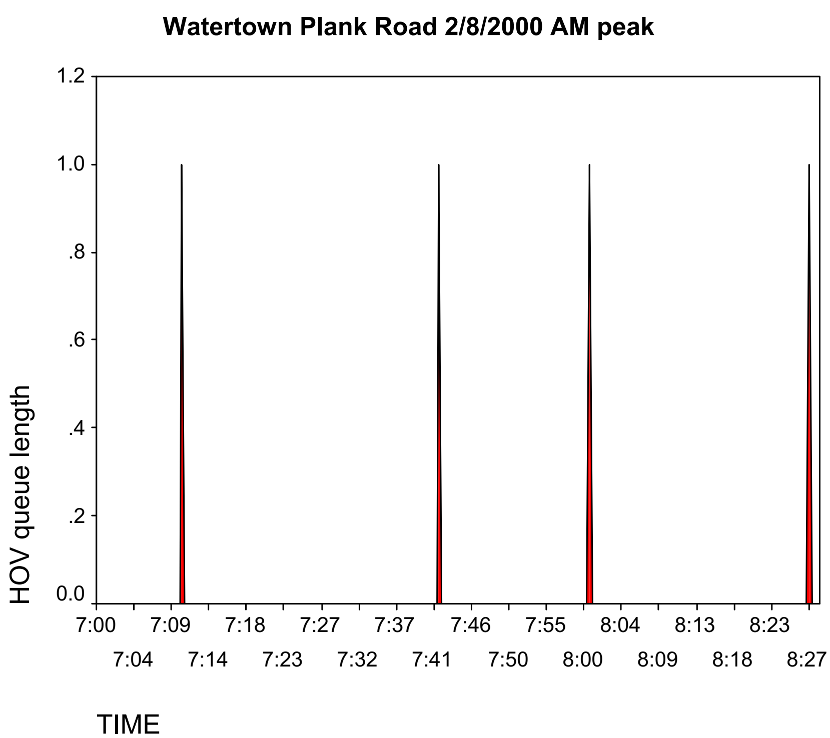
Watertown Plank Road 2/3/2000 AM peak



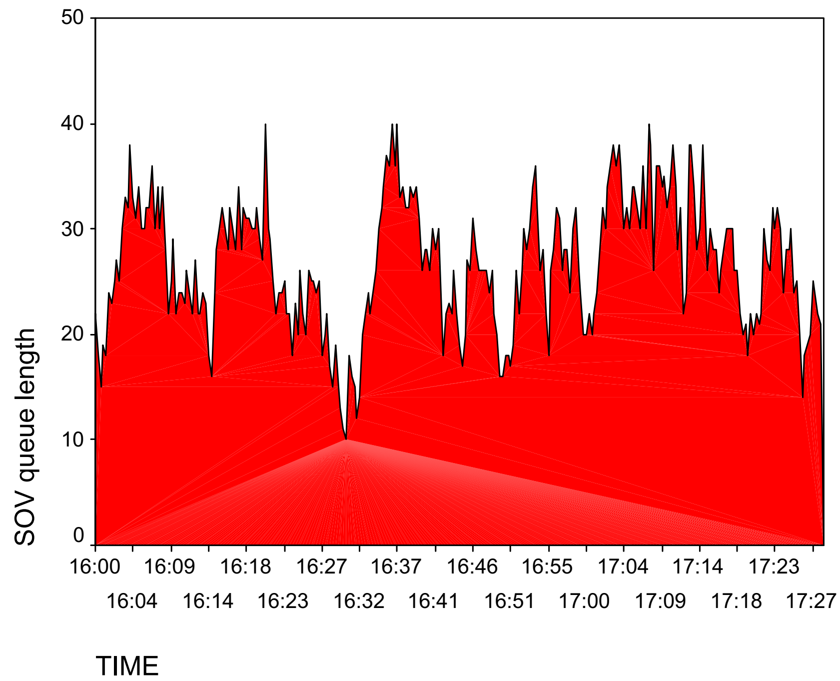
Watertown Plank Road 2/3/2000 PM peak



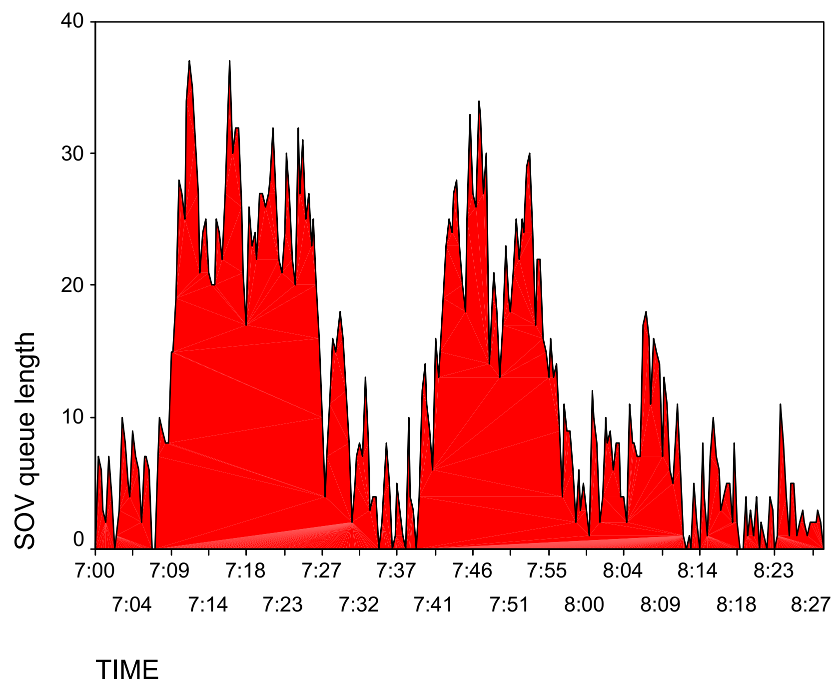

Watertown Plank Road 2/8/2000 AM peak
SOV queue length
40
30
20
10
0
7:00
7:09
7:18
7:27
7:37
7:46
7:55
8:04
8:13
8:23
7:04
7:14
7:23
7:32
7:41
7:50
8:00
8:09
8:18
8:27
TIME

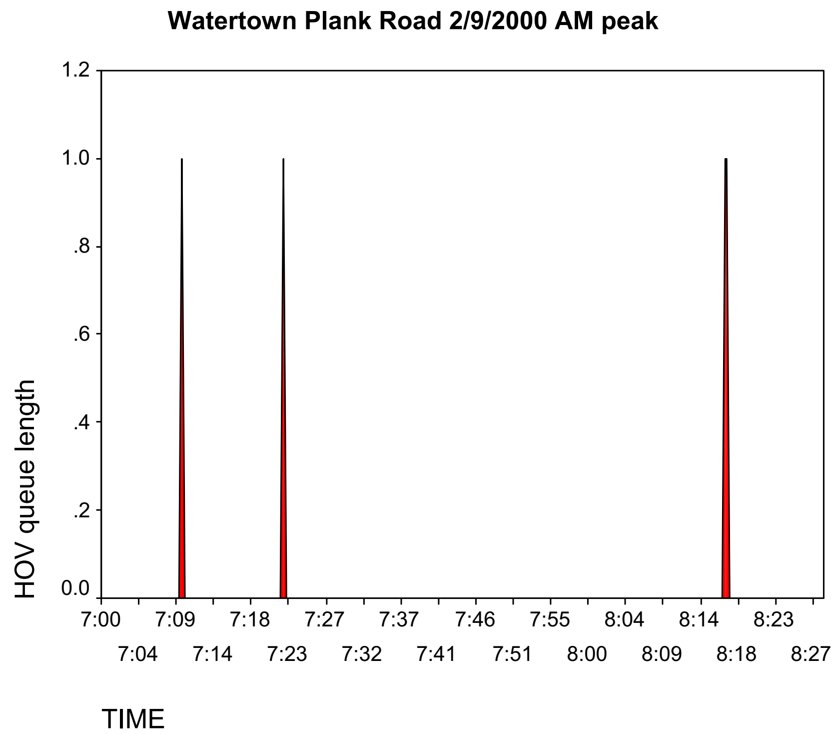

Watertown Plank Road 2/8/2000 AM peak
HOV queue length
1.2
1.0
.8
.6
.4
.2
0.0
7:00
7:09
7:18
7:27
7:37
7:46
7:55
8:04
8:13
8:23
7:04
7:14
7:23
7:32
7:41
7:50
8:00
8:09
8:18
8:27
TIME

Watertown Plank Road 2/8/2000 PM peak

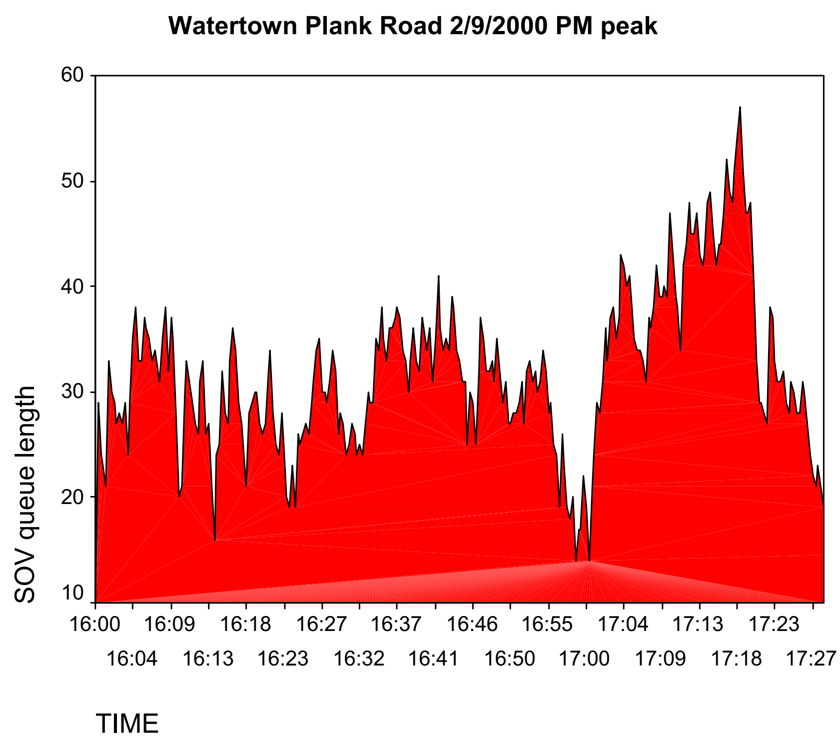


Watertown Plank Road 2/9/2000 AM peak



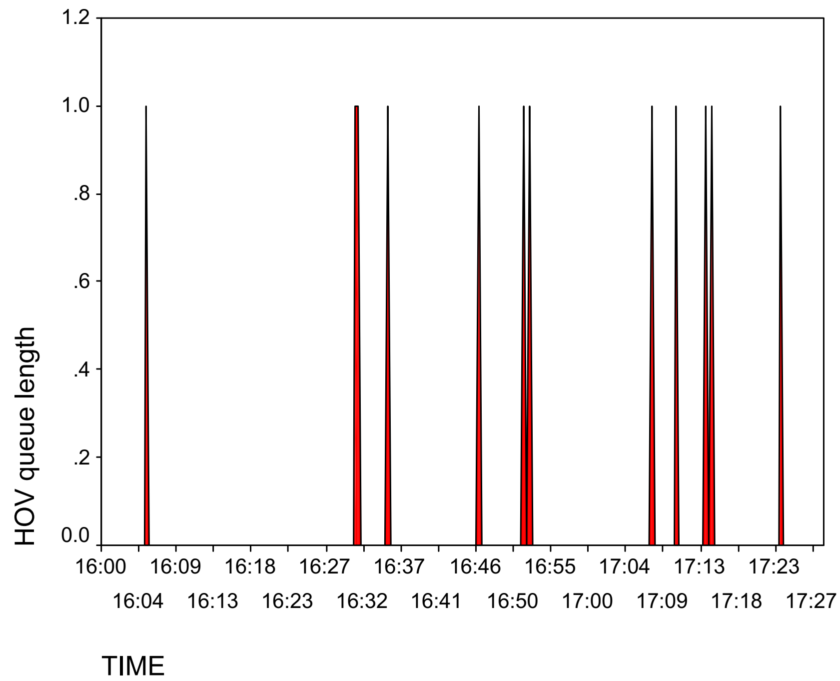


Watertown Plank Road 2/9/2000 AM peak
HOV queue length
1.2
1.0
.8
.6
.4
.2
0.0
7:00
7:04
7:09
7:14
7:18
7:23
7:27
7:32
7:37
7:41
7:46
7:51
7:55
8:00
8:04
8:09
8:14
8:18
8:23
8:27
TIME

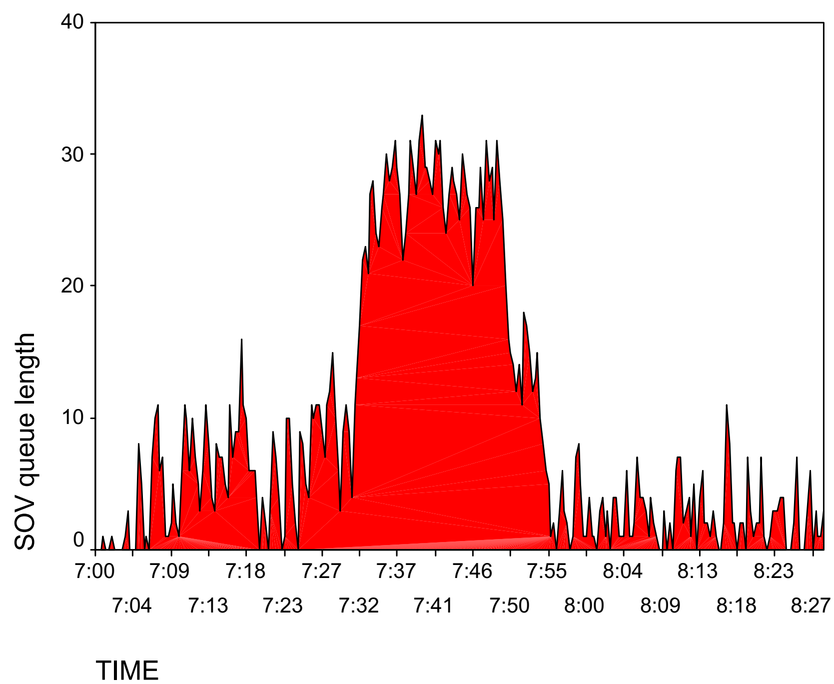


Watertown Plank Road 2/9/2000 PM peak
SOV queue length
60
50
40
30
20
10
16:00
16:04
16:09
16:13
16:18
16:23
16:27
16:32
16:37
16:41
16:46
16:50
16:55
17:00
17:04
17:09
17:13
17:18
17:23
17:27
TIME

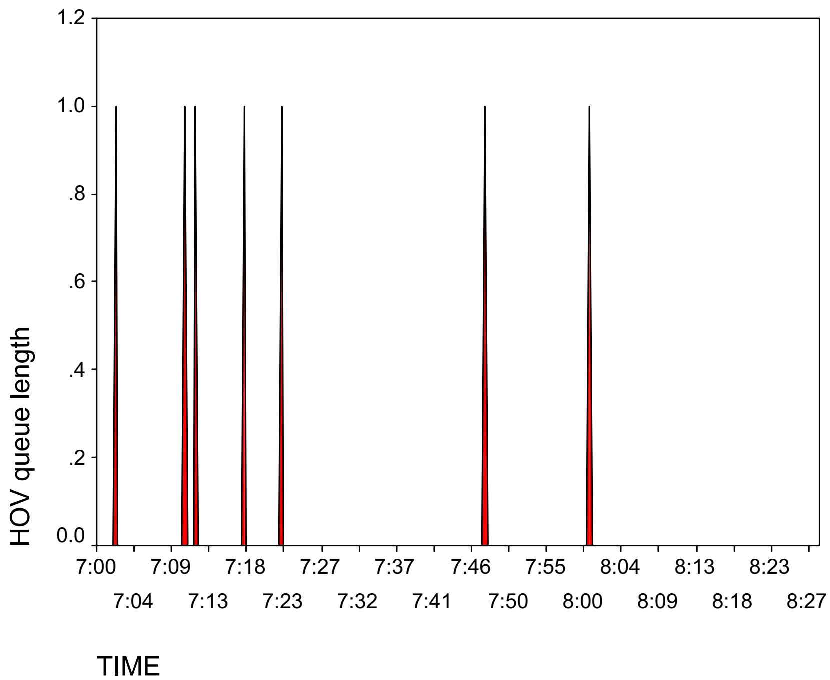
Watertown Plank Road 2/9/2000 PM peak



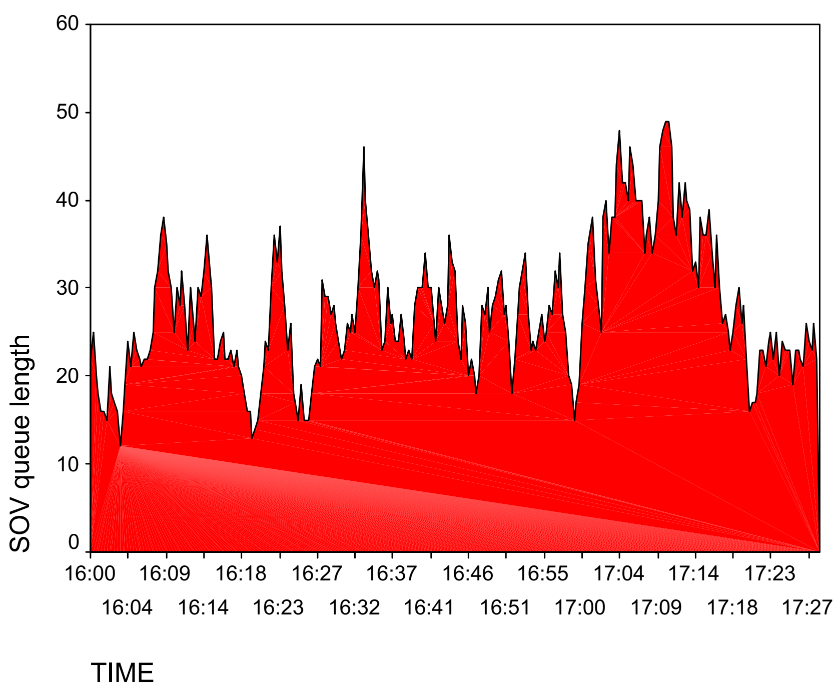
Watertown Plank Road 2/10/2000 AM peak



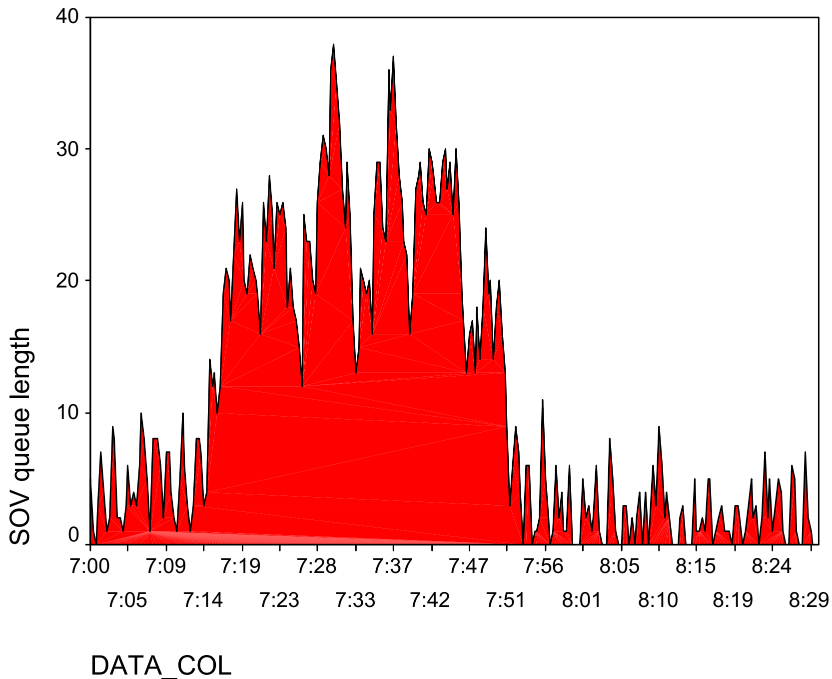
Watertown Plank Road 2/10/2000 AM peak



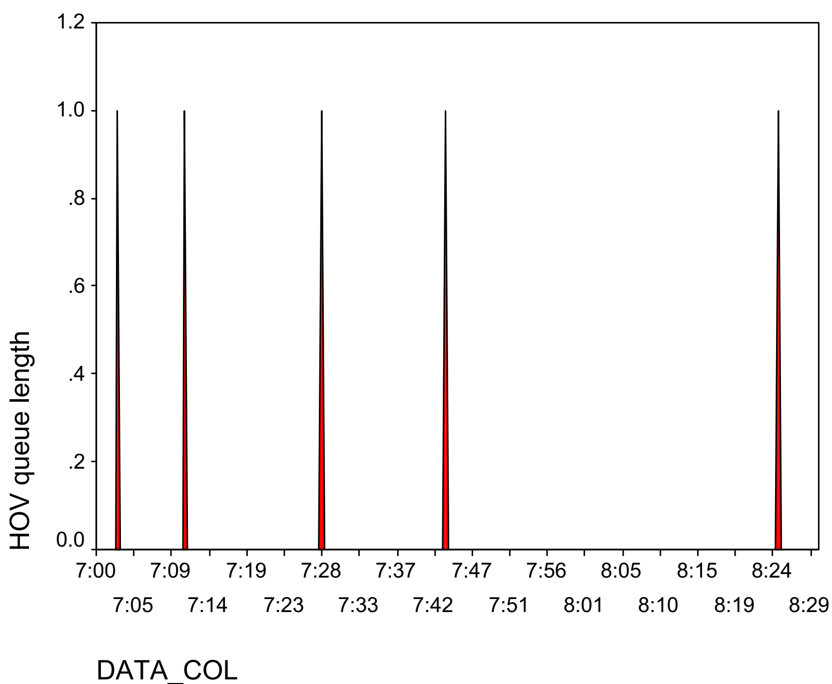
Watertown Plank Road 2/10/2000 PM peak

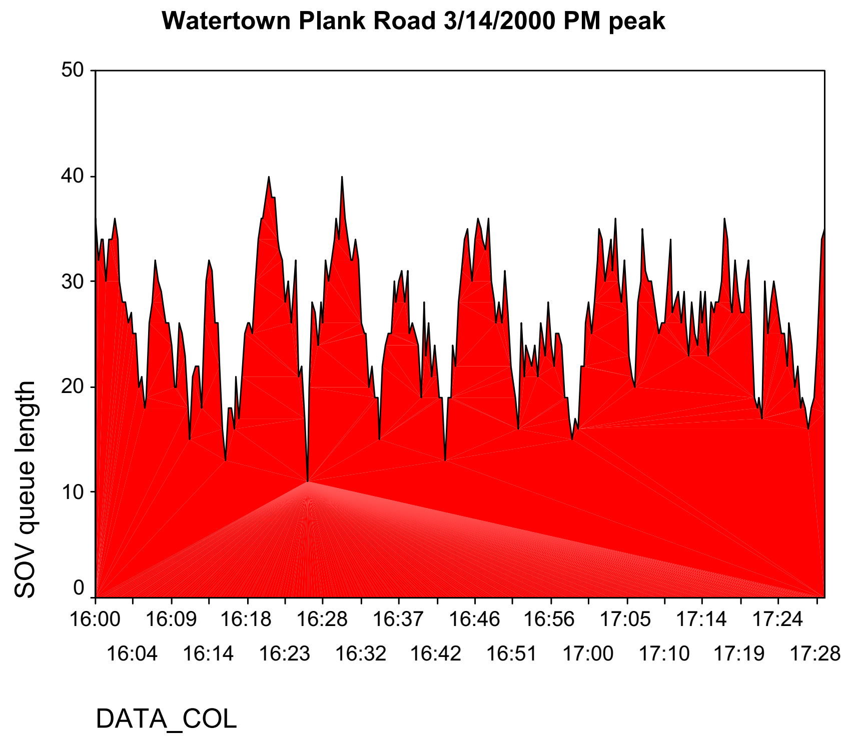


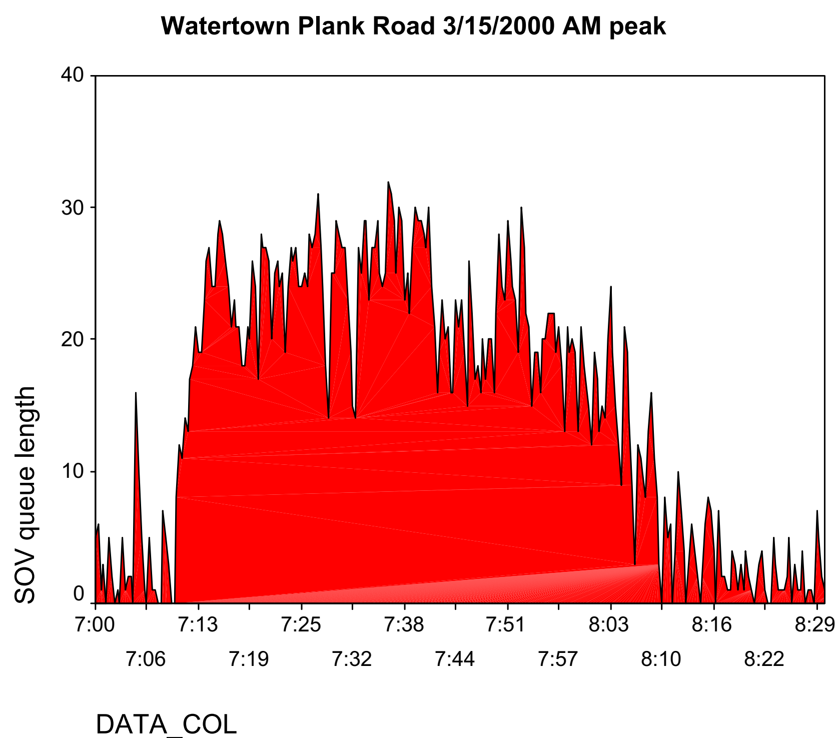
Watertown Plank Road 3/14/2000 AM peak



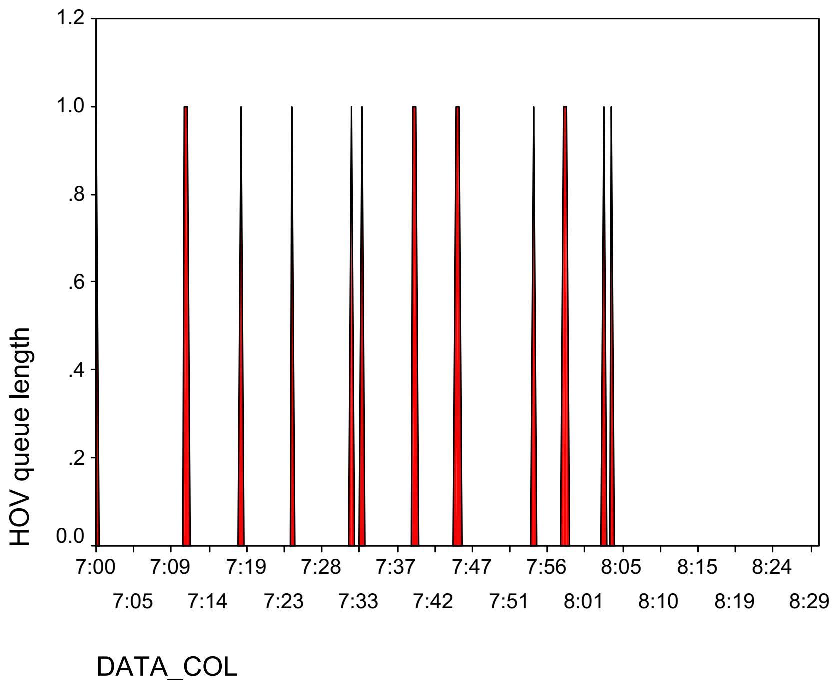
Watertown Plank Road 3/14/2000 AM peak



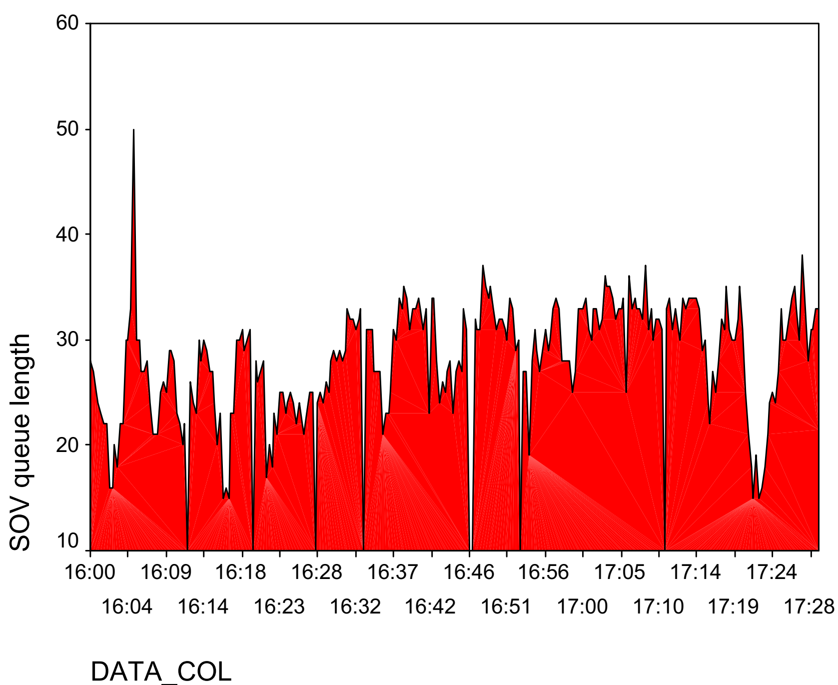

Watertown Plank Road 3/14/2000 PM peak
50
40
30
20
10
0
SOV queue length
16:00
16:09
16:18
16:28
16:37
16:46
16:56
17:05
17:14
17:24
16:04
16:14
16:23
16:32
16:42
16:51
17:00
17:10
17:19
17:28
DATA_COL

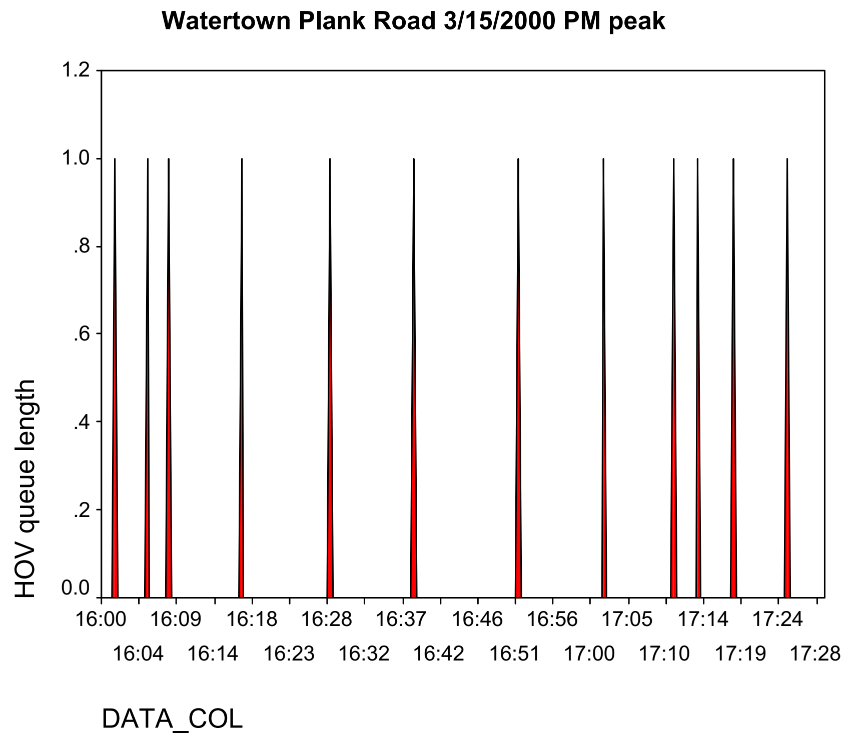

Watertown Plank Road 3/15/2000 AM peak
40
30
20
10
0
SOV queue length
7:00
7:13
7:25
7:38
7:51
8:03
8:16
8:29
7:06
7:19
7:32
7:44
7:57
8:10
8:22
DATA_COL

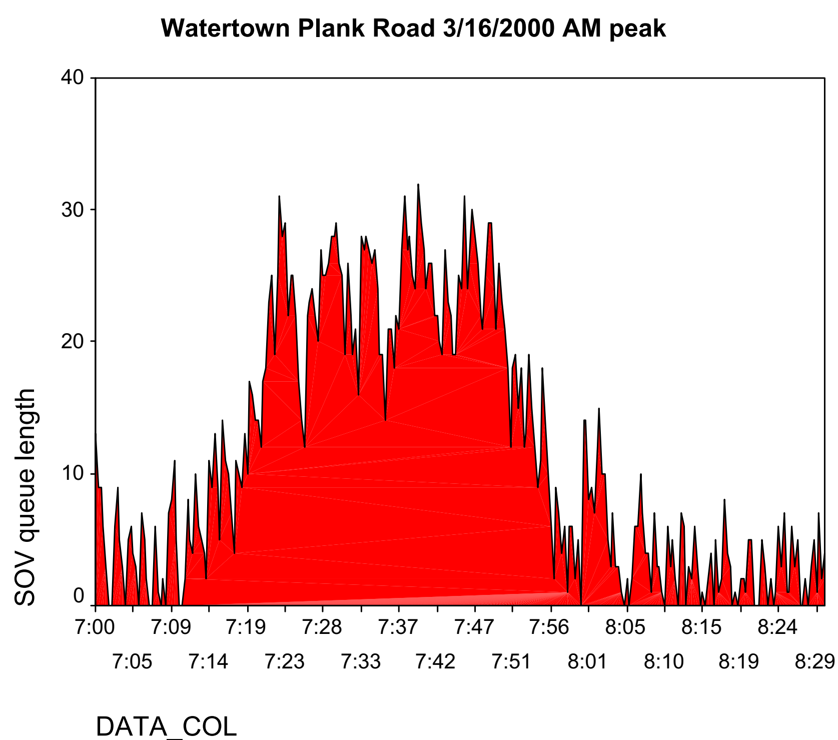
Watertown Plank Road 3/15/2000 AM peak



Watertown Plank Road 3/15/2000 PM peak




Watertown Plank Road 3/15/2000 PM peak
HOV queue length
1.2
1.0
.8
.6
.4
.2
0.0
16:00
16:04
16:09
16:14
16:18
16:23
16:28
16:32
16:37
16:42
16:46
16:51
16:56
17:00
17:05
17:10
17:14
17:19
17:24
17:28
DATA_COL


Watertown Plank Road 3/16/2000 AM peak
SOV queue length
40
30
20
10
0
7:00
7:05
7:09
7:14
7:19
7:23
7:28
7:33
7:37
7:42
7:47
7:51
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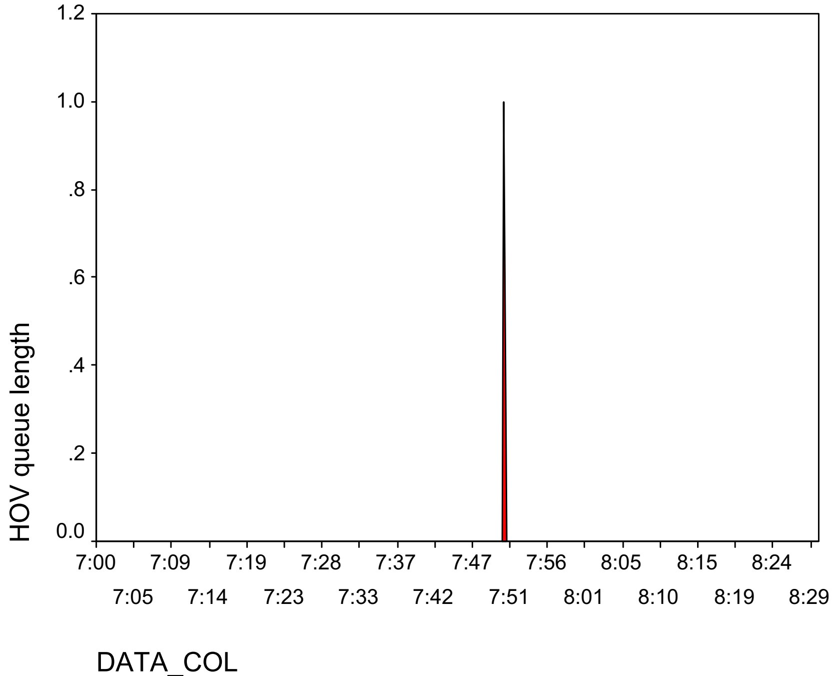
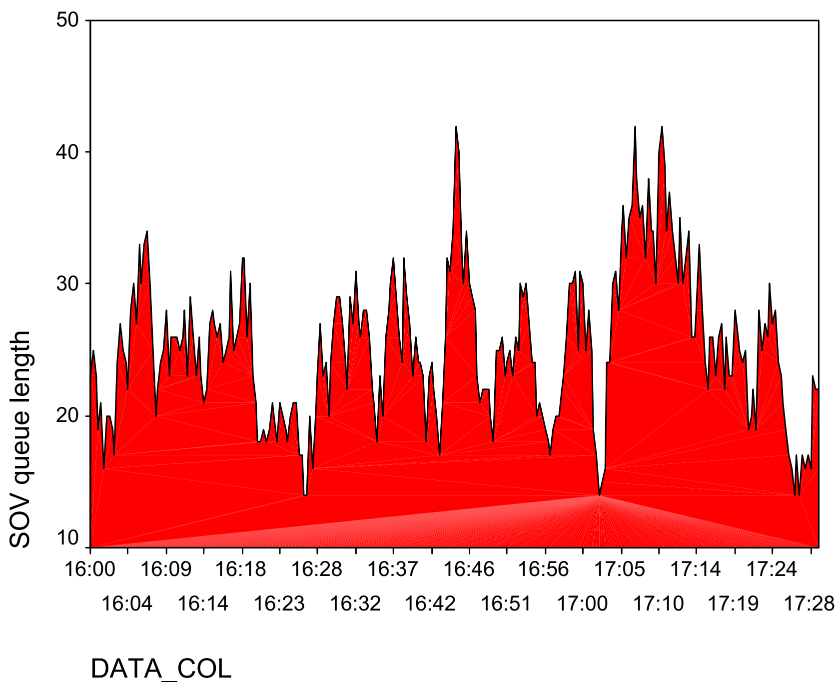
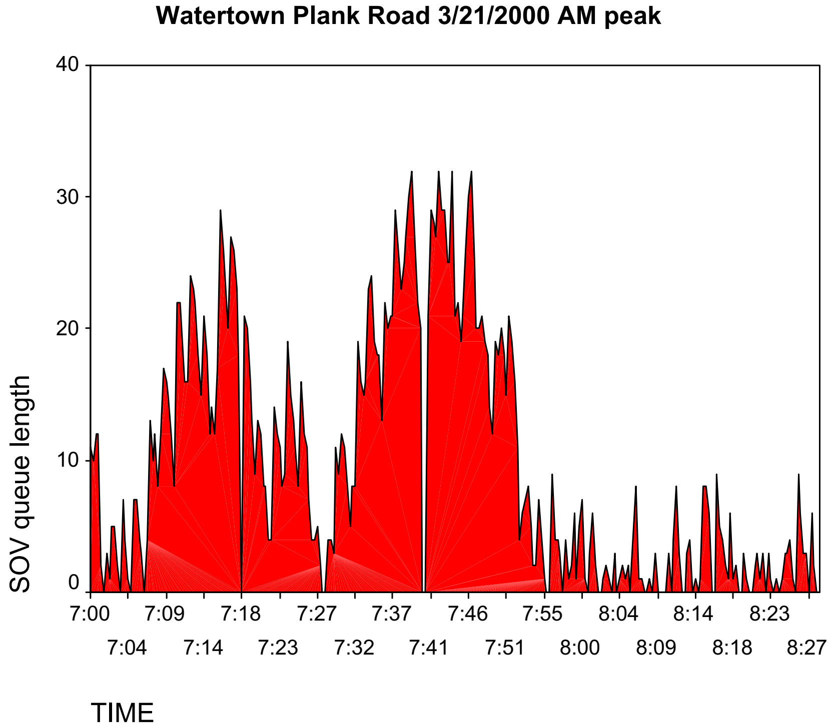
8:05
8:10
8:15
8:19
8:24
8:29
DATA_COL

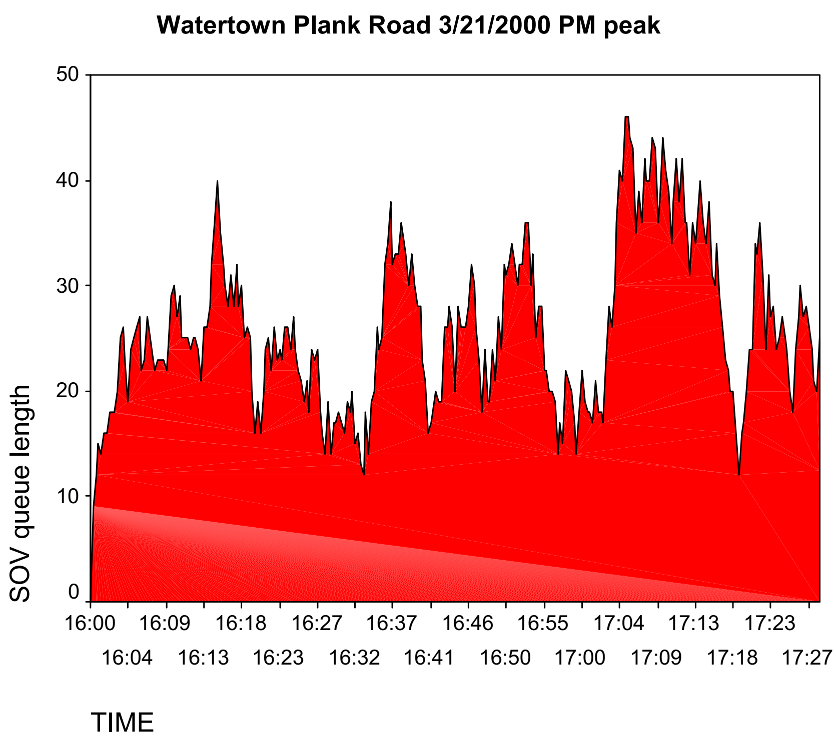
Watertown Plank Road 3/16/2000 AM peak



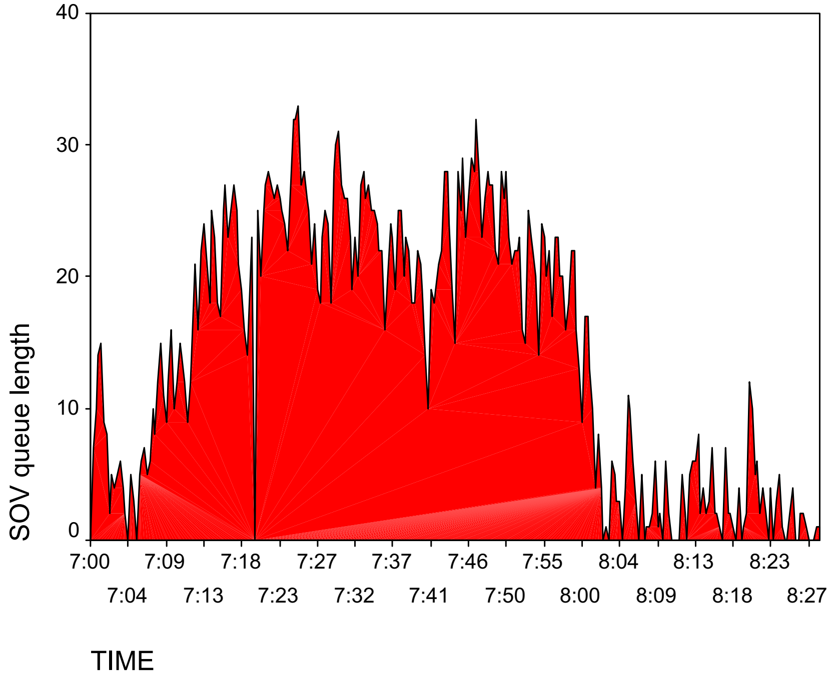
Watertown Plank Road 3/16/2000 PM peak



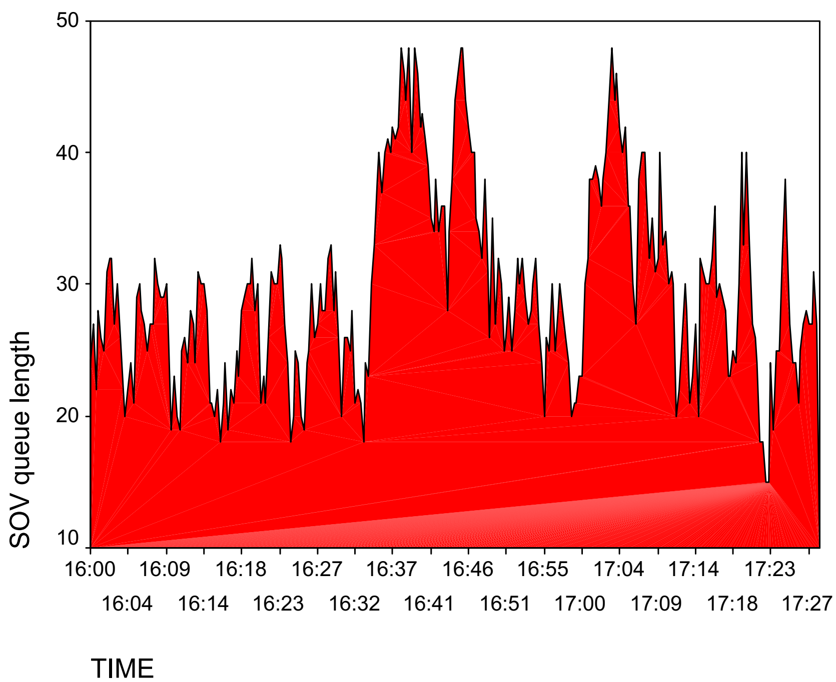

Watertown Plank Road 3/21/2000 AM peak
40
30
20
10
0
SOV queue length
7:00
7:09
7:18
7:27
7:37
7:46
7:55
8:04
8:14
8:23
7:04
7:14
7:23
7:32
7:41
7:51
8:00
8:09
8:18
8:27
TIME


Watertown Plank Road 3/21/2000 PM peak
50
40
30
20
10
0
SOV queue length
16:00
16:09
16:18
16:27
16:37
16:46
16:55
17:04
17:13
17:23
16:04
16:13
16:23
16:32
16:41
16:50
17:00
17:09
17:18
17:27
TIME

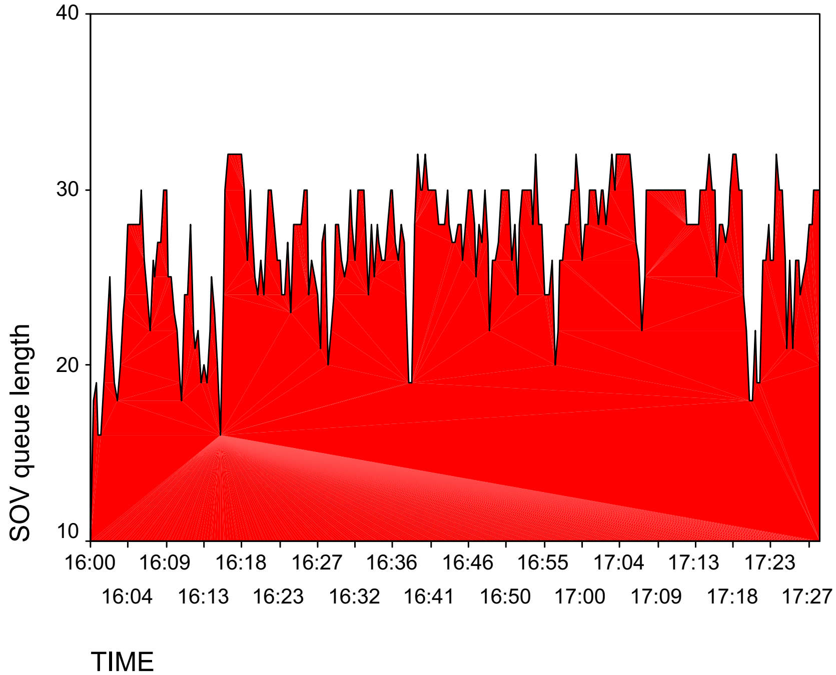
Watertown Plank Road 3/22/2000 AM peak



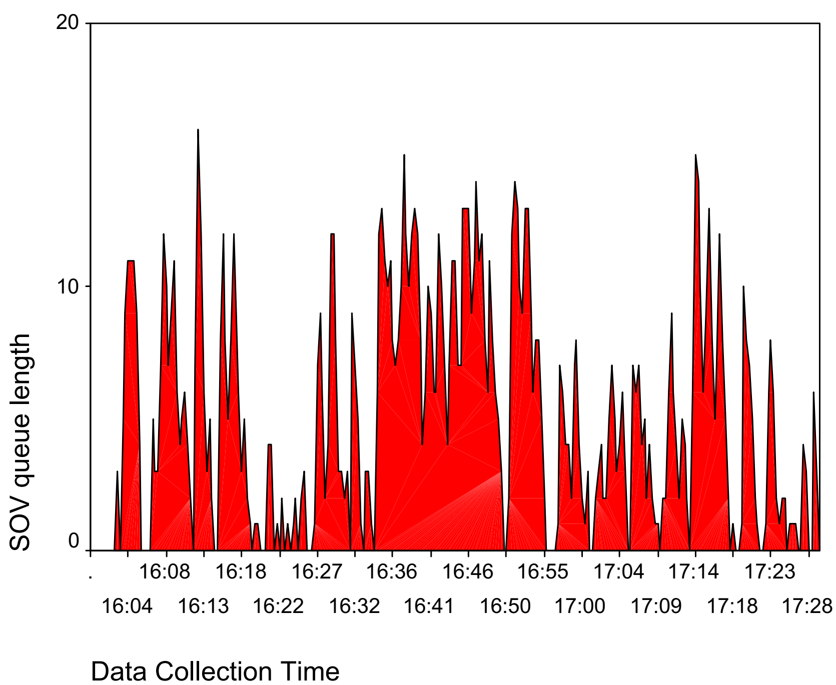
Watertown Plank Road 3/22/2000 PM peak



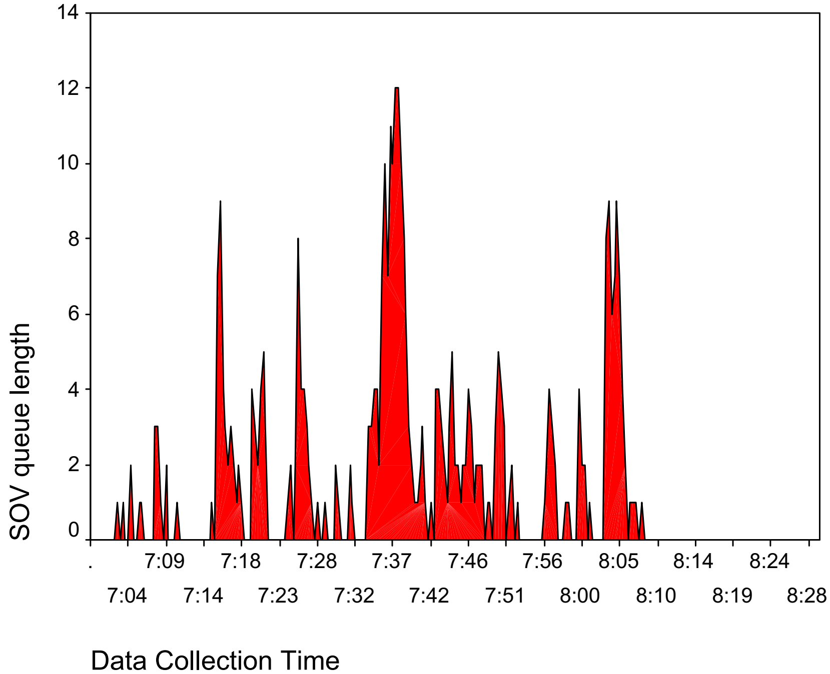
Watertown Plank Road 3/23/2000 PM peak



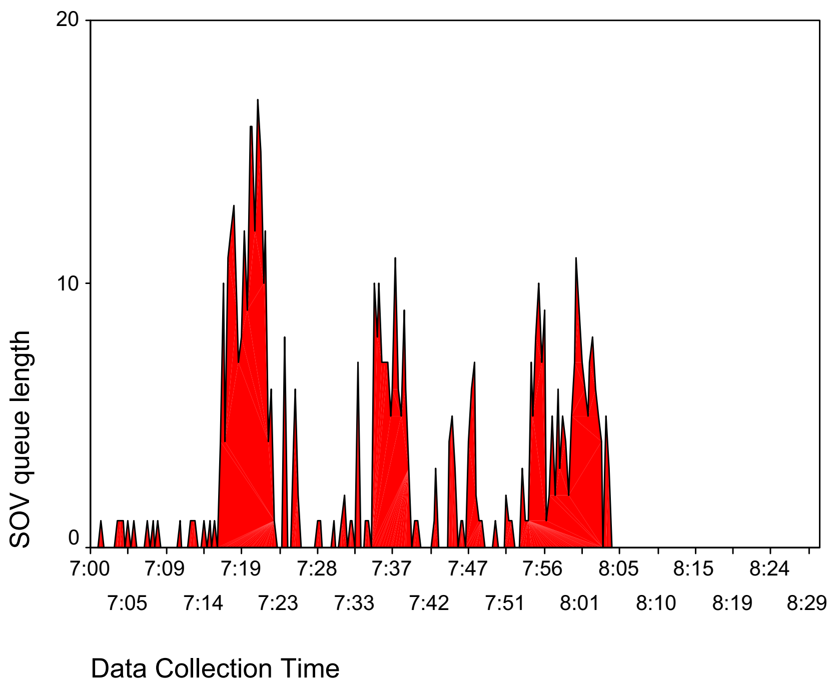
Wisconsin Avenue 2/1/2000 PM peak



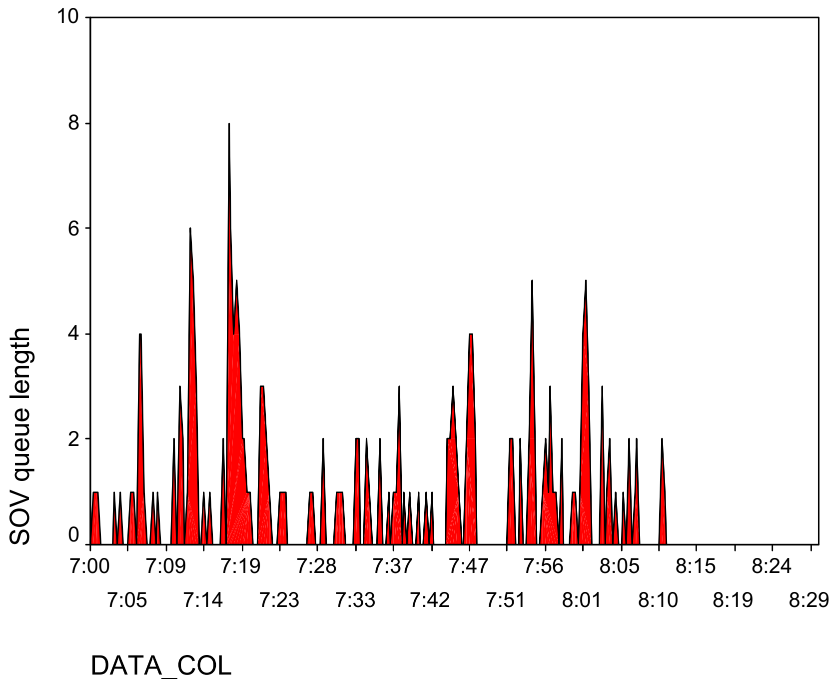
Wisconsin Avenue 2/2/2000 AM peak



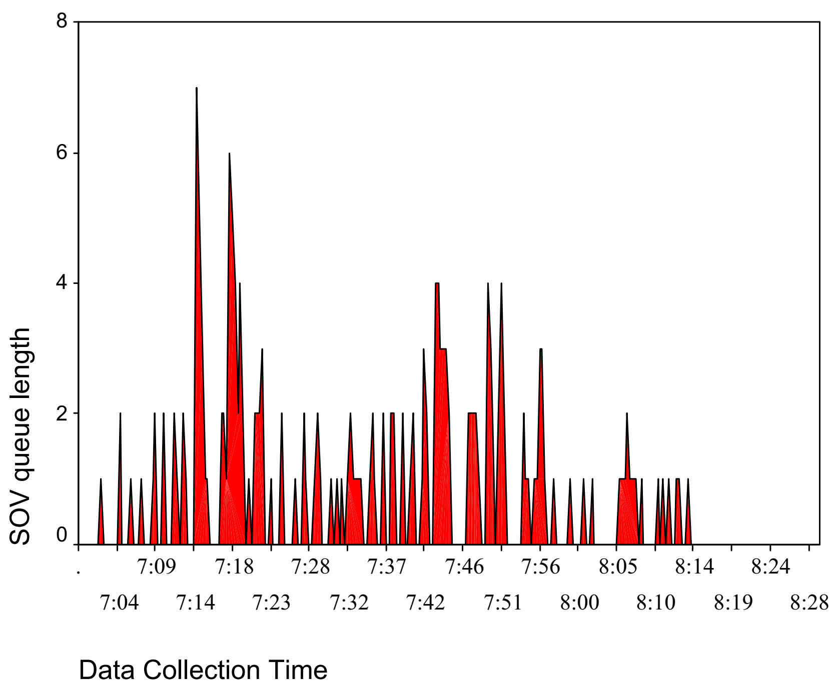
Wisconsin Avenue 2/3/2000 AM peak



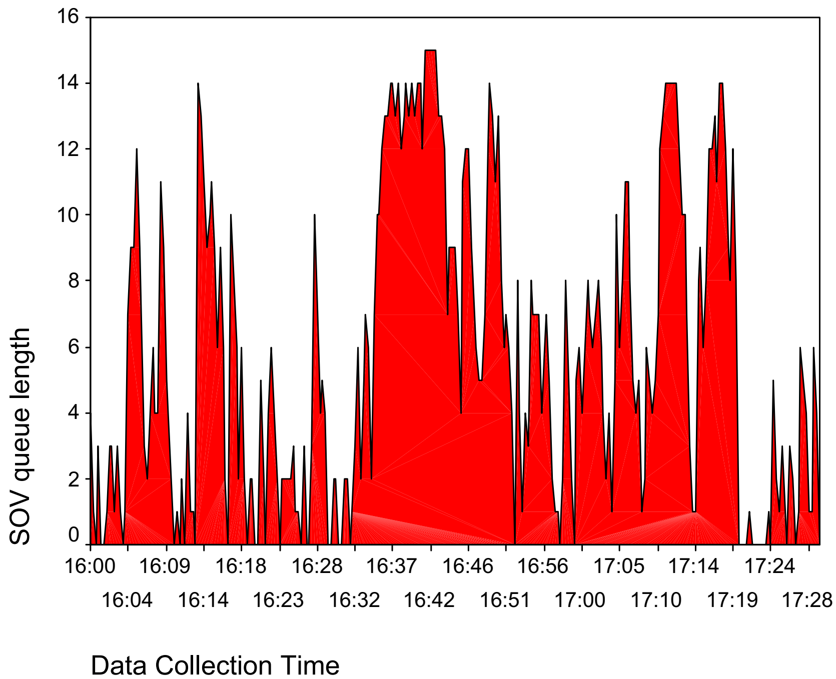
Wisconsin Avenue 2/8/2000 AM peak



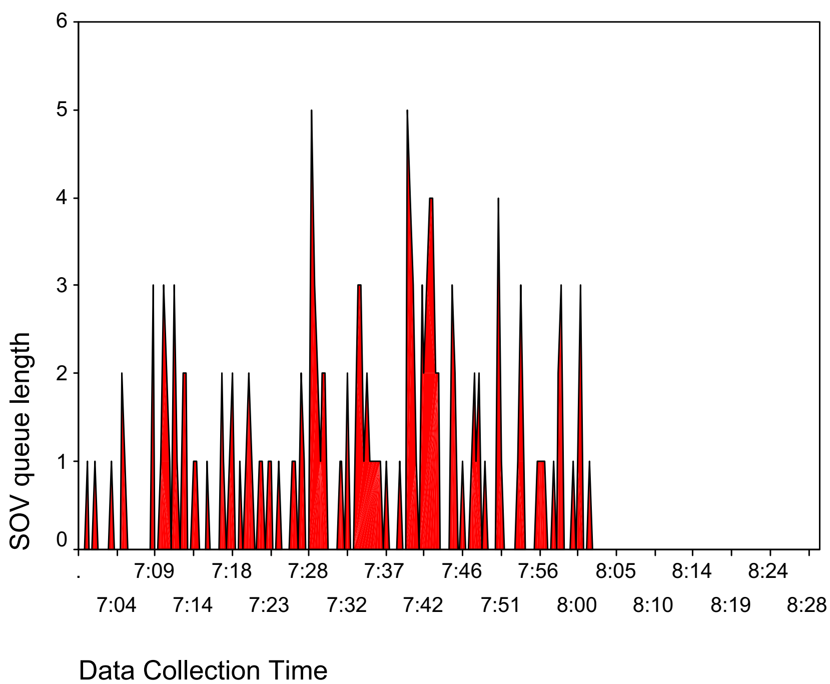
Wisconsin Avenue 2/9/2000 AM peak



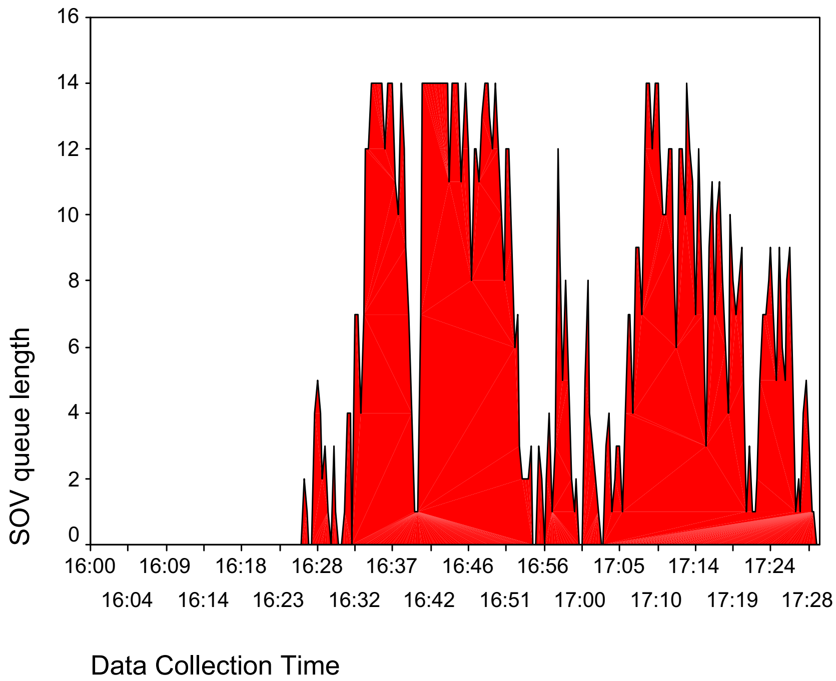
Wisconsin Avenue 2/9/2000 PM peak



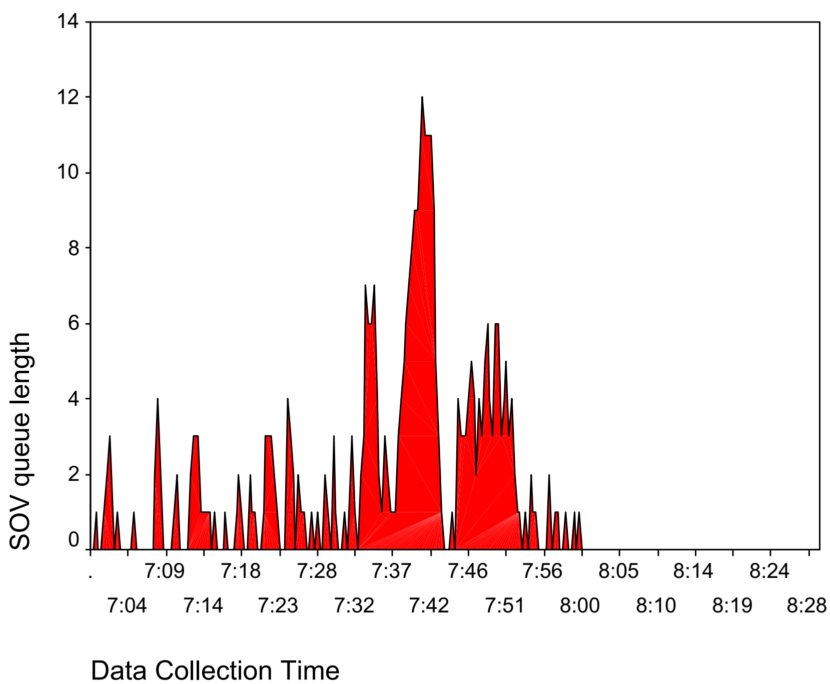
Wisconsin Avenue 2/10/2000 AM peak

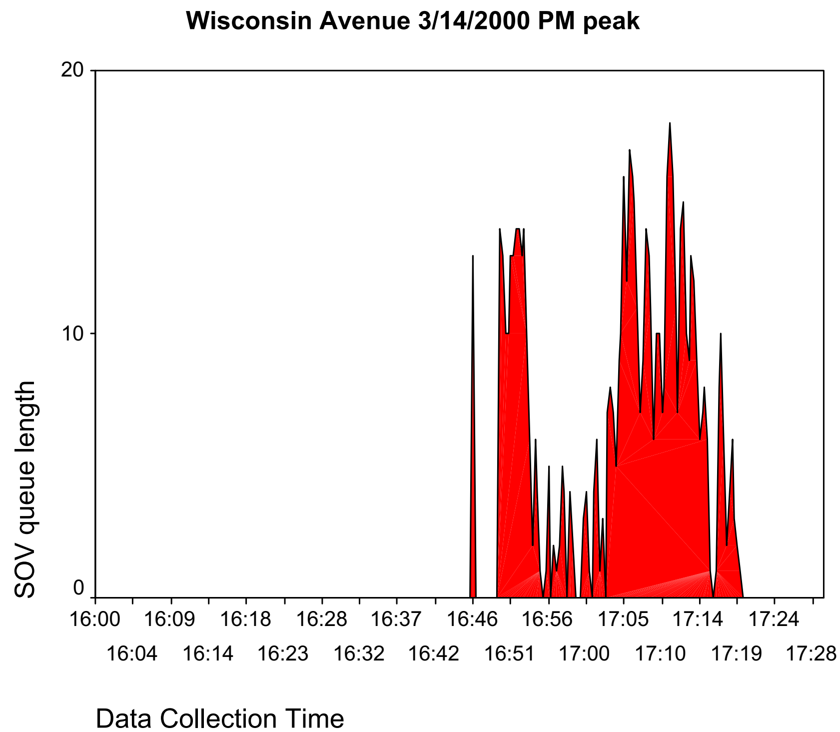


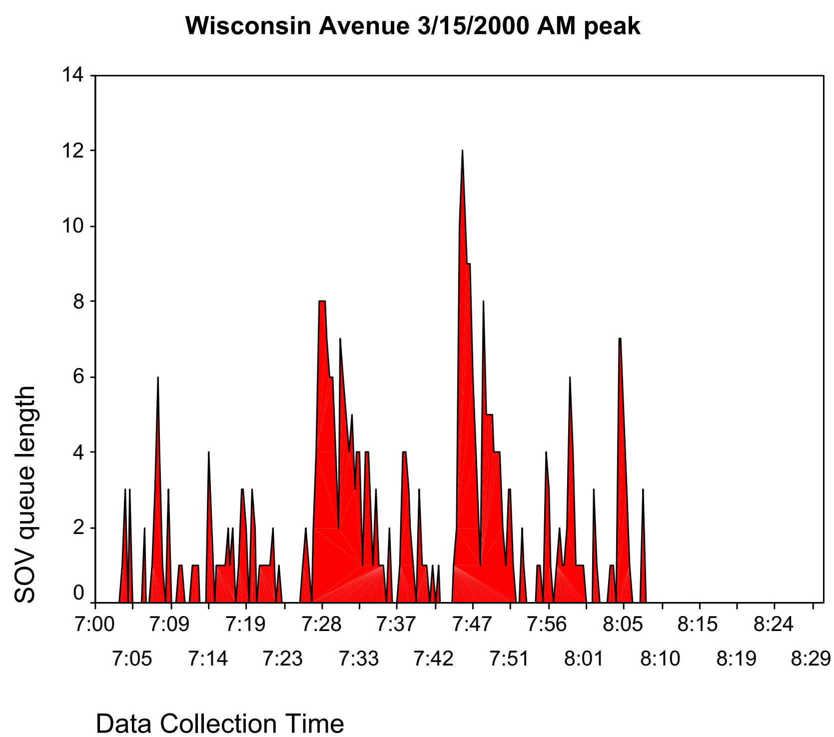
Wisconsin Avenue 2/10/2000 PM peak



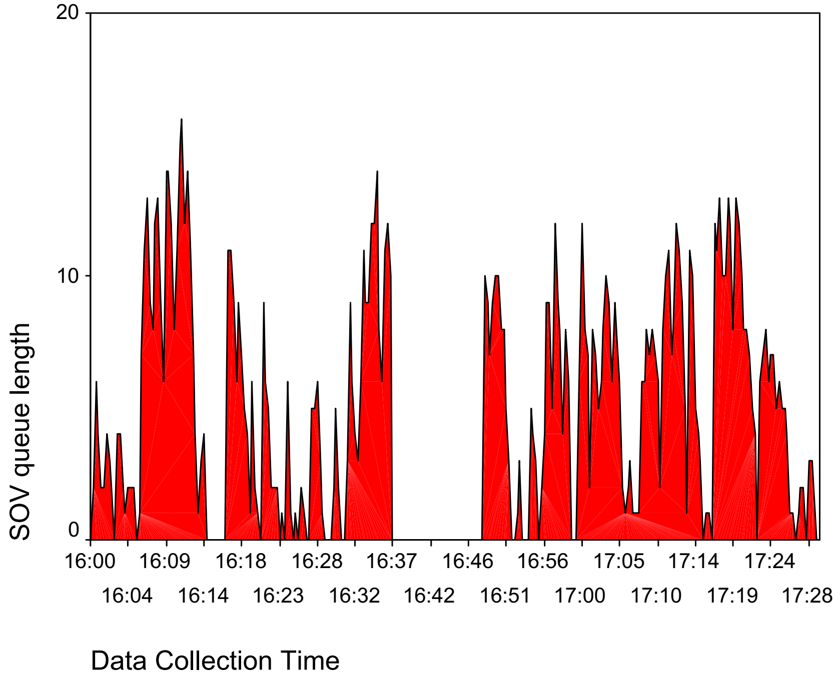
Wisconsin Avenue 3/14/2000 AM peak



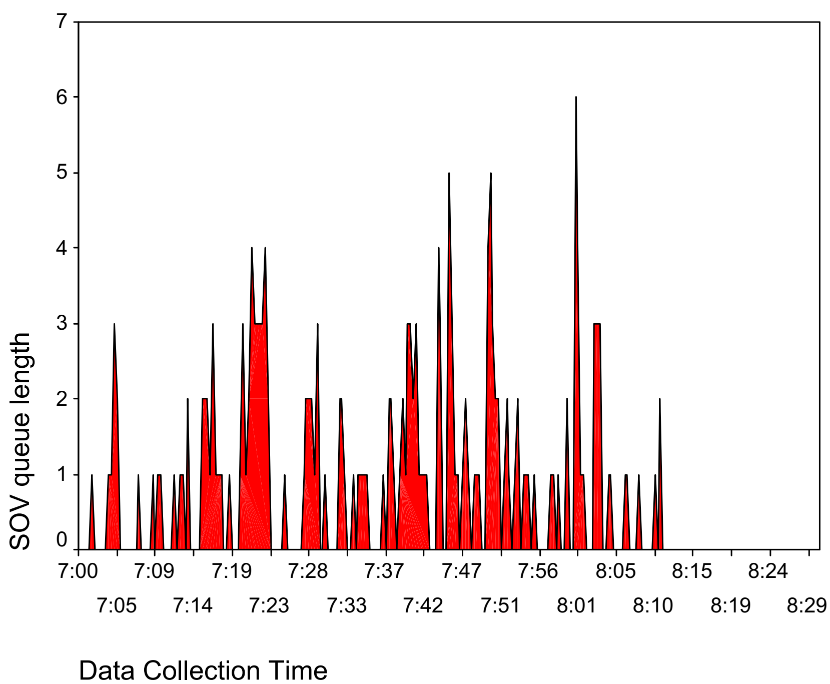

Wisconsin Avenue 3/14/2000 PM peak
SOV queue length
20
10
0
16:00
16:09
16:18
16:28
16:37
16:46
16:56
17:05
17:14
17:24
16:04
16:14
16:23
16:32
16:42
16:51
17:00
17:10
17:19
17:28
Data Collection Time

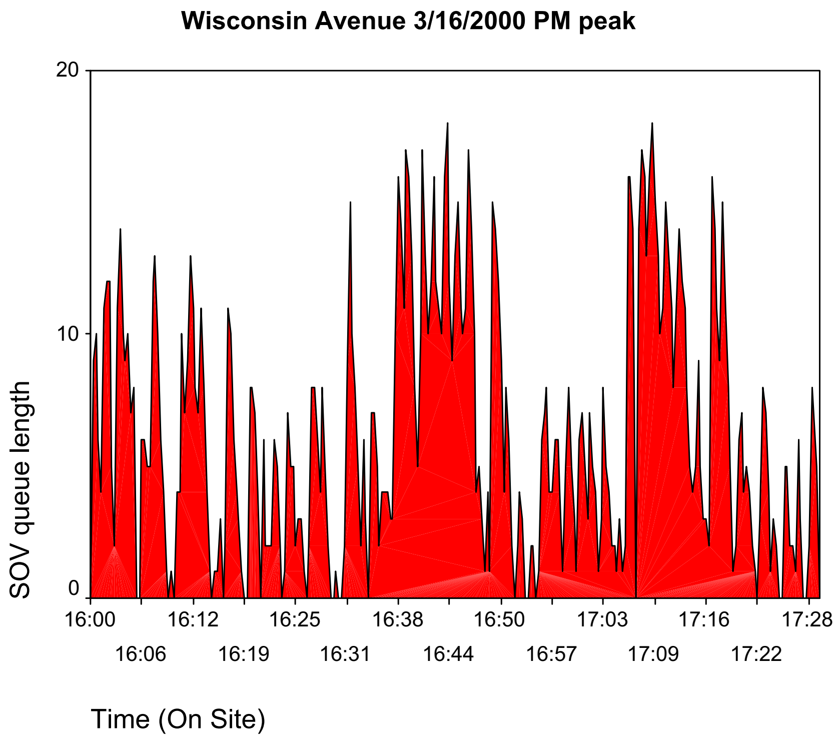

Wisconsin Avenue 3/15/2000 AM peak
SOV queue length
14
12
10
8
6
4
2
0
7:00
7:09
7:19
7:28
7:37
7:47
7:56
8:05
8:15
8:24
7:05
7:14
7:23
7:33
7:42
7:51
8:01
8:10
8:19
8:29
Data Collection Time

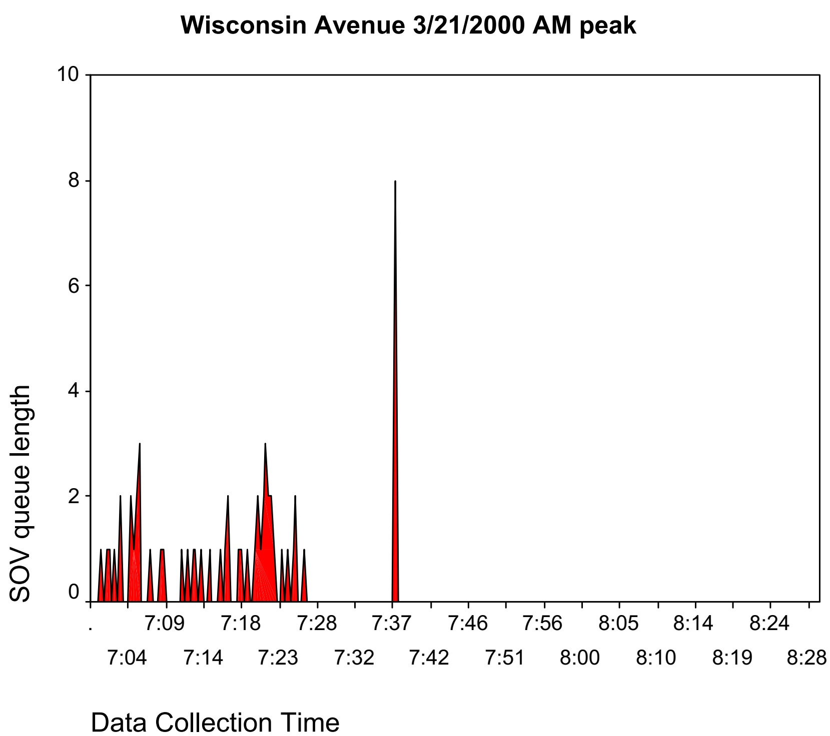
Wisconsin Avenue 3/15/2000 PM peak



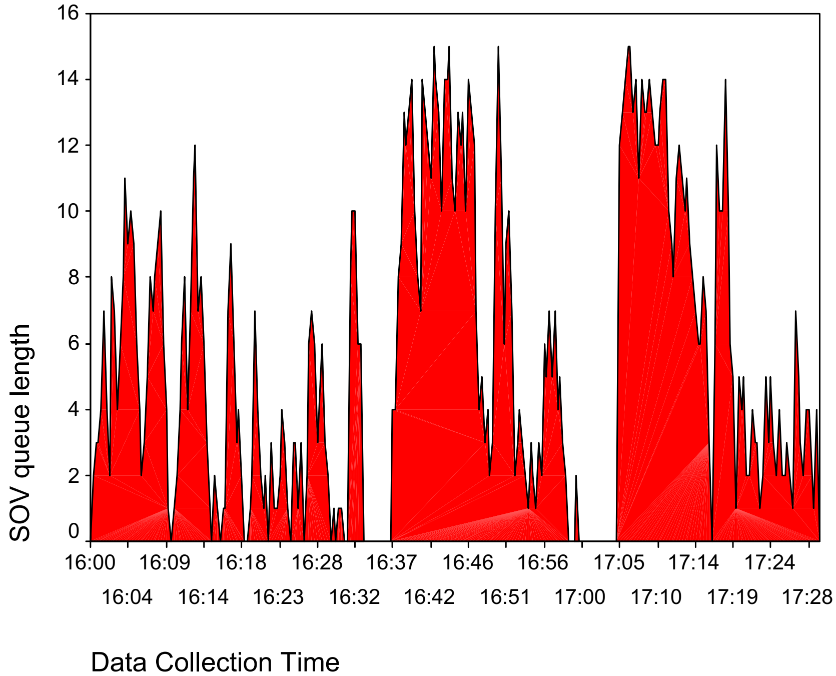
Wisconsin Avenue 3/16/2000 AM peak



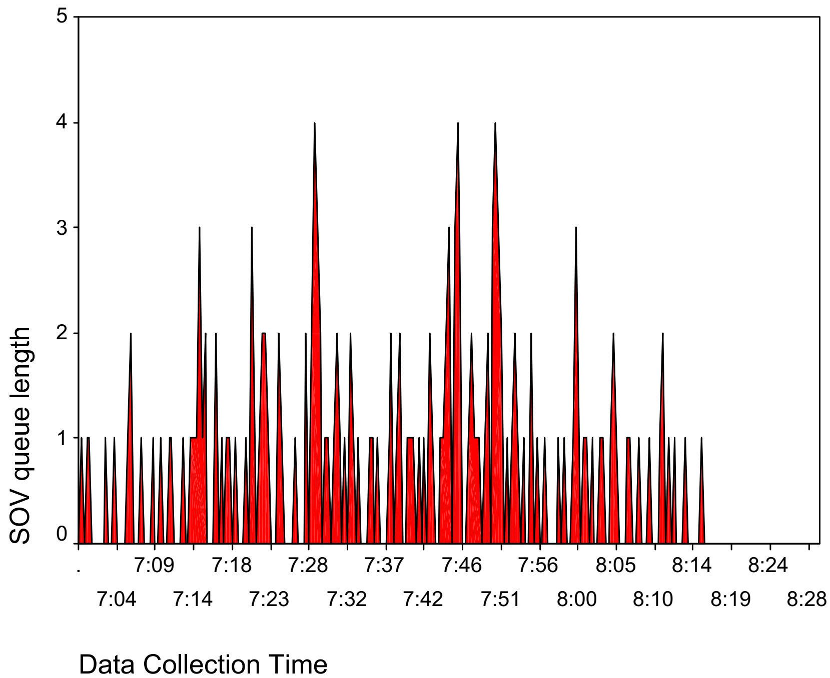

Wisconsin Avenue 3/16/2000 PM peak
20
10
0
SOV queue length
16:00
16:12
16:25
16:38
16:50
17:03
17:16
17:28
16:06
16:19
16:31
16:44
16:57
17:09
17:22
Time (On Site)


Wisconsin Avenue 3/21/2000 AM peak
10
8
6
4
2
0
SOV queue length
.
7:09
7:18
7:28
7:37
7:46
7:56
8:05
8:14
8:24
7:04
7:14
7:23
7:32
7:42
7:51
8:00
8:10
8:19
8:28
Data Collection Time

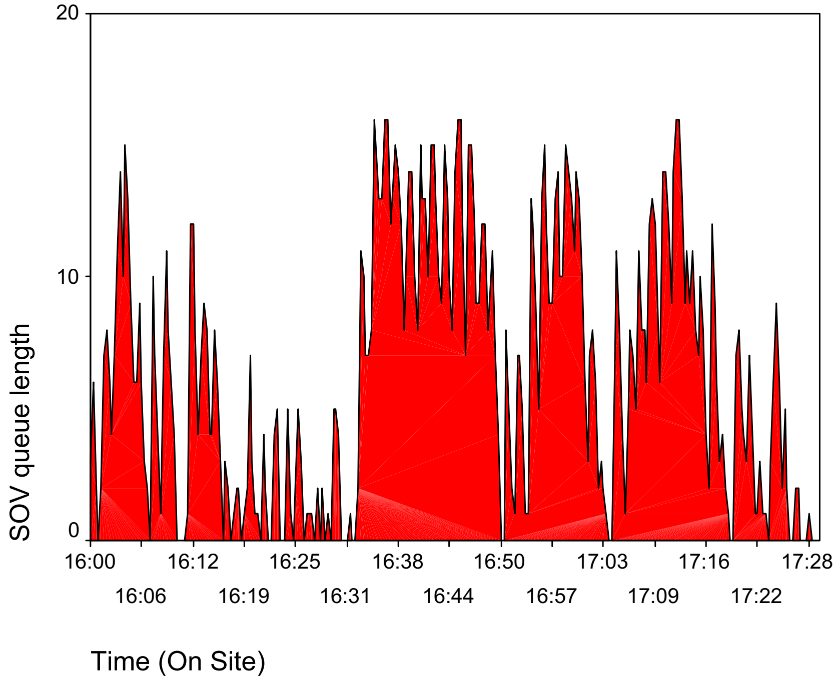
Wisconsin Avenue 3/21/2000 PM peak



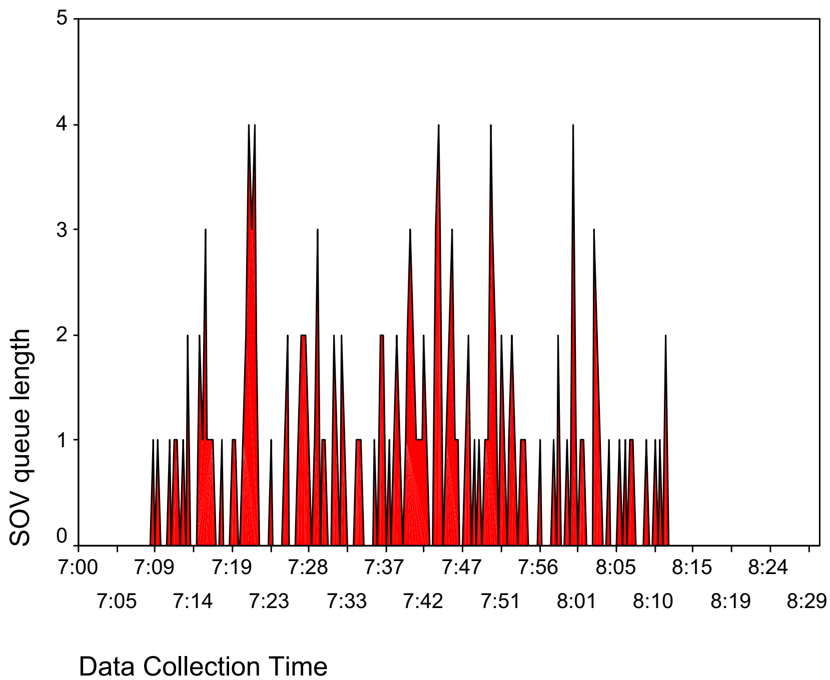
Wisconsin Avenue 3/22/2000 AM peak



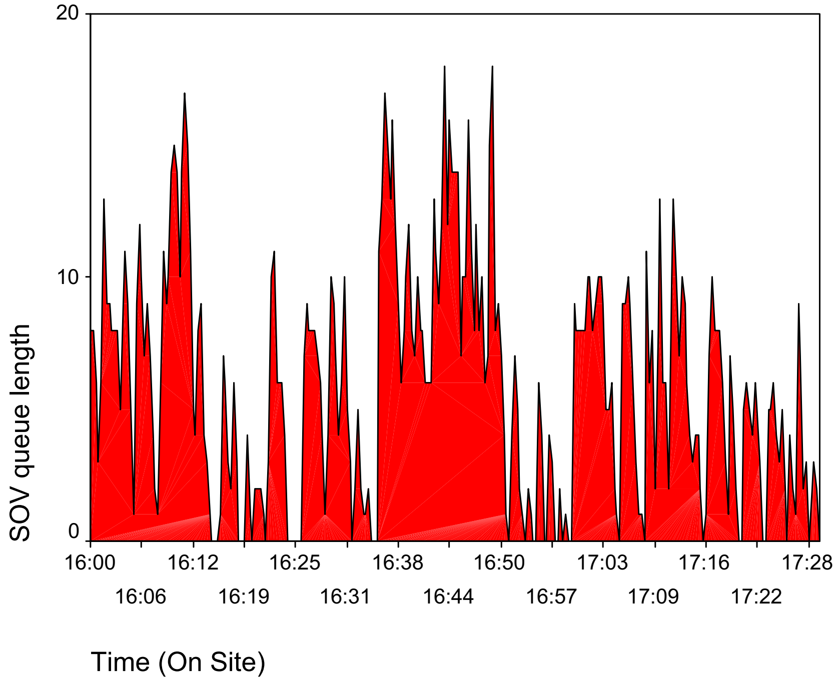
Wisconsin Avenue 3/22/2000 PM peak



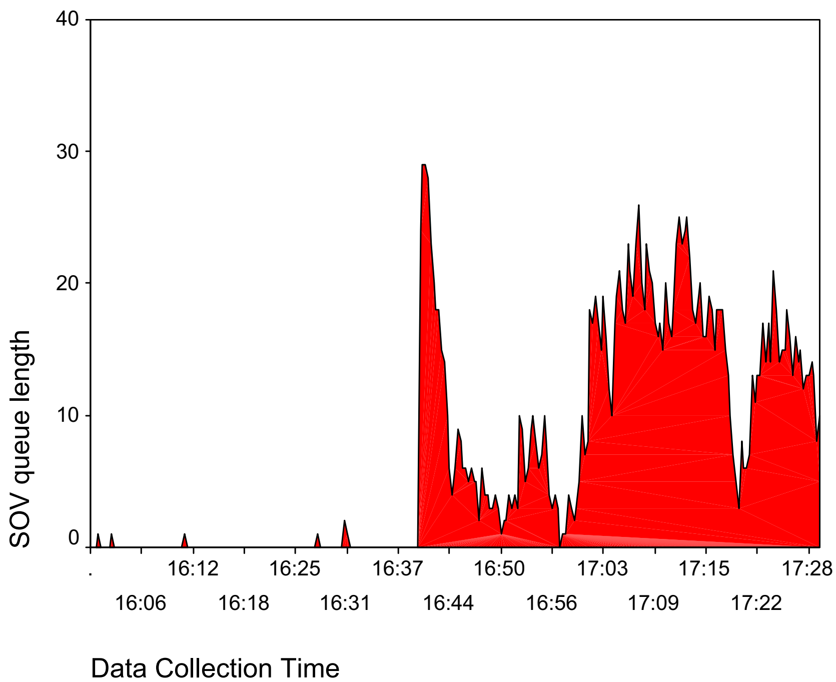
Wisconsin Avenue 3/23/2000 AM peak



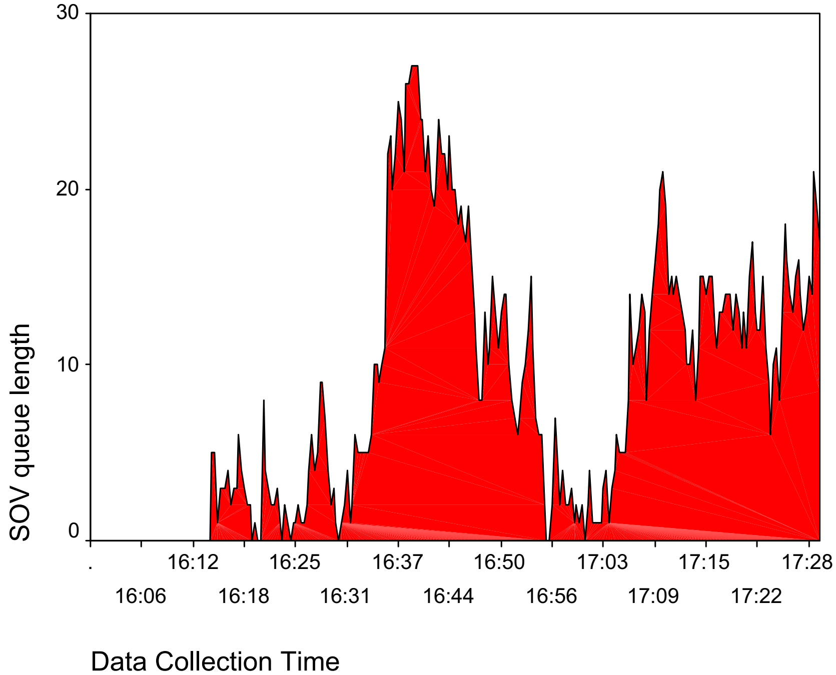
Wisconsin Avenue 3/23/2000 PM peak



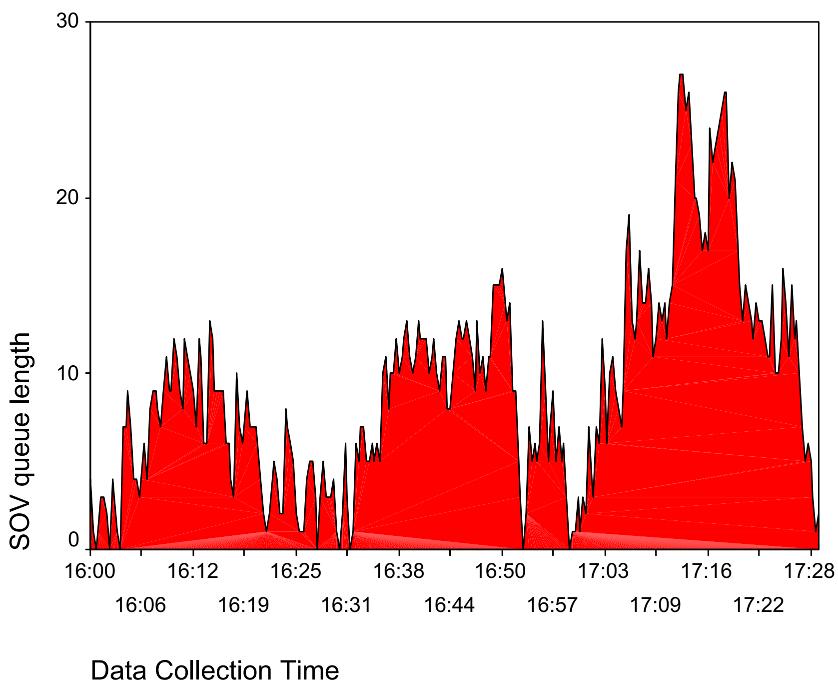
Greenfield Avenue 2/1/2000 PM peak



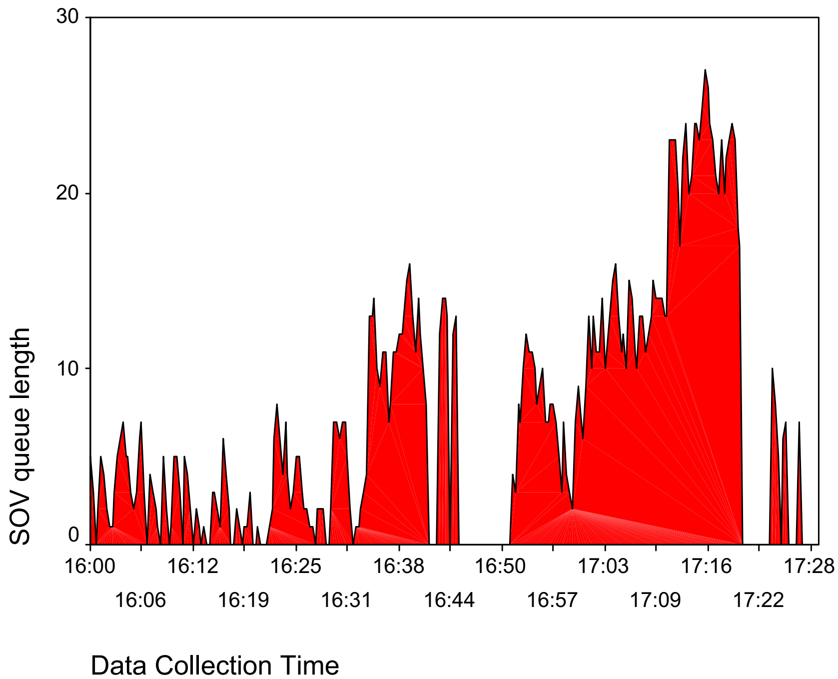
Greenfield Avenue 2/2/2000 PM peak



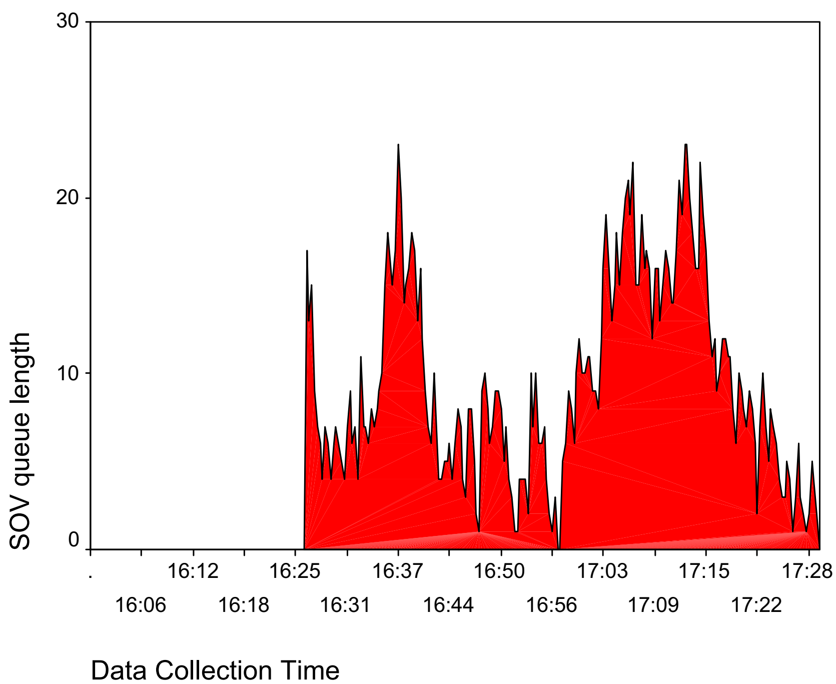
Greenfield Avenue 2/8/2000 PM peak

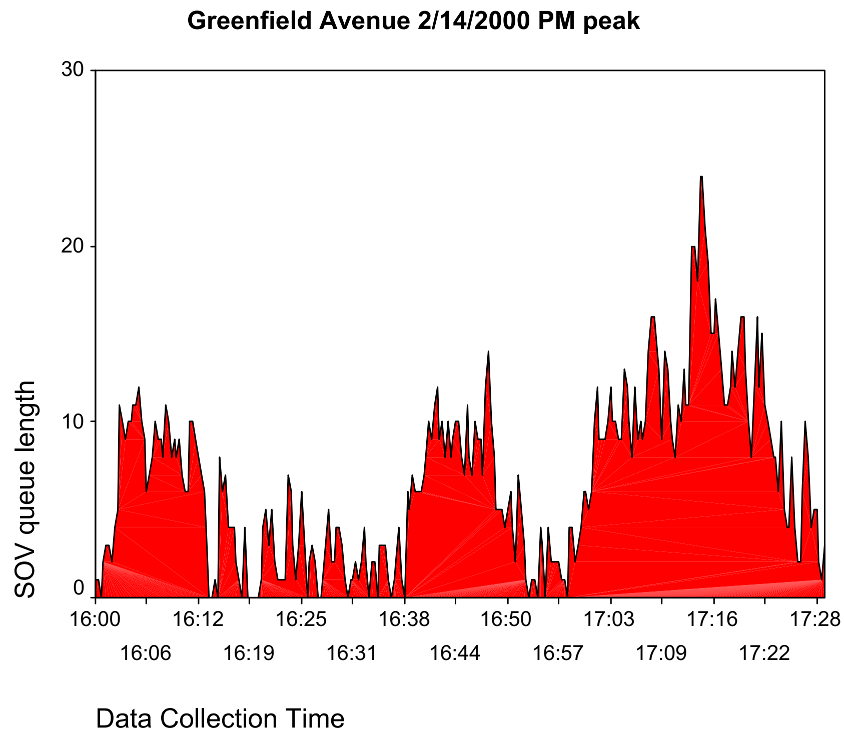


Greenfield Avenue 2/9/2000 PM peak

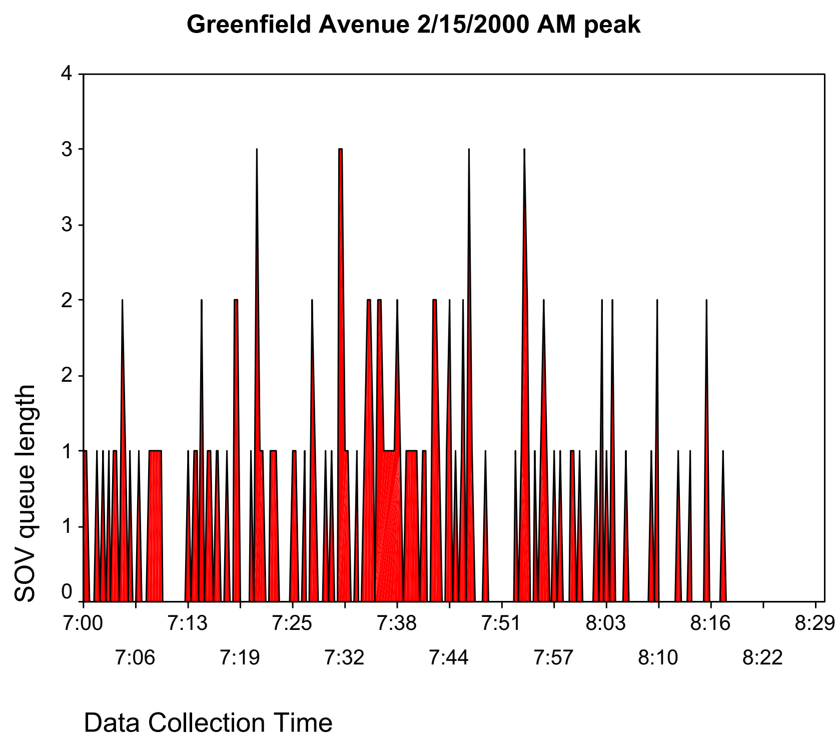


Greenfield Avenue 2/10/2000 PM peak

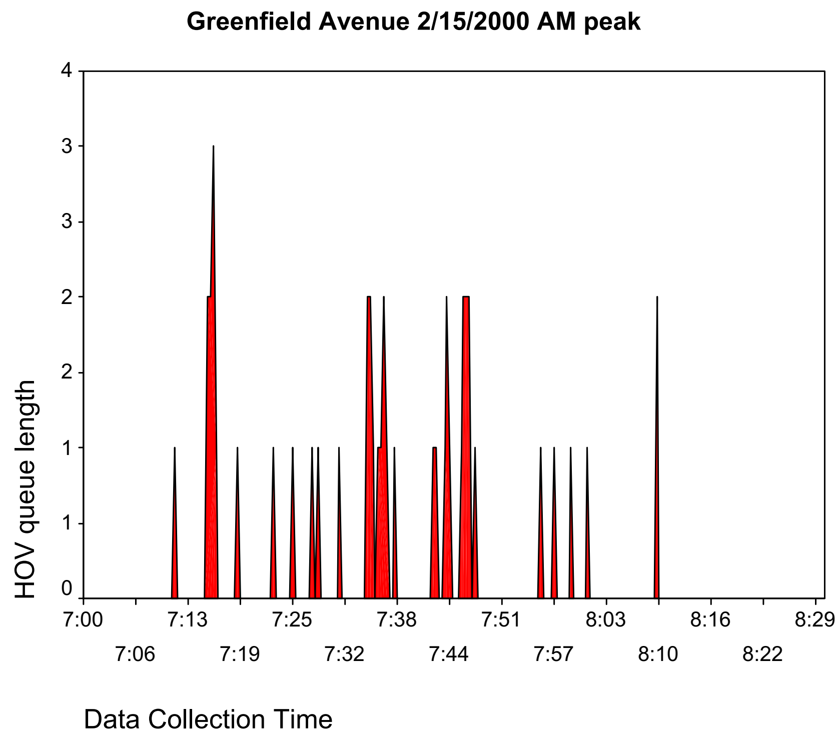


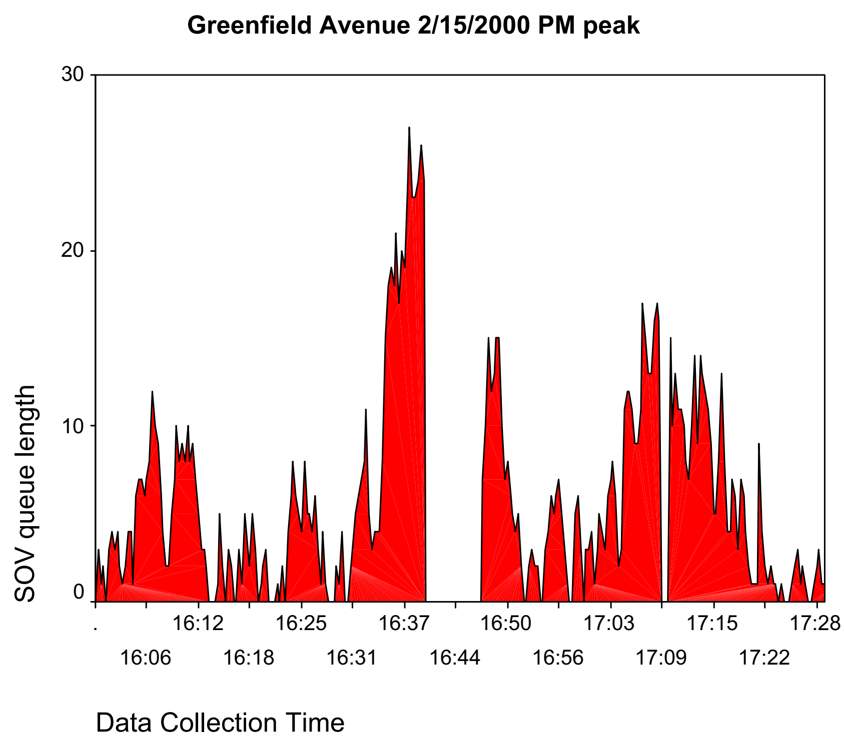


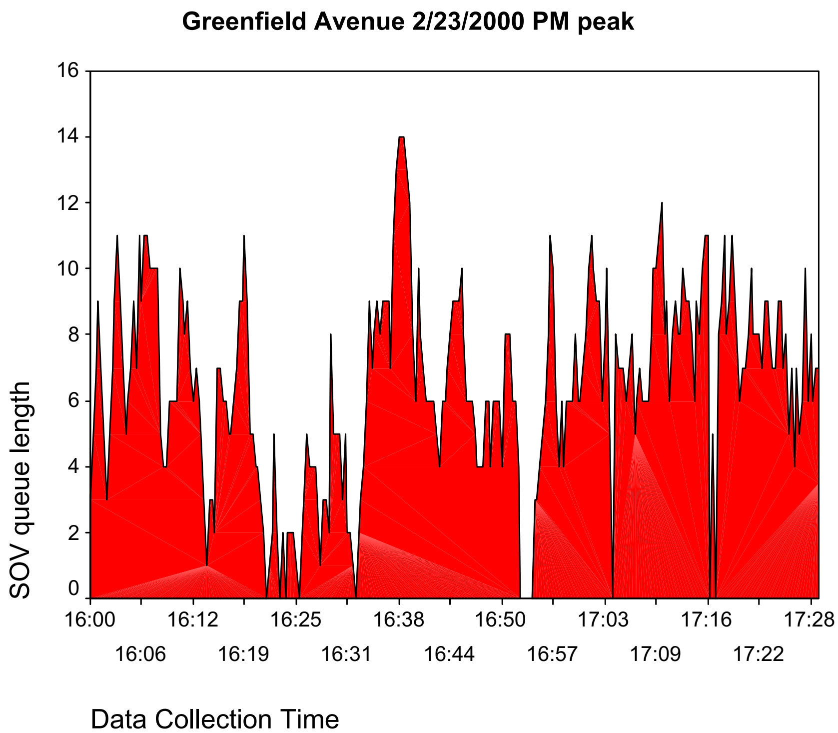
Greenfield Avenue 2/14/2000 PM peak
SOV queue length
30
20
10
0
16:00
16:06
16:12
16:19
16:25
16:31
16:38
16:44
16:50
16:57
17:03
17:09
17:16
17:22
17:28
Data Collection Time



Greenfield Avenue 2/15/2000 AM peak
SOV queue length
4
3
3
2
2
1
1
0
7:00
7:06
7:13
7:19
7:25
7:32
7:38
7:44
7:51
7:57
8:03
8:10
8:16
8:22
8:29
Data Collection Time


Greenfield Avenue 2/15/2000 AM peak
HOV queue length
4
3
3
2
2
1
1
0
7:00
7:06
7:13
7:19
7:25
7:32
7:38
7:44
7:51
7:57
8:03
8:10
8:16
8:22
8:29
Data Collection Time


Greenfield Avenue 2/15/2000 PM peak
SOV queue length
30
20
10
0
16:06
16:12
16:18
16:25
16:31
16:37
16:44
16:50
16:56
17:03
17:09
17:15
17:22
17:28
Data Collection Time


Greenfield Avenue 2/23/2000 PM peak
SOV queue length
0
2
4
6
8
10
12
14
16
16:00
16:06
16:12
16:19
16:25
16:31
16:38
16:44
16:50
16:57
17:03
17:09
17:16
17:22
17:28
Data Collection Time

Appendix B

Wisconsin Avenue Ramp Meter Operation, Afternoon Peak Period (4:00 pm to 5:30 pm) Wednesday, February 9, 2000.

Introduction

The present Appendix provides detailed information on the operation of the Wisconsin Avenue ramp meter during the afternoon peak period (4:00 pm to 5:30 pm) on Wednesday, February 9, 2000. Information presented herein was compiled from data collected through pavement-embedded loop detectors on the ramp and the adjacent mainline lanes.

The Wisconsin Avenue ramp was chosen for this detailed presentation, because a complete set of traffic data was available at this location during the study period; the location coincides with cutline #6 for which additional information is presented elsewhere in the report.

Ramp metering settings for the presented period are shown in **Table B-1** below.

Table B-1. PM Peak Period Ramp Metering Settings-Wisconsin Avenue Ramp

Interval Times

Rate	1	2	3	4	5	6
Green	2.5	2.5	2.5	2.5	2.5	2.5
Yellow	0.0	0.0	0.0	0.0	0.0	0.0
Red	2.5	3.0	3.4	3.8	4.5	6.0

Thresholds

Rate	1	2	3	4	5	6
Volume	1700	1800	1900	2050	2150	2250
Occupancy	19	21	24	27	30	33
Speed	55	50	45	40	35	30
Ramp Occupancy	50	40	35	30	25	20

Time Of Day(TOD)

Time	
15:00	Must/May
15:15	Traffic Responsive Min Plan 1
15:15	Most Restrictive
18:00	Must/May
18:15	Non-Metering

According to information presented in **Table B-1**, under the "*Interval Times*" section of the table, six metering Rates (Rates 1-6) were pre-programmed for the Wisconsin Avenue ramp. All six metering rates allowed for 2.5 seconds of Green; no Yellow indication was present; rates differed in Red interval durations. Rate 1 was the least restrictive, with 2.5 seconds of Red; rate 6 was the most restrictive with 6.0 seconds of Red.

Table B-2 presents ramp metering plan selection information, extracted from the Milwaukee FTMS MONITOR program "Field Equipment Software Reference Manual," prepared by JHK & Associates in 1994. The "TOD[1]" Plan Selection was in effect during the analyzed period ("*Plan Selection"* choice #3).

Under this **Table B-2** choice, afternoon peak ramp metering operation was operational during the hours indicated in the *"Time Of Day (TOD)"* part of **Table B-1**. Explanations of terms are provided below.

15:00 Must/May Explanation: No ramp metering was in effect before 3:00 pm. Ramp metering started at 3:00 pm, if traffic conditions met any of the preset ramp metering controller thresholds (see explanations below) in **Table B-1**.

15:15 Responsive Min Plan 1 Explanation: Metering rate 1 (the least restrictive rate, with a Red duration of 2.5 seconds) would be in effect at this time if traffic did not meet any of the thresholds for a more restrictive metering rate (even if metering rate 1 thresholds were not met).

15:15 Most Restrictive Explanation: Metering rate selection was based on the Volume, Occupancy, or Speed threshold that required the most restrictive rate (longer red interval duration). However, all of these thresholds would be overridden, if queue occupancy values were high enough to dictate a less restrictive metering rate, so the ramp queue could be dissipated before it spilled into an adjacent arterial.

18:00 Must/May Explanation: If traffic conditions met any of the thresholds at 6:00 pm, ramp metering would have continued, otherwise it would have terminated at this time.

18:15 Non-Metering Explanation: Ramp metering would have been turned off at 6:15 pm, regardless of traffic conditions.

Metering rate choice depended on four ramp metering inputs: mainline volume, occupancy and speed, and ramp queue occupancy values indicated under the "*Thresholds*" part of **Table B-1**. Volume, occupancy and speed summary information was received from mainline pavement-embedded detectors; ramp queue occupancy information was received from detectors embedded on the ramp.

[1] TOD: Time of Day

Table B-2. Ramp Metering Plan Selection Information.

PLAN SELECTION LEGEND

Plan Selection

0	All Plan Selection Levels Inhibited
1	Manual Plan Selection
2	Central Plan Selection
3	TOD Plan Selection

Plan Request

0	No Plan Selection
1 - 6	Fixed Ramp Metering Plan
7	Non-Metering
9	Must/May Meter, Ramp Metering Plan 1
10	Must/May Meter, Traffic Responsive Ramp Metering Plan
11 - 16	Traffic Responsive Ramp Metering Plan, Minimum Plan

Current Plan

1 - 6	Ramp Metering Plan
7	Non-Metering

Current Mode *

17	Select Most Restrictive Plan Based on Volume, Occupancy and Vehicle Speed
18	Select Least Restrictive Plan Based on Volume, Occupancy and Vehicle Speed
19	Select Plan Based on Volume
20	Select Plan Based on Occupancy
21	Select Plan Based on Vehicle Speed

* Note: Current Mode is set to reflect Plan Selection mode.

Plan Reason

0	No Plan Selection
1	Queue Failure
2	Fixed Plan
3	Fixed Plan, Queue Override
4	Holiday/Exception Day
5	Clock Failure
6	Traffic Responsive, Minimum Plan
7	Traffic Responsive, Most Restrictive, Volume
8	Traffic Responsive, Most Restrictive, Occupancy
9	Traffic Responsive, Most Restrictive, Vehicle Speed
10	Traffic Responsive, Least Restrictive, Volume
11	Traffic Responsive, Least Restrictive, Occupancy
12	Traffic Responsive, Least Restrictive, Vehicle Speed
13	Traffic Responsive, Volume
14	Traffic Responsive, Occupancy
15	Traffic Responsive, Vehicle Speed
16	Traffic Responsive, Queue Override

Current Interval

0	No Interval (Non-Metering)
1	Metering Initial delay
2	Metering Green Delay
3	Metering Yellow Delay
4	Red
5	Red Extension
6	Red Rest
7	Red Countdown
8	Green
9	Yellow

Metering rate choice example
For example, if detected traffic conditions were between the values shown for Rates 2 and 3, say, volume 1850 vph, occupancy 22%, speed 49 mph, ramp queue override 37%, then ramp metering rate 2 would have been chosen (Green 2.5 sec., Red 3.0 sec.) If all other values remained the same, but speed was 42 mph, which was between the thresholds for rates 3 and 4, rate 3 would have been chosen, the most restrictive rate that the specific traffic conditions warranted. If all other values in the original example remained the same, but the ramp queue occupancy value was 43%, ramp metering rate 1 would have been chosen; ramp occupancy was programmed to override mainline input demands for more restrictive metering rates.

Description of Appendix figures
Figures B-1 through **B-8** in this Appendix are intended to provide a detailed view into the operation of the Wisconsin Avenue ramp on Wednesday, February 9, 2000, between 4:00 pm and 5:30 pm. These figures use the same time axis; they can be superimposed on one-another in order to provide insights into which thresholds were met at specific times, why a certain metering rate was chosen, and how metering rates affected ramp queue length.

Figures B-3 through **B-6** are based on 20-second mainline speed, volume and occupancy data that were averaged using a moving average of six observations (two minutes); **Figure B-6** represents 20-second ramp occupancy observations. Thresholds for each metering rate are marked on each of these graphs for easy reference.

Speed-volume and speed-occupancy graphs (**Figures B-9** and **B-10**) are provided for each quarter hour during this peak period. Similar graphs (**Figures B-11** and **B-12**) are provided for prevailing weekday afternoon peak conditions at this location based on information collected at the same location during all data collection days: February 1, 2, and 3 (week 1), February 8, 9, and 10 (week 2) March 14, 15, and 16 (week 3), and March 21, 22, and 23 (week 4).

A matrix graph (**Figure B-13)** relating volume, speed and occupancy at this location is provided to establish the relationship between all three traffic parameters. Each of the three distinct graphs on the matrix is presented separately on a larger scale for easier reference (**Figures B-14** through **B-16**).

Except for graphs indicating that they are based on two-minute average data, all other information is based on data collected every 20-seconds.

Description of ramp operation
The Wisconsin Avenue ramp queue length is shown in **Figure B-1** (the shaded area represents veh-min of delay). Maximum recorded queue length was 15 vehicles; there were many instances during the peak period that queue lengths were 12 or more vehicles. A characteristic see-saw pattern emerged throughout the peak period, when periods of longer queues were followed by periods of much shorter queues (1 or 2 vehicles-long).

The most persistent presence of long queues was observed approximately between 16:35 and 16:45.

The reason for choosing a certain ramp metering rate during a specific time can be seen in **Figure B-2**. For example, between 16:00 and 16:05, when the "most restrictive" plan was in effect (see **Table B-1**), plan reason #7 (see y-axis) controlled the metering rate. Plan reason #7 corresponds to the entry "Traffic Responsive, Most Restrictive, Volume" in **Table B-2**, indicating that mainline traffic volume was the first ramp metering input that crossed the threshold corresponding to the most restrictive metering rate.

Speeds during this period were 48-50 mph (**Figure B-3**) corresponding to ramp metering rate 2, volumes crossed into metering rates 3 and 4 (**Figure B-4**), mainline occupancies were well below 19% (**Figure B-5**) required for rate 1, and ramp occupancy did not exceed 25% (**Figure B-6**), thus queue override was not called for. The most restrictive metering rate was therefore dictated by mainline traffic volumes. The metering rate in effect at any time is shown in **Figure B-7**—rates 3 and 4 were in effect during these five minutes.

At approximately 16:05, ramp queue length increased to 12 vehicles (**Figure B-1**) within a short period of time, thus ramp occupancy increased as well. **Figure B-2** indicates that between 16:05 and 16:08, plan reason #16 controlled the metering rate ("Traffic Responsive, Queue Override" in **Table B-2**). Indeed, ramp occupancy exceeded 70% (**Figure B-6**), overriding all other inputs, and setting the least restrictive metering rate 1 (see **Figure B-7**) in order to dissipate the ramp queue.

Figure B-8 provides a detailed presentation of metering rates based solely on ramp occupancy. These rates governed only during the time periods that they were less restrictive than the rates demanded by mainline metering inputs.

Although mainline speed and occupancy did not change much during these three minutes, mainline volumes would have demanded rate 5 during this interval, had it not been for ramp queue occupancies overriding this demand and setting rate 1 instead. Thus, more vehicles were released onto the freeway (due to the queue override) at a time when the freeway could handle fewer vehicles because a heavy traffic volume was present.

Ramp operation summary
The most frequent reason for metering rate selection was mainline traffic volume (reason #7 **Table B-2**), which occurred 16 times, for a total of 45 minutes (see **Figure B-2**). Ramp queue override (reason # 16) occurred 14 times during the peak period, for a total 36 minutes. Mainline speed (reason #9) decided metering rate on four occasions for a total of 5 minutes, minimum plan values (reason #8) occurred five times for a total of 2 minutes; and mainline occupancy (reason #11) on one occasion for a total of 2 min.

Table B-3 summarizes how long each metering rate remained in effect when any of the most commonly used plan reasons (mainline volume, ramp queue override and mainline speed) was present. For example, when queue override was the plan reason, metering rate 1 was in effect for a total of 27.7 minutes, metering rate was in effect for 4.3 min., etc.

Table B-3. Reason for Metering Rate Selection and Metering Rate Duration (minutes)

Plan Reason	Current Plan						
	1	2	3	4	5	6	Total
Mainline Volume	0.0	6.0	21.3	14.7	3.3	0.0	45.3
Queue Override	27.7	4.3	2.3	1.3	0.3	0.0	36.0
Mainline Speed	0.0	0.0	0.0	2.7	1.7	0.7	5.0

Traffic characteristics in the vicinity of the ramp
Figures B-9 and **B-10** present mainline speed-volume and speed-occupancy relationships during the analyzed afternoon peak period. Forty-five observations, representing a 20-second interval each are plotted in each 15-minute chart.[2] The Figures indicate that speeds remained above 50 mph, and occupancies did not exceed 22% between 16:00 and 17:00; congestion was present for much of the last 30 minutes.

Figures B-11 through **B-16** present similar information at the same location, based on the 12 afternoon peak periods of the study data collection days. This information is intended to provide a background of traffic conditions at the analyzed location, for comparisons with the afternoon peak period of February 9, 2000, and fine-tuning ramp metering parameters.

Figures B-11 and **B-12** indicate that it was not uncommon for the mainline to be congested during any given quarter of an hour of the afternoon peak period. Congestion often was even more pronounced than during the February 9 afternoon peak, with lower speeds and higher occupancies.

Figure B-13 presents all two-way relationships between mainline volume, speed and occupancy. The peak period volume-speed relation is presented in **Figure B-14**, occupancy-speed in **Figure B-15** and occupancy-volume in **Figure B-16**.

Observations about the February 9, 2000 pm peak period
Overall, much wider ranges of mainline volume, speed and occupancy occurred near the Wisconsin Avenue ramp during the twelve field data collection dates, than the corresponding ranges measured during the February 9 afternoon peak (**Figures B-11** and **B-12**). Congestion was present quite frequently, throughout the afternoon peak period.[3] The most congested part of the afternoon peak was between 16:45 and 17:30.

When mainline volume controlled metering rate, metering rates 3-5 were implemented early-on, rates 2-3 between 16:18 and 16:37, and rates 3-5 later during the peak period.

[2] As expected, a wider variability is present among 20-sec observations than among 2-min averaged observations in **Figures B-3** through **B-6**. For example occupancy values exceeding 40% are present-averaged values do not exceed 24%.
[3] These graphs are based on 20-second data, thus each 15-minute graph is based on 540 observations. Darker parts of the graphs indicate the most frequently occurring values.

Ramp queues built very fast and ramp occupancy values rose sharply very often. These ramp occupancy values exceeded 40%, thus the fastest metering rate 1 was set (green 2.5 sec, red 2.5 sec) during 75% of the duration of ramp metering under queue override control. Under this metering rate, queues dissipated quickly and ramp metering control returned to the volume, speed or occupancy thresholds.

All abrupt changes (changes that skip two or more metering rates) to metering rate 1 during the peak period were the result of queue override taking effect (**Figure B-7**). Unfortunately, queue override most often occurred during periods that mainline volume, occupancy or speed thresholds would have demanded more restrictive metering rates. For example, between 16:37 and 16:47, when a queue override was in effect, mainline volumes would have set a metering rate 4 or 5 (**Figure B-4**).

Ramp queues could build up very fast. In one instance, a 14-vehicle queue built up at 16:13:20, within 20 seconds. This corresponded to an arrival rate of one vehicle every 1.4 seconds (this arrival rate is too fast to be realistic—some rounding error is involved due to sampling at discrete time intervals). The arrival rate of one vehicle every 2 seconds that occurred at 16:17:20, when a queue of 10 vehicles occurred within the next 20 seconds is within reason.

The fastest queue dissipation rate was one vehicle every five seconds (metering rate 1) and the slowest one vehicle every 8.5 sec (metering rate 6). Thus, if a sustained arrival rate of one vehicle every two seconds occurred at any time during the metered period, ramp queue spillover could not have been avoided.

If no ramp queue spillover into adjacent surface streets is to be allowed, queue override must remain in effect, allowing a less restrictive metering rate when the ramp is about to overflow. If, during the same time period, mainline congestion warrants more restrictive metering rates, a compromise must be found between these competing ramp metering goals. A reasonable compromise would be to attempt to precisely manage ramp queue length, avoiding ramp spillover, but also avoiding complete ramp queue dissipation. If this compromise is successfully met, the "valleys" of **Figure B-1** will not reach queue lengths of zero vehicles when mainline volumes require more restrictive metering rates, but will remain at values of, for example 5 or 6 vehicles (thus the shaded part of **Figure B-1** will cover a larger portion of the Figure). This task is quite challenging and perhaps not worth pursuing for the following reasons:

1. Overall, ramp queue delay during the afternoon peak was 4.9% of all freeway delay during the before period, and 7.6% during the after period. The proposed change in ramp metering strategy is likely to affect a very small percentage of ramp delay, representing a negligible percentage of total delay. Labor (and perhaps additional hardware) costs to achieve the proposed strategy may not be justified.
2. The arrival rates of one vehicle every 1.4-2.0 seconds, observed on a couple of occasions following periods when no vehicles were present on the ramp were much higher than the fastest ramp metering rate of one vehicle every 5 seconds.

> Thus, the possibility of ramp overflow would increase if ramp queues were intentionally not allowed to completely dissipate and such an arrival rate were to materialize.

The benefit of spacing out on-ramp vehicle platoons is reaped regardless of how often metering rate 1 is used. However, if mainline congestion is very high when the least restrictive metering rate is set, a number of vehicles released from the stop line would be clustered at the merge area.

Figure B-1. Ramp Delay & Queue Length Wisconsin Ave. 2/9/2000

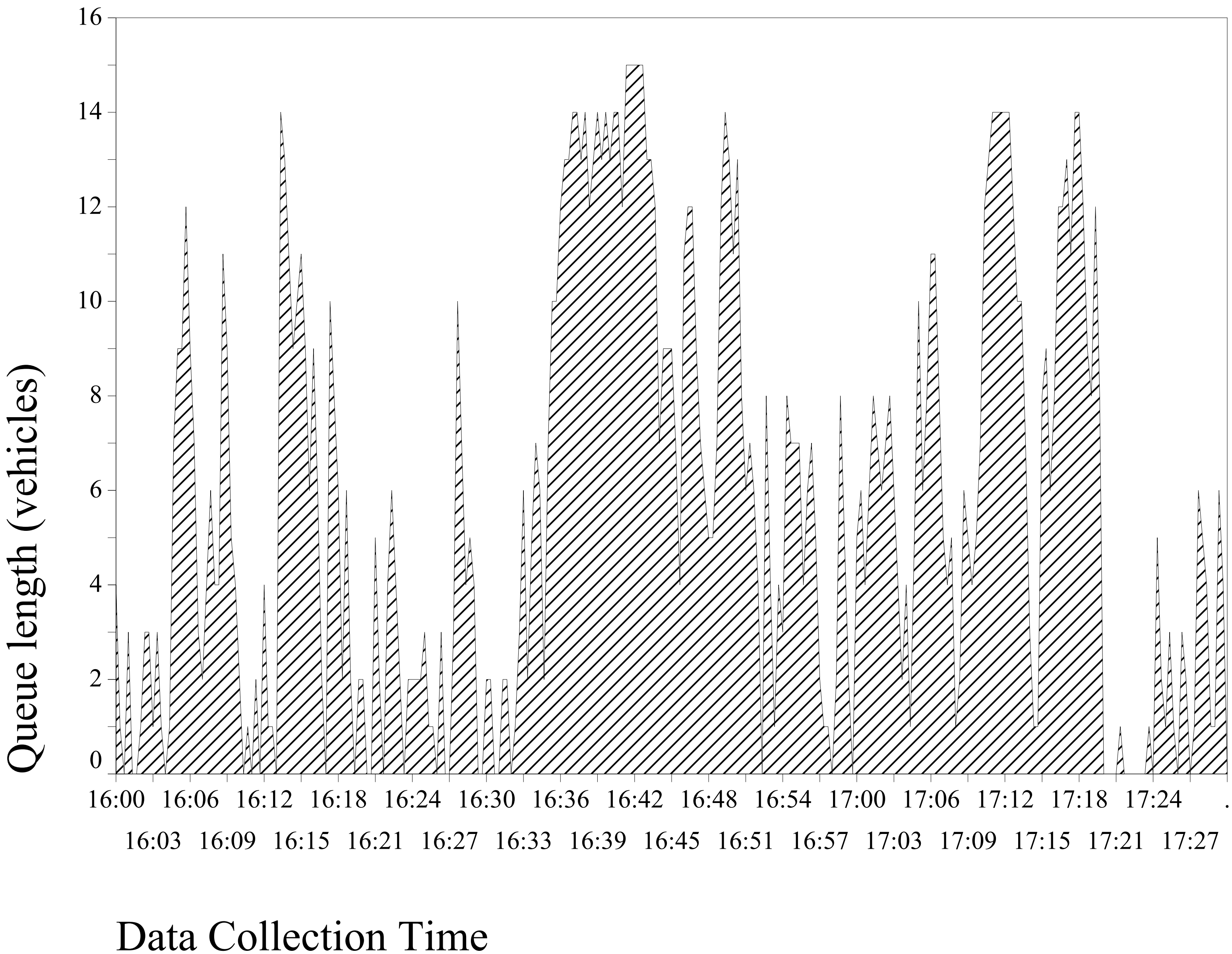


Figure B-2. Reason for Setting Metering Rate

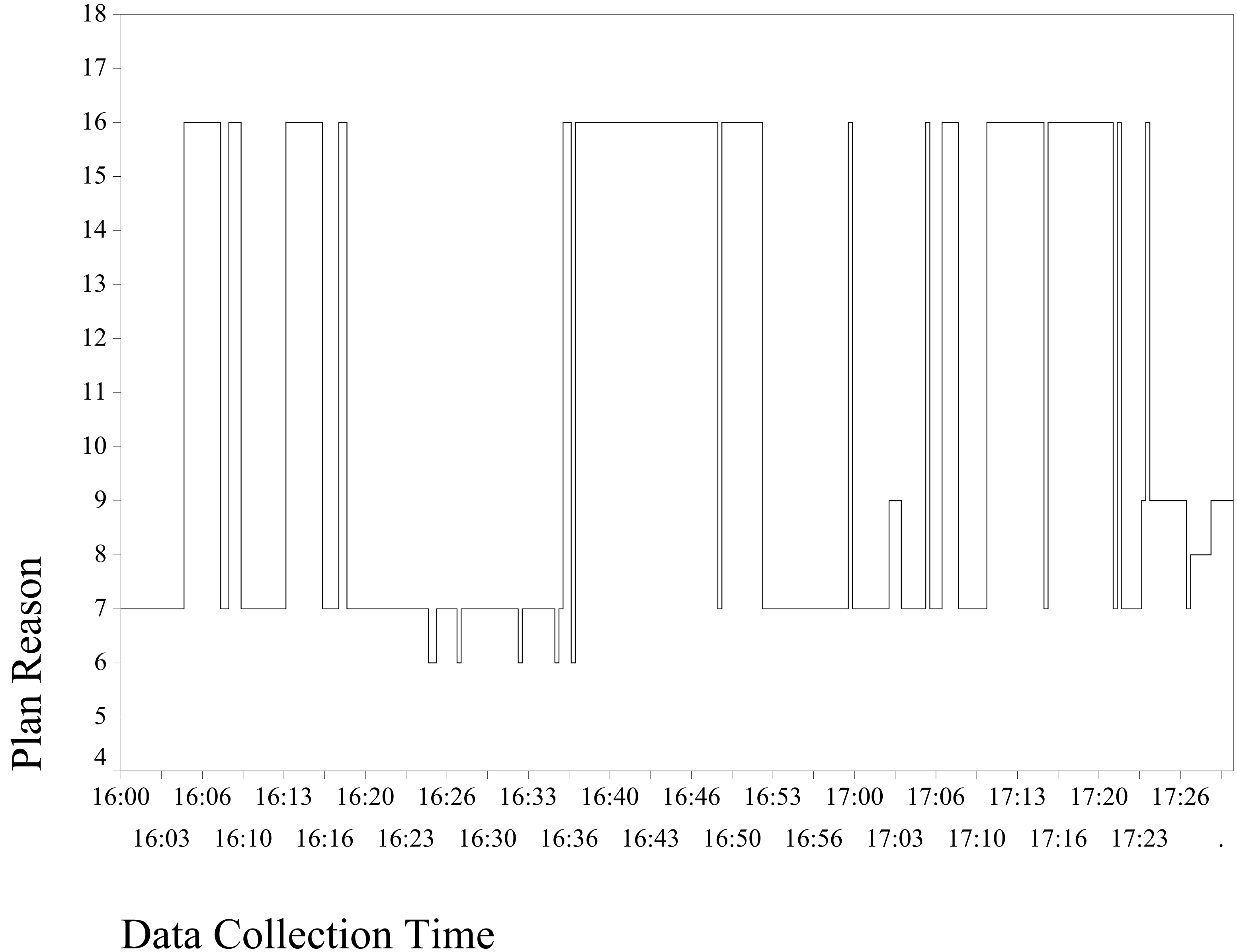


Figure B-3. Mainline Speed.

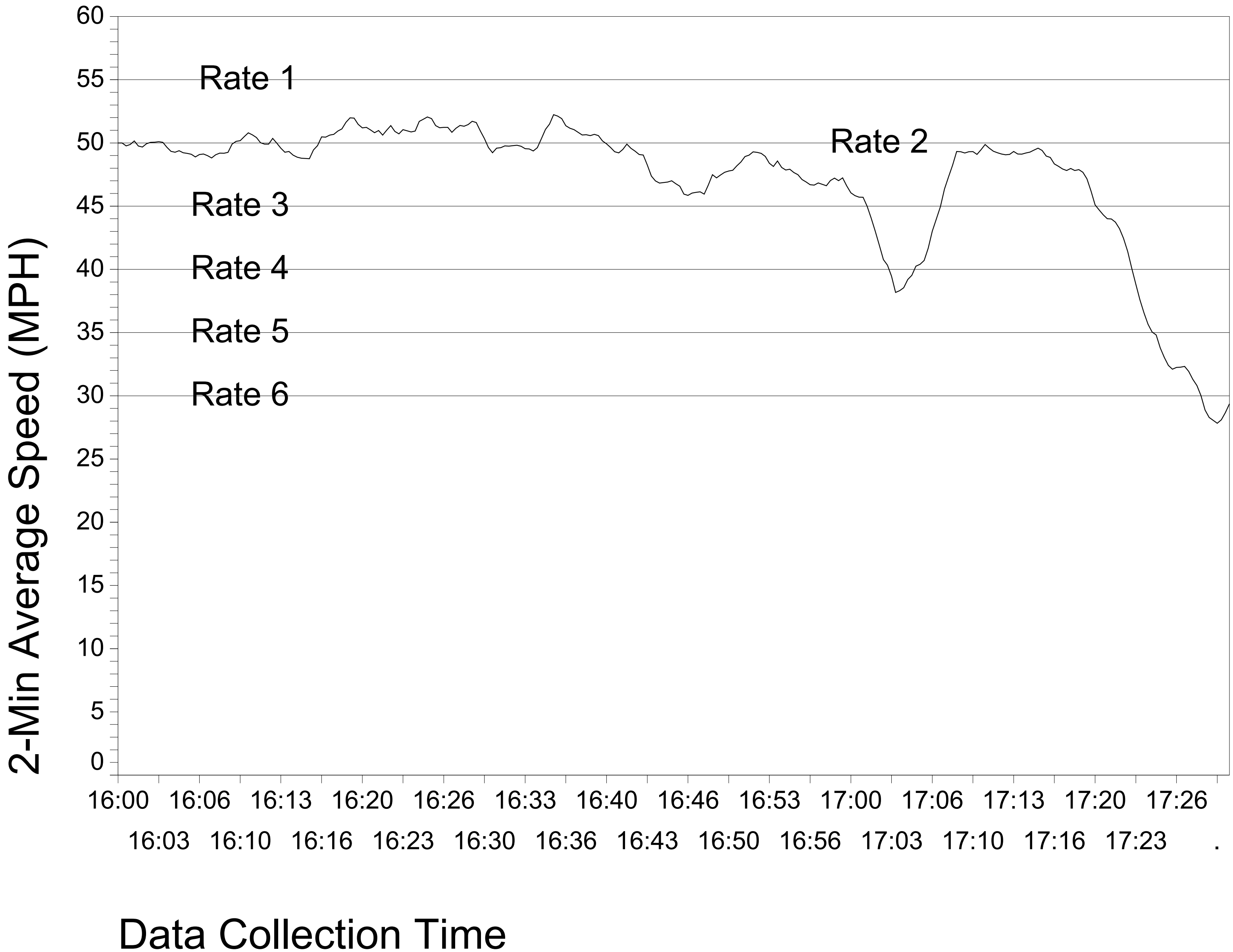


Figure B-4. Mainline Lane Volume.

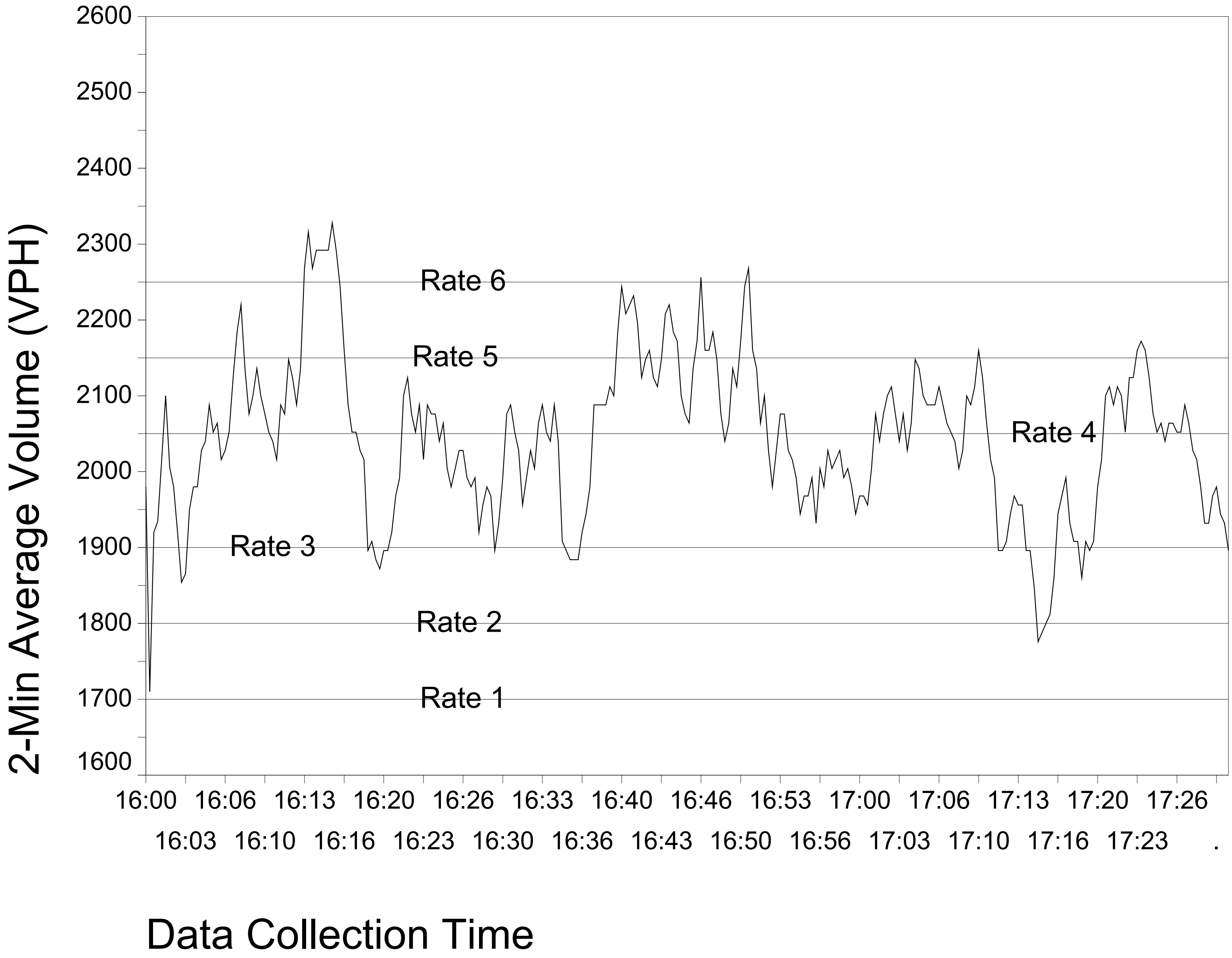


Figure B-5. Mainline Lane Occupancy.

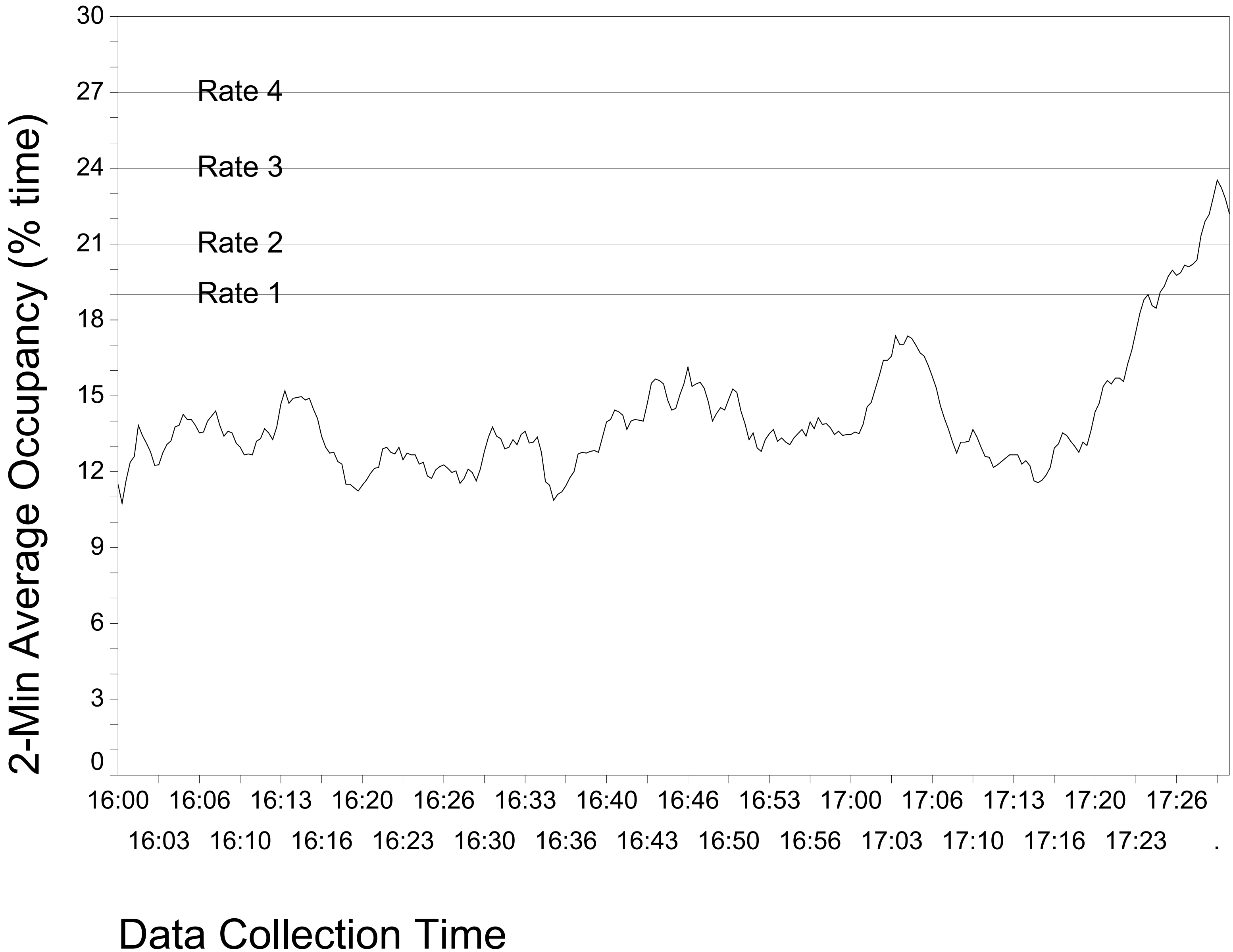


Figure B-6. Ramp Occupancy.

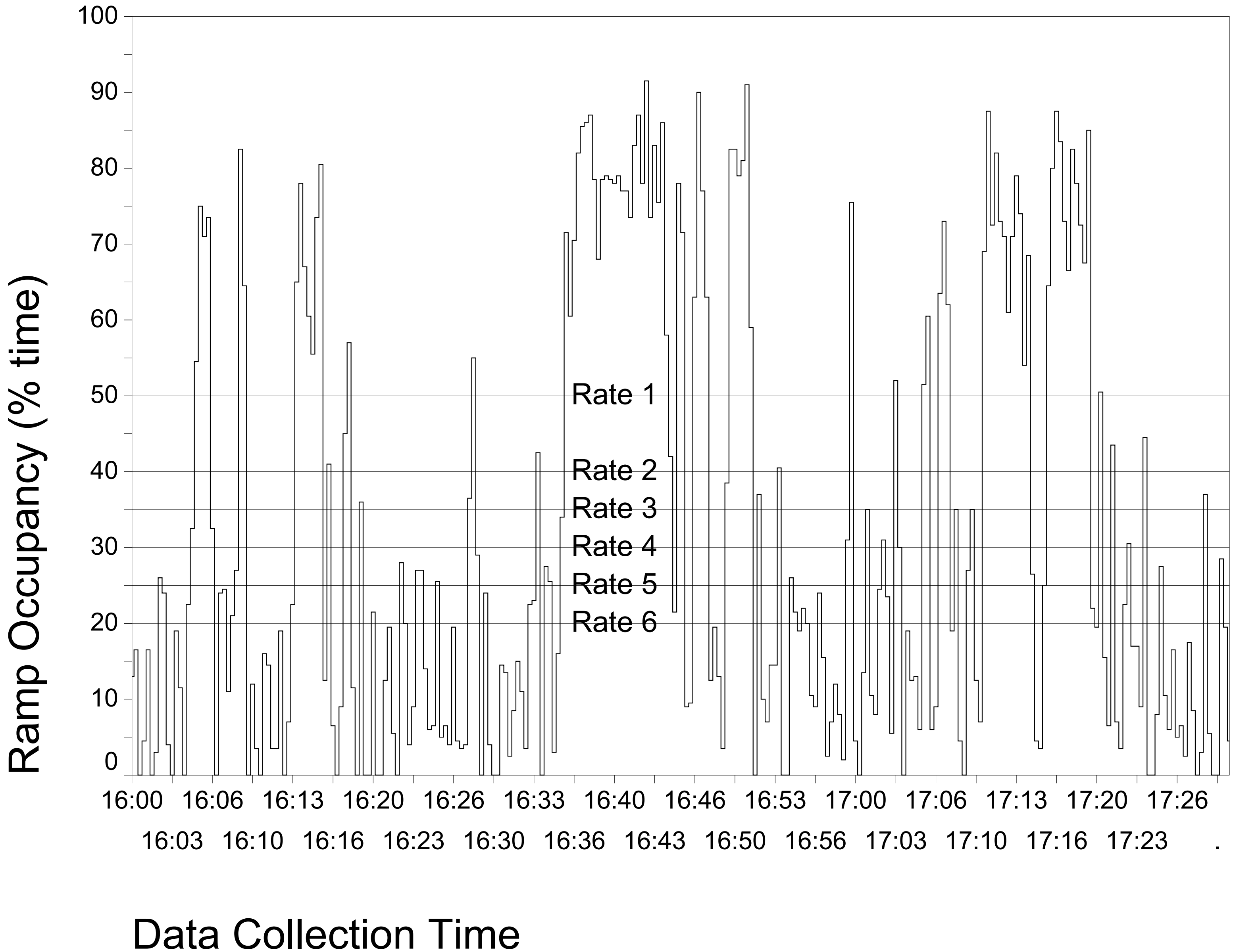


Figure B-7. Chosen Metering Rate.

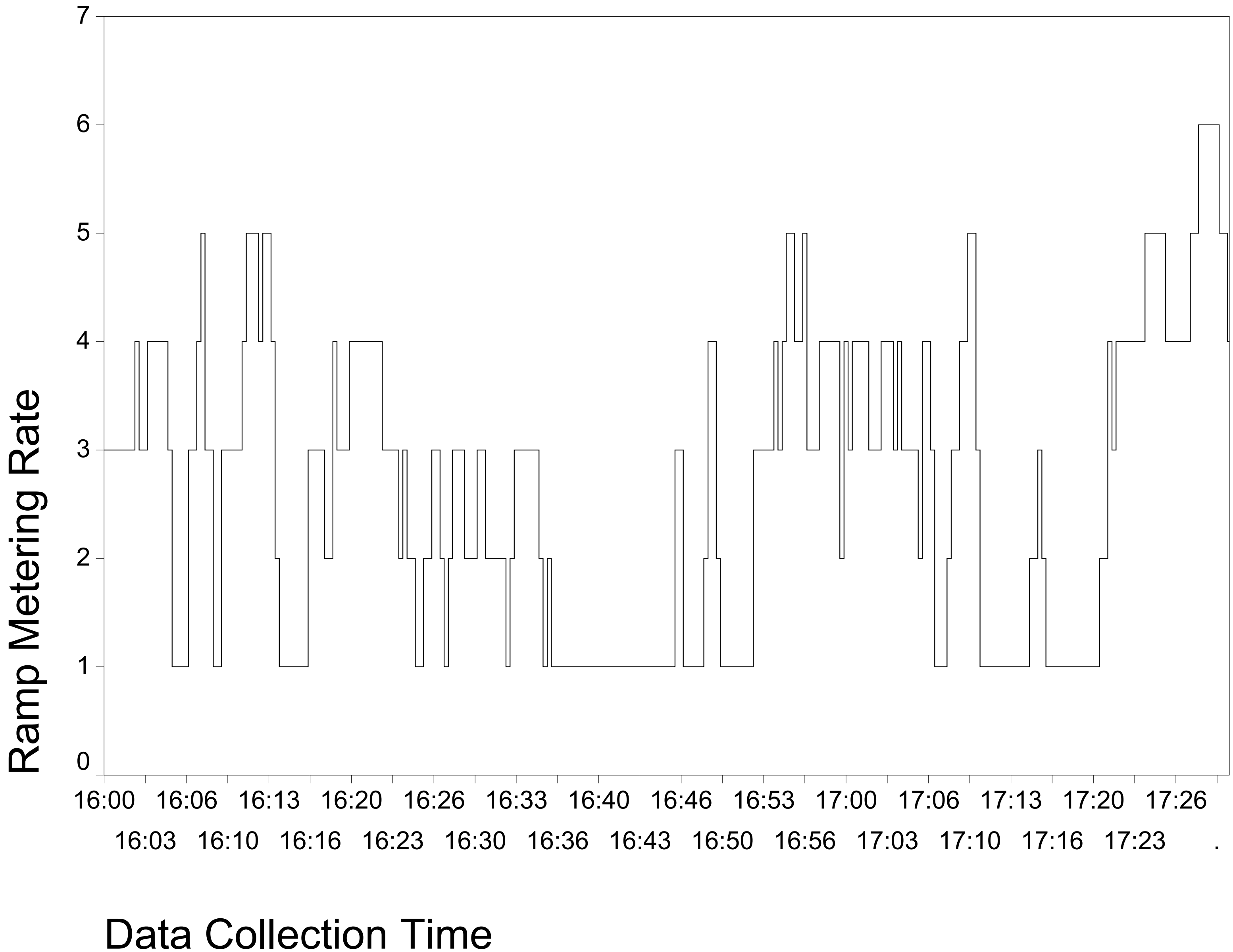


Figure B-8. Metering Rates Based on Ramp Occupancy.

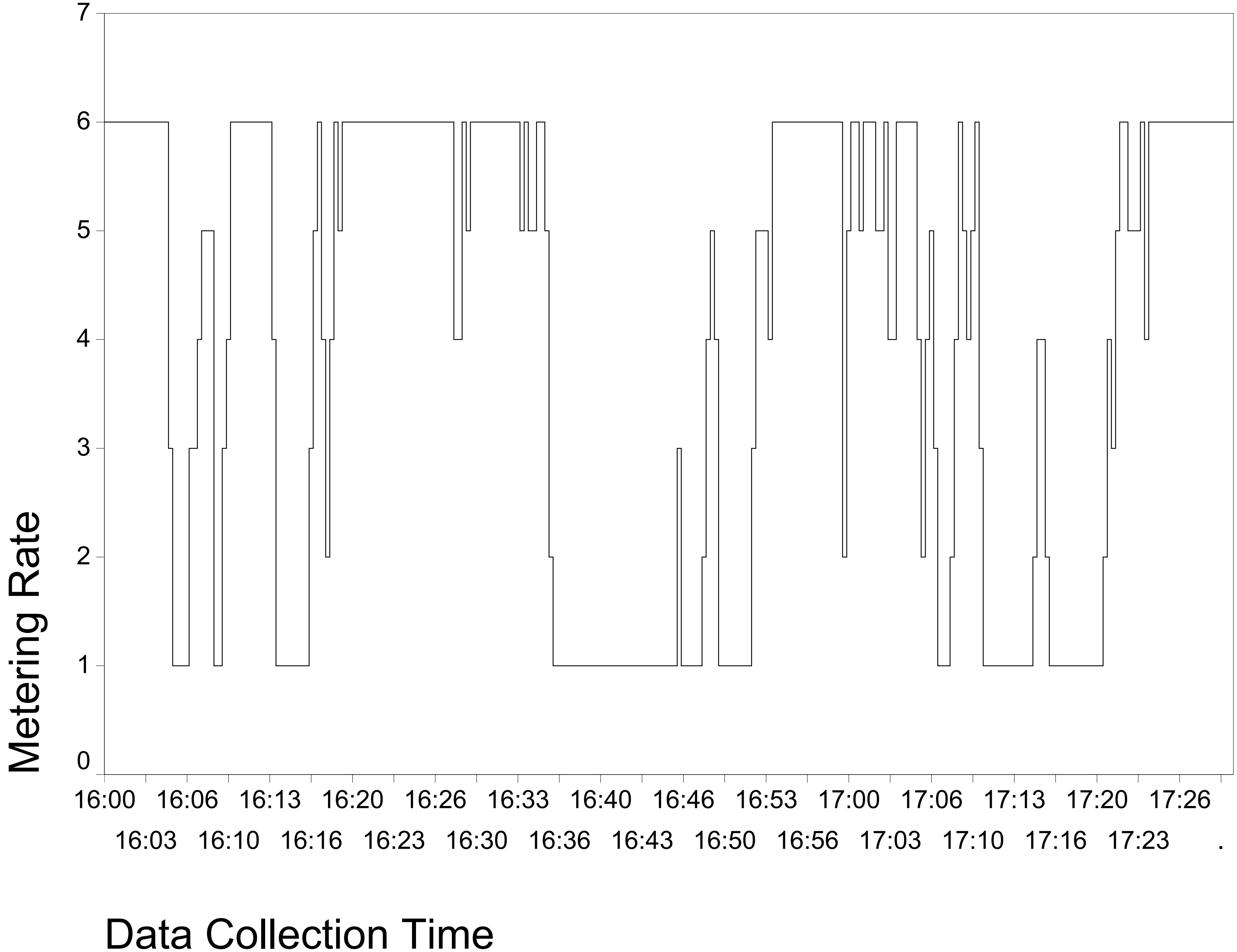


Figure B-9. Mainline Speed-Volume Feb 9, 2000.

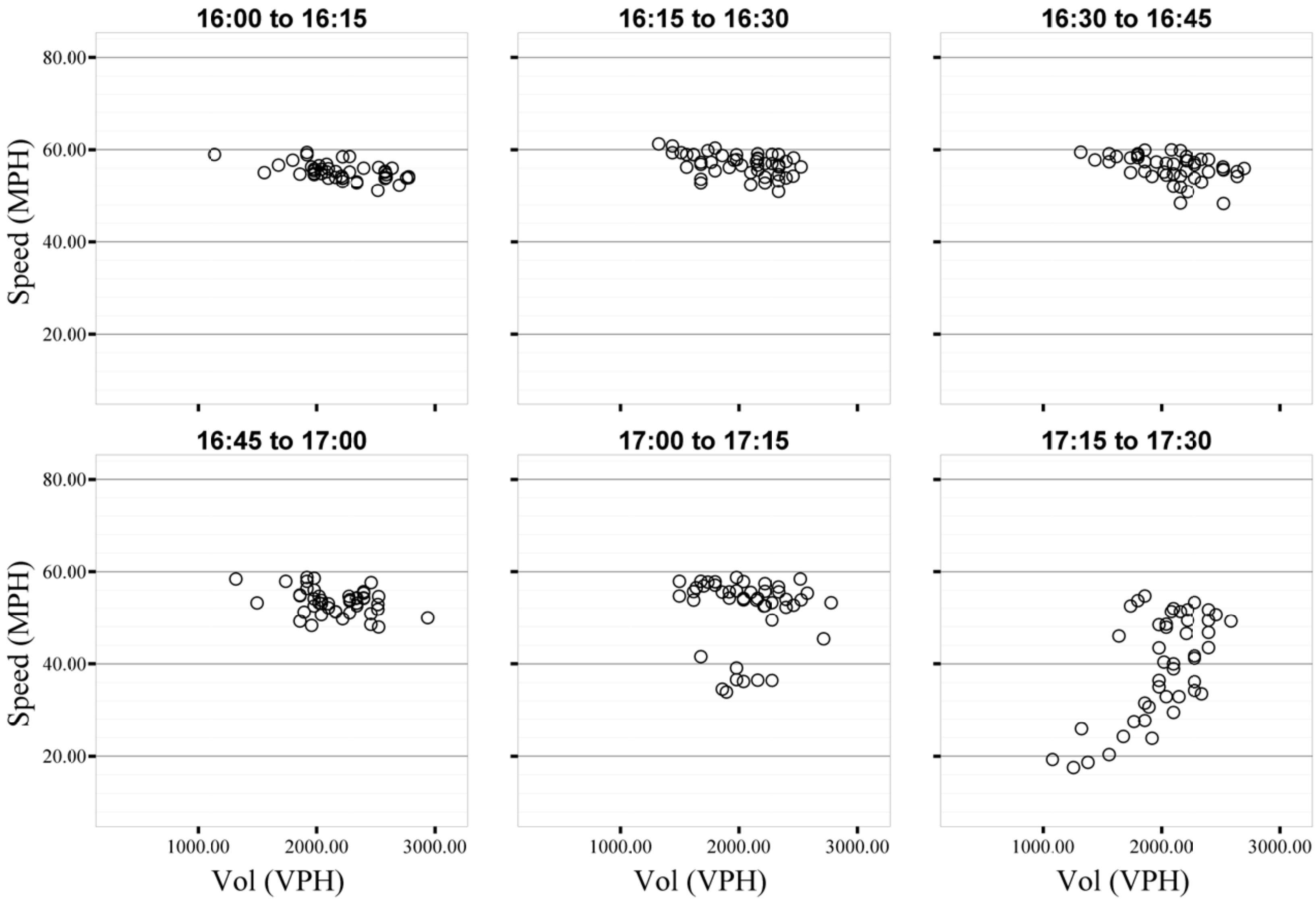


Figure B-10. Mainline Speed-Volume Feb 9, 2000.

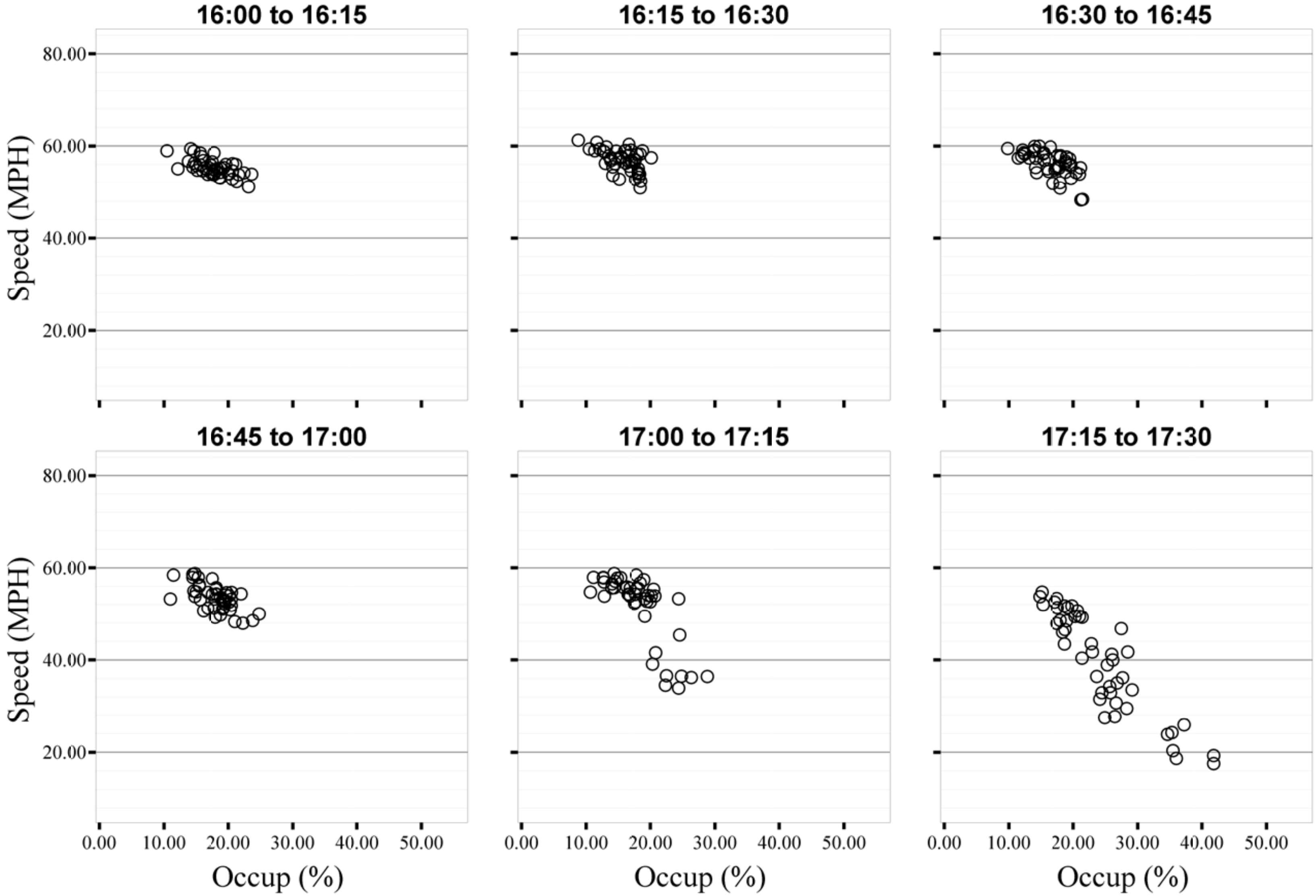


Figure B-11. Mainline Speed-Volume All Days.

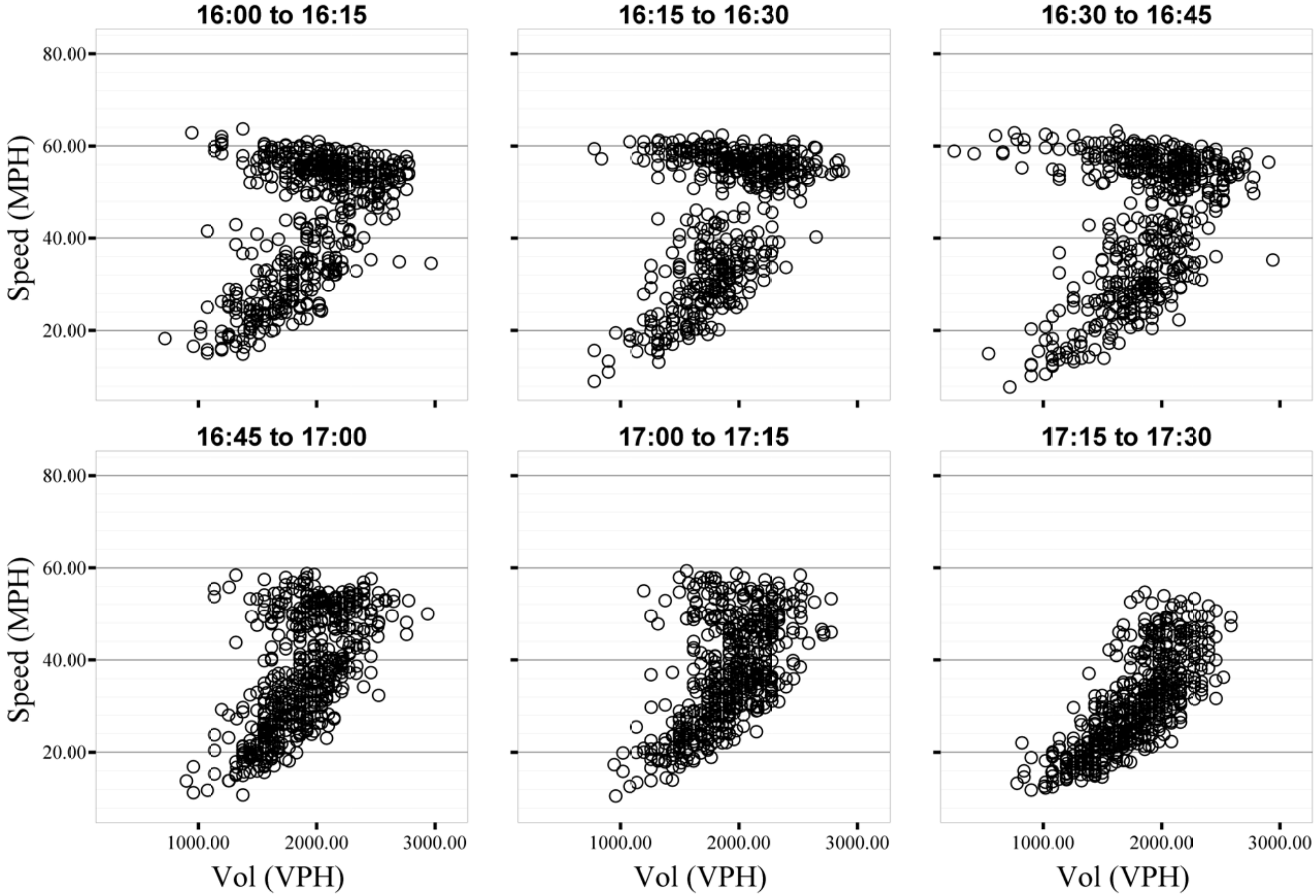


Figure B-12. Mainline Speed-Volume All Days.

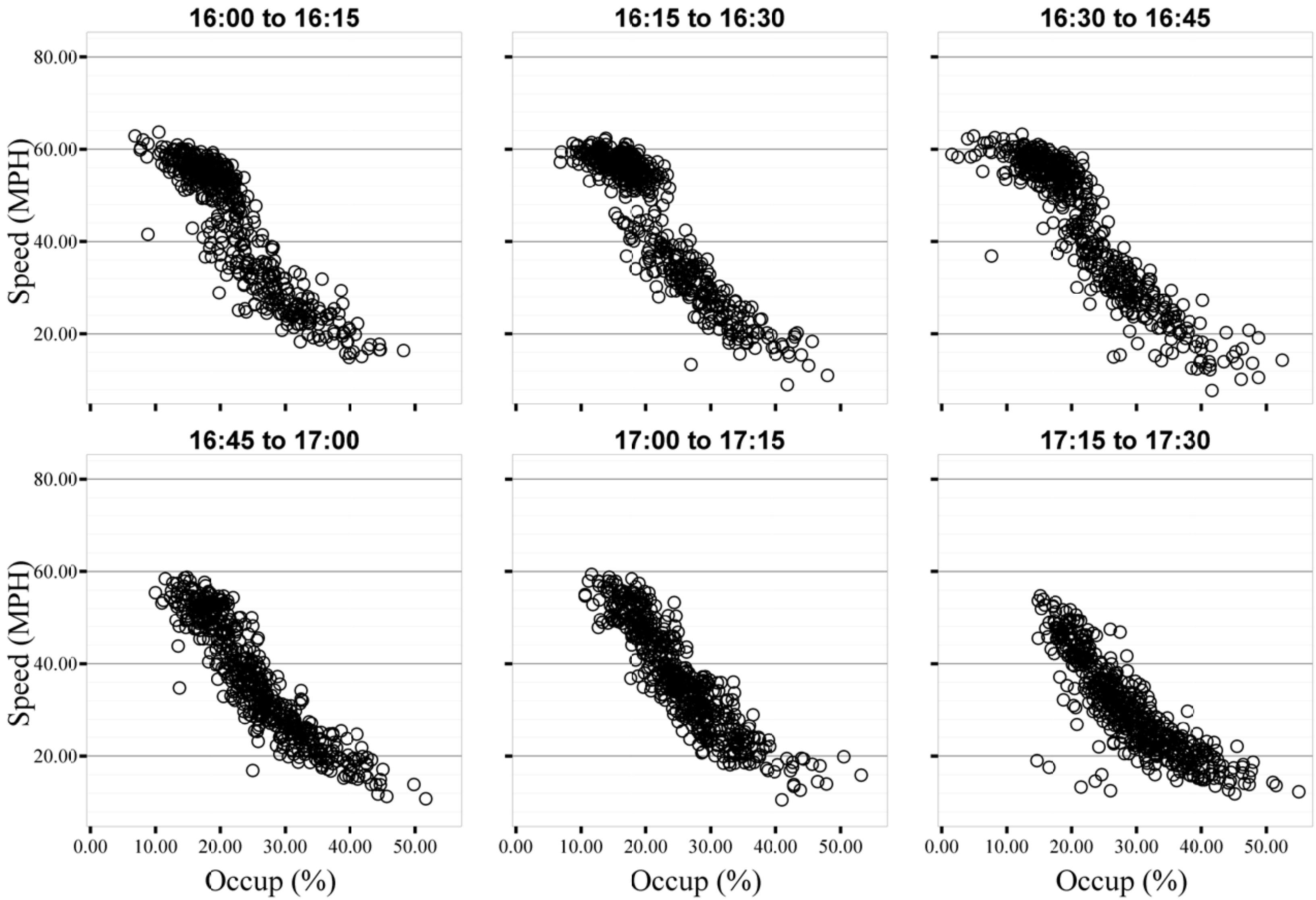


Figure B-13. Mainline Occupancy, Speed and Volume

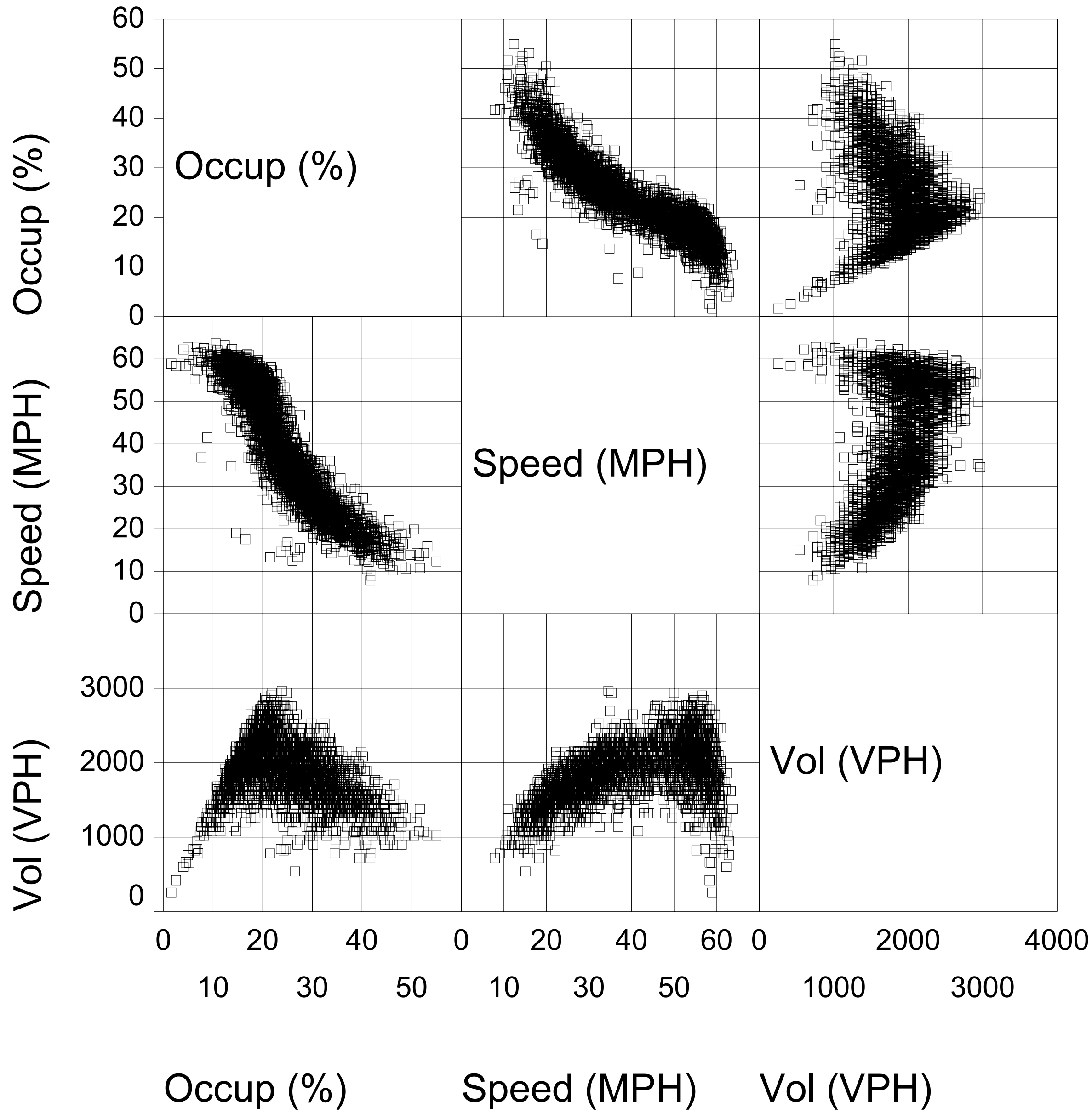


Figure B-14. Mainline Speed and Volume.

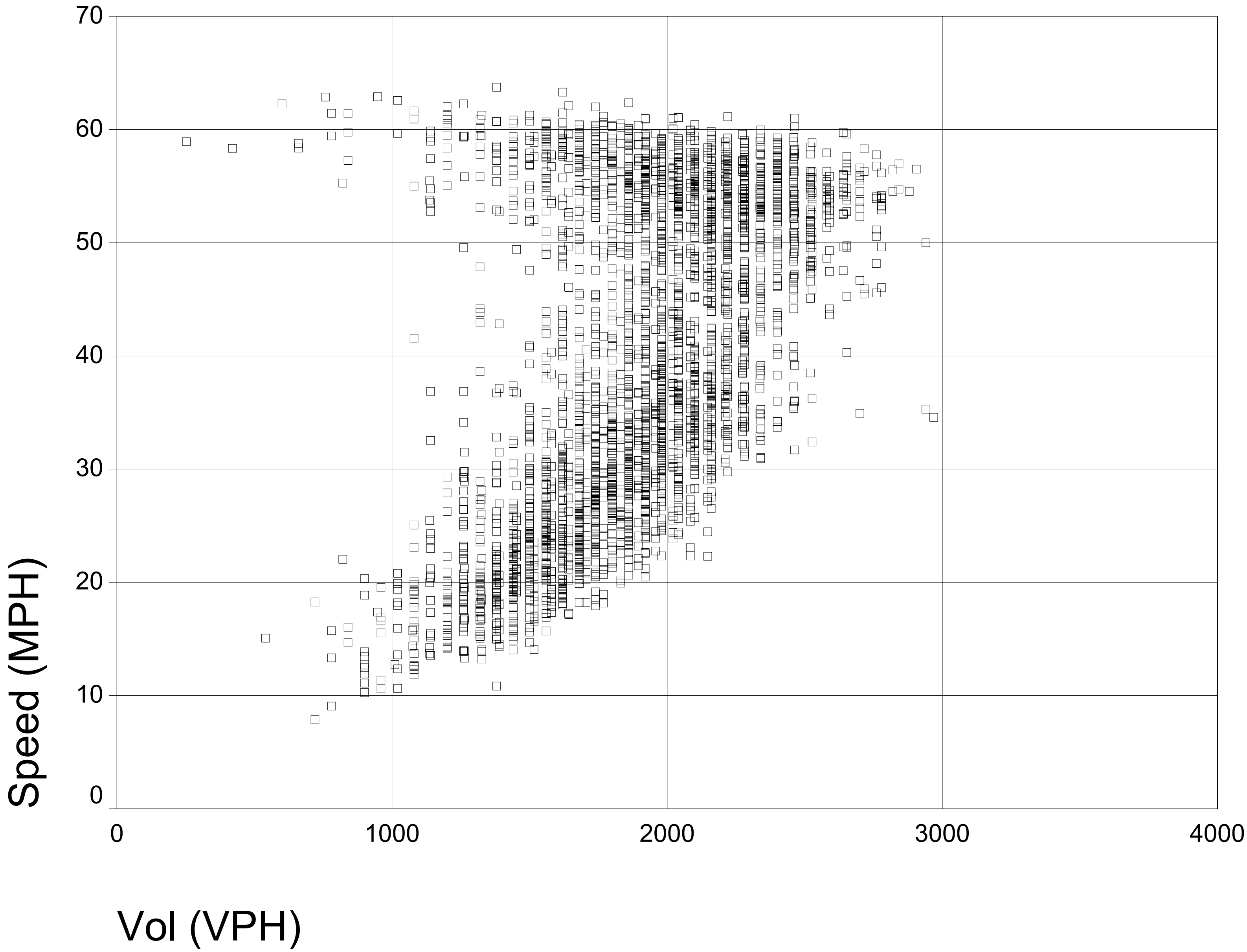


Figure B-15. Mainline Occupancy and Speed.

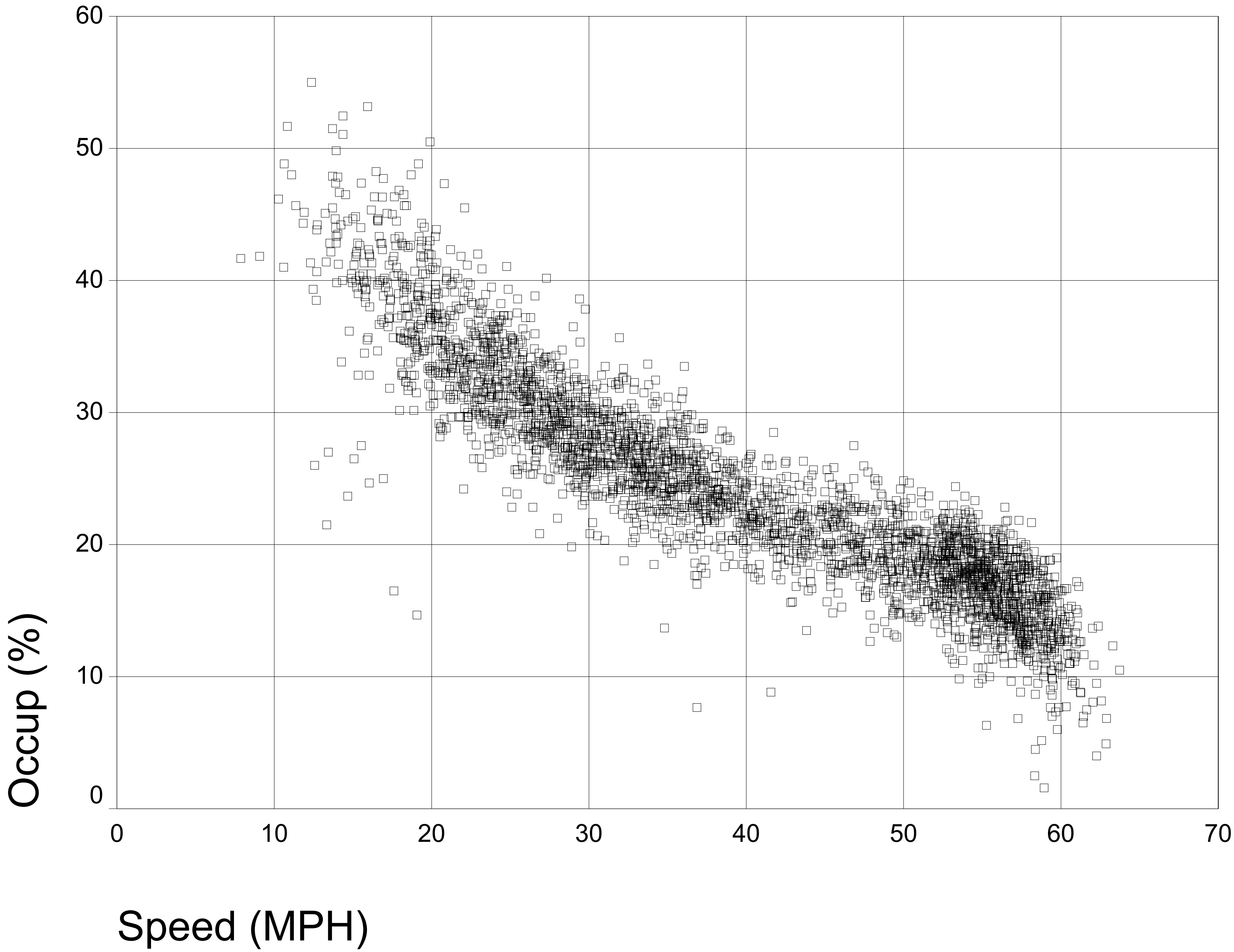
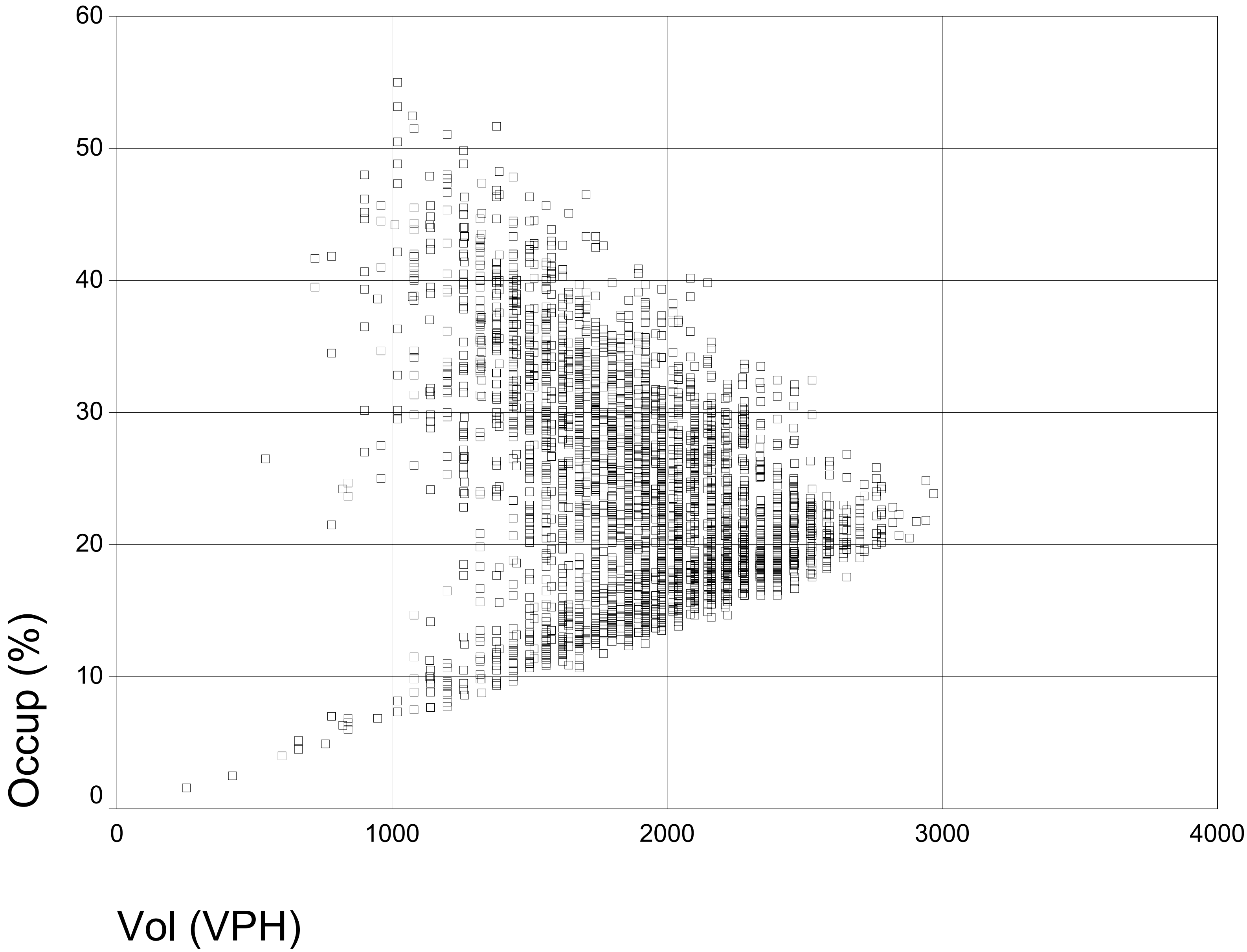


Figure B-16. Mainline Occupancy and Volume.



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