FINAL REPORT

to

THE FLORIDA DEPARTMENT OF TRANSPORTATION
SYSTEMS PLANNING OFFICE

on Project

"**Estimation of Capacities on Florida Freeways**"

FDOT Contract BDV32-977-03

September 2014

by

Lily Elefteriadou, Alexandra Kondyli, and Bryan St. George
Transportation Research Center
The University of Florida

DISCLAIMER

The opinions, findings, and conclusions expressed in this publication are those of the authors and not necessarily those of the State of Florida Department of Transportation.

ACKNOWLEDGEMENT

Ms. Gina Bonyani served as the Project Manager on behalf of the Florida Department of Transportation.

METRIC CONVERSION CHART

U.S. UNITS TO METRIC (SI) UNITS

LENGTH

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
in	inches	25.4	millimeters	mm
ft	feet	0.305	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km

METRIC (SI) UNITS
TO U.S. UNITS

LENGTH

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi

TECHNICAL REPORT DOCUMENTATION PAGE

1. Report No.	2. Government Accession No.	3. Recipient's Catalog No.
4. Title and Subtitle Estimation of Capacities on Florida Freeways		5. Report Date September 2014
		6. Performing Organization Code
7. Author(s) Lily Elefteriadou, Alexandra Kondyli, Bryan St. George		8. Performing Organization Report No.
9. Performing Organization Name and Address Transportation Research Center University of Florida 512 Weil Hall, PO Box 116580 Gainesville, FL 32611-6580		10. Work Unit No. (TRAIS)
		11. Contract or Grant No. FDOT Contract BDV32-977-03
12. Sponsoring Agency Name and Address Florida Department of Transportation 605 Suwannee Street, MS 30 Tallahassee, FL 32399		13. Type of Report and Period Covered Final Report, 3/11/2013 to 9/30/2014
		14. Sponsoring Agency Code
15. Supplementary Notes		

16. Abstract

Current capacity estimates within Florida's travel time reliability tools rely on the Highway Capacity Manual (HCM 2010) to estimate capacity under various conditions. Field measurements show that the capacities of Florida freeways are noticeably lower than the values recommended in the HCM 2010, by an order of a few hundred vehicles. In addition, recent research has shown that maximum freeway throughput may differ between undersaturated and oversaturated conditions, and this is not acknowledged in the HCM 2010.

The main objective of this research was to collect field data at several urban and rural freeway and multilane locations in Florida in order to measure capacity flows and to provide recommended capacity values before and after the initiation of oversaturation. The research team obtained data at urban and rural freeways and multilane highway segments across Florida. The urban freeway data were obtained through the Statewide Transportation Engineering Warehouse for Archived Regional Data (STEWARD) at various types of bottlenecks including merge junctions, weaving segments, as well as geometric bottlenecks (lane drops), while the rural freeway and multilane highway data were obtained from the permanent count stations of FDOT. Incidents and weather data were also obtained to ensure that the final datasets included capacity observations due to excess demand and not due to random events such as incidents or bad weather.

Various capacity measures were investigated, and it is recommended to define pre-breakdown capacity as the 85th percentile of the 15-min average pre-breakdown flow and the post-breakdown capacity as the average discharge flow. A clear drop in throughput between pre-breakdown and discharge capacity values was observed. Recommendations on capacity values as a function of the number of lanes and the segment type for both urban and rural locations are offered. This research also proposes revised density thresholds for defining Level of Service on various types of segments consistent with the recommended capacity values.

17. Key Words	18. Distribution Statement		
capacity, bottlenecks, urban freeways, rural freeways, multilane highways, capacity drop	No restrictions		
19. Security Classif. (of this report)	**20. Security Classif. (of this page)**	**21. No. of Pages**	**22. Price**
Unclassified	Unclassified	63	

EXECUTIVE SUMMARY

The Highway Capacity Manual (HCM) is the publication used most often to estimate capacity. The HCM 2010 indicates that the capacity of a basic freeway segment varies with free-flow speed (FFS), and that under base conditions, it ranges from 2,400 passenger cars per hour per lane (pc/h/ln) for FFS 70 or 75 mi/h, to 2,250 pc/h/ln for FFS 55 mi/h. Similarly, the HCM 2010 indicates that the capacity of a multilane highway segment ranges from 2,200 pc/h/ln (for FFS 60 mi/h) to 1,900 (for FFS 45 mi/h). However, recent research has shown that the maximum freeway throughput may differ between undersaturated and oversaturated conditions and that the difference may be on the order of a 10% drop in throughput after traffic flow breakdown (i.e., beginning of oversaturated conditions.)

Existing models for estimating travel time reliability rely on capacity values in order to estimate travel time under various scenarios, including travel time for undersaturated and oversaturated conditions. Accurate capacity estimates are essential in (a) determining whether demand exceeds capacity and congested conditions are to be anticipated and (b) in estimating the expected travel times under various conditions as a function of the demand and the capacity of a segment.

Current capacity estimates within Florida's travel time reliability tools rely on the HCM 2010 to estimate capacity under various conditions. Field measurements show that the capacities of Florida freeways are noticeably lower than the values estimated in the HCM 2010 by an order of a few hundred vehicles. Also, field measurements seem to indicate that the capacities at merge junctions are lower than the recommended basic freeway segment capacities. No studies have been identified that estimate or measure the capacity of multilane highways in Florida.

The main objective of this research is to collect field data at several urban and rural freeway and multilane locations in Florida in order to capture capacity flows, and to provide recommended capacity values before and after the initiation of oversaturation. The urban freeway data were obtained through STEWARD at various types of bottlenecks including merge junctions, weaving segments, as well as geometric bottlenecks (lane drops), while the rural freeway and multilane highway data were obtained from the permanent count stations of FDOT. In addition to the traffic data, incidents and weather data were also obtained to facilitate the data collection process and ensure that the final datasets include capacity observations due to excess demand and not due to random events such as incidents or bad weather.

All capacity measures presented are related to the occurrence of breakdown events at the study sites, which are identified through sharp speed drops (e.g., at least 10 mi/h between two time intervals). At the multilane highway sites, breakdown events were not observed, thus, a thorough capacity analysis was not performed. Various capacity measures were investigated, such as the breakdown flow, the maximum pre-breakdown flow, the average pre-breakdown flow, and the average discharge flow, as well as a variety of statistics (e.g., 50^{th} percentile, 85^{th} percentile), and these values were considerably lower than the HCM 2010 values. The relationship between capacity and bottleneck type, number of lanes, and free-flow speed was also investigated.

Based on the analysis results, it is recommended to define pre-breakdown capacity as the 85^{th} percentile of the 15-min average pre-breakdown flow and the post-breakdown capacity as the average discharge flow. A clear drop in throughput between pre-breakdown and discharge capacity values was observed. This research provides recommended capacity values as a function of the number of lanes and the segment type for both urban and rural locations. This research also proposes revised density thresholds for defining Level of Service at various bottlenecks as a function of the recommended capacity values.

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

1. INTRODUCTION

1.1 Background

The Highway Capacity Manual is the publication used most often to estimate capacity. The current published version of the HCM (TRB, 2010) defines the capacity of a facility as ". . . the maximum sustainable hourly flow rate at which persons or vehicles reasonably can be expected to traverse a point or a uniform section of a lane or roadway during a given time period, under prevailing roadway, environmental, traffic, and control conditions." The HCM 2010 indicates that the capacity of freeways and multilane highways varies with free-flow speed (FFS). The HCM capacity values for basic freeway and multilane highway segments (in pc/h/ln) are shown in Table 1.1. The weaving segments methodology in the HCM 2010 calculates weaving segment capacities (HCM 2010, Equation 12-5). Based on this calculation, the weaving segment capacity is always less than the capacity of a basic freeway segment with the same FFS. The merge/diverge segments methodology of the HCM 2010 does not provide capacity at those segments, but rather a maximum flow entering the merge (ramp flow plus flow at lanes 1 and 2) or the diverge (flow at lanes 1 and 2) area, which is a function of the FFS. The capacity values shown in Table 1.1, as well as those estimated by equations, represent national averages, and the HCM 2010 indicates that any given location may have higher or lower capacities.

Table 1.1. 2010 HCM (TRB, 2010) Values for Capacity on Basic Freeway and Multilane Highway Segments

Speed (mi/h)	Capacity (pc/h/ln)
Basic Freeway Segments	
70, 75	2,400
65	2,350
60	2,300
55	2,250
Multilane Highways	
60	2,200
55	2,100
50	2,000
45	1,900

Basic freeway segments are rarely bottlenecks (they may form bottlenecks when grades are steep or when other geometric elements are restrictive,) and thus the maximum flows observed at these would not represent capacity unless they are followed by oversaturated conditions. It is not clear whether the values recommended by the HCM represent flows before the breakdown, or maximum flows obtained irrespective of breakdowns.

The adopted FDOT peak hour directional volumes for freeways and multilane highways are shown in Table 1.2. They are provided in units of vehicles rather than PCEs (FDOT, 2013) and thus are noticeably lower than the values provided in the HCM 2010. The FDOT values assume 4 percent heavy vehicles on urbanized freeways and 2 percent on highways. For example, if we assume 4 percent heavy vehicles on level terrain and commuter traffic, then the corresponding capacity values for a four-lane urban freeway is approximately 2100 pc/h/ln (PHF is assumed to be 1). Table 1.2 also provides the corresponding FDOT - recommended capacity values in pc/h/ln based on these truck percentages and for PHF = 1.

Table 1.2. FDOT Peak Hour Directional Volumes and Capacity Values on Various Roadways (FDOT, 2013)

	Urbanized Areas Freeways		**Non-Urbanized Areas Freeways**	
Lanes	**veh/h/ln**	**pce/h/ln**	**veh/h/ln**	**pce/h/ln**
2	1,970	2,010	1,790	1,830
3	2,027	2,070	1,847	1,890
4	2,055	2,100	1,875	1,920
5	2,072	2,120	1,888	1,930
6	2,083	2,130	-	-
	Urbanized Areas Multilane Hwys*		**Non-Urbanized Areas Multilane Hwys***	
Lanes	**veh/h/ln**	**pce/h/ln**	**veh/h/ln**	**pce/h/ln**
2	1,795	1,820	1,720	1,740
3	1,793	1,820	1,723	1,740

* Divided highway

The capacity values of Table 1.2 are a function of the number of lanes, rather than the FFS. This approach is consistent with previous research (Lu and Elefteriadou, 2013) which found that capacity differs by the number of lanes, and is higher for 3-lane facilities than for 2-lane or 4-lane facilities. However, in the FDOT capacity values, capacity increases with the number of lanes even beyond 4-lane facilities. FDOT recommends different capacity values for urbanized vs. non-urbanized facilities.

Research on freeway capacity (Cassidy and Bertini, 1999; Lorenz and Elefteriadou, 2001; Persaud et al., 2001; Brilon, 2005) has examined the conditions under which breakdown occurs, and concluded that it does not occur deterministically under a given set of volumes. Also, several of these articles have shown that this maximum value does not necessarily coincide with the breakdown event. Lastly, it has also been shown that regardless of whether one uses the maximum pre-breakdown flow, or the breakdown flow to define capacity, both values vary widely on a daily basis even for the same site and for similar traffic conditions. This is inconsistent with traditional traffic analysis methods, such as the HCM 2000 (TRB, 2000) and the HCM 2010, which assume that traffic transitions to oversaturated conditions (i.e., breakdown event) when demand reaches a specific maximum value, labeled as capacity.

Recent research has shown that maximum freeway throughput may be different in undersaturated and oversaturated conditions, and that the difference may be in the range between -7.76% and 17.3% drop in throughput after traffic flow breakdown. It should be noted though, that the literature focuses primarily on freeway merging segments, while there is limited information about the capacity drop percent at weaving, diverging segments, or at lane drops.

Current capacity estimates within Florida's travel time reliability tools rely on the HCM 2010 to estimate capacity under various conditions. Field measurements show that the capacities of Florida freeways are noticeably lower than the values estimated in the HCM 2010, by an order of a few hundred vehicles. Also, field measurements seem to indicate that the capacities at merge junctions are lower than basic freeway segment capacities. No studies have been identified that estimate or measure the capacity of multilane highways in Florida.

Existing models for estimating travel time reliability rely on capacity estimates in order to estimate travel time under various scenarios, including travel time for undersaturated and oversaturated conditions. Accurate capacity estimates are essential in a) determining whether demand exceeds capacity and congested conditions are to be anticipated, and b) in estimating the expected travel times under various conditions as a function of the demand and the capacity of a segment. Thus, it is important to obtain accurate capacity estimates considering Florida conditions such as driver populations, degree of aggressiveness, area types, etc., as well as different types of facilities prevalent in the State.

1.2. Objectives

The main objective of this research is to collect field data at several (urban and rural) freeway locations in Florida in order to measure capacity flows, and to provide recommended capacity and the corresponding speed values before and after the initiation of oversaturation. The research team also identified a limited number of suitable locations along multilane highways to conduct a similar analysis. The urban freeway data were obtained through STEWARD at various types of bottlenecks including merge junctions, weaving segments, as well as geometric bottlenecks (lane drops), while the rural freeway and multilane highway data were obtained from the permanent count stations of FDOT.

1.3. Report Organization

The next chapter presents the data collection effort undertaken for this project. Chapter 3 presents the data analysis and derivation of capacity values at each site. Chapter 4 presents the formulated recommendations regarding the measurement of capacity as well as recommended values for various types of facilities and for undersaturated and oversaturated conditions. The literature review conducted for this project related to the capacity drop phenomenon and the definition of capacity is provided in Appendix A.

2. DATA COLLECTION

Data from urban freeways, rural freeways and multilane highways were obtained and analyzed. In addition to the traffic data, incidents and weather data were also obtained to facilitate the data collection process and ensure that the final datasets include capacity observations due to excess demand and not due to random events such as incidents or bad weather.

All capacity measures presented are related to the occurrence of breakdown events at the study sites. If these are not present we cannot be certain that “capacity” has been reached. These breakdown events are identified through sharp speed drops (e.g., at least 10 mi/h between two time intervals) recorded either at the upstream or downstream detector relative to the bottleneck (Figure 2.1).

At ramp merge bottlenecks, the freeway capacity is measured downstream of the on-ramp, which corresponds to the downstream detector shown in Figure 2.1a. At diverge bottlenecks, the freeway capacity is measured upstream of the off-ramp, which corresponds to the upstream detector shown in Figure 2.1b. At weaving segments, the freeway capacity is measured within the weave area, which corresponds to the subject detector shown in Figure 2.1c.

Further information on the breakdown identification algorithm used in this study, can be found in Kondyli et al. (2013).

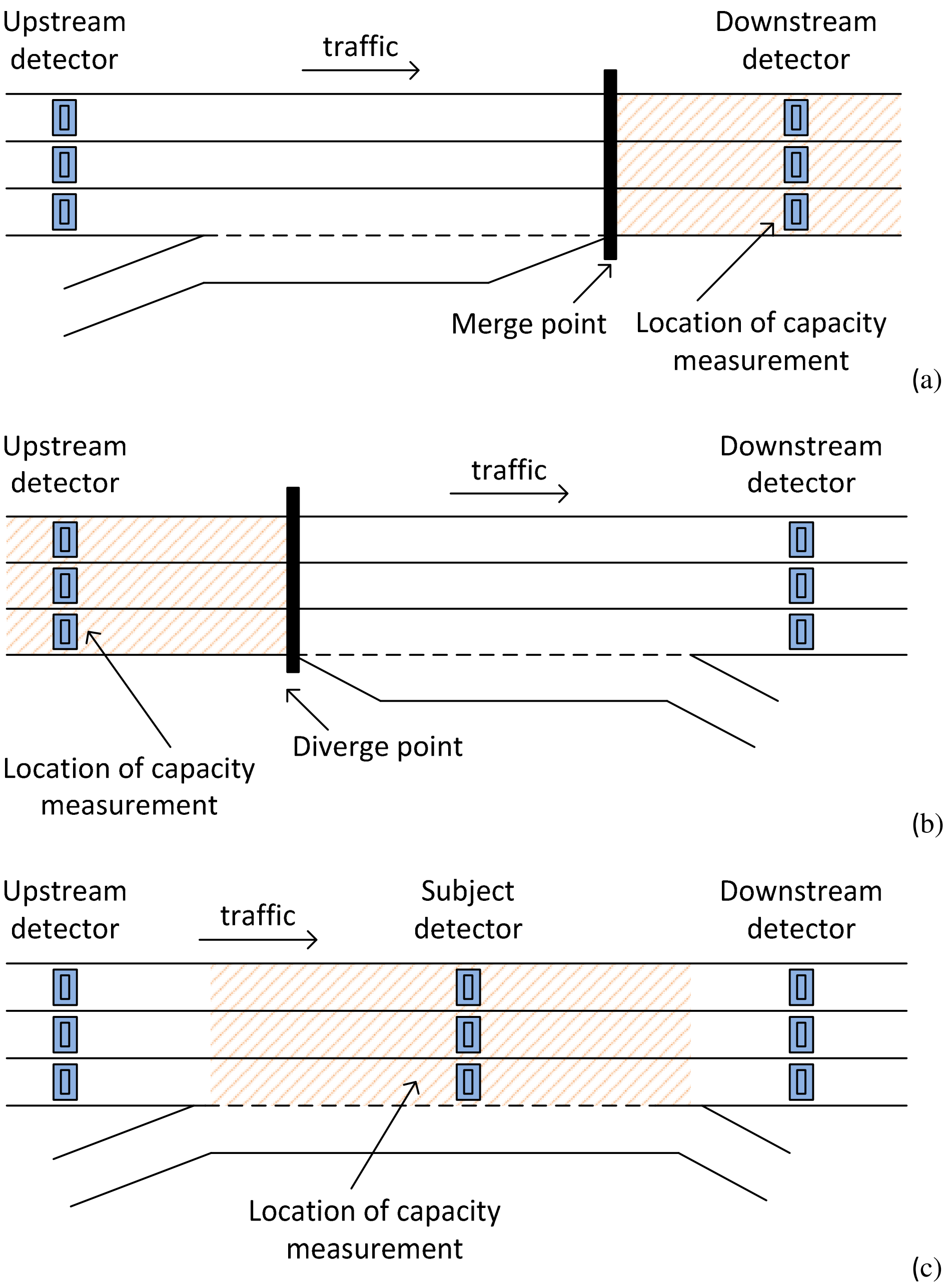

Figure 2.1. Description of capacity measurement location by (a) merge bottleneck, (b) diverge bottleneck, and (c) weaving bottleneck.

2.1. Urban Freeways

Ten urban freeway sites were examined. The sites were identified based on the following sources: FDOT (2011); Washburn et al. (2010). These sites were selected based on the following criteria:

- They experience recurrent congestion due to merging, diverging of weaving operations;
- These bottlenecks are free from downstream congestion;
- Data are available for approximately one year, excluding weekends and holidays;
- Weather and incident data are available;
- The quality of the data is generally considered good.

Speed and flow data were obtained from each site from STEWARD at 1-min increments, excluding days with bad weather or incidents. The weather conditions evaluation was conducted using data from the website http://www.nws.noaa.gov/climate/. Days that experienced precipitation over 0.20 inches or foggy conditions were omitted from the analysis. The incident information was obtained through INRIX and through the CAR database provided by the Florida Department of Transportation (FDOT.) If an incident occurred along the study site and within 5 miles downstream, that day was removed from the analysis. The overall quality of the sensor data was evaluated through various sources, such as INRIX, and the STEWARD quality checks. Truck percentages were also available through FDOT at 1-hour increments.

The remainder of this section provides site descriptions for each bottleneck location analyzed, accompanied by a schematic (not drawn to scale). The schematics specify the presence of nearby on- and off-ramps, the location of the detectors used to obtain capacity values, as well as other detectors available along the study site.

2.1.1. I-95 NB at Butler Boulevard, Jacksonville, FL

This site is located in Jacksonville, Florida, just downstream of the on-ramp from Butler Blvd (Figure 2.2). The bottleneck is activated due to weaving operations, and it consists of three lanes per direction with an auxiliary lane. Data are not available for the auxiliary lane, which was excluded from analysis. The weaving length is 4,400 ft. The speed limit at the segment is 65 mi/h and the AADT is 112,000 vehicles. Speed data were collected for both detectors shown in red in order to identify the breakdown events. Capacity information was collected from the downstream detector, as indicated earlier and in Figure 2.1c.

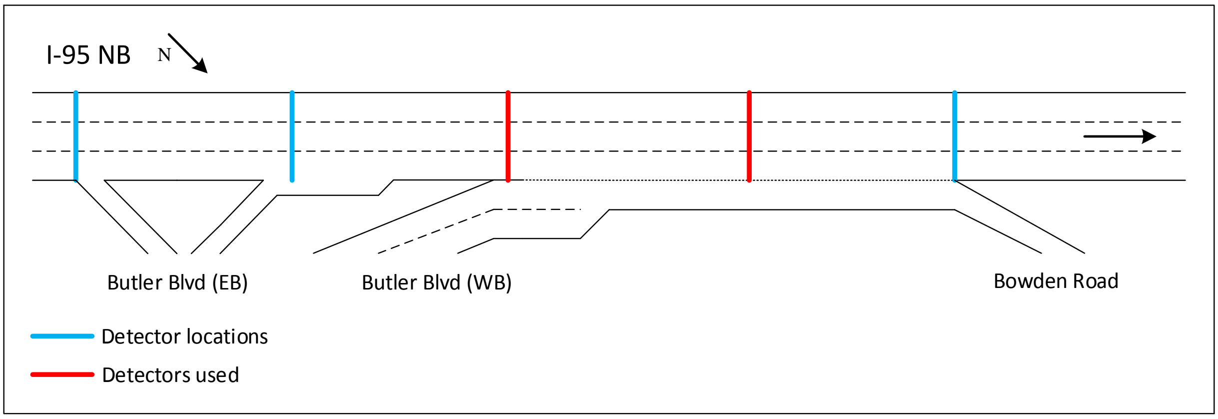

Figure 2.2. Schematic of I-95 NB at Butler Boulevard study section in Jacksonville, FL

2.1.2. I-95 NB at University Boulevard, Jacksonville, FL

This site is also located in Jacksonville, Florida (Figure 2.3). The bottleneck occurs due to an on-ramp merge from University Boulevard. The site has three lanes per direction. The posted speed limit is 65 mi/h and the AADT is 118,000 vehicles. Capacity values are measured according to Figure 2.1a, for merge sites. Both detectors displayed in red were used to obtain speed data and identify the breakdown events.

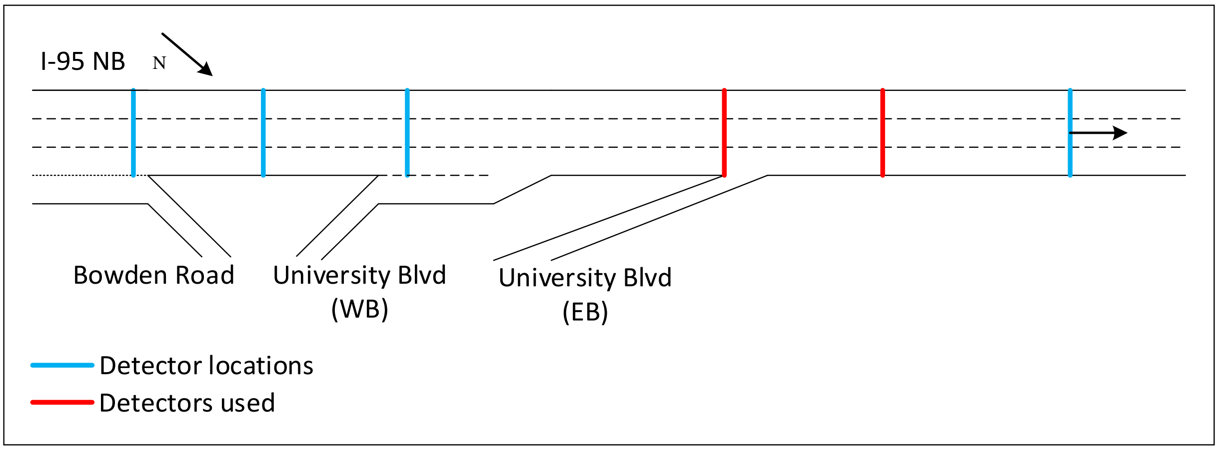

Figure 2.3. Schematic of I-95 NB at University Boulevard study section in Jacksonville, FL

2.1.3. SR-826 EB at NW 47th Avenue, Miami, FL

This site is located in Miami, Florida. The bottleneck is the result of a merge (Figure 2.4). The mainline has three lanes per direction. It has a speed limit of 55 mi/h and the AADT is 142,500 vehicles. Figure 2.4 displays a schematic of the study site. Both detectors shown in red were used to evaluate traffic operations and breakdown events at the merge bottleneck. The final capacity values correspond to the detector located downstream of the merge junction with NW 47th Avenue.

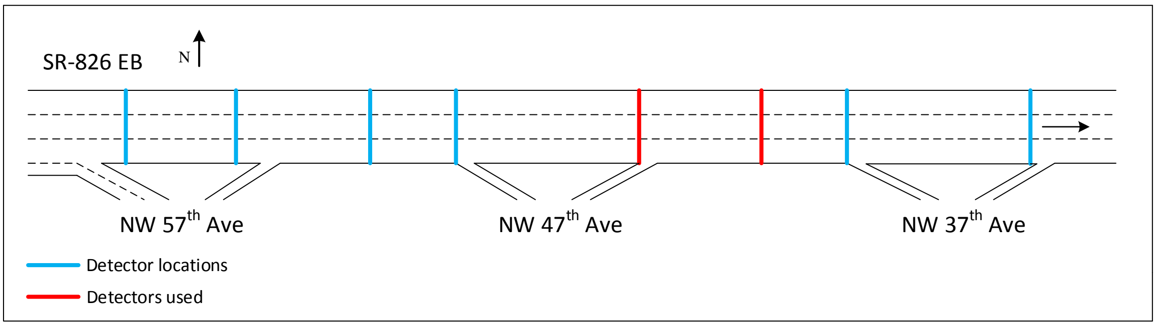

Figure 2.4. Schematic of SR-826 EB at NW 47th Avenue study section in Miami, FL

2.1.4. I-4 EB at SR-408, Orlando, FL

This site is located in Orlando, Florida along the eastbound direction. The bottleneck occurs due to an on-ramp merge from the intersection with SR-408, as well as a left side on-ramp merge with South Street (Figure 2.5). The site has three lanes with an auxiliary on the right side; data were not available for this auxiliary lane. The speed limit is dictated by Variable Speed Limit signs, and its base line speed limit is 50 mph. The AADT is 140,000 vehicles. Figure 2.5 illustrates the study area, along with the detector used.

No data were available for the detector located upstream of the Anderson Street ramp. The detector located downstream of the merge from SR-408 WB is used for the capacity analysis.

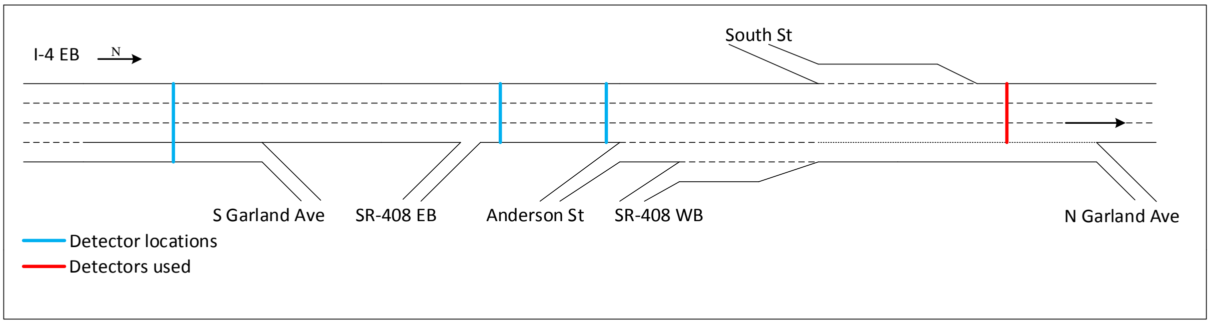

Figure 2.5. Schematic of the I-4 at SR-408 study section in Orlando

2.1.5. I-95 NB at NW 103rd Street, Miami, FL

This segment is located in Miami, Florida, and the bottleneck occurs due to the NW 103rd Street on-ramp (Figure 2.6). The segment has four lanes per direction as well as two HOT lanes; these were not analyzed in this project as they operate independently. The speed limit along the corridor is 55 mi/h and the AADT is 216,000 vehicles. Both detectors shown in red provided speed data to determine breakdown events, while the detector located downstream of the merge is used to gather capacity data.

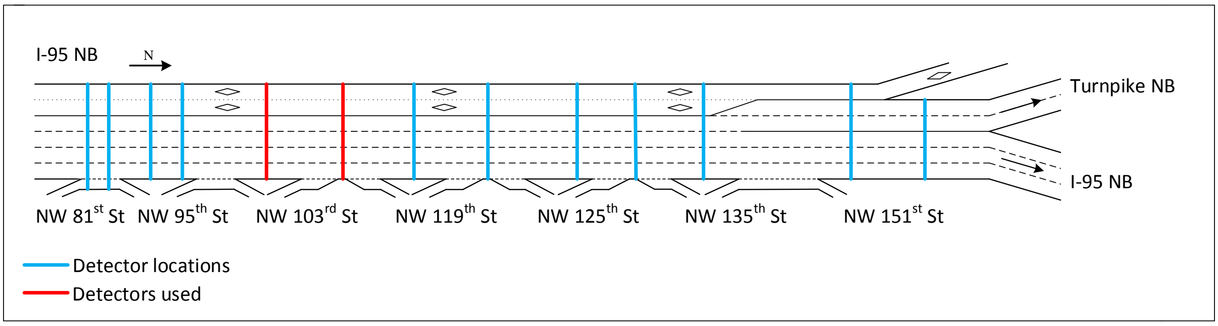

Figure 2.6. Schematic of I-95 NB at NW 103rd Street study section in Miami, FL

2.1.6. I-95 NB at Philips Highway, Jacksonville, FL

This segment is located in Jacksonville, Florida. The bottleneck forms due to the on-ramp from Philips Highway (Figure 2.7). The segment has four lanes along the mainline and the posted speed limit is 65 mph. The AADT is 108,500 vehicles. Speed data were collected at both detectors shown in red, while the detector downstream of the Philips Highway on-ramp was used to gather capacity values.

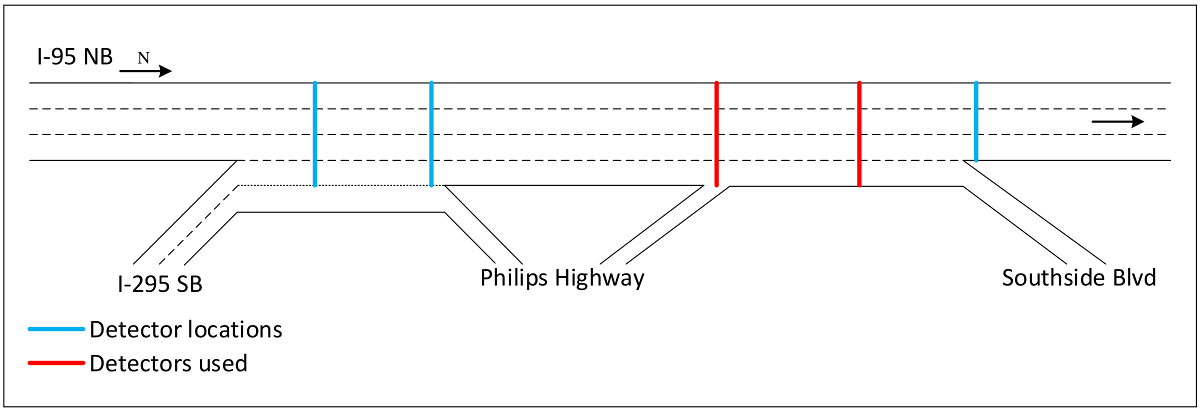

Figure 2.7. Schematic of the I-95 NB at Philips Highway study section in Jacksonville, FL

2.1.7. I-4 EB at I-75, Tampa, FL

This site is located in Tampa, Florida along the eastbound section of I-4. A bottleneck occurs at the on-ramp merge junction from I-75 NB onto I-4 EB (Figure 2.8). The speed limit is 70 mph. The bottleneck section has four lanes downstream of the merge and three lanes upstream (lane addition). The AADT is 143,000 vehicles. Both of the detectors shown in red were used to identify breakdown events, while capacity data were collected from the downstream detector.

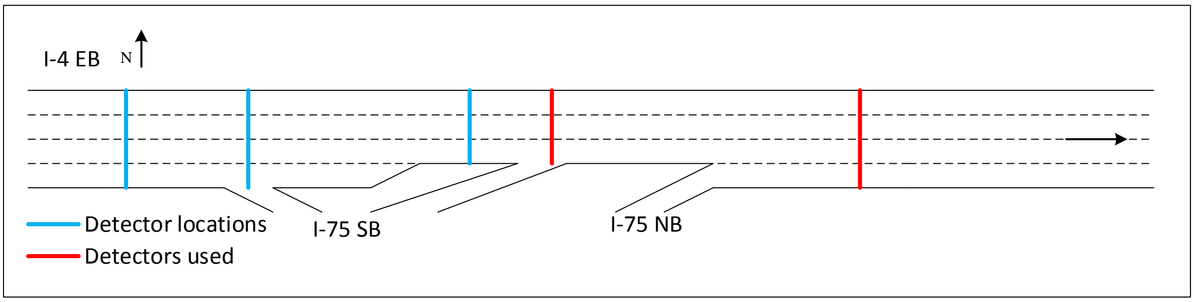

Figure 2.8. Schematic of I-4 EB at I-75 study section in Tampa, FL

2.1.8. I-95 NB at the Turnpike, Miami, FL

This site is located in Miami, Florida. The section is a major diverge bottleneck located along Florida's Turnpike (Figure 2.9). The site has three lanes along the mainline, with two lanes exiting towards the Turnpike. The speed limit at the site is 55 mi/h and the AADT is 225,000 vehicles. Detector data upstream of the diverge were not available, therefore, the detector used for analysis is located immediately downstream of the diverge.

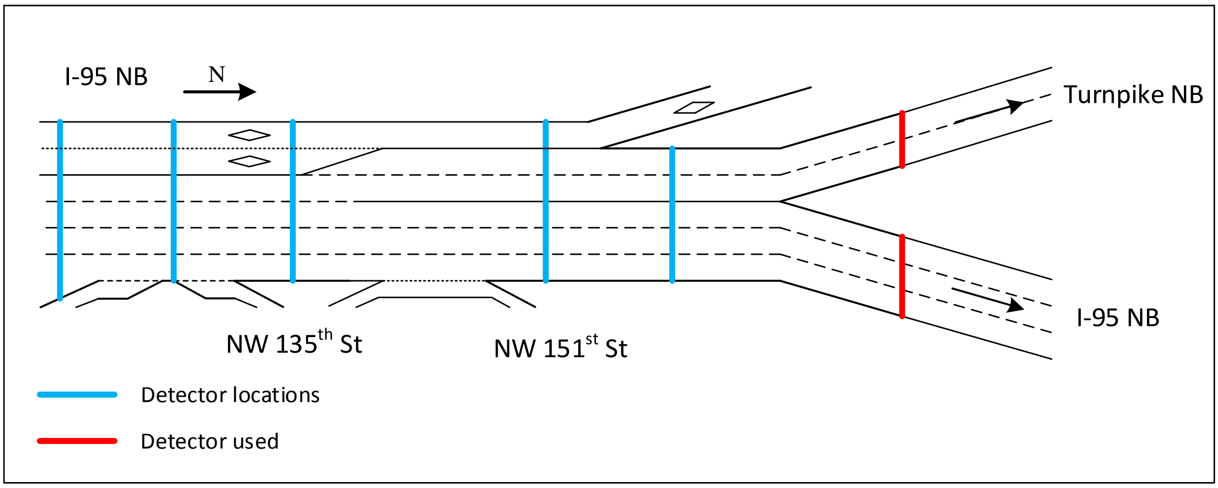

Figure 2.9. Schematic of I-95 NB at Florida's Turnpike study section in Miami, FL

2.1.9. I-95 NB between Baymeadows Rd. and Butler Blvd, Jacksonville, FL

This study site is located in Jacksonville, Florida between Baymeadows Road and Butler Boulevard. The bottleneck is caused by the diverge at Butler Boulevard. The freeway has three lanes per direction and a speed limit of 65 mph. The AADT is 89,500 vehicles. The detectors used for identifying the breakdown events are shown in Figure 2.10. Since this is a diverge bottleneck, the detector located upstream of the Butler Blvd off-ramp was used to calculate all capacity values.

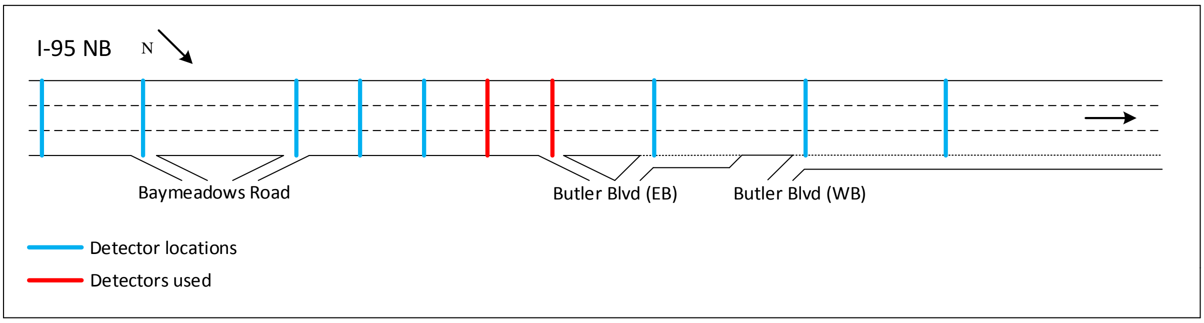

Figure 2.10. Schematic of I-95 NB Between Baymeadows Road and Butler Boulevard study section in Jacksonville, FL

2.1.10. I-4 WB at Lee Road, Orlando, FL

This site is located along a section of I-4 in Orlando, Florida, in the westbound direction. The bottleneck occurs due to a reduction in lanes from four to three downstream of an off-ramp onto Lee Road (Figure 2.11). The speed limit is dictated based on Variable Speed Limit signs, and the baseline speed limit is 50 mph. The AADT is 165,500 vehicles. Capacity values were obtained based on the detector located downstream from the lane drop, while speed information was collected from both detectors labeled in red.

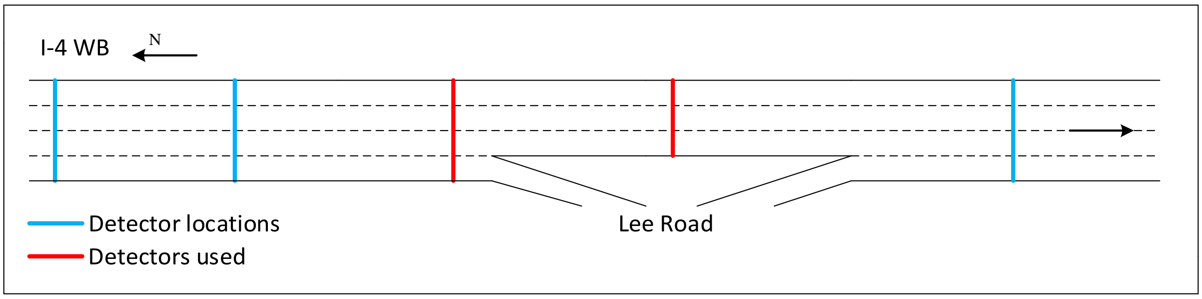

Figure 2.11. Schematic of the I-4 WB at Lee Road study section in Orlando, FL

2.2. Rural Freeways

A total of nine rural freeway sites were analyzed. In contrast to the urban freeway sites, these rural freeways do not experience breakdown regularly because they typically serve lower demands. However, it is possible to observe breakdown events at those sites during some of the highest demand days of the year (e.g., during the Thanksgiving and New Year's periods), in an effort to approximate capacity values.

The study sites were selected based on the results of a previous study (Washburn et al., 2010). In that study, data were collected between November 25, 2009, and November 30, 2009. With the help of FDOT's permanent count stations, additional data were collected at the same sites (both directions of travel) between November 2013 and January 2014, in order to record data during the highest demand period at these facilities. All data were available at 10- or 15-min increments.

With respect to the 2013-2014 data, incident information was not readily available, but after consultation with FDOT, the research team was able to remove days with incidents in the vicinity of the sites. Days that experienced poor weather conditions (precipitation over 0.20

inches or foggy conditions) were also omitted from the analysis. Weather conditions were evaluated using information from the website http://www.nws.noaa.gov/climate/. Since the permanent count stations were programmed by FDOT to collect data at the study sites for this specific project, the research team did not receive any indication by FDOT personnel regarding bad detector quality, and as such all data are considered to be of good quality. The research team does not have any information on incidents and data quality for the 2009 data. Truck percentages were also available for these sites through FDOT at 1-hour increments.

The location of the FDOT's permanent count stations is typically not close to on or off-ramps, and contrary to the urban sites, these count stations are usually very sparsely located. Thus, although one can identify the probable cause of congestion as the proximity to a junction, it is not always possible to infer whether the exact cause is due to merging, diverging or weaving operations. As such, the analysis related to the rural sites does not distinguish between bottleneck types and configurations.

The remainder of this section describes each of the rural freeway study sites and provides the respective schematic (not drawn to scale.) The schematics include nearby on- and off-ramps, as well as the location of the detectors (FDOT's permanent count stations) used for obtaining the speed and flow data.

2.2.1. I-75 at CR 514, West of Coleman, Sumter County, FL

This site is located along I-75 in Sumter County in the vicinity of CR 514 (also called Warm Springs Avenue). The site has two lanes per direction, and its speed limit is 70 mph. The AADT is 40,900. There is a junction at Florida's Turnpike approximately 3.5 miles north of the site, and a junction with N CR 470 approximately 3.1 miles to the south. It is assumed that at the NB direction congestion occurs due to the merge junction with Turnpike, whereas at the SB direction congestion occurs due to the merge with CR 470. A schematic of the study site is shown in Figure 2.12.

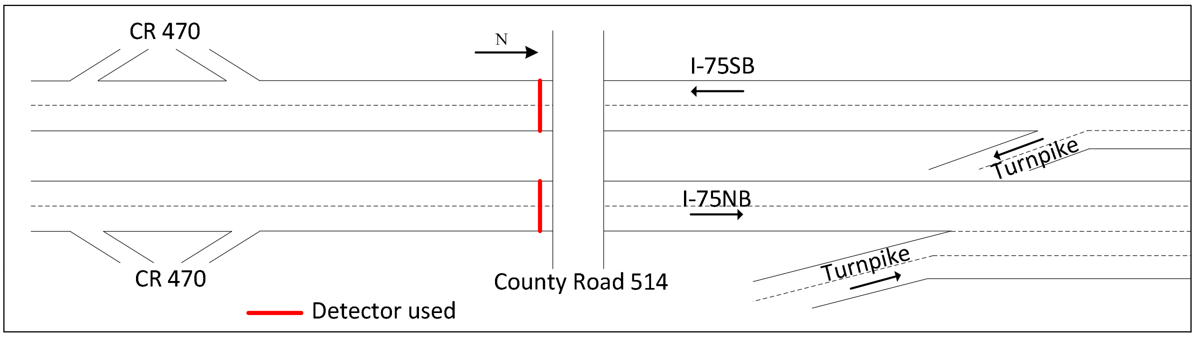

Figure 2.12. Schematic of I-75 at County Road 514 study section, Sumter County

2.2.2. Turnpike, South of County Road 468, East of Coleman, Sumter County, FL

This site is located along the Turnpike in Sumter County, south of County Road 486 (also called SR-91). It has two lanes per direction, and a speed limit of 70 mph. The AADT is 37,893 vehicles. The junction with US-301 is located north of the site, as shown in Figure 2.13.

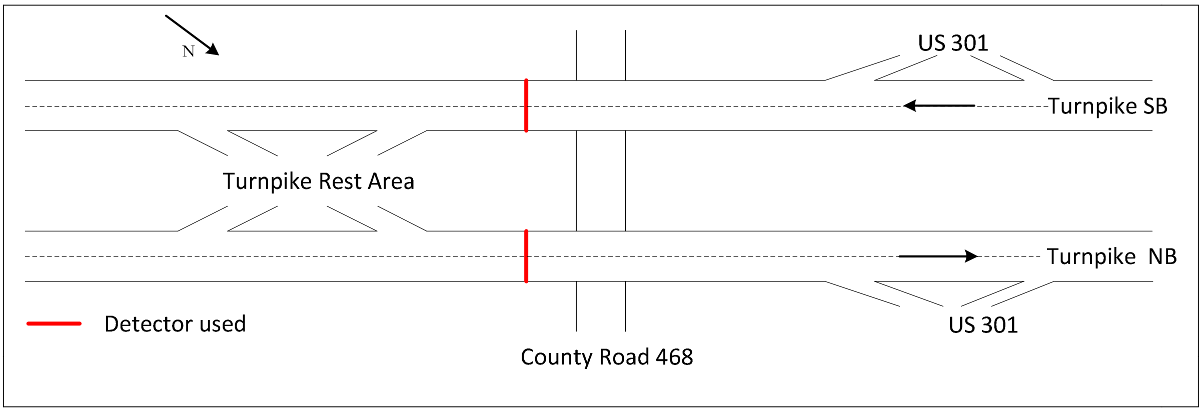

Figure 2.13. Schematic of Turnpike South of County Road 468 study section, Sumter County

2.2.3. I-75, North of SR-48, West of Bushnell, Sumter County, FL

This site is located along I-75 in Sumter County, west of Bushnell, FL. It is north of a major merge junction with SR-48, with on- and off-ramps in close proximity to the study site. The AADT is 38,720 vehicles. The site has two lanes per direction and a 70 mi/h speed limit (Figure 2.14).

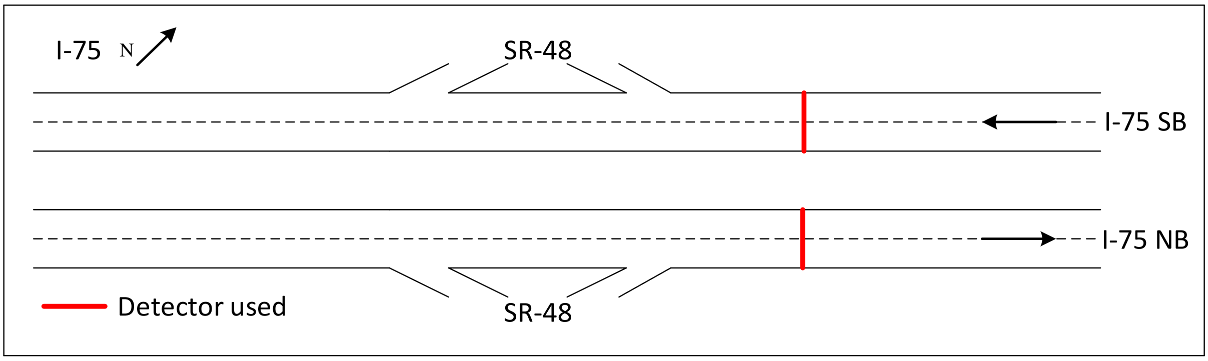

Figure 2.14. Schematic of I-75 north of SR-48 study section, Sumter County

2.2.4. I-95, North of SR-44, West of New Smyrna Beach, Volusia County, FL

The site is along I-95 in Volusia County, west of the city of New Smyrna Beach. The junction with Taylor Rd is located 1.7 miles north and the junction with SR-44 is located 9 miles south. It has two lanes per direction and a speed limit of 70 mph. The AADT is 36,601 vehicles. SR-44 is the nearest junction and is located approximately 2.7 miles north of the site (Figure 2.15).

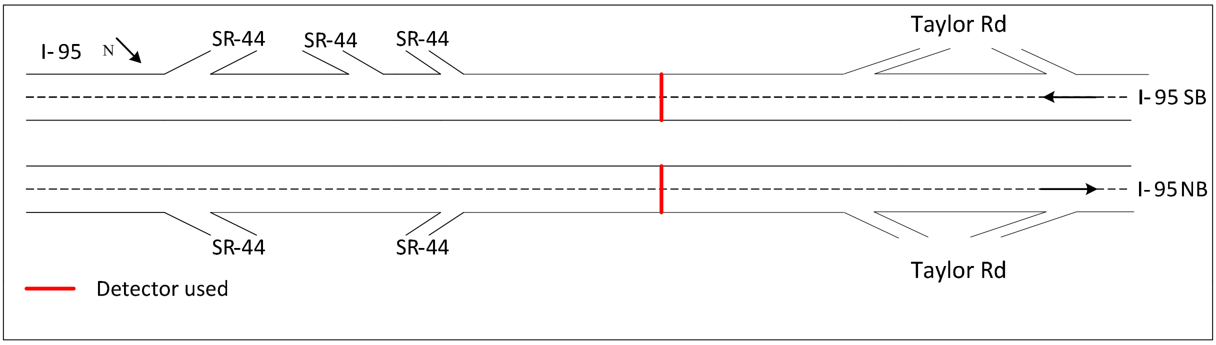

Figure 2.15. Schematic of I-95 north of SR-44 study section, Volusia County

2.2.5. I-75, North of William Road, South of Ocala, Marion County, FL

This site is located along I-75 in Marion County, south of Ocala, FL. It has three lanes per direction and its speed limit is 70 mi/h (Figure 2.16). The detector used is approximately 1.7 miles south of SW College Rd (SR-200) junction. The AADT is 77,544 vehicles. A junction at SW College Road, downstream of the site in the northbound direction, is the access point.

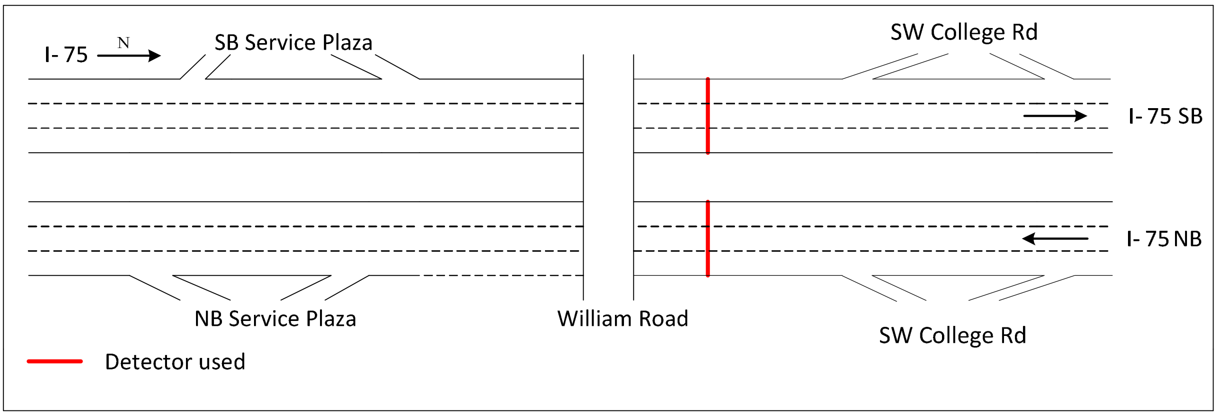

Figure 2.16. Schematic of I-75 north of William Road study section, Marion County

2.2.6. I-95, South of Florida-Georgia State Line, Northwest of Yulee, Nassau County, FL

The site is about 2 miles south of the Florida-Georgia State line along I-95 in Nassau County. The nearest town is Yulee, FL to the southeast of the site. The site has three lanes per direction with a speed limit of 70 mph. The AADT is 55,500 vehicles. The detector used is located just downstream of the junction with US-17, in the northbound direction (Figure 2.17). The junction with US-17 appears to be the bottleneck at this site.

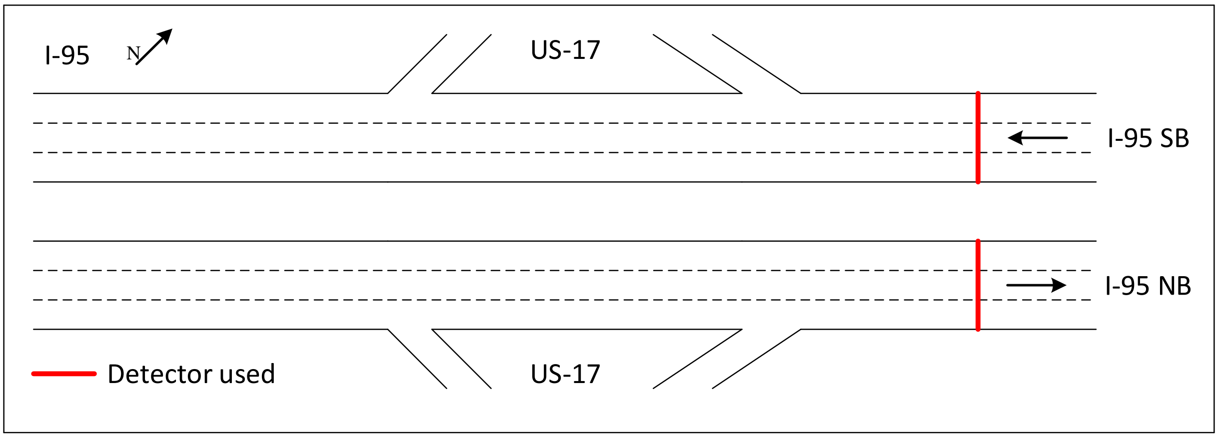

Figure 2.17. Schematic of I-95 south of Florida-Georgia State Line study section, Nassau County

2.2.7. I-75, Between I-10 and US-90, West of Lake City, Columbia County, FL

The site is located in Columbia County, along I-75, between the I-10 and US-90 interchanges. The nearest city is Lake City to the east. It has three lanes per direction, with a speed limit of 70 mi/h (Figure 2.18). The interchange with US-90 appears to be the bottleneck at this site and is located 3.0 miles to the south. The AADT is 44,727 vehicles.

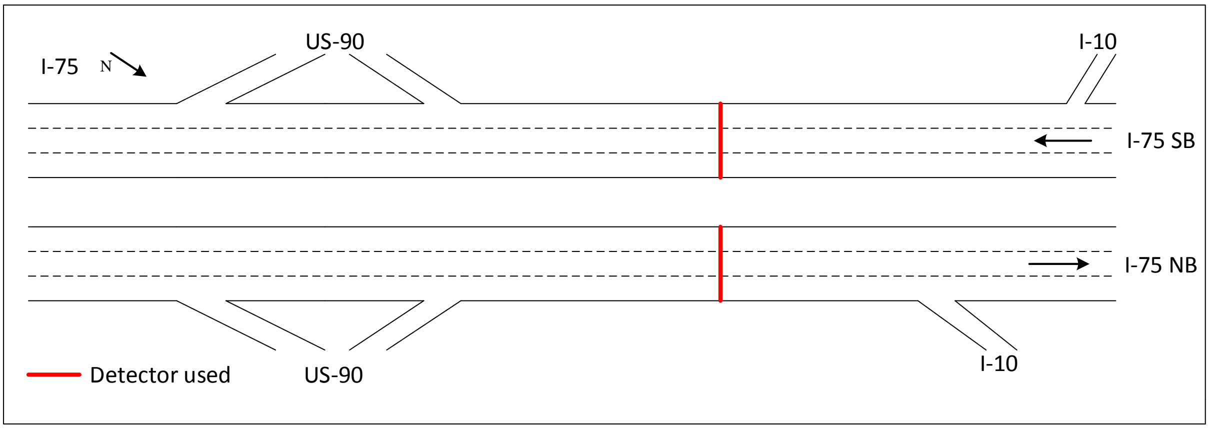

Figure 2.18. Schematic of I-75 between I-10 and US-90 study section, Columbia County

2.2.8. I-95, South of Aurantia Road, North of Titusville, Brevard County, FL

This site is located along I-95 in Brevard County. It has two lanes per direction with a speed limit of 70 mph. The AADT is 26,000 vehicles. The closest major city is Titusville, FL to the south. The detector is located approximately 0.9 miles south of Aurantia Road, which passes underneath I-95. The closest interchange is at Stuckway Rd, which is located 3.0 miles north of the detector (Figure 2.19).

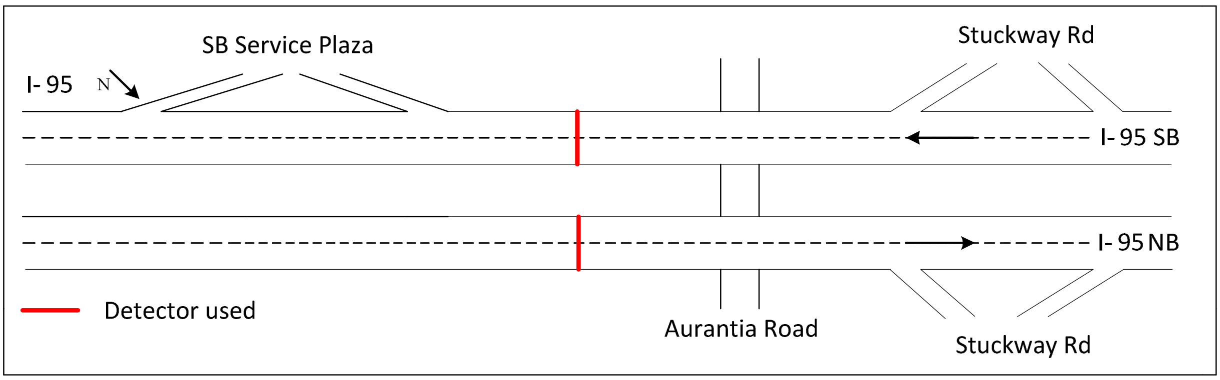

Figure 2.19. Schematic of I-95 south of Aurantia Road study section, Brevard County

2.2.9. I-4, East of Enterprise Road, Deltona, Volusia County, FL

This site is located along I-4 in Volusia County, east of the Enterprise Road overpass in Deltona, FL. It has three lanes per direction, with the Debary Avenue junction closest to the site (1 mile to the south). At the SB direction there is a lane drop approximately 0.7 miles from the detector location. The speed limit is 70 mi/h (Figure 2.20). The AADT is 96,379 vehicles.

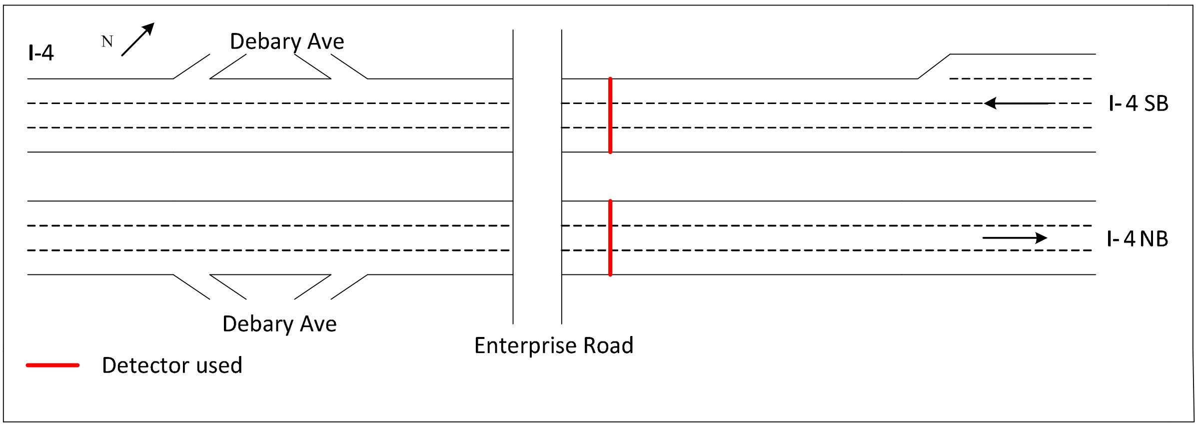

Figure 2.20. Schematic of I-4 east of Enterprise Road study section, Volusia County

2.3. Multilane Highways

Analysis was also performed at three multilane highway sites in Florida. The sites were chosen based on information provided by FDOT, given that these should be at least two miles away from signalized intersections to be categorized as multilane highways according to the HCM 2010 (TRB, 2010). The data were collected between November 21, 2013 and January 6, 2014 for all three sites. All data were available at 15-min increments.

Incident information was not readily available along those sites, but after consultation with FDOT we were able to remove data with incidents occurring in the vicinity of the study sites. Days that experienced poor weather conditions (precipitation over 0.20 inches or foggy conditions according to http://www.nws.noaa.gov/climate/) were also omitted from the analysis. Since the permanent count stations were programmed by FDOT to collect data at the study sites for this specific project, the research team did not receive any indication by FDOT personnel regarding bad detector quality, and as such all data are considered to be of good quality. Truck percentage data were not available at these sites.

Every site description is accompanied by a schematic of the study site (not drawn to scale.) The schematics include nearby on- and off-ramps, as well as the detector location.

2.3.1. US-98, Pensacola Bay Bridge, South of Pensacola, Santa Rosa County, FL

This site is located along US-98 in Santa Rosa County, and it is at the start of the southern end of the Pensacola Bay Bridge, also known as the Three Mile Bridge. The section has two lanes in each direction and has several driveways in its vicinity, leading to marinas as well as residences. The detector is located approximately 0.7 miles north of the nearest signalized intersection, Northcliff Drive/Fairpoint Drive (Figure 2.21). The AADT is 51,831 vehicles. The nearest signalized intersection north of the site is approximately 3.3 miles away, at N 17th Avenue. These intersections are not presented in the figure as they are very unlikely to affect operations at the site. The speed limit along the bridge is 45 mph.

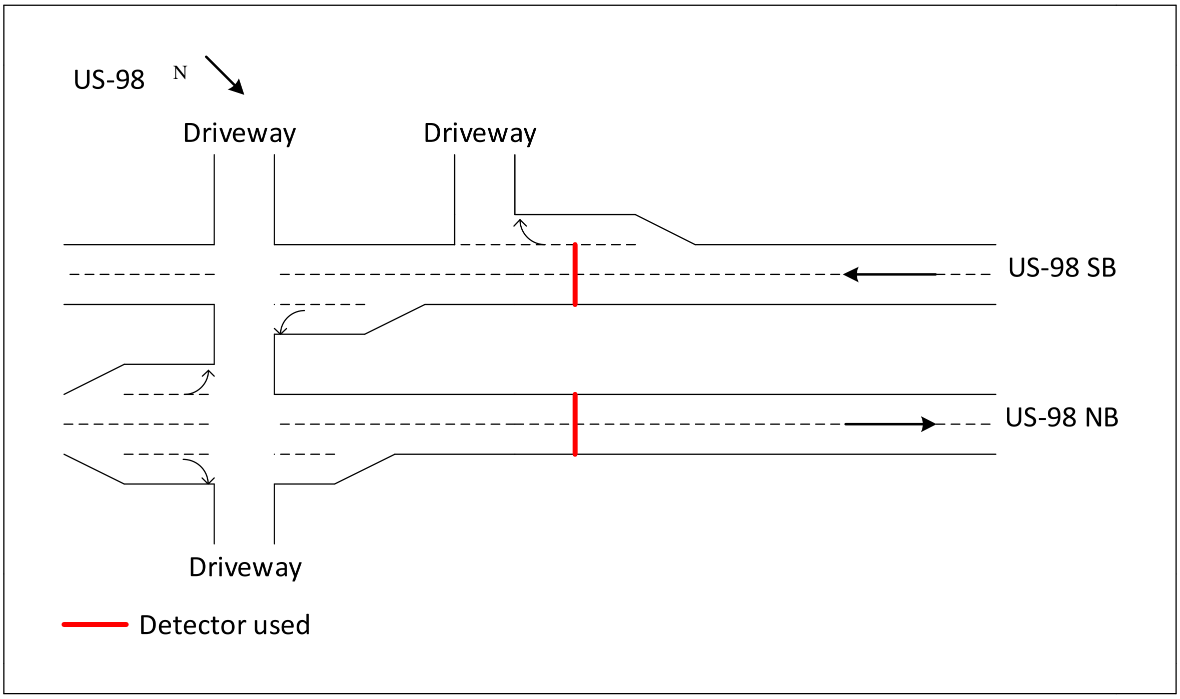

Figure 2.21. Schematic of US-98 at Pensacola Bay Bridge study section, Santa Rosa County

2.3.2. Roosevelt Boulevard, near St. Petersburg Airport, North of St. Petersburg, Pinellas County, FL

This site is located along Roosevelt Boulevard in Pinellas County. It is east of the signalized intersection with 58th Street North. The detector is placed near an unsignalized intersection, along a section with three through lanes and a speed limit of 45 mph. The AADT is 33,346 vehicles. The eastbound direction also has a left-turn lane at the detector location. The site is very close to the St. Petersburg Airport, and it has many unsignalized intersections along its length (Figure 2.22).

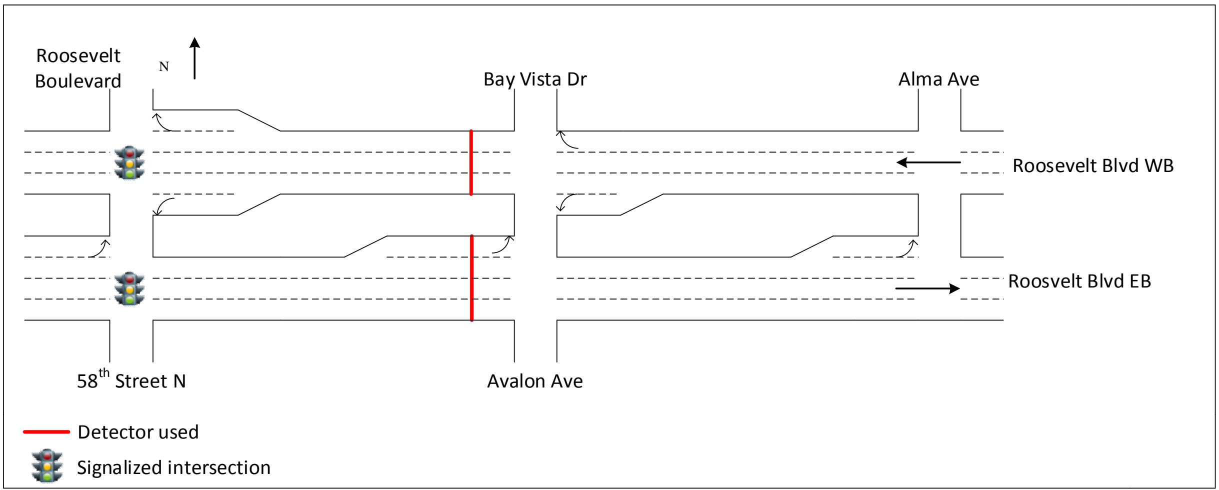

Figure 2.22. Schematic of Roosevelt Boulevard near St. Petersburg Airport study section, Pinellas County

2.3.3. SR-212, East of Hopson Road, Jacksonville, Duval County, FL

The site is located along SR-212, also called Beach Boulevard, in Duval County. It is east of Hopson Road, which provides access to a marina (Figure 2.23). The site is also just east of the Intracoastal Waterway. It has three lanes per direction and the speed limit is 45 mph. The AADT is 39,301 vehicles. The nearest signalized intersection west of the site is San Pablo Road, approximately 1.4 miles from the detector location, which is shown in the schematic of the site.

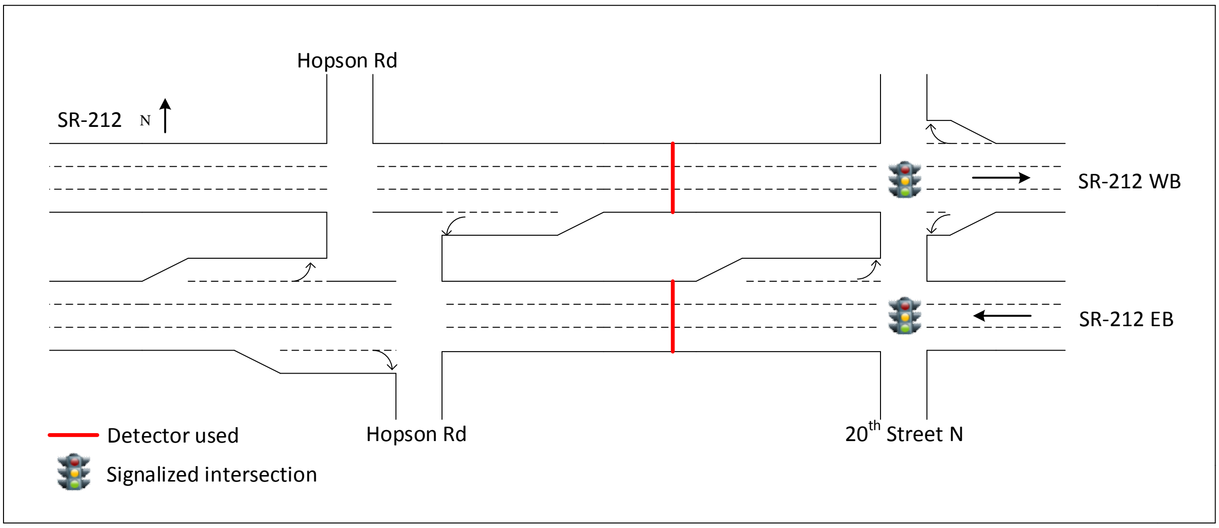

Figure 2.23. Schematic of SR-212 east of Hopson Road study section, Duval County

3. DATA ANALYSIS

This section first provides a description of the six different definitions of capacity for oversaturated and undersaturated conditions. Next, it presents the resulting numerical values for each capacity definition for all study segments.

3.1. Capacity Definitions

Based on the literature review findings, six definitions were considered and their respective values were obtained from the data:

A. *Breakdown flow*: the 1-minute flow per lane immediately before the breakdown event (i.e., before the abrupt speed drop).

B. *Maximum 1-min pre-breakdown flow within 15 minutes*: the 1-min highest flow that occurs during the 15 minutes before the breakdown, i.e., during undersaturated conditions.

C. *Maximum 5-min pre-breakdown flow within 15 minutes*: the 5-min highest flow (rolling average) that occurs during the 15 minutes before the breakdown, i.e., during undersaturated conditions.

D. *Average 5-min pre-breakdown flow*: the average 5-minute flow per lane immediately before the breakdown during undersaturated conditions.

E. *Average 15-min pre-breakdown flow*: the average of the 15-minute flow per lane immediately before the breakdown during undersaturated conditions.

F. *Average discharge flow*: the average flow per lane during oversaturated conditions (i.e., the time interval after breakdown and prior to recovery).

Figure 3.1 identifies the data points that correspond to each of the above capacity definitions in a time series plot.

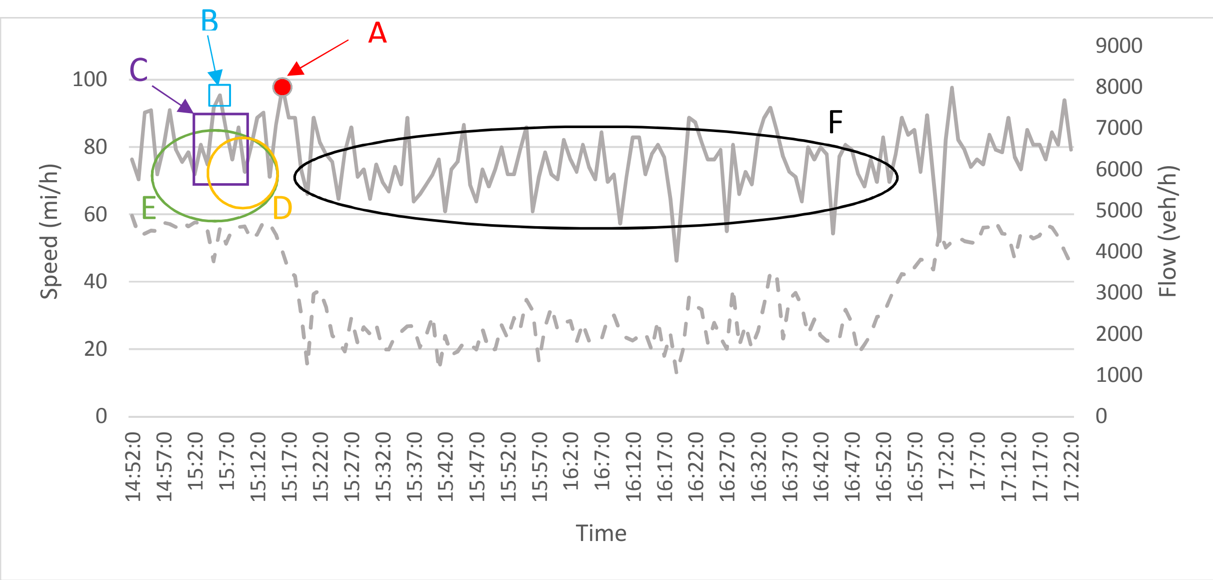

Figure 3.1. Capacity measures under consideration.

For urban freeways, we report the following pre-breakdown capacity measures: breakdown flow; maximum 1-min or 5-min pre-breakdown flows; and the average 5- and 15-minute pre-breakdown flows. For the rural freeways and multilane highways, 5- minute data were not always available; therefore, these were analyzed in 10-minute or 15-minute intervals, depending on data availability.

All capacity measures presented are related to the occurrence of breakdown events at the study sites. When breakdown does not occur, we cannot be certain that the demand has been high enough so that "capacity" can be reached. Breakdown events are identified through sharp speed drops (e.g., at least 10 mi/h between two time intervals), recorded either at the upstream or downstream detector relative to the bottleneck, as discussed in the previous chapter.

3.2. Capacity Estimates

This section provides a summary of the data analysis performed to extract the capacity measures specified earlier: breakdown flow, maximum (1-min or 5-min) pre-breakdown flow, average (5-min or 15-min) pre-breakdown flow and average discharge. Table 3.1 presents the average, minimum, maximum, standard deviation, and 50th and 85th percentiles of the breakdown, pre-breakdown, and discharge capacity measures for the ten urban freeway sites. The results presented in this table are divided into groups based on the type of bottleneck and the number of lanes.

Table 3.1. Capacity Measures for Urban Freeway Sites

Site	Number of Observations (breakdowns)	Statistic	Capacity Values (veh/h/ln)					
			Breakdown	Pre-Breakdown				Discharge
				1-Min Max	5-Min Max	5-Min Avg	15-Min Avg	
Weave, 3 Lanes on Mainline with an Auxiliary Lane								
I-95 NB, At Butler (Jacksonville)	48	Average	2,056	2,380	2,143	2,079	1,981	1,718
		Min	1,480	1,920	1,757	1,684	1,621	1,515
		Max	2,500	2,640	2,348	2,332	2,197	1,945
		St. Dev.	249	166	156	166	138	99
		50th Percentile	2,100	2,390	2,166	2,110	1,997	1,726
		85th Percentile	2,277	2,540	2,287	2,260	2,151	1,800
Merge, 3 Lanes on Mainline								
I-95 NB, At University (Jacksonville)	53	Average	2,138	2,361	2,168	2,092	2,044	1,986
		Min	1,720	1,860	1,688	1,576	1,563	1,716
		Max	2,460	2,680	2,424	2,392	2,377	2,234
		St. Dev.	187	163	173	171	181	121
		50th Percentile	2,160	2,340	2,208	2,124	2,067	1,994
		85th Percentile	2,330	2,530	2,346	2,246	2,223	2,122
SR-826 EB, At NW 47th Ave. (Miami)	99	Average	1,737	1,970	1,788	1,734	1,684	1,617
		Min	1,500	1,640	1,432	1,368	1,344	1,286
		Max	2,080	2,320	2,052	2,052	1,937	1,786
		St. Dev.	128	137	127	136	125	93
		50th Percentile	1,740	1,980	1,920	1,750	1,705	1,646
		85th Percentile	1,920	2,120	2,080	1,884	1,828	1,717
Composite (Right followed by left side) Merge, 3 Lanes on Mainline								
I-4 EB, At SR-408 (Orlando)	145	Average	2,094	2,356	2,120	2,063	1,929	1,849
		Min	1,540	1,780	1,572	1,572	1,475	1,541
		Max	2,520	2,700	2,424	2,332	2,269	2,020
		St. Dev.	190	156	144	149	141	66
		50th Percentile	2,100	2,360	2,144	2,088	1,940	1,858
		85th Percentile	2,316	2,500	2,247	2,194	2,072	1,903
Merge, 4 Lanes on Mainline								
I-95 NB, At NW 103rd St (Miami)	73	Average	1,828	2,048	1,854	1,780	1,754	1,646
		Min	1,485	1,815	1,614	1,482	1,520	1,407
		Max	2,250	2,325	2,190	2,190	2,045	1,962
		St. Dev.	190	124	124	135	112	114
		50th Percentile	1,853	2,055	1,857	1,787	1,764	1,640
		85th Percentile	1,995	2,145	1,979	1,904	1,849	1,731

Table 3.1, continued

Site	Number of Observations (breakdowns)	Statistic	Capacity Values (veh/h/ln)					
			Breakdown	Pre-Breakdown				Discharge
				1-Min Max	5-Min Max	5-Min Avg	15-Min Avg	
I-95, At Philips Hwy (Jacksonville)	54	Average	1,902	2,170	1,962	1,856	1,864	1,590
		Min	1,515	1,800	1,617	1,491	1,554	1,157
		Max	2,415	2,565	2,325	2,253	2,174	1,931
		St. Dev.	263	228	206	222	200	194
		50th Percentile	1,815	2,175	2,019	1,875	1,915	1,624
		85th Percentile	2,220	2,415	2,173	2,118	2,111	1,788
Merge, 3- to 4-Lanes (lane addition)								
I-4 EB, At I-75 (Tampa)	54	Average	1,591	1,781	1,582	1,527	1,494	1,431
		Min	1,410	1,470	1,248	1,248	1,212	1,148
		Max	1,905	1,995	1,830	1,806	1,640	1,606
		St. Dev.	122	117	88	99	77	98
		50th Percentile	1,575	1,770	1,583	1,539	1,496	1,459
		85th Percentile	1,735	1,905	1,657	1,598	1,570	1,522
Major Diverge, 5 Lanes, 3 Lanes on Mainline								
I-95 NB, At Turnpike (Miami)	170	Average	1,735	1,842	1,701	1,650	1,615	1,593
		Min	1,356	1,500	1,327	1,260	1,278	1,215
		Max	2,088	2,100	1,879	1,865	1,815	1,719
		St. Dev.	122	92	93	101	101	77
		50th Percentile	1,728	1,848	1,714	1,663	1,638	1,611
		85th Percentile	1,860	1,920	1,782	1,738	1,690	1,655
Diverge, 3 Lanes on Mainline								
I-95 NB, at Baymeadows and Butler (Jacksonville)	84	Average	2,095	2,429	2,142	2,103	2,066	1,838
		Min	1,600	2,000	1,840	1,840	1,725	1,505
		Max	2,480	2,800	2,420	2,384	2,244	2,096
		St. Dev.	212	158	127	125	111	90
		50th Percentile	2,110	2,420	2,130	2,098	2,080	1,848
		85th Percentile	2,320	2,580	2,280	2,232	2,174	1,903
Diverge, 4 to 3 Lanes (lane drop)								
I-4 WB, At Lee Road (Tampa)	87	Average	1,847	2,178	1,914	1,831	1,796	1,700
		Min	1,440	1,720	1,536	1,440	1,481	1,376
		Max	2,300	2,500	2,256	2,208	2,087	2,029
		St. Dev.	191	173	160	168	146	175
		50th Percentile	1,840	2,180	1,920	1,864	1,800	1,698
		85th Percentile	2,060	2,360	2,056	1,996	1,943	1,924

Table 3.2 presents the capacity measures for the rural freeway sites. The raw data were provided at a 10- or 15-minute aggregation level; therefore the capacity measures reported correspond roughly to the 15-minute flow per lane immediately before the breakdown event and the discharge flow (i.e., the average flow per lane during oversaturated conditions).

A total of nine sites were analyzed. Based on the available data, breakdowns were experienced only at four sites. The site at I-75 north of SR-48 (northbound direction) experienced one breakdown during the days analyzed and as such, no statistical data can be given.

Table 3.2. Capacity Measures for Rural Freeway Sites

Site	Direction	Number of Observations (breakdowns)	Statistic	Capacity Values (veh/h/ln)	
				Breakdown	Discharge
2-Lanes					
I-75, At County Rd 514 [1] (merge)	NB	6	Average	1,427	1,029
			Min	1,120	916
			Max	1,686	1,131
			St. Dev.	231	91
			50th Percentile	1,460	1,036
			85th Percentile	1,643	1,122
	SB	3	Average	1,417	1,342
			Min	1,344	1,322
			Max	1,462	1,356
			St. Dev.	64	18
			50th Percentile	1,444	1,348
			85th Percentile	1,457	1,353
Turnpike, South of County Rd 468 [1] (merge/diverge)	NB	6	Average	1,624	1,301
			Min	1,434	1,184
			Max	1,864	1,494
			St. Dev.	150	108
			50th Percentile	1,611	1,276
			85th Percentile	1,731	1,383
	SB	0	No Breakdowns Observed		
I-75, North of SR-48 [1] (merge/diverge)	NB	1	Average	1,464	1,188
			Min	-	-
			Max	-	-
			St. Dev.	-	-
			50th Percentile	-	-
			85th Percentile	-	-
	SB	0	No Breakdowns Observed		

Table 3.2, continued

Site	Direction	Number of Observations (breakdowns)	Statistic	Capacity Values (veh/h/ln)	
				Breakdown	Discharge
I-95, North of SR-44 [2] (merge/diverge)	NB	0	No Breakdowns Observed		
	SB	0	No Breakdowns Observed		
3-Lanes					
I-75, North of William Rd [1] (merge/diverge)	NB	11	Average	1,596	1,487
			Min	1,225	1,021
			Max	1,816	1,721
			St. Dev.	156	214
			50th Percentile	1,593	1,540
			85th Percentile	1,729	1,669
	SB	16	Average	1,600	1,404
			Min	1,348	1,077
			Max	1,797	1,557
			St. Dev.	129	126
			50th Percentile	1,599	1,440
			85th Percentile	1,737	1,514
I-95, South of FL-GA Line [1] (merge/diverge)	NB	0	No Breakdowns Observed		
	SB	0	No Breakdowns Observed		
I-75, Between I-10 and US-90 [1] (merge/diverge)	NB	0	No Breakdowns Observed		
	SB	0	No Breakdowns Observed		
I-95, South of Aurantia Rd [1] (merge/diverge)	NB	0	No Breakdowns Observed		
	SB	0	No Breakdowns Observed		
I-4, East of Enterprise Rd [1] (merge/diverge)	EB	0	No Breakdowns Observed		
	WB	0	No Breakdowns Observed		

[1] data available every 15 minutes
[2] data available every 10 minutes

At multilane highway sites breakdown events were not observed. Therefore, the results summarized in Table 3.3 correspond to the average of all observed maximum 15-minute flows (vehicles per hour per lane) over all observation days. The range of these maximum values is also provided.

Table 3.3. Analysis Results for Multilane Highways

Site	Direction	Number of Days Observed	Maximum 15-Minute Flow (veh/h/ln)			
			Average	Min	Max	St. Dev.
2 Lanes						
US-98, Pensacola Bay Bridge	EB	47	1,131	534	1,536	287
	WB	47	1,147	544	1,640	327
3 Lanes						
Roosevelt Boulevard, Near St. Petersburg Airport	EB	47	353	117	513	113
	WB	47	560	257	836	195
627SR-212, East of Hopson Road	EB	47	542	236	783	121
	WB	47	523	317	664	85

3.3. Comparison with HCM 2010 and FDOT Default Values

For comparison purposes, the field measurements shown in the previous section and the FDOT (2013) default values were converted to the equivalent passenger car values (i.e., pc/h/ln). The capacity measures presented in the previous section were converted to pc/h/ln by considering the average percentage of trucks at the study sites during the data collection period. Different truck percentages were considered for pre-breakdown capacity measures vs. post-breakdown capacity measures (discharge), to account for the fact that truck percentages are typically lower during oversaturated conditions. To compare with the HCM 2010 capacity estimates, the free-flow speed of the merge and diverge segments (for both urban and rural sites) was calculated from the available data, as the undersaturated speed under low traffic conditions (less than 1,000 veh/h/ln). Thus, the capacities of these segments correspond to Exhibit 13-8 of the HCM 2010, and are equivalent to the basic freeway segments capacities. For the urban weaving segment, the capacity based on the HCM 2010 methodology was calculated using the following equation:

$$c_{IWL} = c_{IFL} - \left[438.2(1+VR)^{1.6}\right] + \left[0.0765L_S\right] + \left[119.8N_{WL}\right] \quad (1)$$

Where:

c_{IWL} = capacity of the weaving segment under equivalent ideal conditions, per lane (pc/h/ln),

c_{IFL} = capacity of a basic freeway segment with the same FFS as the weaving segment under equivalent ideal conditions, per lane (pc/h/ln),

VR = volume ratio,

L_S = length of the weaving segment (ft),

N_{WL} = number of lanes from which a weaving maneuver may be completed with one lane change or no lane changes.

Based on Equation 1, for an FFS 70 mi/h (c_{IFL}= 2,400 pc/h/ln), L_s = 4,500 ft, N_{WL} = 2, and assuming that the volume ratio is VR = 0.30, the capacity of the weaving segment with that particular configuration is calculated as 2,317 pc/h/ln and rounded down to c_{IWL} = 2,300 pc/h/ln. According to the HCM 2010 (page 12-15) segments with N_{WL} = 2 lanes rarely have volume ratios VR greater than the range of 0.40 to 0.50, therefore, an assumption of 0.30 was used to represent average/ expected conditions. It should also be noted that the calculated capacity is very sensitive to the volume ratio, since a 0.1 change in VR suggests approximately a 100 pc/h/ln change in capacity.

The peak hour directional volumes provided by FDOT were also converted to pc/h/ln considering the average truck percentage during pre-breakdown and congested conditions. Truck percentages were not available for the rural freeway segments; thus, default truck percentages were considered in this case (9% trucks for rural freeways) based on FDOT (2013). The average truck percentages both before and after the breakdown events are presented in Table 3.4.

Table 3.4. Truck percentages at urban freeway sites

Location	Pre-breakdown Truck Percentage (%)	Post-breakdown Truck Percentage (%)
I-95 NB, At Butler	6.42%	4.59%
I-95 NB, At University	6.23%	4.59%
SR-826 EB, At NW 47th Ave	7.74%	6.41%
I-4 EB, At SR-408	12.19%	12.27%
I-95 NB, At NW 103rd St	7.65%	6.93%
I-95, At Philips Hwy	6.46%	4.55%
I-4 EB, At I-75	8.65%	7.14%
I-95 NB, At Turnpike	6.35%	5.26%
I-95 NB, at Baymeadows	6.03%	4.42%
I-4 WB, At Lee Road	5.81%	4.57%

Table 3.5 and Table 3.6 summarize the capacity estimates in pc/h/ln for urban and rural freeways. The tables present the HCM 2010 and the FDOT capacity estimates, as well as the 50th percentile and the 85th percentile of the 15-min average and the discharge flow.

Table 3.5. Selected capacity estimates in pc/h/ln for urban freeways

Location	FFS	No. of Lanes	Capacity estimates (pc/h/ln)					
			HCM 2010	FDOT	15-min avg pre-breakdown flow (50th per.)	15-min avg pre-breakdown flow (85th per.)	Discharge (50th per.)	Discharge (85th per.)
I-95 NB, At Butler	70	3	2,300	2,082	2,061	2,220	1,766	1,841
I-95 NB, At University	65	3	2,350	2,081	2,131	2,293	2,040	2,171
SR-826 EB, At NW 47th Ave.	65	3	2,350	2,098	1,765	1,875	1,681	1,756
I-4 EB, At SR-408	60	3	2,300	2,151	2,058	2,198	1,972	2,020
I-95 NB, At NW 103rd St	60	4	2,300	2,130	1,831	1,919	1,697	1,791
I-95, At Philips Hwy	65	4	2,350	2,112	1,977	2,179	1,661	1,829
I-4 EB, At I-75	65	4	2,350	2,136	1,560	1,637	1,511	1,576
I-95 NB, At Turnpike	60	5	2,300	2,085	1,690	1,744	1,653	1,699
I-95 NB, at Baymeadows	70	3	2,400	2,080	2,143	2,240	1,888	1,945
I-4 WB, At Lee Road	55	3	2,250	2,079	1,852	1,999	1,737	1,968

Table 3.6. Selected capacity estimates in pc/h/ln for rural freeways

Location	FFS	No. of Lanes	Capacity estimates (pc/h/ln)					
			HCM 2010	FDOT	Pre-breakdown flow (50th per.)	Pre-breakdown flow (85th per.)	Discharge (50th per.)	Discharge (85th per.)
I-75, At County Rd 514 NB	75	2	2,400	1,871	1,460	1,643	1,036	1,122
I-75, At County Rd 514 SB	75	2	2,400	1,871	1,444	1,457	1,348	1,353
Turnpike, South of County Rd 468 NB	75	2	2,400	1,871	1,611	1,731	1,276	1,383
I-75, North of SR-48 NB *	75	2	2,400	1,871	1,464	1,464	1,188	1,188
I-75, North of William Rd NB	75	3	2,400	1,930	1,593	1,729	1,540	1,669
I-75, North of William Rd SB	75	3	2,400	1,930	1,599	1,737	1,440	1,514

* Only one breakdown event was observed for this site; therefore the reported statistics correspond to the breakdown and discharge values for this event and not the 50^{th} or 85^{th} percentile.

The following figures present graphically a comparison of capacity measures between HCM 2010, FDOT and the field measured capacities for urban and rural freeways. For the purposes of this comparison we selected to show the 85^{th} percentile of the 15-minute average pre-breakdown flow and the 85^{th} percentile discharge flow. Given that breakdown and capacity conditions were not observed at the multilane highway sites, we do not provide a similar comparison for these sites.

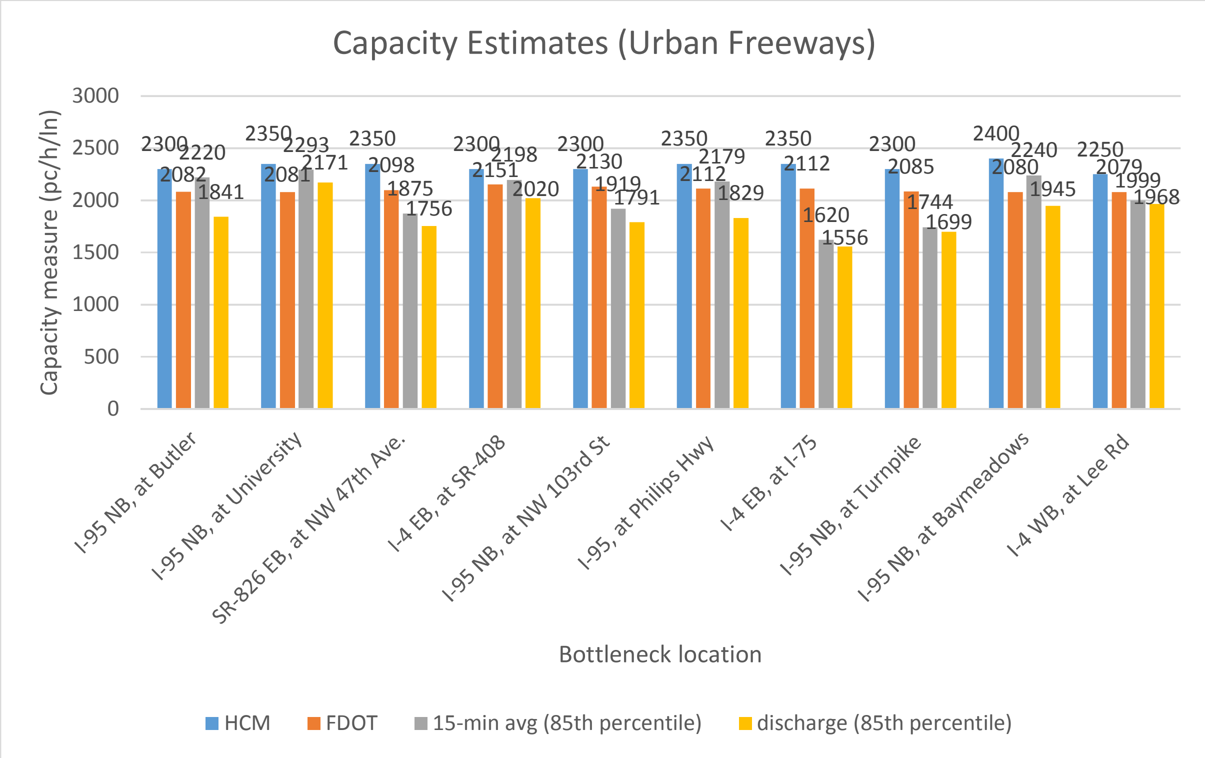

Figure 3.2. Capacity comparison results by bottleneck location for urban freeway segments

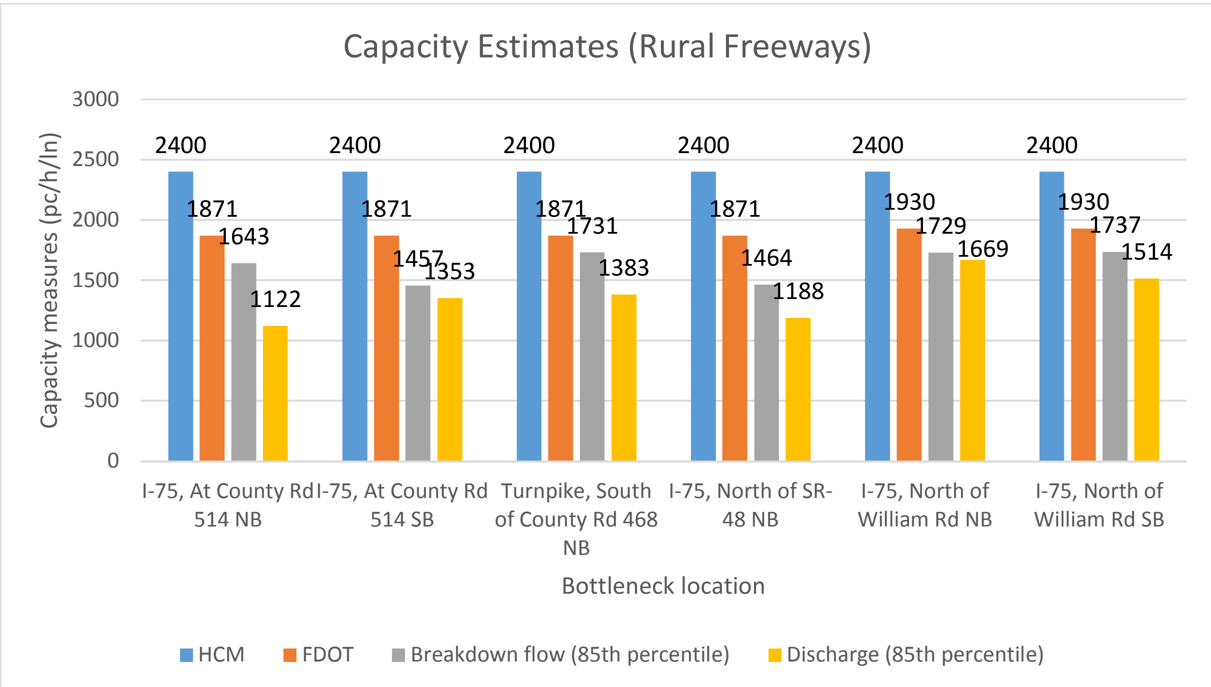

Figure 3.3. Capacity comparison results by bottleneck location for rural freeway segments

As shown in Figure 3.2 and Figure 3.3, the HCM 2010 values are higher than the capacity parameters estimated in this research. The 85th percentile of the 15-min average is closer to the FDOT capacity estimates for the majority of the cases. These differences are larger for the rural freeways analysis, suggesting that the capacity of rural freeways is generally lower than that of urban freeways. For all urban and rural freeways, the discharge flow is considerably less than the pre-breakdown flow, HCM 2010, and FDOT values. Note that for one of the urban freeway sites (I-4 EB at I-75) both capacity estimates seem unrealistically low.

The capacity estimates were further analyzed to evaluate the effect of the segment type and the number of lanes. Figure 3.4 shows the 85th percentile 15-min average pre-breakdown flow, the 85th percentile discharge flow, as well as the HCM 2010 and FDOT capacities by segment type. As shown, the weaving segment studied in this research has slightly higher pre-breakdown capacity than merging or diverging segments. However, this result is based on only one site, whereas a larger sample (with varying weaving configurations) is required to make generalizable conclusions regarding the weaving segments capacities.

Also, the number of lanes seems to affect capacity. The 3-lane merging segments appear to have higher capacity than 4-lane merging segments. Similarly, 3-lane diverging segments have higher capacity than 5-lane diverging segments. This finding is consistent with literature review findings (Lu and Elefteriadou, 2013) which indicate that 3-lane highways have higher per lane capacity than freeways with higher or lower number of lanes. Merging and diverging segments with the same number of lanes had similar values.

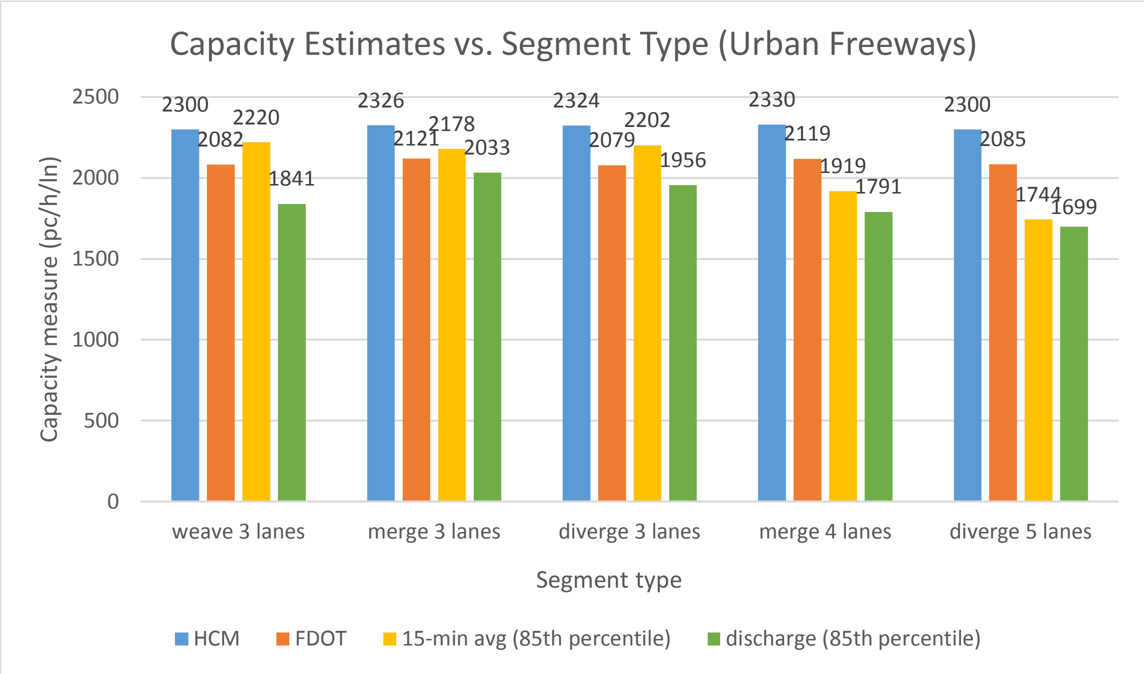

Figure 3.4. Capacity comparison results by number of lanes for urban freeway segments

Figure 3.5 shows the average values across 2-lane and 3-lane rural freeway segments of the 85th percentile 15-min average pre-breakdown flow, the 85th percentile discharge flow, the HCM 2010 and FDOT. Consistent with literature review findings regarding urban freeways (Lu and Elefteriadou, 2013) capacities are higher for 3-lane facilities than for 2-lane facilities.

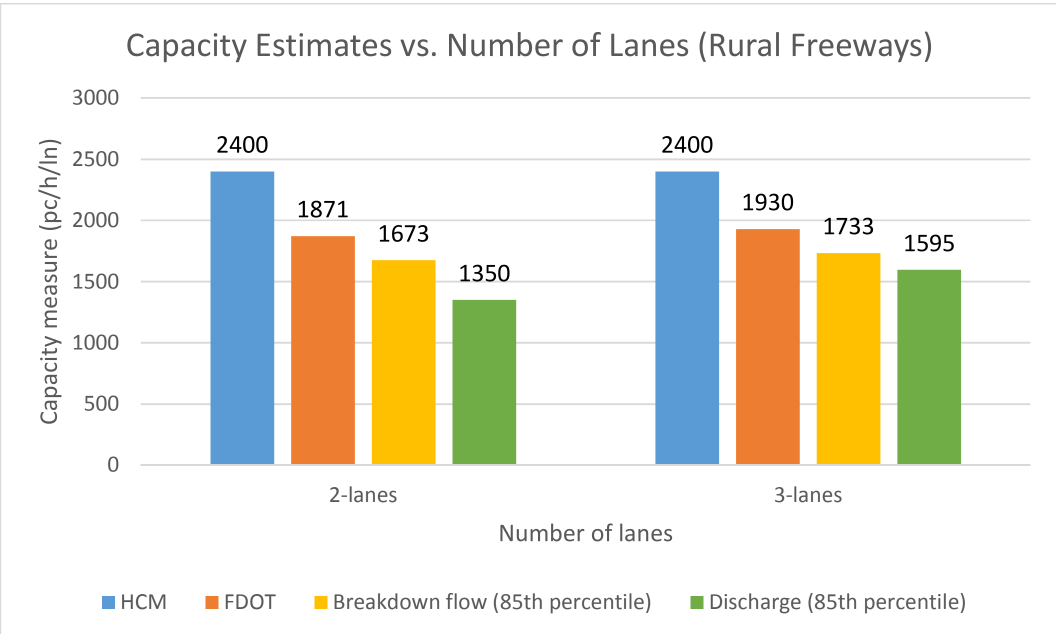

Figure 3.5. Capacity comparison results by number of lanes for rural freeway segments

The relationship between free-flow speed and pre-breakdown capacity was also examined for the urban sites as shown in Figure 3.6. All rural sites have the same FFS, thus, we are not able to evaluate capacity values as a function of FFS for these. Based on the results of the analysis, for the same number of lanes, capacity and FFS are not consistently related. Based on the data collected for this study, the number of lanes is a more significant factor than the FFS in determining capacity.

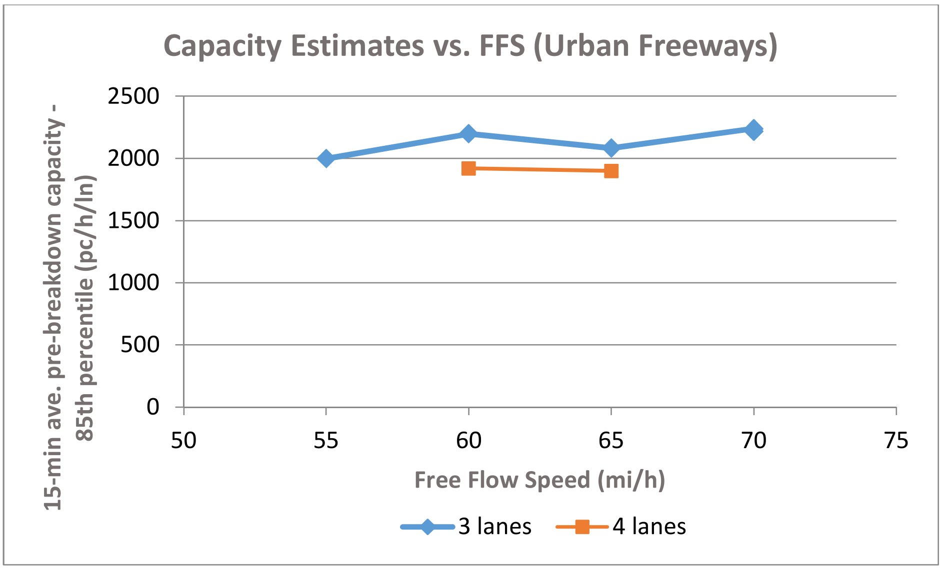

Figure 3.6. Capacity comparison results by FFS for urban freeway sites.

4. CONCLUSIONS AND RECOMMENDATIONS

4.1. Recommended Capacity Values by Segment Type

This section presents recommendations regarding the measurement of capacity as well as recommended values for various types of facilities and for undersaturated and oversaturated conditions. Based on the analysis results, the following recommendations are drawn:

- There is a clear drop in throughput between pre-breakdown and discharge values, generally in the range of 5-10%.
- The 85th percentile of the 15-min average pre-breakdown flow, which can be interpreted to be the closest in definition to that in the HCM 2010, is lower than the HCM 2010 values for all sites.
- The FDOT values are closer to the 85th percentile of the 15-min average pre-breakdown flow.
- The weaving segment investigated in this research appears to have higher pre-breakdown capacity than merging or diverging segments. However, additional sites (with varying weaving configurations) are needed to produce conclusive results regarding weaving segment capacities.
- Merging and diverging segments have comparable capacities (both pre- and post-breakdown).
- Three lane facilities have higher per lane capacities than lower and higher lane facilities, a finding consistent with previous literature.

Based on the analysis provided, we recommend using the capacity values (in pc/h/ln) shown in Table 4.1. The pre-breakdown flow values correspond to the 85th percentile of the 15-min average flow before breakdown. We believe this value is fairly representative of the capacity definition in the HCM. Higher values would be unlikely to be sustained. Lower values could also be used (for example the 50th percentile); however, the drop from existing assumed capacities would be significant. The average discharge values provided represent the entire oversaturated period and represent the 85th percentile of the measured values.

Table 4.1. Recommended capacity values for various types of segments (pc/h/ln)

Segment Type	Number of Lanes	Capacity Values (pc/h/ln)	
		Pre-Breakdown Capacity *	**Discharge Capacity ****
Urban merge and diverge freeway segments	3	2,100	1,900
Urban merge and diverge freeway segments	2; 3>	2,000	1,800
Urban weaving freeway segments	3	2,200	2,000
Urban weaving freeway segments	2; 3>	2,100	1,900
Rural merge/diverge segments	3	1,900	1,700
Rural merge/diverge segments	2; 3>	1,800	1,600

* Coefficient of variation (CV) and standard deviation (SD):
Urban sites: CV= 0.11, SD = 230
Rural sites: CV= 0.08, SD = 130
** Coefficient of variation (CV) and standard deviation (SD):
Urban sites: CV= 0.09, SD = 170
Rural sites: CV= 0.15, SD = 200

It should be noted that the values shown in Table 4.1 represent the types of sites considered in this study, but they are not necessarily appropriate for all freeway bottlenecks. Rather, these can be thought of as values to be used in planning applications. As demonstrated earlier in the data collection stage, each site has its own characteristics and respective capacity measures. For operational analysis purposes, field estimates of capacity at the given site would be more accurate than the values of Table 4.1.

The urban weaving segments capacities shown in Table 4.1 were derived from generalizing the results based on the one site that was analyzed in this study. Although this site had 3 lanes per direction, the recommended capacity values for 2 or greater than 3 weaving segments were derived by extrapolating from the results of the merge/diverge segments. Thus, the weaving segments recommended values should be used with caution.

The selection of an appropriate capacity value has significant implications on the design and operations of a facility. As demonstrated during this data collection (as well as in previous research) flows higher and/or lower than this value could be observed in the field. When a capacity value is sought, the analyst should consider the purpose and use of the value and decide accordingly what the acceptable percentile of the maximum flow should be, to avoid over-design or under-design of the facility.

An equivalent table that provides capacities in veh/h/ln is shown below (Table 4.2). The assumptions for deriving this table are as follows: default truck percentages based on FDOT recommendations (4% for urban freeways and 12% for rural freeways), a PHF of 0.95 for pre-breakdown conditions and a PHF of 1.0 for the discharge flow.

Table 4.2. Recommended capacity values for various types of segments (veh/h/ln)

Segment Type	Number of Lanes	Capacity Values (veh/h/ln)	
		Pre-Breakdown Capacity	**Discharge Capacity**
Urban merge and diverge freeway segments	3	1,950	1,860
Urban merge and diverge freeway segments	2; 3>	1,860	1,760
Urban weaving freeway segments	3	2,040	1,960
Urban weaving freeway segments	2; 3>	1,950	1,860
Rural merge/diverge segments	3	1,700	1,600
Rural merge/diverge segments	2; 3>	1,610	1,500

4.2. Recommended Level of Service Thresholds

Based on the recommended capacity values presented in this report, this section develops and recommends suitable adjustments in the density level of service thresholds.

According to the HCM (TRB, 2010), the Level of Service thresholds for density at the various segment types (basic, weaving, merge/diverge) are provided in Table 4.3. Note that in providing these, the HCM does not differentiate between sites with different numbers of lanes, free-flow speeds and area type. A study by Washburn and Kirschner (2006) proposed new Level of Service threshold values for rural freeway segments, based on travelers' perception. These threshold values are considerably lower than the HCM 2010 values and are also presented in Table 4.3. In contrast to the HCM 2010, the analysis was performed over specific stretches of facilities instead of segments (basic, merge/diverge, or weave).

Table 4.3. LOS criteria for urban and rural basic, weaving, merge/diverge segments

LOS	Density (pc/mi/ln)			
	HCM 2010 (urban and rural freeways)			Washburn and Kirschner, 2006 (rural freeways)
	Basic	Weave	Merge/Diverge	Facility
A	≤11	0-10	0-10	≤6
B	>11-18	>10-20	>10-20	>6-14
C	>18-26	>20-28	>20-28	>14-22
D	>26-35	>28-35	>28-35	>22-29
E	>35-45	>35	>35	>29-39
F	Demand exceeds capacity >45	Demand exceeds capacity	Demand exceeds capacity	>39

Although a density threshold for LOS F is not shown for weaving and merge/diverge segments, the Freeways/Multilane Highways subcommittee of the Highway Capacity and Quality of Service Committee (AHB40) has recently decided to incorporate a threshold density value of 43 pc/mi/ln for weaving segments, in addition to the v/c>1 criterion.

To provide recommended thresholds, the average density at capacity at each study site was calculated from the field data based on the speed-flow curve, and considering that capacity is reached at the values presented in Table 4.1 (i.e., density as well as capacity values are subject to the number of lanes and the area type). An example illustration of the calculation of the density at capacity is presented in Figure 4.1 for the I-95 NB site at NW 103rd St, in Miami, FL (merge site). The density at capacity is calculated based on the recommended capacity of 2,100 pc/h/ln (Table 4.1), divided by the average speed at capacity, which according to the data is 55.8 mi/h.

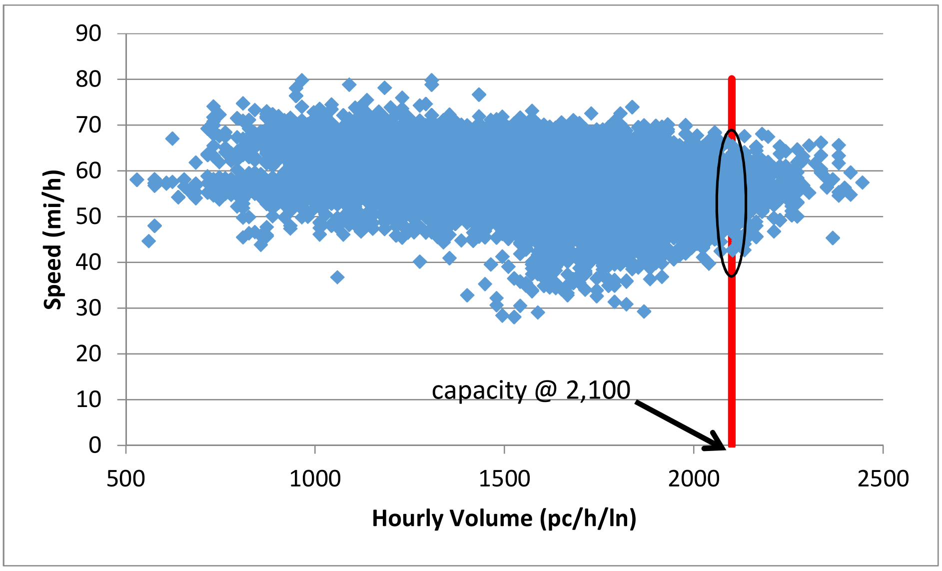

Figure 4.1. Speed-flow curve for I-95 NB, at NW 103rd St., Miami (merge, FFS = 60 mi/h)

The density at capacity for the remaining sites was calculated using the same method. Table 4.4 presents the density at capacity for the urban freeway sites analyzed in this project.

Table 4.4. Density at capacity for urban freeway segments (weave, merge/diverge)

Location	Recommended capacity (pc/h/ln)	FFS (mi/h)	No. of Lanes	Density at capacity (pc/mi/ln)	
				HCM 2010	Field Estimate
I-95 NB, At Butler	2,200	70	3	43	30
I-95 NB, At University	2,100	65	3	-	36
SR-826 EB, At NW 47th Ave	2,100	65	3	-	33
I-4 EB, At SR-408	2,100	60	3	-	40
I-95 NB, At NW 103rd St	2,000	60	4	-	36
I-95, At Philips Hwy	2,000	65	4	-	32
I-4 EB, At I-75	2,000	65	4	-	37
I-95 NB, At Turnpike	2,100	60	5	-	38
I-95 NB, at Baymeadows	2,100	70	3	-	32
I-4 WB, At Lee Road	2,100	55	3	-	41

Therefore, on average, the density at capacity for the 3-lane merge/diverge segments is 37 pc/mi/ln, for the 4-lane merge/diverge segments it is 35 pc/mi/ln, for the weaving segments it is 30 pc/mi/ln. These averages correspond to sites with different free-flow speeds since the data do not show a clear trend between FFS and pre-breakdown capacity. Therefore, at this stage, we cannot distinguish between different speed-flow curves. Based on this analysis, the proposed LOS threshold criteria for density are provided in Table 4.5.

Table 4.5. LOS criteria for weaving, merge/diverge segments (urban freeways)

LOS	Density (pc/mi/ln)			
	3-lane Weave	2; 3> lane Weave	3-lane Merge/Diverge	2; 3> lane Merge/Diverge
A	0-10	0-10	0-10	0-10
B	>10-15	>10-15	>10-18	>10-18
C	>15-20	>15-20	>18-25	>18-25
D	>20-25	>20-25	>25-32	>25-32
E	>25-30	>25-30	>32-37	>32-35
F	Demand exceeds capacity >30	Demand exceeds capacity >30	Demand exceeds capacity >37	Demand exceeds capacity >35

Table 4.6 presents the density at capacity for rural freeway sites analyzed in this project. The estimated density at capacity is considerably lower than the recommended HCM 2010 value, but it reflects the fact that the capacity is also considerably lower. In fact, volumes greater than 1,900 pc/h/ln were hardly observed at these sites, therefore, it was not possible to calculate the speeds at capacity from the data. For the calculation of the density at capacity at the rural sites we considered the speed-flow equations provided by the 2010 HCM for basic freeway segments, since these curves fit the data well (Exhibit 11-3, HCM 2010).

Table 4.6. Density at capacity for rural freeway segments (basic)

Location	Recommended capacity (pc/h/ln)	FFS (mi/h)	No. of Lanes	Density at capacity (pc/mi/ln)	
				HCM 2010	Estimate
I-75, At CR 514 NB/SB	1,800	75	2	45	27
Turnpike, South of CR 468 NB	1,800	75	2	45	27
I-75, North of SR-48 NB	1,800	75	2	45	27
I-75, North of William Rd NB/SB	1,900	75	3	45	29

Figure 4.2 shows the speed-flow curve for I-75 North of William Rd. and for an FFS = 75 mi/h. Based on this assumption, the speed at capacity (1,900 pc/h/ln) is 66.2 mi/h and the resulting

density is approximately 29 pc/mi/ln. The density at capacity at the remaining sites was calculated similarly.

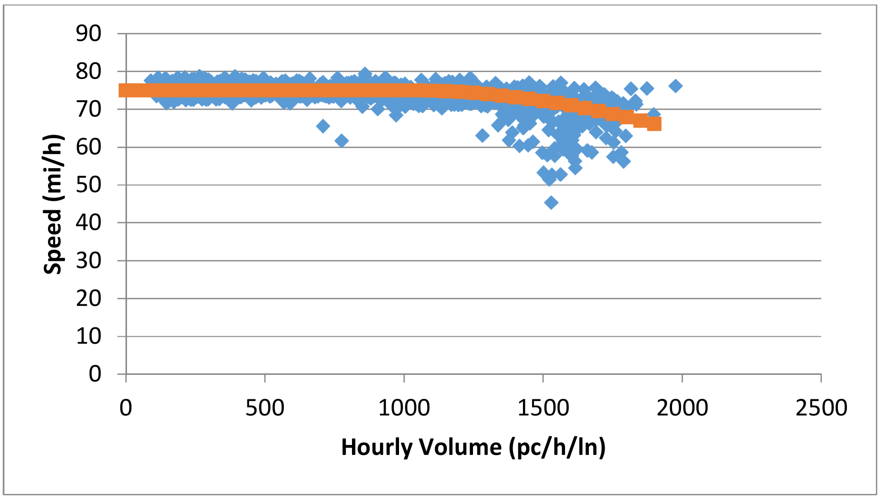

Figure 4.2. Speed-flow curve for I-75 North of William Rd (merge, FFS = 75 mi/h)

According to these results, the recommended thresholds for density are provided in Table 4.7.

Table 4.7. LOS criteria for rural merging/diverging segments

LOS	Density (pc/mi/ln)	
	3-lanes	2; 3> lanes
A	≤8	≤8
B	>8-14	>8-14
C	>14-19	>14-19
D	>19-24	>19-24
E	>24-29	>24-27
F	Demand exceeds capacity >29	Demand exceeds capacity >27

As shown in Table 4.7, the rural freeways densities are considerably lower than the HCM 2010 thresholds for LOS F (shown in Table 4.3). This suggests that the HCM 2010 overestimates capacities at the rural sites and does not account for differences in operation between rural and urban sites. The recommended threshold values presented in Table 4.7 were compared with the threshold values proposed by Washburn and Kirschner (2006) since these concern rural freeways as well. From the comparison with Table 4.3 values it is concluded that the proposed thresholds are lower than the Washburn and Kirschner values, which may be attributed to the differences in the study sites analyzed in both projects.

REFERENCES

Brilon, W. Randomness and Reliability in Freeway Traffic Flow. TRAIL Research School, Delft, Netherlands, 2005.

Cassidy, M. J., and R. L. Bertini. Some Traffic Features at Freeway Bottlenecks. In *Transportation Research Part B: Methodological 33(1)*, 1999, pp. 25–42.

Florida Department of Transportation. *Bottlenecks on Florida SIS*, 2011.

Florida Department of Transportation. *2013 Quality/ Level of Service Handbook*, 2013.

Kondyli, A., L. Elefteriadou, W. Brilon, F. Hall, B. Persaud, S. and Washburn. Development and Evaluation of Methods for Constructing Breakdown Probability Models. In *J. Transp. Eng., 139(9)*, 2013, pp. 931–940.

Lorenz, M., and L. Elefteriadou. Defining Freeway Capacity as a Function of Breakdown Probability. In *Transportation Research Record 1776*, National Academy Press, 2001, pp.43-51.

Lu, C., and L. Elefteriadou. An Investigation of Freeway Capacity Before and During Incidents. In *Transportation Letters 5(3)*, 2013, pp. 144-153.

Persaud, B., S. Yagar, D. Tsui, and H. Look. Study of Breakdown-Related Capacity for a Freeway with Ramp Metering. In *Transportation Research Record 1748*, TRB, National Research Council, Washington D.C., 2001, pp. 110-115.

Transportation Research Board (TRB). *Highway Capacity Manual*, Washington, DC, 2000.

Transportation Research Board (TRB). *Highway Capacity Manual*, Washington, DC, 2010.

Washburn, S. S., and D. S. Kirschner. Rural Freeway Level of Service Based Upon Traveler Perception. In *Transportation Research Record: Journal of the Transportation Research Board 1988*, TRB, Washington D.C., 2006, pp. 31-37.

Washburn, S. S., Y. Yin, V. Modi, and A. Kulshretha. Investigation of Freeway Capacity a) Effect of Auxiliary Lanes on Freeway Segment Volume Throughput and b) Freeway Segment Capacity Estimation for Florida Freeways. TRC-FDOT-73157-2010. Florida Department of Transportation, Tallahassee, 2010.

APPENDIX A: Literature Review

This appendix summarizes the literature review findings related to capacity definitions and the two-capacity phenomenon.

A.1. Capacity Definitions in the Literature

Research on freeway capacity (Cassidy and Bertini, 1999; Lorenz and Elefteriadou, 2001; Persaud et al., 2001; Brilon, 2005) has examined the conditions under which breakdown occurs, and concluded that it does not occur deterministically under a given set of volumes. Also, several of these articles have shown that this maximum value does not necessarily coincide with the breakdown event. Lastly, it has also been shown that regardless of whether one uses the maximum pre-breakdown flow or the breakdown flow to define capacity, both values vary widely on a daily basis even for the same site and for similar traffic conditions. This is inconsistent with traditional traffic analysis methods (HCM 2000, HCM 2010), which assume that traffic transitions to oversaturated conditions (i.e., breakdown event) when demand reaches a specific maximum sustainable value, labeled as capacity.

Researchers have investigated various potential capacity definitions. Lorenz and Elefteriadou (2001) defined capacity as "the rate of flow (expressed in pc/h/ln and specified for a particular time interval) along a uniform freeway segment corresponding to the expected probability of breakdown deemed acceptable under prevailing traffic and roadway conditions in a specified direction." Elefteriadou and Lertworawanch (2003) used three potential definitions of capacity for their study. The first is based on the breakdown flow, defined as the 5- or 15-minute flow immediately before the breakdown. The second definition is based on the maximum pre-breakdown flow defined as the maximum 5- or 15-minute flow observed before the breakdown. The third definition is based on the maximum discharge flow, defined as the maximum 5- or 15-minute flow occurring during oversaturated conditions. Cassidy and Rudjanakanoknad (2005) termed capacity the sustained flow that a freeway discharges from all exits that are unblocked by spillover queues from downstream while the freeway entrances are queued.

Brilon (2005) defined capacity as “the volume at which traffic breaks down from fluent to congested conditions”. This definition is consistent with the breakdown flow defined above. After analyzing a considerable amount of data, the author found that capacity is Weibull-distributed. Brilon (2005) recommended to “use the 50%-percentile of the breakdown probability distribution as the nominal capacity”. Elefteriadou et al. (2006) reviewed several definitions for capacity. Although the authors acknowledge the difficulty in reaching a consensus, they did provide some conclusions regarding the definition of capacity. The definition chosen was the maximum pre-breakdown 5-minute value and they proposed to use either the mean or the 15th percentile of the distribution as the capacity measure. They also suggest measuring the queue discharge flow, which they defined as “the average flow rate for one day over the entire time there is an upstream queue (and no downstream queue affecting the measurement location).” They agree on using the 50th-percentile for this distribution. Dervisoglu, et al. (2009) used the “maximum observed flow during a congested day” for their study.

Of all of the studies found in the literature, there has been no consensus on the single, best way of defining capacity. However, researchers acknowledge the importance of treating capacity as a random variable (thus, considering its distribution), rather than a fixed number. It is possible that different capacity definitions may be more appropriate for different applications. For example, design applications may require more conservative estimates of capacity, while traffic management applications may require multiple capacity values to evaluate and monitor different operating conditions.

A.2. Capacity Drop Estimates in the Literature

This section presents an overview of studies pertaining to the capacity drop, or the two-capacity phenomenon. This event can be described as observing two capacities on a section, one before and one after a breakdown has occurred. This type of research addresses similar issues to those outlined in the previous section, but focusing on the differences in throughput before and after the breakdown.

Banks (1990, 2006) examined the potential for capacity drop at several different freeways. He concluded that there is the possibility of a capacity increase, depending on the roadway. The range of values across both studies is -0.42% to 15.4%. These drops occur over a variety of bottleneck types: merge, lane drop, grade, and diverge. The number of lanes was also found to be a factor in the magnitude of a capacity drop. Hall and Agyemang-Duah (1991) determined the capacity drop at on-ramp merges along Queen Elizabeth Way in Ontario, Canada. A range of values were calculated, and the lowest value was an increase of 7.76%.

Persaud et al. (1998) also observed on-ramp merge bottlenecks. Using only data from the median lane, the breakdown and mean queue discharge flows were obtained. Capacity drop values ranged between 10.6 and 15.3%. Cassidy and Bertini (1999) used a rescaled N-curve to analyze capacity drop. Rescaled N-curves are created from cumulative curves (i.e., curves of cumulative vehicle arrivals). These curves are then shifted based on free flow travel time to the most downstream detector. A reduction factor, q_0, is subtracted from all curves, to create the final N-curves. Tight grouping of the curves indicates consistency on the segment. When the curves diverge, a bottleneck is possible. Using this method on two on-ramp merge bottlenecks the range of capacity drop values was 4% to 10%.

Lorenz and Elefteriadou (2001) performed analysis at two freeway on-ramps, looking at merge bottlenecks along Highway 401 in Toronto, Canada. The authors found that at one site the capacity increased from 1,500 to 1,600 veh/h/ln. The authors concluded that the presence of a drop depended on the flow just before a breakdown; higher flows would most likely result in a drop, while lower flows could result in an increase. An increase in discharge flow after a breakdown is consistent with the results of previous studies performed by Banks (1990) and Hall and Agyemang-Duah (1991).

Bertini and Malik (2004) observed an on-ramp merge bottleneck with a range of capacity drop values between 2% and 5%, with an average value of 4%. Bertini and Leal (2005) observed a lane drop bottleneck with a range of capacity drop values between 6.7% and 10.7%, with an average value of 9.7%.

Cassidy and Rudjanakanoknad (2005) observed merge bottlenecks along the I-805 Northbound in California. The authors compared vehicle accumulations in the shoulder lane and the average of the adjacent lanes. Over all study days, it was found that at the time of breakdown, the accumulation in the shoulder lane was 16 vehicles (averaged over 1 minute intervals) and remained above 16 vehicles for the duration of the breakdown event. The shoulder lane experienced capacity drop values ranging from 8.3% to 17.3%.

Another method of understanding capacity is to examine its relationship to the critical density, as reported by Chung et al. (2007). The densities were calculated over the entire study period. The critical density corresponded with the time a breakdown occurred. The authors analyzed three bottleneck types. The first was a merge bottleneck; the drop in capacity was about 10%, while the densities at the capacity drop were between 208 and 254 veh/km. The second was a lane drop bottleneck; the capacity drop was at least 5%, with densities ranging from 89 to 96 vehicles/km. The third location was on a horizontal curve, where the capacity drops recorded were all above 3%, and the densities varied from 129 to 179 veh/km. The average densities corresponding to the onset of a breakdown were determined for each site. These average values were divided by the number of lanes. All three values were very similar, indicating there may be a possible density threshold to determine whether a capacity drop will occur.

Oh and Yeo (2012) examined merge bottlenecks for roadways with number of lanes ranging from 2 to 5. Each configuration resulted in different average capacity drops. For two lanes it equaled 16.33 %, for three lanes 13.68 %, four lanes 11.61 %, and five lanes 8.85 %. Persaud et al. (1998) also observed on-ramp merge bottlenecks, obtaining a range of capacity drop values between 10.6 % and 15.3 %. An analysis was performed on individual lanes of each of the study sites. The analysis shows that the median lane had the largest capacity no matter how many lanes the section had. The capacity of each lane decreased, moving from the median lane to the shoulder lane. The same situation occurred for the discharge capacity values, thus the capacity drop was proportionally equal across the lanes. The capacity drops of the shoulder lanes (7.31% for 4-lanes and 2.96% for 5-lanes) were less than the lowest observed capacity drop (8.85% for 5-lanes). The authors state that the outer lanes have not reached capacity, while the other lanes have.

Although there is no consensus on the causes of the capacity drop phenomenon, it has been observed in many occasions, mostly on merge bottleneck locations. In addition, different authors have provided different explanations for the capacity drop. Lebacque (2003) surmised that it occurs because vehicles can decelerate much more strongly than they can accelerate. A second explanation of the capacity drop was given by Laval and Daganzo (2006) and is based on lane changing behavior. A third explanation (Treiber et al., 2006) assumes that drivers prefer larger time headways when local traffic dynamics are unstable or largely varying. According to Yeo (2008) the amount of capacity drop depends on the number of deceleration waves initiated by lane changes, as well as the number of lanes in the bottleneck location. Some research has shown that it is possible to have an increase in capacity after the breakdown event. Table A.1 presents a summary of the past capacity drop-related research. One of the important reasons for the differences shown is that the capacity drop values reported vary depending on the capacity measurement and definition used in each study, which are not consistent.

Table A.1. Comparison of capacity drop research

Authors	Bottleneck Type	Number of Lanes	Capacity Definition Used	Capacity Drop (percent)
Banks (1990)	Merge	4	Not defined	0.5 - 4.04
Banks (1990)	Merge	4	Not defined	-0.42 - 1.11
Banks (2006)	Merge	2	Not defined	3.9 - 5.6
	Merge	3	Not defined	5.1
	Merge	3	Not defined	2.3 - 4.4
	Merge	2	Not defined	6.3
	Merge, Horizontal Curve	2	Not defined	8.7
	Merge	2	Not defined	8
	Merge, 3-D Curve	3	Not defined	1.8 - 5.8
	Lane Drop	2	Not defined	9
	Merge, Grade	4	Not defined	10.1
	Merge	4	Not defined	9.6
	Merge, Grade	4	Not defined	11.6
	Grade	4	Not defined	5.1
	Merge, Grade	4	Not defined	5.1
	Weave Exit Leg	4	Not defined	3.1
	Grade, Weave	4	Not defined	4.3
	Merge	4	Not defined	8.4

Table A.1, continued

Banks (2006)	Merge, Grade	5	Not defined	8.4
	Merge	3	Not defined	5.3
	Diverge	2	Not defined	3.5
	Merge	3	Not defined	7.2
	Weave	2	Not defined	15.4
Bertini and Malik (2004)	Merge	2	Rescaled N-curve	2 – 5
Bertini and Leal (2005)	Lane Drop	2	Rescaled N-curve	6.7 - 10.7
Cassidy and Bertini (1999)	Merge	3	Rescaled N-curve	8 – 9
	Merge	3	Rescaled N-curve	4 – 10
Cassidy and Rudjanakanoknad (2005)	Merge	4	Sustained flow a freeway discharges from all exits	8.3 - 17.3
Chung et al. (2007)	Merge	4	Breakdown flow	5 – 18
	Merge, Lane Reduction	2	Breakdown flow	5.1 - 8.5
	Merge, Horizontal Curve	3	Breakdown flow	3 – 12
Hall and Agyemang-Duah (1991)	Merge	3	Not defined	-7.76 - 10.36
Lorenz and Elefteriadou (2001)	Merge	3	Breakdown flow	N/A
	Merge	3	Breakdown flow	-6.7
Oh and Yeo (2012)	Merge	2	Maximum number of vehicles over a 5-min period at free flow speed	16.33
	Merge	3	Maximum number of vehicles over a 5-min period at free flow speed	13.68
	Merge	4	Maximum number of vehicles over a 5-min period at free flow speed	11.61
	Merge	5	Maximum number of vehicles over a 5-min period at free flow speed	8.85
Persaud et al. (1998)	Merge	3	Breakdown flow and mean queue discharge flow	11.6 - 15.3
	Merge	3	Breakdown flow and mean queue discharge flow.	10.6

Based on Table A.1 it can be concluded that the majority of research has dealt with the capacity drop phenomenon at freeway merging sections, while there is limited information about the capacity drop percent at weaving, diverging segments, or at lane drops. The capacity did not drop for all sites reported, with changes in the range between -7.76% and 17.3%. This finding is significant since this capacity increase may be due to differences in the definition and measurement specifications used, or even due to randomness in the pre-breakdown and discharge maximum throughput values.