COMPARISON OF FREEWAY CAPACITY ESTIMATION METHODS

By

VIPUL MODI

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To my parents

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LIST OF ABBREVIATIONS

HCM	Highway Capacity Manual
FDOT	Florida Department of Transportation
VAM	Van Aerde Model
PLM	Product Limit Method
STEWARD	Statewide Transportation Engineering Warehouse for Archived Regional Data
U.S.	United States
CDP	Capacity Data Processor
DBI	Downstream Breakdown Identifier

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Vipul Modi

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Chair: Scott S Washburn
Cochair: Yafeng Yin
Major: Civil Engineering

The capacity of a freeway segment is a critical factor for the assessment of the traffic flow operations on freeway facilities. The Highway Capacity Manual HCM (2000) is considered to be one of the authoritative sources on capacity values for a variety of roadway types in the U.S. It provides a single set of capacity values for basic freeway segments as a function of free-flow speed. These values are considered to be reasonably representative values for freeways located throughout the U.S., but it is recognized that lower or higher values may be more appropriate in any given location. However, the HCM does not provide any guidance on how its recommended values can be adjusted to reflect significant differences in capacity due to local conditions, nor how to directly measure or estimate capacity values. The objective of this research was to investigate various methods that can be used to arrive at an estimate of freeway capacity values, and to recommend one of these methods to the FDOT for use in developing their own estimates of capacity for Florida freeways. Three methods were investigated: one that fits a mathematical function to plotted speed-flow data points, from which the apex of the function is taken as capacity; one that estimates a breakdown probability distribution based on flow rates preceding breakdown events, from which capacity can be taken to correspond to a certain percentile value of the breakdown probability distribution; and one that uses a simple average of a specified

percentage of the highest flow rates observed at a site. Based on the various advantages and disadvantages of each of the methods, the following was concluded. The method based on identifying breakdown events is most suitable for the determination of capacity at a site where a detailed operational analysis is desired. For example, at sites where different operational treatments (e.g., ramp metering) are going to be tried in an effort to improve operations and an estimate of capacity that is as accurate as possible is desired. The method based on fitting a mathematical function to speed-flow data is not as suitable as the previous method for detailed evaluations of operational treatments, but is still appropriate for the determination of general capacity estimates. The average maximum flow rate capacity estimation method is most suitable for planning and preliminary engineering applications.

CHAPTER 1
INTRODUCTION

Background

The maximum number of vehicles that can be carried by a freeway lane is a critical factor for the planning, design, and analysis of freeway facilities. Although definitions vary, the value used to represent the maximum number of vehicles that can be carried by a freeway lane is generally termed ‘capacity’. The Highway Capacity Manual HCM (2000) is considered to be one of the authoritative sources on capacity values for a variety of roadway types in the U.S. It provides a single set of capacity values for basic freeway segments as a function of free-flow speed. These values are considered to be reasonably representative values for freeways located throughout the U.S., but it is recognized that lower or higher values may be more appropriate in any given location. The Florida Department of Transportation (FDOT), for one, believes that capacity values for Florida freeways might be lower than the values provided in the HCM. This belief is based on a preliminary basic analysis of freeway flow data.

Problem Statement

While it is generally recognized that the capacity values provided in the HCM may not be perfectly applicable to all freeway locations, the HCM does not provide any guidance on how its recommended values can be adjusted to reflect significant differences in capacity due to local conditions. Although there are adjustments that can be made to the free-flow speed, which in turn will affect the base capacity value, and also adjustments that can be made to the traffic demand, there is no mechanism for directly adjusting the base capacity values. Furthermore, the HCM does not provide a method that can be used for measuring or estimating capacity values.

Research Objective

The objective of this research was to investigate various methods that can be used to arrive at an estimate of freeway capacity values, and to recommend one of these methods to the FDOT for use in developing their own estimates of capacity for Florida freeways. The following tasks were performed in order to accomplish the desired results of this research objective:

- A detailed review of previous research related to methods used to estimate the capacity of basic freeway segments, as well as a review of the definitions of capacity that are used in these methods.
- From this review, the selection of one or more methods to test with Florida freeway data.
- In addition to methods identified from the literature, a simple method is also developed which is easy to apply and will yield capacity estimates similar to those obtained through more mathematically and/or complex methods.
- A detailed survey of basic freeway segments across Florida to determine suitable sites for the collection of data to use with the selected method, or methods.
- Traffic data are obtained for the respective chosen sites across Florida and are prepared for the subsequent processing and analysis.
- Data were processed and analyzed according to the selected/developed capacity estimation methods.
- Different methods of estimating the capacity values are compared with each other, as well as with the HCM value.

Organization of Report

The remainder of this report is organized as follows. Chapter 2 discusses the previous studies on estimating the capacity values and the various methodologies implemented in order to estimate the capacity values for freeway segments. The chapter also discusses the various factors which affects the capacity values on the freeway segment. Chapter 3 explains the selected methodologies used for estimating the capacity values, the description about the data and procedure applied for the site selection. Chapter 4 gives results of analyzed data for the selected

sites from all the estimation methods. Chapter 5 discusses the summary of the research study, describes the results and provides recommendations for future research on this study.

CHAPTER 2
LITERATURE REVIEW

Introduction

This chapter provides a review of the literature in several related areas. First, the definitions of capacity as discussed historically in the HCM and by other researchers are provided. Second, the concept of breakdown is discussed and various studies are presented which use the concept of breakdown to estimate the capacity on a freeway segment. Third, different capacity estimation methods as suggested and implemented by previous studies are discussed. Fourth, a discussion is provided on factors which may affect capacity values. Fifth, a brief summary on studies that discuss various capacity estimation methods is provided.

Definition of Capacity

The value used to represent the maximum number of vehicles that can be carried by a freeway lane is generally termed ‘capacity’. However, a variety of specific definitions of capacity have been offered by various sources/researchers.

According to Agyemang-Duah and Hall (1991), the capacity values of a freeway segment were defined as the maximum 15 minute flow values for two traffic state conditions namely: pre-queue flows and queue discharge flows. In a study by Van Aerde (1995), the capacity was defined as the apex value of the speed-flow curves obtained by fitting speed flow data points to a mathematical model on the basis of a simple car following model. With more research and studies, the definition of capacity was refined and was defined on the basis of the breakdown flows, which is discussed later in this chapter. Brilon et al. (2005) defined capacity as the expected value of the Weibull distribution function; that is, the value at which there is a 50% probability of occurrence of a breakdown event. This definition of capacity accounted for the stochastic nature of the capacity values.

In the most recent edition of HCM (2000), the capacity of a basic freeway segment is defined as "the maximum hourly rate at which persons or vehicles can be reasonably expected to traverse a point or a uniform section of a lane or roadway during a given time period, under prevailing roadway, traffic and control conditions". As the definition provided by the HCM includes the term "expected", the capacity value for a freeway facility is not considered to be constant and is considered to be stochastic in nature. Also, the flows on different freeways are observed to vary under different conditions. Thus, a single value of the capacity value for a freeway facility does not reflect the real world observations and the capacity values are considered to be stochastic in nature. However, this edition and definition became the focus for determining the capacity as it is the most commonly accepted professional reference for traffic engineering analysis.

The HCM (2000) provides the relationships between speed, flow and density which gives capacity values for a freeway segments free-flow speeds. According to the HCM (2000) and TRB Special Report 209 (1997), under ideal traffic and geometric conditions, freeways will operate with capacities of 2400 pc/h/ln. These conditions and capacity are typically achieved on freeways with speeds of 70 mi/h or greater. As the free-flow speed decreases, there is a slight decrease in the capacity values. For example, the capacity of a basic freeway segment with a free-flow speed of 55 mi/h is expected to be approximately 2250 pc/h/ln whereas for free-flow speed of 70 mi/h, the capacity is given as 2400 pc/h/ln. These capacity values were arrived after observing the maximum flow values on various freeways across the U.S. Within a range of 55 mi/h to 70 mi/h, the variation in the capacity values are observed but the HCM lacks in providing a methodology on how these capacity estimates are obtained.

These capacity estimates are based on the series of speed-flow curves provided in HCM (2000) which gives relationships between speeds and flows for basic freeway segments for different free flow speed values. These speed-flow curves are only provided for the un-congested traffic state and the manual does not mention anything about post congested traffic states. However, it is observed that when the demand on the freeway exceeds the capacity, a transition takes place and the freeway system becomes congested. This state of transition from an un-congested state to a congested state is defined as breakdown. It is thus evident that a breakdown phenomenon has a significant impact on the capacity value for a freeway segment. Although, the maximum flows observed on a freeway facility closely resemble the maximum pre-breakdown flow values, the HCM does not discuss the breakdown flows and the concept of breakdown in its editions. Thus, it becomes important to discuss the concept of breakdown from various studies and research performed in the past. The following section introduces the concept of breakdown and explains the transition states from un-congested to congested flows and vice-versa.

Concept of Breakdown

A number of researchers have investigated the effects of transition from free-flow conditions to congested conditions on freeways. With more research on the transition states, it was found that by studying the breakdown phenomenon, the determination of capacity values can be studied in a more comprehensive manner. It was also observed that the breakdown event does not necessarily occur at a maximum flow and breakdown can occur at flows lower or higher than those traditionally accepted as capacity. It was suggested then, that the breakdown is a better measure to associate specific flows with the capacity flows.

With the concept of a breakdown state affecting the capacity values, the definition of capacity should also incorporate the probabilistic nature of the occurrence of the breakdown events. As a result, a new definition of the capacity value was developed as the volume below

which the facility conditions are acceptable and above this volume, the facility condition becomes unacceptable. This transition between proper operation and non-acceptable flow conditions was defined as “breakdown”. Through a thorough literature review, such a breakdown on a freeway occurs when the average travel speed is reduced from an acceptable speed level to a much lower value of congested conditions. These transitions usually involve a rather sudden speed reduction. The factors which affect the breakdown depend upon one region to another based upon different driving culture and the driver’s behavior across countries and states. The following section gives some of the brief descriptions and discussions on the past research related to breakdown.

Elefteriadou et al. (1995) discussed the probabilistic nature of breakdown at freeway merge junctions. The authors analyzed the probabilistic aspect of ramp merge breakdown by examining and analyzing traffic data at ramp merge junctions for three sites using the NCHRP-Project 3-37 data. It was observed that breakdown was not the direct result of peak volumes. It was also observed that breakdown at ramp-freeway junctions is a probabilistic variable and not deterministic. The authors concluded that capacity does not necessarily occur immediately before breakdown since it might include the cluster of vehicles from ramp which can cause the breakdown. However, the authors didn’t discuss anything related to the breakdown on a basic freeway segment.

A study was conducted by Lorenz and Elefteriadou (2001) to estimate the capacity by the breakdown definition from congested flow and to measure the capacity through discharge flow measurements. The authors conducted an extensive analysis of speed and flow data collected at two freeway-bottleneck-locations in Toronto, Canada, to investigate whether the probabilistic models previously developed replicated reality. The time-series speed plots were examined at

two sites which concluded that a threshold speed at approximately 90 km/h existed between the non-congested and congested regions. This threshold was used to define the breakdown but the cases where the average speeds dropped for a given interval was considered as a true breakdown. The period for this research was taken as 5 minutes or more. The authors observed the frequency of the breakdown events and it was concluded that with increase in flow rate, the probability of a breakdown event to occur increases. The authors also suggested that the capacity depends upon the probability of occurrence of a breakdown event. The researchers also observed that the capacity depends upon the particular flow rate at which the facility breaks down and for a freeway facility the capacity depends upon the discharge flow following breakdown and the flow at which the breakdown occurs. However, the research lacked the discussions on the effects of flows prior to the breakdown event.

To study the pre-breakdown flows, Elefteriadou and Lertworawanich (2003) examined freeway traffic data at two sites over a period of several days which focused on the non-congested state to congested transition state. The authors observed the maximum pre-breakdown flow, the breakdown flow and the flows following the breakdown. It was concluded by the authors that the numerical value of the maximum pre-breakdown flows was larger than the flows following the breakdown but the flows at which a breakdown event occurs is always less than the flows prior to the breakdown and after the breakdown.

Zou and Levinson (2003) discussed that with timely traffic prediction the traffic control facilities can provide rapid and effective response. The authors used frequency domain tools in traffic flow studies instead of using time domain analysis methods. With the use of the frequency domain tools, one can detect the traffic breakdown more effectively. It was found from the research that with the changing rate of the cross-correlation between density dynamics and flow

rate, one can determine the transition from free flow phase to the congestion phase. The method as proposed by the authors suggests that an un-returnable transition will occur only if the changing rate of the cross-correlation exceeds a threshold. A new method was thus developed in order to detect the congested state on a freeway. But, due to complexity involved in this method, the method to detect breakdown would be used as given by Elefteriadou and Lorenz (2001).

Several authors, Minderhoud et al. (1997), Lorenz and Elefteriadou (2001), observed only traffic breakdowns at different flow rates to demonstrate the variability of flows preceding a breakdown. But in order to have a better comprehensive theoretical concept, more systematic analysis should be performed.

Brilon et al. (2005) discussed the concept of stochastic capacities which seems to be more realistic and more useful than the traditional use of single capacity values. The author presented that with a transition of traffic flow from an un-congested state to a congested state, the capacity value of a freeway segment is achieved. The paper examines the traffic flow patterns counted at 5-minute intervals over several months at different sites, which clearly showed that capacity can be well taken as Weibull-distribution with a nearly constant shape parameter which represented the variance in the capacity values. This was identified using the so-called Product Limit Method, which is based on the statistics of lifetime data analysis. The investigations were an extension to the idea being proposed by Minderhoud et al. (1997). The author modified and extended the Product Limit Method to estimate the capacity values. As discussed in the paper, the stochastic analysis of capacity values was coherent with the real world results on German freeways. Overall, it was interpreted that the concept of randomness in capacity values is found to be more applicable as compared to other traffic engineering methodologies.

In an extended research to Brilon et al. (2005), Geistefeldt (2008) discussed the same approach of analyzing the capacity values by using the stochastic capacity concept. The author compared the stochastic capacities for several freeways in Germany with the conventional capacity estimates. The conventional capacities from the speed flow diagram were estimated from the Van Aerde model. The capacity distribution function was used to find the breakdown probability that corresponds to the conventional capacity estimation. The author, after conducting the analysis for several freeways in Germany, concluded that these probability values could be suitably used for designing the freeway segments on the basis of these obtained values. The author summarized that with the availability of five-minute traffic data, 3% of the probability value on the capacity distribution function would fairly represent the freeway segment and with traffic data in one-hour data intervals, 40% of the probability value will represent the respective freeway segment.

Methods to Estimate Capacity

A lot of research has been done on the determination of freeway capacity and its variability. Different studies have performed on the basis of different definitions of capacity values and various methods have been implemented to account for the stochastic nature of the capacity values. The following section summarizes more recent and well-accepted methods of estimating freeway capacity values.

Van Aerde Model

Fitting a mathematical function to speed-flow data plots to determine capacity dates back to Greenshields (1935). Since then, many researchers have tried using various mathematical functional forms to describe the speed-flow relationship. However, in many cases, the functional form either did not fit the data very well or was not necessarily consistent with traffic flow theory. A mathematical model developed by Van Aerde (1995) generally provides a good fit to

speed-flow data and is consistent with traffic flow theory. The mathematical model, as proposed by the author, is a continuous function and has the ability to explain all the traffic states in the fundamental speed-flow diagram.

The model is based on a simple car following model which is based on the minimum headway distance between consecutive cars. The headway is a combination of a constant term, a term which depends on the difference between the speed at any given time and the free flow speed, and a term which depends on the speed at any given time. The model is given by:

$$k = \frac{1}{c_1 + \frac{c_2}{u_f - u} + c_3 \times u} \tag{2-1}$$

Where:

k = traffic density (veh/km)

u = space mean speed (km/h)

u_f = free flow speed (km/h)

c_1, c_2, c_3 as the parameters for the respective three terms

These parameters are calibrated by a non-linear regression analysis as explained in Van Aerde and Rakha (1995). The procedure as proposed by the authors solves an optimization problem which calculates different parameters on the basis of a speed-density relationship. The parameters calibrated from this method are used to determine the free-flow speed, speed at capacity, capacity, and the jam density. From the optimization technique, the model parameters are calculated from the following:

$$c_1 = m \times c_2 \tag{2-2}$$

$$m = \frac{2 \times u_c - u_f}{\left(u_f - u_c\right)^2} \tag{2-3}$$

$$c_2 = \frac{1}{k_j \times \left(m + \frac{1}{u_f}\right)} \tag{2-4}$$

$$c_3 = \frac{-c_1 + \frac{u_c}{q_c} - \frac{c_2}{u_f - u_c}}{u_c} \tag{2-5}$$

where:

c_1 = fixed distance headway constant (km),

c_2 = first variable distance headway constant (km^2/h),

c_3 = second variable distance headway constant (h),

u_f = free-speed(km/h),

u_c = speed at capacity (km/h),

q_c = flow at capacity (veh/h),

k_j = jam density (veh/km), and

m = is a constant used to solve for the three headway constants (per hour)

After the optimization problem is solved, the flow value at the apex of the mathematical model fitted to the speed-flow data points is considered as the capacity estimate for the freeway segment. However, this model is only applied to the freeway segments which have considerable number of congested flow points in the speed flow diagram. With sufficient congested points, it is made sure that the data has flows in the range of the capacity level.

Van Aerde and Rakha (1995) used the multivariate regression analysis procedure for performing the automated fitting of speed-flow relationships for different roads based on loop

detector data, and demonstrated the procedure's flexibility for fitting speed-flow data on a variety of roadway types.

Product Limit Method

The Product Limit Method (PLM) for capacity estimation was proposed by Brilon et al. (2005) to estimate the capacity values by identifying the traffic flow breakdown events. The identification of these breakdown events are determined by application of a suitable algorithm but typically the breakdown events are identified by sudden drops in speed or sudden increase in occupancy values that are sustained for a certain period of time. This method determines the capacity based on the flows which causes the breakdown event. As a result these observed flows are found to be random in nature and accounts for the stochastic nature of the capacity values. To estimate the capacity, it is observed that these flows follow a particular mathematical distribution. These effects make the capacity distribution function as follows:

$$F_c(q) = p\ (\ c \leq q) \tag{2-6}$$

where,

F_c = capacity distribution function

c = capacity

q = traffic volume

As discussed by Minderhoud et al. (1997), Brilon et al. (2005) modified the idea of defining the capacity distribution by proposing an analogy to the statistics of lifetime data analysis. The lifetime distribution function is given by:

$$F(t)=1-S(t) \tag{2-7}$$

where,

$F(t)$ = distribution function of lifetime = $p\ (T \leq t)$

T = lifetime

$S(t)$ = survival function = $p\ (T > t)$

The lifetime distribution functions are based on the life period of the experiment time but there might a possibility of certain lifetimes which exceeds the duration of the experiment. These are accounted as the censored data in the survival analysis function. In a similar manner, the traffic breakdown is regarded as a failure event and it is analogous for estimating the capacity, c, as the lifetime, T, in the lifetime data analysis. The statistics of this lifetime analysis can be then used to estimate the parameters of the distribution function which includes the censored data. To estimate the survival function, Kaplan and Meier (1958) proposed the non-parametric method called as "Product Limit Method". The non-parametric method is described as follows:

$$\hat{S}(t) = 1 - \prod_{j:t_j \leq t} \frac{n_j - d_j}{n_j}{}_j \tag{2-8}$$

where,

$\hat{S}(t)$ = estimated survival function

n_j = number of individuals with a lifetime $T \geq t_j$

d_j = number of deaths t_j

Usually, in this function each observed lifetime is used as one t_j value. For the above equation d_j is always equal to 1. To estimate the distribution function for capacity analysis, a similar analogy to Equation 2-9 is provided. The distribution function for the capacity analysis is given as:

$$F_c(q) = 1 - \prod_{i:q_i \le q} \frac{k_i - d_i}{k_i}\text{; } i \text{ is set of } \{\text{B,C2}\} \tag{2-9}$$

where,

$F_c(q)$ = distribution function of capacity c

q = traffic volume

q_i = traffic volume in interval i

k_i = number of intervals with a traffic volume of $q \geq q_i$

d_i = number of breakdowns at a volume of q_i

{B} = set of breakdown intervals (see below)

Using this equation the traffic flow value, q, is kept into different categories or sets. These sets are obtained and named as set containing all the un-congested flow values. The various sets used for this approach are defined as follows:

- B: The traffic pattern is un-congested in time interval, *i*, but the observed flow value just after the time interval causes a breakdown, i.e. the average speed value drops below the pre-defined threshold value in the next interval *i*+1

- F: The flow values for which the traffic flow is found to be un-congested in interval *i* and in the following interval *i* + 1. This flow value in the interval *i* contains a censored value. This flow interval reveals that the actual capacity in interval *i* is greater than the observed volume q_i

- C1: The flow values in this interval are in congested state for interval *i*, i.e., the average speed is below the threshold value. This flow interval, *i*, provides no information about the capacity value, so these intervals under the congested state are not considered for the data analysis.

- C2: The flow values in this interval, *i*, are considered to be in un-congested state, but the flow value causes a breakdown. However, in contrast to classification B, traffic is congested at a downstream cross section during interval *i* or *i* – 1. For this case, the breakdown at the observation point is supposed to be due to a spillback from downstream. As this flow interval also does not give information about the capacity values at the observation point, these flow values are also not considered for the data analysis.

After the flow datasets have been assigned with the respective set values, the distribution function $F_c(q)$ is plotted for the flow rates in B set. These set of values under B category are thus termed as B-set flows and consequently the B-set values. This distribution function is called as the PLM curves for the rest of the methodological procedure. The value of the distribution function as estimated by the PLM will reach a value of 1 if the maximum observed volume is from the set B and for all the other volumes that are not in B-set are assigned a value of 0. After the breakdown events are identified, the next step is to define the two elements which becomes the basis of using PLM. One of the elements is to select the data time intervals for which the analysis should be performed. This is decided on the basis of the availability of data. Another element is to identify the criteria by which the breakdown events are identified. As a breakdown of traffic flow usually involves a significant speed reduction, breakdown events will be identified if there is drop in the average speed across the lanes for a certain period of time. The threshold speed is estimated using the method as described by Elefteriadou and Lorenz (2001). If the speed value drops in any interval, the traffic flow prior to that interval are considered as B-set flows.

While estimating the capacity functions, it is necessary to know more about the mathematical type of the distribution function $F_c(x)$ or the PLM curves. To find the distribution, Brilon and Zurlinden (2003) suggested various plausible function types like Weibull, Normal and Gamma distribution. To estimate the parameters of the distribution functions, a maximum likelihood technique is used. The likelihood function is given by (Lawless, 2003):

$$L = \prod_{i=1}^{n} f_c(q_c)^{\delta_i} \times [1 - F_c(q_i)]^{1-\delta_i} \tag{2-10}$$

where,

$f_c(q_i)$ = statistical density function of capacity c

$F_c(q_i)$ = cumulative distribution of capacity c

n = number of intervals

δ_i = 1, if uncensored (breakdown classification B and C2)

δ_i = 0, elsewhere

The likelihood function or its natural logarithm has to be maximized to calibrate the parameters of the distribution function as per Lawless (2003). By comparing different types of functions based on the value of the likelihood function, the Weibull distribution is estimated to be the function that fit the observations on all freeway sections and are in accordance with the PLM curves. The expected Weibull distribution function will be:

$$F(x) = 1 - e^{-(\frac{x}{\beta})^{\alpha}} \quad \text{for } x \geq 0 \tag{2-11}$$

Where,

α = shape parameter

β = scale parameter

The Weibull distribution is thus checked if it fits well into the PLM estimation. The PLM can also be also used for traffic densities instead of volumes, q for capacity estimation. After the parameters, i.e. α and β are calibrated, the mean and the standard deviation of the function are calculated that are calculated by the following equations respectively:

$$E(x) = \beta \times \Gamma(1 + \frac{1}{\alpha}) \tag{2-12}$$

$$\sigma(x) = \sqrt{\beta^2 \times \Gamma(1 + \frac{2}{\alpha}) - \mu^2} \tag{2-13}$$

where,

β and α are the parameters estimated by the distribution function

Γ is the Gamma distribution function

μ is the mean or the expected value for the distribution function

According to Brilon et al. (2005), the mean of the capacity distribution function or the Weibull curves gives the capacity estimates for the freeway segments.

Other Methods

A study by Agyemang-Duah and Hall (1991) examined data over 52 days during peak periods to investigate the possibility of a drop in capacity as a queue forms, and to recommend a numerical value for capacity. They compared the plots of pre-queue peak flows and queue discharge flows for 15-minute intervals in which the distribution of the plots was fairly similar. The authors observed the mean value of the 15-minute maximum flows and recommended 2,300 pc/h/ln as the capacity value of freeway segments for stable flow conditions and 2,200 pc/h/ln for post-breakdown conditions. Although the researchers recommended capacity values, they did not discuss the variability in the capacity values observed. Doubts over this concept of capacity as a constant value was raised by Ponzlet (1996), who demonstrated that capacities vary according to external conditions like dry or wet road surfaces, daylight or darkness, and prevailing purpose of the freeway, whether it is used for long distance or metropolitan commuter traffic.

To examine the variability of capacity values, Minderhoud et al. (1997) recommended the use of the PLM to discuss the variability in the values of capacity. The method due to its sound theoretical concepts was used to estimate the capacity distribution using the non-congested flows. The author discussed that the effects of non-congested flow rates for estimating the capacity values. It was observed that higher non-congested flows had a significant impact on the capacity values for a freeway facility. The study also discussed and compared other estimation

methods for the freeways and came to a conclusion that PLM can capture the variability in the capacity values. The study however, did not discuss about the flow rates following a congested state.

Factors that May Affect Capacity Values

The following section discusses factors that have either been shown to affect, or has been hypothesized by some to affect the capacity values.

Free-Flow Speed

The speed-flow and density-flow relationships for a basic freeway segment are provided in HCM (2000) which vary according to the free-flow speeds on the freeways. On the basis of these speed-flow curves and level of service under certain conditions, the capacity is obtained from the curves. It is observed from the values provided by the HCM (2000) that with the decrease in the free flow speed, the capacity value for a freeway will decrease. However, this relationship is only provided for free flow speeds ranging from 50 mi/h to 70 mi/h. The research which led to the development of these curves found that a number of factors affect the free-flow speed which indirectly affects the capacity of a freeway. These factors as discussed in the HCM (2000) are discussed and listed next:

Lane width

When the average lane width across all the lanes is less than 12 ft, the base free-flow speed is reduced. This implies that with decreasing lane widths, the free-flow speed decreases and indirectly the capacity is also reduced.

Lateral clearance

When the right-shoulder lateral clearance is less than 6 ft, the base free-flow speed is reduced. Similarly, it implies that if the lateral clearance is reduced, the free-flow speed and the capacity are also reduced.

Number of lanes

The HCM (2000) considers the freeway segments with five or more lanes (in one direction) have the base conditions with respect to the number of lanes. The manual provides that with decrease in number of lanes, the base free-flow speed decreases and so the capacity of a freeway segment. A detailed discussion on all the previous research is provided in the following section. However, it is interesting to note that this factor has been removed in the forthcoming 2010 HCM.

A limited research has been performed in past for comparing the traffic characteristics on freeways with different numbers of lanes. Al-Kaisy et al. (1999) developed a simulation approach for examining capacity and operational performance at freeway off-ramps areas but the authors didn't look into the effects of the number of lanes on a basic freeway segment. The authors investigated total upstream demand, off-ramp demand, length of deceleration, off-ramp free-flow speed and number of lanes at mainline. The investigation gave an insight that there is a significant impact of number of lanes on capacity and operational performance.

This study was based only on analyzing the freeways at diverge or weaving area but it lacked analysis on the basic freeway segments. However, two studies have been conducted in recent years to see the impact on capacity values if the number of lanes is increased on a freeway. The first study was conducted by Yang and Zhang (2005) which investigates the impact of the number of lanes on highways capacity. The authors give a better understanding of the relationship between highway capacity and its number of lanes upon the statistical analysis of the survey done on the freeways of Shanghai and Beijing with limited sites. The capacity values are estimated using the maximum sustained 15-minute rate of flow which can be accommodated by a uniform highway segment under prevailing and roadway conditions in the specified direction of interest. The statistical test is used to investigate the impact of the number of lanes on highway

capacity. The variance analysis of single factor and *t*-test are applied to test the inequality. It is found that the marginal decrease rate of average capacity per lane with increasing number of lanes is around 6.7%. They explored the possible explanations of the decrease which can be effect of increasing lane changing opportunities and cars interaction with increasing lanes on highways.

The second study was conducted as in extension to the previous research by Yang et al. (2007), which investigates the impact of the number of lanes on freeway traffic characteristics. The research is being conducted and validated upon an extensive field survey of traffic flow in Beijing and Shanghai with more significant improvements observed on two-lane (one-way direction) freeways, then on three lane freeways as compared to those on four-lane freeways. They explored that in congested traffic conditions, at the same densities, both flow rate per lane and average speed decrease with increase in the number of lanes on uninterrupted freeway segments. However, in free flow conditions, the average speed increases with increasing number of lanes on freeways. The regional factor had a little impact on the differences among the flow-density relationships on freeways with different number of lanes. The authors suggest that from the corresponding results observed, it can be interpreted whether building a wide road on a freeway is feasible or building a number of narrow roads around the freeway.

These studies conclude that the average capacity per lane on different lane freeways is not uniform. It might be due to the reason that a driver's behavior and its interactions with other cars change a lot on different lane freeways. This change is observed due to lane-changing activity and its associated disturbance to traffic stream. This observation may not be uniform on different-lane freeways and hence, leads to differences in the speed-flow-density relationships and effectively on the capacity values.

Interchange density

The interchange density of 0.5 interchanges per mile or a 2 mile-interchange spacing is considered as the base conditions to calculate the free-flow speeds. It is provided by the manual that with increasing interchanges per mile, the free-flow speed decreases. It is implied that the free-flow speeds observed on shorter segments are higher as compared to free-flow speeds observed on longer segments under the same conditions. As a result, the capacity for a longer segment is higher than for the short segment under the same conditions. This factor has been changed to ramp density in the forthcoming 2010 HCM.

Merge-Diverge Areas on Freeways

HCM (2000) does not indicate that merging or diverging maneuvers restrict the total capacity of the upstream or downstream basic freeway segments. The influence of merging or diverging vehicles is primarily to add or subtract demand at the ramp-freeway junction. Thus, it is believed that the capacity of a downstream basic freeway segment is not influenced by turbulence in a merge area. The capacity will be the same as if the segment were a basic freeway segment. As on-ramp vehicles enter the freeway at a merge area, the total number of ramp and approaching freeway vehicles that can be accommodated is thus defined as the capacity of the downstream basic freeway segment. Similarly, the capacity of an upstream basic freeway segment is not influenced by the turbulence in a diverge area. The total capacity that may be handled by the diverging junction is limited either by the capacity of the approaching (upstream) basic freeway segment or by the capacity of the downstream basic freeway segment and the ramp itself.

The basic approach to model merge and diverge areas in the HCM (2000) focuses on an influence area of 1500 ft including the acceleration or deceleration lane and the two outside lanes which are in the merging or diverging area. The HCM recognizes that other freeway lanes may

be affected by merging or diverging operations and the impact of congestion in the vicinity of a ramp can extend beyond the 1500 ft influence area, but according to it, this defined area experiences most of the operational impacts across all levels of service. Thus, the operation of vehicles within the ramp influence area is the focus of the computational procedures in the HCM (2000).

The HCM (2000) determines the capacity of a merge area by the capacity of the downstream freeway segment. Thus, the total flow arriving on the upstream freeway and the on-ramp cannot exceed the basic freeway capacity of the departing downstream freeway segment. There is no evidence that the turbulence of the merge area causes the downstream freeway capacity to be less than that of a basic freeway segment. The freeway capacity per lane is always stated as an average across all lanes and the individual lanes always carry proportionally less or more flow. In merge and diverge areas, through vehicles tend to move left to avoid turbulence, resulting in cases where inner lanes are very heavily loaded compared with lanes within the ramp influence area.

Apart from HCM (2000), a study was conducted by Al-Kaisy et al. (1999) to examine the capacity and operational performance at freeway diverge areas. The authors provided models to directly estimate the freeway capacity which were not mentioned in HCM (1997). A computer traffic simulation model INTEGRATION was used to explore the patterns of capacity and operational performance behavior at diverge areas under the impact of some key geometric and traffic variables. One of the control variables which were discussed in this research was number of lanes at mainline which was found to have a significant impact on the capacity and operational performance. The authors studied extensively the effect number of lanes at mainline on freeway capacity at ramp-freeway diverge areas which is explained next. The research showed that for

freeways with more number of lanes, impact due to the spill back was not so drastic as the traffic had enough room to proceed through the diverge section through the left lanes on the freeways. It was concluded that the number of lanes at mainline have an important impact on the capacity of the basic freeway segments when the diverge section is operated at the saturated traffic conditions, given that no queue spill back from the exit ramp is present. It was also concluded that at a particular flow rate, the queue spill back has a significant impact on the through traffic and consequently on overall diverge section operation. This blockage effect was found to become less significant with the increasing number of lanes.

Summary

To estimate the capacity for a basic freeway segment different studies are performed. The HCM (2000) gives fixed values for the capacity but does not consider the stochastic nature of the capacity values. However, different methods have been identified that can be used to estimate the capacity values. First, the method that estimates a breakdown probability distribution based on flow rates preceding breakdown events and accounts for the stochastic nature of capacity values. Second, the method that fits a mathematical function to plotted speed-flow data points, from which the apex of the function is taken as capacity.

CHAPTER 3
RESEARCH APPROACH

Introduction

This chapter describes the approach taken to achieve the objectives of this study. It provides a detailed discussion on the various analysis methods for capacity estimation, data collection, data reduction, and data processing.

Analysis Methods

As discussed in the literature review, capacity is considered as fixed value. Even under uniform traffic and roadway conditions, variability in maximum flow rates (i.e., flow rate before breakdown) is observed. However, different methods of estimating the capacity values are provided from past studies. Thus, the focus of this study is to compare the capacity values as estimated by different methods.

Three methods are used in this study to estimate the capacity of a freeway segment. The first method calculates the capacity value from Van Aerde Model, derived from the fundamental speed-flow diagram. The second method uses a combination of speed-flow data plots and the Product Limit Method, as identified from the literature. The third method is a simple averaging scheme of high flow rates that can be applied easily to estimate the capacity values.

Capacity Estimation from Van Aerde Model

The first method used to estimate the capacity for a basic freeway segment is based on fitting a mathematical model to the speed-flow data points. The relatively recent and widely-accepted mathematical model proposed by Van Aerde (1995) is used, as described in the literature review chapter. The regression analysis procedure that performs the fitting of the Van Aerde model is implemented in the software program SPD_CAL, by Rakha (2007). The SPD_CAL program calibrates the four parameters, free-flow speed, speed at capacity, capacity,

and jam density, for a given data set. The software uses a heuristic hill-climbing technique to determine the optimum parameters. The estimated capacity parameter is considered to be the capacity estimate of the freeway segment, which again, corresponds to the apex of the fitted function through the speed-flow data plot.

Capacity Estimation from Product Limit Method

According to the PLM approach as per Brilon et al. (2005), one of the key elements for the data analysis from PLM is to identify the breakdown events on the freeway segments. These events are typically identified by looking for sudden changes in traffic flow measurements or relationships, such as speed, occupancy, and correlation between volume and occupancy. In this study, it was decided to use average speed as the mechanism to identify traffic breakdown events.

Speed based breakdown identification

The breakdown events in this algorithm are identified using a speed threshold value. When the average speed drops below this threshold value for a specific period of time, a breakdown event is considered to have occurred.

To find the speed threshold value for freeway segments, each study site is analyzed independently as per Elefteriadou and Lorenz (2001). According to the authors, for each of these study sites, the speed and vehicle count data are tabulated in one-minute intervals for individual travel lanes and over all lanes. The vehicle count data are then expressed as equivalently hourly flow rates and the average speeds across all lanes are determined using the volume-weighted average speed of all vehicles crossing the particular detector station. The speed and flow rate data are plotted in time-series over each sample period with time on the x-axis and the speed on the y-axis.

To determine the breakdown condition or the transition state, the speed-flow plots are plotted for a specific period of time for all the sites. It is recommended to examine the daily time-series plot of speed rather than relying on scatter diagrams of many days of accumulated data. The first advantage in observing the daily traffic data is to obtain the relationship of speed with time, which the accumulated data cannot provide. The second advantage is that inspection of the daily plots helps in identifying the points that represent transition between congested and uncongested flow so that one can observe a breakdown state.

While analyzing the time-series speed plot, a threshold or boundary value will exist between the congested and uncongested regions. This threshold or boundary value of speed will be taken into account as an input for the PLM. This threshold value should be evaluated for all the study sites. In this study, only the disturbances that will cause the average speed over all lanes to drop below the threshold value of speed for a certain period will be visualized to identify the breakdowns. A particular event will be considered as a true breakdown if and only if the average speed across all lanes drops below the threshold value for that particular time interval.

Application of speed-based threshold value method

The speed threshold method is applied to each study site on the respective detector location for each day. On the basis of this method, the beginning of congestion times at study sites are identified and stored. If there is more than one event of congestion or breakdown event, the algorithm will run again and a second set of beginning of congestion times are stored separately in another database.

Once the beginning of the congestion periods or breakdown events is identified, the respective output for the particular breakdown is obtained. The output data are stored in a separate database which consists of the flow rates prior to the breakdown event with their respective B-set values. These flow rates are determined on the basis of uncongested datasets as

described in the literature. This output, which is referred as the uncongested flow datasets is developed according to the following steps:

- The B-set value of the flow rate just prior to the flow rate at which breakdown occurs is assigned a value of 1 and is referred as dataset {B}.
- All the other B-set values of volumes are assigned a value of 0 and are referred as dataset {F}.
- All the flow rates under the set {C2} are also obtained for the analysis and are eliminated from the flow rates in the set {F}.

The final output from the above specifications becomes the main dataset for the remaining analysis. The output thus obtained is used for the probabilistic modeling for developing the capacity distribution functions. The Weibull curves for all the study sites are developed from the maximum likelihood approach according to Elefteriadou et al. (2009). Brilon et al. (2005) refers to these Weibull curves as the capacity distribution functions, $F_c(q)$, or the breakdown probability function. These functions give the probability of a breakdown event, as a function of flow rate, on a freeway segment. The capacity distribution functions are first plotted for all the flow rates in B-set. These plots are referred as PLM curves by Brilon et al. (2005) are determined according to Elefteriadou (2009). As per the methodology, the PLM curves fit the Weibull distribution and are referred as Weibull curves. For the data analysis, the fitting of the PLM curves to the Weibull distribution is checked by superimposing the two curves and the applicability of this assumption is validated. Both the PLM curves and the capacity distribution functions are then superimposed with the speed-flow curves as described in Geistefeldt (2008). The flow sets in the speed-flow curves are plotted to their respective speed values. Only the flows from the {F} set, or the uncongested flows, are considered for these curves.

After the plots are superimposed with each other, the capacity values are estimated from these graphs. Different approaches are performed to estimate the capacity values from the

superimposed graphs. One of the approaches as given by Brilon et al. (2005) is to calculate the expected means of all the Weibull distribution curves obtained from analysis of all the study sites from Equation 2-12. This mean value is considered to be one of the capacity values for freeway segments. However, two different approaches are developed in this research study to estimate the capacity values. After the superimposition of these curves, a probability value is chosen from the capacity distribution function that corresponds to a flow rate, determined from the two approaches. On the basis of these specific flow values, the range of the probability values are obtained for all the sites and a suitable probability value is chosen for the capacity estimation. This suitable probability value will be traced back from all the capacity distribution functions onto the speed-flow curves to estimate the respective capacity for all the sites.

The capacity values as estimated by the expected mean and by selecting a suitable probability value are compared with the capacity estimates from the VAM and the values as provided by the HCM (2000). The following section discusses the applicability of the PLM on the basic freeway segments.

Capacity Estimated by Average of Maximum Flow Rates

Apart from the Van Aerde Model and the PLM, a simple methodology was developed to estimate the capacity values. This method is independent of complex mathematical functions and breakdown events. However it is dependent on the analysis period time over which this methodology is implemented. The average maximum flow rate analysis focuses only on the highest flows observed over a period of time at a particular detector location for given any segment. Capacity values were estimated from the average maximum flow rate method according to two different approaches.

For the application of these approaches, the first step is to aggregate the flow rate data into five-minute intervals. The second step is to convert the five-minute flow rate data to hourly flow

rates. The third step is to sort the data in from highest flow rates to lowest flow rates. The remaining steps are specific to each of the two approaches used for the average maximum flow rate analysis.

First, the average of top *x*% highest flows is taken such that the average value equals the capacity estimates as compared with Van Aerde Model and PLM estimation methods. The consistency and the range of the *x* values are observed and a particular *x* value is chosen which would fairly give the capacity estimates as per Van Aerde Model and PLM. Second, the average of all highest flows is taken above a flow rate which is a certain *x*% of the highest flow observed for the respective site. Similarly, on the basis of the consistency and the range of *x* values in this approach, a particular *x* value is chosen which would give the capacity estimates as per Van Aerde Model and PLM. While selecting a particular *x* value for these approaches, the percentage error in the difference of the capacity estimates from average analysis with Van Aerde Model and PLM is least.

It is also required to find a minimum analysis period for which these approaches are applicable. The identification of the minimum analysis period for this approach varies in several ways. These approaches are applied for any given dataset for a certain period of time. The initial testing analysis periods for this procedure are considered as: analysis over three month's data, two month's data, one month's data and two weeks of data. The percentage values as determined from two different approaches are obtained under each testing analysis period. These percentage values are compared with each other under a particular site for different analysis periods and the percentage difference is obtained. The consistency of the percentage values are observed over all the analysis period and on the basis of the minimum error to the percentage values, the minimum analysis period on which these approaches can be performed is obtained.

The methodology and the steps as explained above for the Van Aerde Model, the PLM and the average maximum flow rate analysis are implemented on the selected sites. The following section discusses data collection and data processing.

Data Collection

This section describes the procedure used to identify all suitable field sites for capacity analysis, the sources from where the data were obtained for these sites, the procedure to prepare the data, process the data and analyze the data using different methods. This section also explains the way to implement the methodology and to analyze the data on the selected sites.

Site Selection

Since the study focuses only on basic freeway segments, only freeway segments were finalized which are considered as basic freeway segments. For the purposes of this study, a basic freeway segment was considered to be a freeway segment that was at least 1500 ft in length, had a constant number of mainline lanes throughout this segment, whose operations were are not affected by the merge and diverge areas. However, the freeway segments with unequal number of lanes before the on ramp and after the on ramps are also considered as basic freeway segments provided the length of the freeway segment was at least 1500 ft. The freeway segments with consecutive on-ramps were also considered for the study analysis. On the basis of these characteristics, three types of freeway segments were accepted as the basic freeway segments that are provided in Figure 3-1. Also, all the freeway segments with HOT or HOV lanes were excluded, and sites with significant vertical or horizontal geometry were excluded.

The next step is to identify the desired sites for the districts of Jacksonville, Ft. Lauderdale, Orlando, Miami and Tampa. An extensive study for all the freeway segments was performed to select a number of desired sites on the basis of some criteria. As the main objective for this study was to compare methods for freeway capacity estimation, it is necessary that the candidate sites

regularly experience high flow rates, and more preferably, that experience recurrent congestion due to high traffic demands. This was identified by calculating the average per lane volume for most of the basic freeway segments in Florida. All the freeway segments which had reasonable high flows were considered for the data analysis for this research. However, the final sites were selected on the basis of the data available.

The candidate sites are selected on the basis of a review of maximum recorded flow rates on interstates across Florida. The data source used for these maximum flow rates was the Florida Data Highway DVD (2006) and Florida Traffic Information CD (2006). As the flow rates, available in the FDOT traffic DVD, are aggregated in hour intervals, this data aggregation level is not appropriate for the determination of capacity values. Ideally, the data at one or five minute of aggregation level are necessary for capacity estimation. Therefore, the data for the research study was obtained from the STEWARD (Statewide Transportation Engineering Warehouse for Archived Regional Data).

Data Source

As the scope of this study is limited Florida, it is required to obtain the data for all the potential sites. All the freeway segments across the state are initially considered for this study. The primary concern for data collection on these sites was the availability of useful and non-erroneous data from the sources available. The availability of data was checked for all the freeways in Florida and within the scope of this research, five districts namely: District 2 (Jacksonville), District 4 (Ft. Lauderdale), District 5 (Orlando/Daytona), District 6 (Miami) and District 7 (Tampa) were chosen for the data analysis.

The traffic data were obtained from STEWARD (Statewide Transportation Engineering Warehouse for Archived Regional Data), a website developed at University of Florida sponsored by Florida Department of Transportation and its database server, for districts of Jacksonville, Ft.

Lauderdale, Orlando and Miami. The data are obtained for all the detector locations in these regional areas. All the detectors in these areas are found to be RTMS (Remote Traffic Microwave Sensor) detectors. The detectors at these locations are assumed to be properly functional. The traffic data including the volume, speed, occupancy, etc., are collected by these detectors for all the districts. A comprehensive list of other data parameters that should be collected at each detector location are also developed on the basis of the literature review. The list included elements that previous research found to affect capacity or factors that may affect capacity on a freeway. These factors are listed below:

- Length of the basic freeway segments for all the sites
- Distance of the on-ramps and off-ramps from the basic segment
- Speed Limits on the freeways
- Terrain or grade of each freeway (%)
- Presence of any horizontal curvature on the site
- Volumes, speeds and occupancy by lane for a time period of 1 minute
- Percent heavy vehicles

The data for all these districts are obtained from July 2007 to January 2009. However, the final data used and analyzed for each district is different depending on whether the data are reasonable to use. Apart from the traffic data, the detector configuration files are also obtained from the same source. The data available from STEWARD are then prepared in a compatible format for the data analysis and are converted into one-minute data interval format for all the sites. The final datasets for the analysis are made available in one minute intervals. Table 3-1 provides a typical format of the data available after data processing. Once the data are obtained, the final sites are selected for the analysis on the basis of the availability of the data.

The final sites are selected on the basis of various points discussed in previous sections and are listed in Table 3-2. Figures 3-2 to 3-19 provide the Google Earth images for all the selected

sites with their respective site ID. The next section describes the utility programs developed to process the collected data on the selected sites.

Data Processing

To analyze the data for the PLM, two processing programs were developed: the Capacity Data Processor, and the Downstream Location Breakdown Identifier. The details and various limitations to these processors are described in the following section:

Capacity Data Processor

The Capacity Data Processor (CDP) utility program is used to identify the breakdown events from a file of traffic data for a specific site. The processor is developed based on several breakdown identification algorithms with a specific criterion through which the data can be analyzed to identify the breakdown events. The algorithms contained in the CDP are explained in the section that describes the Product Limit Method. One-minute aggregated interval data are used with the CDP. Some of the parameters, inputs and other elements which are used for the analysis from this processor are discussed next. Figure 3-20 provides a snapshot of the CDP.

The important elements used for the analysis using the Capacity Data Processor are the analysis time period for a given day, the algorithm method for identification of a breakdown event, the respective threshold value, the number of intervals preceding the breakdown, the analysis method and the recovery time period for a breakdown event. These elements used for the data analysis are described next with the respective values used.

Analysis period

The analysis period for the data processor is user-specific. It depends upon the user and the day of time, the user wants to apply the PLM. The ongoing research uses the data for each day from 5 AM to 10 PM. The reason to select such a period is to eliminate the other non-significant breakdown events where the traffic counts are very less. It is assumed that within this specific

time period, all the breakdown events which have traffic flows comparable to capacity values are considered.

Speed threshold value

The speed threshold value and the speed reduction time period are the key elements for the identification of breakdown events. After analyzing the time-series speed plots for the sites for a given period, the threshold value is determined. This value is considered as the threshold value and is used as an input for the capacity data analysis. The speed threshold value determines the congested flow intervals and the uncongested flow intervals.

From the speed threshold value, a breakdown is identified if the speed drops below this threshold value for a period of five minutes. Thus, for a breakdown event to be identified, the average speed (volume-weighted) must remain below the speed threshold value for a period of five minutes. Also, the breakdown event is recovered if the speed increases above this threshold value for a period of five minutes. Thus, for traffic flow to be considered to have recovered from a breakdown event (i.e., return to uncongested flow), the average speed (volume-weighted) must remain above the speed threshold value for a period of five minutes. However, the number of intervals at each breakdown event is also an important measure of this methodology which is discussed next.

Intervals preceding breakdown

According to Brilon et al. (2005), all the time intervals preceding the breakdown from the start time of the data analysis are considered for the data analysis in PLM. If multiple breakdown events occur at the selected site for a particular day, the intervals after a breakdown event was recovered were considered till the next breakdown event is identified.

These parameters are thus used as inputs to CDP to obtain the flow rates and the respective B-set values for rest of the PLM analysis.

Data imputation

In some cases, the traffic flow data for a site has a missing entry for one or more time periods. As the prepared data are available in one-minute aggregations, the data is read by the processor minute by minute. If a single one-minute period of data is missing (i.e., non-consecutive intervals) in the input data, the CDP will impute that entry as the average of the next data entry and the previous data entry. In cases where consecutive time periods of data are missing, imputation does not take place, and these time periods are ignored for purposes of analysis.

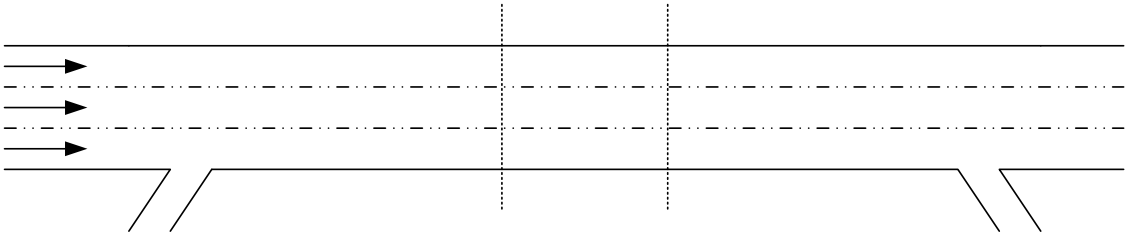
Downstream Breakdown Identifier

Locations that experience congestion due to a downstream bottleneck or incident are not considered in the analysis. Thus, it is necessary to determine if the breakdown at a site is result of downstream congestion that has propagated upstream. The CDP identifies breakdown events regardless of where the congestion initiated. The Downstream Breakdown Identifier (DBI) utility program identifies breakdown events that resulted from downstream congestion.

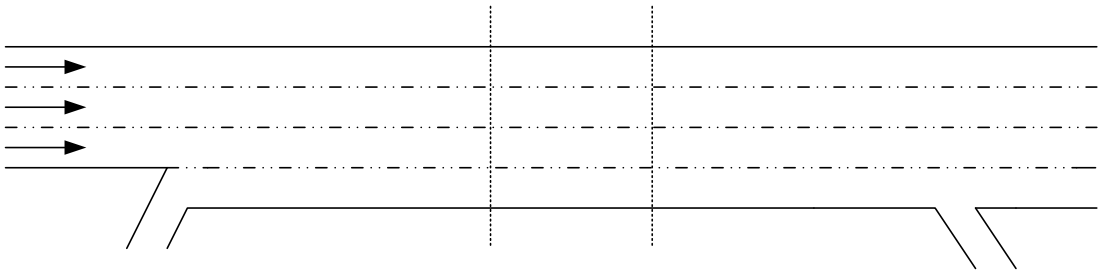
This utility program requires only two inputs: 1) the PLM data file as obtained from CDP for the desired basic freeway segment location, and 2) the results file from the capacity data processor for the detector at the downstream location. The PLM data at the basic freeway segment location is filtered on the basis of the following considerations:

First, if the breakdown event at an downstream location occurs first as compared to that at the upstream detector location, the flow values at the upstream location are discarded from the start time of the breakdown event at the downstream event till that event is recovered. Second, if the breakdown events at both locations occur at the same time, then the flow value preceding the breakdown event at the upstream location is discarded.

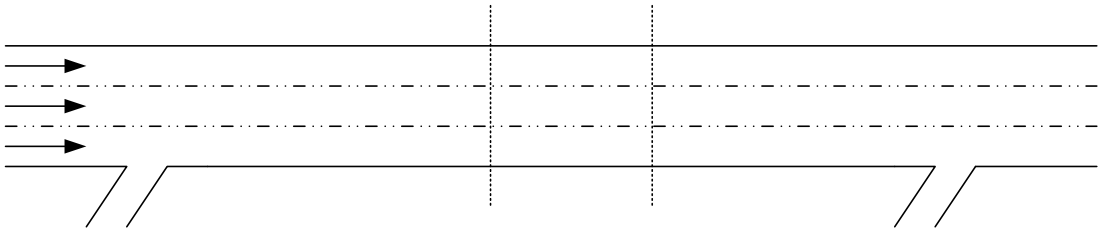
The output flow values, speed values and B-set values from this processor are further used for the PLM analysis. The data is checked and the PLM and Weibull curves are determined to estimate the capacity value as per the methodology. Figure 3-21 provides a snapshot of the PLM data processor used for this analysis.



A)



B)



C)

Figure 3-1. Acceptable freeway segment configurations. A) Freeway segment with constant number of mainline lanes throughout the segment, B) Freeway segments with unequal number of lanes before the on ramp and after the on ramp with atleast 1500 ft in length and C) Freeway segment with consecutive on ramps with at least 1500ft in length

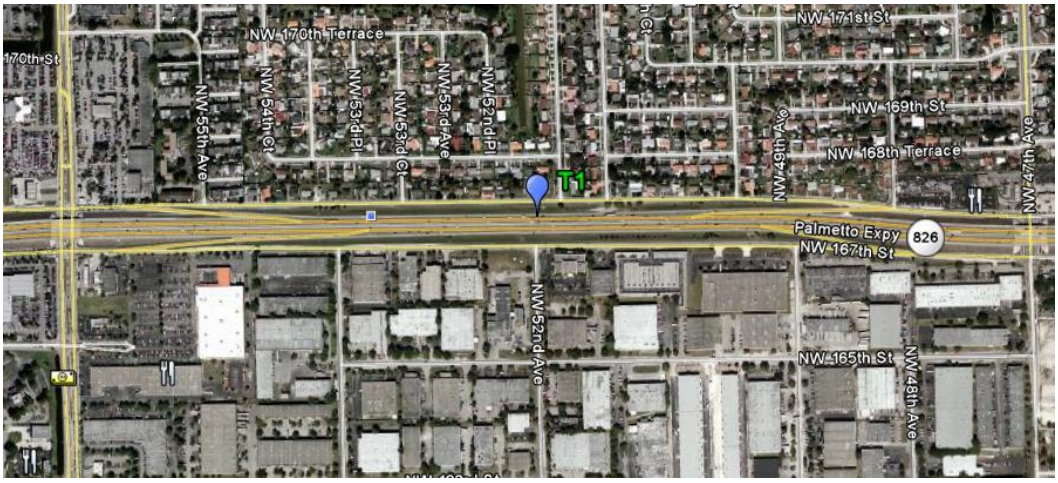


Figure 3-2. Snapshot of site T1 (Source: Google Earth)

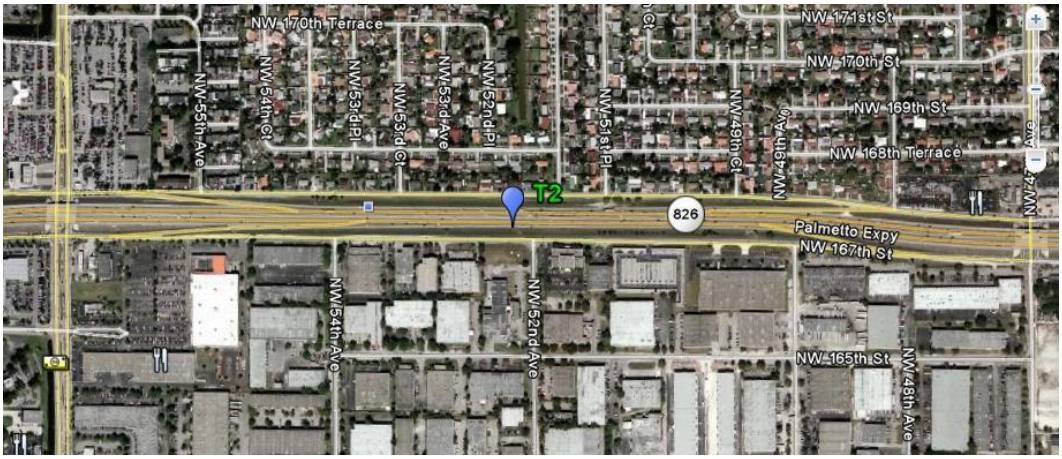


Figure 3-3. Snapshot of site T2 (Source: Google Earth)

Figure 3-4. Snapshot of site T3 (Source: Google Earth)

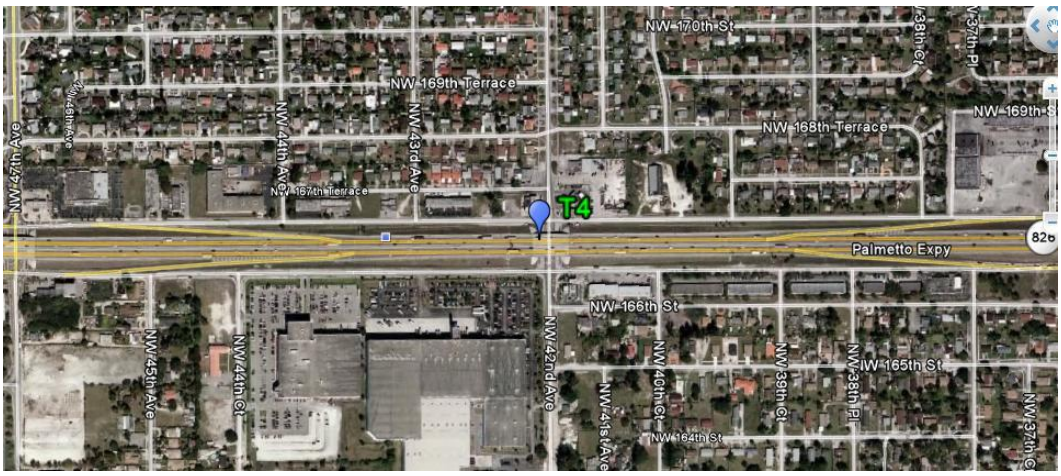


Figure 3-5. Snapshot of site T4 (Source: Google Earth)

Figure 3-6. Snapshot of site T5 (Source: Google Earth)

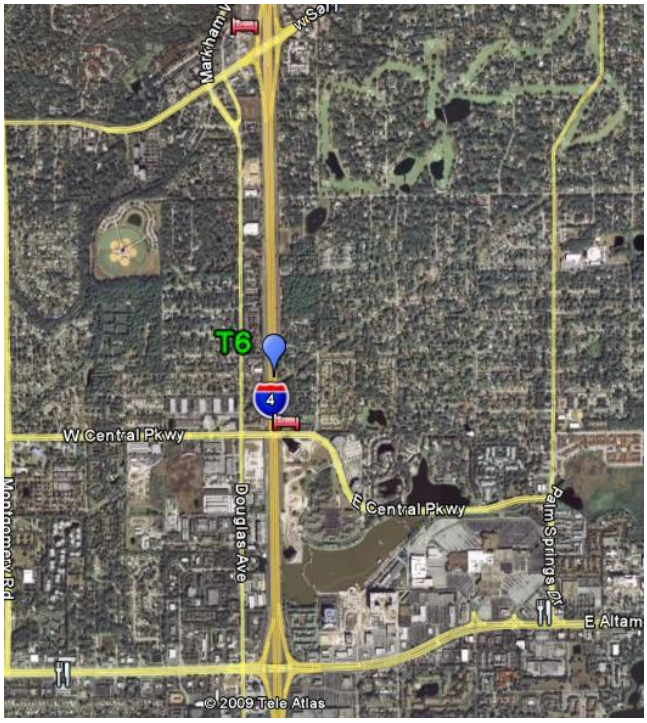


Figure 3-7. Snapshot of site T6 (Source: Google Earth)

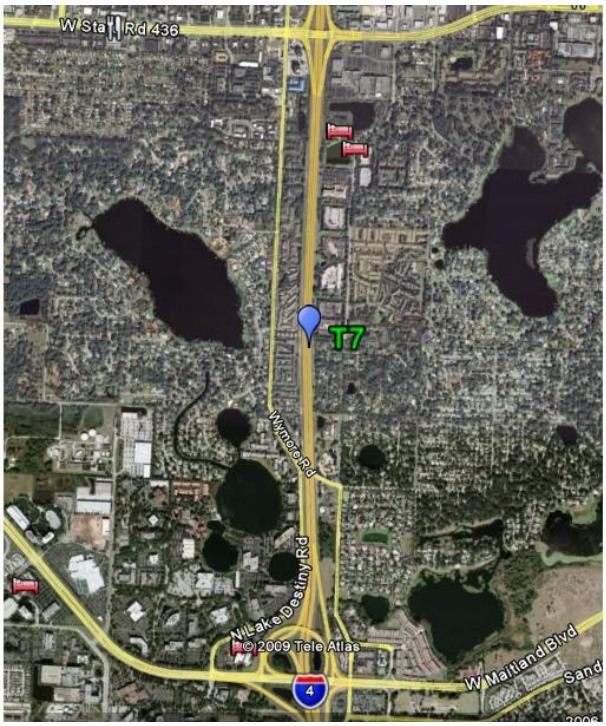


Figure 3-8. Snapshot of site T7 (Source: Google Earth)

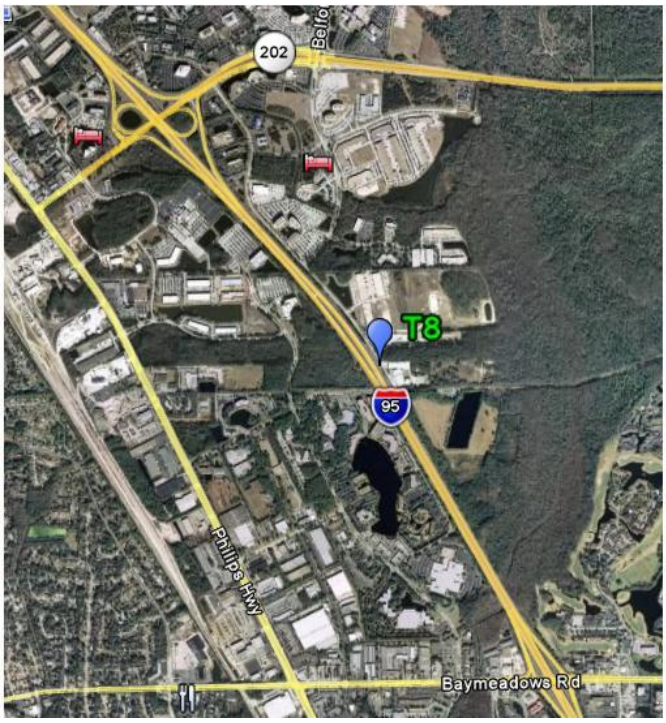


Figure 3-9. Snapshot of site T8 (Source: Google Earth)

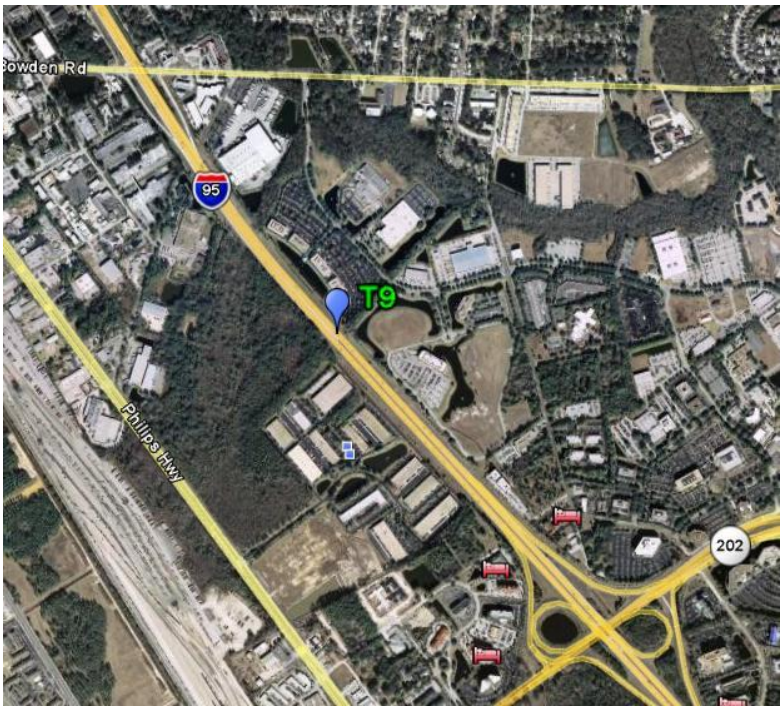


Figure 3-10. Snapshot of site T9 (Source: Google Earth)

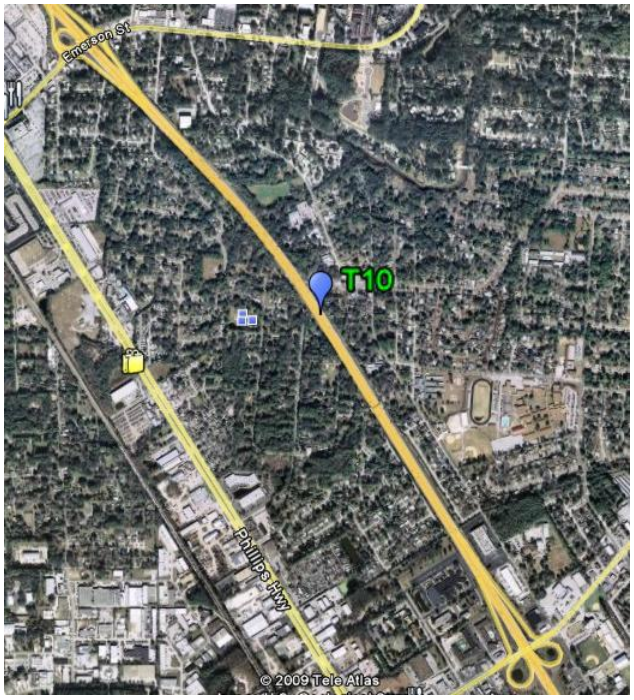


Figure 3-11. Snapshot of site T10 (Source: Google Earth)

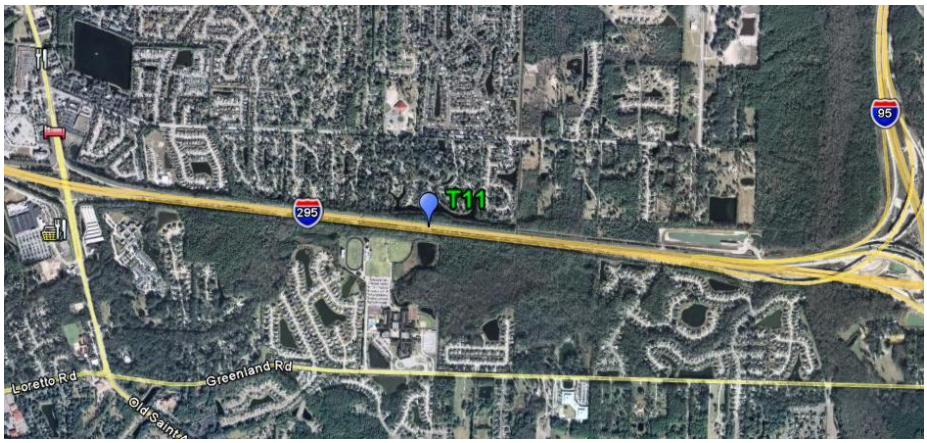


Figure 3-12. Snapshot of site T11 (Source: Google Earth)

Figure 3-13. Snapshot of site F1 (Source: Google Earth)

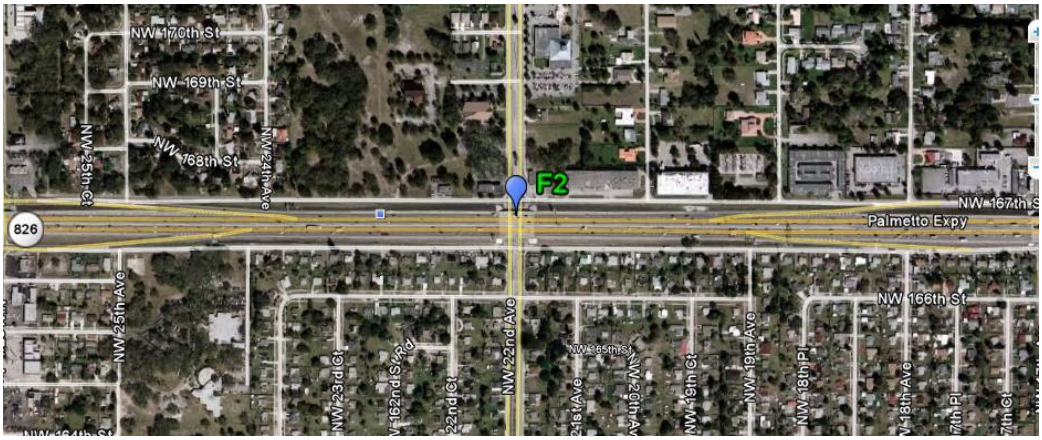


Figure 3-14. Snapshot of site F2 (Source: Google Earth)

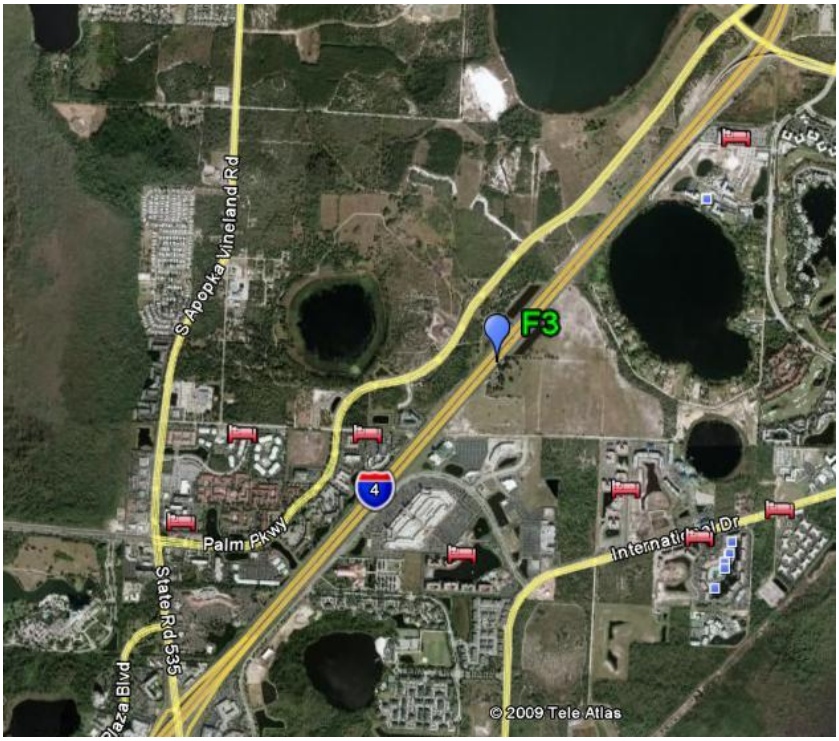


Figure 3-15. Snapshot of site F3 (Source: Google Earth)

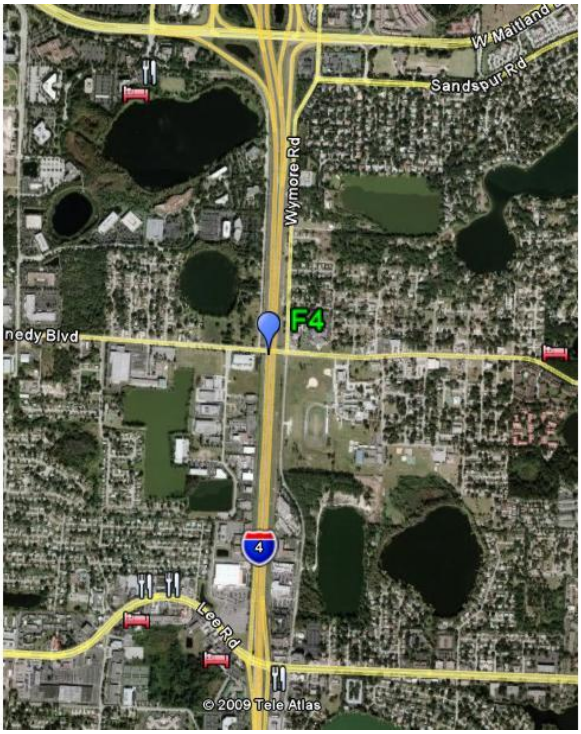


Figure 3-16. Snapshot of site F4 (Source: Google Earth)

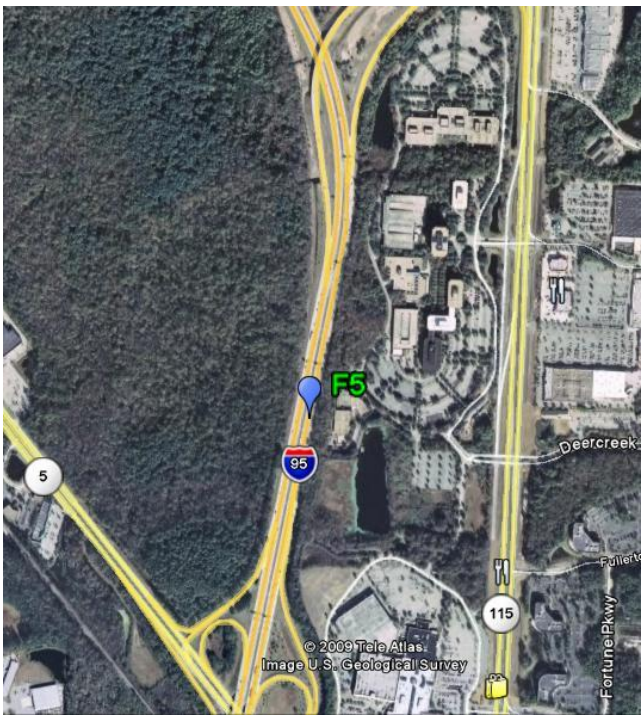


Figure 3-17. Snapshot of site F5 (Source: Google Earth)

Figure 3-18. Snapshot of site F6 (Source: Google Earth)

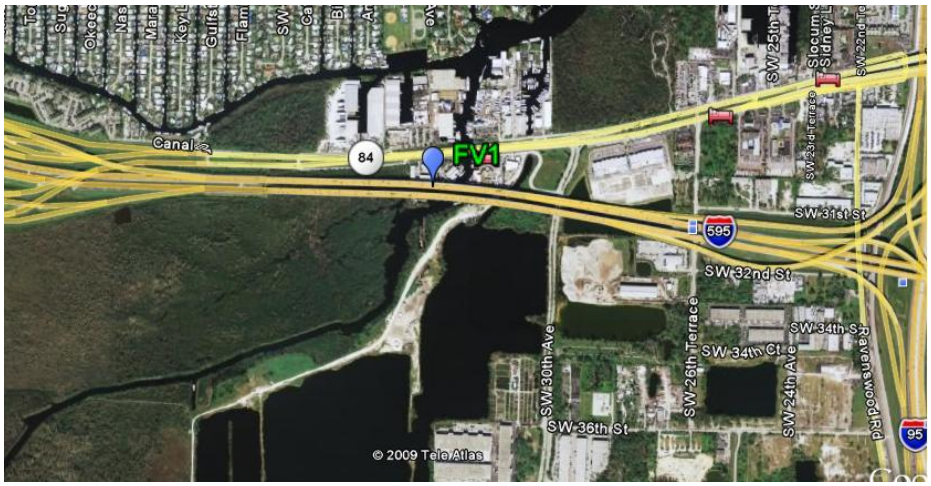


Figure 3-19. Snapshot of site FV1 (Source: Google Earth)

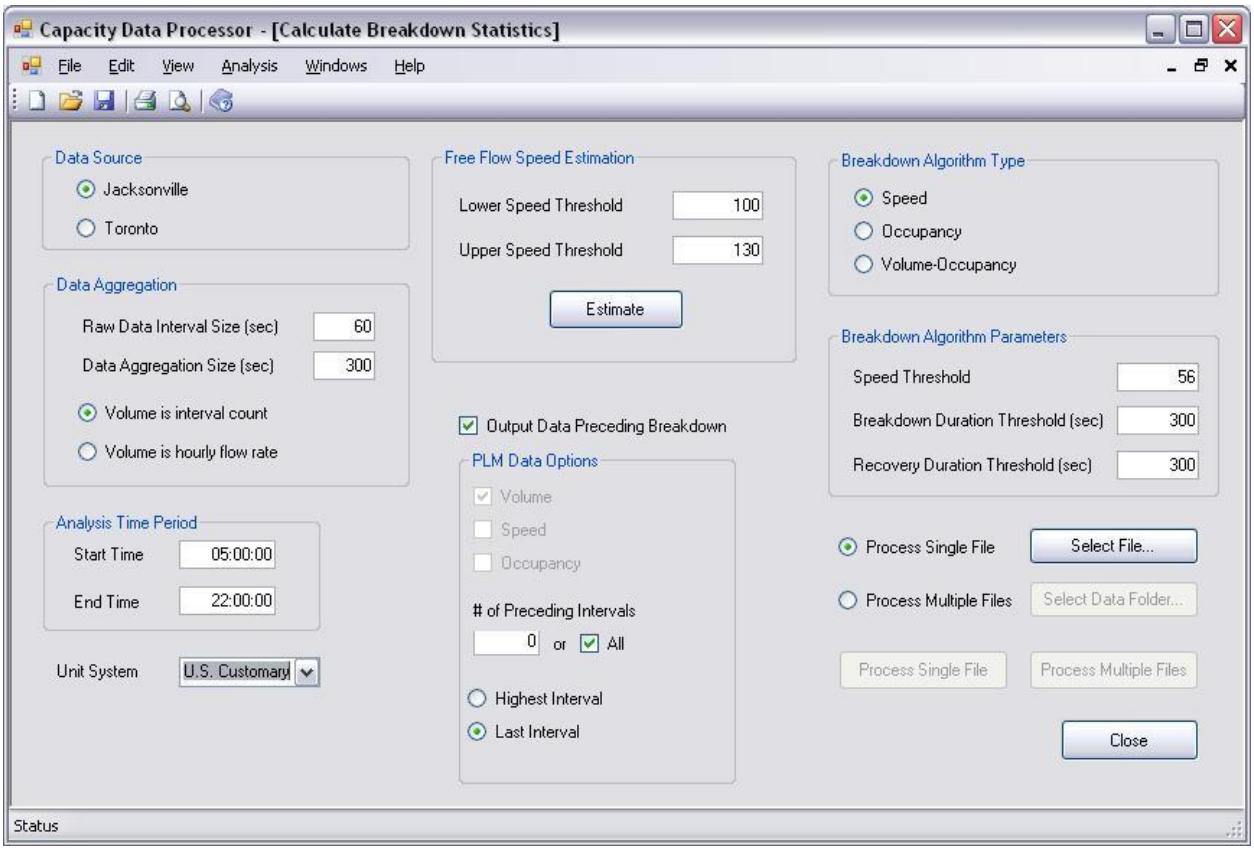


Figure 3-20. Capacity Data Processor utility program user interface

Figure 3-21. Downstream Breakdown Identifier utility program user interface

Table 3-1. Data format for the data obtained from data source

Date	Time	CDWID	FwySpd	FwyVol	FwyOcc	SpdCV	VolRatio	SpdRatio
11/1/2007	0:00:00	210371	58.58	12	1.8	0.71	1.67	1.04
11/1/2007	0:01:00	210371	61.86	7	1.5	2.12	0	0
11/1/2007	0:02:00	210371	58.31	16	2.2	0.71	2.33	1.04
11/1/2007	0:03:00	210371	59	14	3.8	0	4	1.22
11/1/2007	0:04:00	210371	59.75	16	3.2	3.54	2.67	1.12
11/1/2007	0:05:00	210371	60.14	14	2.6	0	1.25	1.07
11/1/2007	0:06:00	210371	60.44	18	4.3	0.71	1.4	1.06
11/1/2007	0:07:00	210371	60.12	17	2.4	2.83	1.75	1.15
11/1/2007	0:08:00	210371	60.27	11	2	3.54	1.33	1.16
11/1/2007	0:09:00	210371	62.38	21	5.2	3.54	1.33	1.15
11/1/2007	0:10:00	210371	62.75	16	1.8	1.41	3.5	1.08
11/1/2007	0:11:00	210371	63.38	16	1.8	4.24	2.67	1.14
11/1/2007	0:12:00	210371	65.5	10	2	4.24	5	1.19
11/1/2007	0:13:00	210371	62.06	16	2.5	2.12	1.2	1.09
11/1/2007	0:14:00	210371	67.8	15	2	6.36	2.67	1.11
11/1/2007	0:15:00	210371	66.47	15	2	1.41	1.5	1.06
11/1/2007	0:16:00	210371	64.63	8	1.5	0.71	2	1.11
11/1/2007	0:17:00	210371	65.46	13	2	2.83	1.25	1.13
11/1/2007	0:18:00	210371	68.62	13	2.7	0	3	1.21
11/1/2007	0:19:00	210371	64.88	16	3.2	0.71	1.5	1.24
11/1/2007	0:20:00	210371	63.89	18	2.5	3.54	1.4	1.21
11/1/2007	0:21:00	210371	64.18	11	2	0	5	1.25
11/1/2007	0:22:00	210371	65.07	15	2.2	3.54	1.5	1.35
11/1/2007	0:23:00	210371	61.68	22	3	7.07	2	1.23
11/1/2007	0:24:00	210371	63.57	14	1.8	1.41	2.33	1.09
11/1/2007	0:25:00	210371	65.38	13	4	2.12	6	1.1
11/1/2007	0:26:00	210371	63.74	19	2.7	0.71	2.67	1.04
11/1/2007	0:27:00	210371	62.21	14	2.2	2.12	1.5	1.07
11/1/2007	0:28:00	210371	61	6	1.8	0	0	0
11/1/2007	0:29:00	210371	62.55	11	2.8	0.71	1.33	1.1
11/1/2007	0:30:00	210371	63.56	16	4.6	0.71	2.67	1.04

Table 3-2. Final selected sites and site description

Site ID	Site Description and Location	Direction	Freeway	Speed[1] (mph)	Data[2]	Type[3]
3 Lanes						
T1	East of NW 57 Avenue	EB	SR-826	55	Aug08-Jan09	A
T2	East of NW 57 Avenue	WB	SR-826	55	Aug08-Jan09	A
T3	East of NW 67 Avenue	EB	SR-826	55	Aug08-Jan09	A
T4	East of NW 47 Avenue	EB	SR-826	55	Aug08-Jan09	A
T5	East of Wymore Rd	WB	I-4	55	Mar08-Nov08	A
T6	East of SR 436	EB	I-4	55	Mar08-Nov08	A
T7	East of Wymore Rd	EB	I-4	55	Mar08-Nov08	A
T8	Between Baymeadows and Butler Blvd	NB	I-95	70	Jul07-Feb08	A
T9	North of Butler Blvd	NB	I-95	70	Jul07-Feb08	A
T10	South of Spring Glen Road	NB	I-95	70	Jul07-Feb08	A
T11	Between Old St. Augustine Rd and I-95	NB	I-295	70	Jul08-Feb09	A
4 Lanes						
F1	South of NW 170 ST	NB	I-75	70	Aug08-Jan09	A
F2	East of NW 27 Avenue	NB	SR-826	55	Aug08-Jan09	B
F3	West of Central Florida Parkway	EB	I-4	70	Mar08-Nov08	A
F4	At Kennedy Blvd	WB	I-4	55	Mar08-Nov08	C
F5	North of Philips Highway	NB	I-95	70	Jul07-Feb08	B
F6	North of I-295S	NB	I-95	70	Jul07-Feb08	B
5 Lanes						
FV1	At SW 26th Terrace/New River	WB	I-595	70	Aug08-Jan09	C

[1]Speed limit, [2]Data available for analysis, [3]Type of basic freeway segment

CHAPTER 4
DATA ANALYSIS AND RESULTS

This chapter describes the application of the analysis methods and the respective results. The analysis methods that are used to estimate the capacity values are termed as VAM capacity estimation from the Van Aerde model, stochastic capacity estimation from the PLM, and average maximum flow rate estimation method. In the end, the capacity values from all the three methods are provided, followed by comparisons of capacity estimates from these three capacity estimation methods with the capacity values provided by HCM.

VAM Capacity Estimation

The VAM capacity estimation method is implemented through the Traffic Stream Calibration Software, SPD_CAL.exe, by Rakha (2007). As described in the methodology, it is an iterative heuristic procedure which calibrates the parameters for the Van Aerde model (1995). The parameters that are obtained from this program, on the basis of the traffic data input file, are the free-flow speed, speed at capacity, capacity, and jam density. The inputs to this program are the flow rates in veh/h, the speeds in km/h and the density in veh/km.

For all the selected sites across Florida, the input files with the flow rates, speeds and density were prepared. The data at the basic freeway segment detector location is used for creating these files. The data used for running this analysis method is similar to that used for the stochastic capacity estimation method. To avoid erroneous data entries in the datasets, all flow values more than 2700 veh/h/ln were eliminated. It should be noted that the minimum flow rate for this input file was considered to be 100 veh/h/ln, and the minimum speed was considered to be 12 mi/h, based on guidance from the software program. Data entries with flow rates less than 100 veh/h/ln were deleted from the datasets. The Van Aerde model curves fitted function to speed-flow data points for two of the randomly selected study sites are provided in Figures 4-1

and 4-2. The calibrated parameters obtained from the Traffic Stream Calibration program, for all sites, are shown in Table 4-1.

Stochastic Capacity Estimation

This section describes the various steps performed to implement the second analysis method, the stochastic capacity estimation method, to estimate the capacity values. First, the applicability of the PLM on basic freeway segments is described, followed by a description of the various steps performed for this analysis method.

Applicability of PLM

Brilon et al. (2005) used the PLM approach for a large number of sites on German freeways. However, the PLM was applied only at sites that had a bottleneck resulting from a lane drop. Brilon et al. (2005) did not apply the PLM at sites where breakdown was the result of friction created at on-ramp merges.

The applicability of the PLM depends on whether a congestion event can occur at a basic freeway segment detector location. Therefore, a small part of the large datasets for two sites was analyzed to check the applicability of the PLM. For each site, breakdown events were observed at all the detector locations along the freeway segment. Breakdown events were identified for the detector station at the location of interest and the detector station immediately downstream of the location of interest, for each day of the selected month. It was observed that in most cases, the breakdown at the downstream of location of interest occurred first and in few cases, the breakdown at the location of interest occurred first. For the cases where the breakdown occurred first at the location of interest, the incident data, obtained from Florida Highway Patrol, were checked and were found not to affect the breakdown at the location of interest.

This small experiment shows that the breakdown can occur at the location of interest and not due to an incident or by the spillback of the queue from the downstream location. In contrast

to the PLM applied by Brilon et al. (2005), only those sites were considered that had a lane drop and the study sites at basic freeway segments were not considered. Therefore, it can be concluded that the PLM is applicable to basic freeway segments for estimation of capacity values. The description of the various steps for this analysis is provided next. These steps are also provided in Elefteriadou et al. (2009), with any significant deviations as implemented for this study denoted.

Determination of Speed Threshold Values

As the PLM is applicable to basic freeway segments, the first step in applying the stochastic capacity estimation method is to determine the speed threshold value. This value is one of the key inputs to the capacity data processor. The capacity of the freeway segments is estimated by identifying the breakdown events at the location of interest or at the basic freeway segments. These breakdown events are defined as breakdown events occurring at the location of interest, and not due to a breakdown at a downstream location. To identify these breakdowns at location of interest, the breakdown events at both the location of interest and downstream location were identified. The speed threshold values were determined for both locations on the freeway segment. The speed threshold value for both locations is determined separately for each freeway segment.

The threshold value is determined on the basis of a speed time-series plot where speed and vehicle count are tabulated in 1-minute intervals for the selected site. The vehicle count is then expressed as an equivalent hourly flow rate and the average speed across all lanes is determined using the volume-weighted average speed of all vehicles crossing the particular detector station. The speed and flow rate data are plotted in time-series over a specific period of time. After extensive examination of the available data, one month of data were used for the time-series plots. For these plots, time is displayed on the x-axis and speed on y-axis. The speed threshold is

then determined visually, on the basis of speed drop for a period of five minutes. If the average speed across the freeway section drops below this threshold value for a five minute of period, a breakdown event will occur. In Elefteriadou et al (2009), a breakdown event is identified at a detector location if the speed drops by 10 mi/h for ten minutes. The speed time series plot for two of the selected sites at its upstream locations are shown in Figure 4-3 and Figure 4-4. Table 4-2 summarizes the speed threshold values used for the analysis at the upstream and downstream locations.

Identification of Breakdown Events

After the speed threshold values for all the sites are determined, the next step is to load the prepared data into the CDP for identification of the breakdown events. The speed threshold values as obtained from the speed time-series plots are used as an input to the CDP. All the breakdown events at the desired location and at the immediate downstream location are identified. These results are then loaded into the DBI, which filters out all the breakdown events at the desired location that were due to a breakdown event at the downstream location.

The next step is to check for erroneous data in the output file obtained from the DBI. All flow rates more than 2700 veh/h/ln are excluded from the data file to obtain another dataset. In Elefteriadou (2009), all flow rates more than 3000 veh/h/ln are excluded. These erroneous data are usually a result of detector malfunctions and are considered as the outliers in the dataset. Also, all the flow rate values preceding a breakdown event, or in the B-set, are checked for any flow rates less than 1000 veh/h/ln are excluded from the datasets. At these flow rate values, the breakdown events are believed to have occurred due to an incident at the freeway segment. The final dataset obtained after excluding the erroneous data contained only the flow rates from the uncongested traffic flow regime. These datasets are used then for the remainder of the analysis.

PLM and Speed-Flow Curves

The PLM is then applied to the datasets obtained from the previous step. This method gives the PLM curves considered to be the capacity distribution function. According to Brilon et al. (2005), it was observed that the Weibull distribution provides the best fit of the PLM curves. Figure 4-3 and Figure 4-4 show that the Weibull distribution curves provide a good fit to the PLM curves, for two sites selected randomly from all 18 analysis sites. Again, the Weibull distribution is considered to be the capacity distribution function. The Weibull parameters are then estimated, for all sites, with the log-likelihood estimation method.

After the PLM and Weibull curves are determined, the speed-flow data points are also plotted. It should be noted that only uncongested flows are used to plot the speed-flow data points. All three plots, i.e., PLM curves, Weibull distribution curves and speed-flow data points are then superimposed with each other in a single graph as provided in Figure 4-5 and Figure 4-6 for all the sites as per Geistefeldt (2008). The left y-axis represents the speed values in mi/h, the right y-axis represents the breakdown probability for the Weibull distribution, and the x-axis represents the flow rate in veh/h/ln.

Estimating Capacity Values

The next step in the data analysis is to superimpose only the speed-flow data points and the Weibull curves. After the superimposition of these two plots, there is a need to find an appropriate value of the breakdown probability from the capacity distribution function, $F(q)$ for all the selected sites. A probability value from the distribution function is chosen which would reasonably represent the occurrence of a breakdown event and would best estimate the capacity for the selected freeway segment. The different approaches that can be used to estimate the capacity values from the breakdown probability function are described next.

First, a capacity value can be identified by visually selecting a point from the plot of speed-flow data points that corresponds to the highest flow rate that is within a critical mass of data points (higher flow rates that are within a sparse area of the speed-flow plot are not considered). However, the results from this method can be significantly biased for a couple of reasons: 1) the size of the data points and resolution of the plot can affect how dense an area of plotted points appears, and 2) the selection of the capacity point to the specific observer's interpretation. Thus, this approach is not recommended. Second, a probability value can be chosen from the capacity distribution function that corresponds to the capacity estimates as determined from the VAM capacity estimation method. Third, for all the selected sites, an average can be taken of all the maximum flow rates that occur within ten minutes of breakdown. The average of all these maximum flow rates is calculated for the respective study sites to find a breakdown probability that corresponds to the average maximum flow rate. However, only flow rates greater than 1500 veh/h/ln were considered for this analysis. This flow rate is termed as the maximum pre-breakdown flow rate. The breakdown probability values from these two approaches, VAM capacity estimation and maximum pre-breakdown flow rate, were determined and are represented by $F_c(q)$. Table 4-3 provides the $F_c(q)$ values for all the sites for each approach.

The next step in the data analysis was to determine an appropriate breakdown probability value that would provide a reasonable estimate of the capacity for the analysis sites. To find this appropriate value, the average of all the $F_c(q)$ values under the respective approach was calculated. The range of the values for $F_c(q)$ under the VAM capacity estimate approach was found to be 0.7% to 10.67% and the respective average of all values was 3.98%. Similarly, the range of the values for $F_c(q)$ under the maximum pre-breakdown flow rate approach was found to be 0.24% to 9.09%, and the respective average of all the values was 3.27%. Geistefeldt (2008)

used the average breakdown probability value under $F_c(q)$ to determines the capacity for a site. However, a value of 4% or 4th percentile is used for this study to estimate the capacity values for all the selected sites. The flow values are then determined after the 5th percentile value is traced back onto the speed-flow data points. This flow is taken as the capacity estimate for the respective site from the stochastic capacity estimation method. Table 4-3 gives the $F_c(q)$ values and the capacity estimates for all the analysis sites, based on a 4th percentile of the capacity distribution function.

Another approach to estimate the capacity is to determine the 50th percentile $F_c(q)$ value as per Brilon et al. (2005). The capacity estimates for the 50th percentile (i.e., mean value) of all the Weibull curves are also provided in Table 4-3. The graphs for the capacity estimation from PLM for all analysis sites are shown in Figure 4-7 to Figure 4-24 with the speed-flow data points plots for uncongested flow rates, speed-flow data points for all the observed data points and the Weibull curves. The next section describes the third, and last, analysis method used for estimating capacity values.

Average Maximum Flow Rate Capacity Estimation Method

The average maximum flow rate capacity estimation is a simple method that was developed to estimate capacity values without the complications of identifying breakdowns and/or estimating complex mathematical functions. Two variants of this approach were tested and are discussed below.

The first step was to aggregate the flow rate data into five-minute intervals. The datasets used for this analysis are for the detector station at the location of interest of that freeway segment. All the available data sets at the respective detector station location were considered for this analysis method. The second step was to convert the five-minute flow rate data to hourly

flow rates (i.e., multiply by 12). The third step was to sort the data from highest flow rates to lowest flow rates. The remaining steps are specific to each of the two variants of this approach.

For the application of the first variant, the average of the top 3% and the top 5% of the highest flow rates were taken. These average flow rates are taken as the capacity estimates for the freeway segments. These capacity values were compared with the capacity values obtained from VAM capacity estimation and stochastic capacity estimation. Table 4-4 tabulates the values of the top 3% highest flows, the top 5% highest flows, the % error in difference of capacity estimates in comparison with VAM capacity estimates and PLM capacity estimates and the average number of breakdowns per day. On the basis of the error in difference in the respective capacity values for a particular site, it was observed that the average of the top 5% highest flow rates gives capacity estimates with less error as compared with VAM capacity estimates and PLM capacity estimates.

In the second variant of this approach, the analysis performed is based on the maximum flow rate observed for the detector station at the location of interest for the freeway segment. The aim of this approach is to calculate the average of all flow rates within a certain percentage of the maximum flow rate observed for the freeway segment of interest. The first step performed in this approach is to find flow rate below the maximum flow rate, such that the average of all flow rates between this maximum flow rate and lower flow rate will match the capacity estimates from the VAM capacity estimation method and stochastic capacity estimation method. The percent value of the maximum flow rate that equals the lower flow rate is then calculated. The percentage values from this analysis with the capacity estimates from the VAM capacity estimation method and the stochastic capacity estimation method are tabulated in Table 4-5. The

average of these percentage values for the VAM capacity estimation method and the stochastic capacity estimation value were found to be 34.89% and 30.29%, respectively.

The percentage values for all the selected sites were compared to each other and on the basis of the average of percentage values, the analysis was performed on the basis of 30% and 35% of the maximum flow rate observed for the freeway segment. The respective lower value that corresponds to 30% and 35% of the maximum flow rate was calculated. The average of all flow rates that are within the 30% and 35% of the maximum flow rate is calculated and is taken as the capacity of the freeway facility. The percentage difference between these capacity estimates and those from the VAM capacity estimation method and stochastic capacity estimation method were calculated. Table 4-6 tabulates the lower flow rate that corresponds to the 30% and 35% of the maximum flow rate observed on the freeway segment, the capacity estimates on the basis of these two lower flow rate, the percentage difference of these capacity estimates with the capacity estimates from the VAM capacity estimation method and stochastic capacity estimation method and the average number of breakdowns per day at each analysis site.

The next step in this approach was to find the minimum time period for which this approach can be implemented. The time periods considered for this step were: three months, two months, one month, and two weeks of data for each analysis site. The start and end dates for each of the time periods were randomly selected from the available datasets, but generally avoiding major holiday periods. This analysis was also based on five-minute flow rates converted to hourly flow rates. For each analysis site, and for each time period, the average of the top *x*% of the highest flow rates was calculated. The value of *x* was determined based on the capacity estimate obtained from the VAM capacity estimation method for the site. In other words, a percentile value was chosen such that the average of these top *x*% flow rates matched with the

VAM estimated capacity value. The *x* value was also obtained relative to the stochastic capacity estimation method results. The *x* values obtained were determined for all the time periods and were compared to each other. The percentage differences in the *x* values for different selected analysis periods were also calculated to check the consistency of datasets. Table 4-7 and Table 4-8 provides all the *x* values for each analysis period and the percentage difference in *x* values as compared in different analysis periods.

Comparisons and Results

This section compares the capacity values provided by the HCM (2000) with the capacity estimates obtained from the analysis methods used in this study. The capacity values from the HCM (2000) are dependent on the free flow speed of the freeway segment, so the capacity value for a freeway segment with a free flow speed of 70 mi/h is given as 2400 pc/h/ln, and the capacity value for a free flow speed of 55 mi/h is given as 2250 pc/h/ln. The capacity estimates from HCM (2000), VAM capacity estimation method, stochastic capacity estimation method and the average maximum flow rate analysis are presented in Table 4-9. It should be noted that the capacity values provided in the HCM (2000) are in passenger cars per hour per lane, but for this research study, due to limited data on truck percentages, flow rates were not adjusted for heavy vehicles and are estimated in vehicles per hour per lane.

It was observed that the capacity values provided by the HCM (2000) are higher than the capacity values estimated from the other three analysis methods. From the stochastic analysis, the average of breakdown probabilities from the capacity distribution function that corresponds to VAM capacity estimates and maximum flow rate within ten minutes of breakdown was found to be 3.98 % and 3.27%. These averages closely resemble the average design breakdown probability provided by Geistefeldt (2008) which is 3%. Also, the range of breakdown probability values were observed to be between 1% and 10%, that were comparable to the range

of breakdown probability values, 0.6% to 5.7%, provided by Gesitefeldt (2008). Therefore, a 4th percentile value of the capacity distribution functions, which is the average of the breakdown probabilities rounded up to the nearest integer percentage, is used to estimate the capacity values for the basic freeway segments. The capacity estimates at the 4th percentile value were found to be lower in comparison with the values provided by HCM (2000). On the other hand, it is observed from the other definition of capacity i.e., the expected mean value or at the 50th percentile of the capacity distribution function, the capacity estimates were higher as compared to capacity values given in HCM (2000).

In the maximum average flow rate analysis that from different averaging schemes, the capacity estimates were found to be lower than the capacity values given in HCM (2000). In the first approach, it was observed that the average of the top 5% of highest flow rates were found to provide capacity estimates with less error over the average of top 3% highest flow rates when compared with capacity estimates from the stochastic capacity estimation method and capacity values given in HCM (2000). Similarly, in the second approach, it was observed that the capacity estimates from the average of highest flow rates above the flow rate that corresponds to the 35% of the maximum flow rate observed for a freeway facility, were found to have less error over the average of highest flow rates above the flow rate that corresponds to the 30% of the maximum flow rate when compared with capacity estimates from VAM capacity estimation method, stochastic capacity estimation method and the capacity values given in HCM (2000).

The errors in the difference of capacity estimates between average maximum flow rate analysis and stochastic capacity estimates were found to be higher for sites with fewer number of breakdown events per day. It was observed that if the number of breakdown events per day is less than 0.5, the capacity estimates from the stochastic capacity analysis method were very high

as compared with capacity estimates from the average maximum flow rate analysis. Also, it was observed that capacity estimates from the stochastic analysis on one of the selected sites with 0.04 breakdown events per day was found to be higher than HCM (2000) capacity value Therefore, with fewer breakdown events per day, the stochastic analysis and the average maximum flow rate analysis may not provide realistic capacity estimates..

To find the minimum analysis period, over which the average maximum flow rate analysis is applicable, it was observed that the data remained consistent of between analysis periods of two and three months. The consistency of data was not maintained when comparisons were made for the other time durations. The inconsistencies observed from this approach might be due to inclusion of a high flow rate period within the analysis period. Therefore, the consistency of the averaging schemes is maintained only for data sets of at least two months in duration.

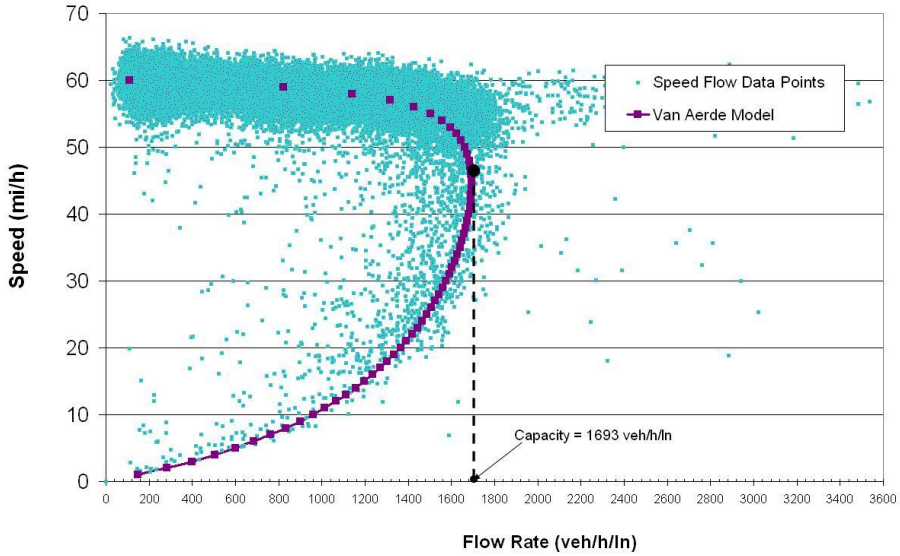


Figure 4-1. Van Aerde Model fit to the speed flow data points for site ID: T1

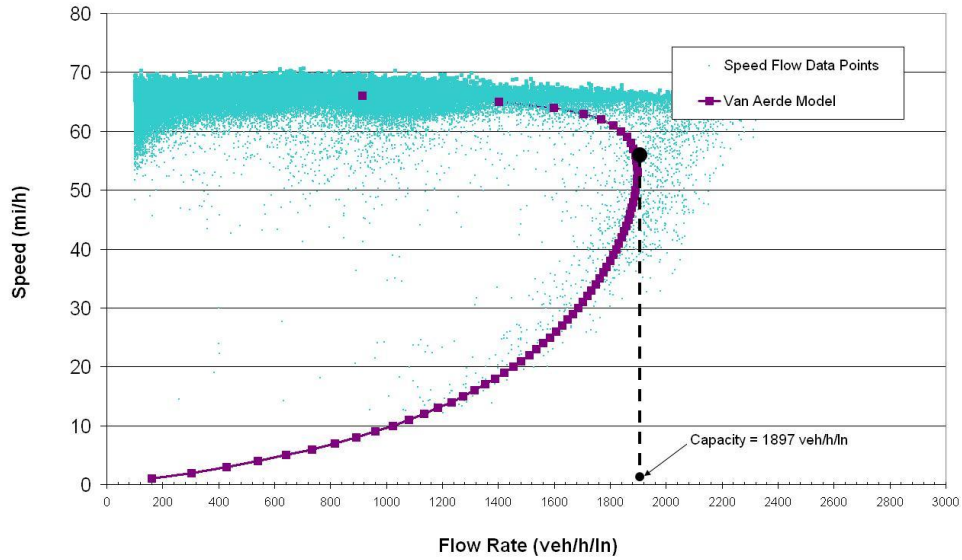


Figure 4-2. Van Aerde Model fit to speed flow points for site ID: T8

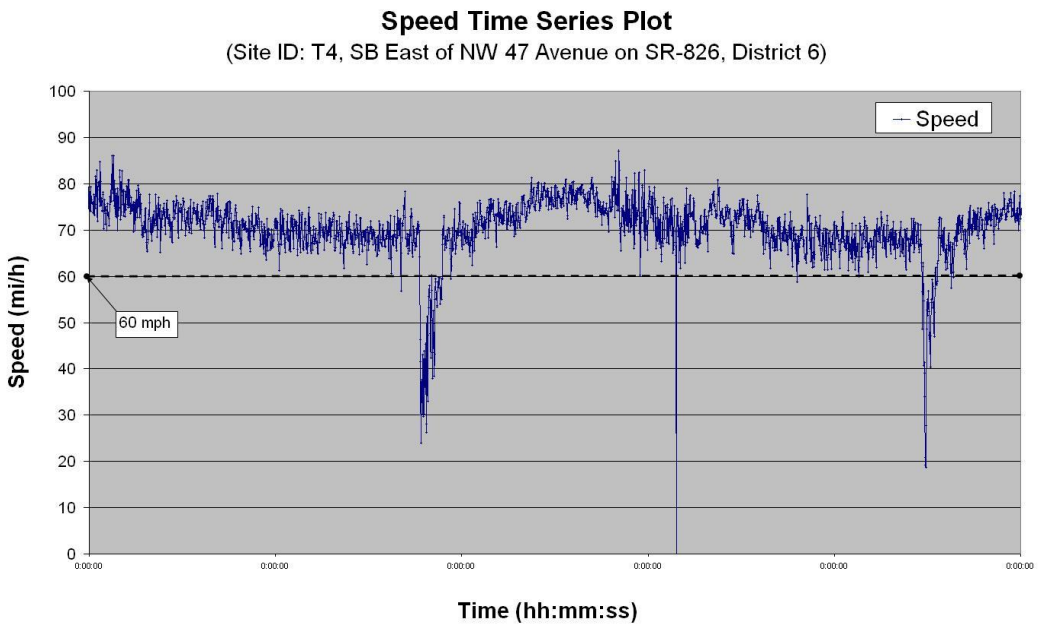


Figure 4-3. Speed time series plot for site T4

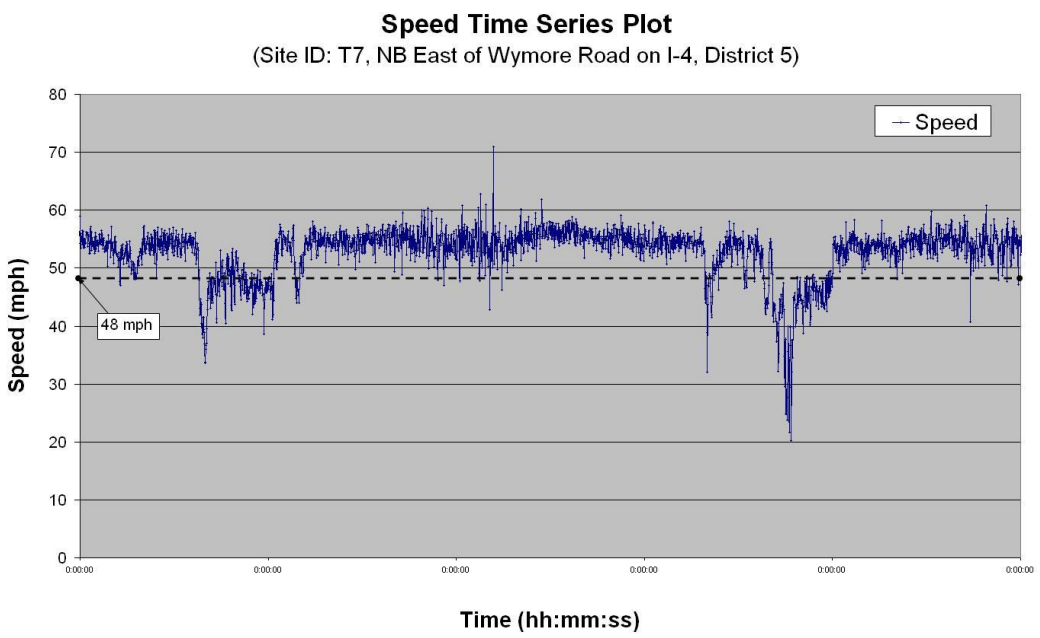


Figure 4-4. Speed time series plot for site T7

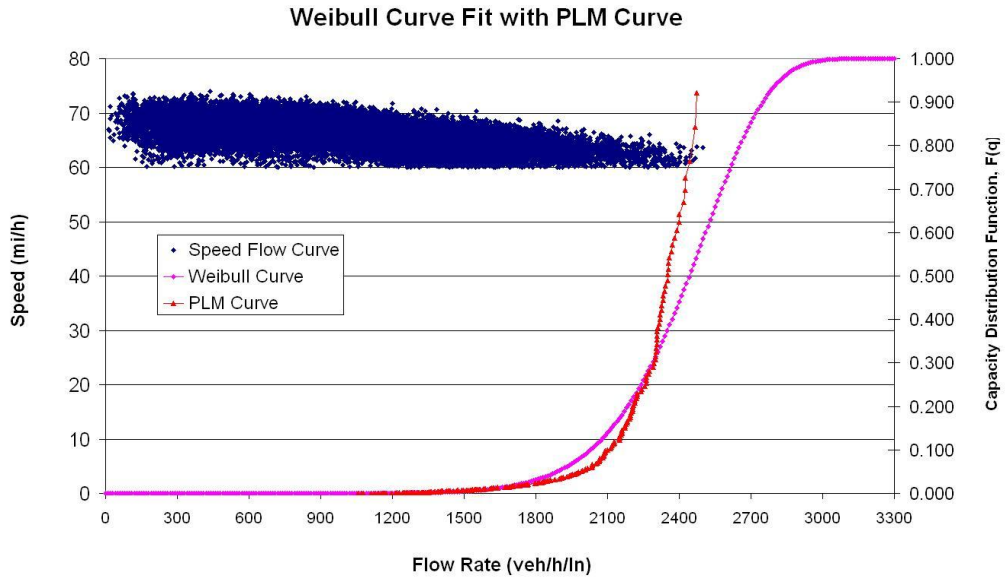


Figure 4-5. Weibull curve fit with PLM curve, Site ID: 210391, NB North of Butler Blvd on I-95, District 2

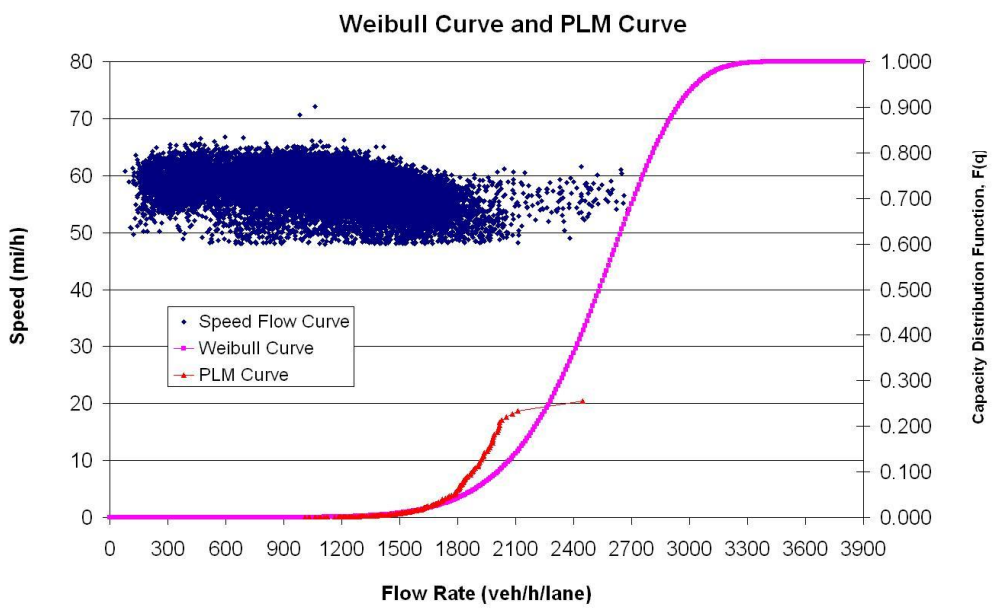


Figure 4-6. Weibull curve fit with PLM curve, Site ID: 610271, East of NW 57 Avenue on SR-826, District 6

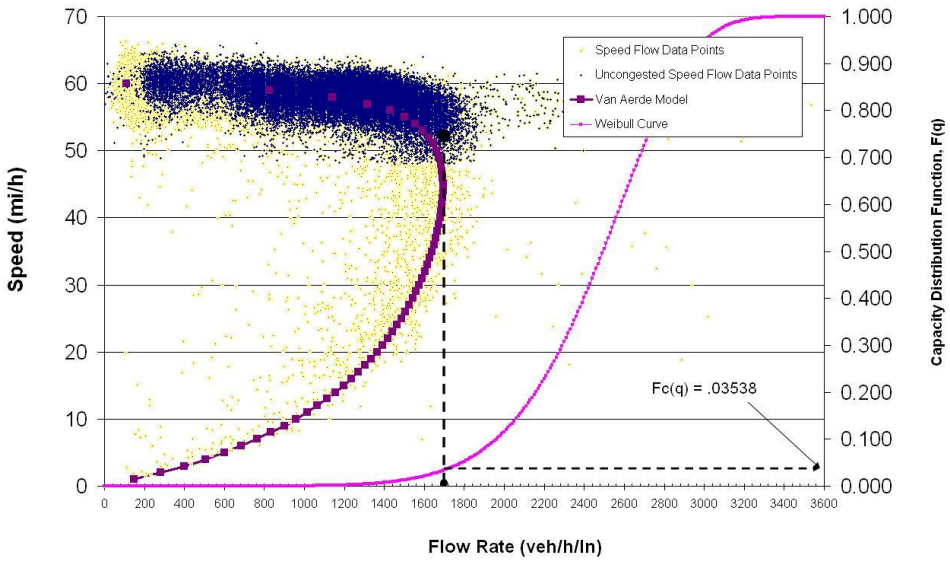


Figure 4-7. Speed Flow, Weibull and Van Aerde Model curves for site ID T1

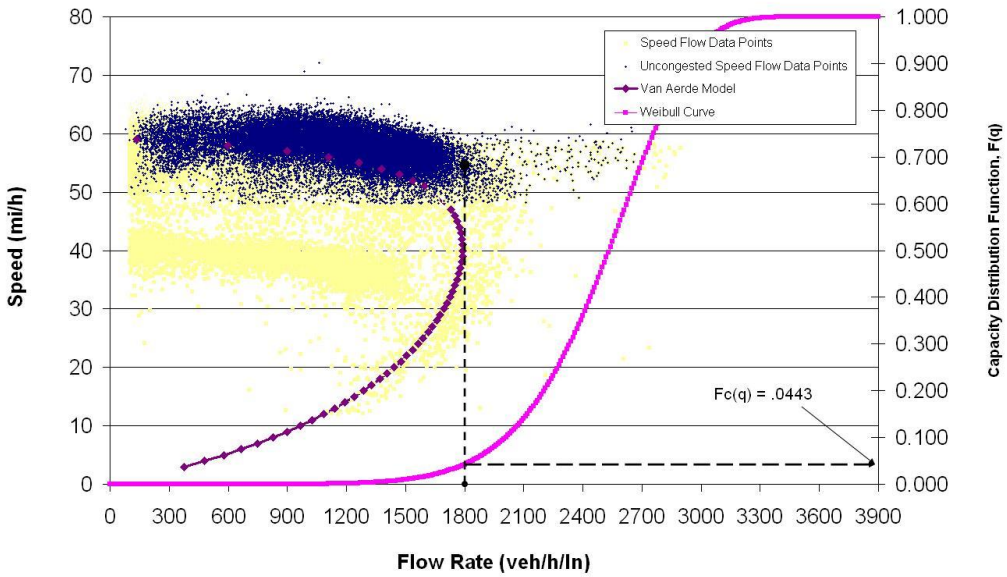


Figure 4-8. Speed Flow, Weibull and Van Aerde Model curves for site ID T2

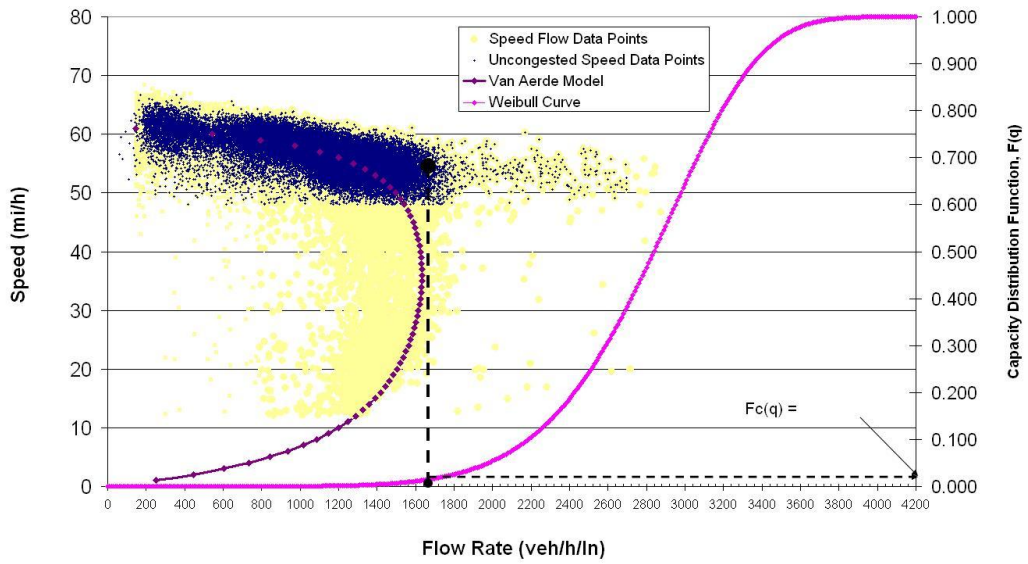


Figure 4-9. Speed Flow, Weibull and Van Aerde Model curves for site ID T3

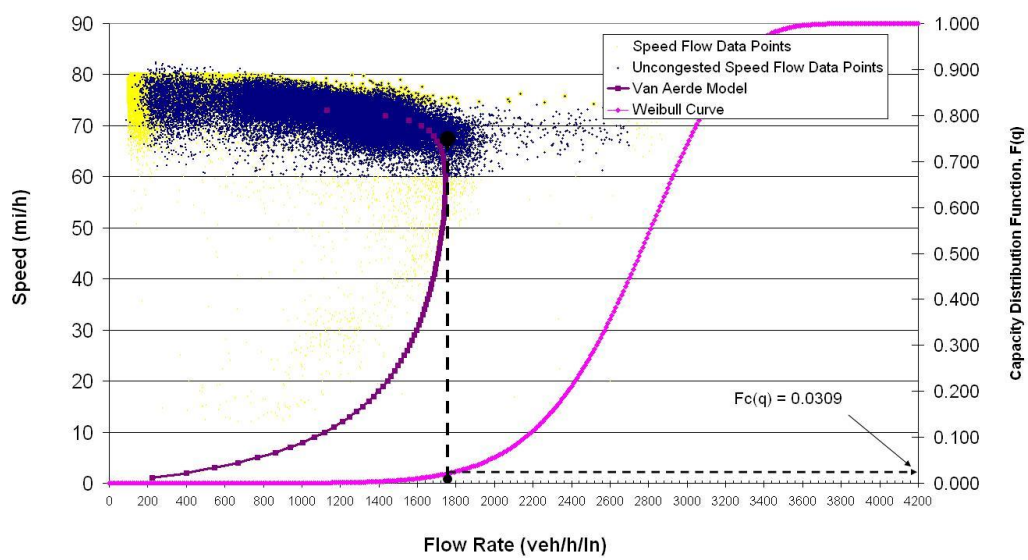


Figure 4-10. Speed Flow, Weibull and Van Aerde Model curves for site ID T4

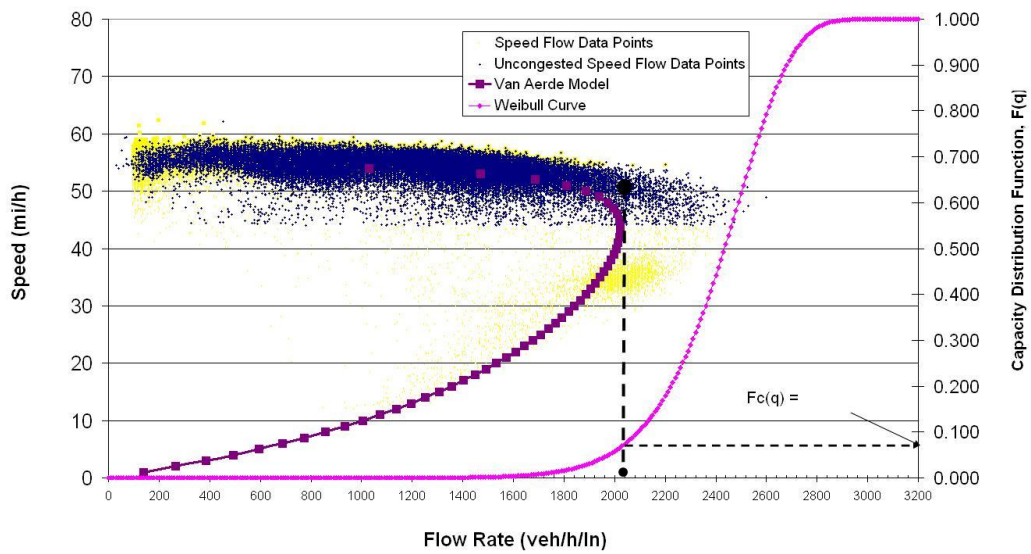


Figure 4-11. Speed Flow, Weibull and Van Aerde Model curves for site ID T5

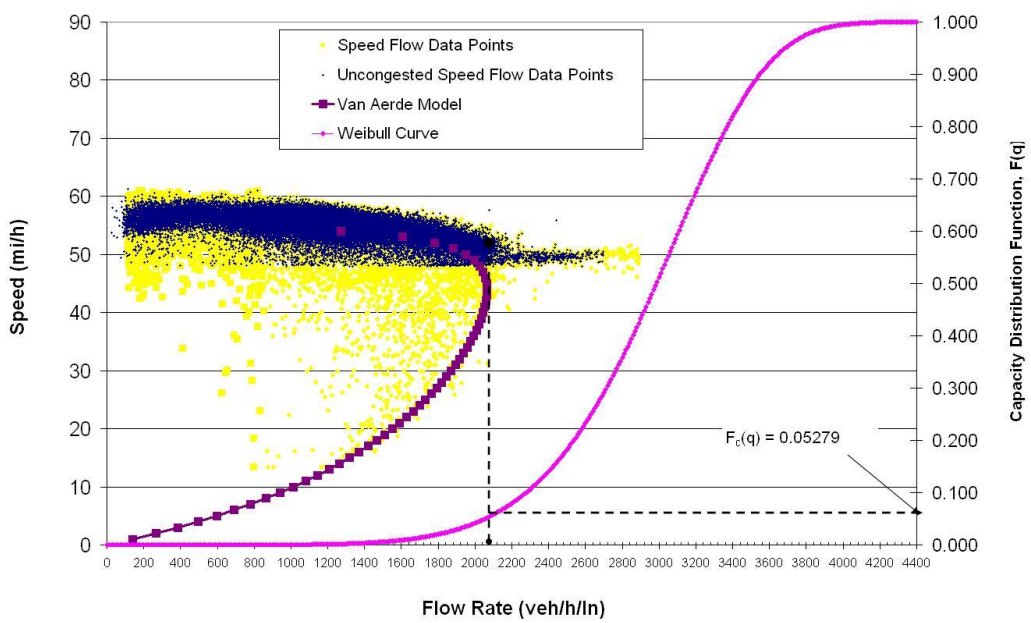


Figure 4-12. Speed Flow, Weibull and Van Aerde Model curves for site ID T6

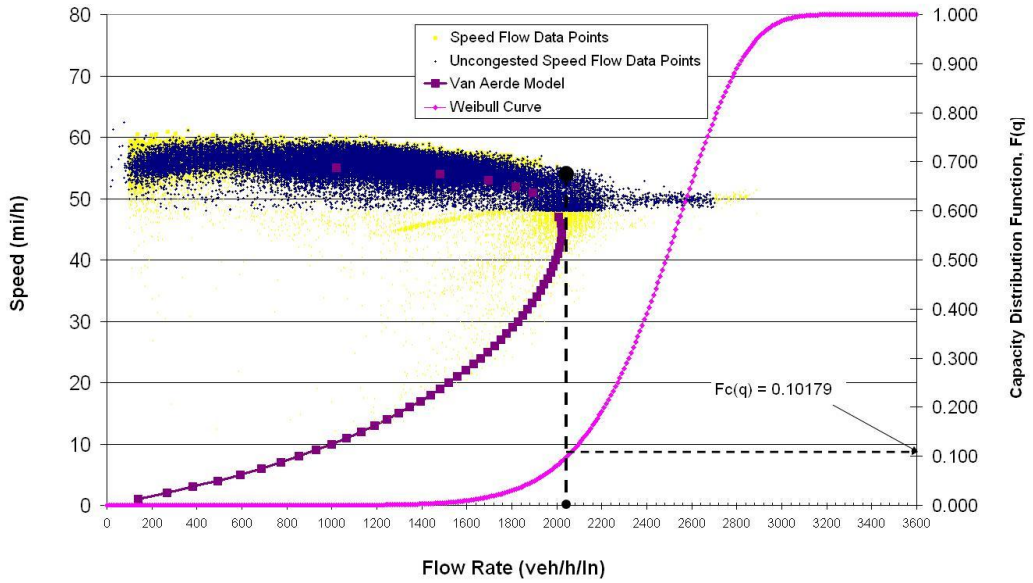


Figure 4-13. Speed Flow, Weibull and Van Aerde Model curves for site ID T7

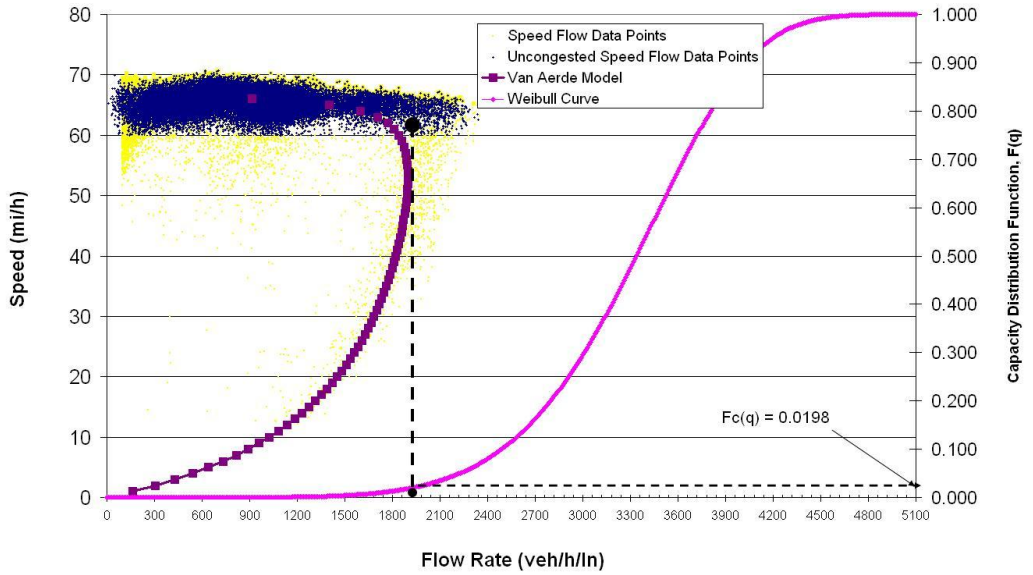


Figure 4-14. Speed Flow, Weibull and Van Aerde Model curves for site ID T8

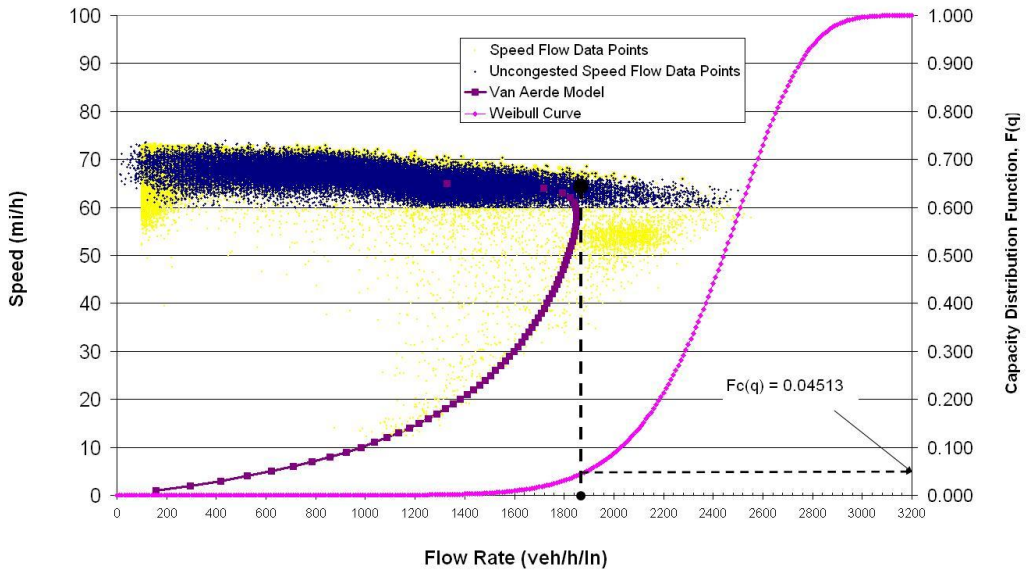


Figure 4-15. Speed Flow, Weibull and Van Aerde Model curves for site ID T9

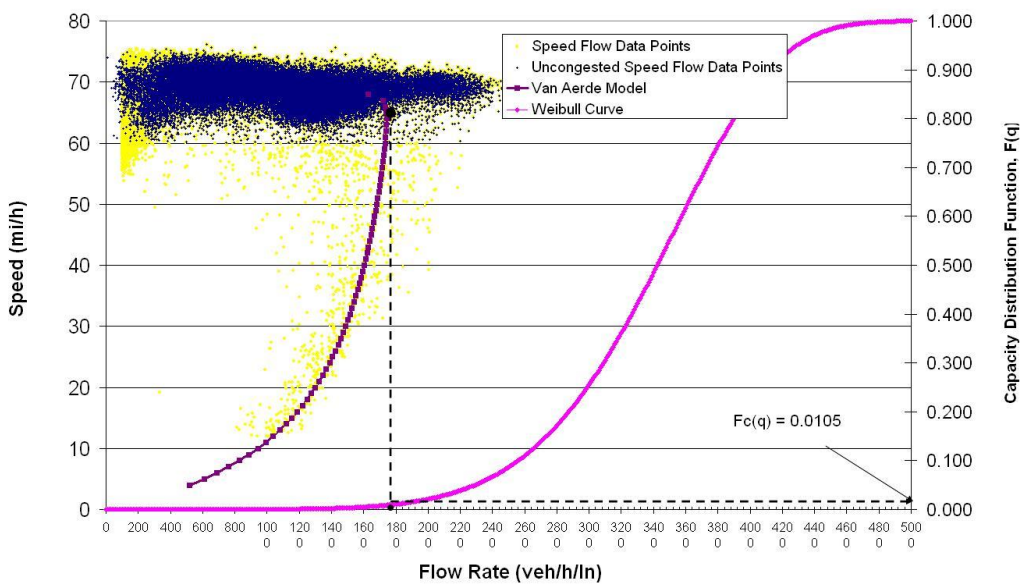


Figure 4-16. Speed Flow, Weibull and Van Aerde Model curves for site ID T10

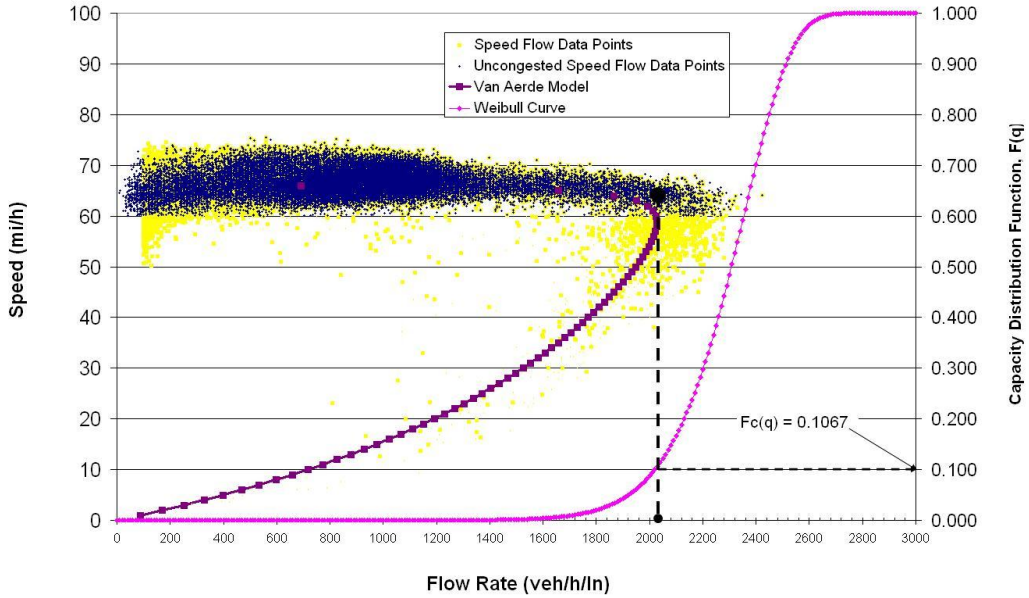


Figure 4-17. Speed Flow, Weibull and Van Aerde Model curves for site ID T11

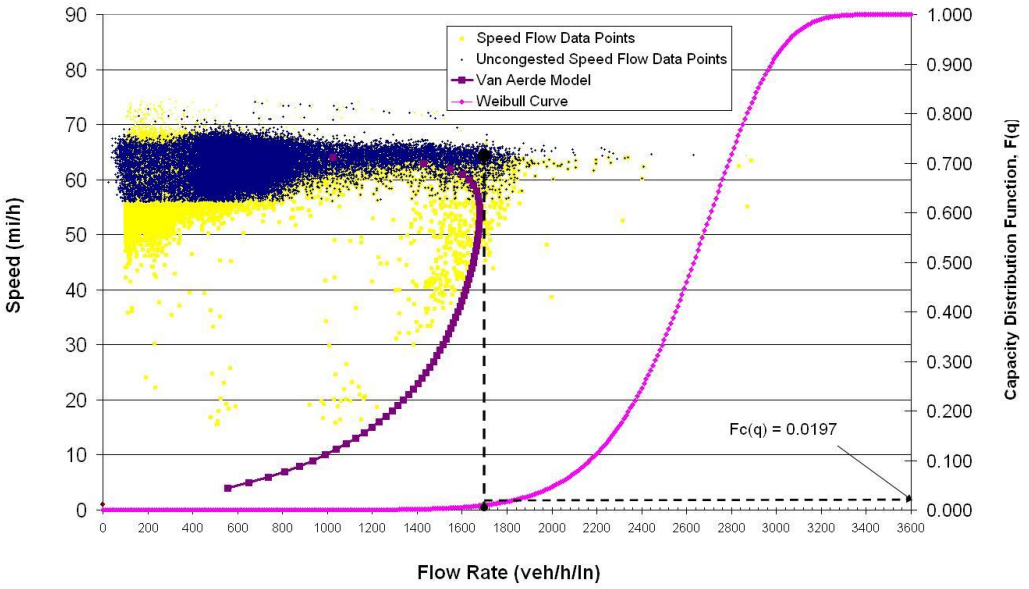


Figure 4-18. Speed Flow, Weibull and Van Aerde Model curves for Site ID F1

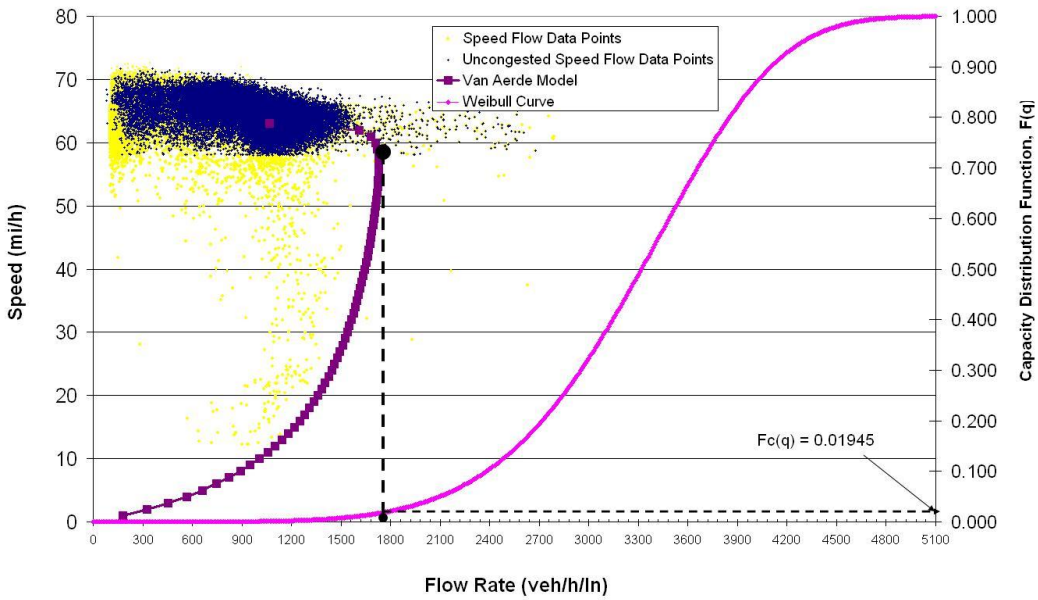


Figure 4-19. Speed Flow, Weibull and Van Aerde Model curves for site ID F2

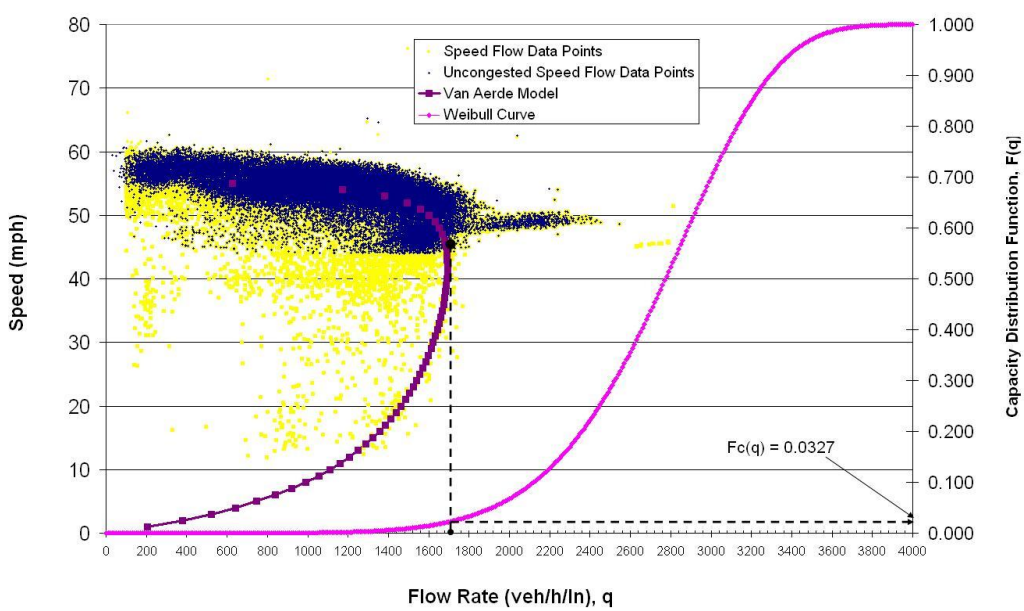


Figure 4-20. Speed Flow, Weibull and Van Aerde Model curves for site ID F3

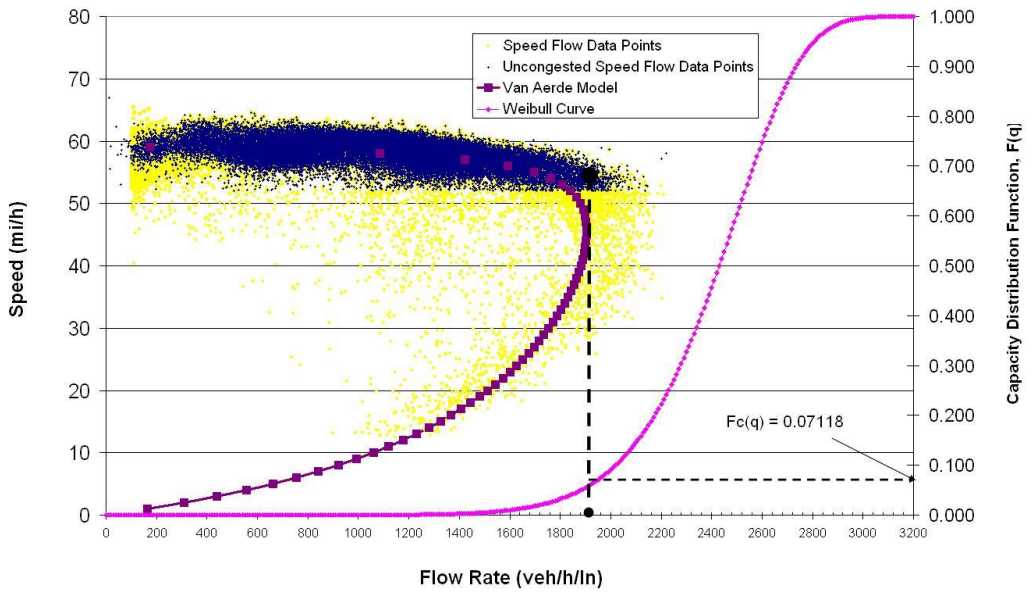


Figure 4-21. Speed Flow, Weibull and Van Aerde Model curves for site ID F4

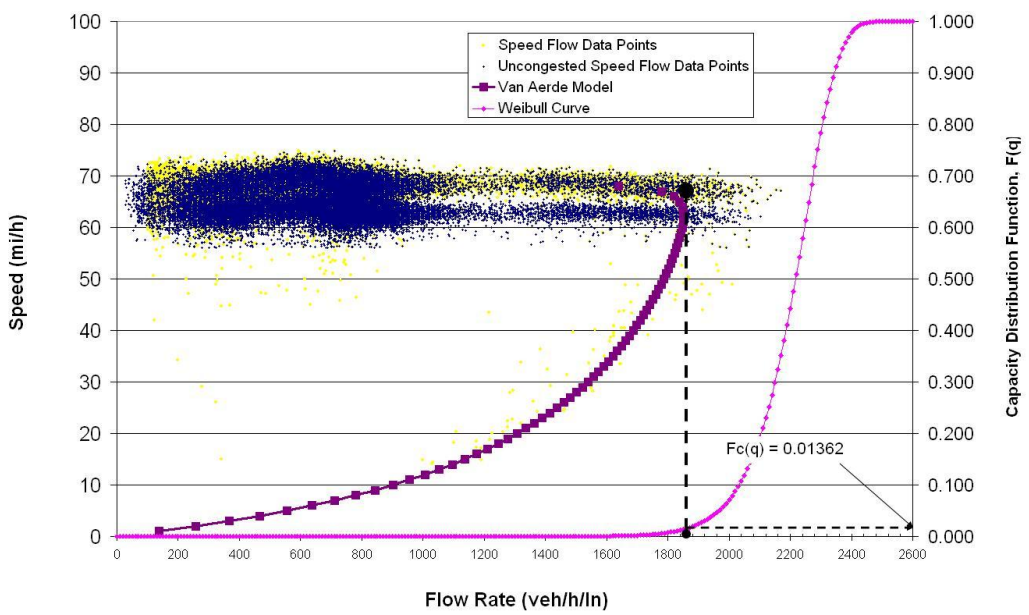


Figure 4-22. Speed Flow, Weibull and Van Aerde Model curves for site ID F5

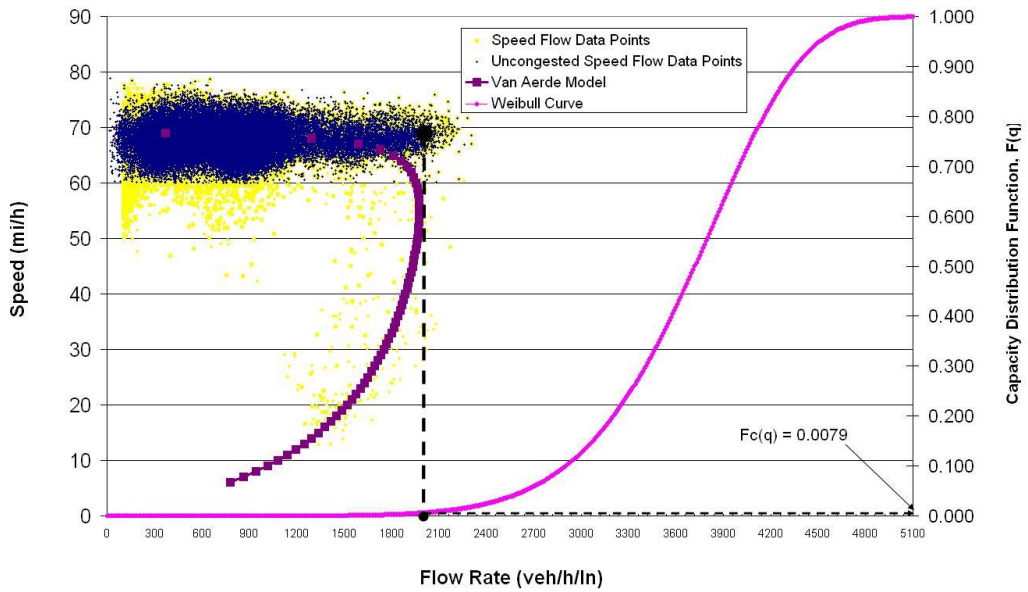


Figure 4-23. Speed Flow, Weibull and Van Aerde Model curves for site ID F6

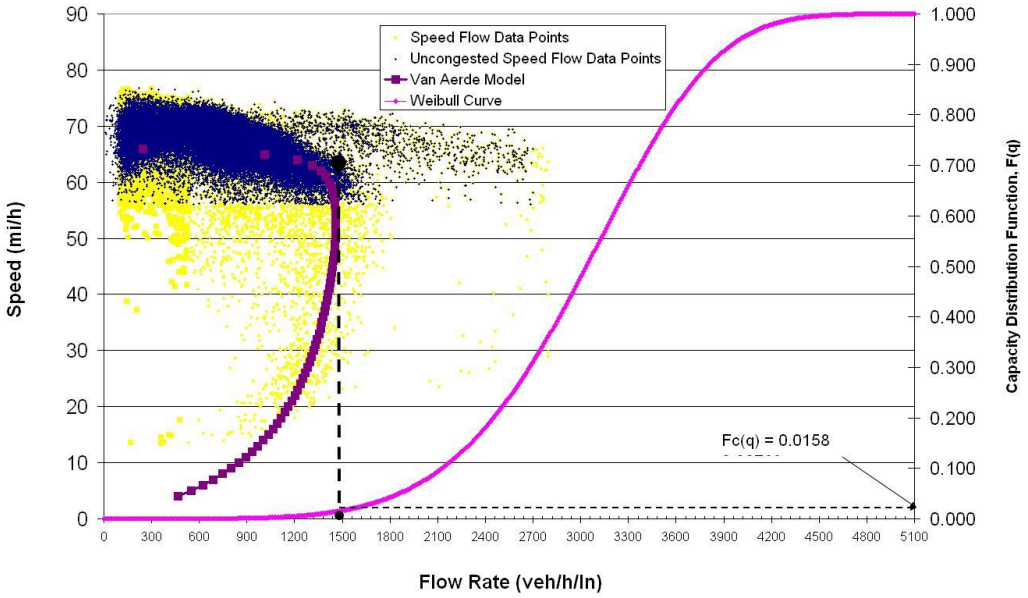


Figure 4-24. Speed Flow, Weibull and Van Aerde Model curves for site ID FV1

Table 4-1. Capacity estimates and other parameters from VAM capacity estimation method

Site ID	Lanes	Capacity (veh/h/ln)	Free-Flow Speed (mi/h)	Speed at Capacity (mi/h)	Jam Density (veh/mi)
T1	3	1693	60.10	44.50	256.47
T2	3	1790	59.23	40.40	229.28
T3	3	1632	61.59	36.67	273.05
T4	3	1746	73.83	58.36	247.95
T5	3	2024	54.88	43.44	144.81
T6	3	2082	56.12	43.51	144.81
T7	3	2021	55.81	44.50	144.81
T8	3	1897	66.63	53.08	172.00
T9	3	1850	65.38	58.30	169.43
T10	3	1743	68.55	64.08	171.36
T11	3	2027	66.19	58.73	194.53
F1	4	1678	64.51	53.82	189.70
F2	4	1729	63.21	56.43	193.40
F3	4	1692	55.44	41.89	228.64
F4	4	1905	59.11	45.56	174.90
F5	4	1851	68.86	62.27	144.81
F6	4	1976	69.17	54.94	186.00
FV1	5	1460	66.13	52.83	156.80

Table 4-2. Speed threshold values for upstream and downstream detectors

			Speed Threshold (mi/h)	
Site ID	Freeway	Lanes	Upstream	Downstream
T1	SR-826	3	48	48
T2	SR-826	3	48	52
T3	SR-826	3	48	60
T4	SR-826	3	60	48
T5	I-4	3	44	44
T6	I-4	3	48	44
T7	I-4	3	48	48
T8	I-95	3	60	64
T9	I-95	3	60	56
T10	I-95	3	60	52
T11	I-295	3	60	60
F1	I-75	4	56	56
F2	SR-826	4	64	56
F3	I-4	4	48	48
F4	I-4	4	52	52
F5	I-95	4	56	60
F6	I-95	4	56	56
FV1	I-595	5	56	60

Table 4-3. Capacity estimates from stochastic capacity estimation method

Site ID	Dir. Lanes	VAM Estimation Approach		Pre-Max Flow Rate[3]		Capacity[1]	Capacity[1]	Breakdowns
		Capacity[1]	$F_c(q)$ %	Capacity[1]	$F_c(q)$ %	50th Percentile	4th Percentile	per day
SR-826 (Mia)	3	1693	3.54	1646	2.64	2494	1789	1.43
SR-826 (Mia)	3	1790	4.43	1729	3.78	2534	1838	1.32
SR-826 (Mia)	3	1632	1.44	1656	1.42	2839	1978	0.55
SR-826 (Mia)	3	1746	3.09	1714	1.74	2757	1969	0.92
I-4 (Orl)	3	2024	7.37	2004	5.95	2434	1976	1.09
I-4 (Orl)	3	2082	5.28	1860	2.54	2980	2054	1.13
I-4 (Orl)	3	2021	10.18	1973	7.14	2485	1900	2.54
I-95 (Jax)	3	1897	1.98	1969	2.32	3338	2224	0.23
I-95 (Jax)	3	1850	4.51	1959	7.14	2442	1890	1.08
I-95 (Jax)	3	1743	1.05	1852	1.29	3424	2290	0.52
I-295 (Jax)	3	2027	10.67	2006	9.09	2308	1920	0.80
I-75 (Mia)	4	1678	1.97	1665	0.80	2632	2015	0.12
SR-826 (Mia)	4	1729	1.95	-[2]	-[2]	3320	2096	0.27
I-4 (Orl)	4	1692	3.27	1577	1.31	2775	1913	0.91
I-4 (Orl)	4	1905	7.12	1728	2.14	2431	1881	0.56
I-95 (Jax)	4	1851	1.36	1902	2.47	2217	1966	0.16
I-95 (Jax)	4	1976	0.79	1766	0.24	3720	2641	0.04
I-595 (Ft.L)	5	1460	1.58	1739	3.56	3036	1857	0.48

[1]Capacity in veh/h/ln

[2]Flow Rates below 1500 veh/h/ln were observed

[3]Flow rate that corresponds to average of maximum flow rate within ten minutes of breakdown

Table 4-4. Capacity estimates from average of top 3% and top 5% highest flows

Site ID	Capacity[1] A	Capacity[2] B	Capacity[3] C	Capacity[4] D	%Change A with C	%Change A with D	%Change B with C	%Change B with D	Breakdowns per day
SR-826 (Mia)	1824	1759	1693	1738	7.74	4.95	3.90	1.21	1.43
SR-826 (Mia)	1866	1787	1790	1787	4.22	4.42	-0.19	0.00	1.32
SR-826 (Mia)	1743	1676	1632	1916	6.78	-9.03	2.68	-12.53	0.55
SR-826 (Mia)	1876	1812	1746	1912	7.48	-1.88	3.81	-5.23	0.92
I-4 (Orl)	2149	2096	2024	1940	6.19	10.77	3.57	8.04	1.09
I-4 (Orl)	2132	2053	2082	1988	2.40	7.24	-1.39	3.27	1.13
I-4 (Orl)	2211	2146	2021	1855	9.41	19.19	6.19	15.69	2.54
I-95 (Jax)	1978	1870	1897	2147	4.27	-7.87	-1.42	-12.90	0.23
I-95 (Jax)	2119	2028	1850	1847	14.54	14.73	9.62	9.80	1.08
I-95 (Jax)	2108	2002	1743	2211	20.94	-4.66	14.86	-9.45	0.52
I-295 (Jax)	2087	2017	2027	1889	2.96	10.48	-0.49	6.78	0.80
I-75 (Mia)	1656	1565	1678	1968	-1.31	-15.85	-6.73	-20.48	0.12
SR-826 (Mia)	1514	1443	1729	2014	-12.43	-24.83	-16.54	-28.35	0.27
I-4 (Orl)	1869	1777	1692	1852	10.43	0.92	5.00	-4.05	0.91
I-4 (Orl)	1961	1908	1905	1840	2.95	6.58	0.16	3.70	0.56
I-95 (Jax)	1826	1730	1851	1946	-1.35	-6.17	-6.54	-11.10	0.16
I-95 (Jax)	1840	1731	1976	2563	-6.88	-28.21	-12.40	-32.46	0.04
I-595 (Ft.L)	1656	1530	1460	1779	13.42	-6.91	4.79	-14.00	0.48

[1]Capacity estimates from average of top 3% highest flow rates (veh/h/ln)

[2]Capacity estimates from average of top 5% highest flow rates (veh/h/ln)

[3]Capacity from VAM estimation method (veh/h/ln)

[4]Capacity from stochastic capacity estimation method (veh/h/ln)

Table 4-5. Threshold values for maximum flow rate in average flow estimation method

		VAM capacity estimation method		Stochastic capacity estimation method	
Site ID	Max. Flow[1]	Threshold Flow[1]	% of Max. Flow	Threshold Flow[1]	% of Max. Flow
T1	2688	1568	41.67	1660	38.24
T2	2684	1643	38.78	1680	37.41
T3	2696	1508	44.07	1720	36.20
T4	2696	1604	40.50	1816	32.64
T5	2528	1836	27.37	1752	30.69
T6	2696	1928	28.48	1900	29.52
T7	2700	1785	33.90	1608	40.45
T8	2352	1664	29.25	2192	6.80
T9	2552	1584	37.93	1624	36.36
T10	2502	1501	40.00	2202	12.00
T11	2424	1856	23.44	1612	33.49
F1	2670	1545	42.13	1836	31.24
F2	2679	1482	44.68	1773	33.82
F3	2691	1545	42.58	1698	36.90
F4	2211	1782	19.40	1734	21.57
F5	2169	1728	20.33	1899	12.45
F6	2307	1866	19.11	-[2]	-[2]
FV1	2690	1229	54.32	1534	42.97

[1]Flow in veh/h/ln,

[2]Threshold value not observed for capacity at 4th percentile

Table 4-6. Capacity estimates from average of flow rates above the flow rate that corresponds to 30th & 35th percentage of maximum flow rate

Site ID	Max. Value	Capacity[1]	Capacity[2]	Flow[3]	Flow[4]	Capacity[5]	% Change	% Change	Capacity[6]	% Change	% Change	Breakdowns per day
		A	B	C	D	E	A with E	A with F	F	B with E	B with F	
T1	2688	1693	1738	1882	1747	2216	30.89	14.18	1933	23.87	8.05	1.43
T2	2684	1790	1787	1879	1745	2058	14.97	7.04	1916	11.97	4.24	1.32
T3	2696	1632	1916	1887	1752	2221	36.09	25.49	2048	12.29	3.54	0.55
T4	2696	1746	1912	1887	1752	2117	21.25	7.62	1879	7.52	-4.57	0.92
T5	2528	2024	1940	1770	1643	1989	-1.73	-6.52	1892	0.66	-4.25	1.09
T6	2696	2082	1988	1887	1752	2043	-1.87	-6.24	1952	-0.54	-4.97	1.13
T7	2700	2021	1855	1890	1755	2067	2.28	-0.84	2004	8.79	5.47	2.54
T8	2352	1897	2147	1646	1529	1887	-0.53	-3.80	1825	-15.15	-17.94	0.23
T9	2552	1850	1847	1786	1659	2011	8.70	3.73	1919	6.40	1.53	1.08
T10	2502	1743	2211	1751	1626	1998	14.63	8.43	1890	-12.75	-17.47	0.52
T11	2424	2027	1889	1697	1576	1959	-3.35	-5.97	1906	2.03	-0.73	0.80
F1	2670	1678	1968	1869	1736	2073	23.54	9.83	1843	2.88	-8.54	0.12
F2	2679	1729	2014	1875	1741	2195	26.95	19.78	2071	4.72	-1.19	0.27
F3	2691	1692	1852	1884	1749	2095	23.82	17.79	1993	9.51	4.18	0.91
F4	2211	1905	1840	1548	1437	1761	-7.56	-12.28	1671	-6.38	-11.16	0.56
F5	2169	1851	1946	1518	1410	1743	-5.83	-8.75	1689	-11.34	-14.09	0.16
F6	2307	1976	2563	1615	1500	1815	-8.15	-11.99	1739	-31.28	-34.15	0.04
FV1	2690	1460	1779	1883	1749	2254	54.38	46.78	2143	21.38	15.40	0.48

[1]Capacity from VAM estimation method (veh/h/ln)

[2]Capacity from stochastic capacity estimation method (veh/h/ln)

[3]Threshold Value for 30% of maximum flow rate (veh/h/ln), [4]Threshold Value for 35% of maximum flow rate (veh/h/ln)

[5]Average value of flow rates above flow rate that corresponds to 30% of max. flow rate (veh/h/ln)

[6]Average value of flow rates above flow rate that corresponds to 35% of max. flow rate (veh/h/ln)

Table 4-7. Average flow analysis *x*-values for VAM capacity estimation method

Site ID	3 Months A (%)	2 Months B (%)	% Change A to B	1 Month C (%)	% Change B to C	2 Weeks D (%)	% Change C to D
T1	14.03	13.32	-5.10	14.37	7.91	40.41	181.25
T2	10.53	10.79	2.47	11.85	9.91	15.89	34.02
T3	4.61	4.33	-6.11	4.07	-5.92	4.28	5.16
T4	1.14	1.17	3.08	1.52	29.85	0.99	-34.87
T5	1.98	2.22	11.93	2.28	2.61	1.81	-20.72
T6	4.84	5.63	16.39	5.81	3.12	5.65	-2.68
T7	1.18	1.08	-7.97	0.82	-24.17	1.29	57.52
T8	7.66	7.52	-1.83	6.97	-7.40	9.80	40.65
T9	9.03	8.45	-6.43	7.63	-9.69	10.89	42.76
T10	2.03	1.94	-4.42	3.60	86.03	7.40	105.34
T11	4.98	4.55	-8.63	4.06	-10.77	5.84	43.84
F1	2.17	2.28	4.83	2.58	13.16	3.47	34.75
F2	10.68	9.31	-12.80	8.77	-5.84	7.11	-18.86
F3	2.58	3.92	51.76	4.39	12.13	3.58	-18.44
F4	7.99	6.16	-22.97	5.50	-10.64	4.85	-11.82
F5	5.94	4.55	-23.40	3.40	-25.27	0.90	-73.53
F6	1.72	1.38	-19.93	1.25	-9.49	0.65	-47.96
FV1	10.02	8.84	-11.81	7.68	-13.12	7.64	-0.52

Table 4-8. Average flow analysis *x*-values for stochastic capacity estimation method

Site ID	3 months A (%)	2 months B (%)	% Change A to B	1 month C (%)	% Change B to C	Two Weeks D (%)	% Change C to D
T1	5.78	4.77	-17.53	4.55	-4.60	3.59	-21.00
T2	4.60	3.43	-25.48	2.62	-23.56	0.77	-70.68
T3	1.57	1.08	-31.17	1.00	-7.79	0.81	-18.67
T4	2.68	1.93	-27.89	1.65	-14.95	1.60	-2.88
T5	9.54	9.24	-3.17	8.63	-6.62	12.14	40.70
T6	1.79	1.71	-4.33	1.30	-24.10	1.99	53.24
T7	14.68	13.78	-6.18	13.09	-4.94	18.03	37.69
T8	0.20	0.21	5.06	0.14	-35.99	0.15	14.04
T9	9.10	9.40	3.33	10.31	9.67	9.71	-5.81
T10	0.57	0.56	-1.16	0.34	-39.24	0.36	5.13
T11	7.41	8.52	14.97	8.86	3.95	8.80	-0.65
F1	0.31	0.47	53.00	0.64	36.24	0.26	-59.43
F2	0.60	0.52	-12.23	0.47	-10.36	0.16	-65.38
F3	0.11	0.02	-84.18	0.04	101.54	0.08	124.00
F4	5.95	5.38	-9.51	4.91	-8.72	6.92	40.76
F5	0.39	0.46	17.91	0.54	17.22	0.42	-21.45
F6[1]	-	-	-	-	-	-	-
FV1[2]	-	-	-	-	-	-	-

[1,2]Flow rates that match the stochastic capacity estimates were not observed

Table 4-9. Comparison of capacity estimates from different analysis methods with HCM 2000

Site ID		Capacity[1]	Capacity[2]	%Change	Capacity[3]	%Change	Capacity[4]	%Change	Capacity[5]	% Change	Breakdowns
		A	B	A with B	C	A with C	D	A with D	E	A with E	per day
SR-826 (Mia)	T1	2250	1693	-24.76	1738	-29.46	1759	-27.91	1933	-14.09	1.43
SR-826 (Mia)	T2	2250	1790	-20.44	1787	-25.91	1787	-25.91	1916	-14.84	1.32
SR-826 (Mia)	T3	2250	1632	-27.47	1916	-17.43	1676	-34.25	2048	-8.98	0.55
SR-826 (Mia)	T4	2250	1746	-22.4	1912	-17.68	1812	-24.17	1879	-16.49	0.92
I-4 (Orl)	T5	2250	2024	-10.04	1940	-15.98	2096	-7.35	1892	-15.91	1.09
I-4 (Orl)	T6	2250	2082	-7.47	1988	-13.18	2053	-9.60	1952	-13.24	1.13
I-4 (Orl)	T7	2250	2021	-10.18	1855	-21.29	2146	-4.85	2004	-10.93	2.54
I-95 (Jax)	T8	2400	1897	-20.96	2147	-11.78	1870	-28.34	1825	-23.96	0.23
I-95 (Jax)	T9	2400	1850	-22.92	1847	-29.94	2028	-18.34	1919	-20.04	1.08
I-95 (Jax)	T10	2400	1743	-27.38	2211	-8.55	2002	-19.88	1890	-21.25	0.52
I-295 (Jax)	T11	2400	2027	-15.54	1889	-27.05	2017	-18.99	1906	-20.58	0.80
I-75 (Mia)	F1	2400	1678	-30.08	1968	-21.95	1565	-53.35	1843	-23.21	0.12
SR-826 (Mia)	F2	2250	1729	-23.16	2014	-11.72	1443	-55.93	2071	-7.96	0.27
I-4 (Orl)	F3	2400	1692	-29.5	1852	-29.59	1777	-35.06	1993	-16.96	0.91
I-4 (Orl)	F4	2250	1905	-15.33	1840	-22.28	1908	-17.92	1671	-25.73	0.56
I-95 (Jax)	F5	2400	1851	-22.88	1946	-23.33	1730	-38.73	1689	-29.63	0.16
I-95 (Jax)	F6	2400	1976	-17.67	2563	6.36	1731	-38.65	1739	-27.54	0.04
I-595 (Ft.L)	FV1	2400	1460	-39.17	1779	-34.91	1530	-56.86	2143	-10.71	0.48

[1]Capacity as provided by HCM 2000 (veh/h/ln)

[2]Capacity as estimated from VAM capacity estimation method (veh/h/ln)

[3]Capacity as estimated from stochastic capacity estimation method (veh/h/ln), [4]Capacity as estimated from average of top 5% highest flow rates (veh/h/ln)

[5]Capacity as estimated from average of flow rates between maximum flow rate and flow rate that corresponds to 35% of maximum flow rate of maximum flow rate (veh/h/ln)

CHAPTER 5
SUMMARY AND CONCLUSIONS

Three methods to estimate the capacity values of basic freeway segments for Florida freeways were investigated and compared to each other: the Van Aerde Model (VAM), stochastic capacity estimation based on the PLM, and the average maximum flow rate. These methods were applied to 18 freeway sites across Florida, with each site experiencing frequent congestion. The STEWARD database server was used to obtain the traffic flow data used in the analysis.

The Traffic Stream Calibration software (Rakha, 2007) was used to generate the VAM capacity estimates. The PLM approach, as proposed by Brilon et al. (2005), was used to generate the stochastic capacity estimates. This approach also consisted of generating Weibull capacity distribution functions and comparing these curves to plotted speed-flow data points to identify appropriate breakdown probability percentile values, and corresponding capacity values. The average maximum flow rate method was developed as a simple alternative to the previous two more complicated methods for estimating capacity. This method consists simply of taking the average of a certain percentage of the highest recorded flow rates. The minimum amount of data (from a time perspective) was also calculated for which this simple averaging method is applicable.

Conclusions

The capacity estimates from the VAM method, the stochastic capacity method, and the average maximum flow rate method were found to be lower than the capacity values given in the HCM (2000) for Florida freeways.

From the stochastic capacity estimation method, it was observed that the average breakdown probability from the Weibull capacity distribution function that corresponds to the

VAM capacity estimates and maximum flow rate within ten minutes of breakdown was 3.98% and 3.27%, respectively. These percentile values, or the average breakdown probabilities, compare closely with the average design breakdown probability provided by Geistefeldt (2008), which was 3% for German freeways. Thus, based on the results of this study and the Geistefeldt study, it appears that the use of a 4th percentile value from the Weibull capacity distribution function will provide reasonable estimates of freeway segment capacities. It is also observed that the most reliable estimates of capacity from the stochastic capacity estimation method will be obtained when the analysis site has 0.5 or more breakdowns per day.

Two variants of the averaging method were investigated. In the first variant, the average of the top 5% highest flow rates were taken as the capacity estimates for the freeway segments. In the second variant, the average of flow rates between maximum flow rate observed and flow that corresponds to 35% of the maximum flow rate were taken as the capacity estimates for the freeway segment. To obtain consistent estimates from this method, it was found that a minimum of two months data should be used.

Advantages and Disadvantages of the Investigated Capacity Estimation Methods

Each of the three methods investigated in this study has its advantages and disadvantages for use in capacity estimation, as described below.

Stochastic estimation method

Advantages:

- Method accounts for the stochastic nature of capacity
- Utilizes the concept of breakdown (without identifying breakdowns, one cannot be sure whether more traffic than the highest observed flow rates could be served)
- Capacity distribution function provides flexibility in choosing capacity value (based on breakdown probability) that is appropriate to given application

Disadvantages:

- Very data processing intensive
- Very computationally intensive
- Determination of appropriate breakdown probability value is not straightforward

Van Aerde model method

Advantages:

- Does not require the identification of breakdowns
- Traffic flow theory basis (i.e., car-following minimum headway rule)
- From a simplicity perspective, a single capacity value is returned
- Is flexible in its application to different types of freeways

Disadvantages:

- Incorporation of congested data points increase accuracy, but capacity values are not tied directly to breakdown events
- Moderately data processing intensive
- Moderately computationally intensive

Average maximum flow rate method

Advantages:

- Easy to understand
- Easy to apply
- Does not require the identification of breakdown events

Disadvantages:

- No theoretical basis for capacity estimates
- Without incorporation of breakdown events or congested data points, accuracy of estimated capacity values is unknown

In summary, the stochastic capacity estimation method is most suitable for the determination of capacity at a site where a detailed operational analysis is desired. For example, at sites where different operational treatments (e.g., ramp metering) are going to be tried in an effort to improve operations and an estimate of capacity that is as accurate as possible is desired. The VAM capacity estimation method is not as suitable as the stochastic estimation method for detailed evaluations of operational treatments, but is still appropriate for the determination of general capacity estimates. The average maximum flow rate capacity estimation method is most suitable for planning and preliminary engineering applications.

Recommendations

The focus of this study was on the comparison of different methods for capacity estimation, with the intention of choosing a method for a follow-on study focused on determining specific capacity values for Florida freeways. Given that the FDOT Systems Planning Office is looking to use these capacity estimates in its planning and preliminary engineering level of service analysis software, it is recommended that the follow-on study apply the VAM method for the initial establishment of these values. However, it is also recommended that the average maximum flow rate capacity estimation method be applied in this follow-on study, for the purpose of identifying appropriate percentile values for this method (in reference to the VAM method). For future updates to the initially established capacity values, the use of the average maximum flow rate method by itself should be sufficient.

This follow-on study will also require more analysis sites. This will be possible as the Central Data Warehouse is expanded to include more Florida cities and more sites within each city. To the extent possible, the follow-on study should also investigate the effect of the following specific roadway and traffic factors on freeway segment capacity: number of lanes (as it relates to per-lane capacity), merge/diverge activity, free-flow speed, and truck percentage.

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BIOGRAPHICAL SKETCH

Vipul Modi was born in the capital city of India, New Delhi. He completed his higher secondary school from Delhi Public School Faridabad and bachelors in Civil Engineering from Indian Institute of Technology (IIT) Bombay in April 2007. He joined University of Florida, Gainesville for his Masters in Transportation Engineering and graduated in December 2009.

He has held many leadership positions in IIT Bombay and at the University of Florida. He held the position of President of ITE (Institute of Transportation Engineers) Student Chapter at University of Florida and has held the position of Treasurer of ITE Student Chapter. He has a passion to learn about different MRTS (Mass Rapid Transit Systems) and has knowledge of all the MRTS in different cities of the world. He has won many prizes at Inter-collegiate festivals in the field of dramatics, choreography and sports. He likes to collect stamps, currencies, travel, photography, drive, listen to music and play all kinds of sports.