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THE FLORIDA DEPARTMENT OF TRANSPORTATION
SYSTEMS PLANNING OFFICE

on Project

"Travel Time Reliability Models for Freeways and Arterials"

FDOT Contract BD-545, RPWO #70 (UF Project 00060228)

September 30, 2007

The University of Florida

DISCLAIMER

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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

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16. Abstract

Travel time reliability is widely recognized as one of, if not the, most important performance measure of highway traveler perceptions. However, determining how to measure, quantify, predict, and report reliability has proved to be elusive. A previous FDOT research project (FDOT Contract BD-545, RPWO #48) on travel time reliability reviewed and recommended a travel time reliability definition to be used by FDOT. It also developed models for estimating travel time realibility on freeways based on field data from Philadelphia, PA. The research summarized in this report focuses on a) modifying and adjusting the previously developed models for freeways with Florida data; b) using FDOT databases to report travel time reliability for freeways throughout Florida; and c) developing a preliminary framework for reporting travel time reliability on arterials. The report presented the data obtained from Florida and the analyses conducted to estimate travel time reliability, including flow and speed data, as well as incident and weather information. The database requirements for obtaining travel time reliability in Florida were provided, along with an assessment of existing FDOT databases for estimating travel time reliability for the SIS. The report also provides a literature review on arterial travel time estimation, along with a preliminary framework for estimating travel time on arterials. The research concluded the following:a) There are several systems being installed in Florida for monitoring various performance measures, including speeds, travel times and incidents. These systems are still being tested and calibrated, therefore this research could not use field data from Florida to recalibrate the existing models developed under FDOT Contract BD-545, RPWO #48. b) The incident information data obtained from District 4 appear to be of good quality; however, there are some inconsistencies observed in the reporting for certain performance measures. c) Weather information can be obtained relatively easily across Florida; however, the analysis can be time-consuming depending on the sampling interval in terms of time, as well as in terms of space (i.e., the number of stations examined across Florida).

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EXECUTIVE SUMMARY

The goal of the Strategic Intermodal System (SIS) is to provide a transportation system that efficiently serves Florida's citizens, businesses and visitors; helps Florida become a worldwide economic leader; enhances economic prosperity and competitiveness; enriches quality of life; and reflects responsible environmental stewardship. The SIS is composed of transportation facilities and services of statewide and interregional significance, including both freeways and arterials. Much research on the SIS and its users is needed. Travel time reliability is widely recognized as one of, if not the, most important performance measure of highway traveler perceptions. However, determining how to measure, quantify, predict, and report reliability has proved to be elusive.

A previous FDOT research project (FDOT Contract BD-545, RPWO #48) on travel time reliability reviewed and recommended a travel time reliability definition to be used by FDOT. At the time this research was initiated, there were no field data available in Florida suitable for this project. Therefore, the models developed were based on field data from Philadelphia, PA. These models predict travel time reliability for freeways.

The research summarized in this report focuses on a) modifying and adjusting the previously developed models for freeways with Florida data; b) using FDOT databases to report travel time reliability for freeways throughout Florida; and c) developing a preliminary framework for reporting travel time reliability on arterials. The report presents the data obtained from Florida and the analyses conducted to estimate travel time reliability, including flow and speed data, as well as incident and weather information. The database requirements for obtaining travel time reliability in Florida are provided, along with an assessment of existing FDOT databases for estimating travel time reliability for the SIS. The report also provides a literature review on arterial travel time estimation, along with a preliminary framework for estimating travel time on arterials.

The following are concluded from the research:

- There are several systems being installed in Florida for monitoring various performance measures, including speeds, travel times and incidents. These systems are still being tested and calibrated, therefore this research could not use field data from Florida to recalibrate the existing models developed under FDOT Contract BD-545, RPWO #48.

- The incident information data obtained from District 4 appear to be of good quality; however, there are some inconsistencies observed in the reporting for certain performance measures.
- Weather information can be obtained relatively easily across Florida; however, the analysis can be time-consuming depending on the sampling interval in terms of time, as well as in terms of space (i.e., the number of stations examined across Florida).

The following are recommendations that resulted from this research:

- The existing data collection systems in Florida should be refined and data should begin to be archived as soon as possible for use in travel time reliability estimation, as well as for other uses.
- It is possible to use the incident data from District 4 to extrapolate to incident occurrences across the SIS. Such data should be recorded and obtained from each district to be able to accurately report on reliability across the SIS.
- The preliminary framework developed for estimating arterial travel time should be simplified so that it can be easily incorporated into the travel time reliability estimation databases, and travel time can be estimated for arterials with data that are typically available from the districts.

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I. BACKGROUND

The goal of the Strategic Intermodal System (SIS) is to provide a transportation system that efficiently serves Florida's citizens, businesses and visitors; helps Florida become a worldwide economic leader; enhances economic prosperity and competitiveness; enriches quality of life; and reflects responsible environmental stewardship. The SIS is composed of transportation facilities and services of statewide and interregional significance, including both freeways and arterials. Much research on the SIS and its users is needed. Travel time reliability is widely recognized as one of, if not the, most important performance measure of highway traveler perceptions. However, determining how to measure, quantify, predict, and report reliability has proved to be elusive.

A previous FDOT research project (FDOT Contract BD-545, RPWO #48) on travel time reliability reviewed and recommended a travel time reliability definition to be used by FDOT. At the time this research was initiated, there were no field data available in Florida suitable for this project. Therefore, the models developed were based on field data from Philadelphia, PA. These models predict travel time reliability for freeways.

The research summarized in this report focuses on a) modifying and adjusting the previously developed models for freeways with Florida data; b) using FDOT databases to report travel time reliability for freeways throughout Florida; and c) developing a preliminary framework for reporting travel time reliability on arterials.

The next section presents the data obtained from Florida and the analyses conducted to estimate travel time reliability, while the third section presents the incident data obtained from District 4. The fourth section summarizes the database requirements along with an assessment of existing FDOT databases for estimating travel time reliability for the SIS. The fifth section presents a brief literature review on arterial travel time estimation, along with a preliminary framework for reporting travel time reliability on arterials, while the last section presents conclusions and recommendations.

II. FIELD DATA FROM FLORIDA FOR ESTIMATING TRAVEL TIME RELIABILITY

Field data were obtained from Ft. Lauderdale and from Jacksonville, Florida, and were evaluated and analyzed to evaluate whether the models developed under the previous FDOT project (FDOT Contract BD-545, RPWO #48) can accurately predict travel times at these two locations. This chapter presents the study sites, discusses the data, and summarizes the research findings for these two locations.

Ft. Lauderdale Data

The study route is a 25- mile section along I-95 in Ft. Lauderdale, FL. A sketch of the area is shown in Figure 1. Data were obtained from multiple sensors across a 25-mile section of I-95, within the city limits of Ft. Lauderdale, FL. In the north, the study area begins at W. Palmetto Park Rd, and ends in the south at W. Hallandale Beach Blvd. The northbound and the southbound sections are analyzed separately. The speed limits for the northbound and southbound directions are 65 mph, while the number of lanes varies significantly through the study area. Figures 2 and 3 show the number of lanes by milepost for each direction of the study facility (for both figures the milestone "0" begins at SW 11th Ave, and ends at Hillsboro Blvd).

There are a total of 52 sensor locations, most of which are for detecting traffic on both the northbound and the southbound sections, while others are one direction only. Data were obtained through the FDOT District 4 RTMS (Remote Traffic Microwave Sensors). These data consist of volume, occupancy, average speed, and percentage of vehicle types classified based on length. The data were obtained in roughly 15-minute intervals (the TMC was implementing the new RTMS system, so the time interval has been changed from time to time, from 1-minute to 5-minutes to 15-minute), from July 18th to August 22nd, 2006. Due to missing data and erroneous information, only data from August 3rd to August 21st, 2006 could be used in the analysis.

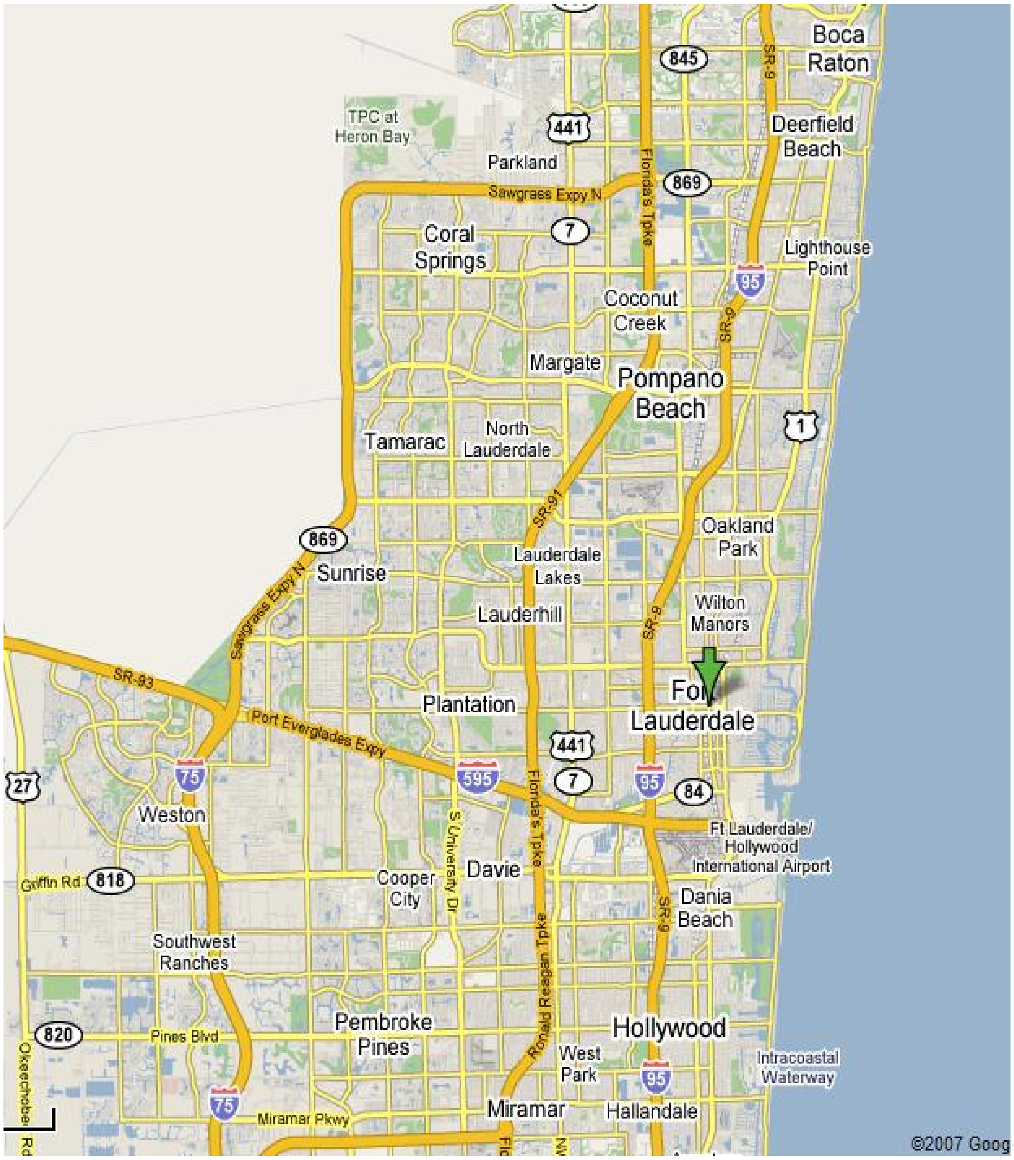

Figure 1 Ft Lauderdale I-95 Coverage Map

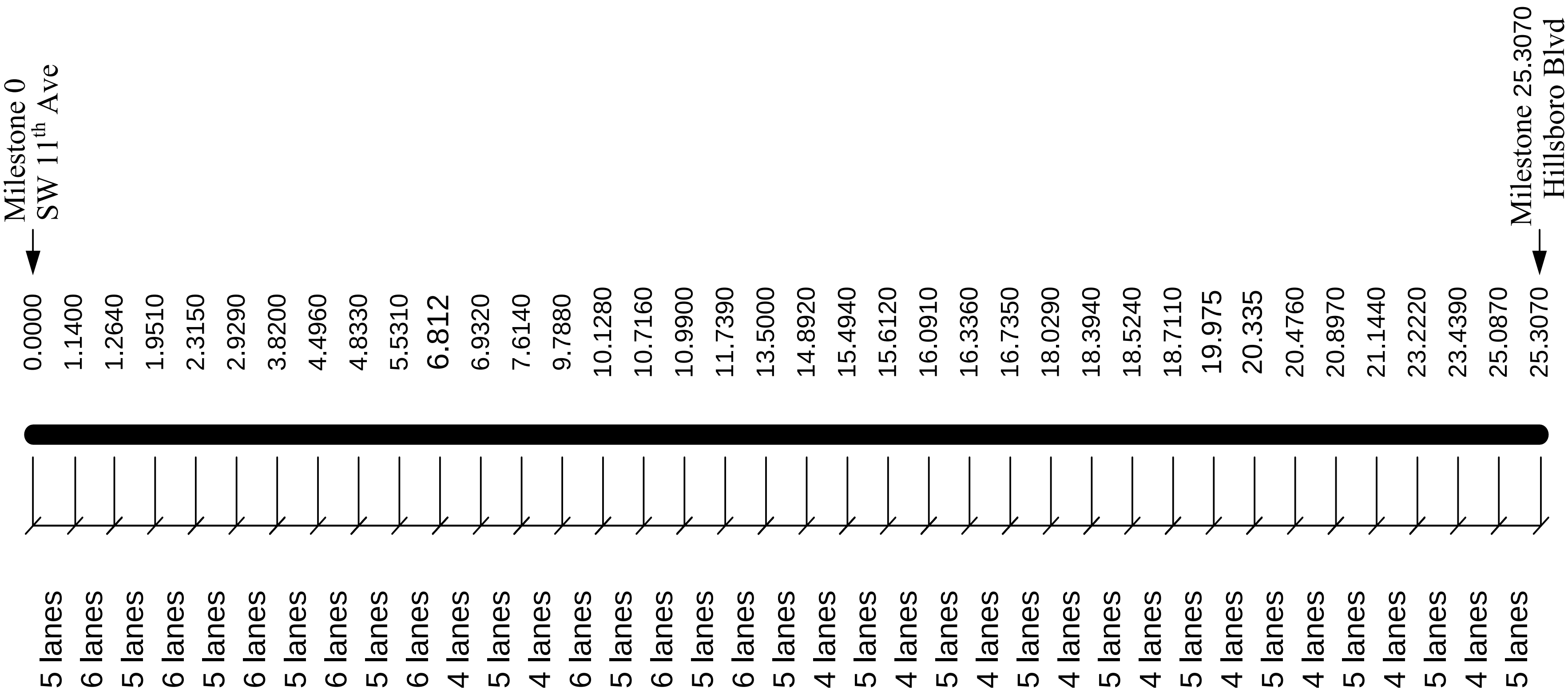

Figure 2 Ft Lauderdale I-95: Number of Lanes in the Southbound Direction

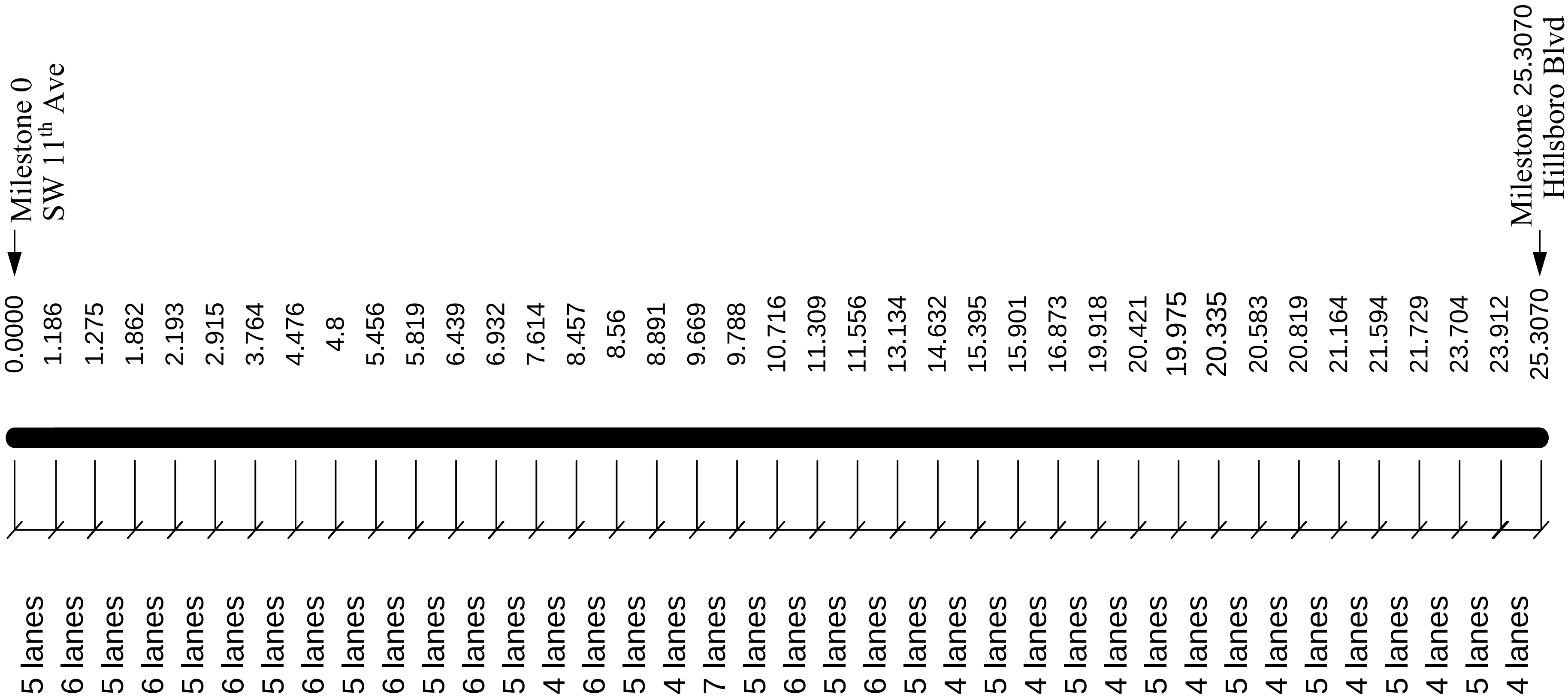

Figure 3   Ft Lauderdale I-95: Number of Lanes in the Northbound Direction

Travel Time Reliability Estimation Using the Ft. Lauderdale Data

There are several different approaches to defining and estimating reliability. Based on the recommendations of the previous FDOT project on reliability, this project defines reliability as the probability of on-time arrival. Two threshold numbers of on-time arrival are considered: (1) travel time made by traveling at ten miles less than the speed limit of each link (2) travel time made by traveling at fifteen miles less than the speed limit of each link.

Fist, the estimated travel time for the northbound and southbound are plotted from 8/3/2006 12:15:00 PM to 8/21/2006 11:15:00 AM, in 15 minute intervals (Figures 4 and 5). As shown, there are certain intervals with excessively high travel times (around time interval 500). There are also certain intervals with unrealistically low travel times (around time interval 413), which are discarded from further analysis.

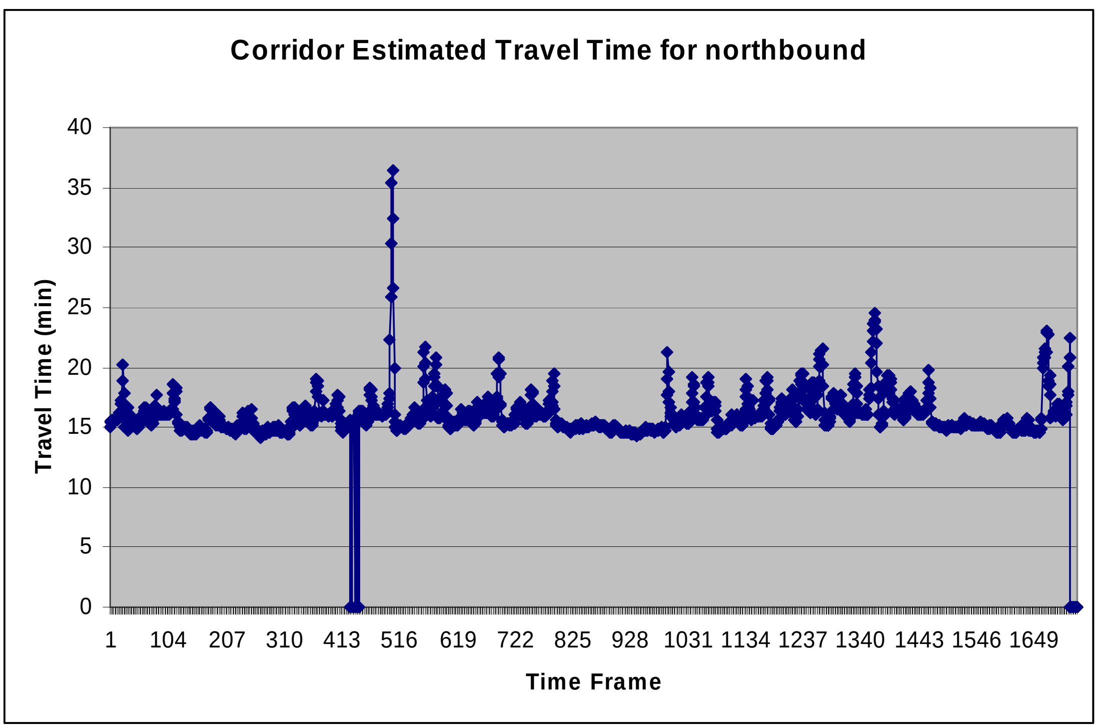

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Figure 4 Estimated Travel Time for Northbound (Ft. Lauderdale)

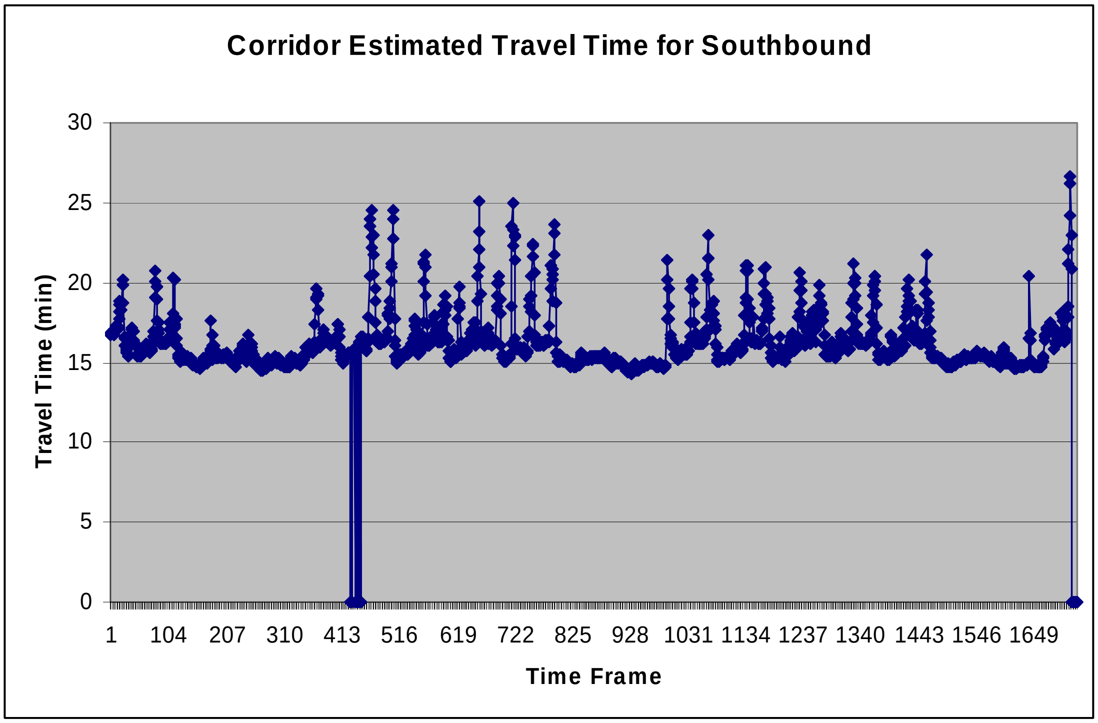

Figure 5 Estimated Travel Time for Southbound (Ft. Lauderdale)

Using the speed limit data and the distance of each link, the unreliability (1-reliability) of each link based on the threshold travel times are calculated and shown in Figures 6 and 7. These two graphs show the unreliability of each link for each direction. As shown, for the northbound direction, the link 16 data point is unrealistically high. Similarly, for the southbound direction, the link 23 data point is equal to 1, which is unrealistic for that particular location.

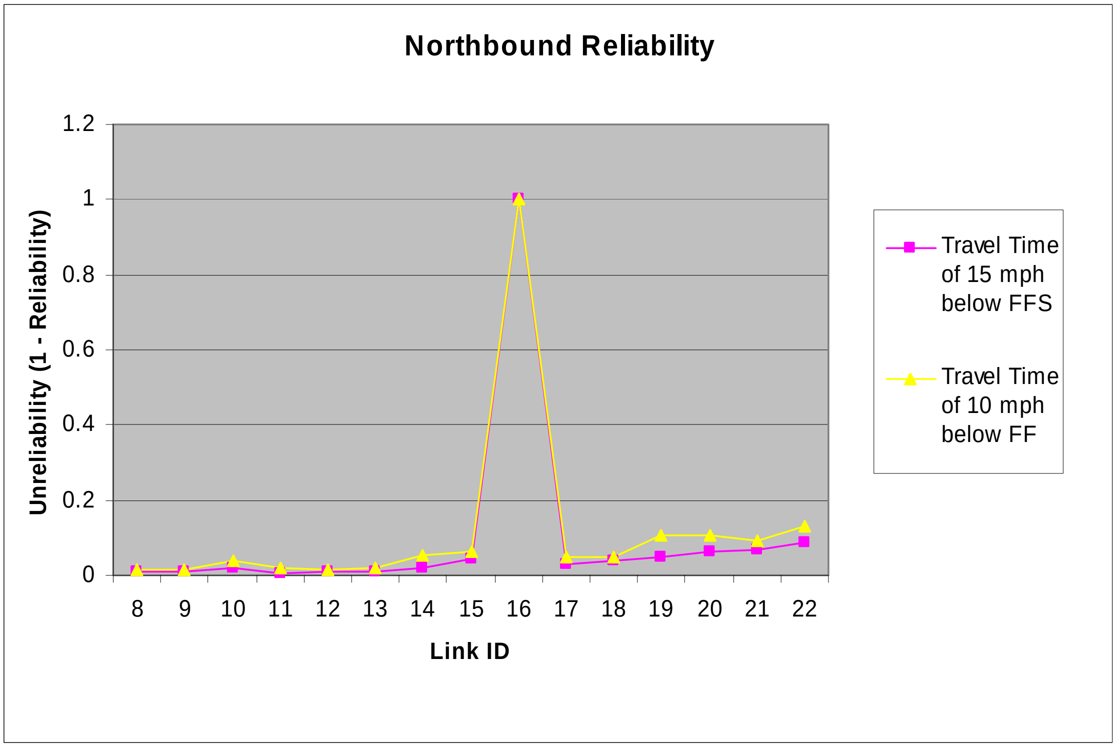

Figure 6 Travel Time Reliability for Northbound (Ft. Lauderdale)

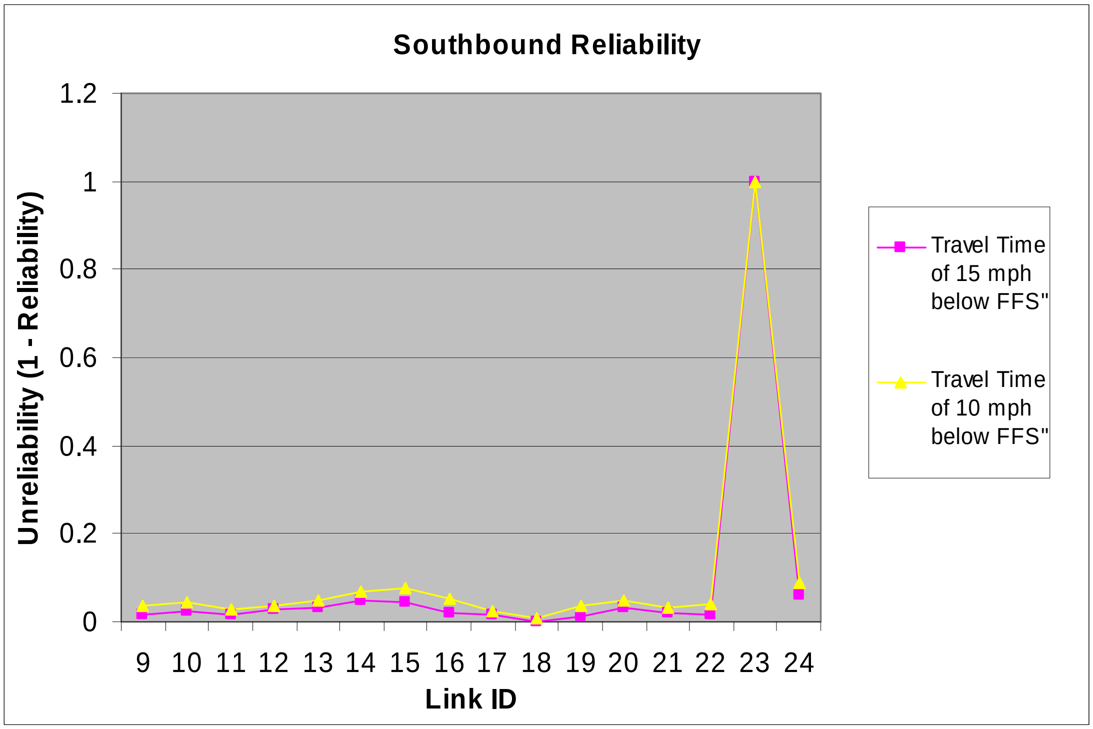

Figure 7 Travel Time Reliability for Southbound (Ft. Lauderdale)

Figures 8 and 9 present the travel time reliability for the entire corridor, and for each direction separately. The shape of the graphs is similar to those produced in previous research. However, there are some spikes which are the result of erroneous data on a specific link.

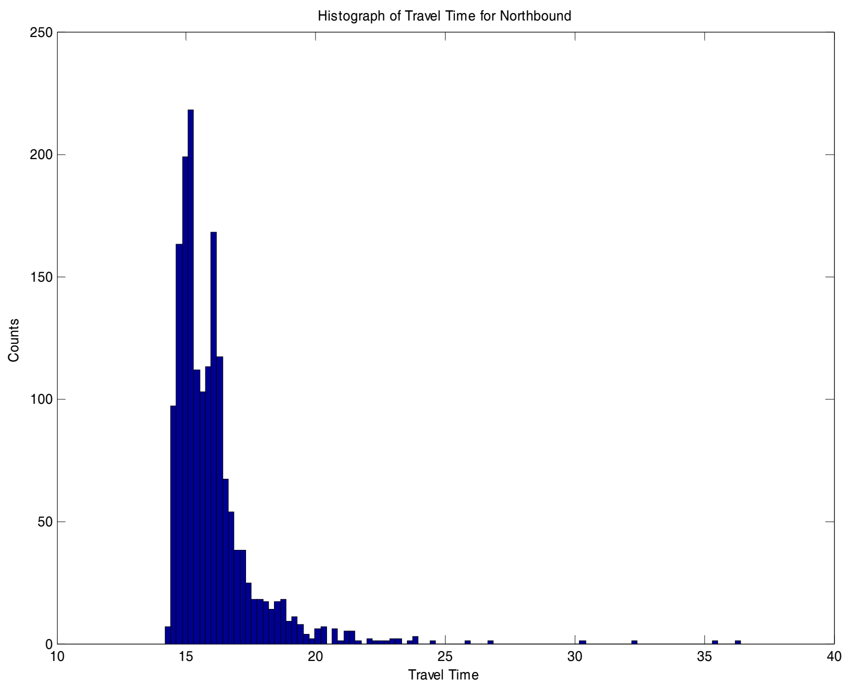

Figure 8 Travel Time Histogram for Northbound (Ft. Lauderdale)

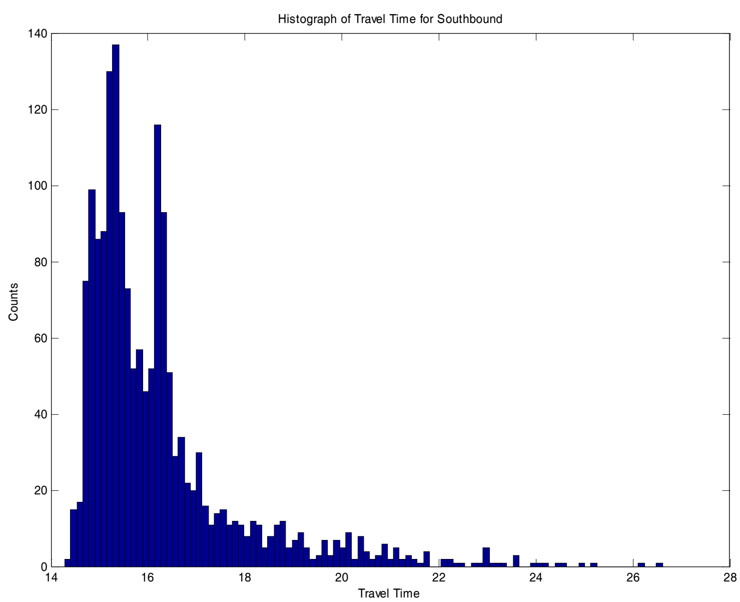

Figure 9 Travel Time Histogram for Southbound (Ft. Lauderdale)

In summary, the data presented above represent approximately 18 days of data collection. A larger database was available; however, it was concluded that the remaining data could not be used because of missing or erroneous data. The RTMS system is still in the process of being tested, therefore data were not available from all sensors for the entire data collection period, and for certain sensors the data collection time interval was frequently changing from 15 minutes to 1 minute or 5 minutes. Even for the 18 days of data collected and analyzed, there were problems in that the speeds shown for certain segments did not accurately represent field conditions. Therefore it was concluded that at this time, flow and speed data from Ft. Lauderdale cannot be used in this research.

Jacksonville Data

The study route is a 25- mile section along I-95 in Jacksonville, FL. A sketch of the area is shown in Figure 10. Data were obtained from multiple sensors across a 25-mile section of I-95, within the city limits of Jacksonville, FL. In the north, the study area begins in the vicinity of the Jacksonville international airport, and ends in the south at Philips. The northbound and the southbound sections are analyzed separately. There are varying speed limits and numbers of lanes along the study area. The speed limits for the northbound and southbound directions are shown in Figures 11 and 12, while the number of lanes through the two routes are shown in Figures 13 and 14.

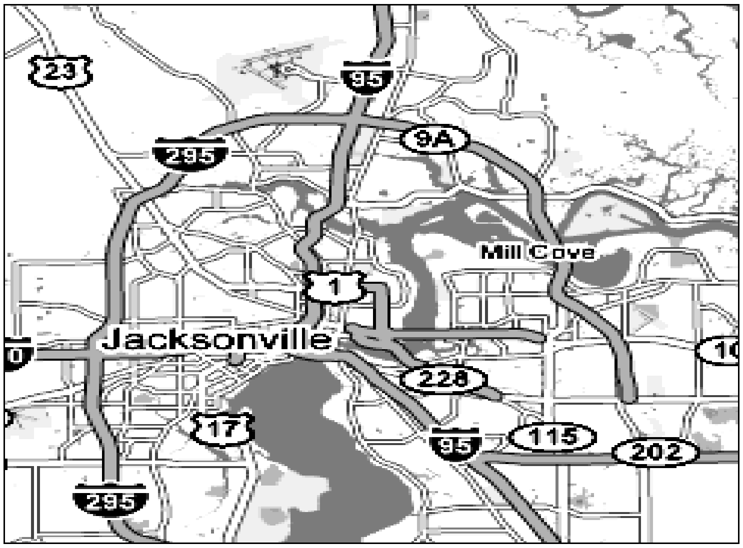

Figure 10 Study Route Along I-95 in Jacksonville, FL
(Source: www.googlemaps.com)

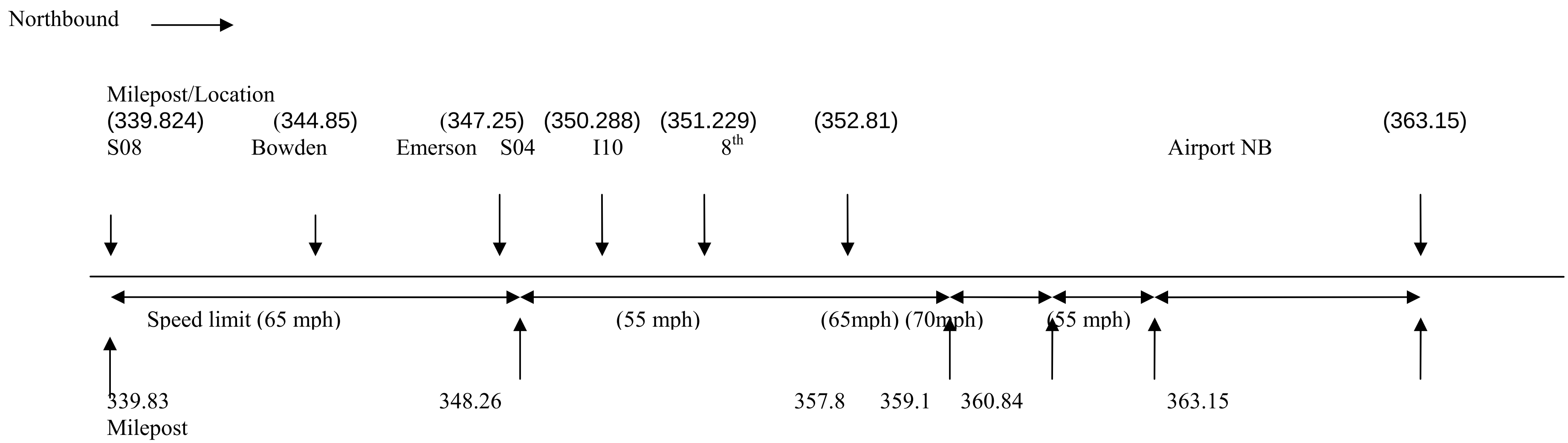

Figure 11 Speed Limits Along I-95 in Jacksonville, FL, Northbound (Source: http://www.dot.state.fl.us/planning/statistics/gis/aboutapps.htm)

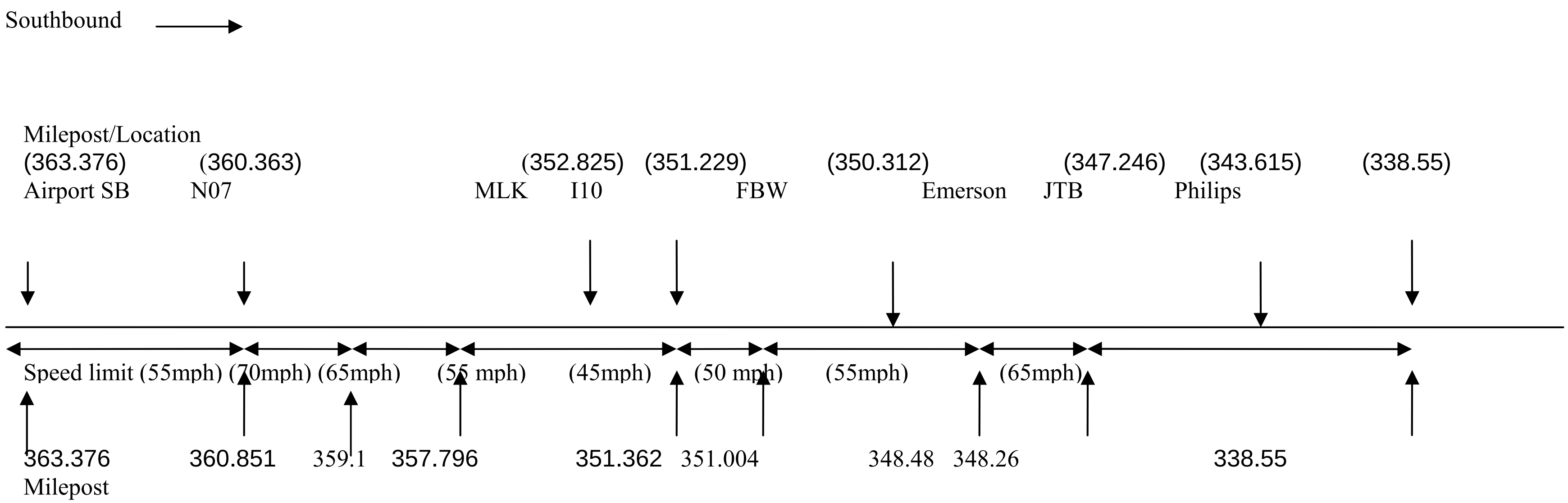

Figure 12 Speed Limits Along I-95 in Jacksonville, FL, Southbound (Source: http://www.dot.state.fl.us/planning/statistics/gis/aboutapps.htm)

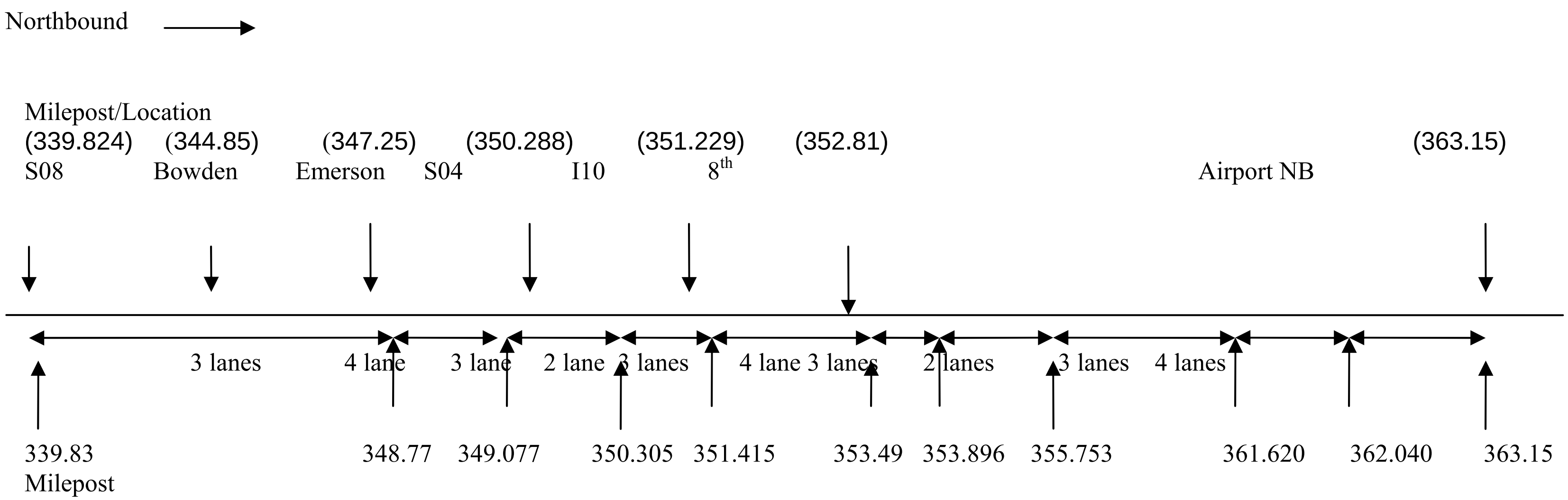

Figure 13 Number of Lanes Along I-95 in Jacksonville, FL, Northbound (Source: http://www.dot.state.fl.us/planning/statistics/gis/aboutapps.htm)

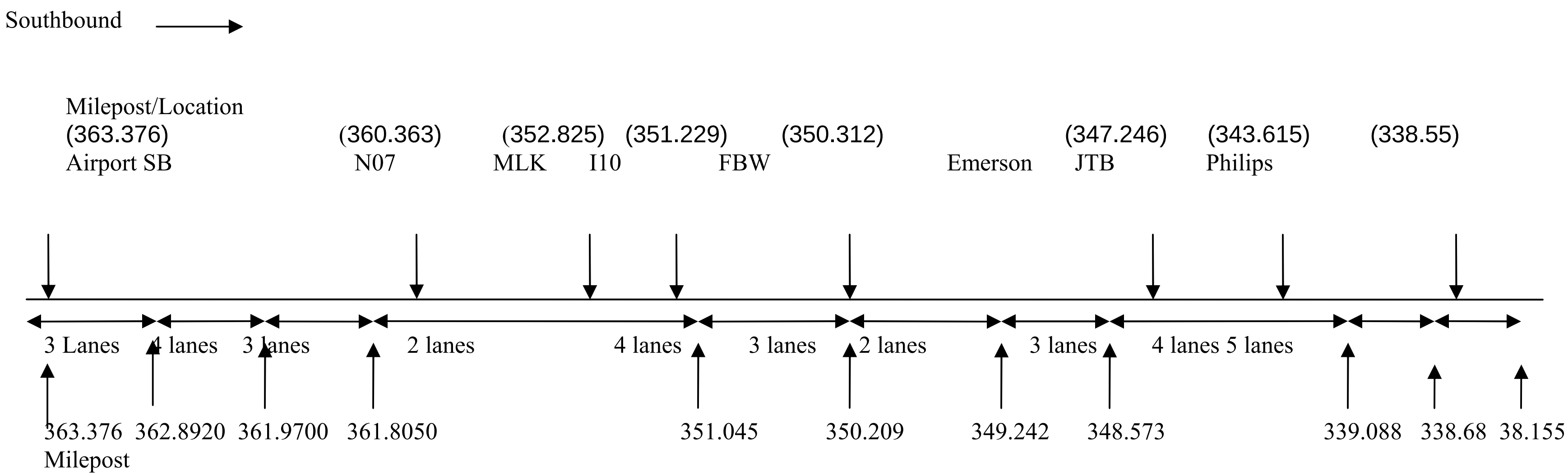

Figure 14 Number of Lanes Along I-95 in Jacksonville, FL, Southbound (Source: http://www.dot.state.fl.us/planning/statistics/gis/aboutapps.htm)

There is a total of 120 sensor locations, 60 along the northbound section and 60 along the southbound section. Data were collected by FDOT District 2 using RTMS. These data consist of volume, occupancy, average speed, and percentage of vehicle types classified based on length. The data were obtained in 5- minute intervals, from May 10th to August 31st, 2006. Due to missing data and erroneous information only data from July 15th to August 31st could be used in the analysis.

Travel time estimates were obtained for specific Origin-Destinations (O-D pairs), which were provided by District 2. These O-D pairs are shown in Table 1. The distance between any O-D pair is calculated using the milepost of the corresponding origin and destination. The distance between all the O-D pairs considered here is shown in Table 2 (the vertical axis represents the origin, while the horizontal axis represents the destination.) Figure 15 shows the general vicinity of each origin and destination. Travel time data were obtained from SunGuide for the O-D pairs presented above. These travel times are estimated using sensor data, and are not independently obtained. In essence, the link travel time is computed from the average speed and the length of the link.

Table 1 O-D Pairs

Origin	Destinations
AirportSB	MLK, Philips, I-10, JTB
N07	MLK, I-10
S08	I-10, Bowden, AirportNB
FWB	JTB, Emerson
S04	I-10, 8th

Table 2 O-D Pair Distance Matrix (in miles)

	MLK	Philips	I-10	Bowden	JTB	Airport NB	Emerson	8th
AirportSB	10.551	24.826	12.147		19.761			
N07	7.538		9.134					
S08			11.405	5.027		23.326		
FWB					6.697		3.066	
S04			0.941					2.523

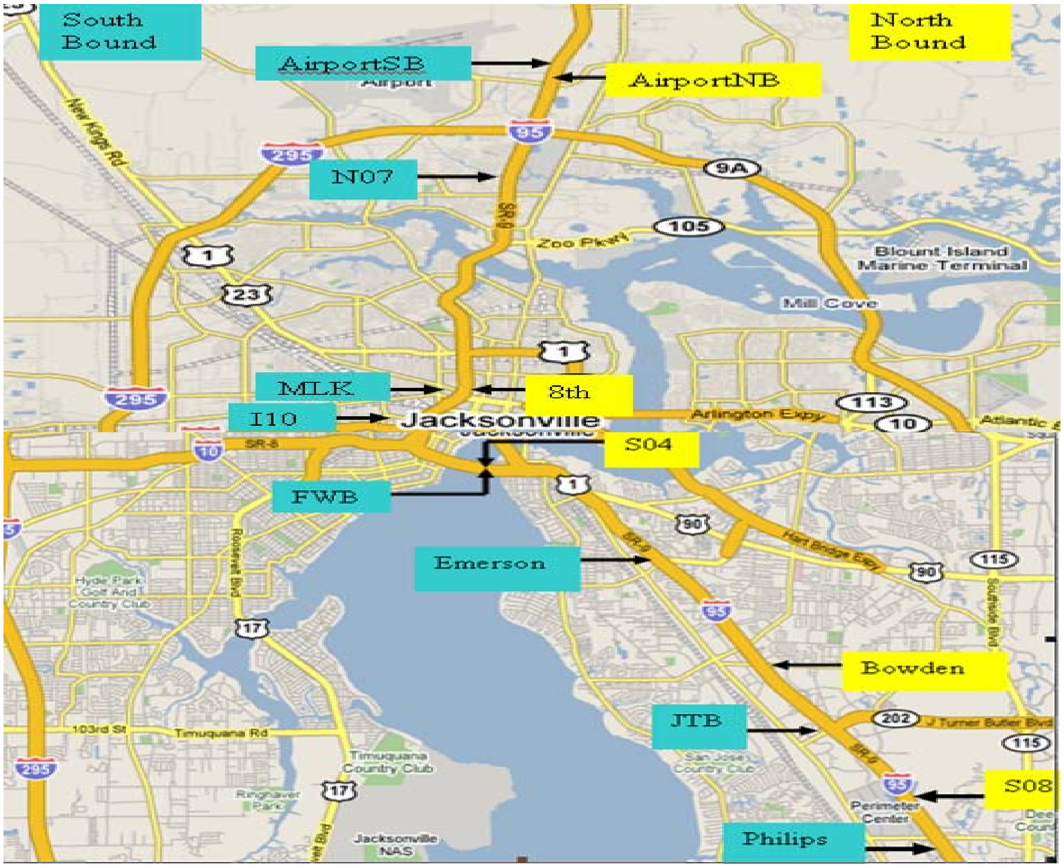

Figure 15 Origin and destination locations along the study section (Source www.googlemaps.com)

As mentioned above, preliminary analysis of the travel time data revealed that significant portions of the data required were missing. To review the data availability for the entire route, a graph of data vs. time was prepared and is shown in Figure 6. The red areas indicate missing data, and the green available data. As shown in Figure 16, only the data from June 14th To August 31st could be used for further analysis. Moreover for one O-D pair (S08 to 8th) data were available only from July 17th to August 31st.

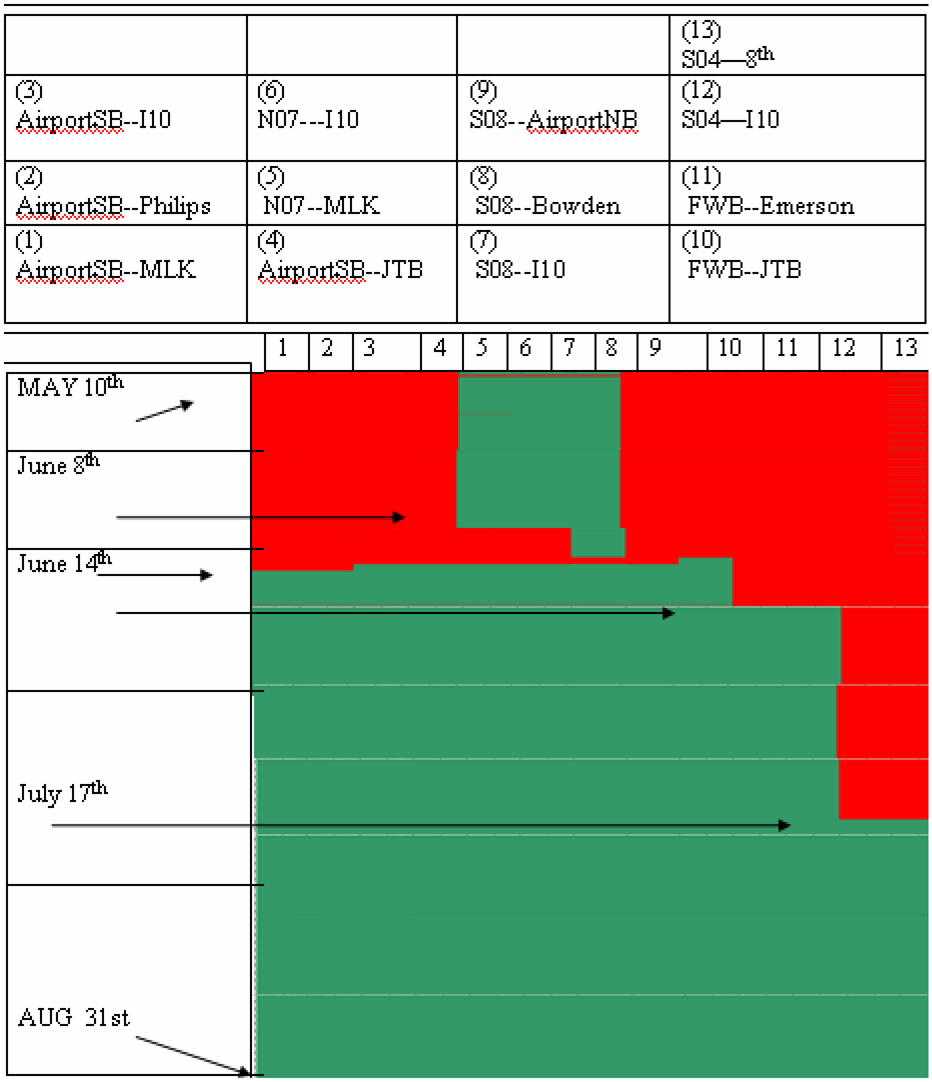

Figure 16 Data Availability

As for the analysis of the Ft. Lauderdale data, two threshold numbers of on-time arrival are considered: (1) travel time made by traveling at ten miles less than the speed limit of each link (2) travel time made by traveling at fifteen miles less than the speed limit of each link. Using the speed limit data and the distance of each link the threshold travel times are calculated and shown below in Tables 3 and 4.

Table 3 Threshold/On-Time Reliable Travel Times (Speed Limit – 10 Mph)

Travel time in minutes								
	MLK	Philips	I-10	Bowden	JTB	AirportNB	Emerson	8th
Airport SB	13.0896	30.405	15.1189		24.88			
N07	9.7229		11.7523					
S08			13.3350	5.4796		28.45		
FWB					8.116		4.1676	
S04			1.4					3.429

Table 4 Threshold/On-Time Reliable Travel Times (Speed Limit - 15 Mph)

Travel time in minutes								
	MLK	Philips	I-10	Bowden	JTB	AirportNB	Emerson	8th
AirportSB	14.630	33.9390	16.9135		27.8694			
N07	10.843		13.1260					
S08			14.772	6.0276		31.6855		
FWB					9.05357		4.7095	
S04			1.575					3.858

Figures 17 and 18 show the travel time frequencies and the travel time reliability for each of the two threshold values for the southbound route. As shown, there is a relatively high frequency of high travel times present along the study section. This shape is unusual, in that the frequencies of high travel times usually taper off as travel time increases. Similar plots were prepared for other O-Ds and the shapes are similar to those shown in Figures 17 and 18.

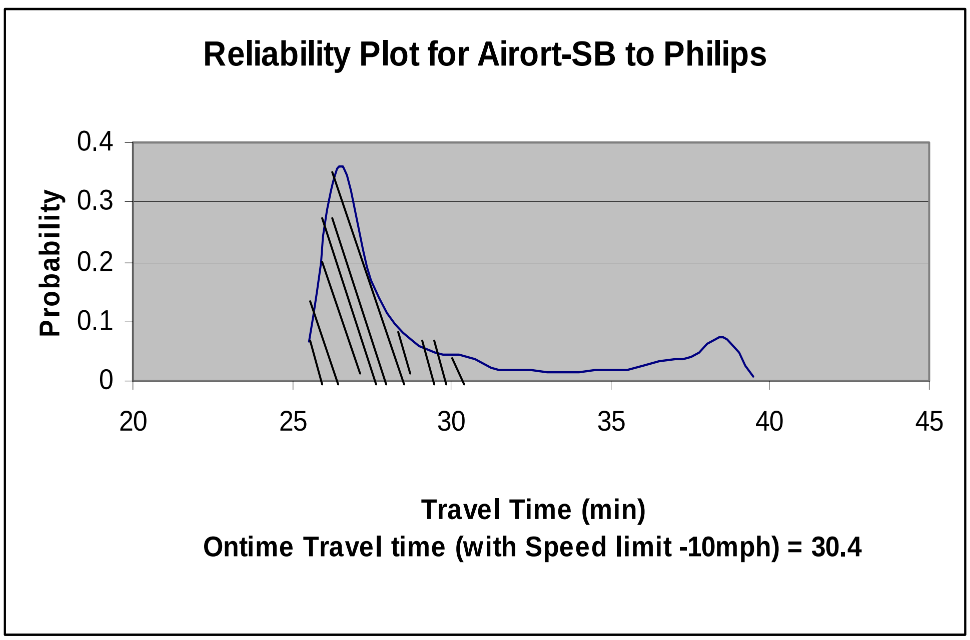

Figure 17 Reliability Plot For O-D Pair Airport-SB To Philips (Speed Limit – 10 Mph)

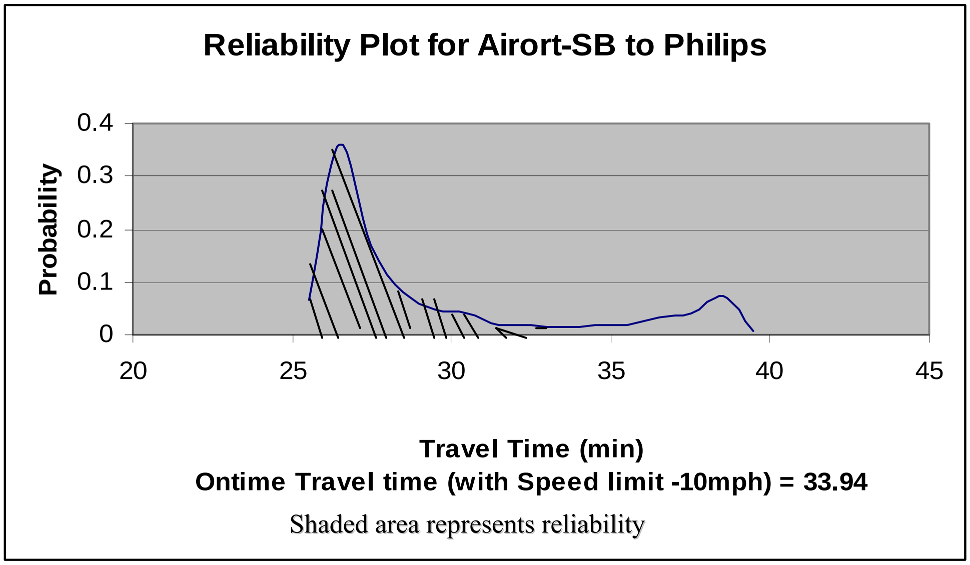

Figure 18 Reliability Plot For O-D Route Airport-SB To Philips (Speed Limit – 15 Mph)

Additional plots were constructed to evaluate the travel time reliability across the time of day. The travel time reliability estimates for the southbound section, i.e., Airport-SB to Philips, are shown in Figure 19, while those for the O-D pair N07 to MLK are shown in Figure 20. What was completely unexpected in these graphs is that the speed, and thus the reliability during the night time is much lower than that during the day time. After discussions with District 2 staff regarding the possible presence of night time work zones, it was concluded that these data are not accurate. The data showed that the speeds were low even for low flows under non-congested conditions, which was not accurate based on field observations. Therefore these data from Jacksonville cannot be used in the travel time reliability modeling. Additional data obtained after the District 2 detectors were recalibrated showed that there still is a significant number of missing observations, and thus the I-95/Jacksonville data cannot be used currently.

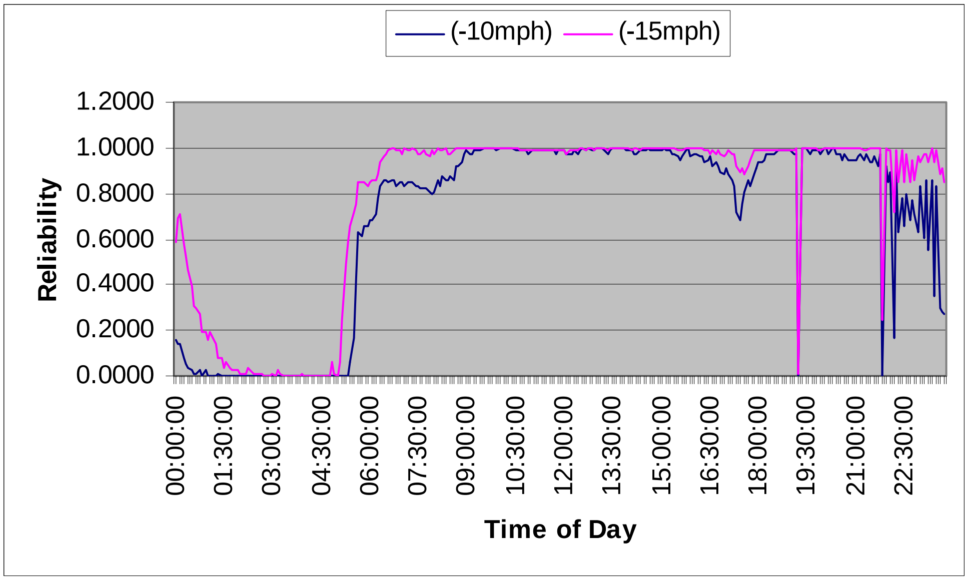

Figure 19 Reliability Plot For Airport SB To Philips

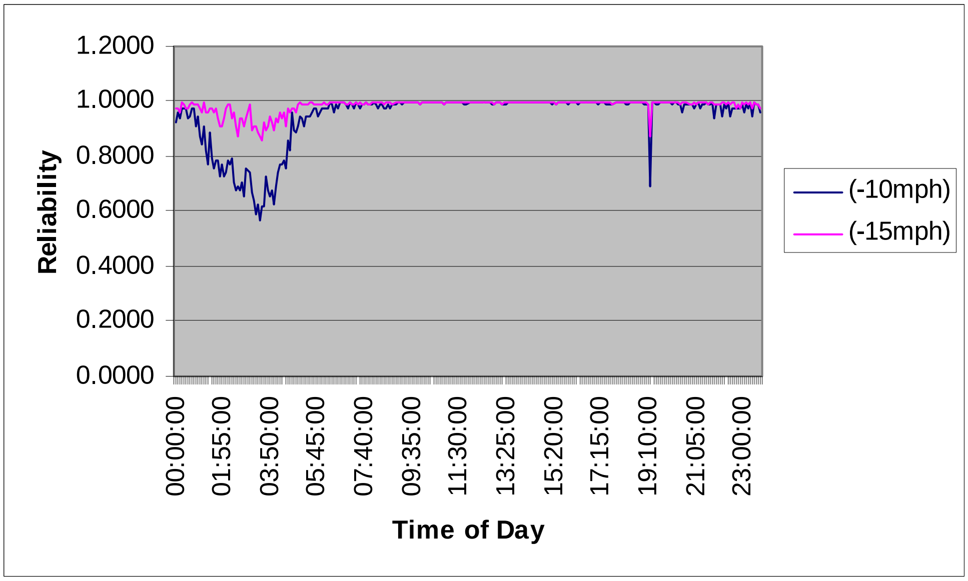

Figure 20 Reliability Plot For N07 To MLK

III. INCIDENT AND WEATHER DATA

This section of the report presents the data gathered with respect to incidents and weather for Florida.

Incident Data

Incident data documented by District 4 were obtained and analyzed to obtain frequencies and other information that can be used to estimate travel time reliability for the SIS. All the data used in this analysis was obtained from the public information section of the SMART SUNGUIDE (FDOT District 4) website (www.smartsunguide.com), in which FDOT District 4's transportation management center (TMC) programs provide ITS related information. These data provide the most comprehensive information found in Florida with respect to freeway incidents. In the subfolder titled “Performance Measures” and under “Weekly Reports,” one report per month (usually the first week) from January 2006 till May 2007 was chosen and was used in the data analysis reported here.

From each weekly report, the hourly-based incident information (calculated from the section of Road Ranger / SIRV Performance by Time of Day, as an average for the respective week) was obtained. All the data from those samples were imported into an Excel spreadsheet, by hour and month of the year. Average and standard deviation values were calculated both by hour and month of year, and a yearly total number of incidents (over 365 days) was estimated by extrapolating from the samples obtained. Table 5 provides a summary of these data, while Figures 21 and 22 show graphically the incidents by hour and by month. As shown, the level of incidents does not vary much by month or week. However, the number of incidents varies by hour, and it follows the shape of the traffic demand within a day. Finally, the yearly average and total number of incidents for each hour are calculated, by extrapolation from the weekly samples.

Table 5 Total Number Of Incidents For A Week In Each Of Those Months. (Using RR/SIRV Responses)

	Jan-06	Feb-06	Mar-06	Apr-06	May-06	Jun-06	Jul-06	Aug-06	Sep-06	Oct-06	Nov-06	Dec-06	Jan-07	Feb-07	Mar-07	Apr-07	May-07	Sum	Average	Yearly Est
12AM-1AM	20	24	15	24	26	13	23	26	13	14	21	15	20	15	7	28	18	322	18.9	988
1AM-2AM	18	14	13	13	13	13	13	18	14	15	16	11	23	8	10	9	16	237	13.9	727
2AM-3AM	9	12	11	7	13	7	22	7	13	5	14	7	7	3	11	5	13	166	9.8	509
3AM-4AM	16	4	8	13	18	6	15	11	16	11	2	12	5	18	3	11	9	178	10.5	546
4AM-5AM	17	13	10	15	25	20	14	20	35	31	20	22	18	12	8	10	12	302	17.8	926
5AM-6AM	14	16	15	16	12	10	20	8	10	25	10	19	17	9	6	11	16	234	13.8	718
6AM-7AM	17	40	44	36	37	52	51	56	51	33	53	61	69	37	59	33	62	791	46.5	2426
7AM-8AM	51	51	68	91	77	85	74	78	86	94	80	99	70	81	74	67	75	1301	76.5	3990
8AM-9AM	53	71	77	69	74	76	76	72	80	89	75	91	78	68	72	72	65	1258	74	3859
9AM-10AM	41	58	56	83	75	67	69	69	80	92	73	79	67	71	60	66	83	1189	69.9	3647
10AM-11AM	47	56	53	45	66	74	49	55	48	68	55	57	78	72	60	57	66	1006	59.2	3086
11AM-12PM	46	55	65	51	59	64	62	63	49	69	54	59	71	46	64	44	59	980	57.6	3006
12PM-1PM	28	54	36	61	48	70	52	54	54	58	61	49	69	66	48	38	65	911	53.6	2794
1PM-2PM	34	41	42	68	55	57	49	68	65	69	45	53	50	67	55	47	56	921	54.2	2825
2PM-3PM	41	59	48	65	36	61	47	56	62	65	52	40	66	52	57	64	65	936	55.1	2871
3PM-4PM	42	49	53	57	61	64	68	75	65	77	70	58	55	44	55	45	65	1003	59	3076
4PM-5PM	41	44	77	67	68	74	58	88	62	69	68	68	60	64	59	70	71	1108	65.2	3398
5PM-6PM	47	50	59	76	51	61	60	62	63	90	61	79	56	56	63	62	58	1054	62	3233
6PM-7PM	22	39	39	47	37	50	34	52	53	64	42	43	42	39	39	46	71	759	44.6	2328
7PM-8PM	29	30	35	32	49	32	42	45	38	30	40	38	33	22	25	47	44	611	35.9	1874
8PM-9PM	34	41	39	52	31	31	27	41	43	33	29	29	36	26	28	29	33	582	34.2	1785
9PM-10PM	27	32	26	28	26	28	16	17	23	30	20	22	19	28	23	18	24	407	23.9	1248
10PM-11PM	27	18	22	34	21	20	24	14	21	31	27	34	28	27	20	29	37	434	25.5	1331
11PM-12AM	15	28	24	34	35	33	21	29	30	16	21	26	21	16	15	35	31	430	25.3	1319
Summation	736	899	935	1084	1013	1068	986	1084	1074	1178	1009	1071	1058	947	921	943	1114			

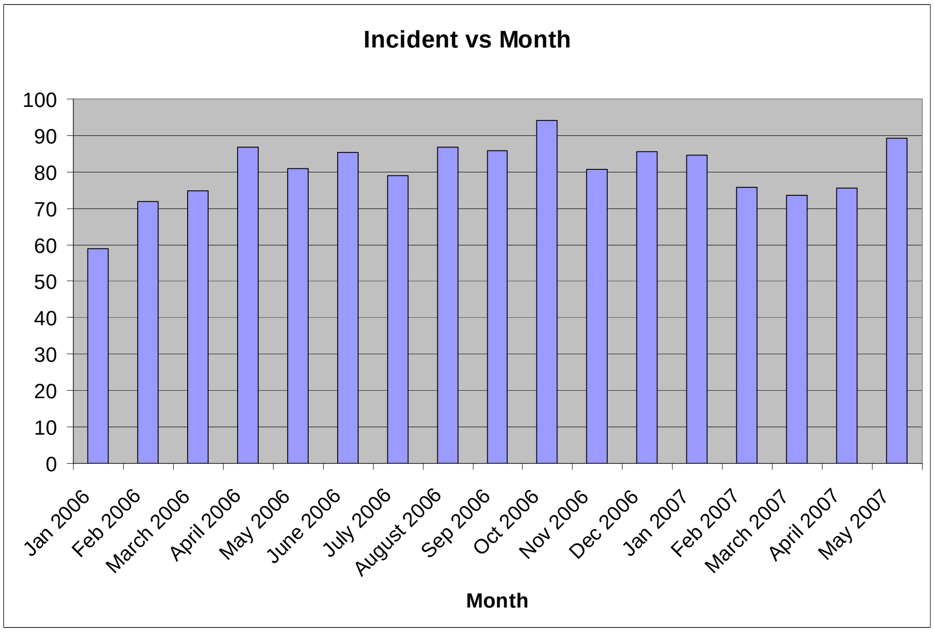

Figure 21 Total Number Of Incidents By Month For A Week In Each Of Those Months. (Using RR/SIRV Responses)

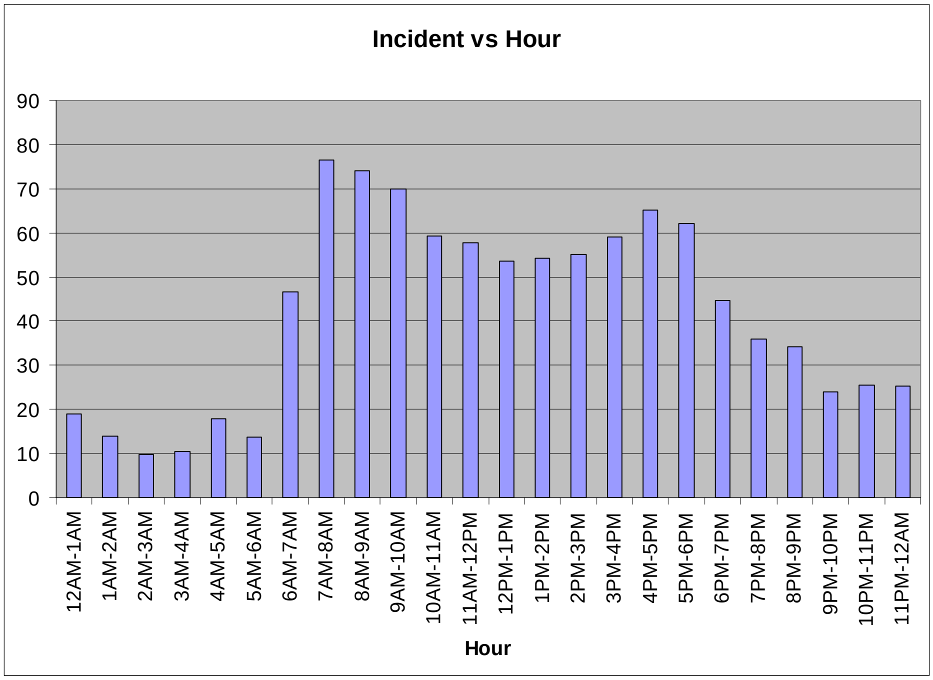

Figure 22 Total Number Of Incidents By Hour For A Week In Each Of Those Months. (Using RR/SIRV Responses)

Table 6 summarizes the length of each facility in the system along with the number of lanes (range), and provides the total lane miles for the entire system monitored. That number is then used to estimate the incidents per lane mile for each month, and by hour of the day, in Figures 23 and 24.

Table 6 Facilities Length, Number Of Lanes, And Incidents Per Lane Mile

Facility	Area Covered	Length (Miles)	Number of Lanes (Range)	Total Lane Miles
SR-84 (excluding the part shared with I-595)	Between Eisenhower Blvd and junction of SR-84 and I-595	4.3	3	12.9
I-595	Between Federal Hwy and Sawgrass Expy	12.7	4 - 5	62.512
I-95	Between W. Palmetto Park Rd and W. Hallandale Beach Blvd	25.307	4 - 6	123.473
I-75	Between Palmetto Hwy and Sawgrass Expy	18.734	4 - 5	92.758
TOTAL		61.041		291.643

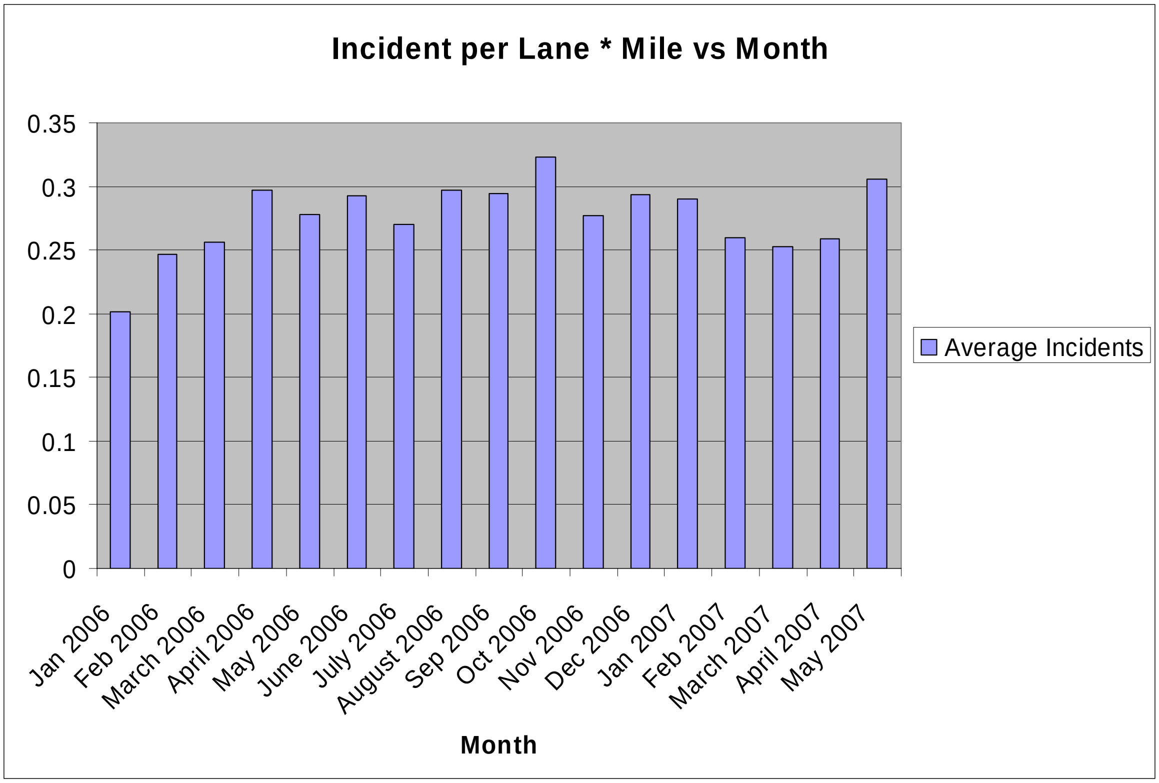

Figure 23 System-Wide Number Of Incidents Per Lane Mile By Month For A Week In Each Of Those Months

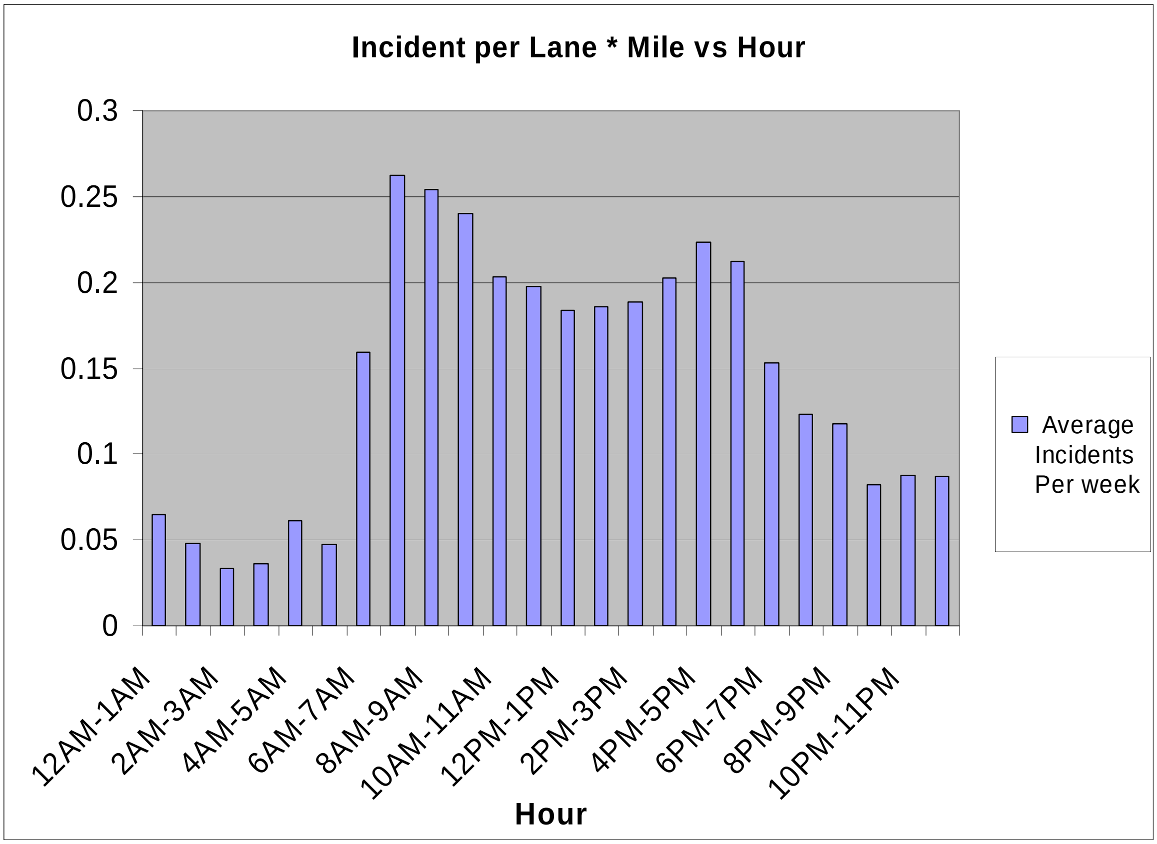

Figure 24 System-Wide Number Of Incidents Per Lane*Mile By Hour For A Week In Each Of Those Months.

From the same weekly reports the monthly-based incident blockage information (calculated from the section of Performance Measures by Severity) and incident severity, were also obtained. The level of blockage for each incident is divided by the SMART SUNGUIDE into four categories: Level 1 (Any lane blocked for less than 30 minutes), Level 2 (Any lane blocked between 30 to 120 minutes), Level 3 (ALL lanes blocked for any period of time, or individual lanes blocked more than 120 minutes), and Other (Non-blocking). The ratio of the number of incidents with no lane blocked to the total number of incidents is calculated and compared for each weekly set of data. Since there are no hourly data provided for each of these levels, the data are analyzed by month only. The annual number of incidents for each of those four levels was also estimated by extrapolation. Table 7 includes the total number of incidents by month and by level, while Figure 25 plots the number of incidents by month, and by blockage level (at least one lane blocked vs. no lanes blocked). As shown, there are no significant trends in the blockage vs. season or month.

Table 7 Number Of Incidents By Level)

	At least one lane blocked				No lane blocked
	Level 1	Level 2	Level 3	Sum	Other
Jan-06	12	11	16	39	663
Feb-06	20	38	18	76	791
Mar-06	21	22	14	57	842
Apr-06	23	38	8	69	974
May-06	25	22	43	90	874
Jun-06	32	19	15	66	964
Jul-06	30	21	11	62	891
Aug-06	28	44	10	82	962
Sep-06	30	25	7	62	970
Oct-06	40	44	14	98	1041
Nov-06	39	34	7	80	891
Dec-06	28	35	4	67	957
Jan-07	28	29	9	66	954
Feb-07	29	42	13	84	815
Mar-07	31	9	9	49	843
Apr-07	29	38	15	82	814
May-07	40	34	27	101	957
Sum	485	505	240	1230	15203
Avg	28.5	29.7	14.1	72.4	894.3
Yearly EST	1487.6	1549	736.1	3772.7	46631.1

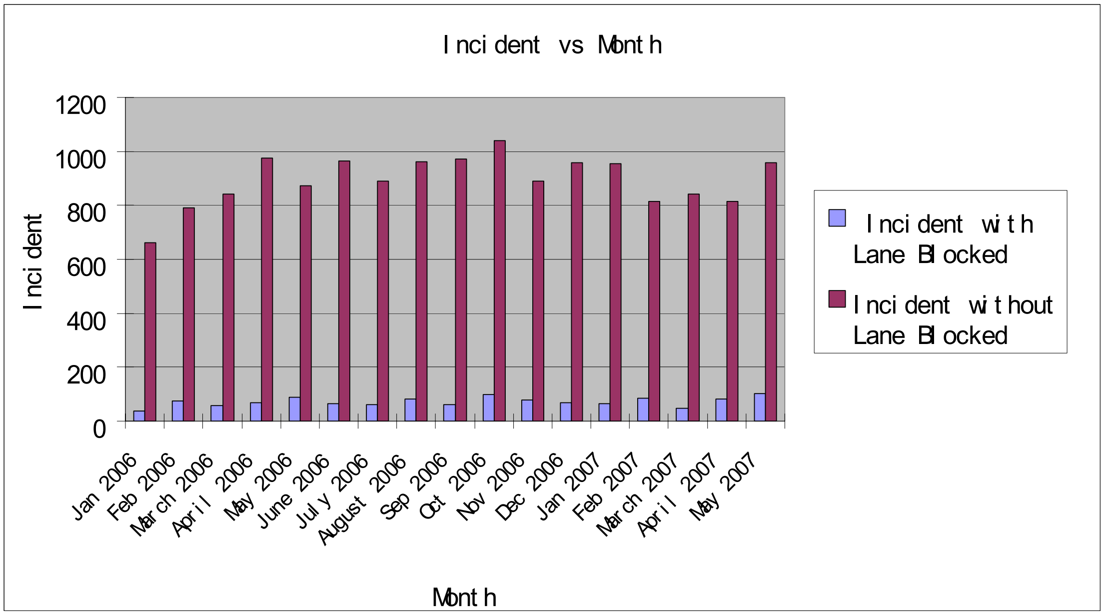

Figure 25 Number Of Incidents By Level

Incident duration data were obtained from the weekly reports based on the values for Roadway Clearance by Maximum Number of Lanes Blocked. The average duration of incidents by maximum number of lanes closed during that incident is obtained again on a weekly basis for one week per month. There are seven categories of the maximum number of lanes closed during that incident (0, 1, 2, 3, 4, 5, and 6). There is no information provided in the weekly reports regarding the frequency of those occurrences and blockages. Averages were calculated for each type of blockage within a year. Table 8 presents the average incident duration as a function of the maximum number of lanes closed during the incident, while Figure 26 tabulates the same information.

Table 8 Average Duration Of Incidents (In Min) By Maximum Number Of Lanes Closed During That Incident

Lane Blocked	0	1	2	3	4	5	6	Avg (min)
Jan-06	0	34.7	35.6	124.3	101.3	0	0	74
Feb-06	0	55.2	64.1	141.1	72.7	267.5	0	120.1
Mar-06	0	61.7	36.5	189.3	30.1	49.9	0	73.5
Apr-06	0	32.6	35	67.7	50.6	49	0	47
May-06	0	115	55.7	90.4	66.3	164.9	0	98.5
Jun-06	0	18	36.3	77	80.9	0	0	53
Jul-06	0	27.5	36.6	130.3	0	0	0	64.8
Aug-06	0	27.1	52.1	41.9	83.7	0	0	51.2
Sep-06	0	33	39.2	49	0	148	0	67.3
Oct-06	0	28.3	58.8	51.5	59.7	166.3	0	72.9
Nov-06	0	27.5	39.4	84.5	0	0	0	50.4
Dec-06	0	24.2	39.8	104.9	35.2	3.4	0	41.5
Jan-07	0	22.7	34.1	47.1	0	74.3	0	44.5
Feb-07	0	35.4	41.7	109.3	23.3	0	0	52.4
Mar-07	10.6	24.7	30.1	45.5	0	34.5	0	24.2
Apr-07	0	35.3	48.1	36	204.6	0	0	81
May-07	0	22.3	47.2	92.4	84.4	37.7	0	56.8
Avg	0.6	36.8	42.9	87.2	52.5	58.6	0	63.1

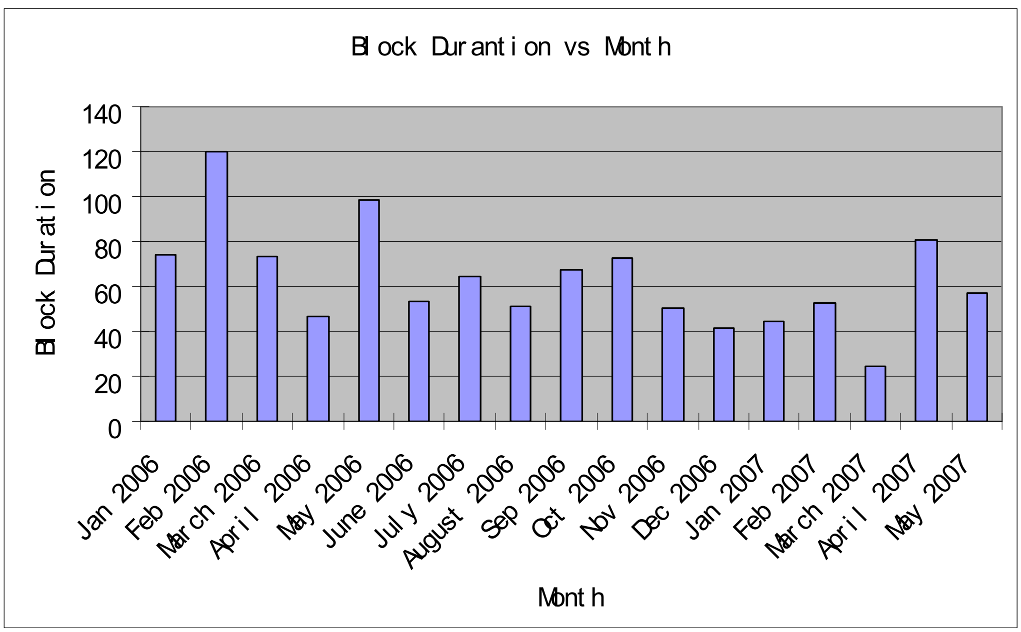

Figure 26 Average Duration Of Incidents (In Min) By Maximum Number Of Lanes Closed During That Incident

Weather Data

Precipitation data from January 1st, 2006 until December 31st, 2006 were extracted from the website: www.wunderground.com, for the observation station in Pompano Highlands (33064, Lat N 26 ° 17 ' 9 ", Lon W 80 ° 6 ' 9 "), which is in the vicinity of the Ft. Lauderdale target area (zip code 33303). These data provide precipitation every 10 or 15 minutes. Three days (usually the 1st, 11th, 21st of each month, or adjacent days when these were not available) per month were sampled, and precipitation was aggregated in average hourly rates over those days. The 24-hour aggregated data (one data point for each hour) for the three days per month are averaged to estimate the mean 24-hour precipitation for each month. Thus we obtained the average precipitation by hour of the day, for each month of the year. These results were aggregated to provide the average hourly precipitation for the time periods 6:00-10:00AM, 10:00AM-2:00PM, 2:00-6:00PM and 6:00PM-6:00AM. These data are provided in Table 9, while the hourly precipitation probability weighted by the three-day sample of each of those 12 months are shown in Table 10.

Table 9 Average Precipitation Probability By Time and Month

	6:00 am - 10:00 am	10:00 am - 2:00 pm	2:00 pm -6:00 pm	6:00 pm – 6:00 am	Daily EST
6-Jan	0.00	0.00	0.00	0.00	0.00
6-Feb	0.00	0.00	0.00	0.00	0.00
6-Mar	0.00	0.00	0.00	0.00	0.00
6-Apr	0.00	0.00	0.00	0.00	0.00
6-May	0.00	0.00	0.00	0.00	0.00
6-Jun	0.00	0.00	0.17	0.00	0.03
6-Jul	0.00	0.00	0.00	0.06	0.03
6-Aug	0.00	0.00	0.00	0.14	0.07
6-Sep	0.00	0.00	0.08	0.11	0. 07
6-Oct	0.00	0.00	0.00	0.00	0.00
6-Nov	0.17	0.00	0.00	0.00	0.03
6-Dec	0.00	0.00	0.00	0.00	0.00
Yearly EST	0.01	0.00	0.02	0.03	

Table 10 Average Hourly Precipitation Probability

Hour	Precipitation Probability
0 - 1	0.03
1 - 2	0.03
2 - 3	0.03
3 - 4	0.03
4 - 5	0.03
5 - 6	0.00
6 - 7	0.00
7 - 8	0.03
8 - 9	0.03
9 - 10	0.00
10 - 11	0.00
11 - 12	0.00
12 - 13	0.00
13 - 14	0.00
14 - 15	0.03
15 - 16	0.03
16 - 17	0.00
17 - 18	0.03
18 - 19	0.03
19 - 20	0.03
20 - 21	0.06
21 - 22	0.06
22 - 23	0.00
23 - 24	0.00

Since there is only one available weather station around the target area, these estimates may not reflect the exact conditions on the freeway. Furthermore, choosing 3 days per month as sampling data still can lead to biased aggregation. Missing data (substitute day needed) may compound the problem.

There are 209 weather stations similar to the one discussed across Florida, therefore weather information can be obtained for various locations across Florida.

IV. USE OF FDOT DATABASES TO REPORT TRAVEL TIME RELIABILITY ON FREEWAYS

The objective of this task is to calculate travel time reliability on Florida freeways by utilizing the data available in existing FDOT databases. To complete this task, first the research team evaluated the availability of the specific data required by the models previously developed. The team met with FDOT personnel (Doug McLeod and Gordon Morgan) on January 11th, 2007, and also received input from Kittelson and Associates.

Table 11 provides a list of the data required by the models previously developed, for each analysis time period. Table 12 provides a preliminary list of pertinent data that are available in various FDOT databases and have currently been assembled by Kittelson and Associates. As can be seen from these two tables, some of the data in the existing databases can be applied in the travel time estimation. There are still several data items that cannot be obtained from the existing databases (for example, the number of on-ramps, number of off-ramps, congestion level, etc.)

It is possible that additional databases are available through other offices of FDOT (for example, the safety office may have incident-related road closure information), which can be integrated into these existing databases. Also, it is possible that for data that are not available, reasonable assumptions can be made after consulting with the FDOT technical review team.

Table 11 Basic Data Required for Travel Time Estimation for Each Analysis Time Period

Column ID	Columns	Data Types	Description
1	Number of Lanes	Integer	This column records the number of lanes for the observed facility segment.
2	Number of On-Ramps	Integer	This column records the number of on-ramps for the current segment of freeway.
3	Number of Off-Ramps	Integer	This column records the number of off-ramps for the current segment of freeway.
4	Congestion Level	Integer	This column identifies the congestion level for the observed time period. For travel time estimation purposes, the parameter has been categorized into three levels: Non-congested, semi-congested, and fully-congested.
5	Weather Condition	Text or integer	This column identifies the weather conditions for the observed time period. For travel time estimation purposes, the parameter has been categorized into two levels: Rain and No Rain.
6	Incident Level	Text or Integer	This column identifies the incident level for the observed time period. For travel time estimation purposes, the parameter has been categorized into two levels: Incident and No Incident.
7	Workzone	Text or Integer	This column identifies the workzone situation for the observed time period. For travel time estimation purposes, the parameter has been categorized into two levels: Work zone and No Incident.
8	Precipitation	Double	This column records the amount of rain, if any, for the subject time period.
9	Number of lanes closed due to construction	Integer	This column records the number of lanes closed for the work zone, if any.
10	Number of lanes closed due to incident	Integer	This column records the number of lanes closed due to an incident, if any.
11	Hourly flow per lane	Double	This column records the flow rate per lane.
12	Speed	Double	This column records the operating speed for the specific facility segment

Table 12 Data Items Available In FDOT Databases (Source: Kittelson And Associates)

Column ID	Columns	Data Types	Description
1	FACILITY	text	facility name
2	ROADWAY	integer	ID of a facility segment
3	BEGIN_POST	float	start station
4	END_POST	float	end station
5	ATYPE	text	Area Type: urban or rural
6	FTYPE	text	Facility Type: arterial, freeway or other.
7	CTYPE	text	Classification: I, II, III
8	LANES05	integer	Number of lanes
9	AADT05	integer	AADT
10	LOS05	text	Level of Service

V. LITERATURE REVIEW ON ARTERIAL TRAVEL TIME

There are three potential methods for estimating arterial travel time: a) measure it directly, b) develop a theoretical model; c) simulate the geometric characteristics and traffic conditions in a simulation package, and then obtain the travel time. This section reviews articles related to each of these three general methods.

Measurement of Arterial Travel Time

Several methods have been used in the literature (Sen et.al, 1996) to measure arterial travel time:

1. Probe vehicle: Observer in the vehicle records travel time, and other information throughout the trip.
2. Observation through video technology: Observer matches license plates, or vehicle type, at an origin and a destination.
3. Loop detectors: Spot speed converted to travel time.

Each of these methods has distinct advantages and disadvantages. When using the probe vehicle method, one can obtain detailed information about the entire route, including number of stops, causes of stops, etc. The sample size obtained is limited, and the method is relatively time-consuming and expensive. With video technology, one can obtain a larger sample, relatively quickly. Identifying a specific vehicle at the destination point manually can be tedious, and previous studies have shown that trucks may be favored since they are more easily identifiable. Also, if there are several intersections between the origin destination, there is a high probability that a given vehicle will “disappear” to a different exit point. Loop detectors can provide a very large sample; however, they need to be closely spaced (1/3 to 1/2 miles apart) for the travel time estimate to be accurate.

Quiroga and Bullock (1999) used GPS data to measure intersection control delay and evaluated the accuracy of new intersection delay models. With a GPS device, control delay can be calculated as:

$$D = TravelTime - \frac{Link\_Length}{S_f}$$

The paper compared the control delay measured in the field to the control delay calculated by the HCM method. The authors concluded that the deceleration and acceleration lengths were much longer than usually anticipated so that the part of the control delay that occurred during acceleration and deceleration is not negligible.

Modeling of Arterial Travel Time

Several methods have been developed for estimating arterial travel time. In general, arterial travel time can be decomposed into two parts: vehicles' link travel time, and intersection delay. Previous research efforts have typically estimated these two parts separately. In some studies, link travel time has been assumed to be constant and not affected by signalization factors. In all studies, intersection delay estimation is essential in determining travel time along the arterial. There are several methods for estimating signalized intersection delays: the HCM intersection delay model, shockwave analysis, deterministic queuing analysis, and stochastic queuing analysis. This section briefly discusses signalized intersection delay estimation, and arterial travel time modeling and estimation.

Signalized Intersection Delay Estimation Methods

In the US, the HCM delay calculation has been used as the standard delay estimation for intersections. The HCM chapter 16 presents the estimation method for isolated intersection delay. The model is based on Webster's (1958) equation.

Benekohal and Kim (2005) indicated that for oversaturated conditions, the HCM 2000 model does not apply the progression adjustment factor (PF) when there is an initial queue. The problem caused is that the delay with an initial queue is sometimes less than the delay without an initial queue. Also, the model yields the same uniform delay values for all arrival types when there is an initial queue. This paper proposed a new model for estimating uniform delay for oversaturated conditions. The authors concluded that the new model gives a better estimation of delay then the HCM 2000 delay model.

Ahmed and Abu-Lebdeh (2005) proposed a model for estimating delay at signalized intersections that is caused by a downstream traffic disturbance such as queue spillback. The authors suggested adding one more delay term in the HCM 2000 delay model to represent the downstream induced delay, which is called d_4. They developed an equation for d_4, which considers the downstream queue length, the availability of space downstream, and the speed on the link. The authors used CORSIM and the HCM 2000 delay model to obtain the intersection delay for queue spillback conditions and they concluded that the new delay model can estimate delay well when compared to CORSIM.

Arterial Travel Time Estimation Methods

In the HCM 2000 Chapter 15, travel time on arterials is estimated by the following equation:

$$T.T. = \frac{L}{S_A} + intersection\_delay$$

$$S_A = \frac{3600L}{T_R + d}$$

Where:

L : Link length between two intersections

S_A: Average speed along the arterial

T_R: running time

d : Control delay for through movements

$intersection\_delay$: Isolated intersection delay

The basic method in the HCM is to divide travel time into two parts; one is the actual running time on the link. The average speed can be found in a table provided in the HCM as a function of the link length and the free flow speed. The second part is the delay encountered at intersections. The HCM uses the isolated intersection delay model to represent the delay vehicles encounter along the arterial. This approach has significant shortcomings. First, the equation does not consider demand flow. Second, the operating speed changes over time, and it cannot be accurately estimated by the average speed. Third, the isolated intersection delay is not the actual delay that vehicles encounter at intersections.

The National Institute of Statistical Science (NISS) performed two consecutive studies on arterial travel time; these studies were completed on year 1996 and 1998. In 1996, Sen, et al. conducted a travel time study which presents a comparison of some arterial travel time models based on data from probe vehicles and loop detectors. The authors suggested that the estimated travel time is the sum of cruise time and stopped delay. Cruise time is the period of time when a vehicle is moving, stopped delay is the period of time when a vehicle stops at a red signal or is slowly moving in a queue. Their conclusion was that variation in travel time mostly depends on variation in stopped delay.

The second study from NISS was performed by Graves et al. (1998) as an extension of the previous one. In this study, the authors focused on travel time variability and they performed the study on a selected arterial in a Chicago suburban area. Travel time data were collected with a probe vehicle and loop detectors. The findings are that travel time variability relies on the signal status and the traffic volume when a particular vehicle enters the link. This paper presented a new variable in estimating arterial travel time: Relative Entry Time (RET). RET is related to the status of the downstream signal. In the paper, they developed a general travel time model and then adjusted the model for through, left turning, and right turning movements. Factors considered in travel time estimation are red time, green time, free flow travel time, time for each vehicle to clear the queue, and traffic volume.

Lum and Fan (1998) studied the speed-density relationship for arterials in Singapore and proposed a travel time-density model for interrupted flow. This method can be used for estimating travel time. They collected data on traffic volume and travel time to build the revised speed-density relationship. Since an arterial has interrupted flow, signals can affect the speed-density relationship. To account for these, the number of intersections along the arterial is incorporated into the relationship.

Zhang (1999) developed a model to estimate the journey speed along an arterial; the idea is if the speed can be estimated correctly, arterial travel time can be determined as the link length divided by the speed. In his model, detectors provide occupancy, which in turn is used to estimate journey speed. The model must be calibrated to predict speeds at a given location. The

"Illinois Model" (1999) presents a regression model for delay estimation, where two of the regression parameters are also based on data from detectors.

Fu and Hellinga (2000) studied the variability of delays at signalized intersections. They suggest delays that individual vehicles experienced at signalized intersections are subject to variations caused by random arrival and interruption from signal control. In this study, delay was defined as the sum of uniform delay and overflow delay. The variation of total delay is the sum of the variation of uniform delay and overflow delay. They concluded that there are no analytical models to account for these variations, which can be very important in estimating delays at intersections.

Xie and Cheu (2001) proposed a model to estimate travel time based on arterial link travel time, and signal delay. The model was tested with field data and the authors concluded that the model is more accurate than previous models.

Lin, Kulkarni, and Mirchandani (2003) developed a model to estimate arterial travel time. In the model, they reduced travel time estimation to intersection delay estimation: in their paper, travel time is first defined as the sum of link travel time and intersection delay. They concluded that link travel time is not so sensitive to the traffic flow when the flow is medium or high. They suggested that link travel time should be estimated as a constant value. This method is similar to urban street travel time estimation in the HCM in that link travel time is not considered as a variable. However, in this paper, the authors suggested that delay occurs with a certain probability, which is not the same for all the vehicles. They developed a transition matrix to determine what is the probability that a vehicle is in state i if its previous state is j. There are two states defined in this study: 0 and 1: 0 represents the condition where the vehicle is not delayed at the intersection i, while 1 represents the condition where the vehicle is delayed at the intersection i. Therefore a 2×2 matrix is generated as follows:

$$P = \begin{bmatrix} p_{00}^{i} & 1 - p_{00}^{i} \\ 1 - p_{11}^{i} & p_{11}^{i} \end{bmatrix}$$

Delay at intersections is calculated by using the conditional probability and Webster's delay equation. This paper is an improvement in estimating arterial travel time since not all vehicles encounter intersection delay, but delay occurs with a certain probability. The transition matrix would be more realistic if the paper considered signal timing, offset, traffic flow, speed, and arrival type when it calculated conditional probabilities. The paper applied the model in four simulated scenarios. They concluded that the results are promising but need to be verified with field data.

Liu and Shuldiner (2004) predicted travel time by the time series method. They collected field data on Massachusetts Route 9 for about 3.7 miles along an arterial with the aid of AVI. With the AVI system, two cameras were used at the beginning and ending points of the road segment. Cameras can capture the license plate of vehicles, and when the two plates match, one particular travel time data point is generated. Three time series models were developed: Auto Regressive Integrated Moving Average (ARIMA), ARIMA with traffic volumes as additional input variables, and Autoregressive Error Model. The three models were tested and they proved to be reliable in predicting near-future travel time; however, these models have shortcoming in that they can only predict travel time under non-congested, no-incident conditions. When the travel time increases greatly and rapidly in a short period of time, which means there is congestion, these models are not good predictors.

Pueboobpaphan et al. (2005) estimated travel time by modifying the conventional shockwave diagram so that it can present the traffic conditions more realistically. For example, the conventional shockwave analysis assumes that vehicles accelerate instantly from 0 to x mph. The authors suggested that delay at intersections would be estimated more accurately by modeling the speed changes over time. They modified the shockwave diagram and compared their results with simulated data. The results proved that the new model is more accurate. This paper improved the estimation of speed changes around intersections, but in this paper it is assumed that vehicles move with constant speed on the link.

Mark, Sadek, and Dickason (2005) applied Artificial Neural Networks (ANN) methods in predicting arterial travel time. The study tested the possibility for using the ANN method based on detector data. The results show that ANN is quite applicable in predicting arterial travel time.

Geroliminis and Skabardonis (2005) used Markov Decision Process (MDP) to predict arrival profiles and queue length along signalized arterials. Markov Decision Process (MDP) was used to model two consecutive traffic signals. They indicated that arterial travel time is greatly affected by platoon size and dispersion at intersections. Platoon size and dispersion can help to establish the relationships between two consecutive intersections. If the relationship can be established, then the arrival profile at the downstream intersection can be predicted by the entering flow at the first upstream intersection. This approach can predict the arrival profile at downstream intersections, and the method can be applied to estimate travel time. The model works well for undersaturated conditions, but it cannot predict well when conditions are saturated.

Simulation Methods in Arterial Travel Time Estimation

Simulation packages are often used in arterial analysis in determining intersection delay and travel time. Each package uses its own assumptions regarding delay estimation, thus the results from various simulators cannot be easily compared. Most micro-simulators provide travel time as an output, thus they are a useful tool in travel time estimation. As previously discussed, several articles used simulation as a validation tool, in lieu of field data.

Summary

As shown in several studies, arterial travel time arterial travel time can be decomposed into two parts: vehicles' link travel time, and intersection delay. Because of the importance of intersection delay, some of the studies focused on intersection delay when studying arterial travel time so that they may simplify travel time estimation to intersection delay estimation. One of the studies even concluded that link travel time is not so sensitive to the traffic flow when the flow is medium or high. Some studies have indicated that the time a vehicle arrives during the cycle impacts its delay, and thus each vehicle experiences a different delay. In essence, arterial delay and travel time can be represented by a distribution. This variability in travel times, and the resultant distribution should be considered when developing travel time reliability models.

Appendix A presents a preliminary framework for estimating travel time on arterials. This framework will need to be refined and simplified for use in estimating travel time on SIS arterial facilities.

V. CONCLUSIONS AND RECOMMENDATIONS

The research summarized in this report focuses on a) modifying and adjusting the previously developed models for freeways with Florida data; b) using FDOT databases to report travel time reliability for freeways throughout Florida; and c) developing a preliminary framework for reporting travel time reliability on arterials. The report presented the data obtained from Florida and the analyses conducted to estimate travel time reliability, including flow and speed data, as well as incident and weather information. The database requirements for obtaining travel time reliability in Florida were provided, along with an assessment of existing FDOT databases for estimating travel time reliability for the SIS. The report also provides a literature review on arterial travel time estimation, along with a preliminary framework for estimating travel time on arterials.

The following are concluded from the research:

- There are several systems being installed in Florida for monitoring various performance measures, including speeds, travel times and incidents. These systems are still being tested and calibrated, therefore this research could not use field data from Florida to recalibrate the existing models developed under FDOT Contract BD-545, RPWO #48.
- The incident information data obtained from District 4 appear to be of good quality; however, there are some inconsistencies observed in the reporting for certain performance measures.
- Weather information can be obtained relatively easily across Florida; however, the analysis can be time-consuming depending on the sampling interval in terms of time, as well as in terms of space (i.e., the number of stations examined across Florida).

The following are recommendations that resulted from this research:

- The existing data collection systems in Florida should be refined and data should begin to be archived as soon as possible for use in travel time reliability estimation, as well as for other uses.

- It is possible to use the incident data from District 4 to extrapolate to incident occurrences across the SIS. Such data should be recorded and obtained from each district to be able to accurately report on reliability across the SIS.
- The preliminary framework developed for estimating arterial travel time should be simplified so that it can be easily incorporated into the travel time reliability estimation databases, and travel time can be estimated for arterials with data that are typically available from the districts.

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APPENDIX A. GENERAL FRAMEWORK FOR ESTIMATING TRAVEL TIME ON ARTERIALS

This appendix provides an overview of a methodology developed for estimating travel time on arterials. This methodology is based on Xiao Cui's dissertation "Travel Time Estimation for Signalized Arterials Using Probabilistic Modeling" (2006).

The methodology is based on the premise that travel time can be considered as the sum of "travel time in motion" and "time in queue". Time in queue includes "waiting time in queue" and "moving time in queue". These three parts of travel time need to be estimated separately and then combined together to estimate the arterial travel time.

1. The Base Case: Travel Time on a Link

This section presents the estimation of travel time on a single link. The methodology can later be expanded to include a series of links. This relatively simple case illustrates travel time estimation for vehicles encountering different conditions along a link (e.g., presence of queue, green signal, red signal, etc.). Figure 1-1 provides a sketch of a link, along with several variables to be used in travel time estimation.

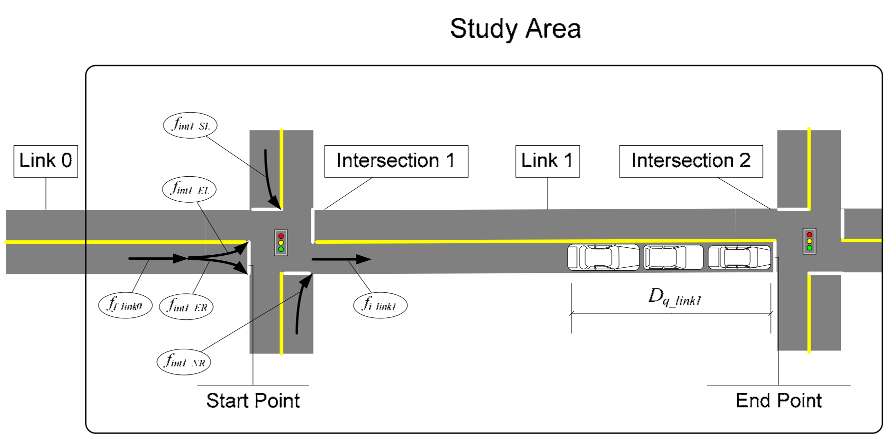

Figure 1-1: Sketch of A Link and Illustration of Significant Variables

The comprehensive list of variables used in the methodology is provided below:

ATT Arterial Travel Time, total travel time on the arterial

T_M Time in Motion, time period when the vehicles travel before they join the queue

T_Q Time in Queue, time period from when the vehicles join the queue to when they leave the queue

WT_Q Waiting Time in the Queue, time that vehicles have to wait in the queue

MT_Q Moving Time in the Queue, time that vehicles move when they are in the queue

w_{AB} Shockwave speed between state A and B

w_{BC} Shockwave speed between state B and C

w_{AC} Shockwave speed between state A and C

q_A Arrival rate (actual flow rate)

q_C At capacity flow rate

k_A Density at arrival rate, can be converted from flow divided by speed

k_B Jam density

k_C Density at capacity

C Cycle length

r Effective red time

s Total length of the link (exclude the length of intersection)

s_A Accelerating distance

s_C Constant speed distance

s_D The distance from the end point of S_2 till the end of the link

D_q Distance over which the queue extends

V_{i_link1} Vehicles' entering speed (initial speed) when they enter link 1

V_m Vehicles' maximum operating speed when they travel on the link

a_a Vehicles' acceleration rate

a_d Vehicles' deceleration rate

D_{q_link1} Distance over which the queue extends at link 1

V_{d_link1} Discharging speed at link 1

$avg.D_{q\_link1}$ Average distance over the queue extends at link 1

Q_M Maximum queue length

V_{f_link0} Leaving speed (final speed) for link 0

D_{q_link0} Distance over which the queue extends at link 0

V_{d_link0} The discharging speed from link 0

Q_R The residual queue

Q_M The maximum queue

$Diff_{cond2}$ The difference between the completed Condition 2 and the terminated Condition 2

f_{i_link1} Entering (initial) flow rate at link 1

f_{f_link0} Final flow rate at link 0

$f_{\text{int}1\_WR}$ Flow rate at intersection 1 from westbound right turning movement

$f_{\text{int}1\_WL}$ Flow rate at intersection 1 from westbound left turning movement

$f_{\text{int}1\_NR}$ Flow rate at intersection 1 from southbound right turning movement

$f_{\text{int}1\_SL}$ Flow rate at intersection 1 from northbound left turning movement

f_{i_link2} Entering (initial) flow rate at link 2

f_{f_link1} Final flow rate at link 1

f_{int2_WR} Flow rate at intersection 2 from westbound right turning movement

f_{int2_WL} Flow rate at intersection 2 from westbound left turning movement

f_{int2_NR} Flow rate at intersection 2 from southbound right turning movement

f_{int2_SL} Flow rate at intersection 2 from northbound left turning movement

There are several conditions that a vehicle may encounter when entering a link, and these affect its travel time on that link. Figure 1-2 presents nine posible speed profiles for a vehicle traveling through a particular link. These speed profiles are based on the upstream and downstream signal conditions. Condition 1 is defined as entering or departing the link from a full stop; Condition 2 is defined as entering or departing the link from a discharging moving queue; Condition 3 is defined as entering or departing the link at the desired speed. For example, the top left box in Figure 1-2 presents the speed profile of a vehicle that stops at two consecutive intersections. As it departs Intersection 1, it accelerates to its desired speed (if the length of the link allows it), and then decelerates to join the queue at Intersection 2.

The expected travel time on a link can be calculated based on the travel times for each of those profiles and the frequency at which each scenario is expected to occur. For example, for good progression, the frequency of the scenario shown in the bottom right of Figure 1-2 should be high, while the frequency of the scenario shown in the top left should be low.

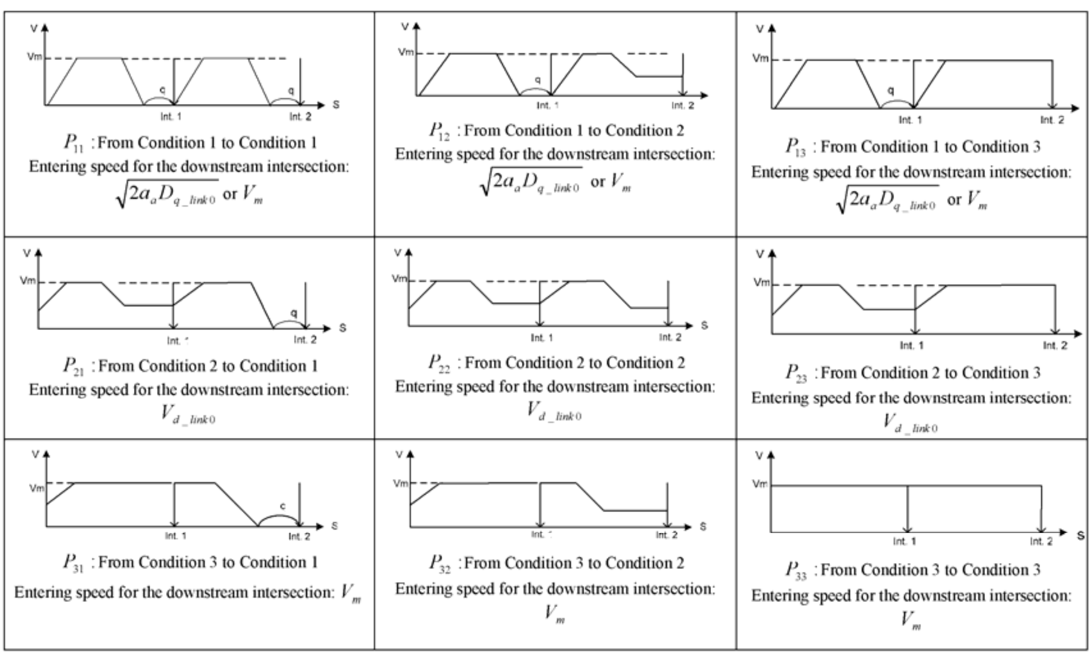

Figure 1-2. Speed Profiles for a Vehicle Traveling Through a Link

2. Example

The following example illustrates an application of this method for a single link between two signalized intersections. The expected travel time for a single direction along this link is estimated. A link is defined to start at the stop bar of the upstream intersection and end at the stop bar of the downstream interesection.

Inputs for the Model

- The link length between intersection 1 and 2 is 1320 ft, or 0.25 mile.
- The arterial has two-way traffic, with one lane per direction.
- Maximum operating speed: The maximum operating speed is assumed to be equal to the speed limit of the arterial, which is 30 mph.
- Traffic signal control at the two intersections: The cycle length is 80 seconds; the main arterial effective green time is 60 seconds, and effective red time is 20 seconds. The offset between intersections is 30 seconds.
- Traffic volume: *ff_link0* is 900 veh/h/ln, *ff_link1* is 950 veh/h/ln

- Acceleration and deceleration rates are assumed to be 10 *ft/s2*.
- At the first intersection, traffic volumes are as follows: NB and SB LT is 50 veh/hr/ln, NB and SB RT is 100 veh/hr/ln, EB LT is 5% of the *ff_link*0, EB RT is 10% of the *ff_link*0, EB TH is 85% of the *ff_link*0.
- At the second intersection, traffic volumes are as follows: NB and SB LT is 55 veh/hr/ln, NB and SB RT is 110 veh/hr/ln, EB LT is 10% of the *ff_link*1, EB RT is 10% of the *ff_link*1, EB TH is 80% of the *ff_link*1.

Step 1: Calculate the entering flows at the two intersections

Intersection 1:

$$f_{i\_link1} = f_{f\_link0} - f_{int1\_ER} - f_{int1\_EL} + f_{int1\_NR} + f_{int1\_SL} = 900 - 900*10\% - 900*5\% + 50 + 100$$
$$= 915\text{veh/h}$$

Intersection 2:

$$f_{i\_link2} = f_{f\_link1} - f_{int1\_ER} - f_{int1\_EL} + f_{int1\_NR} + f_{int1\_SL} = 915 - 915*10\% - 900*10\% + 55 + 110$$
$$= 897\text{veh/h}$$

Step 2: Calculate the probabilities for Conditions 1, 2, and 3

Using intersection 1 as an example, the probability for Conditions 1, 2, and 3 can be calculated as:

$$w_{AB} = \frac{-q_A}{k_B - k_A}, w_{BC} = \frac{-q_C}{k_B - k_C}, w_{AC} = \frac{q_A - q_C}{k_A - k_C}$$

In this example, based on the assumption of a linear function for the speed-density relationship, the calculations are as follows:

$q_{A\_int1} = 915\text{veh/hr}$, $q_{C\_int1} = 1800\text{veh/hr}$

$$k_{C\_int1} = \frac{q_{C\_int1}}{V_m/2} = \frac{1800}{30/2} = 120\text{veh/mile}$$

$$k_{B\_int1} = \frac{q_{C\_int1}}{V_m/2} * 2 = \frac{1800}{30/2} * 2 = 240\text{veh/mile}$$

Based on the four known values, the mathematical expression of the flow-density curve can be calculated as: Y = −15X2 + 3600X:

q_{A_int1} is 915 veh/hr, thus, q_{A_int1} = -15 k_{A_int1}2 + 3600 k_{A_int1}, k_{A_int1}=0.254 veh/mi

$$w_{AB} = \frac{-q_A}{k_B - k_A} = \frac{-915}{240 - 0.254} = -3.817, w_{BC} = \frac{-q_C}{k_B - k_C} = \frac{-1800}{240 - 120} = -15,$$

$$w_{AC} = \frac{q_A - q_C}{k_A - k_C} = \frac{915 - 1800}{0.254 - 120} = 7.39$$

$$\Pr\{\text{Condition 1}\} = \frac{R + R(\frac{w_{AB}}{w_{BC} - w_{AB}})}{C} = \frac{R(\frac{w_{BC}}{w_{BC} - w_{AB}})}{C} = \frac{20(\frac{-15}{-15 + 3.817})}{80} = 0.335$$

$$\Pr\{\text{Condition 2}\} = \frac{R(\frac{-w_{AB}w_{BC}}{(w_{BC} - w_{AB})w_{AC}})}{C} = \frac{20(\frac{15 * -3.817}{(-15 + 3.817) * 7.39})}{80} = 0.173$$

$$\Pr\{\text{Condition 3}\} = 1 - \frac{R(\frac{w_{BC}}{w_{BC} - w_{AB}})}{C} - \frac{R(\frac{-w_{AB}w_{BC}}{(w_{BC} - w_{AB})w_{AC}})}{C} = 1 - 0.335 - .0173 = 0.492$$

The same calculations are performed for intersection 2:

$q_{A\_\text{int}2} = 897\text{veh/hr}$, $q_{C\_\text{int}2} = 1800\text{veh/hr}$

$$k_{C\_\text{int}2} = \frac{q_{C\_\text{int}1}}{V_m/2} = \frac{1800}{30/2} = 120\text{veh/mile}$$

$$k_{B\_\text{int}2} = \frac{q_{C\_\text{int}1}}{V_m/2} * 2 = \frac{1800}{30/2} * 2 = 240\text{veh/mile}$$

Based on the four known values, the mathematical expression of the flow-density curve can be calculated as: Y = −15X2 + 3600X:

$q_{A\_\text{int}2}$ is 897 veh/hr, thus, $q_{A\_\text{int}2}$ = -15 $k_{A\_\text{int}2}$ 2 + 3600 $k_{A\_\text{int}2}$, $k_{A\_\text{int}2}$ =0.249 veh/mi

$$w_{AB} = \frac{-q_A}{k_B - k_A} = \frac{-897}{240 - 0.249} = -3.741, w_{BC} = \frac{-q_C}{k_B - k_C} = \frac{-1800}{240 - 120} = -15,$$

$$w_{AC} = \frac{q_A - q_C}{k_A - k_C} = \frac{897 - 1800}{0.249 - 120} = 7.541$$

$$\Pr\{\text{Condition 1}\} = \frac{R + R(\frac{w_{AB}}{w_{BC} - w_{AB}})}{C} = \frac{R(\frac{w_{BC}}{w_{BC} - w_{AB}})}{C} = \frac{20(\frac{-15}{-15 + 3.741})}{80} = 0.333$$

$$\Pr\{\text{Condition 2}\} = \frac{R(\frac{-w_{AB}w_{BC}}{(w_{BC} - w_{AB})w_{AC}})}{C} = \frac{20(\frac{15 * -3.741}{(-15 + 3.741) * 7.541})}{80} = 0.165$$

$$\Pr\{\text{Condition 3}\} = 1 - \frac{R(\frac{w_{BC}}{w_{BC} - w_{AB}})}{C} - \frac{R(\frac{-w_{AB} w_{BC}}{(w_{BC} - w_{AB}) w_{AC}})}{C} = 1 - 0.333 - 0.165 = 0.502$$

Step 3: Calculate the queue lengths and determine Vi_link1 for the three conditions

Dq_link1 for condition 1 and Condition 2

Dq_link1 is one factor in calculating *TM* for vehicles arriving at the intersection in Condition 1 and Condition 2. The average *Dq_link1* for the stopped queue in Condition 1 and the moving queue in Condition 2 are approximately the same.

$$avg\ D_{q\_Link1} = \frac{Q_M}{2} = \frac{r}{7200} \frac{-w_{AB} w_{BC}}{(w_{BC} - w_{AB})} * 5280 = \frac{20 * 5280}{7200} \frac{3.817 * -15}{(3.817 - 15)} = 75.09\ ft$$

Calculate *Vi_link1* for Condition 1, Condition2, and Condition3:

Entering speed for Condition 1:

$$V_{i\_Link1} = \sqrt{2 a_a D_{q\_link0}} = \sqrt{2 * 10 * 75.09} = 38.75\text{ft/s} = 26.42\text{mph} < V_m$$

Thus, *Vi_link1* =26.42 mph

Entering speed for Condition 2:

$$V_{i\_Link1} = V_{d\_Link0} = \frac{V_m}{2} = 15\text{mph} < V_m$$

Entering speed for Condition 3:

$$V_{i\_Link1} = V_m = 30\text{mph}$$

Step 4: Calculate the minimum and maximum travel time for each possible flow profile.

For vehicles that start from Condition 1 at the first intersection, calculate the travel time for the first vehicle in the group, and then locate the time point on the travel time bar for the next intersection:

tt11=27.38 sec, it is found that the time point for this travel time on the time bar for the next intersection is 54.21 sec, which is smaller than the end time point of Condition 1 on the next intersection (56.95 sec). Thus, the minimum travel time point is 54.21 seconds.

Find the time intervals for each possible flow profile and then calculate the probability of occurrence for each. For example, for vehicles that arrive in Condition 1 at the first intersection, there is only one possible time interval. The probability is

$$P_{11} = \frac{56.95 - 54.21}{56.95 - 54.21} = 1.0$$

Step 5: Calculate travel time components *TM*, *MTQ*, and *WTQ*, the total travel time for 9 flow profiles, the expected travel time, the standard deviation of travel time, and the travel time distribution

TM values are calculated based on the trajectory of the vehicles *WTQ* : is approximately calculated as half of the red time according to shockwave analysis. *MTQ* : is based on the queue spacing and the acceleration rate. Travel times can be calculated for 9 different flow profiles. The expected travel time can be calculated for the three different groups of vehicles that arrive in different conditions at intersection 1.

$$E(TT_{Cond1}) = TT_{11} * Scenario_{11}\% + TT_{12} * Scenario_{12}\% + TT_{13} * Scenario_{13}\%$$

$$E(TT_{Cond2}) = TT_{21} * Scenario_{21}\% + TT_{22} * Scenario_{22}\% + TT_{23} * Scenario_{23}\%$$

$$E(TT_{Cond3}) = TT_{31} * Scenario_{31}\% + TT_{32} * Scenario_{32}\% + TT_{33} * Scenario_{33}\%$$

The total expected travel time is calculated as follows:

$$E(TT) = TT_{Cond1} * Cond_1\% + TT_{Cond2} * Cond_2\% + TT_{Cond3} * Cond_3\%$$

$$E(TT) = 41.29 * 0.41 + 32.29 * 0.59 = 35.94 \sec$$

The standard deviation of travel time is:

$$\sigma = \sqrt{E(x^2) - E^2(x)} = \sqrt{\sum_i p_i x_i^2 - (\sum_i p_i x_i)^2} = 4.74 \sec$$

In conclusion, the expected (i.e., average) travel time on this link is 35.94 seconds, and the standard deviation of that travel time is 4.74 sec.