FINAL REPORT

to

THE FLORIDA DEPARTMENT OF TRANSPORTATION
SYSTEMS PLANNING OFFICE

on Project

Investigation of Freeway Capacity:
a) Effect of Auxiliary Lanes on Freeway Segment Volume Throughput, and
b) Freeway Segment Capacity Estimation for Florida Freeways

FDOT Contract BDK-75-977-08, (UF Project 00073157)

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University of Florida
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SI* (MODERN METRIC) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

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

APPROXIMATE CONVERSIONS FROM SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
		AREA		
mm^2	square millimeters	0.0016	square inches	in^2
m^2	square meters	10.764	square feet	ft^2
m^2	square meters	1.195	square yards	yd^2
ha	hectares	2.47	acres	ac
km^2	square kilometers	0.386	square miles	mi^2
		VOLUME		
mL	milliliters	0.034	fluid ounces	fl oz
L	liters	0.264	gallons	gal
m^3	cubic meters	35.314	cubic feet	ft^3
m^3	cubic meters	1.307	cubic yards	yd^3
		MASS		
g	grams	0.035	ounces	oz
kg	kilograms	2.202	pounds	lb
Mg (or "t")	megagrams (or "metric ton")	1.103	short tons (2000 lb)	T
		TEMPERATURE (exact degrees)		
°C	Celsius	1.8C+32	Fahrenheit	°F
		ILLUMINATION		
lx	lux	0.0929	foot-candles	fc
cd/m^2	candela/m^2	0.2919	foot-Lamberts	fl
		FORCE and PRESSURE or STRESS		
N	newtons	0.225	poundforce	lbf
kPa	kilopascals	0.145	poundforce per square inch	lbf/in^2

*SI is the symbol for the International System of Units. Appropriate rounding should be made to comply with Section 4 of ASTM E380. (Revised March 2003)

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16. Abstract

Auxiliary lanes are generally used to reduce the traffic turbulence created by merging and diverging movements and are primarily used by vehicles either entering or exiting the freeway. The Highway Capacity Manual (HCM) does not offer explicit guidance on the benefit of adding an auxiliary lane between an on- and off-ramp. The objective of this part of the project was to quantify the additional traffic volume that can be accommodated on a freeway segment by connecting an on-ramp to an off-ramp with an auxiliary lane. The approach used was to identify the traffic volume level at which each level of service density threshold was met for the conditions of with and without an auxiliary lane. The CORSIM simulation program was used to generate the data upon which to establish the quantitative effect of an auxiliary lane. Two versions of an adjustment equation that gives the percentage increase in volume throughput due to adding an auxiliary lane were developed. The developed equations are simply a function of the number of mainline lanes, as other factors were not found to significantly affect the percentage increase in volume throughput.

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 part of the project 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 its own estimates of capacity for Florida freeways. Three methods were investigated: one that fits a mathematical function to 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 flow rate corresponding to a specified percentile within a specified range of maximum flow rates observed at a site. It is recommended that this latter method is most suitable for planning and preliminary engineering applications.

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Report Organization

The content for Part A of this report was prepared by Mr. Ashish Kulshrestha under the supervision of Dr. Scott Washburn. The content for Part B of this report is essentially the master's thesis prepared by Mr. Vipul Modi under the supervision of Drs. Scott Washburn and Yafeng Yin. The front matter that was relevant only to the graduate school of the University of Florida was deleted, and some minor editorial and formatting revisions were also performed.

Executive Summary

Part A

Auxiliary lanes are generally used to reduce the traffic turbulence created by merging and diverging movements and are primarily used by vehicles either entering or exiting the freeway. The Highway Capacity Manual (HCM) does not offer explicit guidance on the benefit of adding an auxiliary lane between an on- and off-ramp. The FDOT has previously developed its own guidelines for quantifying the benefit of adding an auxiliary lane in terms of the additional traffic volume that can be accommodated on the freeway segment for any given level of service. However, the FDOT has not previously conducted a study to validate their selection of these guidelines.

The objective of this part of the project was to quantify the additional traffic volume that can be accommodated on a freeway segment by connecting an on-ramp to an off-ramp with an auxiliary lane. The approach used was to identify the traffic volume level at which each level of service density threshold was met for the conditions of with and without an auxiliary lane. Comparisons were made between results obtained using the HCM 2010 analysis methodologies and microscopic simulation using CORSIM. Ultimately, CORSIM was selected for use to generate the data upon which to establish the quantitative effect of an auxiliary lane.

Two versions of an adjustment equation that gives the percentage increase in volume throughput due to adding an auxiliary lane were developed. The developed equations are simply a function of the number of mainline lanes, as other factors were not found to significantly affect the percentage increase in volume throughput.

Part B

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.

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, 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.

The objective of this part of the project 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 its own estimates of capacity for Florida freeways. To achieve the objective of this task, a detailed review of previous research related to methods used to estimate the capacity of basic freeway segments was completed. From this review, three methods were investigated: one that fits a mathematical function to 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 flow rate corresponding to a specified percentile within a specified range of maximum 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 capacity estimation method based on a specified percentile within a specified range of maximum flow rates is most suitable for planning and preliminary engineering applications. For Florida freeways, the capacity estimates from all the three methods were found to be lower than the capacity values given in the HCM (2000).

These estimates are based on specific percentile flow rate values that fall between the speed-flow plotted capacity estimates and the maximum observed flow rates. A simple and limited capacity analysis on rural freeways was also performed that determines the maximum hourly flow rates at the respective site locations.

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 percentile of maximum hourly flow rates (with a lower bound of the average of the highest 6.5% hourly flow rates), based on a 5-minute aggregation interval, be applied. Furthermore, it is recommended that the capacity estimates pertaining to percentile values between 60%-80% are likely the most appropriate and be used for freeway capacity estimation. It is recommended that a follow-on study be conducted that will focus on investigating 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. This study will require considerably more data and analysis sites than were used for this study.

Part A

EFFECT OF AUXILIARY LANES ON FREEWAY SEGMENT VOLUME THROUGHPUT

TABLE OF CONTENTS

Introduction

Auxiliary lanes[1] are generally used to reduce the traffic turbulence created by merging and diverging movements. Given that these lanes are used primarily by vehicles either entering or exiting the freeway, it is uncommon for them to realize capacity values similar to those of a regular lane on a basic freeway segment. However, if the distance between the connecting on-ramp and off-ramp becomes great enough, it is plausible that the auxiliary lane will be used by some amount of through traffic.

The Highway Capacity Manual (HCM) 2000 does not offer explicit guidance on the capacity of auxiliary lanes. The typical interpretation of this issue by analysts is as follows:

- Less than 2500 ft in length, it is analyzed as a weaving section
- Greater than 3000 ft in length, it is considered to have the same capacity as its adjacent regular freeway lanes

These distance thresholds are based on a general interpretation of the distance guidelines given in the HCM related to the analysis of weaving sections and merge/diverge areas. More specifically, the HCM 2000 currently recommends that freeway segments with an auxiliary lane of 2500 ft or less be analyzed with the freeway weaving procedure[2]. For longer sections, the ramp junctions analysis procedure should be applied to both the on-ramp and off-ramp areas. The HCM ramp junctions analysis procedure assumes that the influence area of a ramp extends 1500 ft downstream/upstream of an on-ramp/off-ramp. Thus the selection of the 3000 ft threshold (the combined value of adjacent on-ramp and off-ramp influence areas) for the full-capacity value. For distance values between 2500 and 3000 ft, analysts assume a wide range of capacity values due to the lack of any guidance whatsoever on this specific distance range.

The FDOT has developed its own guidelines for the capacity of an auxiliary lane, which are currently implemented in its FREEPLAN software program, as follows:

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[1] Auxiliary lanes, as defined in this study, consist of lanes connecting on-ramps to off-ramps. Furthermore, auxiliary lanes that are physically separated from the adjacent freeway lanes, such as collector-distributor lanes are also not considered.

[2] This length threshold is no longer applicable in the updated weaving analysis procedure for the HCM 2010 (which became available after this contract was initiated).

Auxiliary Lane Length (mi)	Proportion of Full Capacity
< 0.5	0.6
>=0.5 and < 1	0.7
>=1 and < 2	0.8
>=2 and < 3	0.9
>=3	1.0

However, the FDOT has not previously conducted a study to validate their selection of these distance-capacity values.

The objective of this task was to determine the relative traffic operations performance benefit of connecting an on-ramp to an off-ramp with an auxiliary lane by comparing the results of a weaving analysis (with the new HCM 2010 methodology) to the results of isolated ramp merge/diverge analyses.

This task consisted of the following sub-tasks:

1. Identify the key parameters and develop the experimental scenarios (number of lanes, freeway volume, free flow speed, on-ramp/off-ramp volume, length of acceleration and deceleration lanes, distance between merge and diverge section).
2. Perform the HCM analysis for merge and diverge segments (per the HCM 2010 procedures) for each experimental scenario.
3. Connect the on-ramp acceleration lane to the off-ramp deceleration lane (i.e., add an auxiliary lane) for each scenario and perform the HCM analysis for a weaving segment (per the HCM 2010 weaving procedure).
4. Perform a microscopic simulation (using CORSIM) of each scenario (merge/diverge and weaving).
5. Analyze and evaluate the results obtained from the HCM 2010 and CORSIM simulation.
6. Compare the results obtained with the HCM 2010 and CORSIM analysis methods and choose the most appropriate analysis method to use for the following sub-task.
7. Develop quantitative guidelines on the performance benefits of adding an auxiliary lane between an on-ramp and an off-ramp junction.

HCM 2010 WEAVING ANALYSIS METHODOLOGY

A recent National Cooperative Highway Research Program (NCHRP) project (#3-75) resulted in the development of a new weaving segment analysis methodology, which will be incorporated in the next edition of the HCM (slated for release in late 2010). The major difference between the HCM 2010 weaving analysis methodology and the HCM 2000 weaving analysis methodology is that the maximum length of weaving operations is no longer a constant value. In the HCM 2000 methodology, the maximum weaving segment length was fixed at 2500 ft. In the HCM 2010 methodology, the maximum weaving length depends on the volumes and the configuration characteristics.

Another difference between the HCM 2000 and 2010 weaving methodologies is that weaving segments in the HCM 2000 methodology were classified into Type A, Type B and Type C; whereas in the HCM 2010 methodology, there is no such classification and all weaving segments are analyzed in the same way depending on the input parameters and configuration characteristics. The HCM 2010 weaving analysis methodology is described in detail in Appendix A.

To get an idea about how many freeway segments with auxiliary lanes in Florida might be able to be analyzed as a weaving segment per the HCM 2010 weaving analysis methodology, a large sampling of sites with auxiliary lanes across Florida were identified. Using available peak hour traffic volume data, assumptions regarding weaving demands, and the length of each segment, a determination was made whether each site would be considered as weaving segment (for the given volumes) per the HCM 2010 weaving analysis methodology. The sites examined are shown in Table 1. Of these sites, 93% (37/40) would be considered weaving segments for the peak hour traffic demands under the HCM 2010 weaving analysis methodology. Under the HCM 2000 weaving methodology, only 68% of these sites would be considered weaving segments for analysis purposes (i.e., a segment length <= 2500 ft).

Table 1. Summary of Lengths and Locations of Identified Auxiliary Lane Sites in Florida

No.	Location	Interstate	Direction	Length (mi)	Location
1	Jacksonville	I-95	Southbound	0.41	Exit to Phillips Hwy
2	Jacksonville	I-95	Northbound	0.32	Kings road and W 8th St.
3	Jacksonville	I-95	Southbound	0.37	Kings road and W 8th St.
4	Jacksonville	I-95	Southbound	0.25	Between 20th Street Expressway and W 8th St.
5	Jacksonville	I-95	Northbound	0.33	Between 20th Street Expressway and W 8th St.
6	Jacksonville	I-95	Southbound	0.25	W 23rd and 30th St.
7	Jacksonville	I-95	Northbound	0.21	W 23rd and 30th St.
8	Jacksonville	I-295	Northbound	0.50	Between I-10 and SR 228
9	Jacksonville	I-10	Westbound	0.27	After the Intersection with I-295
10	Jacksonville	I-10	Westbound	0.40	Before the Intersection with I-295
11	Jacksonville	I-10	Eastbound	0.49	Before the Intersection of I-95 and I-10
12	Orlando	I-4	Southbound	0.25	Between W Gore St. and W Kaley St.
13	Orlando	I-4	Northbound	0.23	Between W Gore St. and W Kaley St.
14	Orlando	I-4	Southbound	0.60	Between SR 423 and Conroy windermere Rd.
15	Orlando	I-4	Northbound	0.70	Between SR 423 and Conroy windermere Rd.
16	Orlando	I-4	Southbound	0.39	Between Florida Turnpike and Conroy windermere Rd.
17	Orlando	I-4	Northbound	0.46	Between Florida Turnpike and Conroy windermere Rd.
18	Orlando	I-4	Southbound	0.58	Between Epcot Center Drive and SR 535
19	Orlando	I-4	Northbound	0.50	Between Epcot Center Drive and SR 535
20	Orlando	I-4	Southbound	0.75	Between SR 435/ South Kirkman and Florida Turnpike
21	Orlando	I-4	Northbound	0.46	Between SR 435/ South Kirkman and Florida Turnpike
22	Orlando	I-4	Northbound	0.50	Over West Colonial Drive
23	Orlando	I-4	Northbound	0.76	Between West Sand Lake Road and Universal Bld.
24	Fort Lauderdale	I-95	Southbound	0.35	Between NW 36th St. and W Copans Rd.
25	Fort Lauderdale	I-95	Northbound	0.35	Between W Copans Rd. and NW 36th St.
26	Fort Lauderdale	I-95	Southbound	0.26	Between N Andrews Ave. and W Commercial Blvd.
27	Fort Lauderdale	I-95	Northbound	0.53	Between W Commercial Blvd. and N Andrews Ave.
28	Fort Lauderdale	I-95	Southbound	0.28	Between W Sunrise Blvd. and NW 6th St.
29	Fort Lauderdale	I-95	Northbound	0.50	Between I-595 and SR 818
30	Fort Lauderdale	I-95	Southbound	0.38	Between SR 818 and 848
31	Fort Lauderdale	I-95	Northbound	0.35	Between SR 818 and 848
32	Fort Lauderdale	I-95	Southbound	0.33	Between SR 818 and 822
33	Fort Lauderdale	I-95	Northbound	0.32	Between SR 818 and 822
34	Fort Lauderdale	I-95	Southbound	0.36	Between SR 820 and 824
35	Fort Lauderdale	I-95	Northbound	0.33	Between SR 820 and 824
36	Miami	I-95	Northbound	0.49	Between Opa Locka Blvd and NW 151st St.
37	Miami	I-95	Southbound	0.39	Between NW 79th St. and 69th St.
38	Miami	SR-826	Eastbound	0.46	Between NW 57th Ave and 67th Ave
39	Miami	SR-826	Westbound	0.30	Between NW 154th St. and exit to I-75
40	Miami	I-75	Eastbound	0.57	Under NW 87th Av. Leading to Gratigny Expressway

Research Approach

The basic foundation of the research approach was to compare traffic performance measures from freeway segment configurations with and without an auxiliary lane. Comparisons were made using the HCM analysis methodologies and microscopic simulation. These comparisons were then used to quantify the effect, with regard to throughput, of an auxiliary lane. The rest of this chapter describes each of the steps in the research approach in detail along with the analysis and results.

Identification of the key parameters and development of the experimental scenarios

The first step in the research approach was to identify the key parameters. Based on the new HCM 2010 methodology of weaving analysis, factors which may affect the performance benefit of connecting an on-ramp to an off-ramp with an auxiliary lane were considered. Parameters which were identified are area type, number of lanes, freeway volume, free flow speed, on-ramp/off-ramp volume, length of acceleration and deceleration lanes, distance between merge and diverge section. Experimental scenarios were then developed using the appropriate values for these parameters.

For developing the experimental scenarios, 1 mi and 2 mi of distance between the merge and diverge areas was considered for the urban area type. For transitioning and rural area types, 3 mi of distance was considered. A summary of developed experimental scenarios are given in Table 2.

Table 2. Summary of Experimental Scenarios

Area Type	Urban				Urban				Transitioning/Rural			
FFS (mi/h)	65.0				65.0				70.0			
Ramp Speed (mi/h)	35.0				35.0				40.0			
La (ft)	1,000				1,000				1,000			
Ld (ft)	450				450				450			
Interchange Spacing (ft)	5,280 (1.0 mi)				10,560 (2.0 mi)				15,840 (3.0 mi)			
No. of Lanes	3		4		3		4		2		3	
Weaving Volume	High (20%)	Low (10%)	High (20%)	Low (10%)	High (20%)	Low (10%)	High (20%)	Low (10%)	High (20%)	Low (10%)	High (20%)	Low (10%)
Mainline Demand Volume*												
LOS A	1763	1923	2350	2564	1763	1923	2350	2564	1280	1396	1920	2095
LOS B	2938	3205	3917	4273	2938	3205	3917	4273	2120	2313	3180	3469
LOS C	4171	4550	5562	6067	4171	4550	5562	6067	2960	3229	4440	4844
LOS D	5229	5704	6972	7605	5229	5704	6972	7605	3600	3927	5400	5891
LOS E	5875	6409	7833	8545	5875	6409	7833	8545	4000	4364	6000	6545

***** The volumes were selected to correspond to the maximum service volume for each level of service, A-E

Identification of the appropriate analysis tool to observe the effect of an auxiliary lane

The next step in the research approach was to identify the appropriate analysis tool which could be used to observe and quantify the effect of adding an auxiliary lane. Test analyses were run with the HCM 2000 weaving analysis methodology, the HCM 2010 weaving analysis methodology, and CORSIM. To be able to use the HCM 2000 weaving analysis methodology, the test segments had to be on the order of 2500 ft in length. Since many interchanges in urban areas are spaced at approximately ½ mile intervals, a test segment length of 2640 ft was used. Although this is slightly longer than the 2500 ft limit for the HCM 2000 methodology, it was felt this small difference in length would introduce little error. For this comparison, experimental scenarios for an urban area (see Table 3) were developed. The following analyses were done for these experimental scenarios:

a. HCM analysis for merge and diverge segments (per the HCM 2010 and 2000 procedures) for each experimental scenario

In this step, the experimental scenarios for urban area type with 0.5 mi length were then analyzed using the HCM methodologies for merge/diverge segments. Since the influence areas of the on and off-ramps overlap (1500 ft for each), the performance measures for the critical junction (i.e., the one with the highest density) were used as the performance measures for the whole freeway segment.

b. HCM analysis for a weaving segment (per the HCM 2010 and 2000 weaving procedure) for each experimental scenario

In this step, experimental scenarios for urban area type with 0.5 mi length were then analyzed using the HCM methodologies for weaving segment by connecting the on-ramp and off-ramp with an auxiliary lane. In addition to the performance measures obtained from both methodologies, total segment capacity was also obtained from HCM 2010 methodology for all the scenarios.

c. Perform a microscopic simulation (using CORSIM) for each experimental scenario (merge/diverge)

In this step, experimental scenarios for urban area type with 0.5 mi length were run using CORSIM for merge/diverge segments. An isolated freeway segment of 0.5 mi length with on- and off-ramp junctions was coded in CORSIM. Experimental scenarios with appropriate input parameters were executed and the averages of performance measures for 10 runs for each scenario were obtained.

d. Perform a microscopic simulation (using CORSIM) for each experimental scenario (weaving)

In this step, experimental scenarios were run using CORSIM for a weaving segment. An isolated freeway segment of 0.5 mi length with an auxiliary lane connecting the on- and off-ramp junctions was coded in CORSIM. Experimental scenarios with appropriate input parameters were executed and the averages of performance measures for 10 runs for each scenario were obtained.

A summary of results obtained for average segment speed and segment density performance measures are shown in Tables 4 and 5 respectively.

Table 3. Experimental Scenarios for Urban Area Type (0.5 mi Segment Length)

Scenario No.	Scenario 1	Scenario 2	Scenario 3	Scenario 4	Scenario 5	Scenario 6	Scenario 7	Scenario 8	Scenario 9	Scenario 10
AREA TYPE	Urban	Urban	Urban	Urban	Urban	Urban	Urban	Urban	Urban	Urban
Fwy FFS (mi/h)	65	65	65	65	65	65	65	65	65	65
Ramp FFS (mi/h)	35	35	35	35	35	35	35	35	35	35
Length Accel (ft)	1000	1000	1000	1000	1000	1000	1000	1000	1000	1000
Length Decel (ft)	450	450	450	450	450	450	450	450	450	450
No. of Lanes	3	3	3	3	3	3	3	3	3	3
Weaving Volume	Low (10%)	Low (10%)	Low (10%)	Low (10%)	Low (10%)	High (20%)	High (20%)	High (20%)	High (20%)	High (20%)
Demand Volume (veh/h)	1923	3205	4550	5704	6409	1763	2938	4171	5229	5875
Level-of-Service	LOS A	LOS B	LOS C	LOS D	LOS E	LOS A	LOS B	LOS C	LOS D	LOS E
Interchange Spacing (ft)	2640	2640	2640	2640	2640	2640	2640	2640	2640	2640
Scenario No.	Scenario 11	Scenario 12	Scenario 13	Scenario 14	Scenario 15	Scenario 16	Scenario 17	Scenario 18	Scenario 19	Scenario 20
AREA TYPE	Urban	Urban	Urban	Urban	Urban	Urban	Urban	Urban	Urban	Urban
Fwy FFS (mi/h)	65	65	65	65	65	65	65	65	65	65
Ramp FFS (mi/h)	35	35	35	35	35	35	35	35	35	35
Length Accel (ft)	1000	1000	1000	1000	1000	1000	1000	1000	1000	1000
Length Decel (ft)	450	450	450	450	450	450	450	450	450	450
No. of Lanes	4	4	4	4	4	4	4	4	4	4
Weaving Volume	Low (10%)	Low (10%)	Low (10%)	Low (10%)	Low (10%)	High (20%)	High (20%)	High (20%)	High (20%)	High (20%)
Demand Volume (veh/h)	2564	4273	6067	7605	8545	2350	3917	5562	6972	7833
Level-of-Service	LOS A	LOS B	LOS C	LOS D	LOS E	LOS A	LOS B	LOS C	LOS D	LOS E
Interchange Spacing (ft)	2640	2640	2640	2640	2640	2640	2640	2640	2640	2640

Table 4. Analysis Results for Average Speed of Segment

Scenario No.	Average Speed (mi/h)										
	Ramp Junction Analysis								Weaving Analysis		
	HCM				CORSIM				HCM 2000	HCM 2010	CORSIM
	All Lanes		Ramp Influence Area		All Lanes		Ramp Influence Area				
	On-Ramp	Off-Ramp	On-Ramp	Off-Ramp	On-Ramp	Off-Ramp	On-Ramp	Off-Ramp			
Scenario 1	60.6	58.5	58.9	54.8	62.7	62.0	62.6	63.5	64.2	59.5	63.0
Scenario 2	59.7	58.7	58.4	54.5	61.7	61.8	61.7	62.3	60.2	56.4	62.2
Scenario 3	58.2	58.4	57.0	54.2	60.3	60.1	60.3	60.5	56.1	53.0	61.0
Scenario 4	55.7	57.9	54.2	54.0	58.8	58.2	58.5	58.3	53.3	50.3	60.0
Scenario 5	52.8	57.5	50.6	53.8	57.4	56.4	56.9	56.1	51.6	48.6	59.3
Scenario 6	60.5	57.7	58.9	54.4	61.7	62.4	61.6	63.4	60.2	57.4	62.3
Scenario 7	59.6	57.3	58.3	53.9	60.8	60.7	60.8	61.6	55.6	53.4	61.4
Scenario 8	57.8	56.7	56.3	53.4	59.3	58.9	59.2	59.5	50.9	49.3	60.3
Scenario 9	53.7	56.2	51.5	53.0	57.5	56.4	57.1	56.5	47.4	45.6	59.3
Scenario 10	48.1	55.8	44.7	52.7	55.6	54.1	54.7	53.5	43.6	43.4	58.3
Scenario 11	61.6	62.1	58.9	54.6	62.6	63.0	62.0	63.4	64.2	58.4	63.0
Scenario 12	60.4	61.5	58.5	54.3	61.7	61.7	61.2	61.8	60.2	54.7	62.2
Scenario 13	58.9	60.5	57.4	53.9	60.4	60.0	59.7	59.6	56.3	50.8	61.0
Scenario 14	57.0	59.6	55.2	53.6	58.8	57.9	57.7	57.0	53.3	47.4	59.8
Scenario 15	55.0	59.1	52.8	53.4	57.0	55.1	55.1	52.8	51.6	45.4	59.0
Scenario 16	61.2	61.1	58.8	54.2	61.8	62.2	60.9	62.8	58.4	55.9	62.2
Scenario 17	60.4	60.5	58.3	53.5	60.8	60.5	60.0	60.9	53.1	50.9	61.3
Scenario 18	58.7	59.4	56.7	52.9	59.2	58.1	58.0	57.7	48.8	45.7	60.0
Scenario 19	56.2	58.5	53.4	52.3	56.9	53.9	54.9	51.6	45.8	41.1	58.8
Scenario 20	53.4	57.9	49.4	51.9	53.4	48.9	49.6	43.5	44.2	38.2	57.6

Table 5. Analysis Results for Density of Segment

Scenario No.	Density (pc/mi/ln)										
	Ramp Junction Analysis								Weaving Analysis		
	HCM				CORSIM				HCM 2000	HCM 2010	CORSIM
	All Lanes		Ramp Influence Area		All Lanes		Ramp Influence Area				
	On-Ramp	Off-Ramp	On-Ramp	Off-Ramp	On-Ramp	Off-Ramp	On-Ramp	Off-Ramp			
Scenario 1	10.4	11.6	9.7	13.4	8.9	10.0	11.6	11.4	11.0	8.9	8.4
Scenario 2	17.9	19.2	16.7	21.1	15.2	17.0	19.6	19.5	19.5	15.6	14.2
Scenario 3	25.9	27.3	24.0	28.6	22.0	24.9	28.3	28.2	29.9	23.9	20.4
Scenario 4	33.0	34.5	30.3	34.7	28.3	32.2	36.3	36.4	39.2	31.2	26.1
Scenario 5	37.4	39.0	34.5	38.2	32.6	37.3	41.8	42.2	45.5	36.3	29.7
Scenario 6	10.3	11.7	10.1	14.1	9.1	10.1	12.2	11.7	11.6	9.2	8.4
Scenario 7	17.7	19.5	17.4	23.3	15.4	17.3	20.3	19.9	21.1	16.5	14.3
Scenario 8	25.6	27.6	25.9	32.9	22.4	25.4	29.2	29.0	32.8	25.4	20.7
Scenario 9	32.4	34.7	33.5	41.2	29.0	33.3	37.8	37.9	44.1	34.4	26.4
Scenario 10	36.5	39.1	38.3	46.2	33.6	38.9	44.0	44.2	54.6	40.6	30.1
Scenario 11	10.6	11.1	9.1	12.0	9.4	10.3	11.8	11.7	11.0	9.7	8.9
Scenario 12	18.2	18.5	15.7	19.9	15.9	17.5	20.2	20.2	19.5	17.2	15.1
Scenario 13	26.4	26.6	22.6	28.2	23.2	25.6	29.2	29.6	29.6	26.3	21.8
Scenario 14	33.7	33.7	28.5	35.3	29.8	33.2	37.9	38.8	39.3	35.3	27.9
Scenario 15	38.6	38.1	32.1	39.6	34.6	39.2	44.6	46.9	45.5	41.5	31.8
Scenario 16	12.4	11.2	11.4	13.1	9.6	10.4	12.9	12.7	12.1	10.1	9.0
Scenario 17	18.0	18.6	17.2	21.6	16.2	17.8	21.7	21.7	22.1	18.5	15.3
Scenario 18	26.1	26.6	24.7	30.6	23.7	26.4	31.6	32.3	34.2	29.2	22.2
Scenario 19	33.2	33.7	31.2	38.3	30.9	35.8	41.6	44.7	45.7	40.8	28.4
Scenario 20	37.7	38.1	35.1	43.0	37.0	44.2	51.1	53.5	53.2	49.3	32.6

Table 6. Weaving Segment Capacity (HCM 2010) for Different Scenarios

	Maximum Length L_{max} (ft)	Scenario No.	Weaving Segment Capacity (HCM 2010)	
			(veh/h)	(veh/h/ln)
3 Lanes Low Weaving Volume	4260	**Scenario 1**	8718	2180
		Scenario 2	8719	2180
		Scenario 3	8726	2182
		Scenario 4	8719	2180
		Scenario 5	8719	2180
3 Lanes High Weaving Volume	5763	**Scenario 6**	7580	1895
		Scenario 7	7578	1895
		Scenario 8	7577	1894
		Scenario 9	7581	1895
		Scenario 10	7581	1895
4 Lanes Low Weaving Volume	4260	**Scenario 11**	10897	2179
		Scenario 12	10898	2180
		Scenario 13	10899	2180
		Scenario 14	10898	2180
		Scenario 15	10898	2180
4 Lanes High Weaving Volume	5763	**Scenario 16**	7576	1515
		Scenario 17	7581	1516
		Scenario 18	7578	1516
		Scenario 19	7578	1516
		Scenario 20	7580	1516

The capacity of weaving segment obtained for different scenarios using the HCM 2010 methodology are shown in Table 6. Segment capacity per lane was found to be same for low weaving volume scenarios of 3 lanes and 4 lanes. However, the total segment capacity was same for high weaving volume scenarios of 3 lanes and 4 lanes. As per the weaving methodology of HCM 2010, the capacity of a weaving segment is controlled by one of two conditions:

1. Breakdown of a weaving segment is expected to occur when the average density of all vehicles in the segment reaches 43 pc/mi/ln

$$c_{IWL} = c_{IFL} - [438.2(1 + VR)^{1.6}] + [0.0765L_s] + [119.8N_{WL}]$$

$$c_W = c_{IWL} N f_{HV} f_p$$

where

c_{IWL} = capacity of weaving segment per lane (pc/h/ln)
c_{IFL} = capacity of basic freeway segment per lane
c_W = total capacity of weaving segment
VR = volume ratio
L_s = length of weaving segment
N_{WL} = number of lanes *from which* a weaving maneuver may be made with one or no lane changes (2 in our case)

2. Breakdown of a weaving segment is expected to occur when the total weaving demand flow rate exceeds 2400 pc/h

$$c_{IW} = \frac{2400}{VR} \qquad for\ N_{WL} = 2$$

$$c_W = c_{IW} f_{HV} f_p$$

where

c_{IW} = capacity of all lanes in the weaving segment (pc/h)
c_W = total capacity of weaving segment

For low weaving scenarios (1-5) and (11-15), the capacity of weaving segment is controlled by the first criterion of density. Since the volume ratio (VR) and length of segment (L_s) for all scenarios (1-5) and (11-15) are the same, we get the same value of capacity per lane. For the high weaving scenarios (6-10) and (16-20), the capacity of weaving segment is controlled by the second criterion of weaving flow rate. That is, there is a practical limit on how many vehicles can cross each other's path without causing a breakdown. Thus, even though there is one less lane for scenarios (6-10) than scenarios (16-20), the total segment capacity is the same because of the practical constraint on weaving flow rate (these scenarios have the same *VR*). Therefore, if the weaving flow rate is high, the total segment capacity will remain the same (for N_{WL}= 2) for a given volume ratio and does not depend on how many additional lanes are there in the segment.

Analyze and evaluate the results obtained from the HCM 2010 methodologies and CORSIM simulation

The next step was to compare the results from the HCM 2010 methodologies and the CORSIM microscopic simulation and to choose the one with more plausible results for the further analysis. Comparisons for traffic performance measures from freeway segment configurations with and without an auxiliary lane were made.

A comparison of average speed and density performance measures obtained from HCM 2010 and CORSIM analyses for the merge/diverge segments and weaving segments did not show a close match between the values. Results obtained from very few of the scenarios for the HCM 2010 analysis methodologies followed the basic intuition for the effect of adding an auxiliary lane. Most of the scenarios indicated lower segment speeds after adding an auxiliary lane as compared to the segment speed without an auxiliary lane. Similarly, many scenarios indicated higher segment densities after adding an auxiliary lane as compared to the segment density without an auxiliary lane. These results obtained from the HCM 2010 methodologies raised doubts about the validity of the procedures and thus were not for further analyses. On the other hand, results obtained from CORSIM simulation were consistent for all the scenarios in terms of the expected effect of adding an auxiliary lane and hence CORSIM simulation was chosen as the analysis tool for the further analyses.

The objective of this study was to compare the traffic performance measures for a freeway segment with and without an auxiliary lane and to quantify the effect of adding an auxiliary lane with regard to throughput. Therefore, the basic premise for estimating the effect of an auxiliary lane was to determine the additional traffic throughput that could be accommodated with an auxiliary lane relative to the no auxiliary lane condition while keeping the same performance level of the segment (i.e., density). The density threshold values for LOS A to E were identified and then the next step was to estimate the throughput for a merge/diverge segment and a weaving segment (all other geometric characteristics being equal) for the same threshold density value using CORSIM.

For theses analyses, experimental networks for a freeway segment with and without an auxiliary lane were developed in CORSIM for an urban area type (1 mi and 2 mi interchange spacing) and a transitioning/rural area type (3 mi interchange spacing).

CORSIM analysis for urban area type with 1-mile interchange spacing

Experimental scenarios were developed for 2, 3, 4 and 5 mainline lanes for an urban area type with an interchange spacing of 1.0 mi. Weaving volumes of 10% and 20% of the mainline volume for each test scenario was used in the analysis. Results for throughput obtained using CORSIM for merge/diverge and weaving analysis along with the difference in throughput are shown in Table 7. Some of the highlighted cases in the results are because of the limit on input volume in CORSIM for higher density, but in looking at the trend of the results obtained, it is likely that the result for the highlighted case would be similar to the other results.

Table 7. Percentage Increase in Throughput for Urban Area (1-mile Interchange Spacing)

	Density (veh/mi/ln)	Total Volume (veh/h)		Additional Volume (veh/h)	Percentage Increase in Volume
		Ramp Analysis	Weaving Analysis		
2 Lanes - Low (10%) Weaving Volume	10.0	1276	1903	627	49.14
	17.0	2134	3157	1023	47.94
	24.0	2948	4367	1419	48.13
	31.0				
	39.0				
2 Lanes - High (20%) Weaving Volume	10.0	1284	1896	612	47.66
	17.0	2136	3144	1008	47.19
	24.0	2964	4344	1380	46.56
	31.0	3744	5496	1752	46.79
	39.0				
3 Lanes - Low (10%) Weaving Volume	10.0	1914	2519	605	31.61
	17.0	3201	4224	1023	31.96
	24.0	4450	5863	1414	31.77
	31.0	5643	7403	1760	31.19
	39.0				
3 Lanes - High (20%) Weaving Volume	10.0	1920	2508	588	30.63
	17.0	3204	4224	1020	31.84
	24.0	4464	5832	1368	30.65
	31.0	5640	7368	1728	30.64
	39.0				
4 Lanes - Low (10%) Weaving Volume	10.0	2552	3157	605	23.71
	17.0	4279	5313	1034	24.16
	24.0	5929	7337	1408	23.75
	31.0	7535	9284	1749	23.21
	39.0				
4 Lanes - High (20%) Weaving Volume	10.0	2544	3156	612	24.06
	17.0	4284	5256	972	22.69
	24.0	5928	7308	1380	23.28
	31.0	7536	9228	1692	22.45
	39.0				
5 Lanes - Low (10%) Weaving Volume	10.0	3190	3784	594	18.62
	17.0	5368	6358	990	18.44
	24.0	7425	8789	1364	18.37
	31.0				
	39.0				
5 Lanes - High (10%) Weaving Volume	10.0	3204	3780	576	17.98
	17.0	5316	6312	996	18.74
	24.0	7404	8760	1356	18.31
	31.0	9360	11040	1680	17.95
	39.0				

CORSIM analysis for urban area type with 2-mile interchange spacing

Similarly, experimental scenarios were developed for 2, 3, 4 and 5 mainline lanes for an urban area type with an interchange spacing of 2.0 mi. Weaving volumes of 10% and 20% of the mainline volume for each test scenario was used in the analysis. Results for throughput obtained using CORSIM for merge/diverge and weaving analysis along with the difference in throughput are shown in Table 8.

Table 8. Percentage Increase in Throughput for Urban Area (2-mile Interchange Spacing)

	Density (veh/mi/ln)	Total Volume (veh/h)		Additional Volume (veh/h)	Percentage Increase in Volume
		Ramp Analysis	Weaving Analysis		
2 Lanes - Low (10%) Weaving Volume	10.0	1276	1903	627	49.14
	17.0	2107	3168	1061	50.36
	24.0	2943	4378	1435	48.76
	31.0				
	39.0				
2 Lanes - High (20%) Weaving Volume	10.0	1272	1896	624	49.06
	17.0	2124	3168	1044	49.15
	24.0	2952	4368	1416	47.97
	31.0	3732	5544	1812	48.55
	39.0				
3 Lanes - Low (10%) Weaving Volume	10.0	1914	2530	616	32.18
	17.0	3190	4235	1045	32.76
	24.0	4422	5863	1441	32.59
	31.0	5616	7403	1787	31.82
	39.0				
3 Lanes - High (20%) Weaving Volume	10.0	1908	2532	624	32.70
	17.0	3204	4212	1008	31.46
	24.0	4428	5868	1440	32.52
	31.0	5640	7416	1776	31.49
	39.0				
4 Lanes - Low (10%) Weaving Volume	10.0	2541	3157	616	24.24
	17.0	4268	5291	1023	23.97
	24.0	5907	7337	1430	24.21
	31.0	7535	9295	1760	23.36
	39.0				
4 Lanes - High (20%) Weaving Volume	10.0	2544	3156	612	24.06
	17.0	4260	5292	1032	24.23
	24.0	5904	7320	1416	23.98
	31.0	7512	9276	1764	23.48
	39.0				
5 Lanes - Low (10%) Weaving Volume	10.0	3179	3806	627	19.72
	17.0	5335	6358	1023	19.18
	24.0	7403	8822	1419	19.17
	31.0				
	39.0				
5 Lanes - High (10%) Weaving Volume	10.0	3180	3792	612	19.25
	17.0	5328	6336	1008	18.92
	24.0	7404	8784	1380	18.64
	31.0	9360	11112	1752	18.72
	39.0				

CORSIM analysis for transitioning/rural area type with 3-mile interchange spacing

Experimental scenarios were developed for 2, 3 and 4 mainline lanes for a transitioning/rural area type with an interchange spacing of 3.0 mi. Weaving volumes of 10% and 20% of the mainline volume for each test scenario was used in the analysis. Results for throughput obtained using CORSIM for merge/diverge and weaving analysis along with the difference in throughput are shown in Table 9.

Table 9. Percentage Increase in Throughput for Transitioning/Rural Area (3-mile Interchange Spacing)

	Density (veh/mi/ln)	Total Volume (veh/h)		Additional Volume (veh/h)	Percentage Increase in Volume
		Ramp Analysis	Weaving Analysis		
2 Lanes - Low (10%) Weaving Volume	10.0	1364	2057	693	50.81
	17.0	2266	3394	1128	49.78
	24.0	3146	4697	1551	49.30
	31.0				
	39.0				
2 Lanes - High (20%) Weaving Volume	10.0	1356	2040	684	50.44
	17.0	2268	3396	1128	49.74
	24.0	3144	4692	1548	49.24
	31.0	3984	6000	2016	50.60
	39.0				
3 Lanes - Low (10%) Weaving Volume	10.0	2046	2728	682	33.33
	17.0	3410	4543	1133	33.23
	24.0	4730	6292	1562	33.02
	31.0	5995	7909	1914	31.93
	39.0				
3 Lanes - High (20%) Weaving Volume	10.0	2052	2724	672	32.75
	17.0	3420	4536	1116	32.63
	24.0	4740	6276	1536	32.41
	31.0	6012	7920	1908	31.74
	39.0				
4 Lanes - Low (10%) Weaving Volume	10.0	2739	3410	671	24.50
	17.0	4565	5676	1111	24.34
	24.0	6325	7865	1540	24.35
	31.0	8030	9933	1903	23.70
	39.0				
4 Lanes - High (20%) Weaving Volume	10.0	2736	3396	660	24.12
	17.0	4572	5676	1104	24.15
	24.0	6324	7848	1524	24.10
	31.0	8022	9900	1878	23.41
	39.0				

Adjustment Equation

As seen from Tables 7, 8 and 9, the percentage increase in volume throughput of the segment by adding an auxiliary lane is essentially a fixed value for a particular number of through lanes. In addition, the proportional increase does not depend on weaving volume or interchange spacing. The average percentage increase in throughput volume based on number of lanes is shown in Table 10.

Table 10. Average Percentage Increase in Volume by Adding an Auxiliary Lane

Number of Through Lanes ***N***	Percentage Increase in Volume
2	48.87
3	32.03
4	23.81
5	18.71

Using the values obtained from CORSIM, two models were developed for the percentage increase in volume throughput due to auxiliary lane for a given number of mainline lanes. The general specification of the two models is given by:

Model 1: $percentage\ increase = 16.0 + 10 \times (5 - N)$

Model 2: $percentage\ increase = 65.4 - (10.0 \times N)$

where

N = Number of through lanes

The two models give very similar results. The key difference is that the first model implies that it is valid only for freeway segments with a maximum of five lanes. While this was the maximum number of lanes used in the test scenarios in this study, it is possible that this relationship will hold reasonably for freeway segments with more than five lanes. Thus, if one is comfortable with that notion, the second equation could be specified. Table 11, shows the comparison between the percentage increase in volume by adding an auxiliary lane obtained from CORSIM and the two models for given number of lanes.

Table 11. Comparison of Percentage Increase in Volume by Adding an Auxiliary Lane

Number of Through Lanes *N*	Percentage Increase in Volume		
	CORSIM	**Model 1**	**Model 2**
2	48.87	46.00	45.40
3	32.03	36.00	35.40
4	23.81	26.00	25.40
5	18.71	16.00	15.40

Appendix A: Overview of HCM 2010 Weaving Analysis Methodology

A recent National Cooperative Highway Research Program (NCHRP) Project (#3-75) resulted in the development of a new weaving segment analysis methodology, which will be incorporated in the next edition of the HCM (planned for release in late 2010).

Where HCM analyses were used in this study, the HCM 2010 methodology was used, given its imminent release. This new methodology has some significant differences from the HCM 2000 weaving analysis methodology. The remainder of this section will provide a brief overview of the HCM 2010 weaving analysis methodology.

Introduction

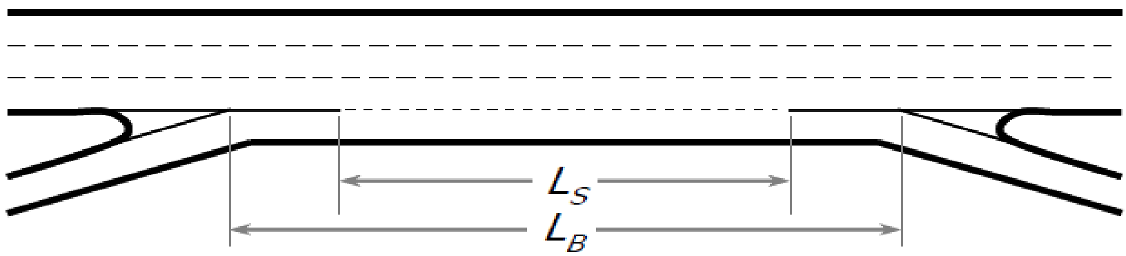
There are three geometric characteristics that affect a weaving segment's operating conditions:

- Length
- Width
- Configuration

Length is the distance between the merge and diverge forming the weaving segment. *Width* refers to the number of lanes within the weaving segment. *Configuration* is defined by the way entry and exit lanes are aligned with respect to each other. All have an impact on the critical lane-changing activity that is the unique operating feature of a weaving segment. The new proposed HCM 2010 methodology for analyzing the operation of weaving segments is based on these characteristics, as well as a segment's free-flow speed and the demand flow rates for each movement within a weaving segment.

Length of a Weaving Segment

There are two measures of weaving segment length that are relevant, short length (L_S) and base length (L_B). In HCM 2010 weaving methodology, short length (L_S) will be used in all the cases. The use of short length is not to suggest the lane-changing in a weaving segment is restricted to this length. Some lane-changing does take place over barrier markings and even painted gore areas but research has shown that short length is the better predictor of operating characteristics within the weaving segment.



L_S = Short length, the distance in feet between the end points of any barrier markings that prohibits or discourage lane-changing
L_B = Base length, the distance in feet between points in the respective gore areas where the left edge of the ramp traveled way and right edge of the freeway traveled way meet

If, no barrier markings are used in the weaving segment then in that case the two lengths are same, i.e., $L_S = L_B$. In dealing with future designs in which the details of markings are unknown, a default value should be based on general marking policy (e.g., $L_S = 0.77\,L_B$)

Maximum Weaving Length

Maximum length is the length at which weaving turbulence no longer has an impact on the capacity of the weaving segment. The maximum length of weaving (in ft) is computed as:

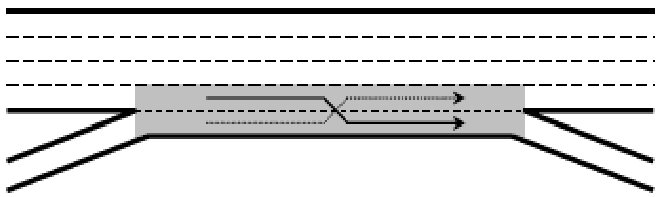
$$L_{MAX} = [5{,}728(1 + VR)^{1.6}] - [1{,}566 N_{WL}]$$

v_W= weaving demand flow rate in weaving segment, veh/h, $v_W = v_{RF} + v_{FR}$
v_{NW}= non-weaving demand flow rate in weaving segment, veh/h, $v_{NW} = v_{RF} + v_{FR}$
v = total demand flow rate in weaving segment, veh/h, $v = v_W + v_{NW}$
VR = volume ratio = v_W / v
N_{WL}= number of lanes from which a weaving maneuver may be made with one or no lane changes

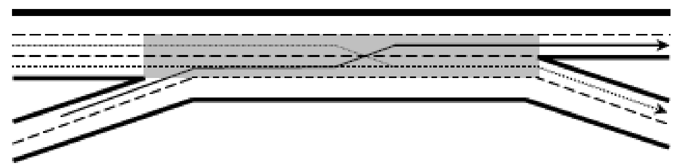
As *VR* increases, it is expected that influence of weaving turbulence would extend for longer distances. All values of N_{WL} are 2 or 3 (one-sided weaving segments). The value of L_{MAX} is used to determine whether the freeway segment can be analyzed as weaving segment or not.
If $L_s < L_{MAX}$, configuration should be analyzed as a weaving segment otherwise configuration should be analyzed as separate merge and the diverge junctions.

Configuration of a Weaving Segment

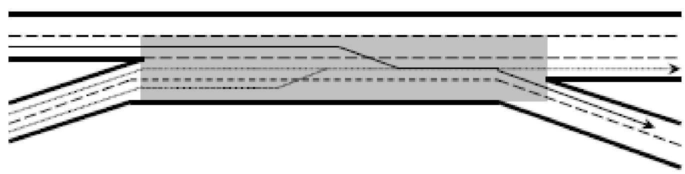
Configuration of a weaving segment refers to the way that entry and exit lanes are "linked." The configuration determines how many lane changes a weaving driver must make to successfully complete the weaving maneuver.



(a) Five-Lane Ramp-Weave Segment
($N_{WL} = 2$)



(b) Four-Lane Major Weave Segment without Lane Balance
($N_{WL} = 2$)



(c) Four-Lane Major Weave Segment with Lane Balance
($N_{WL} = 3$)

For standard auxiliary lane configurations (i.e., ramp-weave segment), N_{WL} is 2. Segments with $N_{WL} = 3$ exist in major weaving segments with lane balance at the exit gore. When $N_{WL} = 2$, even for on-ramp and off-ramp volumes equal to 5% of the mainline volume, the maximum weaving length L_{MAX} will be 3493 ft. Furthermore, when $N_{WL} = 3$, and the on-ramp and off-ramp volumes are greater than 12% of the mainline volume (which is likely to occur with a major weave segment), L_{MAX} will be greater than 3000 ft. Hence, for most practical conditions, when $L_S \geq L_{MAX}$, L_S will be greater than 3000 ft.

Figure 1 is a flowchart illustrating the basic steps that defines the HCM 2010 weaving analysis methodology for analyzing freeway weaving segments. The methodology uses several types of predictive algorithms, all of which are based upon a mix of theoretical and regression models.

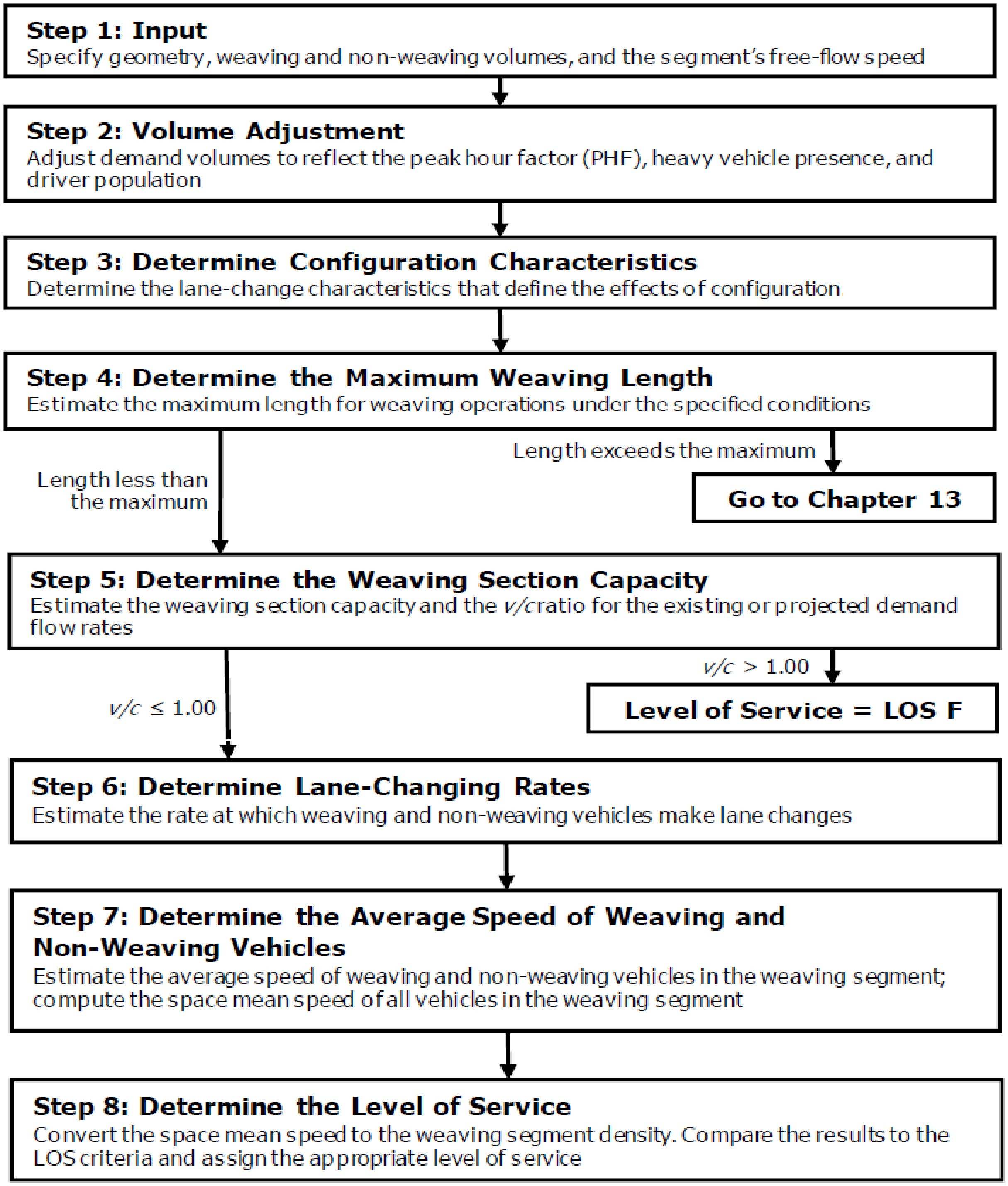


Figure 1. HCM 2010 Weaving Methodology Flowchart

Part B

FREEWAY SEGMENT CAPACITY ESTIMATION FOR FLORIDA FREEWAYS

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

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
RTMS	Remote Traffic Microwave Sensor
TTMS	Telemetered Traffic Monitoring Sensor
PTMS	Portable Traffic Monitoring Sensor

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 its 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
- Development of a simple, easy to apply, method that 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
- Obtaining traffic data for the respective chosen sites across Florida and preparing the data for the subsequent processing and analysis
- Processing and analyzing the data according to the selected/developed capacity estimation methods
- Comparison of the different methods for estimating the capacity values as well as comparison with the HCM values

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 can affect the capacity values on the freeway segment. Chapter 3 describes the selected methodologies used for estimating the capacity values, the data obtained for analysis, and the procedure applied for the site selection. Chapter 4 provides the results of the data analysis for the selected sites from each of the tested estimation methods. Chapter 5 provides a summary of the research study, the study conclusions, and recommendations for future research on this topic.

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: 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 uncongested 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 uncongested 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 uncongested 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 flows 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 depends 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 unreturnable 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 Lorenz and Elefteriadou (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 suggested that with a transition of traffic flow from an uncongested 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 capacity estimates from Van Aerde Model. These 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 VAM 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:

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

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 (h^{-1})

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:

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

where,

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

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

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:

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

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 grouped into different categories or sets. These sets are obtained and named as a set containing all the uncongested flow values. The various sets used for this approach are defined as follows:

- B: The traffic pattern is uncongested 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 uncongested 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 uncongested 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. 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

The HCM (2000) gives fixed values for the capacity of a freeway segment but does not consider the stochastic nature of the capacity values. However, different methods have been identified that can be used to estimate freeway segment capacity values. For example, one method estimates a breakdown probability distribution based on flow rates preceding breakdown events and accounts for the stochastic nature of capacity values. Another method fits a mathematical function to speed-flow data points, from which the apex of the function is taken as the value of capacity. Finally, the different factors (e.g., free-flow speed, merge/diverge areas) that may affect freeway segment capacity values were discussed.

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 for urban freeways. Along with the discussion on the analyses for urban freeways, a simple discussion is also provided for the rural freeways.

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 to 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 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 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) refer 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 to as PLM curves by Brilon et al. (2005) and 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, and had operations that were 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 freeway segments with high occupancy toll (HOT) or high occupancy vehicle (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 estimate freeway capacities by comparing different capacity estimation methods, 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 1-hour intervals, this data aggregation level is not appropriate for the determination of capacity values. Ideally, the data at one or five minutes 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), Courage and Lee (2009).

Apart from the basic freeway segments across the urban areas, several freeway segments outside the urban or metropolitan area were selected that had reasonable high flows. The candidate sites at rural locations were selected on the basis of maximum recorded flow rates on interstates across Florida, obtained from Florida Highway Data cum Traffic Information DVD (2008). The final sites were selected on the basis of the data availability at these locations.

Data Source

As the scope of this study is limited to 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 Florida freeways and within the scope of this research. Five districts were chosen for the data analysis: District 2 (Jacksonville), District 4 (Ft. Lauderdale), District 5 (Orlando/Daytona), District 6 (Miami), and District 7 (Tampa).

The traffic data were obtained from STEWARD (Statewide Transportation Engineering Warehouse for Archived Regional Data), a website-accessible database developed at the University of Florida and sponsored by Florida Department of Transportation. The database contains data for the urban areas of Jacksonville, Ft. Lauderdale, Orlando, Miami and Tampa. 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. To check the quality control and the performance of the data collected, the traffic counts from RTMS sensors were compared with the FDOT's PTMS detectors for which a detailed analysis and comparison is discussed later in this section. 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 September 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 3-23 provide the Google Earth images for all the selected sites with their respective site ID.

For the sites at the rural locations, the traffic data (counts and speed) were obtained from FDOT statistics office. The TTMS (Telemetered Traffic Monitoring Sensors), installed by FDOT on Florida roadways were used for collecting the traffic data. These detectors are originally configured to collect the daily data in fifteen minutes of data interval and archive it at its respective location. However, for the rural analysis, the TTMS detectors were re-configured to collect traffic data in five minutes interval instead of fifteen minutes. Due to data storage and hardware issues at the detector locations, two sites collected the traffic data in intervals of ten minutes and hourly traffic data. The data were collected from 11/25/2009 to 11/30/2009, to capture the high flows experienced at these locations due to holiday season. The final selected sites are listed in Table 3-3. Figures 3-24 to 3-33 provide the Google Earth images for all the selected rural freeway sites. The following section compares the traffic counts data as obtained from RTMS detectors with the FDOT installed telemetered sensors.

Comparison of RTMS counts and PTMS counts

The traffic data for the Portable Traffic Monitoring Sensors (PTMS) were obtained from the FTI DVD (2008). The historical data are stored by FDOT in form of synopsis reports for these sensors that are provided in the FTI DVD (2008). These reports were obtained for sites that are located in proximity of the location of RTMS detectors. In order to check the quality of the data collected by RTMS detectors, the traffic counts collected by these detectors were compared with the traffic counts as provided in the synopsis reports for PTMS detectors. Although, the data was available for a 24-hr period, the analyses/comparisons were performed only for AM peak and PM peak time period. A summary of all the comparisons performed between these two detectors are provided in Table 3-4.

A detailed comparison for a 24-hr period is provided in Appendix A, Table A-1 to Table A-5. These tables compare the RTMS counts with the PTMS counts from 5AM to 10 PM. The overall difference in the percentages of counts from these two detectors is also calculated. The percentage difference in the counts is observed to be less than 10% for almost all the sites for all the days of analysis, which indicates that the counts from the RTMS detectors are good. Since the data obtained for the research are good, the data are then processed in the next step. The following 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 that are necessary before 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.

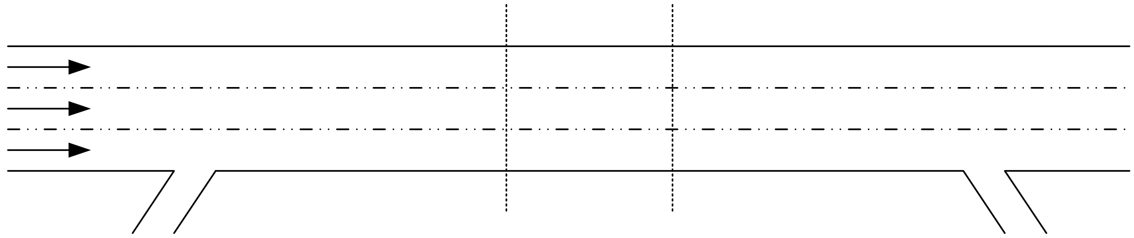
Downstream Breakdown Identifier

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.

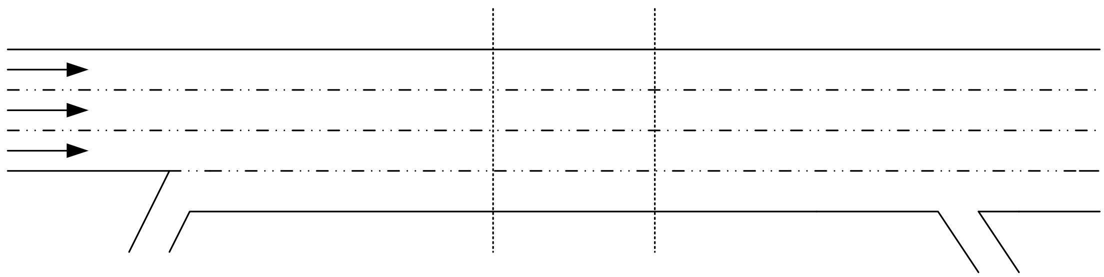
This utility program requires only two inputs: 1) the PLM data file as obtained from CDP for the desired basic freeway segment location, and 2) the results file from the capacity data processor for the detector at the downstream location. The PLM data at the basic freeway segment location is filtered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

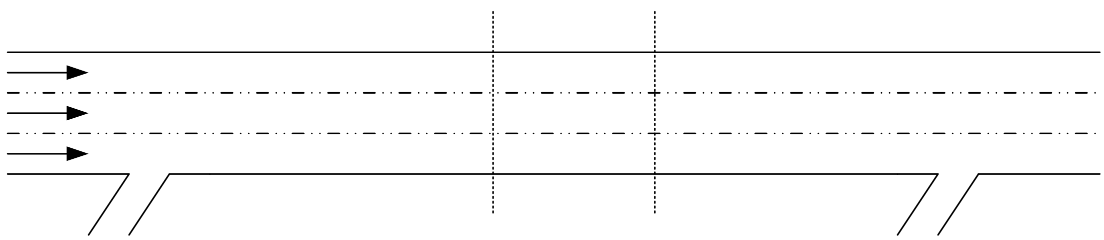
The output flow values, speed values and B-set values from this processor are further used for the PLM analysis. The data is checked and the PLM and Weibull curves are determined to estimate the capacity value as per the methodology. Figure 3-21 provides a snapshot of the PLM data processor used for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



A) Freeway segment with constant number of mainline lanes throughout the segment



B) Freeway segment at least 1500 ft in length with an unequal number of lanes before and after the on-ramp



C) Freeway segment at least 1500 ft in length with consecutive on-ramps

Figure 3-1. Acceptable freeway segment configurations

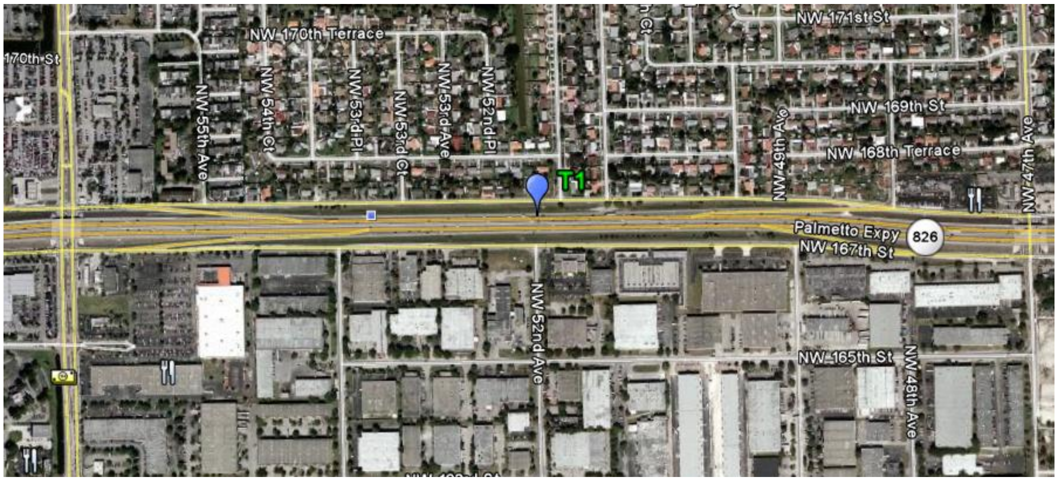


Figure 3-2. Aerial photo of site T1 (Source: Google Earth)

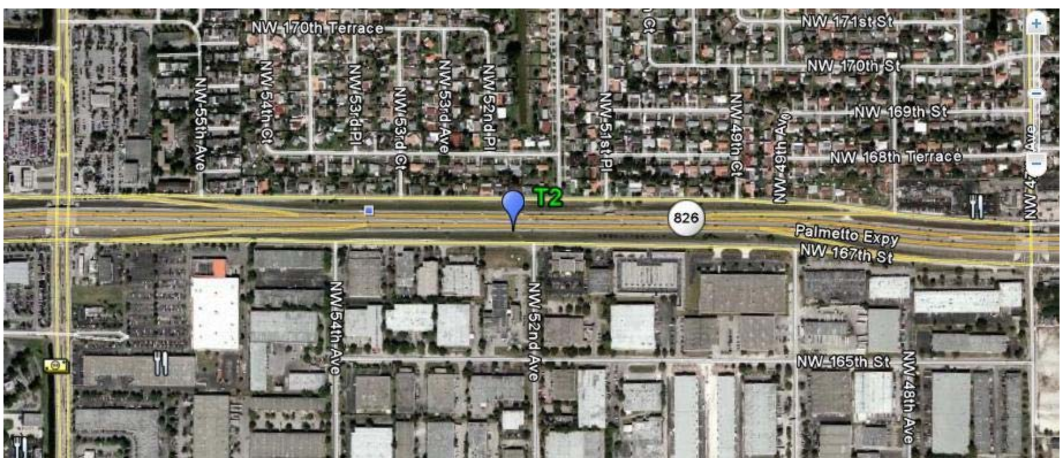


Figure 3-3. Aerial photo of site T2 (Source: Google Earth)

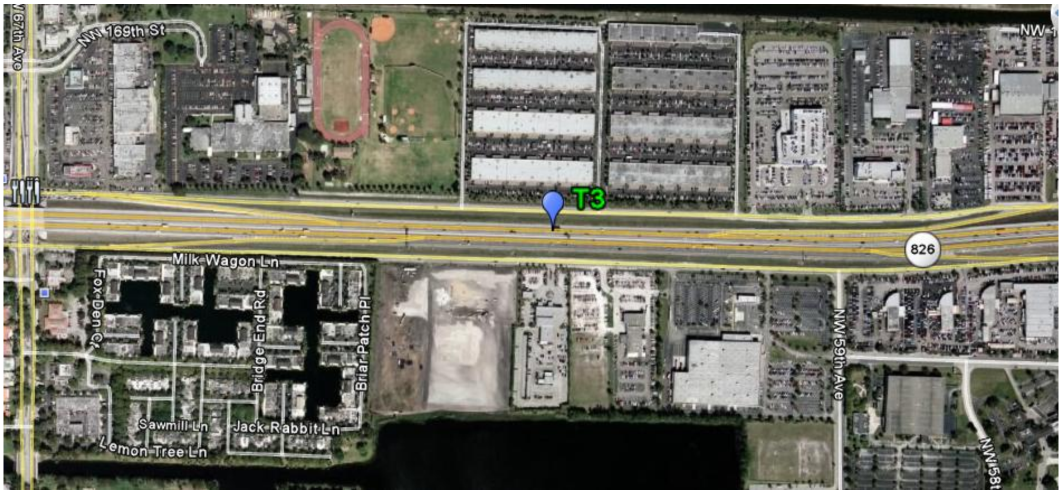


Figure 3-4. Aerial photo of site T3 (Source: Google Earth)

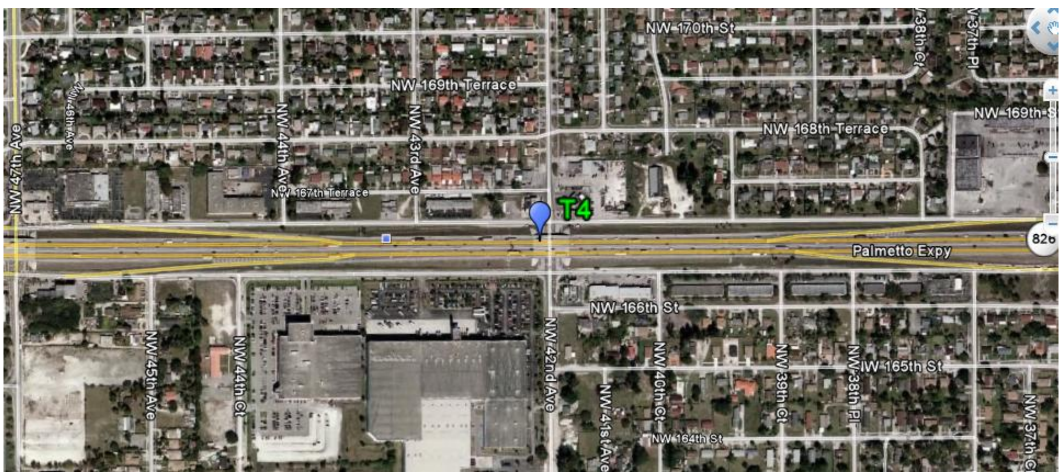


Figure 3-5. Aerial photo of site T4 (Source: Google Earth)

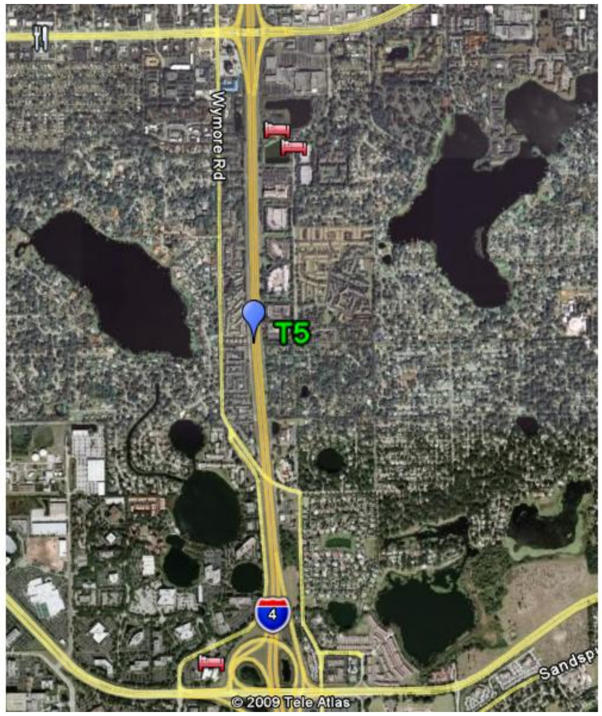


Figure 3-6. Aerial photo of site T5 (Source: Google Earth)

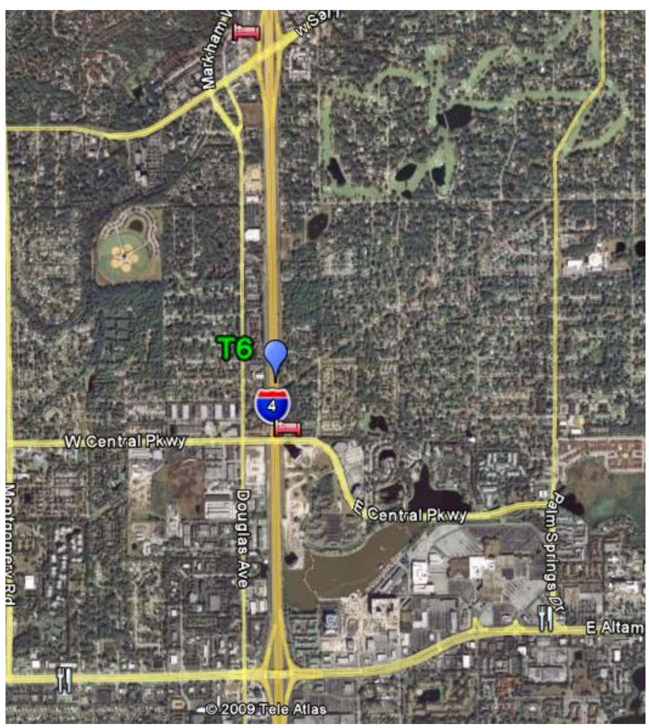


Figure 3-7. Aerial photo of site T6 (Source: Google Earth)

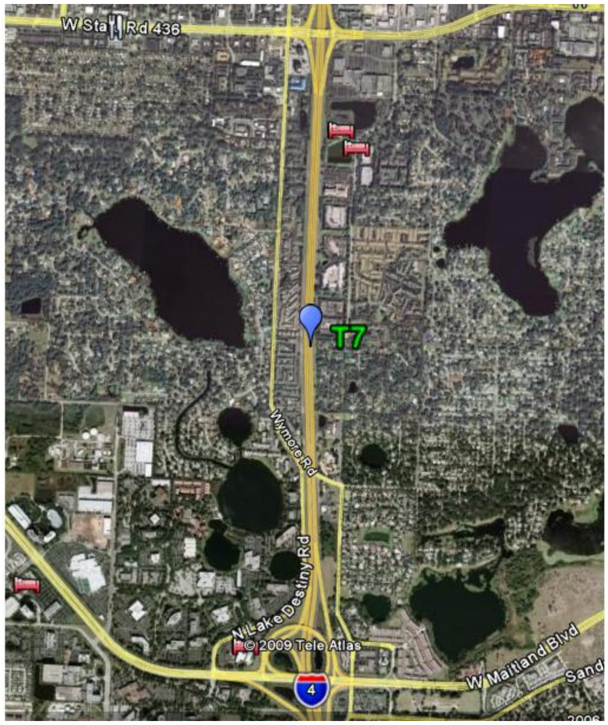


Figure 3-8. Aerial photo of site T7 (Source: Google Earth)

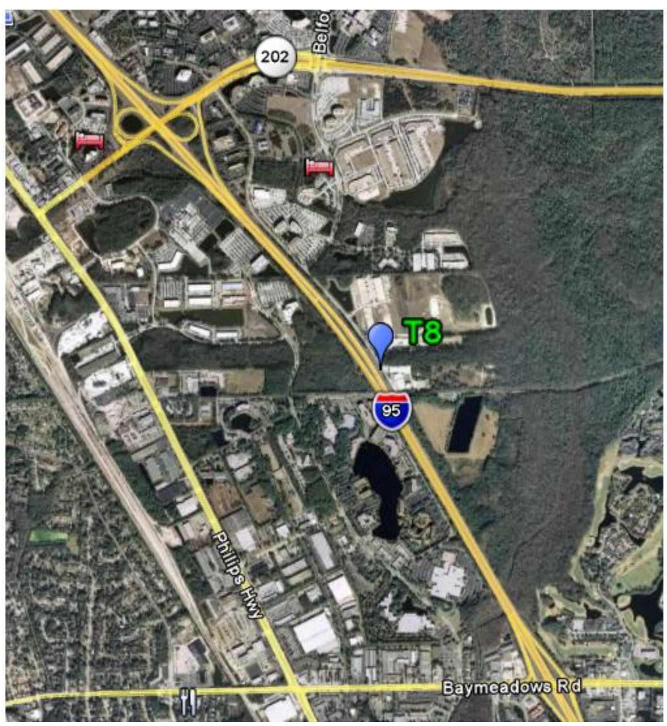


Figure 3-9. Aerial photo of site T8 (Source: Google Earth)

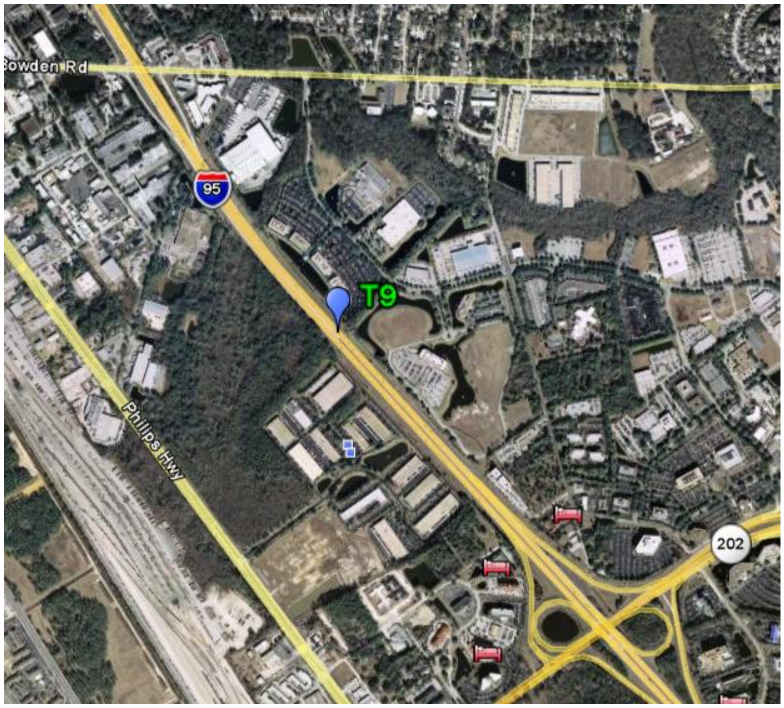


Figure 3-10. Aerial photo of site T9 (Source: Google Earth)

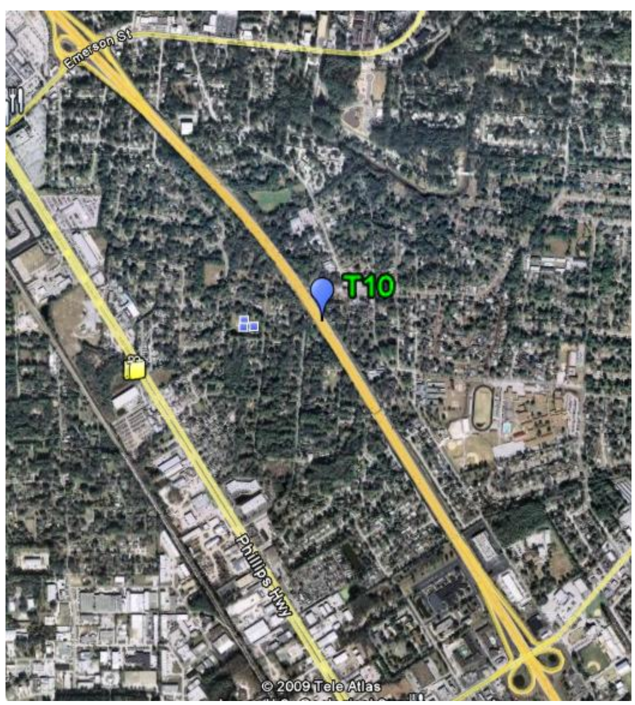


Figure 3-11. Aerial photo of site T10 (Source: Google Earth)

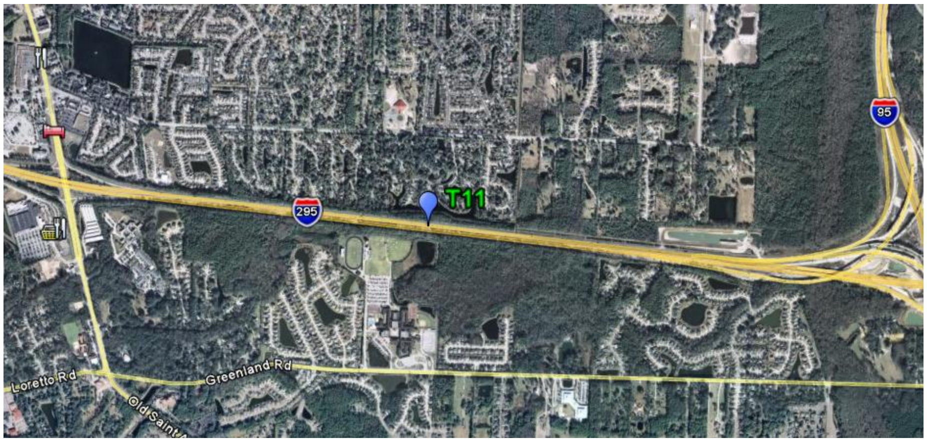


Figure 3-12. Aerial photo of site T11 (Source: Google Earth)

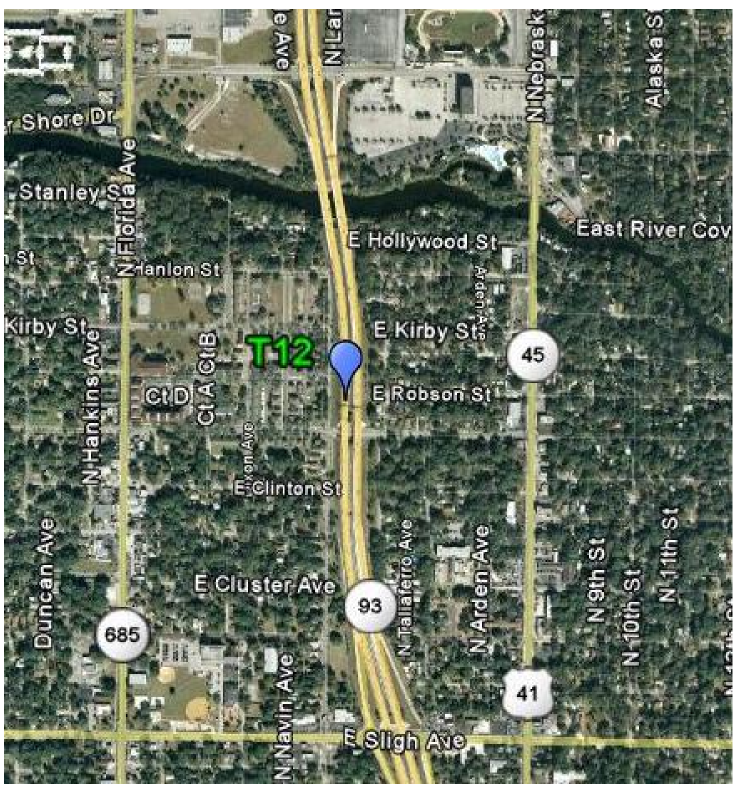


Figure 3-13. Aerial photo of site T12 (Source: Google Earth)

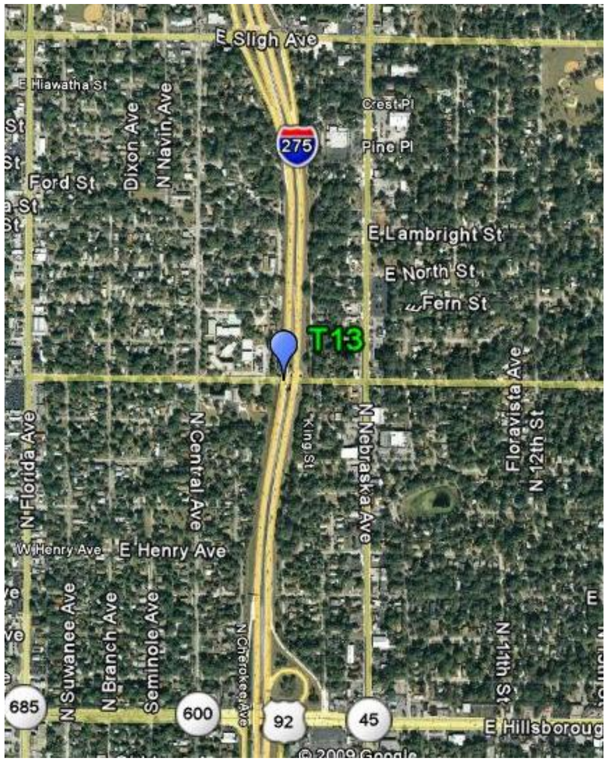


Figure 3-14. Aerial photo of site T13 (Source: Google Earth)

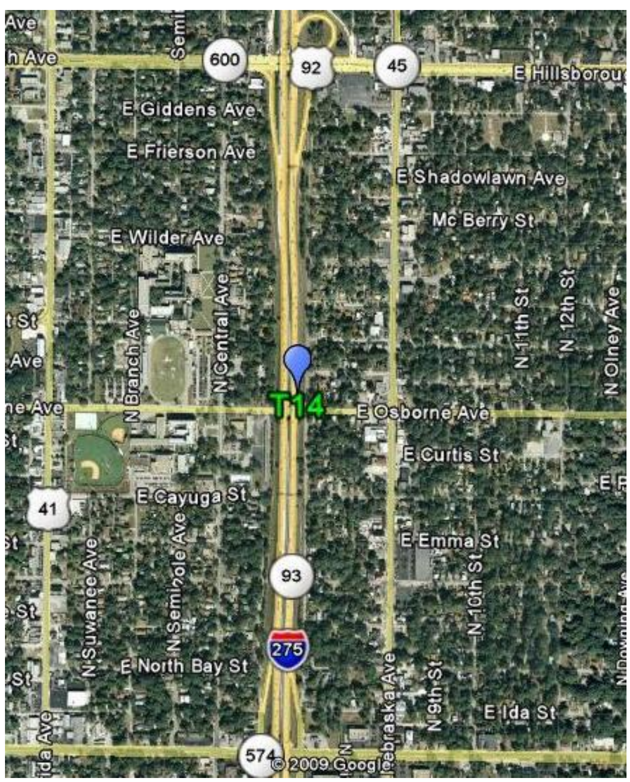


Figure 3-15. Aerial photo of site T14 (Source: Google Earth)

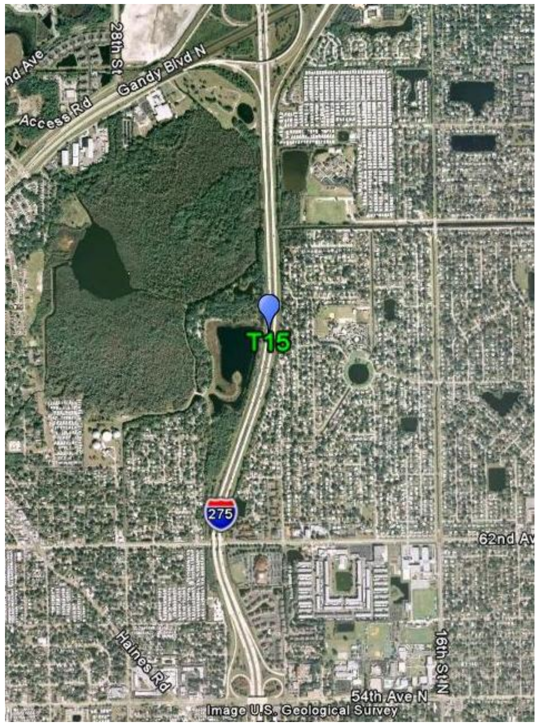


Figure 3-16. Aerial photo of site T15 (Source: Google Earth)

Figure 3-17. Aerial photo of site F1 (Source: Google Earth)

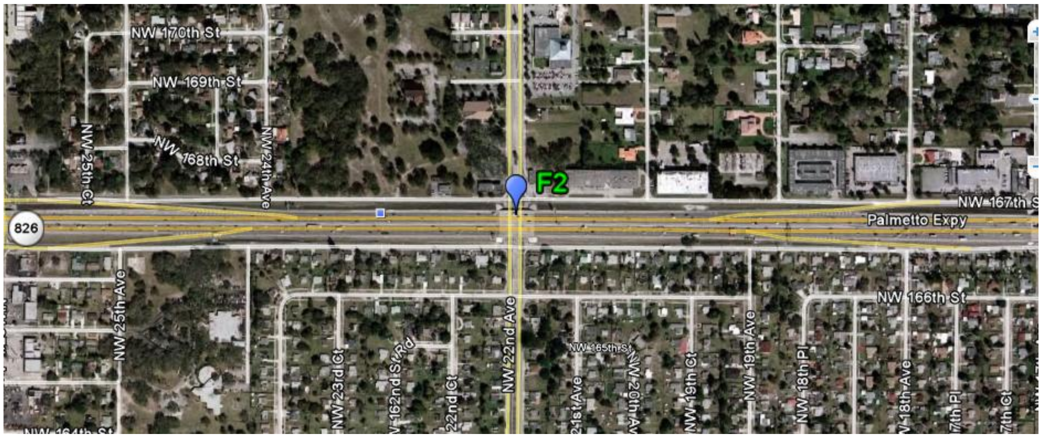


Figure 3-18. Aerial photo of site F2 (Source: Google Earth)

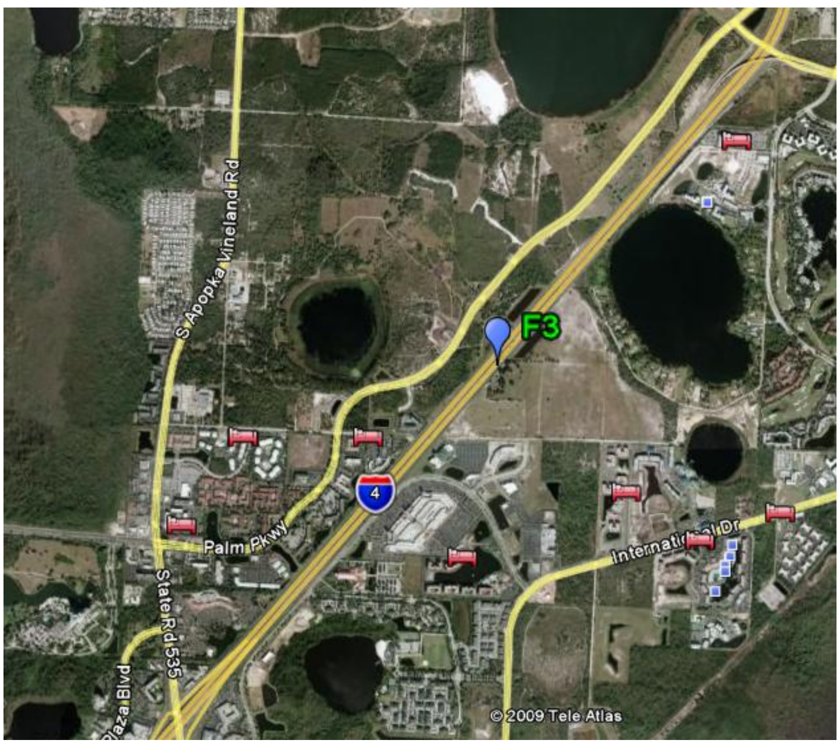


Figure 3-19. Aerial photo of site F3 (Source: Google Earth)

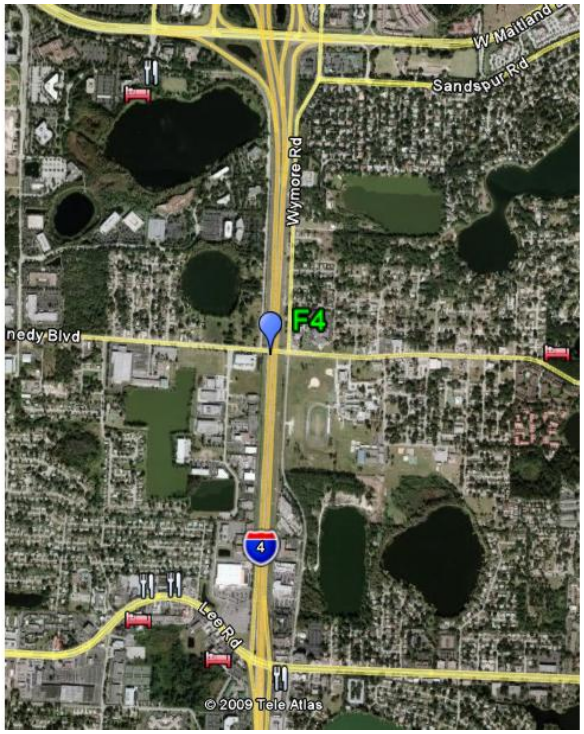


Figure 3-20. Aerial photo of site F4 (Source: Google Earth)

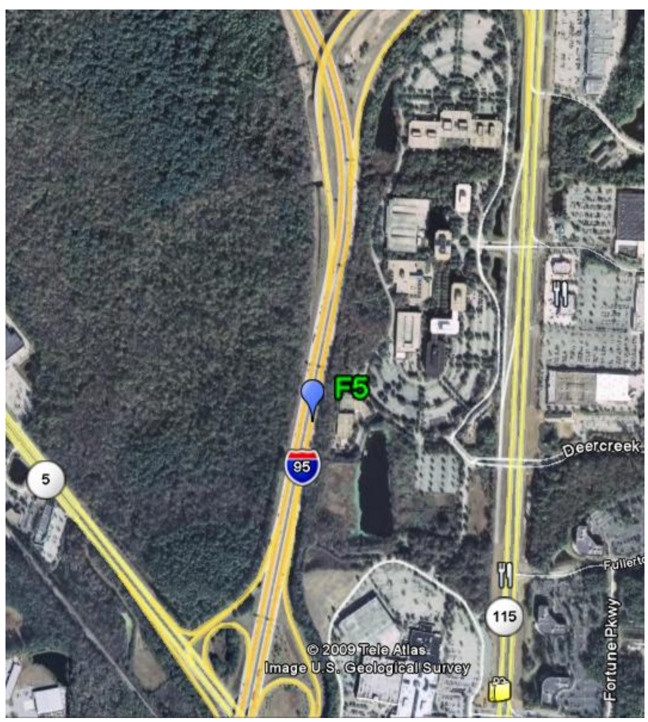


Figure 3-21. Aerial photo of site F5 (Source: Google Earth)

Figure 3-22. Aerial photo of site F6 (Source: Google Earth)

Figure 3-23. Aerial photo of site FV1 (Source: Google Earth)

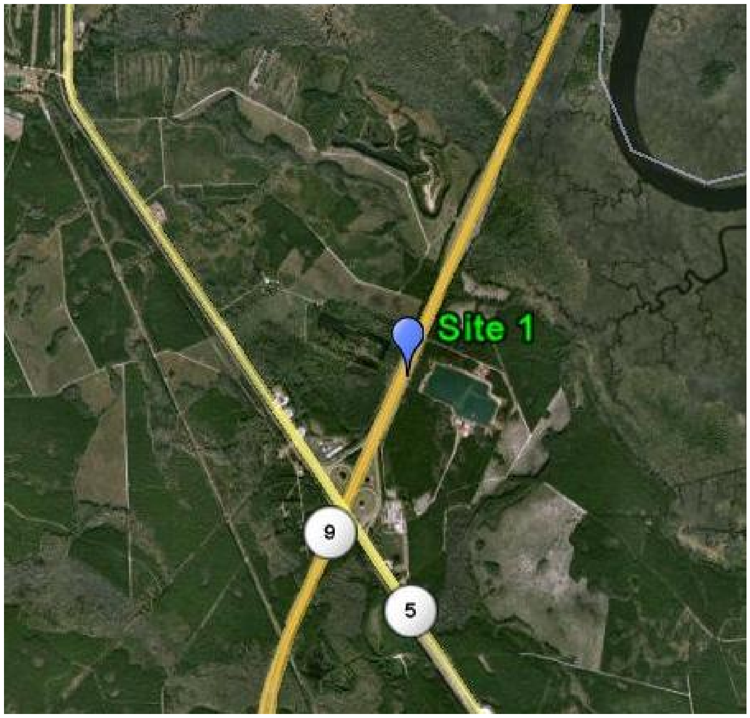


Figure 3-24. Aerial photo of rural site 1 (Source: Google Earth)

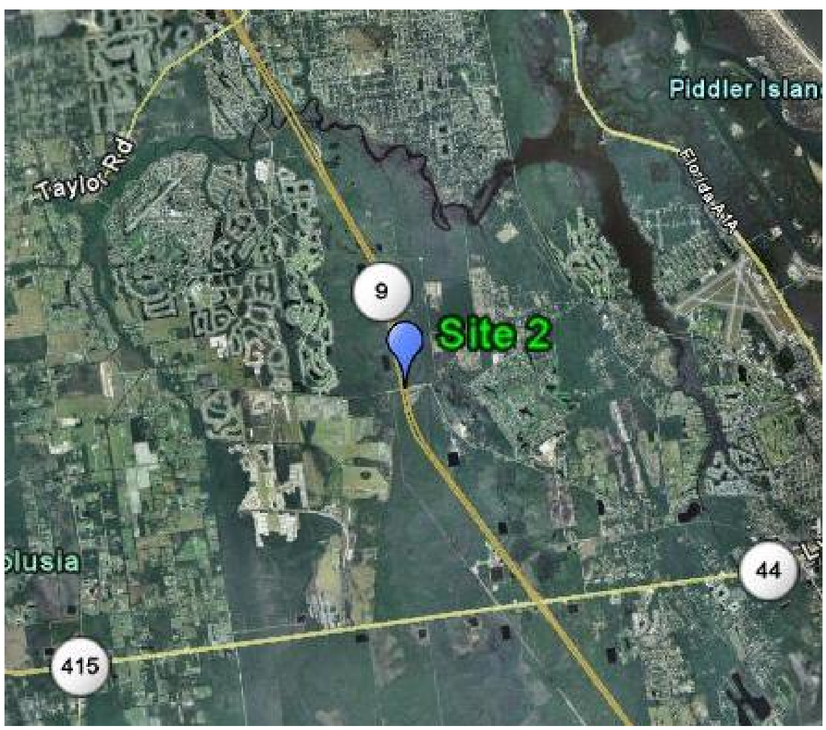


Figure 3-25. Aerial photo of rural site 2 (Source: Google Earth)

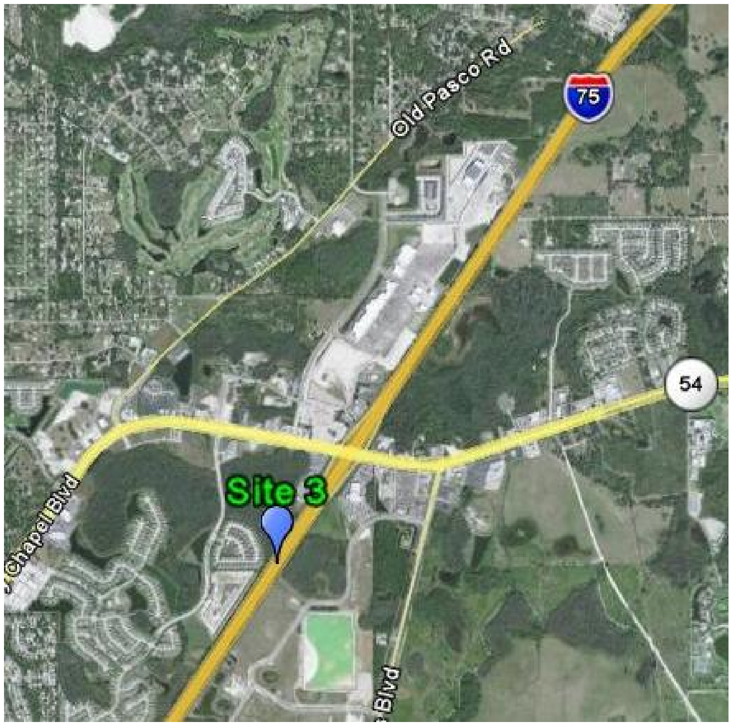


Figure 3-26. Aerial photo of rural site 3 (Source: Google Earth)

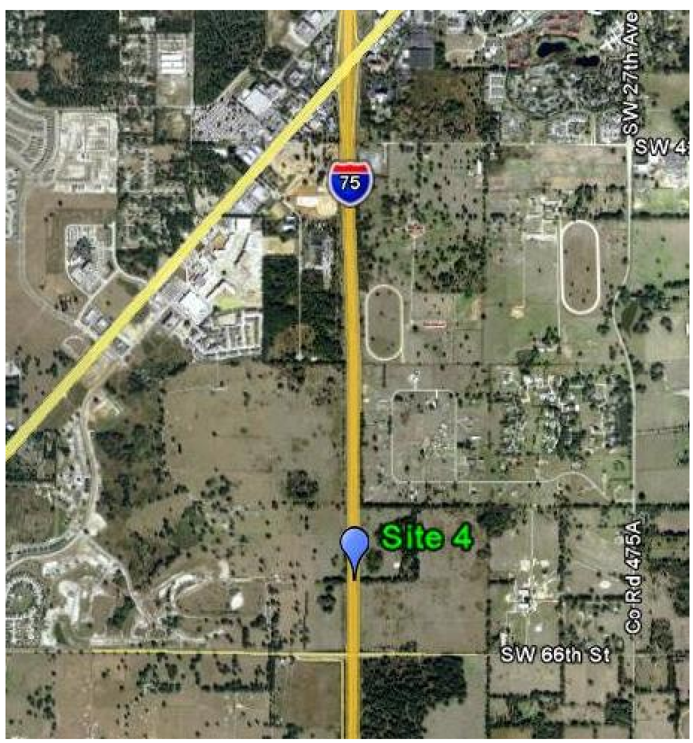


Figure 3-27. Aerial photo of rural site 4 (Source: Google Earth)

Figure 3-28. Aerial photo of rural site 5 (Source: Google Earth)

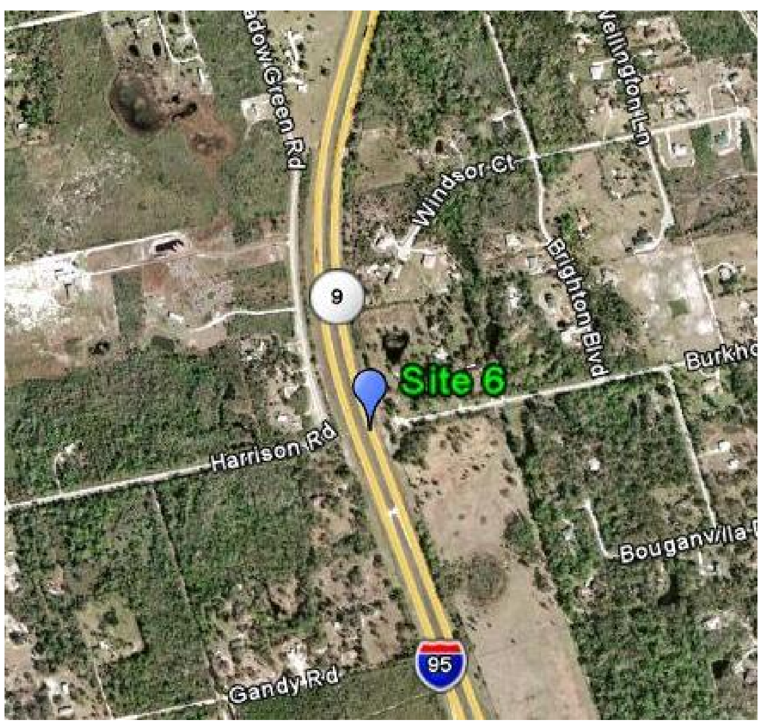


Figure 3-29. Aerial photo of rural site 6 (Source: Google Earth)

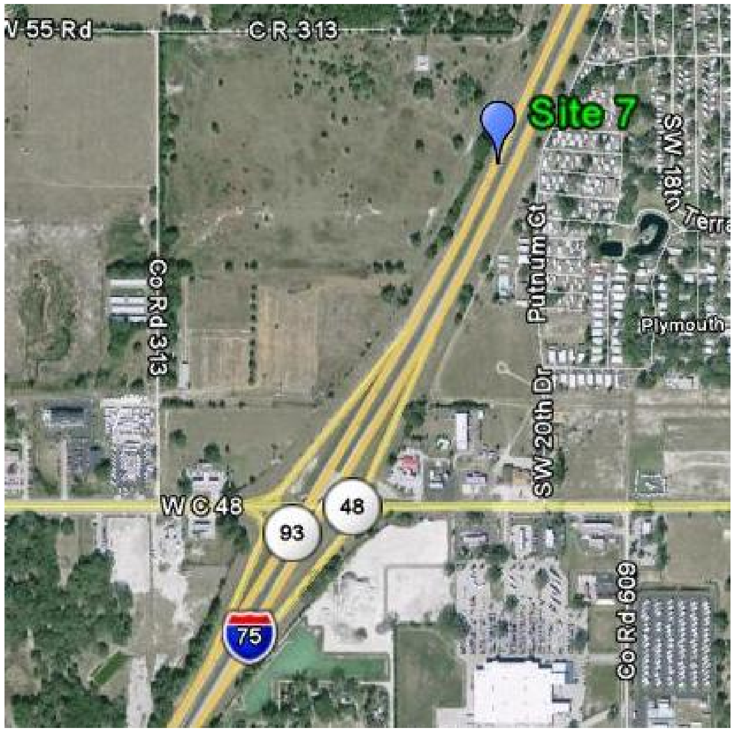


Figure 3-30. Aerial photo of rural site 7 (Source: Google Earth)

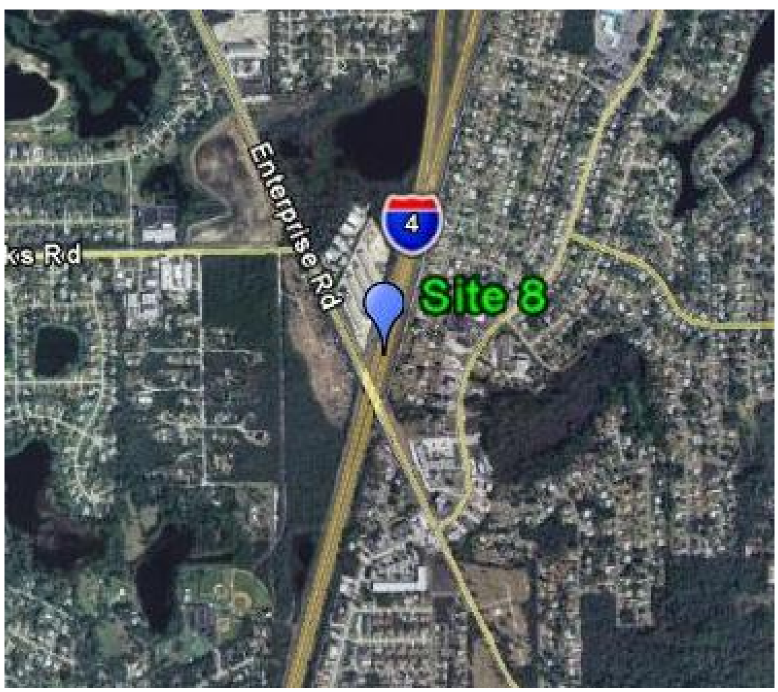


Figure 3-31. Aerial photo of rural site 8 (Source: Google Earth)

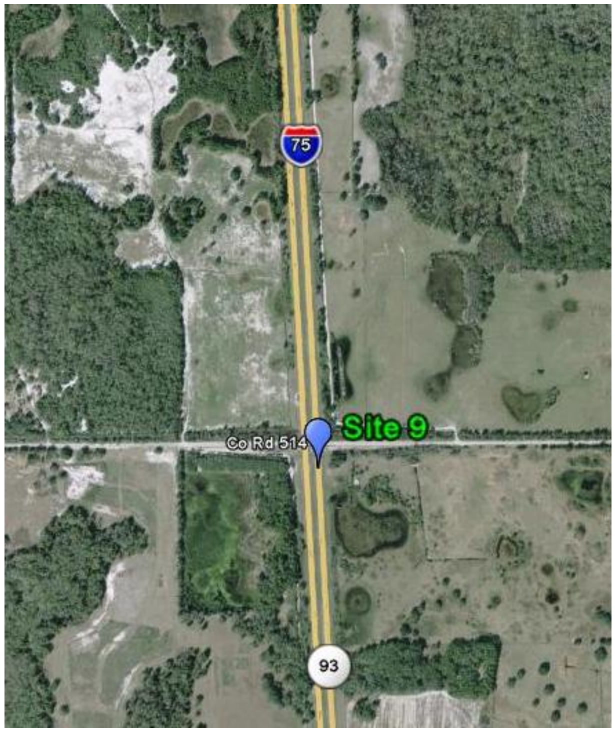


Figure 3-32. Aerial photo of rural site 9 (Source: Google Earth)

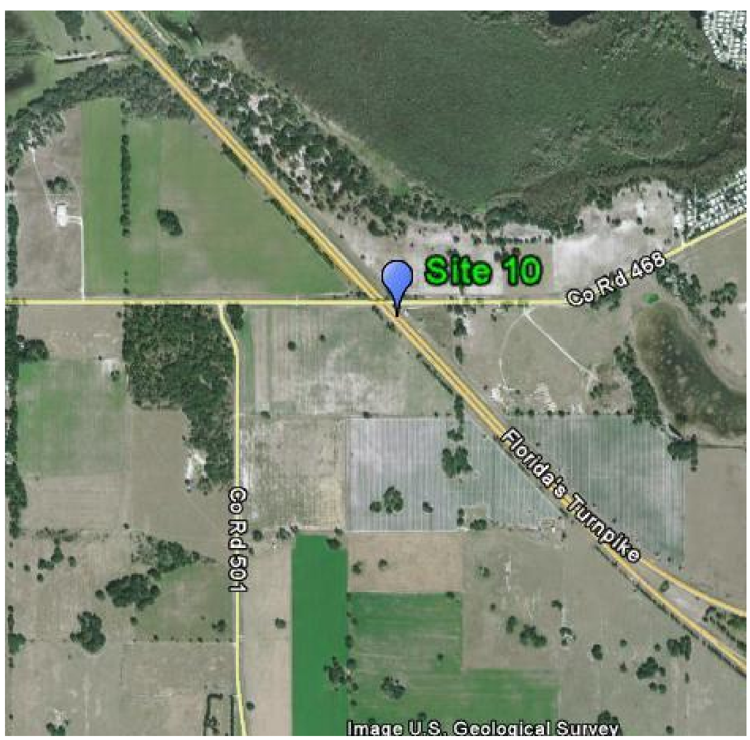


Figure 3-33. Aerial photo of rural site 10 (Source: Google Earth)

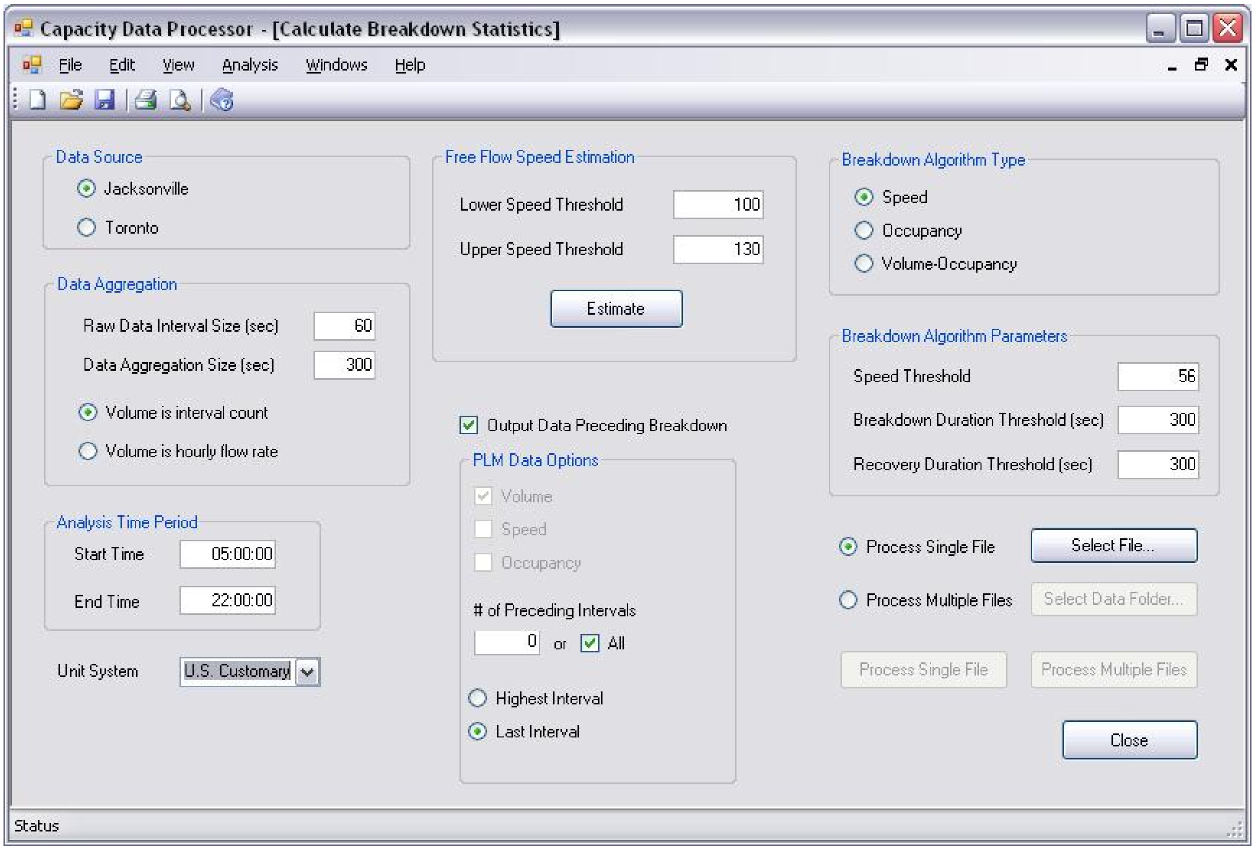


Figure 3-34. Capacity Data Processor utility program user interface

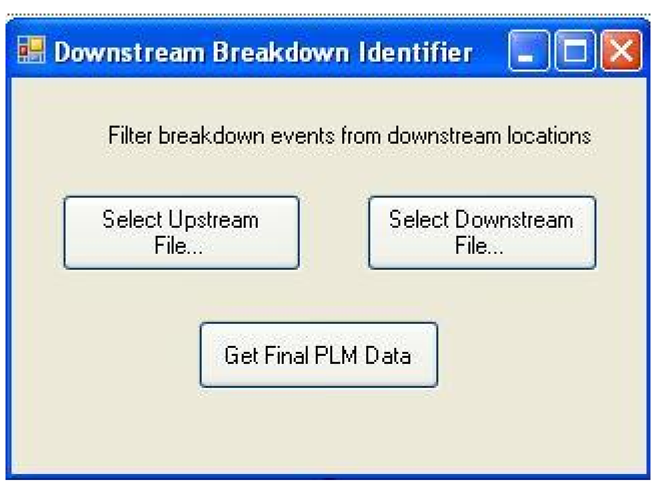


Figure 3-35. Downstream Breakdown Identifier utility program user interface

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	1.80	0.71	1.67	1.04
11/1/2007	0:01:00	210371	61.86	7	1.50	2.12	0.00	0.00
11/1/2007	0:02:00	210371	58.31	16	2.20	0.71	2.33	1.04
11/1/2007	0:03:00	210371	59.00	14	3.80	0.00	4.00	1.22
11/1/2007	0:04:00	210371	59.75	16	3.20	3.54	2.67	1.12
11/1/2007	0:05:00	210371	60.14	14	2.60	0.00	1.25	1.07
11/1/2007	0:06:00	210371	60.44	18	4.30	0.71	1.40	1.06
11/1/2007	0:07:00	210371	60.12	17	2.40	2.83	1.75	1.15
11/1/2007	0:08:00	210371	60.27	11	2.00	3.54	1.33	1.16
11/1/2007	0:09:00	210371	62.38	21	5.20	3.54	1.33	1.15
11/1/2007	0:10:00	210371	62.75	16	1.80	1.41	3.50	1.08
11/1/2007	0:11:00	210371	63.38	16	1.80	4.24	2.67	1.14
11/1/2007	0:12:00	210371	65.50	10	2.00	4.24	5.00	1.19
11/1/2007	0:13:00	210371	62.06	16	2.50	2.12	1.20	1.09
11/1/2007	0:14:00	210371	67.80	15	2.00	6.36	2.67	1.11
11/1/2007	0:15:00	210371	66.47	15	2.00	1.41	1.50	1.06
11/1/2007	0:16:00	210371	64.63	8	1.50	0.71	2.00	1.11
11/1/2007	0:17:00	210371	65.46	13	2.00	2.83	1.25	1.13
11/1/2007	0:18:00	210371	68.62	13	2.70	0.00	3.00	1.21
11/1/2007	0:19:00	210371	64.88	16	3.20	0.71	1.50	1.24
11/1/2007	0:20:00	210371	63.89	18	2.50	3.54	1.40	1.21
11/1/2007	0:21:00	210371	64.18	11	2.00	0.00	5.00	1.25
11/1/2007	0:22:00	210371	65.07	15	2.20	3.54	1.50	1.35
11/1/2007	0:23:00	210371	61.68	22	3.00	7.07	2.00	1.23
11/1/2007	0:24:00	210371	63.57	14	1.80	1.41	2.33	1.09
11/1/2007	0:25:00	210371	65.38	13	4.00	2.12	6.00	1.10
11/1/2007	0:26:00	210371	63.74	19	2.70	0.71	2.67	1.04
11/1/2007	0:27:00	210371	62.21	14	2.20	2.12	1.50	1.07
11/1/2007	0:28:00	210371	61.00	6	1.80	0.00	0.00	0.00
11/1/2007	0:29:00	210371	62.55	11	2.80	0.71	1.33	1.10
11/1/2007	0:30:00	210371	63.56	16	4.60	0.71	2.67	1.04

Table 3-2. Final selected sites and site description

Site ID	Florida District	Urban Area	Site Description and Location	Direction	Freeway	Speed[1] (mph)	Data[2]	Type[3]
3 Lanes								
T1	6	Miami	East of NW 57 Avenue	EB	SR-826	55	Aug08-Jan09	A
T2	6	Miami	East of NW 57 Avenue	WB	SR-826	55	Aug08-Jan09	A
T3	6	Miami	East of NW 67 Avenue	EB	SR-826	55	Aug08-Jan09	A
T4	6	Miami	East of NW 47 Avenue	EB	SR-826	55	Aug08-Jan09	A
T5	5	Orlando	East of Wymore Rd	WB	I-4	55	Mar08-Nov08	A
T6	5	Orlando	East of SR 436	EB	I-4	55	Mar08-Nov08	A
T7	5	Orlando	East of Wymore Rd	EB	I-4	55	Mar08-Nov08	A
T8	2	Jacksonville	Between Baymeadows and Butler Blvd	NB	I-95	70	Jul07-Feb08	A
T9	2	Jacksonville	North of Butler Blvd	NB	I-95	70	Jul07-Feb08	A
T10	2	Jacksonville	South of Spring Glen Road	NB	I-95	70	Jul07-Feb08	A
T11	2	Jacksonville	Between Old St. Augustine Rd and I-95	NB	I-295	70	Jul08-Feb09	A
T12	7	Tampa	Between Bird St and Sligh Ave	SB	I-275	55	Jan09-Sep09	A
T13	7	Tampa	Between Sligh Ave and Hillsborough Ave	SB	I-275	55	Jan09-Sep09	A
T14	7	Tampa	Between Hillsborough Ave and MLK Jr Blvd	NB	I-275	55	Jan09-Sep09	A
T15	7	Tampa	Between Gandy Blvd and 54th Ave	SB	I-275	65	Jan09-Sep09	A
4 Lanes								
F1	6	Miami	South of NW 170 ST	NB	I-75	70	Aug08-Jan09	A
F2	6	Miami	East of NW 27 Avenue	NB	SR-826	55	Aug08-Jan09	B
F3	5	Orlando	West of Central Florida Parkway	EB	I-4	70	Mar08-Nov08	A
F4	5	Orlando	At Kennedy Blvd	WB	I-4	55	Mar08-Nov08	C
F5	2	Jacksonville	North of Philips Highway	NB	I-95	70	Jul07-Feb08	B
F6	2	Jacksonville	North of I-295S	NB	I-95	70	Jul07-Feb08	B
5 Lanes								
FV1	4	Ft. Lauderdale	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

Table 3-3. Selected rural freeway sites and description

Site ID	Lanes	County	County	Site	Location Description	Interstate	Data[1]	Roadway ID	Rdwy Mile
Site 1	3	Nassau	74	0132	2.0 Mile S of GA State Line	I-95	25/11/09-30/11/09	74160000	10.038
Site 2	2	Volusia	79	0133	2.7 Mile N of SR 44 @CR44	I-95	25/11/09-30/11/09	79002000	19.000
Site 3	2	Pasco	14	0190	0.6 Mile S of SR 54	I-75	25/11/09-30/11/09	14140000	4.500
Site 4	3	Marion	36	0317	0.35 Mile N of William Road	I-75	25/11/09-30/11/09	36210000	12.270
Site 5	3	Columbia	29	0320	Between I-10 and US-90	I-75	25/11/09-30/11/09	29180000	22.421
Site 6	3	Brevard	70	0332	0.9 Mile S of Aurantia Rd	I-95	25/11/09-30/11/09	70225000	25.800
Site 7	2	Sumter	18	0358	North of SR 48	I-75	25/11/09-30/11/09	18130000	8.160
Site 8	3	Volusia	79	9906	169' E of Enterprise Road	I-4	25/11/09-30/11/09	79110000	4.668
Site 9	2	Sumter	18	9920	At Co Rd 514	I-75	25/11/09-30/11/09	18130000	17.589
Site 10	2	Sumter	97	9931	Just South of SR 91 or Co Road 468	Turnpike	25/11/09-30/11/09	18470000	3.364

[1]Data available for analysis

Table 3-4. Summary of comparisons of RTMS counts with PTMS counts

					AM Peak		PM Peak	
Facility	RTMS Site	RTMS Site Description	PTMS Site	PTMS Site Description	Day 1	Day 2	Day 1	Day 2
I-4	T5	East of Wymore Road	75-3080	I-4, 0.635 miles NE of SR-414	11.27	-	5.06	-
I-595	FV1	At SW 26th Terrace/New River	86-2806	0.8 mile E of SR 7/US441, I-595	10.55	-	14.15	-
I-295	F6	North of I-295S	72-0864	0.5 mile of SR-5 (US1), I-95	5.90	-	3.84	-
SR-826	T2	East of NW 57 Avenue	87-0405	East of NW 57 Avenue	44.35	3.44	8.10	4.49
SR-826	F2	East of NW 27 Avenue	87-0579	East of NW 27 Avenue	8.02	8.45	8.66	8.09
I-75	F1	South of NW 170th St.	87-2501	200' S. Miami Gardens	2.23	-	13.60	-

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. The individual analyses are followed by providing the capacity estimates from all the three methods, followed by comparisons of capacity estimates from these three capacity estimation methods with the capacity values provided by HCM. Apart from the analysis on the selected urban freeways, a simple analysis is also performed for rural freeways. In the end, an alternate method to estimate the capacity value is discussed and provided.

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 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 4th 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 70% and 65% of the maximum flow rate observed for the freeway segment. The respective lower flow rate that corresponds to 70% and 65% of the maximum flow rate was calculated. The average of all flow rates that are within the 70% and 65% 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 70% and 65% of the maximum flow rate observed on the freeway segment, the capacity estimates on the basis of these two lower flow rates, 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 Geistefeldt (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 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 65% 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 70% 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 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 period with high flow rates within the analysis period. Therefore, the consistency of the averaging schemes is maintained only for data sets of at least two months in duration.

Percentile of Maximum Flow-Rates Estimation Method

From the discussions and results of the previous capacity estimation methods (i.e., Van Aerde Model (VAM) estimation method and stochastic estimation method), it was felt that the VAM capacity estimates were generally too low and the stochastic method capacity estimates were generally too high. For the VAM capacity estimation results, it was observed from the speed-flow data plots that the capacity estimates were generally at the lower end of the region containing the highest flow rates. For the stochastic estimation method, reliable capacity estimates were observed only for sites with 0.5 or more breakdown events per day and the estimates were very high for anything other than very small percentile thresholds. Therefore, in an attempt to determine “reasonable” capacity estimates across Florida freeways, an alternative approach is developed that seeks to find a compromise in the estimated capacity values between the VAM and stochastic estimation methods. This approach uses a simple methodology based on a selected percentile value of the highest observed flow rates. The flow rates at different percentile values are calculated for a subset of flow rates containing the highest flow-rates, with the most appropriate percentile value giving the capacity values.

The first step in this approach is to determine the subset of flow rates that constitute high flow rates. While the maximum flow rate is obviously used for the upper bound of this subset, identifying the lower bound is not so clear cut. Two approaches can be performed to estimate/calculate the lower boundary value. One approach is to estimate the flow rate by visually selecting a point from the plot of speed-flow data points that is contained within the critical mass of data points corresponding to the highest flow rates. However, this method was not applied as the size of the plotted data points and resolution of the plot can affect the selection of the point and is also somewhat subjective. The chosen approach was a simple mathematical procedure. In this approach, the lower bound is determined by selecting a flow rate that corresponds to a certain percentile of a subset of the total number of flow rate values. However, given that the selection of this percentile value can also be subjective, the percentile value was based on the Van Aerde Model (VAM) estimated capacity values, as described in more detail below.

In selecting the lower bound of the subset containing the highest flow rates, the first step was to aggregate the flow-rate data into five-minute and fifteen-minute intervals. The flow rate data in five minute and fifteen minute data interval were then converted to hourly flow rates (i.e., multiplying by 12 and 4 respectively). The data were then sorted from highest flow rates to lowest flow rates. The next step was to calculate the average of top x% highest flow rates for both aggregated data intervals. The x value for each selected site for two data aggregation intervals is calculated such that the average of top x% highest flow rates corresponds to the respective VAM capacity estimates. On the basis of the range and the average of x values of all the sites, two specific values of *x*, 5% and 6.5%, were chosen for rest of the analyses. Two different subsets were created with different lower bound values, each corresponding to the two *x* values. The lower bounds for the two subsets were calculated as the average of top 5% and 6.5% highest flow rates for both five-minute and fifteen-minute data intervals.

After the lower boundary values were calculated for the chosen *x* values, flow rates were estimated at multiple percentile values within the subset containing the highest flow rates. However, in order to avoid potentially erroneous data entries, all flow rates above 2700 veh/hr/ln were excluded. Based on the upper bound values (i.e., highest flow rate observed, less than or equal to 2700) and lower bound values, the remaining analyses were performed. The flow rates were calculated for 55^{th} through 85^{th} percentile values at an increment of 5 percent. For example, the 75^{th} percentile value within this subset corresponds to the flow rate value for which 75% of the flow rate values in this subset are less than. Table 4-10 to Table 4-13 provides flow rates at different percentile values for the two specific *x* values. The "VAM" represents the capacity estimates from VAM estimation method. The "Top *x*%" represents the *x* value such that the average of top *x*% highest flows corresponds to the VAM capacity estimates. The "Average: Top 5%" column represents the lower boundary values for the respective sites. The "Percentile Values" columns identify the flow rate corresponding to the specific percentile value.

The next step for the analyses is to select a set of set of capacity estimates that would fairly represent the traffic conditions at the respective freeway locations. Different sets of aggregated data are available to determine the capacity values, but the use of five-minute data aggregation intervals is most appropriate to estimate the capacity values for a freeway segment. Therefore, it is recommended to use the set of capacity values that are determined by analyzing the 5-minute aggregated data. Also, the use of the average of the top 6.5% flow rates is preferred as this percentage yields capacity estimates closest to those estimated by the VAM method. Thus, Table 4-11 is recommended to be used for selecting the estimated capacity values for the selected freeway segments. Considering the variance in selecting the percentile value within this table, a value between 75th and 85th percentile for areas of District 2 (Jacksonville), District 5 (Orlando) and District 7 (Tampa) looks reasonable. For other District 4 (Ft. Lauderdale) and District 6 (Miami), due to limited resources of data on several freeways and inclusion of HOT/HOV lanes, an appropriate percentile value is not reachable. However, any percentile value between, and including, the 60th to 85th percentile values would fairly represent the capacity values for freeway segments at these locations.

Rural Freeway Analysis

The rural freeway analysis was performed by simply determining the highest flow rate observed on the selected freeway segment within the five days of available traffic counts and speed data. The data were aggregated into five-minute intervals (except for sites 2 and 8, as explained earlier in chapter 3 under the data collection section) and the highest hourly flow rates for all the selected sites were calculated on the basis of 5-minute, 15-minute, and 60-minute aggregations. The time of day and the date were also obtained at which the highest flow rate was observed. Tables 4-14, 4-15 and 4-16 provide the maximum flow rates observed at the selected sites for 5-, 15- and 60-minute aggregations, respectively. All the analyses were done separately for each direction of the roadway.

After observing the speed-flow plots, most of the sites were found to be uncongested even during the highest flow-rate periods. On this basis, it is highly likely that the capacity of the respective roadway segment was never reached. However, for two sites, congestion was observed from the speed-flow plots and hence, it is likely that the capacity of those respective roadways were reached. However, it is possible that the observed breakdowns were due to an incident, weather, or some other "external" influence. Figures 4-29 and 4-30 provide the speed-flow plots for the sites that were found to be congested. For these sites, the maximum flow rate with a five-minute data aggregation interval was found to be 1974 veh/h/ln and 1824 veh/h/ln, respectively.

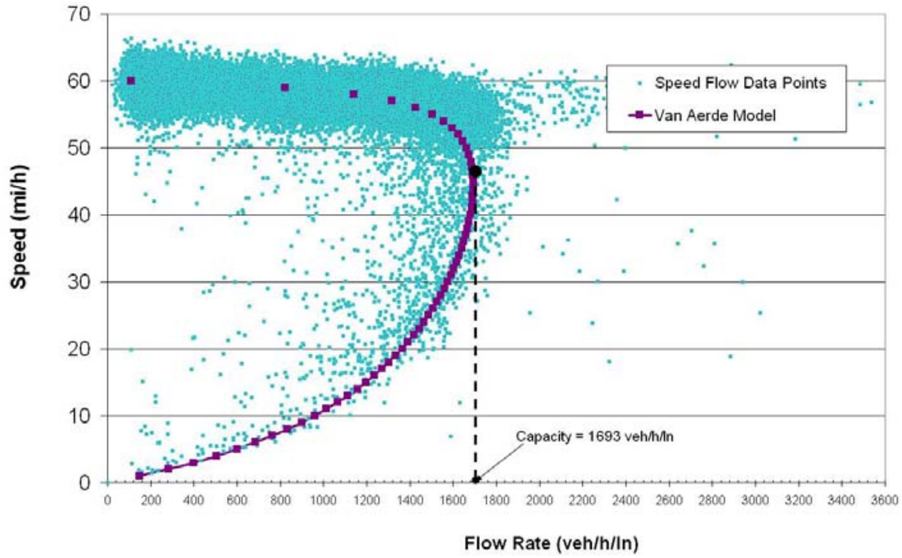


Figure 4-1. Van Aerde Model fit to the speed flow data points for site ID: T1

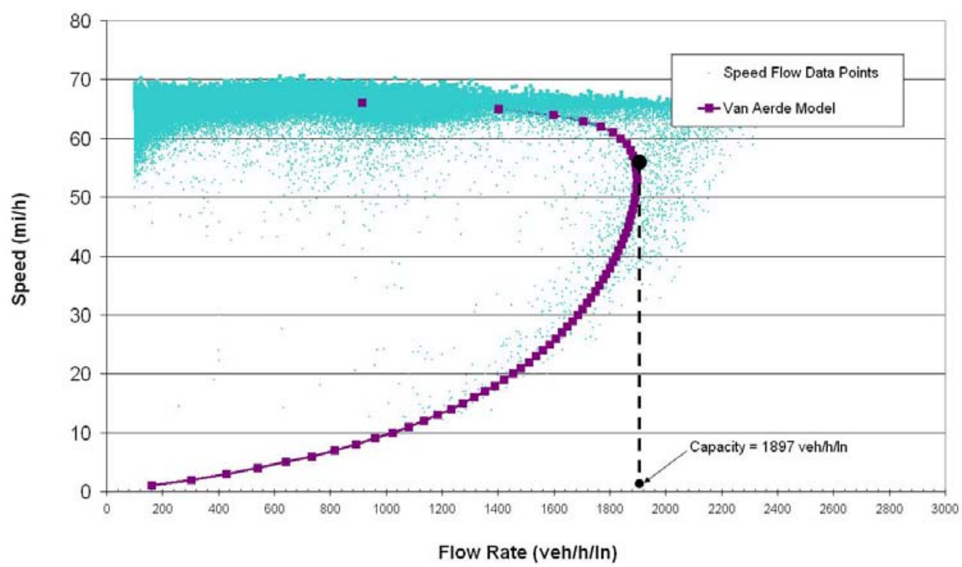


Figure 4-2. Van Aerde Model fit to speed flow points for site ID: T8

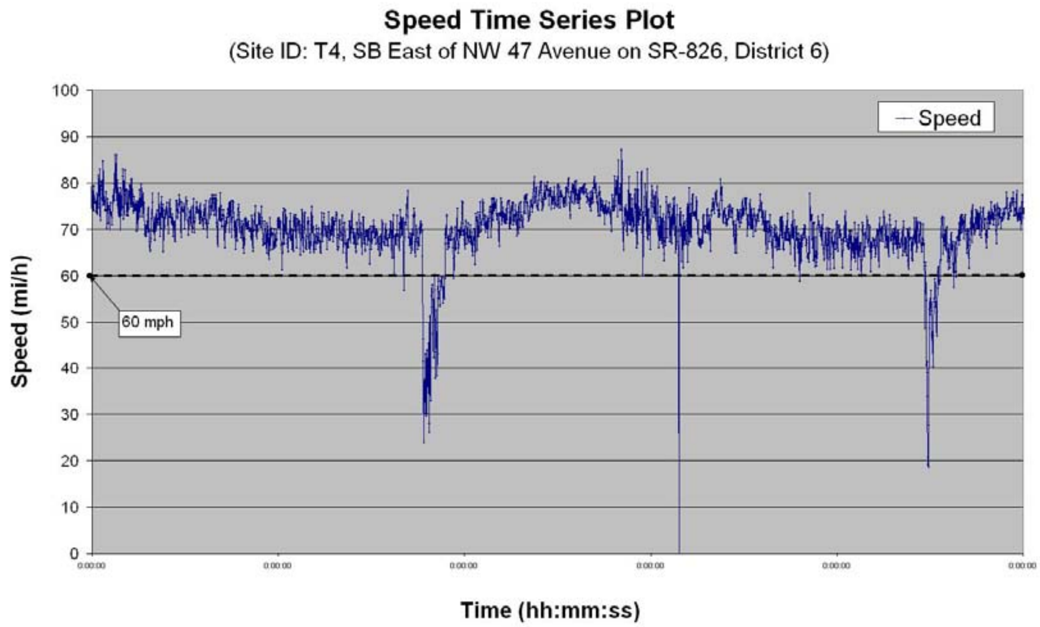


Figure 4-3. Speed time series plot for site T4

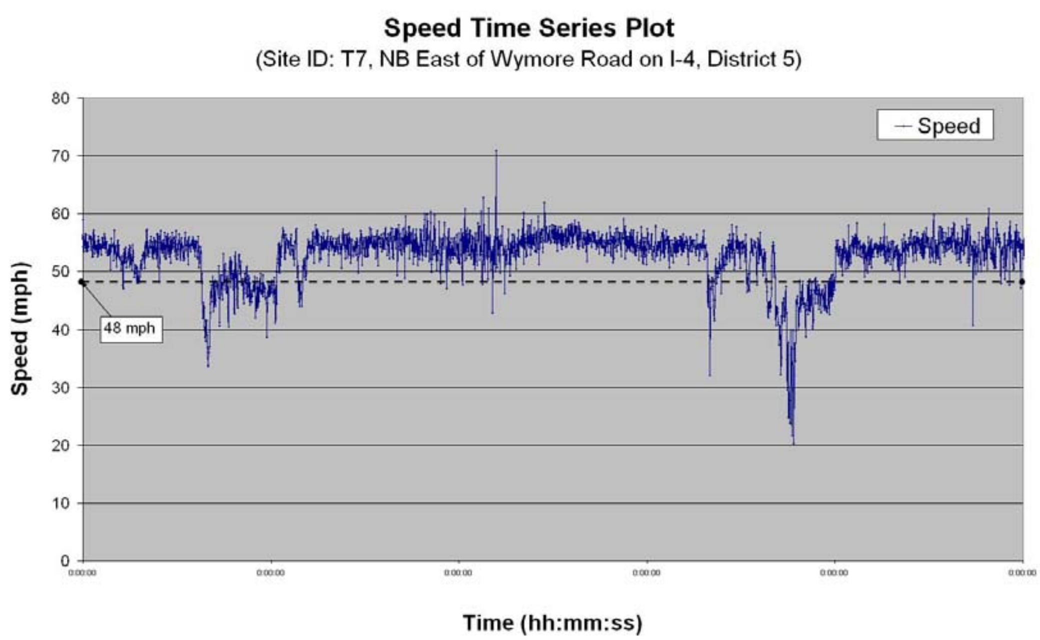


Figure 4-4. Speed time series plot for site T7

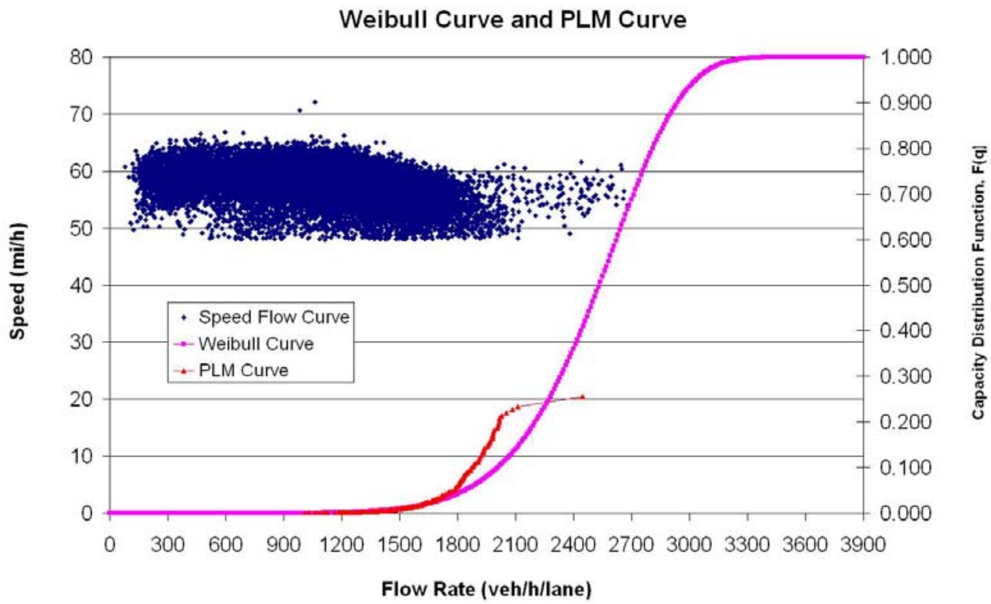


Figure 4-5. Weibull curve fit with PLM curve for site T1, East of NW 57 Avenue on SR-826

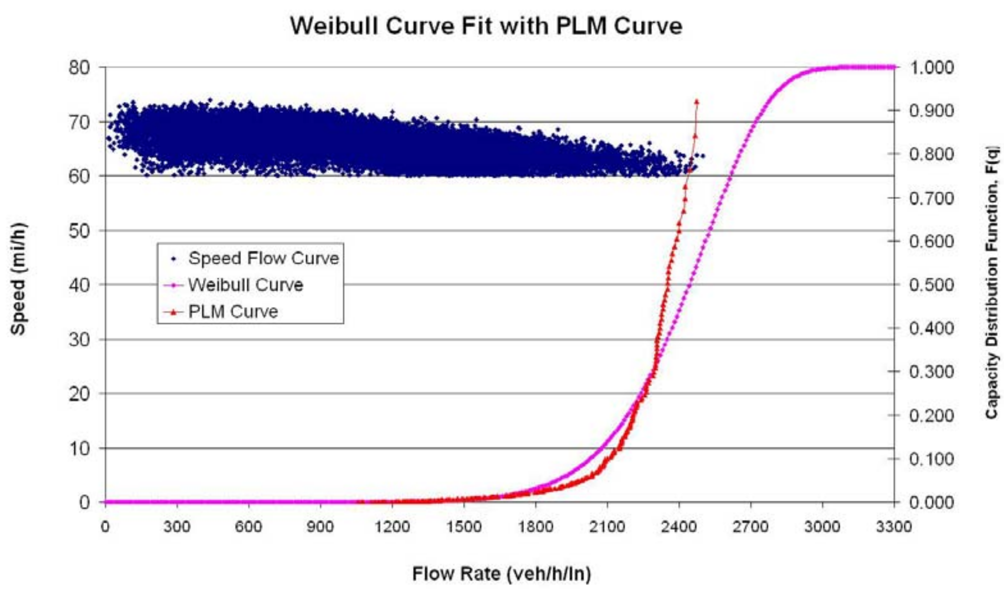


Figure 4-6. Weibull curve fit with PLM curve for site T9, NB North of Butler Blvd on I-95

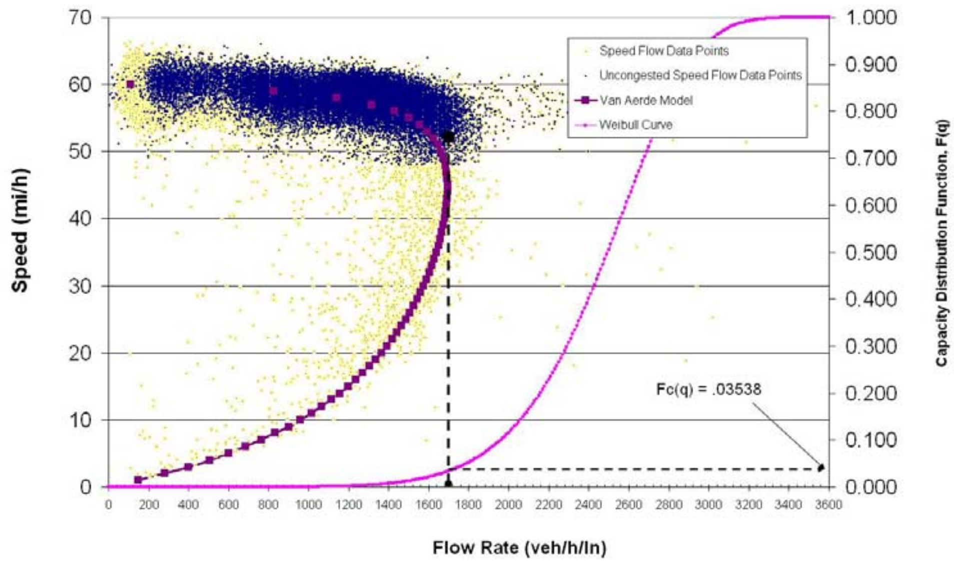


Figure 4-7. Speed Flow, Weibull and Van Aerde Model curves for site ID T1

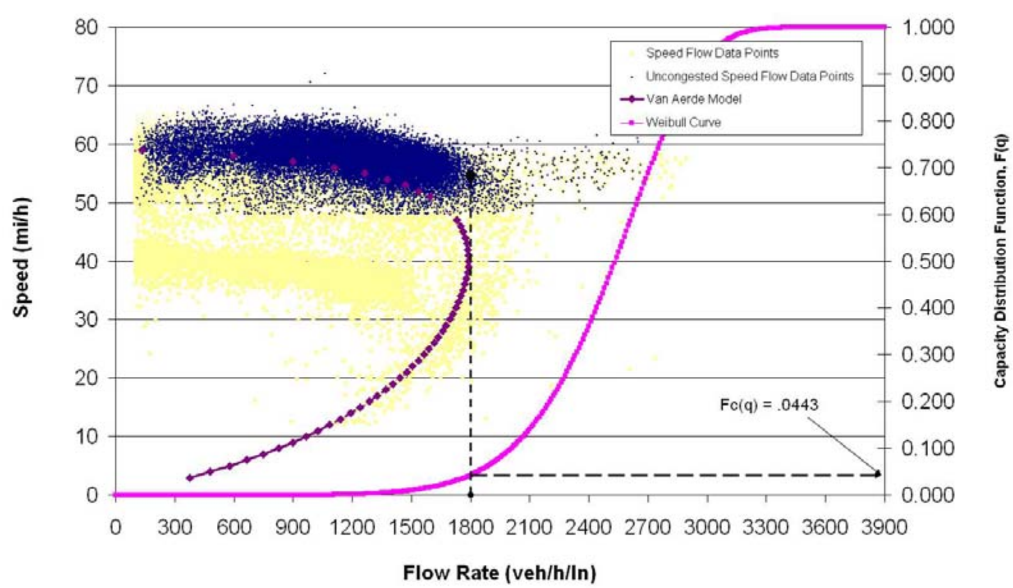


Figure 4-8. Speed Flow, Weibull and Van Aerde Model curves for site ID T2

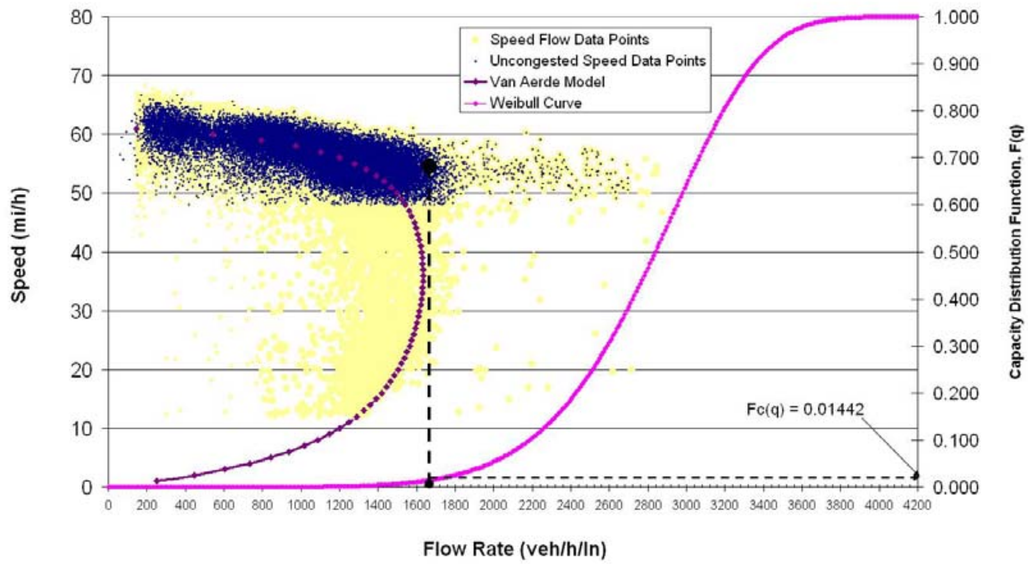


Figure 4-9. Speed Flow, Weibull and Van Aerde Model curves for site ID T3

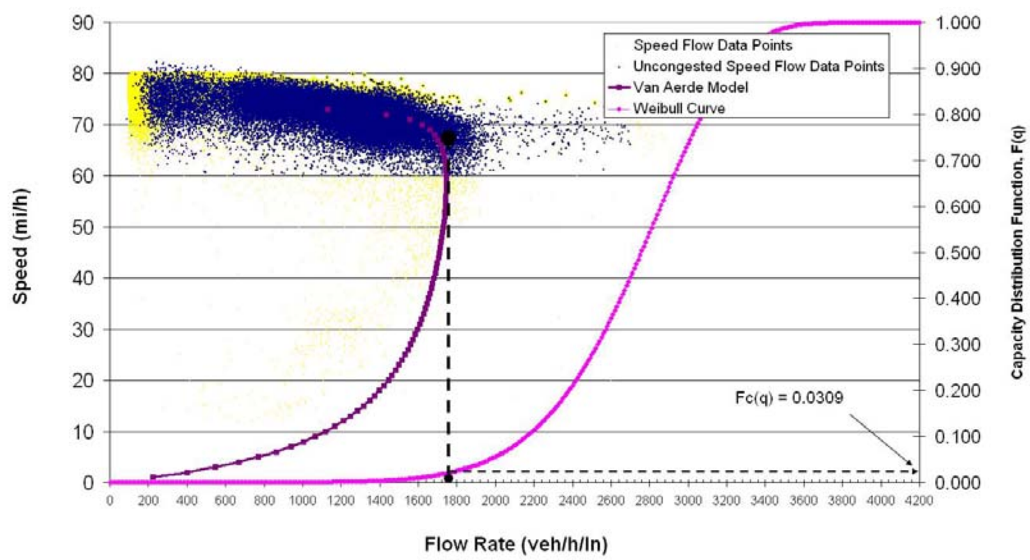


Figure 4-10. Speed Flow, Weibull and Van Aerde Model curves for site ID T4

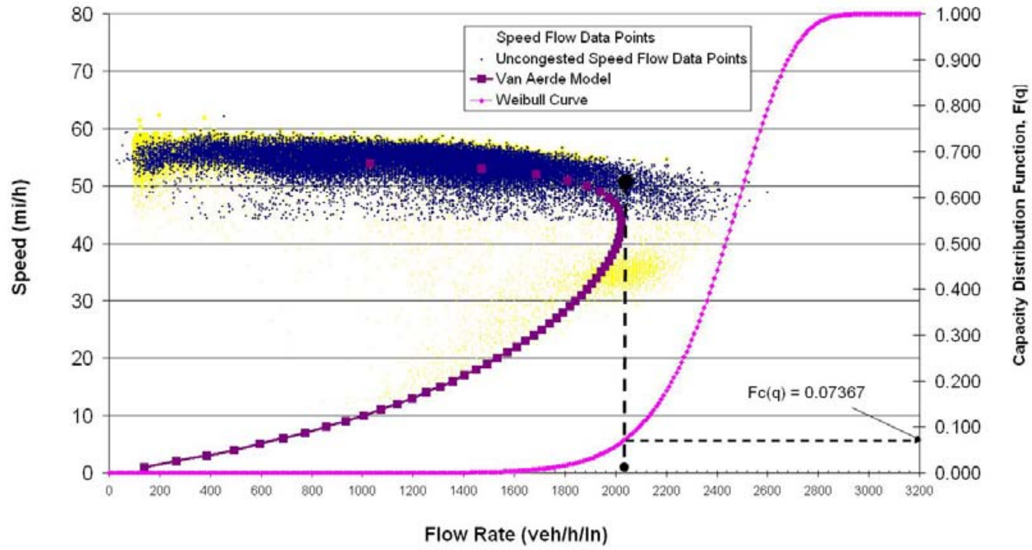


Figure 4-11. Speed Flow, Weibull and Van Aerde Model curves for site ID T5

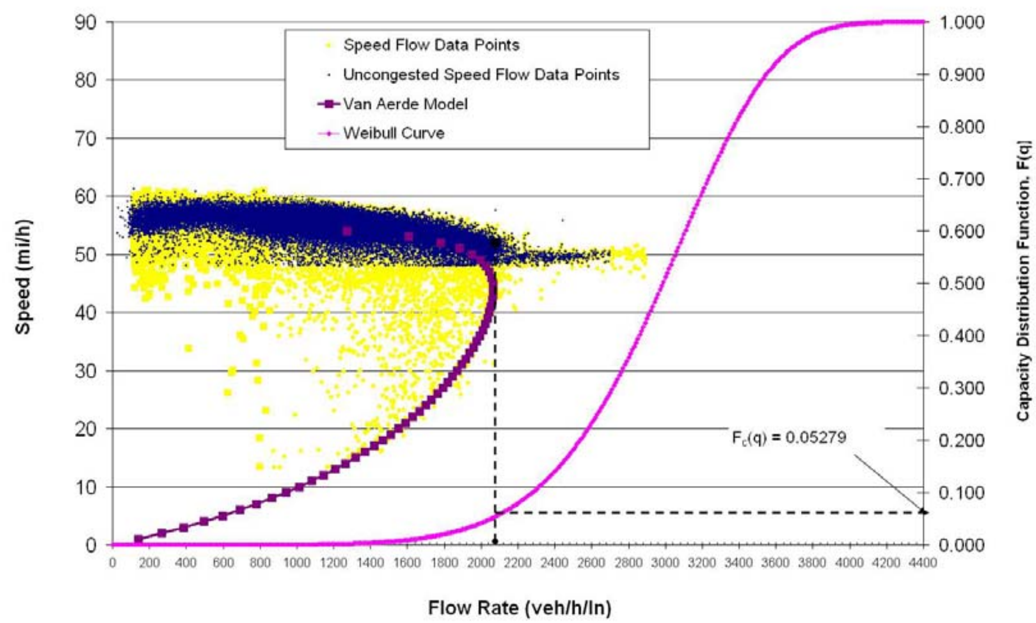


Figure 4-12. Speed Flow, Weibull and Van Aerde Model curves for site ID T6

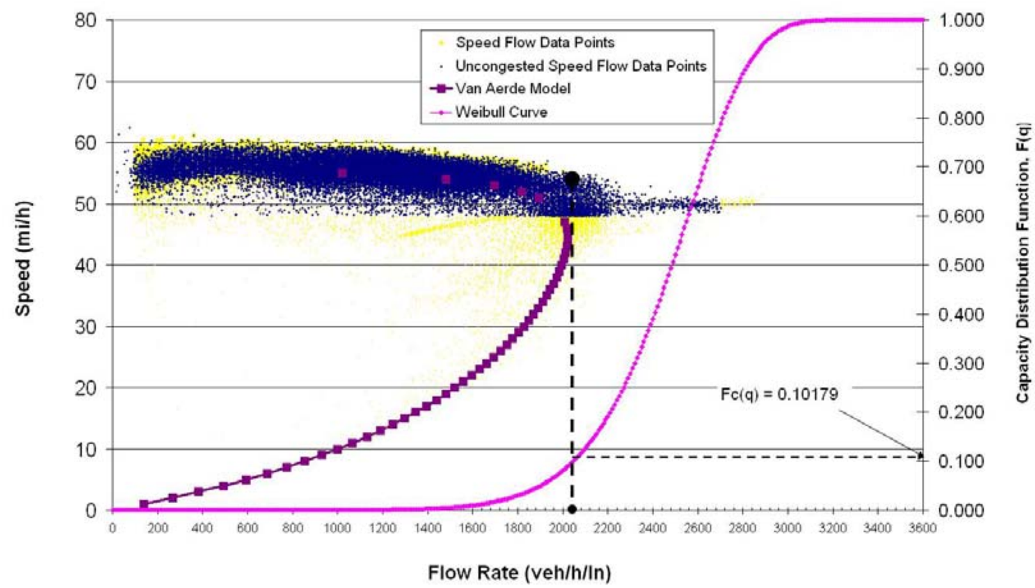


Figure 4-13. Speed Flow, Weibull and Van Aerde Model curves for site ID T7

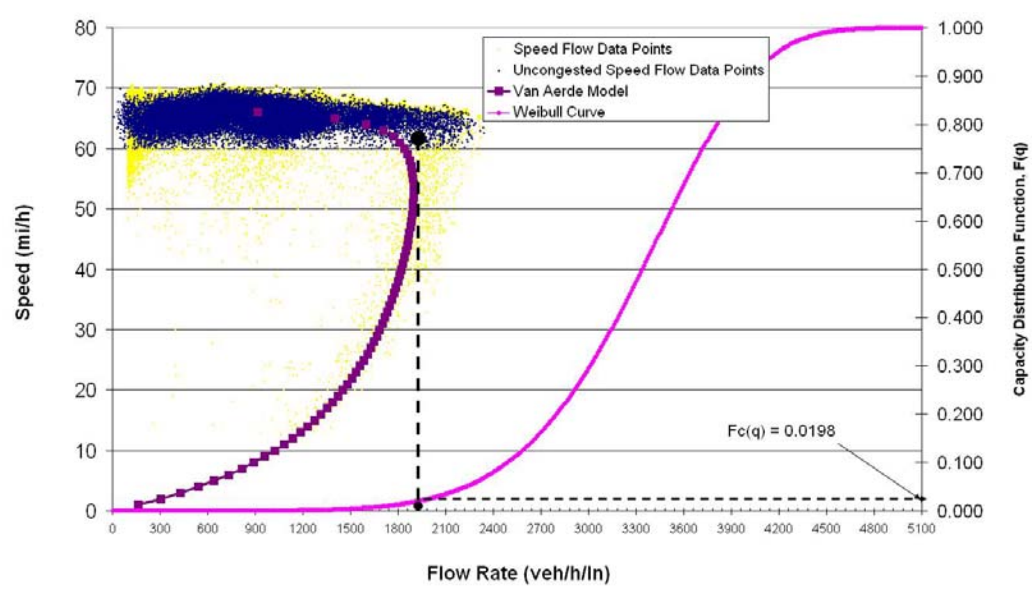


Figure 4-14. Speed Flow, Weibull and Van Aerde Model curves for site ID T8

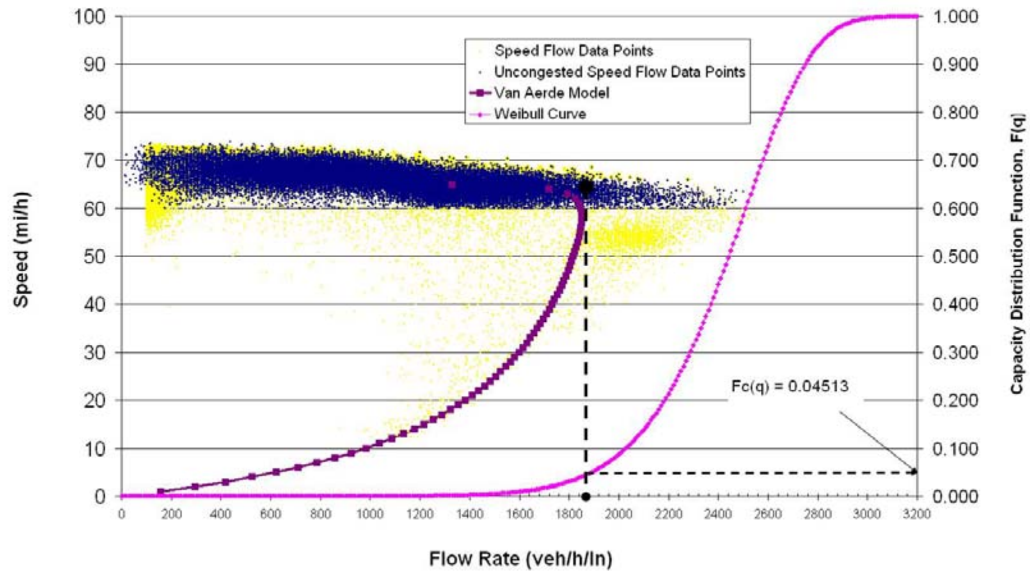


Figure 4-15. Speed Flow, Weibull and Van Aerde Model curves for site ID T9

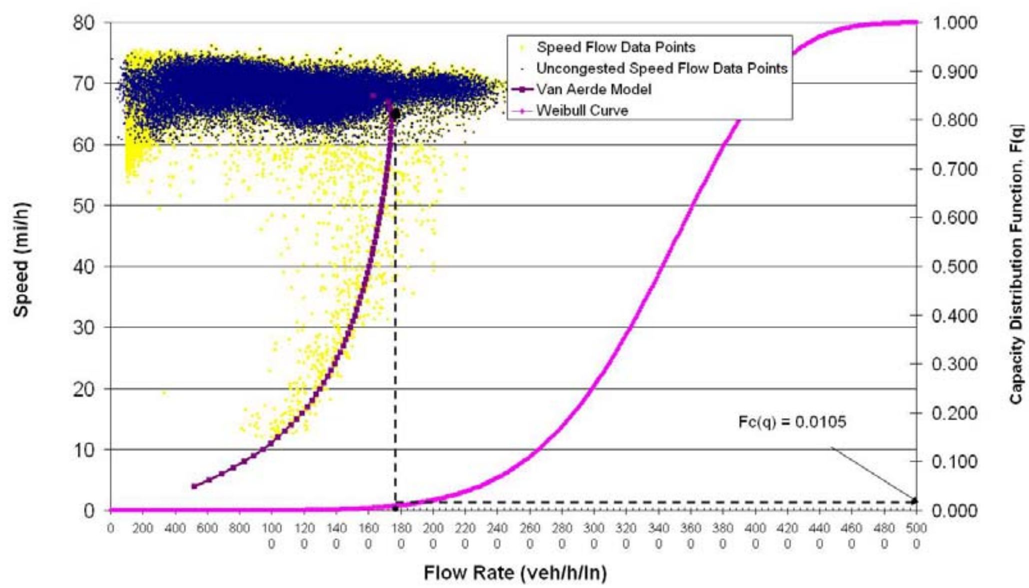


Figure 4-16. Speed Flow, Weibull and Van Aerde Model curves for site ID T10

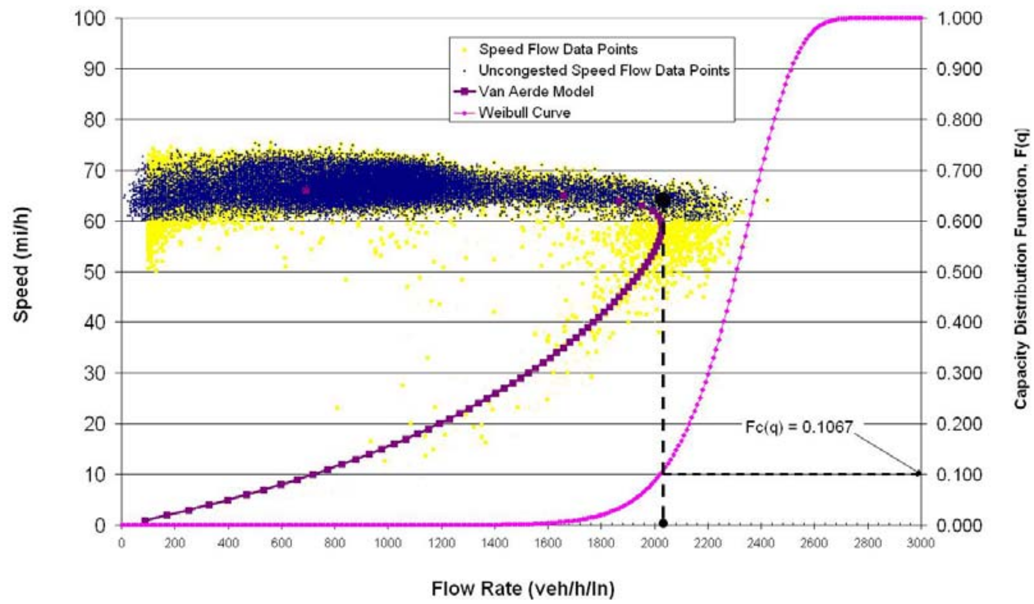


Figure 4-17. Speed Flow, Weibull and Van Aerde Model curves for site ID T11

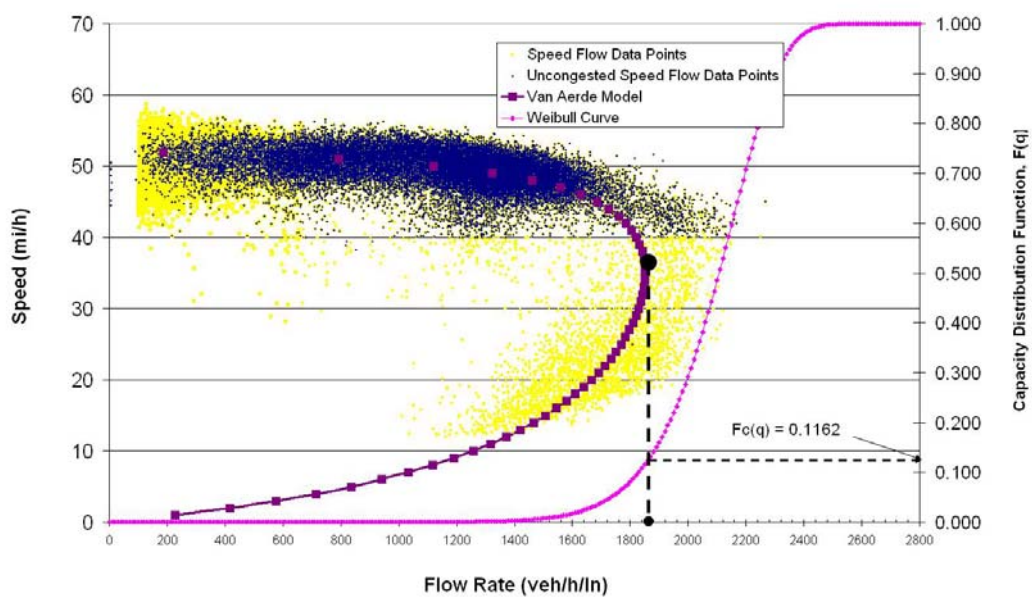


Figure 4-18. Speed Flow, Weibull and Van Aerde Model curves for site ID T12

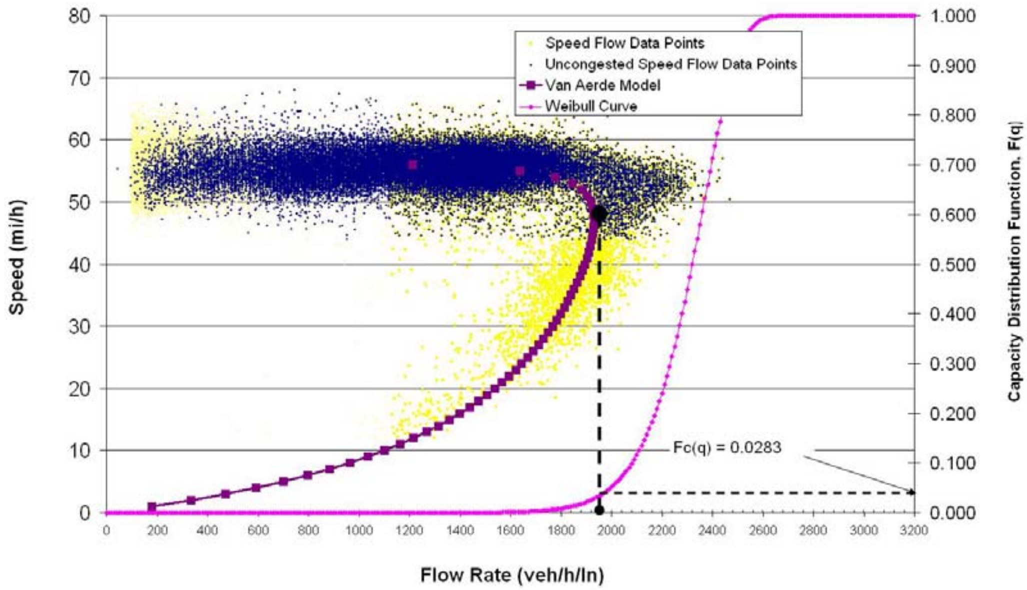


Figure 4-19. Speed Flow, Weibull and Van Aerde Model curves for site ID T13

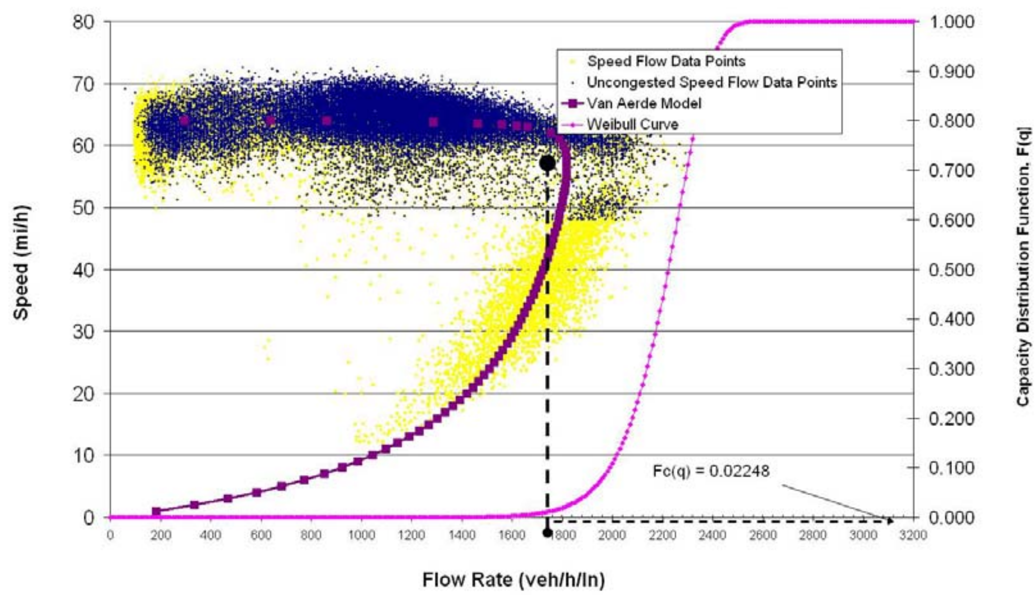


Figure 4-20. Speed Flow, Weibull and Van Aerde Model curves for site ID T14

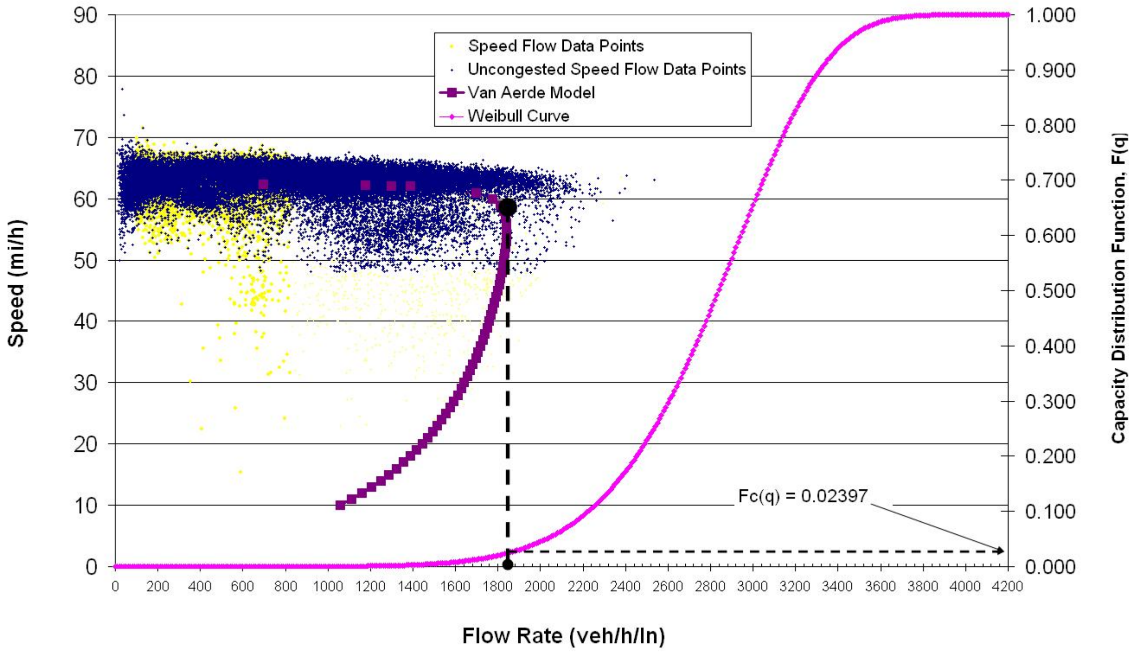


Figure 4-21. Speed Flow, Weibull and Van Aerde Model curves for site ID T15

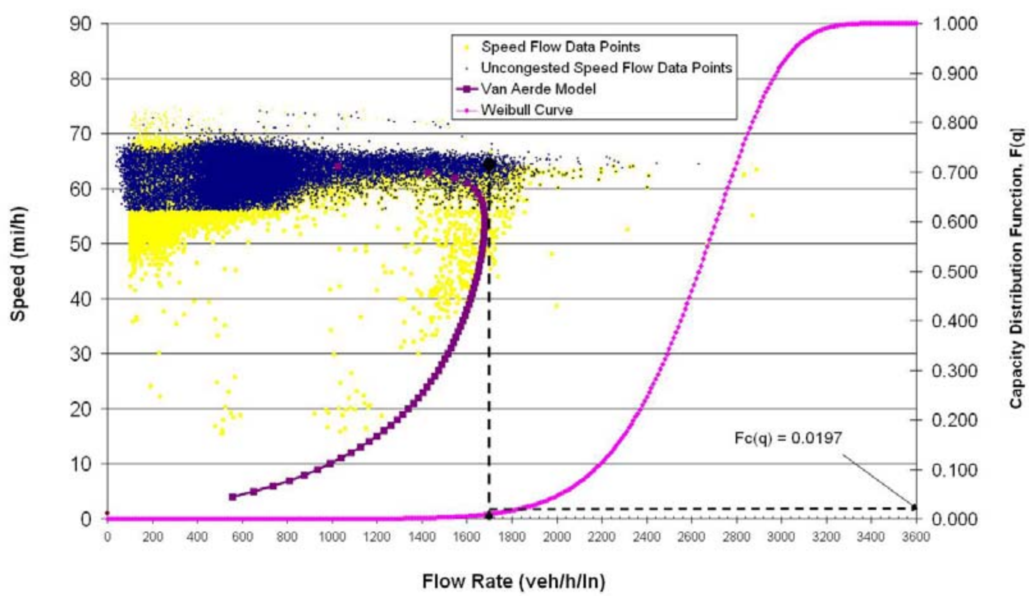


Figure 4-22. Speed Flow, Weibull and Van Aerde Model curves for Site ID F1

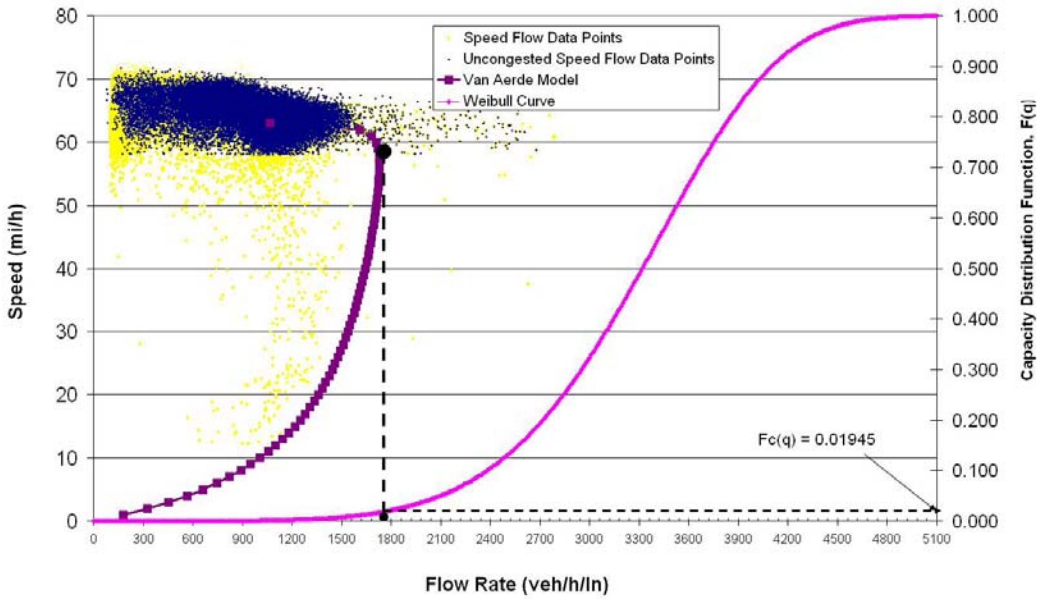


Figure 4-23. Speed Flow, Weibull and Van Aerde Model curves for site ID F2

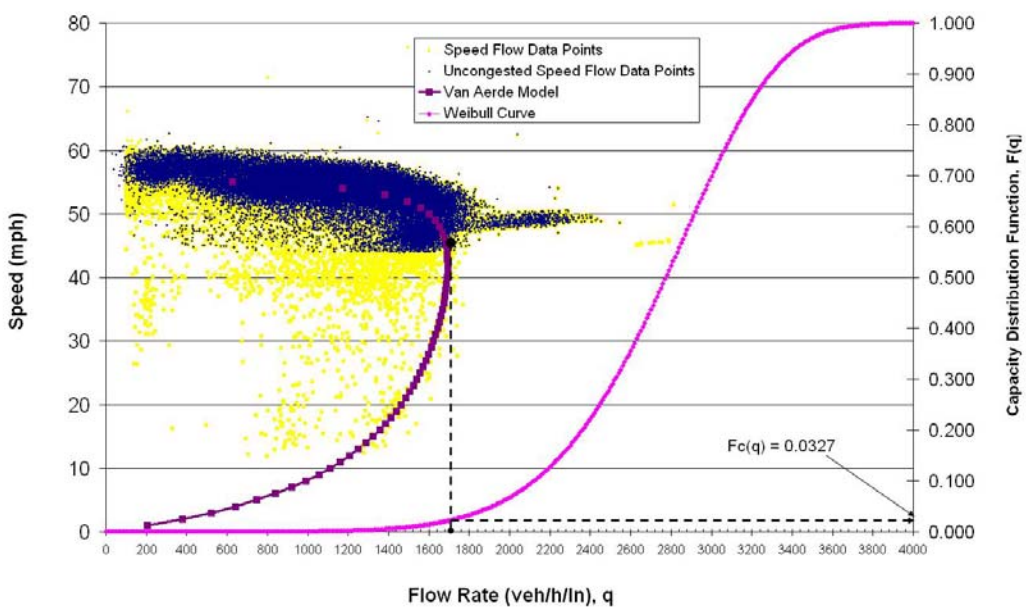


Figure 4-24. Speed Flow, Weibull and Van Aerde Model curves for site ID F3

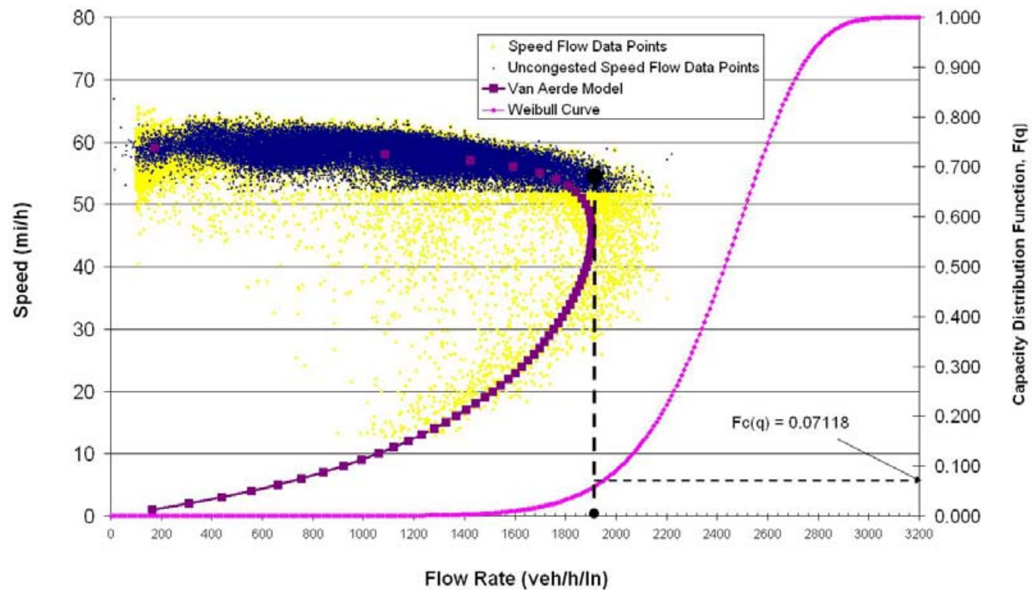


Figure 4-25. Speed Flow, Weibull and Van Aerde Model curves for site ID F4

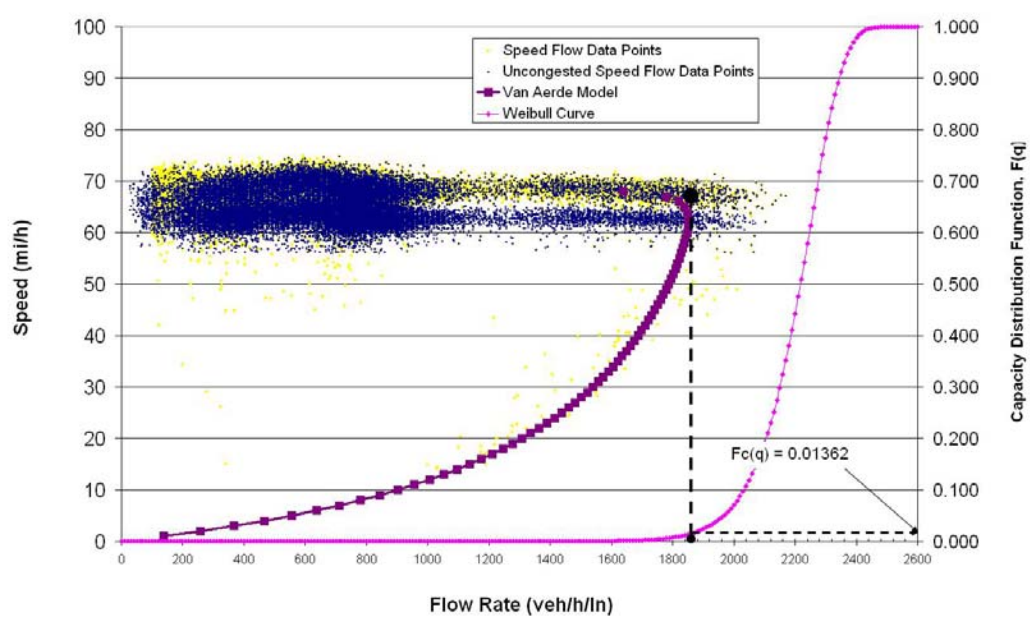


Figure 4-26. Speed Flow, Weibull and Van Aerde Model curves for site ID F5

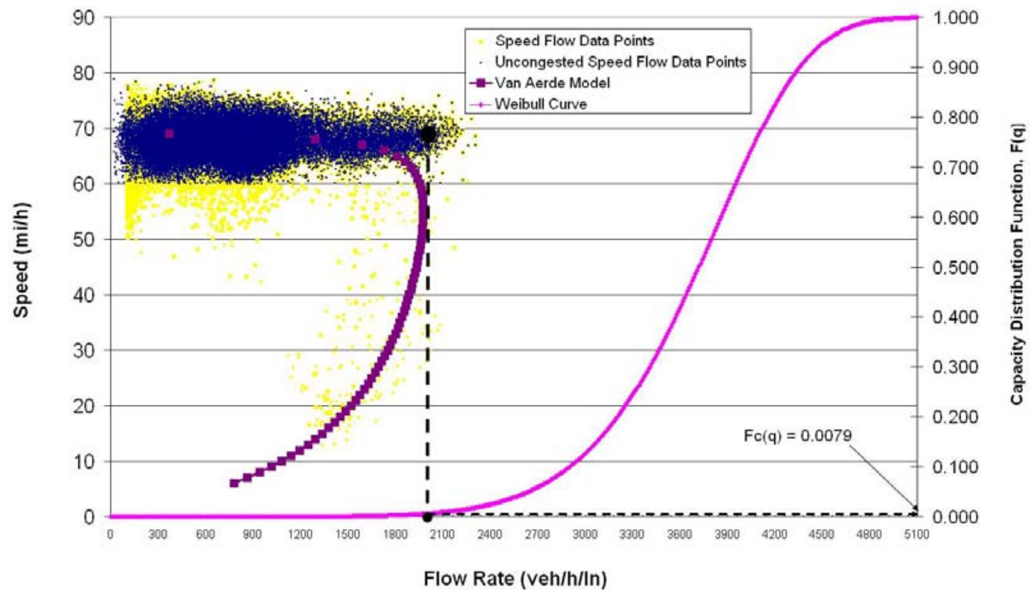


Figure 4-27. Speed Flow, Weibull and Van Aerde Model curves for site ID F6

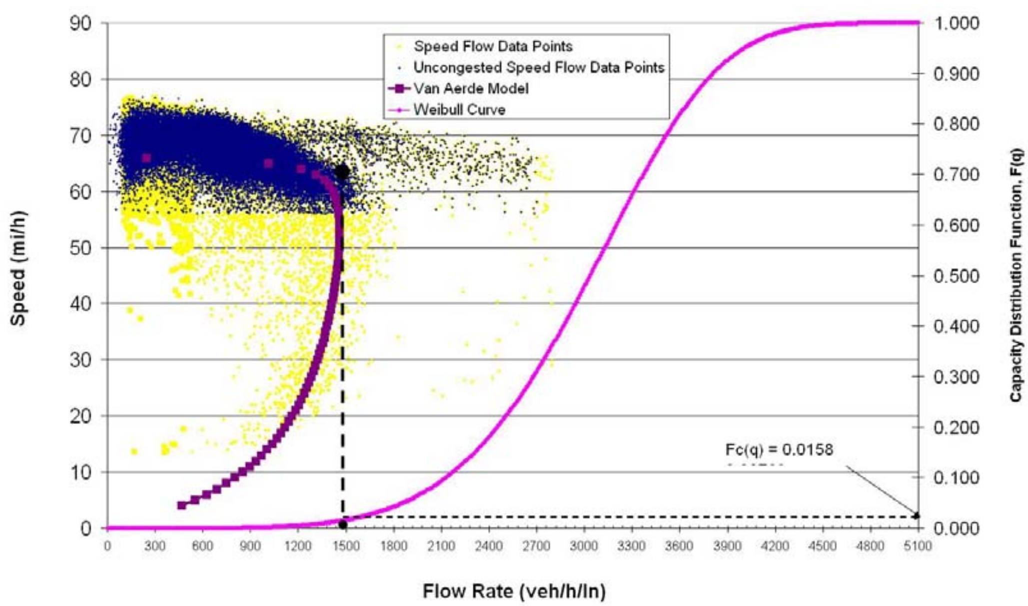


Figure 4-28. Speed Flow, Weibull and Van Aerde Model curves for site ID FV1

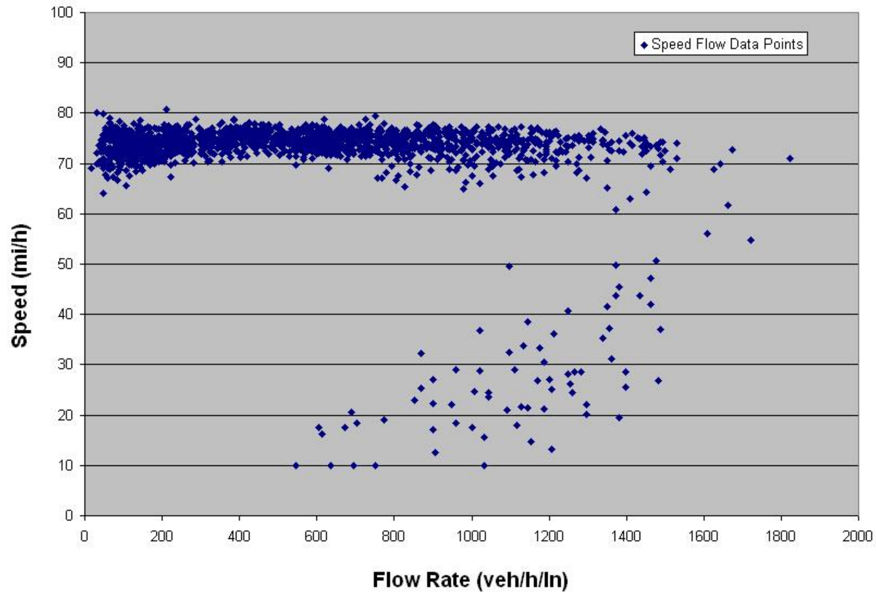


Figure 4-29. Speed flow plot for rural freeway site 9 (on I-75, At Co Rd 514)

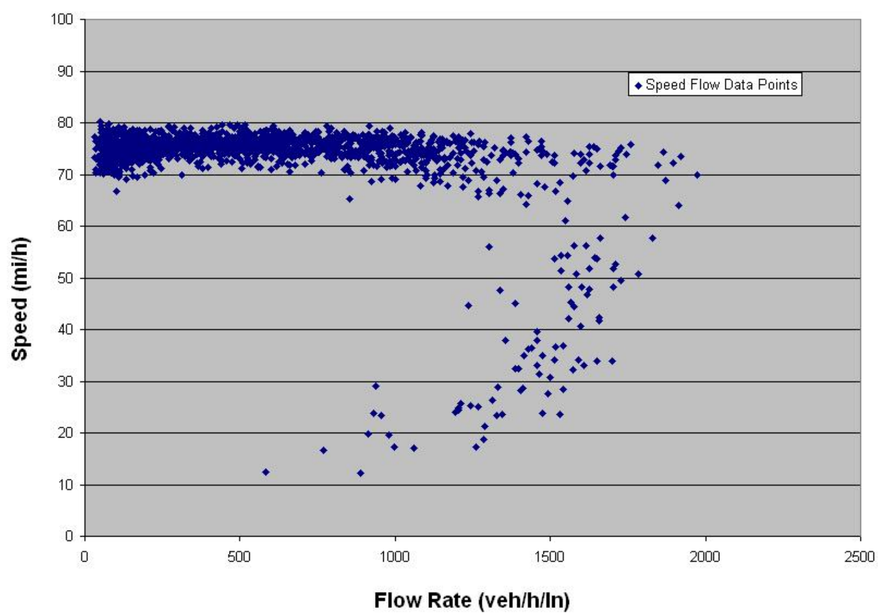


Figure 4-30. Speed flow plot for rural freeway site 10 (on Turnpike, Just South of SR 91 or Co Road 468)

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 (Mia.)	3	1693	60.10	44.50	256.47
T2 (Mia.)	3	1790	59.23	40.40	229.28
T3 (Mia.)	3	1632	61.59	36.67	273.05
T4 (Mia.)	3	1746	73.83	58.36	247.95
T5 (Orl.)	3	2024	54.88	43.44	144.81
T6 (Orl.)	3	2082	56.12	43.51	144.81
T7 (Orl.)	3	2021	55.81	44.50	144.81
T8 (Jax.)	3	1897	66.63	53.08	172.00
T9 (Jax.)	3	1850	65.38	58.30	169.43
T10 (Jax.)	3	1743	68.55	64.08	171.36
T11 (Jax.)	3	2027	66.19	58.73	194.53
T12 (Tpa.)	3	1851	52.20	34.98	247.67
T13 (Tpa.)	3	1931	56.61	47.11	193.60
T14 (Tpa.)	3	1819	64.13	56.92	227.23
T15 (Tpa.)	3	1841	62.51	54.68	197.62
F1 (Mia.)	4	1678	64.51	53.82	189.70
F2 (Mia.)	4	1729	63.21	56.43	193.40
F3 (Orl.)	4	1692	55.44	41.89	228.64
F4 (Orl.)	4	1905	59.11	45.56	174.90
F5 (Jax.)	4	1851	68.86	62.27	144.81
F6 (Jax.)	4	1976	69.17	54.94	186.00
FV1 (FtL.)	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 (Mia.)	SR-826 (Mia)	3	48	48
T2 (Mia.)	SR-826 (Mia)	3	48	52
T3 (Mia.)	SR-826 (Mia)	3	48	60
T4 (Mia.)	SR-826 (Mia)	3	60	48
T5 (Orl.)	I-4 (Orl)	3	44	44
T6 (Orl.)	I-4 (Orl)	3	48	44
T7 (Orl.)	I-4 (Orl)	3	48	48
T8 (Jax.)	I-95 (Jax)	3	60	64
T9 (Jax.)	I-95 (Jax)	3	60	56
T10 (Jax.)	I-95 (Jax)	3	60	52
T11 (Jax.)	I-295 (Jax)	3	60	60
T12 (Tpa.)	I-275 (Tpa)	3	40	48
T13 (Tpa.)	I-275 (Tpa)	3	44	48
T14 (Tpa.)	I-275 (Tpa)	3	56	48
T15 (Tpa.)	I-275 (Tpa)	3	48	52
F1 (Mia.)	I-75 (Mia)	4	56	56
F2 (Mia.)	SR-826 (Mia)	4	64	56
F3 (Orl.)	I-4 (Orl)	4	48	48
F4 (Orl.)	I-4 (Orl)	4	52	52
F5 (Jax.)	I-95 (Jax)	4	56	60
F6 (Jax.)	I-95 (Jax)	4	56	56
FV1 (FtL.)	I-595 (Ft.L)	5	56	60

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

Site ID	Dir. Lanes	VAM Estimation Method		Pre-Breakdown Max Flow Rate[3]		Capacity[1]	Capacity[1]	Breakdowns per day
		Capacity[1]	$F_c(q)$ %	Capacity[1]	$F_c(q)$ %	50th Percentile	4th Percentile	
T1 (Mia.)	3	1693	3.54	1646	2.64	2494	1738	1.43
T2 (Mia.)	3	1790	4.43	1729	3.78	2534	1787	1.32
T3 (Mia.)	3	1632	1.44	1656	1.42	2839	1916	0.55
T4 (Mia.)	3	1746	3.09	1714	1.74	2757	1912	0.92
T5 (Orl.)	3	2024	7.37	2004	5.95	2434	1940	1.09
T6 (Orl.)	3	2082	5.28	1860	2.54	2980	1988	1.13
T7 (Orl.)	3	2021	10.18	1973	7.14	2485	1855	2.54
T8 (Jax.)	3	1897	1.98	1969	2.32	3338	2147	0.23
T9 (Jax.)	3	1850	4.51	1959	7.14	2442	1847	1.08
T10 (Jax.)	3	1743	1.05	1852	1.29	3424	2211	0.52
T11 (Jax.)	3	2027	10.67	2006	9.09	2308	1889	0.80
T12 (Tpa.)	3	1851	11.62	1867	12.94	2107	1703	1.17
T13 (Tpa.)	3	1931	2.83	2052	7.91	2320	1971	1.33
T14 (Tpa.)	3	1818	2.25	1965	8.18	2223	1882	0.67
T15 (Tpa.)	3	1841	2.40	1734	1.52	2839	1969	0.38
F1 (Mia.)	4	1678	1.97	1665	0.80	2632	1703	0.12
F2 (Mia.)	4	1729	1.95	-[2]	-[2]	3320	1971	0.27
F3 (Orl.)	4	1692	3.27	1577	1.31	2775	1852	0.91
F4 (Orl.)	4	1905	7.12	1728	2.14	2431	1840	0.56
F5 (Jax.)	4	1851	1.36	1902	2.47	2217	1946	0.16
F6 (Jax.)	4	1976	0.79	1766	0.24	3720	2563	0.04
FV1 (FtL.)	5	1460	1.58	1739	3.56	3036	1779	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
T1 (Mia.)	1824	1759	1693	1738	7.74	4.95	3.90	1.21	1.43
T2 (Mia.)	1866	1787	1790	1787	4.22	4.42	-0.19	0.00	1.32
T3 (Mia.)	1743	1676	1632	1916	6.78	-9.03	2.68	-12.53	0.55
T4 (Mia.)	1876	1812	1746	1912	7.48	-1.88	3.81	-5.23	0.92
T5 (Orl.)	2149	2096	2024	1940	6.19	10.77	3.57	8.04	1.09
T6 (Orl.)	2132	2053	2082	1988	2.40	7.24	-1.39	3.27	1.13
T7 (Orl.)	2211	2146	2021	1855	9.41	19.19	6.19	15.69	2.54
T8 (Jax.)	1978	1870	1897	2147	4.27	-7.87	-1.42	-12.90	0.23
T9 (Jax.)	2119	2028	1850	1847	14.54	14.73	9.62	9.80	1.08
T10 (Jax.)	2108	2002	1743	2211	20.94	-4.66	14.86	-9.45	0.52
T11 (Jax.)	2087	2017	2027	1889	2.96	10.48	-0.49	6.78	0.80
T12 (Tpa.)	1912	1840	1851	1703	3.30	12.27	-0.59	8.04	1.17
T13 (Tpa.)	2100	2033	1931	1971	8.75	6.54	5.28	3.15	1.33
T14 (Tpa.)	1953	1895	1818	1882	7.43	3.77	4.24	0.69	0.67
T15 (Tpa.)	1829	1743	1841	1969	-0.65	-7.11	-5.32	-11.48	0.38
F1 (Mia.)	1656	1565	1678	1968	-1.31	-15.85	-6.73	-20.48	0.12
F2 (Mia.)	1514	1443	1729	2014	-12.43	-24.83	-16.54	-28.35	0.27
F3 (Orl.)	1869	1777	1692	1852	10.43	0.92	5.00	-4.05	0.91
F4 (Orl.)	1961	1908	1905	1840	2.95	6.58	0.16	3.70	0.56
F5 (Jax.)	1826	1730	1851	1946	-1.35	-6.17	-6.54	-11.10	0.16
F6 (Jax.)	1840	1731	1976	2563	-6.88	-28.21	-12.40	-32.46	0.04
FV1 (FtL.)	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

Site ID	Max. Flow[1]	VAM capacity estimation method		Stochastic capacity estimation method	
		Lower Flow Rate[1]	% of Max. Flow	Lower Flow Rate[1]	% of Max. Flow
T1 (Mia.)	2688	1568	58.33	1660	61.76
T2 (Mia.)	2684	1643	61.22	1680	62.59
T3 (Mia.)	2696	1508	55.93	1720	63.80
T4 (Mia.)	2696	1604	59.50	1816	67.36
T5 (Orl.)	2528	1836	72.63	1752	69.31
T6 (Orl.)	2696	1928	71.52	1900	70.48
T7 (Orl.)	2700	1785	66.10	1608	59.55
T8 (Jax.)	2352	1664	70.75	2192	93.20
T9 (Jax.)	2552	1584	62.07	1624	63.64
T10 (Jax.)	2502	1501	60.00	2202	88.00
T11 (Jax.)	2424	1856	76.56	1612	66.51
T12 (Tpa.)	2268	1692	74.60	1520	67.02
T13 (Tpa.)	2468	1736	70.34	1784	72.29
T14 (Tpa.)	2292	1636	71.38	1744	76.09
T15 (Tpa.)	2380	1692	71.09	1864	78.32
F1 (Mia.)	2670	1545	57.87	1836	68.76
F2 (Mia.)	2679	1482	55.32	1773	66.18
F3 (Orl.)	2691	1545	57.42	1698	63.10
F4 (Orl.)	2211	1782	80.60	1734	78.43
F5 (Jax.)	2169	1728	79.67	1899	87.55
F6 (Jax.)	2307	1866	80.89	-[2]	-[2]
FV1 (FtL.)	2690	1229	45.68	1534	57.03

[1]Flow in veh/h/ln,

[2]Threshold/Lower Flow Rate 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 70th & 65th percentage of maximum flow rate

Site ID	Max. Value	Capacity[1] A	Capacity[2] B	Flow[3] C	Flow[4] D	Capacity[5] E	% Change A with E	% Change A with F	Capacity[6] F	% Change B with E	% Change B with F	Breakdowns per day
T1 (Mia.)	2688	1693	1738	1882	1747	2216	30.89	14.18	1933	27.50	11.22	1.43
T2 (Mia.)	2684	1790	1787	1879	1745	2058	14.97	7.04	1916	15.17	7.22	1.32
T3 (Mia.)	2696	1632	1916	1887	1752	2221	36.09	25.49	2048	15.92	6.89	0.55
T4 (Mia.)	2696	1746	1912	1887	1752	2117	21.25	7.62	1879	10.72	-1.73	0.92
T5 (Orl.)	2528	2024	1940	1770	1643	1989	-1.73	-6.52	1892	2.53	-2.47	1.09
T6 (Orl.)	2696	2082	1988	1887	1752	2043	-1.87	-6.24	1952	2.77	-1.81	1.13
T7 (Orl.)	2700	2021	1855	1890	1755	2067	2.28	-0.84	2004	11.43	8.03	2.54
T8 (Jax.)	2352	1897	2147	1646	1529	1887	-0.53	-3.80	1825	-12.11	-15.00	0.23
T9 (Jax.)	2552	1850	1847	1786	1659	2011	8.70	3.73	1919	8.88	3.90	1.08
T10 (Jax.)	2502	1743	2211	1751	1626	1998	14.63	8.43	1890	-9.63	-14.52	0.52
T11 (Jax.)	2424	2027	1889	1697	1576	1959	-3.35	-5.97	1906	3.71	0.90	0.80
T12 (Tpa.)	2268	1851	1703	1588	1474	1770	-4.38	-10.43	1658	3.93	-2.64	1.17
T13 (Tpa.)	2468	1931	1971	1728	1604	1922	-0.47	-6.27	1810	-2.49	-8.17	1.33
T14 (Tpa.)	2292	1818	1882	1604	1490	1798	-1.10	-5.23	1723	-4.46	-8.45	0.67
T15 (Tpa.)	2380	1841	1969	1666	1547	1823	-0.98	-5.81	1734	-7.41	-11.93	0.38
F1 (Mia.)	2670	1678	1968	1869	1736	2073	23.54	9.83	1843	5.34	-6.35	0.12
F2 (Mia.)	2679	1729	2014	1875	1741	2195	26.95	19.78	2071	8.99	2.83	0.27
F3 (Orl.)	2691	1692	1852	1884	1749	2095	23.82	17.79	1993	13.12	7.61	0.91
F4 (Orl.)	2211	1905	1840	1548	1437	1761	-7.56	-12.28	1671	-4.29	-9.18	0.56
F5 (Jax.)	2169	1851	1946	1518	1410	1743	-5.83	-8.75	1689	-10.43	-13.21	0.16
F6 (Jax.)	2307	1976	2563	1615	1500	1815	-8.15	-11.99	1739	-29.18	-32.15	0.04
FV1 (FtL.)	2690	1460	1779	1883	1749	2254	54.38	46.78	2143	26.70	20.46	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 70% of maximum flow rate (veh/h/ln), [4]Threshold Value for 65% of maximum flow rate (veh/h/ln)

[5]Average value of flow rates above flow rate that corresponds to 70% of max. flow rate (veh/h/ln)

[6]Average value of flow rates above flow rate that corresponds to 65% 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 (Mia.)	14.03	13.32	-5.06	14.37	7.88	40.41	181.21
T2 (Mia.)	10.53	10.79	2.47	11.85	9.82	15.89	34.09
T3 (Mia.)	4.61	4.33	-6.07	4.07	-6.00	4.28	5.16
T4 (Mia.)	1.14	1.17	2.63	1.52	29.91	0.99	-34.87
T5 (Orl.)	1.98	2.22	12.12	2.28	2.70	1.81	-20.61
T6 (Orl.)	4.84	5.63	16.32	5.81	3.20	5.65	-2.75
T7 (Orl.)	1.18	1.08	-8.47	0.82	-24.07	1.29	57.32
T8 (Jax.)	7.66	7.52	-1.83	6.97	-7.31	9.80	40.60
T9 (Jax.)	9.03	8.45	-6.42	7.63	-9.70	10.89	42.73
T10 (Jax.)	2.03	1.94	-4.43	3.60	85.57	7.40	105.56
T11 (Jax.)	4.98	4.55	-8.63	4.06	-10.77	5.84	43.84
T12 (Tpa.)	4.62	4.75	2.81	3.00	-36.80	5.87	95.47
T13 (Tpa.)	8.42	9.39	11.52	10.69	13.84	10.91	2.09
T14 (Tpa.)	8.24	7.59	-7.86	6.98	-8.08	4.79	-31.31
T15 (Tpa.)	1.76	2.30	30.54	1.86	-19.16	2.72	46.15
F1 (Mia.)	2.17	2.28	5.07	2.58	13.16	3.47	34.50
F2 (Mia.)	10.68	9.31	-12.83	8.77	-5.80	7.11	-18.93
F3 (Orl.)	2.58	3.92	51.94	4.39	11.99	3.58	-18.45
F4 (Orl.)	7.99	6.16	-22.90	5.50	-10.71	4.85	-11.82
F5 (Jax.)	5.94	4.55	-23.40	3.40	-25.27	0.90	-73.53
F6 (Jax.)	1.72	1.38	-19.77	1.25	-9.42	0.65	-48.00
FV1 (FtL.)	10.02	8.84	-11.78	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 (Mia.)	5.78	4.77	-17.47	4.55	-4.61	3.59	-21.10
T2 (Mia.)	4.60	3.43	-25.43	2.62	-23.62	0.77	-70.61
T3 (Mia.)	1.57	1.08	-31.21	1.00	-7.41	0.81	-19.00
T4 (Mia.)	2.68	1.93	-27.99	1.65	-14.51	1.60	-3.03
T5 (Orl.)	9.54	9.24	-3.14	8.63	-6.60	12.14	40.67
T6 (Orl.)	1.79	1.71	-4.47	1.30	-23.98	1.99	53.08
T7 (Orl.)	14.68	13.78	-6.13	13.09	-5.01	18.03	37.74
T8 (Jax.)	0.20	0.21	5.00	0.14	-33.33	0.15	7.14
T9 (Jax.)	9.10	9.40	3.30	10.31	9.68	9.71	-5.82
T10 (Jax.)	0.57	0.56	-1.75	0.34	-39.29	0.36	5.88
T11 (Jax.)	7.41	8.52	14.98	8.86	3.99	8.80	-0.68
T12 (Tpa.)	9.69	10.13	4.55	7.65	-24.44	12.09	57.97
T13 (Tpa.)	7.04	7.92	12.48	9.11	15.00	9.30	2.10
T14 (Tpa.)	5.86	5.20	-11.18	4.61	-11.34	2.80	-39.17
T15 (Tpa.)	0.54	0.76	40.24	0.54	-28.68	0.85	56.77
F1 (Mia.)	0.31	0.47	51.61	0.64	36.17	0.26	-59.38
F2 (Mia.)	0.60	0.52	-13.33	0.47	-9.62	0.16	-65.96
F3 (Orl.)	0.11	0.02	-81.82	0.04	100.00	0.08	100.00
F4 (Orl.)	5.95	5.38	-9.58	4.91	-8.74	6.92	40.94
F5 (Jax.)	0.39	0.46	17.95	0.54	17.39	0.42	-22.22
F6 (Jax.)[1]	-	-	-	-	-	-	-
FV1 (FtL.)[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

Facility	Site ID	Capacity[1] A	Capacity[2] B	%Change A with B	Capacity[3] C	%Change A with C	Capacity[4] D	%Change A with D	Capacity[5] E	% Change A with E	Breakdowns per day
SR-826 (Mia)	T1	2143	1693	-20.99	1738	-18.89	1759	-17.91	1933	-9.79	1.43
SR-826 (Mia)	T2	2143	1790	-16.47	1787	-16.61	1787	-16.61	1916	-10.59	1.32
SR-826 (Mia)	T3	2143	1632	-23.84	1916	-10.59	1676	-21.79	2048	-4.43	0.55
SR-826 (Mia)	T4	2143	1746	-18.52	1912	-10.77	1812	-15.44	1879	-12.31	0.92
I-4 (Orl)	T5	2143	2024	-5.55	1940	-9.47	2096	-2.19	1892	-11.71	1.09
I-4 (Orl)	T6	2143	2082	-2.84	1988	-7.23	2053	-4.19	1952	-8.91	1.13
I-4 (Orl)	T7	2143	2021	-5.69	1855	-13.43	2146	0.15	2004	-6.48	2.54
I-95 (Jax)	T8	2286	1897	-17.01	2147	-6.07	1870	-18.19	1825	-20.16	0.23
I-95 (Jax)	T9	2286	1850	-19.06	1847	-19.19	2028	-11.28	1919	-16.04	1.08
I-95 (Jax)	T10	2286	1743	-23.74	2211	-3.27	2002	-12.41	1890	-17.31	0.52
I-295 (Jax)	T11	2286	2027	-11.32	1889	-17.36	2017	-11.76	1906	-16.61	0.8
I-75 (Tpa)	T12	2143	1851	-13.62	1703	-20.53	1840	-14.13	1658	-22.63	1.17
I-75 (Tpa)	T13	2143	1931	-9.89	1971	-8.02	2033	-5.13	1810	-15.53	1.33
I-75 (Tpa)	T14	2143	1819	-15.11	1882	-12.17	1895	-11.57	1723	-19.59	0.67
I-75 (Tpa)	T15	2238	1841	-17.74	1969	-12.02	1743	-22.12	1734	-22.52	0.38
I-75 (Mia)	F1	2286	1678	-26.59	1968	-13.90	1565	-31.53	1843	-19.37	0.12
SR-826 (Mia)	F2	2143	1729	-19.31	2014	-6.01	1443	-32.66	2071	-3.35	0.27
I-4 (Orl)	F3	2286	1692	-25.98	1852	-18.98	1777	-22.26	1993	-12.81	0.91
I-4 (Orl)	F4	2143	1905	-11.10	1840	-14.13	1908	-10.96	1671	-22.02	0.56
I-95 (Jax)	F5	2286	1851	-19.02	1946	-14.86	1730	-24.31	1689	-26.11	0.16
I-95 (Jax)	F6	2286	1976	-13.55	2563	12.13	1731	-24.27	1739	-23.92	0.04
I-595 (Ft.L)	FV1	2286	1460	-36.13	1779	-22.17	1530	-33.06	2143	-6.24	0.48

[1] Capacity as provided by HCM 2000 converted to (veh/h/ln) from pc/h/ln assuming 10% of heavy vehicles on level terrain

[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 65% of maximum flow rate of maximum flow rate (veh/h/ln)

Table 4-10. Flow rates (veh/h/ln) for multiple percentile value in 5 minute data intervals for lower bound on basis of average of top 5% flows as lower bound

		VAM		Average	Flow Rates at Percentile Values (Lower Bound: Average Top 5%)						
District	Site ID	5 min	Top x%	Top 5%	55%	60%	65%	70%	75%	80%	85%
Jacksonville	T8	1897	4.46	1868	2016	2032	2048	2064	2084	2108	2136
	T9	1850	10.12	2028	2136	2148	2164	2184	2204	2228	2268
	T10	1743	13.44	2000	2136	2148	2164	2180	2196	2216	2244
	T11	2027	4.85	2024	2112	2120	2132	2144	2160	2172	2188
	F5	1851	2.59	1736	1854	1869	1881	1902	1926	1938	1959
	F6	1976	1.15	1731	1887	1902	1920	1938	1956	1983	2010
Ft. Lauderdale	FV1	1460	2.34	1364	1464	1474	1488	1507	1522	1546	1570
Orlando	T5	2024	8.05	2096	2168	2180	2192	2204	2224	2248	2268
	T6	2082	4.12	2052	2212	2240	2292	2332	2368	2408	2460
	T7	2021	10.88	2144	2284	2328	2368	2404	2448	2504	2556
	F3	1692	8.67	1776	2052	2079	2100	2127	2148	2169	2196
	F4	1905	5.09	1908	1974	1986	1995	2007	2019	2037	2055
Miami	T1	1693	8.43	1756	1848	1860	1892	1936	2068	2160	2264
	T2	1790	4.87	1788	1908	1924	1944	1968	1992	2028	2092
	T3	1632	7.05	1676	1768	1792	1828	1892	1972	2092	2192
	T4	1746	8.17	1812	1884	1896	1916	1940	1976	2036	2184
	F1	1678	2.56	1564	1680	1692	1701	1713	1731	1752	1770
	F2	1729	0.96	1444	1530	1542	1578	1626	1710	1797	1914
Tampa	T12	1851	4.66	1840	1936	1948	1960	1972	1984	1996	2016
	T13	1931	8.84	2032	2128	2140	2152	2164	2176	2196	2216
	T14	1818	8.21	1896	1976	1984	1996	2012	2020	2040	2052
	T15	1841	2.77	1744	1868	1884	1904	1920	1940	1964	1992
		Average	6.01								

Table 4-11. Flow rates (veh/h/ln) for multiple percentile value in 5 minute data intervals for lower bound on basis of average of top 6.5% flows as lower bound

		VAM		Average	Flow Rates at Percentile Values (Lower Bound: Average Top 6.5%)						
District	Site ID	5 min	Top x%	Top 6.5%	55%	60%	65%	70%	75%	80%	85%
Jacksonville	T8	1897	4.46	1796	1980	1996	2016	2036	**2060**	2080	2112
	T9	1850	10.12	1968	2104	2116	2132	2148	**2172**	2200	2232
	T10	1743	13.44	1936	2108	2124	2140	2160	**2180**	2196	2224
	T11	2027	4.85	1968	2080	2092	2104	2116	**2132**	2148	2168
	F5	1851	2.59	1660	1821	1836	1851	1872	**1890**	1917	1941
	F6	1976	1.15	1656	1842	1860	1881	1902	**1929**	1950	1983
Ft. Laud.	FV1	1460	2.34	1328	1435	1450	1464	1478	**1498**	1519	1550
Orlando	T5	2024	8.05	2060	2144	2152	2164	2176	**2192**	2216	2248
	T6	2082	4.12	2012	2128	2152	2192	2236	**2300**	2348	2404
	T7	2021	10.88	2112	2196	2216	2252	2308	**2364**	2420	2496
	F3	1692	8.67	1736	1998	2031	2061	2097	**2127**	2151	2178
	F4	1905	5.09	1872	1953	1959	1971	1983	**1998**	2016	2037
Miami	T1	1693	8.43	1724	1792	1804	1820	1848	**1880**	1944	2124
	T2	1790	4.87	1748	1876	1892	1908	1936	**1964**	1992	2036
	T3	1632	7.05	1644	1724	1736	1756	1780	**1820**	1928	2064
	T4	1746	8.17	1780	1852	1864	1876	1892	**1916**	1948	2024
	F1	1678	2.56	1500	1641	1656	1674	1692	**1704**	1725	1752
	F2	1729	0.96	1408	1485	1500	1518	1536	**1578**	1650	1779
Tampa	T12	1851	4.66	1796	1912	1928	1936	1952	**1968**	1980	2000
	T13	1931	8.84	1988	2092	2108	2124	2140	**2156**	2172	2196
	T14	1818	8.21	1860	1952	1960	1972	1988	**2004**	2020	2040
	T15	1841	2.77	1692	1832	1848	1864	1888	**1908**	1936	1968
		Average	6.01								

Table 4-12. Flow rates (veh/h/ln) for multiple percentile value in 15 minute data intervals for lower bound on basis of average of top 5% flows as lower bound

		VAM		Average	Flow Rates at Percentile Values (Lower Bound: Average Top 5%)						
District	Site ID	15 min	Top x%	Top 5%	55%	60%	65%	70%	75%	80%	85%
Jacksonville	T8	1897	4.17	1856	1987	1997	2011	2029	2048	2069	2087
	T9	1759	12.76	2004	2112	2120	2136	2149	2167	2187	2212
	T10	1743	12.37	1984	2113	2131	2144	2159	2175	2197	2217
	T11	2016	4.70	2008	2073	2081	2088	2097	2104	2112	2129
	F5	1793	2.93	1700	1814	1822	1833	1841	1854	1863	1878
	F6	1928	1.47	1720	1876	1889	1901	1915	1932	1954	1979
Ft. Laud.	FV1	1389	3.22	1340	1426	1435	1446	1464	1470	1485	1499
Orlando	T5	2015	7.94	2080	2145	2156	2165	2177	2188	2203	2221
	T6	2008	5.47	2020	2163	2216	2251	2293	2336	2380	2427
	T7	2010	10.68	2116	2243	2308	2333	2375	2432	2493	2537
	F3	1692	6.96	1736	2038	2074	2102	2123	2146	2162	2188
	F4	1836	7.54	1892	1945	1950	1958	1963	1971	1981	1997
Miami	T1	1638	10.81	1716	1781	1801	1825	1872	1941	2065	2129
	T2	1722	6.24	1752	1889	1900	1911	1921	1944	1972	2011
	T3	1565	9.22	1620	1684	1696	1721	1736	1773	1835	1943
	T4	1725	8.17	1776	1835	1847	1861	1880	1921	2017	2100
	F1	1653	2.73	1556	1659	1666	1683	1693	1703	1721	1739
	F2	1481	2.28	1400	1456	1465	1471	1481	1506	1542	1597
Tampa	T12	1858	3.36	1804	1897	1907	1917	1928	1939	1952	1965
	T13	1940	7.71	2008	2100	2109	2120	2131	2144	2159	2175
	T14	1831	6.83	1868	1936	1940	1947	1957	1969	1980	1988
	T15	1767	3.67	1716	1831	1843	1861	1881	1900	1921	1940
		Average	6.42								

Table 4-13. Flow rates (veh/h/ln) for multiple percentile value in 15 minute data intervals for lower bound on basis of average of top 6.5% flows as lower bound

		VAM		Average	Flow Rates at Percentile Values (Lower Bound: Average Top 6.5%)						
District	Site ID	15 min	Top x%	Top 6.5%	55%	60%	65%	70%	75%	80%	85%
Jacksonville	T8	1897	4.17	1784	1952	1968	1988	2001	2021	2044	2072
	T9	1759	12.76	1940	2081	2096	2108	2121	2144	2159	2187
	T10	1743	12.37	1916	2089	2101	2116	2136	2153	2172	2201
	T11	2016	4.70	1956	2051	2057	2069	2079	2088	2101	2109
	F5	1793	2.93	1628	1791	1808	1817	1829	1838	1855	1865
	F6	1928	1.47	1644	1831	1848	1868	1887	1903	1924	1953
Ft. Laud.	FV1	1389	3.22	1308	1405	1414	1424	1436	1451	1468	1485
Orlando	T5	2015	7.94	2048	2116	2127	2139	2155	2165	2183	2201
	T6	2008	5.47	1984	2065	2087	2115	2179	2243	2304	2359
	T7	2010	10.68	2088	2141	2157	2185	2228	2312	2372	2449
	F3	1692	6.96	1700	1955	1990	2027	2085	2108	2138	2162
	F4	1836	7.54	1860	1923	1932	1940	1947	1957	1965	1979
Miami	T1	1638	10.81	1692	1747	1760	1768	1781	1811	1852	2013
	T2	1722	6.24	1716	1861	1876	1893	1905	1917	1949	1988
	T3	1565	9.22	1596	1648	1660	1669	1691	1713	1745	1812
	T4	1725	8.17	1752	1808	1815	1821	1841	1861	1896	1993
	F1	1653	2.73	1492	1626	1641	1656	1666	1687	1698	1721
	F2	1481	2.28	1372	1440	1445	1454	1463	1472	1494	1527
Tampa	T12	1858	3.36	1764	1875	1885	1899	1912	1924	1937	1955
	T13	1940	7.71	1972	2072	2087	2099	2109	2127	2141	2160
	T14	1831	6.83	1840	1920	1928	1936	1944	1952	1969	1981
	T15	1767	3.67	1668	1801	1816	1831	1848	1876	1899	1928
		Average	6.42								

Table 4-14. Maximum flow rates (veh/h/ln), date and day of time of occurrence for rural freeway sites for 5 minute aggregated data

Site ID	Interstate	County	Site	Location Description	Maximum Flow Rate (NB)			Maximum Flow Rate (SB)		
					veh/h/ln	Date	Time	veh/h/ln	Date	Time
Site 1	I-95	74	0132	2.0 Mile S of GA State Line	1548	11/29	11:05	1400	11/29	13:55
Site 2	I-95	79	0133	I-95, 2.7 Mile N of SR 44 @CR44	1263	11/28	11:00	1488	11/29	14:00
Site 3	I-75	14	0190	SR-93/I-75, 0.6 Mile S of SR 54	1830	11/28	10:25	1710	11/29	14:05
Site 4	I-75	36	0317	I-75, 0.35 Mile N of William Road	1832	11/28	9:40	1816	11/25	16:30
Site 5	I-75	29	0320	SR-93/I-75, Between I-10 and US-90	1536	11/29	15:25	1704	11/29	15:05
Site 6	I-95	70	0332	SR-9/I-95, 0.9 Mile S of Aurantia Rd	1048	11/25	15:25	1144	11/25	7:45
Site 7	I-75	18	0358	North of SR 48	1734	11/28	13:55	1896	11/28	20:50
Site 8	I-4	79	9906	On I-4, 169' E of Enterprise Road	1351	11/25	15:00	1391	11/30	7:00
Site 9	I-75	18	9920	At Co Rd 514	1824	11/28	13:20	1866	11/28	20:30
Site 10	Turnpike	97	9931	Just South of SR 91 or Co Road 468	1974	11/28	10:45	1590	11/29	14:05

Table 4-15. Maximum flow rates (veh/h/ln), date and day of time of occurrence for rural freeway sites for 15 minute aggregated data

Site ID	Interstate	County	Site	Location Description	Maximum Flow Rate (NB)			Maximum Flow Rate (SB)		
					veh/h/ln	Date	Time	veh/h/ln	Date	Time
Site 1	I-95	74	0132	2.0 Mile S of GA State Line	1392	11/29	12:05	1400	11/29	11:15
Site 2	I-95	79	0133	I-95, 2.7 Mile N of SR 44 @CR44	1228	11/28	11:00	1484	11/29	15:00
Site 3	I-75	14	0190	SR-93/I-75, 0.6 Mile S of SR 54	1788	11/28	10:15	1710	11/29	14:50
Site 4	I-75	36	0317	I-75, 0.35 Mile N of William Road	1719	11/28	9:30	1816	11/25	16:10
Site 5	I-75	29	0320	SR-93/I-75, Between I-10 and US-90	1414	11/29	14:55	1704	11/29	14:55
Site 6	I-95	70	0332	SR-9/I-95, 0.9 Mile S of Aurantia Rd	996	11/25	15:15	1144	11/25	7:40
Site 7	I-75	18	0358	North of SR 48	1654	11/28	13:15	1896	11/29	17:35
Site 8	I-4	79	9906	On I-4, 169' E of Enterprise Road	1351	11/25	15:00	1391	11/30	7:00
Site 9	I-75	18	9920	At Co Rd 514	1686	11/28	13:20	1866	11/29	17:05
Site 10	Turnpike	97	9931	Just South of SR 91 or Co Road 468	1914	11/28	10:40	1590	11/29	13:45

Table 4-16. Maximum flow rates (veh/h/ln), date and day of time of occurrence for rural freeway sites for hourly aggregated data

					Maximum Flow Rate (NB)			Maximum Flow Rate (SB)		
Site ID	Interstate	County	Site	Location Description	veh/h/ln	Date	Time	veh/h/ln	Date	Time
Site 1	I-95	74	0132	2.0 Mile S of GA State Line	1303	11/29	12 Noon	1200	11/29	11:00
Site 2	I-95	79	0133	I-95, 2.7 Mile N of SR 44 @CR44	1175	11/28	11:00	1336	11/29	14:00
Site 3	I-75	14	0190	SR-93/I-75, 0.6 Mile S of SR 54	1560	11/28	10:00	1550	11/29	14:00
Site 4	I-75	36	0317	I-75, 0.35 Mile N of William Road	1458	11/25	14:00	1722	11/25	16:00
Site 5	I-75	29	0320	SR-93/I-75, Between I-10 and US-90	1347	11/29	15:00	1490	11/29	15:00
Site 6	I-95	70	0332	SR-9/I-95, 0.9 Mile S of Aurantia Rd	906	11/25	15:00	903	11/25	7:00
Site 7	I-75	18	0358	North of SR 48	1582	11/28	13:00	1433	11/29	15:00
Site 8	I-4	79	9906	On I-4, 169' E of Enterprise Road	1351	11/25	15:00	1391	11/30	7:00
Site 9	I-75	18	9920	At Co Rd 514	1508	11/28	13:00	1458	11/29	14:00
Site 10	Turnpike	97	9931	Just South of SR 91 or Co Road 468	1777	11/28	10:00	1379	11/29	14:00

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 22 freeway sites across Florida, with each of these sites 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 65% 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.

Generally, it was felt that the VAM capacity estimates were too low and the stochastic method capacity estimates were too high. In order to find a "compromise" between the estimates of the two methods, another set of capacity estimates were developed. These estimates are based on specific percentile flow rate values that fall between the VAM capacity estimates and the maximum observed flow rates. While the all the values in Tables 4-10 – 4-13 represent what can be considered to be reasonable capacity estimates, it is felt that the values in Table 4-11 are the most reasonable estimates of capacity, as these correspond to a 5-minute data aggregation interval and the lower capacity boundary corresponding to the average of top 6.5% of highest hourly flow rates most closely corresponds to the VAM capacity estimates. Furthermore, the capacity estimates pertaining to percentile values 75%-85% are likely the most appropriate. The maximum hourly flow rate observed on rural freeway segments that did not experience any congestion was found to be 1832 veh/h/ln, based on a 5-minute data aggregation interval. However for the sites that experienced congestion, the maximum hourly flow rate was found to be 1974 veh/h/ln. At this time, no specific conclusions can be reached regarding capacity estimates for rural freeway segments, and considerably more data and analysis are needed. The challenge, of course, to identifying capacity values for rural freeway segments is that these segments rarely reach capacity conditions. While a special data collection effort was made to collect data from several rural freeway sites (the Central Data Warehouse data is only available for urban areas), these data only span five days and represent possibly somewhat atypical travel conditions (i.e., holiday travel).

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

Percentile 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.

Unfortunately, the results of this study did not allow conclusions to be reached regarding the effect of factors such as merge/diverge activity, number of lanes, free-flow speed, and truck percentage on estimated capacity values. This was due to the relatively small number of analysis sites and the considerable variance in maximum flow rates within each analysis site.

Recommendations

The focus of this study was on the comparison of different methods for capacity estimation, with the intention of identifying a method that will be suitable for use by the FDOT for 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 percentile of maximum hourly flow rates (with a lower bound of the average of the highest 6.5% hourly flow rates), based on a 5-minute aggregation interval, be applied. Furthermore, it is recommended that a percentile value between 60%-80% be used for this method.

It is recommended that a follow-on study be conducted that will focus on investigating 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. This type of study will require considerably more analysis sites than were used in this study. However, as the Florida Central Data Warehouse, Courage and Lee (2009), continues to obtain data from more Florida cities and more sites within each city, it will soon be feasible to obtain data from many more sites.

To obtain the additional data that is necessary to fully investigate capacity estimates for rural freeways, it is recommended that the data collection equipment at the rural freeway sites in Florida that routinely experience the highest flow rates be reprogrammed to save the data in 5-minute aggregation intervals, rather than the 60-minute intervals currently used.

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APPENDIX A

Table A-1. Comparisons of RTMS counts with PTMS counts for District 2

County:	72	County:	72
PTMS Station:	0864	RTMS Station:	210711
Description:	SR-9 (I-95) 0.5 Mi of SR-5 (US-1)	Description:	NB North of I-295S
Start Date:	7/1/2008	Start Date:	7/1/2008

	MINUTES/VEHICLES - DIRECTION: NORTHBOUND												TOTALS:		
	15			30			45			60					
Time	RTMS	PTMS	%	RTMS	PTMS	%	RTMS	PTMS	%	RTMS	PTMS	%	RTMS	PTMS	%
5.00	257	252	1.95	338	248	26.63	462	405	12.34	585	502	14.19	1,642	1,407	14.31
6.00	776	650	16.24	1,121	911	18.73	1,488	1,297	12.84	1623	1,564	3.64	5,008	4,422	11.70
7.00	1,745	1,644	5.79	1,983	1,858	6.30	1,962	2,042	-4.08	1947	1,993	-2.36	7,395	7,391	0.05
8.00	1,943	1,788	7.98	1,741	1,712	1.67	1,663	1,704	-2.47	1522	1,536	-0.92	5,982	6,146	-2.74
9.00	1,131	1,265	-11.85	854	1,118	-30.91	1,087	1,106	-1.75	944	1,041	-10.28	4,063	4,322	-6.37
10.00	894	610	31.77	901	910	-1.00	899	886	1.45	917	899	1.96	3,499	3,198	8.60
11.00	864	838	3.01	789	803	-1.77	894	881	1.45	879	853	2.96	3,435	3,330	3.06
12.00	813	810	0.37	798	758	5.01	838	785	6.32	857	865	-0.93	3,324	3,253	2.14
13.00	837	802	4.18	816	793	2.82	854	818	4.22	770	800	-3.90	3,277	3,213	1.95
14.00	778	779	-0.13	826	814	1.45	738	764	-3.52	672	749	-11.46	3,014	3,106	-3.05
15.00	717	761	-6.14	688	708	-2.91	782	768	1.79	726	758	-4.41	2,913	2,995	-2.81
16.00	742	735	0.94	707	686	2.97	816	754	7.60	786	779	0.89	3,051	2,954	3.18
17.00	784	743	5.23	799	765	4.26	790	770	2.53	681	693	-1.76	3,054	2,971	2.72
18.00	693	645	6.93	702	659	6.13	578	557	3.63	607	601	0.99	2,580	2,462	4.57
19.00	591	565	4.40	528	463	12.31	456	438	3.95	482	469	2.70	2,057	1,935	5.93
20.00	447	401	10.29	365	342	6.30	361	303	16.07	357	328	8.12	1,530	1,374	10.20
21.00	381	323	15.22	344	324	5.81	349	316	9.46	315	289	8.25	1,389	1,252	9.86
TOTALS	15,509	14,565	**6.09**	15,472	14,775	**4.50**	16,148	15,505	**3.98**	15700	15,586	**0.73**	62,829	60,431	**3.82**

Table A-2. Comparisons of RTMS counts with PTMS counts for District 4

County: 86
PTMS Station: 2806
Description: SR 862/I-595 - 0.8 MI E of SR 7/US 441
Start Date: 05/29/08

County: 86
RTMS: 420292
Description: SR 862/I-595 0.8 MI E of SR 7/US 441
Start Date: 05/29/2008

	MINUTES/VEHICLES - DIRECTION: WESTBOUND														
	0			15			30			45			TOTALS:		
Time	**RTMS**	**PTMS**	**%**	**RTMS**	**PTMS**	**%**	**RTMS**	**PTMS**	**%**	**RTMS**	**PTMS**	**%**	**RTMS**	**PTMS**	**%**
6.00	428	471	-10.05	595	624	-4.87	807	863	-6.94	917	1,042	-13.63	2,747	3,000	-9.21
7.00	981	1,069	-8.97	1,047	1,318	-25.88	1,258	1,546	-22.89	1,358	1,553	-14.36	4,948	5,728	-15.76
8.00	1,334	1,472	-10.34	1,351	1,560	-15.47	1,381	1,473	-6.66	1,200	1,325	-10.42	4,861	5,394	-10.96
9.00	1,001	1,072	-7.09	946	1,124	-18.82	1,024	1,071	-4.59	1,047	1,106	-5.64	4,024	4,361	-8.37
10.00	1,004	955	4.88	952	1,112	-16.81	1,020	1,082	-6.08	836	1,069	-27.87	3,861	4,186	-8.42
11.00	990	1,137	-14.85	1,001	1,080	-7.89	1,031	1,134	-9.99	1,059	1,210	-14.26	4,180	4,624	-10.62
12.00	1,087	1,178	-8.37	1,100	1,143	-3.91	1,180	1,167	1.10	1,014	1,257	-23.96	4,441	4,750	-6.96
13.00	1,030	1,076	-4.47	1,160	1,148	1.03	1,013	1,155	-14.02	1,163	1,293	-11.18	4,366	4,672	-7.01
14.00	1,161	1,269	-9.30	1,246	1,342	-7.70	1,209	1,420	-17.45	1,248	1,538	-23.24	4,864	5,569	-14.49
15.00	1,343	1,596	-18.84	1,404	1,653	-17.74	1,464	1,735	-18.51	1,624	1,687	-3.88	5,835	6,671	-14.33
16.00	1,564	1,758	-12.40	1,792	1,904	-6.25	1,585	1,910	-20.50	1,812	1,982	-9.38	6,753	7,554	-11.86
17.00	1,821	2,097	-15.16	1,707	2,104	-23.26	1,650	1,915	-16.06	1,785	1,795	-0.56	6,963	7,911	-13.61
18.00	1,332	1,576	-18.32	1,453	1,623	-11.70	1,239	1,513	-22.11	1,190	1,407	-18.24	5,214	6,119	-17.36
19.00	1,097	1,231	-12.22	1,058	1,189	-12.38	875	1,044	-19.31	855	949	-10.99	3,885	4,413	-13.59
20.00	771	941	-22.05	800	895	-11.88	687	792	-15.28	698	701	-0.43	2,956	3,329	-12.62
21.00	596	731	-22.65	671	732	-9.09	602	660	-9.63	538	629	-16.91	2,407	2,752	-14.33
TOTALS	17,540	19,629	**-11.91**	18,283	20,551	**-12.40**	18,025	20,480	**-13.62**	18,344	20,543	**-11.99**	72,192	81,203	**-12.48**

Table A-3. Comparisons of RTMS counts with PTMS counts for District 5

County: 75 | County: 75
PTMS Station: 3080 | RTMS: T5
Description: I-4, 0.635 miles NE of SR-414 | Description: East of Wymore Road
Start Date: 4/7/2008 | Start Date: 4/7/2008

	MINUTES/VEHICLES - DIRECTION: WB															
	0			15			30			45			TOTALS:		
Time	**RTMS**	**PTMS**	**%**	**RTMS**	**PTMS**	**%**	**RTMS**	**PTMS**	**%**	**RTMS**	**PTMS**	**%**	**RTMS**	**PTMS**	**%**
5.00	112	381	-240.18	337	570	-69.14	532	695	-30.64	759	845	-11.33	1,740	2,491	-43.16
6.00	783	1059	-35.25	1033	1474	-42.69	1492	1618	-8.45	1,561	1633	-4.61	4,869	5,784	-18.79
7.00	1643	1635	0.49	1515	1577	-4.09	1524	1559	-2.30	1,274	1394	-9.42	5,956	6,165	-3.51
8.00	1150	1480	-28.70	1392	1546	-11.06	1197	1507	-25.90	982	1438	-46.44	4,721	5,971	-26.48
9.00	1422	1351	4.99	1384	1377	0.51	1380	1370	0.72	1,458	1231	15.57	5,644	5,329	5.58
10.00	1295	1181	8.80	1097	1078	1.73	1113	1189	-6.83	1,161	1125	3.10	4,666	4,573	1.99
11.00	1114	1076	3.41	1116	1169	-4.75	190	1100	-478.95	782	1039	-32.86	3,202	4,384	-36.91
12.00	1094	1080	1.28	1107	1072	3.16	1088	1138	-4.60	1,078	1014	5.94	4,367	4,304	1.44
13.00	1124	1099	2.22	1118	1108	0.89	980	1154	-17.76	1,143	1207	-5.60	4,365	4,568	-4.65
14.00	1178	1076	8.66	1132	1081	4.51	1117	1203	-7.70	1,123	1116	0.62	4,550	4,476	1.63
15.00	1118	1154	-3.22	1097	1150	-4.83	1216	1108	8.88	1,070	1211	-13.18	4,501	4,623	-2.71
16.00	1148	1102	4.01	1126	1112	1.24	1245	1138	8.59	1,232	1241	-0.73	4,751	4,593	3.33
17.00	1369	1270	7.23	1267	1380	-8.92	1252	1232	1.60	1,037	1271	-22.57	4,925	5,153	-4.63
18.00	1094	1037	5.21	935	1083	-15.83	778	943	-21.21	790	764	3.29	3,597	3,827	-6.39
19.00	747	823	-10.17	599	717	-19.70	575	610	-6.09	587	565	3.75	2,508	2,715	-8.25
20.00	595	607	-2.02	527	574	-8.92	503	522	-3.78	545	508	6.79	2,170	2,211	-1.89
21.00	537	561	-4.47	474	542	-14.35	392	454	-15.82	367	392	-6.81	1,770	1,949	-10.11
22.00	407	374	8.11	340	393	-15.59	314	337	-7.32	257	316	-22.96	1,318	1,420	-7.74
23.00	269	256	4.83	268	262	2.24	207	278	-34.30	0	200	#DIV/0!	744	996	-33.87
TOTALS	**18,199**	**18,602**	**-2.21**	**17,864**	**19,265**	**-7.84**	**17095.00**	**19,155**	**-12.05**	**17,206**	**18,510**	**-7.58**	**70,364**	**75,532**	**-7.34**

Table A-4. Comparisons of RTMS counts with PTMS counts for District 6-i

County:	87	County:	87
PTMS Station:	2501	RTMS:	650151
Description:	SR 93/I-75, 200' S. Miami Gardens DR/SR 860	Description:	I-75 South of NW 170 St.
Start Date:	09/16/2008	Start Date:	09/16/2008

	MINUTES/VEHICLES - DIRECTION: NORTHBOUND														
	0			15			30			45			**TOTALS:**		
Time	**RTMS**	**PTMS**	**%**	**RTMS**	**PTMS**	**%**	**RTMS**	**PTMS**	**%**	**RTMS**	**PTMS**	**%**	**RTMS**	**PTMS**	**%**
5.00	105	102	2.86	161	173	-7.45	183	206	-12.57	188	206	-9.57	637	687	-7.85
6.00	227	262	-15.42	398	420	-5.53	545	532	2.39	536	492	8.21	1706	1,706	0.00
7.00	558	513	8.06	697	661	5.16	762	738	3.15	754	738	2.12	2816	2,704	3.98
8.00	695	693	0.29	742	715	3.64	653	688	-5.36	636	629	1.10	2502	2,524	-0.88
9.00	515	509	1.17	518	514	0.77	446	485	-8.74	504	459	8.93	1961	1,955	0.31
10.00	504	445	11.71	496	502	-1.21	251	511	-103.59	562	492	12.46	1870	1,987	-6.26
11.00	484	515	6.40	553	539	2.53	517	532	-2.90	535	525	1.87	2129	2,155	-1.22
12.00	417	616	47.72	593	583	1.69	720	584	18.89	673	616	8.47	2489	2,438	2.05
13.00	625	630	0.80	679	622	8.39	740	714	3.51	724	701	3.18	2768	2,667	3.65
14.00	709	727	2.54	766	732	4.44	869	836	3.80	884	842	4.75	3228	3,137	2.82
15.00	950	910	4.21	1,020	1,041	-2.06	1,110	1,071	3.51	577	1,105	-91.51	3657	4,127	-12.85
16.00	1,847	1,115	39.63	1,486	1,278	14.00	1,498	1,281	14.49	1,553	1,334	14.10	6384	5,008	21.55
17.00	1,722	1,477	14.23	1,782	1,614	9.43	1,770	1,488	15.93	1,703	1,414	16.97	6977	5,993	14.10
18.00	1,602	1,452	9.36	1,609	1,320	17.96	1,520	1,304	14.21	1,303	1,209	7.21	6034	5,285	12.41
19.00	1,141	1,043	8.59	1,012	953	5.83	910	819	10.00	789	764	3.17	3852	3,579	7.09
20.00	776	785	1.16	748	700	6.42	675	562	16.74	578	525	9.17	2777	2,572	7.38
21.00	545	510	6.42	532	352	33.83	556	475	14.57	453	549	-21.19	2086	1,886	9.59
TOTALS	14,571	13,599	**6.67%**	14,716	13,949	**5.21%**	14,637	13,954	**4.67%**	13,740	13,654	**0.63%**	57664	55,156	**4.35%**

Table A-5. Comparisons of RTMS counts with PTMS counts for District 6-ii

County: 87 | County: 87
PTMS Station: 0579 | RTMS: 610091
Description: SR 826/PALMETTO EXPWY, 1000' E NW 27 AV | Description: EAST OF NW 27 AVENUE
Start Date: 10/14/2008 | Start Date: 10/14/2008

	MINUTES/VEHICLES - DIRECTION: EASTBOUND (NB)														
	0			15			30			45			TOTALS:		
Time	**RTMS**	**PTMS**	**%**	**RTMS**	**PTMS**	**%**	**RTMS**	**PTMS**	**%**	**RTMS**	**PTMS**	**%**	**RTMS**	**PTMS**	**%**
5.00	282	291	-3.19	428	416	2.80	573	518	9.60	610	597	2.13	1893	1,822	3.75
6.00	789	714	9.51	1,008	973	3.47	1,155	1,086	5.97	1,235	1,110	10.12	4187	3,883	7.26
7.00	1,366	1,140	16.54	1,360	1,213	10.81	1,396	1,258	9.89	1,372	1,246	9.18	5494	4,857	11.59
8.00	1,320	1,157	12.35	1,159	1,086	6.30	1,059	1,006	5.00	1,175	995	15.32	4713	4,244	9.95
9.00	1,012	990	2.17	1,041	1,014	2.59	1,085	978	9.86	992	953	3.93	4130	3,935	4.72
10.00	531	853	-60.64	897	876	2.34	1,519	950	37.46	879	895	-1.82	3826	3,574	6.59
11.00	648	866	-33.64	1,308	910	30.43	1,234	945	23.42	977	880	9.93	4167	3,601	13.58
12.00	987	878	11.04	1,101	977	11.26	1,000	925	7.50	1,024	913	10.84	4112	3,693	10.19
13.00	1,036	947	8.59	1,116	976	12.54	1,065	1,009	5.26	1,079	948	12.14	4296	3,880	9.68
14.00	1,081	972	10.08	1,097	1,030	6.11	1,127	1,046	7.19	1,148	1,046	8.89	4453	4,094	8.06
15.00	968	1,056	-9.09	1,338	1,028	23.17	1,199	1,068	10.93	1,151	1,080	6.17	4656	4,232	9.11
16.00	1,155	1,084	6.15	1,208	1,065	11.84	1,156	1,049	9.26	1,153	1,074	6.85	4672	4,272	8.56
17.00	1,184	1,049	11.40	1,186	1,097	7.50	1,107	1,001	9.58	1,107	1,050	5.15	4584	4,197	8.44
18.00	1,127	1,047	7.10	1,088	1,046	3.86	1,081	982	9.16	938	915	2.45	4234	3,990	5.76
19.00	1,002	873	12.87	941	887	5.74	950	819	13.79	924	801	13.31	3817	3,380	11.45
20.00	877	775	11.63	762	733	3.81	716	653	8.80	673	636	5.50	3028	2,797	7.63
21.00	646	603	6.66	697	662	5.02	643	579	9.95	647	619	4.33	2633	2,463	6.46
22.00	647	600	7.26	578	579	-0.17	557	562	-0.90	461	453	1.74	2243	2,194	2.18
TOTALS	**17,812**	**17,004**	**4.54**	**19,489**	**17,678**	**9.29**	**19,727**	**17,486**	**11.36**	**18,600**	**17245**	**7.28**	**75628**	**69,413**	**8.22**