Development of Speed Models for Improving Travel Forecasting and Highway Performance Evaluation

Final Report

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by

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16. Abstract

Most travel forecasting models require the use of appropriate values of free flow speed and practical capacity as well as appropriate forms of volume delay functions and their calibrated parameters. One-year field data of hourly volumes and hourly speeds from permanent traffic count stations maintained by the Florida Department of Transportation were used to determine the prevailing free flow speeds and practical capacities of highway facilities categorized by area type and facility type. The area types were grouped as rural, urban, and residential while facilities were categorized as uninterrupted flow facilities or interrupted flow facilities. Field data of speed and volume were plotted and used to fit and calibrate four volume delay functions (VDFs) – that is, Modified BPR, Modified Davidson, Akcelik and Conical functions. The calibrated VDFs were then tested into the FSUTMS using the Orlando Urban Area Transportation Study (OUATS) region. The results of free flow speed and practical capacity estimation produced values that were in line with those found in published literature although capacity on rural freeway segments could not be estimated because of uncongested operations on those segments. Updating the FSUTMS SPEEDCAP lookup table is recommended. Although the testing of the volume delay functions on OAUTS region revealed a reasonable match between estimated counts and field counts for Modified BPR and Modified Davidson volume delay functions, the results need to be validated further due to lack of permanent counts stations in the test region.

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EXECUTIVE SUMMARY

Background

Travel forecasting utilizes models to predict future travel demand based on present status of the transportation system and its use. The challenge faced by a transportation modeler is the development, validation, and calibration of existing models to ensure that the predicted travel demand is as close to reality as possible. Nested within travel forecasting models is an important step known as traffic assignment. In the traffic assignment step, the decay in travel time is predicted based on the level of traffic volume using models commonly referred to as volume-delay functions (also known as link-congestion functions). Travel time, however, is related to speed and therefore volume-delay functions tend to model speed as a function of traffic demand using roadway-specific values particularly free flow speed and practical capacity as non-changing quantities.

A number of theoretical volume-delay functions have been proposed with some gaining wide practical applications. The major practical volume-delay functions (VDFs) include Bureau of Public Roads (PBR) function, the Davidson function, the Conical function, and the Akcelik function. The predictive accuracy of these models is heavily dependent on accurately specifying the free flow speed and practical capacity of the highway under study. In addition, properly calibrated parameters of these models (α, β, etc.) are of paramount importance in ensuring realistic results.

The heterogeneity of highway networks in Florida poses challenges in determining appropriate values of free flow speed, practical capacity, and VDFs parameters since a set of these values applicable for one highway segment type might not be applicable for another highway segment type. Driver behavior and driver expectation have been documented to differ by area type – urban, rural, or residential – and by facility type – limited-access, non-limited access, etc. Transportation planners have generally responded to this challenge by first categorizing the highway network by facility type and area type, then creating a lookup table for different facility-area type combinations. Appropriate free flow speed, practical capacity, and VDF parameters are thus fixed by area type and facility type in these lookup tables.

The lookup tables such as those used in the Florida Standard Urban Transportation Modeling Structure (FSUTMS) for statewide modeling were created using limited field data. Generally, as data and more information become available, the lookup tables are updated together with the volume-delay functions' parameters. This research was prompted by recent developments in data acquisition, particularly by private vendors who have been collecting data to provide travelers with congestion information especially in urban areas. In addition, improvements in point detection technologies are enabling highway agencies to monitor more highways of different facility-area type combinations, thus providing more data that can be used to improve travel forecasting models.

Objectives

The overall goal of this project was to improve travel forecasting and highway performance evaluation in the State of Florida. The Systems Planning Office of the Florida Department of Transportation developed and maintains a computer-based modeling package known as the Florida Standard Urban Transportation Modeling Structure (FSUTMS). One of the missions of the System Planning Office is to provide support for all FSUTMS users by continually improving FSUTMS to incorporate new methodologies and improvements of existing models. Consistent with this mission, the main objective of this research project was to evaluate the efficacy of various traffic assignment models and test their parameters using empirical data collected from traffic monitoring sites operated by the Florida Department of Transportation and, where applicable, utilize data collected by private vendors for traveler information purposes. With the realization that the major inputs of a traffic assignment model is free flow speed and practical capacity, this research was also aimed at updating these traffic parameters using area wide data covering most area types and facility types.

Findings and Conclusions

Field data were collected from roadways of different area type/facility type combinations. The area types contained in the FSUTMS look-up tables were collapsed into three categories – that is, rural, urban, and residential. Data collection and analysis were further subdivided into uninterrupted flow and interrupted flow facilities due to the fact that speed-volume relationship are not the same on these facilities. Vehicle speed and volume data from over 76 permanent count stations were acquired for a period beginning July 1, 2010 and ending June 30, 2011. In addition, data acquired from the permanent count stations were supplemented with active field data collected from 20 sites in the City of Tallahassee for use in the analysis of interrupted flow facilities. Following screening and validation of the collected field data, analysis of the data was carried out with the following aims:

(a) estimating free flow speeds,
(b) estimating practical capacities,
(c) scatter plotting of speed vs. flow,
(d) fitting volume delay functions to the speed-flow scatter plots,
(e) developing optimum volume delay function parameters, and
(f) testing the volume delay functions and their parameters in the FSUTMS.

<u>Free Flow Speeds</u>

Free flow speed, a major input into volume delay functions, were analyzed using one-year data from 76 permanent count sites installed on uninterrupted flow facilities. The analysis involved determining which vehicles were free flowing and what percentile speed was appropriate in the estimation of free flow speed. Traffic was considered to be free-flowing if the density corresponding to low volume (≤ 200 passenger cars per hour per lane) was ≤ 5 passenger cars per mile per lane. The hourly space mean speeds in the one-year data meeting these conditions were ranked in ascending order and the 85^{th} percentile speed was calculated to estimate practical free flow speed, the results of which are shown in Table 1 for both non-toll and toll limited access highways. In addition to area type, the free flow speeds are further categorized by speed limit and number of lanes. Because of lack of permanent count stations on some segments with certain speed limits, a speed prediction model was developed to predict free flow speed in those segments. The predicted free flow speeds are shown in red color. Overall, the field data results confirm the influence of posted speed limit and area type on free flow speed. Table 1 shows that rural segments had higher free flow speeds while segments with high posted speed limit experienced higher free flow speeds.

Table 1. Recommended Free Flow Speeds

Area type	Non-toll Uninterrupted Flow Facilities				Toll Facilities			
	2-Lanes	3-Lanes	4-Lanes	5-Lanes	2-Lanes	3-Lanes	4-Lanes	5-Lanes
Speed Limit = 55 MPH								
Urban	64	65	64	70	65	67	63	69
Residential	63	65	67	67	73	65	67	67
Rural	67	68	68	69	67	68	68	69
Speed Limit = 60 MPH								
Urban	66	66	67	67	69	66	67	67
Residential	68	68	69	69	68	68	69	69
Rural	69	70	70	71	69	70	70	71
Speed Limit = 65 MPH								
Urban	68	67	69	69	68	76	69	69
Residential	70	72	71	70	70	68	71	73
Rural	71	72	72	73	77	72	72	73
Speed Limit = 70 MPH								
Urban	70	70	71	71	70	70	71	71
Residential	73	74	72	73	79	78	77	73
Rural	74	74	74	75	74	78	74	75

Practical Capacities

The hourly volumes on uninterrupted flow facilities were analyzed by first converting the hourly vehicular flows into passenger car equivalents. The practical capacities of the non-toll and toll facilities were determined by area type – that is, urban, residential, and rural areas. Although the maximum hourly flow rates were determined from the field data, the practical capacity was defined as the 99th percentile flow to reduce chances of outliers. The results of the analysis are shown in Table 2. Practical capacities were undeterminable for some desired toll segments because such segments either did not exist or there were no permanent count stations installed in them. The results showed that hourly volumes on rural Florida freeways were below normal ranging from 937 passenger cars per hour per lane on 4-lane rural freeways to 1,362 passenger cars per hour per lane on 6-lane rural freeways. However, for the urban and residential segments for which data were available, the practical capacities were in line with those found in published literature.

Table 2. Observed 99th Percentile Flows on Non-toll and Toll Uninterrupted Flow Facilities

Area Type	Number of Lanes	99th Percentile Flow (pcphpln)	
		Freeways	Tolls
Urban	2	1,504	1748
Urban	3	2,056	1938
Urban	4	1,410	-
Urban	5	1,569	-
Residential	2	1,277	2074
Residential	3	1,765	-
Residential	4	2,209	-
Residential	5	1,641	-
Rural	2	937	1772

Plotting Speed vs. Volume Using Observed Field Data

Scatter plots of speed versus flow were produced using field data from telemetered traffic monitoring sites. These plots depict the relationship between hourly speeds and hourly volumes on a highway segment. The majority of speed-flow scatter plots plotted using data from various monitoring sites had a parabolic relationship consistent with those prevalent in literature. However, fitting of volume delay functions required extending the *v/c* ratio on the *x*-axis to include oversaturated conditions in which the *v/c* ratio exceeds 1.0. To accomplish this, the following equation was used to determine demand volume, v_{demand}, above capacity for those TTMS sites which experienced oversaturated conditions for certain periods:

$$v_{demand} = \begin{cases} v_c + (v_c - v), & \text{if } \frac{U}{v} \leq \frac{U_c}{v_c} \text{ and } U \leq U_c \\ v, & \text{otherwise} \end{cases}$$

where v_c is flow at capacity, u_c is optimum speed at capacity, h_c is optimum average headway at capacity given by U_c/v_c. This equation produced reasonable results for uninterrupted flow facilities and will be useful to practitioners modeling speed-volume relationships. Figure 1 shows scatterplot of speed vs. volume for one of the analysis segments.

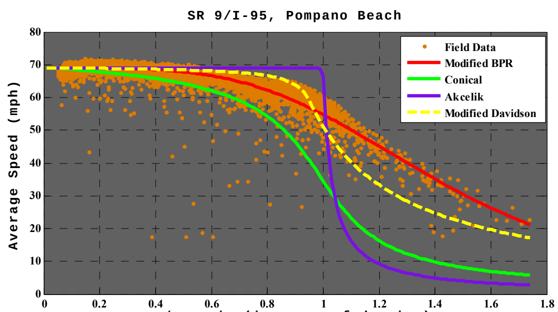


Figure 1. Fitting of Volume Delay Functions to Sample Field Data

Fitting Volume Delay Functions

Four volume delay functions were analyzed to determine their efficacy in predicting delay based on the loading on a facility represented by the volume-to-capacity (v/c) ratio. The fitting of the volume delay functions on the scatter plots of field speed-volume data was carried out with the purpose of determining the best functional forms and the attendant optimum parametric values that can reasonably fit the field data for the purpose of enabling prediction of operating speeds at various traffic flow levels. The model parameters were estimated using Gauss-Newton (GN) method of solving the nonlinear least squares problem while the goodness-of-fit was evaluated using root-mean-square error (*RMSE*), the root-mean-square percent error (*RMSPE*), the mean error (*ME*), and the mean percent error (*MPE*) statistics as measures of performance. The optimum VDFs parameters are shown in Table 3. The analysis of the fitted models showed that the modified BPR and the Modified Davidson model equations fitted the field data better than Conical and Akcelik equations for both non-toll and toll uninterrupted flow facilities. The degree of fit was evident from virtual observation of the curves (Figure 1) as well as statistical analysis of the goodness-of-fit measures.

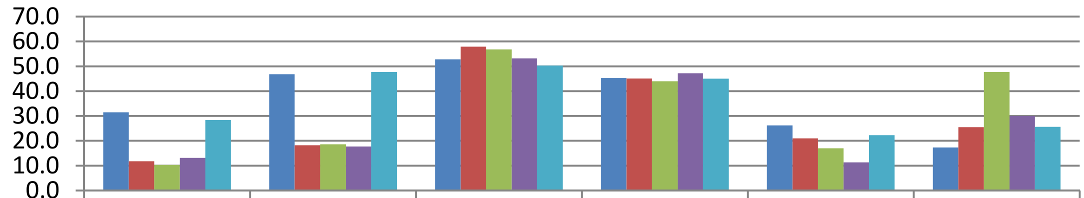
Table 3. Estimated VDF Parameters

Facility Type	Area Type	**Fitted BPR**		**Conical**		**Modified Davidson**		**Akcelik**
		α	β	β	A	J	μ	J
Freeway	Urban	0.263	6.869	18.390	1.029	0.009	0.950	0.100
	Residential	0.286	5.091	18.390	1.029	0.009	0.949	0.101
	Rural	0.150	5.610	15.064	1.036	0.010	0.951	0.099
Toll Road	Urban	0.162	6.340	18.390	1.029	0.008	0.940	0.110
	Residential	0.250	7.900	15.064	1.036	0.010	0.952	0.098
	Rural	0.320	6.710	15.064	1.036	0.010	0.940	0.097
HOV/HOT	Residential	0.320	8.400	18.550	1.028	0.009	0.950	0.090
	Urban	0.330	8.600	18.700	1.028	0.009	0.947	0.080
Divided Arterial - Signalized, ≤ 35 MPH	Residential	0.215	8.135	1.029	18.390	0.008	0.945	0.105
	Urban	0.240	7.895	1.033	16.599	0.010	0.951	0.099
Divided Arterial - Signalized, ≥ 40 MPH	Residential	0.250	8.460	1.028	18.550	0.009	0.950	0.090
	Urban	0.260	8.650	1.028	18.700	0.009	0.947	0.080
Undivided Arterial - Signalized, ≤ 35 MPH	Residential	0.215	8.135	1.029	18.390	0.008	0.945	0.105
	Urban	0.240	7.895	1.033	16.599	0.010	0.951	0.099
Undivided Arterial - Signalized, ≥ 40 MPH	Residential	0.250	8.460	1.028	18.550	0.009	0.950	0.090
	Urban	0.260	8.650	1.028	18.700	0.009	0.947	0.080

Performance of Volume Delay Functions

The Orlando Urban Area Transportation Study (OUATS) region was selected as the travel forecasting region to be used for testing the volume delay functions and their parameters that were developed in this study. The OUATS model is a daily travel forecasting model with a region that covers all or parts of six counties in central Florida, namely: Volusia (west), Lake, Seminole, Orange, Polk (northeast), and Osceola.

The results of the VDF tests were reviewed from a comparative perspective, i.e., the performance of each VDF was compared to the performance of the other volume delay functions. Figure 2 shows the percent RMSE for the estimated volume versus observed counts by facility type for morning peak period of 8-9 AM and for the afternoon peak period of 4-5 PM. These results show that the Modified BPR, the Modified Davidson, and the Conical functions performed better overall than the Standard BPR and the Akcelik functions.



	8-9 AM: Freeway	4-5 PM: Freeway	8-9 AM: Toll Road	4-5 PM: Toll Road	8-9 AM: Div-Arterial (Signalized)	4-5 PM: Div-Arterial (Signalized)
SBPR	31.5	46.8	52.8	45.3	26.2	17.3
MBPR	11.8	18.2	57.9	45.1	21.0	25.5
MDAVIDSON	10.3	18.6	56.8	44.0	17.0	47.7
CONICAL	13.1	17.7	53.2	47.2	11.3	30.0
AKCELIK	28.4	47.7	50.3	45.0	22.3	25.6

Figure 2. Percent RMSE for Estimated Volume vs. Counts for Freeway, Toll Road, Divided Signalized Arterials (8-9 AM and 4-5 PM)

In addition, comparison of the percent RMSE by volume per lane for the tested facility types (freeway, toll road and signalized divided arterial) for the 4-5 PM hour showed that the Modified BPR, Modified Davidson and Conical functions were the best performing VDFs. However, the Modified BPR function was the best performing volume delay function overall.

Benefits

Accurate travel forecasting models are essential as significant miscalculation of actual future traffic can lead to misallocation of resources and result in delays to travelers. This project was aimed at further improving models used by the State of Florida through the Florida Standard Urban Transportation Modeling Structure for travel forecasting and highway performance evaluation. The use of field data in calibrating and developing model parameters has created a cornerstone on which further improvements in the FSUTMS model can be made.

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CHAPTER ONE

INTRODUCTION

1.1 Background

Travel forecasting utilizes models to predict future travel demand based on present status of the transportation system and its use. Models are mathematical relationships used to represent and predict human behavior in making travel choices – how, where, and when to travel – based on reasonable assumptions and a relevant baseline dataset. The challenge faced by a transportation modeler is the development, validation, and calibration of models such that the predicted travel demand matches reality. One of the important steps in the urban transportation planning process is the traffic assignment step. There is no shortage of models proposed by various authors and agencies for traffic assignment. The models used for traffic assignment are commonly known as volume-delay functions (or link-congestion functions) that tend to quantify decay in travel speed (and thus increase in travel time) based on increased traffic demand on a particular highway.

A number of theoretical volume-delay functions have been proposed with some gaining wide practical applications. The major practical volume-delay functions include Bureau of Public Roads (PBR) function, the Conical function, Akcelik function, and Davidson function. The predictive accuracy of these models is heavily dependent on accurately specifying the free flow speed and practical capacity of the highway under study. In addition, properly calibrated parameters of these models (α, β, etc.) are crucial in producing realistic results.

The heterogeneity of highway networks in most States poses challenges in determining appropriate values of free flow speed, practical capacity, and modeling parameters since a set of these values applicable for one highway segment type might not be applicable for another highway type. Driver behavior and driver expectation have been documented to differ by area type – urban, rural, residential – and by facility type – limited-access, non-limited access, etc. Transportation planners have generally responded to this challenge by first categorizing the highway network by facility type and area type then creating a lookup table. Appropriate free flow speed, practical capacity, and modeling parameters are thus fixed by area type and facility type in these lookup tables.

The lookup tables such as those used in Florida were created using limited field data. Generally, as data and more information become available, the lookup tables are updated together with volume-delay functions. This research was prompted by recent developments in data acquisition, particularly by private vendors who have been collecting data to provide travelers with congestion information especially in urban areas. In addition, the improvement in point detection technologies is enabling highway agencies to monitor more highways of different functional classes thus providing data that can be used to improve travel forecasting models.

1.2 Goal and Objectives

The overall goal of this project was to improve travel forecasting and highway performance evaluation in the State of Florida. The Florida Department of Transportation, through the System Planning Office, developed and maintains a computer-based modeling package known as the Florida Standard Urban Transportation Modeling Structure (FSUTMS). One of the missions of the System Planning Office is to provide support for all FSUTMS users by continually improving FSUTMS to incorporate new methodologies and improvements of existing models. Consistent with this goal, the main objective of this research project was to evaluate the efficacy of various traffic assignment models and test their parameters using empirical data collected from traffic monitoring sites operated by the Florida Department of Transportation and where applicable utilize data collected by private vendors for traveler information purposes. Knowing that the major inputs of a traffic assignment model is free flow speed and practical capacity, this research was aimed at updating these traffic parameters using area wide data covering most area types and facility types.

1.3 Methodology

The calibration and validation of link congestion functions for use in travel forecasting and highway performance evaluation is a data-driven undertaking requiring extensive and robust data of traffic speed and volume. Thus, the cornerstone of this research project was the acquisition of data from government and private sources followed by rigorous assessment of the data and statistical manipulation to ensure that the data are suitable for extracting free flow speeds and practical capacities for highways of different geometry located in different area types. The data also have to be suitable to produce speed-flow curves on which various volume delay functions can be fitted. Once volume-delay functions have been fitted following proper calibration and validation process, model parameters of these volume-delay functions applicable to different highway categories were to be produced.

Fitting of the volume-delay functions requires plotting of speed versus volume for a particular highway using data gathered at time intervals not exceeding one hour, preferably every 15-minutes. Thus, following the selection of geographically distributed segments to represent a wide range of area types and facility types, scatterplots of speed versus volume had to be generated followed by attempts to fit a function over the plots. A number of statistical methods for assessing the goodness-of-fit were selected including root mean square error (RMSE), root mean square percent error (RMSPE), mean error (ME), mean percent error (MPE), and the Theil's inequality coefficient (TIC). The volume delay functions whose efficiency in fitting existing field data exceeded the specified statistical threshold were selected for further analysis which included testing them in the FSUTMS software using various testbeds such as the Orlando Urban Area Transportation Study (OUATS) Model. Recommendations to the FDOT Systems Planning Office to update the FSUTMS software were to be made based on the results of the model runs using the volume delay functions and their parameters analyzed.

1.4 Report Format

This report is organized as follows. Chapter 1 introduces the reader to the nature of the problem, the objectives of the research project together with the expected deliverables. In addition, the methodology used to accomplish various project tasks are detailed in this chapter. Chapter 2 reviews the relevant literature to determine the state-of-the-art and the state-of-practice in transportation planning with special emphasis on link-congestion functions and their performance as used in the traffic assignment step of the urban transportation planning process. Chapter 3 reviews the Highway Network Model incorporated into the Florida Standard Urban Transportation Model Structure (FSUTMS). This chapter informs the reader how FSUTMS categorize highways by area and facility type. It is important the reader grasps this categorization given that the collection of data and analysis of volume-delay functions are profoundly influenced by the selection of homogenous highway segments by area type and facility type. This chapter also discusses the major input variables into the Highway Network Model, i.e., free flow speed and practical capacity. The methodology used to select highway segments for analysis is also explained in Chapter 3. Chapter 4 discusses the acquisition of speed and volume data from sources maintained by the government, in this case the Florida Department of Transportation and from private vendor sources. Detailed explanation of the strengths and weaknesses of the acquired data and the statistical manipulation conducted to make the data amenable for model building is given in this chapter. Chapter 5 covers the analysis of interrupted flow facilities describing the geographical coverage of the telemetered traffic monitoring sites located in these facilities. Statistical analyses are then conducted to estimate free-flow speeds and practical capacities for this facility type. The speed-volume curves for each site are displayed in this chapter. This chapter also documents the modeling efforts in which various volume-delay functions are fitted into the developed speed-volume curves. Various statistical techniques are used in this chapter to assess the efficiency of the volume-delay functions. Modeling parameters are then derived. Chapter 6 is a mirror image of Chapter 5 except that this chapter covers analysis of interrupted flow facilities. Chapter 7 discusses the testing of various volume-delay functions and their optimized parameters. The Orlando Urban Area Transportation Study (OUATS) model was selected as the testbed. Chapter 8 gives a summary of the project efforts and recommendations for further research.

CHAPTER TWO

LITERATURE REVIEW

2.1. Background

Travel demand modeling has undergone various modifications in the past few years in order to appraise more complex policy actions resulting from legislation such as Moving Ahead for Progress in the 21st Century Act (MAP-21) and the Clean Air Act. As travel demand models have become more intricate, so have the procedures needed to update, calibrate, and validate them. Regularly, there is a compromise between increasing confidence in the level of accuracy of the models and the cost of resources needed to collect data for updating, calibrating, and validating these models. The methods used to assess the reliability of the models range from a simple assessment of the reasonableness of model outputs based on professional experience to the use of sophisticated statistical techniques.

In the current traffic assignment methods, the effect of highway capacity on travel speeds or travel times is specified by means of volume delay functions (VDFs), also known as link-congestion functions. These functions are used to express the travel time (or cost) on a highway link as a function of the traffic volume. A volume delay function can be represented in terms of speed on a link, $U_v = S(\theta, v)$, where U_v is the estimated speed on the link carrying traffic volume, v and θ represents a vector of parameters that describe the characteristics of the link. The function starts with a finite free travel speed, U_0, and then the actual travel speed decreases with increasing volume. The rate of decrease is small at low volumes but accelerates as the volume builds up towards the capacity of the link.

The factors that control the final assigned travel speeds are the free-flow speed, U_0, and capacity, *c,* which are both link-based. Typically, most highway links in the FSUTMS derive free-flow speeds and link capacities via a look-up table that relates these variables to the facility type or functional class of the link and the type of the area surrounding the link. These input parameters from look-up tables need to be updated regularly to ensure that modeled speeds on highway networks closely reflect actual speeds. Acquisition of accurate speed data is necessary for robust model development. Reliable calculation of vehicle hours traveled (VHT), time-of-day traffic assignments, development of Congestion Management Plans (CMPs), highway and transit corridor analyses, and air quality emissions analysis depend on the use of robust models. In order to implement these, travel speeds which are routinely obtained from travel demand models at the link level are used. However, conventional travel demand forecasting procedures do not typically generate sufficiently resolved or accurate enough speed estimates (Stopher and Fu, 1998). Recent advances in traffic modeling are expected to produce a more realistic estimation of speed but the majority of the recent techniques (e.g.,, simulation) are difficult to use for regional scale modeling due to significant data requirements and high computational overhead (Bai *et al.,* 2007). Two published research studies (Dowling & Skabardonis, 1993*;* Helali & Hutchinson, 1994) proposed speed post-processing as a cost-efficient alternative to simulation. Although most post-processing approaches are reported to produce speeds comparable to those derived from operational or simulation models, there has been little research exploring comparatively lower cost approaches of using extensive data currently being collected by public and private sector to update and calibrate the VDFs for reliable speed outputs. In this research, the researchers developed speed models on Florida highways consistent with the consensus of the Florida Standard Urban Transportation Model Structure (FSUTMS) Task Force which found that speed models and data used for traffic projection and model validation in Florida needed to be improved. The main sources of data used in this study were from Telemetered Traffic Monitoring Sites (TTMS) operated by FDOT Statistics Office and the Statewide Transportation Engineering Warehouse for Archived Regional Data (STEWARD) database.

The process of refining the outputs of regional travel demand models depends on data that reflects regional travel activities. Each step of the traditional four-step travel demand model needs to be calibrated, validated, and updated regularly to cope with the changes in trends of travel demand and behavior. The common practice is to calibrate and validate each step individually and not the entire model at once. This is done in order to control and minimize propagation of errors from one step to other subsequent steps. The estimation, calibration, validation, and updating of the first three steps in the travel demand models (i.e., trip generation, distribution, and mode choice) use census and survey data collected locally. The level of complexity differs in each step. With highway assignment models, it has been difficult to use locally collected data from traffic counts due to limited resources to make the data usable for modeling. The data used in this approach was acquired from TTMS which was aggregated hourly for a full year period. This extensive data coverage of Florida highways captures all important traffic variability necessary for accurate modeling.

2.2. Desirable Characteristics of Volume Delay Functions

Regional models used for traffic assignment generally require that travel speed be a monotonically decreasing function of volume. This ensures that the model will be able to find a single user-equilibrium solution to the traffic assignment problem. Speed-flow models must possess two important characteristics in order to permit capacity constrained equilibrium assignment to be executed by travel demand models. First, the speed-flow models must be monotonically decreasing and continuous functions of the volume/capacity ratio (v/c) in order for the equilibrium assignment process to arrive at a single unique solution (Spiess, 1990). Second, as a practical matter, the speed-flow models should also be asymptotic to the horizontal axis and never intersect it leading to the predicted travel speed never reaching a value of zero.

2.3. State-of-practice in Volume Delay Functions

The notations used for each function are, U_v the speed as a function of demand volume v, U_0 representing free-flow speed, v is link volume, c is the capacity, v/c stands for v/c ratio, t_v is travel time per unit distance corresponding to demand volume v, t_0 is the travel time under free flow conditions, and T designates the analysis period.

2.3.1 Standard and Modified BPR Functions

The standard BPR curve was derived in the late 1960s by the Bureau of Public Roads, now the Federal Highway Administration (FHWA) by fitting a polynomial equation to the freeway speed-flow curves in the Highway Capacity Manual developed in 1965. Efforts have been made by various metropolitan planning organizations (MPOs) to advance and modernize the original formulation of the BPR curve by fitting local data or hypothetical data from simulation models. This resulted in different forms in the BPR curve throughout the United States. The general form of the function is given below as:

$$U_v = \frac{U_0}{[1+\alpha({}^{v}/_{c})^{\beta}\ ]} \quad \text{.......... 2.1}$$

This function presumes that coefficient α, often set at 0.15 is the ratio of travel time per unit distance at practical capacity to that at free flow, and that parameter β (often set at 4) determines how fast the curve of estimated average link speed, U_v versus ${}^{v}/_{c}$ ratio decreases from free-flow speed. With higher values of β, the onset of congestion effects becomes more and more sudden (Spiess, 1990).

The BPR function became widely used in transportation modeling due to its minimum data input requirements and its simple mathematical form. In addition, Dowling and Skabardonis (1997) argue that it is also easier to develop efficient algorithms for finding the equilibrium solution of the BPR function by differentiating it. However, the standard BPR curve has a number of limitations. Its derivation is based on data that do not reflect current operating conditions and does not take into account facility characteristics, such as signalization conditions on arterials (Dowling and Skabardonis, 1997). These drawbacks led several planning organizations to propose alternative BPR curves to match local travel activities.

Early proposals were made by Dowling and Skabardonis (1993) to modify of the BPR function to deal with the requirements of air quality-transportation modeling. However, the function they proposed did not make any distinction between freeways and arterials, as they stated that estimating intersection delay is too dependent on data that was practically unavailable. For under-saturated conditions (i.e., v/c ≤ 1), the proposed BPR coefficient, α, and the exponent, β, are 1.0 and 10, respectively. This function was intended to fit speed-flow curves that were incorporated into the 1985 Highway Capacity Manual. For oversaturated conditions, the speed was computed using queue analysis.

Improvements of the model proposed by Dowling and Skabardonis (1993) were made by Helali and Hutchinson (1994) by characterizing arterial streets and freeways in isolation. In this model, the under-saturated conditions in freeways were treated by using the so-called Greater Toronto Area link performance function (Data Management Group, 1991), which is the modified BPR function with α = 1.0 and β = 6; the over-saturated scenarios were solved using queuing analysis. In addition the calibrated Davidson's function (Davidson, 1966; Davidson 1978) was proposed for speed estimation for under-saturated arterial streets. The proposed function has the following form:

$$U_v = \max\left(\frac{U_0}{1+10J}, \frac{U_0}{1+\frac{Jv}{c-v}}\right) \quad \text{.......... 2.2}$$

The value of J is 0.211 for CBD, 0.187 for metropolitan areas and 0.170 for other locations. The procedure to deal with oversaturated arterial streets is the same, i.e., queuing analysis used by Dowling and Skabardonis (1993).

New developments in the 1994 Highway Capacity Manual prompted the need to modify the BPR function to cater for the changes made in the manual. Skabardonis and Dowling (1997) proposed a new BPR function shown in Equation 2.3 that better fits the Highway Capacity Manual speed-flow curve.

$$U_v = \frac{U_0}{[1+0.2(^{v}/_{c})^{10}]} \qquad 2.3$$

In this case, v is the demand discharging at $t + \Delta t$ plus the residual queue carried over from time period *t*. In addition to the proposed function, the use of queuing analysis in oversaturated regions was discarded. However, the analysis of queue length was still maintained in order to determine speed variations within a relatively long peak period. The presence of a queue would not directly affect the speed calculation at a given time space, *t,* instead the queue would influence speeds at time period, $t + \Delta t$, as a residual flow.

For signalized facilities where signals are spaced at less than or equal to 2 miles apart the coefficient in the denominator in Equation 2.3 changes from 0.2 to 0.05 and speeds are worked out using the "Updated BPR" curve based contained in the *NCHRP Report 387* (1997). The equation for signalized facilities is shown below:

$$U_v = \frac{U_0'}{[1+0.05(^{v}/_{c})^{10}]} \qquad 2.4$$

where U_0' is the adjusted free flow speed considering the presence of signals. The modified speed is then dependent upon factors such as number of signals (*N*), link length *(l)* and the intersection delay *(d)* given in the 1994 Highway Capacity Manual. The modified free flow speed is given by:

$$U_0' = \frac{l}{\frac{l}{U_0}+\frac{dN}{3600}} \qquad 2.5$$

Similar in form to the modified BPR Equations 2.3 and 2.4, Michael Baker Associates (1999) developed an equation for undersaturated conditions on non-limited-access facilities in Virginia:

$$U_v = \frac{U_0^p}{[1+0.8\left(^{v}/_{c_p}\right)^2]} \qquad 2.6$$

There are at least two differences noted between the previous modified BPR Equations 2.3 and 2.4 and the Michael Baker Associates' Equation 2.6. In Equation 2.6 the parameters U_0^p (practical free flow speed) and c_p (practical capacity) are introduced. The practical free flow speed is the corridor free flow speed divided by 1.15. According to the 1997 *NCHRP Report 387*, the value of practical capacity is 80 percent of the capacity at level of service E.

For oversaturated non-limited-access facilities, Equation 2.7 was proposed and interestingly the results were exactly equivalent to those found using Equation 2.6 at capacity.

$$U_v = \frac{l}{1.8t_0+0.2(\frac{v}{c}-\frac{c_p}{c})} \qquad 2.7$$

For limited-access highways, Michael Baker Associates proposed the coefficient in the denominator of Equation 2.6, should be changed from 0.8 to 0.15 and the exponent from 2 to

13.29. While the change of coefficients improved performance, changing the exponent caused the resulting function to have substantial errors for predicting speeds on limited-access facilities (Miller *et al.*, 2004). To overcome these drawbacks, Miller *et al.* changed the coefficient from 0.8 to 0.15 but retained the exponent of 2 thus leading to Equation 2.8, applicable to limited-access facilities:

$$U_v = \frac{U_0^p}{[1+0.15\left(v/c_p\right)^2]} \qquad 2.8$$

In the Northern Virginia District, the use of standard BPR equation (Equation 2.1) proved to be significantly useful. However, the Virginia Department of Transportation made modifications to standard BPR when volume to practical capacity ratio exceeded 2.0 such that the coefficient in the denominator was changed from 0.15 to 0.60 and the exponent from 4 to 2, leading to higher estimated speeds than would otherwise be obtained without the modifications. The District staff observed that, at volume/practical capacity ratios above 2.0, these higher predicted speeds were more realistic in their planning applications than would have been obtained with standard BPR equation (Miller *et al*., 2004).

2.3.2 Other Link Congestion Models

In traffic network modeling it is often essential to describe the overall traffic performance of a facility (e.g., a route, link or junction) by a single function, rather than to apply separate functions for free flowing and interrupted traffic (Taylor, 1997). The relationship between the amount of traffic using a network element and the travel time (or travel speed), and the delay incurred on that facility is known as a congestion function or volume delay function. The estimated travel speed along a network facility is inversely related to the traffic volume using that facility. As volume increases so does the travel time due to decreased speeds. The rate of increase in travel time accelerates as volume-to-capacity ratio (*v/c*) of the facility approaches 1.0.

Davidson's delay model

A typical congestion function was developed by Davidson (1978):

$$U_v = \frac{U_0}{1+\frac{J(v/c)}{(1-v/c)}} \qquad 2.9$$

in which v/c is the degree of saturation of the network element, J is an environmental parameter that reflects road type, design standard and abutting land use development, and c is the absolute link capacity.

The Davidson function has proved popular in economic analysis and travel demand modeling for road networks, largely on account of its flexibility and its ability to cater for a wide range of traffic conditions and environments (Taylor, 1997). However, the original Davidson function as shown in Equation 2.9 has one serious flaw – it cannot define a travel time for link volumes which exceed the capacity *(c)*. This leads to computational problems in network models which determine link volumes iteratively and as a result may occasionally overload some links in computing its intermediate solutions (Taylor, 1997). A modification involving the addition of a linear extension term as a second component to the function was proposed by Tisato (1991) as follows:

$$U_v = \begin{cases} \dfrac{U_0}{1+\frac{J(\frac{v}{c})}{(1-{}^{v}/_{c})}}, & for\ {}^{v}/_{c} \le \mu \\ \\ \dfrac{U_0}{1+\frac{J\mu}{(1-\mu)}+\frac{J({}^{v}/_{c}-\mu)}{(1-\mu)^2}}, & for\ {}^{v}/_{c} > \mu \end{cases} \quad \text{..........} 2.10$$

where μ is a user-selected proportion, usually in the range (0.85, 0.95), which provides a finite definition of the function for all finite ${}^{v}/_{c}$ ratios. Equation 2.10 also allows for link oversaturation.

Akcelik delay function

Akcelik (1991) proposed a time-dependent form of the Davidson function. Using the coordinate transformation technique, the Akcelik function attempts to incorporate intersection delay which provides a significant part of the total link travel time. The Akcelik function takes the following form:

$$U_v = \frac{U_0}{\left(1+0.25U_0\left[\left(\frac{v}{c}-1\right)+\sqrt{\left(\frac{v}{c}-1\right)^2+8J\frac{\left(\frac{v}{c}\right)}{cT}}\right]\right)} \quad \text{..........} 2.11$$

where J is a delay parameter. Akcelik suggested lower values of J be used for freeways/coordinated signal systems while higher values should be used for arterial roads without signal coordination. Akcelik's function has been tested for planning applications and was observed to often provide the best fit when comparing various speed delay functions to data collected from 119 freeway segments located in California (Skabardonis and Dowling, 1997). Singh (1999) also indicated that the use of Akcelik's function in traffic assignment has some other advantages, such as better convergence and more realistic speed estimation under congested conditions. Akcelik function is also useful because of its conciseness – that is, a uniform functional form can be used everywhere while avoiding complex parameters in computing intersection delay.

Conical delay model

Spiess (1990) proposed the conical link-congestion function to overcome the drawbacks associated with high exponent β values of the BPR function. Spiess found that high values of β reduce the rate of convergence by giving undue penalties to overloaded links during the first few iterations of an equilibrium assignment and can also cause numerical problems, such as overflow conditions and loss of precision. Additionally, for links with volumes that are far below capacity, the BPR function with high β values always yields free-flow times that do not match those of the actual traffic volumes. The conical link-congestion function proposed by Spiess is defined as:

$$U_v = \frac{U_0}{\left[2+\sqrt{\beta^2\left(1-\frac{v}{c}\right)^2+\alpha^2}-\beta\left(1-\frac{v}{c}\right)-\alpha\right]} \quad \text{..........} 2.12$$

where $\alpha = \frac{2\beta-1}{2\beta-2}$, and $\beta > 1$. The parameter β corresponds to exponent β of the BPR function. The use of this function proved a remarkable improvement in the convergence of equilibrium assignment when switching from the previously used BPR functions to the corresponding conical functions in a transportation study conducted in the City of Basel, Switzerland (Spiess, 1990).

It should be noted that all of the congestion functions (or VDFs) discussed above are 'steady-state' functions in that they are based on the assumption that the flow v will persist indefinitely. In other words, most of the VDFs used in travel demand forecasting models are static and don't take into account propagations of congestion. However, the prediction level they provide is adequate for long-range transportation planning purposes.

CHAPTER THREE

THE HIGHWAY NETWORK IN FSUTMS

3.1 Overview of the FSUTMS Model

To understand the representation of the highway network model within the FSUTMS, a brief overview of the FSUTMS is covered in this section. The Florida Standard Urban Transportation Modeling Structure (FSUTMS) is a computer-based modeling package used by agencies throughout the State of Florida for travel forecasting and highway performance evaluation. The FSUTMS is currently implemented in Cube Voyager developed by Citilabs and follows the relationship shown in Figure 3.1.

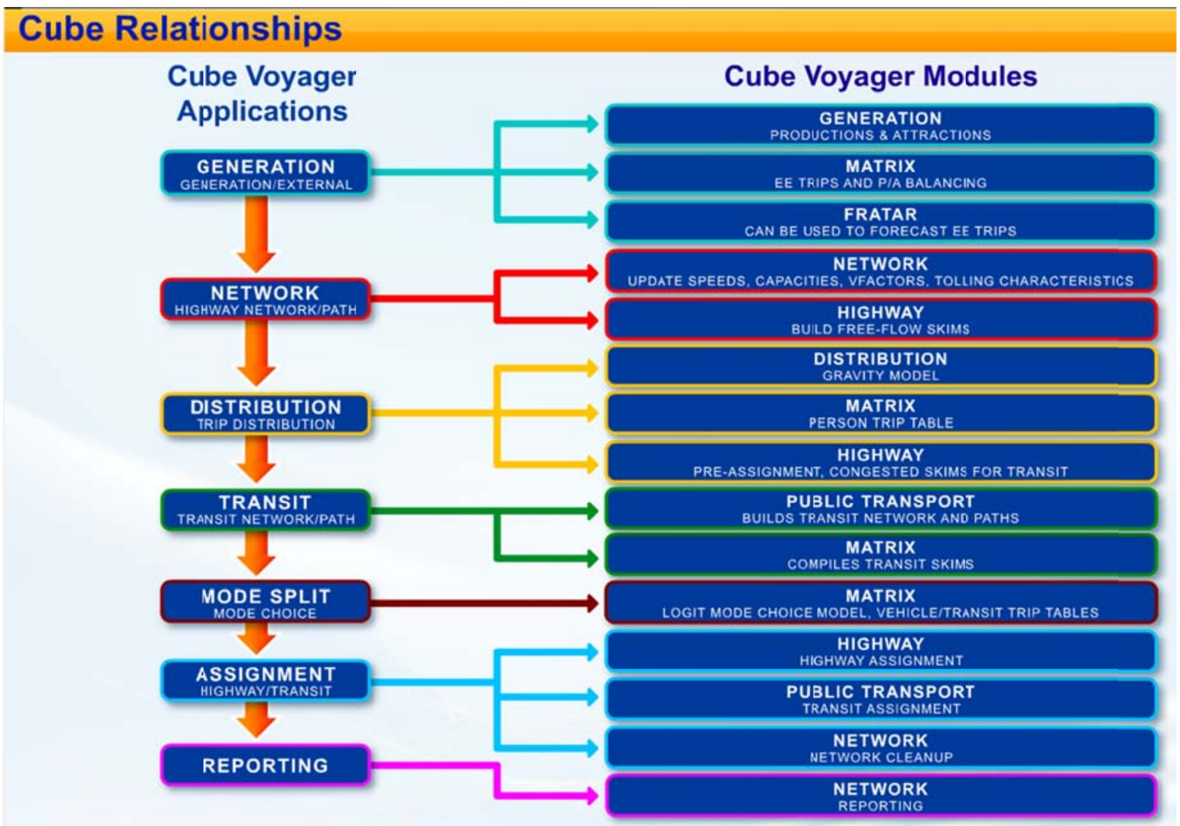


Figure 3.1 FSUTMS Implementation in Cube Voyager *(Source: FSUTMS WBT online)*

Figure 3.1 shows that the computerization of an existing or proposed transportation system is a second step in the FSUTMS model chain. A transportation system is comprised of a network of highways and/or a network of transit services. A highway network consists of links and nodes that represent a roadway system in the study area of interest. Because traffic assignment on a highway network is influenced by trips generated from a particular zone and the characteristics of the highway itself, transportation modelers generally classify highways by area type, facility type, and facility size as discussed in the following sections.

3.2 Area Type

Travel activity is heavily influenced by population and land use activity. Travel conditions in a particular area elicit different considerations in travel forecasting and highway performance evaluation. When modeling population centers with significant roadway congestion, there may be a need to employ models with loop-feedback and capacity-constraints so that the effects of congestion on travel behavior are reasonably captured by the model. Consequently, the implementation of volume-delay functions in congested areas poses challenges that are different from the implementation of these functions in less congested areas such as those found in rural Florida. In addition, unlike rural areas where more travel occurs on non-limited access facilities, a significant portion of daily trips in the congested areas are undertaken on limited access facilities. In a nutshell, although the functional form of the volume-delay function might be the same for rural and urban facilities, the modeling parametric values (σ, β, etc.) will be different.

In 1998, the HNET Enhancements Study recommended to the Model Task Force (MTF) implementation of the following area types in the FSUTMS:

1. CBD areas
2. CBD fringe areas
3. Residential areas
4. Outlying Business District (OBD) areas
5. Rural areas

These five area types are further subdivided as follows:

<u>CBD Areas</u>
Urbanized Area (over 50,000) Primary City Central Business District
Urbanized Area (under 50,000) Primary City Central Business District
Other Urbanized Area Central Business District and Small City Downtown
Non-urbanized Area Small City Downtown

<u>CBD Fringe Areas</u>
Typical Central Business District (CBD) Areas
CBD Fringe Strip Commercial

<u>Residential Areas</u>
Residential Area of Urbanized Areas
Undeveloped Portion of Urbanized Areas
Transitioning Areas/Urban Areas Over 5,000 Population
Beach Residential

<u>Outlying Business District (OBD) Areas</u>
High Density Outlying Business District
Other Outlying Business District
Beach Outlying Business District

<u>Rural Areas</u>
Developed Rural Areas/Small Cities Under 5,000 Population
Undeveloped Rural Areas

3.3 Facility Type

Research has shown that driver behavior varies by facility type. Therefore, volume-delay models should be developed based on the geometric characteristics of roadway facilities. Limited access facilities, e.g., v/c ratio, freeways, are built to very high standards with wide multiple lanes, lateral clearance, and low interchange density. This type of facility enables very high speeds and very high throughput of traffic. On the other hand, urban arterials that have short signal spacing with many roadside driveways elicit drivers to be more cautious and therefore speeds and volume relationships on these roadways would be different from those pertaining to freeways.

In 1998, the HNET Enhancements Study also recommended to the Model Task Force (MTF) the implementation of the following facility types in the FSUTMS:

1. Freeways and Expressways
2. Divided Arterials
3. Undivided Arterials
4. Collectors
5. Centroid Connectors
6. One-way Facilities
7. Ramps
8. HOV Facilities
9. Toll Facilities

These nine facility types are further subdivided into subtypes as follows:

<u>Freeways and Expressways</u>
Urban Freeway Group 1 (cities of 500,000 or more)
Other Freeways (not in Group 1)
Collector/Distributor Lane
Controlled Access Expressway
Controlled Access Parkway

<u>Divided Arterials</u>
Divided Arterial Unsignalized (55 mph)
Divided Arterial Unsignalized (45 mph)
Divided Arterial Class 1a
Divided Arterial Class 1b
Divided Arterial Class II/III

<u>Undivided Arterials</u>
Undivided Arterial Unsignalized with Turn Bays
Undivided Arterial Class 1a with Turn Bays
Undivided Arterial Class 1b with Turn Bays
Undivided Arterial Class II/III with Turn Bays
Undivided Arterial Unsignalized without Turn Bays
Undivided Arterial Class 1a without Turn Bays
Undivided Arterial Class 1b without Turn Bays
Undivided Arterial Class II/III without Turn Bays

Collectors
Major Local Divided Roadway
Major Local Undivided Roadway with Turn Bays
Major Local Undivided Roadway without Turn Bays
Other Local Divided Roadway
Other Local Undivided Roadway with Turn Bays
Other Local Undivided Roadway without Turn Bays
Low Speed Local Collector
Very Low Speed Local Collector

Centroid Connectors
Basic Centroid Connector
External Station Centroid Connector

One-Way Facilities
One-Way Facility Unsignalized
One-Way Facility Class 1a
One-Way Facility Class 1b
One-Way Facility Class II/III
Frontage Road Unsignalized
Frontage Road Class 1a
Frontage Road Class 1b (default for all Frontage Roads)
Frontage Road Class II/III

Ramps
Freeway On-Ramp
Freeway Loop On-Ramp
Other On-Ramp
Other Loop On-Ramp
Freeway Off-Ramp
Freeway Loop Off-Ramp
Other Off-Ramp
Other Loop Off-Ramp
Freeway to Freeway High-Speed Ramp

HOV Facilities
Freeway Group 1 HOV Lane (Barrier Separated)
Other Freeway HOV Lane (Barrier Separated)
Freeway Group 1 HOV Lane (Non-Separated)
Other Freeway HOV Lane (Non-Separated)
Non Freeway HOV Lane
AM&PM Peak HOV Ramp
AM Peak Only HOV Ramp
PM Peak Only HOV Ramp
All Day HOV Ramp

Toll Facilities
Freeway Group 1 Toll Facility

Other Freeway Toll Facility
Expressway/Parkway Toll Facility
Divided Arterial Toll Facility
Undivided Arterial Toll Facility
Toll On-Ramp
Toll Off-Ramp
Toll Plaza

3.4 Facility Size

Are traffic operating characteristics different between, say, a 3-lane and a 4-lane directional freeway? Are free flow speeds on 3-lanes one-direction highways higher than 4-lane highways? Also, does practical capacity measured as passenger cars per hour per lane, differ between freeways with different number of lanes? Numerous research findings have indicated that on basic freeway segments, the number of lanes affects free flow speed which in turn affects the freeway's capacity, measured in passenger cars per hour per lane.

Table 3.1 shows that the impact of lateral geometrics on free flow speed (FFS) depends on both the distance of obstruction and the number of lanes in one direction on the basic freeway segment. A lateral clearance restriction causes vehicles in the right lane to move somewhat to the left, thus affect operations in the next lane. As the number of lanes increases, the overall effect on freeway operations decreases.

Table 3.1 Impact of Lateral Geometrics on FFS (HCM2010, Exhibit 11-19)

Right-Side Lateral Clearance (ft)	**Lanes in One Direction**			
	2	3	4	≥5
≥ 6	0.0	0.0	0.0	.0.
5	0.6	0.4	0.2	0.1
4	1.2	0.8	0.4	0.2
3	1.8	1.2	0.6	0.3
2	2.4	1.6	0.8	0.4
1	3.0	2.0	1.0	0.5
0	3.6	2.4	1.2	0.6

In addition, HCM2010 Exhibit 11-2 as reproduced in Figure 3.2 below shows that free flow speed has a major influence on practical capacity of a freeway. Combining the results of Table 3.1 and Figure 3.2, it can be shown that if all geometrics of two freeways are the same, and the only difference is the number of lanes in one direction, the freeway with more lanes will generally have higher free flow speed and higher practical capacity. This is a fact that has been noted by transportation modelers and included in the lookup tables in the FSUTMS.

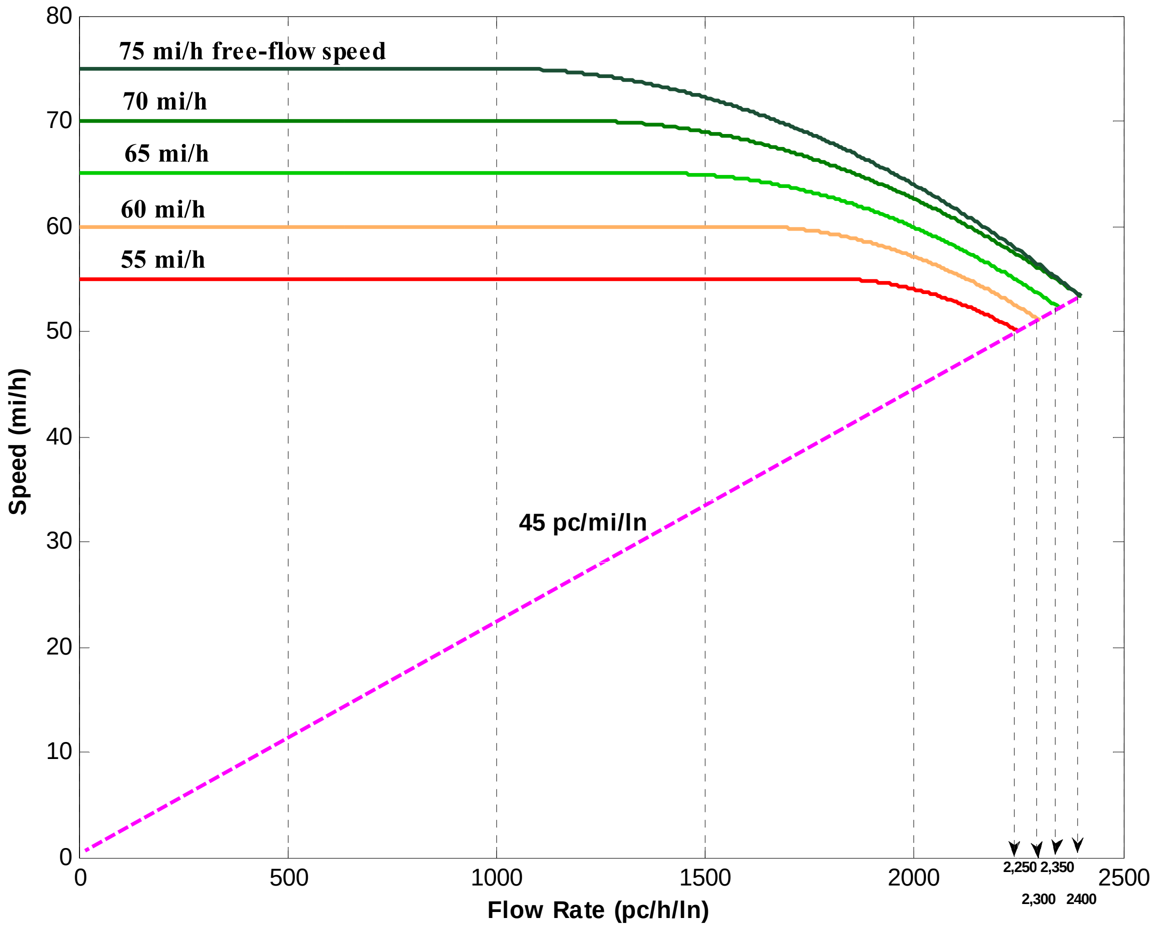


Figure 3.2 Speed-Volume Curve for a Freeway Segment (HCM2010, Exhibit 11-2).

3.5 Inputs to FSUTMS Highway Network Model

To perform traffic assignment on a highway network, the Highway Network model reads a number of input files which include speed/capacity lookup table; turn penalty and prohibitors; toll link and toll plaza information; and variable factors. The speed/capacity lookup table refers to the free flow speed and practical capacity which are a product of the geometrics and traffic characteristics of a highway. The variable factors (VFACTOR) file is used to store parameters for different facility types. These parameters are UROAD factor, CONFAC, BPR LOS, and BPR EXP. Of importance in this research are the BPR LOS and BPR EXP factors which comes from the Bureau of Public Roads (BPR) equation:

$$T = T_0 \times \lfloor 1 + BPRLOS \times ({}^{v}/_{c})^{BPREXP} \rfloor \quad (3.1)$$

in which T = travel time

T_0 = travel time at free flow speed

v/c = ratio of assigned volume-to-practical capacity

$BPRLOS$ = BPR level-of-service (LOS) value, commonly referred to as α in literature. The current default value in FSUTMS is 0.15.

$BPR\ EXP$ = BPR exponent, commonly referred to as β in literature.

The scope of this project is limited to researching and updating the speed/capacity lookup table and modeling factors.

3.6 Selection of Segments by Area-Facility Type

Analysis of the FSUTMS speed/capacity lookup tables shows free flow speed and practical capacity values specified for 16 area types and 49 facility types resulting into 784 distinct speed/capacity values. The updating of these values would require the study of traffic operating characteristics on 784 different segments (not counting replication) in order to confidently propose changes of the existing speed/capacity values. Due to limited financial and time resources allocated to this project, it was impossible to evaluate all area-facility type combinations found in FSUTMS.

To pare down the number of segments to be studied that could have significant impact of most area-facility type combinations, a rational method had to be devised to select homogenous segments for further evaluation. Generally, the representative facility type should possess theoretical and practical characteristics of the group type. In addition, the facility types should be randomly selected while covering all geographical regions within the state. The selection should also be as diverse as possible to minimize the effects of geographic driving patterns. As discussed in the preceding sections, there are five area types used in the FSUTMS model. These are central business district (CBD); fringe area of CBD; residential area; outside business district (OBD); and rural areas. It was decided that the five area types be collapsed into three area types – i.e., urban, residential, and rural. As for facility types, the information above shows that there are nine facility types specified in FSUTMS with each type being subdivided further into additional distinct types with different default capacity and free flow speed. A decision was made to collapse these facility types into seven categories as shown in Table 3.2. The next challenge therefore was to find 21 homogenous segments that would represent these area-facility type combinations.

Table 3.2 Area-Facility Types Combinations to be Studied

	Area Type		
Facility Type	CBD	Residential	Rural
Freeways	✓	✓	✓
Divided arterials	✓	✓	✓
Undivided arterials	✓	✓	✓
Collectors	✓	✓	✓
One way streets	✓	✓	✓
Ramps	✓	✓	✓
HOV lanes	✓	✓	✓

CHAPTER FOUR

SPEED VOLUME DATA ACQUISITION

4.1. Overview

The operational performance of a highway is conducted by measuring the supply and the demand. The demand on a highway system is known to vary temporally, spatially, modally, and compositionally. The measurement of demand depends on the viewpoint of the transportation analyst, i.e., looking at traffic flow at microscopic viewpoint or at macroscopic viewpoint as shown in Table 4.1.

Table 4.1 Measurement of Traffic

Traffic Characteristic	Microscopic measures	Macroscopic measures
Speed	Individual vehicle speed	Average speed of a group of vehicles
Flow	Time headway	Flow rate
Density	Distance headway	Density rate

At the microscopic level, an analyst is looking at traffic behavior by following individual vehicles and assessing their speeds and how they follow each other in time (by measuring time headway) and in space (by measuring distance headway). Collection of traffic flow data at microscopic level enables the analyst to evaluate string stability, stochastic queues, intersection signalization, and speed limit violations. At macroscopic level, the analyst is looking at the behavior of a group of vehicles by measuring the average speeds per unit of time, e.g., per hour and by measuring traffic volumes, densities, and occupancies, e.g., per hour. From this high level, the analyst can use the data to analyze highway segment capacities and free flow speeds; conduct shock wave analysis; assess congestion and travel time reliability; and plot speed-density, speed-volume, and volume-density curves.

Transportation planners are generally interested in macroscopic measures of traffic flow as it is these measures that are used as input into travel forecasting models. Traditionally, macroscopic data have been collected using loop detectors installed at strategic points on the highway system. However, recent advances in microelectronics and computing power are enabling other techniques of capturing traffic data, particularly speed and travel time. The data used in this study were acquired from two sources – hereinafter referred to as government data and private vendor data.

4.2. Government Data

The data was supplied by the Transportation Statistics Office of the Florida Department of Transportation. This state agency is a central clearinghouse and the principal source for highway and traffic data. The office operates temporary and permanent count stations strategically placed at various locations on the state highway system. The data collected by electronic equipment installed at these stations include individual vehicle records composed of number of axles per vehicle, axle spacing, overall vehicle length, and operating speed. The individual vehicle records data are then used to derive a number of traffic variables including operating speed distribution, hourly volumes, the annual average daily traffic (AADT), and classification of vehicles into Federal Highway Administration's Scheme F. The Florida Department of Transportation operates other sites categorized as weigh-in-motion (WIM) sites that are set up to additionally collect individual axle weights and overall gross vehicle weight. Figure 4.1 shows the location of traffic monitoring sites in the State of Florida as they existed in year 20101.

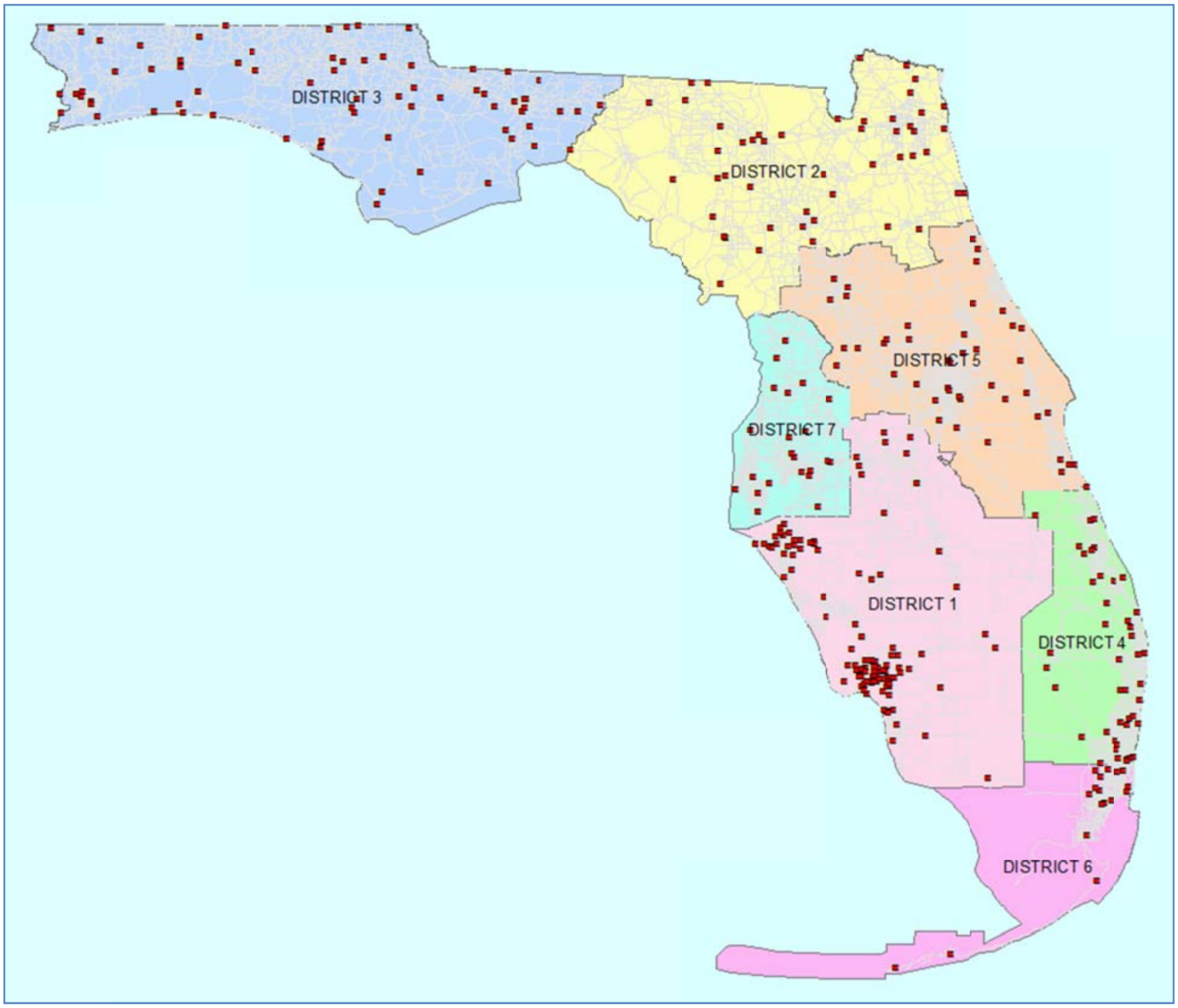


Figure 4.1 Geographical Distribution of Permanent Traffic Monitoring Sites

Figure 4.1 shows that there are 256 TTMS sites. The data files showed that 21 of these sites collected vehicle speeds only while 235 sites collected both vehicle speeds and classification counts. Table 4.2 shows the distribution of TTMS sites by type of data collected, area type, and facility type. Sites with HOV lanes are all located on freeways and are counted in the Freeway & Expressway facility type category.

Table 4.2 Distribution of TTMS Sites by Area and Facility Type

Facility Type	TTMS Sites					
	Speed Counts Only	Speed and Classification Counts				Total
		Urban	Residential	Rural	Sub-Total	
Freeway & Expressway	2	9	23	19	51	53
Divided Arterials	10	21	34	29	84	94
Undivided Arterials	7	7	11	56	74	81
Collectors			2	7	9	9
One-Way Facilities						
Ramps						
Toll Roads	2	2	6	9	17	19
HOV Lanes*		3	1		4	4
Total	**21**	**39**	**76**	**120**	**235**	**256**

* Sites with HOV lanes are counted only in the Freeway & Expressway facility type category.

Table 4.2 shows that speed and vehicle classification monitoring TTMS sites on divided arterials provided 41.2 percent of the hourly records in the main data set. Speed and vehicle classification monitoring TTMS sites on freeways/expressways and undivided arterials provided 28.3 percent and 19.4 percent of the hourly records respectively. The main data set supplied by the Florida Department of Transportation contained 8,580,315 records of hourly counts by lane for the 256 sites shown above (Table 4.2) for the period beginning July 1, 2010 and ending June 30, 2011.

4.2.1 File Format and Data Structure

Since this project started in mid-year, the data that were analyzed covered the period from July 1, 2010 to June 30, 2011. Two sets of data were thus provided to the research team in ASCII format. One file set consisting of 96,553 speed count data files and 86,891 vehicle classification count data files contained traffic data recorded from January 1, 2010 to December 31, 2010 and another file set comprising 47,525 speed count data files and 42,153 vehicle classification count data files covered the period beginning January 1, 2011 and ending June 30, 2011. Each TTMS hourly speed and vehicle classification count data file contained records for a particular count unit at a particular TTMS site for a particular date for each travel lane. In the hourly speed count data files each record is organized into twenty-six fields as shown in Table 4.3.

Table 4.3 Data Structure of Speed Count Data File.

Description	Position	Start Column	End Column
Record Type	1	1	3
County	2	4	5
Site ID	3	6	9
ATR Lane	4	10	11
Year	5	12	14
Month	6	15	16
Day	7	17	18
Hour	8	19	20
Minute	9	21	22
Source	10	23	26
1 to 20 mph	11	27	31
21 to 25 mph	12	32	35
26 to 30 mph	13	36	39
31 to 35 mph	14	40	43
36 to 40 mph	15	44	47
41 to 45 mph	16	48	51
46 to 50 mph	17	52	55
51 to 55 mph	18	56	59
56 to 60 mph	19	60	63
61 to 65 mph	20	64	67
66 to 70 mph	21	68	71
71 to 75 mph	22	72	75
76 to 80 mph	23	76	79
81 to 85 mph	24	80	83
85+ mph	25	84	87
Total	26	88	93

The vehicle counts for each record are contained in 15 speed bins according to the speed of the vehicle. One speed bin is used for all vehicles travelling at or below 20 miles per hour (mph), one bin for vehicles travelling at speeds greater than 85 mph, and 13 speed bins at 5 mph intervals for vehicles traveling at speed greater than 20 mph to 85 mph. Each record in the hourly speed count data file represents a single lane at the TTMS site. Table 4.3 above shows the data structure of the file while Figure 4.2 below shows an extract from a typical TTMS hourly speed count data file. The structure of the hourly vehicle classification count data files is shown in Appendix B together with the structure and data dictionary of merged data file.

```
SPD930010 1 1001010100 060    0   0   1   4  32  68  54  17   5   1   0   0   0   0   0  182
SPD930010 2 1001010100 060    0   0   0   0   0   0   0   0   0   0   0   0   0   0   0    0
SPD930010 3 1001010100 060    0   0   0   5  35  67  17  10   2   0   0   0   0   0   0  136
SPD930010 4 1001010100 060    0   1   6  28  92 101  51  16   1   0   0   0   0   0   0  296
SPD930010 1 1001010200 060    0   0   1   4  39  85  52  15   3   0   0   0   0   0   0  199
SPD930010 2 1001010200 060    0   0   0   0   0   0   0   0   0   0   0   0   0   0   0    0
SPD930010 3 1001010200 060    0   0   0   4   6  52  30  12   1   0   0   0   0   0   0  105
SPD930010 4 1001010200 060    0   0   1   7  45 131  50  16   2   0   0   0   0   0   0  252
SPD930010 1 1001010300 060    0   0   1   3  15  45  39  17   4   0   0   0   0   0   0  124
SPD930010 2 1001010300 060    0   0   0   0   0   0   0   0   0   0   0   0   0   0   0    0
SPD930010 3 1001010300 060    0   0   0   2   5  23  19   4   2   0   0   0   0   0   0   55
SPD930010 4 1001010300 060    0   2   1   6  15  63  36  14   5   1   0   0   0   0   0  143
SPD930010 1 1001010400 060    0   0   0   0   6  11  24  11   2   2   0   0   0   0   1   57
```

Figure 4.1 Extract from Typical TTMS Hourly Speed Count Data File

As seen in Appendix B, the vehicle counts for each record are contained in 15 vehicle classification bins based on the FHWA Classification Scheme F. Classes 1 to 3 are motorcycles, automobiles, and light trucks; Classes 4 to 13 are trucks and buses; Class 14 is not currently used; and Class 15 is unclassifiable vehicles. Each record in the file represents a single lane at the TTMS site.

4.2.2 Data Augmentation, Cleaning and Validation

Other data files were acquired and merged into the main dataset to augment the TTMS count data and to aid in the data cleaning process. The files that were acquired are described below.

- *Lane Relationship data file (LaneRel.csv).* This file contains information for all lanes at all TTMS sites and each record in the file provides information about a single lane. The information in each record includes TTMS Site ID, Unit No., ATR Lane number and direction of travel for the lane.
- *Florida State 2010 and 2011 Holidays*. Traffic flow on holidays is atypical and thus there was a need to identify, flag, and discard counts that were recorded on holidays. A list of dates for 2011 holidays was obtained from the Florida Department of Management Services (DMS) website. The dates for 2010 holidays were generated by adjusting the dates from the 2011 list of State holidays. These were added to a list containing each Day of Week for 2010 and 2011 by date. Weekdays on which a holiday was observed were flagged as holidays. The 2010 and 2011 holiday and day of week information were merged into the main data set using a merge key created from data in the year, month, and day fields. All weekday (Monday to Friday) records in the main data set are for non-holiday weekdays. Each record in the main data set is associated with one of eight "Day of Week" types – namely, Monday; Tuesday; Wednesday; Thursday; Friday; Saturday; Sunday; and Holiday.
- *2010 and 2011 TTMS "Bad Counts" data files.* These files listed the dates when counts at a particular TTMS was deemed as bad data based on data audits conducted by the Florida Department of Transportation data analysts. This information was merged into the main data set – using a merge key created form the Site ID, year, month, day, and direction of travel fields – and used to flag corresponding records as bad counts. These "bad" records were excluded from the main dataset during the data cleaning process.
- *TTMS Site Description data file.* This file provided several details about each TTMS site including: number of lanes by direction; location by road section, road name and coordinates; active status of the site; and whether or not the site counts vehicles by classes.
- *Florida Statewide Model Facility Type and Area Type Data file*. Files in the highway network of the Florida Statewide model (version 5.1.2 Release 1) were used to obtain the

facility type and area type of the roadway on which the TTMS site was located. The highway network was visually compared to a GIS map of the Florida highway system to relate each TTMS site to a link in the Statewide model highway network. This relationship was used to assign the facility type and area type attributes to each TTMS site based on the attributes of its associated statewide model highway network link. This information was added to each record of the main dataset using a merge key created from the data in the Site ID field.

- *Posted Speed Limits at TTMS sites file.* This file contained information on the posted speed limits at TTMS sites. This information was merged into the main data set using a merge key created from data in the Site ID and Direction fields.
- *Special Events file.* This file contained information about the dates on which the counts at TTMS sites were affected by special event traffic. This information was merged into the main data set using a merge key created from data in the TTMS Site ID, year, month and date key.

The data check process revealed that 2,784 records from 29 of the TTMS hourly speed count data files were found to have a data structure that was different from the other TTMS hourly speed count data files. The records in those 29 files included unit number and direction of travel while the other speed data count files did not. The data structure of the records in the 29 files was made consistent with the other TTMS hourly speed data files before the records were added to the main data set. In addition, 288 records in the TTMS hourly speed count data had a 2-digit year of 20 (implying year 2020). These include 108 records with 0 lane volumes between the hours of midnight and 7:00 p.m. All 288 hourly records were excluded from the main dataset.

The records in the TTMS hourly count data files (for all except the 29 files mentioned above) did not include information about direction of travel and also did not include enough information to enable deduction of the direction data from other sources. However, the name of each count data file included a unit number that when combined with values from the "SITE_ID" and "ATRLane" fields in each record provided enough information to determine the direction of travel for each lane using data from the Lane Relationship file. It was therefore necessary to add the Unit Number value contained in each TTMS count data file name to each record of the associated count data file. This information was subsequently used to add the direction of travel to each count record. The lane direction information in the lane relationship file was merged into the main dataset using a merge key created from data in the TTMS Site ID, Unit No and ATR Lane number fields.

Upon completion of data processing and cleaning, the number of records in the main dataset was reduced from 9,182,224 to 8,580,315. Each record contained one hour counts for each lane at each TTMS site and descriptive information about each lane and the TTMS site.

4.2.3 Data Variables of Interest

Following data validation process, the following variables were synthesized – County, Lane Number, Month, Day, Hour, Minute, Speed Bins (15 bins in 5-mph increments including < 20 mph and > 85 mph), Total Volume by Speed, Total Volume by Classification, Light Vehicles, Heavy Vehicles, %Heavy Vehicles, Direction of Travel, TTMS Location, Urban Size, Functional Classification, AADT, K-Factor, Facility Type, Area Type, Posted Speed Limit, and Day of the Week. Figure 4.3 shows a spreadsheet extract from the main dataset with a view of some of the column titles. Each record represents one-hour counts for a lane. The complete data structure and data dictionary for the main dataset are shown in Appendix B.

	Cnty	Site_ID	Unit	Direction	ATR_Lane	Year	Month	Day	Hour	Minute	Source	mph_1_20	mph_21_25	mph_26_30	mph_31_35	mp
1	93	0010	1	N	1	10	7	1	1	0	60	0	0	0	2	
2	93	0010	1	N	1	10	7	1	2	0	60	0	0	1	0	
3	93	0010	1	N	1	10	7	1	3	0	60	0	0	0	0	
4	93	0010	1	N	1	10	7	1	4	0	60	0	0	0	0	
5	93	0010	1	N	1	10	7	1	5	0	60	0	0	0	0	
6	93	0010	1	N	1	10	7	1	6	0	60	0	0	0	4	
7	93	0010	1	N	1	10	7	1	7	0	60	0	0	0	1	
8	93	0010	1	N	1	10	7	1	8	0	60	0	0	1	7	
9	93	0010	1	N	1	10	7	1	9	0	60	1	0	5	15	
10	93	0010	1	N	1	10	7	1	10	0	60	6	2	7	12	
11	93	0010	1	N	1	10	7	1	11	0	60	5	0	0	0	
12	93	0010	1	N	1	10	7	1	12	0	60	0	0	0	0	
13	93	0010	1	N	1	10	7	1	13	0	60	0	0	0	0	

Figure 4.2 Spreadsheet Display of the Main Dataset

4.3. Private Vendors Data

The proliferation of traffic data collection by private vendors for traveler information purposes presents a great opportunity for transportation planners and engineers to tap into this maturing data collection alternative to acquire data that can be used for highway system performance evaluation and for highway system planning. Private vendors generally rely on in-vehicle cellular and/or GPS-enabled devices to probe location and time, thereby enabling the determination of speed and travel time. The commercial traffic information providers reached a consensus that enabled them to create private sector highway links known as Traffic Message Channel (TMC) network on which traffic data, both real-time and historical, are collected. The TMC network, currently maintained by NAVTEQ and TeleAtlas, seems to be a de facto standard network used virtually by all private traffic information data collectors.

The TMC network data have the advantage that they cover entire links thus providing space mean speed which is more suitable for modeling compared to time mean speed (or spot speed) that is usually collected by most States' traffic monitoring programs utilizing loop detectors. Space mean speeds reflects the average travel speeds across a link, thus average travel times (or delays) calculated using space mean speeds are more accurate that those estimated using spot speeds. However, because most TMC data do not have traffic volume information – due to the fact that only a few vehicle probes are used to capture average speeds – there is a potential of fusing data collected through loop detectors and TMC data for the purposes of improving transportation modeling process and development of congestion maps and indices. TMC data are becoming ubiquitous and are slowly maturing therefore there is a need for researching the efficacy of such data for all kinds of highway operational and planning studies.

4.3.1 Solicitation of Private Probe Data

A quick search of information revealed that there are number of providers of real time traffic information in the United States. These companies with their level of penetration (as of 2011 Ref) is shown below.

INRIX	(113 markets)
NAVTEQ	Unknown
Airsage	(127 markets)

Total Traffic Network	(95 markets)
TrafficCast	(146 markets)
SpeedInfo	(14 markets)

Following preliminary analysis of the efficacy of using probe data for the purposes of this research, a Request For Quotes (RFQ) was prepared by the Florida State University and sent to companies to provide data for this research. A number of requirements was specified in the RFQ. It was important that the provider provide complete one-year data of traffic speeds and/or volume collected throughout the 24-hr period of a particular day, seven days a week. The vendor was expected to supply data that was accurate and reflected the true ground conditions as far as traffic flow is concerned. Also, it was expected that the vendor will provide data that represents the entire Florida highway system. The expectation here was sufficient data coverage of all area and facility types in the State of Florida. Two companies responded and INRIX was chosen based on the above requirements and the cost associated with the data acquisition.

4.3.2 Overview of INRIX Data

INRIX provided speed data collected from July 1, 2010 to June 30, 2011 and covered 18,010 centerline miles of major roads and arterials in the state of Florida, on 33,700 Traffic Message Channel (TMC) links. A summary of the INRIX data is shown in Table 4.4. The time and date values in the original INRIX dataset were in Universal Time Coordinated (UTC) format. During the data processing stage, the time and date were converted to the Eastern Standard Time and Eastern Daylight Saving Time for all data records collected in the Eastern Time Zone and to Central Standard Time and Central Daylight Saving Time for all data records collect in the Central Time Zone.

In the United States, Daylight Saving Time begins at 2:00 A.M. on the second Sunday in March and ends at 2:00 A.M. on the first Sunday in November. However, the Florida Department of Transportation uses the convention of adjusting the time on traffic counters at midnight at the start and end of Daylight Saving Time. In order to adhere to the Florida DOT practice, midnight was used as the transition time when the researchers converted the INRIX data from UTC time format to Eastern Standard and Daylight Time and Central Standard and Daylight Time.

Table 4.4 Summary of INRIX Speed Data

Period Covered	July 1, 2010 to June 30, 2011
Data Collection Method	Cell phone and GPS probe
Spatial Resolution	18, 010 centerline miles (statewide)
Temporal Resolution	5-minute interval, 24-hours per day
Lane Resolution	Speed data averaged across all lanes
Variables in Data File	➢ TMC ID with lat/long information ➢ Data/Time (UTC format) ➢ Average speed (5-minute interval)
Number of Speed Records	711,351,697
Size of the Original Data File	Approximately 30 GB

It is noteworthy that INRIX uses Traffic Message Channel (TMC) system in collecting and summarizing speed data across a link. Figure 4.4 shows the extent of TMC coverage in the State of Florida.

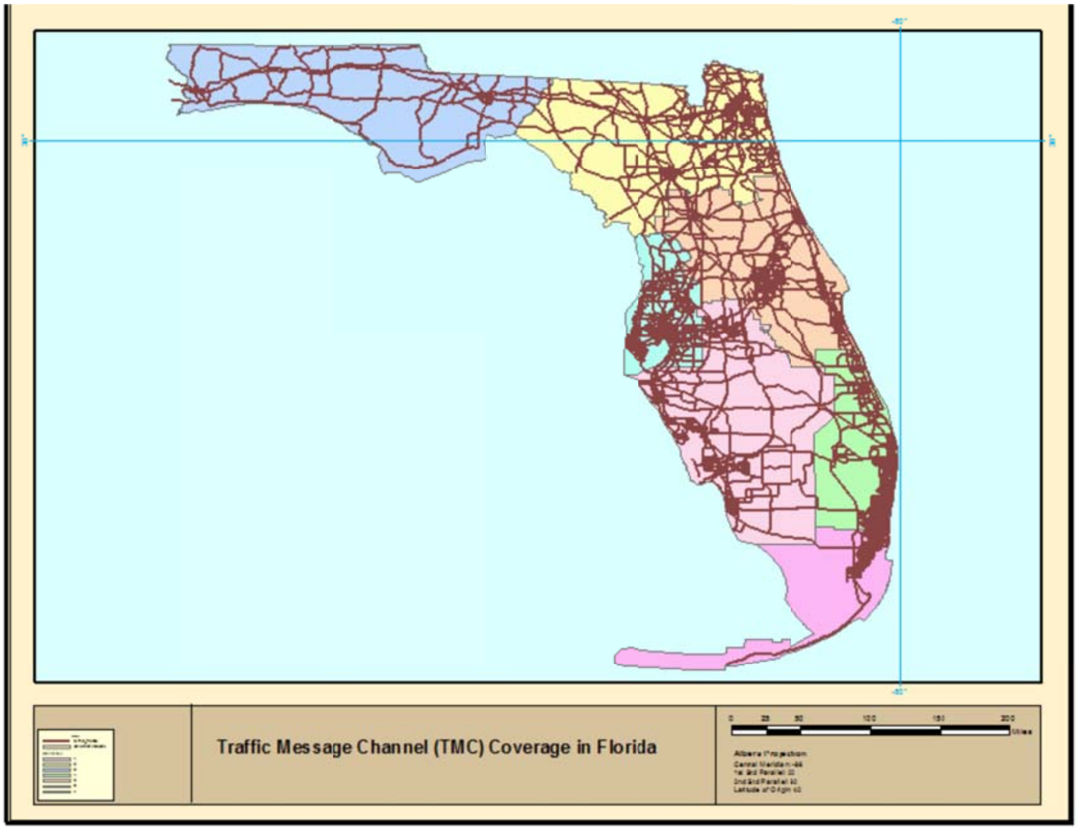


Figure 4.3 INRIX TMC Coverage in Florida

4.3.3 INRIX Data Processing and Mapping

The INRIX data set consisted of 711,351,697 average speed records based on speed observations collected at 5-minute intervals over a 12-month period on 33,696 TMCs in Florida, an average of about 58 average speed records per TMC per day. The Florida TMCs vary in length from 0.00149 miles (7.87 feet) to 42.58 miles with a median length of 0.51 miles and an average length of 1.07 miles. Table 4.5 shows the descriptive statistics for Florida TMCs and Figure 4.4 shows the frequency distribution of Florid TMCs by length.

Table 4.5 Descriptive Statistics for Florida TMCs

Number of TMCs	33,696	
Mean length	1.06960886	miles
Median length	0.51377352	miles
Minimum length	0.00149136	miles
Maximum length	42.57820372	miles
Sum	36,041.54021650	miles
Percentiles 25	0.06394206	miles
50	0.51377352	miles
75	1.22291520	miles

INRIX acquires and reports traffic flow information at the Traffic Message Channel (TMC) link level and provides coverage on all major roads and most local arterials in the USA. The traffic data is derived from:[1]

- Traffic sensors (inductive loops, radar sensors, toll tag readers) maintained by local DOTs
- Probe vehicles (trucks, busses and passenger cars) with onboard GPS devices and the capability to anonymously report vehicle speed and location.
- INRIX's Smart Dust Network which derived speed by combining data from one or more physical sensors and from probe vehicles within a specific segment of the road for a particular time window. A patented system then evaluates the input data and calculates the average speed on the road segment.

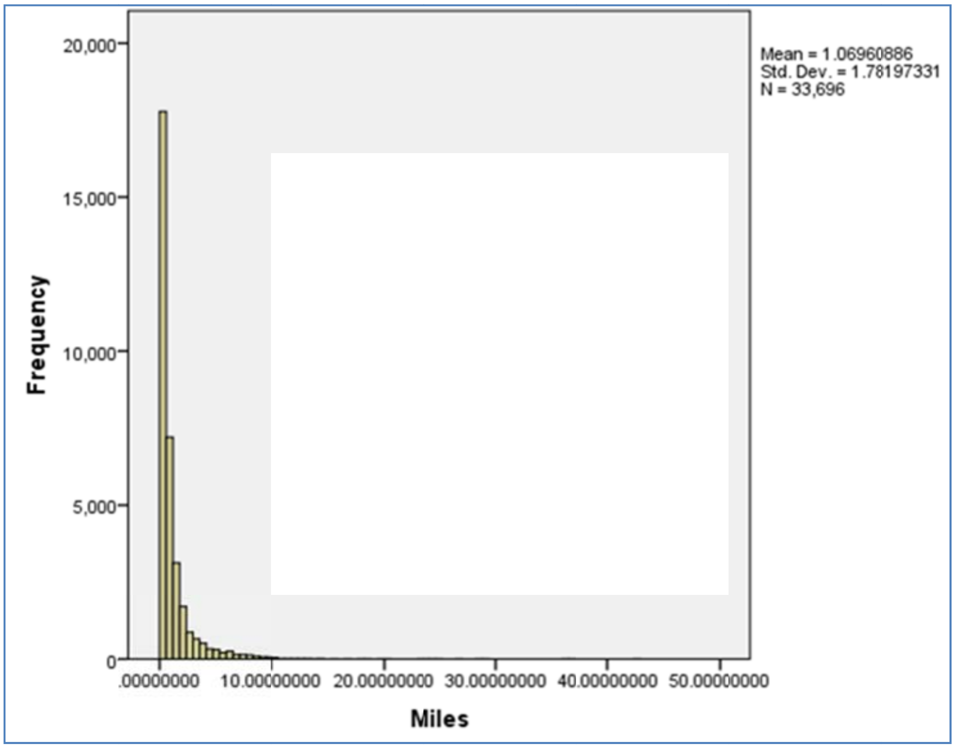


Figure 4.4 Frequency Distribution of TMCs by Length

A spatial comparison of the location of Florida DOT TTMS sites relative to the INRIX TMC paths was done to evaluate the percentage of TTMS that were covered by the INRIX TMC paths. Figure 4.4 shows that 92 percent of the TTMS that are on general use links in the Florida Statewide model are covered by INRIX TMC paths and there is 100 percent coverage of TTMS for most facility type/area type combinations. The lowest percentage of coverage for TTMS sites on general use lanes occurs on undivided arterials in rural areas where only 77 percent of the TTMS sites are covered by the INRIX TMC paths.

[1] INRIX, Inc. (2008). *INRIX Historical Traffic Flow Product Interface Guide.* INRIX Inc., Kirkland, WA. USA.

Table 4.6 Location of TTMS (by Facility and Area Type) Relative to TMC Paths

FTYPE - ATYPE (GP Lanes)*	Number of TTMS Sites			Percent	
	On TMC	Off TMC	Total	On TMC	Not on TMC
Freeway - OBD	8		8	100.0	0.0
Freeway - Fringe	1		1	100.0	0.0
Freeway - Residential	26		26	100.0	0.0
Freeway - Rural	21		21	100.0	0.0
Divided Arterial - CBD	1		1	100.0	0.0
Divided Arterial - Fringe	2		2	100.0	0.0
Divided Arterial - OBD	26	1	27	96.3	3.7
Divided Arterial - Residential	37	2	39	94.9	5.1
Divided Arterial - Rural	31		31	100.0	0.0
Undivided Arterial - CBD	1		1	100.0	0.0
Undivided Arterial - OBD	5	1	6	83.3	16.7
Undivided Arterial - Residential	12	1	13	92.3	7.7
Undivided Arterial - Rural	48	14	62	77.4	22.6
Collect - Residential	2		2	100.0	0.0
Collect - Rural	5	2	7	71.4	28.6
Toll - OBD	2		2	100.0	0.0
Toll - Residential	7		7	100.0	0.0
Toll - Rural	11		11	100.0	0.0
Grand Total	248	21	269	92.2	7.8

*Assuming INRIX data was collected from General Use lanes only.

INRIX TMC paths are not lane specific so it is not possible to differentiate speed data for HOV lanes that are adjacent to general use lanes. In such cases, the historical speed data reported by INRIX could be either the average speed in the general use lanes, the average speed in the HOV lane or the average speed across all lanes (general use and HOV). This would present an issue when using INRIX speed data for TMC paths that include both general use and HOV lanes.

Table 4.7 Location of TTMS at HOV Facilities relative to TMC Paths

FTYPE - ATYPE (HOV Lanes)*	Num. of TTMS Sites			Percent	
	On TMC	Off TMC	Total	On TMC	Not on TMC
HOV - OBD	??	??	3		
HOV - Residential	??	??	1		
Grand Total	??	??	4		

*INRIX data not lane specific so cannot deliver speeds on HOV lanes.

Figure 4.5 shows a chart of the number of TTMS (by Facility Type/Area Type Combinations) and their location relative the TMC Paths and Figure 4.6 the spatial and geographic relationship of the TTMS site to the TMC paths.

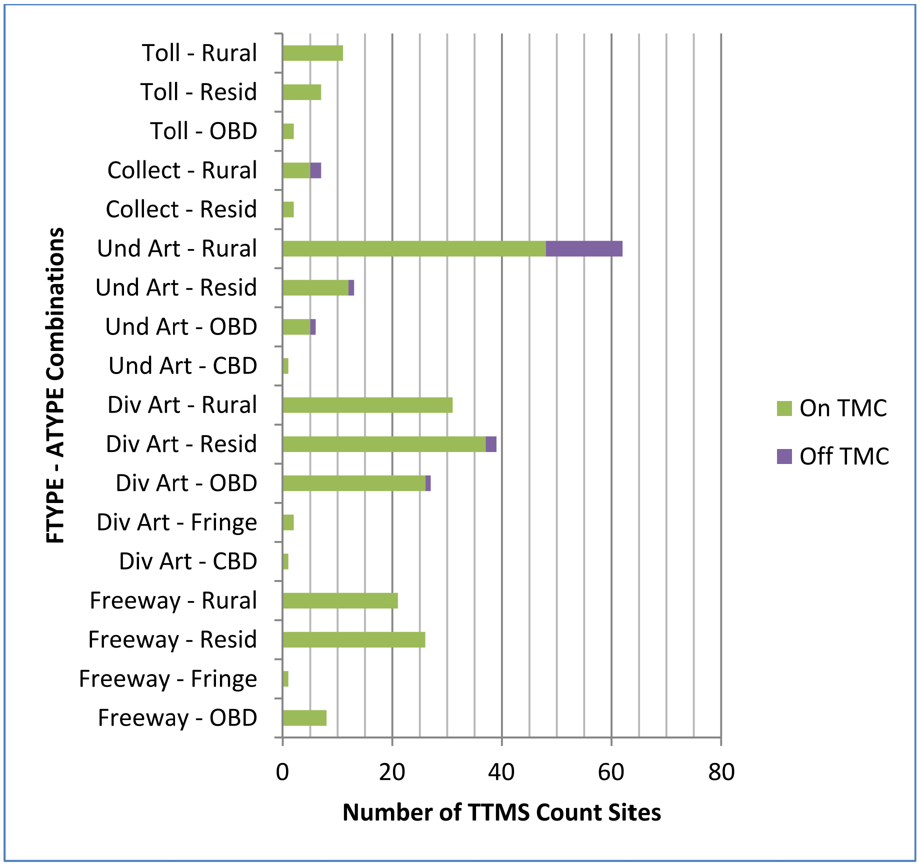


Figure 4.5 Number of TTMS (by Facility Type and Area Type) relative to TMC Paths

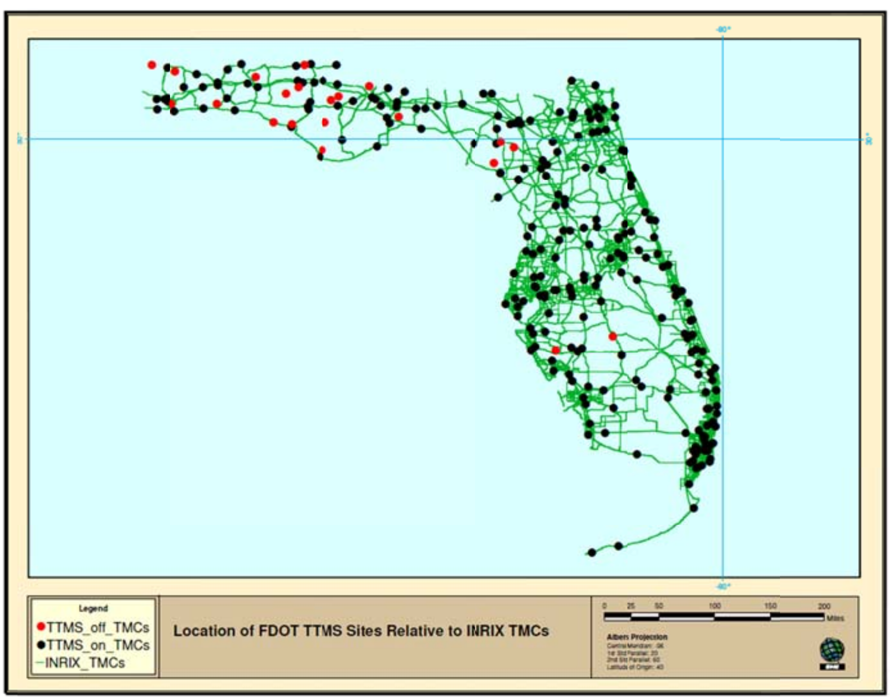


Figure 4.6 Map of TTMS Sites Relative to TMC Paths

4.3.4 Evaluation of Sample INRIX Speed Data Relative to TTMS Speed Data

This evaluation was done to understand commonalities and differences in the two data sets and to obtain insights in potential issues that would need to be considered and addressed when using the INRIX speed data during the study. The sample data consisted of INRIX speed data reported at 5-minute interval from 12 TMC paths identified in Table 4.8.

Table 4.8 TMC paths s for data evaluation

TMC	Miles	County	TTMS	Road Name	FT_AT_Desc_GP *	FT_AT_Desc_HOV **
102-04122	0.055926	86	0331	I-95	Freeway – OBD	HOV - OBD
102+04123	0.099424					
102-04865	0.729524	10	9922	I-275	Freeway – OBD	
102+04866	0.744375					
102+05696	1.232609	87	9947	US-27	Div Art – OBD	
102-05695	1.193585					
102+06802	0.900657	72	0062	US-90	Div Art – Resid	
102-06801	0.901216					
102+16121	5.274195	16	0275	SR-554	Collect – Resid	
102-16120	5.279725					
102-09884	2.167381	11	0246	SR-44	Collect – Rural	
102+09885	2.165579					

* Facility Type/Area Type Description (General Use Lanes)
**Facility Type/Area Type Description (HOV Lanes)

Sample speed data for three days (April 5, 2011 to April 7, 2011) was compared to TTMS data for the same dates at the corresponding TTMS locations shown in Table 4.8. The evaluation identified the following:

- The date and time in the INRIX data are in Corrdinated Universal Time (UTC) format while the TTMS data was in Eastern Daylight Time (EDT) for the period evaluated. Care should be takes when using a entire year of INRIX's data for all Florida locations because:
  - Some TTMS sites are in the eastern time zones while a few are in the central time zone.
  - The time in the TTMS data is affected by bi-annual adjustments for daylight saving time.
  - Converting UTC to eastern daylight time or eastern standard time would result not only in a change of time but also a change of date for some data.
- Generally, the evaluation found that the speeds in the TTMS data are higher than the INRIX speeds on non-rural arterials.
- The INRIX data shows substantial fluctuations in average speeds at 15-minute intervals.
- On urban interstate facilities, the peak period speeds in the INRIX data are significantly lower than the peak period speeds in the TTMS data for the locations and period evaluated.
- There are several hours of missing speed data in the early morning hours (generally before 7:00 AM) and at night (generally after 9:00 PM) in the INRIX sample data set for all arterials. This is also an issue (but to a much reduced extent) on interstate facilities.

The results of the speed data evaluation are shown at Appendix F to this report.

CHAPTER FIVE

ANALYSIS OF UNINTERRUPTED FLOW FACILITIES

5.1 Overview of TTMS Installed on Non-Toll Uninterrupted Flow Facilities

The Statistics Office of the Florida Department of Transportation installs and maintains traffic monitoring sites on both interrupted and uninterrupted flow facilities. Table 5.1 shows the distribution of the Telemetered Traffic Monitoring Sites (TTMS) installed on Florida limited access non-toll highways categorized by area type, number of lanes, and speed limit. The data displayed in Table 5.1 exclude uninterrupted flow facilities in which toll is collected such as the Florida's Turnpike. Table 5.1 shows that there are no sites installed on freeways with posted speed limit of 60 MPH. Overall, the distribution of the TTMS sites shows that there are data deficiencies that need to be filled to ensure complete coverage of all combinations of area type, number of lanes, and speed limit.

Table 5.1 Distribution of TTMS on Non-toll Uninterrupted Flow Facilities

Area Type	Number of Non-HOV Lanes (by direction)			
	2	3	4	5
Speed Limit = 55 MPH				
Urban[2]		2	1	1
Residential	1	1		
Rural				
Speed Limit = 60 MPH				
Urban				
Residential				
Rural				
Speed Limit = 65 MPH				
Urban	1	2		2
Residential		3		1
Rural				
Speed Limit = 70 MPH				
Urban				
Residential[3]	6	10	1	
Rural	14	5		

[2] Site 0137 has three lanes in one direction and four lanes in the opposite direction.
[3] Site 0361 has two lanes in one direction and three lanes in the opposite direction.

5.2 Analysis of Speed on Uninterrupted Flow Facilities

Table 5.2 shows the distribution of speeds on non-toll interrupted flow facilities using data collected from the TTMS sites that were displayed in Table 5.1. The statistics of interest in this analysis were the 50th percentile speed (i.e., the median speed), the 85th percentile speed, and the estimate of free flow speed, denoted in Table 5.2 as FFS. Since the raw data acquired from FDOT had speed bins aggregated on hourly basis, the first step towards generating the required statistical parameters was to calculate the harmonic mean of speeds on hourly basis using the following formula:

$$Harmonic\ Mean\ Speed = \frac{\sum_{b=1}^{b=15}(Count_{(b)})}{\sum_{b=1}^{b=15}\left(\frac{Count_{(b)}}{Speed_{(b)}}\right)} \quad \text{.......... } 5.1$$

where *b* is the speed bin index (1 to 15), $Count_b$ is the number of vehicles in speed bin "*b*", and $Speed_b$ is the mid-point of the speed range in bin "*b*".

Table 5.2 Speed Characteristics on Non-toll Uninterrupted Flow Facilities[4]

Area Type	2-Lanes			3-Lanes			4-Lanes			5-Lanes		
	50th	85th	99th	50th	85th	99th	50th	85th	99th	50th	85th	99th
Speed Limit = 55 MPH												
Urban				63	66	69	60	63	64	66	69	71
Residential	61	62	65	62	63	64						
Rural												
Speed Limit = 60 MPH												
Urban												
Residential												
Rural												
Speed Limit = 65 MPH												
Urban	67	68	71	65	66	68				67	68	70
Residential				69	71	74				68	69	71
Rural												
Speed Limit = 70 MPH												
Urban												
Residential	71	73	75	72	73	76	67	68	72			
Rural	72	73	75	70	71	74						

[4] For all days, including weekends, holidays and special event days.

5.3 Estimation and Prediction of Free Flow Speed

The hourly harmonic mean speeds calculated using Equation 5.1 above were then ranked and plotted in order to determine the average speed of vehicles for all hours. The determination of free flow speed (FFS) was based on the HCM 2010 definition of free flow speed as the average running speed under very low volume conditions. In this study, the researchers chose low volume to be ≤ 200 passenger cars per hour per lane. Thus, the hourly harmonic mean speeds were again ranked in ascending order for only those hours that had volume ≤ 200 passenger cars per hour per lane. However, since the relationship between speed and volume is parabolic for uninterrupted flow facilities, there are two flow regimes in which volume is equal to or less than 200 passenger cars per hour per lane. One regime is a free flow and the other is congested flow. In order to separate these regimes, speed density relationship which is somehow linear, as shown in Figure 5.1(b), was used with additional assumption that free flow speed on uninterrupted flow facilities would generally be higher than the posted speed limit. Traffic was considered to be free-flowing if the density corresponding to low volume (≤ 200 passenger cars per hour per lane) was ≤ 5 passenger cars per mile per lane. All observations which met these criteria were extracted and analyzed. The 50th, 85th, and 90th percentile speeds were computed as estimates of free flow speed given that the research team was not sure which percentile would reasonably represent free flow speed.

As noted in Section 5.1 above there is not sufficient number of traffic monitoring stations to cover all possible combinations of area type, number of lanes, and speed limit. The calculated speed percentiles from sites where TTMS sites are available were used as input into the model to predict missing data. A number of studies have investigated factors influencing free flow speed (HCM 2010; Bonneson *et al*., 2011; Fitzpatrick *et al*., 2003). Most of the reported studies indicated that the significant factors which influence free flow speed were speed limit, access point density, median type, curb presence, segment length, number of lanes, and area type.

In uninterrupted flow facilities, access points are controlled in such a way that they don't pose significant influence on free flow speed. In addition, curbs are mostly not included in the design of uninterrupted flow facilities, and medians are consistently designed not to affect the flow in uninterrupted flow facilities. Thus, factors with most influence on free flow speed in uninterrupted flow highways are speed limit and geometric characteristics. The 50th, 85th, and 90th percentile speeds were analyzed to study the consistency among them and decide which percentile is appropriate for estimating the free-flow speed. The model developed for predicting speed percentiles for missing data used maximum speed limit, number of lanes, and area type as the predictor variables for the percentiles. These models were specified using the following linear regression equation:

$$\text{Speed} - \text{Percentile} = \beta_0 + \sum_{i=1}^{N} \beta_i X_i + \varepsilon_i, \qquad \text{with i} = 1, 2, 3. \quad \ldots\ldots 5.2$$

where β_0 is an intercept of the model, β_1 are the coefficients of a predictor variables X_i and ε_i are error terms. In this case, X_1 is the speed limit, X_2 is the area type, and X_3 is the number of lanes. The statistical analysis of the models is shown in Table 5.3.

Table 5.3 Regression model outputs

Model	Variable	Label	Parameter estimate	Standard error	*t*-value	Pr>|*t*|
50th %ile	Intercept	Intercept	39.3439	8.4341	4.66	0.0009
	Speed limit	SPL	0.3631	0.1178	3.08	0.0116
	No. of lanes	NumLan	0.5782	0.5474	1.06	0.3157
	Area type	ATYPE	1.9907	0.7214	2.76	0.0201
85th %ile	Intercept	Intercept	40.2118	8.9151	4.51	0.0011
	Speed limit	SPL	0.3870	0.1245	3.11	0.0111
	No. of lanes	NumLan	0.4421	0.5786	0.76	0.4625
	Area type	ATYPE	1.6984	0.7626	2.23	0.0401
99th %ile	Intercept	Intercept	39.8958	9.4705	4.21	0.0018
	Speed limit	SPL	0.4249	0.1323	3.21	0.0093
	No. of lanes	NumLan	0.3767	0.6146	0.61	0.5537
	Area type	ATYPE	1.5967	0.8101	1.97	0.047

Notes: ATYPE is a categorical variable with 3 three levels: ATYPE= 1 for Urban; ATYPE=2 for Residential; ATYPE = 3 for Rural areas.

In all three models shown in Table 5.3, the variables with significant predictive power are the speed limit and the area type due to extremely low *p*-values. The significance of speed limit on the prediction of speed percentiles has also been reported in other studies (Fitzpatrick *et al.*, 2003). The variable number of lanes shows higher *p*-values suggesting that this variable does not have significant influence on the free flow speed. However, this variable is retained in the speed percentile models because our analysis is disaggregating the facilities by area type, speed limit as well as the number of lanes. Table 5.4 displays the predicted free flow speeds following the application of the models displayed in Table 5.3. The predicted values are highlighted in red. Again, it should be noted that the free flow speed was predicted only on segments in which data were not available either because such segments do not exist in Florida or no permanent count stations are installed in those segments.

Table 5.4 Free Flow Speeds on Non-toll Uninterrupted Flow Facilities[5]

Area type	2-Lanes			3-Lanes			4-Lanes			5-Lanes		
	50th	85th	99th	50th	85th	99th	50th	85th	99th	50th	85th	99th
Speed Limit = 55 MPH												
Urban	62	64	66	64	65	67	63	64	65	67	70	72
Residential	61	63	65	63	65	66	66	67	68	66	67	68
Rural	66	67	69	67	68	69	68	68	70	68	69	70
Speed Limit = 60 MPH												
Urban	64	66	68	65	66	68	65	67	68	66	67	69
Residential	66	68	69	67	68	70	67	69	70	68	69	70

[5] For all days, including weekends, holidays and special event days.

Area type	2-Lanes			3-Lanes			4-Lanes			5-Lanes		
	50th	85th	99th	50th	85th	99th	50th	85th	99th	50th	85th	99th
Rural	68	69	71	69	70	71	69	70	72	70	71	72
Speed Limit = 65 MPH												
Urban	67	68	70	66	67	67	67	69	71	66	69	70
Residential	68	70	71	69	72	74	69	71	72	68	70	71
Rural	70	71	73	71	72	73	71	72	74	72	73	74
Speed Limit = 70 MPH												
Urban	68	70	72	68	70	72	69	71	73	70	71	73
Residential	71	73	75	72	74	75	71	72	74	72	73	75
Rural	72	74	76	72	74	76	73	74	76	74	75	76

Table 5.4 contains 50th, 85th, and 99th percentile speeds. The question is – which of these percentiles reasonably represents free flow speed? Further analysis of the relationship between speed limit and the percentiles was needed in order to arrive at the correct percentile for free flow speed prediction. In Figure 5.1 below, the 90th percentile speed has higher *R*-squared value compared to 50th and 85th percentile speeds. This would suggest the use of 90th percentile speed as the estimate of the free flow speed. However, since most agencies report using the 85th percentile speed as the basis for their speed limits (Fitzpatrick *et al.* 2003), and given a strong correlation between free flow speed and speed limit, therefore the 85th percentile speed would be a consistent estimate of free flow speed.

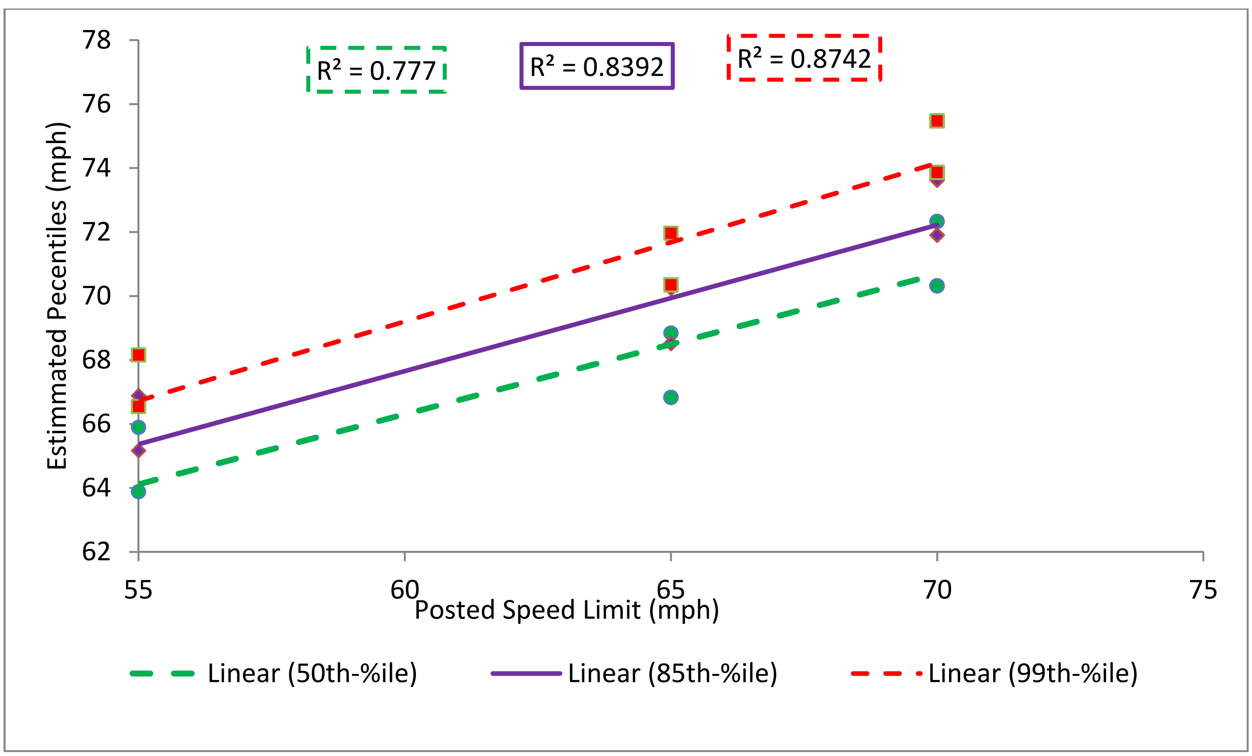


Figure 5.1 Variation of percentile speeds with speed limit

5.4 Analysis of Volume on Uninterrupted Flow Facilities

Presence of trucks in traffic affects capacity of a highway; therefore, it was important to analyze truck traffic. The truck percentage was calculated as the percentage of heavy vehicles using FHWA Scheme F. In this scheme buses, trucks, and other vehicles with six or more tires are classified from Class 4 to Class 13[6]. The analysis dataset included hourly vehicle classification counts. However, in the TTMS data used to create the analysis dataset, the total of the vehicle classification count did not always equal the total of the speed count for each record. Consequently, the number of trucks for each record was not derived directly from the vehicle classification count. Instead, the vehicle classification count was used to determine the percentage of trucks for each record. The percentage of trucks was then applied to the speed count for that record to obtain the estimate of the percentage of trucks as shown in Equation 5.3.

$$\#\,\text{of trucks}_{(SiteD,Hr)} = \text{Total Speed Count}_{(SiteD,Hr)} \text{x} \frac{\sum_{c=4}^{c=13}\left[\text{Count}_{(c,SiteD,Hr)}\right]}{\sum_{c=1}^{c=15}\left[\text{Count}_{(c,SiteD,Hr)}\right]} \text{x}100 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots 5.3$$

where c is vehicle class ranging from Class 1 to Class 15, $SiteD$ is the TTMS site by direction of traffic travel, Hr is the hour of the day from 1 to 24, and $\text{Count}_{(c,SiteD,Hr)}$ is the number of vehicles in the particular vehicle class for a particular hour at a particular site.

Table 5.5 shows the descriptive statistics of traffic volumes in vehicles per hour for all sites on non-toll uninterrupted flow facilities. Although the data was screened to remove outliers both on the lower and upper ends, a decision was made to calculate the 99th percentile volume and use this value rather than maximum value as an indicator of practical capacity because observed maximum values may still have happened at random due to incidents that could have happened at the time of data collection.

Table 5.5 Volume Characteristics on Non-Toll Uninterrupted Flow Facilities

Area Type	Speed Limit	# of Lanes	# of Obs.	Min.	Max.	Mean	99th Percentile	Std. Dev.	% Trucks
Residential	70	3	50,376	40	2,277	698	2,031	469	12.19
	70	4	17,245	34	2,417	804	2,037	536	20.40
	65	5	17,052	60	2,028	650	1,766	394	5.53
	55	2	40,651	1	1,674	512	1,498	319	5.36
Urban	65	3	48,985	28	2,354	877	2,034	570	14.74
	65	5	27,812	41	2,016	722	1,798	430	8.78
	65	2	39,203	14	1,627	469	1,591	298	6.63
	55	3	38,049	21	1,937	664	1,591	461	7.69
	55	4	8,605	68	1,698	748	1,491	409	6.67
Rural	70	3	17,345	1	1,685	684	1,511	408	27.36
	70	2	232,367	1	1,628	306	983	213	27.41

[6] Traffic Monitoring Handbook, Florida Department of Transportation. Available at www.dot.state.fl.us/planning/statistics/tmh/tmh.pdf.

The results in Table 5.5 suggest that the rural freeway sections for which data were acquired through TTMS are operating way below capacity. In addition, urban and residential freeways have higher hourly volumes but not seem to reach congestion levels seen in other United States metropolitan areas. It should be noted that the values in Table 5.5 are in vehicles per hour while it is common in literature for hourly flows and capacities to be expressed in passenger car per hour. The truck percentages shown in Table 5.5 were used to calculate equivalent passenger cars as discussed in the following section.

5.4.1 Conversion of Vehicles to Equivalent Passenger Cars

A PCE factor of 1.5 was used to convert the hourly number of trucks to an equivalent number of passenger cars. The value of this factor is consistent with the PCE factor that is used in the Florida Statewide model.

5.4.2 Determination of Practical Capacity

The volumes displayed in Table 5.5 were converted into passenger car equivalents using conversion procedures and factors discussed in the preceding section. The resulting values are displayed in Table 5.6. A more detailed display of the observed volumes including statistical analysis by TTMS site is shown in Appendix C.

Table 5.6 Observed volumes on Uninterrupted Non-toll Facilities in pce/hr/lane

Area Type	Number of Lanes	Minimum Flow	Maximum Flow	99th Percentile Flow	FDOT
Urban	2	57	1,697	1,504	1920
Urban	3	28	2,354	2,056	2025
Urban	4	68	1,491	1,410	2055
Urban	5	41	1,798	1,569	2075
Residential	2	1	1,627	1,277	1790
Residential	3	15	2,277	1,765	1845
Residential	4	34	2,417	2,209	1875
Residential	5	60	1,766	1,641	1890
Rural	2	1	1,628	937	1750
Rural	3	1	1,685	1,362	1800

Some facilities did not experience optimal flow conditions on any hour of the day during the period July 1, 2010 to June 30, 2011 and in such cases it is expected that the 99th percentile flow would be less than the practical capacity for those facilities. In comparison to the values recommended by FDOT, the observed 99th percentile flows are lower except for 3-lane urban freeways and 4-lane residential freeways.

5.5 Analysis of Toll Facilities and HOV Lanes

The same procedure used to analyze uninterrupted flow (non-HOV/Toll) facilities was followed in the analysis of facilities in which drivers pay tolls or of which HOV lanes exist. There were 20 permanent count stations (TTMS) installed on Toll/HOV facilities. The speed and flow data from these sites was screened to identify extreme values (outliers) and clean out invalid observations before further analysis was conducted. Free flow speeds were determined from sites in which data exists and in some areas free flow speeds were predicted using algorithms described earlier. The results are displayed in Table 5.7.

Table 5.7 Free Flow Speeds on Toll Facilities

Area Type	2-Lanes			3-Lanes			4-Lanes			5-Lanes		
	50th	85th	99th	50th	85th	99th	50th	85th	99th	50th	85th	99th
Speed Limit = 55 MPH												
Urban	64	65	67	65	67	68	62	63	66	68	69	71
Residential	67	73	75	63	65	66	66	67	68	66	67	68
Rural	66	67	69	67	68	69	68	68	70	68	69	70
Speed Limit = 60 MPH												
Urban	66	69	71	65	66	68	65	67	68	66	67	69
Residential	66	68	69	67	68	70	67	69	70	68	69	70
Rural	68	69	71	69	70	71	69	70	72	70	71	72
Speed Limit = 65 MPH												
Urban	67	68	70	67	76	83	67	69	71	66	69	70
Residential	68	70	71	65	68	70	69	71	72	65	73	79
Rural	73	77	79	71	72	73	71	72	74	72	73	74
Speed Limit = 70 MPH												
Urban	68	70	72	68	70	72	69	71	73	70	71	73
Residential	74	79	82	71	78	81	68	77	80	72	73	75
Rural	72	74	76	72	78	81	73	74	76	74	75	76

In most cases, the speed values observed and predicted in toll facilities are higher than those found in non-toll facilities with the same posted speed limit. This could be explained by the level of enforcement congestion in toll facilities compared to non-toll facilities.

5.6 Development of Speed-Volume Curves

In order to establish the validity of the TTMS data, the relationships among traffic variables were analyzed. The objective of this analysis was to identify highway segments, in which TTMS are installed, which have traffic flow relationships conforming to fundamental traffic flow diagrams. Plots of speed-volume, speed-density, volume-density, and speed-headway were examined to determine if the traffic flow relationships are typical. Figure 5.2 shows fundamental traffic flow relationships from TTMS data collected on Interstate 95 in Pompano Beach, Florida. These plots seem to be reasonable and follow trends similar to those found in literature. Similar plots were produced for all sites. The sites which showed traffic behavior similar to that in Figure 5.2 were classified as "well-behaving" sites and were used in the next state of the analysis, which was to fit volume delay functions. These plots were also used in deriving flow regimes for situations in which demand exceeds capacity.

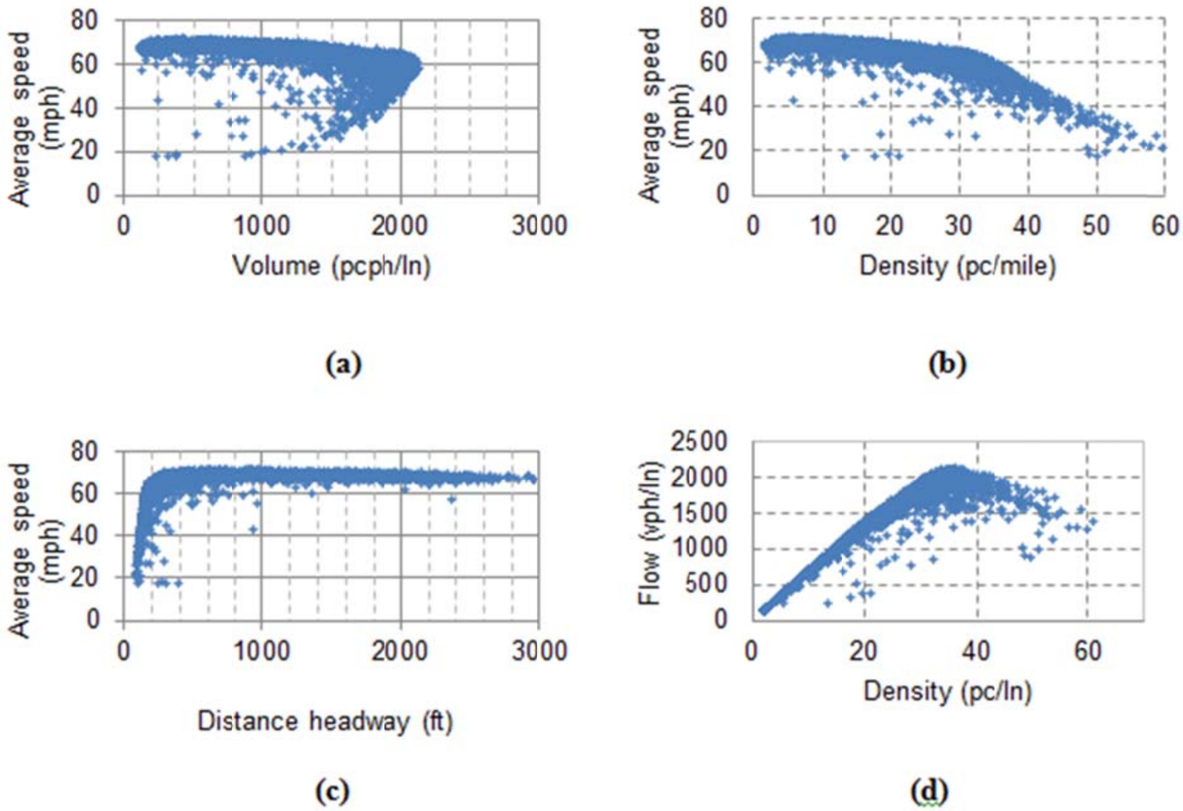


Figure 5.2 Traffic flow plots in a segment on I-95 in Pompano Beach, Florida.

The traffic assignment algorithms used in travel demand forecasting models require the computation of travel speeds in both undersaturated conditions and oversaturated conditions in which the demand-to-capacity ratios exceed 1.0. Similarly, fitting, calibration, and validation of VDFs entail performance tests for the regime in which the v/c ratio or the demand-to-capacity ratio is greater than 1.0. An analyst can visualize the different flow regimes through speed-flow, speed-density, flow-density, and speed-headway plots together with the use of queue and shockwave theories to determine the threshold between undersaturated conditions and oversaturated conditions. Some studies (Dowling and Skabardonis, 1993; Skabardonis and Dowling, 1997) attempted to calibrate VDFs for conditions in which v/c was ≥ 1.0. Because it was very difficult to observe and collect traffic flow data in situations in which demand exceeds capacity, these studies used simulation models instead to generate the requisite data. Two other studies (Huntsinger & Rouphail, 2011; Hansen *et al*., 2005) utilized local data from detectors and applied the concept of measuring the queue at a bottleneck. The number of vehicles queued was then added to the capacity at the bottleneck to estimate demand for each time interval. In these studies, the data was collected from detectors located on at least three locations along the study link – that is, upstream, midsection, and downstream. This set up enabled the use of queue lengths measured at the bottleneck to derive demand for the link segment.

In our current study, data from a single point location loop detector was used to estimate demand above the capacity of the segment. The procedure developed to estimate this demand was based on the understanding of the traffic flow dynamics. In traffic flow, headway and speed decrease gradually as volume increases towards capacity. This is illustrated in Figure 5.3 as region A. Headway and speed decrease then becomes steeper in the congested flow regime (region B). At capacity, v_c, the headway and speed reach optimum values, h_c and U_c, respectively. These values were used to derive flow in region C based on flow in region B.

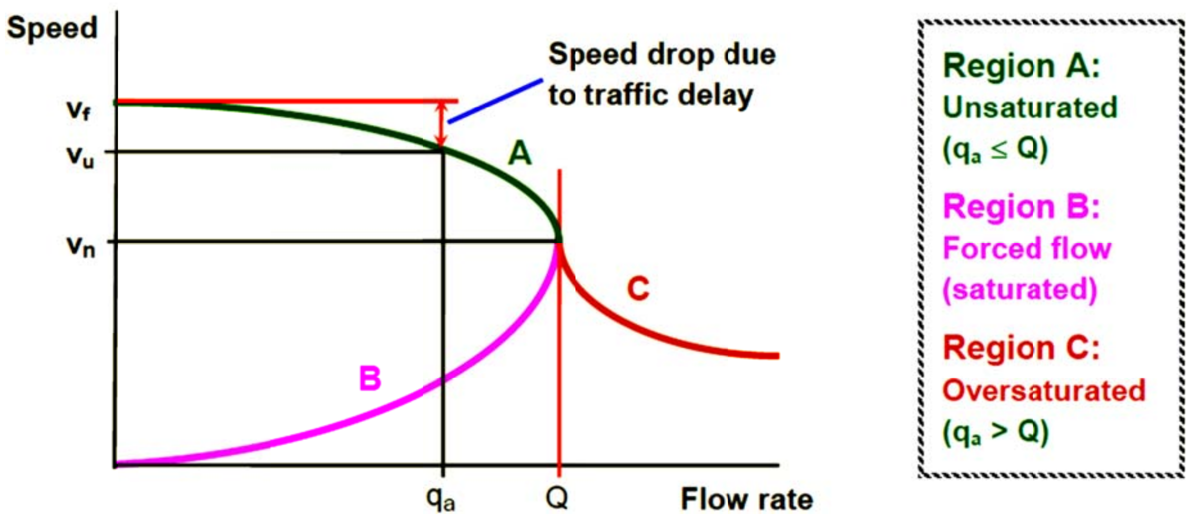


Figure 5.3 Speed versus flow rate in uninterrupted traffic stream (Akcelik, 2003)

To estimate the demand (flow in region C) the following steps were followed:

(i) Determine flow at capacity, v_c

(ii) Determine the optimum speed at capacity, u_c

(iii) Calculate the optimum average headway at capacity, $h_c = u_c/v_c$

(iv) Calculate average headway for all data points, $h = u/v$

(v) Estimate the demand as:

$$v_{demand} = \begin{cases} v_c + (v_c - v), \ if \ \frac{U}{v} \leq \frac{U_c}{v_c} \text{ and } U \leq U_c \\ v, otherwise \end{cases} \quad 5.4$$

Following the derivation of demand volume from field data, the relationship between demand volume and average travel speed was further scrutinized. The v/c ratio was calculated and plotted against the average travel speed. Figure 5.4 shows one of the plots. The data used for the plot in Figure 5.4 is from a basic freeway segment on Interstate 95 in Davie, Florida. Other curves of speeds against *v/c* ratio were plotted for other TTMS sites. The fitting of the volume delay functions utilized these plots.

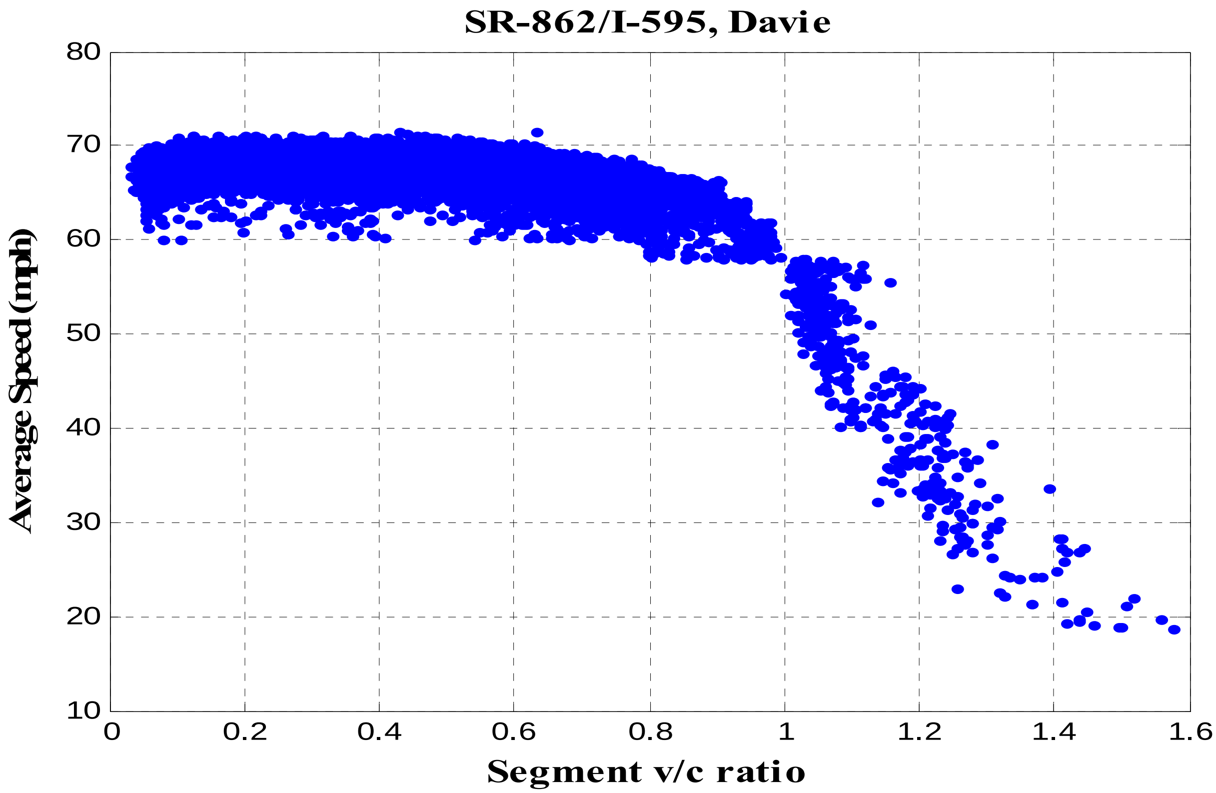


Figure 5.4 Speed variation as a function demand-to-capacity ratio

5.7 Fitting of Volume Delay Functions

A number of volume delay functions discussed in Chapter 2 were analyzed to determine their efficacy in predicting delay based on the loading on a facility represented by the v/c ratio. The fitting process discussed in this Chapter is for uninterrupted flow facilities only that includes non-toll and toll facilities. Also analyzed are volume delay functions suitable for fitting operations in HOV lanes. The following sections describe the process of fitting volume delay functions to the field data acquired from the telemetered traffic monitoring stations (TTMS).

5.7.1 Model Parameters, Predictor, and Response Variables

In traffic forecasting models, traffic is assigned to available transit or roadway routes using a mathematical algorithm that determines the amount of traffic as a function of time, volume, capacity, or impedance factor. There are three common methods for trip assignment – all or nothing assignment, diversion assignment, and capacity restraint assignment. The Florida Standard Urban Transportation Modeling Structure (FSUTMS) uses capacity restraint assignment model in the form of a mathematical equation commonly known as volume delay function (VDF), which can be represented as $U = S(v, \theta)$ where U is the vector of estimated speed on the link carrying traffic volume in vector, v and θ represent vector of parameters that describe the characteristics of the link. Different capacity restraint equations have been developed and tested and are available for use. There are two basic characteristics common to capacity restraint models: (i) they are non-linear relationships, and (ii) they use the v/c ratio as a common factor. The underlying premise of a capacity restraint model is that the travel time on any link is related to the traffic volume on that link.

The major task in the analysis of volume delay functions was to estimate the model parameters to reflect how link average travel speed (or travel time) will be affected given variation in link volume. As noted above, most capacity restraint models have non-linear relationships; thus, establishing the VDF parameters was somewhat tricky and laborious since ordinary regression techniques could not be applied. The following section describes the technique that was used in tweaking the model to enable reasonable fit to the field data.

5.7.2 Estimation of Parameters

The Gauss-Newton (GN) method is a well-known iterative technique used regularly for solving the nonlinear least squares problem (NLSP). The method is specified as

$$\min \Phi(\theta) = {}_{\theta} \frac{1}{2} \|f(\theta)\|_2^2 \quad \text{.......... 5.5}$$

where θ is an *n*-dimensional real vector and *f* is an *m*-dimensional real vector function of θ (Ortega & Rheinboldt, 1970; Pereyra, 1967). Problems of this nature arise commonly from engineering applications in optimal control, filtering, and in data fitting.

In our data, there are m observed data of link volumes and average travel speeds (v_i, U_i) that need to be fitted with a model S(θ,U), determined by a vector θ of n parameters. If the i^{th} component of $f(\theta)$ is defined as $f_i(\theta) = S(\theta, v_i) - U_i$, then the solution to the NLSP (Equation 5.5) gives the best model fit to the data in the sense of producing the minimum sum of square errors. In the nonlinear least squares problem (NLSP) defined in (Equation 5.5), the assumption is that $f: \mathbb{R}^n \to \mathbb{R}^m$ is a nonlinear, twice continuously Fréchet differentiable function (Stoer & Bulirsch, 1980).

The Jacobian of the function *f is* denoted by $J(\theta) = f'(\theta)$. The gradient and Hessian of $\Phi(\theta)$ are then given by Ortega & Rheinboldt (1970):

$$\nabla \Phi(\theta) = \mathrm{J}^T(\theta)\mathrm{f}(\theta) \quad \text{.......... 5.6}$$

$$\nabla^2 \Phi(\theta) = \mathrm{J}^T(\theta)\mathrm{J}(\theta) \quad \text{.......... 5.7}$$

Therefore, finding the stationary points of Φ is equivalent to solving the gradient equation using the Newton's method:

$$\mathrm{G}(\theta) = \nabla \Phi(\theta) = \mathrm{J}^T(\theta)\mathrm{f}(\theta) = 0 \quad \text{.......... 5.8}$$

This is an iterative method which was implemented in MATLAB using the steps narrated below:

Step 0: Choose initial $\theta_0 \in \mathbb{R}^n$
Step 1: Repeat until convergence:
Step 1(a): Solve $\mathrm{J}^T(\theta_k)\mathrm{J}(\theta_k)\varepsilon_k = -\mathrm{J}^T(\theta_k)\mathrm{f}(\theta_k)$
Step 1(b): Set $\theta_{k+1} = \theta_k + \varepsilon_k$

This iterative process is stopped when the convergence criterion, $|\theta_{k+1} - \theta_k| = 10^{-16}$, is met.

5.7.3 Goodness-of-fit Measures

A number of goodness-of-fit measures can be used to evaluate the overall performance of predictive models. Popular among them are the root-mean-square error (*RMSE*), the root-mean-square percent error (*RMSPE*), the mean error (*ME*), and the mean percent error (*MPE*) statistics. These statistics quantify the overall error of the model. Percent error measures provide information on the magnitude of the errors relative to the average measurement directly. The *RMSE* and *RMSPE* penalize large errors at a higher rate relative to small errors. The two measures are given by

$$RMSE = \sqrt{\frac{1}{N}\sum_{i=1}^{N}\left(U_i^{pred} - U_i^{obs}\right)^2} \quad \text{..........} \quad 5.9$$

$$RMSPE = \sqrt{\frac{1}{N}\sum_{i=1}^{N}\left(\frac{U_i^{pred} - U_i^{obs}}{U_i^{obs}}\right)^2} \quad \text{..........} \quad 5.10$$

where U_i^{obs} and U_i^{pred} are the averages of observed and predicted speeds at time period *i* calculated from all available data – i.e., several days of observations.

Another measure that provides information on the relative error is Theil's inequality coefficient, TIC:

$$TIC = \frac{\sqrt{\frac{1}{N}\sum_{i=1}^{N}\left(U_i^{pred} - U_i^{obs}\right)^2}}{\sqrt{\frac{1}{N}\sum_{i=1}^{N}\left(U_i^{pred}\right)^2} + \sqrt{\frac{1}{N}\sum_{i=1}^{N}\left(U_i^{obs}\right)^2}} \quad \text{..........} \quad 5.11$$

where *TIC* is circumscribed as $0 \leq TIC \leq 1$. When $TIC = 0$, it signifies perfect fit between observed and predicted speeds from the model. When *TIC* = 1 it implies the worst possible fit.

5.7.4 Results of the Fitted VDFs

Estimation of model parameters for four most commonly used volume-delay functions (VDFs) was conducted using curve fitting algorithm described in section 5.7.2 which was implemented in MATLAB. The estimation was conducted for four facility types namely freeways or expressways, toll roads and HOV or HOT lanes as shown in Table 5.8. Each category of facility type comprises of three area types distinguished by land uses; urban, residential and rural.

Table 5.8 Parameter Estimates for Fitted Models

		Facility and Area Type							
		Freeways/Expressways			**Toll Roads**			**HOV/HOT Lanes**	
Function	**Parameters**	**Urban**	**Resid.**	**Rural**	**Urban**	**Resid.**	**Rural**	**Urban**	**Resid.**
Fitted BPR	α	0.263	0.286	0.15	0.162	0.25	0.32	0.32	0.33
	β	6.869	5.091	5.61	6.34	7.9	6.71	8.4	8.6
Conical	β	18.390	18.39	15.06	18.39	15.064	15.064	18.55	18.7
	α	1.029	1.029	1.04	1.029	1.036	1.036	1.028	1.028
Modified Davidson	J	0.009	0.0092	0.0099	0.008	0.0099	0.0099	0.009	0.0089
	μ	0.950	0.949	0.951	0.94	0.952	0.940	0.95	0.947
Akcelik	J	0.100	0.101	0.099	0.11	0.098	0.097	0.09	0.08

The models fitted were BPR function, conical delay function, modified Davidson's function, and Akcelik function. The models were then plotted against observed field data for visual analysis as shown in Figure 5.5 below. This figure shows results of a typical urban freeway segment located in Pompano Beach, Florida on Interstate 95 (I-95).

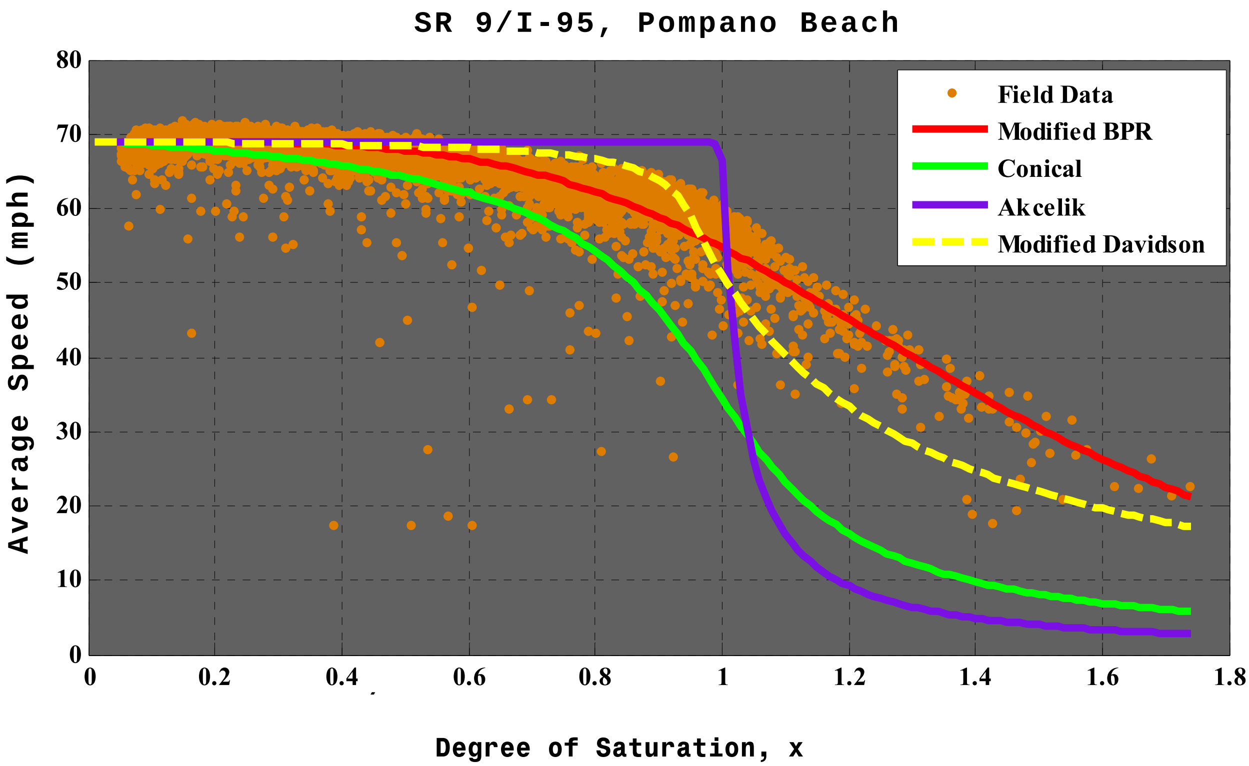


Figure 5.5. Speed-volume relationship for fitted VDFs and field data

The results in Figure 5.5 show that modified or fitted BPR fits the data well, followed by modified Davidson, conical delay function, and lastly Akcelik function. However, these results are not the final judgment to which VDF performs better compared to others. This is due to the fact that, in a congested network, a VDF will perform differently given different facility types. For that reason, the selection of VDF for a particular facility type and area type needs sturdy knowledge of transportation network behavior under different congestion levels and different traffic controls. It is obvious that, the effect of change in congestion, near or at capacity, will have different impact on travel speed for a freeway link compared to a signalized arterial link. Speed tends to deteriorate faster in shorter links than in longer links when demand is close to capacity. Therefore, the selection of a VDF for a particular facility type should not only rely on statistical performance measures (Table 5.9) such as root mean square error (RMSE) or coefficient of determination (R^2) but also to account for sensitivity of link travel speed to change in congestion or demand, and the performance when implemented in the travel demand forecasting model. The sensitivity tests and the results of the model run which will assist in the recommendations on the suitability of each VDF will be discussed in Chapter 7.

Table 5.9. Goodness of Fit Statistics for Calibrated VDFs

Model	RMSE	MSE	TIC	Iterations	R-Square
Fitted BPR	2.888	8.339	0.0020	17	0.710
Conical	5.074	25.745	0.0440	6	0.551
Modified Davidson	2.214	4.902	0.0018	8	0.878
Akcelik	4.374	19.134	0.0040	11	0.610

CHAPTER SIX

ANALYSIS OF INTERRUPTED FLOW FACILITIES

6.1 Overview

The Highway Capacity Manual describes interrupted flow facilities as a category of roadways characterized by signals, stop signs, or other fixed causes of intermittent delay or disruption to the traffic stream. Traffic flow patterns on interrupted flow facilities are the result not only of vehicle interactions and the facility's geometric characteristics, but also of the traffic control used at intersections and the frequency of access points along the facility (FDOT, 2013). Traffic signals, for instance, allow designated movements to occur only during portions of the signal cycle, and therefore affect both flow and capacity as the facility is not available for continuous use. Traffic signals also create platoons of vehicles that travel along the facility as a group. By contrast, the all-way STOP controlled intersections and roundabouts discharge vehicles more randomly, creating periodic but sometimes small gaps in traffic at downstream locations.

Estimation of practical capacity and free flow speed, which are the basic inputs to a volume delay function necessary for determining travel time on a roadway, is more challenging in interrupted flow facilities than in uninterrupted flow facilities. The challenge can be explained as follows. In uninterrupted flow operations, the speed-flow relationship is discernibly consistent with fundamental diagrams of traffic flow commonly seen in literature as reproduced in Figure 6.1b. The data in Figure 6.1b was collected from a basic segment along Interstate 95 freeway in Pompano Beach, Florida. In Figure 61.b, the speed has a defined relationship with flow in undersaturated and oversaturated conditions. Thus, it is relatively easy to estimate free flow speed and capacity from the resulting speed-volume curves, thus rendering modeling of the decay of travel speed relative to the increase in traffic volume on a roadway segment.

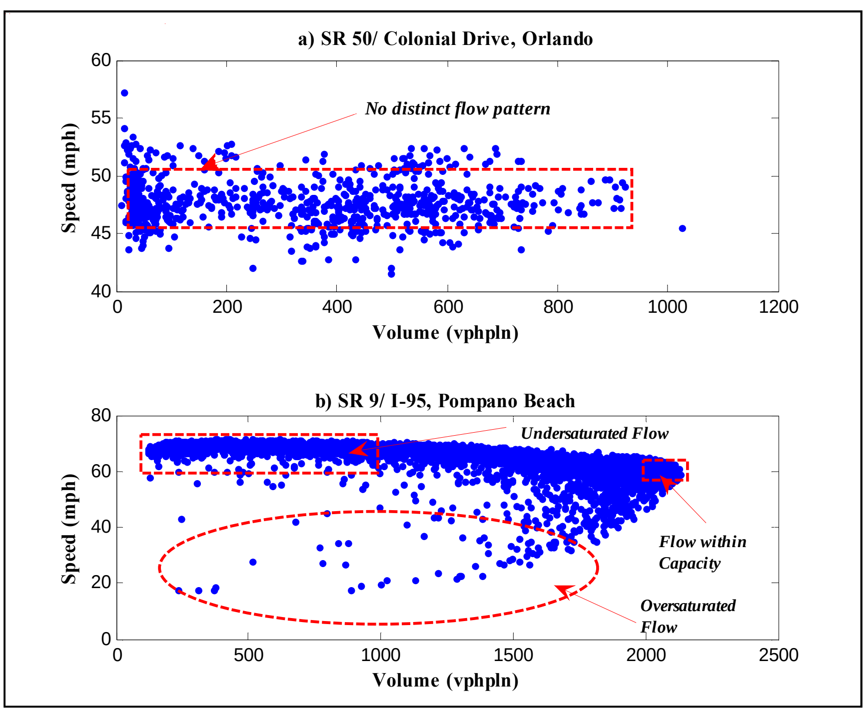


Figure 6.1 Speed-Volume Curves in Uninterrupted and Interrupted Flow Facilities

Traffic flow analysis of interrupted flow operations in a corridor characterized by intersecting streets poses a challenge of modeling speed (and volumes) in the corridor as mid-block speed and volume may not only be different from other mid-blocks but is also different from intersection traffic speeds (and volumes) due to turning movements and the alternating stop-and-go operations of intersections. Thus, collecting data using TTMS installed in the midblock would not capture traffic flow characteristics close to the intersection and therefore free flow speed or practical capacity at midblock are not necessarily the same as intersection capacity or operating speed. In addition, in oversaturated situations, queue form upstream of intersections resulting in difficulty in modeling speed-flow relationship downstream.

At the beginning of this study, it was envisaged that TTMS data will be used for estimation of volume delay functions and updating the FSUTMS free-flow speed and capacity lookup tables for both uninterrupted and interrupted flow facilities. The inconsistency of speed-flow relationship in signalized arterials (Figure 6.1b) dictated the need to study the characteristics of this group of facility type in isolation from uninterrupted flow facility types. The analysis of interrupted flow facilities as far as the estimation of free flow speed, estimation of practical capacity, and fitting of volume delay functions required a robust process of data collection. The data needed for analysis included speed data, volume data, and traffic signal data. Because most of TTMS on arterial streets are installed midblock away from intersections, it was important to acquire additional data related to intersection turning movements. To this end, traffic simulation was used to supplement data collected in midblock.

6.2 Site Selection

In the Florida Standard Urban Transportation Modeling Structure (FSUTMS) speed-capacity lookup tables there are 16 area types and 30 interrupted facility types summing up to 480 distinct speed-capacity values. To analyze and update all these values would require the collection of data on 480 different segments. Due to time and funding constraints, the workload was reduced by collapsing the area types into three categories, namely urban, residential, and rural. The urban category is comprised of CBD, fringe area of CBD and OBD. The original FSUTMS classification of residential and rural area categories was retained. The facility types were grouped according to their median type (divided or undivided) and by the speed limit. The categorization by speed limit was consistent with the classification found in the Florida Department of Transportation 2013 Quality/Level of Service Handbook (FDOT 2013). The handbook classifies signalized arterials with posted speed limit of 40 MPH or higher as Class I and Class II are signalized arterials with speed limit of 35 MPH or lower. Table 6.1 shows the end result of the classification system used in this project.

An equitable representative method was devised to ensure that various segments that could have significant influence in most area-facility type combinations were represented in the data collection process and subsequent analyses. Generally, a representative facility type represents typical characteristics of the group type. In addition, the selection of facility types has to ensure that the selected segments represent different congestion levels experienced in the network. In addition to selecting segments which represent diverse characteristics, the researchers had to ensure that the segments also met traffic criteria necessary for fitting volume delay functions. In travel demand forecasting models, the traffic assignment algorithms compute travel speeds in both undersaturated conditions and oversaturated conditions. In order to develop a function that can accomplish such computation precisely, the fitting, calibration, and validation of VDFs need performance tests for the regime in the demand-to-capacity ratio exceeding 1.0.

Table 6.1 Arterial Classes by Speed Limit

Area Type	Class I (≥ 40 MPH)		Class II (≤ 35 MPH)	
	Lanes	Median	Lanes	Median
Urban	1	Undivided	1	Undivided
	2	Divided	2	Divided
	≥ 3	Divided	≥ 3	Divided
Residential	1	Undivided	1	Undivided
	2	Divided	2	Divided
	≥ 3	Divided	≥ 3	Divided
Rural	1	Undivided	1	Undivided
	2	Divided	2	Divided
	≥ 3	Divided	≥ 3	Divided

A total of 84 TTMS segments from interrupted flow facilities selected from the database provided by FDOT. The preliminary analysis (as displayed in the sample plots shown in Appendix D) showed that it is not was not possible to deduce free flow speeds and capacity from these plots. The plots do not show any distinctions between undersaturated and oversaturated flow conditions. In order to overcome these challenges the research team conducted additional data collection from segments in which traffic and geometric variables could be controlled. The plan was to collect data from different cities but this was not possible due to time and funding constraints. Only 20 sites were selected and all were located in the City of Tallahassee. The selection of the 20 sites was based on various speed limits, facility type, and facility size to ensure diversity in the overall analysis.

6.3 Traffic and Geometric Characteristics of the Selected Sites

Prior to fitting of volume delay functions, it is important to first determine segment free flow speeds and practical capacities of interrupted flow facilities. Collection of data for the estimation of arterial free flow speeds, and capacities is tied to the ability to reasonably collect field traffic and geometric data for use in the estimation free-flow speed and practical capacity. In the analysis of signalized arterials for planning applications, free flow speed and capacity are the input variables influenced by a number of geometric, traffic, and signalization variables. The travel demand modeling does not require geometric variables as inputs into the model but they facilitate the categorization of network links in the model. In FSUTMS, the categorization of links by facility type and area type is based on the aggregation of segments from the same functional class, area type, and similar geometric and traffic characteristics. Table 6.2 displays the characteristics of the 20 segments from speed data was to be collected.

Table 6.2 shows that 1 segment had speed limit of 25, 6 segments had speed limit of 30, 8 segments had speed limit of 35, 1 segments had speed limit of 40, and 4 segments had speed limit of 45. The review of traffic volume supplied by the City of Tallahassee showed that 9 segments had Average Daily Traffic (ADT) between 7, 000 and 23, 000 vehicles per day (vpd) while 11 segments had ADT between 24, 000 and 50, 000 vpd. The length of these segments ranges from 1056 feet up to 5808 feet. The minimum effective green to cycle length ratio (g/C) is 0.35 and the maximum is 0.55.

Table 6.2 Traffic and Geometric Characteristics of Selected Sites

Road Name	Speed Limit (MPH)	g/C ratio	Length (ft)	No. of Through Lanes	ADT[7] (vpd)
N Macomb St	30	0.45	2,112	2	17,903
Lake Bradford	35	0.55	2,640	2	28,690
Thomasville Rd	35	0.50	2,112	2	30,484
Thomasville Rd	35	0.55	2,112	3	30,484
Tennessee St	35	0.45	1,056	2	29,696
Blair Stone Rd	30	0.45	1,584	2	20,715
Blair Stone Rd	35	0.50	3,696	2	23,073
Orange Ave	35	0.35	2,112	2	22,929
Apalachee Pkwy	45	0.40	5,808	2	38,439
Tharpe St	30	0.45	1,584	2	27,626
Tennessee St	30	0.41	2,640	3	39,753
W Pensacola St	40	0.50	2,112	2	30,431
S Adams	45	0.45	2,640	2	21,964
N Monroe St	25	0.40	1,056	2	30,852
S Monroe St	35	0.40	1,584	2	19,890
Paul Russel Rd	30	0.44	1,056	2	7,427
Capital Circle NE	45	0.41	5,808	3	50,000
Capital Circle NE	45	0.41	3,168	3	47,032
Miccosukee Rd	35	0.45	4,752	2	19,494
Miccosukee Rd	30	0.48	2,112	1	8,553

6.4 Determination Free flow Speed

In planning applications and especially in travel demand forecasting, when dealing with large networks most of geometric and traffic factors are grouped facility and area types. This grouping method reduces the number of segments in which data should be collected for model calibration and validation. Estimation of free flow speed on interrupted flow facilities was accomplished using data collected on 20 segments shown in Table 6.2 and from 84 segments from TTMS database. The speed data from the TTMS sites were converted from hourly spot speed into space mean speed using the harmonic mean speed formula shown in Equation 6.1 below

$$Harmonic\ Mean\ Speed = \frac{\sum_{b=1}^{b=15}(Count_{(b)})}{\sum_{b=1}^{b=15}\left(\frac{Count_{(b)}}{Speed_{(b)}}\right)} \quad \text{......} \quad 6.1$$

where b is the speed bin index (1 to 15), $Count_b$ is the number of vehicles in speed bin "*b*", and $Speed_b$ is the mid-point of the speed range in bin "*b*".

[7] The average daily traffic (ADT) information was acquired from the Traffic Engineering Division of the City of Tallahassee.

For the 20 sites from which speed data was manually collected, individual vehicle records were acquired. The collection of 3-day speed data on 20 roadway segments in the City of Tallahassee using pneumatic tube counters provided an opportunity to determine free flow speed in accordance with HCM 2010 procedure as specified in Chapter 30 of the HCM 2010 publication. The HCM 2010 procedure has three main steps:

Step 1. Conduct a spot-speed study at a midsegment location during low volume conditions. Record the speed of 100 or more free-flowing passenger cars. A car is free-flowing when it has a headway of 8 seconds or more to the vehicle ahead and 5 seconds or more to the vehicle behind in the same traffic lane. This is illustrated in Figure 6.2.

Step 2. Compute the average of the spot speeds S_{spot} and their standard deviation σ_{spot}.

Step 3. Compute the segment free-flow speed S_f as a space mean speed using equation

$$S_f = S_{spot} - \frac{{\sigma_{spot}}^2}{S_{spot}} \qquad 6.2$$

where S_f is free-flow speed (mph), S_{spot} is the average spot speed (mph), and σ_{spot} is the standard deviation of spot speeds (mph).

These three steps were applied to the speed data collected by the pneumatic tube counters. A computer program was written to extract vehicles whose leading and following gaps were more than 8 seconds and more than 5 seconds, respectively. The speeds of these vehicles were then summed up and averaged. The final step was the calculation of the space mean speed using Equation 9. It should be noted that only daytime speeds of passenger cars (i.e., two-axle vehicles) were used in the analysis.

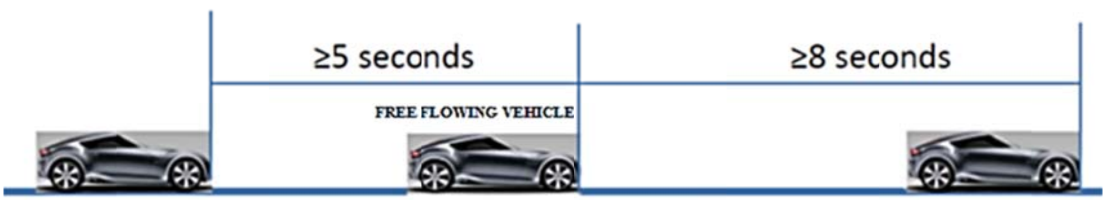


Figure 6.2 Illustration of HCM 2010 Procedure of Determining Free Flow Vehicles

After extracting free-flowing vehicles from the field data, descriptive statistics of the subset of extracted data was produced. The statistics of interest were the 50th percentile speed (i.e., the median speed), the 85th percentile speed, and the 99th percentile speed. In an uninterrupted flow speed analysis, the free-flow speed was determined directly as 85th percentile. Speed analysis in interrupted flow facilities generally requires the inclusion of the effect of signal delay in the calculation of the segment free flow speed. This was accomplished through the use of the NCHRP Report 387 equation shown in Equation 6.3:

$$S_f = \frac{1}{\left[\frac{1}{S_{mb}} + \varphi\left(\frac{D}{3600}\right)\right]} \qquad 6.3$$

where S_f is the free flow speed for an urban interrupted flow facility in miles per hour, S_{mb} is the midblock free flow speed in miles per hour, φ is the signal density given by $\frac{N}{L}$, L is the length of the facility in miles, N is the number of signalized intersections in the analysis segment, D is the average delay per signal in seconds equal to $DF\times0.5\times C\times(1-g/C)^2$, DF is delay adjustment factor equal to $(1-P)/(1-\frac{g}{C})$, P is the proportion of vehicles arriving in green, g is the effective green time in seconds, and C is the cycle length in seconds. Table 6.3 shows the free flow speed for the 20 Tallahassee segments.

Table 6.3 Estimated Free Flow Speed for the 20 Tallahassee Segments

Road Name	FTYPE	ATYPE	Speed Limit (mph)	g/C	Length (ft)	Delay (sec)	S_{mb} (mph)	S_f (mph)
N Macomb St	44	21	30	0.45	2112	19.81	37	25
Lake Bradford	20	21	35	0.55	2640	17.24	43	30
Thomasville Rd	20	31	35	0.50	2112	19.81	42	27
Thomasville Rd	20	31	35	0.55	2112	20.12	40	26
Tennessee St	20	21	35	0.45	1056	18.60	43	20
Blair Stone Rd	20	42	30	0.45	1584	21.18	37	21
Blair Stone Rd	20	31	35	0.50	3696	18.91	44	33
Orange Ave	20	31	35	0.35	2112	18.91	43	27
Apalachee Pkwy	20	21	45	0.40	5808	19.06	51	41
Tharpe St	20	31	30	0.45	1584	18.45	34	22
Tennessee St	20	42	30	0.41	2640	18.60	37	27
W Pensacola St	20	42	40	0.50	2112	20.12	46	28
S Adams	20	42	45	0.45	2640	17.09	52	35
N Monroe St	20	12	25	0.40	1056	19.81	32	17
S Monroe St	20	21	35	0.40	1584	21.02	41	23
Paul Russel Rd	20	42	30	0.44	1056	19.97	36	18
Capital Circle NE	20	42	45	0.41	5808	16.64	52	43
Capital Circle NE	20	42	45	0.41	3168	16.79	52	37
Miccosukee Rd	51	42	35	0.45	4752	17.85	41	33
Miccosukee Rd	30	31	30	0.48	2112	20.87	36	24

6.5 Determination Practical Capacity

According to the 2010 Highway Capacity Manual of analyzing interrupted flow facilities, capacity represents the maximum number of vehicles that can pass a point during a specified time period under prevailing roadway, traffic, and control conditions. One of the most significant variables used in calculating highway capacity on a signalized arterial is the through movement's effective green time to signal cycle length ratio, commonly known as the g/C ratio. The second most significant variable in determining capacity of an arterial corridor is the number of through lanes. The 2010 HCM capacity equation is

$$c = Ns(\frac{g}{c}) \text{ ....} 6.4$$

Where c is the capacity in vehicles per hour, N is the number of through lanes, and s is the adjusted saturation flow rate in vehicles per hour. This equation was used to estimate the capacities for the analysis signalized segments. Appropriate statistical analysis was employed in determining practical capacity from reduced data free of outliers and inconsistent records. Though the data was cleaned to remove outliers in both on the lower and upper ends, the research team decided to calculate the 99th percentile volume and use this value rather than maximum value as an indicator of practical capacity because observed maximum values may not be repeatable as they could have happened by chance or caused by incidents occurring at the time of data collection. The obtained values were compared with the default values from the FDOT 2013 Q/LOS Handbook and the values calculated using Equation 6.3. Table 6.4 shows the estimated practical capacities from the signalized arterial segments.

Table 6.4 Analysis of Practical Capacities

				Observed Volumes (vphph)					**Default Capacities (pcphpl)**	
Road Name	**FTYPE**	**ATYPE**	**Lanes**	**Min.**	**Mean**	**StDev**	**Max.**	**99th %**	**HCM**	**FDOT**
N Macomb St	44	21	2	27	446.41	150.42	616	589	625	720
Lake Bradford	20	21	2	45	341.12	222.62	985	858	800	800
Thomasville Rd	20	31	2	34	376.32	231.74	936	842	825	800
Thomasville Rd	20	31	3	55	296.13	208.66	942	849	825	807
Tennessee St	20	21	2	38	376.23	202.62	966	838	800	800
Blair Stone Rd	20	42	2	55	426.00	245.87	986	860	800	800
Blair Stone Rd	20	31	2	44	452.23	198.46	954	887	800	800
Orange Ave	20	31	2	57	522.04	288.97	937	924	800	800
Apalachee Pkwy	20	21	2	43	374.54	261.16	990	976	800	910
Tharpe St	20	31	2	36	368.74	258.15	892	852	800	800
Tennessee St	20	42	3	26	334.32	271.19	890	844	800	807
W Pensacola St	20	42	2	48	448.43	263.24	984	923	825	910
S Adams	20	42	2	42	446.64	249.37	988	940	825	910
N Monroe St	20	12	2	196	631.83	209.99	922	920	825	800
S Monroe St	20	21	2	62	526.13	254.22	928	910	825	800
Paul Russell Rd	20	42	2	38	376.23	202.62	966	848	800	800
Capital Circle NE	20	42	3	34	456.53	253.34	978	950	825	914
Capital Circle NE	20	42	3	46	476.24	263.11	969	942	825	914
Miccosukee Rd	51	42	2	14	384.74	281.17	880	847	800	800
Miccosukee Rd	30	31	1	16	332.61	242.22	852	806	800	720

The results in Table 6.4 shows that practical capacity estimated using field data is somewhat different from the default values suggested by the 2010 HCM and by the FDOT methodology. The 99th percentile flows and the default capacity values from 2010 HCM and FDOT were tested for differences at significance level of 0.05 using ANOVA test. The results as shown in Figure 6.3 show that there are significant differences (p-value = 0.0033) among the capacity values determined by the three methods. The differences among the values could be attributed by the fact that default values are suggested to encompass a wide area and a more inclusive analysis and therefore they may not be site-specific.

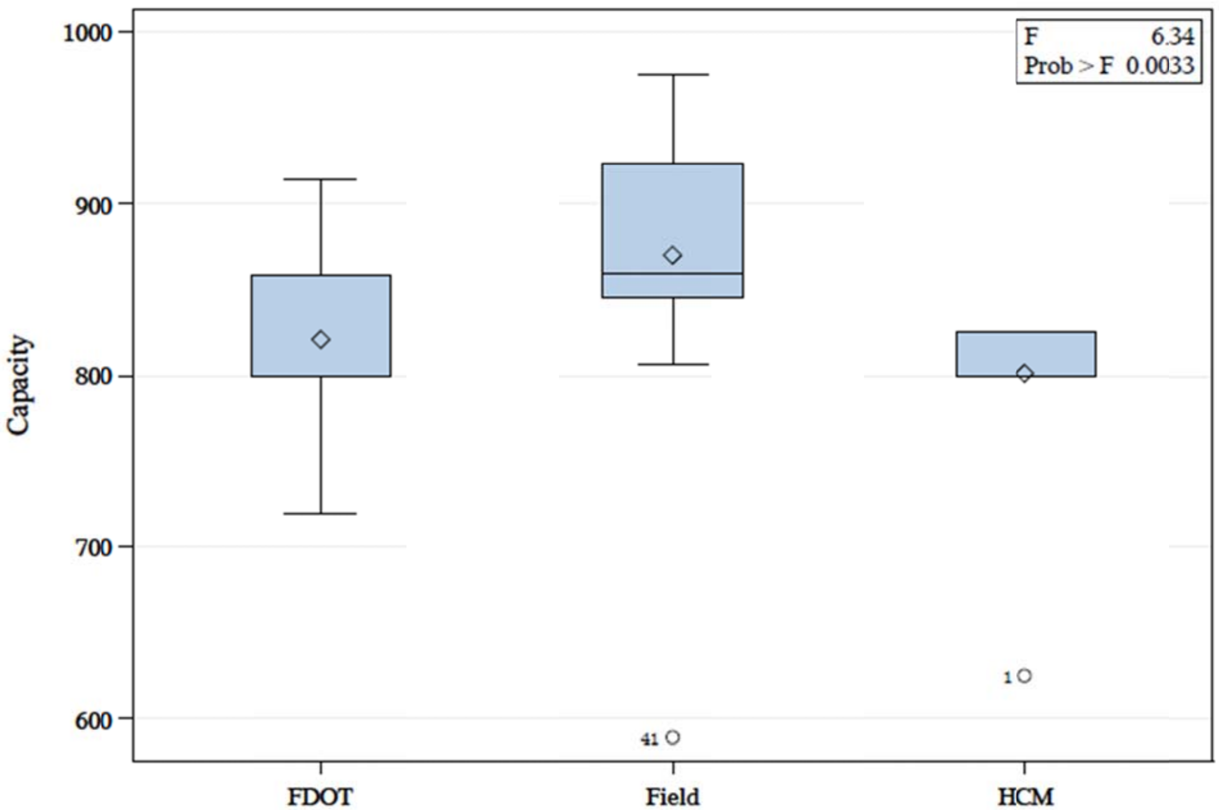


Figure 6.3 Distribution of Capacity by Estimation Method

6.6 Fitting of Volume Delay Functions

The behavior of volume delay functions in interrupted flow facilities is mostly affected by intersections' signal operations along the segment. Signalization factors which have to be considered when deriving VDF parameters are the effective green to cycle length ratio, signal coordination, and signal density. Though these factors are not directly input into the VDFs, the parameters that influence the behavior of the VDFs (free-flow speed and capacity) have to be adjusted to reflect the effects of flow interruptions caused by signals. The fitting process discussed in this chapter is for signalized interrupted flow facilities. The functions fitted in this part are mathematically the same as those discussed in Chapter 5, only their parameters and coefficients differ. Data preparation and initial inputs (free-flow speed and capacity) are more time consuming in interrupted flow facilities than in uninterrupted flow facilities. This is due to complex traffic operations in interrupted facilities compared to uninterrupted ones.

6.6.1 *Generating Simulated Data*

Volume delay functions (VDFs) are generally fitted in speed-flow curves in which flow ranges from undersaturated to oversaturated. Field data collection of speed-flow data in undersaturated conditions is simple while collection of speed-flow data in oversaturated conditions on signalized arterial streets is cumbersome for reasons that were discussed earlier. Both data from the 84 TTMS sites used in the analysis and the 20 Tallahassee sites did not capture oversaturated situations because the vehicle sensors were generally installed in midblock away from intersection areas where oversaturation occurs. Therefore, simulation methodology was chosen to generate data to supplement the midblock data collected by the vehicle sensors.

The field traffic, geometric, and signal timing data was used to simulate link operations at different traffic loading including situations in which volume exceeded capacity. The resulting speeds were recorded in order to produce speed-volume curves covering a wide range of operating conditions. The simulation software used were Synchro 7 and Cube Dynasim 4. However, due to lack of enough calibration data required by Cube Dynasim 4, only Synchro 7 results were used. The outputs from Synchro 7 were iteratively fed into an optimization program written in C++ and MATLAB with the aim of closely matching the field data with the simulated data. The simulated data was trained to match the portion of field data in which flow is in under-saturated regime. Once the process reached convergence criteria (a gap of 10^{-6}) the simulated data was further used to simulate all flow conditions including oversaturation (in which v/c >1.0). The purpose of training was to build the confidence that, if the simulated data could mimic the field in the under saturated conditions, then generation of simulated data in oversaturated flow conditions would be reasonably accurate. Figure 6.4 shows a flow chart of the simulation process.

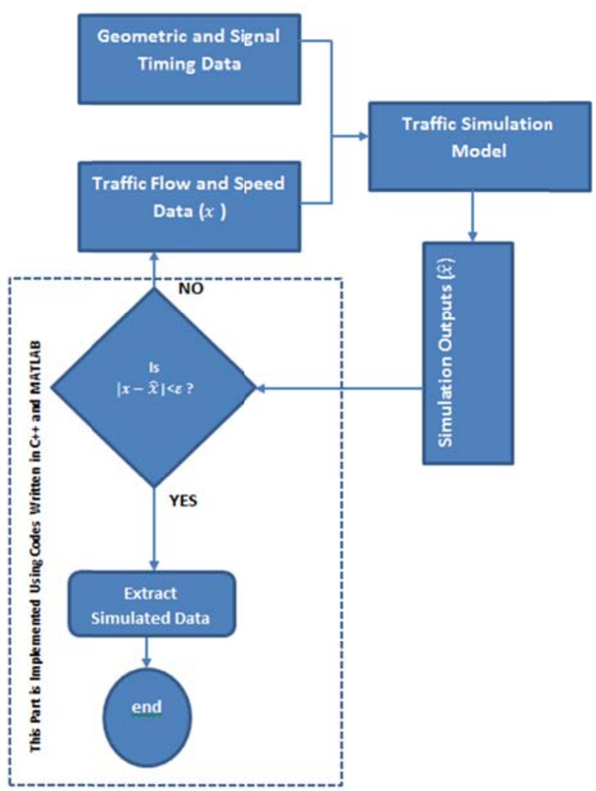


Figure 6.4. Flow Diagram for Segment Simulated Data Generation

6.6.2 Results and Discussion

After generating the data that populated uncongested and congested regimes in signalized arterial segments, the VDF parameters were estimated using the non-linear curve fitting algorithm implemented in MATLAB. This is same algorithm which was used for fitting VDFs in uninterrupted flow facilities as was discussed in Chapter 5. A number of goodness-of-fit measures were used to evaluate the overall performance of fitted functions. Goodness-of-fit measures used were the root-mean-square error (RMSE), the root-mean-square percent error (RMSPE), the mean error (ME), the mean percent error (MPE) statistics and Theil's inequality coefficient, TIC. The parameters of the fitted VDFs were summarized by speed limit and aggregate by area types as shown in Table 6.5.

Table 6.5 Volume Delay Functions and their Estimated Parameters

		30 MPH		**35 MPH**		**≤ 35 MPH**		**40 & 45 MPH**		**AGGREGATED**	
Function	Parameter	Urban	Resid.	Urban	Resid.	Urban	Resid.	Urban	Resid.	Urban	Resid.
Fitted BPR	α	0.220	0.240	0.210	0.240	0.215	0.240	0.250	0.260	0.240	0.260
	β	8.840	7.830	7.530	7.950	8.135	7.895	8.460	8.650	6.500	6.200
Conical	β	18.390	18.390	18.390	15.064	18.390	16.599	18.550	18.700	18.800	18.800
	α	1.029	1.029	1.029	1.036	1.029	1.033	1.028	1.028	1.030	1.030
Modified Davidson	J	0.009	0.009	0.008	0.010	0.008	0.010	0.009	0.009	0.010	0.010
	μ	0.950	0.949	0.940	0.952	0.945	0.951	0.950	0.947	0.950	0.950
Akcelik	J	0.100	0.101	0.110	0.098	0.105	0.099	0.090	0.080	0.100	0.100

As indicated in Table 6.5, the VDFs fitted were BPR function, conical delay function, modified Davidson's function, and Akcelik function. These functions were then plotted against segment simulated data for visual analysis as shown in Figure 6.4 below. This figure shows results of a characteristic urban signalized arterial segment with speed limit of 45. In the analysis of signalized arterials, it was important to categorize facilities in terms of their speed limit for consistency with the arterial classification in the 2013 Quality/Level of Service Handbook (see Table 6.1).

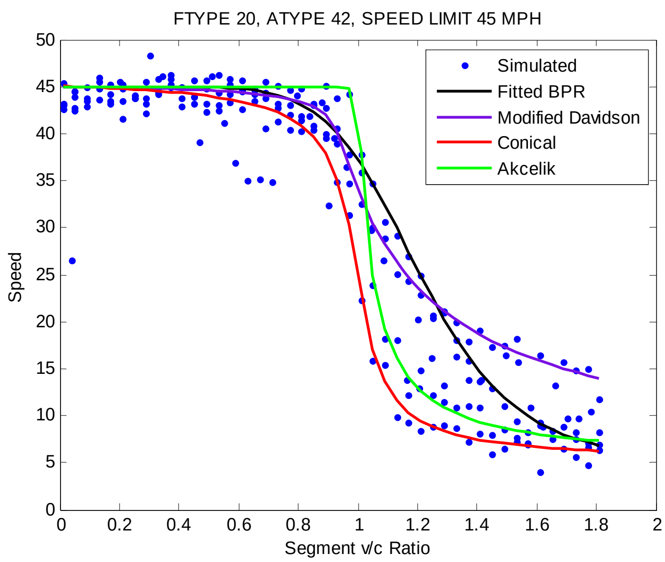


Figure 6.4 Fitted VDFs and simulated data

Visually, in the Figure 6.4 above, the results show that Akcelik function and fitted BPR curve fit the data well, followed by modified Davidson, and lastly conical delay function. However, the same arguments as in section 5.7.4 was used, that these results are not the final judgment to which VDF performs better compared to others. The selection of a VDF for a particular facility type or congestion level should rely on statistical performance measures, sensitivity of link travel speed to change in congestion or demand, and the performance when implemented in the travel demand forecasting model which will be discussed in Chapter 7.

CHAPTER SEVEN

TESTING OF VOLUME DELAY FUNCTIONS IN FSUTMS

7.1 Overview of OAUTS Model

The Orlando Urban Area Transportation Study (OUATS) Model was selected as the travel forecasting model to be used for testing the volume delay functions and their parameters that were developed during the study. The OUATS model is a daily travel forecasting model with a region that covers all or parts of six counties in central Florida, namely: Volusia (west), Lake, Seminole, Orange, Polk (northeast), and Osceola. A map of the OUATS model region 8 is shown in Figure 7.1.

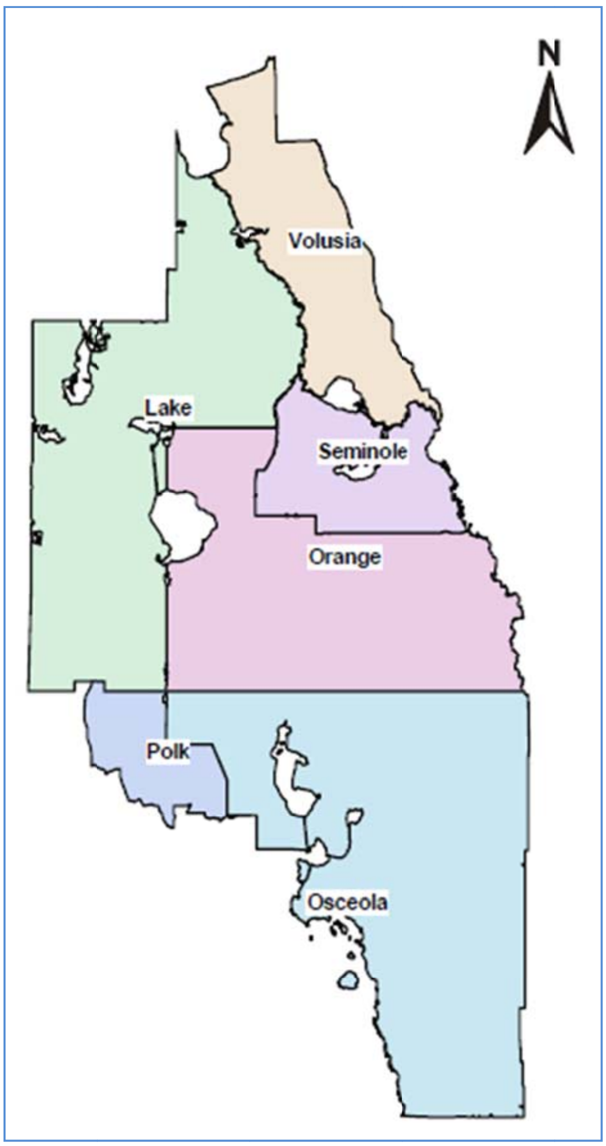


Figure 7.1 OUATS Model Region

The OUATS model is a typical FSUTMS trip-based model and for the purpose of this study a new application group described as the “Speed Study Highway Assignment” application group was added to the model. Groups 2 to 8, shown in Figure 7.2, contain the original components of the OUATS model and the Speed Study Highway Assignment Group (Group 9) contains the sub-groups used for testing the volume delay functions. The sub-groups in the Speed Study Highway Assignment Group are shown in Figure 7.3 and are described in Section 7.3.

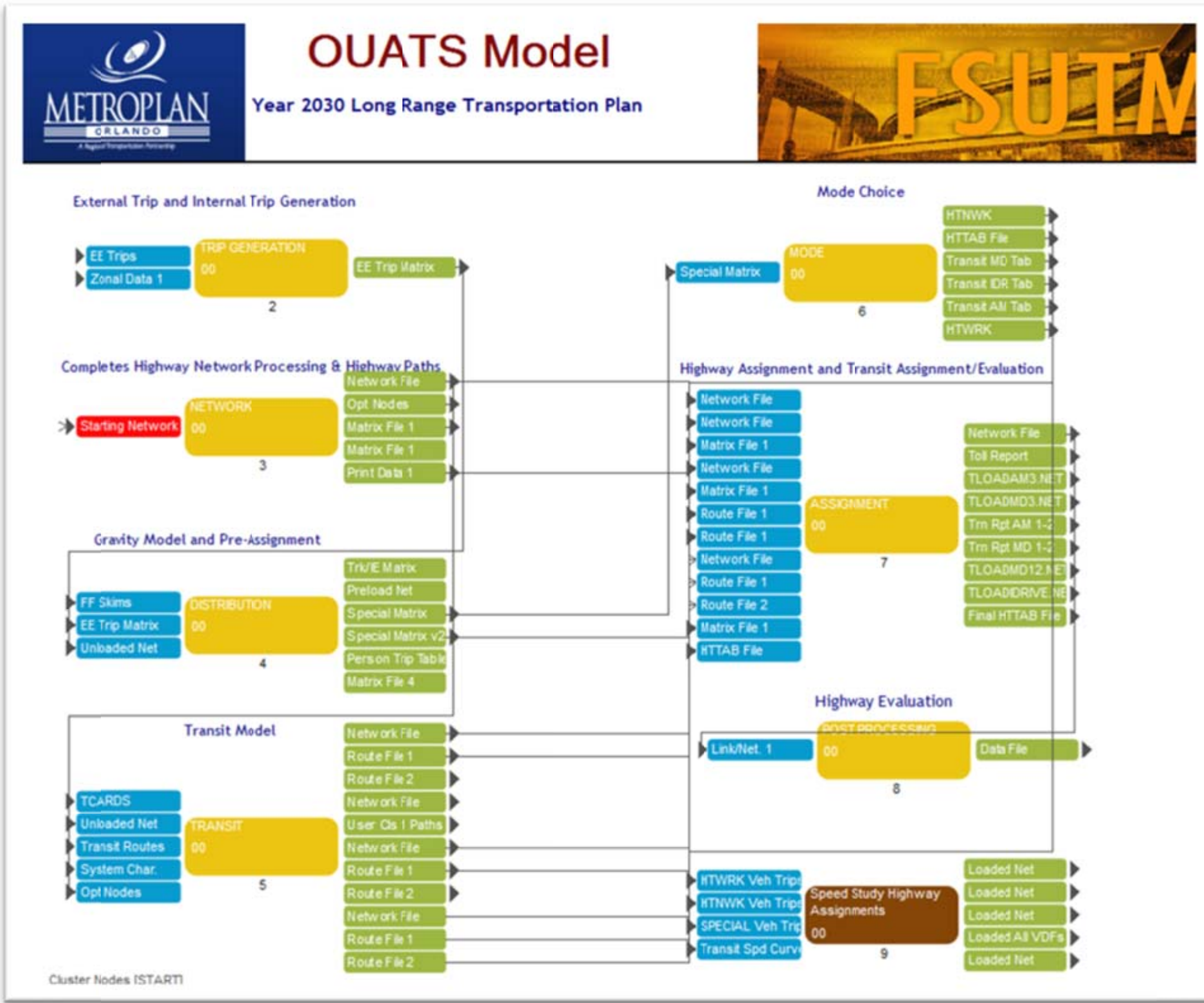


Figure 7.2 Overview of the Test Model

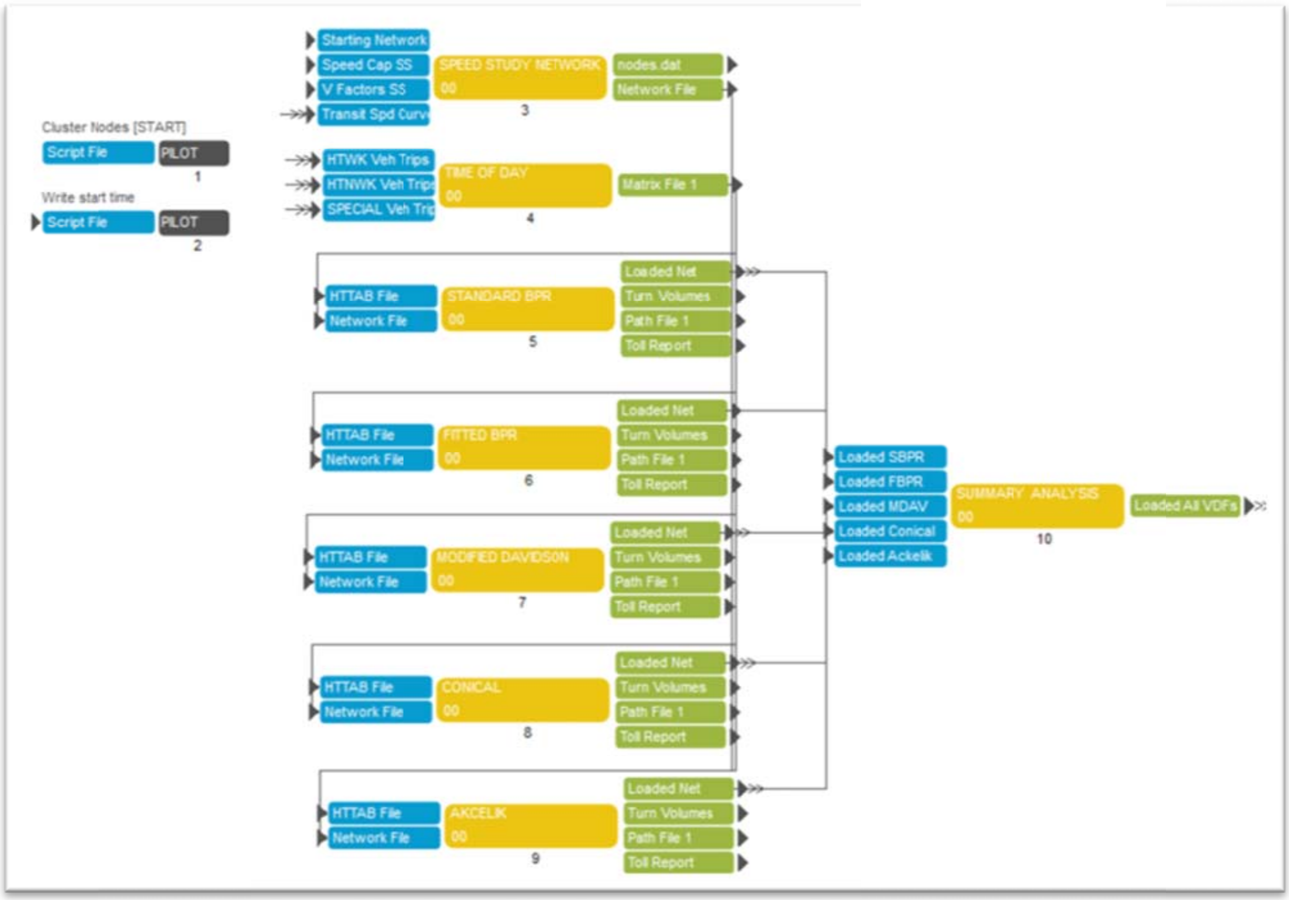


Figure 7.3 Overview of Speed Study Highway Assignment Sub-Group

7.2 Overview of the VDF Testing Methodology

The methodology used to test the volume delay functions consisted of the following steps:

1. Run the OUATS travel forecasting model with Year 2010 input data (socioeconomic data, highway networks, transit networks, etc.) and original model parameters to obtain daily origin-destination trip tables for the model region.
2. Use the OUATS 2010 input highway network to create a set of twenty-four 1-hour input highway networks for each VDF to be tested.
3. Populate each set of input 1-hour highway networks with appropriate free flow speeds, capacities, and parameters for the VDF to be tested.
4. Populate each input 1-hour highway network with the observed average weekday count, average weekday PCE and observed average weekday speeds for the particular 1-hour period.
5. Create twenty-four 1-hour trip tables by applying static time of day factors to the daily trip table obtained from the OUATS model run.
6. Run twenty-four 1-hour highway assignments for each VDF being tested.
7. Compare the output link volumes and congested speeds on TTMS links with the observed average weekday counts and average weekday volumes.

7.3 Modification of the OAUTS Model

The OAUTS model was modified in order to accommodate a number of changes necessary for testing the developed VDFs and their parameters.

7.3.1 The Speed Study Highway Assignments Application Group

A new application group was added to the OUATS model to accommodate the procedures use for implementing the VDF testing methodology. The "Speed Study Highway Assignments" group includes the following sub-groups:

1. *Speed Study Networks Sub-Group*: This sub-group was used to create twenty-four 1-hour input highway networks which were subsequently used as inputs to the highway assignment process for each of the five VDFs tested. Each highway 1-hour networks was populated with the attributes for a particular 1-hour time of day. The attributes include:
   a. Parameters for each VDF being tested.
   b. Revised free flow speed (for all except the Standard BPR)
   c. Revised link capacity (for all networks except the Standard BPR)
   d. Average weekday vehicle count, observed on links with TTMS sites,
   e. Average weekday PCE, computed on links with TTMS sites, and
   f. Average weekday speed, observed on links with TTMS sites.

The average weekday vehicle speeds and counts at TTMS sites incorporated into the travel demand model network were obtained from the analysis data set of TTMS speeds and counts for Tuesdays, Wednesdays and Thursdays that did not have special event traffic, for the period July 1, 2010 to June 30, 2011. A PCE conversion factor of 1.5 was used to convert heavy truck to passenger car equivalents as described earlier.

2. *Time-of-Day Sub-Group*: This sub-group was used to factor the daily 2010 trip table obtained from the OUATS model into twenty-four 1-hour trip tables using static time-of-day (TOD) factors. The time of day factors were obtained from previous Florida DOT sponsored research done by Cambridge Systematics (2010) which developed time of day factors for Florida regions[8].

3. *The VDF Sub-Groups*: Each of the VDF subgroups was used to run twenty-four 1-hour highway assignments using the appropriate VDF parameters. The CUBE script for each highway assignment was revised to include the formulation for the particular VDF being tested. The following volume delay functions were tested:
   a. Standard BPR VDF
   b. Fitted BPR VDF
   c. Modified Davidson VDF
   d. Conical VDF
   e. Akcelik VDF

[8] Cambridge Systematics, Inc (2010). *Incorporating Time of Day into FSUTMS, Phase 1: Factoring and Procedure Development, Draft Final Report (Prepared for FDOT)*. Tallahassee, FL: Author

The Standard BPR Sub-Group ran the same highway assignment as the original OUATS model, the only difference being the hourly input networks and trip tables. All other VDFs used parameters that were estimated during the study.

4. *Summary Analysis*: This sub-group was used to process the model run statistics.

7.3.2 Updated Facility Type Classification for Arterials

The Florida Department of Transportation (FDOT) recently changed the classifications of arterials to a two class system based on the speed limit of the facility[9]. Consequently, the VFD parameters estimated for signalized arterials were classified into the two speed limit based classes (≤ 35 MPH and ≥ 40 MPH). It was therefore necessary to update the input highway networks with the new arterial classifications. The speed limit for signalized arterial links were obtained from FDOT's maximum speed for state roads, and where otherwise necessary, the speed limit for the link was derived from Speed Category data in the NAVTEQ Florida street database. The new arterial classifications were only used when testing the new VDFs, the original facility type classification was used when testing the Standard BPR from the original OUATS model.

The NAVTEQ Speed category classifies the speed trend of a road based on posted or legal speed and also considers other factors such as physical restrictions or access characteristics. The Speed Category value can therefore differ from the legal speed limit[10]. Table 7.1 shows the equivalency table that was developed and used to convert the NAVTEQ Speed Category to an arterial speed limit based classification.

Table 7.1 NAVTEQ Speed Category Conversion

NAVTEQ Speed Category	NAVTEQ Speed Trend	Speed Limit based Classification
1	> 80 mph	≥ 40 mph
2	65 - 80 mph	≥ 40 mph
3	55 - 64 mph	≥ 40 mph
4	41 - 54 mph	≥ 40 mph
5	31 - 40 mph	≤ 35 mph
6	21 - 30 mph	≤ 35 mph
7	6 - 20 mph	≤ 35 mph
8	< 6 mph	≤ 35 mph

7.3.3 The Speed Study Highway Assignments Application Group

The variable factors (VFACTORS) file contains parameters used by the OUATS model. These include the parameters for the volume delay function used in the model. For the purpose of this study the VFACTORS file was updated to include the parameters for all the VDF forms that were tested. Originally, the OUATS travel model considered only the facility type when selecting the VDF parameters to be used on a particular highway network link. However, during

[9] Florida Department of Transportation. *2013 Quality/Level of Service Handbook*. Tallahassee, FL: Author

[10] NAVTEQ. NAVTEQ Street Data Reference Manual v3. Chicago, IL. 2008

this study VDF parameters were estimated for facilities stratified by area type and facility type. The VFACTORS file was therefore updated accordingly for all the VDF tests (except for the Standard BPR) and both the facility type and area type were considered when selecting the VDF parameters for each highway network link. The estimated VDF parameters that were tested are shown in Table 7.2 and discussed in Section 5 and Section 6 of this report.

The speed and capacity (SPDCAP) file in the OAUTS model contains the free flow speed and capacity for facilities stratified by facility type and area type. Where applicable, the SPDCAP file was updated to include the free flow speeds and capacities developed during the study. The updated free flow speeds and capacities are discussed in Section 5 and Section 6 of this report. The estimated free flow speeds used in the VDF tests are shown in Table 7.2

Table 7.2 Estimated VDF Parameters and Free Flow Speeds

Facility Type	Area Type	Free Flow Speed	**Fitted BPR**		**Conical**		**Modified Davidson**		**Akcelik**
		(MPH)	α	β	β	α	J	μ	J
Freeway	Urban	68	0.263	6.869	18.390	1.029	0.009	0.950	0.100
	Residential	69	0.286	5.091	18.390	1.029	0.009	0.949	0.101
	Rural	71	0.150	5.610	15.064	1.036	0.010	0.951	0.099
Toll Road	Urban	70	0.162	6.340	18.390	1.029	0.008	0.940	0.110
	Residential	70	0.250	7.900	15.064	1.036	0.010	0.952	0.098
	Rural	74	0.320	6.710	15.064	1.036	0.010	0.940	0.097
HOV/HOT	Residential	72	0.320	8.400	18.550	1.028	0.009	0.950	0.090
	Urban	71	0.330	8.600	18.700	1.028	0.009	0.947	0.080
Divided Arterial - Signalized, ≤ 35 MPH	Residential	30	0.215	8.135	1.029	18.390	0.008	0.945	0.105
	Urban	29	0.240	7.895	1.033	16.599	0.010	0.951	0.099
Divided Arterial - Signalized, ≥ 40 MPH	Residential	39	0.250	8.460	1.028	18.550	0.009	0.950	0.090
	Urban	37	0.260	8.650	1.028	18.700	0.009	0.947	0.080
Undivided Arterial - Signalized, ≤ 35 MPH	Residential	29	0.215	8.135	1.029	18.390	0.008	0.945	0.105
	Urban	27	0.240	7.895	1.033	16.599	0.010	0.951	0.099
Undivided Arterial - Signalized, ≥ 40 MPH	Residential	37	0.250	8.460	1.028	18.550	0.009	0.950	0.090
	Urban	35	0.260	8.650	1.028	18.700	0.009	0.947	0.080

7.4 TTMS in the OAUTS Model Region

Twenty-two TTMS sites are located within the OUATS model region. Fifteen of the twenty-two sites monitor vehicle count, vehicle speed and vehicle classification (see Table 7.3). During the VDF testing process the traffic volume and speeds estimated by the OUATS model, when the new VDFs were applied, were compared to the observed vehicle counts and speeds at eight of the TTMS sites. Figure 7.4 shows the location of the TTMS and the associated facility type and area type in the OUATS model.

Table 7.3 TTMS Locations in the OUATS Model Region

COSITE	ROADWAY	DESCRIPTION	LOCATION	FACILITY TYPE	AREA TYPE
770343	77160000	SR-400/I-4 1.6 MI E OF SR-434 SEMINOLE CO	5.135	Freeway	Residential
799906	79110000	ON I-4 169 FT E OF ENTERPRISE RD OVERPASS VOLUSIA CO	4.668	Freeway	Rural
750204	75002000	SR-528/BEELINE EXPWY 2.26 MI W OF SR-15 ORANGE CO	10.71	Toll	Residential
750336	75002000	SR-528 0.7 MI W OF SR-520 ORANGE CO	29.64	Toll	Rural
970428	11470000	SR-91 M/L 765 FT S OF CR-561	7.683	Toll	Rural
970429	92471000	SR-91 M/L 163 FT S OF NEPTUNE RD UNDERPASS/CR-525	33.46	Toll	Rural
770102	77010000	ON US-17 AND 92 1.6 MI S OF SR-46 SEMINOLE CO	9.991	Arterial - Divided (Signalized)	Urban
770197	77120000	SR-434 1.6 MI E OF I-4 SEMINOLE CO	6.626	Arterial - Divided (Signalized)	Residential
160310	16180000	SR-25/US-27 280 FT S OF S HOLLY HILL TANK RD POLK CO	20.526	Arterial - Divided (Unsignalized) *	Rural
110177	11010000	SR-500/US-441 0.3 MI E OF CR-44 LAKE CO	8.58	Arterial - Divided (Unsignalized) *	Urban
750104	75060000	SR-50 0.19 MI W. OF SR-520 NEAR BITHLO ORANGE CO	19.42	Arterial - Divided (Unsignalized) *	Rural
770299	77040000	SR-46 0.4 MI W OF ST. JOHNS RIVER BRG SEMINOLE CO	5.291	Arterial - Divided (Unsiginalized) *	Rural
920065	92030000	US-192 2 MI W OF SR-15 HOLOPAW OSCEOLA CO	22.153	Arterial - Divided (Unsiginalized) *	Rural
110262	11100000	SR-19 1.2 MI N OF CR-42E 68 FT N OF PALM ST LAKE CO	6.32	Arterial - Undivided (Unsignalized) *	Rural
110246	11110000	SR-44 720 FT EAST OF CR-44 LAKE CO	2.125	Collector *	Rural

Note: * implies – facility type not included in VDF testing.

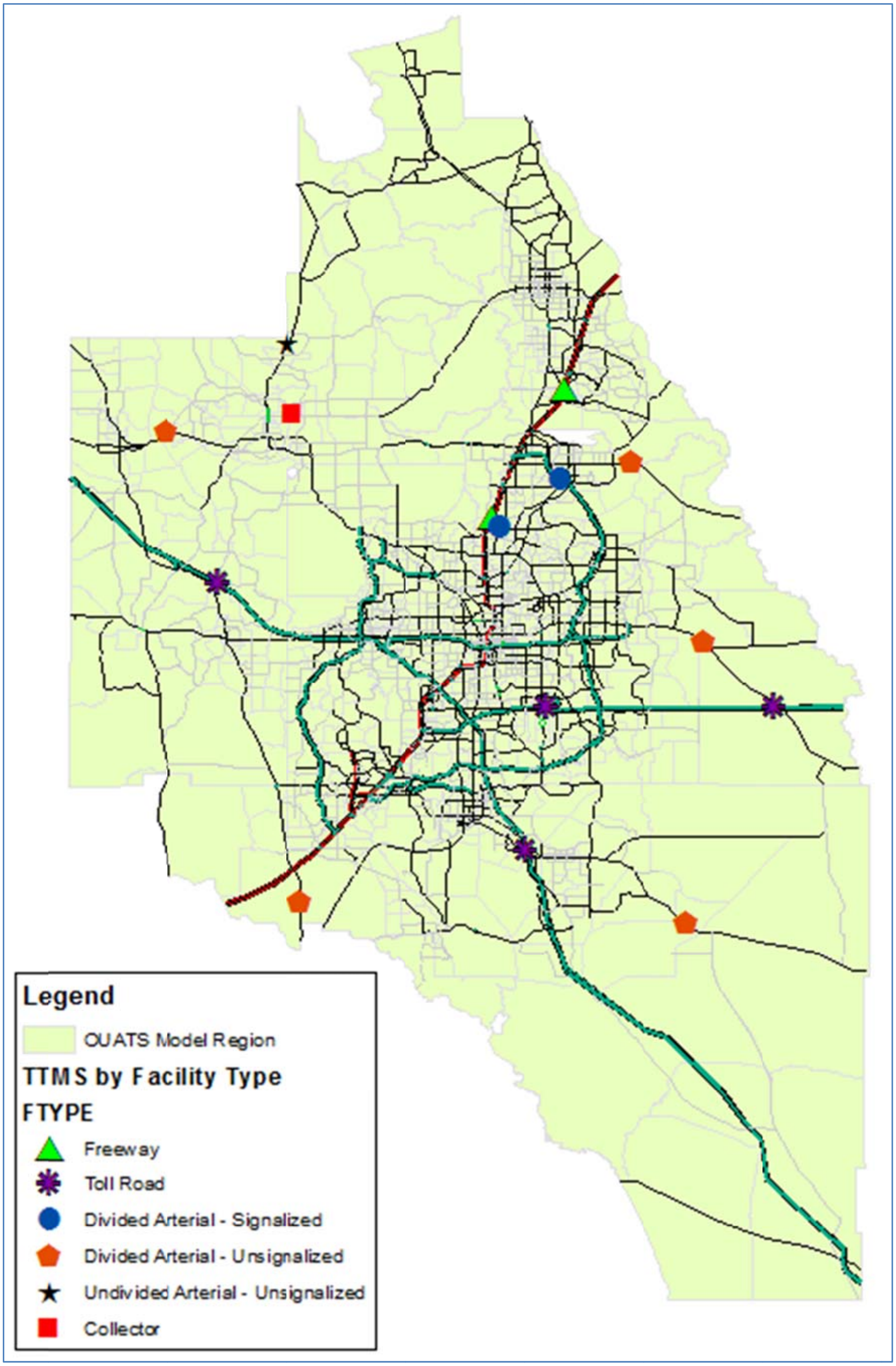


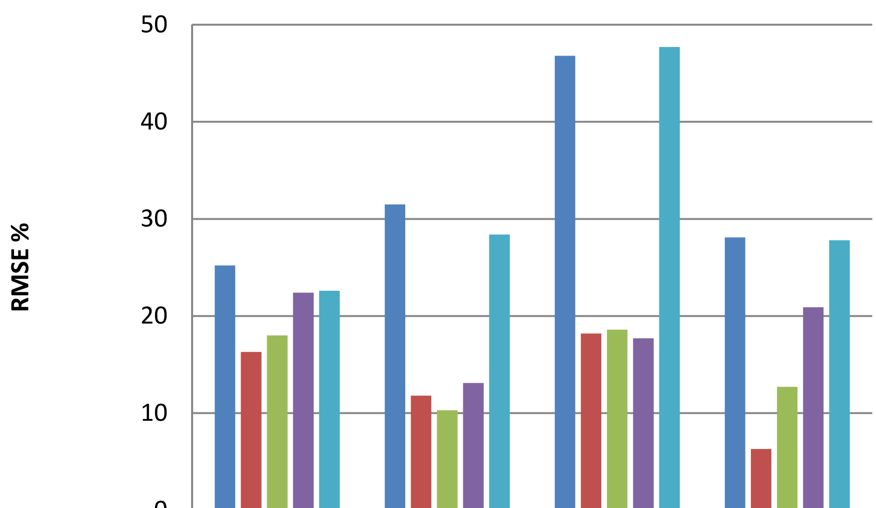
Figure 7.4 Location of TTMS in OUATS Model Region

7.5 Results of VDF Parameter Testing

This section discussed the results of the VDF parameter testing for the peak period hours of 7:00-8:00 AM, 8:00-9:00 AM, 5:00-6:00 PM and 6:00-7:00 PM. To ensure a tight convergence of the highway assignment procedure, a Gap closure criteria of $1x10^{-6}$ was used during the model runs. The results are presented for each group of facility types for which VDF parameters were developed and tested. Additional RMSE percent charts and tables can also be found at Appendix E.

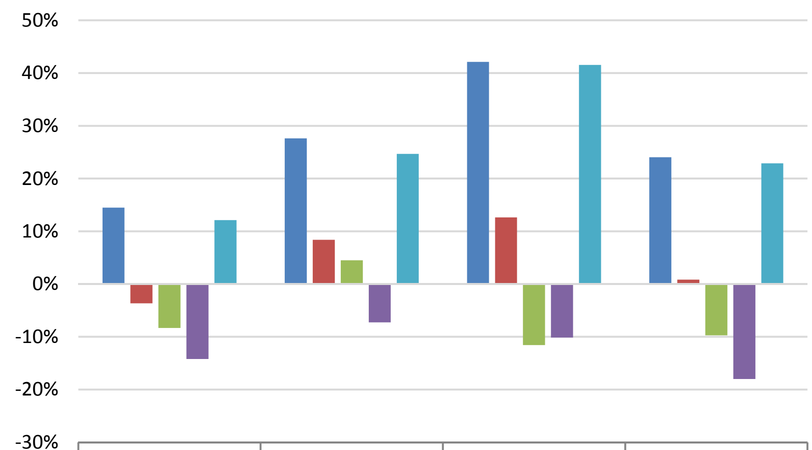
7.5.1 Uninterrupted Flow Facilities

There are only two TTMS sites (both located on the I-4 Interstate) that monitor vehicle counts, vehicle speeds and vehicle classification on uninterrupted facilities in the OUATS model region. This limited the available data points for comparing the model estimates to observed data to four, i.e., two TTMS sites by two directions. The results of the VDF test show that volume of traffic estimated on uninterrupted facilities during the morning and evening peak periods are generally a relatively close match to the TTMS average weekday vehicle/lane counts for the corresponding hours for at least three of the five VDFs tested. The fitted BPR and Modified Davidson VDFs estimated traffic volumes with RMSE of 18.6 percent or less for each of the four peak hours in the AM and PM peak periods (Figure 7.5).



	7-8 AM	8-9 AM	4-5 PM	5-6 PM
SBPR	25.2	31.5	46.8	28.1
FBPR	16.3	11.8	18.2	6.3
MDAVIDSON	18.0	10.3	18.6	12.7
CONICAL	22.4	13.1	17.7	20.9
AKCELIK	22.6	28.4	47.7	27.8

Figure 7.5 RMSE % for Freeway Estimated Volume vs. Counts



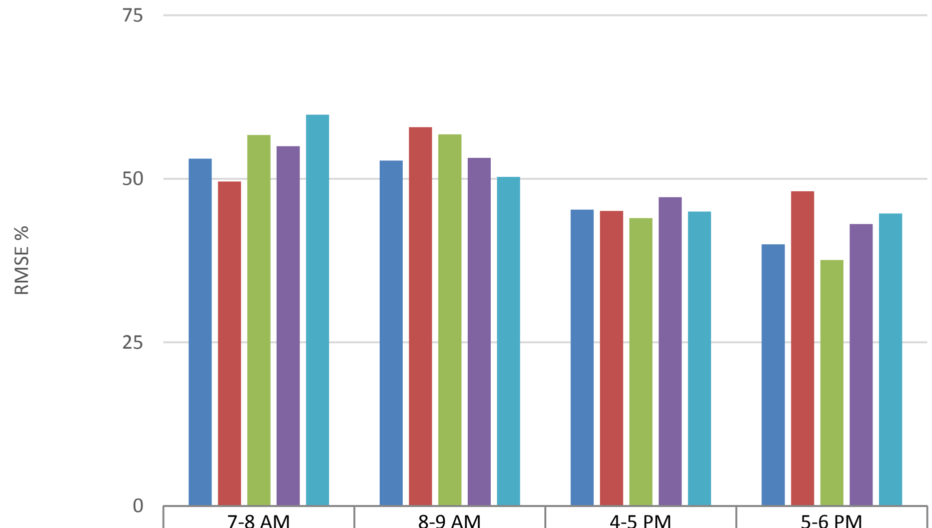
	7-8 AM	8-9 AM	4-5 PM	5-6 PM
SBPR	0.14	0.28	0.42	0.24
FBPR	-0.04	0.08	0.13	0.01
MDAVIDSON	-0.08	0.05	-0.12	-0.10
CONICAL	-0.14	-0.07	-0.10	-0.18
AKCELIK	0.12	0.25	0.42	0.23

Figure 7.6 Freeway Volume to Count Percent Difference

The Modified Davidson and Conical VDFs generally underestimated the traffic volume for uninterrupted facilities in the tests by 18 percent or less. Overall the Fitted BPR parameters performed the best while the standard BPR volume delay function (from the original OUATS model) and the Akcelik VDF parameters produces the worse estimates with relatively high overestimation of the traffic volume (see Figure 7.6).

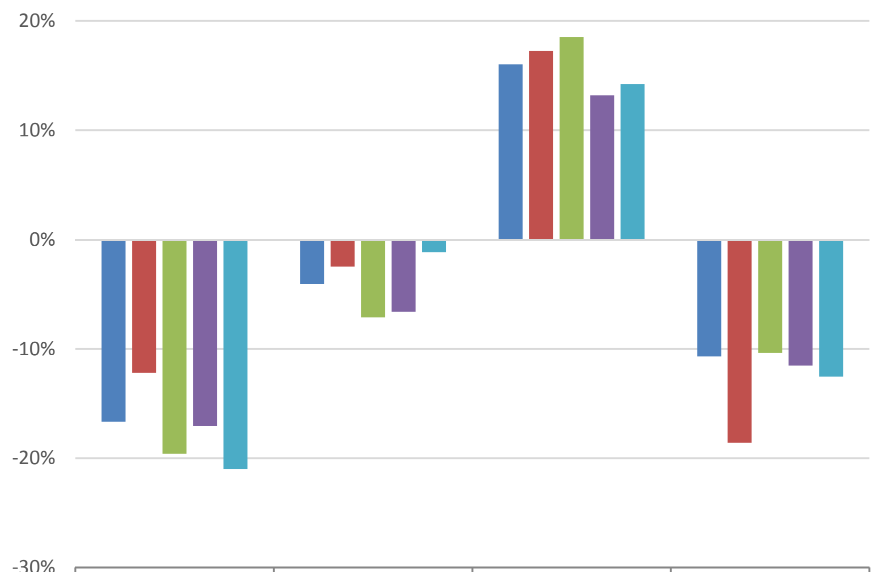
7.5.2 Toll Facilities

Four TTMS that monitor vehicle counts, vehicle speeds and vehicle classification on toll facilities in the OUATS model region and these provide eight data points for comparison of estimated traffic volumes and speeds from the model to observed vehicle counts and speeds. The performances of the VDFs when applied to toll facilities in the OUATS model were very similar with no single VDF performing significantly better than any other VDF. RMSE for estimated volume compared to average weekday counts were relatively high, 50 percent to 60 percent RMSE for the 7:00-8:00 AM and 8:00-9:00 AM hours for each VDF that was tested (see Figure 7.7). Performance was somewhat better for the 4:00-5:00 PM and 5:00-6:00 PM hours where the estimated volume compared to average weekday counts showed 38 percent to 48 percent RMSE). Generally, the VDFs underestimated traffic volumes during the 7:00-8:00 AM, 8:00-9:00 AM and 5:00-6:00 PM hours as shown in Figure 7.8. The RMSE for estimated speed compared to observe speed (shown at Appendix E) was also consistent across all the VDF with values mainly in the range of 14 percent to 15 percent RMSE.



	7-8 AM	8-9 AM	4-5 PM	5-6 PM
SBPR	53.1	52.8	45.3	40.0
FBPR	49.6	57.9	45.1	48.1
MDAVIDSON	56.7	56.8	44.0	37.6
CONICAL	55.0	53.2	47.2	43.1
AKCELIK	59.8	50.3	45.0	44.7

Figure 7.7 RMSE Percent for Toll Road Estimated Volume vs. Counts

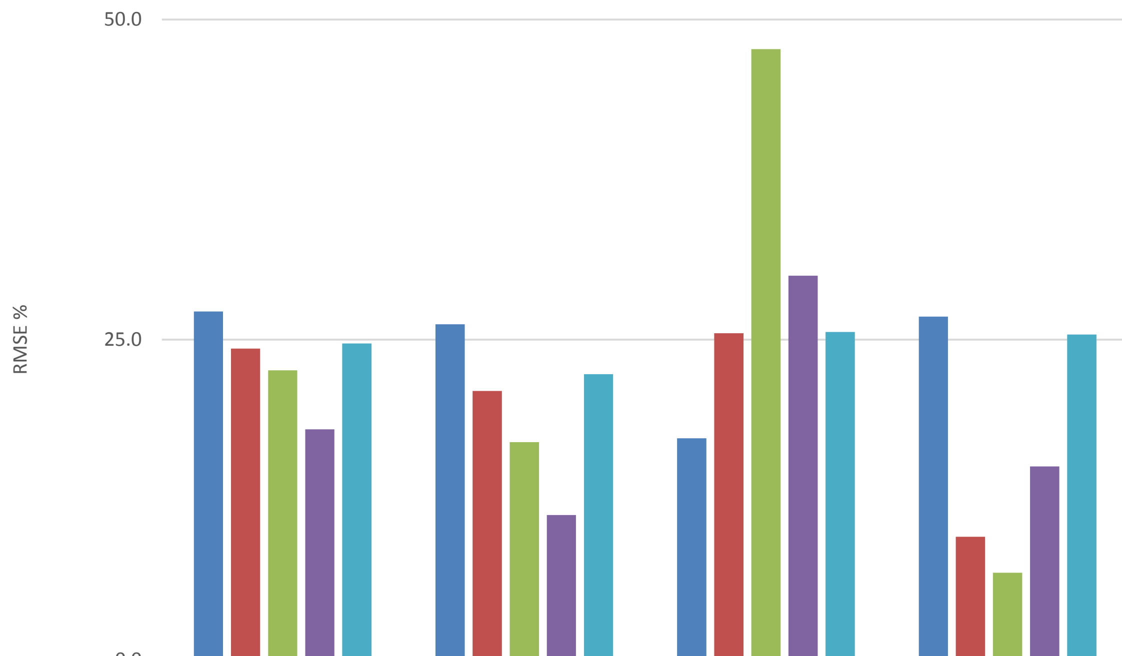


	7-8 AM	8-9 AM	4-5 PM	5-6 PM
SBPR	-0.17	-0.04	0.16	-0.11
FBPR	-0.12	-0.02	0.17	-0.19
MDAVIDSON	-0.20	-0.07	0.19	-0.10
CONICAL	-0.17	-0.07	0.13	-0.12
AKCELIK	-0.21	-0.01	0.14	-0.13

Figure 7.8 Toll Road Volume to Count Percent Difference

7.5.3 Divided Arterials -- Signalized

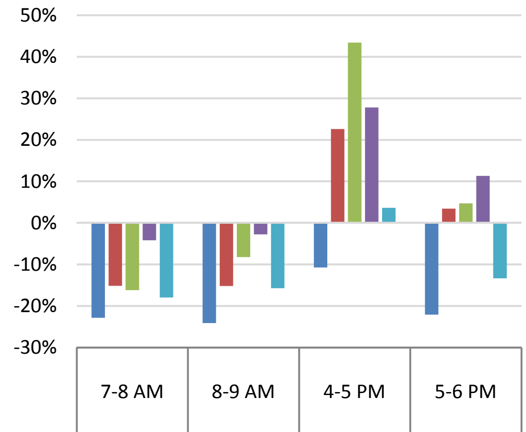
Only two TTMS that monitor vehicle counts, speeds and vehicle classifications are located on signalized divided arterials in the OUATS model region. When the VDFs were applied to signalized divided arterials in the OUATS model, the estimated traffic volumes were less than the observed average weekday vehicle counts at the TTMS. It should be noted that while the VDF parameters for uninterrupted and toll facilities were estimated from TTMS data, the parameters for signalized divided arterials were estimated from field data collected in the Tallahassee area. Comparison of the estimated volumes to observed vehicle counts showed that the VDFs generally underestimated the traffic volumes during three of the four hours in the peak periods (7:00-8:00 AM, 8:00-9:00 AM and 5:00-6:00 PM) as shown in Figure 7.9. The Standard BPR generally performed worse that the other VDFs and the Conical VDF performed slightly better than the other VDFs for the four peak period hours with smaller levels of traffic volume underestimation and relatively low RMSE for three of the four peak period hours (see Figure 7.10).



	7-8 AM	8-9 AM	4-5 PM	5-6 PM
SBPR	27.2	26.2	17.3	26.8
FBPR	24.3	21.0	25.5	9.6
MDAVIDSON	22.6	17.0	47.7	6.8
CONICAL	18.0	11.3	30.0	15.1
AKCELIK	24.7	22.3	25.6	25.4

SBPR FBPR MDAVIDSON CONICAL AKCELIK

Figure 7.9 RMSE Percent for Signalized Divided Arterials



	7-8 AM	8-9 AM	4-5 PM	5-6 PM
SBPR	-0.23	-0.24	-0.11	-0.22
FBPR	-0.15	-0.15	0.23	0.03
MDAVIDSON	-0.16	-0.08	0.43	0.05
CONICAL	-0.04	-0.03	0.28	0.11
AKCELIK	-0.18	-0.16	0.04	-0.13

Figure 7.10 Signalized Divided Arterial Volume to Count Percent Difference

7.5.4 Overall Indicators

Generally, the number of data points (TTMS sites with observed vehicle counts and speeds) available in the model region for comparing the traffic volumes estimated by the VDFs is limited for some facility types and non-existent for others (e.g., HOV, ramps, etc.). However, the results of the tests do suggest that that some VDF parameters perform better than others on particular facility types. Table 7.4 shows that the Fitted BPR and Modified Davidson VDFs generally performed better than the other VDFs during these test and the performance of the Conical VDF did improve when testing signalized arterial, a facility type for which it may be better suited.

Table 7.4 RMSE % Values for Estimated Volumes vs. Observed Counts (4-5 PM)

Volume per Lane	Standard BPR	Modified BPR	Modified Davidson	Conical	Akcelik	# of TTMS (Directional)
1 - 200						
200 - 400	64.0%	65.8%	70.1%	66.8%	59.2%	4
400 - 600	17.3%	25.5%	47.7%	30.0%	25.6%	4
600 - 800	39.1%	36.0%	25.8%	33.2%	38.5%	3
800 - 1000	52.0%	31.0%	0.5%	10.7%	44.9%	1
1000 - 1200	25.4%	25.7%	25.4%	34.3%	31.5%	1
1200 - 1400	16.6%	0.7%	24.4%	18.4%	12.8%	1
1400 - 1600	69.6%	28.0%	5.0%	0.8%	72.6%	1
1600 - 1800						
1800 - 2000	33.2%	0.7%	21.9%	22.6%	35.6%	1
2000 - 2200						
2200 - 2400						
2400 - 10000						
1 - 10000	50.3%	29.9%	32.6%	30.9%	51.3	16

7.6 Characteristic Response of VDFs to Change in Congestion Levels

In this study, the VDFs' coefficients and their input parameters (capacities and free-flow speeds) were calibrated using field data and simulation techniques. Subsequently, the VDFs were tested in the travel demand forecasting model. The VDFs showed different performances in curve fitting and in the travel demand forecasting model. The performance results were consistent in both curve fitting and VDF testing processes. In these processes, it was observed that the modified or fitted BPR aligned well with the field data, followed by modified Davidson, Conical delay function, and lastly the Akcelik function.

Considering different network characteristics and travel behavior from different metropolitan planning regions, the results from the curve fitting and VDF testing procedures should not be the final in judging which volume delay function performs better compared to others. In a congested network, a VDF will perform differently given different facility types. For this reason, the selection of VDF for a particular facility type or area type needs sturdy knowledge of transportation network behavior under different congestion levels and different traffic controls. It is obvious that the effect of change in congestion, near or at capacity, will have different impact on travel speed for a freeway link compared to a signalized arterial link.

Speed tends to deteriorate faster in shorter links (urban signalized arterials) than in longer links (uninterrupted flow facilities such as freeways and expressways) when demand is close to capacity. Therefore, the selection of a VDF for a particular facility type should not only rely on statistical performance measures such as root mean square error (RMSE) or coefficient of determination (R^2) but also to account for sensitivity of link travel speed to change in congestion or demand, which is measured as a slope of a VDF at a given congestion level which, in this case, is represented by v/c ratio.

Figure 7.11 shows the behavior of each fitted VDF as congestion or demand level, x, changes. When demand is lower than capacity (up to $x \approx 0.7$), the slopes of the VDFs remain fairly unchanged ($\frac{du}{dx} \approx 0$), meaning that the users in the link are free-flowing. The slopes become steeper when demand approaches capacity. Conical, Akcelik and modified Davidson reach their steepest slopes at capacity (x =1.0) different from fitted BPR which reaches its steepest slope at a demand 20% higher than capacity (x =1.2). At higher demands ($x \gg 1$) when the link is already congested the slopes change from steep to gentle ($\frac{\mathrm{du}}{\mathrm{dx}} \approx 0$ at higher values of x). This means that the speed becomes less sensitive to increasing demand when the link already oversaturated. With respect to network performance, the results in Figure 7.11 can be interpreted as follows: A link is robust to change in demand if either the demand or travel speed is low – that is, changes in demand have lesser effect to travel speed if there are a few travelers in the link (free-flow condition), or if the link is already highly congested, and therefore the speed will not deteriorate much further.

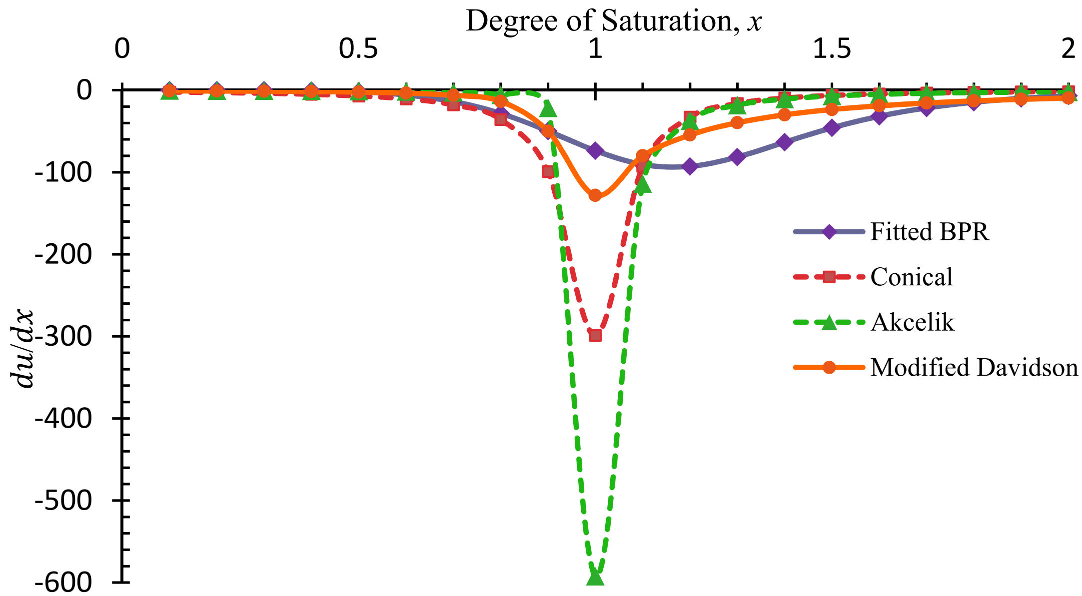


Figure 7.11 Sensitivity of Link Travel Speed to Change in Congestion Level

Different sensitivity characteristics shown by the VDFs in Figure 7.11 indicate the suitability of a function for use in a particular facility type. The stability manifested by the fitted BPR, is consistent with the behavior of long stretches of uninterrupted flow facilities, such as basic freeway segments with low ramp density. The Modified Davidson, matches facilities with medium access density such as freeways and expressways in fringe or outlying business district (OBD), segments of toll roads with medium spaced toll plazas, or multilane highways with highly isolated signals. Speed modeling in freeways with high ramp density, toll roads with closely spaced toll booths or plazas, or multilane highways with medium spaced signals, can be achieved by using Conical delay function. The behavior depicted by Akcelik function, is a characteristic of urban streets with closely spaced signalized intersections.

CHAPTER EIGHT

CONCLUSIONS AND RECOMMENDATIONS

8.1. Conclusions

Traffic assignment is an important step in the Florida statewide four-step travel forecasting model, commonly known as the Florida Standard Urban Transportation Modeling Structure (FSUTMS), maintained by the Systems Planning Office of the Florida Department of Transportation. The sequential four steps are trip generation, trip distribution, mode choice, and traffic route assignment. Traffic is assigned to a particular highway link based on predicted travel time which is in itself heavily influenced by demand volume in the link – the higher the demand, the lower is the travel speed and the longer is the travel time along the link. Volume-delay functions are used by transportation modelers to relate the decay in travel time (hence increased delay) to the operating volumes on the roadway. Appropriate volume delay functions used in travel forecasting are arrived at by first determining the best functional form; fitting the VDF on speed-volume plots produced from field data collected from roadways in different area types, facility types, and facility sizes; and then calibrating and validating the VDFs to produce appropriate VDF parameters for use in travel forecasting.

Field data was collected from roadways from different area/facility type combinations. The area types contained in the FSUTMS look-up tables were collapsed into three categories – that is, rural, urban, and residential. Data collection and analysis were further subdivided into uninterrupted flow and interrupted flow facilities due to the fact that speed-volume relationship are not the same on these facilities. Data from 76 permanent count stations were acquired for a period beginning July 1, 2010 to June 30, 2011. In addition, data acquired from the permanent count stations were supplemented with active field data collection from 20 sites in the City of Tallahassee for use in the analysis of interrupted flow facilities, mainly signalized arterial corridors. Following screening and validation of the collected field data, analysis was carried out with the following aims:

(a) estimating free flow speeds
(b) estimating practical capacities,
(c) plotting scatter plots of speed vs. flow,
(d) fitting volume delay functions to the speed-flow scatter plots,
(e) developing optimum VDFs parameters, and
(f) testing the VDF and their optimum parameters in the FSUTMS.

8.1.1 Free Flow Speeds

Free flow speeds, the major input of volume delay functions, were analyzed using one-year data from 76 permanent count sites installed on uninterrupted flow facilities. The analysis involved determining which vehicles were free flowing and what percentile speed was appropriate in the estimation of free flow speed. Traffic was considered to be free-flowing if the density corresponding to low volume (≤ 200 passenger cars per hour per lane) was ≤ 5 passenger cars per mile per lane. The hours in the one-year data meeting these conditions were ranked in ascending order and the 50th, 85th, and 99th percentile speeds were calculated as possible estimates of practical free flow speed. For non-toll and toll uninterrupted flow facilities, the analysis showed that free flow speed was influenced by area type and speed limit consistent with results commonly found in published literature. Because of lack of permanent count stations on some segments with certain speed limits, a speed prediction model was developed to predict free flow speed in those segments. The results of the speed prediction were in line with free flow speeds found in literature suggesting that the developed speed regression model is appropriate for use in determining free flow speeds for travel forecasting purposes.

Speed modeling was also conducted for interrupted flow facilities generally characterized by signalized intersections. Determination of free flow speeds in these types of facilities requires the inclusion of the effect of signal delay. The segment average free flow speed is calculated by adjusting the mid-block free flow speed to incorporate the effect of intersection approach delay. The procedure contained in the NCHRP Report No. 387 was adopted in the determination of the 50th, 85th, and 99th percentile segment free flow speeds. The results showed that the free-flow speeds were lower than the segments' posted speeds. These results are consistent with values in the FSUTMS standards in the Cube Framework, Default Model Parameters.

8.1.2 Practical Capacities

The hourly volumes on uninterrupted flow facilities were analyzed by first converting the hourly vehicular flows into passenger car equivalents. Practical capacities of the non-toll and toll facilities were determined by area type – that is, urban, residential, and rural areas. Although the maximum hourly flow rates were determined, the practical capacity was defined as the 99th percentile flow to reduce chances of outliers. The results showed that hourly volumes on rural Florida freeways were below normal ranging from 937 passenger cars per hour per lane on 4-lane rural freeways to 1,362 passenger cars per hour per lane on 6-lane rural freeways. However, for the urban and residential segments for which data were available, the practical capacities were in line with those published in literature. In facility types where the observed 99th flows were lower than the proposed default capacities in the FSUTMS standards, the default values were retained, otherwise the capacity values were updated to reflect the observed 99th flows.

8.1.3 Speed-Flow Scatter Plots

The speed-flow scatter plots were developed using field data from telemetered traffic monitoring sites. These plots show the relationship between hourly speeds as a function of increasing hourly demands on a highway segment. The majority of speed-flow scatter plots plotted using data from various TTMS sites had a parabolic relationship consistent with those prevalent in literature. However, fitting of volume delay functions required extending the *v/c* ratio on the *x*-axis to include oversaturated conditions in which the *v/c* ratios exceed 1.0. To accomplish this, the following equation was used to determine demand, v_{demand}, above capacity for those TTMS sites which experienced oversaturated conditions for certain periods:

$$v_{demand} = \begin{cases} v_c + (v_c - v), & \text{if } \dfrac{U}{v} \leq \dfrac{U_c}{v_c} \text{ and } U \leq U_c \\ v, & \text{otherwise} \end{cases}$$

where v_c is flow at capacity, u_c is optimum speed at capacity, h_c is optimum average headway at capacity given by u_c/v_c. This equation produced reasonable results for uninterrupted flow facilities and will be useful to practitioners in the modeling speed-volume relationships.

8.1.4 Volume Delay Functions

Four volume delay functions were analyzed to determine their efficacy in predicting delay based on the loading on a facility represented by the v/c ratio. The fitting of volume delay functions on field speed-volume scatter plots developed above is done with the purpose of determining the best function forms and the accompanying optimum parametric values that can reasonably fit field data for the purpose of enabling prediction of operating speeds at various traffic flow levels. The model parameters were estimated using the Gauss-Newton (GN) method of solving the nonlinear least squares problem while the goodness-of-fit were evaluated using root-mean-square error (*RMSE*), the root-mean-square percent error (*RMSPE*), the mean error (*ME*), and the mean percent error (*MPE*) statistics as measures of performance. The results showed that the Modified Davidson model and the modified BPR equation fitted the field data better than Conical and Akcelik equations for the uninterrupted flow facilities for both toll and non-toll facilities. The degree of fit was evident from virtual observation of plots as well as statistical summary of the goodness-of-fit measures.

8.1.5 Performance of Volume Delay Functions

The Orlando Urban Area Transportation Study (OUATS) Model was selected as the travel forecasting model to be used for testing the volume delay functions and their parameters that were developed during the study. The OUATS model is a daily travel forecasting model with a region that covers all or parts of six counties in central Florida, namely: Volusia (west), Lake, Seminole, Orange, Polk (northeast), and Osceola. The results of the VDF test show that volume of traffic estimated on uninterrupted facilities during the morning and evening peak periods are generally a relatively close match to the TTMS average weekday vehicle/lane counts for the corresponding hours for at least three of the five VDFs tested. The fitted BPR and Modified Davidson VDFs estimated traffic volumes with RMSE of 18.6 percent or less for each of the four peak hours in the AM and PM peak periods. The Modified Davidson and Conical VDFs generally underestimated the traffic volume for uninterrupted facilities in the tests by 18 percent or less. Overall the Fitted BPR parameters performed the best while the standard BPR volume delay function (from the original OUATS model) and the Akcelik VDF parameters produces the worse estimates with relatively high overestimation of the traffic volume.

8.2. Recommendations

This project was aimed at the performance of the FSUTMS in travel forecasting and highway performance monitoring. A number of inputs are necessary for forecasting volume and speeds in a transportation network of which SPEEDCAP table and VDF factors are a subset. It is therefore recommended that there is a need to experiment with the SPEEDCAP values and VDF factors for uninterrupted flow facilities developed by this study. Only the OAUTS Region model was used to test the developed volume delay functions. Thus, further testing of these functions in other regions is recommended. Appendix F shows implementation guidelines that can be followed in using lookup tables developed by this study or in further generating data necessary for modeling purposes. In addition, an electronic file containing revised speed-cap values and VDF factors is available and can be requested from the Project Manager.

Due to different characteristics of network links and travel activities from different metropolitan planning regions, sensitivity test on volume delay functions should be performed. In this study, the behavior of each calibrated curve was studied by conducting sensitivity analysis. The stability manifested by the fitted BPR, was consistent with the behavior of long stretches of uninterrupted flow facilities with low ramp densities. Facilities in fringe or outlying business district (OBD) areas, toll roads with medium spaced toll booths or plazas, or multilane highways with highly isolated signals could be characterized by modified Davidson's function. Speed modeling in freeways with high ramp density, toll roads with closely spaced toll plazas, or multilane highways with medium spaced signals, could be described by the Conical delay function. The behavior depicted by the Akcelik function was a characteristic of urban streets with closely spaced signalized intersections. Therefore, given that the evaluated VDFs have strengths and weaknesses when used in particular facility and area types, it is possible to apply more than one volume delay function in the urban model region in order to achieve the desired forecasting accuracy.

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APPENDIX A – Structure and Data Dictionary of Merged Data File

Table A.1. Structure and Data Dictionary of Merged Data File

Variable	Position	Format	Description	Data Source
Rec_Type_SPD	1	A3	Record Type: SPD = Record includes traffic count volume by speed bins	TTMS Hourly Speed data files
Cnty	2	A2	County Code	TTMS Hourly Speed data files
Site_ID	3	A4	Count Site ID	TTMS Hourly Speed data files
ATR_Lane	4	F1	ATR Lane Number	TTMS Hourly Speed data files
Year	5	F2	Year (2-digit)	TTMS Hourly Speed data files
Month	6	F2	Month (1 or 2 digits)	TTMS Hourly Speed data files
Day	7	F2	Day of Month (1 or 2 digits)	TTMS Hourly Speed data files
Hour	8	F2	Hour (1 to 24 hourly periods)	TTMS Hourly Speed data files
Minute	9	F2	Minute	TTMS Hourly Speed data files
Source	10	F4	TTMS source number	TTMS Hourly Speed data files
mph_1_20	11	F4	Traffic volume travelling at less than or equal to 20 mph.	TTMS Hourly Speed data files
mph_21_25	12	F4	Traffic volume travelling at 21 to 25 mph.	TTMS Hourly Speed data files
mph_26_30	13	F4	Traffic volume travelling at 26 to 30 mph.	TTMS Hourly Speed data files
mph_31_35	14	F4	Traffic volume travelling at 31 to 35 mph.	TTMS Hourly Speed data files
mph_36_40	15	F4	Traffic volume travelling at 36 to 40 mph.	TTMS Hourly Speed data files
mph_41_45	16	F4	Traffic volume travelling at 41 to 45 mph.	TTMS Hourly Speed data files
mph_46_50	17	F4	Traffic volume travelling at 46 to 50 mph.	TTMS Hourly Speed data files
mph_51_55	18	F4	Traffic volume travelling at 51 to 55 mph.	TTMS Hourly Speed data files
mph_56_60	19	F4	Traffic volume travelling at 56 to 60 mph.	TTMS Hourly Speed data files
mph_61_65	20	F4	Traffic volume travelling at 61 to 65 mph.	TTMS Hourly Speed data files
mph_66_70	21	F4	Traffic volume travelling at 66 to 70 mph.	TTMS Hourly Speed data files
mph_71_75	22	F4	Traffic volume travelling at 71 to 75 mph.	TTMS Hourly Speed data files
mph_76_80	23	F4	Traffic volume travelling at 76 to 80 mph.	TTMS Hourly Speed data files
mph_81_85	24	F4	Traffic volume travelling at 81 to 85 mph.	TTMS Hourly Speed data files
mph_gt_85	25	F4	Traffic volume travelling at greater than 85 mph.	TTMS Hourly Speed data files
Total_SPD	26	F6	Total traffic volume counted in lane.	TTMS Hourly Speed data files
Filename_SPD	27	A18	Name of speed data source file	Extracted from TTMS Hourly Speed data filenames
DATECHECK_FLAG	28	F1	Flag used to check consistency of record and file name dates. (1 = Dates consistent; 0= Dates not consistent)	Computed
DST_FLAG	29	F1	Daylight Saving Time Flag. (1 = Record date is "Spring Forward" or "Fall Backward" day; 0 = All other dates)	Computed
UNIT	30	F1	Traffic Sensor Unit Number (1 or 2)	Extracted from TTMS Hourly Speed data filenames
Rec_Type_CLS	31	A3	Record Type: CLS = Record include traffic count volume by vehicle type.	TTMS Hourly Classification data files
Class_01	32	F6	Volume: Motorcycles	TTMS Hourly Classification data files
Class_02	33	F6	Volume: All Cars	TTMS Hourly Classification data files
Class_03	34	F6	Volume: Pickups and Vans	TTMS Hourly Classification data files
Class_04	35	F6	Volume: Buses	TTMS Hourly Classification data files

Variable	Position	Format	Description	Data Source
Class_05	36	F6	Volume: 2-Axle Single Unit	TTMS Hourly Classification data files
Class_06	37	F6	Volume: 3-Axle Single Unit	TTMS Hourly Classification data files
Class_07	38	F6	Volume: 4-Axle Single Unit	TTMS Hourly Classification data files
Class_08	39	F6	Volume: 2-Axle Tractor with 1- or 2-Axle Trailer, 3-Axle Tractor, 1-Axle Trailer	TTMS Hourly Classification data files
Class_09	40	F6	Volume: 3-Axle Tractor with 2-Axle Trailer, 3-Axle Truck, with 2-Axle Trailer	TTMS Hourly Classification data files
Class_10	41	F6	Volume: Tractor w/single Trailer (6 or 7 Axles)	TTMS Hourly Classification data files
Class_11	42	F6	Volume: 5-Axle Multi-Trailer	TTMS Hourly Classification data files
Class_12	43	F6	Volume: 6-Axle Multi-Trailer	TTMS Hourly Classification data files
Class_13	44	F6	Volume: 7+ Axles	TTMS Hourly Classification data files
Class_14	45	F6	Volume: Not used.	TTMS Hourly Classification data files
Class_15	46	F6	Volume: Unknown Vehicle Type	TTMS Hourly Classification data files
Total_CLS	47	F6	Volume: Total vehicle classification count	TTMS Hourly Classification data files
Filename_CLS	48	A18	Name of classification data source file	Extracted from TTMS Hourly Classification data filenames
LightVeh	49	F6	Volume: Passenger Vehicles, Vans and Pickups	Computed
HeavyVeh	50	F6	Volume: Buses, SU Trucks, Combination Trucks and Unknown Vehicles	Computed
PctHeavyVeh	51	F6.2	Percent heavy vehicles	Computed
GoodRec_Flag	52	F1	Good Record Flag (1 = Good; 2 = Bad)	Computed
DataStruct_Flag	53	F1	Source File Data Structure Flag (1 = Good; 0 = Bad)	Computed
DIRECTION	54	A1	Direction of travel. (Valid values: N, S, E, W)	A1
Dir_Flag	55	F1	Direction Flag (1 = N; 2 = E; 3 = S; 4 = W; 9 = Unknown)	Lookup: Lane Relationship file
BadDay_Flag	56	F1	Bad Daily Record Flag (1 = Bad record; 0 = Good record)	Lookup: Bad Daily Record Table
FHWA_LANE	57	F2	FHWA Lane number (1 to 14; 99 = Unknown)	Lookup: Lane Relationship File
DirLanes	58	F3	Number of lanes in the particular lane's direction of travel	Computed
Lane_Position	59	A10	Relative position of lane	Computed
SECTION	60	A8	TTMS Site: 8-digit County/Section/Subsection number	Lookup: TTMS Location File
LOCATION	61	F6.3	TTMS Site: Milepost of site location	Lookup: TTMS Location File
URBSIZE	62	A1	TTMS Site: Rural/urban/urbanized code, from RCI Feature 124	Lookup: TTMS Location File
HWYLOC	63	A1	TTMS Site: Highway location, from RCI Feature 124	Lookup: TTMS Location File
FUNCL	64	A2	TTMS Site: Functional Classification	Lookup: TTMS Location File
NUMLANE	65	F2	TTMS Site: Number of lanes	Lookup: TTMS Location File
ASCDIR	66	A1	TTMS Site: Ascending Direction of survey (Valid values: B = Bi-directional; N; S; E; W)	Lookup: TTMS Location File
DSCDIR	67	A1	TTMS Site: Descending Direction of survey (Valid values: B = Bi-directional; N; S; E; W)	Lookup: TTMS Location File
ASCLANE	68	F2	TTMS Site: Number of lanes in ascending direction	Lookup: TTMS Location File
DSCLANE	69	F2	TTMS Site: Number of lanes in descending direction	Lookup: TTMS Location File

Variable	Position	Format	Description	Data Source
SFCAT	70	A4	TTMS Site: Seasonal Factor Category	Lookup: TTMS Location File
AFCAT	71	A4	TTMS Site: Axle Factor Category	Lookup: TTMS Location File
SITETYPE	72	A1	TTMS Site: Site Type (Valid value = T, for TTMS)	Lookup: TTMS Location File
COMM	73	A69	TTMS Site: Additional descriptive data	Lookup: TTMS Location File
ACTIVE	74	A1	TTMS Site: (Valid values: 'Y' = active; 'N' = inactive)	Lookup: TTMS Location File
LATITUDE	75	F12.5	TTMS Site: Latitude	Lookup: TTMS Location File
LNGITUDE	76	F12.5	TTMS Site: Longitude	Lookup: TTMS Location File
AADT	77	F6	TTMS Site: AADT	Lookup: TTMS Location File
CLASSD	78	A3	TTMS Site: Vehicle Classification Count Site	Lookup: TTMS Location File
KFCTR	79	F5.2	TTMS Site: K Factor	Lookup: TTMS Location File
DFCTR	80	F5.2	TTMS Site: D Factor	Lookup: TTMS Location File
TFCTR	81	F5.2	TTMS Site: T Factor	Lookup: TTMS Location File
FTYPE	82	F2	Facility Type (General Use facilities/lanes)	Statewide Model
ATYPE	83	F2	Area Type	Statewide Model
Classd_Flag	84	F1	Classification Count Site Flag (1 = Yes; 0 = No)	Computed
Active_Flag	85	F1	Active Site Flag (1 = Yes; 0 = No)	Computed
Gen_FT_AT	86	F2	FTYPE/ATYPE Combination (Note: FTYPE is for General Use facilities/lanes only)	Computed
TIMEZONE	87	A1	Time Zone of Site (E = Eastern; C = Central)	Lookup: Posted Speed Limit File
Posted_Speed	88	F3	Posted Speed Limit	Lookup: Posted Speed Limit File
CLS_Data_Flag	89	F1	Classification Data Flag (1 = Record includes CLS Data; 0 = No CLS data is in the record)	Computed
SPD_Data_Flag	90	F1	Speed Data Flag (1 = Record includes SPD Data; 0 = No SPD data is in the record)	Computed
SPD_CLS_Data_Flag	91	F1	Speed and Classification Data Flag (1 = Record includes both SPD and CLS Data; 0 = No)	Computed
TotVol_AmtDiff	92	F6	Amount difference in Total classification and speed volumes	Computed
TotVol_PctDiff	93	F6.2	Percent difference in Total classification and speed volumes	Computed
VolEqual_Flag	94	F1	Volume Equal Flag (1 = SPD and CLS total volumes are equal; 2 = Volumes are not equal)	Computed
SpEvent_Flag	95	F11	Special Event Flag (1 = Special Event; 2 = No Special Event)	Lookup: Special Event File
SpEvent_Code	96	A13	Special Event Code	Lookup: Special Event File
SpEvent_Desc	97	A114	Description of Special Event	Lookup: Special Event File
DayOfWeek	98	A9	Day of Week	Lookup: Day Of Week and Holiday
rDayOfWeek	99	F1	Recoded Day of Week (1 = SUN; to, 7 = SAT; 8 = Holiday)	Computed

APPENDIX B – Analysis of TTMS Speed and Volume on Uninterrupted Flow Facilities

Table B. 1. Observed Flow

Facility Type	Area Type	Number of Lanes	Number of TTMs Sites	N (number of hourly records)	Minimum Flow	Maximum Flow	Mean Flow	Standard Error on Mean	Standard Deviation	99th Percentile Flow
					pcph/ln	*pcph/ln*	*pcph/ln*			*pcph/ln*
Freeway	Urban	2	1	15,792	57	1697	713	3.191	400.980	1,504
Freeway	Urban	3	4	55,248	28	2354	886	2.397	563.355	2,056
Freeway	Urban	4	1	8,605	68	1491	748	4.411	409.160	1,410
Freeway	Urban	5	3	27,812	41	1798	722	2.580	430.291	1,569
Freeway	Residential	2	7	96,907	1	1627	468	1.005	312.934	1,277
Freeway	Residential	3	14	222,874	15	2277	611	0.911	430.182	1,765
Freeway	Residential	4	1	17,245	34	2417	804	4.079	535.686	2,209
Freeway	Residential	5	1	17,052	60	1766	650	3.019	394.228	1,641
Freeway	Rural	2	14	232,367	1	1628	306	0.442	212.881	937
Freeway	Rural	3	5	81,091	1	1685	446	1.106	314.846	1,362

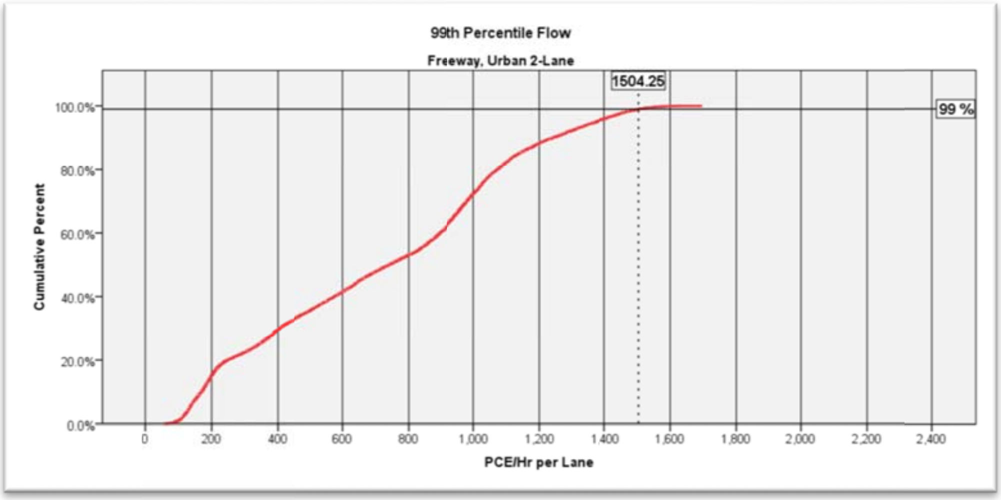

Figure B.1. Urban Freeway 2-Lane, Observed Maximum and 99th Percentile Flow

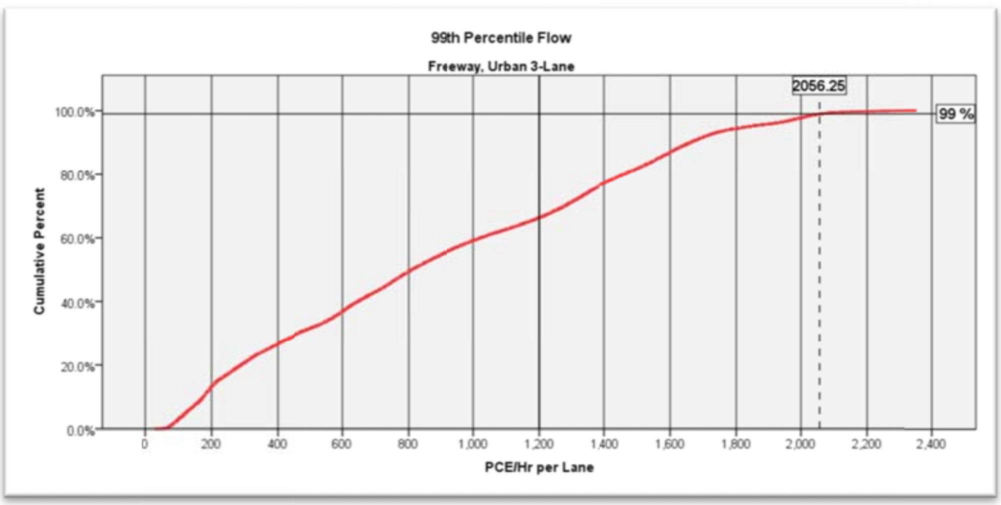

Figure B.2. Urban Freeway 3-Lane, Observed Maximum and 99th Percentile Flow

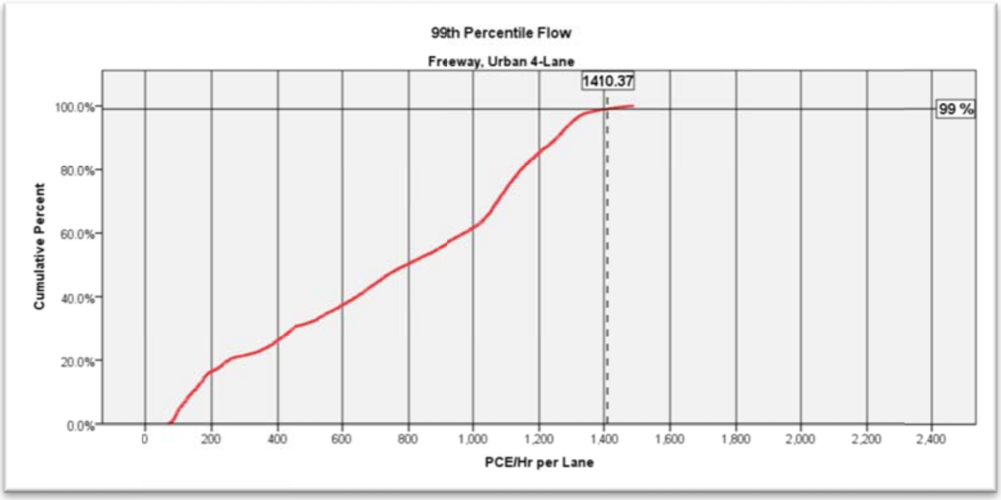

Figure B. 3. Urban Freeway 4-Lane, Observed Maximum and 99th Percentile Flow

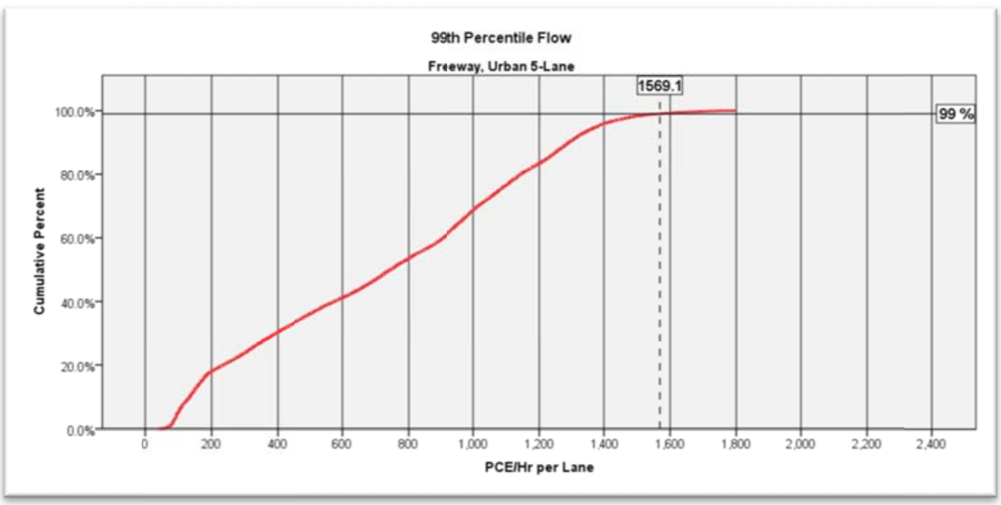

Figure B.4. Urban Freeway 5-Lane, Observed Maximum and 99th Percentile Flow

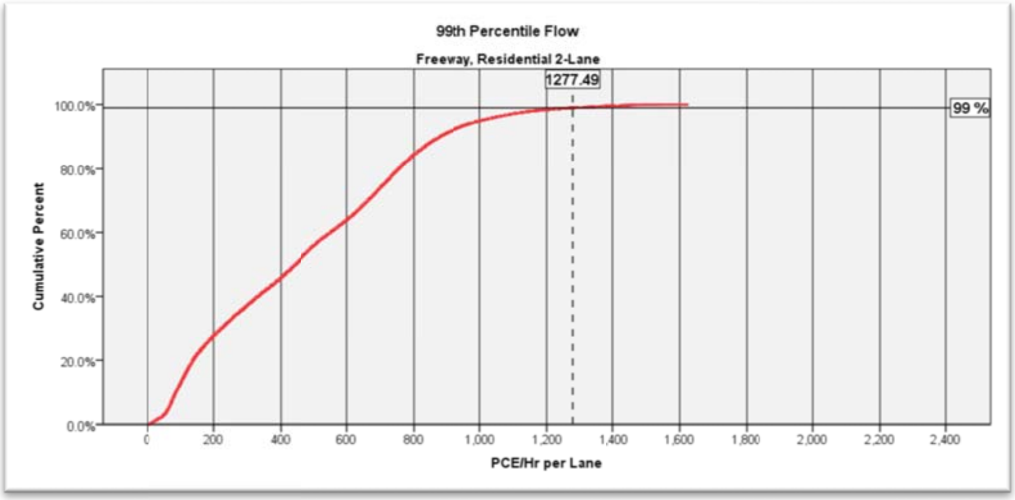

Figure B.5. Residential Freeway 2-Lane, Observed Maximum and 99th Percentile Flow

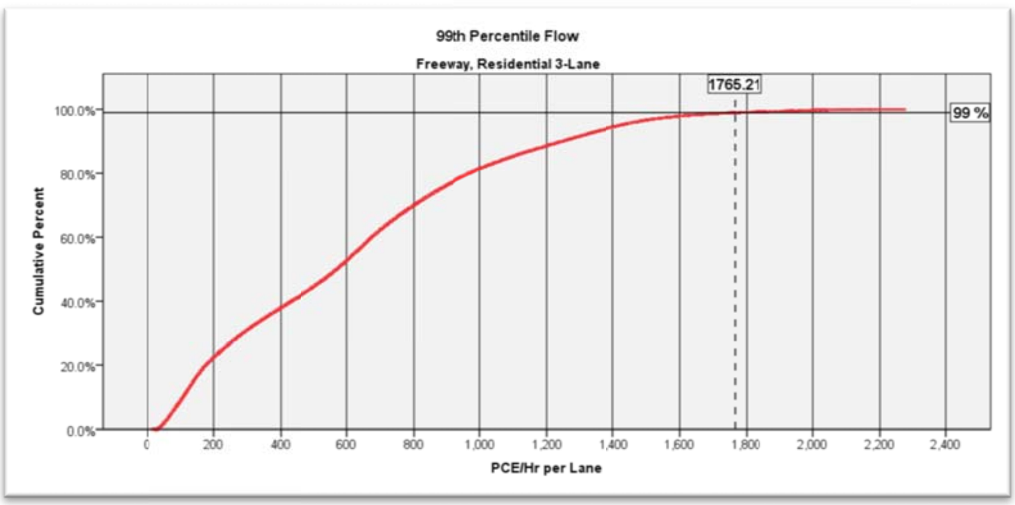

Figure B.6. Residential Freeway 3-Lane, Observed Maximum and 99th Percentile Flow

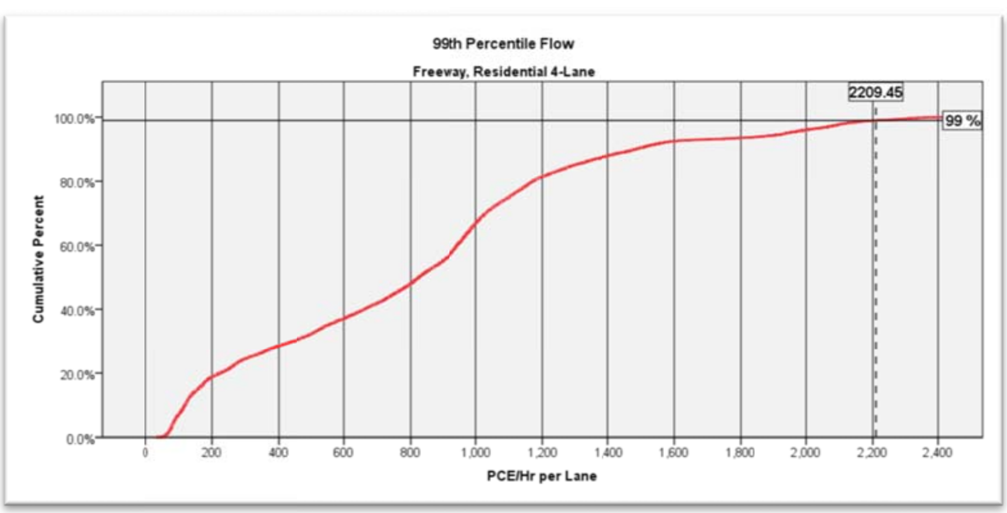

Figure B.7. Residential Freeway 4-Lane, Observed Maximum and 99th Percentile Flow

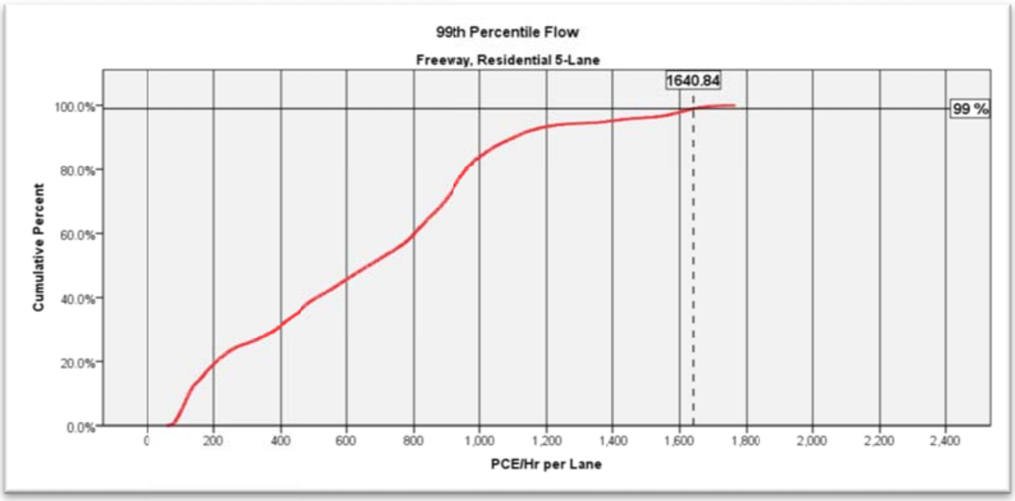

Figure B.8. Residential Freeway 5-Lane, Observed Maximum and 99th Percentile Flow

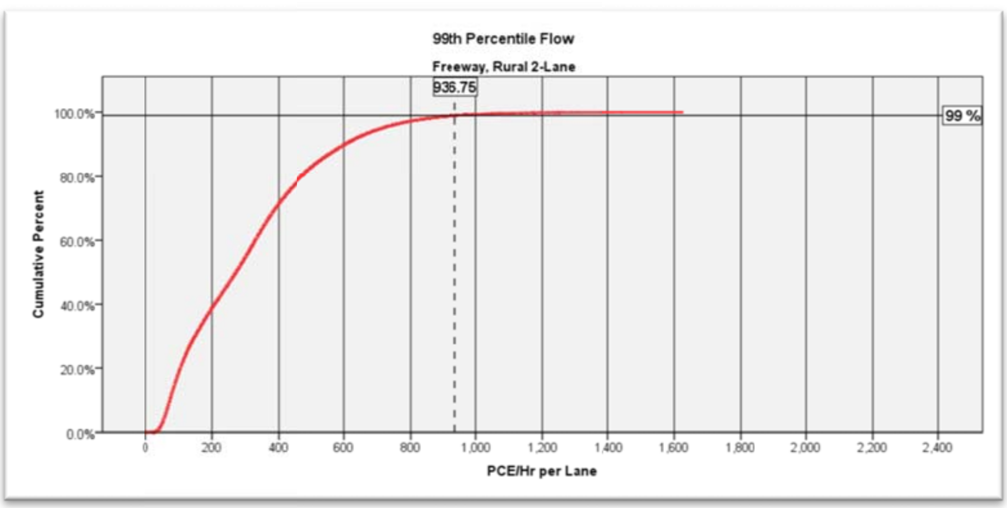

Figure B.9. Rural Freeway 2-Lane, Observed Maximum and 99th Percentile Flow

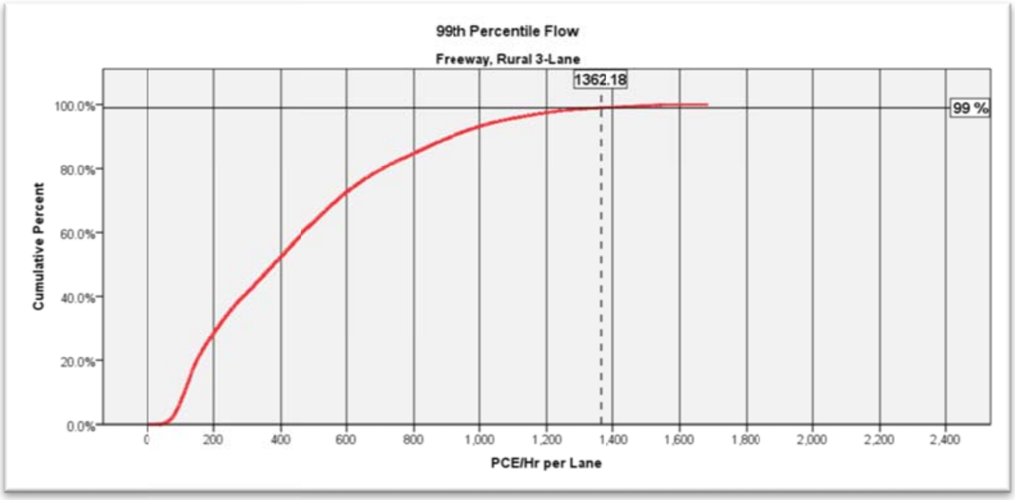

Figure B.10. Rural Freeway 3-Lane, Observed Maximum and 99th Percentile Flow

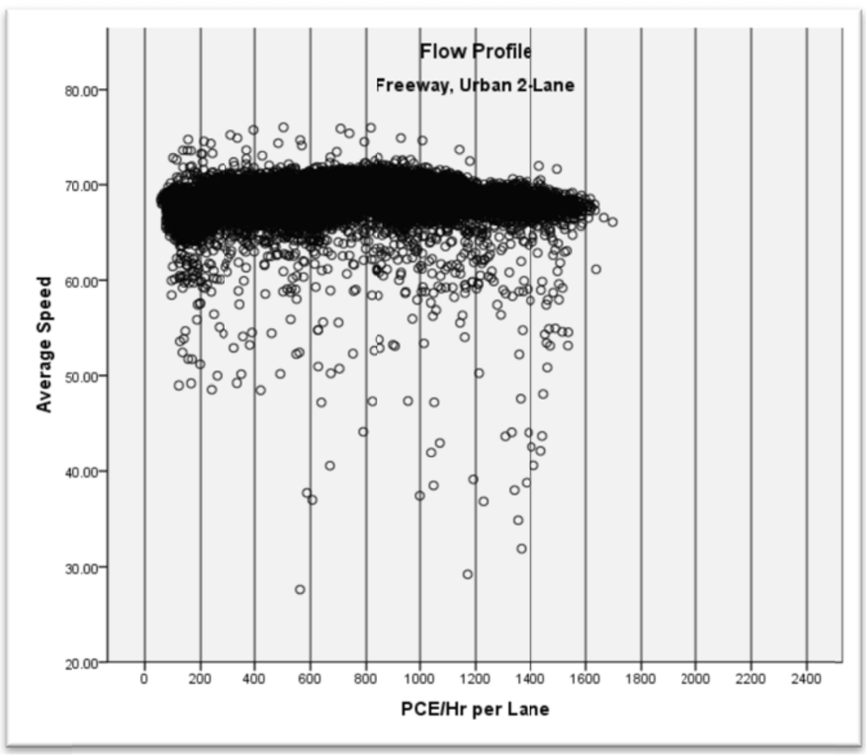

Figure B.11. Observed Speed v Flow - Urban Freeway, 2-Lane

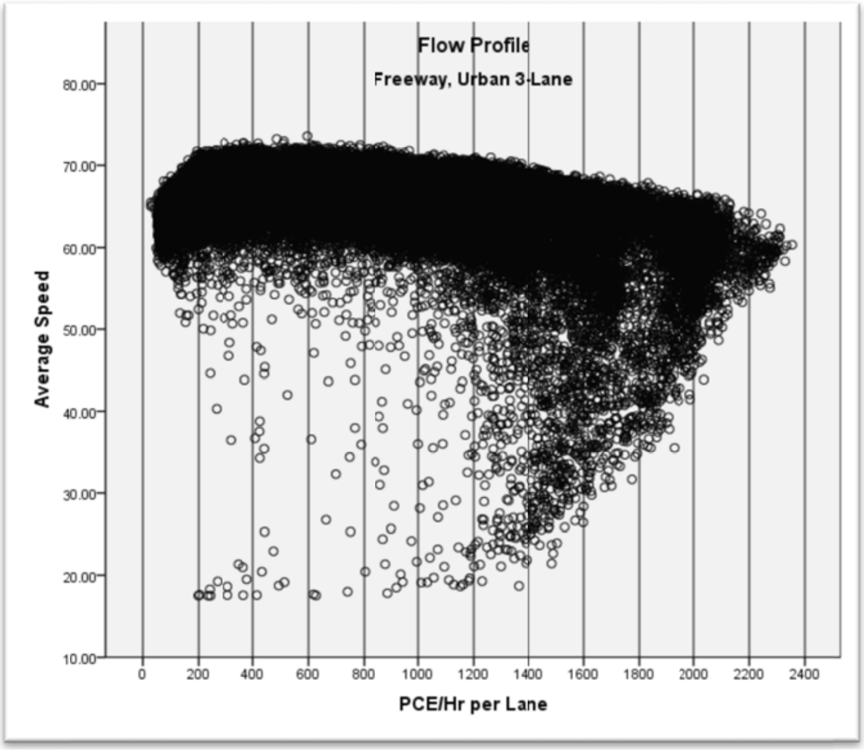

Figure B.12. Observed Speed v Flow - Urban Freeway, 3-Lane

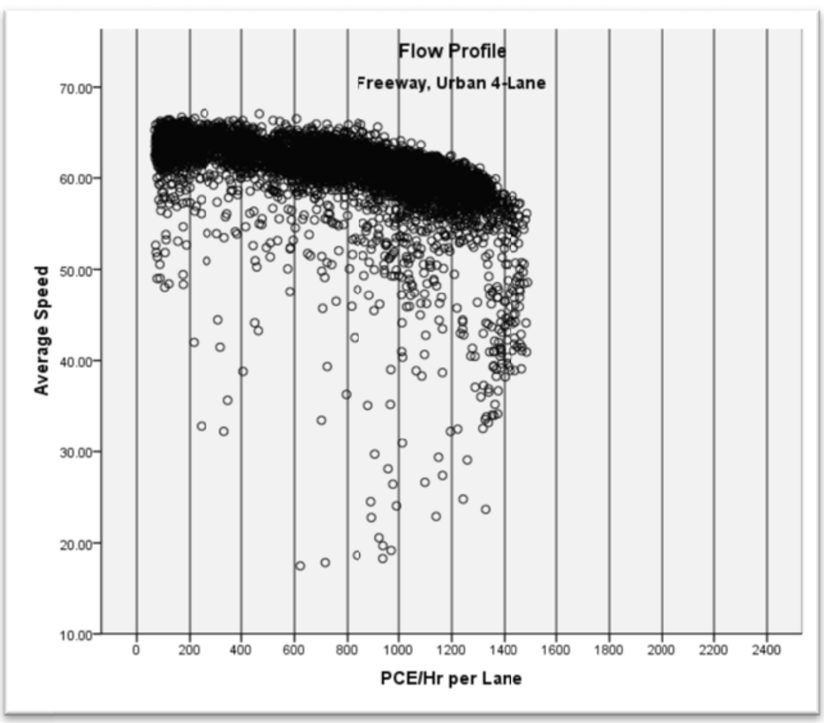

Figure B.13. Observed Speed v Flow - Urban Freeway, 4-Lane

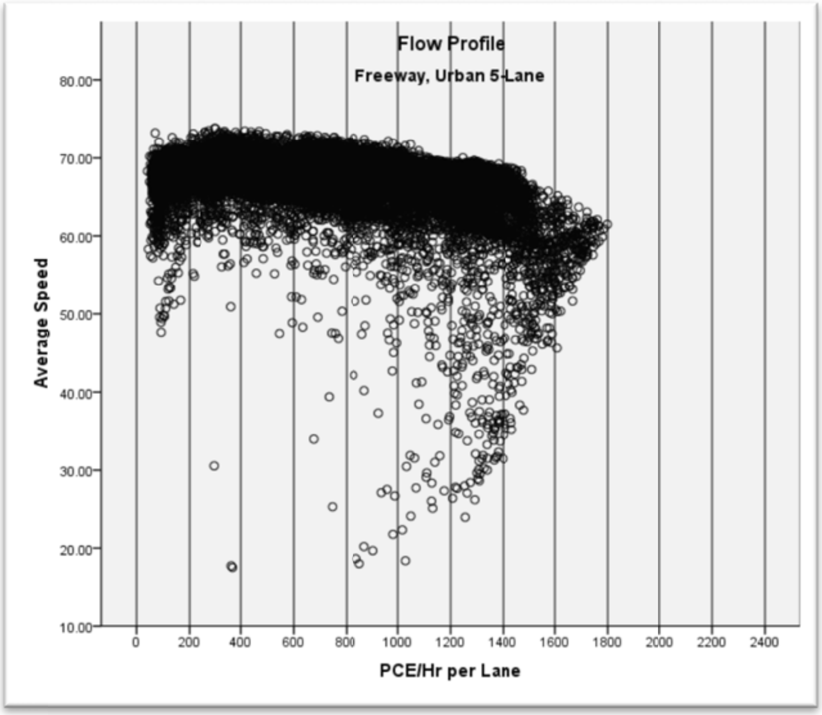

Figure B.14. Observed Speed v Flow - Urban Freeway, 5-Lane

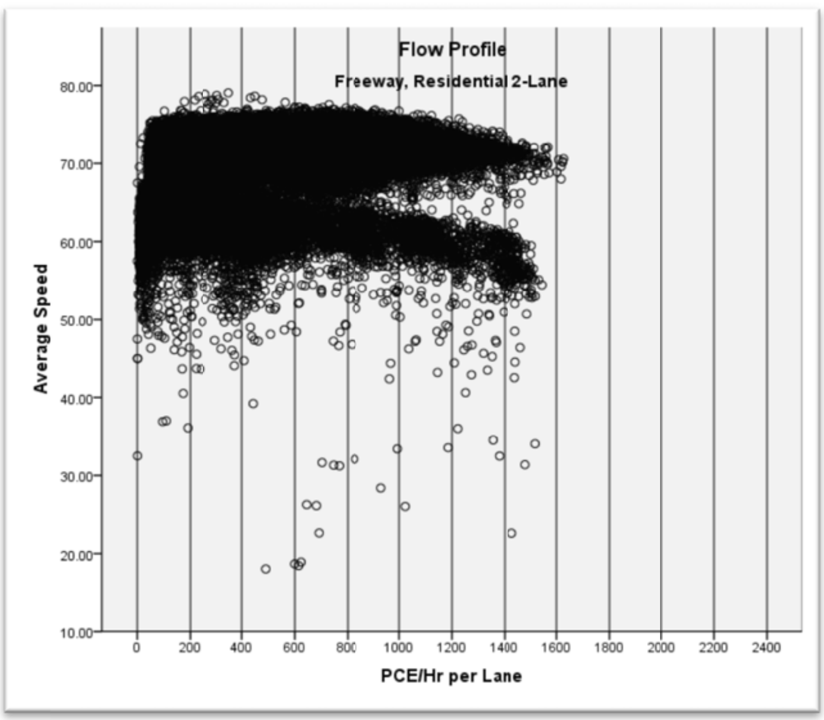

Figure B.15. Observed Speed v Flow - Residential Freeway, 2-Lane

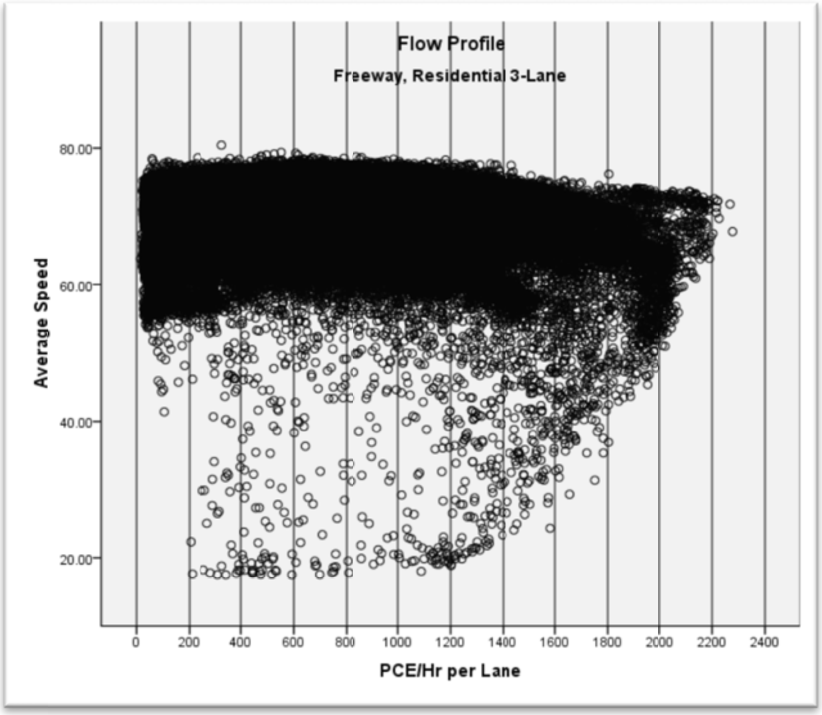

Figure B.16. Observed Speed v Flow - Residential Freeway, 3-Lane

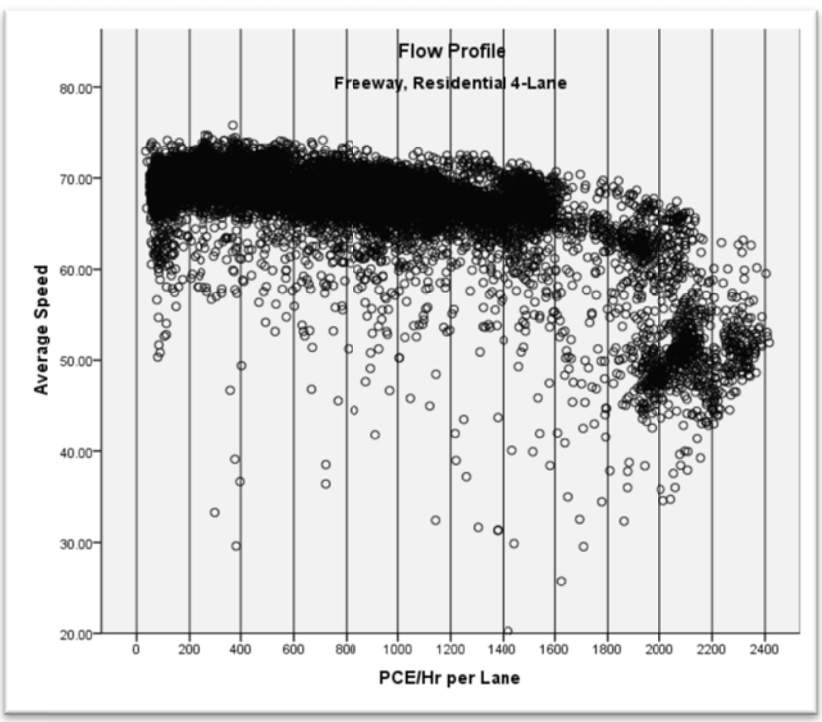

Figure B.17. Observed Speed v Flow - Residential Freeway, 4-Lane

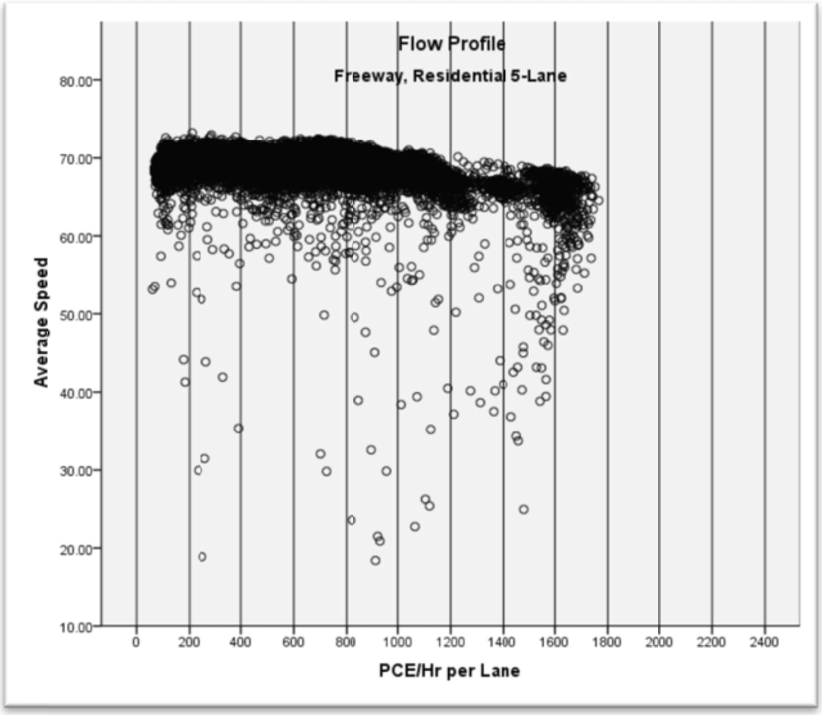

Figure B.18. Observed Speed v Flow - Residential Freeway, 5-Lane

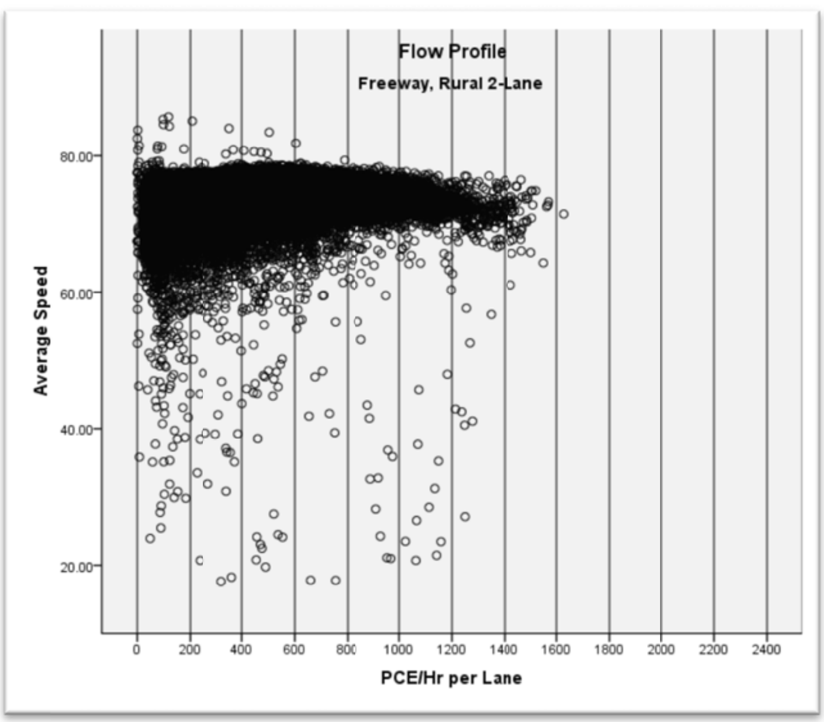

Figure B.19. Observed Speed v Flow - Rural Freeway, 2-Lane

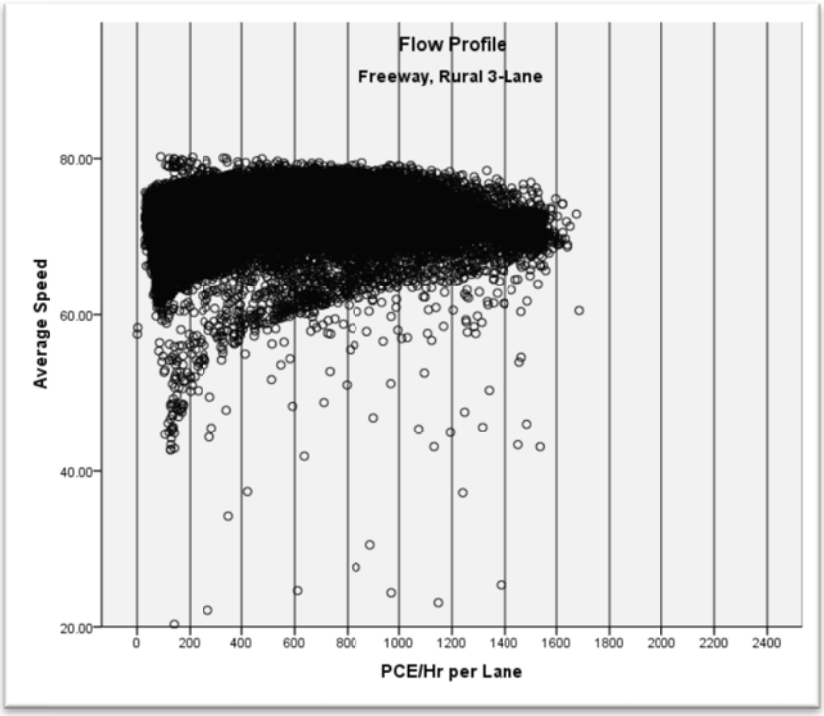

Figure B. 20. Observed Speed v Flow - Rural Freeway, 3-Lane

Table B.2. Observed Mean Free-Flow Speeds

Facility Type	Area Type	Number of Lanes	Number of TTMS Sites	N (number of hourly records)	Test Speed	Degrees of Freedom	Sample Mean Free-Flow Speed	Standard Deviation	Standard Error	t-value	Difference in Mean	95% CI Lower	95% CI Upper	Free-Flow Speed (Rounded)
Freeway	Urban	2	1	2,409	55	2408	67.0090	1.75181	0.03569	336.464	12.00899	11.9390	12.0790	67
Freeway	Urban	3	4	7,301	55	7300	65.0687	2.20884	0.02585	389.493	10.06867	10.0180	10.1193	65
Freeway	Urban	4	1	1,412	55	1411	63.6358	1.92704	0.05128	168.395	8.63577	8.5352	8.7364	64
Freeway	Urban	5	3	5,022	55	5021	67.9340	2.07574	0.02929	441.569	12.93399	12.8766	12.9914	68
Freeway	Residential	2	7	26,861	65	26860	68.5053	4.26629	0.02603	134.658	3.50526	3.4542	3.5563	69
Freeway	Residential	3	14	49,974	65	49973	70.4729	3.38039	0.01512	361.927	5.47288	5.4432	5.5025	70
Freeway	Residential	4	1	3,250	65	3249	69.1274	1.91465	0.03359	122.893	4.12740	4.0615	4.1933	69
Freeway	Residential	5	1	3,263	65	3262	69.1402	1.45531	0.02548	162.507	4.14017	4.0902	4.1901	69
Freeway	Rural	2	14	90,288	70	90287	71.2234	2.16601	0.00721	169.721	1.22343	1.2093	1.2376	71
Freeway	Rural	3	5	23,102	70	23101	70.6059	2.62153	0.01725	35.127	0.60585	0.5720	0.6397	71

Notes:

- N: Number of hourly speed records with flow < 200 PCE/hr/lane.
- Test Speed: Value to which the sample mean speed is compared in the T-test.
- CI: Confidence Interval

APPENDIX C – Results of INRIX Data Evaluation

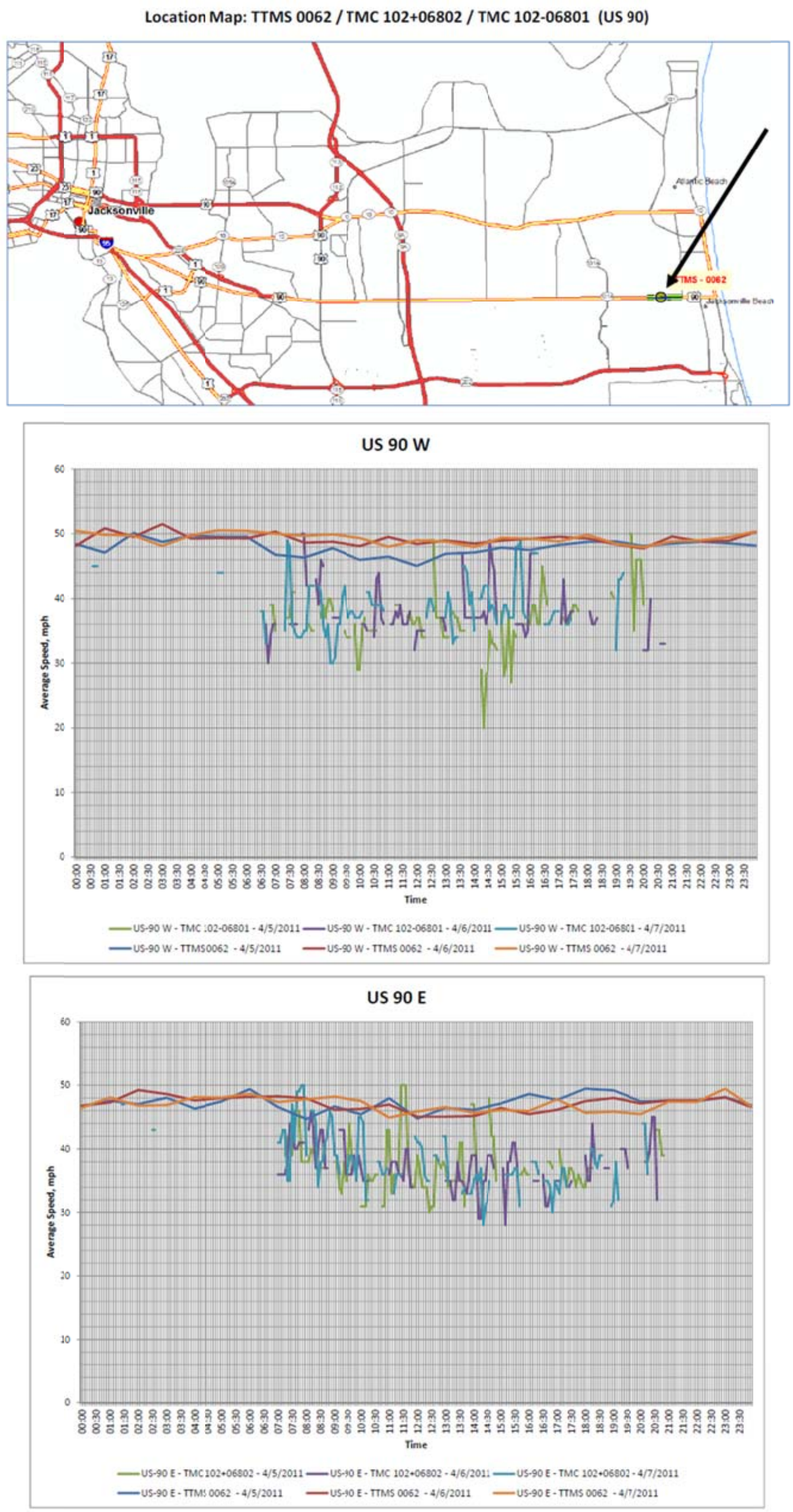

Figure C.1. Comparison of INRIX speeds to TTMS speeds at TTMS 0062

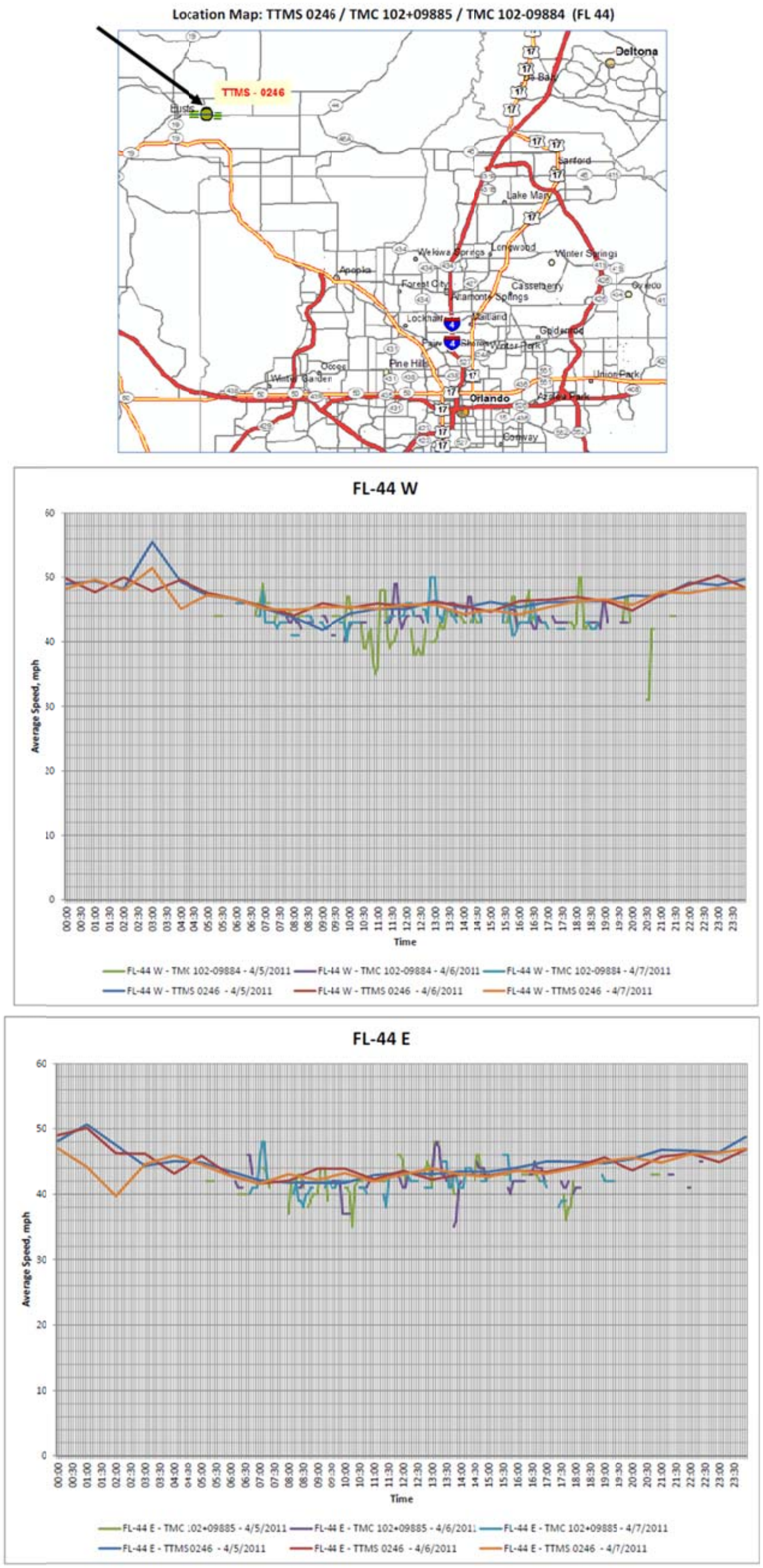

Figure C.2. Comparison of INRIX speeds to TTMS speeds at TTMS 0246

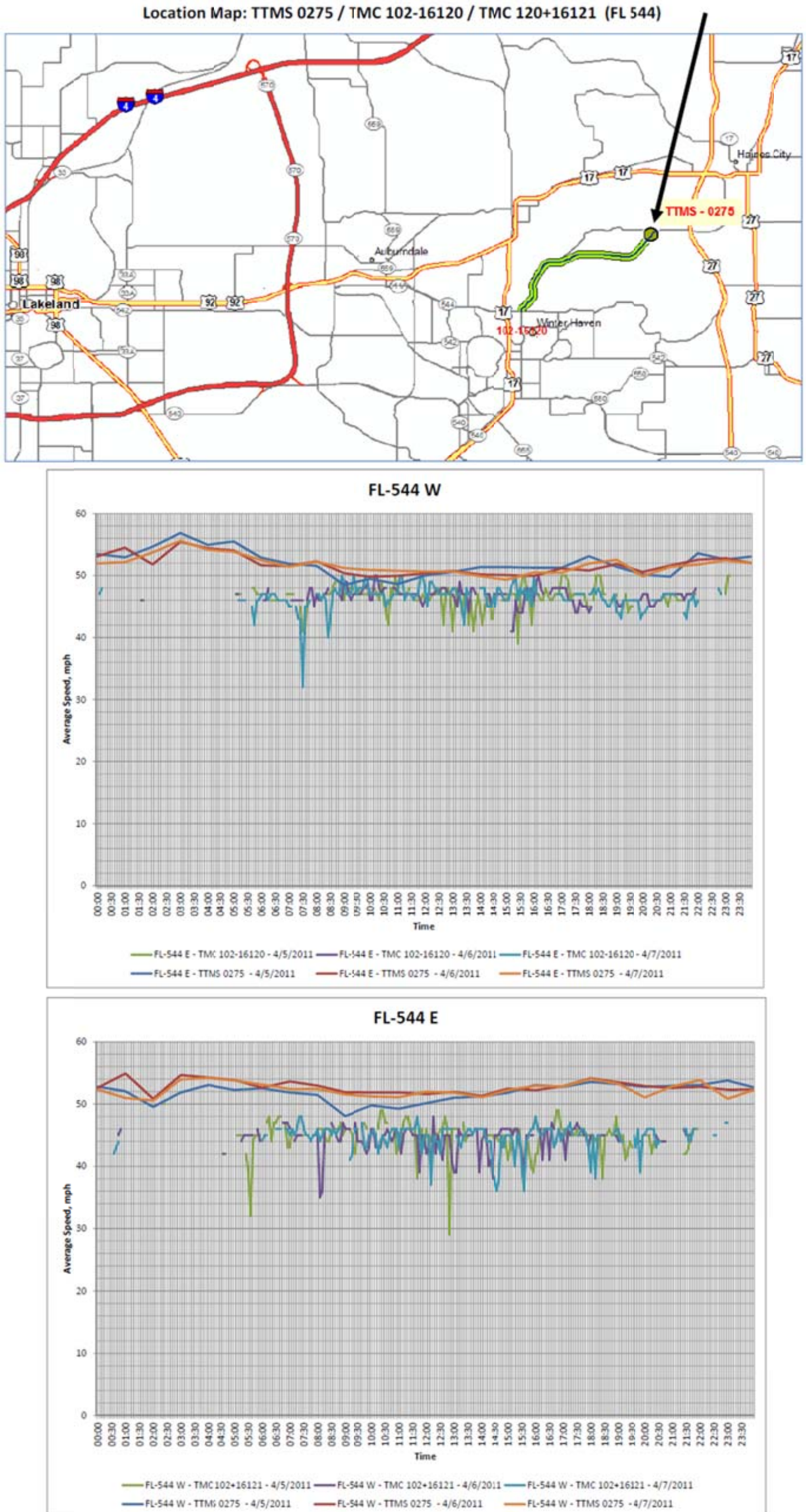

Figure C.3. Comparison of INRIX speeds to TTMS speeds at TTMS 0275

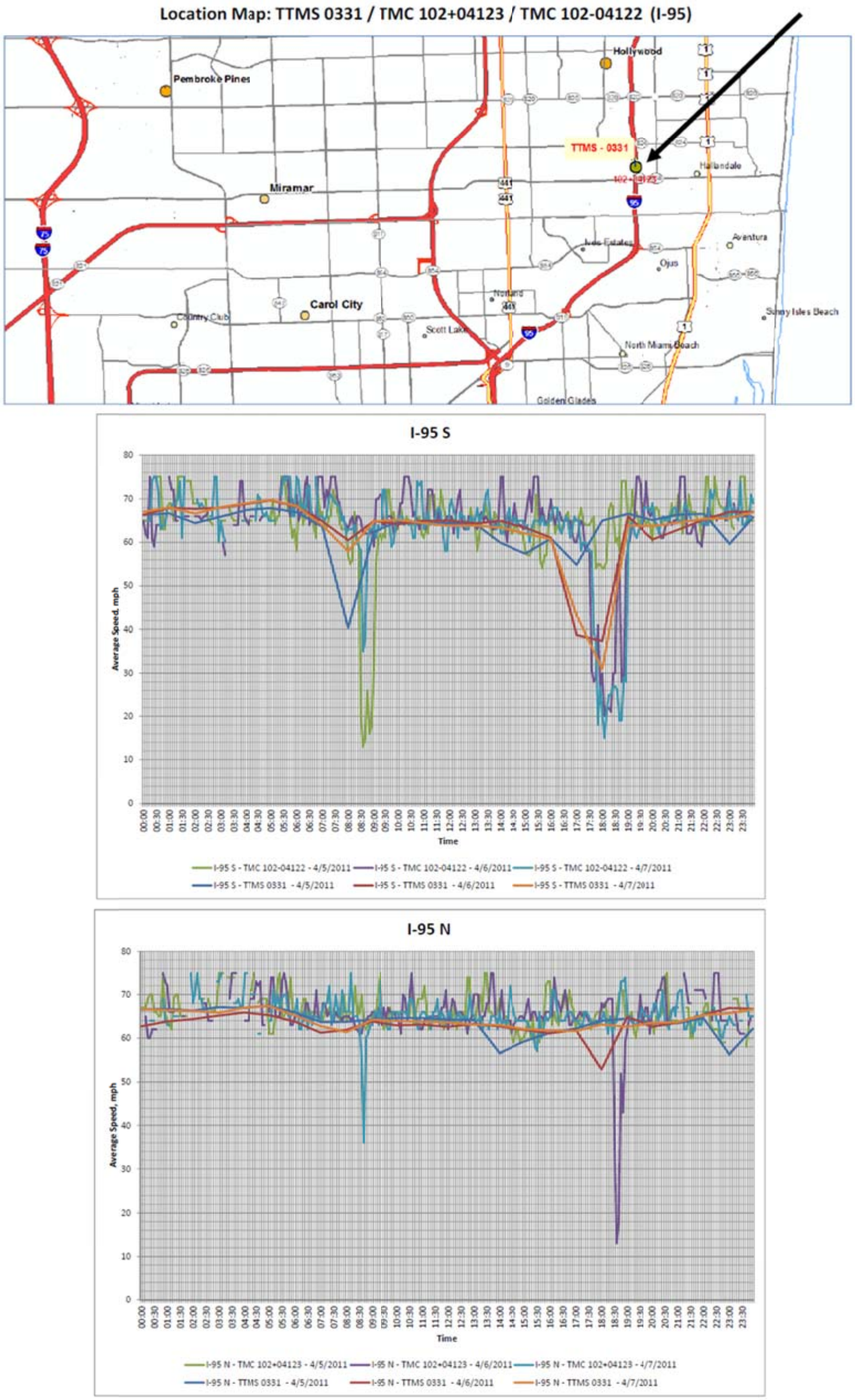

Figure C.4. Comparison of INRIX speeds to TTMS speeds at TTMS 0331

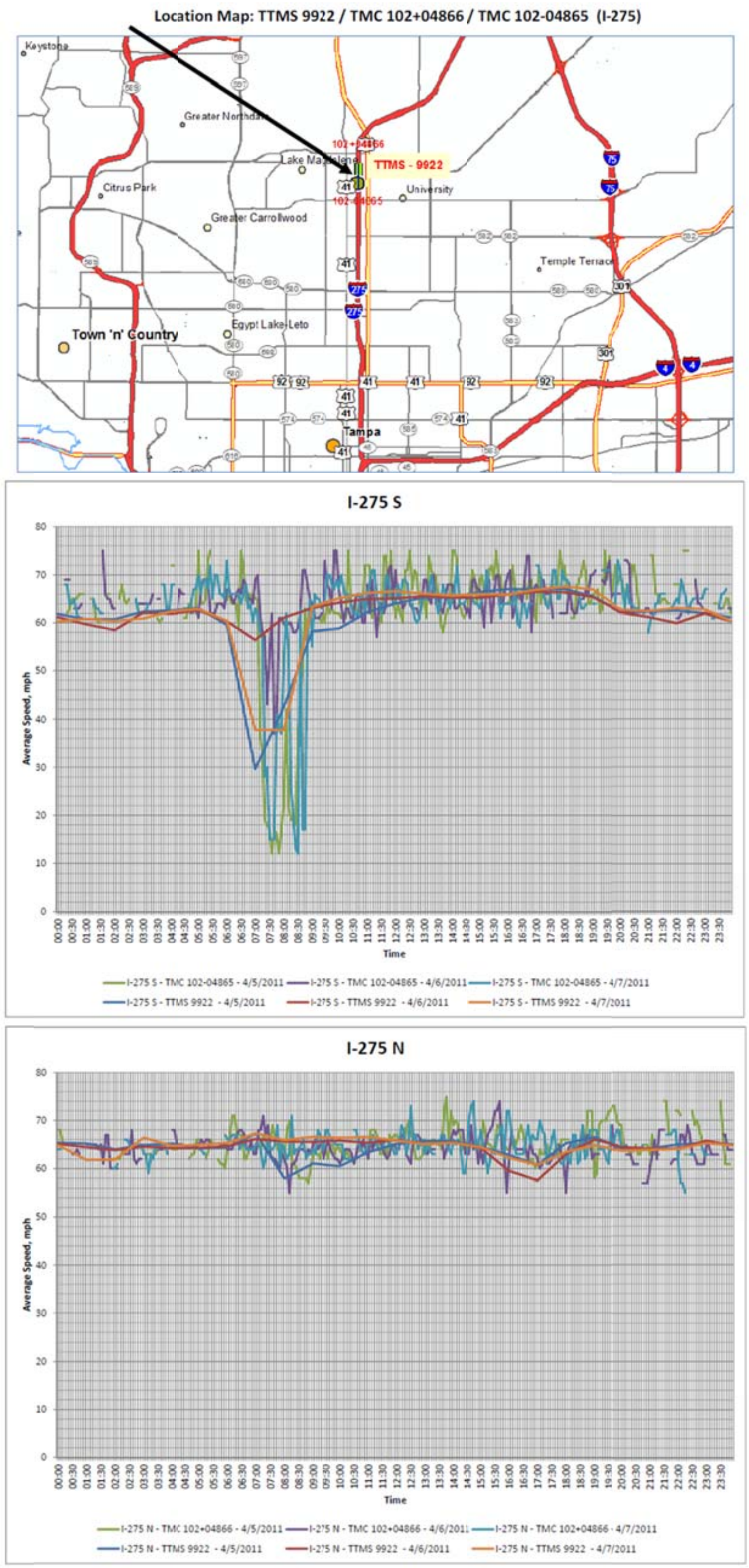

Figure C.5. Comparison of INRIX speeds to TTMS speeds at TTMS 9922

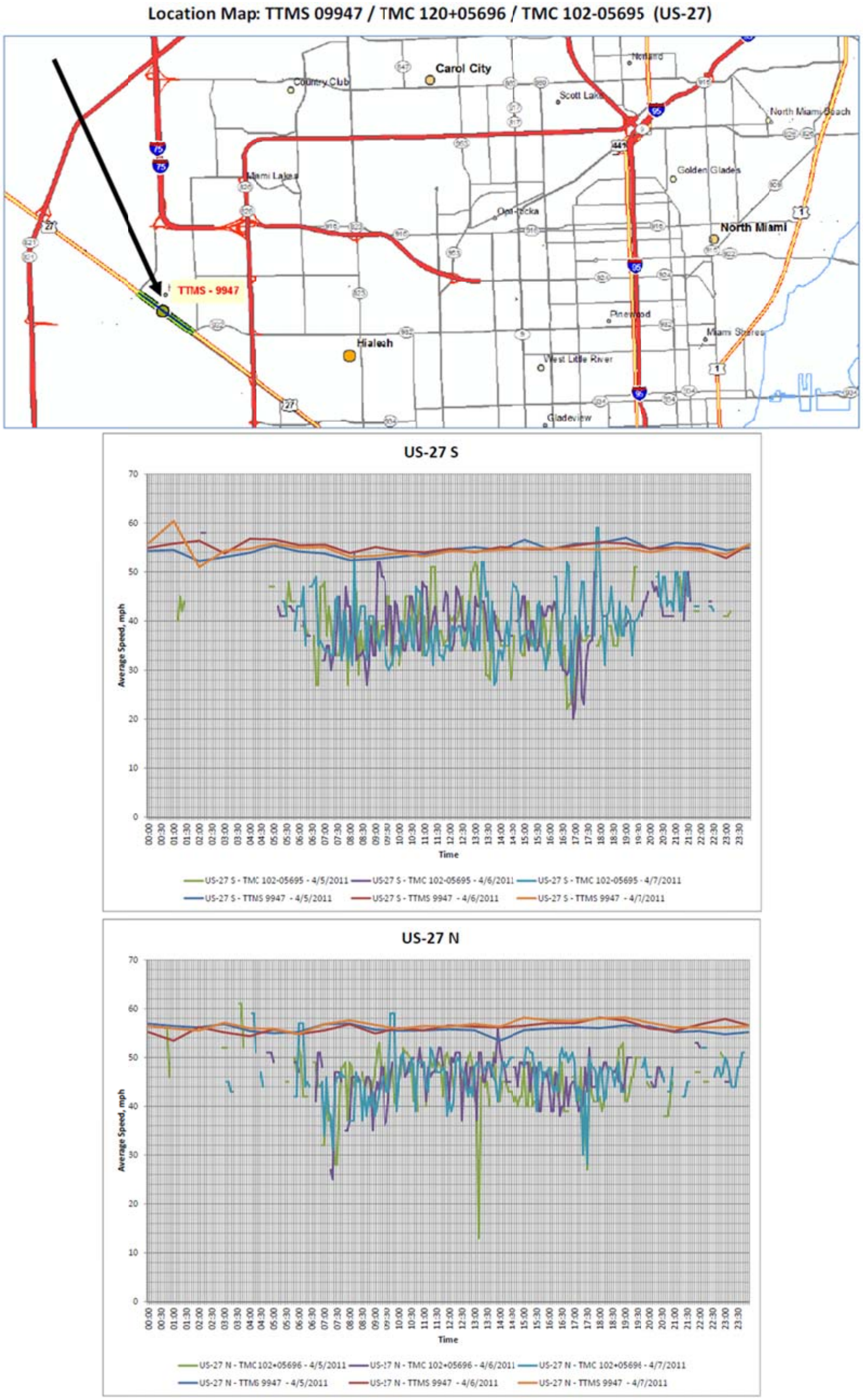

Figure C.6. Comparison of INRIX speeds to TTMS speeds at TTMS 9947

APPENDIX D –TTMS Speed and Volume Data on Uninterrupted Flow Facilities

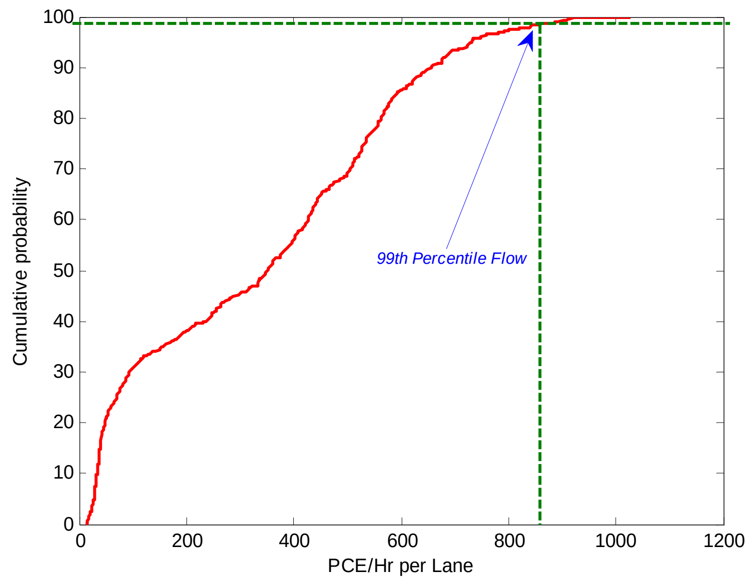

Figure D.1. Urban Undivided Arterial, 3-Lane, Flow Distribution

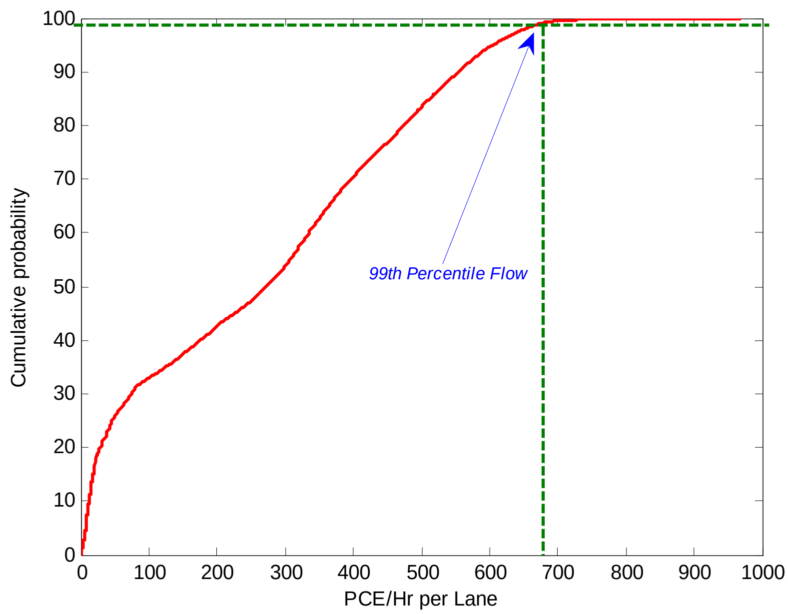

Figure D.2. Urban Divided Arterial, 2-Lane, Flow Distribution

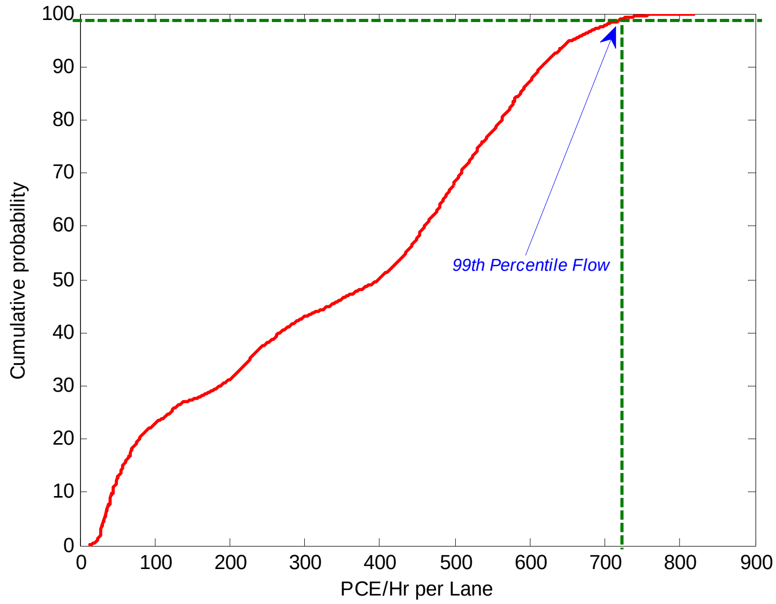

Figure D.3. Urban Divided Arterial, 3-Lane, Flow Distribution

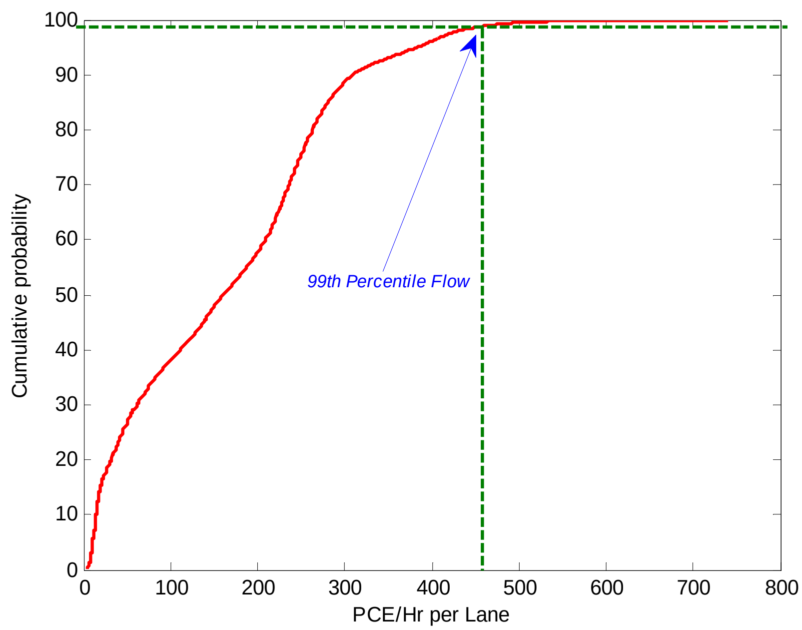

Figure D.4. Rural Divided Arterial, 2-Lane, Flow Distribution

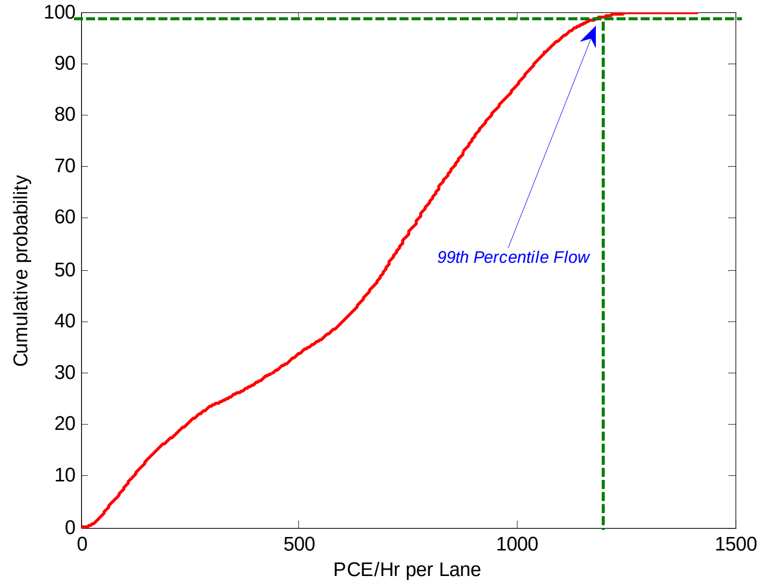

Figure D.5. Urban Divided Arterial, 2-Lane, Flow Distribution

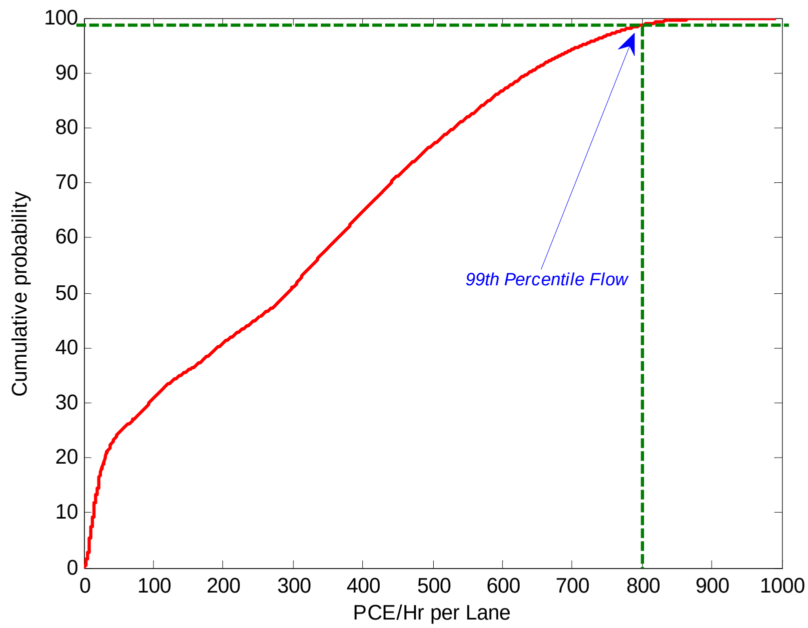

Figure D.6. Residential Divided Arterial, 2-Lane, Flow Distribution

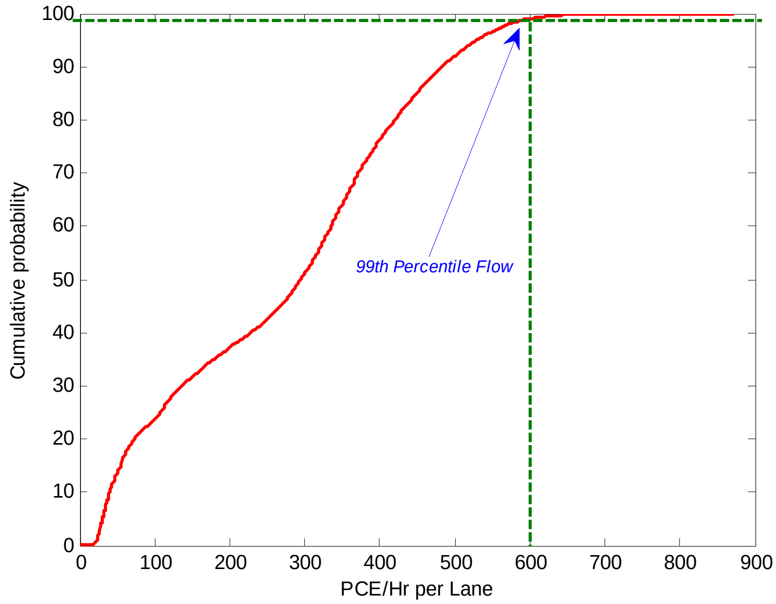

Figure D.7. Urban Undivided Arterial, 3-Lane, Flow Distribution

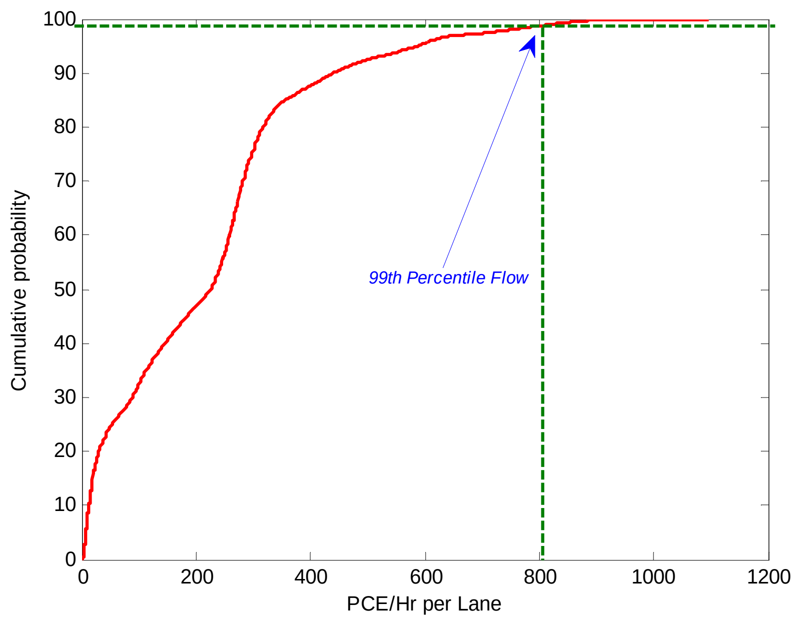

Figure D.8. Urban Undivided Arterial, 3-Lane, Flow Distribution

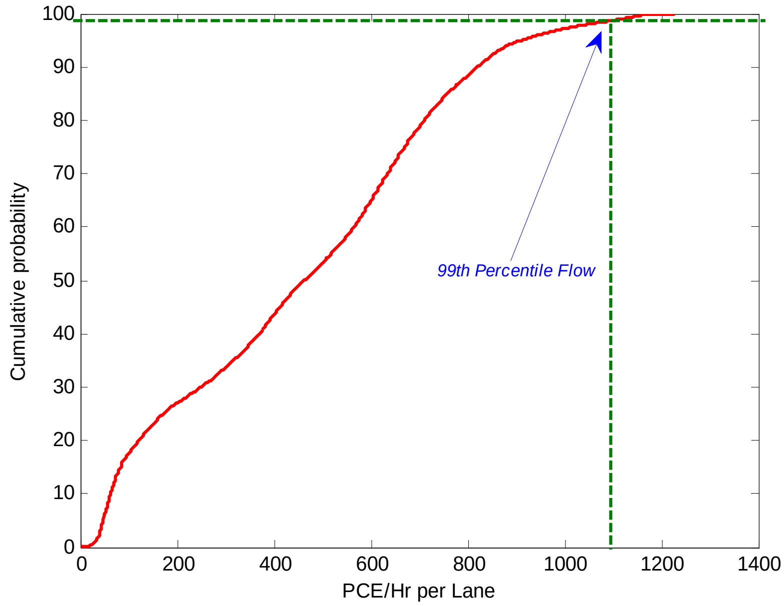

Figure D.9. Residential Undivided Arterial, 3-Lane, Flow Distribution

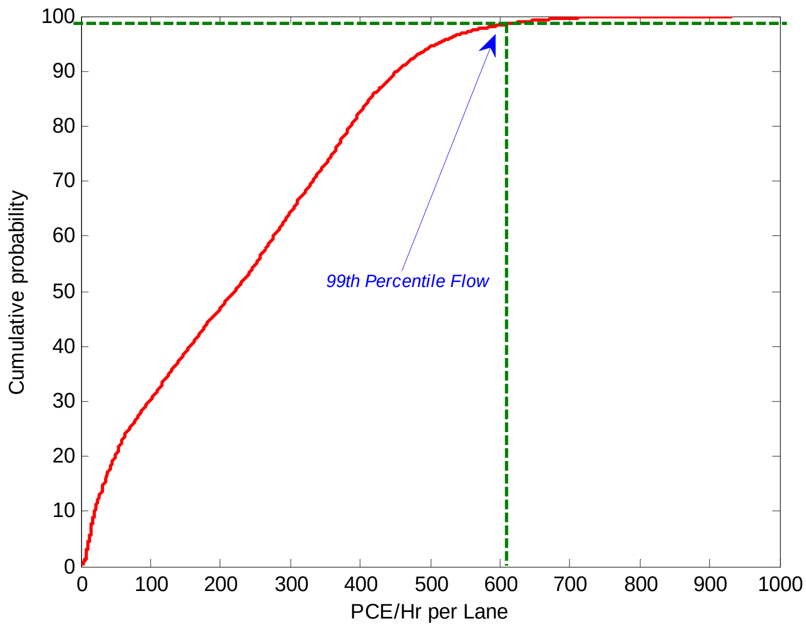

Figure D.10. Residential Undivided Arterial, 2-Lane, Flow Distribution

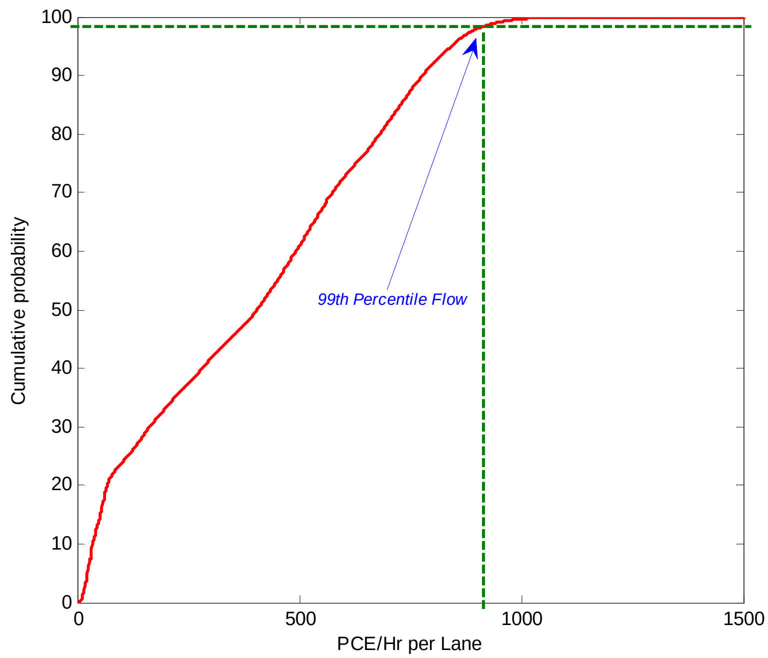

Figure D.11. Residential Undivided Arterial, 4-Lane, Flow Distribution

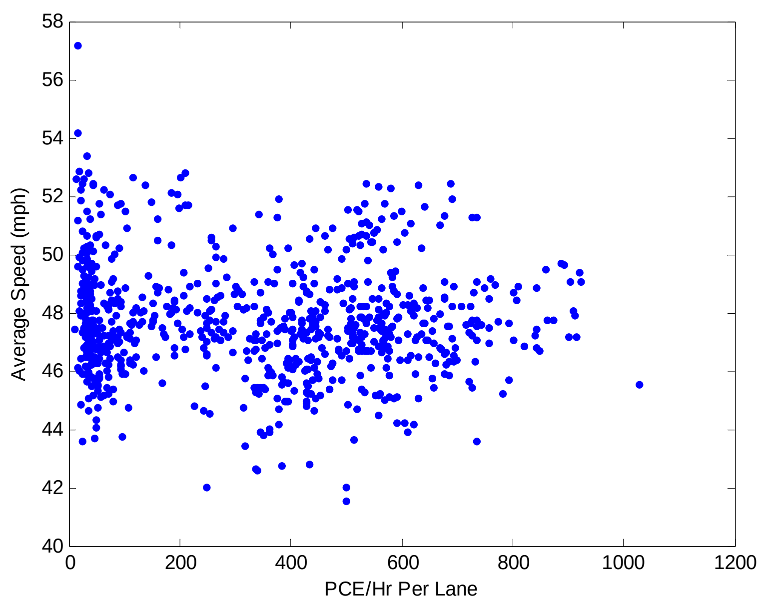

Figure D.12. Observed Speed vs. Flow - Urban Undivided Arterial, 3-Lane

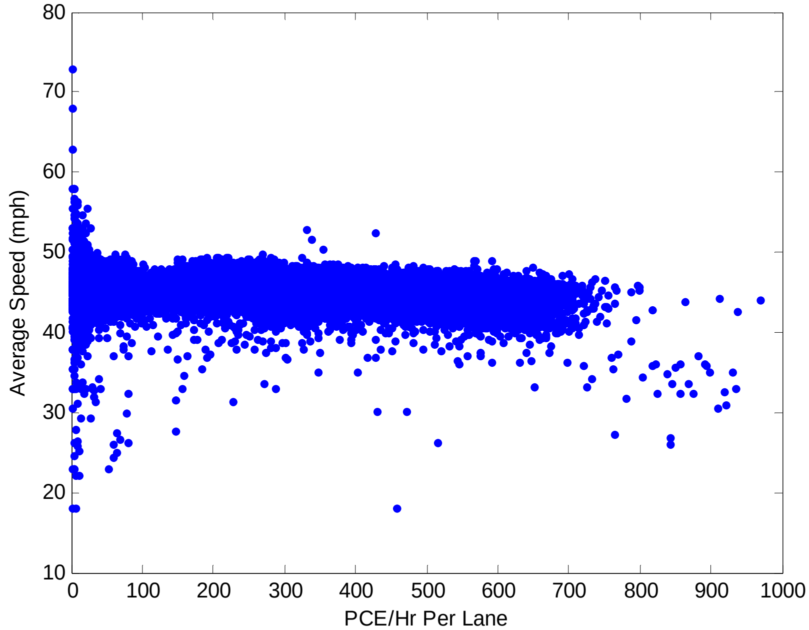

Figure D.13. Observed Speed v Flow - Urban Divided Arterial, 2-Lane

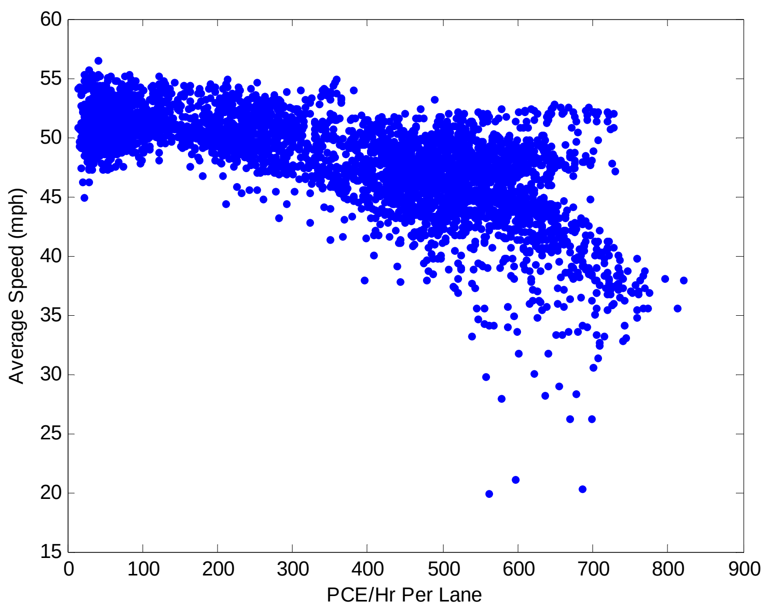

Figure D.13. Observed Speed v Flow - Urban Divided Arterial, 3-Lane

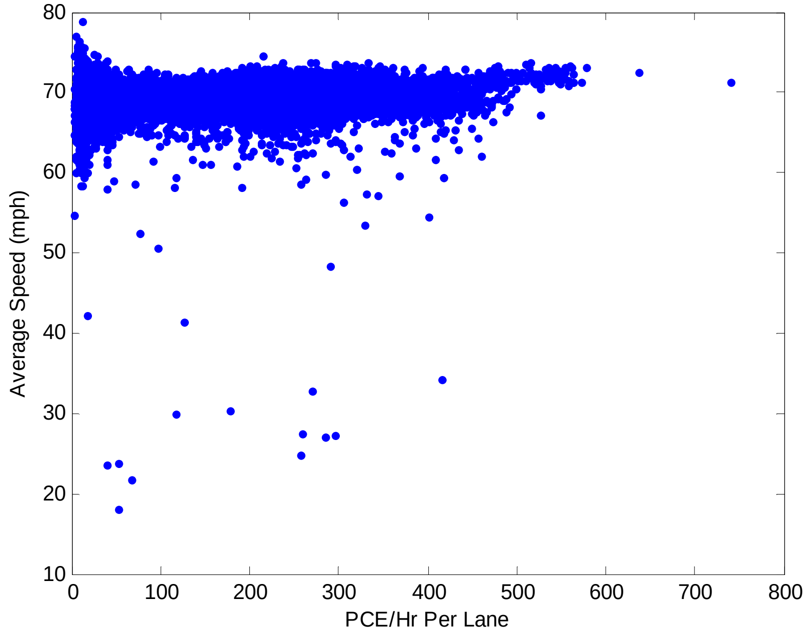

Figure D.14. Observed Speed v Flow - Rural Divided Arterial,   2-Lane

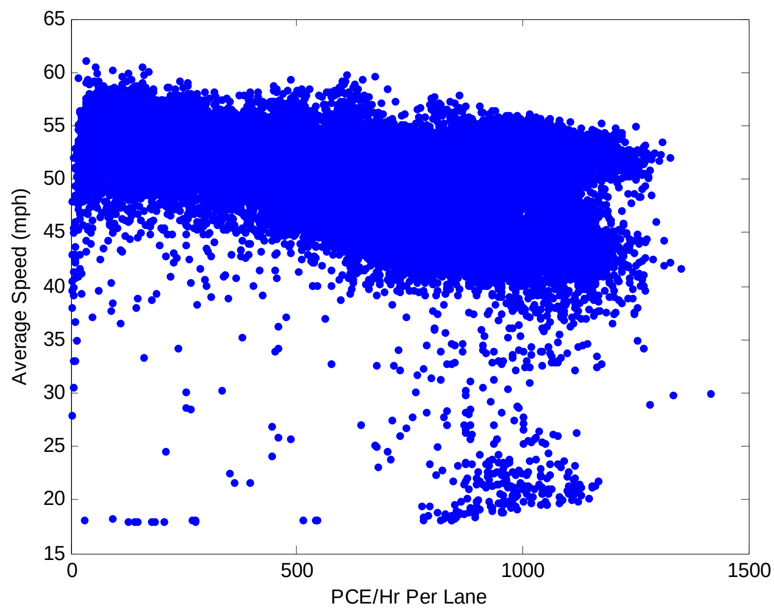

Figure D.15. Observed Speed v Flow - Urban Divided Arterial,   2-Lane

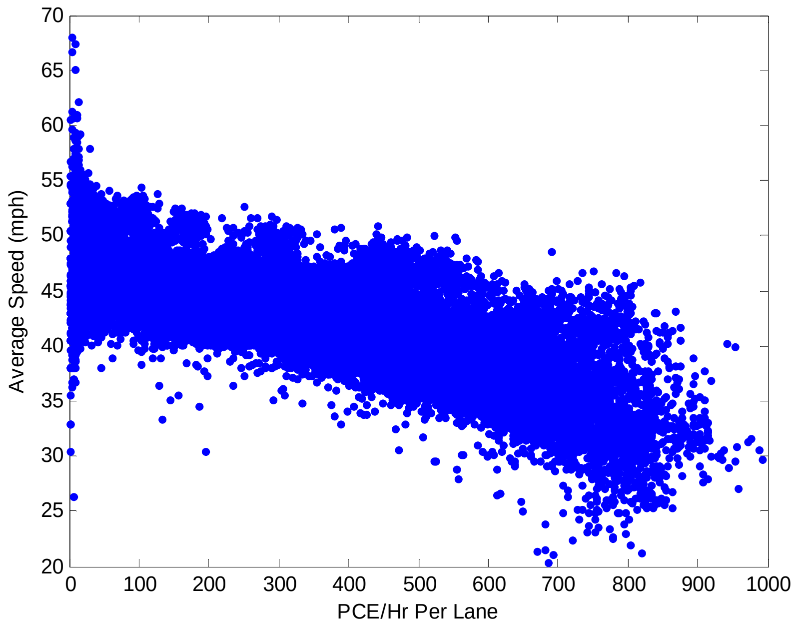

Figure D.16. Observed Speed v Flow - Residential Divided Arterial, 2-Lane

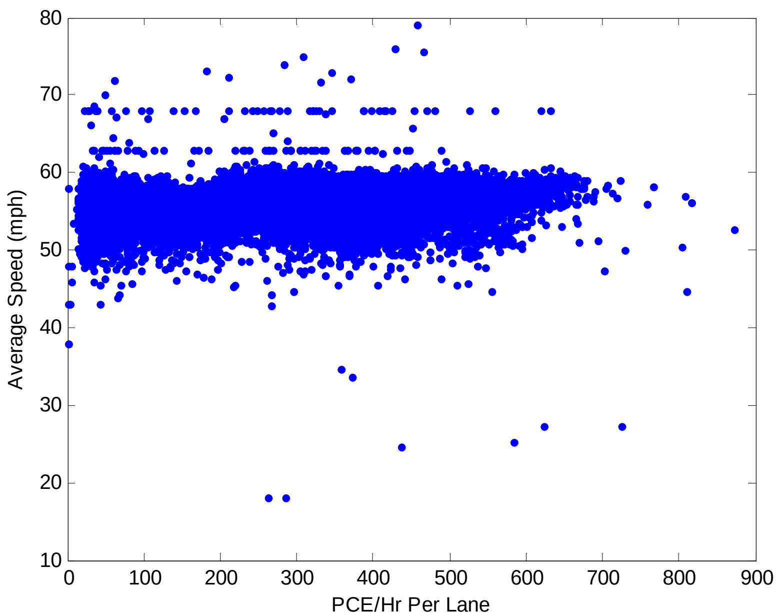

Figure D.17. Observed Speed v Flow - Urban Undivided Arterial, 3-Lane

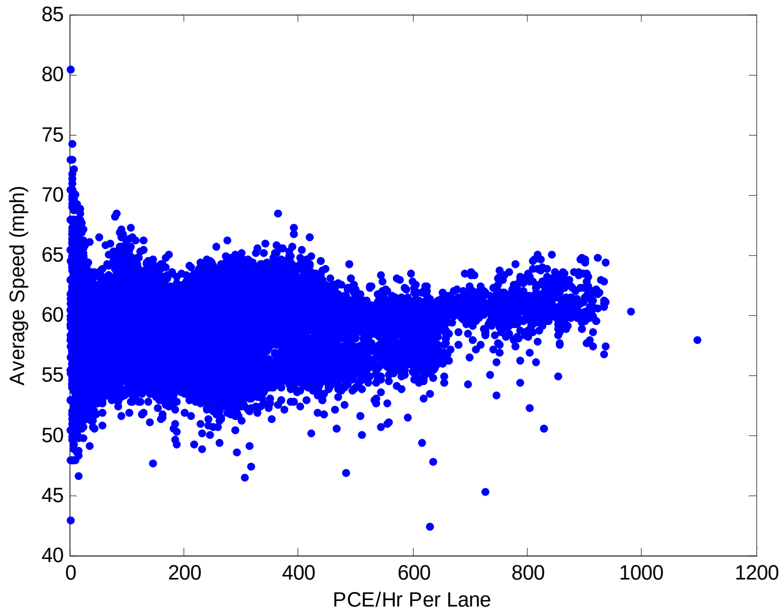

Figure D.18. Observed Speed v Flow - Urban Undivided Arterial, 3-Lane

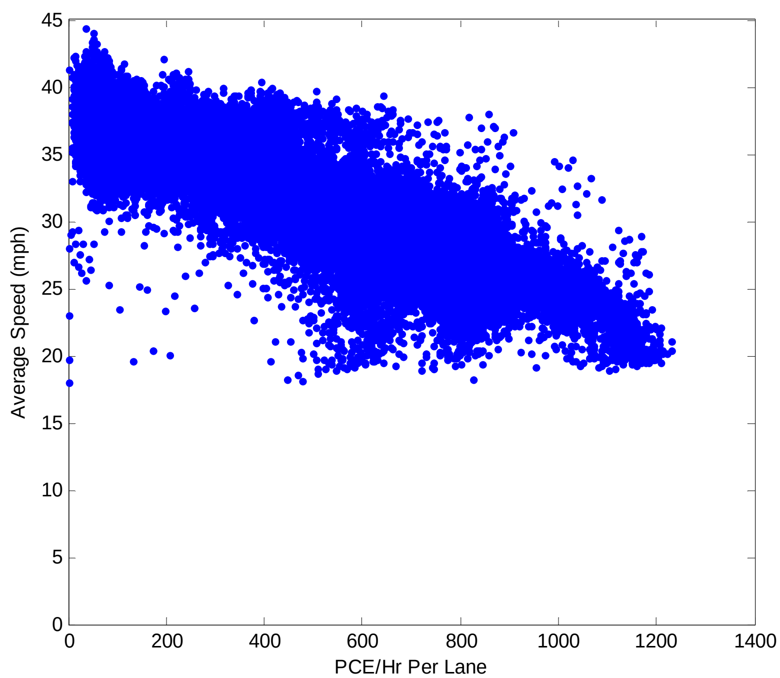

Figure D.19. Observed Speed v Flow - Residential Undivided Arterial, 3-Lane

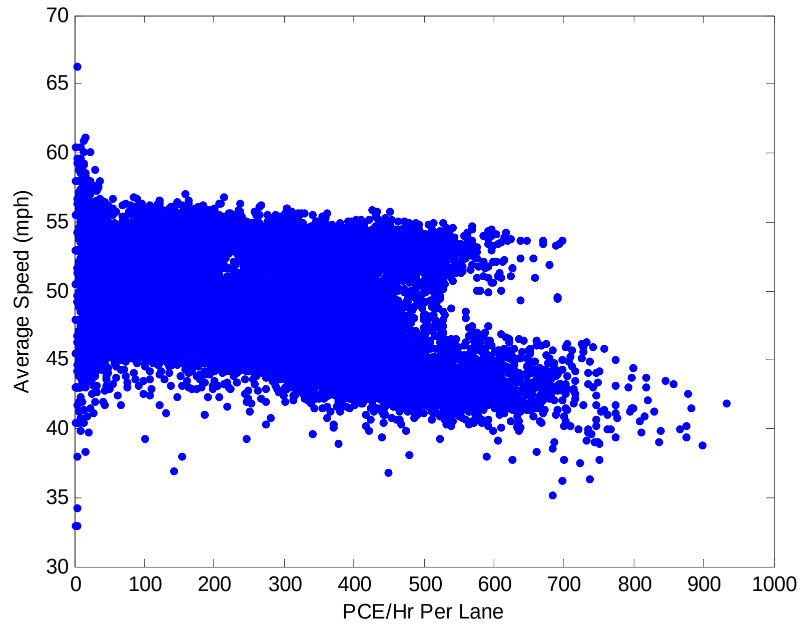

Figure D.20. Observed Speed v Flow - Residential Undivided Arterial, 2-Lane

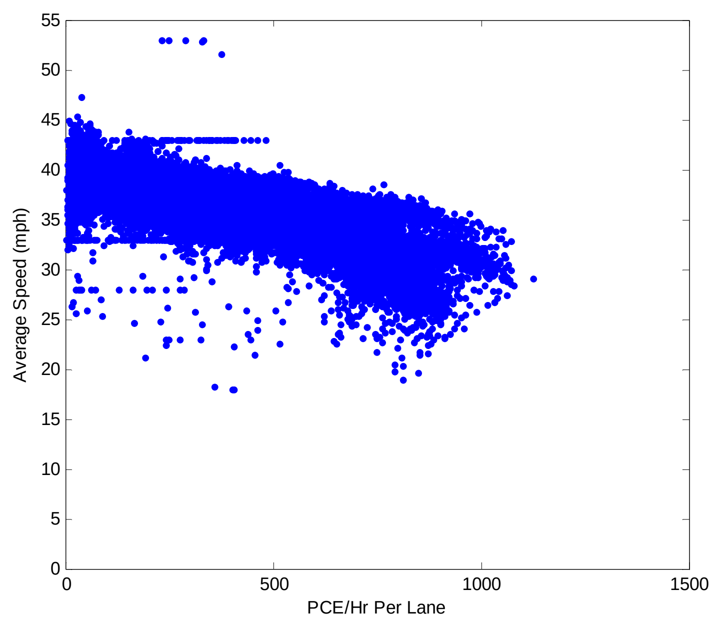

Figure D.21. Observed Speed v Flow - Residential Undivided Arterial, 4-Lane

APPENDIX E – Results from Implementation of Volume Delay Functions in FSUTMS

Table E.1. RMSE Percent for Estimated Volume vs. Observed Count (7:00-8:00 AM at 8 TTMS Sites, Freeways, Toll Roads and Signalized Divided Arterials)

Lane Flow, (vphpl)	Standard BPR VDF	Fitted BPR VDF	Modified Davidson VDF	Conical VDF	Akcelik VDF	TTMS Sites (Directional)
1-100						
100-200	98.30%	104.10%	95.60%	99.60%	96.00%	1
200-400	13.60%	19.20%	8.40%	23.30%	15.60%	4
400-600	26.20%	23.80%	23.40%	17.00%	25.80%	5
600-800	37.60%	28.10%	34.70%	34.30%	40.20%	2
800-1,000						
1,000-1,200	51.10%	46.70%	53.60%	50.50%	56.50%	1
1,200-1,400						
1,400-1,600	30.80%	18.10%	22.10%	20.90%	28.30%	2
1,600-1,800						
1,800-2,000	9.6%	11.7%	10.1%	23.6%	6.7%	1
2,000-2,200						
2,200-2,400						
1-2,400	37.00%	29.80%	33.30%	35.10%	37.30%	16

Table E.2. RMSE Percent for Estimated Volume vs. Observed Count (8:00-9:00 AM at 8 TTMS Sites, Freeways, Toll Roads and Signalized Divided Arterials)

Lane Flow, (vphpl)	Standard BPR VDF	Fitted BPR VDF	Modified Davidson VDF	Conical VDF	Akcelik VDF	TTMS Sites (Directional)
1-100						
100-200	113.40%	122.80%	112.30%%	114.90%	112.10%	1
200-400	30.80%	31.80%	29.80%	31.70%	29.40%	4
400-600	28.10%	27.20%	22.00%	19.30%	26.00%	5
600-800	40.70%	30.90%	34.40%	32.40%	34.40%	2
800-1,000						
1,000-1,200	34.30%	38.20%	37.80%	34.60%	32.60%	2
1,200-1,400						
1,400-1,600	33.00%	8.60%	4.40%	10.30%	30.10%	2
1,600-1,800						
1,800-2,000						
2,000-2,200						
2,200-2,400						
1-2,400	41.00%	31.50%	30.20%	29.30%	37.70%	16

Table E.3. RMSE Percent for Estimated Volume vs. Observed Count (4:00-5:00 PM at 8 TTMS

Sites, Freeways, Toll Roads and Signalized Divided Arterials)

Lane Flow, (vphpl)	Standard BPR VDF	Fitted BPR VDF	Modified Davidson VDF	Conical VDF	Akcelik VDF	TTMS Sites (Directional)
1-100						
100-200						
200-400	64.00%	65.80%	70.1%,	66.80%	59.20%	4
400-600	17.30%	25.50%	47.7%,	30.00%	25.60%	4
600-800	39.10%	36.00%	25.8%,	33.20%	38.50%	3
800-1,000	52.00%	31.00%	0.5%,	10.78	44.9	1
1,000-1,200	25.40%	25.70%	25.4%,	34.30%	31.50%	1
1,200-1,400	16.60%	0.70%	24.4%,	18.40%	12.80%	1
1,400-1,600	69.60%	28.00%	5.0%,	0.80%	72.60%	1
1,600-1,800	33.20%	0.70%	21.9%,	22.50%	35.60%	1
1,800-2,000	50.30%					
2,000-2,200						
2,200-2,400						
1-2,400	50.80%	29.90%	32.60%	30.90%	51.30%	16

Table E.4. RMSE Percent for Estimated Volume vs. Observed Count (5:00-6:00 PM at 8 TTMS Sites, Freeways, Toll Roads and Signalized Divided Arterials)

Lane Flow, (vphpl)	Standard BPR VDF	Fitted BPR VDF	Modified Davidson VDF	Conical VDF	Akcelik VDF	TTMS Sites (Directional)
1-100						
100-200						
200-400	30%	31%	32%	31%	30%	3
400-600	29%	15%	20%	24%	28%	5
600-800	35%	52%	43%	40%	36%	2
800-1,000	14%	14%	13%	12%	13%	1
1,000-1,200	27%	16%	1%	2%	24%	1
1,200-1,400	40%	46%	30%	41%	47%	1
1,400-1,600	10%	6%	12%	24%	1%	1
1,600-1,800	38%	0%	6%	16%	38%	1
1,800-2,000	25%	2%	17%	25%	25%	1
2,000-2,200						
2,200-2,400						
1-2,400	35%	25%	23%	30%	36%	16

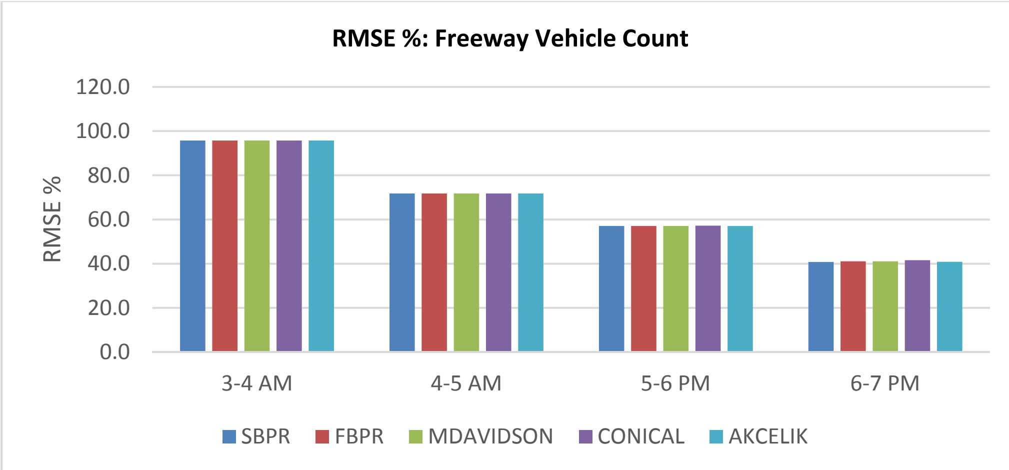

Figure E.1. RMSE Percent for Estimated Volumes

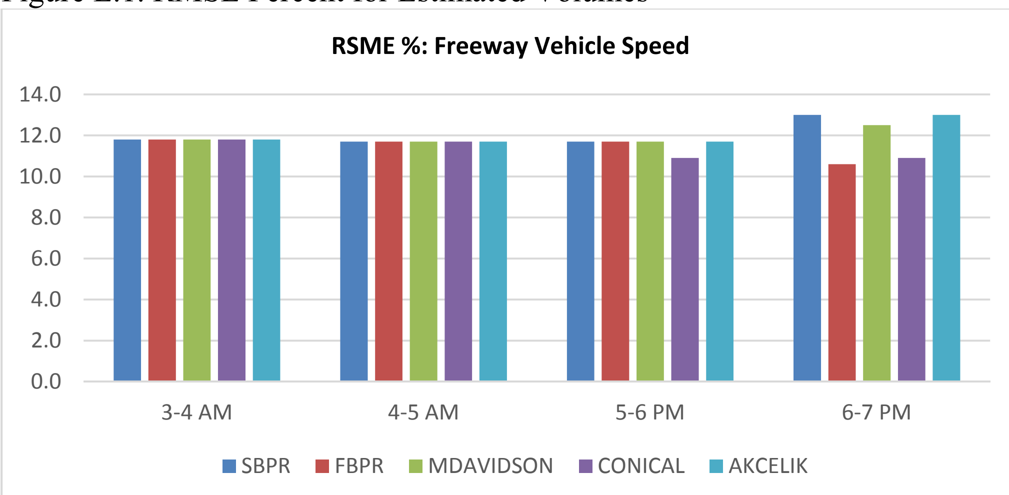

Figure E.2. RMSE for Estimated Speeds

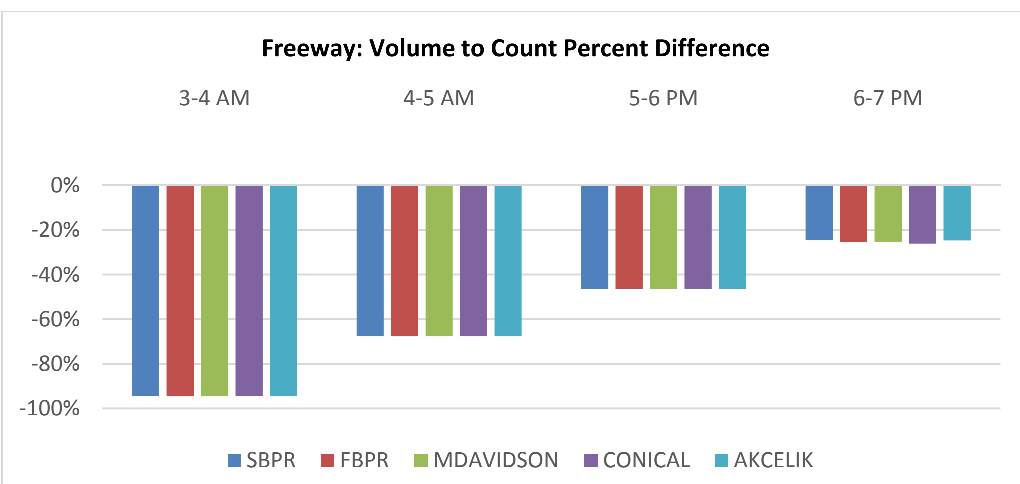

Figure E.3. Volume to Count Percent Difference by Time of Day

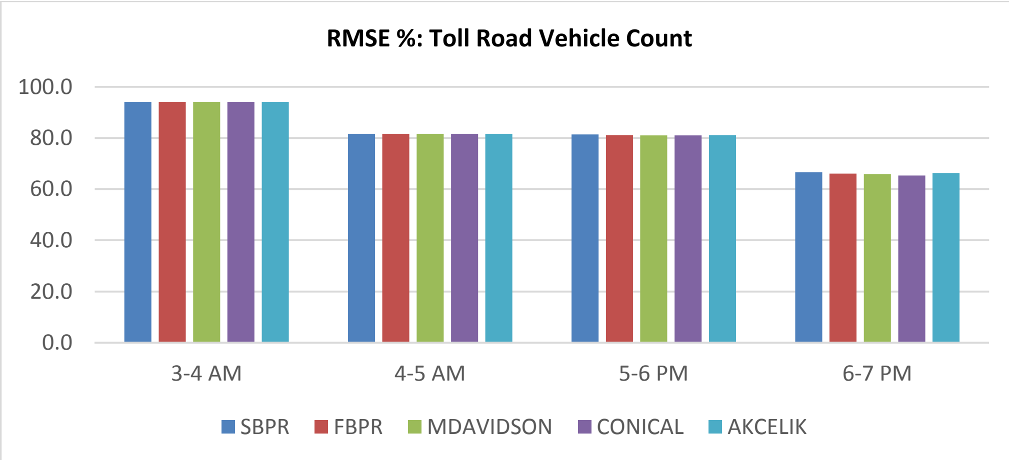

Figure E.4. RMSE Percent for Estimated Volumes

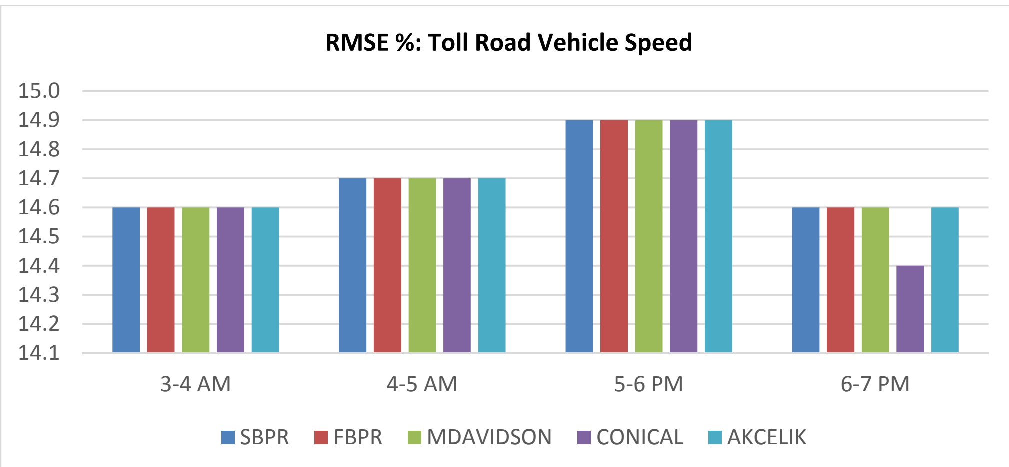

Figure E.5. RMSE for Estimated Speeds

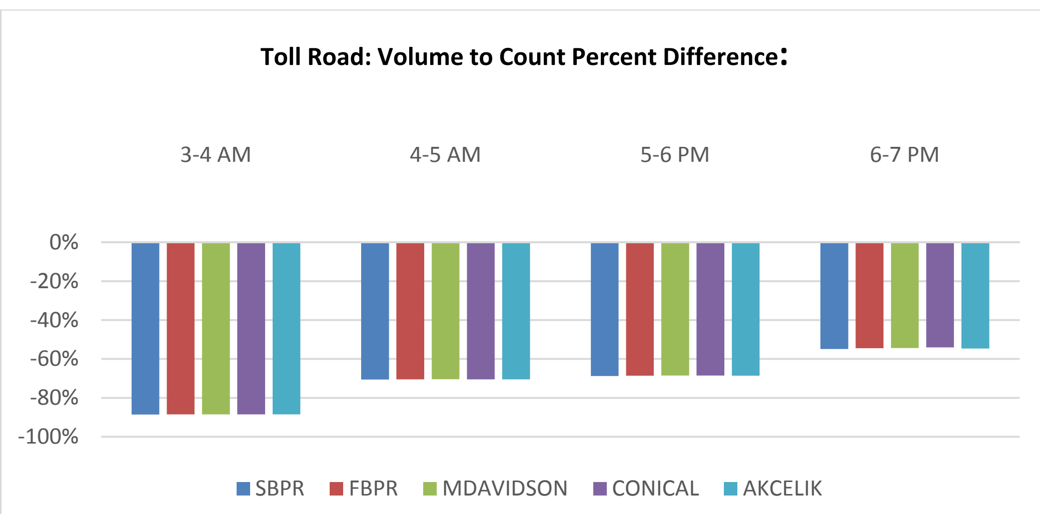

Figure E.6. Volume to Count Percent Difference by Time of Day

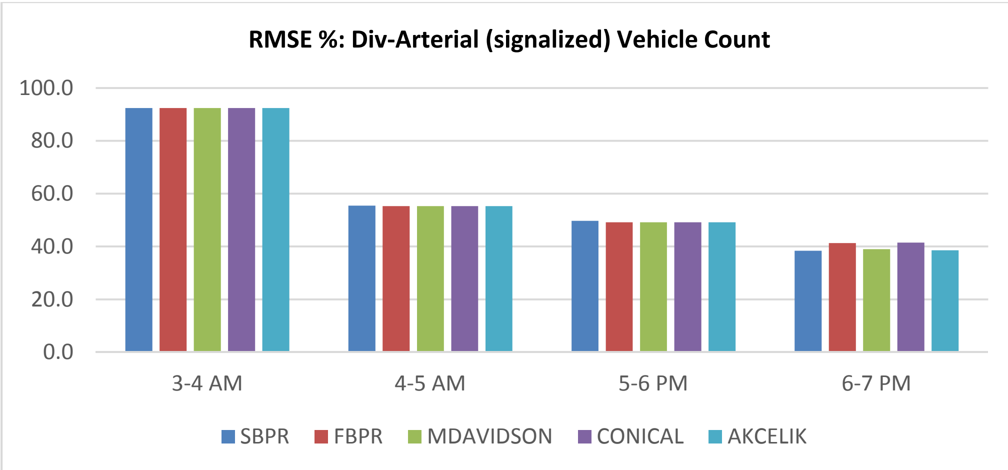

Figure E.7. RMSE Percent for Estimated Volumes

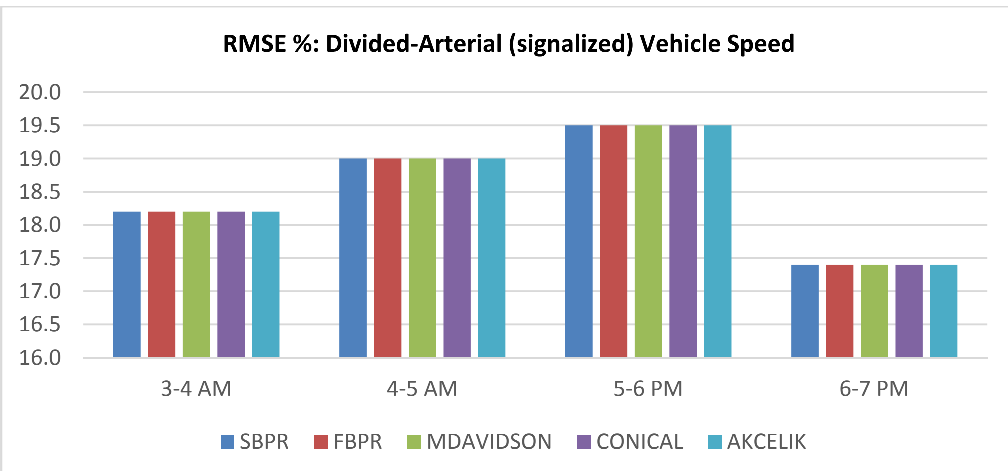

Figure E.8. RMSE for Estimated Speeds

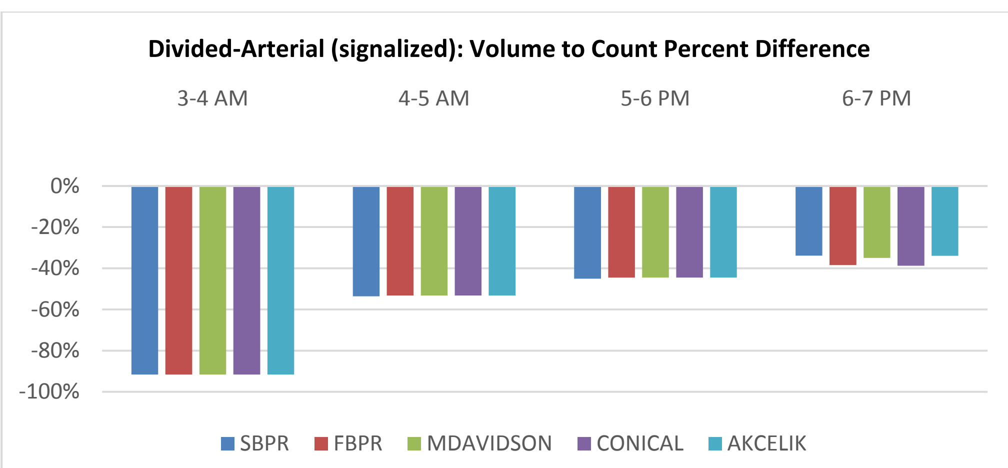

Figure E.9. Volume to Count Percent Difference by Time of Day

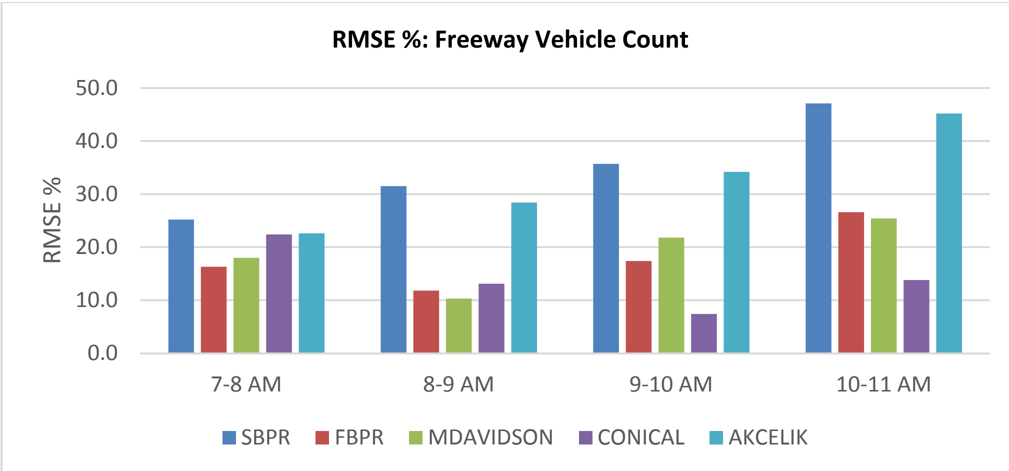

Figure E.10. RMSE Percent for Estimated Volumes

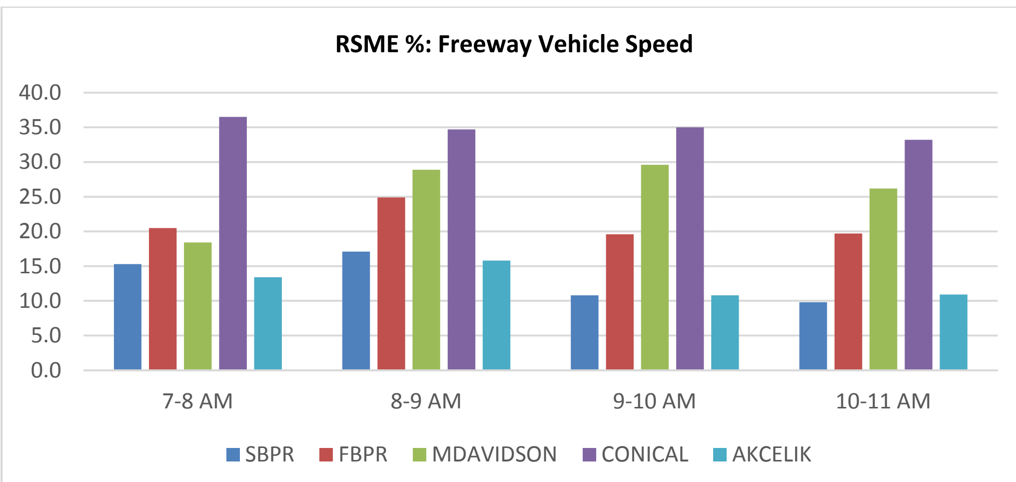

Figure E.11. RMSE for Estimated Speeds

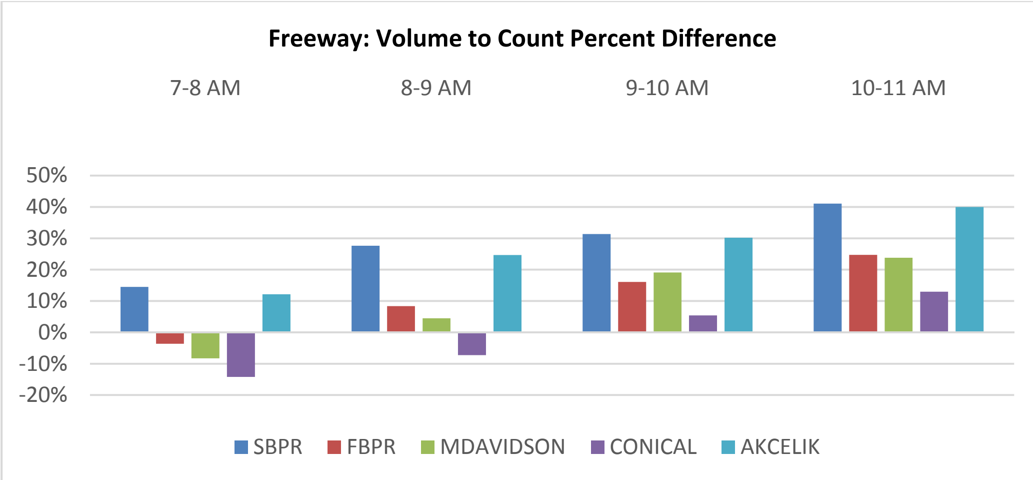

Figure E.12. Volume to Count Percent Difference by Time of Day

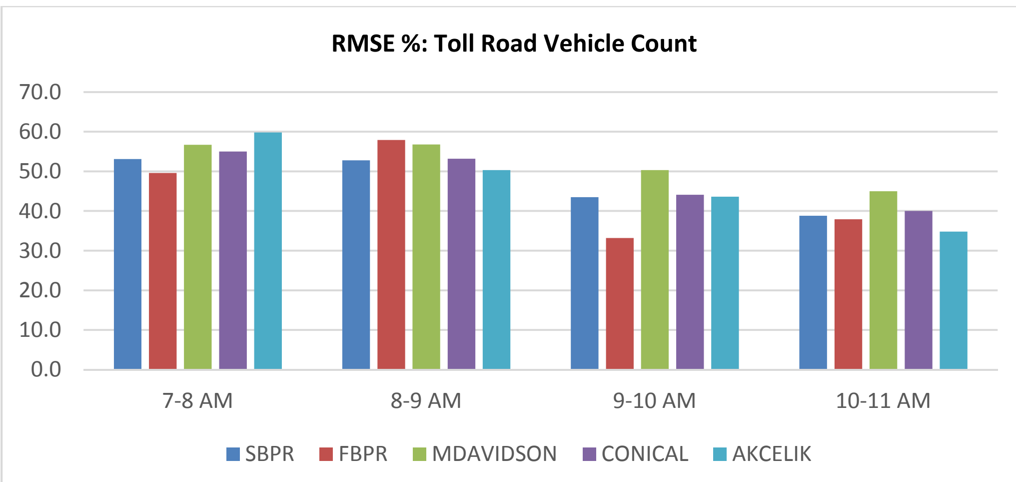

Figure E.12. RMSE Percent for Estimated Volumes

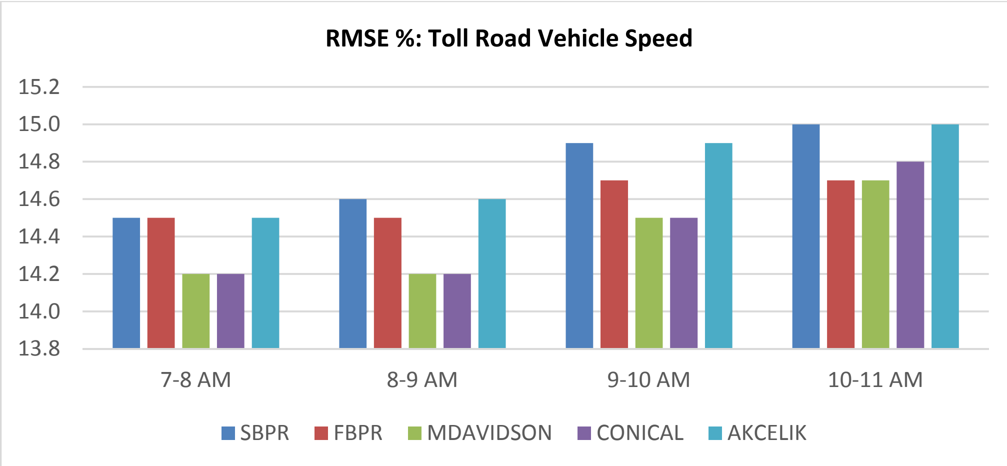

Figure E.13. RMSE for Estimated Speeds

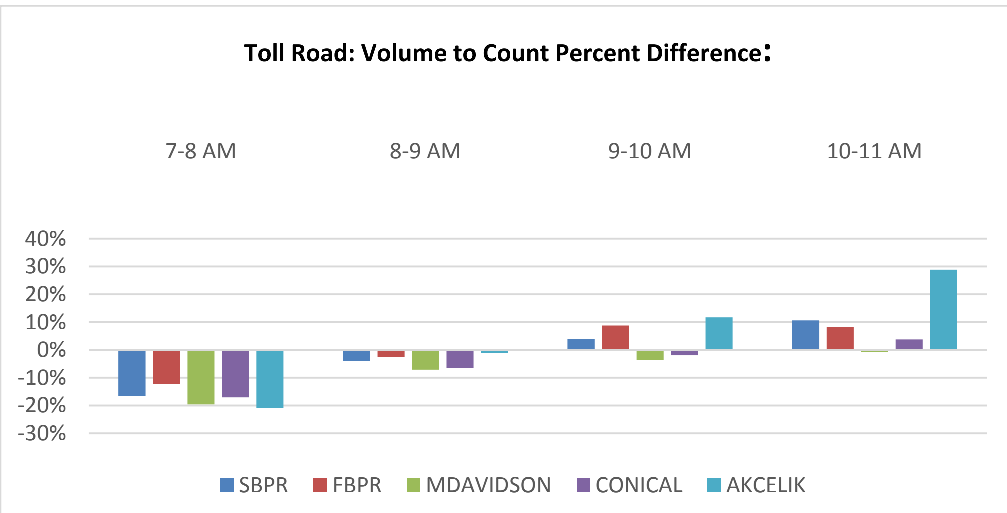

Figure E.14. Volume to Count Percent Difference by Time of Day

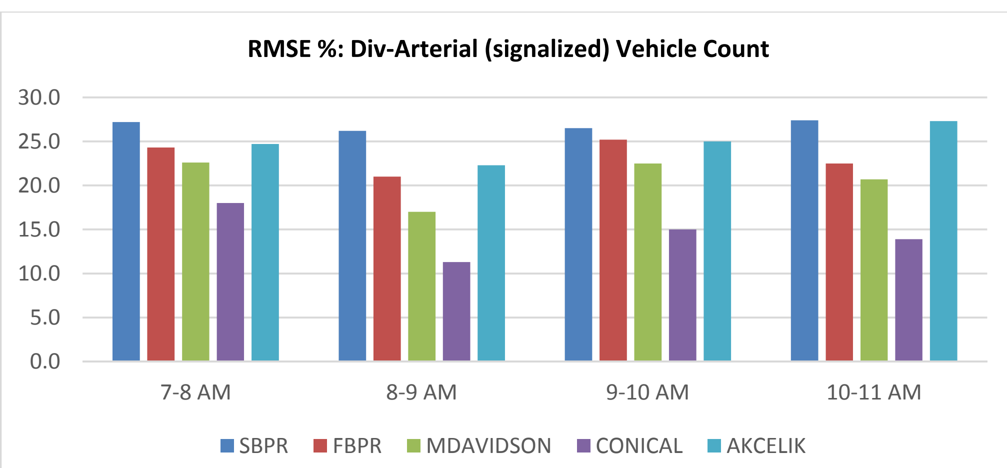

Figure E.15. RMSE Percent for Estimated Volumes

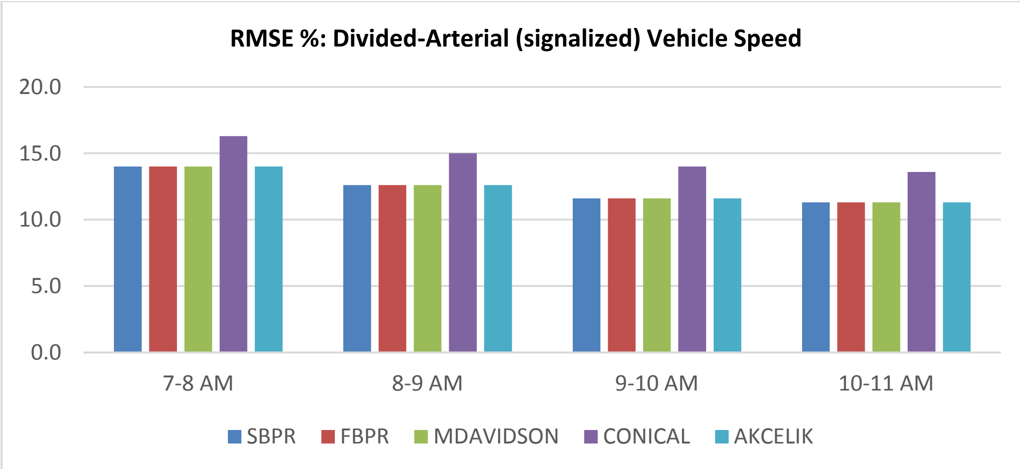

Figure E.16. RMSE for Estimated Speeds

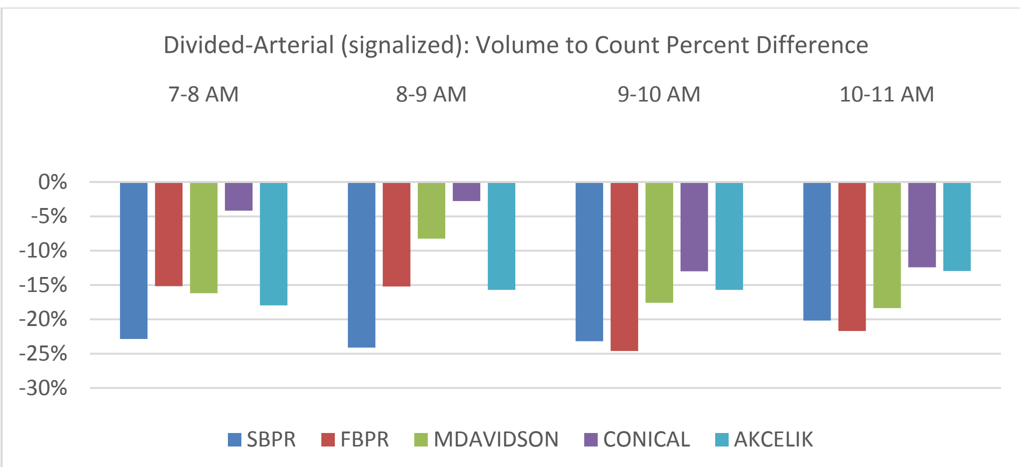

Figure E.17. Volume to Count Percent Difference by Time of Day

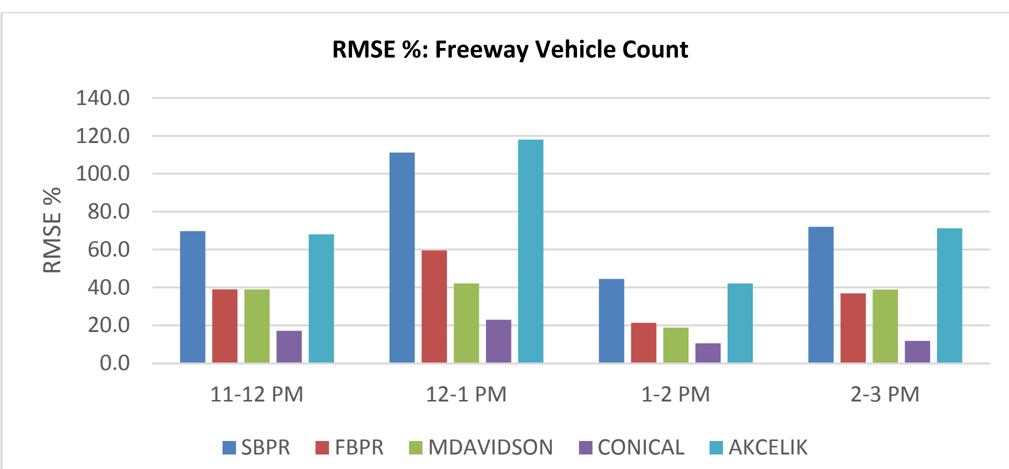

Figure E.18. RMSE Percent for Estimated Volumes

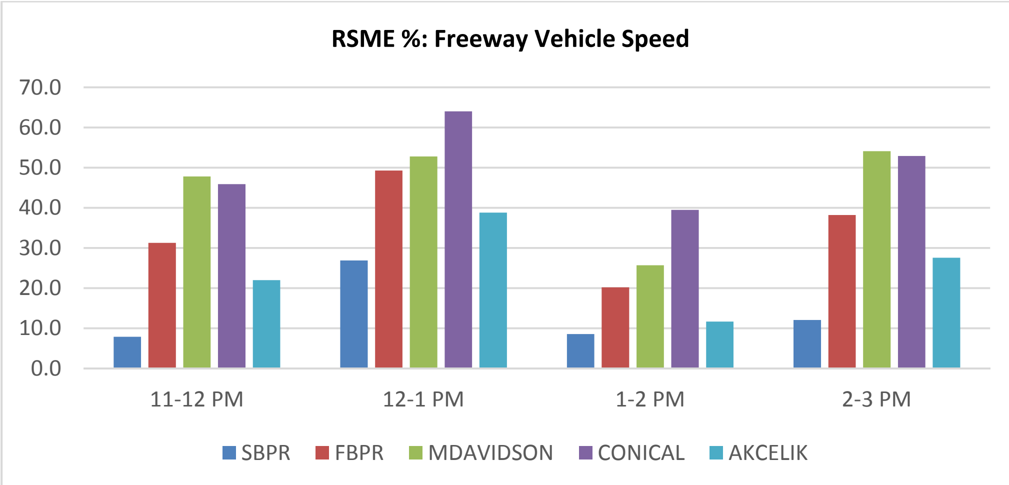

Figure E.19. RMSE for Estimated Speeds

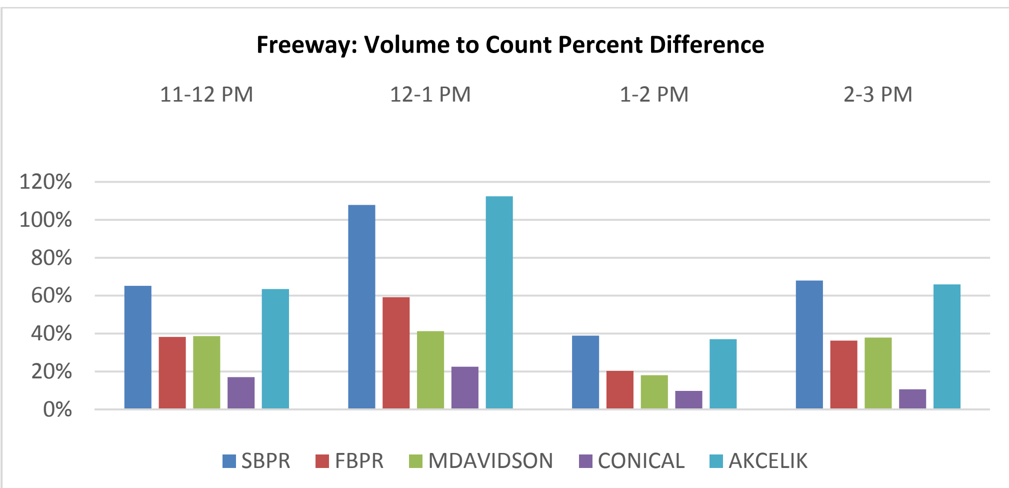

Figure E.20. Volume to Count Percent Difference by Time of Day

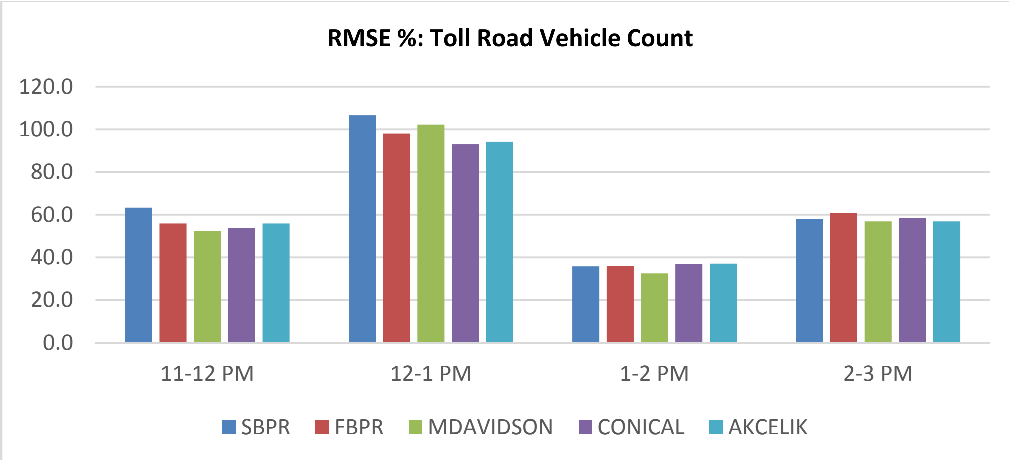

Figure E.21. RMSE Percent for Estimated Volumes

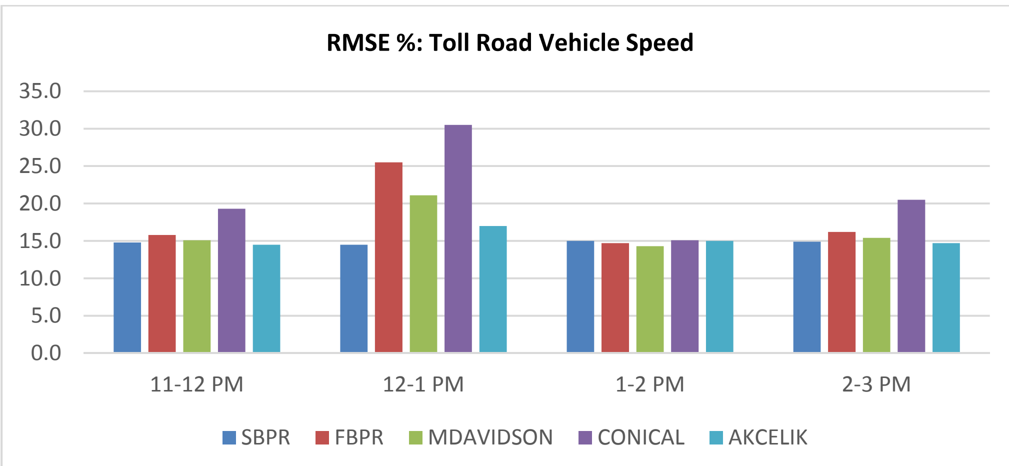

Figure E.22. RMSE for Estimated Speeds

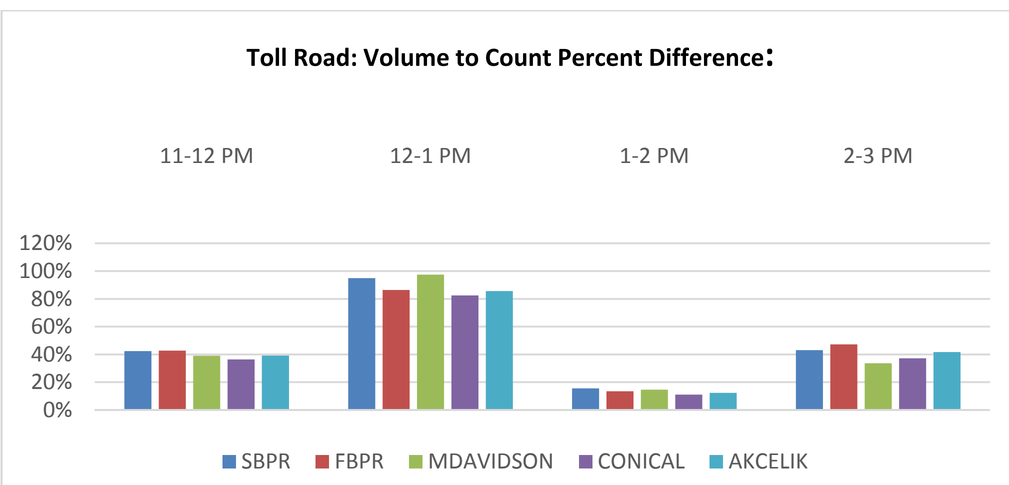

Figure E.22. Volume to Count Percent Difference by Time of Day

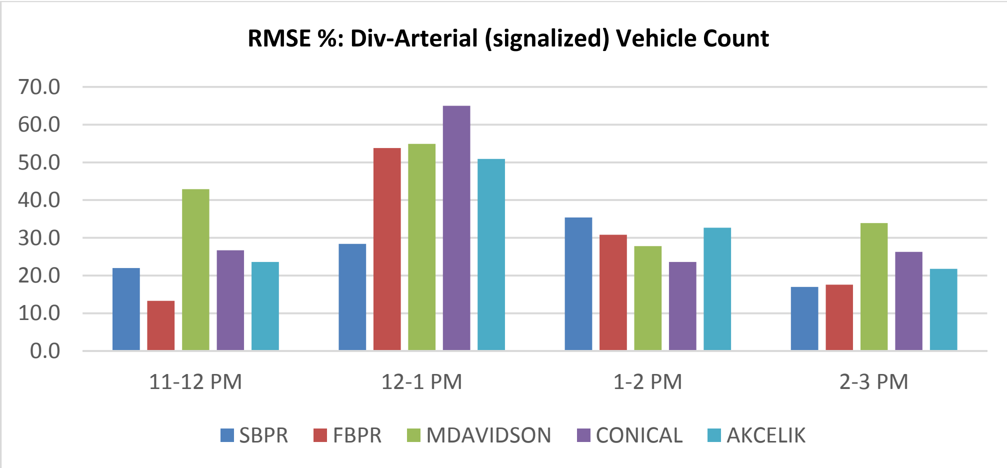

Figure E.23. RMSE Percent for Estimated Volumes

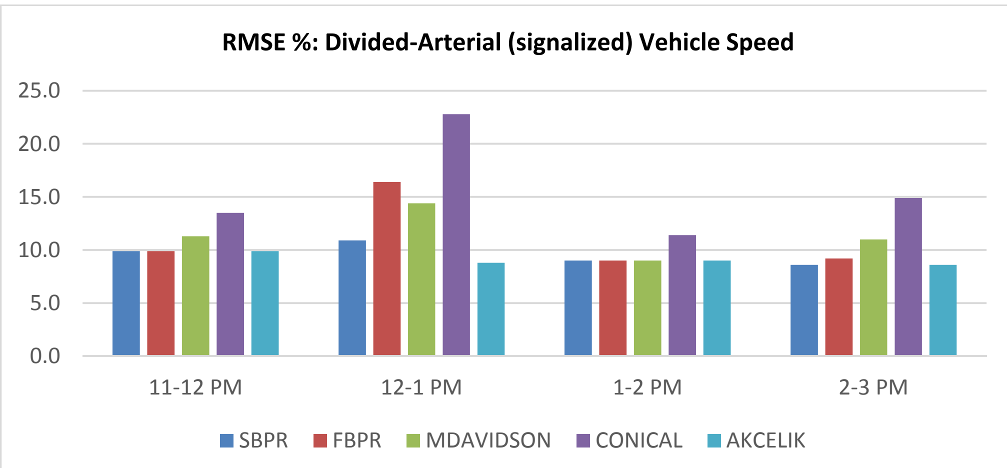

Figure E.24. RMSE for Estimated Speeds

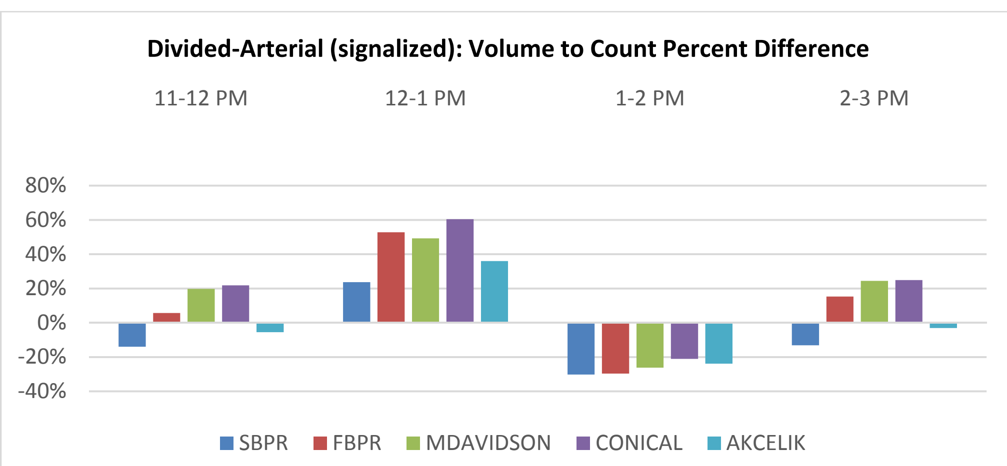

Figure E.25. Volume to Count Percent Difference by Time of Day

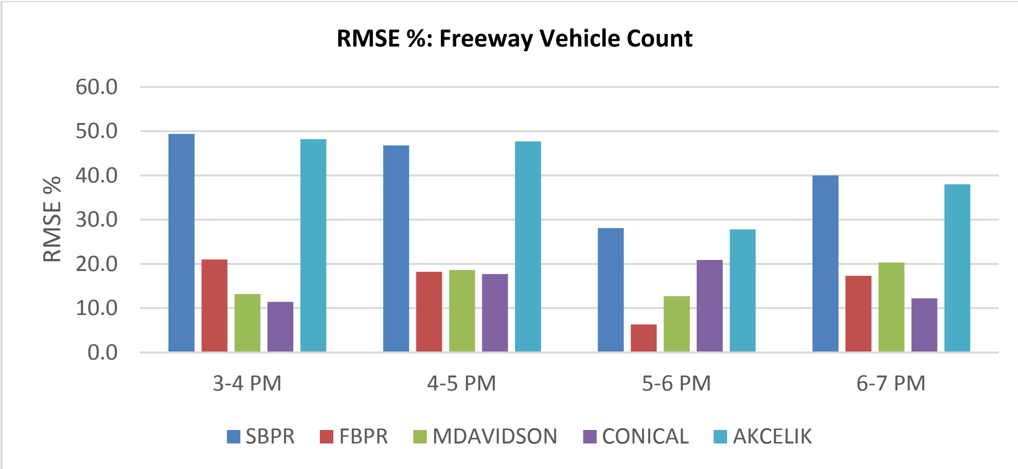

Figure E.26. RMSE Percent for Estimated Volumes

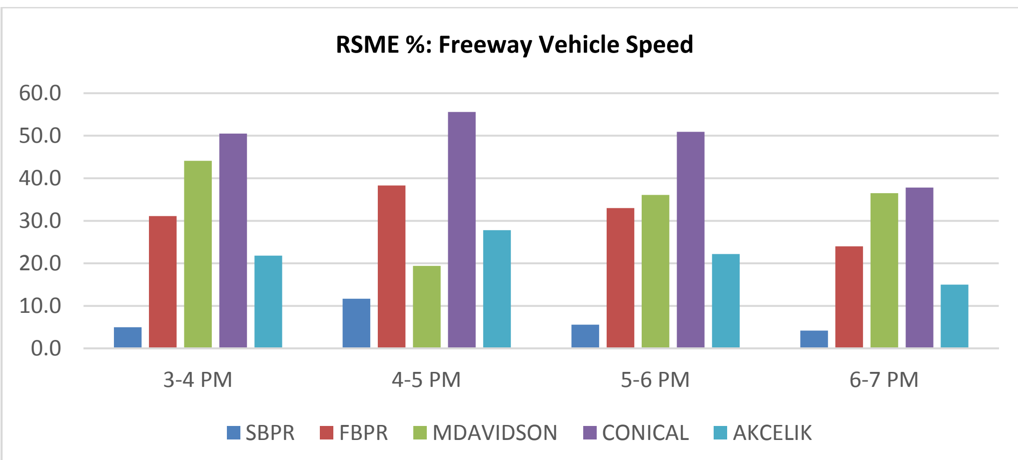

Figure E.27. RMSE for Estimated Speeds

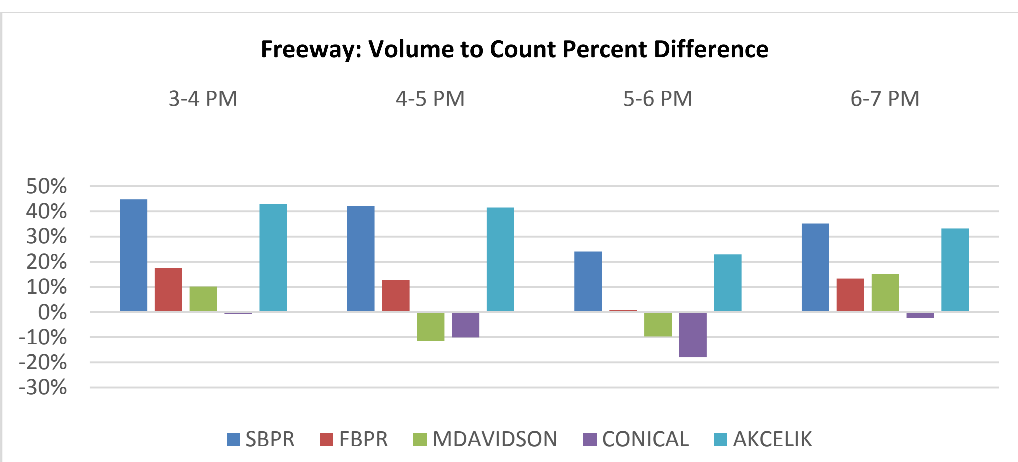

Figure E.28. Volume to Count Percent Difference by Time of Day

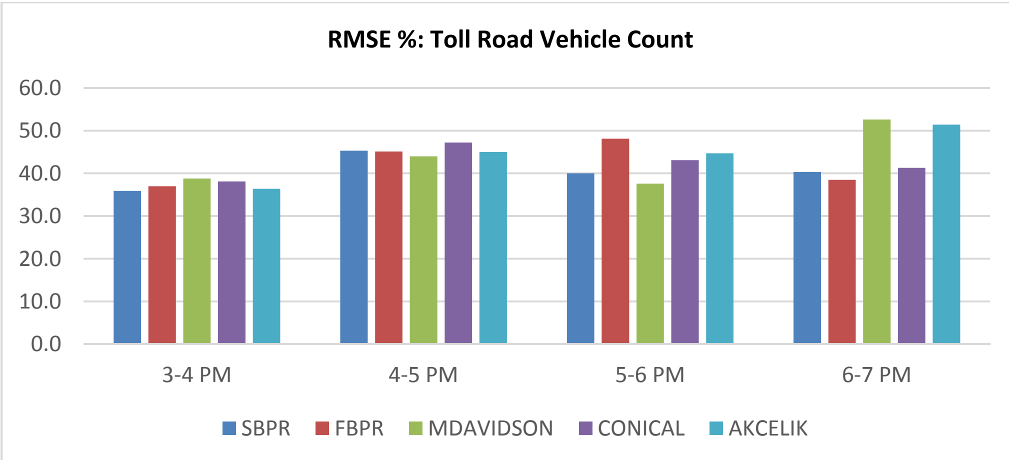

Figure E.29. RMSE Percent for Estimated Volumes

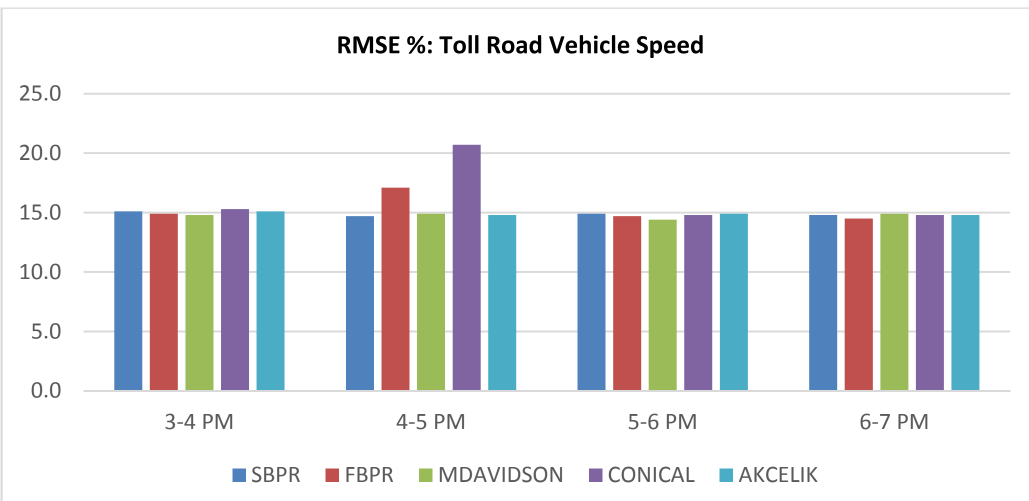

Figure E.30. RMSE for Estimated Speeds

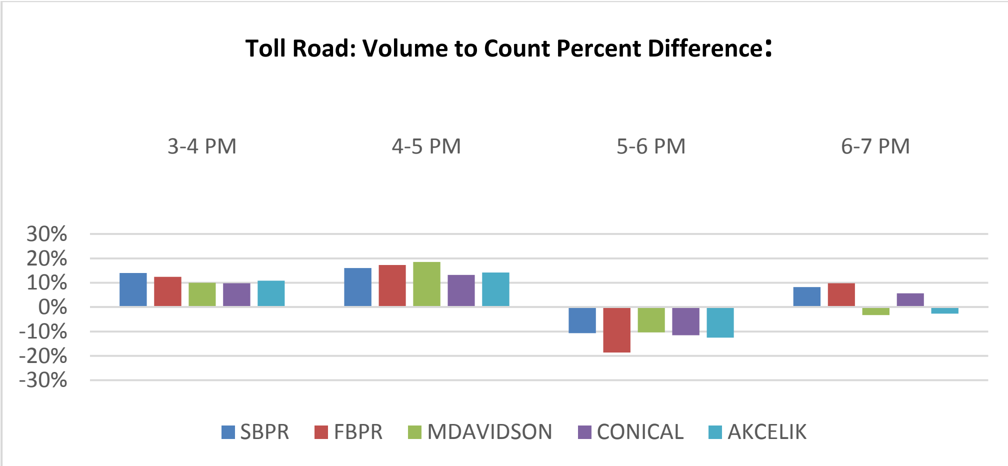

Figure E.31. Volume to Count Percent Difference by Time of Day

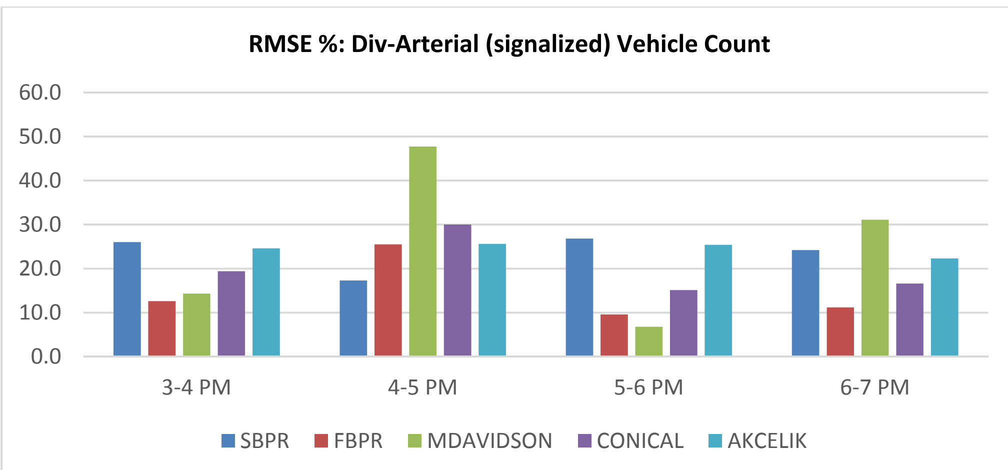

Figure E.32. RMSE Percent for Estimated Volumes

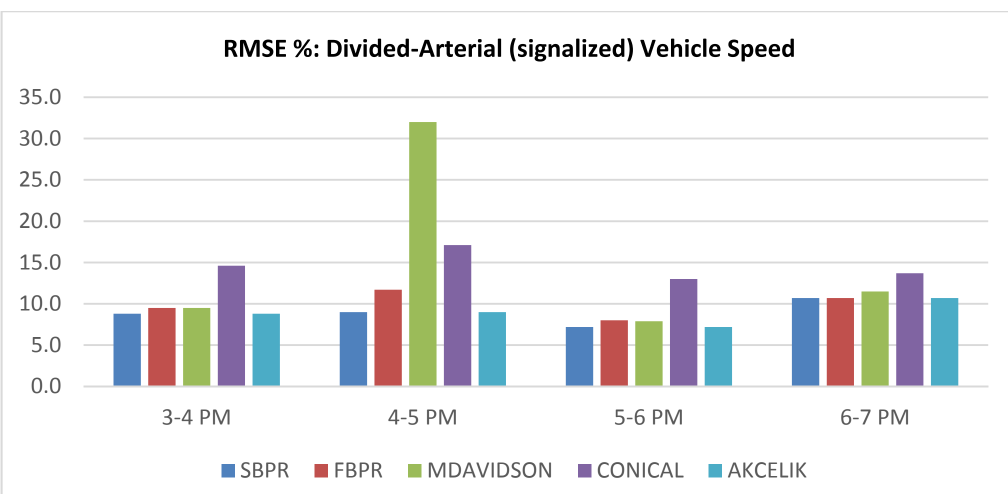

Figure E.33. RMSE for Estimated Speeds

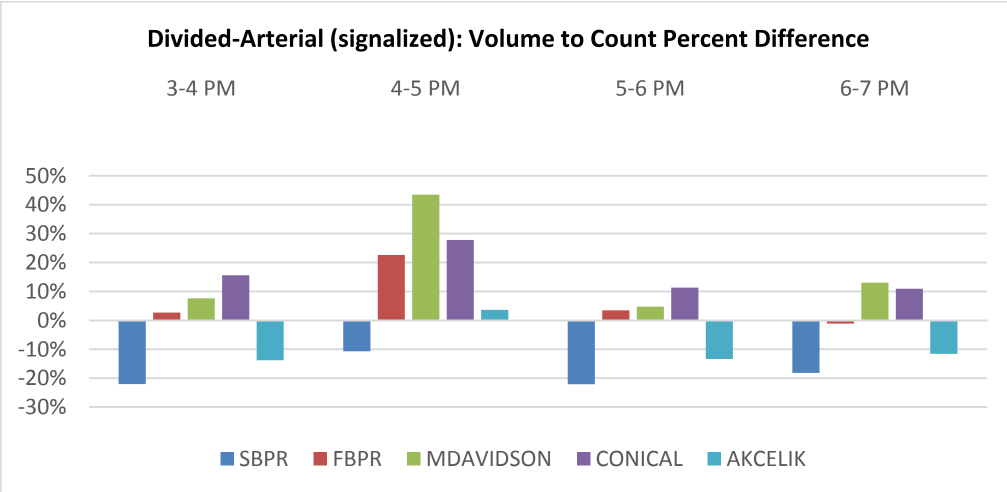

Figure E.34. Volume to Count Percent Difference by Time of Day

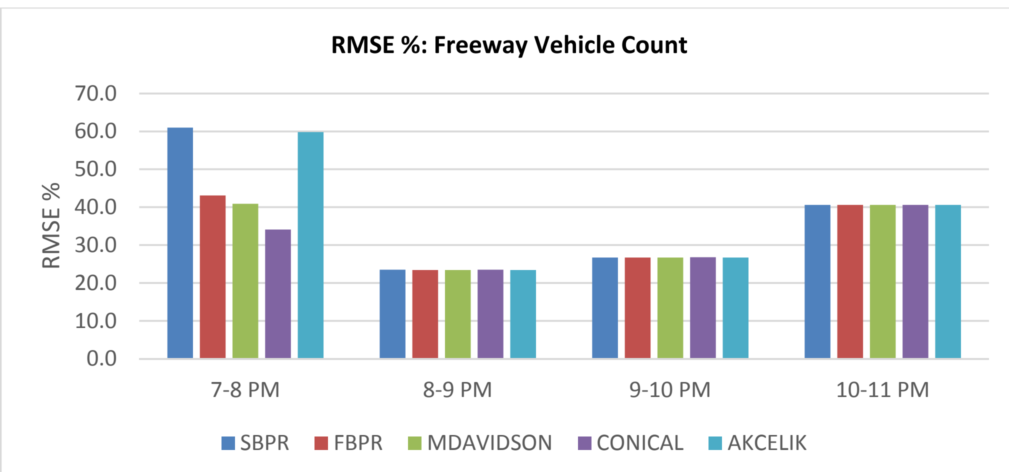

Figure E.35. RMSE Percent for Estimated Volumes

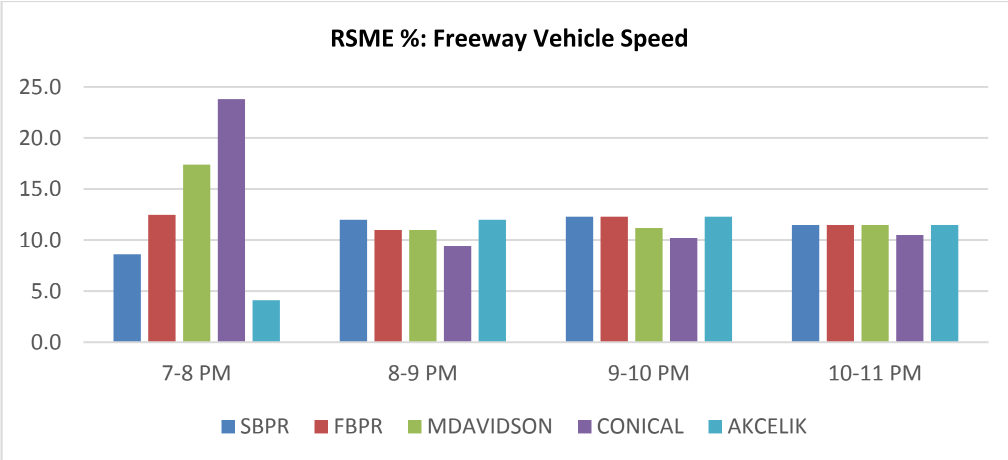

Figure E.36. RMSE for Estimated Speeds

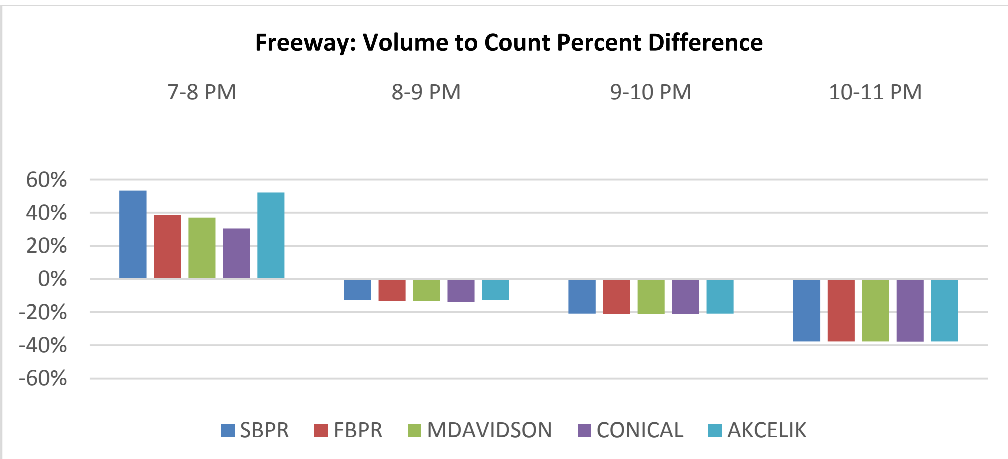

Figure E.37. Volume to Count Percent Difference by Time of Day

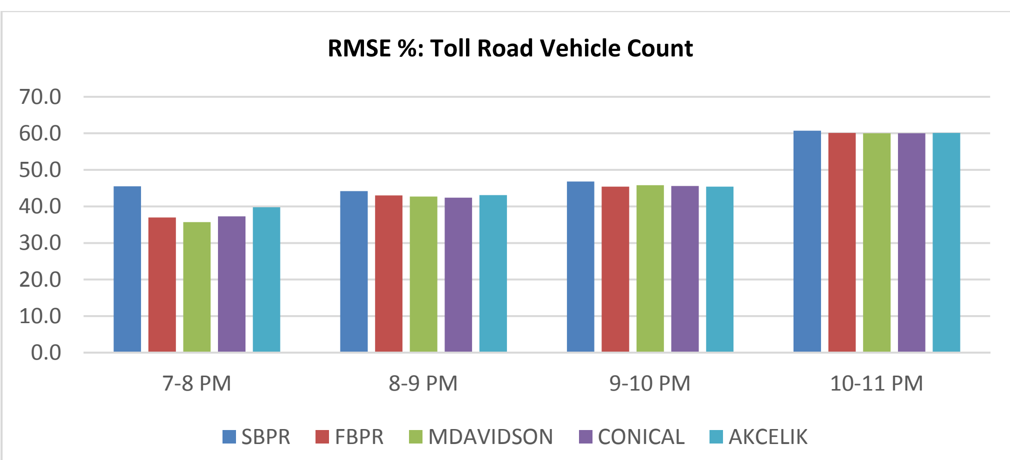

Figure E.38. RMSE Percent for Estimated Volumes

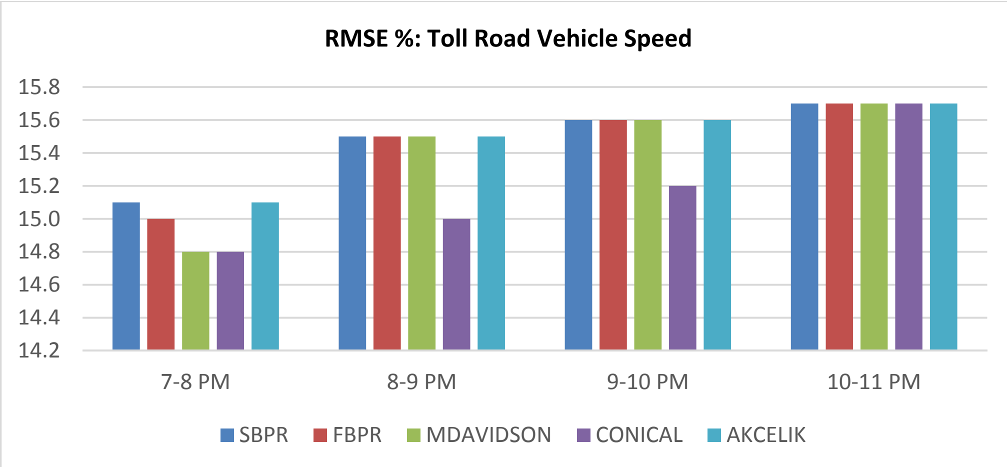

Figure E.39. RMSE for Estimated Speeds

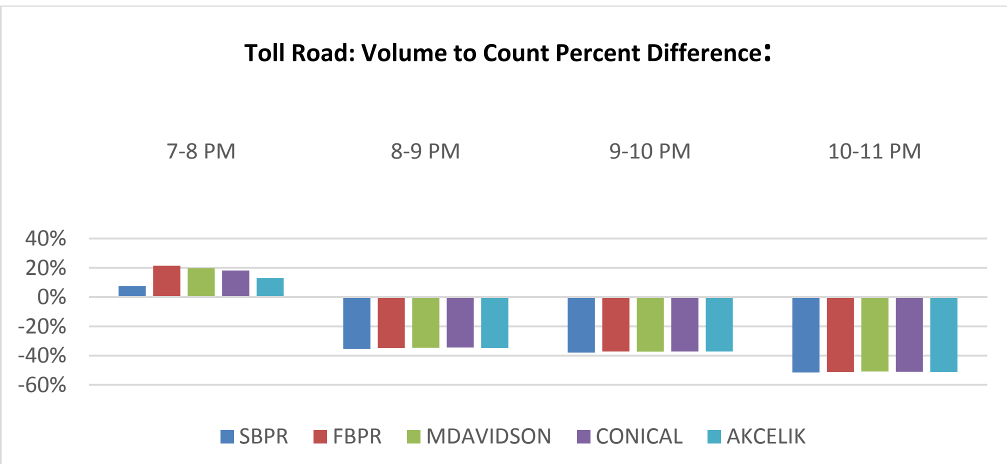

Figure E.40. Volume to Count Percent Difference by Time of Day

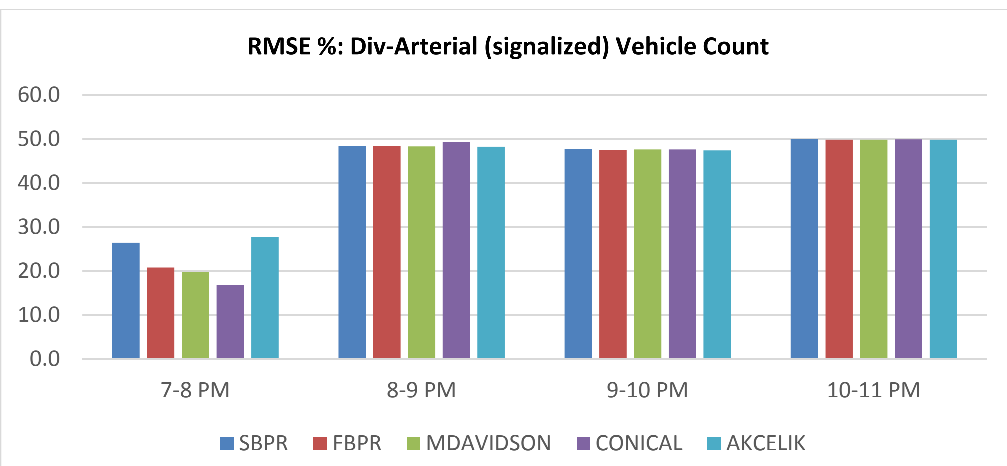

Figure E.41. RMSE Percent for Estimated Volumes

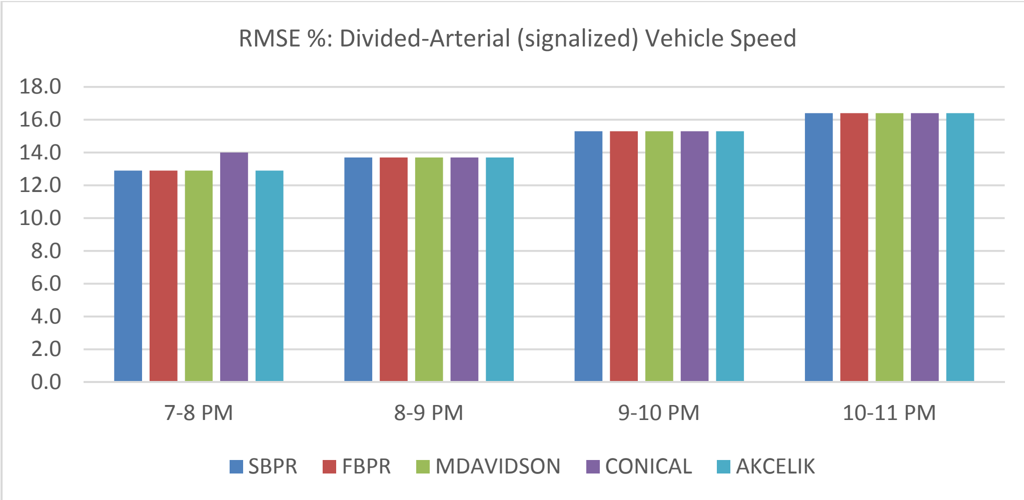

Figure E.42. RMSE for Estimated Speeds

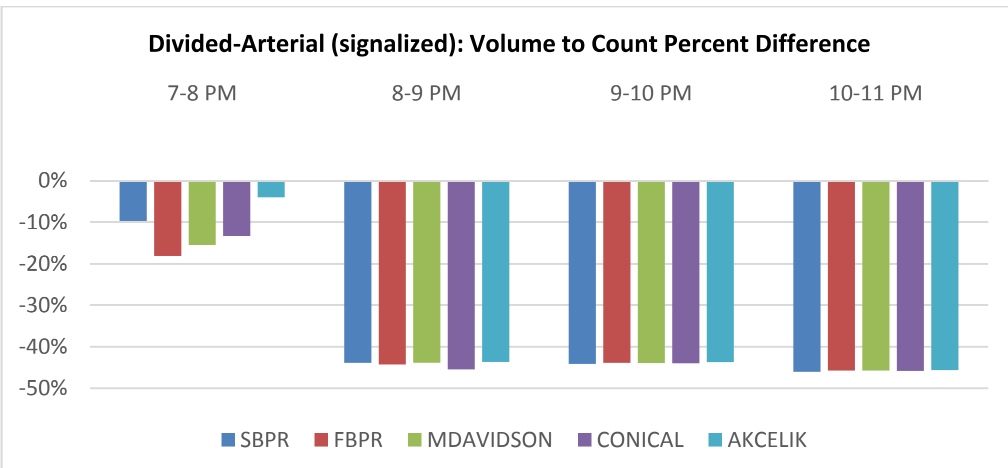

Figure E.43. Volume to Count Percent Difference by Time of Day

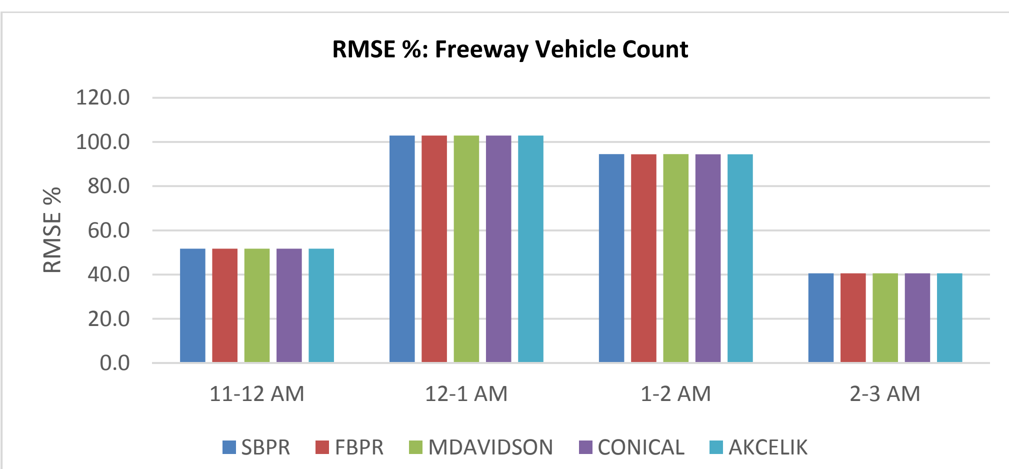

Figure E.44. RMSE Percent for Estimated Volumes

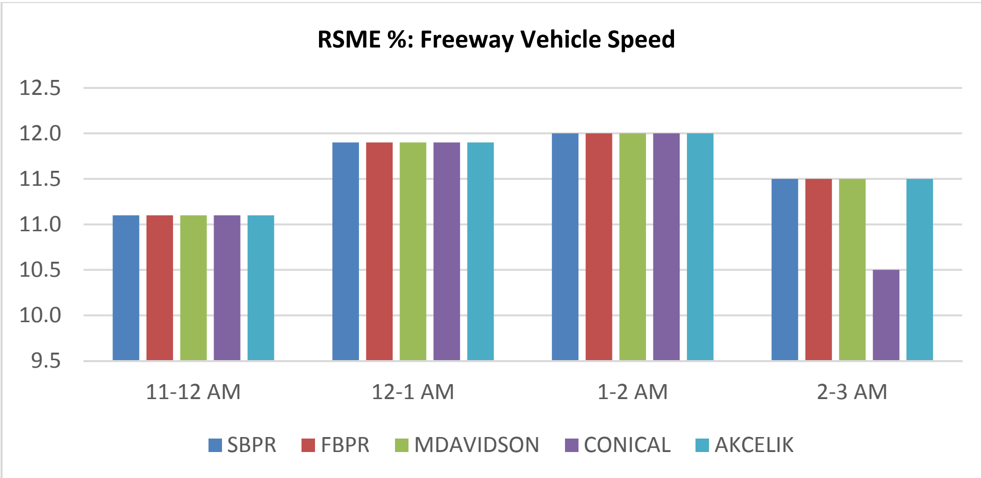

Figure E.45. RMSE for Estimated Speeds

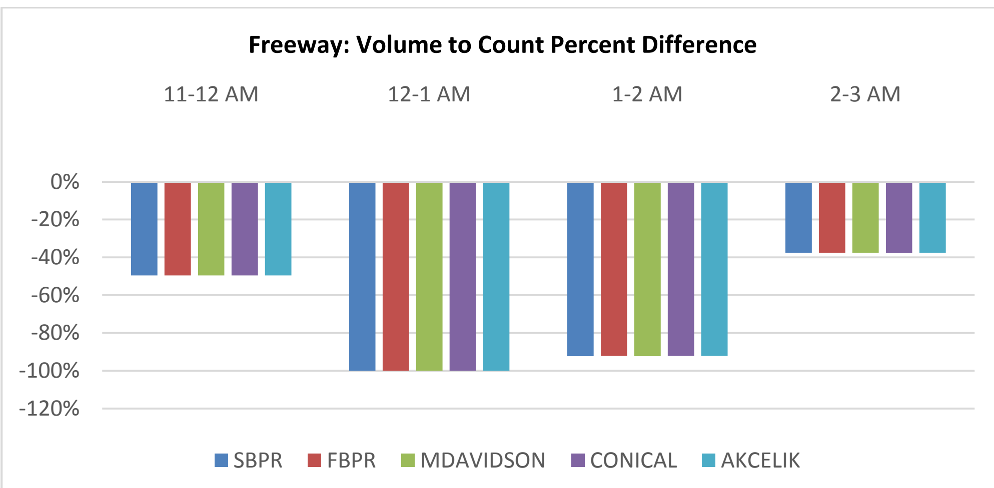

Figure E.46. Volume to Count Percent Difference by Time of Day

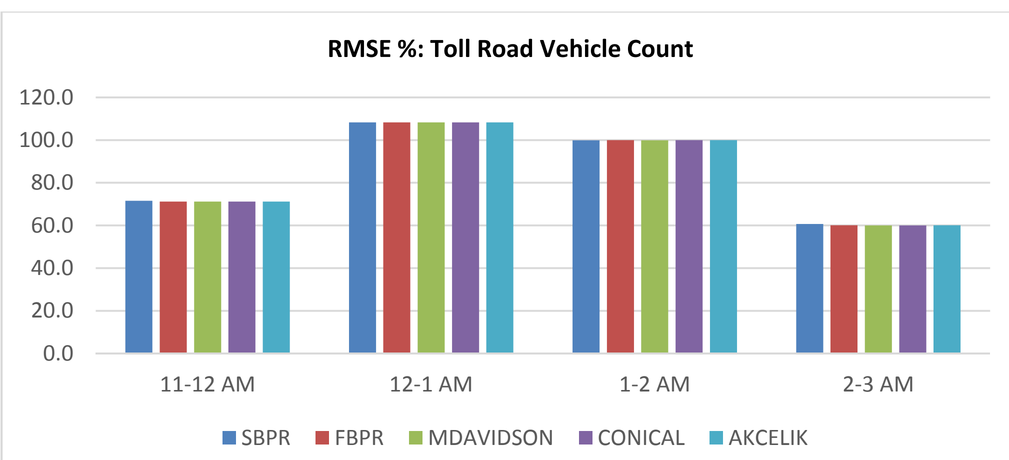

Figure E.47. RMSE Percent for Estimated Volumes

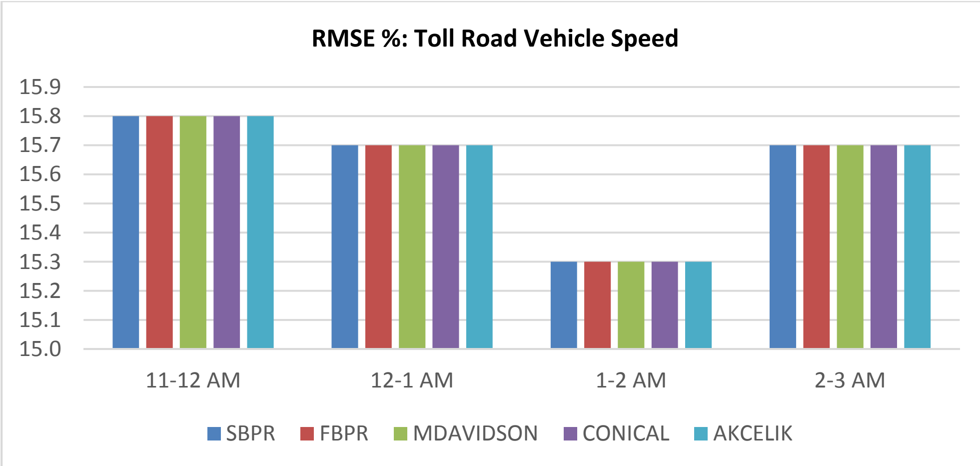

Figure E.48. RMSE for Estimated Speeds

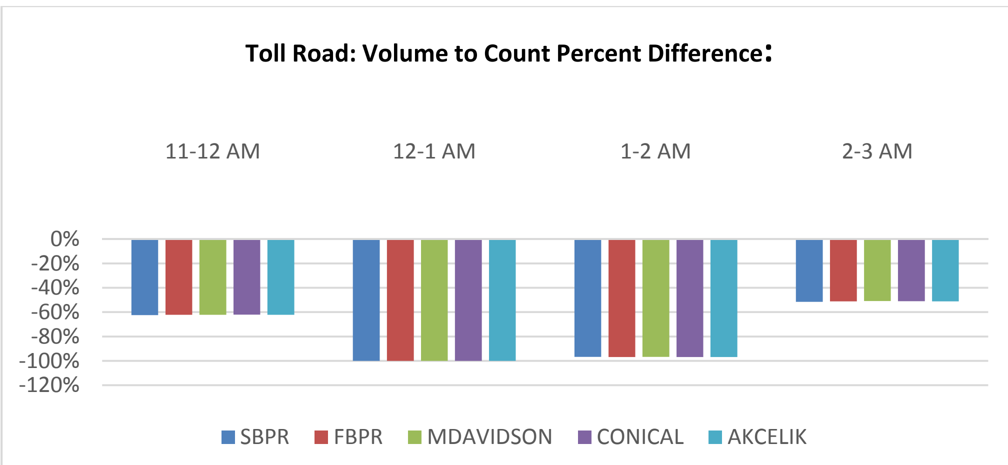

Figure E.49. Volume to Count Percent Difference by Time of Day

APPENDIX F – Implementation Guidelines

1. Overview

The purpose of this section of this report is to provide a quick and portable access to methodologies and updated FSUTMS inputs which were generated by this project. These guidelines cover the following issues:

(a) estimation of free flow speeds,
(b) estimation of practical capacities,
(c) estimation and fitting of volume delay functions,
(d) development of speed-capacity lookup tables, and
(g) development of VDF factors lookup tables.

2. Estimation of Free Flow Speeds

The estimation of free flow speeds was conducted separately for uninterrupted flow facilities and for interrupted flow facilities.

2.1 Estimation Procedure for Uninterrupted Flow Facilities

If field speed data is available, the free-flow speed can be defined as the 85th percentile speed of the speed observations recorded under low volume conditions (i.e., ≤ 200 passenger cars per hour per lane) and density conditions (≤ 5 passenger cars per mile per lane). The density criterion is important to be used because of the parabolic shape of speed-flow plots. In field data of speed observations is unavailable, segment free flow speed can be estimated using the following equation

$$FFS = 40.2118 + 0.3870 \times \text{SPL} + 1.6984 \times \text{ATYPE}$$

where ATYPE is a categorical variable with 3 three levels: ATYPE= 1 for urban area; ATYPE=2 for residential; and ATYPE = 3 for rural area. The SPL variable is the speed limit. This equation is suggested for use for freeways, access-controlled expressways, tolled freeways, HOV/HOT, and facilities whose operational characteristics are classified as uninterrupted or access controlled.

2.2 Estimation Procedure for Interrupted Flow Facilities

For interrupted flow facilities, collection of speed data for estimation of free flow speed requires targeting the speed of free flowing vehicles. In this project, the HCM 2010 definition of free flowing vehicle was used. The HCM 2010 defines a free flowing vehicle as a vehicle which has a headway of 8 seconds or more to the vehicle ahead and 5 seconds or more to the vehicle behind in the same traffic lane. The 85th percentile speed resulting from the cumulative frequency of the speeds of those vehicles is considered as midblock free flow speed, S_{mb}. To obtain segment free flow speed which takes into account delay at signalized intersections, the midblock 85th percentile speed is then adjusted as follows:

$$S_f = \frac{1}{\left[\frac{1}{S_{mb}} + \varphi(\frac{D}{3600})\right]}$$

where S_f is the free flow speed for an urban interrupted flow facility in miles per hour, φ is the signal density given by $\frac{N}{L}$, L is the length of the analysis segment in miles, N is the number of signalized intersections in the analysis segment, D is the average delay per signal in seconds equal to $DF \times 0.5 \times C \times (1 - g/C)^2$, DF is delay adjustment factor equal to $(1 - P)/(1 - \frac{g}{C})$, P is the proportion of vehicles arriving in green, g is the effective green time in seconds, and C is the cycle length in seconds.

3. Estimation of Practical Capacities

The estimation of practical capacities was conducted separately for uninterrupted flow facilities and for interrupted flow facilities.

3.1 Estimation Procedure for Uninterrupted Flow Facilities

The determination of practical capacities from field data collected from uninterrupted flow facilities requires initial examination of the fundamental traffic flow diagrams in order to identify segment traffic behavior. All segments which comply with theoretical and practical traffic characteristics are picked as representative segments. These segments are then grouped by area type and facility type. We recommend using the 99^{th} percentile of observed volume as the estimate of capacity rather than the observed maximum volume. This is because the maximum values may not be repeatable as they could have occurred by chance or caused by incidents occurring at the time of data collection. If field data are not available, the analyst can use the recommended procedures in the 2010 HCM or use default values from the FDOT 2013 Q/LOS Handbook.

3.2 Estimation Procedure for Interrupted Flow Facilities

Similar to the procedure recommended for uninterrupted flow facilities, the 99^{th} percentile observed volume is recommended as practical capacity of an interrupted flow segment. However, if field data are not available, the analyst can use the HCM 2010 procedure which, however, requires signal timing data for intersections within the analysis corridor. The default values from the FDOT 2013 Q/LOS Handbook can also be used in this case.

4. Development of Speed-Capacity and VDF Factors Lookup Tables

Based on the results from field data analysis and modeling, speed-capacity and VDF factors lookup tables were developed. Table F1, categorization by facility and area type, shows combinations that were analyzed (checked in green) and that were not analyzed (checked in red). Due to limitation in data availability, some facility types were not analyzed as shown. These are mainly one-way streets and interrupted flow facilities located in rural areas. For in uninterrupted flow facilities, the research did not analyze all combinations of facility type-area type by speed limit and by number of lanes. The speed and capacity values from segments of which TTMS data was available were used to project values for segments without TTMS data but which had similar operational characteristics. For the segments in which this methodology couldn't work because there were no similar segments with TTMS data, default values from the FDOT 2013 Q/LOS Handbook or FSUTMS Standards.

Table F1. Area and Facility Type Analyzed

Facility Type	Area Type	Description	Changes
Urban			
1X	1X and 2X	Freeway CBD Fringe	☑
2X	2X	Divided Arterial in Fringe	☑
3X	4X	Undivided Arterial in OBD	☑
4X	4X	Divided Local Collector in OBD	☒
6X	1X	One Way Urban Street	☒
8X	4X	Urban Freeway with HOV lane	☑
9X	4X	Toll Road in OBD	☑
Residential			
1X	3X	Freeway in Residential Area	☑
2X	3X	Divided Arterial in Residential Area	☑
3X	3X	Undivided Arterial in Residential Area	☑
4X	3X	Collector in Residential Area	☑
6X	3X	One Way Street in Residential Area	☒
8X	3X	HOV lane in Residential Area	☑
9X	3X	Toll in Residential Area	☑
Rural			
1X	5X	Freeway in Rural Area (Developed)	☒
2X	5X	Divided Arterial in Rural Area	☒
3X	5X	Undivided Arterial in Rural Area	☒
4X	5X	Divided Local Collector in Rural Area	☒
6X	5X	One Way Facility in Rural Area	☒
8X	5X	HOV in Rural Area	NA
9X	5X	Toll Road in Rural	☑

5. Template for Applying the New VDFs in a Regional Model

A template in Cube Voyager that can be used to implement the new VDFs in a regional or statewide model has been developed. This template is intended to run for the EC2010 scenario only and not the "base year" scenario since the base year highway network uses the old facility type classification for interrupted flow facilities. This template does not include the speed limit data for the classification of interrupted facilities in the base year. The template consists of two modules, HIGHWAY NETWORK and HIGHWAY ASSIGNMENT as shown in Figure F1.

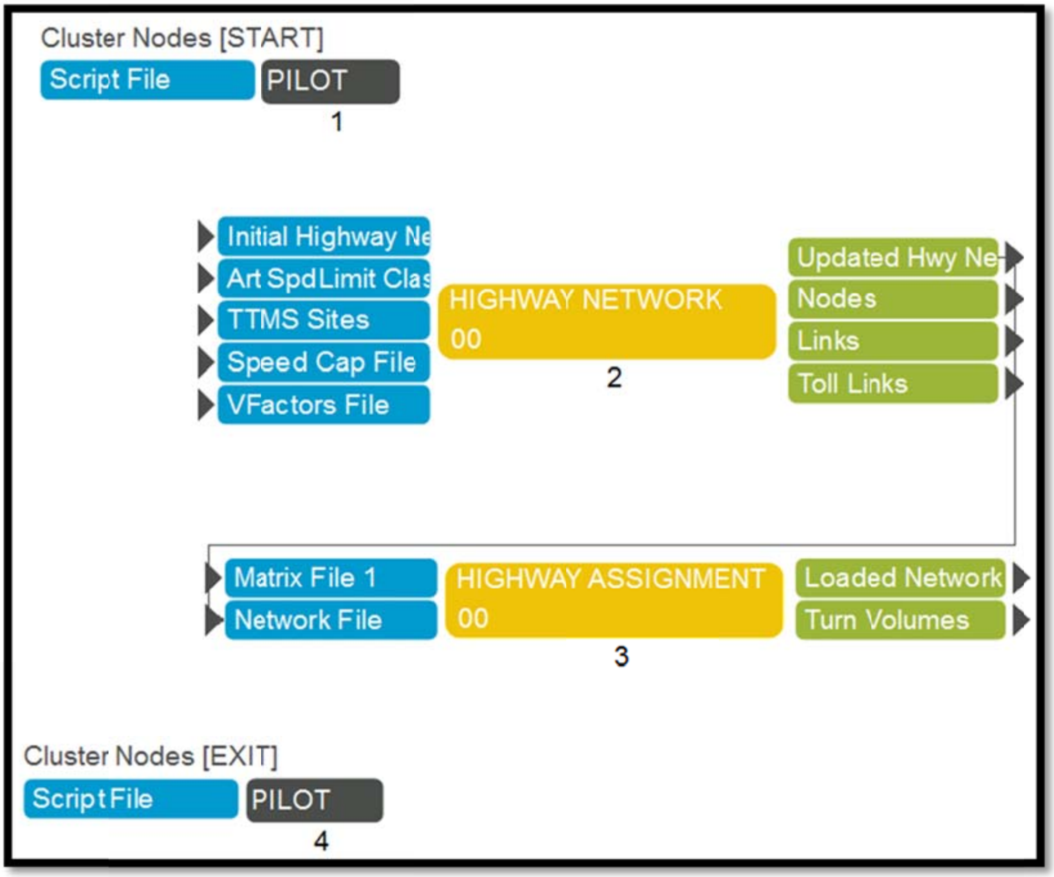

Figure F1. Modules in the Template Application

The HIGHWAY NETWORK module shows how the new arterial facility type code and the new VDF parameters can be added to an existing network. The user should replace the input SPDCAP.DBF and VFACTORS.DBF files with their own verified values or with those developed in this study. If the user decides to use the tables developed by this research, values for the facilities which were not covered should be adopted from the FDOT 2013 Q/LOS Handbook or FSUTMS Standards. The parameter files used in the template are for demonstration purposes only and should not be used for running the model scenarios.