Facility Performance Model Enhancements for Multimodal Systems Planning (Part II)

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16. Abstract

There were two main research components to this project. The first component addressed the issue of performance measures and level of service assessment for rural freeways. The second component addressed the issue of freeway capacity values and the impacts of ramp merge and diverge influence areas.

In the first component, several rural freeway level of service models were developed based upon traveler perceptions obtained from surveys of traffic and roadway conditions depicted in in-vehicle videos. A level of service model based on a more comprehensive set of traffic and roadway variables is recommended, as opposed to the current HCM assessment method that is based only on density.

In the second component, two issues were investigated. The first issue was the interpretation of the language in Chapter 25 (Ramps and Ramp Junctions) of the HCM with regard to the capacity of a freeway segment in a ramp influence area and situations that result in unrealistic flow rates in certain lanes of the freeway segment. The second issue dealt with the use of archived loop detector data to measure freeway throughput values and the potential for measuring the impact of ramp activity on freeway capacity values.

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Report Organization

There were two research components to this project. The first component, known as Task 3 in the original scope of work, addresses the issue of performance measures and the assessment of level of service for rural freeways. The second component, known as Task 4 in the original scope of work, addresses the issue of freeway capacity values and the impacts of ramp merge and diverge influence areas.

Although both components deal with freeway issues, the specific issues addressed in each task are relatively unique. As such, this project report has been organized into two separate sections, one for Task 3 and one for Task 4, labeled Part I and Part II, respectively. Additionally, Task 4 has been further divided into parts 'a' and 'b'.

The report content for Task 3, under the section titled "Development of a Rural Freeway Level of Service Model Based upon Traveler Perception", is in large part the Masters thesis prepared by Mr. David Kirschner under the supervision of Dr. Scott Washburn. The front matter that was relevant only to the graduate school of the University of Florida was deleted, and some additional editorial and formatting revisions were also performed.

The report content for Task 4a, under the section titled "Highway Capacity Manual Guidance on Freeway Capacity in Merge/Diverge Areas", was developed by Dr. Scott Washburn with the assistance of graduate student Ms. Rohini Bobba.

The report content for Task 4b, under the section titled "Freeway Capacity Measurements from Inductance Loop Detector Data", is in large part the Masters report prepared by Mr. Mudassar Alam under the supervision of Dr. Scott Washburn. The front matter that was relevant only to the graduate school of the University of Florida was deleted, and some additional editorial and formatting revisions were also performed.

Part I

Development of a Rural Freeway Level of Service Model Based upon Traveler Perception (Task 3)

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ABSTRACT

The concept of Level of Service (LOS) is meant to reflect the trip quality a traveler will experience on a roadway or other transportation facility. The objective of this study is to provide insight into how road users perceive trip quality on rural freeways, and to examine how the existing service measure (density) relates to the perceived trip quality.

Study participants were shown a series of video clips of rural freeway travel from a driver's perspective and then filled out survey forms indicating their opinion of the trip quality provided by the conditions in the video clip. The survey participants were also asked to give background information about themselves and their driving habits.

The data from the surveys were analyzed using an ordered probit model. The first model used only density as a predictive factor. The second took into account other roadway and traffic characteristics, and the third examined all the significant factors obtained from the survey. The 'density only' model confirmed that density is a strong indicator of travelers' perceptions of trip quality. The other models showed the significance of other traffic and roadway factors in the perception of trip quality in addition to density, as well as some socio-economic information and personal driving habits. A set of LOS thresholds was also calculated for the 'density only' model using the survey participants' responses. The thresholds estimated from the survey participants' responses were considerably lower than the HCM thresholds for all LOS rankings. This suggests that travelers' tolerance of congestion is lower on rural freeways than the HCM indicates.

CHAPTER 1
INTRODUCTION

Background

Transportation infrastructure investment decisions are often heavily influenced by the results of level of service (LOS) analyses conducted according to the methodologies of the *Highway Capacity Manual* (HCM) [1]. In the HCM facility analysis methodologies, the assignment of a LOS is based on designated performance measures and corresponding threshold values. In the 2000 HCM, LOS is divided into six categories—A through F. LOS A indicates excellent service and LOS F indicates extremely poor service. The currently designated service measure(s) (i.e., the performance measure(s) used to assess LOS) for each facility is (are) based on the collective experience and judgment of the members of the Highway Capacity and Quality of Service (HCQS) committee. The same is true with the selection of the threshold values for the various LOS designations. There is currently no quantitative procedure to define which values are used as LOS thresholds. The LOS determination process, therefore, is based on the perspective of transportation professionals. The selection of service measures by the HCQS committee is, however, guided by two principles: 1) the service measure for each facility should represent speed and travel time, freedom to maneuver, traffic interruptions, and comfort and convenience in a manner most appropriate to characterizing quality of service for the particular facility being analyzed, and 2) the service measure chosen for a facility should be sensitive to traffic flow such that the service measure accurately describes the degree of congestion experienced by users of the facility [2]. It is also the committee's intent that the selected service measures and corresponding thresholds be highly correlated with public perception, but this is not known for sure [3]. Since billions of dollars of transportation investment decisions are made every year based upon the outcome of HCM level of service analyses, it is desirable that the transportation engineers' assessments of the impact of these investments be consistent with traveler perception of the investment impacts.

In this study, the issue of level of service perception for freeways in rural areas was addressed. Rural freeways can differ significantly from urban freeways. For example, rural freeways typically have greater distances between interchanges, higher speed limits, and a higher percentage of social and recreational trips (and lower percentage of work and shopping trips) than urban freeways. Despite these differences, urban and rural freeway segments not only use the same service measure, density, but also the same LOS thresholds. This study was initiated by the Florida Department of Transportation (FDOT) Systems Planning Office due to their belief that this generalized approach to freeway segments was not appropriate. The FDOT believed that traveler expectations and perceptions of quality of service were different for rural and urban freeways. While urban freeways experience the full range of traffic congestion conditions, rural freeways rarely experience enough traffic congestion to cause significant travel speed reductions. Rural freeway travelers likely expect these free-flowing conditions. While urban freeway travelers are concerned with their overall travel time and the reliability of this travel time, rural freeway drivers take travel time for granted. Furthermore, urban freeway drivers expect their ability to change lanes to be restricted, while a restricted ability to change lanes negatively impacts a rural freeway user's perceived quality of service [4].

Research Objective and Tasks

The objective of this study was to develop a model, and corresponding thresholds, for assessing level of service on rural freeway segments based upon traveler perceptions. The following tasks were carried out in supporting the above research objective:

- Reviewing and summarizing relevant literature
- Determination of an appropriate research approach
- Determining appropriate rural freeway sites to perform field data collection
- Collection of video of roadway and traffic conditions from these sites
- Collection of traffic data from FDOT count stations at these sites
- Production of video clips to be shown to survey participants
- Development of a survey instrument
- Recruitment of survey participants
- Conducting survey sessions
- Performing an analysis of survey responses
- Development of candidate level of service models

CHAPTER 2
LITERATURE REVIEW

The *Highway Capacity Manual* [1] states that the level of service of a roadway segment should be consistent with the perceptions of travelers, yet the development of the current freeway segment LOS methodology did not directly take these perceptions into account. There have been some recent studies performed seeking travelers' opinions about what factors and qualities are important to them in assessing the quality of their trip. A literature review was conducted to identify these studies and note their findings with regard to travelers' perceptions of LOS. This summary follows an overview of the freeway segment LOS analysis methodology in the 2000 edition of the HCM, as outlined in the following section.

HCM Freeway Segment LOS Methodology

A freeway is a segment of divided roadway with controlled access and two or more lanes in one direction. Within this definition there are significant differences between urban and rural freeways. Rural freeways have greater distances between interchanges than urban freeways, higher speed limits than urban freeways, and a higher percentage of social and recreational trips than urban freeways. Urban freeways have a higher percentage of work and shopping trips than rural freeways. Despite these differences, urban and rural freeways both use density as their service measure with the same thresholds for LOS.

Traveler expectations and perceptions of quality of service are different for rural and urban freeways. While urban freeways experience the full range of LOS conditions from A to F, rural freeways rarely drop below LOS C. Rural freeway travelers have come to expect these higher levels of service, therefore while urban freeway travelers are concerned with their overall travel time and the reliability of this travel time, rural freeway drivers take travel time for granted. Urban freeway drivers expect their ability to change lanes to be restricted, while a restricted ability to change lanes negatively impacts a rural freeway user's perceived quality of service [5].

The original HCM had a basic three-point scale to define level of capacity. In 1963 the Level of Service concept was introduced and replaced the previous scale. In 1965 the six-point LOS scale (from A to F) was introduced. In 1985 this six-point scale was redefined to use traffic density (vehicles per unit length of roadway) as the service measure for defining LOS on freeway sections. This is the method that is still used today. Although the concept of LOS is meant to reflect the operational conditions as perceived by motorists, no freeway LOS methodology in the history of the HCM has been based on driver perception studies. Therefore, there can be no way to make sure that the LOS thresholds freeways (as well as any other type of facility) accurately reflect users' perception of the quality of service they receive.

Under the existing LOS methodology, rural and urban freeways have the same service measure – density, as well as the same thresholds for each rank on the LOS scale. These thresholds for all freeway segments are shown in Table 1.

Table 1. HCM Level of Service Thresholds

Level of Service	Density (pc/mi/ln)
A	0-11
B	11-18
C	18-26
D	26-35
E	35-45
F	> 45

Do these thresholds accurately reflect the quality of service perceived by travelers on all freeways, urban and rural? In particular, the studies by Hostovsky [5] and Washburn [4] indicate that rural freeway travelers may judge the quality of their trip based on different qualities and criteria.

Studies Investigating Traveler Perception of LOS

A study by Pécheux et al. [6] noted that the HCQS committee recognized a need to improve the HCM methodology of assessing LOS. Specifically, concerns were raised that the LOS of a roadway section did not correspond to road users' perceptions. The authors felt that for LOS to accurately reflect travelers' perception of quality of service they would first have to find out what performance measures were significant to travelers. The study method involved test participants driving along a pre-selected 40-minute route, encompassing mostly arterial streets, accompanied by an interviewer and a traffic engineer. The participant would discuss what factors they personally found important to the quality of their trip. Participants identified over 40 factors that were important. These included such factors as intersection efficiency—if the intersection was being utilized by opposing traffic while travelers were waiting, and the aesthetic qualities of the intersection. Neither of these topics is currently covered by the HCM. The study concluded that more research was needed to focus on traveler perception.

A study by Hostovsky et al. [5] used focus groups to identify factors important to trip quality on rural freeways and then compared those findings with those from a focus group study using regular urban commuters and commercial truck drivers. The participants in the rural freeway focus group identified three factors that were most important to trip quality—low density, predictable travel time, and maintaining a steady travel speed. Other topics discussed were the safety issues inherent to the isolated locations of rural freeways, aesthetics, speed differential between vehicles, the presence of heavy vehicles, and the need for better traveler information. Urban commuters placed high importance on the overall speed of their trip, where rural freeway travelers felt that the ability to choose their speed was a positive. This reflects the fact that urban drivers rarely have the opportunity to choose their speed in the traffic stream, so a faster speed is usually preferable over a slower one. Urban commuters were also not as concerned with the ability to change lanes and move about the facility at will. Most of the urban drivers were happy if they could stay in one lane and maintain a desired speed for their trip. The rural drivers were pleased if the density of the freeway section was low enough to allow movement between lanes and passing at will. This study was significant due to the fact that it recognized the differences in how travelers rate their trip quality on an urban versus a rural freeway.

A study by Nakamura et al. [7] evaluated traffic flow conditions along an expressway in Japan from a driver's viewpoint. The study intended to quantitatively analyze the relationship between traffic flow conditions, drivers' perceptions, and drivers' behaviors. The field data portion of this study collected data on drivers' behavior and perception under various flow conditions. Drivers had a video camera mounted in their own vehicle and were asked to drive a section of expressway. After each trip the subject was asked to complete a survey about the traffic flow conditions. Twenty-two subject vehicles were used and 105 surveys were collected. The behavioral data collected were number of lane changes, travel time by lane, and percent time spent following. This study found that the most important factor influencing drivers' satisfaction with their trip was the traffic flow rate. Other factors affecting trip quality were found to be number of lane changes, and the percent time spent following.

Several studies have been identified using road-user surveys and video selections to evaluate LOS methodology. The first study, by Sutaria and Haynes [8], used a road user survey to evaluate the LOS methodology for signalized intersections. Over 300 drivers were shown video clips taken both from a driver's perspective and from an overhead camera at an intersection. The video segments were specifically chosen to represent a specific LOS and were intended to be shown to drivers for one or two signal cycles. The final compilation shown to drivers included both types of view and the clips were put in a random order. Their road-user survey consisted of two parts—a group attitude survey and a video review survey. The group attitude survey used a questionnaire, to be answered before the video portion of the survey. The questionnaire included demographic information such as gender, age, and education, as well as questions about the participants' driving experience and the type of roadways the participants usually drove on. They were then asked to give the relative importance of factors including delay, number of stops, traffic congestion, heavy vehicle density, and ability to change lanes as these factors applied to the quality of service at an intersection. After the initial questionnaire the participants were shown the video clips, consisting of a driver's view of a vehicle approaching, waiting, and passing through an intersection. After each of these clips the participants rated the quality of service they felt the intersection provided. At the end of the video portion the participants were again asked to rate the factors important to quality of service at an intersection to see if their initial opinion had changed. The results from the survey showed delay to be the most important factor both before and after the video portion of the survey. This study provided the first results that took into account the perceptions of travelers and changed the performance measure used by the HCM to evaluate LOS in the 1985 edition. This study also recommended further similar studies, and for further studies to simultaneously collect video and traffic flow data to allow for accurate measurements of what is depicted on the video.

A study conducted by Pécheux et al. [9] addressed the issue of developing a study method to assess the perceived LOS at signalized intersections. The first objective was to determine how well the current LOS methodology reflects the opinion of road users. The second objective was to determine the factors affecting users' perceptions at signalized intersections. The participants in this study represented a wide range of ages, education levels, and incomes. The participants were first shown a series of approaches to signalized intersections from a driver's perspective. After being shown a sequence of these clips, the participants were asked to fill out a survey including their attitudes about certain driving situations as well as their socio-economic information. After filling out these surveys the participants were asked to discuss the factors that influenced their perception of quality of service as a group. The study results showed that on average, the participants' delay estimates were fairly accurate, however individual delay perceptions varied significantly. Fifteen factors were identified that contributed significantly to quality of service. Finally, the study found that participants tended to perceive service quality on three or four distinct levels as opposed to the six levels currently defined by the HCM.

Another study using video clips and road user surveys was performed by Choocharukul et al. [10] with the intention of evaluating the current HCM methodology of assessing LOS. This study provided a multivariate statistical analysis of the factors that were important to road users' perception of trip quality as well as an assessment of the adequacy of the current service measure, density. The data for this study were collected from several urban freeway segments. Cameras were positioned on overpasses and focused on areas that included inductance loop detectors so density could be calculated from the speed and flow measurements. Thus, the camera position was static and recorded traffic flowing through the field of view. Twelve video clips were prepared, two for each of the six LOS designations (according to the calculated density and HCM thresholds), with one clip at the higher end of the LOS category and one clip at the lower end. The survey participants were provided with the descriptions of the six HCM LOS designations. They then watched the twelve video clips and ranked the LOS they thought best described the conditions. The participants were also surveyed for demographic and driving habit information. There were two groups of survey participants, one consisting of students, transportation professionals, and environmental management professionals, and the other consisting of commercial truck drivers and clerical and support staff. This study used an ordered probit statistical model to assess how users perceive the LOS of the freeway operating conditions. The results of the survey and analysis revealed that perceived levels of service do not closely follow the HCM. Almost all the participants in this study had a lower tolerance for LOS A than the HCM threshold, with the average cut-off for LOS A among the study participants shown to be 7 passenger cars per mile per lane (pc/mi/ln) as opposed to the HCM cutoff of 11 pc/mi/ln. The HCM threshold for LOS F also does not correspond with the findings of this study, with the participants selecting an average of 82 pc/mi/ln as the upper bound of LOS F as opposed to the HCM LOS F of 45 pc/mi/ln. The study also found that factors other than density strongly influence road users' perception of quality of service, such as number of lanes, average speed, speed variance, headway variance, percentage of trucks, and some socio-demographic variables. It should be noted that the use of an overhead view of traffic could likely affect survey participants' perceptions in a different way than that of an in-vehicle view of traffic and roadway conditions.

A study by Washburn et al. [4] investigated what factors are important to drivers when evaluating the quality of their trip on a rural freeway. A driver intercept survey approach was used. This survey approach was implemented at rest areas and service plazas along rural sections of freeway in Florida. These locations were chosen due to their access to travelers in the process of a rural freeway trip. It was believed that this in-field survey approach would provide more reliable data, than mail-back surveys for example, as the drivers' experiences would still be fresh in their minds when filling out the surveys. A total of 233 surveys were collected. Travelers were asked to rank the factors that contributed to the quality of their trip on a scale from 1 to 7. The most important factor, ranked in the top three 64.3% of the time, was the ability to consistently maintain the desired travel speed. The factor with the next highest ranking was the ability to change lanes freely and pass other vehicles. This was ranked in the top three 33.3% of the time. The third most important factor was the ability to maintain a speed no less than the posted speed limit. This factor was ranked in the top three 33.0% of the time. This preliminary study showed that while density is important to rural freeway travelers, it is not the most important factor in determining trip quality. It also showed that drivers consider many other factors when determining trip quality.

Conclusions

The studies detailed in this chapter have shown that, while some research has been done on travelers' perception of quality of service, there is a need for more study. The current HCM methodologies for evaluating LOS may be insufficient for determining the perceived quality of service from the traveler's point of view. The studies summarized in this chapter have shown that it is possible to understand and approximate a traveler's perception of quality of service using the factors that are found to be important to them. This type of research may ultimately assist decision makers when planning roadway infrastructure investments.

CHAPTER 3
RESEARCH APPROACH

This chapter describes the methods used in collecting the sample data for this study as well as the methods used to refine the data for the official survey efforts. Detailed within the chapter are the choices of a survey method, the selection of data collection sites, the creation of video clips, the developed survey form, and the process for conducting a road user survey.

Alternative Survey Methods

Common methods of data collection include the following: focus groups, field surveys, in-vehicle surveys driven by a researcher, in-vehicle surveys driven by the research participant, driving simulators, and video surveys.

- Focus Groups – This consists of recruiting test participants in order to arrange a roundtable-type discussion about rural freeway travel. Participants would discuss their rural freeway trip experiences and relate which aspects of rural freeway travel are most important to them when evaluating the quality of their trip. The advantage of a focus group is the relative ease of the survey, there is no video data collection, field work, or liability on the part of the researchers. The disadvantage is that participants may influence each other's responses and one particularly vocal participant could swing the rest of the group towards his or her opinion. Another disadvantage is the lack of a control element for the researchers – there is no one experience on which the participants are basing their opinions, so the researchers can not look at the data or video record to interpret the responses. Additionally, the potential lack of quantitative feedback upon which to build an analytical model limits researchers in their ability to predict the responses of other travelers faced with similar roadway and traffic conditions.
- Field Surveys – Researchers distribute survey forms at locations frequented by rural freeway travelers, such as rest stops or service plazas. The participants give their opinions on rural freeway travel in a survey form, rating which factors are most important when they judge their trip quality. One advantage to this method is that participants surveyed have recently driven on a rural freeway and have this experience fresh in their mind. Another advantage is that it is relatively easy to recruit participants for this sort of survey; there is always a ready supply of people in this type of location. The disadvantages are similar to the focus group.
- In-Vehicle Surveys (driven by research personnel) – Participants are recruited and driven along a section of rural freeway, then surveyed about their perception of the trip quality. Advantages to this method include – all participants would have the same experience to draw upon for their responses, and there would be no need to attempt to simulate the driving experience as participants would be experiencing the conditions

firsthand. The disadvantages to this method include the liability to the researchers should the vehicle be involved in an accident, and the time and effort involved in conducting a survey of this manner. The controllability and repeatability of the conditions are also disadvantages because it is not possible to ensure the same conditions will be experienced by multiple survey participants.

- In-Vehicle Surveys (driven by research participants) – Participants are recruited to drive along a section of rural freeway and provide the researchers with feedback on their trip once they return. Once again this method is advantageous in that it would provide participants with a firsthand look at the conditions involved. The disadvantages to this are similar to the previous method in that there is significant liability attached to a method like this, and this method would be even more time-consuming than the previous one. This method also suffers from the same lack of control and repeatability as any in-car survey.
- Driving Simulator – Participants are put behind the wheel of a real vehicle, but the driving environment is simulated with the use of computer animation and video display monitors. They would then participate in the virtual driving of a rural freeway segment. This would give participants a closer likeness of actual freeway travel without the liability of having them drive a real section themselves. Disadvantages include cost (simulator time is expensive) and the well-documented motion sickness problem for participants (which increases recruitment time and costs).
- Video Surveys – This method involves participants viewing pre-recorded video scenes from actual rural freeway sites. The clips could be from one of two perspectives:
    - Overhead View – A camera placed over the test section of rural freeway records the traffic flow for survey participants to review at a later time. While this method does not give a simulation of actually driving the freeway section, it does give the participant a broader overview of the traffic stream.
    - Driver's Perspective – A vehicle is equipped with a video camera to record the rural freeway trip from the driver's perspective. This method would better simulate actual rural freeway travel than an overhead view.

After considering all advantages and disadvantages, the video survey method (from the driver's perspective) was chosen for this initial effort. This method would allow larger groups of people to be surveyed simultaneously while giving a reasonably realistic depiction of rural freeway travel. This method will ensure that all survey participants experience the same conditions and limit the liability issues inherent in an in-vehicle survey, as well as provide for efficiencies in cost and time for data collection.

Site Selection

The sites at which the video clips were captured were all within Florida. Reasons for this include the proximity to the University of Florida and the access to the Florida Department of Transportation's (FDOT) network of more than 7,500 traffic monitoring stations. The FDOT maintains a network of inductance loop detectors (ILD) along Florida's Intrastate Highway System. There are two types of ILD stations – permanent and portable. The permanent stations were used in this study since they are continuously recording data 24 hours a day, 365 days a year. The portable stations require a data recorder to be installed at the location of the ILD station in a roadside cabinet. The permanent stations are telemetered such that data can be downloaded and archived on a daily basis. The data archived from the permanent stations are compiled every year by the FDOT and published on the Florida Traffic Information (FTI) CD. Also included on this CD for each state highway are the number of lanes in each direction, the ILD station type (permanent or portable), a description of the site, the Average Annual Daily Traffic (AADT), the percentage of trucks on the highway, and the peak hour volume in each direction.

Multiple sites were examined around the state. There were several factors leading to the final site selections. South Florida was excluded due to the limited number of rural freeway segments and the long distance. The panhandle area was also not considered due to driving distance. These conditions hinged upon the availability of suitable sites in north-central Florida. The final sites selected represented a mix of four-lane and six-lane freeways, level and rolling terrain, truck percentages, and a wide range of volume. This information was obtained from the Florida Traffic Information CD published by the FDOT [11]. All sites selected were permanent count stations instead of portable stations. Additional data were used from a similar study conducted months before at the University of Florida. A list of the data collection sites and their associated traffic data is shown in Table 2. Maps of the locations of the data collection sites can be found in Appendix A.

Table 2. Data Collection Sites and Traffic Data

Site[1]	Description	Avg. Daily Volume Direction 1	Avg. Daily Volume Direction 2	AADT (Dir. 1 + Dir. 2)	K Factor	D Factor	Truck %
189920	SR-93/I-75, 3.5 mi south of Turnpike, Sumter Co.	20472 N	21250 S	41722	10.1	59.84	21.66
360317	I-75, 0.35 mi north of Williams Rd overpass, Marion Co.	37630 N	37844 S	75474	11.14	55.41	21.76
140190	SR-93/I-75, 0.6 mi. south of SR 54, Pasco County	37443 N	37203 S	74646	8.76	53.67	11.71
730292	SR-9/I-95, 1.4 mi south of Palm Coast Pkwy, Flagler Co.	29276 N	29980 S	59256	9.91	54.92	17.82
269904	SR-93/I-75, 3 mi. north of Marion Co. line, Alachua Co.	31304 N	31023 S	62327	11.96	55.97	19.09
970428	SR-91/Fl. Turnpike, 797 ft. south of CR561, Lake Co.	17655 N	18088 S	35743	11.09	55.42	12.34

[1] All sites are telemetered traffic monitoring stations

Video Data Collection

The data collection method included three tasks – Equipment and Setup, Field Video Capture, and Video Clip Creation.

Equipment and Setup

The objective of the video data collection was to depict travel along a section of rural freeway from a driver's perspective in a reasonably realistic manner. Three in-vehicle video cameras were used to capture three different fields of view: 1) the view through the front windshield, including a view of the interior rear-view mirror, 2) the view of the vehicle's driver-side rear-view mirror, and 3) the view of the speedometer.

The vehicle used for the video data collection was a minivan. As mentioned above, three cameras were placed in the vehicle in order to capture different aspects of the rural freeway trip. The various camera mounting positions are shown in Figure 1. It was found during a preliminary test that the instrument cluster needed to be shaded to reduce glare, so the image would not appear washed-out. The video images were captured by three portable VCRs placed inside the vehicle. A microphone was also connected to one of the VCRs allowing the researcher to announce when they crossed a loop detector station and any other potentially important information. This would allow the researcher to match the captured video clip to the collected loop detector data. All these devices were powered by three 12-volt batteries. A schematic of the equipment and connections is found in Figure 2.

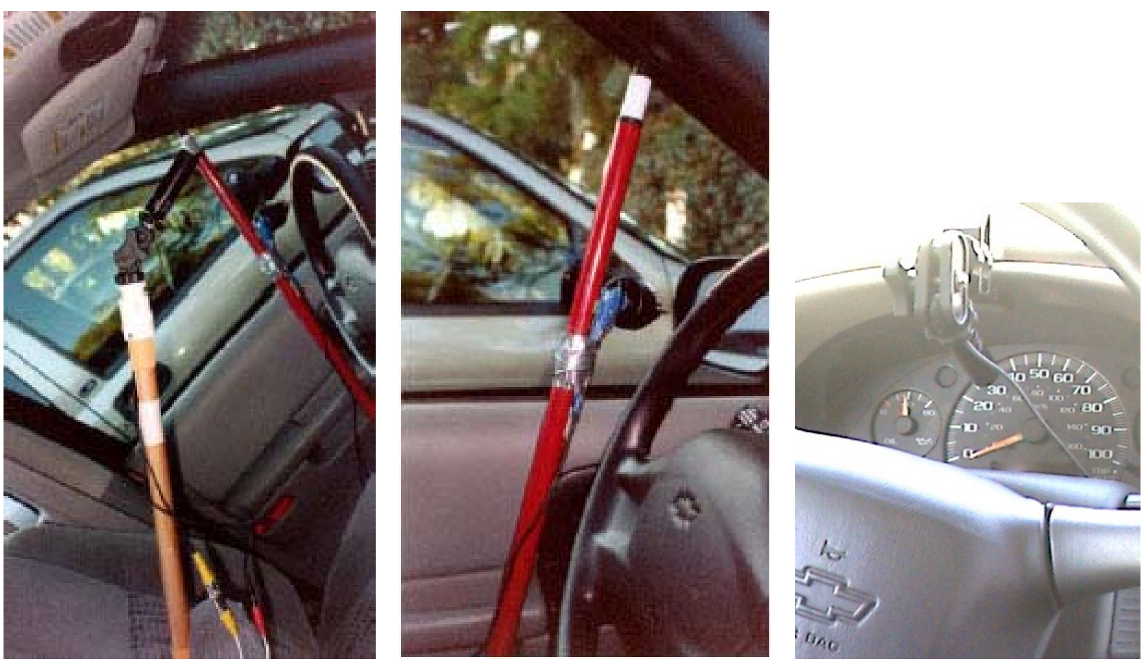

Figure 1. Camera Setup-Front View, Side View, Speedometer

Field Video Capture

The capture of video in the field involved the following steps: the researcher would simultaneously activate the three VCRs and start recording (with a single remote control); the researcher then merged onto the freeway from an on-ramp; the cameras captured conditions between the entrance and exit ramps; the researcher would record (by speaking into microphone) the travel direction, site number, and exact time when the loop detector station (between the ramps) was crossed. Up to four runs were made at each location, providing several video scenes to choose from when creating the clips for the survey. The data collection for this project was performed during November 2003 and March 2004. A summary of each video data collection session is shown in Table 3.

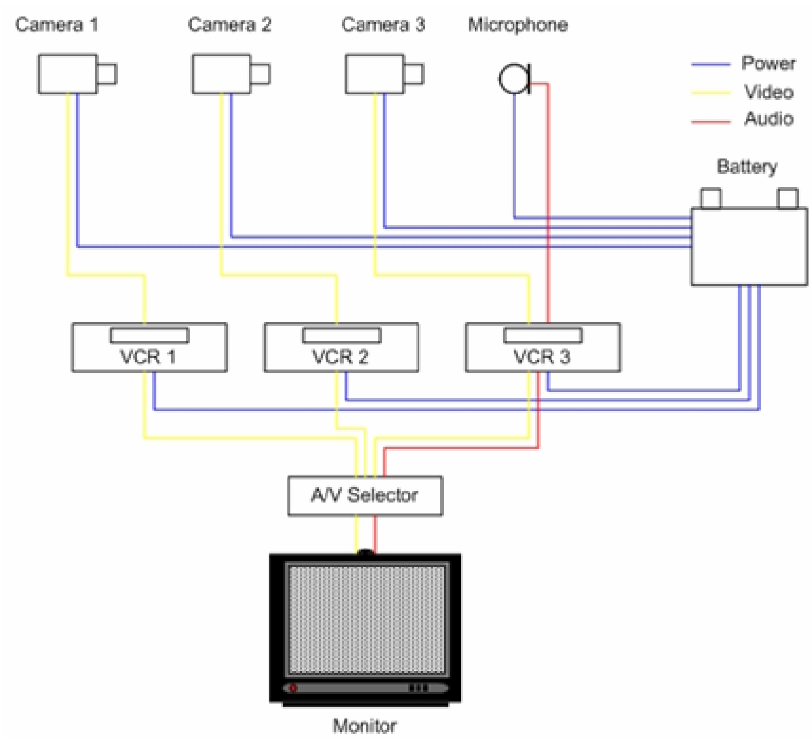

Figure 2. In-Vehicle Equipment Setup

Table 3. Data Collection Times, Locations, and Directions

Date	Site	Freeway	Direction	Time
11/4/2003	730292	I-95	NB	12:55
11/5/2003	269904	I-75	SB	11:01
11/21/2003	140190	I-75	NB	6:48
11/21/2003	140190	I-75	SB	7:04
11/21/2003	140190	I-75	SB	7:14
11/21/2003	140190	I-75	SB	2:31
3/7/2004	730292	I-95	SB	2:35
3/7/2004	730292	I-95	NB	2:47
3/8/2004	189920	I-75	SB	12:36
3/8/2004	360317	I-75	SB	11:50
3/8/2004	360317	I-75	NB	12:04
3/8/2004	360317	I-75	SB	2:07
3/8/2004	970428	Turnpike	SB	1:49

After reviewing the video data gathered on the first day of data collection, it was deemed unusable. The mounting for the front windshield camera allowed too much vibration in the picture and the video would not work for the survey effort. Although the second round of video data collection was scheduled for March 6-8, the runs made on the first day needed to be redone due to problems with the camera placement, necessitating a fourth day of data collection on March 9, 2004.

Video Clip Creation

The survey participants were shown a single video display that contained the video scenes of the front windshield and interior rear-view mirror, the driver's side rear-view mirror, and the speedometer. The clips were assembled using a video-editing program [12]. They first had to be captured from the VHS tapes using an analog-to-digital converter [13]. After they were stored on the computer hard drive they were combined into a single composite scene using the video editing software. A screenshot from one of the video clips used in the survey is shown in Figure 3. Screenshots from all 13 clips can be found in Appendix B. Each created final video clip ranged from 1.5 to 2.5 minutes in length. The length of an individual clip was chosen based on events in the video that the researchers wanted to include or exclude, as well as with a survey participant's attention span in mind.

Figure 3. Sample Video Screenshot

Video Clip Selection

There were thirteen video clips chosen for the final survey. The final number of clips chosen was a result of several pilot test sessions, providing a balance between coverage of alternatives and attention span of participants. The pilot tests revealed that many participants lost interest after two minutes and had already started writing their opinions down. The final video clips were chosen to represent a variety of conditions in categories including lane configuration, traffic density, terrain, truck percentage, the presence of a median or guardrail, and shoulder configuration. The relevant data for each video clip included in the final survey is shown in Table 4 and Table 5.

Inductance Loop Detector Data Collection

To investigate the correlation of density with the user perceptions of LOS, it was necessary to collect speed and volume data at each data collection site during the video recording runs. These data were collected from the inductance loop detector (ILD) stations at each test site. FDOT personnel programmed the detectors at the sites selected for the study to record data in five-minute intervals (the hardware minimum interval) rather than the usual one-hour interval. This shorter interval provided traffic data that more accurately reflected the conditions depicted in the video clips. It should be noted that even with a five-minute data collection interval the conditions shown in the video clips could potentially vary from the average provided by the ILD data.

The ILD data were provided in the form seen in Appendix D. When available, there were three data files for each site – speed, count, and class. In the speed file, counts are provided for each speed range. The midpoints of the speed ranges are shown at the top of the table. In the class file, descriptions such as "CL01" are given to the columns. These refer to the specific class of vehicle counted in that group and are explained by the figure provided. These ILD data were used to categorize the collected video data and provided a starting point for selecting a range of conditions to be represented in the survey. From the data provided (speed, volume, and vehicle classification by lane) it was possible to calculate descriptive statistics for the traffic flow at each site, such as the percentage of heavy vehicles in the traffic stream, the total 5-minute volume, the average speed, and the density.

Table 4. Traffic Data for 13 Video Clips

					Observed or Estimated from Video Clip[1]							
Clip #	**Road**	**Dir.**	**Lanes**	**Clip Length**	**Terrain**	**Volume (veh/h/ln)**	**Avg. Speed (mi/h)**	**Density (veh/mi/ln)**	**Speed Diff. (mi/h)**	**Wtd. Speed Diff.**	**Truck %**	**Wtd. Truck %**
1	I-75	S	2	2:10	flat	550	75	7.3	4	2.2	0	0.0
2	I-75	S	3	1:52	flat	1200	70	17.1	9	10.8	30	36.0
3	I-75	N	2	2:00	flat	1300	65	20.0	12	15.6	0	0.0
4	I-95	N	2	1:35	flat	1400	54	25.9	8	11.2	10	14.0
5	I-75	S	3	1:40	rolling	600	72	8.3	10	6.0	10	6.0
6	I-95	S	2	1:59	flat	1100	72	15.3	7	7.7	0	0.0
7	I-75	S	2	2:00	flat	1550	70	22.1	6	9.3	30	46.5
8	I-75	N	3	2:01	flat	1350	70	19.3	15	20.3	20	27.0
9	I-75	S	2	2:00	flat	1900	63	30.2	7	13.3	0	0.0
10	I-95	N	2	1:43	flat	1150	73	15.8	6	6.9	30	34.5
11	I-75	S	3	1:26	flat	1000	75	13.3	6	6.0	10	10.0
12	I-75	S	2	1:27	flat	1800	65	27.7	8	14.4	15	27.0
13	Turnpike	S	2	2:03	rolling	700	75	9.3	12	8.4	40	28.0

[1] Density was calculated from volume and speed; speed differential is the difference in average speed between the inside lane and outside lane; truck percentage was rounded to nearest 5 percent; the weighted speed differential and truck percentage values are derived by multiplying the respective value by the volume and then dividing by 1000.

Table 5. Clip Sites, Dates, and Times

Clip #	Clip	Site	Time	Date	Closest City
1	189920 run 1	189920	12:36	3/8/2004	Wildwood
2	360317 run 1	360317	11:50	3/8/2004	Ocala
3	Tampa 0648	140190	6:48	11/21/2003	Tampa
4	730292 run 4	730292	14:47	3/7/2004	Daytona Beach
5	Micanopy 1101	269904	11:01	11/5/2003	Micanopy
6	730292 run 3	730292	14:35	3/7/2004	Daytona Beach
7	Tampa 0714	140190	7:14	11/21/2003	Tampa
8	360317 run 2	360317	12:04	3/8/2004	Ocala
9	Tampa 0704	140190	7:04	11/21/2003	Tampa
10	Daytona 1255	730292	12:55	11/4/2003	Daytona Beach
11	360317 run 3	360317	12:07	3/8/2004	Ocala
12	Tampa 1431	140190	14:31	11/21/2003	Tampa
13	970428 run 1	970428	13:49	3/8/2004	Winter Garden

Survey Sessions

Survey Form and Participant Instructions

The first section of the survey form addressed personal information and rural freeway travel habits, such as education level, income, number of years possessing a driver's license, the number of rural freeway trips taken per month, the average length of the rural freeway trips taken, and level of driving aggressiveness.

The second section of the survey was for recording the participants' opinions and rankings of the video clips. It was divided into two parts for each of the thirteen clips. The first part asked the participant to rank the quality of the trip depicted in the video clip with one of the following descriptors: 'Very Poor', 'Poor', 'Fair', 'Good', 'Very Good', or 'Excellent'. For this preliminary study, the number of levels was kept at six for general correspondence with the six levels of the HCM (A-F). Certainly, it has yet to be determined whether travelers actually perceive six different levels of service, but this issue was beyond the scope of this study. The second part asked the participant to record why they ranked the video clip as they did, listing all factors that significantly contributed to their ranking. The participants were to then number these according to their relative significance.

Finally the form included questions about the survey itself. These included the participant's opinion about the video clips as a representation of rural freeway travel and whether the participant would have changed their rankings based on the purpose of the trip (e.g., business, recreational, or social).

A one page written survey instruction sheet was developed because there was a significant amount of information that needed to be communicated to the participants in order for them to complete the survey form in a manner which would be useful as study data. The survey form and participant instruction sheet are provided in Appendix C.

Conducting the Survey Sessions

Survey participants were recruited from various sources. They include the following:

- Undergraduate students in the University of Florida civil engineering program, recruited from the introductory transportation engineering course
- Graduate students in the University of Florida civil engineering program, recruited from the transportation degree program
- Employees of the University of Florida Technology Transfer Center
- Employees of the Florida Department of Transportation
- Alachua county residents (Random participants recruited by the Florida Survey Research Center)

The undergraduate students were recruited from the introductory transportation engineering course (TTE 4004) during the Fall 2004 semester. The graduate students were those enrolled in a transportation engineering degree program during the Fall 2004 semester. The University of Florida Technology Transfer Center is an organization that provides training and technical assistance to Florida's transportation and public works professionals. Their survey session was conducted at their off-site headquarters in Gainesville, FL, with participants ranging from high-school educated support staff to professionals with graduate degrees. The FDOT survey session was conducted at the central office in Tallahassee, FL. This session also included participants of varying backgrounds and demographics. The public sample was comprised of Alachua county residents, recruited by the University of Florida Survey Research Center (UFSRC). The UFSRC was instructed to recruit individuals with varying socio-demographic characteristics and also make sure that the participants had experience driving on rural freeways. Additionally, they did not recruit college students as there was already a sufficient number in this group. The UFSRC contacted individuals for potential study participation through a random telephone number generation process. The telephone numbers were randomly generated, as opposed to being randomly selected from a source such as a phone book, so that people with unlisted phone numbers would not be systematically excluded. A total of 126 people participated in this study. The locations, dates, and groups of participants taking the survey during each session are listed in Table 6.

Table 6. Dates and Locations of Survey Sessions

Survey Session	Date	City	Location	Participants	# of Surveys
1	8/4/04	Gainesville	UF Technology Transfer Center	T^2 employees	16
2	11/16/04	Tallahassee	Florida DOT Central Office	DOT employees	11
3	12/2/04	Gainesville	University of Florida	undergraduate students	14
4	12/2/04	Gainesville	University of Florida	undergraduate students	9
5	12/4/04	Gainesville	UF Hilton Conference Center	public[1]	13
6	12/4/04	Gainesville	UF Hilton Conference Center	public[1]	15
7	12/4/04	Gainesville	UF Hilton Conference Center	public[1]	11
8	12/9/04	Gainesville	University of Florida	undergraduate students	20
9	1/22/05	Gainesville	University of Florida	public[1]	9
10	1/27/05	Gainesville	University of Florida	graduate students	8
			Total Number of Surveys		**126**

[1] Participants were recruited through the University of Florida Survey Research Center and paid a participation fee of $50 each.

With the video format of the survey, multiple survey participants could be accommodated during each session. The video clips were shown with a video projector and wall-mounted screen, located between 10 and 20 feet away from the participants depending on the specific survey location. The viewing room was arranged so that each participant had an unobstructed view of the screen, which was placed as close as possible to eye level so it approximated looking through a vehicle’s windshield. The setup of one of the survey sessions is depicted below in Figure 4.

Before viewing the clips the participants were given the instruction sheet and time to read it. These written instructions were also verbally reviewed by the session moderator, as well as some supplemental information. The participants were also told that they could ask interpretation questions in-between the viewing of the video clips.

Two example clips, each 20 seconds long, were shown to the survey participants to demonstrate the upper and lower ends of the range of possible traffic flows. The first was a nearly empty four-lane freeway and the second was stop-and-go traffic along a four-lane freeway. The participants were then shown each of the 13 video clips and instructed to watch each clip entirely before writing their responses. The order of the presented clips was different for each survey session.

Figure 4. Setup of a Survey Session

CHAPTER 4
ANALYSIS AND RESULTS

This chapter contains information about the methodology used to analyze the survey data, as well as the results of these analyses.

Analysis Method

To determine the extent to which the participants' responses correspond to the six LOS rankings, a statistical analysis was performed to predict the probability of selecting discrete rankings (1-6 as included in the survey). An ordered probit model was chosen for the statistical analysis approach. This model is well suited to the analysis of discrete choice data, particularly data that have an ordinal (or ranking) nature to the response range [14]. The ordered probit model is derived by defining an unobserved variable, z, that is the basis for modeling the ordinal ranking of data (in this case the six clip rankings) [15]. This variable is specified as a linear function for each observation n such that

$$z_n = \boldsymbol{\beta X_n} + \varepsilon_n \tag{1}$$

where $\boldsymbol{X_n}$ is a vector of independent variables (such as traffic conditions) influencing the clip ranking for observation n, $\boldsymbol{\beta}$ is a vector of estimable parameters, and ε_n is a random disturbance. In this analysis, y is defined as each participant's evaluation of each of the 13 video clips. Using this equation, the observed clip ranking (Excellent = 1, …, Very Poor = 6), y_n, for each observation is written as,

$$\begin{aligned} y_n &= 1 \text{ if } z_n \leq 0 \\ y_n &= 2 \text{ if } 0 < z_n \leq \mu_1 \\ y_n &= 3 \text{ if } \mu_1 < z_n \leq \mu_2 \\ y_n &= 4 \text{ if } \mu_2 < z_n \leq \mu_3 \\ y_n &= 5 \text{ if } \mu_3 < z_n \leq \mu_4 \\ y_n &= 6 \text{ if } z_n \geq \mu_4 \end{aligned} \tag{2}$$

where the μ values are estimable parameters, referred to as thresholds, that are used in the determination of the probability for y_n [15]. The μ values are estimated jointly with the model parameters ($\boldsymbol{\beta}$). The estimation problem then becomes one of determining the probability that a participant will select a particular ranking for each clip. In using the ordered probit model, it is assumed that the error term, ε_n, is normally distributed with a mean of 0 and a variance of 1. A positive increase in the $\boldsymbol{\beta}$ term implies that an increase in x will increase the probability of the highest response category (i.e., y = 6). An increase in the $\boldsymbol{\beta}$ term also implies that the probability of the lowest response category ($y = 1$) is decreased. The resulting response category selection probabilities can be calculated as,

$$
\begin{aligned}
P(y_n = 1) &= \boldsymbol{\Phi}(-\boldsymbol{\beta X_n}) \\
P(y_n = 2) &= \boldsymbol{\Phi}(\mu_1 - \boldsymbol{\beta X_n}) - \boldsymbol{\Phi}(-\boldsymbol{\beta X_n}) \\
P(y_n = 3) &= \boldsymbol{\Phi}(\mu_2 - \boldsymbol{\beta X_n}) - \boldsymbol{\Phi}(\mu_1 - \boldsymbol{\beta X_n}) \\
P(y_n = 4) &= \boldsymbol{\Phi}(\mu_3 - \boldsymbol{\beta X_n}) - \boldsymbol{\Phi}(\mu_2 - \boldsymbol{\beta X_n}) \\
P(y_n = 5) &= \boldsymbol{\Phi}(\mu_4 - \boldsymbol{\beta X_n}) - \boldsymbol{\Phi}(\mu_3 - \boldsymbol{\beta X_n}) \\
P(y_n = 6) &= 1 - \boldsymbol{\Phi}(\mu_4 - \boldsymbol{\beta X_n})
\end{aligned}
\quad (3)
$$

In the above equations, $\Phi(\cdot)$ represents the cumulative normal distribution,

$$\Phi(u) = \frac{1}{\sqrt{2\pi}} \int_{-\infty}^{u} e^{-\frac{1}{2}w^2} dw \quad (4)$$

Figure 5 provides a visual illustration of how Equations 3 and 4 are used to calculate the probability of the respective clip rankings.

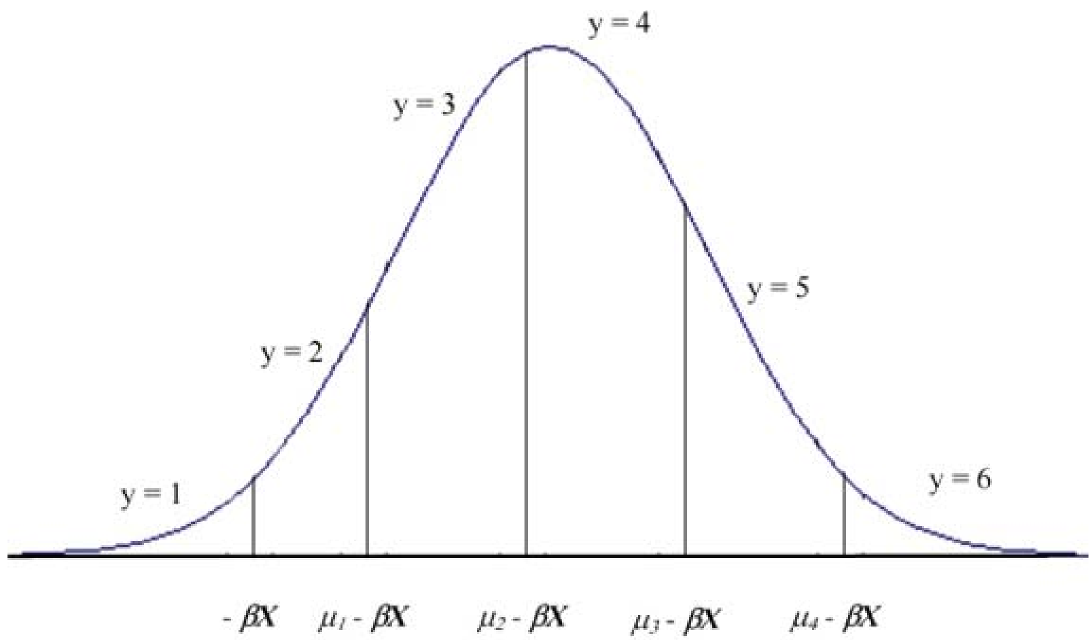

Figure 5. Illustration of the Ordered Probit Model (with $\mu_0 = 0$)

A positive value for the $\boldsymbol{\beta}$ term implies that an increase in x will increase the probability that the highest category response will be returned (in this case, $y = 6$). Likewise, a negative value for the $\boldsymbol{\beta}$ term implies that an increase in x will increase the probability of returning the lowest category response (i.e., $y = 1$). This is illustrated in Figure 6.

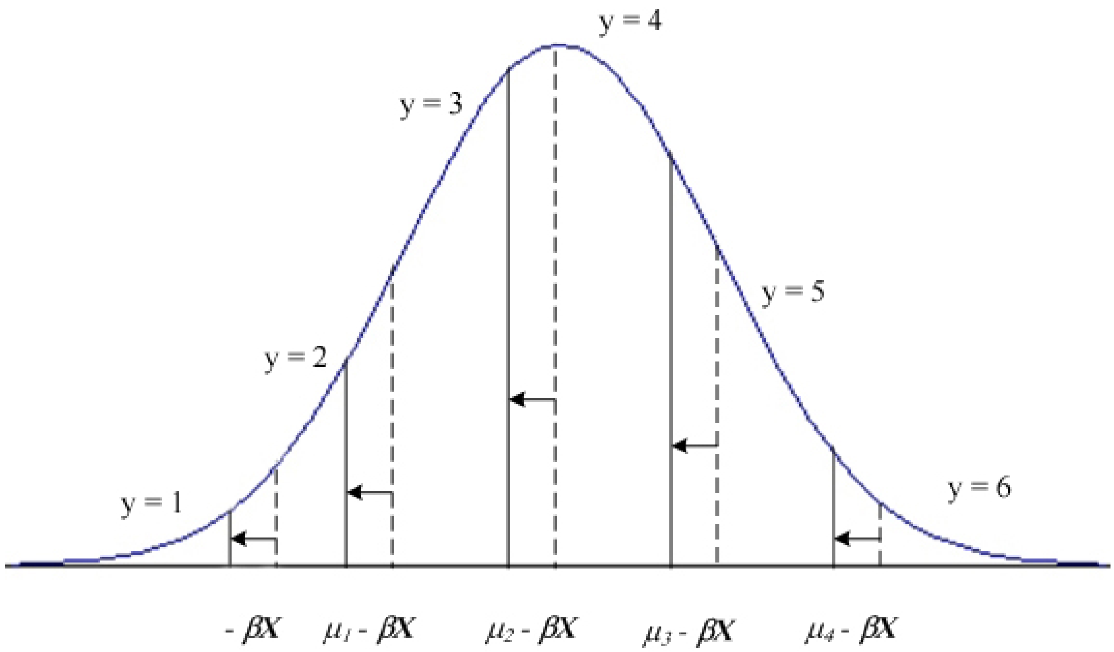

Figure 6. Illustration of an Ordered Probit Model with an Increase in $\boldsymbol{\beta X}$

This model can be estimated using maximum likelihood procedures. However, an adjustment must be made to the estimation procedure. Since each of the 126 participants viewed 13 clips and thus generated 13 observations (for a total of 1638 observations), there are unobserved characteristics that are unique to each participant that will be reflected in all 13 of their rankings. If this is not accounted for in the model, the model will be estimated as though each of the 1638 observations came from a unique participant. This approach would result in lower standard errors in the model's estimated parameters, leading to inflated *t*-statistics and potential biases in parameter estimates.

The situation can be handled with a standard random effects approach, by rewriting Eq. (1) as,

$$z_{ic} = \boldsymbol{\beta X_{ic}} + \varepsilon_{ic} + \varphi_i \qquad (5)$$

where i denotes each participant (i = 1,…,126), the c denotes each video clip (c = 1,…,13), φ_i is the individual random effect term and all other terms are as previously defined. The random effect term φ_i is assumed to be normally distributed with mean 0 and variance σ^2. An output from this random effects model estimation is an estimate of σ, the significance of which determines the significance of the random effects model relative to the standard ordered probit model [14].

Statistical Analysis

Three models were estimated[1] in this study, and the results for all three are shown in Table 8. The first analysis explored how the quality of service perceptions of the survey participants correlated with just the measure of density and the HCM LOS thresholds. The positive coefficient calculated for density indicates that, as density increases, the likelihood of a traveler perceiving a worse LOS increases. The very high *t*-statistic (coefficient divided by its standard error) for density indicates that it is certainly significant in the model. The standard deviation of the random effects term, σ, is also highly significant, meaning that the choice of a random effects model for this data set was justified. The overall model goodness-of-fit measure, adjusted ρ^2, was 0.355. The ρ^2 value can range between 0 and 1. A ρ^2 value of 1.0 indicates a perfect model fit. The adjusted ρ^2 value is calculated as,

$$\bar{\rho}^2 = 1 - \frac{LL(\beta) - K/2}{LL(0)} \tag{6}$$

where $\bar{\rho}^2$ represents the adjusted ρ^2 value, K represents the number of variables in the model, $LL(\beta)$ represents the log likelihood at convergence, and $LL(0)$ represents the log likelihood at zero [15]. The adjusted ρ^2 value compensates for the number of independent variables in the model, as increasing the number of variables inherently increases the ρ^2 value, even for variables that are not statistically significant.

Using the calculated values in Table 8, the threshold values for density can be calculated as $(\mu_k - \beta_0)/\beta_1$, where k designates the four threshold values for this model in Table 8 (assuming the lowest threshold is 0, i.e., $\mu_0 = 0$). A comparison between the calculated threshold values from this model and the HCM LOS thresholds is given in Table 7. These thresholds are lower than the HCM thresholds across all LOS rankings, indicating the participants in this study had a lower tolerance for traffic congestion than is generally indicated by the HCM LOS thresholds.

Table 7. Comparison of Estimated and HCM LOS Thresholds

LOS	Estimated Thresholds (pc/mi/ln)	HCM thresholds (pc/mi/ln)
A	0-6	0-11
B	>6-14	>11-18
C	>14-22	>18-26
D	>22-29	>26-35
E	>29-39	>35-45
F	>39	>45

[1] The LIMDEP software package was used [16].

Table 8. Model Estimation Results

	Model 1		Model 2		Model 3	
Independent Variable	**Parameter**	***t*-statistic**	**Parameter**	***t*-statistic**	**Parameter**	***t*-statistic**
Constant	-0.730	-9.34	1.750	2.28	1.702	2.16
Traffic Characteristics						
Density (pc/mi/ln)	0.127	44.41	0.097	11.48	0.097	11.14
Average Speed (mi/h)			-0.032	-3.30	-0.032	-3.18
Volume Weighted Speed Differential			0.014	1.47	0.014	1.38
Volume Weighted Truck %			0.012	5.36	0.012	5.29
3 Lanes (1 - Yes, 0 - No)			-0.107	-1.67	-0.105	-1.57
Personal and Travel Characteristics						
Age > 25 (1 - Yes, 0 - No)					-0.251	-2.26
Income (thousands of $)					-0.004	-2.78
Avg. # of Rural Freeway Trips per Month					0.029	1.68
Average One-Way Trip Distance > 100 miles? (1 - Yes, 0 - No)					0.294	2.35
Threshold Values						
μ_1	1.004	24.62	1.024	24.20	1.017	23.94
μ_2	2.060	43.60	2.102	42.65	2.094	41.81
μ_3	3.005	58.61	3.062	59.03	3.050	58.20
μ_4	4.221	68.41	4.294	65.31	4.278	63.63
Standard Deviation of Random Effects						
σ	0.481	9.40	0.492	9.422	0.435	8.99
Number of Observations		1638		1638		1625
Log Likelihood at Zero		-3500.30		-3500.30		-3465.70
Log Likelihood at Convergence		-2257.68		-2233.07		-2207.58
$\bar{\rho}^2$		0.3547		0.3612		0.3616

Note: A *t*-statistic of 1.282 corresponds to a 90% confidence level for a one-tailed *t*-test.

The second analysis incorporated additional traffic and roadway characteristics into the random effects model to predict perceived LOS. The additional traffic variables found to be significant have intuitive coefficient signs. As expected, a higher average speed resulted in a more favorable LOS ranking. Speed differential (calculated as the difference in average speeds between the inner lane and the outer lane) as weighted by traffic volume, has a positive coefficient sign, indicating that as the average speed difference between lanes increases, participants were more likely to assign a worse LOS. Likewise, an increase in the truck percentage, as weighted by traffic volume, resulted in a higher probability of a worse LOS ranking. The speed differential and truck percentage variables were weighted by traffic volume since these variables were more significant than their non-weighted versions. This makes some intuitive sense as lane speed differentials and higher percentages of trucks probably only begin to affect motorists' perceptions of trip quality under higher volume conditions. Survey participants were also more likely to give a better LOS ranking to three-lane (in one direction) roadway cross sections than two lanes. The rationale given by the vast majority of the participants for this was because they felt they had more movement opportunities for any given traffic conditions and more "outs" in case something went wrong. While density was still a significant variable, its *t*-statistic was considerably lower in this model. Furthermore, the adjusted ρ^2 value improved, although not by a large amount, to 0.361. The standard deviation of the random effects term was again significant in this analysis, justifying the use of a random effects model.

The third analysis incorporated all of the variables of the second analysis, as well as personal and rural freeway travel characteristics. The indicator variable, 'Age > 25' indicates that participants over 25 years of age are more likely to assign a given set of conditions a better LOS, and likewise for those with higher household incomes (this variable was coded as a continuous variable, using the midpoints of the presented ranges). Travelers who drive on rural freeways more frequently are more likely to perceive a worse LOS, as are those whose average rural freeway trip is over 100 miles in one-way length. This would imply that those individuals who spend more time traveling on rural freeways are probably more critical of any given set of conditions on those roadways. The same traffic characteristic variables included in this model as in the second model maintained very similar coefficient and *t*-statistic values. While the personal and travel characteristics variables added in the third model were statistically significant by virtue of their respective *t*-statistics, overall they added very little additional predictive power of LOS perceptions as indicated by a very small improvement in the adjusted ρ^2 value. From a practical standpoint, this result is encouraging as it does not make a case for the collection of data other than roadway and traffic characteristics. The standard deviation of the random effects term was once again significant.

Participant Rating of Video Survey Method

The last question on the survey form (as seen in Appendix A) asked participants to rate how well the video clips simulated the driving experience for the conditions depicted on the screen. The majority of participants found the survey to be a "very good" representation of the actual driving experience, with 95% of the participants rating the survey as a "good" or better representation of the actual driving experience. The responses to this question are tabulated in Table 9. As shown in this table, the average response from participants was approximately a 2 out of 6, corresponding to "very good".

Table 9. Video Survey Method Ratings

Ranking	**Excellent**	**Very Good**	**Good**	**Fair**	**Poor**	**Very Poor**
	1	2	3	4	5	6
Frequency	21	64	36	5	1	0
Percent of Total Responses (%)	17	50	28	4	1	0
Average Rank	**2.2**					

CHAPTER 5
CONCLUSIONS AND RECOMMENDATIONS

Three model formulations were explored in this study. The first model demonstrated two important points. First, density is definitely highly correlated with traveler perceptions of trip quality on rural freeways. Second, travelers are less tolerant of traffic congestion on rural freeways than is currently suggested by the HCM. If the single service measure of density is to be retained in the HCM for freeways, the concept of having different sets of thresholds for rural and urban freeways should be considered. This would be consistent with the treatment of arterials in the urban streets chapter of the HCM, where there are currently four different sets of average speed thresholds for four different arterial classifications.

The second model, with the inclusion of additional traffic and roadway variables showed some improvement in the replication of LOS perceptions. Density was still very significant, but this model indicates that the incorporation of additional traffic and roadway variables into the level of service methodology should be considered, especially since these variables are easily collected with the traffic monitoring infrastructure and roadway inventories available to almost all transportation agencies.

The third model identified some significant personal and travel factors, as well as the roadway and traffic characteristics from the previous model. The results of this model indicated that the personal and travel characteristics of the individual road user can influence their perception of LOS. However, from a practical standpoint, the implementation of this model versus the second model is hardly warranted due to the negligible gain in model fit and the complexity of trying to measure these variables.

This study provided some preliminary insight into travelers' perception of trip quality on rural freeways. However, additional research is certainly needed to further define the complex relationship between traveler perceptions of LOS and the interrelated factors of traffic, roadway, and personal characteristics corresponding to those perceptions. For example, a more regionally diverse population might be surveyed. Additionally, an expanded sample, both geographically and in roadway conditions, could provide more comprehensive coverage of the roadway and traffic condition combinations. Eventually, the results from this type of video-based study should also be compared to results obtained from a comparable in-field driving experiment. If the video survey is shown to be an accurate method of simulating traffic conditions, it can be used in future studies and will be more effective than in-field surveys. Ultimately, a better understanding of travelers' perceptions of quality of service will lead to better decisions about infrastructure investments.

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13. ADS Pyro A/V Link. Last Accessed March 15, 2005, from http://www.adstech.com/products/API-555/intro/api555_intro.asp?pid=API-555

14. Greene, William H., (2003). *Econometric Analysis, Fifth Edition*. Prentice Hall. Upper Saddle River, NJ.

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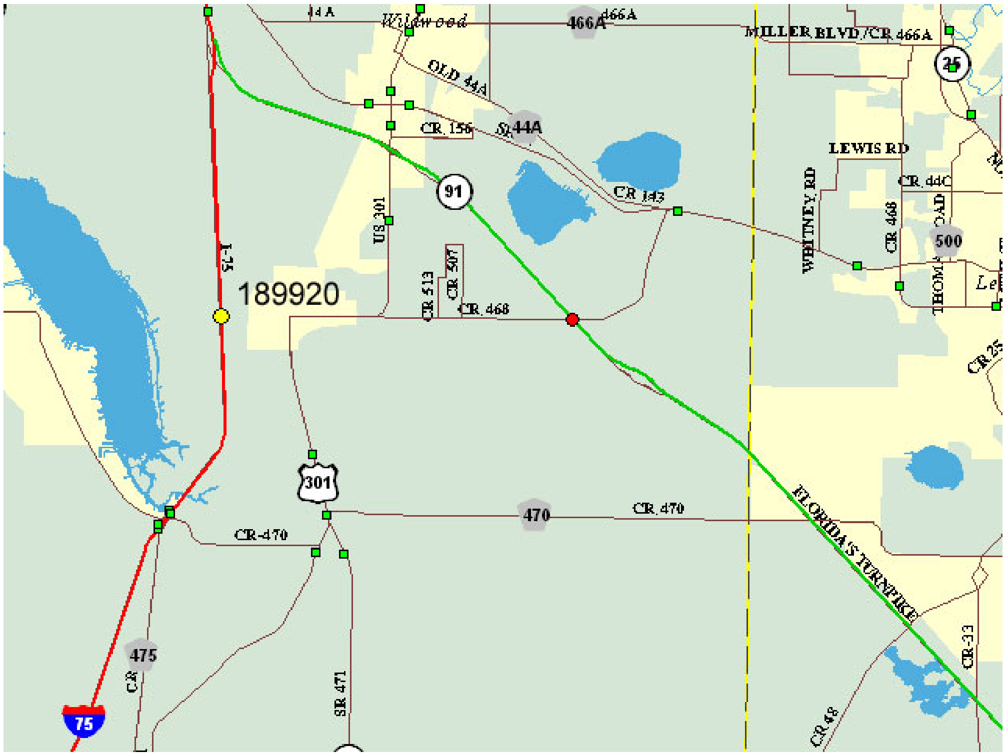
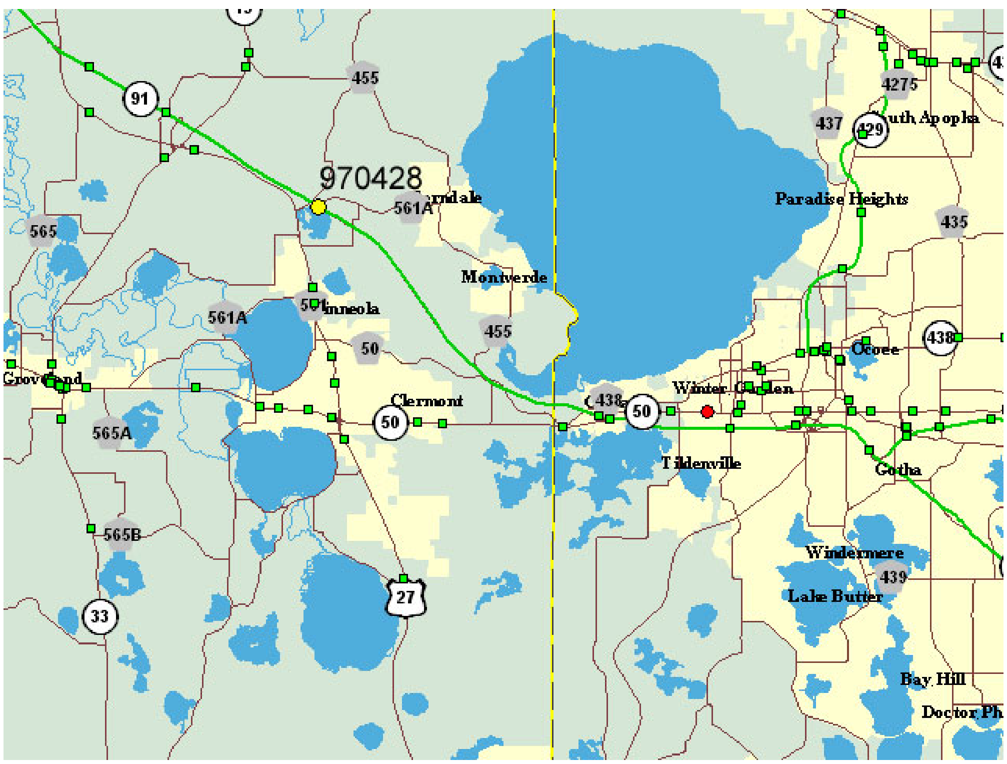
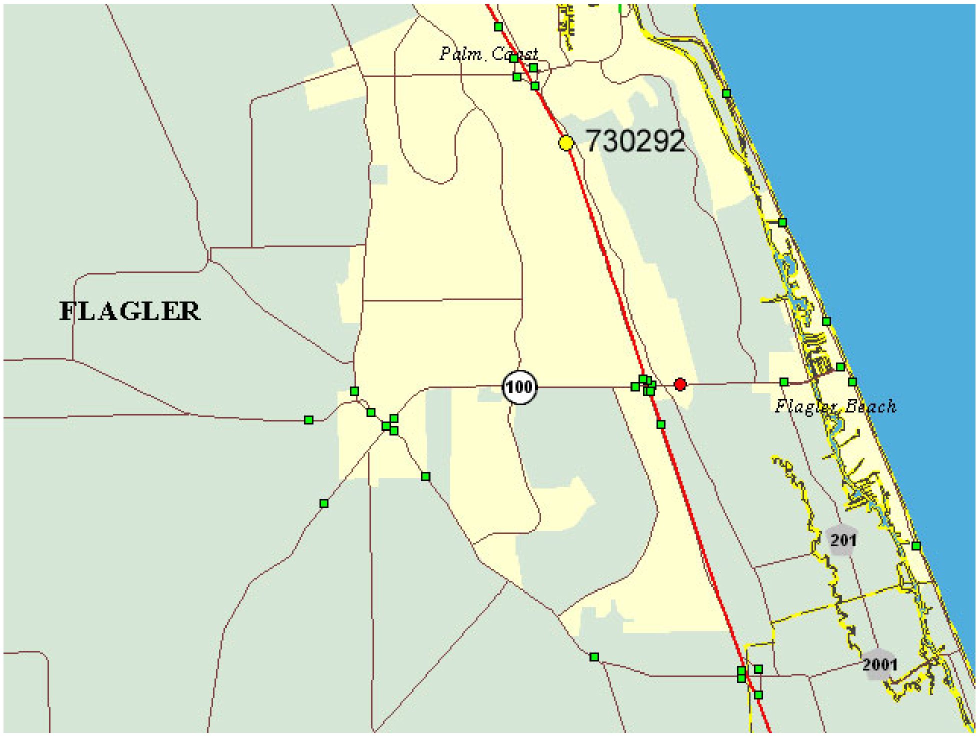
16. LIMDEP 8.0. http://www.limdep.com Econometric Software, Inc. Last Accessed March 23, 2005.

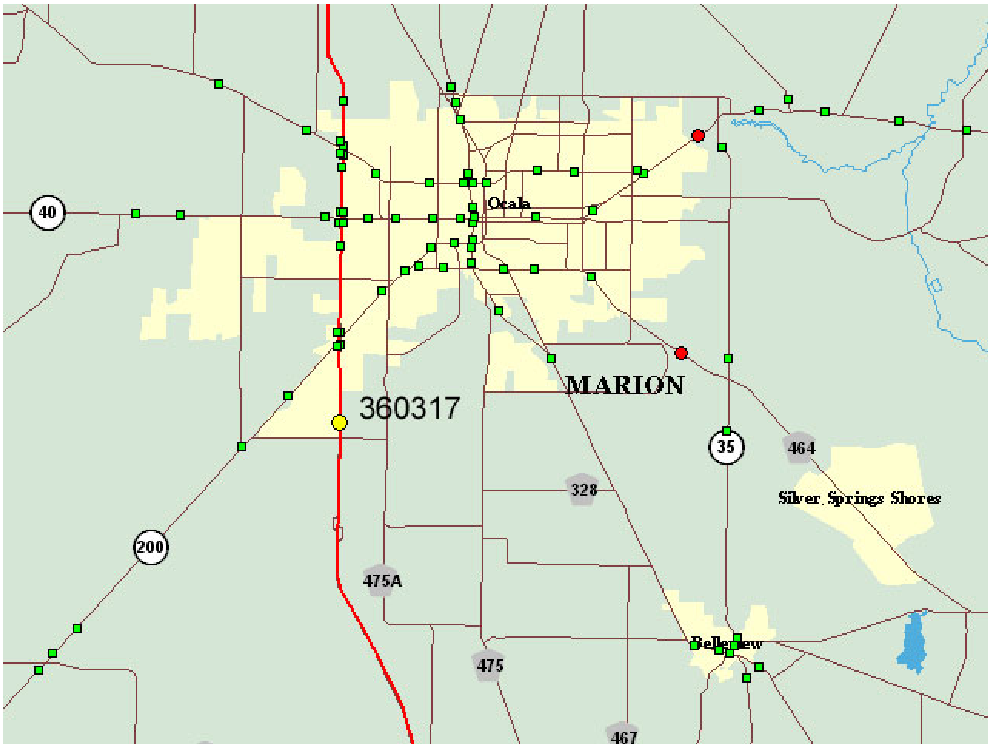
APPENDIX A
LOCATIONS OF DATA COLLECTION SITES

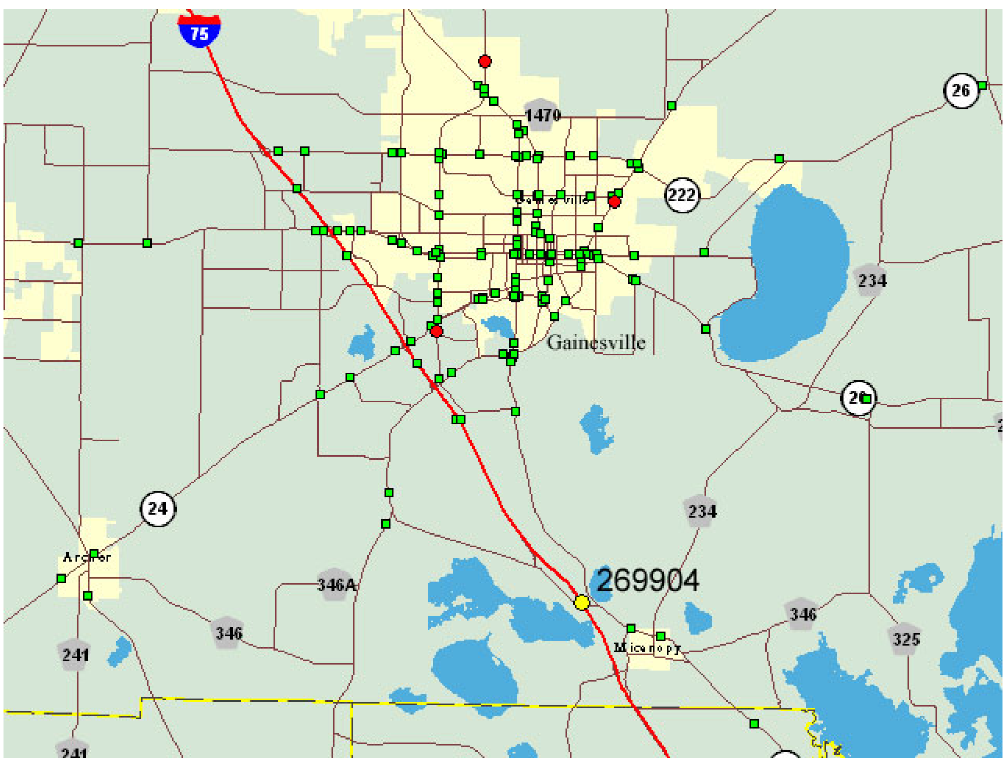

Wildwood
466A
MILLER BLVD / CR 466A
25
OLD 44A
CR 156
44A
LEWIS RD
91
CR 143
WHITNEY RD
CR 44C
CR 468
500
US 301
I-75
189920
CR 513
CR 507
CR 468
301
470
CR 470
CR-470
FLORIDA'S TURNPIKE
475
75
SR 471
CR 33
CR 48


455
91
970428
561A
Montverde
565
561A
50
455
565A
Clermont
50
438
50
Winter Garden
Tildenville
Ocoee
438
Paradise Heights
435
4275
437
429
Gotha
Windermere
439
Lake Butler
Bay Hill
565B
33
27


Palm Coast
730292
FLAGLER
100
Flagler Beach
201
2001


54
140190
Land O' Lakes
581
579
597
Wesley Chapel
Pebble Creek
Cheval
Lutz
41
HILLSBOROUGH
75
579
597
581
301
Lake Magdalene
University
582A
582
Greater Carrollwood
Thonotosassa

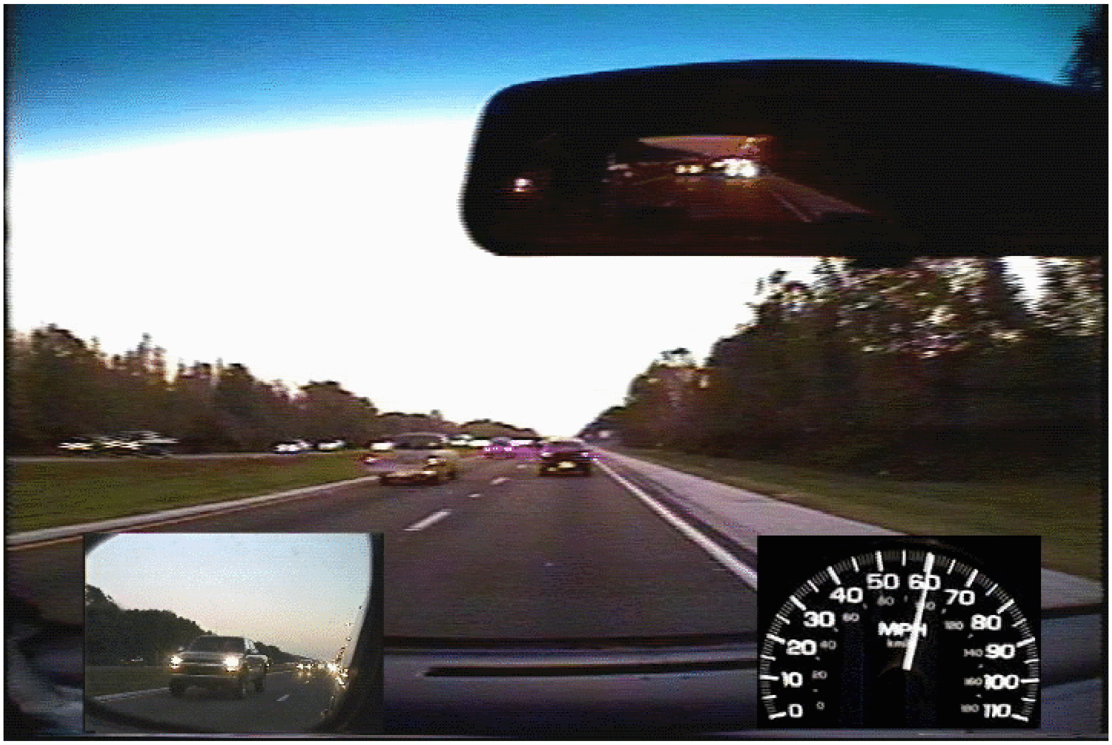

40
Ocala
MARION
360317
35
464
328
Silver Springs Shores
200
475A
475
467


75
1470
26
222
234
Gainesville
24
234
269904
Archer
346A
346
346
325
241
Micanopy
241

APPENDIX B
VIDEO CLIP SCREENSHOTS

(Clip 1)

(Clip 2)


0
10
20
30
40
50
60
70
80
90
100
110
MPH

(Clip 3)


0
10
20
30
40
50
60
70
80
90
100
110
MPH

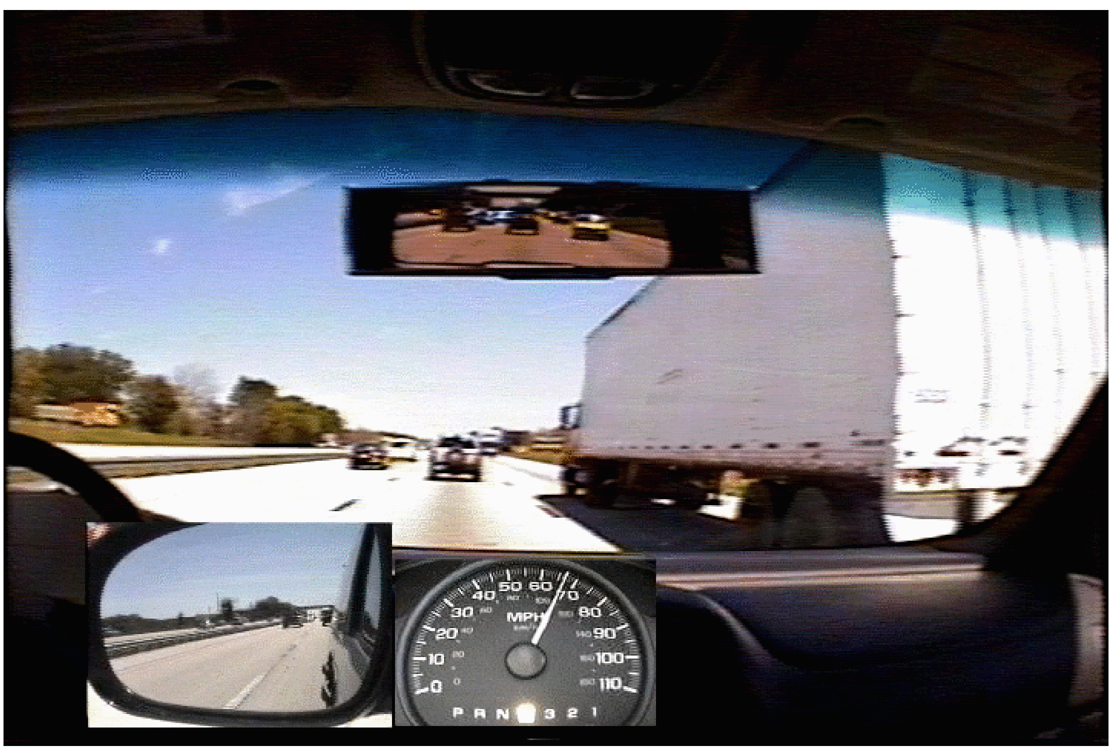
(Clip 4)

(Clip 5)

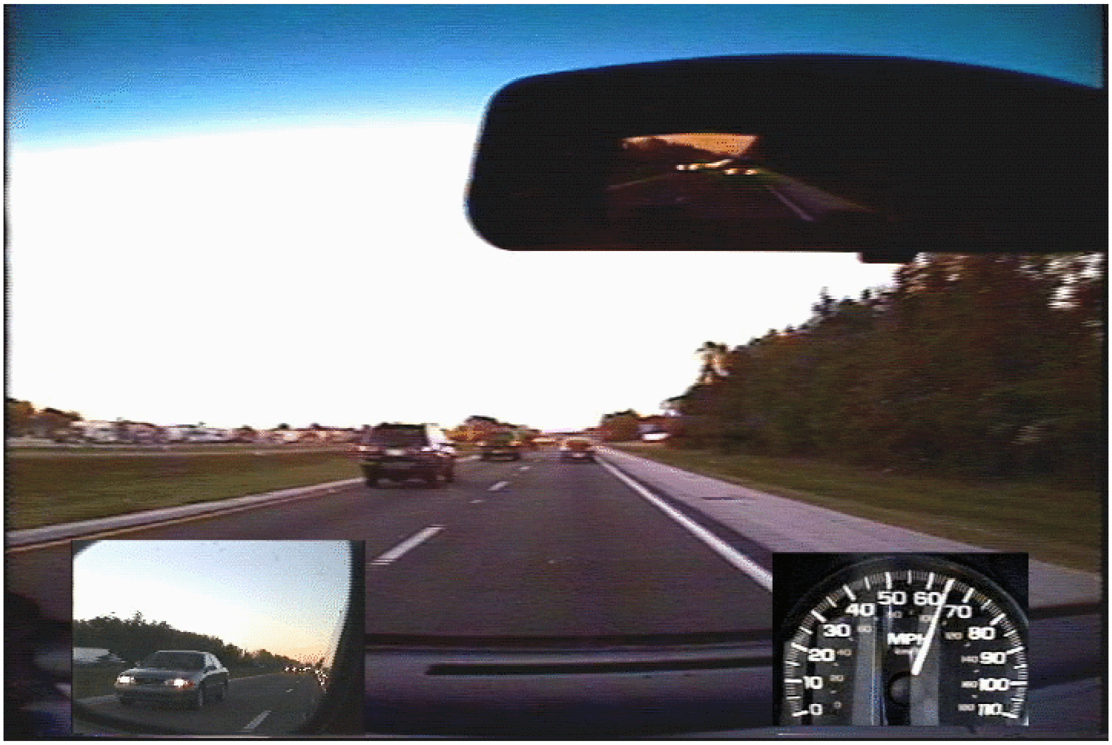
(Clip 6)


0
10
20
30
40
50 60
70
MPH
80
90
100
110

(Clip 7)

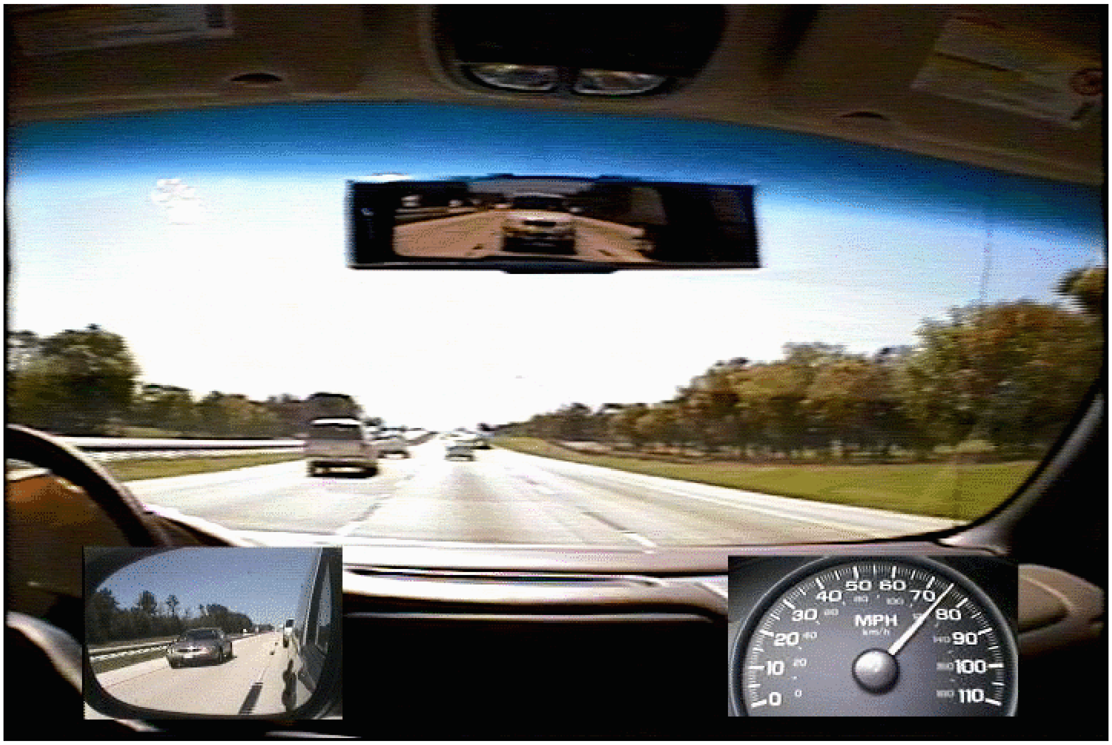

0
10
20
30
40
50 60
70
MPH
80
90
100
110
P R N 3 2 1

(Clip 8)

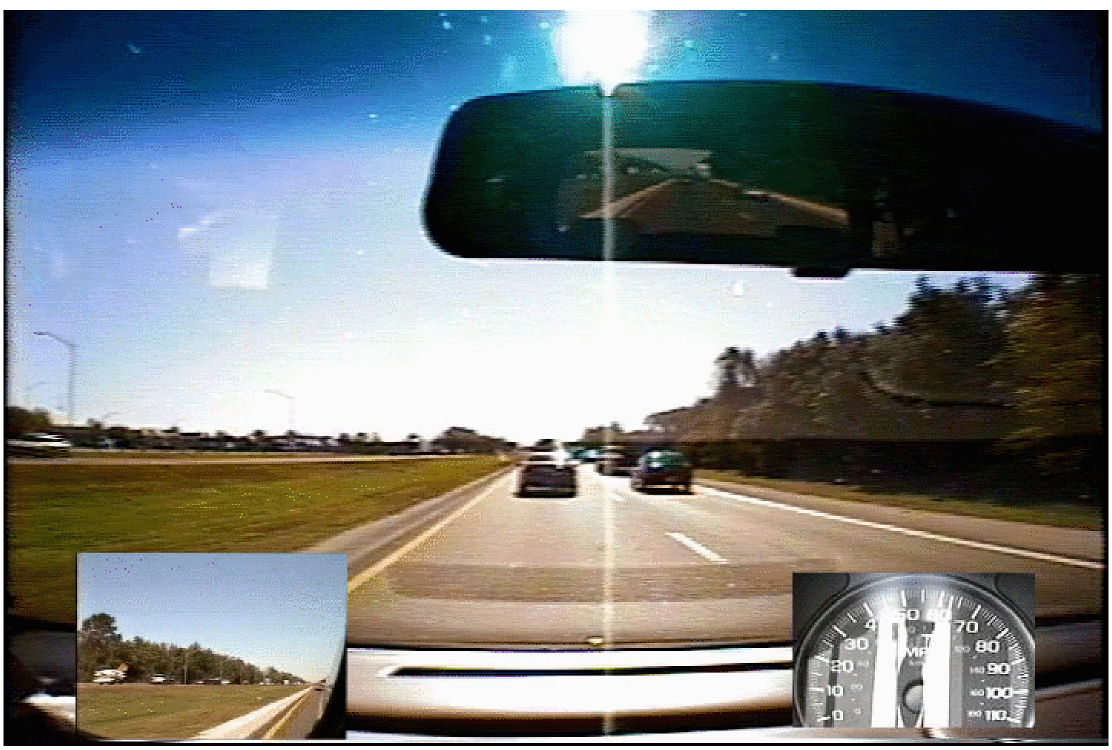


(Clip 9)

(Clip 10)



(Clip 11)



(Clip 12)

(Clip 13)

APPENDIX C
PARTICIPANT INSTRUCTIONS AND SURVEY FORM

UNIVERSITY OF FLORIDA

TRC
Transportation Research Center

Rural Freeway Trip Quality Survey

In the exercise you are about to participate in, you will be watching a series of 13 short video segments of various roadway and traffic conditions on rural freeways. A rural freeway is a freeway that travels through relatively unpopulated areas. Rural freeways are typically used for longer trips, such as city-to-city trips. All freeway segments (whether in urban, rural, or other types of areas) are characterized by opposing directions of traffic being separated by either a physical barrier or open space. All freeways are also characterized by limited access, that is, entry to and exit from a freeway can only be made at interchanges (on- and off-ramps). For rural freeways, interchanges are spaced much further apart than along freeways in urban areas.

Each of the video clips is approximately 1.5 to 2 minutes in length. Each clip is intended to give you a "snapshot" of the typical conditions experienced over the course of an extended trip on a rural freeway. When watching each video clip, please imagine and/or keep the following points in mind:

- The conditions viewed on the video clip for about 2 minutes are intended to be representative of what you would experience for a much longer trip (30 minutes or more).
- Imagine how you would personally drive, or try to drive, in the given conditions. You are not limited to the driving behavior of the vehicle from which the video is being viewed. The intent of the video vehicle is to provide you with a reasonable representation of the typical conditions being experienced by ALL motorists on that section of rural freeway. Therefore, your survey responses should not be specific to how the video vehicle was being driven. If you feel like you would, and could, drive differently under the given conditions, then base your survey responses on that. It is important that your survey responses reflect how the given conditions affect your perception of trip quality based upon your own desired driving behavior.

After watching each video clip, we ask that you do the following on the survey form:

- Rank (from Very Poor to Excellent) the travel conditions
- In the space provided, briefly list the reasons/factors for why you ranked the conditions in that video clip as you did. Please be as specific as possible—for example, you might say 'opportunities to pass other vehicles in order to maintain my desired speed were limited', as opposed to 'speed was too low'.

The video clips are intended to be weather neutral—that is, in developing the video clips it was not our intent to have weather be a significant factor in your trip quality perceptions. Although the lighting conditions may vary somewhat, please do not factor in the environmental conditions unless you feel very strongly about a certain condition.

If you recognize the freeway section, disregard previous knowledge and experience and base your ranking strictly upon the conditions observed in the video clip.

Thank you for your cooperation and participation.

UNIVERSITY OF FLORIDA

TRC
Transportation Research Center

About Yourself

Gender: ☐ Male ☐ Female

Age: ☐ 16 to 25 years ☐ 26 to 45 years ☐ 46 to 65 years ☐ Over 65 years

Marital Status: ☐ Single ☐ Married ☐ Separated/Divorced ☐ Widowed

Highest level of education:

☐ Some or no high school ☐ High school diploma or equivalent
☐ Technical college degree (A.A.) ☐ College degree ☐ Post-graduate degree

Approximate annual household income:

☐ No income ☐ Under $25,000 ☐ $25,000 – 49,999 ☐ $50,000 – 74,999
☐ $75,000 – 99,999 ☐ $100,000 – 149,999 ☐ $150,000 or more

Number of years possessing a driver's license: _________

About Your Rural Freeway Driving

Typical number of rural freeway round trips made during a month?

☐ 1 to 2 ☐ 3 to 4 ☐ 5 to 6 ☐ 7 to 8 ☐ 9 to 10 ☐ 11 to 12 ☐ Over 12

Typical percentage of these trips made as a driver ______, as a passenger ______ (should sum to 100)

Typical one-way length of trip made on a rural freeway (in miles)?

☐ less than 16 miles ☐ 16 to 30 ☐ 31 to 45 ☐ 46 to 60 ☐ 61 to 75 ☐ 76 to 100
☐ 101 to 125 ☐ 126 to 150 ☐ 151 to 175 ☐ 176 to 200 ☐ Over 200

Vehicle type most often used for rural freeway trips:

☐ Sedan ☐ Sports car ☐ Pickup truck ☐ SUV ☐ Minivan
☐ Full-size van ☐ RV/Motorhome ☐ Motorcycle ☐ Other _______________

Typical number of passengers in vehicle for rural freeway trips?

☐ 0 – Driver only ☐ 1 ☐ 2 ☐ 3 ☐ 4 or more

Typical driving style on rural freeways (on a scale from 1-5, with 1 being 'Very Conservative' and 5 being 'Very Aggressive'): ________

When driving alone, versus driving with passengers, does your driving style become:

☐ Less aggressive ☐ Stay the same ☐ More aggressive

Your Opinions

Rank the overall quality of your trip (Excellent, Very Good, Good, Fair, Poor, Very Poor) for the given roadway and traffic conditions observed in each video clip. In the space provided, list all the significant factors/reasons that influenced your ranking of the trip quality for each video clip. After listing the factors, please number them from most significant to least significant (with 1 being the most significant).

Video Clip	Rank	Comments
1		
2		
3		
4		
5		
6		
7		
8		
9		

10		
11		
12		
13		

In general, how would the purpose of your trip (such as business, recreational, social) affect the trip quality rankings assigned above (e.g., higher, lower, not at all)?

____

____

____

If the conditions in the video clips were encountered in an urban setting, and the trip length was relatively short, how would this affect the trip quality rankings assigned above (e.g., higher, lower, not at all)?

____

____

____

How would you rate this exercise in terms of its ability to give you a reasonable feel for the traffic and roadway conditions you would experience if you were actually driving your vehicle along this roadway under these traffic conditions?

☐ Excellent ☐ Very Good ☐ Good ☐ Fair ☐ Poor ☐ Very Poor

APPENDIX D
SAMPLE LOOP DETECTOR DATA

Tag	County	Site	Lane	Year	Month	Day	Hour	Min	Int	15	23	28	33	38	43	48	53	58	63	68	73	78	83	91	Total Vol.	Avg. Spd[1]	5 min vol[1]	veh/hr/ln[1]	Density[1]
SPD	18	9920	1	04	03	08	00	05	005	0	0	0	0	0	0	0	0	2	1	3	4	3	1	1	15	72.2			
SPD	18	9920	2	04	03	08	00	05	005	0	0	0	0	0	0	0	0	0	0	0	2	5	1	0	8	77.4	23	138	1.86
SPD	18	9920	3	04	03	08	00	05	005	0	0	0	1	0	0	0	0	0	0	0	2	4	1	1	9	73.9			
SPD	18	9920	4	04	03	08	00	05	005	0	0	0	0	0	0	0	0	1	2	6	10	5	1	0	25	71.8	34	204	2.82
SPD	18	9920	1	04	03	08	00	10	005	0	0	0	0	0	0	0	0	1	2	3	3	7	2	0	18	73.3			
SPD	18	9920	2	04	03	08	00	10	005	0	0	0	0	0	0	0	0	0	0	0	3	6	1	2	12	79.3	30	180	2.38
SPD	18	9920	3	04	03	08	00	10	005	0	0	0	0	0	0	0	0	0	0	0	5	7	2	1	15	77.9			
SPD	18	9920	4	04	03	08	00	10	005	0	0	0	0	0	0	0	1	0	1	5	10	4	1	0	22	71.9	37	222	2.99
SPD	18	9920	1	04	03	08	00	15	005	0	0	0	0	0	0	0	0	0	2	2	7	3	0	2	16	74.3			
SPD	18	9920	2	04	03	08	00	15	005	0	0	0	0	0	0	0	0	0	0	0	1	2	2	3	8	83.5	24	144	1.86
SPD	18	9920	3	04	03	08	00	15	005	0	0	0	0	0	0	0	0	0	0	1	7	5	1	0	14	75.1			
SPD	18	9920	4	04	03	08	00	15	005	0	0	0	0	1	0	0	0	2	5	8	7	4	1	0	28	68.5	42	252	3.56
SPD	18	9920	1	04	03	08	00	20	005	0	0	0	0	0	1	0	0	0	5	3	6	4	0	1	20	70.2			
SPD	18	9920	2	04	03	08	00	20	005	0	0	0	0	0	0	0	0	0	0	4	0	3	0	1	8	74.6	28	168	2.35
SPD	18	9920	3	04	03	08	00	20	005	0	0	0	0	0	0	0	0	0	2	3	3	6	1	2	17	75.4			
SPD	18	9920	4	04	03	08	00	20	005	0	0	0	0	0	0	0	0	0	4	4	8	12	0	0	28	73.0	45	270	3.65
SPD	18	9920	1	04	03	08	00	25	005	0	0	0	0	0	0	0	0	0	0	5	5	3	2	1	16	74.8			
SPD	18	9920	2	04	03	08	00	25	005	0	0	0	0	0	0	0	0	0	0	0	2	3	2	0	7	78.0	23	138	1.82
SPD	18	9920	3	04	03	08	00	25	005	0	0	0	0	0	0	0	0	0	0	0	0	2	2	0	4	80.5			
SPD	18	9920	4	04	03	08	00	25	005	0	0	0	0	0	0	0	0	0	1	7	2	5	0	0	15	71.7	19	114	1.55
SPD	18	9920	1	04	03	08	00	30	005	0	0	0	0	0	0	2	0	0	1	2	9	4	0	0	18	70.2			
SPD	18	9920	2	04	03	08	00	30	005	0	0	0	0	0	0	0	0	0	0	0	3	2	1	0	6	76.3	24	144	2.01
SPD	18	9920	3	04	03	08	00	30	005	0	0	0	0	0	0	0	0	0	0	1	2	1	1	2	7	79.6			
SPD	18	9920	4	04	03	08	00	30	005	0	0	0	0	0	0	0	0	0	2	5	8	4	2	1	22	73.6	29	174	2.32
SPD	18	9920	1	04	03	08	00	35	005	0	0	0	0	0	0	0	0	3	3	3	6	3	5	0	23	71.9			
SPD	18	9920	2	04	03	08	00	35	005	0	0	0	0	0	0	0	0	0	0	0	5	1	0	0	6	73.8	29	174	2.41
SPD	18	9920	3	04	03	08	00	35	005	0	0	0	0	0	0	0	0	0	0	0	3	2	4	0	9	78.6			
SPD	18	9920	4	04	03	08	00	35	005	0	0	0	0	0	0	0	0	0	1	7	11	8	0	0	27	72.8	36	216	2.91
SPD	18	9920	1	04	03	08	00	40	005	0	0	0	0	0	0	0	0	0	2	6	7	2	1	0	18	71.3			
SPD	18	9920	2	04	03	08	00	40	005	0	0	0	0	0	0	0	0	0	0	0	1	2	1	0	4	78.0	22	132	1.82
SPD	18	9920	3	04	03	08	00	40	005	0	0	0	0	0	0	0	0	0	0	0	1	4	1	2	8	81.3			
SPD	18	9920	4	04	03	08	00	40	005	0	0	0	0	0	0	0	0	2	2	3	7	6	1	1	22	72.7	30	180	2.40

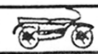
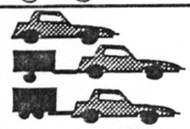
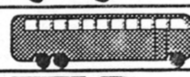
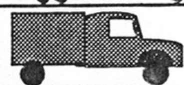
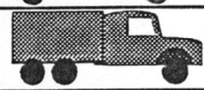
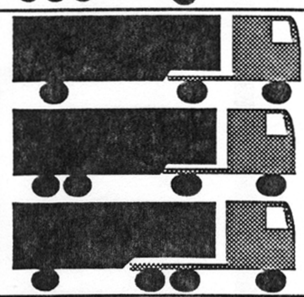
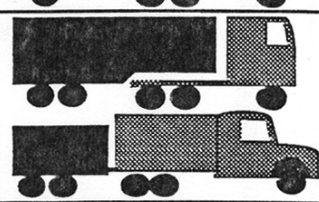
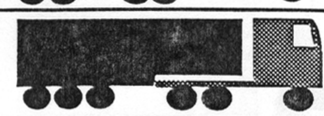
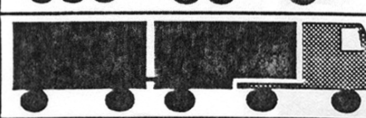
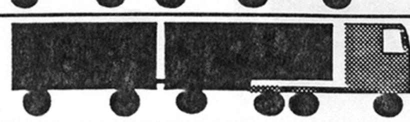
[1] These categories were calculated from the given loop detector data and added to the speed data spreadsheets.

Tag	County	Site	Yr.	Mo.	Day	Hour	Min	Int	Lane	#	Lane	#	Lane	#	Lane	#	Total NB	Total SB	Total Volume
CNT	18	9920	04	03	08	00	05	005	1	15	2	8	3	9	4	25	23	34	57
CNT	18	9920	04	03	08	00	10	005	1	18	2	12	3	15	4	22	30	37	67
CNT	18	9920	04	03	08	00	15	005	1	16	2	8	3	14	4	28	24	42	66
CNT	18	9920	04	03	08	00	20	005	1	20	2	8	3	17	4	28	28	45	73
CNT	18	9920	04	03	08	00	25	005	1	16	2	7	3	4	4	15	23	19	42
CNT	18	9920	04	03	08	00	30	005	1	18	2	6	3	7	4	22	24	29	53
CNT	18	9920	04	03	08	00	35	005	1	23	2	6	3	9	4	27	29	36	65
CNT	18	9920	04	03	08	00	40	005	1	18	2	4	3	8	4	22	22	30	52
CNT	18	9920	04	03	08	00	45	005	1	13	2	4	3	12	4	28	17	40	57
CNT	18	9920	04	03	08	00	50	005	1	8	2	4	3	14	4	29	12	43	55
CNT	18	9920	04	03	08	00	55	005	1	16	2	4	3	8	4	22	20	30	50
CNT	18	9920	04	03	08	01	00	005	1	12	2	3	3	4	4	24	15	28	43
CNT	18	9920	04	03	08	01	05	005	1	19	2	3	3	8	4	19	22	27	49
CNT	18	9920	04	03	08	01	10	005	1	6	2	2	3	8	4	25	8	33	41

Tag	County	Site	Lane	Year	Month	Day	Hour	Min	Int	CL 01	CL 02	CL 03	CL 04	CL 05	CL 06	CL 07	CL 08	CL 09	CL 10	CL 11	CL 12	CL 13	CL 14	CL 15	Total Vol.	Buses[1]	Trucks[1]	HV[1]	%HV[1]	Total % HV[1]
CLS	18	9920	1	04	03	08	00	05	005	0	4	5	0	0	0	0	0	5	0	1	0	0	0	0	15	0	6	6	0.4	
CLS	18	9920	2	04	03	08	00	05	005	0	6	2	0	0	0	0	0	0	0	0	0	0	0	0	8	0	0	0	0	0.26
CLS	18	9920	3	04	03	08	00	05	005	0	6	2	0	0	0	0	0	0	0	0	0	0	0	1	9	0	1	1	0.11	
CLS	18	9920	4	04	03	08	00	05	005	0	9	1	0	0	0	0	0	13	0	0	0	0	0	2	25	0	15	15	0.6	0.47
CLS	18	9920	1	04	03	08	00	10	005	0	9	5	1	0	0	0	0	2	0	0	0	0	0	1	18	1	3	4	0.22	
CLS	18	9920	2	04	03	08	00	10	005	0	9	1	0	0	0	0	0	2	0	0	0	0	0	0	12	0	2	2	0.17	0.2
CLS	18	9920	3	04	03	08	00	10	005	0	11	0	0	0	0	0	0	3	1	0	0	0	0	0	15	0	4	4	0.27	
CLS	18	9920	4	04	03	08	00	10	005	0	5	4	0	0	0	0	1	10	0	1	0	0	0	1	22	0	13	13	0.59	0.46
CLS	18	9920	1	04	03	08	00	15	005	0	8	3	0	1	1	0	0	3	0	0	0	0	0	0	16	0	3	3	0.19	
CLS	18	9920	2	04	03	08	00	15	005	0	7	1	0	0	0	0	0	0	0	0	0	0	0	0	8	0	0	0	0	0.13
CLS	18	9920	3	04	03	08	00	15	005	0	13	0	0	0	0	0	0	1	0	0	0	0	0	0	14	0	1	1	0.07	
CLS	18	9920	4	04	03	08	00	15	005	0	11	7	0	0	0	0	0	7	0	1	0	0	0	2	28	0	10	10	0.36	0.26
CLS	18	9920	1	04	03	08	00	20	005	0	9	3	0	1	1	0	0	6	0	0	0	0	0	0	20	0	6	6	0.3	
CLS	18	9920	2	04	03	08	00	20	005	0	6	0	0	0	0	0	0	1	0	1	0	0	0	0	8	0	2	2	0.25	0.29
CLS	18	9920	3	04	03	08	00	20	005	0	8	3	0	1	0	0	0	5	0	0	0	0	0	0	17	0	5	5	0.29	
CLS	18	9920	4	04	03	08	00	20	005	0	7	3	0	1	0	0	1	16	0	0	0	0	0	0	28	0	17	17	0.61	0.49
CLS	18	9920	1	04	03	08	00	25	005	0	8	0	0	1	0	0	0	6	0	0	0	0	0	1	16	0	7	7	0.44	
CLS	18	9920	2	04	03	08	00	25	005	0	5	2	0	0	0	0	0	0	0	0	0	0	0	0	7	0	0	0	0	0.30
CLS	18	9920	3	04	03	08	00	25	005	0	4	0	0	0	0	0	0	0	0	0	0	0	0	0	4	0	0	0	0	
CLS	18	9920	4	04	03	08	00	25	005	0	5	2	0	0	0	0	1	7	0	0	0	0	0	0	15	0	8	8	0.53	0.42
CLS	18	9920	1	04	03	08	00	30	005	0	9	2	0	0	1	0	0	5	0	0	0	0	0	1	18	0	6	6	0.33	
CLS	18	9920	2	04	03	08	00	30	005	0	5	1	0	0	0	0	0	0	0	0	0	0	0	0	6	0	0	0	0	0.25
CLS	18	9920	3	04	03	08	00	30	005	0	2	4	0	1	0	0	0	0	0	0	0	0	0	0	7	0	0	0	0	
CLS	18	9920	4	04	03	08	00	30	005	0	9	2	0	1	0	0	0	10	0	0	0	0	0	0	22	0	10	10	0.45	0.34
CLS	18	9920	1	04	03	08	00	35	005	0	12	1	0	1	0	0	0	9	0	0	0	0	0	0	23	0	9	9	0.39	
CLS	18	9920	2	04	03	08	00	35	005	0	6	0	0	0	0	0	0	0	0	0	0	0	0	0	6	0	0	0	0	0.31
CLS	18	9920	3	04	03	08	00	35	005	0	5	3	0	0	0	0	0	1	0	0	0	0	0	0	9	0	1	1	0.11	
CLS	18	9920	4	04	03	08	00	35	005	0	9	2	0	1	0	0	1	13	0	0	0	0	0	1	27	0	15	15	0.56	0.44

[1]These categories were calculated from the given loop detector data and added to the class data spreadsheets.

CLASSIFICATION SCHEME "F"

CLASS. GROUP		DESCRIPTION	NO. OF AXLES
1		MOTORCYCLES	2
2		ALL CARS CARS W/ 1-AXLE TRLR CARS W/2-AXLE TRLR	2 3 4
3		PICK-UPS & VANS 1 & 2 AXLE TRLRS	2, 3, & 4
4		BUSES	2 & 3
5		2-AXLE, SINGLE UNIT	2
6		3-AXLE, SINGLE UNIT	3
7		4-AXLE, SINGLE UNIT	4
8		2-AXLE TRACTOR, 1-AXLE TRLR(2S1) 2-AXLE TRACTOR, 2-AXLE TRLR(2S2) 3-AXLE TRACTOR, 1-AXLE TRLR(3S1)	3 4 4
9		3-AXLE TRACTOR, 2-AXLE TRLR(3S2) 3-AXLE TRUCK, W/2-AXLE TRLR	5 5
10		TRACTOR W/ SINGLE TRLR	6 & 7
11		5-AXLE MULTI-TRLR	5
12		6-AXLE MULTI-TRLR	6
13	ANY 7 OR MORE AXLE		7 or more

System Usage Data 1/9/90

Part II

Highway Capacity Manual Guidance on Freeway Capacity in Merge/Diverge Areas (Task 4a)

Table of Contents

Introduction

The Highway Capacity Manual (HCM) contains procedures for analyzing the operational conditions of different components of a freeway facility, including basic freeway segments, ramp junctions, and weaving areas. The Florida Department of Transportation (FDOT) contracted with Dr. Elena Prassas (Polytechnic University) to develop a freeway facility analysis program based upon the procedures of the 2000 edition of the HCM (hereinafter referred to as HCM2000). This program is called FREEPLAN and can be obtained from http://www.dot.state.fl.us/planning/systems/sm/los/los_sw2.htm.

After the development and release of this software program, it was later discovered that the freeway segment capacity values being used for ramp merge and diverge areas differed from those being used in the Highway Capacity Software (HCS), another software program that implements the analytical procedures of the HCM. HCS is developed by McTrans at the University of Florida. More information on this program can be found at http://mctrans.ce.ufl.edu/hcs/.

Consequently, this raised the question as to which program was using the correct capacity values. Thus, the objective of this task was to review the applicable HCM2000 chapters and determine if the HCM offered clear guidance on which capacity values should be used. The following sections will give an overview of the applicable HCM methodologies and guidance, the current interpretation of this language by the FDOT in their implementation of FREEPLAN, the current interpretation of this language by McTrans in their implementation of HCS, and conclusions and recommendations.

Overview of HCM Methodologies

Basic Freeway Segments

Chapter 23 covers the analysis procedure for basic freeway segments. The freeway analysis methodology of the HCM2000 uses a capacity value of 2400 pc/h/ln for basic freeway segments with free-flow speeds of 70-75 mi/h and lower capacity values for free-flow speeds under 70 mi/h. These values are shown in Exhibit 23-2 of the HCM2000. Capacity is defined in the HCM [Chapter 5] as the maximum sustainable flow rate at which vehicles or persons reasonably can be expected to traverse a point or a uniform segment of a lane or a roadway during a specified time period under given roadway, geometric, traffic, environmental, and control conditions.

Ramp Junctions

Merge Areas

Chapter 25 covers the analysis procedure for ramp junctions. The analysis of merge areas takes into consideration the traffic flow in the outer two lanes (V_{12}) of the freeway segment. The proportion of traffic entering the outer two lanes (P_{FM}) is calculated according to the equations of Exhibit 25-5. Thus, the total volume in the ramp merge influence area is calculated according to the following equation:

$$V_{R12} = V_F \times P_{FM} + V_R$$

Where:

V_{R12} = Total flow entering the ramp merge influence area,

V_F = Mainline freeway volume,

$P_{FM)}$ = Proportion of freeway mainline traffic remaining in lanes 1 of 2 immediately upstream of merge (note that $V_F \times P_{FM} = V_{12}$), and

V_R = On-ramp volume.

The Level of Service (LOS) is based on the density in the merge influence area, as given by Equation 25-5. However, before calculating the density of the merge area, capacity values must be checked. If certain capacity values are exceeded, then the LOS is F and no further calculations are necessary.

The capacity of a merge area is determined by the capacity of the downstream freeway segment. Thus, the total flow arriving on the upstream freeway segment (V_F) and the on-ramp (V_R) must be less than or equal to the capacity of the downstream freeway segment. The HCM also states that there is no evidence that the turbulence in the merge area causes the downstream freeway capacity to be less than that of a basic freeway segment. Therefore the capacity values of the basic freeway segment are used in the analysis of merge areas.

The HCM addresses the following two cases in the capacity analysis of merge areas:

Case 1: The total departing flow exceeds the capacity of the downstream freeway segment. In this case, queues are expected to form upstream of the merge area and therefore LOS F is assigned without considering the flow or density in the ramp influence area.

Case 2: The total flow entering the ramp influence area ($V_{12} + V_R$) exceeds the maximum desirable level but the total freeway flow does not exceed the downstream freeway segment capacity. In this case, locally high densities may occur but no queuing is expected on the freeway. The actual lane distribution of entering vehicles is likely to consist of more vehicles in the outer lanes than is indicated by the models herein. Operations are expected to remain stable and LOS F is not expected to occur. LOS is determined by estimating the density in the ramp influence area.

The capacity values for merge areas are given in Exhibit 25-7 of the HCM. It is included below for convenience.

EXHIBIT 25-7. CAPACITY VALUES FOR MERGE AREAS

Freeway Free-Flow Speed (mi/h)	Maximum Downstream Freeway Flow, v (pc/h)				Max Desirable Flow Entering Influence Area, v_{R12} (pc/h)
	Number of Lanes in One Direction				
	2	3	4	> 4	
≥ 70	4800	7200	9600	2400/ln	4600
65	4700	7050	9400	2350/ln	4600
60	4600	6900	9200	2300/ln	4600
55	4500	6750	9000	2250/ln	4600

In this table, it can be seen that the per lane downstream freeway capacities are consistent with the capacity values given in Chapter 23 (Basic Freeway Segments). However, the last column of the table implies that the maximum flow in the two lanes of the merge influence area (two outside lanes) should not exceed 4600 pc/h, which is 100 pc/h/ln less than the capacity of a basic freeway segment lane (2300 vs. 2400).

Implicit in this table is that the downstream capacity check applies to the segment as a whole (i.e., a single value for the overall segment) and not on an individual lane basis. Thus, it can be inferred that LOS F conditions will not occur as long as the total downstream flow does not exceed the applicable per lane value of capacity times the number of lanes, regardless of the actual flow in any individual lane.

Although the total departing flow may be less than the downstream freeway capacity and the total flow entering the ramp influence area may be less than its corresponding desired maximum flow, the analysis procedure may yield flow rates in the inner lanes of the freeway (assuming more than 2 lanes in the analysis direction) that exceed the per lane capacity (e.g., 2400 pc/h/ln). This kind of a situation might arise when the total flow approaching the merge area is nearing the basic freeway capacity. The following examples illustrate this issue. These examples were analyzed using the HCS and the summary of the results is provided below.

Example 1: The total departing flow is less than the capacity of the downstream freeway segment.

Freeway Data	Ramp Data	Assumptions
FFS = 70 mi/h Volume = 9000 veh/h Number of lanes in each direction = 4	Type of ramp: On-Ramp FFS = 45 mi/h Volume = 200 veh/h Number of lanes = 1 Accel lane length = 500 ft	PHF = 1.0 Percent heavy vehicles = 0 Therefore, f_{HV} = 1.0 Level terrain

The first step is the determination of the proportion of freeway flow remaining in lanes 1 and 2 immediately upstream of the merge area (P_{FM}). This is determined from Equation 4 of Exhibit 25-5.

P_{FM} = 0.317

The total flow in the outer two lanes of the freeway (at a point immediately upstream of the merge) is

$$
\begin{aligned}
V_{12} &= V_F \times P_{FM} \\
&= 9000 \times 0.317 \\
&= 2850 \text{ pc/h}
\end{aligned}
$$

Therefore, the total flow in the inner two lanes is 6150 (9000 – 2850) pc/h. The per lane flow is 3075 (6150/2) pc/h/ln.

For a FFS of 70 mi/h, the maximum freeway flow downstream of a merge junction is 2400 pc/h/ln (Exhibit 25-7).

The maximum per lane flow rate obtained from calculations is 3075 pc/h/ln. Therefore, the flow in each of the inner lanes is exceeding capacity by 675 (3075 – 2400) pc/h/ln (approximately 28%).

The total flow at a point immediately downstream of the merge is 9200 pc/h (9000 + 200) and V_{R12} is 3050 (2850 + 200), which are below the 9600 and 4600 capacity value checks from Exhibit 25-7. Thus, the LOS is not F, by definition, and can be calculated by Equation 25-5. This yields a density of 26 pc/mi/ln and corresponding LOS of C.

Figure 1 shows the final volume distributions from the HCS calculations for this example.

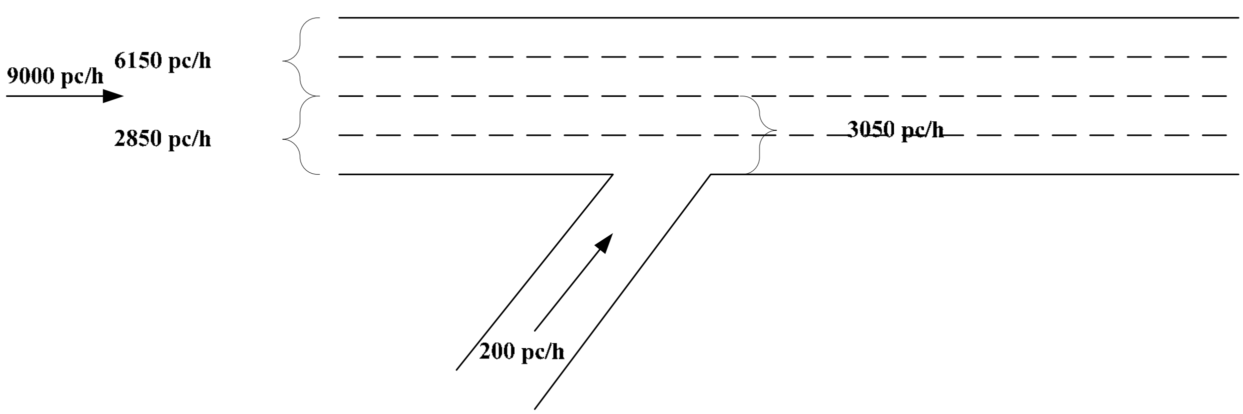


Figure 1: Final volume distributions for Example 1

Example 2: The total departing flow is equal to the capacity of the downstream freeway segment.

Freeway Data	Ramp Data:	Assumptions
FFS = 70 mi/h Volume = 9400 pc/h Number of lanes in each direction = 4	Type of ramp: On-Ramp FFS = 45 mi/h Volume = 200 pc/h Number of lanes = 1 Accel lane length = 500 ft	PHF = 1.0 Percent of heavy vehicles = 0 Therefore, f_{HV} = 1.0 Level terrain

From Equation 4 of Exhibit 25-5, $P_{FM} = 0.317$

The total flow in the outer two lanes of the freeway (at a point immediately upstream of the merge) is

$$V_{12} = V_F \times P_{FM}$$
$$= 9400 \times 0.317$$
$$= 2980 \text{ pc/h}$$

Therefore, the total flow in the inner two lanes is 6420 (9400 – 2980) pc/h. The per lane flow is 3210 (6420/2) pc/h/ln.

For a FFS of 70 mi/h, the maximum freeway flow downstream of a merge junction is 2400 pc/h/ln (Exhibit 25-7).

The maximum per lane flow rate obtained from calculations is 3210 pc/h/ln. Therefore, the flow in each of the inner lanes is exceeding capacity by 810 (3210 – 2400) pc/h/ln (approximately 33%).

The total flow at a point immediately downstream of the merge is 9600 pc/h (9400 + 200) and V_{R12} is 3180 (2980 + 200), which are below or equal to the 9600 and 4600 capacity value checks from Exhibit 25-7. Thus, the LOS is not F, by definition, and can be calculated by Equation 25-5. This yields a density of 27 pc/mi/ln and corresponding LOS of C.

Figure 2 shows the final volume distributions from the HCS calculations for this example.

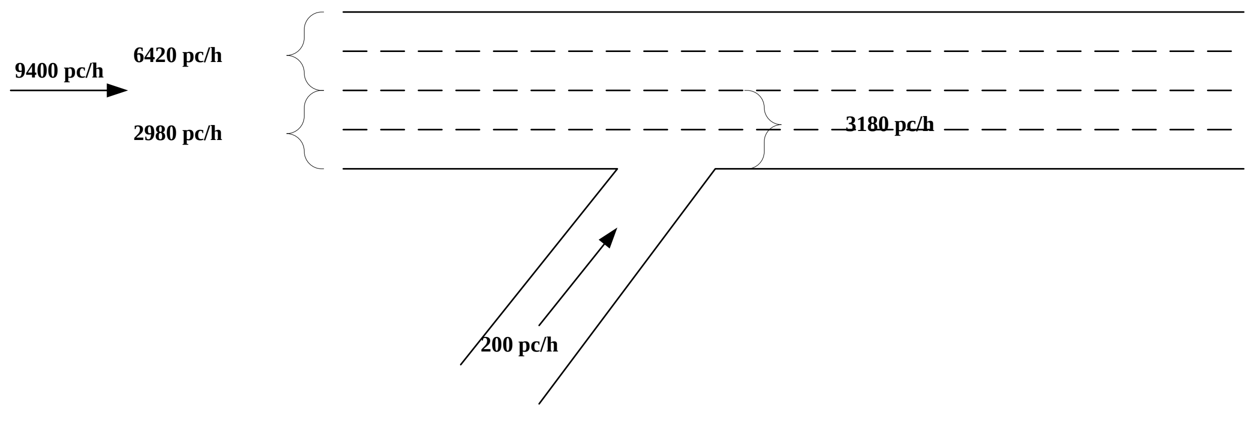


Figure 2: Final volume distributions for Example 2

Example 3: The total departing flow is equal to the capacity of the downstream freeway segment.

Freeway Data	Ramp Data	Assumptions
FFS = 70 mi/h Volume = 9000 veh/h Number of lanes in each direction = 4	Type of ramp: On-Ramp FFS = 45 mi/h Volume = 600 veh/h Number of lanes = 1 Accel lane length = 500 ft	PHF = 1.0 Percent heavy vehicles = 0 Therefore, f_{HV} = 1.0 Level terrain

From Equation 4 of Exhibit 25-5, P_{FM} = 0.267

The total flow in the outer two lanes of the freeway (at a point immediately upstream of the merge) is

$$
\begin{aligned}
V_{12} &= V_F \times P_{FM} \\
&= 9000 \times 0.267 \\
&= 2400 \text{ pc/h}
\end{aligned}
$$

Therefore, the total flow in the inner two lanes is 6600 (9000 – 2400) pc/h. The per lane flow is 3300 (6600/2) pc/h/ln.

For a FFS of 70 mi/h, the maximum freeway flow downstream of a merge junction is 2400 pc/h/ln (Exhibit 25-7).

The maximum per lane flow rate obtained from calculations is 3300 pc/h/ln. Therefore, the flow in each of the inner lanes is exceeding capacity by 900 (3300 – 2400) pc/h/ln (approximately 38%).

The total flow at a point immediately downstream of the merge is 9600 pc/h (9000 + 600) and V_{R12} is 3000 (2400 + 600), which are below or equal to the 9600 and 4600 capacity value checks from Exhibit 25-7. Thus, the LOS is not F, by definition, and can be calculated by Equation 25-5. This yields a density of 25.5 pc/mi/ln and corresponding LOS of C.

Figure 3 shows the final volume distributions from the HCS calculations for this example.

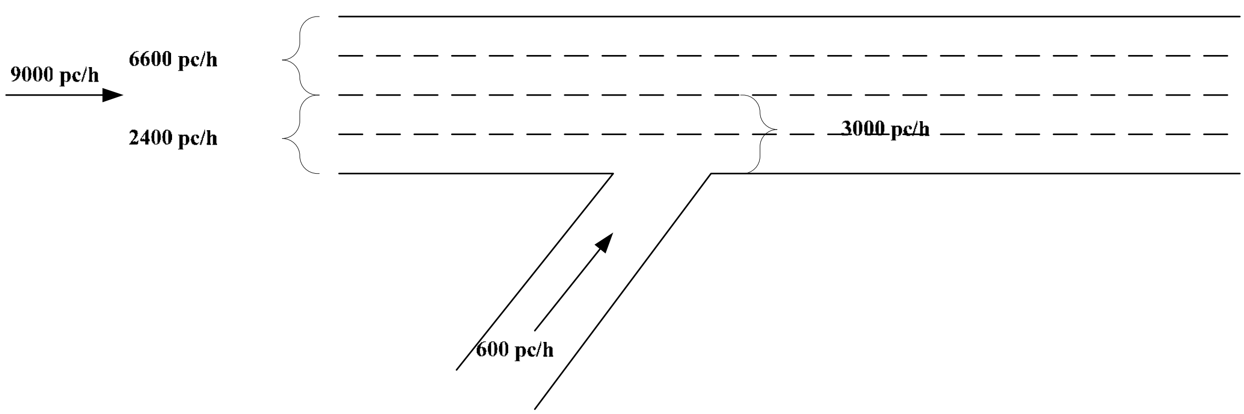


Figure 3: Final volume distributions for Example 3

Diverge Areas

The analysis of diverge areas follows a similar process as that for merge areas. The traffic flow in the outer two lanes (V_{12}) of the freeway segment must be determined. The proportion of through traffic remaining in the outer two lanes (P_{FD}) is calculated according to the equations of Exhibit 25-12. Thus, the total flow rate immediately upstream of the diverge is calculated according to the following equation:

$$V_{12} = V_R + (V_F - V_R) \times P_{FD}$$

Where:

V_{12} = Total flow entering the ramp diverge influence area,
V_F = Mainline freeway volume,
$P_{FD)}$ = Proportion of freeway through traffic remaining in lanes 1 of 2 immediately upstream of diverge, and
V_R = Off-ramp volume.

The Level of Service (LOS) is based on the density in the diverge influence area, as given by Equation 25-10. However, before calculating the density of the diverge area, capacity values must be checked. If certain capacity values are exceeded, then the LOS is F and no further calculations are necessary.

Three capacity values must be checked for a diverge analysis: 1) the total flow that can depart from the diverge area; 2) the capacity of the departing freeway leg or ramp or both; and 3) the maximum flow that can enter on lanes 1 and 2 prior to the deceleration lane. Another major difference to be considered in diverge analysis is that the flow entering the diverge influence area (V_{12}) includes the off-ramp flow (V_R). The capacity values for diverge areas are given in Exhibit 25-14 of the HCM. It is included below for convenience.

EXHIBIT 25-14. CAPACITY VALUES FOR DIVERGE AREAS

Freeway Free-Flow Speed (mi/h)	Maximum Upstream, v_{FI}, or Downstream Freeway Flow, v (pc/h)				Max Flow Entering Influence Area, v_{12} (pc/h)
	Number of Lanes in One Direction				
	2	3	4	> 4	
≥ 70	4800	7200	9600	2400/ln	4400
65	4700	7050	9400	2350/ln	4400
60	4600	6900	9200	2300/ln	4400
55	4500	6750	9000	2250/ln	4400

In this table, it can be seen that the per lane upstream and downstream freeway capacities are consistent with the capacity values given in Chapter 23 (Basic Freeway Segments). However, the last column of the table implies that the maximum flow in the two lanes of the diverge influence area (two outside lanes) should not exceed 4400 pc/h, which is 200 pc/h/ln less than the capacity of a basic freeway segment lane (2200 vs. 2400).

Again, it is implicit in this table that the freeway segment capacity checks apply to the freeway segment as a whole and not on an individual lane basis. As for the merge area analysis, it can be inferred that LOS F conditions will not occur as long as the total upstream and downstream flow does not exceed the applicable per lane value of capacity times the number of lanes, regardless of the actual flow in any individual lane.

Thus, even if the methodology predicts flow rates for the inside lanes of the freeway that exceed the per lane capacity, the LOS will not be F as long as the total upstream and downstream flow rates do not exceed the per lane capacity times the number of lanes. In this case, the LOS is determined from the density of the diverge influence area. The following example illustrates this issue.

Example 4: The total flow entering the diverge area is equal to the upstream freeway capacity

Freeway Data	Ramp Data	Assumptions
FFS = 70 mi/h Volume = 9600 veh/h Number of lanes in each direction = 4	Type of ramp: Off-Ramp FFS = 45 mi/h Volume = 200 veh/h Number of lanes = 1 Decel lane length = 500 ft	PHF = 1.0 Percent heavy vehicles = 0 Therefore, f_{HV} = 1.0 Level terrain

The first step is the determination of the proportion of freeway flow remaining in lanes 1 and 2 immediately upstream of the diverge area (P_{FD}). This is determined from Equation 8 of Exhibit 25-12.

$$P_{FD} = 0.436$$

The total flow in the outer two lanes of the freeway (at a point immediately upstream of the diverge) is

$$\begin{aligned} V_{12} &= V_R + (V_F - V_R) \times P_{FD} \\ &= 200 + (9600 - 200) \times 0.436 \\ &= 4298 \text{ pc/h} \end{aligned}$$

Therefore, the total flow in the inner two lanes is 5302 (9600 – 4298) pc/h. The per lane flow is 2651 (5302/2) pc/h/ln.

For a FFS of 70 mi/h, the maximum freeway flow upstream/downstream of diverge junction is 2400 pc/h/ln (Exhibit 25-14).

The maximum per lane flow rate obtained from calculations is 2651 pc/h/ln. Therefore, the flow in each of the inner lanes is exceeding capacity by 251 (2651 – 2400) pc/h/ln.

The total flow at a point immediately upstream of the diverge area is 9600 pc/h, the total flow immediately downstream of the diverge is 9400 pc/h (9600 - 200), and V_{12} is 4298, which are below or equal to the 9600 and 4400 capacity value checks from Exhibit 25-14. Thus, the LOS is not F, by definition, and can be calculated by Equation 25-10. This yields a density of 36.7 pc/mi/ln and corresponding LOS of E.

Figure 4 shows the final volume distributions from the HCS calculations for this example.

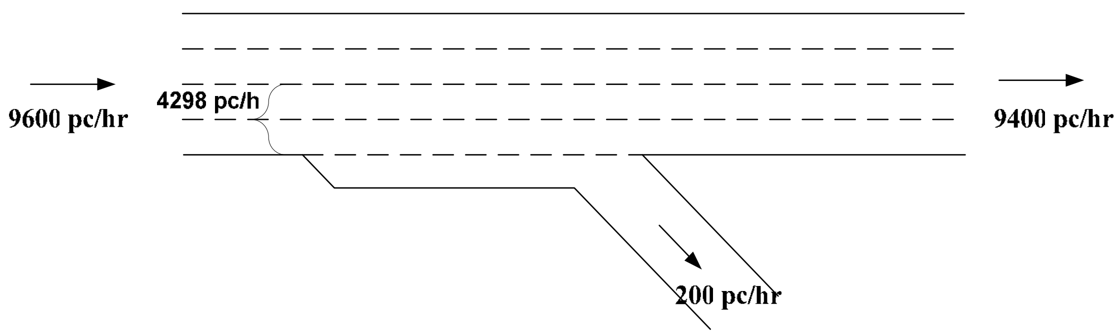


Figure 4: Final volume distributions for Example 4

FDOT Implementation of the HCM Chapter 25

The previous sections outlined the HCM methodology for the analysis of ramp junctions, and demonstrated, by example, how the Highway Capacity Software implements the methodology. This section will describe how the HCM Chapter 25 methodology is being implemented by the FREEPLAN software developed for the FDOT. This implementation will again be demonstrated by example.

Example 3 was reanalyzed using FREEPLAN and the calculations and the results are shown below.

Calculations

From Equation 4 of Exhibit 25-5, P_{FM} = 0.267

The total flow in the outer two lanes of the freeway (at a point immediately upstream of the merge) is

$$\begin{aligned} V_{12} &= V_F \times P_{FM} \\ &= 9000 \times 0.267 \\ &= 2400 \text{ pc/h} \end{aligned}$$

Therefore, the total flow in the inner two lanes is 6600 (9000 – 2400) pc/h. The per lane flow is 3300 (6600/2) pc/h/ln.

The total volume entering the merge influence area is 9600 pc/h.

Based on Exhibit 25-7, FREEPLAN calculates the maximum capacity in the merge influence area is as follows:

2400 × (Total Lanes – 2) + 4600
2400 × (4-2) + 4600 = 9400 pc/h

Thus, the FDOT is interpreting the maximum desirable entering volume (V_{R12}) of 4600 as a capacity value for the two outside lanes. This is then added to the normal capacity value of 2400 pc/h/ln for each of the two additional lanes.

As a result, FREEPLAN identifies the merge influence area as being over-capacity (9600 > 9400). Nonetheless, it still proceeds with a density calculation for the merge influence area, but also performs a queue calculation for oversaturated conditions. These analysis output values can be seen in Figures 5 and 6. The merge influence area capacity of 9400 is circled. For the downstream basic freeway segment, FREEPLAN still uses the full input volume of 9600 (also circled), despite this value exceeding the capacity (9400) identified for the upstream merge influence area. Thus, the calculation results are essentially identical to those of the HCS, with the exception that FREEPLAN uses a lower capacity for the ramp influence area and then performs queuing calculations when this value is exceeded.

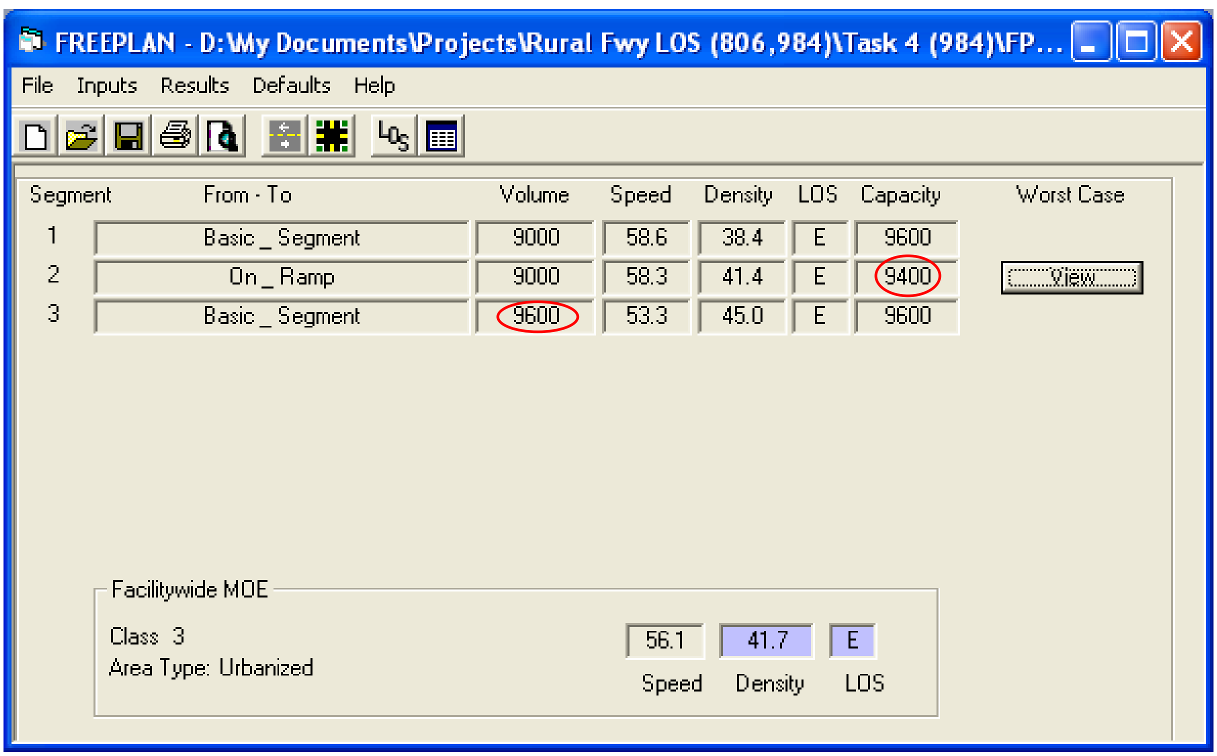


Figure 5: Output from FREEPLAN analysis

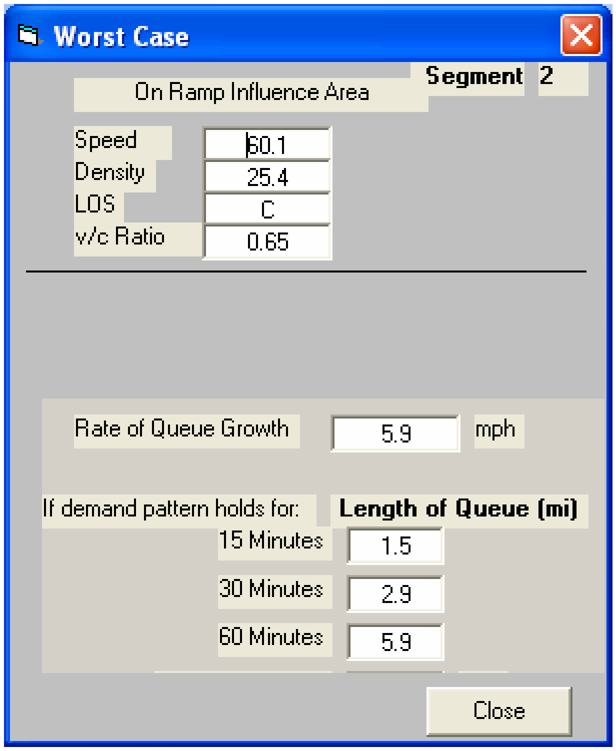


Figure 6: FREEPLAN merge influence area results for Example 3

Summary and Recommendations

The above discussion and examples highlights two important questions for the methodology of HCM Chapter 25:

1. What is the proper interpretation for the capacity of a merge/diverge influence area?
2. Is it reasonable for the methodology to output per lane flow rates that are greatly in excess of the standard per lane capacity values?

The first question came to light due to the interpretation/implementation differences between the HCS and FREEPLAN implementations of Chapter 25. The second question came to light from computational examples used to investigate the implementation differences between the HCS and FREEPLAN.

At the mid-year meeting in Atlanta (July 2003) of the HCQS committee, this question was raised to Dr. Roger Roess (Polytechnic University), a member of the committee and principal investigator of NCHRP project 3-37 (*Capacity of Ramp-Freeway Junctions*), and he indicated that it was not the intent for the total capacity of the merge influence area to be less than the basic freeway segment capacity (i.e., basic segment per lane capacity times number of lanes). Thus, the implication is that the inner lanes would carry more than 2400 pc/h/lane (for FFS ≥ 70 mi/h) to compensate for reduced capacity in outside lanes, with all lanes averaging out to 2400 pc/h/ln. To a certain extent, this is a reasonable interpretation, as it is generally assumed that the 2400 pc/h/ln capacity value represents an average across all lanes of freeway segment. He did acknowledge that 4-lane freeways (2 lanes in each direction), however, would operate at the reduced capacity of 2300/2200 pc/h/ln in merge/diverge influence areas.

The current HCS implementation is consistent with Dr. Roess' interpretation of Chapter 25, but a natural question is whether it is reasonable to have results from an HCM analysis of a ramp junction output individual lane flow rates that greatly exceed the typical per lane capacity of a basic freeway segment. For Example 3, the average flow rates of 3300 pc/h/ln for the two inside lanes, according to the analysis results, is highly unrealistic, as well as the LOS C ranking for the merge influence area.

It is more logical that as a freeway segment approaches capacity, the traffic volume distribution across lanes will become more balanced, but with somewhat higher percentages still realized in the inside lanes. It is recommended that the HCQS committee revisit the language in this chapter and consider revising and clarifying it with regard to the overall capacity in a ramp influence area and under what conditions (such as near capacity) the volume proportions predicted for lanes 1 and 2 (i.e., P_{FM} and P_{FD}) are likely to not be reliable.

As was paraphrased earlier in this document, the following language is from the top of page 25-8 of the HCM[1].

"The second condition occurs when the total flow entering the ramp influence area (V_{R12}) exceeds its maximum desirable level but the total freeway flow (V) does not exceed the capacity of the downstream freeway segment. In this case, locally high densities are expected, but no queuing is expected on the freeway. *The actual lane distribution of entering vehicles is likely to consist of more vehicles in the outer lanes than is indicated by the models herein.* Overall, operation will remain stable, and LOS F is not expected to occur.

When the total downstream flow exceeds the basic freeway capacity of the downstream segment, LOS F exists. In such cases, no further computations are needed, and LOS F is assigned. For all other cases, including cases in which V_{R12} exceeds its stated limit, LOS is determined by estimating the density in the ramp influence area."

The italicized sentence partially addresses this issue, but is not specific enough to the issue of when V_{R12} does not exceed its maximum desirable level, yet the methodology may predict extremely unrealistic volumes for the inner freeway lanes. Furthermore, this language raises another potential ambiguity as it implies that LOS F conditions will not occur when V_{R12} does exceed its maximum desirable level, as long as the total downstream freeway segment capacity is not exceeded. However, the worksheet provided in the HCM essentially indicates that if V_{R12} is greater than 4600, LOS F is assigned to the conditions, regardless of whether the downstream capacity is exceeded or not. This is also the implementation in the HCS, as demonstrated by the following screen shots. These screen shots are an excerpt from an on-ramp analysis in which V_{R12} is greater than 4600 but the downstream flow rate is still below capacity. In this case, HCS has assigned a LOS of F even though the calculated density value would put it at LOS D.

Again, it is recommended that these issues be brought forward for discussion at a future HCQS committee meeting so that the current ambiguities in the chapter can be clarified. Subsequently, the appropriate revisions can be made in FREEPLAN and/or HCS to ensure that they are consistent with each other as well as the HCM.

[1] Similar language is also present under the diverge analysis section of the chapter (lower part of page 25-14)

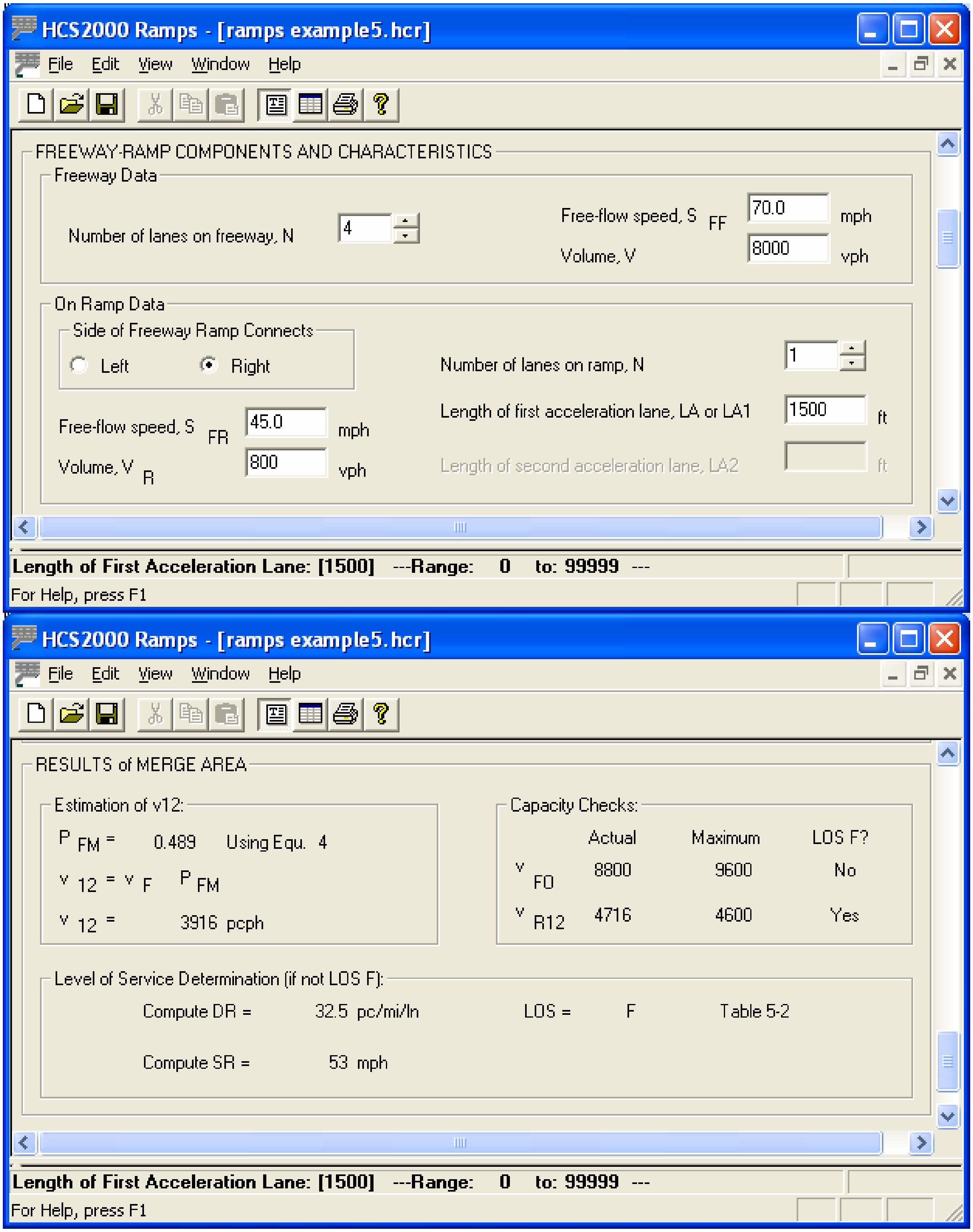


Figure 7. HCS screen shots for V_{R12} example

Part II

Freeway Capacity Measurements from Inductance Loop Detector Data
(Task 4b)

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INTRODUCTION

The capacity of a facility is defined by the Highway Capacity Manual (HCM) [1] as the maximum hourly rate at which persons or vehicles reasonably can be expected to traverse a point or a uniform section of a lane or roadway during a given time period under prevailing roadway, traffic and control conditions.[1]

Capacity analysis of freeways is based on segments with uniform traffic and roadway conditions. If any of the prevailing conditions change significantly, the capacity of the segment and its operating conditions change as well. Therefore, each uniform segment should be analyzed separately. The determination of capacity involves the observation of highways of various types operating under high-volume conditions. The direct observation of capacity is difficult to achieve for several reasons. The recording of a high, or even a maximum, flow rate for a given facility does not insure that a higher flow could not be accommodated at another time.[2][1]

The freeway analysis methodology of the 2000 edition of the HCM (hereinafter referred to as HCM2000) uses a capacity value of 2400 pc/hr/ln for basic freeway segments with free-flow speeds of 70-75 mi/hr and lower capacity values for free-flow speeds under 70 mi/hr. Exhibit 8-19 of HCM2000 contains observations of values higher than this standard, but these are the maximums reported on a given freeway and are not expected to be achieved on most other freeway segments[2]. [1]

Problem Statement

It is generally accepted that the friction effects of merging and diverging create a reduction in capacity for vehicles on the freeway mainline in the vicinity of entrance and exit ramps. Past research has not been able to establish a satisfactory model for predicting the extent of this reduction, and no such reduction is therefore recognized by the HCM. This might be overcome by better understanding regional and site-specific effects.

It can be argued that all capacity measurements, even if actually collected within a basic segment, reflect the metering effects of an upstream interchange area, and thus do not represent the capacity of a “true” basic freeway segment[3]. If this statement is considered to be true, the numbers that are in the HCM, supposedly for basic segments, actually reflect the effects of interchanges. If there is a reduction in capacity for the lanes in the ramp influence area (freeway segments influenced by ramp merge and diverge movements), then the current implication of the HCM is that the remaining lanes outside of the influence area will absorb the extra traffic and potentially operate at a higher capacity, thus providing the same overall capacity of a segment not under the influence of ramp traffic (see Part II, Task 4a for more on this issue).

Another thing that needs to be distinguished is whether the capacity of a lane in the vicinity of an on- and/or off-ramp is actually lower, or just that the lane throughput of the outside lane is lower than the inner lanes due to vehicles moving out of the outer lane and into the inner lanes to avoid the merging and/or diverging friction. The latter does not meet the technical definition of capacity, but could result in the same difference for practical purposes.

---

[1] HCM2000 *Chapter 2 – Capacity and Level of Service Concepts, page 2-2*

[2] HCM2000 *Chapter 8 – Traffic Characteristics*

[3] Typically, capacity-level flows can only be found in urban areas, which have frequent interchange spacing.

Research Objective

The objective of this project was to evaluate maximum traffic flow rate measurements for a variety of freeway segments within and across different geographic areas. The intent of this study was to perform an exploratory investigation of capacity differences between freeway segments with varying interchange spacing.

OVERVIEW OF THE FREEWAY CAPACITY CONCEPTS AND ANALYSIS METHODOLOGIES IN THE HCM2000

This section provides an overview of the current concepts and analysis methodologies contained in the HCM2000 as they pertain to freeway capacity. The HCM2000 was the focus of this study since it is the most commonly accepted professional reference for traffic engineering analyses. The HCM2000 contains several chapters that deal with freeway analysis methodologies, concepts, segments, and facilities, which incorporate the basic segment, weaving, and ramp analysis methodologies.

A *freeway* may be defined as a divided highway with full control of access and two or more lanes for the exclusive use of traffic in each direction. Operating conditions on a freeway primarily result from interactions among vehicles and drivers in the traffic stream and between vehicles and their drivers and the geometric characteristics of the freeway. Operations can also be affected by environmental conditions, such as weather or lighting conditions, by pavement conditions, and by the occurrence of traffic incidents.[1][1]

A freeway facility consists of three component parts, basic freeway segments, weaving areas and ramp junctions. A *basic freeway segment* is a segment of the freeway that is outside of the influence area of ramps or weaving areas. A *weaving area* is a segment of the freeway where two or more vehicle flows must cross each other's path along a length of the freeway. They are usually formed when merge areas are followed closely by diverge areas. They are also formed when an on-ramp is followed by an off-ramp and the two are connected by an auxiliary lane. A *ramp junction* is a point at which on- and off-ramps join the freeway. The junction formed at this point is an area of turbulence because of concentrations of merging or diverging vehicles. A ramp is a length of roadway providing an exclusive connection between two highway facilities. On freeways, all entering and exiting maneuvers take place on ramps that are designed to facilitate smooth merging of on-ramp vehicles into the freeway traffic stream and smooth diverging of off-ramp vehicles from the freeway traffic stream onto the ramp. A sketch showing a basic segment of a freeway is shown in Figure 1.

[1] HCM2000 *Chapter 13 – Freeway Concepts*

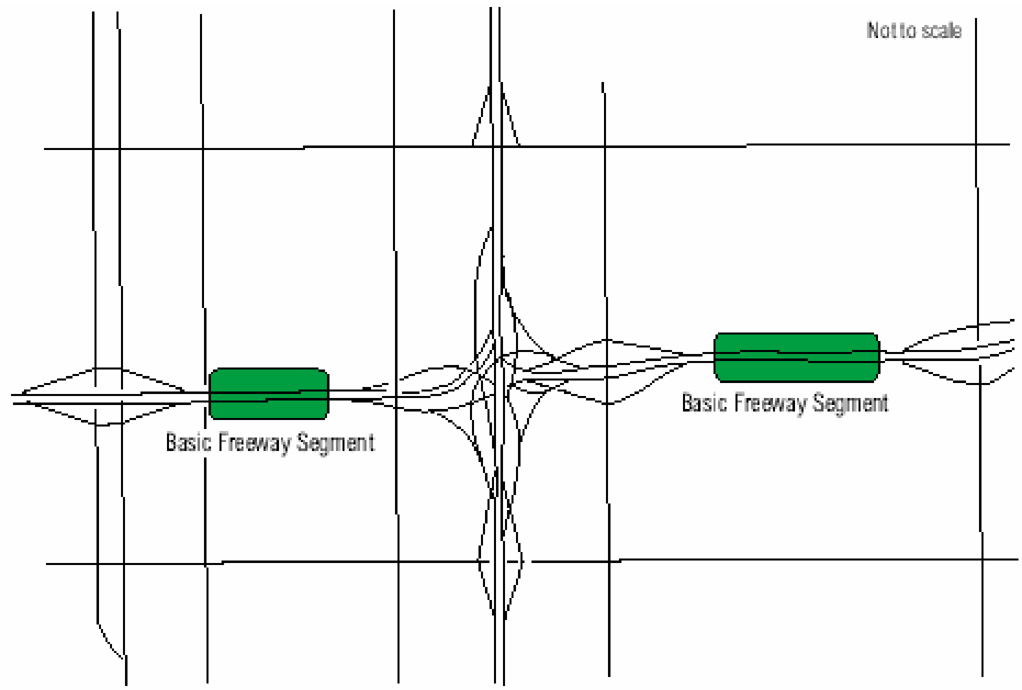


Figure 1: An illustration of a basic segment between interchanges[1]

The base conditions under which the full capacity of a basic freeway segment is achieved include good weather, good visibility, and no incidents or accidents. If any of these conditions fail to exist, the speed and capacity of the freeway segment is typically reduced. Other base conditions which impact free flow speed, and subsequently capacity, include lane width, traffic stream composition, lateral clearance, lane distribution, curvature, etc.

According to the HCM2000, there is no evidence that merging or diverging maneuvers restrict the total capacity of the upstream or downstream basic freeway segments. Their influence is primarily to add or subtract demand at the ramp-freeway junction. Thus, the capacity of a downstream basic freeway segment is not influenced by turbulence in a merge area. The capacity will be the same as if the segment were a basic freeway segment. As on-ramp vehicles enter the freeway at a merge area, the total number of ramp and approaching freeway vehicles that can be accommodated is the capacity of the downstream basic freeway segment.

Similarly, the capacity of an upstream basic freeway segment is not influenced by the turbulence in a diverge area. The total capacity that may be handled by the diverge junction is limited either by the capacity of the approaching (upstream) basic freeway segment or by the capacity of the downstream basic freeway segment and the ramp itself.[2] [1]

The basic approach to modeling merge and diverge areas in the HCM focuses on an influence area of 1500 ft including the acceleration or deceleration lane and lanes 1 and 2 of the freeway. The HCM recognizes that other freeway lanes may be affected by merging or diverging operations and the impact of congestion in the vicinity of a ramp can extend beyond the 1500 ft influence area, but according to it, this defined area experiences most of the

[1] HCM2000 *Exhibit 13-1 Chapter 13 – Freeway Concepts*
[2] HCM2000 *Chapter 13-Freeway Concepts, page 13-22*

operational impacts across all levels of service. Thus, the operation of vehicles within the ramp influence area is the focus of the computational procedures in the HCM2000.

The capacity of a merge or diverge area is always controlled by the capacity of its entering and exiting roadways, that is, the freeway segments upstream and downstream of the ramps, or by the capacity of the ramp itself. Research has shown that the turbulence due to merging and diverging maneuvers does not affect the capacity of the roadways involved, although there may be local changes in lane distribution and use.[1] [1]

The HCM determines the capacity of a merge area primarily by the capacity of the downstream freeway segment. Thus, the total flow arriving on the upstream freeway and the on-ramp cannot exceed the basic freeway capacity of the departing downstream freeway segment. There is no evidence that the turbulence of the merge area causes the downstream freeway capacity to be less than that of a basic freeway segment.[2] [1]

The freeway capacity per lane is always stated as an average across all lanes and the individual lanes always carry proportionally less or more flow. In merge and diverge areas, through vehicles tend to move left to avoid turbulence, resulting in cases where inner lanes are very heavily loaded compared with lanes within the ramp influence area (i.e., lanes 1 and 2). [1]

RESEARCH APPROACH

Due to the limited resources and exploratory nature of this study, the scope was limited to the use of existing data sources. The identification of appropriate data sources was the first task. Several traffic management centers (TMCs) around the country archive inductance loop detector data. After determining the availability of reliable data, the second task was the selection of suitable sites within the respective TMC's domain. The relevant data for the selected sites was collected from the sources provided by the TMCs. The time frame of the data collected was a crucial step in the process of acquiring data. A time frame that could best represent the conditions to be studied was decided upon and volume data were collected for the selected sites. Statistical analyses were then performed on the collected volume data.

This section gives details of the site selection, data collection methodology and time frame selection. Some of the limitations of the data collection process are also described.

Data Source

Although the HCM contains a method for estimating the capacity of a freeway segment, the best way to determine capacity is to measure flow rates in the field. Different techniques can be used for traffic flow measurement, such as manual counts, pneumatic tubes, loop detectors, radar, microwave, and video image processing. This project concentrated on the usage of existing loop detector data.

Potential Data Sources

The primary concern was the availability of useful data from traffic management centers. As described earlier, the factor determining the usefulness of data was the availability of loop detector data for freeway segments of varying lengths. Only traffic management

[1] HCM2000 *Chapter 25-Ramps and Ramp Junctions*

centers that had historical volume data were considered. Availability of loop data from the following sources was checked:

- Washington State Department of Transportation (WSDOT)
- Florida Department of Transportation (FDOT)
- Arizona Department of Transportation (ADOT)
- Texas Department of Transportation (TxDOT)
- Virginia Department of Transportation (VDOT)
- Minnesota Department of Transportation (Mn/DOT)

Archived loop data did not exist for all the above mentioned departments of transportation. After careful evaluation of the available data, it was decided that the data from the WSDOT website [2] and the FDOT's I-4 Traffic information website [3] would be used.

Site Selection

Once the sources for the archived loop data were selected, the next task was to select sites.

Desirable Site Characteristics

The next step was to identify the desirable characteristics of potential data collection sites. Finding sites with close interchange spacing was, of course, not difficult. The challenge was to find some sites with high flow rates that had an interchange spacing of at least a mile or more. The physical attributes of the potential data collection site were also considered. A mix of four-, six-, and eight-lane cross sections were desired. Four-lane cross sections are desirable because existing research has shown that the outer two lanes are most impacted; hence in the case of a four-lane section, both lanes would be subjected to turbulence of the ramp traffic. However, four-lane freeways in urban areas (where capacity conditions occur) are rare (the SR-520 floating bridge section in the Seattle region is an example of one, but there were no loop detectors at that location).

Sections with very high traffic volumes were to be considered; thus, only urban sections of the freeway were used. Segments under construction at the time of data collection were not used. A summary of the criteria for suitable data collection is given in Table 1.

Table 1: Site selection Criteria

Category	Desirable Locations/Attributes
Physical Attributes	• Longer segments as well as shorter segments • Six lane (three lanes in each direction) or wider segment • Sections with a lane drop • No construction zones
High Traffic Locations	• Sections with near or at-capacity flow rates (thus limited to urban sites) • Stop-and-go traffic conditions to be avoided
Data	Loop detector data available for mainline segments
Time Period	Times of year with maximum traffic flows, e.g., holidays

Seattle Region

After evaluating different parts of the freeway network in the Seattle area and keeping the site selection criteria under consideration, the following sites were selected:

1. S1[1]: I-90, City of Issaquah area (East of Seattle)
2. S2: I-90, Lake Washington Floating Bridge
3. S3: I-5, South Seattle (Boeing Field Area)
4. S4: I-5, Ship Canal Bridge (Central Seattle)
5. S5: I-5, North Seattle. (NE 155th Street)
6. S6: I-5, North Seattle (NE 185th Street)

Brief details and snapshots[2] of the locations are given below and diagrams showing loop detector locations are given in Appendix A.

- S1: I-90, Issaquah (East of Seattle): The section of the interstate from Exit 13 and Exit 15 was considered. This provided a segment length of approximately 1 mile between the ramps with loop detectors approximately midway on the mainline segment.

[1] In "S1", the 'S' stands for Seattle and the number '1' stands for the site number. A list of the sites can be seen in Table 2.
[2] Photos obtained from the WSDOT website, http://www.wsdot.wa.gov/traffic/seattle/

(looking west)

(looking east)

Figure 2: Snapshot of I-90 segment in Issaquah

- S2: I-90, Lake Washington Floating Bridge: This site provided a 2-mile segment of the freeway between the ramps on either side of the bridge, with loop detectors on the bridge segment.

(looking east)

Figure 3: Snapshot of I-90 Floating Bridge

- S3: I-5, South Seattle (Boeing Field Area): The section of the freeway between Exit 158 and Exit 161 was considered. This provided a segment of approximately 2.16 miles between adjacent ramps with loop detectors on the mainline segment.

(looking south)

(looking north)

Figure 4: Snapshot of I-5 close to Boeing Access Road

(looking south)

(looking north)

Figure 5: Snapshot of I-5 near Boeing Field

- S4: I-5, Ship Canal Bridge: The section of the interstate between Exit 168A and Exit 169 was considered. This provided a mainline segment of approximately 1 mile between adjacent ramps with loop detectors on the mainline segment.

Figure 6: Snapshot of I-5 near Ship Canal Bridge (looking north)

- S5: I-5, North Seattle (NE 155th Street): The section of the interstate between Exit 175 and Exit 176 was considered. This provided a segment of approximately 1 mile between adjacent ramps with loop detectors on the mainline segment.

(looking south)

(looking north)

Figure 7: Snapshot of I-5 at 145th Street

(looking south)

(looking north)

Figure 8: Snapshot of I-5 at 175th Street

- S6: I-5, North Seattle (NE 185th Street): Capacity conditions were likely to occur at this site as the loop detector was located just after a lane drop on the northbound freeway segment.

(looking south) (looking north)

Figure 9: Snapshot of I-5 at 195th Street

Some aspects of the sites selected in Florida are discussed below:

Orlando Region

Only the highest hourly volume could be determined from the data available on the Florida Traffic Information (FTI2002) CD-ROM[1]. An average per lane volume was determined by dividing this by the number of lanes. This was then compared to the data obtained from the Traffic Info website. [3] Initially six sites were considered for data collection in the Orlando region. Appendix A shows the locations of these sites. After careful evaluation of these sites, four locations were chosen for further analysis.

- O1: Site 0343[2] - SR-93/I-75, 0.6 miles south of SR-54, Pasco County

Figure 13 shows the location of the permanent traffic monitoring station. This site provided a mainline segment of approximately 4 miles between interchanges.

[1] The FTI2002 CD-ROM contains a graphical interface to access traffic data collected for over 7,500 traffic count locations on the State Highway System More information about the FTI2002 CD-ROM is given in following sections

[2] Site number refers to site numbers as given in the FTI2002 CD-ROM[4] and are represented by the red circles in Figure 13 - 20

Figure 10: O1 - Site 0343 (SR-93/I-75, 0.6 Mi S of SR-54, Pasco Co)

- O2: Site 0196 - SR-400/I-4, @SR-408 Overpass, Orange County

Figure 14 shows the location of the permanent traffic monitoring station. This site provided a mainline segment of approximately 0.85 miles between interchanges.

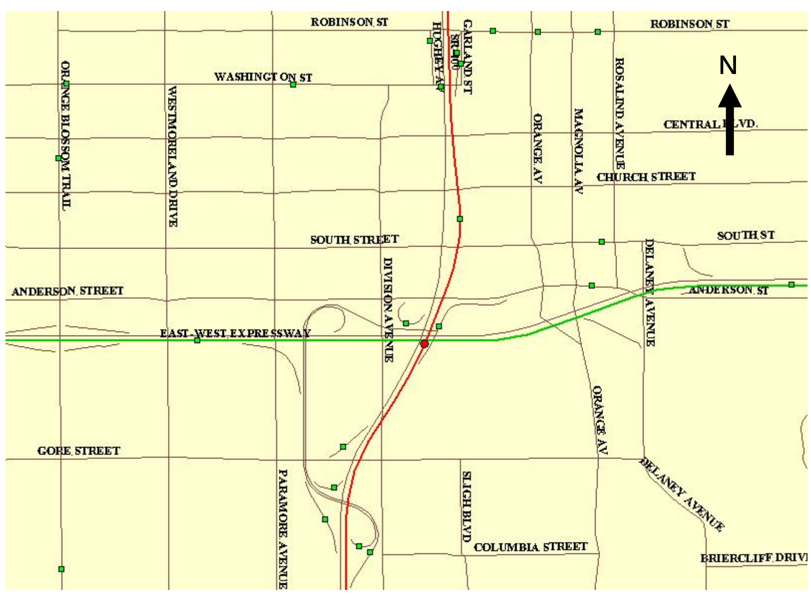


Figure 11: O2 - Site 0196 (SR-400/I-4, @SR-408 Overpass, Orange Co)

- O3: Site 0130 - SR-400/I-4, 0.8 miles south of SR-482, Orange County

Figure 15 shows the location of the permanent traffic monitoring site. This provided a mainline segment of approximately 1.55 miles between interchanges.

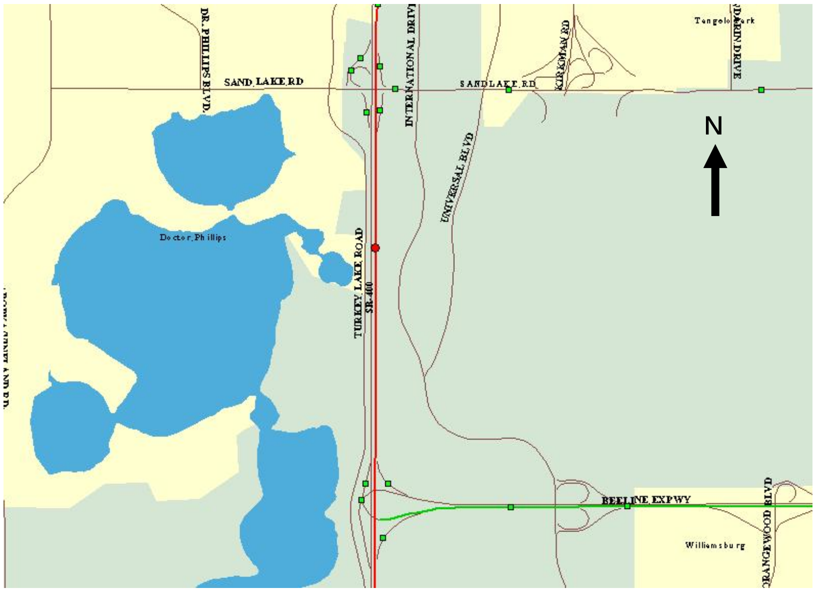


Figure 12: O3 - Site 0130 (SR-400/I-4, 0.8 Mi S of SR-482, Orange Co)

- O4: Site 0303 - On I-4, 0.5 miles southwest of Orange Co Line, Osceola County

Figure 16 shows the location of the permanent traffic monitoring station. This provided a mainline segment of approximately 0.86 miles.

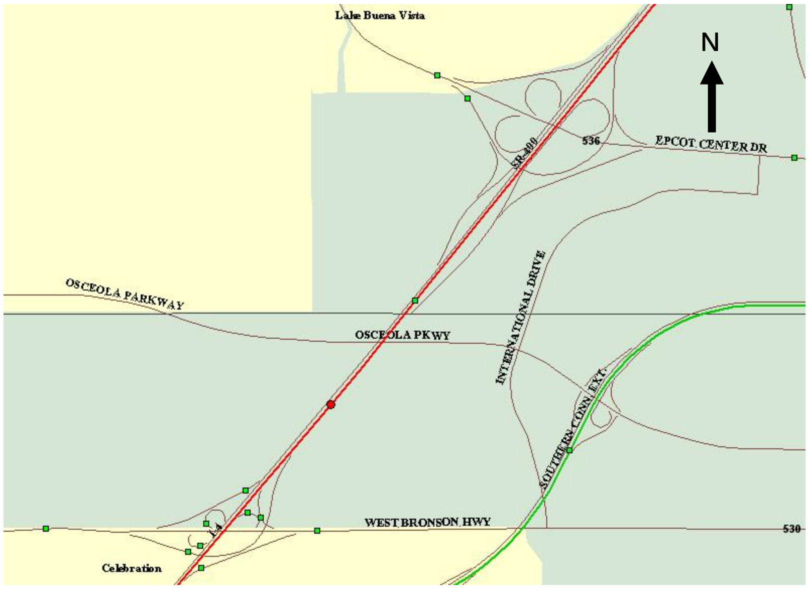


Figure 13: O4 - Site 0303 (On I-4, 0.5 Mi SW of Orange Co Line, Osceola Co)

Tampa Region

Sections of I-75, I-275 and I-4 in the Tampa region were used. Initially, eight sites were considered for data collection and are shown in Appendix A. After careful evaluation of the site locations and the data available, four sites were chosen for further analysis.

- T1: Site 0190[1] - SR-93/I-75, 0.6 miles south of SR-54, Pasco County

Figure 17 shows the location of the permanent traffic monitoring station. This site provided a mainline segment of approximately 1.3 miles between interchanges.

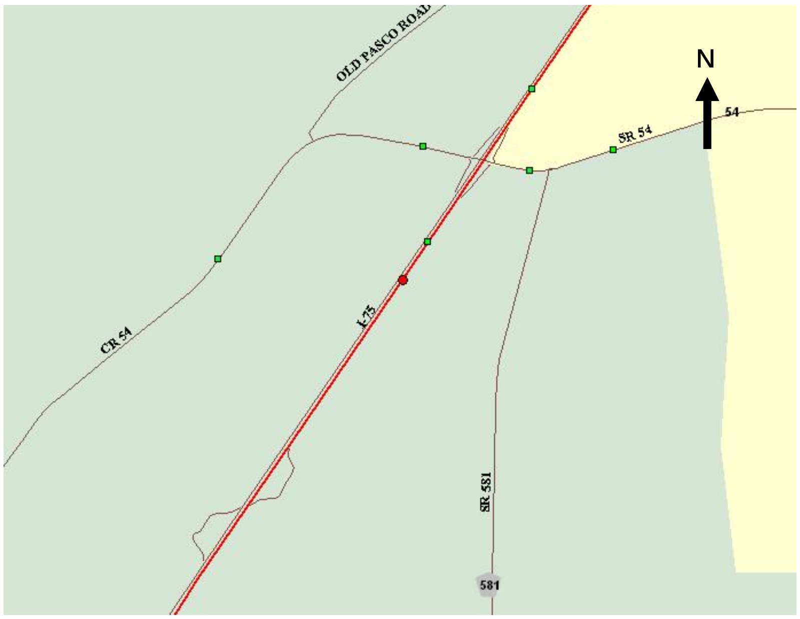


Figure 14: T1 - Site 0190 (SR-93/I-75, 0.6 Mi S of SR-54, Pasco Co)

- T2: Site 0110 - SR-93/I-275, 1.3 miles east of Howard Franklin Br, Hills. County

Figure 18 shows the location of the permanent traffic monitoring station. This site provided a mainline segment of more than 4 miles between interchanges. The loop detector was at a location just before the ramp junction, but still showed reasonably high volumes.

[1] Site number refers to site numbers as given in the FTI2002 CD-ROM[4] and are represented by the red circles in Figure 13 - 20

Figure 15: T2 - Site 0110 (SR-93/I-275, 1.3 Mi E of Howard Franklin Br, Hills. Co)

- T3: Site 0194 - SR-93A/I-75,0.6 miles south of US-301, Hillsborough County

Figure 19 shows the location of the permanent traffic monitoring station. This site provided a mainline segment of approximately 2.6 miles between interchanges.

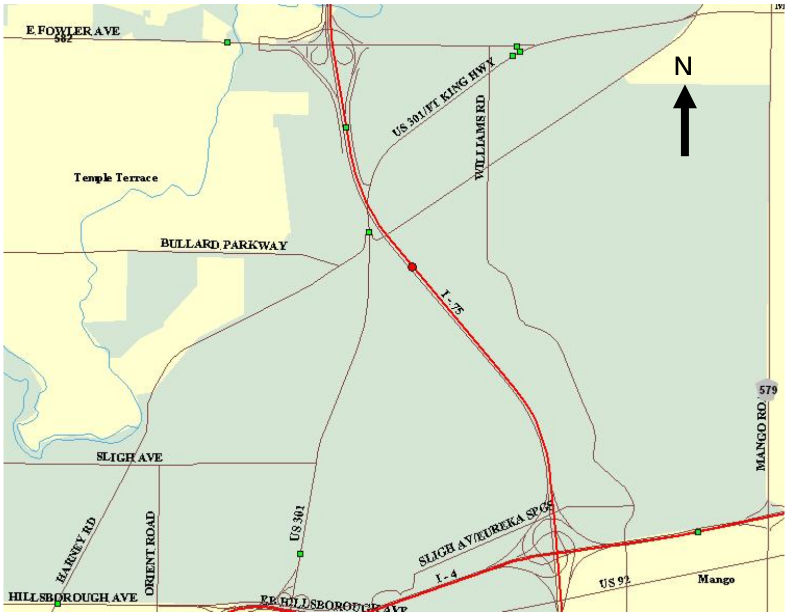


Figure 16: T3 - Site 0194 (SR-93A/I-75, 0.6 Mi S of US-301, Hills. Co)

- T4: Site 9926 - SR-93A/I-75, 1.25 miles north of SR-60, Tampa, Hillsborough County

Figure 20 shows the location of the permanent traffic monitoring station. This site provided a mainline segment of approximately 1.9 miles between interchanges.

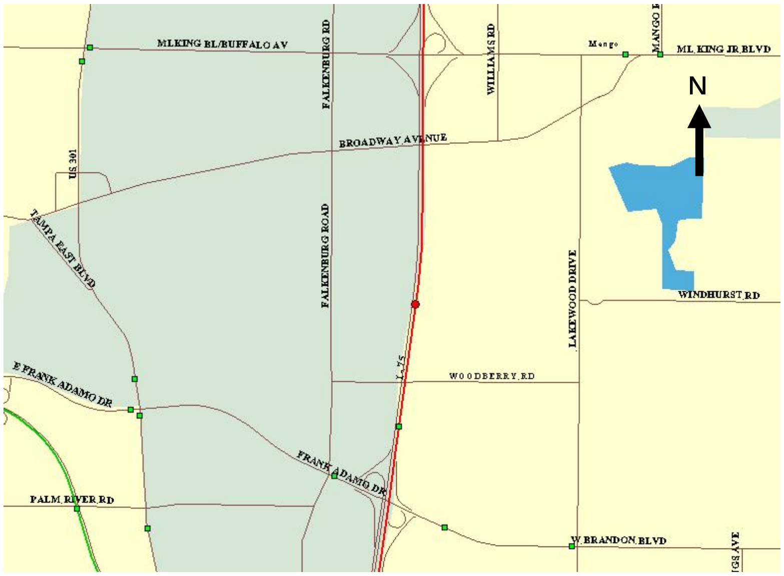


Figure 17: T4 - Site 9926 (SR-93A/I-75, 1.25 Mi N of SR-60, Tampa, Hills. Co)

Data Collection

Only mainline volumes were considered for this phase of the project. Data showing near and at-capacity conditions were used. Under-capacity or over-saturated conditions were avoided. For this reason, the times of the year considered for data collection were carefully selected. Another important issue to be considered was whether capacity flow was being measured, or queue discharge flow. It should be noted that in some instances this is a difficult distinction to make with only loop data.

It was decided to collect data from times of the year at which maximum traffic flow is expected. The different times considered were:

- New Year’s day period (first week of January)
- Spring break period (March 8 to March 25)
- Independence day period (first week of July)
- Thanksgiving period (November 25 to December 3)
- Christmas period (December 20 to December 31)

Other time periods that might yield capacity flows were also searched for as part of this task.

The data for the Washington (Seattle) area were present on the Traffic Data Acquisition and Distribution (TDAD) website [2]. The TDAD database collects the output from the ILD network monitored and maintained by WSDOT’s traffic systems management center (TSMC). Every twenty seconds, a new "snapshot" of the current highway conditions is added to the database. The TDAD Query Interface provides a web-based front end to the system. Using this tool, a query can be refined and submitted, and a text file containing the requested data is made available for download. The data can then be imported into other programs for further analysis. Figure 18 shows a view of the TDAD query interface.

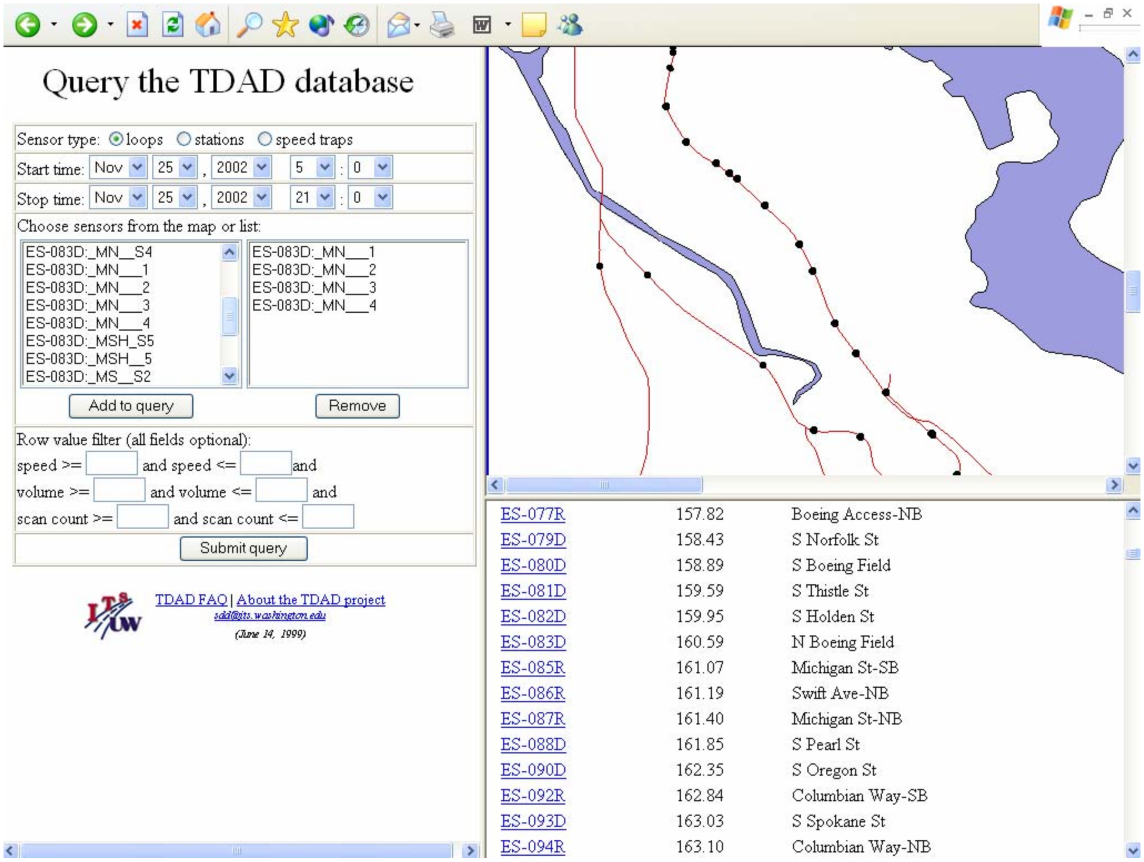


Figure 18: Screen-shot of the TDAD query interface

For the purpose of this project, the highest hourly volume was needed. This required some processing of the data. We first chose to use Microsoft Excel for this purpose. A typical Excel spreadsheet obtained by importing the data from the downloaded text file is shown in Appendix B. As described earlier, this has the traffic counts for every twenty second interval. To obtain the maximum hourly volume, columns were added to the spreadsheet and the volume after every five, fifteen and sixty minutes was determined. A typical Excel spreadsheet after these changes is shown in Appendix B. The data rows in a typical Excel spreadsheet (for one loop detector) ranged from 30,000 to more than 60,000 depending on the number of lanes and ramp type. In some cases the number of rows exceeded the maximum number allowed by Microsoft Excel. In such cases the file was broken down into two files. In order to make the data in the files easy to understand and use, the 'Data Filter' function of Microsoft Excel was used. This made it possible to view the required information without cumbersome and time consuming scrolling and searching through the whole file.

To determine the suitable time period, the daily traffic counts were used [5]. Each night at midnight, the previous day's data is analyzed to sum the volume measurements from each of the 'station' sensors. These values represent the total number of vehicles, in all mainline lanes, that have passed a particular location within a given 24-hour interval. The resulting daily counts are then placed in a database table. The primary freeway system in the Seattle area also contains high-occupancy vehicle (HOV) lanes. The volume data from these lanes was excluded because they typically do not experience capacity flows. This added to the data reduction complexity.

Considering the time required to organize and analyze the data downloaded from the TDAD website for each loop detector, the time period for the Seattle region sites was decided on by using the daily traffic counts from the TDAD website. Thirty days with the highest annual average daily traffic from the year 2002 were considered. Out of these thirty, the volumes with obvious below-capacity conditions were excluded from the analysis. Appendix C gives the data considered for analysis.

For the Florida sites, two resources were used for acquiring the necessary data, the I-4 Traffic Info website [3] and the Florida Traffic Information 2002 (FTI2002) CD-ROM [4].

The I-4 Traffic Info website [3] provides near real-time traffic information to the traveling public as a service of the University of Central Florida's Center for Advanced Transportation System Simulation (CATSS). It also has archived historical traffic data. The entire length of I-4 within the City of Orlando region was considered, as this entire length was included in the website database.

The volume data available on the website was in the form of a web page (HTML format). Using the data in this format was very cumbersome and time consuming. Importing data into other software like Microsoft Excel, in a usable format, required considerable manipulation. Dr. Haitham Al-Deek, the founder of this website and the Principal Investigator of the UCF Data Warehouse project, was contacted to see if an alternative format for the data was available. We were informed that at the time, the data were only available from the website in HTML format. [6]

The FTI2002 CD-ROM contains a graphical interface to access traffic data collected for over 7,500 traffic count locations on the State Highway System. These data are available from one of the most comprehensive traffic count programs in the country. The Florida Traffic Information program allows users to locate, identify, and access this information from the thousands of traffic count sites monitored in 2002. Two types of traffic monitoring sites are used for this purpose, the Telemetered Traffic Monitoring Site (Permanent) and the Portable Traffic Monitoring Site (Temporary). The permanent sites collect data on a continuous basis, whereas portable sites only collect data when a traffic recorder is installed at the roadside cabinet. The data present on the CD-ROM comes primarily from the permanent count station sites. Data at different sites available on this CD-ROM were studied. The Orlando region (I-4), the Tampa region (I-75, I-275, I-4), the Bridge(s) across the St. Johns River (I-295) in Jacksonville and other Jacksonville sites were considered.

It was decided that information for I-4 in the Orlando region would be obtained from this CD-ROM and compared to the data obtained from the Traffic Info website. Data from the FTI CD-ROM would be used for the Tampa region.

The '200 Highest Hour' report generated by the FTI2002 CD-ROM was used to determine a suitable time period. The '200 Highest Hour' traffic count report is an annual report that provides traffic count information for the highest 200 hours of the year at all permanent traffic monitoring sites. For sites with incomplete data, the 200 Highest Hour report was not generated by the CD-ROM. In such cases the 'Hourly Continuous Count' report was used. This report displays traffic counts for every hour of each day. The traffic counts are arranged by month and direction of traffic flow for a permanent traffic monitoring site. The 'Hourly Continuous Count' report for each month of the year was scanned for the highest volumes. Considering the above mentioned site selection criteria, the sites selected and their attributes are given in **Table** 2.

Table 2: Locations and Attributes selected for research

<table>
<tr><th>Category</th><th>Selected Locations/Attributes</th></tr>
<tr><td>Traffic Management Center</td><td>Washington (WSDOT)
Florida (FDOT)</td></tr>
<tr><td>Selected Locations</td><td>Washington:
Seattle
S1: I-90 City of Issaquah area (East Seattle)(1 mile)
S2: I-90 Lake Washington floating bridge (2 miles)
S3: I-5 South Seattle (Boeing Field Area) (2.16 miles)
S4: I-5 Ship Canal Bridge (1 mile)
S5: I-5 North Seattle (NE 155th Street) (1 mile)
S6: I-5 North Seattle (NE 185th Street) (1 mile)
Florida:
Orlando
O1: Site 0343 - SR-400/I-4, 1.6 Mi E of SR-434, Seminole County (4 miles)
O2: Site 0196 - SR-400/I-4, @SR-408 Overpass, Orange County (0.85 miles)
O3: Site 0130 - SR-400/I-4, 0.8 Mi S of SR-482, Orange County (1.55 miles)
O4: Site 0303 - On I-4, 0.5 Mi SW of Orange County Line, Osceola County (0.86 miles)
Tampa
T1: Site 0190 - SR-93/I-75, 0.6 Mi S of SR-54, Pasco County (1.3 miles)
T2: Site 0110 - SR-93/I-275, 1.3 Mi E of Howard Franklin Br, Hills. County (4 miles)
T3: Site 0194 - SR-93A/I-75, 0.6 Mi S of US-301, Hillsborough County (2.6 miles)
T4: Site 9926 - SR-93A/I-75, 1.25 Mi N of SR-60, Tampa, Hillsborough County (1.9 miles)</td></tr>
<tr><td>Time Period</td><td>Washington:
Thirty days with the highest annual average daily traffic were considered and then the data which were obviously below capacity were excluded from the analysis.
Florida:
The highest fifty hours from the "200 Highest Hour" report or the "Hourly Continuous Count" report generated by the FTI2002 CD-ROM</td></tr>
</table>

RESULTS AND ANALYSIS

Three different analyses were performed on the obtained traffic volume data: Determination of maximum hourly flow rate means and variances at each site; comparison of maximum hourly flow rate means and variances across sites within the same region (Seattle, Orlando and Tampa); and comparison of maximum hourly flow rate means and variances across regions.

Data Limitations

The study relied on existing archived loop detector data. The volumes obtained from both the traffic management centers considered for this project, WSDOT and FDOT, had some limitations.

The data for the Tampa and Orlando regions were obtained mainly from the FTI2002 CD-ROM. Data were missing for many of the sites. Mainly the permanent traffic monitoring sites were used, as the CDROM contained hourly data for these stations only. The highest 200 hourly count data were missing for some sites under consideration. Data from the hourly continuous count reports were used as much as possible to overcome this discrepancy, but this did not ensure the usage of the highest flow rates for a few sites. Another issue was the usage of the hourly counts instead of the peak 15 minutes. The FTI2002 only provides the hourly counts, whereas it is always preferable to use the peak 15-minute count to calculate the maximum hourly volume (via the peak hour factor). Loop detector data with five-minute aggregations were available for site T1. A comparison of the usage of the peak 15-minute count versus the peak-hour count is presented in **Table** 3.

Table 3: Comparison of usage of peak hour versus peak 15 minutes (Site T1)

Date	Peak Hour Count (veh/hr)		Per lane volume (veh/hr/lane)	
	Using peak 15 minutes	Using peak hour directly	Using peak 15 minutes	Using peak hour directly
11/13/03	4128	3800	2064	1900
11/21/03	3840	3606	1920	1803

From **Table** 3, the potential limitation of the data used is evident, but the above example was for only one particular site at which the peak in traffic volume did not occur for a full hour.

The Seattle data were obtained from the TDAD (Traffic Data Acquisition and Distribution) website. The usage of the hourly volume count was not an issue for these data, but the enormous amount of time required to process the data obtained from the website was.

Data Analysis

Table 4 provides a summary of the data collected. The data obtained from the various sites were transferred into a suitable format to be imported into a spreadsheet. Microsoft Excel was used mainly for data processing. Microsoft Excel and MINITAB were used for detailed statistical analyses.

Table 4: Summary of data collected for various sites

Site Description	Site Name	Length (miles)	Loop Detector/Site #	Direction of Traffic Flow	Average Volume (veh/hr/ln)
I-90 City of Issaquah area (East Seattle)	S1	1	924 928 932	W W W	1991 1951 1968
I-90 Lake Washington floating bridge	S2	2	852 855 858 861	E E E E	1944 1915 1893 1912
I-5 South Seattle (Boeing Field Area)	S3	2	79 80 81 82 83	N N N N N	2076 1959 1904 1820 1748
I-5 Ship Canal Bridge	S4	1	130	N S	1931 1955
I-5 North Seattle (NE 155th Street)	S5	1	170 172	N N S	1872 1879 2318
I-5 North Seattle (NE 185th Street)	S6	1	177	N S	2368 2214
SR-400/I-4, 1.6 Mi E of SR-434, Seminole County	O1	4	0343	E W	1699 1784
SR-400/I-4, @SR-408 Overpass, Orange County	O2	1	0196	W	2433
SR-400/I-4, 0.8 Mi S of SR-482, Orange County	O3	2	0130	E W	1702 1764
On I-4, 0.5 Mi SW of Orange County Line, Osceola County	O4	1	0303	E W	1513 1554
SR-93/I-75, 0.6 Mi S of SR-54, Pasco County	T1	1	0190	N S	1814 1719
SR-93/I-275, 1.3 Mi E of Howard Franklin Br, Hills. County	T2	4	0110	E W	1901 1944
SR-93A/I-75, 0.6 Mi S of US-301, Hillsborough County	T3	3	0194	N S	1806 1925
SR-93A/I-75, 1.25 Mi N of SR-60, Tampa, Hills. County	T4	2	9926	N S	1883 1796

A step-by-step explanation of the data analysis performed is given below for the segments chosen in the Seattle region and the two regions in Florida.

Washington

The loop detector data collected and imported as worksheets (Appendix B) were arranged according to different sites. The data present on the TDAD website could be queried according to date, location and exact loop detectors. Single loop detector data were queried for specific dates at one time as manipulating data for more than one loop detector location was too cumbersome and time consuming as the file size became too large. One file (Appendix B) provided the volume data for the different lanes at one location of the freeway.

The loop detector data obtained for the Seattle region are shown in Appendix C. The data gives the maximum flow rate in vehicles per hour per lane at the loop detector location in each direction on the date indicated. For the location of loop detectors, refer to Appendix A.

The *F*-test was performed to compare the means of the collected samples. First, data collected at different loop detector locations at each site were analyzed to check similarity of data within a site, and then data collected for different sites were analyzed. All of the data collected were not used as some locations did not have capacity flow levels.

- **S1: I-90, Issaquah**

The data used and the corresponding analysis of variance (ANOVA) for this site are shown in **Table** 5. The manual calculations for the *F*-test are shown at the bottom of the first part of the table. The *F*-test results given by using the Data Analysis ANOVA tool in Microsoft Excel are also shown at the bottom of **Table** 5, which match with the manual calculations.

Table 5: Sorted data and *F*-test performed for the S1 site

	W924	W928	W932	
	2169	2041	2074	
	2121	2032	2040	
	2071	2021	2036	
	2056	2020	2031	
	2043	2019	2031	
	2039	2016	2028	
	2036	2016	2025	
	2027	2008	2024	
	2022	2003	2013	
	2020	1999	2005	
	2012	1987	2002	
	2012	1977	1999	
	2009	1976	1982	
	2005	1976	1980	
	1999	1973	1944	
	1998	1972	1939	
	1993	1959	1908	
	1991	1959		
	1989	1957		
	1989	1940		
	1953	1938		
	1913			
Average =	2021.24	1989.94	2003.65	
Variance =	2777.21	919.08	1774.52	
Std Dev =	52.70	30.32	42.13	
n =	22.00	21.00	17.00	
(n-1)var =	58321.37	18381.58	28392.26	
SS_{Total} =	115684.71			
SS_E =	105095.21			
SS_T =	10589.50			
MS_E =	1843.78			
MS_T =	5294.75			
$F = MS_T/MS_E$ =	2.872	<	$F_{0.05,2,69}$ =	3.15

SUMMARY

Groups	*Count*	*Sum*	*Average*	*Variance*
W924	22	44467.29	2021.24	2777.21
W928	21	41788.79	1989.94	919.08
W932	17	34062.02	2003.65	1774.52

ANOVA

Source of Variation	*SS*	*df*	*MS*	*F*	*P-value*	*F crit*
Between Groups	10589.5	2	5294.748	2.872	0.065	3.159
Within Groups	105095.2	57	1843.776			
Total	115684.7	59				

The analysis shows that the maximum flow rate values are not statistically significantly different at the 95% confidence level.

- **S2: I-90, Floating Bridge**

The data and analysis results are presented in **Table** 6.

Table 6: Sorted data and *F*-test performed for the S2 site

855E	858E	861E
2083	2047	2063
2080	2037	2045
2072	2033	2041
2072	2032	2041
2057	2031	2033
2025	1993	2023
2009	1980	2017
2007	1977	1997
2007	1975	1983
2005	1972	1980
2004	1967	1976
1987	1952	1973
1983	1948	1967
1975	1942	1961
1972	1940	1961
1961	1936	1957
1956	1932	1935
1955	1932	1929
1951	1920	1929
1934	1916	1918
1923	1915	1907
1916		

SUMMARY

Groups	*Count*	*Sum*	*Average*	*Variance*
855E	22	43933.52	1996.98	2604.43
858E	21	41376.94	1970.33	1882.28
861E	21	41637.49	1982.74	2134.15

ANOVA

Source of Variation	*SS*	*df*	*MS*	*F*	*P-value*	*F crit*
Between Groups	7650.532	2	3825.266	1.728	0.186	3.148
Within Groups	135021.7	61	2213.470			
Total	142672.2	63				

The analysis shows that the maximum flow rate values are not statistically significantly different at the 95% confidence level.

- **S3: I-5, South Seattle**

The data and analysis results are presented in **Table** 7. This site also had two more loop detector locations within the mainline segment considered, but the data obtained from them had lower volumes and more missing observations. This was attributed to reliability and/or undercount problems.

Table 7: Sorted data and *F*-test performed for the S3 site

79N	80N	81N
2297	2097	2099
2195	2051	2078
2194	2043	2042
2190	2028	2021
2189	2019	2016
2171	2018	2014
2151	2018	2004
2147	2015	2003
2144	2015	1982
2128	2012	1971
2128	2001	1967
2118	1994	1960
2117	1990	1959
2117	1980	1948
2091	1973	1945
2084	1968	1944
2077	1968	1925
2077	1960	1923
2069	1959	1919
2058	1959	1909
2052	1936	1900
2050	1931	
2042	1910	
2031	1909	
2029	1907	
2006	1900	

SUMMARY

Groups	*Count*	*Sum*	*Average*	*Variance*
79N	26	54951.65	2113.525	4526.774
80N	26	51562.08	1983.157	2438.493
81N	21	41529.3	1977.586	2938.596

ANOVA

Source of Variation	*SS*	*df*	*MS*	*F*	*P-value*	*F crit*
Between Groups	295835.7	2	147917.9	44.45724	3.45E-13	3.127681
Within Groups	232903.6	70	3327.194			
Total	528739.3	72				

The analysis shows that the maximum flow rate values are statistically significantly different at the 95% confidence level. The next step was to determine which sites were significantly different from one another using the Tukey-Kramer pair-wise comparison method, as shown in Table 8.

Table 8: Tukey-Kramer pair-wise comparison results for the S3 site

$Q_{0.05,3,70}$ = 3.39

Comparison			Mean Δ	HSD	|Δ| > HSD?
N79	vs.	N80	130.37	38.35	**Y**
N79	vs.	N81	135.94	38.35	**Y**
N80	vs.	N81	5.57	40.57	N

It can be seen that the mean of the maximum flow rates for the northbound direction of station 79 was significantly different (at the 95% confidence level) than the northbound direction of stations 80 and 81.

- **Comparison of different sites in the Seattle region**

Separate *F*-tests were not carried out for the S4, S5 and S6 sites due to the small number of loop detector locations within these segments. The capacities of five freeway segments, namely S1, S2, S3, S4, S5 (only northbound) were compared with each other. The southbound direction of the S5 and S6 sites were not included in this comparison due to obvious differences in volumes. These differences were attributed to the lane drop at S6 and fewer lanes at S5 in the southbound direction than northbound. The data for these different locations were compared separately.

For the purpose of these comparisons, the data collected at different loop detector locations were averaged for each site and the resulting values were used in the analysis. These data and results are shown in **Table** 9.

Table 9: Sorted data and *F*-test performed for comparison of the five Seattle sites

S1	S2	S3	S4	S5 (N)
2095	2064	2164	2110	2021
2064	2054	2108	2085	2000
2043	2049	2093	2057	1995
2036	2048	2080	2026	1982
2031	2040	2075	2018	1974
2028	2014	2068	2018	1942
2026	2002	2058	1987	1934
2020	1994	2055	1980	1932
2013	1988	2047	1978	1929
2008	1986	2037	1974	1927
2000	1982	2032	1974	1922
1996	1971	2024	1972	1918
1989	1966	2022	1966	1917
1987	1959	2015	1960	1917
1972	1958	2003	1953	1911
1970	1952	1999	1950	1911
1953	1941	1990	1947	
1975	1939	1987	1940	
1973	1933	1982	1932	
1965	1923	1975	1926	
1946	1915	1963	1916	
1913	1916	1991	1913	
		1976		
		1970		
		1968		
		1953		

SUMMARY

Groups	*Count*	*Sum*	*Average*	*Variance*
S1	22	44001.01	2000.05	1738.30
S2	22	43593.07	1981.50	2218.95
S3	26	52633.57	2024.37	2721.64
S4	22	43577.50	1980.80	2750.73
S5 (N)	16	31128.50	1945.53	1290.22

ANOVA

Source of Variation	*SS*	*df*	*MS*	*F*	*P-value*	*F crit*
Between Groups	68020.64	4.00	17005.16	7.67	1.9E-05	2.46
Within Groups	228261.75	103.00	2216.13			
Total	296282.39	107.00				

The analysis shows that the maximum flow rates are statistically significantly different at the 95% confidence level. To determine which values were significantly different from each other, the Tukey-Kramer pair-wise comparison method was used as before. The results are shown in Table 10.

Table 10: Tukey-Kramer pair-wise comparison results for five segments of the Seattle region

$Q_{0.05,5,103}$ = 3.95

Comparison			Mean Δ	HSD	|Δ| > HSD?
S1	**vs.**	**S2**	**18.54**	**39.64**	**N**
S1	**vs.**	**S3**	**-24.32**	**38.09**	**N**
S1	**vs.**	**S4**	**19.25**	**39.64**	**N**
S1	vs.	S5	54.51	43.20	Y
S2	vs.	S3	-42.87	38.09	Y
S2	**vs.**	**S4**	**0.71**	**39.64**	**N**
S2	**vs.**	**S5**	**35.97**	**43.20**	**N**
S3	vs.	S4	43.57	38.09	Y
S3	vs.	S5	78.84	41.78	Y
S4	**vs.**	**S5**	**35.26**	**43.20**	**N**

A ‘Y’ in the last column indicates that the flow rate values for the compared sites are statistically significantly different.

Another comparison was done between the data collected from the loop detectors in the North Seattle region (S5 and S6). For this comparison, only the data showing abnormally high volumes were used. These data and results are shown in **Table** 11.

Table 11: Sorted data and *F*-test performed for three North Seattle locations

172S	177N	177S
2452	2499	2356
2419	2480	2344
2412	2473	2317
2395	2469	2296
2393	2468	2288
2391	2467	2288
2387	2464	2279
2383	2459	2265
2376	2456	2264
2372	2449	2264
2368	2448	2261
2336	2443	2256
2331	2440	2215
2327	2424	2204
2313	2399	2201
2289	2396	
2289	2391	
2252	2381	
2224	2363	
	2349	
	2349	
	2285	
	2277	

SUMMARY

Groups	*Count*	*Sum*	*Average*	*Variance*
172S	19	44708.00	2353.05	3528.21
177N	23	55628.67	2418.64	3737.12
177S	15	34098.67	2273.24	2060.82

ANOVA

Source of Variation	*SS*	*df*	*MS*	*F*	*P-value*	*F crit*
Between Groups	192771.87	2	96385.94	29.81	2E-09	3.168
Within Groups	174575.92	54	3232.89			
Total	367347.79	56				

The Tukey-Kramer pair-wise comparison results for these sites are shown in **Table** 12.

Table 12: Tukey-Kramer pair-wise comparison results for the North Seattle sites

$Q_{0.05,3,62}$ = 3.42

Comparison			Mean Δ	HSD	|Δ| > HSD?
172S	vs.	177N	-65.59	42.628	Y
172S	vs.	177S	79.81	47.492	Y
177N	vs.	177S	145.39	45.634	Y

These results indicate that the flow rates were all statistically significantly different. The abnormally high volumes at loop detector site 177 could be attributed to the lane drop between loop detector sites 172 and 177.

Florida

The data collected from the FTI2002 CD-ROM were organized into worksheets and the 50 highest recorded hourly volumes were determined (Appendix C). The means and standard deviations of the flow rate data for each site were determined and statistical comparisons were made for the selected sites.

- **Tampa Region**

The thirty-five highest measured volumes were considered for analysis. A summary of the *F*-test performed on the selected capacity data for Tampa region is given in **Table** 13. The data are presented in Appendix C.

Table 13: *F*-test performed on data for the Tampa region

SUMMARY

Groups	*Count*	*Sum*	*Average*	*Variance*
T1-N	35	63483.50	1813.81	2677.43
T1-S	35	60162.50	1718.93	3632.93
T2-E	35	66531.67	1900.90	2892.17
T2-W	35	68050.33	1944.30	730.38
T3-N	35	63201.00	1805.74	778.08
T3-S	35	67356.67	1924.48	243.89
T4-N	35	65903.67	1882.96	1969.26
T4-S	35	62853.67	1795.82	1808.43

ANOVA

Source of Variation	*SS*	*df*	*MS*	*F*	*P-value*	*F crit*
Between Groups	1451736	7	207390.8	112.62	1.41E-76	2.043
Within Groups	500907.6	272	1841.572			
Total	1952643	279				

The analysis shows that the flow rate values are statistically significantly different at the 95% confidence level. To determine which ones were significantly different from one another, the Tukey-Kramer pair-wise comparison method was again used (**Table** 14).

Table 14: Tukey-Kramer pair-wise comparison results for the Tampa region data

$Q_{0.05,8,272}$ = 4.29 HSD = 31.12

Comparison			Mean Δ	HSD	|Δ| > HSD?
T1-N	vs.	T1-S	94.89	31.12	Y
T1-N	vs.	T2-E	-87.09	31.12	Y
T1-N	vs.	T2-W	-130.48	31.12	Y
T1-N	**vs.**	**T3-N**	**8.07**	**31.12**	**N**
T1-N	vs.	T3-S	-110.66	31.12	Y
T1-N	vs.	T4-N	-69.15	31.12	Y
T1-N	**vs.**	**T4-S**	**18.00**	**31.12**	**N**
T1-S	vs.	T2-E	-181.98	31.12	Y
T1-S	vs.	T2-W	-225.37	31.12	Y
T1-S	vs.	T3-N	-86.81	31.12	Y
T1-S	vs.	T3-S	-205.55	31.12	Y
T1-S	vs.	T4-N	-164.03	31.12	Y
T1-S	vs.	T4-S	-76.89	31.12	Y
T2-E	vs.	T2-W	-43.39	31.12	Y
T2-E	vs.	T3-N	95.16	31.12	Y
T2-E	**vs.**	**T3-S**	**-23.57**	**31.12**	**N**
T2-E	**vs.**	**T4-N**	**17.94**	**31.12**	**N**
T2-E	vs.	T4-S	105.09	31.12	Y
T2-W	vs.	T3-N	138.55	31.12	Y
T2-W	**vs.**	**T3-S**	**19.82**	**31.12**	**N**
T2-W	vs.	T4-N	61.33	31.12	Y
T2-W	vs.	T4-S	148.48	31.12	Y
T3-N	vs.	T3-S	-118.73	31.12	Y
T3-N	vs.	T4-N	-77.22	31.12	Y
T3-N	**vs.**	**T4-S**	**9.92**	**31.12**	**N**
T3-S	vs.	T4-N	41.51	31.12	Y
T3-S	vs.	T4-S	128.66	31.12	Y
T4-N	vs.	T4-S	87.14	31.12	Y

The Tukey-Kramer pair-wise comparison test indicates that the means of the data collected for sites T2 and T3 are similar. Both of these sites had relatively long mainline segments between interchanges. The northbound data for site T3 was similar to the southbound data for site T4 and the data for the opposite directions of flow were also similar. These two sites also had reasonably long mainline segments between interchanges.

- **Orlando region**

The thirty-five highest volume hours were used from the data collected to perform the *F*-test for this region. The *F*-test results are shown in **Table** 15.

Table 15: *F*-test performed on data for the Orlando region

SUMMARY

Groups	*Count*	*Sum*	*Average*	*Variance*
O1-E	35	59448.67	1698.53	229.11
O1-W	35	62424.67	1783.56	355.83
O2-W	35	85158.67	2433.10	610.19
O3-E	35	59578.33	1702.24	861.04
O3-W	35	61735.67	1763.88	2084.95
O4-E	35	52940.00	1512.57	2698.44
O4-W	35	54379.67	1553.70	2256.33

ANOVA

Source of Variation	*SS*	*df*	*MS*	*F*	*P-value*	*F crit*
Between Groups	19677273	6	3279545.51	2523.87	2.6E-212	2.137
Within Groups	309260.4	238	1299.41			
Total	19986533	244				

The analysis indicates that the capacity values are statistically significantly different for the 95% confidence interval. The Tukey-Kramer pair-wise comparison results are shown in **Table** 16.

Table 16: Tukey-Kramer pair-wise comparison results for the Orlando region data

$Q_{0.05,7,343}$ = 4.29 HSD = 26.14

Comparison			Mean Δ	HSD	|Δ| > HSD?
O1-E	vs.	O1-W	-85.03	26.14	Y
O1-E	vs.	O2-W	-734.57	26.14	Y
O1-E	**vs.**	**O3-E**	**-3.70**	**26.14**	**N**
O1-E	vs.	O3-W	-65.34	26.14	Y
O1-E	vs.	O4-E	185.96	26.14	Y
O1-E	vs.	O4-W	144.83	26.14	Y
O1-W	vs.	O2-W	-649.54	26.14	Y
O1-W	vs.	O3-E	81.32	26.14	Y
O1-W	**vs.**	**O3-W**	**19.69**	**26.14**	**N**
O1-W	vs.	O4-E	270.99	26.14	Y
O1-W	vs.	O4-W	229.86	26.14	Y
O2-W	vs.	O3-E	730.87	26.14	Y
O2-W	vs.	O3-W	669.23	26.14	Y
O2-W	vs.	O4-E	920.53	26.14	Y
O2-W	vs.	O4-W	879.40	26.14	Y
O3-E	vs.	O3-W	-61.64	26.14	Y
O3-E	vs.	O4-E	189.67	26.14	Y
O3-E	vs.	O4-W	148.53	26.14	Y
O3-W	vs.	O4-E	251.30	26.14	Y
O3-W	vs.	O4-W	210.17	26.14	Y
O4-E	vs.	O4-W	-41.13	26.14	Y

From the analysis, it can be seen that the maximum flow rate means of sites O1 and O3 are similar. Interestingly, both of these sites had relatively long mainline segments between interchanges.

Finally, an *F*-test was done to check the similarity between the means of the maximum flow rates in the Tampa and Orlando regions. The Tukey-Kramer pair-wise comparison results are shown Table **17**.

Table 17: Tukey-Kramer pair-wise comparison results for sites in Orlando and Tampa

Comparison			Mean Δ	HSD	|Δ| > HSD?
O1-E	vs.	T1-N	-115.28	32.34	Y
O1-E	**vs.**	**T1-S**	**-20.40**	**32.34**	**N**
O1-E	vs.	T2-E	-202.37	32.34	Y
O1-E	vs.	T2-W	-245.76	32.34	Y
O1-E	vs.	T3-N	-107.21	32.34	Y
O1-E	vs.	T3-S	-225.94	32.34	Y
O1-E	vs.	T4-N	-184.43	32.34	Y
O1-E	vs.	T4-S	-97.29	32.34	Y
O1-W	**vs.**	**T1-N**	**-30.25**	**32.34**	**N**
O1-W	vs.	T1-S	64.63	32.34	Y
O1-W	vs.	T2-E	-117.34	32.34	Y
O1-W	vs.	T2-W	-160.73	32.34	Y
O1-W	**vs.**	**T3-N**	**-22.18**	**32.34**	**N**
O1-W	vs.	T3-S	-140.91	32.34	Y
O1-W	vs.	T4-N	-99.40	32.34	Y
O1-W	**vs.**	**T4-S**	**-12.26**	**32.34**	**N**
O2-W	vs.	T1-N	619.29	32.34	Y
O2-W	vs.	T1-S	714.18	32.34	Y
O2-W	vs.	T2-E	532.20	32.34	Y
O2-W	vs.	T2-W	488.81	32.34	Y
O2-W	vs.	T3-N	627.36	32.34	Y
O2-W	vs.	T3-S	508.63	32.34	Y
O2-W	vs.	T4-N	550.14	32.34	Y
O2-W	vs.	T4-S	637.29	32.34	Y
O3-E	vs.	T1-N	-111.58	32.34	Y
O3-E	**vs.**	**T1-S**	**-16.69**	**32.34**	**N**
O3-E	vs.	T2-E	-198.67	32.34	Y
O3-E	vs.	T2-W	-242.06	32.34	Y
O3-E	vs.	T3-N	-103.50	32.34	Y
O3-E	vs.	T3-S	-222.24	32.34	Y
O3-E	vs.	T4-N	-180.72	32.34	Y
O3-E	vs.	T4-S	-93.58	32.34	Y
O3-W	vs.	T1-N	-49.94	32.34	Y
O3-W	vs.	T1-S	44.95	32.34	Y
O3-W	vs.	T2-E	-137.03	32.34	Y
O3-W	vs.	T2-W	-180.42	32.34	Y
O3-W	vs.	T3-N	-41.87	32.34	Y
O3-W	vs.	T3-S	-160.60	32.34	Y
O3-W	vs.	T4-N	-119.09	32.34	Y
O3-W	**vs.**	**T4-S**	**-31.94**	**32.34**	**N**
O4-E	vs.	T1-N	-301.24	32.34	Y
O4-E	vs.	T1-S	-206.36	32.34	Y
O4-E	vs.	T2-E	-388.33	32.34	Y
O4-E	vs.	T2-W	-431.72	32.34	Y
O4-E	vs.	T3-N	-293.17	32.34	Y
O4-E	vs.	T3-S	-411.90	32.34	Y

(Table 17 Continued)

Comparison			Mean Δ	HSD	|Δ| > HSD?
O4-E	vs.	T4-N	-370.39	32.34	Y
O4-E	vs.	T4-S	-283.25	32.34	Y
O4-W	vs.	T1-N	-260.11	32.34	Y
O4-W	vs.	T1-S	-165.22	32.34	Y
O4-W	vs.	T2-E	-347.20	32.34	Y
O4-W	vs.	T2-W	-390.59	32.34	Y
O4-W	vs.	T3-N	-252.04	32.34	Y
O4-W	vs.	T3-S	-370.77	32.34	Y
O4-W	vs.	T4-N	-329.26	32.34	Y
O4-W	vs.	T4-S	-242.11	32.34	Y

Sites with similar lengths of mainline segments between interchanges did have more similar mean flow rate values.

SUMMARY AND RECOMMENDATIONS

High volume traffic flow measurements were obtained from archived loop detector data for a variety of freeway segments in the Seattle, Orlando and Tampa regions. The objective was to obtain capacity-level volume measurements across freeway segments of varying lengths and test for significant differences.

The use of archived loop detector data provided valuable insight on the data collection requirements to ultimately investigate the specific issue of the impact of on- and off-ramps on freeway capacity. Some limitations of the use of archived loop data for this purpose were:

- Ambiguity as to whether the flow measurements were actually at capacity
- Selection of the right time period for data collection
- Ambiguity as to whether capacity flow rates or queue discharge flow rates were being measured at lane drop sites
- Use of peak-hour volumes instead of peak 15-minute volumes (converted to an hourly flow rate)

Since project constraints only allowed for a relatively small sample of data to be examined, it is possible that the highest flow rates experienced on the chosen segments were not obtained. Ideally, data for an entire year (or even multiple years) should be obtained, thus ensuring that the highest volumes will be acquired. Additionally, from the volume data alone, it could not be conclusively determined whether the flow rates were not representative of queue discharge conditions, which is a strong possibility for the segments that included a lane drop. The use of speed data along with the volume data could have been useful in determining whether the flow measurements were in the capacity region of the speed-flow curve. The absence of complete speed data on the WSDOT website and the extensive data manipulation required to determine the peak volumes at each loop detector location made it very difficult to ensure that capacity flows were being obtained. The FTI-2002 CD-ROM used for archived data for the Florida regions did not have speed data. The traffic info website provided speed data for some regions, but again the extensive data manipulation and formatting requirements made it very difficult to determine the flow conditions. Furthermore, since heavy vehicle percentages were not available for the obtained loop detector volumes, conversion from units of veh/hr to pc/hr (which is how capacity values are reported by the HCM2000) was not possible. This issue can certainly be responsible for much of the variance in maximum flow rates between sites.

To reach any firm conclusions from the statistical analysis of the data collected, it is essential to obtain a sufficient sample size. The FTI-2002 CD-ROM provided the highest two hundred volumes for any particular site, so sample size was not an issue in the case of the sites chosen in Florida. But for the Seattle region, due to the time constraints and the labor-intensive procedure required to determine the days with the highest flows the sample size was limited to about twenty flow rate measurements.

For the Florida region, hourly volumes were used due to the absence of 15-minute count data for all the sites. The hourly flow rate will not represent a capacity volume if peak traffic flow conditions occur for less than an hour (e.g., such as at the T1-Tampa site which peaks for only about 30 minutes).

From the data collected for this study, it is difficult to draw any statistically valid conclusions. There was a lot of variance in flow rates within each site as well as across sites. Due to limitations in the data set, it is not possible to pinpoint the sources of variance. In general, sites with longer mainline segments between interchanges had higher flow rates than those with shorter segments. However, with the limited number of sites, the number of other variables not accounted for, and the data issues previously mentioned, this relationship cannot be established with any reasonable level of statistical confidence.

This exploratory study illuminated the many difficulties with trying to directly quantify the effect of ramp friction on freeway segment capacities. While many regions have a fairly substantial network of inductance loop detectors, the types of data collected and aggregation intervals of these data can be highly variable. For example, the Seattle data were mostly obtained from single-loop installations, which do not provide speed or vehicle classification data. However, it was archived in very small time intervals. The Florida data was mostly based on dual-loop installations, but was aggregated in one-hour intervals. The Florida data also did not contain ramp volume.

The experimental design required to study this issue and arrive at statistically valid conclusions would be extensive. Besides controlling for interchange spacing, a number of other variables must also be accounted for, such as free-flow speed, percentage of heavy vehicles, geometric characteristics (e.g., grade, horizontal curvature, ramp characteristics), number of lanes, and weather conditions. This would require a substantial number of data collection sites. Additionally, due to the inherent variability in traffic flows at any one site from one day to the next, a very large number of days of data must be collected. The logistics of carrying out such an experimental design would certainly be a giant undertaking. Data collection from dual-loop detector stations and/or video surveillance is preferred as speed and vehicle classifications could be obtained. The use of video would also assist in determining whether capacity or queue discharge flows were being measured. Surveillance equipment already in place as part of traffic management infrastructure could potentially be utilized. Video would likely provide more insight into the factors causing variance at each site (e.g., local driver behavior)

REFERENCES

1. Highway Capacity Manual 2000. Transportation Research Board. National Research Council, Washington, D.C. 2000.

2. Traffic Data Acquisition and Distribution (TDAD) website. Web address: *http://www.its.washington.edu/tdad/*. Accessed for loop detector data acquisition between July 2003 and December 2003.

3. Traffic Info website. Center for Advanced Transportation System Simulation (CATSS), University of Central Florida. Web address: *http://www.trafficinfo.org*. Accessed for loop detector data acquisition between July 2003 and December 2003.

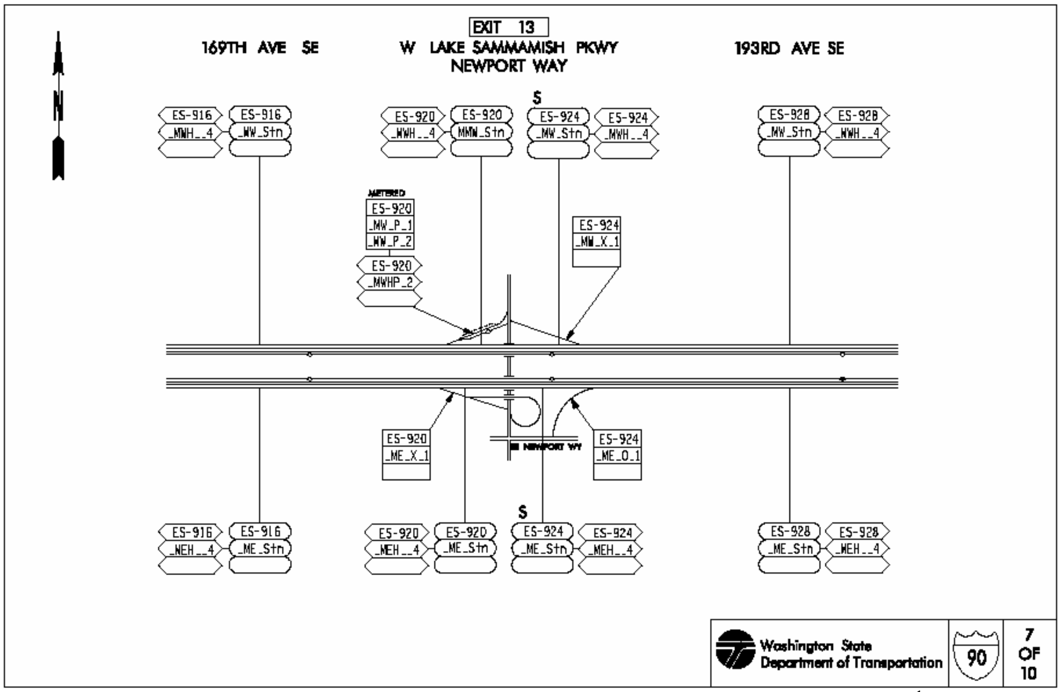
4. Florida Traffic Information 2002 CD-ROM produced by the Florida Department of Transportation. Version 6.0. Tallahassee, Florida.

5. Daily traffic count from Traffic Data Acquisition and Distribution (TDAD) website. Web address: http://www.its.washington.edu/tdad/DailyVolume.html. Accessed to determine daily traffic counts and in turn days with highest traffic flow between July 2003 and December 2003.

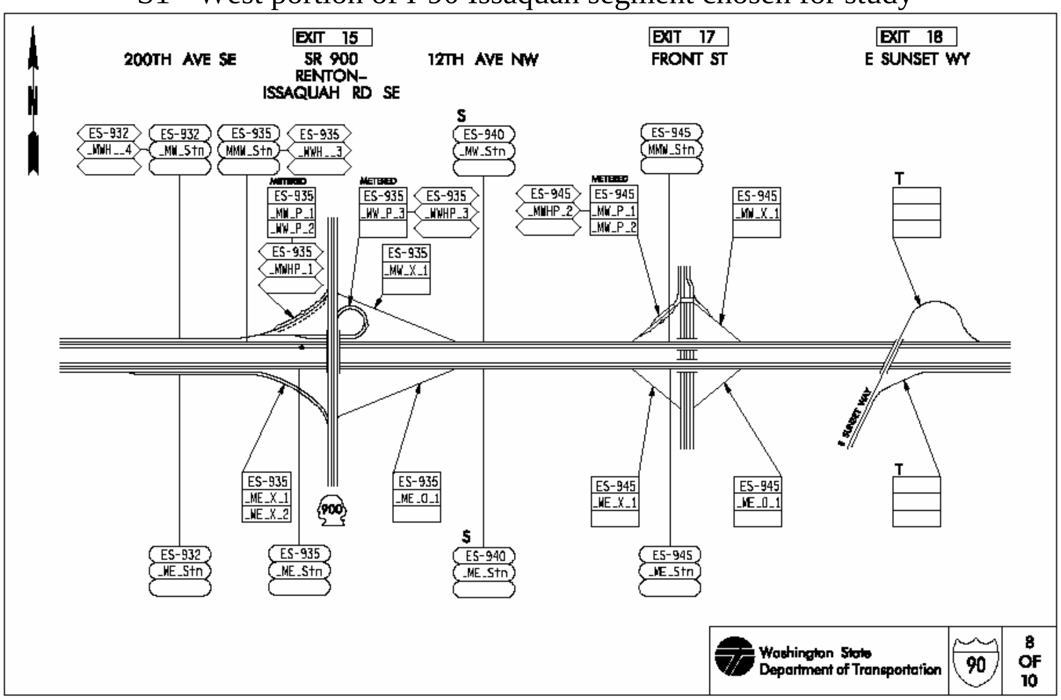
6. Dr. Haitham Al-Deek (haldeek@mail.ucf.edu). E-mail correspondence on November 4 and November 5, 2003.

APPENDIX A

Loop Detector Locations for Segments in the Seattle Region

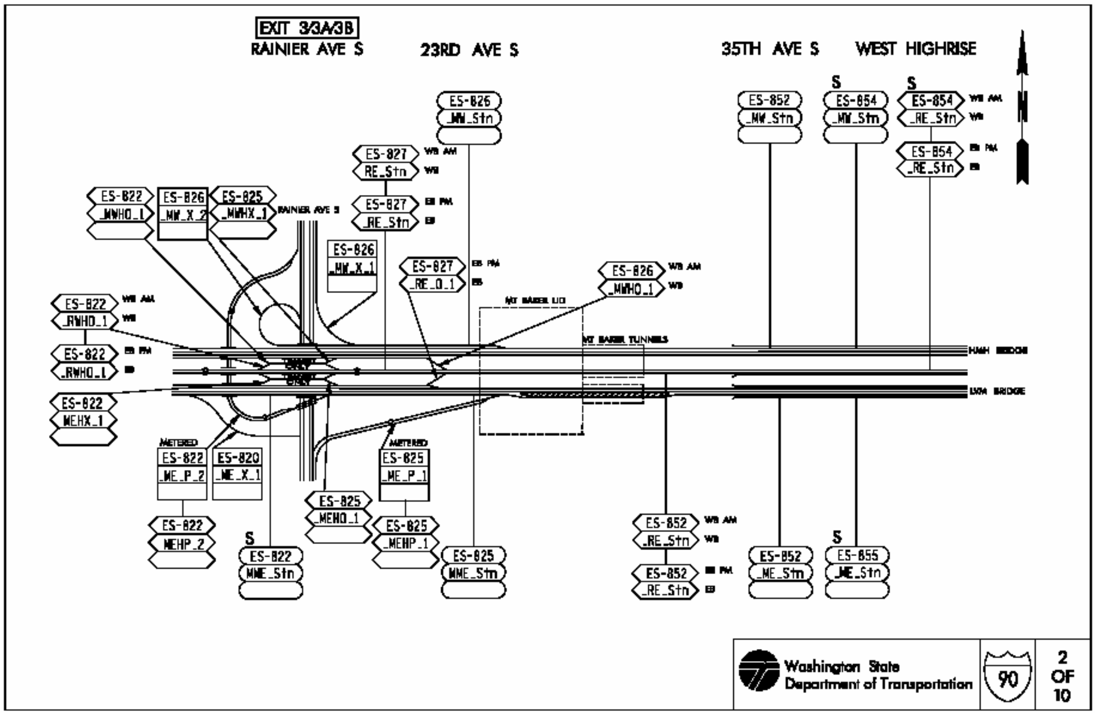


S1 - West portion of I-90 Issaquah segment chosen for study[1]

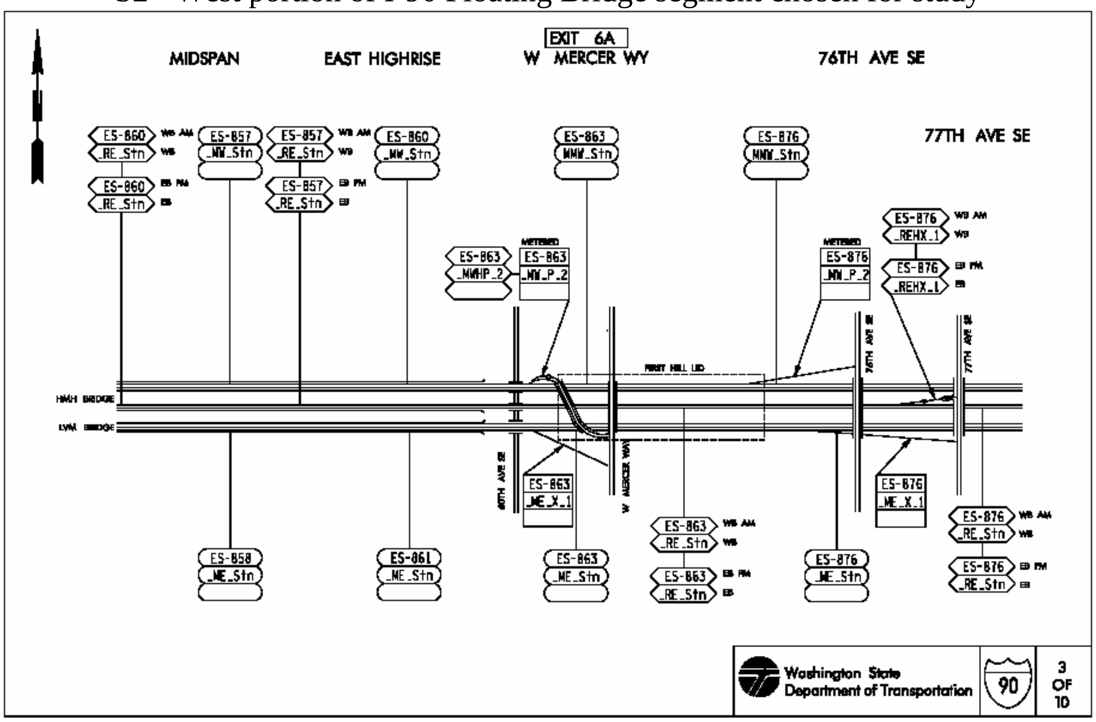


S1 - East portion of I-90 Issaquah segment chosen for study

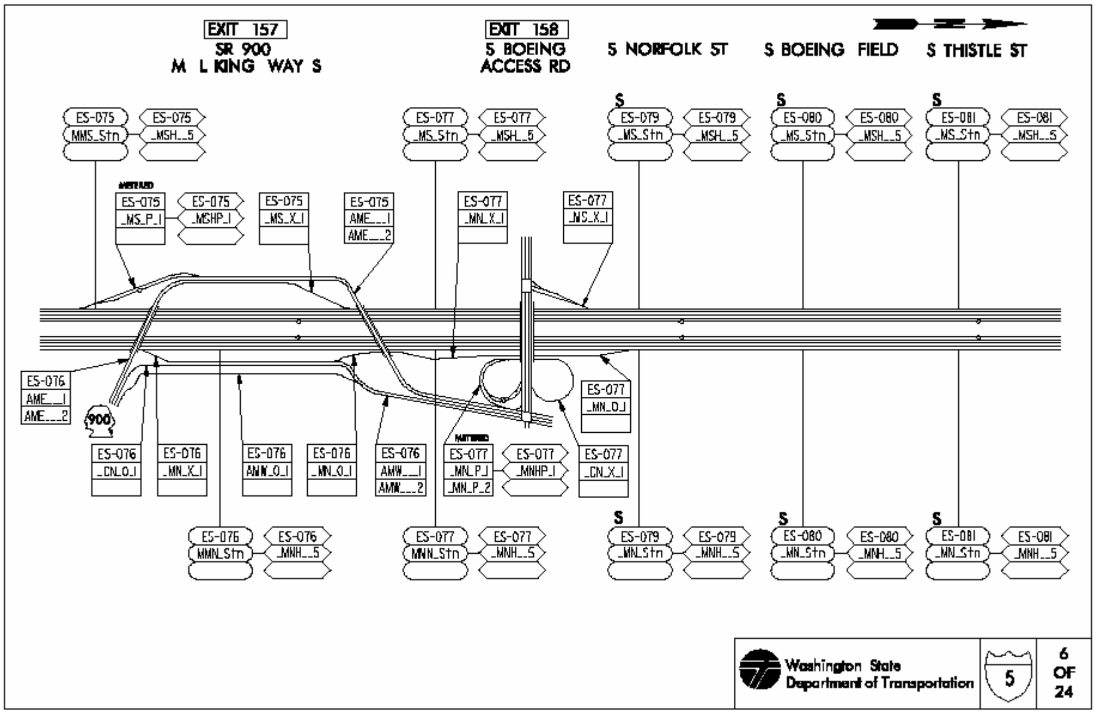
[1] The labels on both sides of the freeway give the loop detector names.[2]



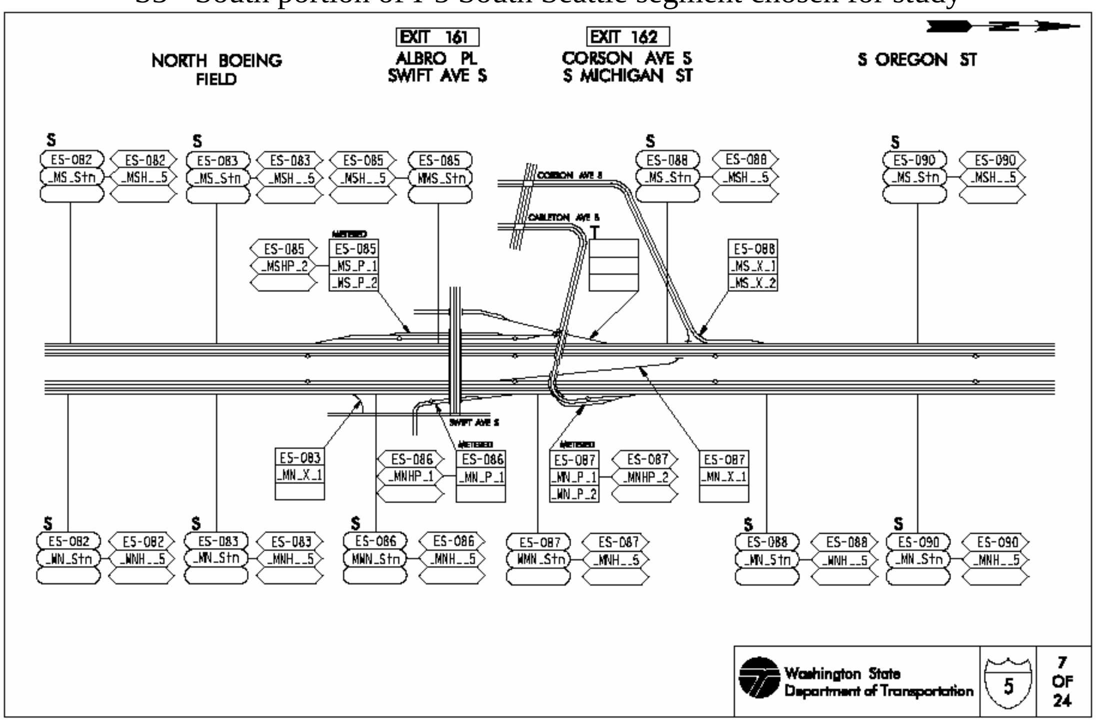
S2 - West portion of I-90 Floating Bridge segment chosen for study



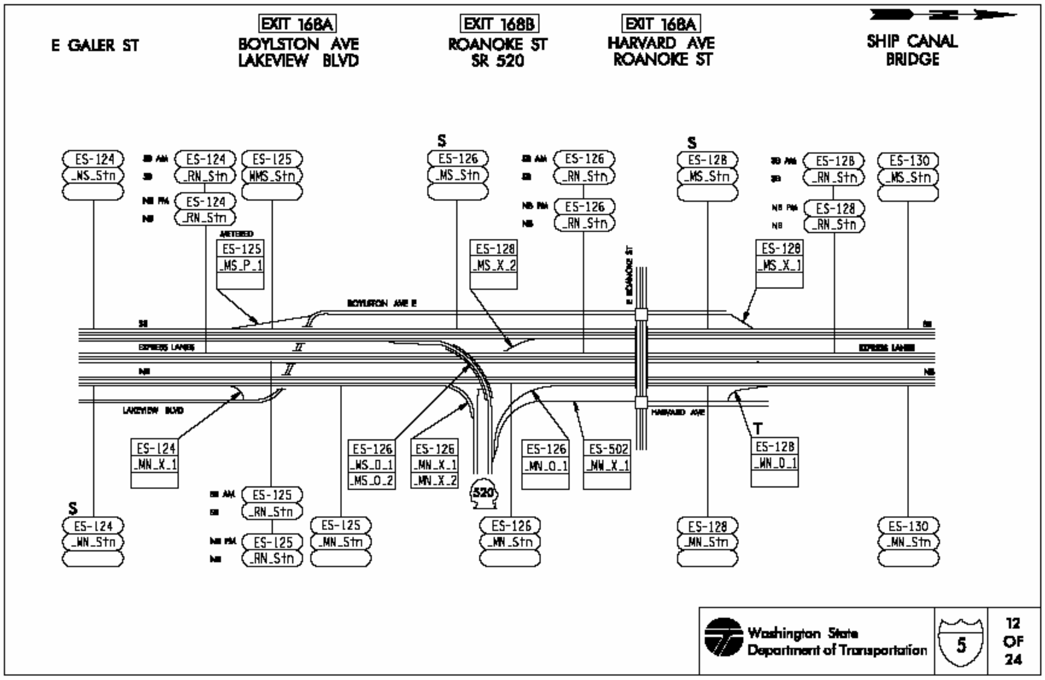
S2 - East portion of I-90 Floating Bridge segment chosen for study



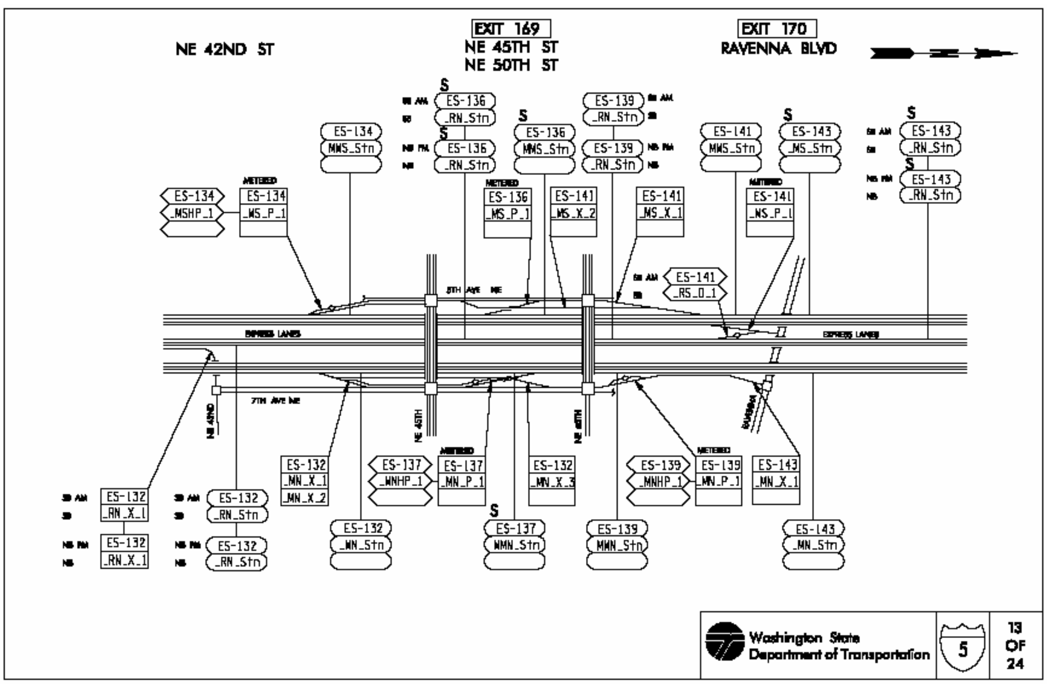
S3 - South portion of I-5 South Seattle segment chosen for study



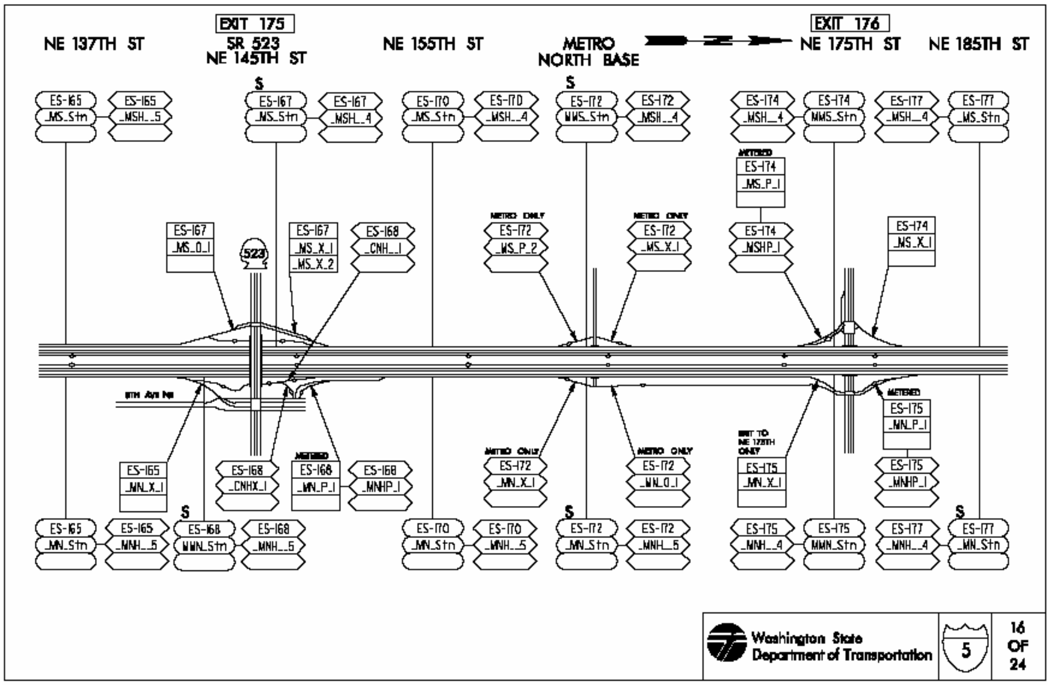
S3 - North portion of I-5 South Seattle segment chosen for study



S4 - South Portion of the I-5 Ship Canal Bridge selected for data collection

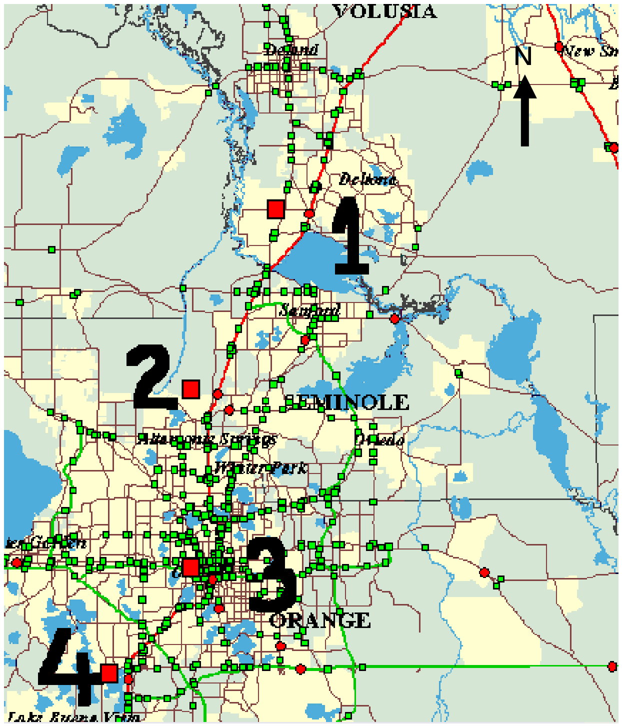


S4 - North Portion of I-5 Ship Canal Bridge selected for study

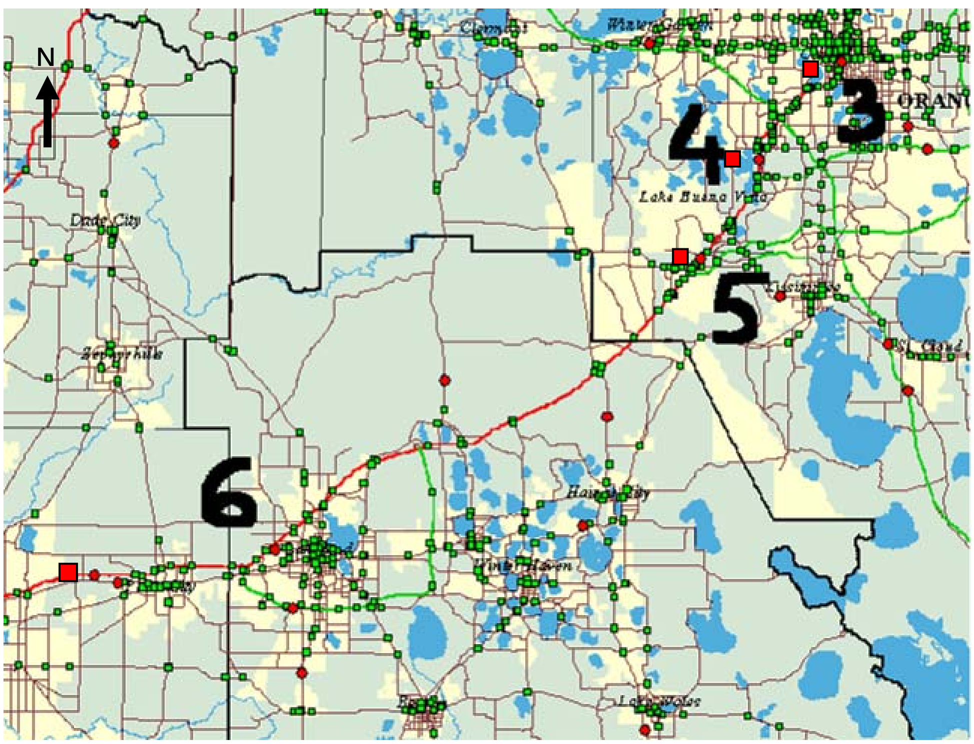


S5 & S6 - Portion of I-5 between exit 175 and exit 176 selected for data collection

Initial Sites Selected for Data Collection in the Orlando Region

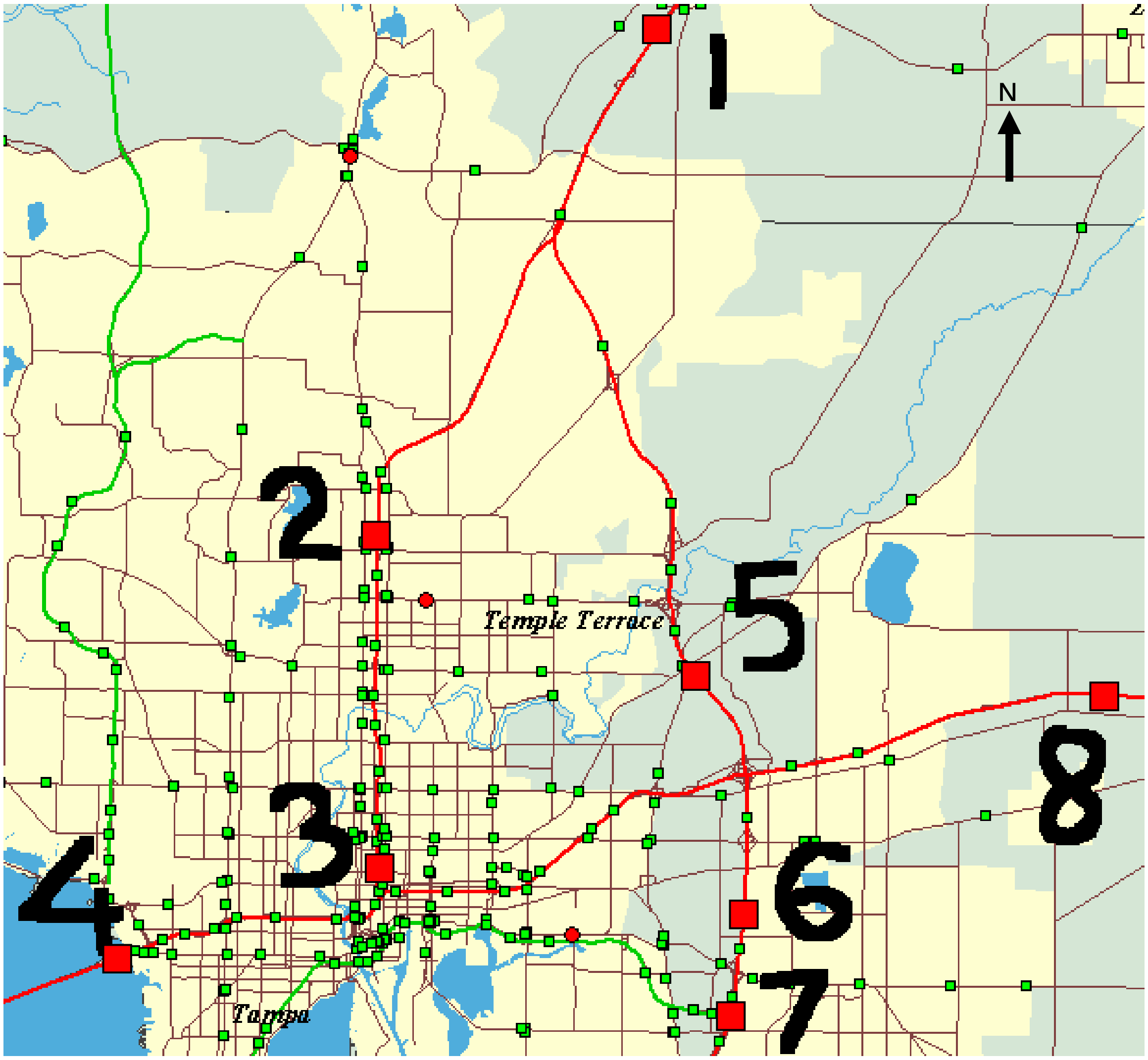


East part of the I-4 Orlando region chosen for the study



West part of the I-4 Orlando region chosen for the study

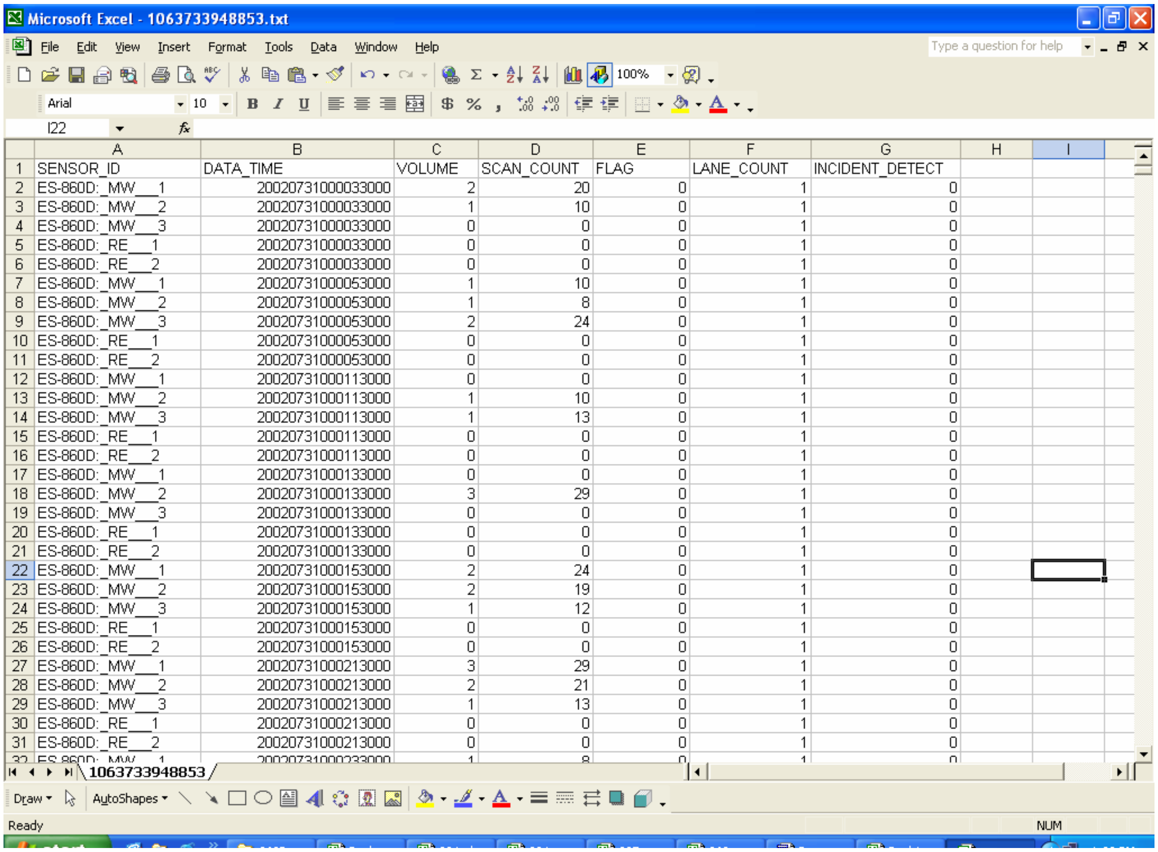
Initial Sites Selected for Data Collection in the Tampa Region



Tampa region in Florida chosen for study

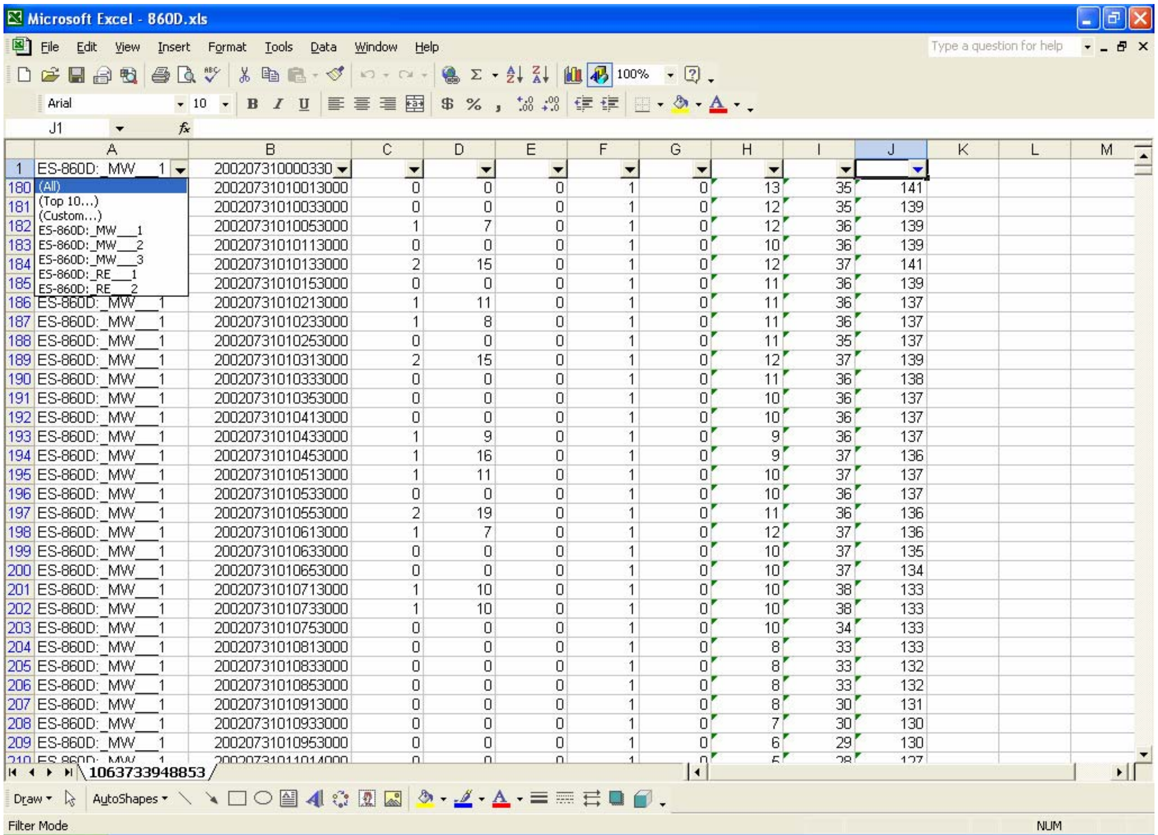
APPENDIX B

Typical spreadsheet obtained by importing data from a TDAD file



	A	B	C	D	E	F	G
1	SENSOR_ID	DATA_TIME	VOLUME	SCAN_COUNT	FLAG	LANE_COUNT	INCIDENT_DETECT
2	ES-860D:_MW__1	20020731000033000	2	20	0	1	0
3	ES-860D:_MW__2	20020731000033000	1	10	0	1	0
4	ES-860D:_MW__3	20020731000033000	0	0	0	1	0
5	ES-860D:_RE__1	20020731000033000	0	0	0	1	0
6	ES-860D:_RE__2	20020731000033000	0	0	0	1	0
7	ES-860D:_MW__1	20020731000053000	1	10	0	1	0
8	ES-860D:_MW__2	20020731000053000	1	8	0	1	0
9	ES-860D:_MW__3	20020731000053000	2	24	0	1	0
10	ES-860D:_RE__1	20020731000053000	0	0	0	1	0
11	ES-860D:_RE__2	20020731000053000	0	0	0	1	0
12	ES-860D:_MW__1	20020731000113000	0	0	0	1	0
13	ES-860D:_MW__2	20020731000113000	1	10	0	1	0
14	ES-860D:_MW__3	20020731000113000	1	13	0	1	0
15	ES-860D:_RE__1	20020731000113000	0	0	0	1	0
16	ES-860D:_RE__2	20020731000113000	0	0	0	1	0
17	ES-860D:_MW__1	20020731000133000	0	0	0	1	0
18	ES-860D:_MW__2	20020731000133000	3	29	0	1	0
19	ES-860D:_MW__3	20020731000133000	0	0	0	1	0
20	ES-860D:_RE__1	20020731000133000	0	0	0	1	0
21	ES-860D:_RE__2	20020731000133000	0	0	0	1	0
22	ES-860D:_MW__1	20020731000153000	2	24	0	1	0
23	ES-860D:_MW__2	20020731000153000	2	19	0	1	0
24	ES-860D:_MW__3	20020731000153000	1	12	0	1	0
25	ES-860D:_RE__1	20020731000153000	0	0	0	1	0
26	ES-860D:_RE__2	20020731000153000	0	0	0	1	0
27	ES-860D:_MW__1	20020731000213000	3	29	0	1	0
28	ES-860D:_MW__2	20020731000213000	2	21	0	1	0
29	ES-860D:_MW__3	20020731000213000	1	13	0	1	0
30	ES-860D:_RE__1	20020731000213000	0	0	0	1	0
31	ES-860D:_RE__2	20020731000213000	0	0	0	1	0
32	ES-860D:_MW__1	20020731000233000	1	8	0	1	0

Typical spreadsheet after calculation of required volumes



	A	B	C	D	E	F	G	H	I	J
1	ES-860D:_MW___1	200207310000330								
180		200207310100130000	0	0	0	1	0	13	35	141
181		200207310100330000	0	0	0	1	0	12	35	139
182		200207310100530000	1	7	0	1	0	12	36	139
183		200207310101130000	0	0	0	1	0	10	36	139
184		200207310101330000	2	15	0	1	0	12	37	141
185		200207310101530000	0	0	0	1	0	11	36	139
186	ES-860D:_MW___1	200207310102130000	1	11	0	1	0	11	36	137
187	ES-860D:_MW___1	200207310102330000	1	8	0	1	0	11	36	137
188	ES-860D:_MW___1	200207310102530000	0	0	0	1	0	11	35	137
189	ES-860D:_MW___1	200207310103130000	2	15	0	1	0	12	37	139
190	ES-860D:_MW___1	200207310103330000	0	0	0	1	0	11	36	138
191	ES-860D:_MW___1	200207310103530000	0	0	0	1	0	10	36	137
192	ES-860D:_MW___1	200207310104130000	0	0	0	1	0	10	36	137
193	ES-860D:_MW___1	200207310104330000	1	9	0	1	0	9	36	137
194	ES-860D:_MW___1	200207310104530000	1	16	0	1	0	9	37	136
195	ES-860D:_MW___1	200207310105130000	1	11	0	1	0	10	37	137
196	ES-860D:_MW___1	200207310105330000	0	0	0	1	0	10	36	137
197	ES-860D:_MW___1	200207310105530000	2	19	0	1	0	11	36	136
198	ES-860D:_MW___1	200207310106130000	1	7	0	1	0	12	37	136
199	ES-860D:_MW___1	200207310106330000	0	0	0	1	0	10	37	135
200	ES-860D:_MW___1	200207310106530000	0	0	0	1	0	10	37	134
201	ES-860D:_MW___1	200207310107130000	1	10	0	1	0	10	38	133
202	ES-860D:_MW___1	200207310107330000	1	10	0	1	0	10	38	133
203	ES-860D:_MW___1	200207310107530000	0	0	0	1	0	10	34	133
204	ES-860D:_MW___1	200207310108130000	0	0	0	1	0	8	33	133
205	ES-860D:_MW___1	200207310108330000	0	0	0	1	0	8	33	132
206	ES-860D:_MW___1	200207310108530000	0	0	0	1	0	8	33	132
207	ES-860D:_MW___1	200207310109130000	0	0	0	1	0	8	30	131
208	ES-860D:_MW___1	200207310109330000	0	0	0	1	0	7	30	130
209	ES-860D:_MW___1	200207310109530000	0	0	0	1	0	6	29	130

APPENDIX C

Loop detector data for the selected segments in the Seattle region

S1 - I90 Issaquah					
Date	**924 W**	**Date**	**928 W**	**Date**	**932 W**
7-Mar	1953	**7-Mar**	1987	**7-Mar**	2002
8-Mar	2022	**8-Mar**	1938	**8-Mar**	2074
3-May	2121	**10-May**	2016	**31-May**	1885
16-May	2043	**31-May**	1976	**20-Jun**	1999
17-May	2009	**6-Jun**	1976	**21-Jun**	1939
23-May	2169	**7-Jun**	1973	**25-Jun**	2013
30-May	2012	**11-Jun**	2021	**26-Jun**	2036
31-May	2012	**13-Jun**	2019	**27-Jun**	2025
6-Jun	1993	**14-Jun**	1895	**3-Jul**	2031
7-Jun	2027	**20-Jun**	1972	**9-Jul**	2040
11-Jun	2036	**21-Jun**	1915	**11-Jul**	1980
14-Jun	1913	**25-Jun**	1959	**12-Jul**	1944
19-Jun	2071	**26-Jun**	2016	**18-Jul**	2031
21-Jun	1999	**27-Jun**	1999	**19-Jul**	1807
25-Jun	1989	**11-Jul**	2041	**24-Jul**	2024
27-Jun	2020	**12-Jul**	1940	**25-Jul**	2005
12-Jul	1989	**15-Jul**	2003	**26-Jul**	1848
25-Jul	2005	**18-Jul**	2008	**30-Jul**	1869
26-Jul	1880	**24-Jul**	2020	**31-Jul**	1884
31-Jul	1897	**25-Jul**	1957	**16-Aug**	1908
15-Aug	2056	**26-Jul**	1843	**17-Aug**	1982
16-Aug	1891	**31-Jul**	2032	**25-Nov**	2028
17-Aug	2039	**16-Aug**	1977		
20-Sep	1991	**17-Aug**	1902		
25-Nov	1998	**25-Nov**	1959		
Mean	2005		1974		1971

Note: volume in units of veh/hr/lane

S2 - I90 Floating Bridge			
Date	**855 E**	**858 E**	**861 E**
7-Mar	1890	1915	2017
8-Mar	1934	1855	1858
4-Apr	2083	2037	2045
24-Apr	2057	2032	2033
10-May	2004	1972	1973
31-May	1955	1942	1834
6-Jun	1983	1948	1967
7-Jun	2072	2047	2063
11-Jun	2007	1967	1961
12-Jun	1987	1952	1961
13-Jun	2005	1977	1983
21-Jun	2080	2031	2041
25-Jun	1923	1932	1929
23-Jul	2072	2033	2041
24-Jul	1961	1932	1929
25-Jul	2025	1993	1976
26-Jul	1975	1936	1957
31-Jul	1834	1840	1840
14-Aug	2009	1980	1997
15-Aug	1857	1940	1918
16-Aug	1972	1844	1907
17-Aug	1951	1916	1883
20-Sep	1956	1920	1935
26-Sep	2007	1975	1980
25-Nov	1916	1857	2023
Mean	1981	1951	1962

S3 - I-5 South Seattle			
Date	79 N	80 N	81 N
15-Feb	2128	1980	1971
7-Mar	2077	1907	1887
8-Mar	2118	2018	1959
22-Mar	2194	2043	2042
19-Apr	2117	2001	2004
10-May	2297	2097	2099
17-May	2189	2028	2021
23-May	2195	2051	2078
24-May	2077	1959	1944
31-May	2144	1990	2003
14-Jun	2147	2012	2014
21-Jun	2084	1960	1967
25-Jun	2069	1931	1925
25-Jul	2128	1973	1656
26-Jul	2006	1910	1909
31-Jul	2029	1909	1894
1-Aug	2190	2019	2016
2-Aug	2031	1900	1900
9-Aug	2050	1959	1919
15-Aug	2042	2018	1960
16-Aug	2117	1968	1948
17-Aug	2058	2015	1865
20-Sep	2171	2015	1982
7-Oct		1968	1945
22-Nov	2091	1936	1923
25-Nov	2151	1994	1899
20-Dec	2052	1893	1876
Mean	2114	1983	1950

S4 - I-5 Canal Bridge		
Date	130	
	N	**S**
1-Mar	2028	2085
7-Mar	2084	2136
8-Mar	1974	1851
22-Mar	2065	2105
19-Apr	1981	2071
10-May	1980	2056
24-May	1888	1976
31-May	2001	2034
14-Jun	1931	2028
24-Jun	1991	1915
25-Jun	1913	1966
2-Jul	2008	1948
12-Jul	1898	1934
25-Jul	1981	1950
31-Jul	1925	1995
9-Aug	1855	1934
14-Aug	1871	1908
15-Aug		1947
16-Aug		1926
12-Sep	1910	1990
18-Sep	1969	2004
20-Sep	1959	1988
26-Sep	1924	2020
25-Nov	1975	1972
Mean	1960	1989

S5 - I-5 NE 155th St			
Date	**170**	**172**	
	N	**N**	**S**
7-Mar	2003	1986	2387
8-Mar	1886	1971	
22-Mar	1933	1950	2289
19-Apr	1974	2025	2376
3-May	1856	1880	2336
10-May	1997	2044	2383
24-May	1910	1944	2331
31-May	1960	1908	2452
7-Jun	1878	1868	2393
14-Jun	1924	1940	2412
25-Jun	1895	1865	2368
12-Jul	1913	1922	2327
25-Jul	1972	1991	2313
26-Jul	1958	1990	2224
31-Jul	1906	1928	2391
2-Aug	1901	1921	2372
3-Aug	1751	1772	2041
9-Aug	1836	1852	2419
15-Aug	1915	1907	2395
16-Aug	1804	1807	2289
20-Sep		1917	2109
25-Nov		1922	2252
Mean	1826	1847	2327

S6 - I-5 Lane Drop		
Date	177	
	N	S
7-Mar	2443	2317
8-Mar	2448	
22-Mar	2449	2196
5-Apr	2473	2296
19-Apr	2499	2288
10-May	2459	2265
16-May	2440	2344
17-May	2469	
31-May	2363	2264
7-Jun		2261
14-Jun	2456	2288
25-Jun	2277	2279
12-Jul	2391	2204
25-Jul	2424	2264
26-Jul	2464	2141
29-Jul	2467	2140
30-Jul	2468	
31-Jul	2381	2356
2-Aug	2480	2201
9-Aug	2349	2215
15-Aug	2285	2256
16-Aug	2349	2113
20-Sep	2396	2123
25-Nov	2399	2127
Mean	2419	2235

50 Highest Recorded Hourly Volumes for each permanent traffic monitoring site

Orlando Region

O1

County: **77**
Site: **343**
Description: SR-400/I-4,1.6 MI E OF SR-434,SEMINOLE CO.
Location: 77160000 **Milepost:** 5.14

East (pm)			West (am)		
Date	**Volume**	**Per lane**	**Date**	**Volume**	**Per lane**
26-Nov	5197	1732	31-Oct	5484	1828
5-Aug	5171	1724	30-Oct	5465	1822
25-Feb	5170	1723	29-Oct	5457	1819
27-Aug	5165	1722	5-Nov	5453	1818
10-Oct	5155	1718	13-Nov	5397	1799
4-Dec	5142	1714	17-Apr	5395	1798
30-Sep	5137	1712	30-Apr	5391	1797
20-Nov	5134	1711	4-Nov	5391	1797
12-Jun	5134	1711	6-Nov	5390	1797
14-Nov	5131	1710	22-May	5376	1792
23-Sep	5130	1710	11-Dec	5375	1792
8-Feb	5127	1709	25-Sep	5375	1792
11-Feb	5114	1705	16-Apr	5370	1790
25-Nov	5107	1702	27-Feb	5369	1790
10-Dec	5100	1700	9-Apr	5367	1789
16-Dec	5100	1700	12-Nov	5360	1787
26-Sep	5093	1698	27-Mar	5345	1782
15-Oct	5086	1695	13-May	5342	1781
7-Jan	5086	1695	14-Feb	5341	1780
12-Sep	5071	1690	19-Nov	5338	1779
5-Mar	5069	1690	7-May	5337	1779
19-Feb	5067	1689	4-Dec	5325	1775
8-Oct	5067	1689	22-Nov	5316	1772
7-Oct	5067	1689	21-Oct	5312	1771
29-Oct	5059	1686	14-May	5305	1768
5-Dec	5056	1685	26-Feb	5301	1767
12-Feb	5052	1684	28-Mar	5300	1767
22-Aug	5052	1684	22-Apr	5297	1766
18-Sep	5051	1684	2-Dec	5296	1765
23-Oct	5046	1682	18-Jan	5295	1765
14-Feb	5045	1682	19-Apr	5295	1765
4-Mar	5045	1682	26-Aug	5294	1765
7-Nov	5041	1680	27-Aug	5282	1761

East (pm)		
Date	**Volume**	**Per lane**
1-Jul	5040	1680
21-Nov	5039	1680
24-Jan	5039	1680
28-Feb	5039	1680
18-Nov	5038	1679
19-Sep	5025	1675
18-Apr	5024	1675
17-Dec	5020	1673
3-Jan	5018	1673
23-May	5014	1671
11-Dec	5011	1670
7-Mar	5010	1670
21-May	5005	1668
14-May	5000	1667
2-Dec	4999	1666
6-Dec	4999	1666
23-Jan	4998	1666
Average =	5072	1691
Std Dev =	53.41	17.80

West (am)		
Date	**Volume**	**Per lane**
20-Nov	5276	1759
1-May	5262	1754
23-May	5252	1751
3-Apr	5248	1749
15-Apr	5248	1749
20-Feb	5246	1749
21-Mar	5245	1748
18-Mar	5245	1748
18-Nov	5245	1748
13-Feb	5241	1747
8-May	5241	1747
5-Mar	5240	1747
26-Mar	5235	1745
21-Feb	5233	1744
15-May	5233	1744
21-May	5227	1742
1-Apr	5225	1742
	5318	1773
	69.66	23.22

O2

County: **75**
Station: **0196**
Description: SR-400/I-4,@SR-408 OVERPASS,ORANGE CO.
Location: 75280000 **Milepost:** 17.06

West (pm)		
Date	**Volume**	**Per lane**
20-May	7448	2483
4-Feb	7429	2476
21-May	7420	2473
16-Apr	7413	2471
5-Dec	7410	2470
5-Feb	7408	2469
17-Dec	7395	2465
20-Nov	7388	2463
9-Apr	7365	2455
15-May	7352	2451
2-Oct	7336	2445
16-Jan	7335	2445
8-Oct	7301	2434
4-Mar	7290	2430
4-Dec	7287	2429
31-Dec	7280	2427
14-May	7280	2427
6-Dec	7279	2426
4-Nov	7273	2424
18-Jun	7265	2422
13-Feb	7259	2420
30-Jan	7258	2419
9-Jan	7257	2419
12-Nov	7244	2415
19-Nov	7243	2414
3-Apr	7241	2414
26-Feb	7240	2413
22-Jan	7234	2411
14-Aug	7233	2411
28-Feb	7230	2410
11-Sep	7226	2409
27-Feb	7224	2408
18-Dec	7217	2406
8-Aug	7208	2403
21-Nov	7208	2403
15-Oct	7208	2403
22-Aug	7200	2400
8-Jul	7199	2400
25-Nov	7199	2400
4-Jun	7196	2399
2-Dec	7192	2397

West (pm)		
Date	**Volume**	**Per lane**
10-Dec	7192	2397
27-Aug	7191	2397
19-Feb	7190	2397
11-Dec	7188	2396
22-May	7185	2395
7-Jan	7182	2394
1-Oct	7179	2393
26-Nov	7178	2393
8-Jan	7174	2391

Average =	7267	2422
Std Dev =	79.92	26.64

O3

County: **75**
Station: **0130**
Description: SR-400/I-4,0.8 MI S OF SR-482,ORANGE CO.
Location: 75280000 **Milepost:** 7.50

East (pm)			West (am)		
Date	**Volume**	**Per lane**	**Date**	**Volume**	**Per lane**
23-Jan	5340	1780	8-Feb	5524	1841
4-Apr	5307	1769	8-Mar	5517	1839
9-May	5254	1751	3-Apr	5512	1837
17-May	5248	1749	27-Mar	5495	1832
17-Apr	5229	1743	4-Apr	5491	1830
26-Mar	5229	1743	28-Mar	5487	1829
5-Apr	5159	1720	2-Apr	5457	1819
3-Apr	5156	1719	18-Jan	5453	1818
25-Jan	5136	1712	26-Mar	5453	1818
5-Mar	5136	1712	15-Mar	5390	1797
27-Mar	5134	1711	5-Apr	5378	1793
19-Mar	5132	1711	10-Mar	5361	1787
2-Apr	5127	1709	1-Mar	5358	1786
26-Apr	5123	1708	14-Mar	5302	1767
8-May	5112	1704	7-Mar	5293	1764
4-Jan	5111	1704	21-Mar	5287	1762
6-Mar	5101	1700	12-Feb	5259	1753
23-May	5070	1690	1-Apr	5258	1753
7-Mar	5067	1689	24-May	5225	1742
14-Feb	5061	1687	24-Mar	5218	1739
28-Mar	5060	1687	21-Feb	5205	1735
30-Apr	5055	1685	11-Feb	5201	1734
22-May	5055	1685	29-Mar	5198	1733
10-May	5044	1681	22-Mar	5192	1731
9-Apr	5039	1680	17-Feb	5183	1728
16-May	5034	1678	20-Jun	5180	1727
13-Mar	5033	1678	15-Feb	5173	1724
12-Feb	5031	1677	25-Mar	5166	1722
21-May	5029	1676	27-Apr	5159	1720
16-Feb	5027	1676	9-Mar	5155	1718
29-Mar	5026	1675	1-Feb	5151	1717
19-Apr	5025	1675	20-Feb	5143	1714
16-Jan	5017	1672	20-Mar	5133	1711
24-Jan	5017	1672	23-May	5130	1710
3-Jan	5011	1670	12-Mar	5120	1707
2-May	5010	1670	9-Feb	5104	1701
15-May	5001	1667	22-May	5096	1699
25-Apr	4999	1666	19-Mar	5086	1695
8-Feb	4988	1663	13-Mar	5083	1694
25-Mar	4984	1661	25-Jan	5080	1693
24-May	4974	1658	17-Apr	5072	1691

East (pm)		
Date	**Volume**	**Per lane**
20-Feb	4961	1654
12-Mar	4959	1653
16-Apr	4934	1645
14-Mar	4929	1643
24-Feb	4917	1639
7-May	4908	1636
10-Apr	4906	1635
19-Feb	4903	1634
28-Apr	4895	1632
Average =	5060	1687
Std Dev =	104.99	35.00

West (am)		
Date	**Volume**	**Per lane**
30-Mar	5071	1690
14-Feb	5049	1683
12-Jun	5029	1676
11-Apr	5024	1675
16-Mar	5022	1674
19-Apr	5020	1673
7-Jun	5019	1673
21-Jun	5017	1672
26-Apr	5013	1671
	5220	1740
	160.01	53.34

O4

County: **92**
Site: **303**
Description: ON I-4,0.5 MI SW OF ORANGE CO LINE,OSCEOLA CO.
Location: 92130000 **Milepost:** 7.34

East (pm)			West (am)		
Date	**Volume**	**Per Lane**	**Date**	**Volume**	**Per Lane**
15-May	4923	1641	30-Nov	5066	1689
26-Dec	4824	1608	26-Dec	4945	1648
26-Dec	4806	1602	27-Dec	4938	1646
23-Dec	4763	1588	26-Dec	4836	1612
23-Dec	4718	1573	28-Dec	4810	1603
29-Dec	4712	1571	30-Dec	4801	1600
27-Dec	4711	1570	29-Nov	4784	1595
28-Dec	4680	1560	29-Dec	4768	1589
30-Dec	4679	1560	30-Dec	4762	1587
15-May	4662	1554	29-Dec	4760	1587
27-Mar	4619	1540	27-Dec	4702	1567
27-Dec	4616	1539	4-Apr	4700	1567
27-Dec	4585	1528	17-Aug	4687	1562
27-Dec	4552	1517	30-Mar	4682	1561
31-Dec	4536	1512	19-Oct	4681	1560
28-Dec	4522	1507	28-Dec	4668	1556
29-Dec	4518	1506	3-Apr	4659	1553
31-Dec	4497	1499	3-Aug	4622	1541
27-Nov	4446	1482	5-Apr	4619	1540
31-Dec	4438	1479	28-Dec	4584	1528
14-Mar	4438	1479	15-Feb	4578	1526
30-Dec	4437	1479	10-Aug	4570	1523
8-Mar	4424	1475	21-Feb	4565	1522
30-Dec	4419	1473	15-Mar	4555	1518
8-Mar	4414	1471	20-Jul	4551	1517
29-Dec	4414	1471	25-May	4544	1515
27-Dec	4406	1469	23-Dec	4542	1514
3-Apr	4395	1465	3-Jul	4542	1514
2-Apr	4391	1464	1-Apr	4540	1513
27-Mar	4388	1463	30-Dec	4523	1508
7-Mar	4384	1461	23-Dec	4517	1506
15-Mar	4383	1461	19-Jul	4515	1505
26-Dec	4380	1460	6-Mar	4512	1504
21-Feb	4378	1459	16-Mar	4507	1502
23-Dec	4362	1454	14-Mar	4504	1501
15-Mar	4354	1451	24-Jul	4503	1501
26-Mar	4338	1446	29-Mar	4502	1501
1-Nov	4322	1441	27-Dec	4499	1500
27-Mar	4321	1440	31-Dec	4496	1499
4-Apr	4312	1437	25-Mar	4492	1497
23-Dec	4307	1436	7-Mar	4490	1497

East (pm)		
Date	**Volume**	**Per Lane**
28-Dec	4302	1434
27-Nov	4300	1433
26-Jul	4300	1433
8-Nov	4290	1430
1-Mar	4282	1427
28-Mar	4277	1426
12-Jul	4265	1422
25-Jul	4262	1421
5-Apr	4261	1420
Average =	4466	1489
Std Dev =	170.95	56.98

West (am)		
Date	**Volume**	**Per Lane**
23-Mar	4489	1496
26-May	4482	1494
6-Apr	4457	1486
8-Mar	4455	1485
20-Feb	4454	1485
13-Apr	4453	1484
20-Mar	4452	1484
2-Apr	4451	1484
28-Jul	4451	1484
	4605	1535
	147.12	49.04

Tampa Region

T1

County: 14
Site: 190
Description: SR-93/I-75,0.6 MI S OF SR-54,PASCO CO.
Location: 14140000 **Milepost:** 4.50

North			South		
Date	Volume	Per lane	Date	Volume	Per lane
15-Nov	3897	1949	11-Mar	3726	1863
30-Nov	3848	1924	28-Feb	3657	1829
25-Oct	3797	1899	28-Dec	3637	1819
30-Nov	3791	1896	28-Dec	3635	1818
25-Oct	3762	1881	1-Mar	3610	1805
20-Dec	3739	1870	22-Mar	3569	1785
22-Nov	3727	1864	29-Dec	3567	1784
15-Nov	3695	1848	28-Dec	3553	1777
1-Nov	3687	1844	27-Dec	3550	1775
5-Apr	3683	1842	29-Dec	3549	1775
22-Nov	3676	1838	29-Dec	3466	1733
30-Aug	3658	1829	30-Mar	3461	1731
30-Aug	3641	1821	1-Dec	3441	1721
15-Feb	3640	1820	29-Dec	3439	1720
8-Nov	3640	1820	27-Dec	3437	1719
1-Mar	3631	1816	28-Dec	3416	1708
30-Mar	3630	1815	28-Dec	3408	1704
1-Nov	3585	1793	27-Dec	3403	1702
30-Mar	3582	1791	23-Dec	3395	1698
27-Nov	3580	1790	29-Dec	3392	1696
8-Nov	3576	1788	1-Dec	3389	1695
20-Sep	3568	1784	31-Mar	3383	1692
29-Mar	3566	1783	30-Mar	3366	1683
8-Mar	3563	1782	30-Mar	3363	1682
5-Apr	3555	1778	30-Dec	3358	1679
30-Aug	3544	1772	23-Mar	3340	1670
1-Dec	3539	1770	28-Dec	3340	1670
27-Nov	3538	1769	30-Dec	3338	1669
18-Jan	3533	1767	23-Mar	3315	1658
15-Nov	3521	1761	27-Nov	3310	1655
5-Apr	3521	1761	27-Nov	3309	1655
29-Mar	3520	1760	23-Dec	3306	1653
8-Nov	3514	1757	28-Mar	3300	1650
5-Apr	3513	1757	28-Dec	3300	1650
23-Mar	3507	1754	29-Mar	3297	1649
29-Mar	3499	1750	29-Mar	3285	1643
27-Nov	3497	1749	27-Dec	3285	1643
4-Oct	3473	1737	30-Dec	3278	1639
3-Jul	3471	1736	26-Dec	3259	1630

North		
Date	**Volume**	**Per lane**
19-Jul	3458	1729
15-Mar	3456	1728
31-Mar	3453	1727
18-Jan	3451	1726
11-Jan	3439	1720
14-Jun	3434	1717
27-Sep	3430	1715
28-Jun	3429	1715
26-Nov	3428	1714
29-Mar	3426	1713
25-Oct	3424	1712
Average =	3575	1787
Std Dev =	119.49	59.74

South		
Date	**Volume**	**Per lane**
26-Dec	3254	1627
27-Dec	3252	1626
30-Mar	3238	1619
26-Dec	3229	1615
30-Dec	3224	1612
30-Nov	3206	1603
27-Nov	3202	1601
27-Dec	3190	1595
23-Mar	3189	1595
1-Dec	3187	1594
27-Dec	3186	1593
	3376	1688
	140.15	70.07

T2

County: **10**
Site: **110**
Description: SR93/I275,1.3 MI E OF HOWARD FRANKLIN BR,HILLS. CO
Location: 10190000 **Milepost:** 1.34

East (am)		
Date	**Volume**	**Per lane**
26-Apr	6316	2105
29-Jan	5998	1999
1-Apr	5988	1996
27-Mar	5871	1957
2-Apr	5809	1936
4-Mar	5799	1933
20-Feb	5792	1931
5-Mar	5791	1930
24-Apr	5788	1929
4-Apr	5759	1920
27-Feb	5756	1919
25-Mar	5741	1914
19-Mar	5730	1910
12-Feb	5702	1901
5-Apr	5694	1898
21-Feb	5693	1898
18-Apr	5686	1895
28-Mar	5684	1895
13-Feb	5679	1893
21-Mar	5658	1886
14-Mar	5654	1885
3-Apr	5650	1883
25-Apr	5607	1869
1-Mar	5593	1864
16-Apr	5592	1864
10-Apr	5590	1863
23-Jul	5587	1862
4-Nov	5577	1859
30-Jul	5565	1855
29-Aug	5550	1850
26-Mar	5546	1849
15-Feb	5545	1848
7-Mar	5542	1847
4-Dec	5539	1846
23-Sep	5524	1841
9-Apr	5502	1834
29-Apr	5495	1832
25-Feb	5479	1826
25-Jun	5476	1825
13-Mar	5447	1816
14-Feb	5423	1808

West (pm)		
Date	**Volume**	**Per lane**
12-Feb	6012	2004
27-Feb	5967	1989
13-Mar	5940	1980
10-Jun	5940	1980
1-Mar	5933	1978
3-May	5928	1976
25-Apr	5926	1975
21-Feb	5921	1974
2-Apr	5915	1972
13-Feb	5915	1972
17-Jan	5878	1959
4-Mar	5877	1959
18-Apr	5850	1950
11-Jun	5840	1947
25-Jan	5839	1946
7-Mar	5837	1946
20-Feb	5832	1944
21-Nov	5829	1943
28-Feb	5820	1940
5-Mar	5818	1939
9-Apr	5799	1933
7-Jun	5793	1931
3-Jan	5777	1926
14-Feb	5771	1924
1-Mar	5762	1921
8-Jul	5762	1921
12-Jun	5756	1919
22-Mar	5753	1918
18-Sep	5751	1917
25-Feb	5748	1916
6-Mar	5740	1913
15-Feb	5739	1913
9-Jul	5736	1912
26-Nov	5725	1908
5-Dec	5722	1907
4-Jun	5718	1906
13-Jun	5717	1906
5-Feb	5716	1905
19-Apr	5714	1905
26-Feb	5706	1902
30-Jul	5706	1902

East (am)		
Date	**Volume**	**Per lane**
23-Dec	5382	1794
19-Dec	5238	1746
27-Mar	5168	1723
4-Apr	5129	1710
8-Aug	5126	1709
28-Mar	5125	1708
21-Nov	5107	1702
20-Mar	5083	1694
25-Apr	5053	1684
Average =	5577	1859
Std Dev =	254.71	84.90

West (pm)		
Date	**Volume**	**Per lane**
10-Apr	5706	1902
25-Nov	5701	1900
14-Nov	5698	1899
3-Jun	5695	1898
8-Feb	5695	1898
6-Feb	5692	1897
28-Aug	5689	1896
17-Sep	5687	1896
21-Mar	5680	1893
	5793	1931
	91.16	30.39

T3

County: **10**
Station: **0194**
Description: SR-93A/I-75,0.6 MI S OF US-301,HILLSBOROUGH CO.
Location: 10075000 **Milepost:** 29.38

North (pm)		
Date	**Volume**	**Per lane**
7-Nov	5670	1890
23-Sep	5600	1867
27-Sep	5546	1849
6-Sep	5520	1840
14-Nov	5509	1836
26-Aug	5488	1829
25-Sep	5484	1828
18-Nov	5480	1827
17-Sep	5478	1826
28-Feb	5476	1825
28-Aug	5444	1815
18-Sep	5440	1813
3-Sep	5434	1811
7-Oct	5433	1811
13-Nov	5413	1804
20-Feb	5402	1801
29-Aug	5396	1799
4-Sep	5393	1798
15-Aug	5391	1797
22-Nov	5388	1796
8-Nov	5382	1794
20-Nov	5379	1793
9-Sep	5378	1793
1-Aug	5372	1791
27-Aug	5366	1789
4-Mar	5354	1785
19-Sep	5348	1783
24-Oct	5345	1782
21-Mar	5344	1781
30-Aug	5337	1779
6-Nov	5336	1779
25-Oct	5334	1778
23-Oct	5320	1773
5-Mar	5315	1772
6-Mar	5308	1769
1-Oct	5302	1767
4-Oct	5295	1765
20-Sep	5292	1764
20-Mar	5288	1763
22-Aug	5275	1758
13-Aug	5258	1753

South (am)		
Date	**Volume**	**Per lane**
21-Feb	5876	1959
27-Feb	5874	1958
6-Nov	5856	1952
21-Nov	5848	1949
7-Mar	5840	1947
20-Feb	5838	1946
25-Feb	5829	1943
21-May	5809	1936
22-May	5809	1936
24-Apr	5787	1929
9-May	5785	1928
28-Aug	5784	1928
21-Mar	5782	1927
28-Jan	5773	1924
8-May	5772	1924
26-Feb	5770	1923
2-Apr	5765	1922
3-Oct	5762	1921
23-Sep	5761	1920
30-Oct	5758	1919
15-May	5756	1919
1-May	5754	1918
16-Apr	5742	1914
31-Oct	5736	1912
5-Sep	5733	1911
6-Mar	5732	1911
25-Apr	5730	1910
9-Sep	5730	1910
4-Mar	5729	1910
20-Mar	5728	1909
7-May	5728	1909
20-May	5727	1909
17-Apr	5723	1908
30-Apr	5723	1908
28-May	5721	1907
20-Aug	5720	1907
16-May	5716	1905
3-Apr	5709	1903
12-Nov	5703	1901
8-Apr	5698	1899
11-Apr	5697	1899

North (pm)		
Date	**Volume**	**Per lane**
13-Sep	5258	1753
1-Mar	5252	1751
16-Aug	5249	1750
21-Feb	5247	1749
30-Sep	5246	1749
23-Aug	5231	1744
12-Aug	5229	1743
15-Feb	5222	1741
26-Sep	5218	1739
Average =	5369	1790
Std Dev =	102.68	34.23

South (am)		
Date	**Volume**	**Per lane**
23-Apr	5694	1898
30-Sep	5693	1898
8-Mar	5690	1897
18-Mar	5687	1896
1-Mar	5685	1895
6-May	5684	1895
23-May	5684	1895
29-Aug	5683	1894
20-Jun	5676	1892
	5750	1917
	53.86	17.95

T4

County: **10**
Station: **9926**
Description: SR-93A/I-75,1.25 MI N OF SR-60,TAMPA,HILLS CO.
Location: 10075000 **Milepost:** 24.06

North (am)			South (pm)		
Date	**Volume**	**Per lane**	**Date**	**Volume**	**Per lane**
1-Apr	6020	2007	26-Nov	5705	1902
5-Apr	5975	1992	22-Nov	5637	1879
2-Apr	5876	1959	22-Mar	5622	1874
18-Sep	5853	1951	5-Apr	5557	1852
23-Oct	5808	1936	29-Mar	5550	1850
30-Oct	5767	1922	2-Apr	5504	1835
20-Nov	5739	1913	13-Nov	5495	1832
15-Jul	5738	1913	3-May	5479	1826
11-Apr	5692	1897	18-Apr	5461	1820
13-Nov	5676	1892	4-Apr	5460	1820
19-Mar	5675	1892	23-Aug	5449	1816
2-Oct	5671	1890	30-Oct	5443	1814
4-Apr	5656	1885	15-Mar	5439	1813
28-May	5652	1884	14-Jun	5422	1807
26-Aug	5652	1884	30-Aug	5420	1807
6-Oct	5648	1883	17-May	5385	1795
4-Dec	5647	1882	11-Apr	5375	1792
18-Mar	5640	1880	12-Jul	5354	1785
11-Dec	5632	1877	1-Apr	5354	1785
12-Nov	5628	1876	16-Oct	5344	1781
1-May	5614	1871	20-Sep	5331	1777
22-May	5596	1865	25-Jun	5330	1777
22-Mar	5572	1857	18-Mar	5321	1774
19-Dec	5563	1854	26-Aug	5298	1766
1-Oct	5557	1852	24-May	5293	1764
21-May	5544	1848	7-May	5291	1764
15-Nov	5537	1846	19-Mar	5288	1763
8-May	5532	1844	22-Apr	5286	1762
26-Nov	5527	1842	26-Mar	5271	1757
19-Nov	5524	1841	1-May	5267	1756
20-Sep	5516	1839	18-Sep	5266	1755
14-Jun	5514	1838	24-Apr	5235	1745
23-Jul	5498	1833	7-Jun	5233	1744
12-Jun	5497	1832	28-May	5218	1739
24-Sep	5475	1825	8-May	5178	1726
10-Sep	5468	1823	17-Apr	5168	1723
22-Nov	5463	1821	25-Apr	5138	1713
28-Dec	5443	1814	23-Oct	5129	1710
4-Jun	5426	1809	15-Jul	5096	1699
30-Aug	5421	1807	5-Jun	5096	1699
12-Jul	5418	1806	22-May	5093	1698

North (am)		
Date	**Volume**	**Per lane**
5-Jun	5402	1801
23-Aug	5392	1797
18-Dec	5392	1797
Average =	5603	1868
Std Dev =	150.03	50.01

South (pm)		
Date	**Volume**	**Per lane**
12-Jun	5091	1697
10-Sep	5088	1696
21-May	5086	1695
23-Jul	5080	1693
29-Apr	5076	1692
4-Jun	5059	1686
22-Jul	5053	1684
26-Jul	5038	1679
15-Apr	5028	1676
	5298	1766
	175.81	58.60