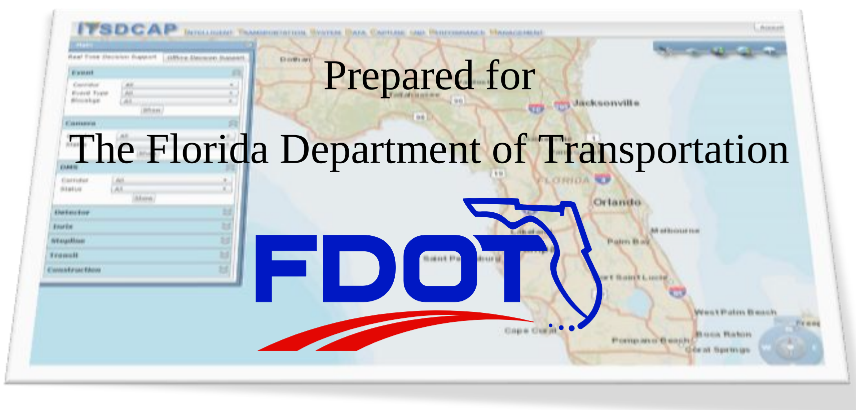
Decision Support Systems for Transportation System Management and Operations (TSM&O)

Final Report

FDOT Project BDV29-977-09



Prepared for

The Florida Department of Transportation

by

Lehman Center for Transportation Research

Florida International University

December 2015

Disclaimer

Metric Conversion Chart

APPROXIMATE CONVERSIONS TO SI UNITS

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

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

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16. Abstract

There is a need for the development of tools and methods to support off-line and real-time planning and operation decisions associated with the Transportation System Management and Operations (TSM&O) program. The goal of this proposed project is to research and produce a data analytic environment that supports the objectives and activities of the TSM&O program. Two previously developed tools, the ITS Data Capture and Performance Management (ITSDCAP) and the Integrated Regional Information Sharing and Decision Support System (IRISDS), have been integrated into a single Web-based environment in the project. In addition, new methods and modules have been developed to provide the functionality required by this environment. Some of the addressed functions in this research include the extension of system performance estimation and prediction, the production of performance dashboard, the development of benefit/cost estimation module for arterial incident management evaluation, the assessment of construction zone impacts, methods for identification of arterial performance problems, and review of the related previous Florida Department of Transportation research projects for possible implementation in the environment.

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Executive Summary

To meet the objectives of the Transportation System Management and Operations (TSM&O) program, there is a need for the development of tools and methods to support off-line and real-time planning and operation decisions associated with the TSM&O. Such developments would be integrated into a data analytic environment that captures data from multiple sources and utilizes the data to support TSM&O partner agency decisions.

Two tools were developed as part of previous Florida Department of Transportation (FDOT) Research Center projects and can be used as bases for the development of the decision support environment mentioned above. The two tools are ITS Data Capture and Performance Management (ITSDCAP), developed as part of the FDOT Research Center Project BDK80-977-11 (Hadi et al., 2012), and the Integrated Regional Information Sharing and Decision Support System (IRISDS), developed as part of FDOT Research Center Project BDK80-977-09 (Hadi e al., 2013).

ITSDCAP, developed in Project BDK80-977-11, captures data from multiple sources, estimates various performance measures (mobility, reliability, safety and environmental), performs data mining techniques, support benefit-cost analysis, and allows the visualization of data. To perform these functions, ITSDCAP utilizes data from multiple sources, including SunGuide data, central data warehouses such as the Statewide Transportation Engineering Warehouse for Archived Regional Data (STEWARD) and Regional Integrated Transportation Information System (RITIS), incident databases, FDOT planning statistics office data, weather data, pricing rates, construction database, crash data such as Crash Analysis Reporting (CAR) System, 511 traveler information systems, Automatic Vehicle Identification (AVI) data, and private sector data. However, the original version of ITSDCAP was a desktop tool that required the installation of add-on software. In addition, it mainly focused on freeway corridor performance measurements.

IRISDS is a proof-of-concept Web-based system that displays regionally shared information in real-time and provides a decision support environment for transportation system management agencies in a region. One of the tools included in IRISDS allows the prediction and visualization of incident impacts in real-time (duration, delays, queues, secondary incidents, and diversion rate). Another tool allows the estimation of general traffic travel time based on bus Automatic Vehicle Location (AVL) data.

The goal of the proposed project was to produce a decision support environment that supports the objectives and activities of the TSM&O program. The specific objectives were to allow:

- Integrate the ITSDCAP and IRISDS tools developed in previous efforts in a single Web-based user friendly environment

- Extend the estimation and analysis of system performance to include further performance measures and to produce performance dashboards based on user needs

- Extend the benefit-cost analysis module of ITSDCAP to allow the estimation of the benefits of incident management on signalized arterials and to produce required inputs to other benefit-cost analysis tools based on data from multiple sources

- Produce modules for the estimation of the impacts of construction and maintenance activities on system performance and integrate these modules into ITSDCAP

- Develop and test a method for real-time prediction of breakdown conditions on arterial streets

- Develop methods for identification of arterial performance problems and influencing factors

- Review past FDOT research projects related to TSM&O activities for potential incorporation in future versions of the tool.

A summary of the activities of this project follows.

Conversion of ITSDCAP and IRISDS to an Integrated Environment: The first task of this project was to convert the IRISDS and ITSDCAP into a Web-based environment that integrates the off-line and real-time utilization of data to support TSM&O decision making processes. Figures E-1 and E-2 show an example of the newly developed ITSDCAP user interface.

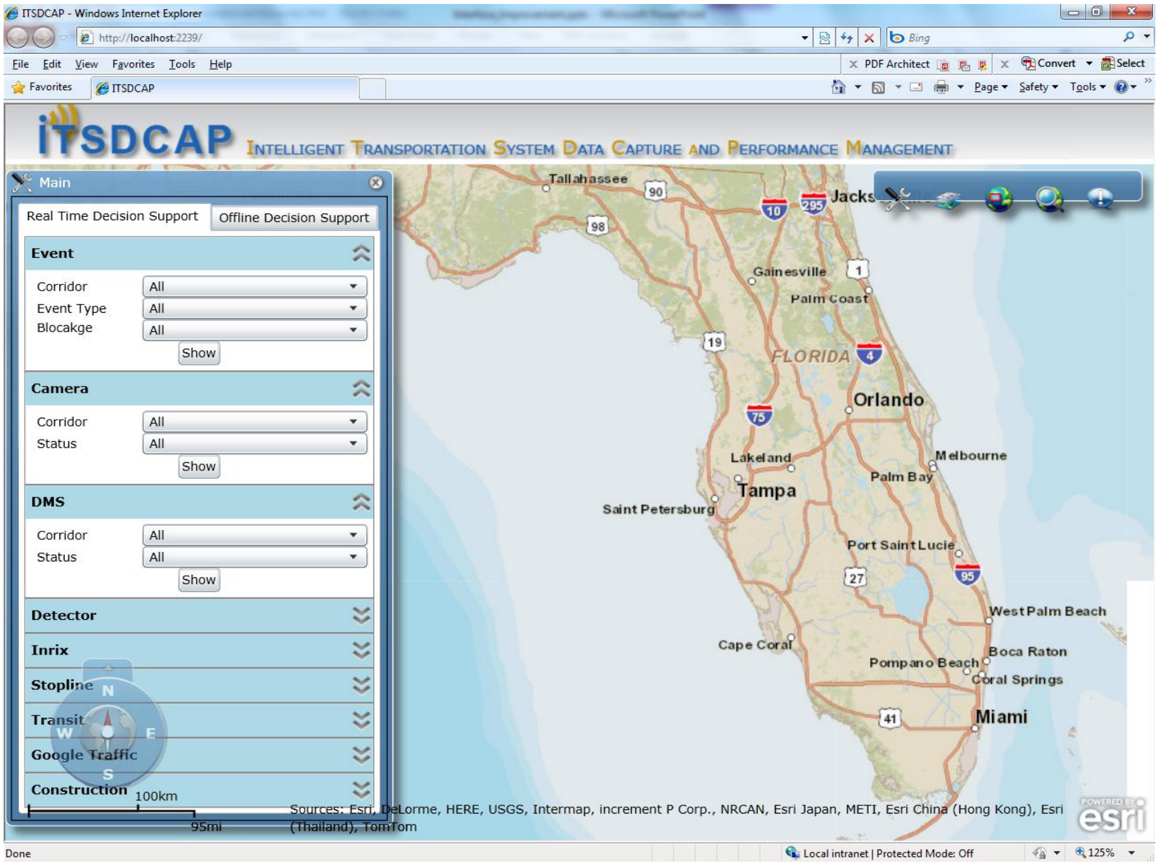

Figure E-1 Example of ITSDCAP Entry Display

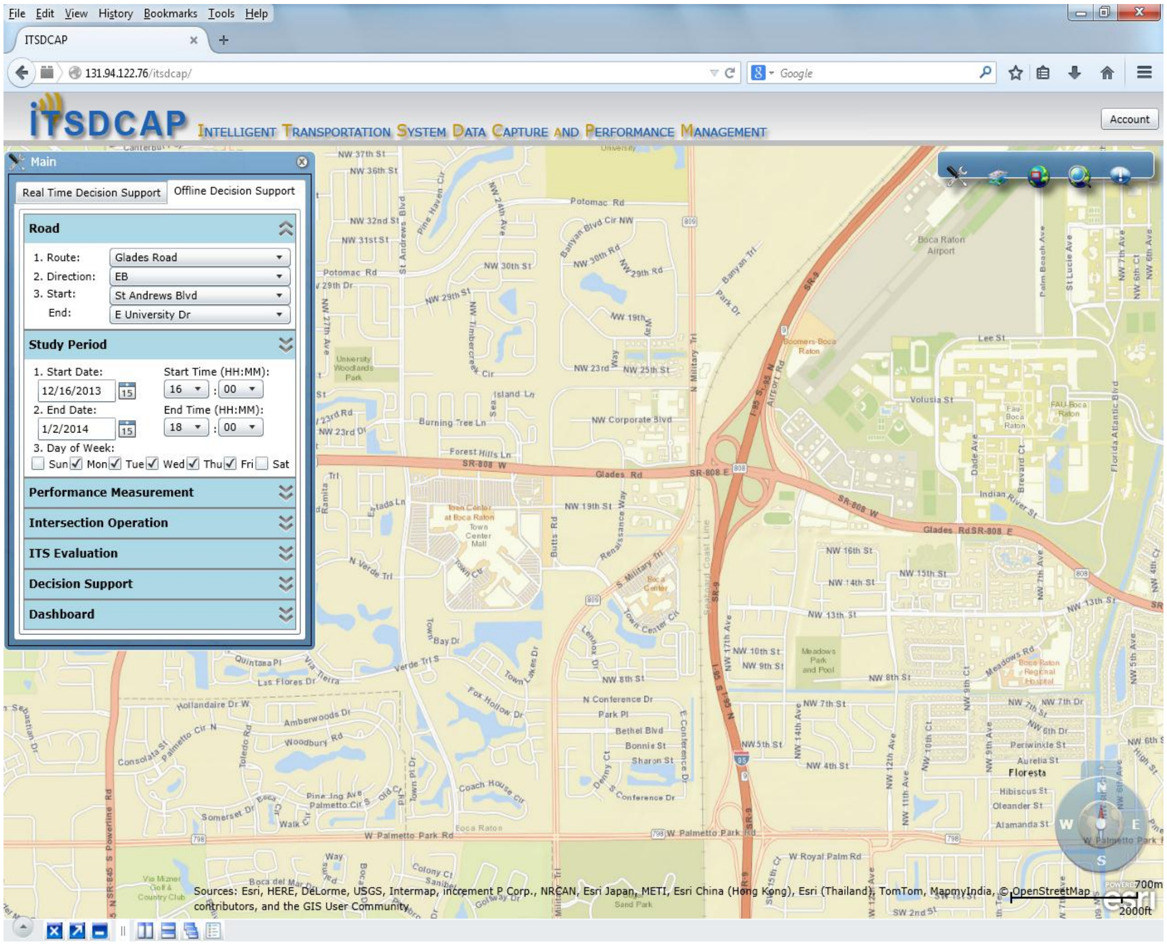

Figure E-2 Example of ITSDCAP Corridor Level Display

Support of TSM&O Performance Dashboard: A module was included in the original version ITSDCAP to estimate various performance measures including mobility, reliability, safety, and pollutant emission. In this project, the ITSDCAP tool was upgraded to allow performance measurement of both freeways and arterials. In addition, the enhanced ITSDCAP tool allows for the creation of performance dashboards based on user requirements. Figure E-3 presents an example of the dashboard produced for FDOT District 4 Broward County TSM&O arterial networks produced by ITSDCAP. Figure E-4 shows an example of safety measures interfaces of ITSDCAP.

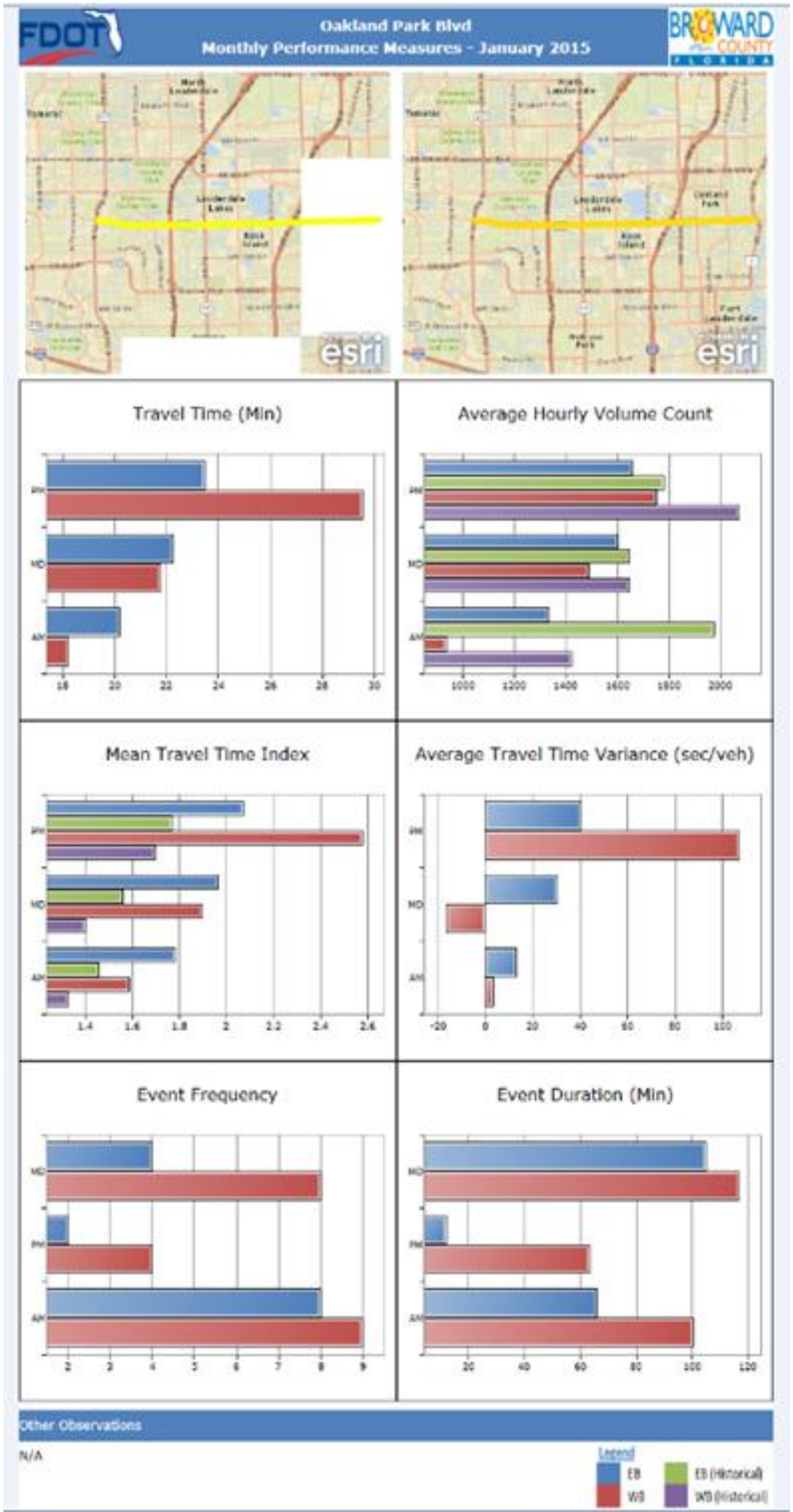

Figure E-3 Example of ITSDCAP Dashboards

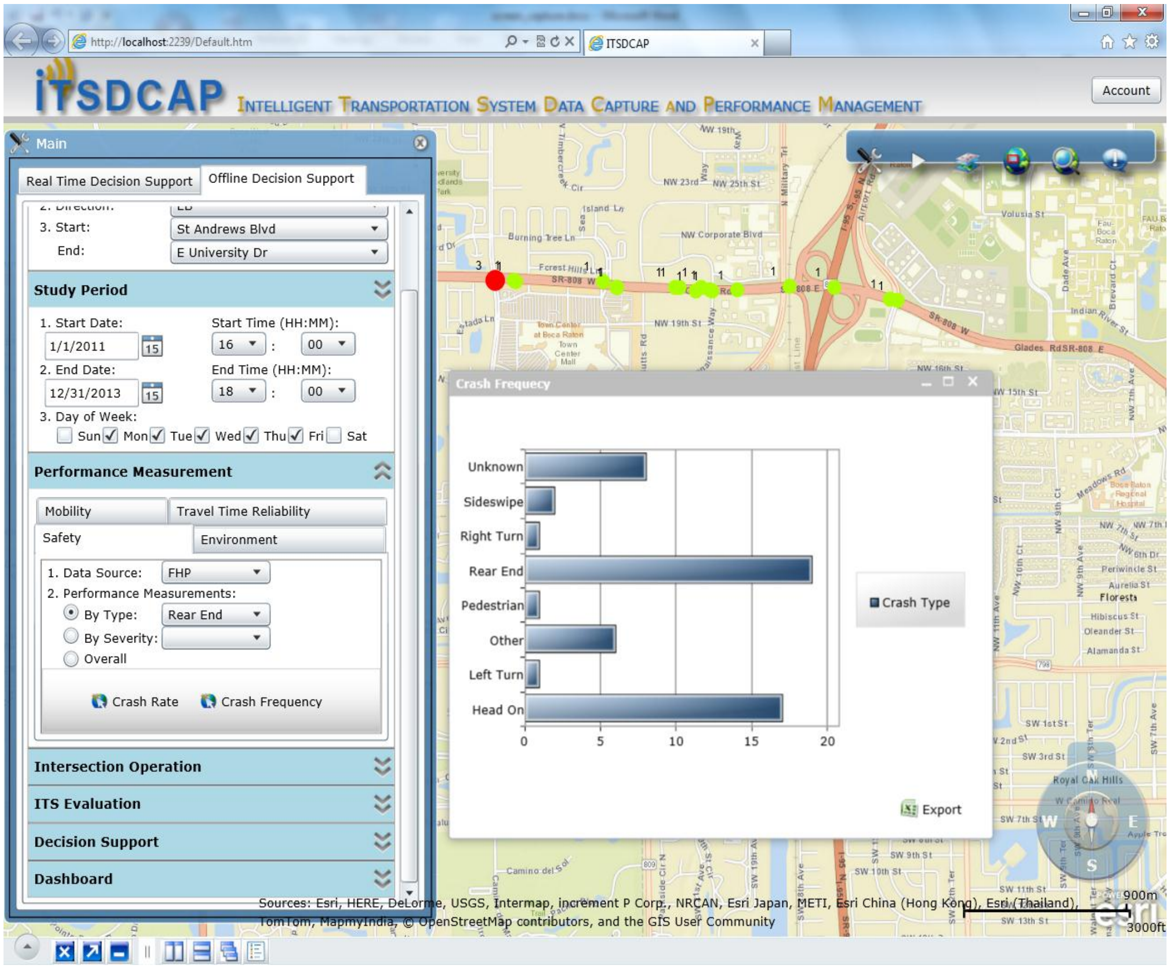

Figure E-4 Example of Rear-End Crash Frequency for Glades Road Eastbound in ITSDCAP

Incorporating the Probability of Breakdown: There are studies about predicting traffic breakdown on freeways in the literature, but few of these addressed the prediction of traffic breakdown on arterials. This project investigates approaches to predict breakdown on arterial streets. The breakdown prediction models were integrated into the ITSDCAP tool for real-time prediction of probability of breakdown. Figure E-5 presents a decision tree developed in this study to predict breakdown probability on arterials.

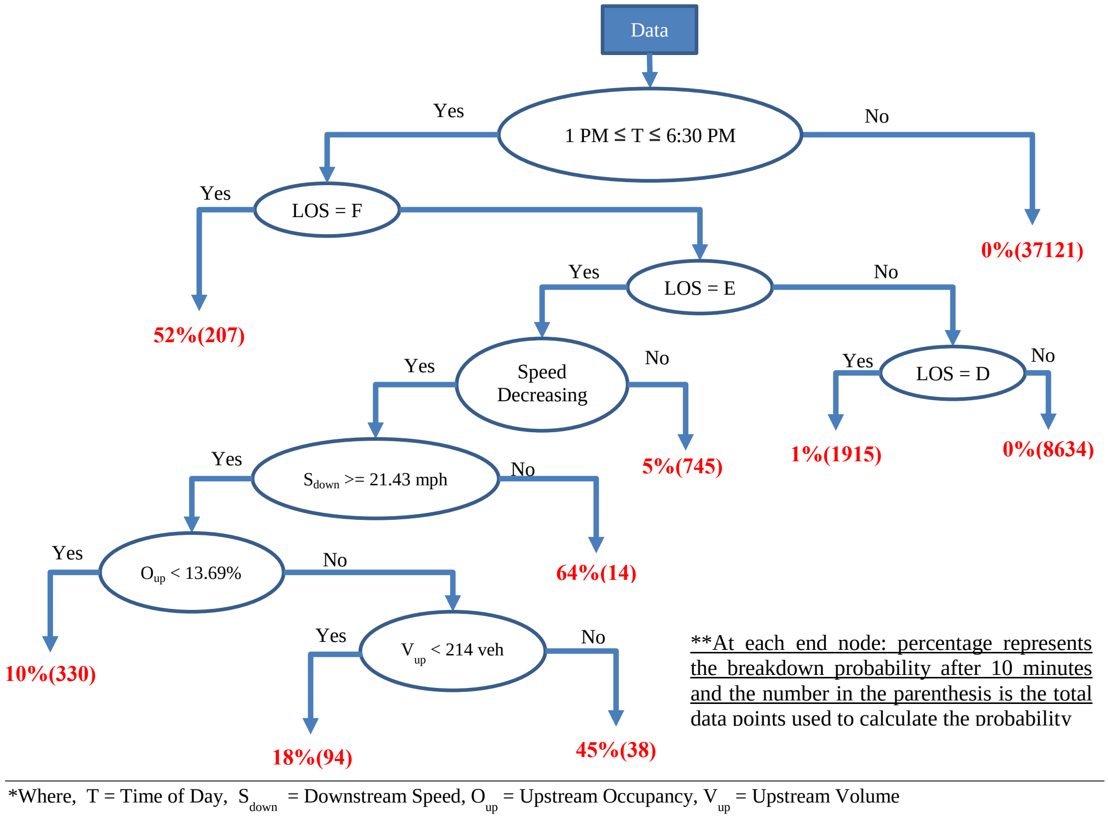

Figure E-5 Developed Decision Tree to Predict Breakdown on Glades Road in Boca Raton

Extension of the Benefit-Cost Module of ITSDCAP: A benefit-cost evaluation module was developed and incorporated into the original ITSDCAP tool. In the Web-based version of ITSDCAP developed in this study, two types of benefit-cost assessment supports are available. The first provides the input required for other ITS evaluation tools such as the Florida ITS Evaluation Tool (FITSEVAL) and Tool for Operations Benefit Cost Analysis (TOPS-BC). The second estimates the benefits directly based on data and modeling. For this second type of the benefit evaluation support, the incident management benefit module, originally developed for freeways was extended in this study to allow the assessment of the benefits of incident management on arterials. Figure E-6 shows a snapshot of the ITSDCAP benefit-cost module support function.

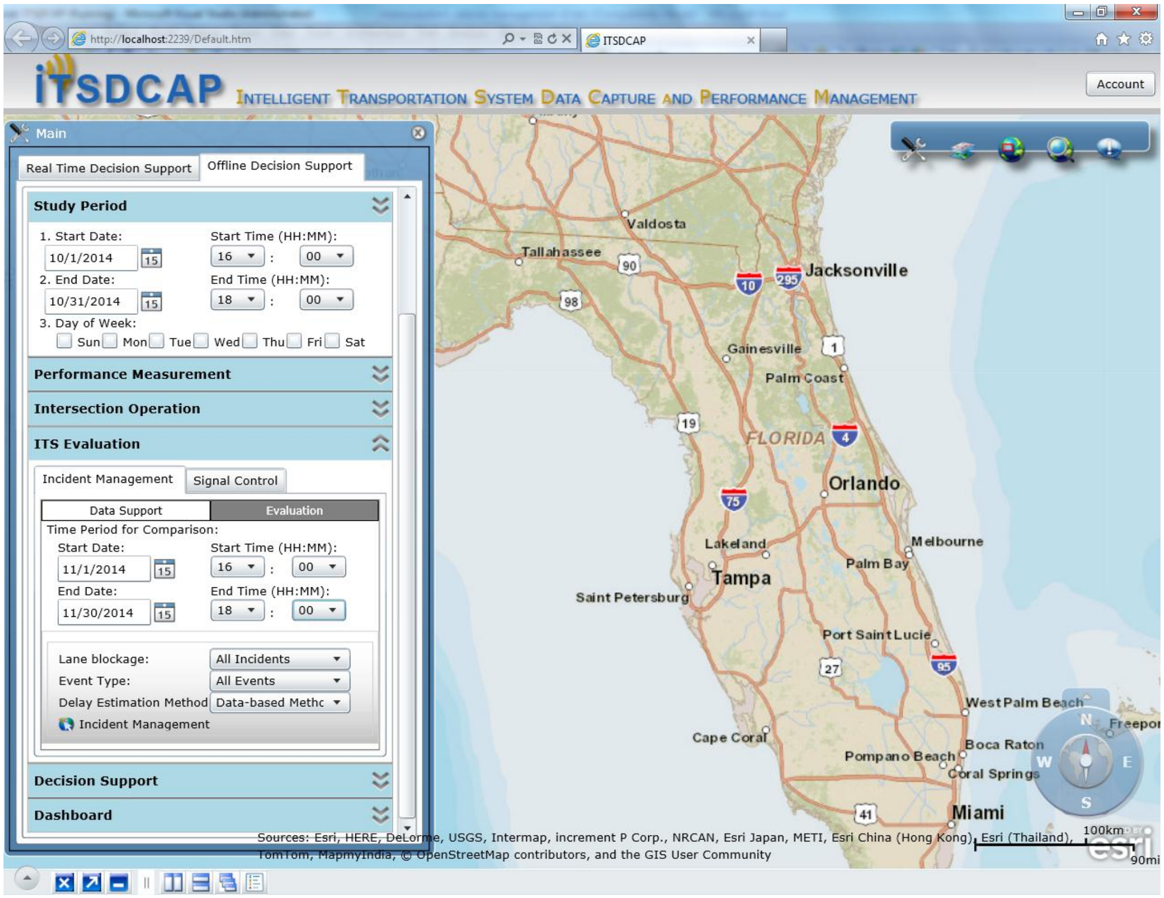

Figure E-6 Screenshot of ITSDCAP Interface for the Benefit-Cost Support Function

Estimation of Construction Impacts: In this task, we developed a module within the ITSDCAP environment to provide the data analysis and modeling support for construction impact analysis. A work zone evaluation module based on real-world data is implemented in ITSDCAP, as part of this task. In addition, the developed environment provides the required inputs for external modeling tools such as the demand and capacity values at the work zone. An example of Construction Impact Assessment Interface in ITSDCAP is shown in Figure E-7.

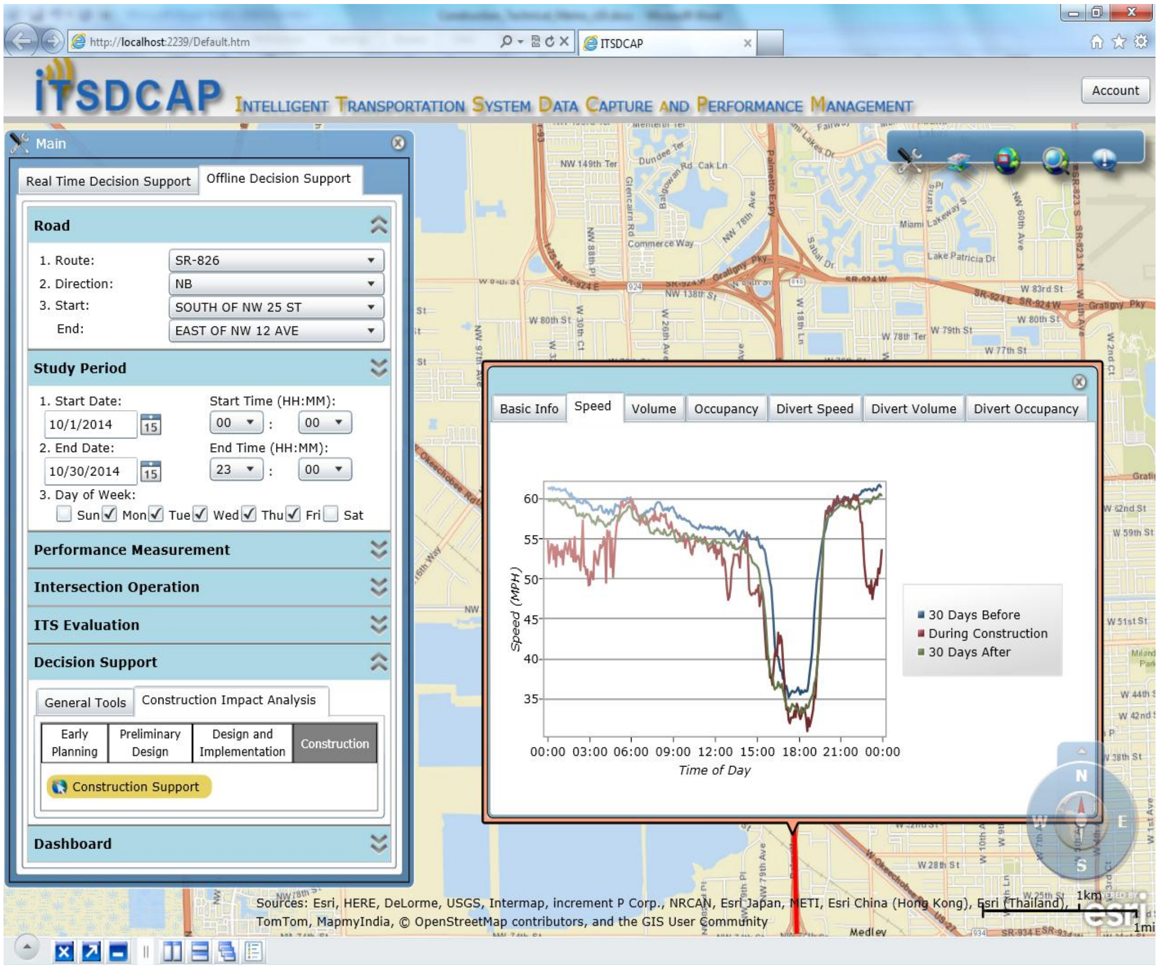

Figure E-7 Example of Construction Impact Assessment Interface in ITSDCAP

Signal Timing Diagnostic System Based on Existing Data Sources: This task involved an initial effort to develop a signal timing diagnostic system that uses a combination of existing relatively low-cost data from Wi-Fi or Bluetooth readers combined with data from existing signal controllers to provide information for diagnosing signal operations. An overview of the developed decision support scheme is shown in Figure E-8.

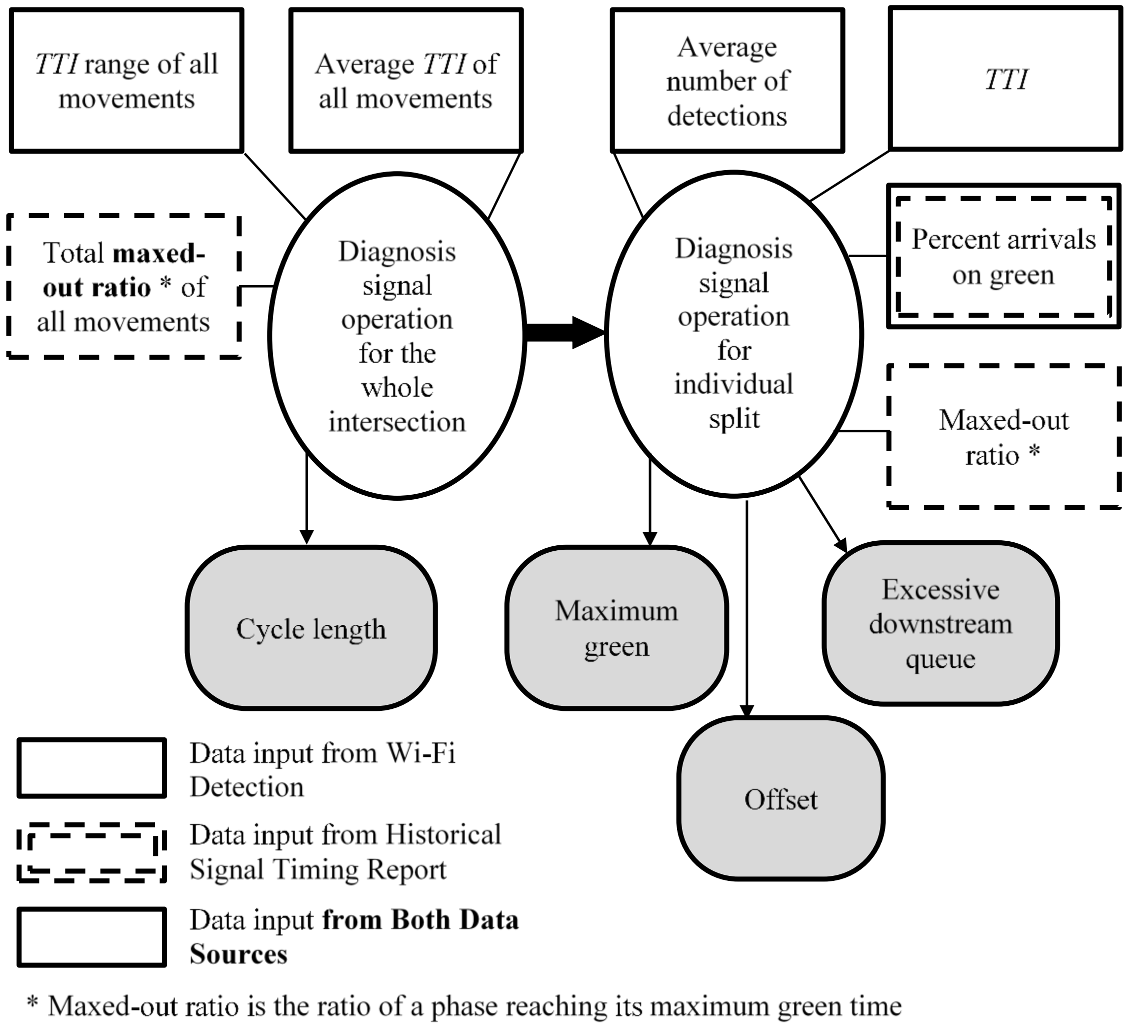

Figure E-8 Developed Decision Support Signal Operation Diagnosis Scheme

Utilization the HCM Procedures for the Estimation of Travel Time with Consideration of Rain Impacts: This task focused on investigating the potential for real-time prediction of travel time on urban street facilities under rainy conditions utilizing the Highway Capacity Manual (HCM) urban street procedures. The travel time estimation is validated based on real-world measurements of traffic performance in conditions with different rain intensities. Once validated, this task examines the accuracy of using HCM 2010 urban street facility procedure with these factors to predict weather impacts on travel time in real-time operations. The results of the prediction assessment are shown in Table E-1.

Table E-1 Travel Time Prediction Results

Scenario		**Medium Rain**				
		MAPE	**RMSE**	**NRMSE**	**MSPE**	**RMSPE**
No Prediction	15 min	0.107	13.326	0.132	0.016	0.127
	30 min	0.117	18.668	0.192	0.012	0.108
	45 min	0.111	15.890	0.175	0.010	0.101
	60 min	0.210	43.012	0.391	0.050	0.223
Prediction Using “Normal” Day Demands as Input	15 min	0.096	17.294	0.171	0.010	0.099
	30 min	0.103	23.187	0.239	0.013	0.115
	45 min	0.097	19.867	0.218	0.011	0.104
	60 min	0.219	46.868	0.426	0.050	0.223
Prediction Using Instantaneous Demands as Input	15 min	0.059	12.111	0.125	0.004	0.063
	30 min	0.061	12.561	0.127	0.004	0.063
	45 min	0.043	8.513	0.094	0.002	0.045
	60 min	0.148	34.157	0.311	0.024	0.155
Prediction with Forecasted Demands as Input	15 min	0.048	10.700	0.106	0.003	0.055
	30 min	0.045	8.913	0.098	0.002	0.047
	45 min	0.045	6.087	0.072	0.004	0.061
	60 min	0.088	11.627	0.117	0.008	0.092
		Heavy Rain				
No Prediction	15 min	0.126	17.103	0.244	0.019	0.139
	30 min	0.208	32.016	0.508	0.051	0.227
	45 min	0.121	11.597	0.153	0.009	0.096
	60 min	0.160	21.840	0.240	0.019	0.138
Prediction Using “Normal” Day Demands as Input	15 min	0.116	16.347	0.234	0.014	0.118
	30 min	0.108	16.523	0.262	0.013	0.116
	45 min	0.100	14.874	0.196	0.010	0.100
	60 min	0.146	26.217	0.288	0.022	0.149
Prediction Using Instantaneous Demands as Input	15 min	0.015	2.948	0.042	0.000	0.017
	30 min	0.086	16.895	0.268	0.008	0.092
	45 min	0.028	3.619	0.048	0.001	0.031
	60 min	0.044	10.675	0.117	0.003	0.054
Prediction with Forecasted Demands as Input	15 min	0.015	2.948	0.042	0.000	0.017
	30 min	0.043	7.432	0.118	0.003	0.056
	45 min	0.020	2.658	0.035	0.000	0.021
	60 min	0.036	6.768	0.078	0.001	0.037

Review of Previous FDOT Projects on Traffic Management: The development of the ITSDCAP tool in this project provides an opportunity to incorporate decision support tools produced based on previously conducted research projects into a single environment. The last task of this project was to review of the related FDOT research projects for potential incorporation in ITSDCAP. This review is presented in Chapter 7.

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List of Selected Acronyms and Abbreviations

Acronyms and abbreviations used in the report are listed below.

ATIS	Advanced Traveler Information System
CAR	Crash Analysis Reporting System
CSV	Comma-Separated Value
DLL	Dynamic Link Library
DMS	Dynamic Message Sign
DTA	Dynamic Traffic Assignment
FDOT	Florida Department of Transportation
EPA	Environmental Protection Agency
FHP	Florida Highway Patrol
FHWA	Federal Highway Administration
GIS	Geographic Information System
GUI	Graphic User Interface
HCM	Highway Capacity Manual
HOV	High Occupancy Vehicle
ITS	Intelligent Transportation System
Matlab	A numerical computing environment developed by MathWorks
RWIS	Road Weather Information System
SIRV	Severe Incident Response Vehicle
STEWARD	Statewide Transportation Engineering Warehouse for Archived Regional Data
TMC	Traffic Management Center
TSS	Traffic Sensor System
TVT	Travel Time

1 Introduction

1.1 Background

The Transportation System Management and Operations (TSM&O) program of the Florida Department of Transportation (FDOT) has seven objectives, which are listed in the TSM&O Tier 2 business plan. Two important objectives of the program are “continually measure success of TSM&O by developing the ability to measure and report TSM&O performance gains” and “improve the performance of the network.” To meet the above objectives, there is a need for the development of tools and methods for off-line and real-time measurement of performance, benefit-cost analysis, and the support of decisions associated with active management strategies.

The Regional Integrated Transportation Information System (RITIS) produced by the University of Maryland has been selected as the FDOT’s new central data warehouse (University of Maryland CATTI Lab, 2015). This system has a powerful data archiving and visualization components and will provide one source of data for this project. There is a need, however, for the development of a data analytic tool to capture data from RITIS and multiple other sources and to utilize the data in combination with methods and models developed in previous FDOT research projects and new research conducted as part of this project to support TSM&O partner agency decisions. Such development will utilize data mining and traffic analysis to add significant values to the archived data with the goal of supporting TSM&O activities.

Two tools were developed as part of previous FDOT Research Center projects that can provide a strong platform for the development required to support the TSM&O program activities. The two tools are ITS Data Capture and Performance Management (ITSDCAP), developed as part of the FDOT Research Center Project BDK80-977-11 (1), and the Integrated Regional Information Sharing and Decision Support System (IRISDS), developed as part of FDOT Research Center Project BDK80-977-09 (2).

ITSDCAP, developed in Project BDK80-977-11, captures data from multiple sources, estimates various performance measures (mobility, reliability, safety and environmental), performs data mining techniques, supports benefit-cost analysis, and allows the visualization of data. To perform these functions, ITSDCAP utilizes data from multiple sources, including SunGuide data, central data warehouses (STEWARD and RITIS), incident databases, FDOT planning statistics office data, weather data, pricing rates, construction database, crash data such as Crash Analysis Reporting (CAR) System, 511 traveler information systems, Automatic Vehicle Identification (AVI) data, and private sector data. However, the original version of ITSDCAP was a desktop tool that required the installation of add-on software. In addition, it mainly focused on freeway corridor performance measurements.

IRISDS is a proof-of-concept Web-based system that displays regionally shared information in real-time and provides a decision support environment for transportation system management agencies in a region. One of the tools included in IRISDS allows the prediction and visualization of incident impacts in real-time (duration, delays, queues, secondary incidents, and diversion rate). Another tool allows the estimation of general traffic travel time based on bus Automatic Vehicle Location (AVL) data.

This project extends and builds on the existing tools and methods developed in previous FDOT efforts to produce an effective decision support environment that supports the objectives and activities of the TSM&O program. This project integrates the ITSDCAP and IRISDS tools mentioned above in an integrated Web-based environment that supports both real-time and off-line analysis. This project also involves conducting further research and development of tools and methods to support TSM&O planning and operations decisions for freeways and arterials.

1.2 Project Goal and Objectives

The goal of the proposed project is to produce a decision support environment that supports the objectives and activities of the TSM&O program. The specific objectives are to allow:

- Integrate the ITSDCAP and IRISDS tools developed in previous efforts in a single Web-based user friendly environment

- Extend the estimation and analysis of system performance to include further performance measures and to produce performance dashboards based on user needs

- Extend the benefit-cost analysis module of ITSDCAP to allow the estimation of the benefits of incident management on signalized arterials and to produce required inputs to other benefit-cost analysis tools based on data from multiple sources

- Produce modules for the estimation of the impacts of construction and maintenance activities on system performance and integrate these modules in ITSDCAP

- Develop and test a method for real-time prediction of breakdown conditions on arterial streets

- Develop methods for identification of arterial performance problems and influencing factors

- Review past FDOT research projects related to TSM&O activities for potential incorporation in future versions of the tool

1.3 Project Activities and Report Organization

The list below presents a summary of the activities of this project and associates these activities with the sections of this report:

- ***Conversion of ITSDCAP and IRISDS to an Integrated Environment:*** The first task of this project is to convert the IRISDS and ITSDCAP in to a Web-based environment that integrates the off-line and real-time utilization of data to support TSM&O decision making processes. This conversion is discussed in Section 1.4 of this report.

- ***Support of TSM&O Performance Dashboard:*** A module was included in the original version ITSDCAP to estimate various performance measures including mobility, reliability, safety, and pollutant emission. In this project, the ITSDCAP tool is upgraded to allow performance measurement of both freeways and arterials. In addition, the enhanced ITSDCAP tool allows for the creation of performance dashboards based on user requirements. The development of this task is detailed in Chapter 2.

- ***Incorporating the Probability of Breakdown:*** Studies have been conducted to predict traffic breakdown on freeways but limited studies addressed the prediction of traffic breakdown on arterials. This project investigates approaches to predict breakdown on arterial streets. The breakdown prediction models are integrated in the ITSDCAP tool for real-time prediction of probability of breakdown. The incorporation of the probability of breakdown is discussed in Chapter 3.

- ***Extension of the Benefit-Cost Module of ITSDCAP***: A benefit-cost evaluation module was developed and incorporated in the original ITSDCAP tool. In the Web-based version of ITSDCAP developed in this study, two types of benefit-cost assessment supports are available. The first is to provide the input required for other ITS evaluation tools such as the Florida ITS Evaluation Tool (FITSEVAL) and TOPS-BC. The second is to estimate the benefits directly based on data and modeling. For this second type of the benefit evaluation support, the incident management benefit module, originally developed for freeways is extended in this study to allow the assessment of the benefits of incident management on arterials. The benefit-cost evaluation support is discussed in Chapter 4.

- ***Estimation of Construction Impacts:*** This task aims at developing a module within the ITSDCAP environment to provide the data analysis and modeling support for construction impact analysis. A work zone evaluation module based on real-world data is implemented in ITSDCAP, as part of this task. In addition, the developed environment provides the required inputs for external modeling tools such as the demand and capacity values at the work zone. The estimation of the construction costs is addressed in Chapter 5.

- ***Signal Timing Diagnostic System based on Existing Data Sources:*** This task involves an initial effort to develop a signal timing diagnostic system that use a combination of existing relatively-low-cost data from Wi-Fi or Bluetooth readers combined with data from existing signal controllers to provide information for diagnosing signal operations. This initial development is discussed in Chapter 6 and will be extended in future efforts.

- ***Utilization the HCM Procedures for the Estimation of Travel Time with Consideration of Rain Impacts:*** This task focuses on investigating the potential for real time prediction of travel time on urban street facilities under rainy condition utilizing the Highway Capacity Manual (HCM) urban street procedures. The travel time estimation is validated based on real-world measurements of traffic performance in conditions with different rain intensities. Once validated, this task examines the accuracy of using HCM 2010 urban street facility procedure with these factors to predict weather impacts on travel time in real-time operations. The results from this task are also discussed in Chapter 6.

- ***Review of Previous FDOT Projects on Traffic Management:*** The development of the ITSDCAP tool in this project provides an opportunity to incorporate decision support tools produced based on previously conducted research projects in a single environment. The last task of this project is to review of the related FDOT research projects for potential incorporation in ITSDCAP. This review is presented in Chapter 7.

1.4 Conversion of ITSDCAP and IRISDS to an Integrated Web-Based Environment

As stated earlier, the previously developed version of IRISDS was a Web-based tool. ITSDCAP was originally developed as a desktop tool. In this project, the existing IRISDS and ITSDCAP tools were integrated in a Web-based environment and became accessible from the same user interface. Details of the original modules of ITSDCAP and IRISDS are included in References (Hadi et al., 2012 and 2013). New modules and developments are further discussed in this document.

The Web-based environment was developed using Microsoft Silverlight and Esri ArcGIS API for Silverlight. Microsoft Silverlight provides a cross-browser, cross-platform development environment for building and delivering interactive and expressive applications for the web. The ArcGIS API for Silverlight enables integrating the ArcGIS and the Bing Maps services and capabilities in a Silverlight application. The Microsoft Visual Studio 2010 was used as the programming environment in the development. The utilized programming language was Visual C#.

The framework of the developed environment is a client-server architecture that includes three tiers:

- Presentation tier
- Application tier
- Data tier

The presentation tier is the topmost level. It allows the users to access the website using browsers such as Internet Explorer, Firefox, etc. It also provides other applications with XML feeds such data as real-time center-to-center (C2C) data stream connection with traffic management centers.

The application tier is the logical tier located at the website server controlling the website's functionality. There are two groups of applications. One is the real-time applications, and the other is the off-line applications.

The data tier is the back-end data store. It comprises a central Oracle database server storing the historical data collected from the data archives different agencies and data collected in real-time from remote servers from agencies sharing the data with the XML feeds.

Figure 1-1 shows the main interface of the developed Web-based tool. The web page consists of three areas:

- The title and an account button are located on the top of the page, allowing the user to perform account related tasks such as changing password or logging out. The user needs to request and activate a user account so that they can access the website. Password protected views can be set for each agency views to protect the information of the agency if needed.

- The main operation area is located in the In the middle of the page. The Bing map at the background is to show the space related results. The non-space related results such as tabular data or charts are shown in the windows floating on top of the map. A main control panel also floating on top of the map allows the user to input values and perform decision support tasks. A taskbar located at the top-right corner provides the user with more controls of the map. For example, the user can change the base map type to the satellite imagery, open an overview map, or toggle the visibility of the main control panel window, an overview map, or a magnifying glass, etc.

- On the bottom of the page is a toolbox bar allowing the user to close or control the layout of the floating result windows. For example, the user can maximize or minimize the windows. To compare the results of different scenarios, it is more convenient for the user to line up the windows horizontally or vertically.

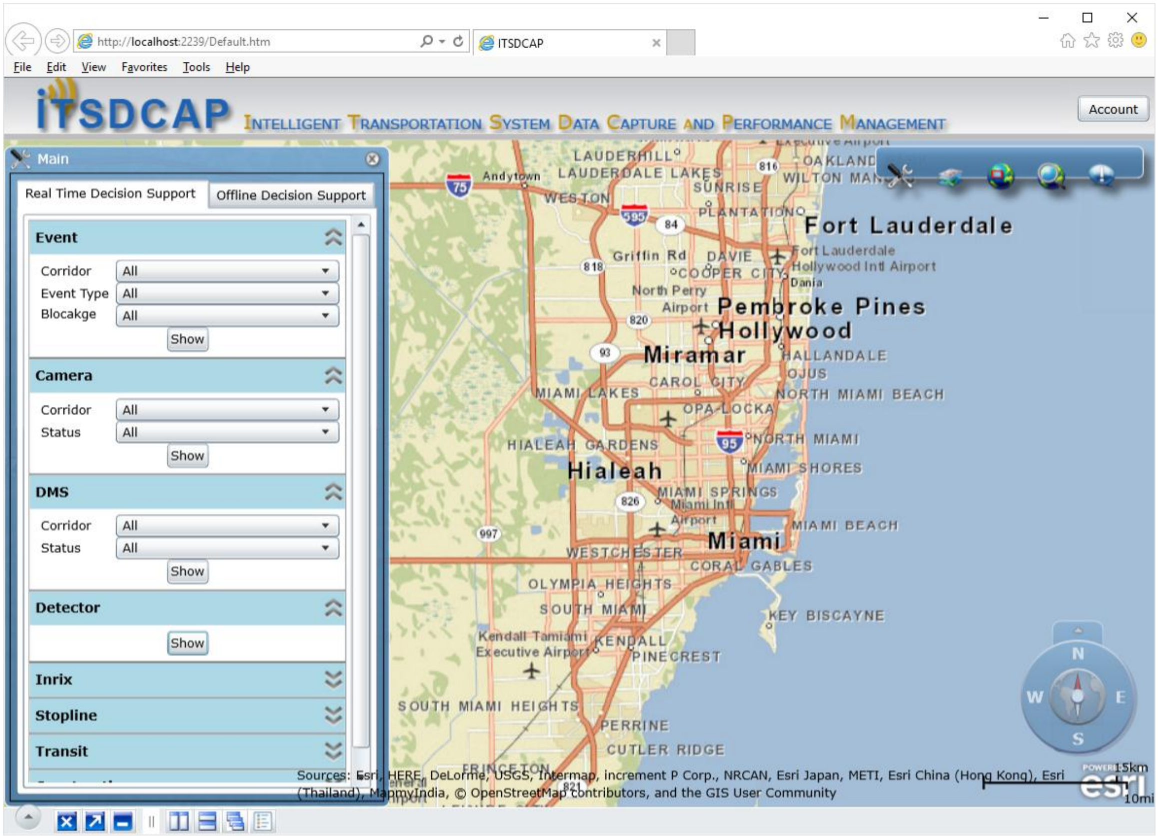


Figure 1-1 Main Interface of the ITSDCAP Website

Figures 1-2 to 1-4 show some examples of the user interface with results. Figure 1-2 shows the real-time event and detector data in the map for FDOT District 6. Figure 1-3 shows the average speed of I-95 NB from 4:00 PM to 6:00 PM based on historical data of November, 2012. Figure 1-4 shows the unreliability contribution result for I-95 NB in March, 2012.

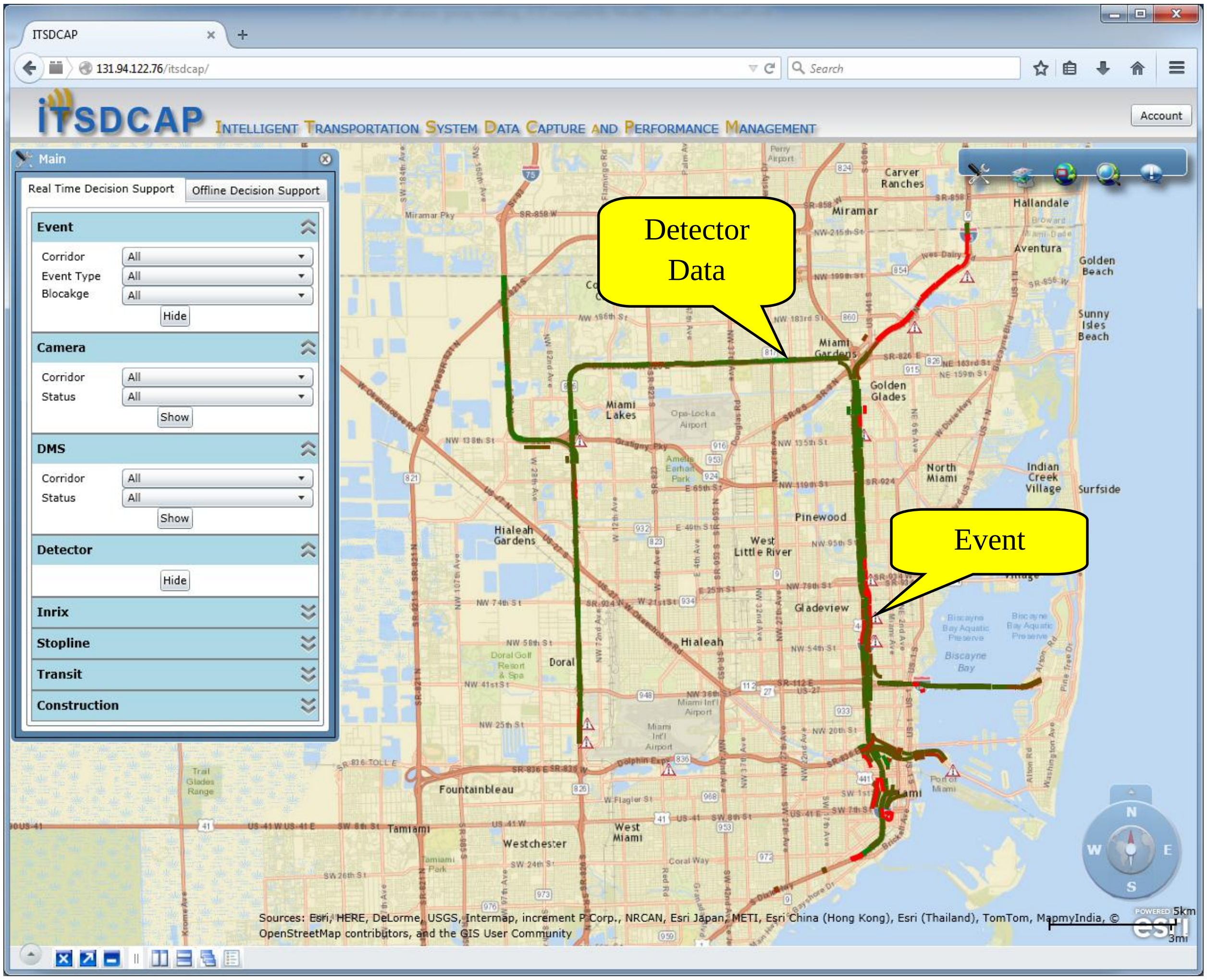


Figure 1-2 ITSDCAP Website Showing Real-time Event and Detector Data

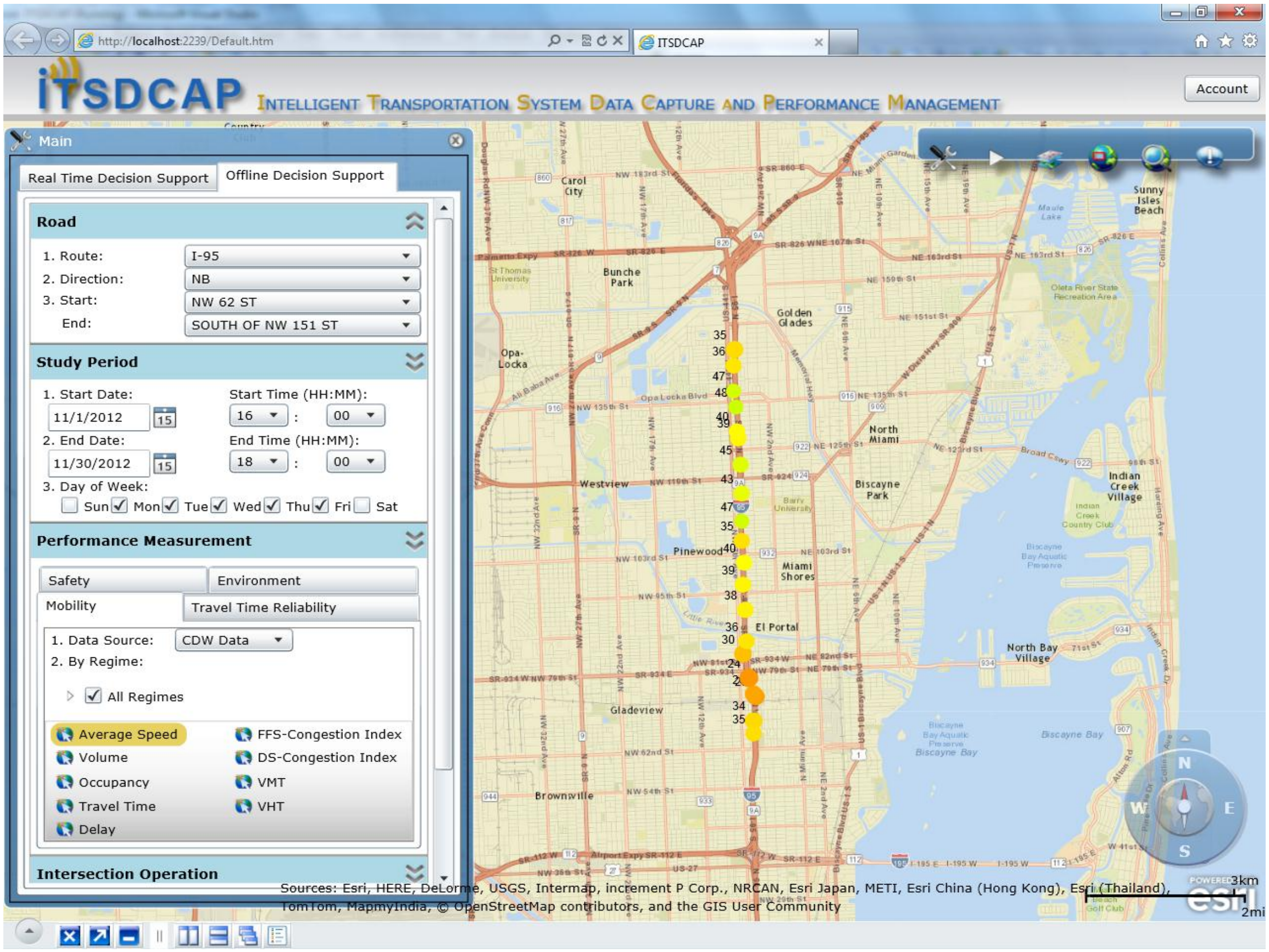


Figure 1-3 ITSDCAP Website Showing Off-line Average Speed for I-95 NB in Nov, 2012

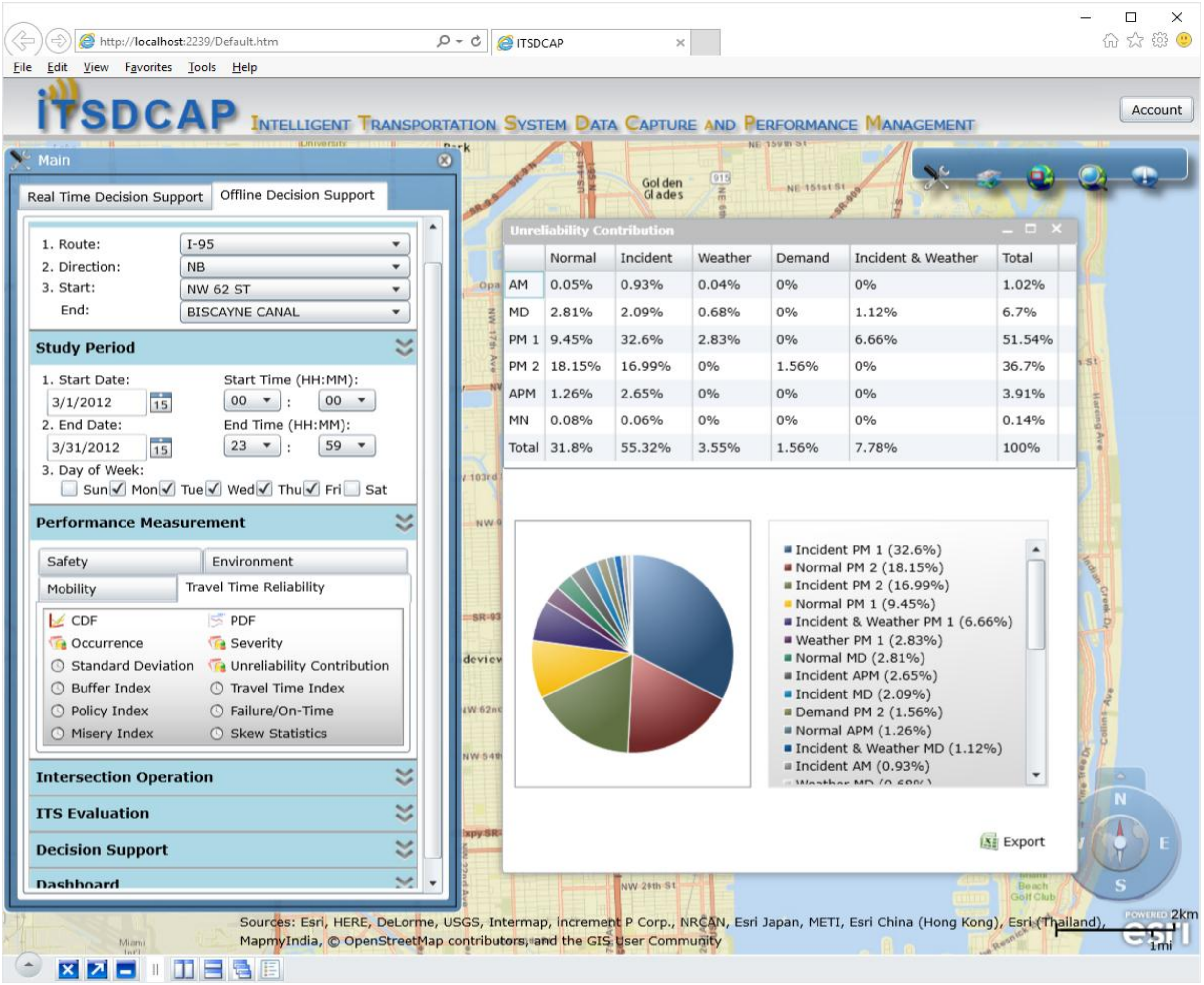


Figure 1-4 ITSDCAP Website Showing Unreliability Contribution for I-95 NB in March, 2012

2 PERFORMANCE MEASUREMENT AND DASHBOARD SUPPORT

2.1 Introduction

The importance of performance measurement and management has been increasingly realized by transportation agencies. The Moving Ahead for Progress in the 21st Century Act's (MAP-21) requirements for performance measurements have increased this realization. The Transportation System Management and Operations (TSM&O) program of the Florida Department of Transportation (FDOT) has seven objectives. These objectives are listed in the TSM&O Tier 2 business plan. Two important objectives of the program are "continually measure success of TSM&O by developing the ability to measure and report TSM&O performance gains" and "improve the performance of the network." To meet the above objectives, there is a need for the development of tools and methods for performance measurement estimation and management.

The wide deployment of point detectors and automatic vehicle identification (AVI) devices based on technologies such as Bluetooth readers, Wi-Fi readers, magnetometers, and electric toll tag readers provide a rich data environment for performance measurement estimation and monitoring. The performance measures that are most commonly monitored and reported by agencies include mobility and safety. Reliability measures have also been considered. A number of metrics for travel time reliability have been proposed and assessed by Strategic Highway Research Program 2 (SHRP2) reliability projects. Among these, the SHRP2 L02 project developed a detailed data-based travel time reliability monitoring procedure (Institute for Transportation Research and Education, et al., 2012 and 2013).

In a previous FDOT research project (Hadi et al., 2012), the Florida International University (FIU) research team developed an Intelligent Transportation System Data Capture and Performance Management (ITSDCAP) tool. This tool can capture data from multiple sources, estimate various performance measures (mobility, reliability, safety and environment), perform data mining techniques, support benefit-cost analysis, and allow for the visualization of data. However, as a desktop version tool, ITSDCAP required the installation of add-on software. In addition, it mainly focused on freeway corridor performance measurements. In this research project, the ITSDCAP tool was upgraded to a Web-based version that allows performance measurement of both freeways and arterials. In addition, the enhanced ITSDCAP allows for the creation of performance dashboards to support the operations of the TSM&O program, based on user requirements.

2.2 Review of Performance Dashboard

As part of the ITSDCAP enhancement effort, a review was conducted of Web-based dashboards that are being used by transportation agencies around the United States, as well as the performance measures listed in these dashboards. This review was meant to provide inputs regarding the potential formats and contents of the dashboards. It should be emphasized that it is anticipated that different agencies will select different dashboard designs and contents based on their individual requirements. ITSDCAP allows for the flexibility of producing different dashboards by the tool administrator based on agency requirements. The tool administrator refers to the team maintaining the ITSDCAP. An agency can contact the team and the team can work with the agency on designing and customizing the dashboard based on the needs of the agency. Different agencies can have their own dashboard styles.

2.2.1 Alaska Department of Transportation and Public Facilities (DOT&PF) Dashboard

The Alaska Department of Transportation and Public Facilities (DOT&PF) developed a Web-based dashboard to share their performance measures with the public (Alaska DOT&PF, 2011). Figure 2-1 shows a snapshot of this dashboard. As shown in this figure, this dashboard includes the following performance indicators:

- Number of centerline miles of National Highway System roads meeting department standards
- Traffic fatalities
- Number of occupational injuries and illnesses
- Seasonally closed airports
- Percentage change in deck area of structurally deficient or functionally obsolete bridges
- Alaska Marine Highway on-time departures
- Alaska Marine Highway vessel car deck capacity utilization
- Rural airport revenues
- Aeronautical surveys for rural airports
- Commercial motor vehicle weight compliance rate
- Percentage of administrative and engineering costs on projects

Among the 11 measures listed above, the first ten items are maintenance and operations-related safety measures, and the last is an infrastructure-related measure.

More detailed information about each indicator can be retrieved by clicking on the desired gauge. An example of the indicators is demonstrated in Figure 2-2, which presents a snapshot of the traffic fatality measure from the Alaska DOT&PF dashboard. The importance and the calculation method of the traffic fatality measure are explained in the dashboard. Also, the period of analysis, actual number of fatalities, and the target values are shown in a bar chart. The year with an actual number of fatalities greater than the target values are indicated by a red square.

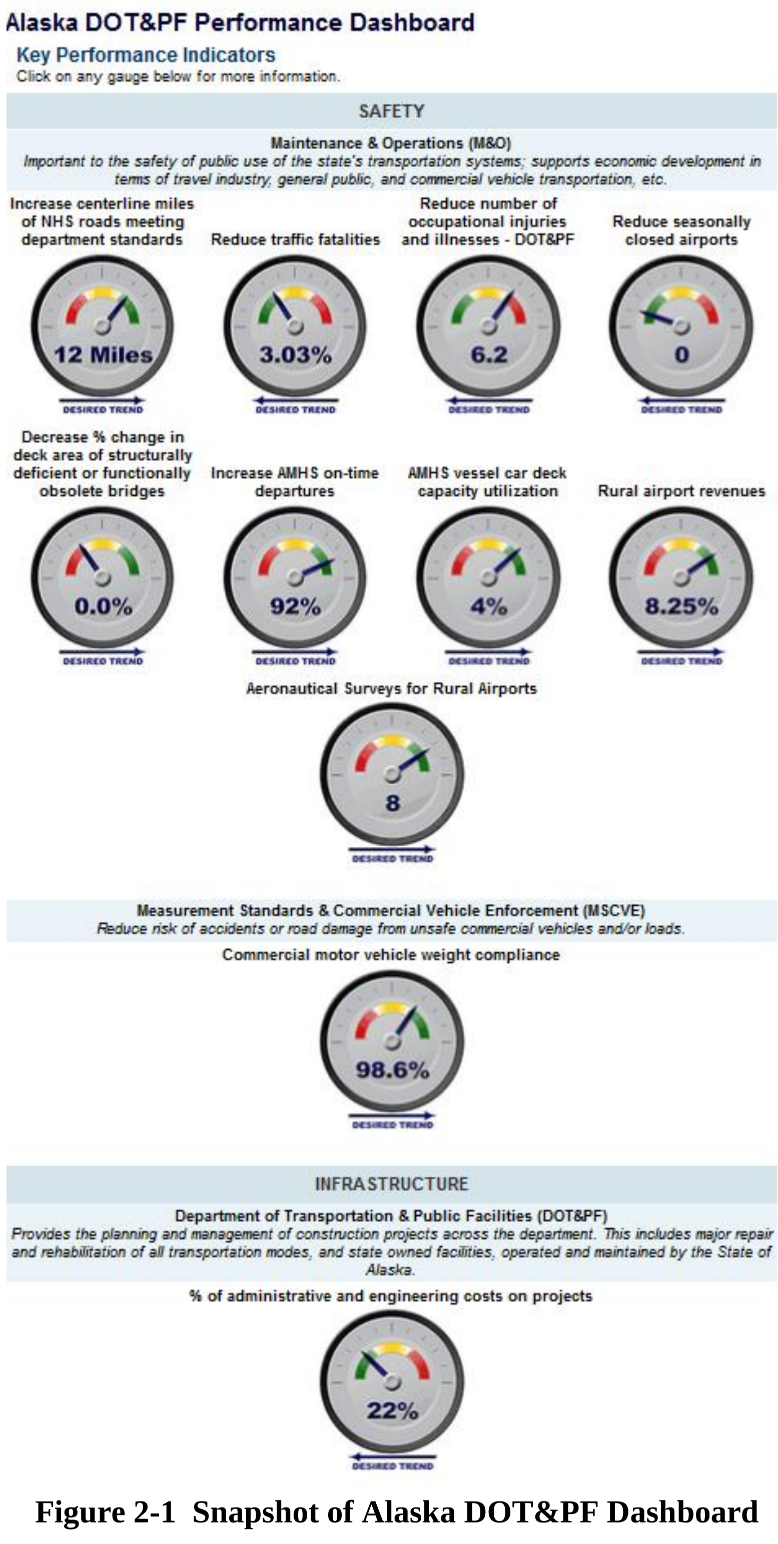
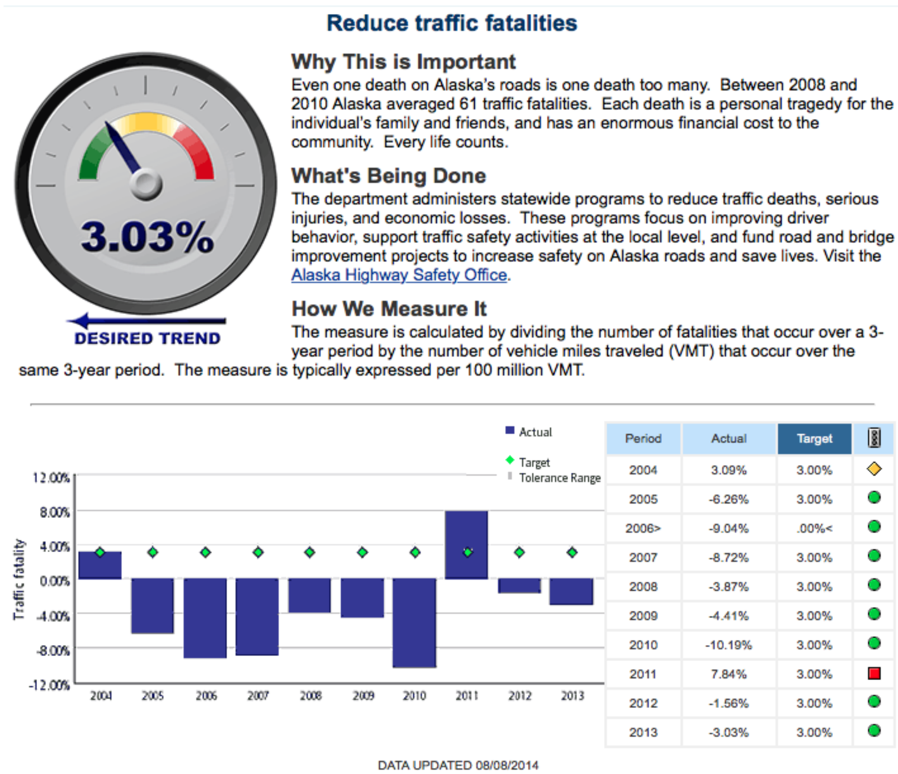


Figure 2-1 Snapshot of Alaska DOT&PF Dashboard



Period	Actual	Target	
2004	3.09%	3.00%	
2005	-6.26%	3.00%	
2006>	-9.04%	.00%<	
2007	-8.72%	3.00%	
2008	-3.87%	3.00%	
2009	-4.41%	3.00%	
2010	-10.19%	3.00%	
2011	7.84%	3.00%	
2012	-1.56%	3.00%	
2013	-3.03%	3.00%	

Figure 2-2 Snapshot of Traffic Fatality Measure from Alaska DOT&PF Dashboard

2.2.2 District of Columbia Department of Transportation (DDOT) Dashboard

Six sets of performance indicators are included in the District of Columbia's Department of Transportation's (DDOT) District Transportation Access Portal of Washington, DC, including safety, roadway conditions, project on-time, transit on-time performance, finance, and customer service (DDOT, 2015). Figure 2-3 shows an example of this dashboard. Similar to the dashboard of Alaska DOT&PF, the user can access tables or charts, with a summary of the measures used for each indicator by clicking on each gauge. For instance, the "Safety" gauge presents the user with the information of the number of crashes, pedestrian fatalities, bicycle fatalities, motor cycle fatalities and overall fatalities, as shown in Figure 2-4.

Figure 2-3 Snapshot of DDOT Transportation Access Portal

District Department of Transportation

crash report

2011 Crash Summary

Performance Measures	1st Quarter	2nd Quarter	3rd Quarter	4th Quarter	Total
Number of Crashes	4,061	4,773	4,467	4,650	17,951
Number of Pedestrian Fatalities	3	2	2	2	9
Number of Bicycle Fatalities	0	2	0	0	2
Number of Motor Cycle Fatalities	2	1	1	1	5
Number of Overall Fatalities	10	7	7	3	27

2012 Crash Summary

Performance Measures	1st Quarter	2nd Quarter	3rd Quarter	4th Quarter[1]	Total
Number of Crashes	4,463	4,860	4,619	2,589	16,531
Number of Pedestrian Fatalities	0	1	0	1	2
Number of Bicycle Fatalities	0	0	1	0	1
Number of Motor Cycle Fatalities	1	1	2	0	4
Number of Overall Fatalities	6	5	3	6	20

Note: [1] Includes crash data from October 1st, 2012 to November 23rd, 2012.

District Department of Transportation 55 M Street, SE, Suite 400 Washington, DC 20003

Figure 2-4 Safety Measures from the DDOT Transportation Access Portal

2.2.3 Georgia Department of Transportation (GDOT) Dashboard

To support the state's strategic plan, the Georgia Department of Transportation (GDOT) launched a performance management dashboard, which contains three components: Safety, infrastructure, and planning & construction, as shown in Figure 2-5 (GDOT, 2014).

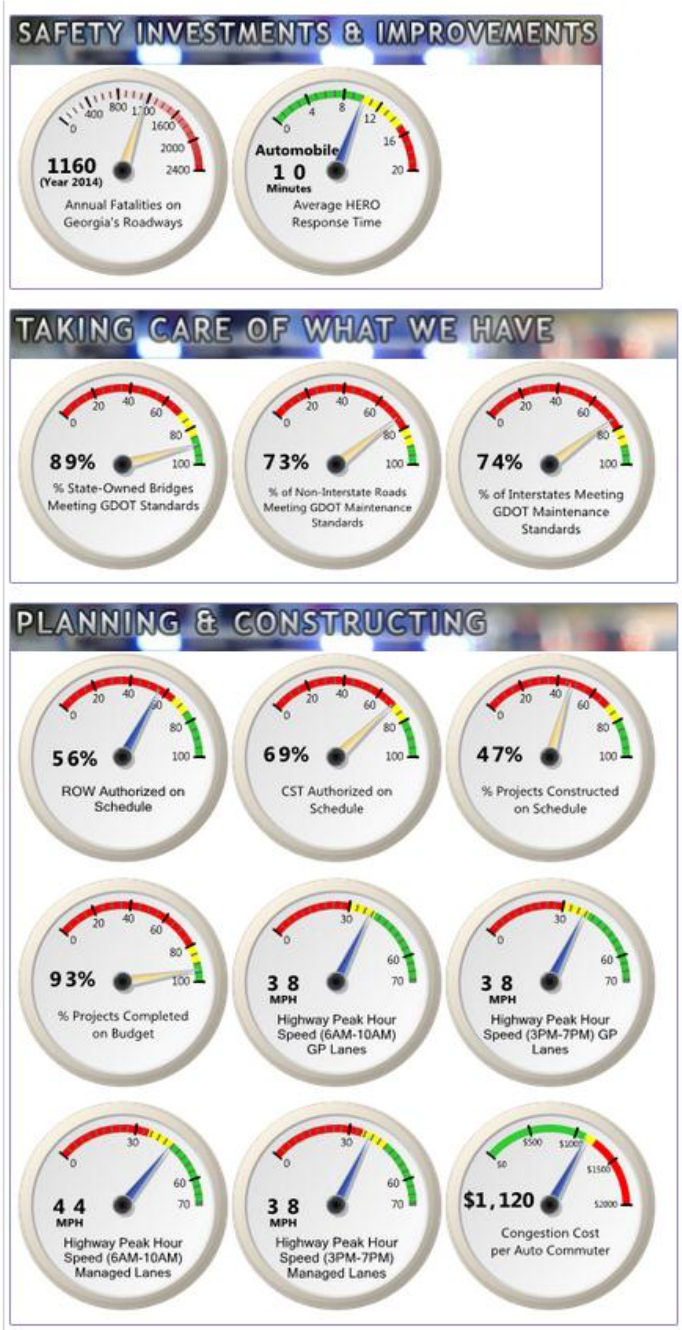


Figure 2-5 Snapshot of GDOT Performance Measure Dashboard

A further description of each performance indicator and the strategic objective for this indicator, along with the corresponding chart, is also provided by the GDOT. Figure 2-6 shows the display for the number of annual fatalities. Following is a list of all of the performance indicators included in the GDOT dashboard.

- Number of fatalities on Georgia’s roadways
- Average service patrol response time
- Percentage of state-owned bridges meeting GDOT standards
- Percentage of state-owned non-interstate roads meeting maintenance standards
- Percentage of interstates meeting maintenance standards
- Right-of-way authorized on schedule
- CST authorized on schedule
- Percentage of projects constructed on schedule
- Percentage of projects completed within the budget
- Morning peak-hour freeway speeds (general purpose lanes)
- Evening peak-hour freeway speeds (general purpose lanes)
- Morning peak-hour speeds (managed lanes)
- Evening peak-hour speeds (managed lanes)
- Congestion costs per auto commuter

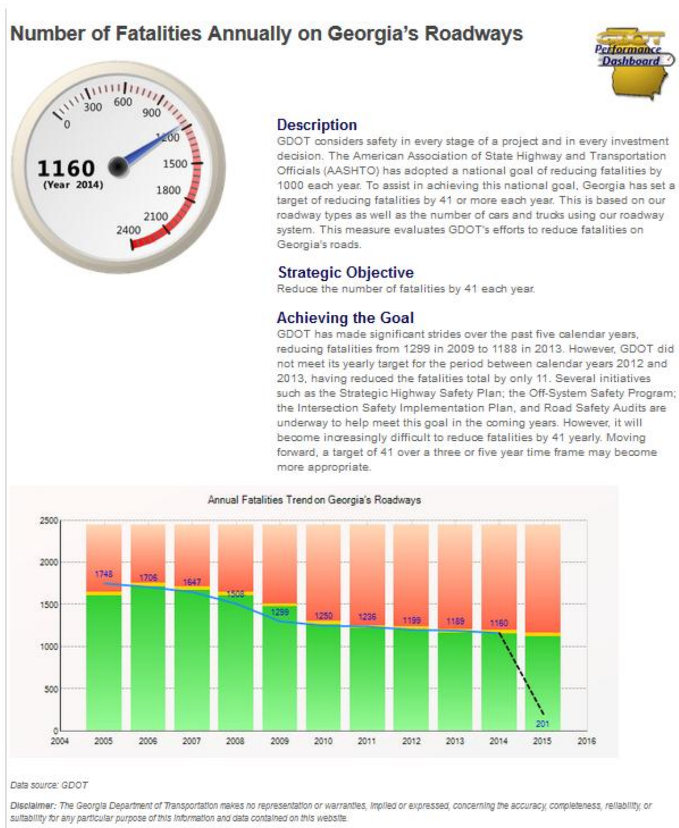


Figure 2-6 Number of Annual Fatalities Display of the GDOT Dashboard

2.2.4 Idaho Department of Transportation (IDOT) Dashboard

Ten performance measure indicators are included in the transportation system dashboard of the Idaho Department of Transportation (Idaho Transportation Department, 2015), as follows:

- Five-year fatality rate
- Percent of time highways clear of snow/ice during winter storms
- Percent of pavement in good or fair conditions
- Percent of bridges in good condition
- Percent of highway project designs completed on time
- Final construction cost as a percent of the contract award
- Construction cost at award as a percent of budget
- Days to process vehicle titles
- DMV transactions processed on the internet

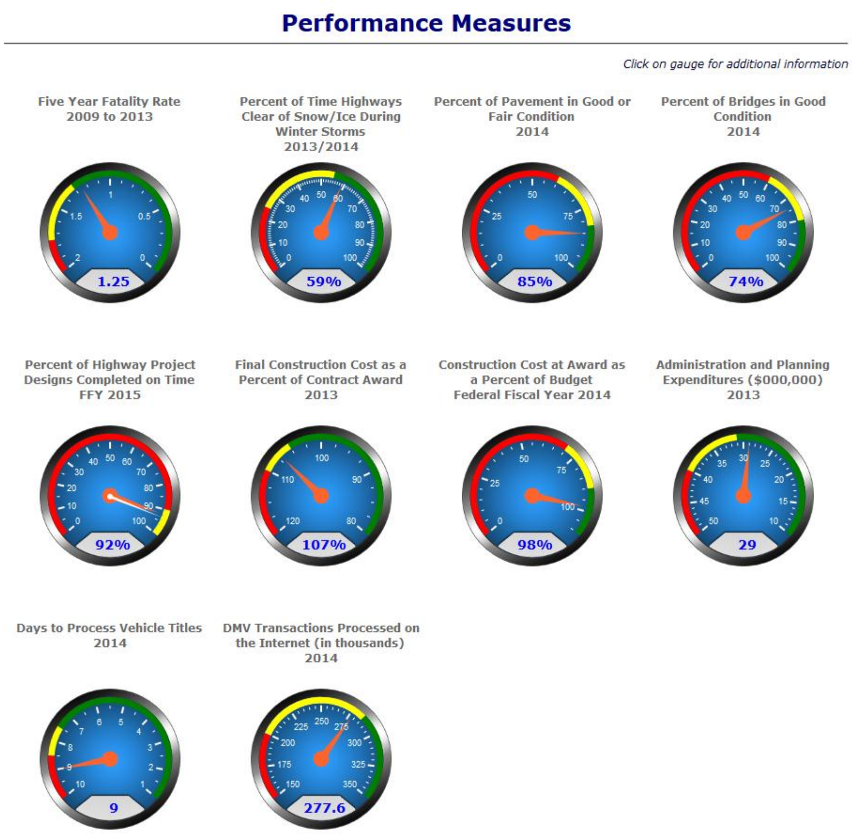
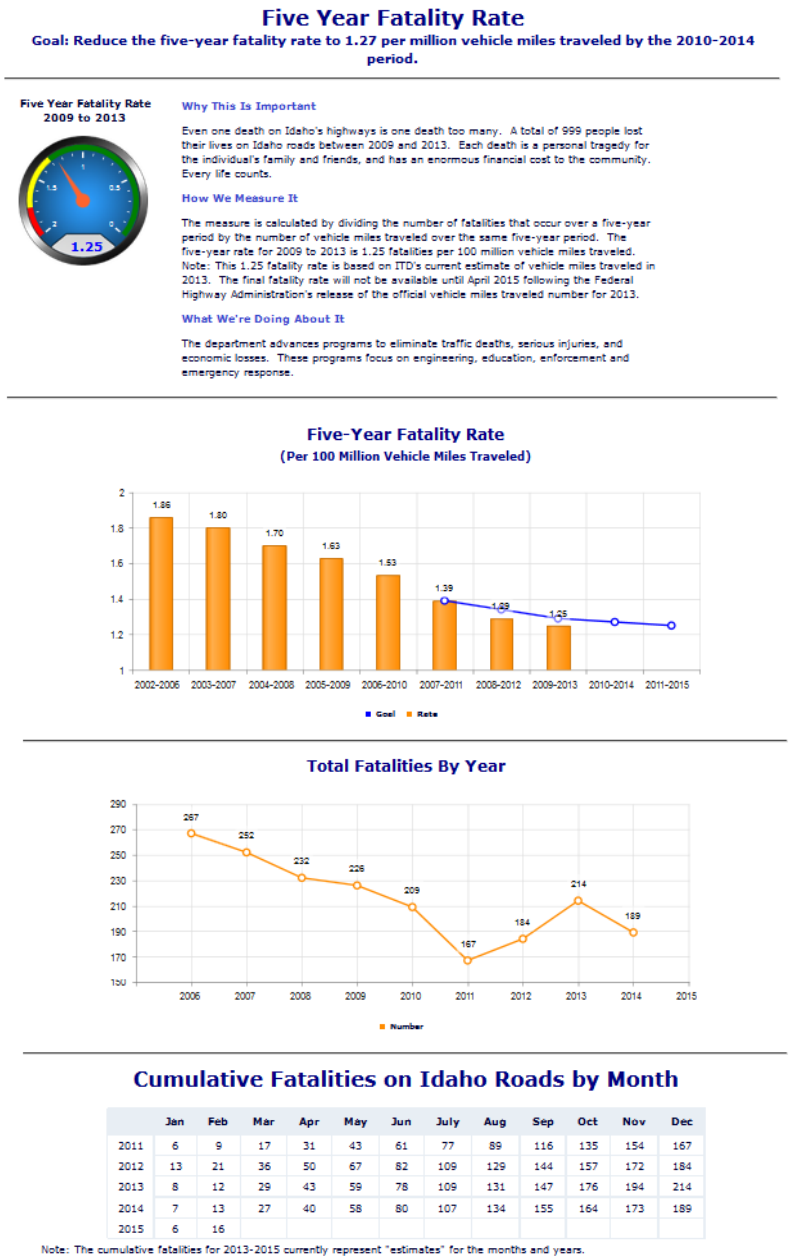


Figure 2-7 Idaho DOT Performance Measures

Similar to other reviewed dash boards, the user can click on each gauge and obtain additional information about each measure. As shown in Figure 2-8, the display includes a target for each measure, the importance of the measure, how it is calculated, what the state is doing to improve it, and associated charts and graphics.



	Jan	Feb	Mar	Apr	May	Jun	July	Aug	Sep	Oct	Nov	Dec
2011	6	9	17	31	43	61	77	89	116	135	154	167
2012	13	21	36	50	67	82	109	129	144	157	172	184
2013	8	12	29	43	59	78	109	131	147	176	194	214
2014	7	13	27	40	58	80	107	134	155	164	173	189
2015	6	16										

Note: The cumulative fatalities for 2013-2015 currently represent "estimates" for the months and years.

Figure 2-8 Example of Five-Year Fatality Rate displayed in Idaho DOT Dashboard

2.2.5 North Carolina Department of Transportation Dashboard

An organizational performance dashboard was developed by the North Carolina Department of Transportation (North Carolina DOT, 2015). This dashboard consists of five main performance measures:

- Fatality rate
- Incident duration
- Infrastructure health
- Delivery rate
- Employee engagement

Figure 2-9 Snapshot of North Carolina DOT Organizational Performance Dashboard

Additional performance measures are also available in gauge, table, and chart formats by clicking the corresponding link, as shown in Figure 2-9. These measures can be filtered by counties or requested as statewide measures. The following is a complete list of all of the performance measures that are included in the North Carolina DOT dashboard.

- Number of crashes
- Number of fatalities
- Number of injuries
- Yearly statistics (including the information on number of crashes, fatalities, injuries, vehicle-miles traveled, crash rate, fatality rate, and injury rate)
- Incident clearance time
- Ferry service reliability
- Rail service customer satisfaction

- Public transit utilization
- Highway reliability
- Statewide infrastructure health
- Statewide yearly statistics of bridge health, pavement condition, and roadside feature condition
- Percentage of plans completed and bids opened on time
- Percentage of right-of-way plans completed on time
- Percentage of construction projects completed on schedule
- Percentage of construction projects completed on budget
- Average state environmental compliance score
- Employee engagement in terms of commitment, discretionary effort, and intent to stay

2.2.6 Utah Department of Transportation Dashboard

The public dashboard developed by the Utah Department of Transportation reflects their four strategy goals: Preserve infrastructure, optimize mobility, zero fatalities, and strengthen the economy (Utah DOT, 2015). The measures shown in their dashboards are:

- Current interstate travel times
- Number of fatalities
- Percentage of construction projects on time
- Percentage of construction project on budget
- Employee associated with construction projects
- Total projects under construction
- Historic and predicted pavement conditions
- Historic and predicted bridge conditions

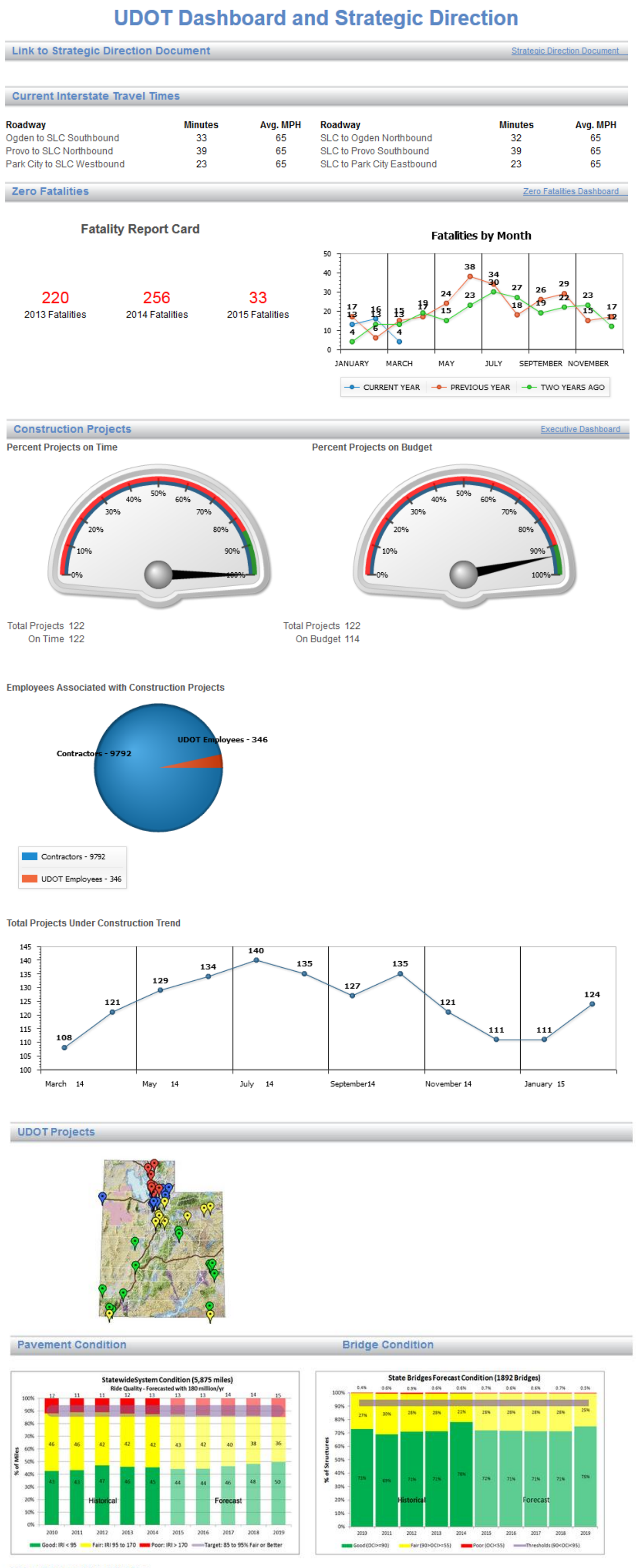


Figure 2-10 Snapshot of Utah DOT Performance Dashboard

2.2.7 Virginia Department of Transportation Dashboard

Seven key performance measures are clearly presented on the front page of the Virginia Department of Transportation (VDOT) dashboard (VDOT, 2015), as shown in Figure 2-11. The measures are divided into two groups, highway-related and VDOT performance-related measures, as listed below.

- Highway
    1. Performance – congestion on interstates daily updates
    2. Safety – highway deaths since the beginning of the year
    3. Condition – quality of road surface
    4. Finance – year-to-date planned vs. actual expenditures (variance)
- VDOT Performance
    1. VDOT management – management performance areas
    2. Projects – on time
    3. Citizen survey results – interaction with the public

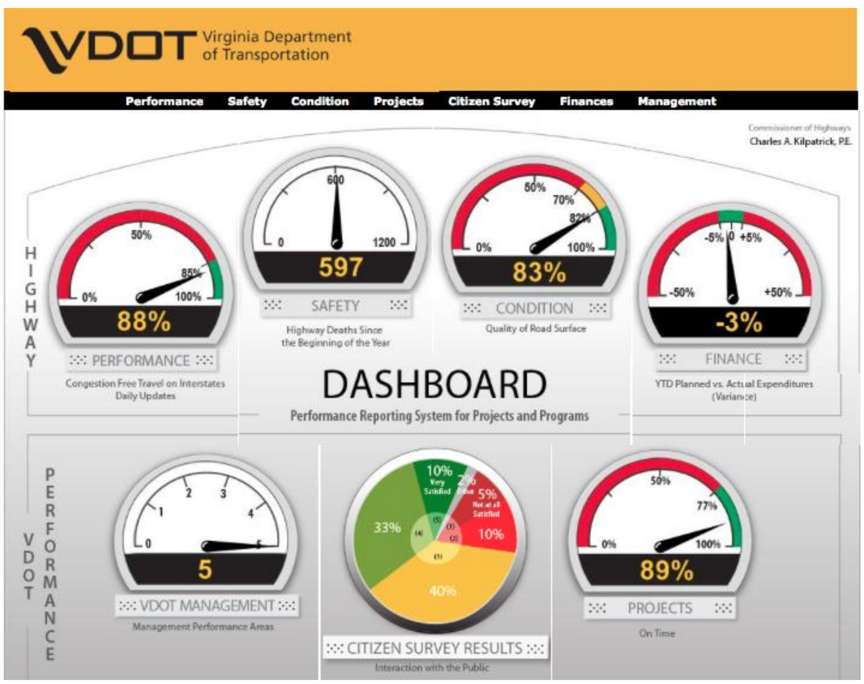


Figure 2-11 Snapshot of Virginia DOT Dashboard

Each measure has additional information associated with it. Figure 2-12 presents an overview of highway performance measures. As shown in this figure, the dashboard reports the percentage of vehicle miles in three levels of services (good, marginal, and poor), which indicates congestion at various interstate locations. The percentages of travel above 45 mph during the AM and PM peak periods are presented to reflect the travel speed performance. The average travel times during the peak hours are also listed and compared to travel time at the speed limits. In addition, the distribution of incident durations (in terms of percentage of incidents and number of incidents) and the average annual hours of delay per traveler during the peak periods can be retrieved, as shown in Figures 2-13 and 2-14. Figure 2-15 displays the safety measures in the dashboard. As shown in this figure, the number of crashes, injuries, deaths, and work zone crashes in the past 12 months and last 3 years are reported in both chart and table formats. Additional information related to the other measures can also be retrieved by clicking the corresponding gauges.

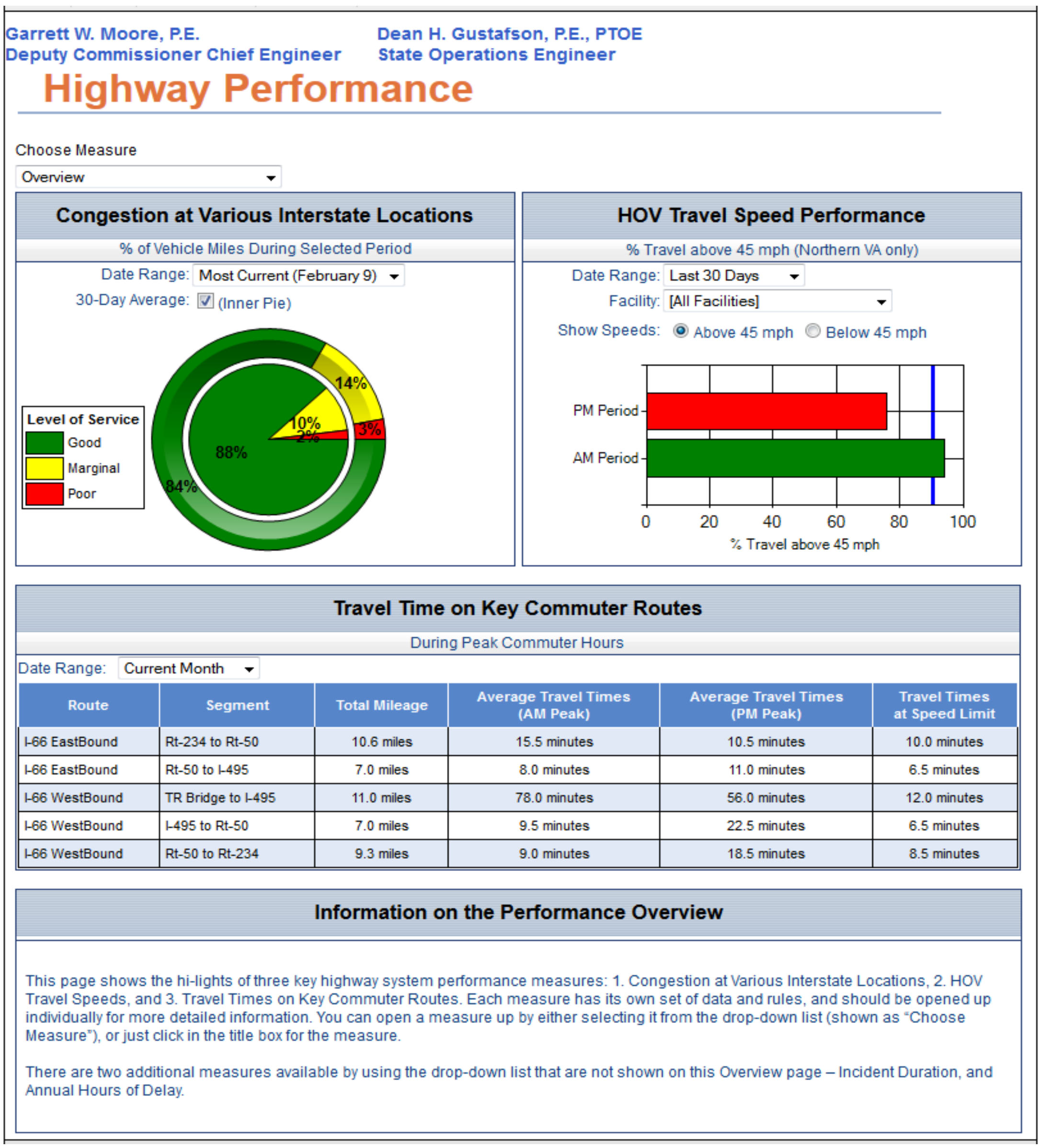


Figure 2-12 Overview of Virginia DOT Highway Performance

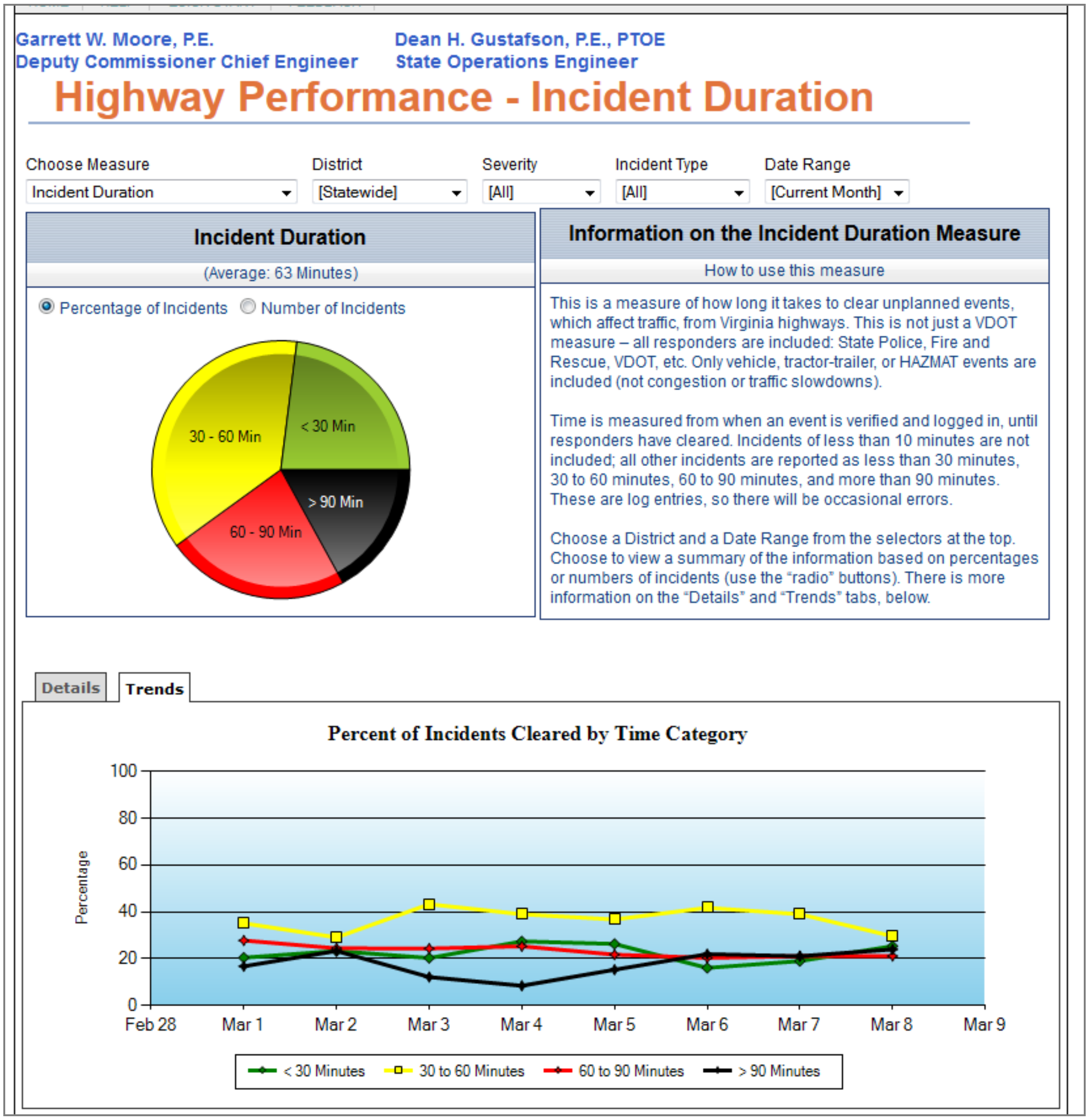


Figure 2-13 Incident Duration of Virginia DOT Highway Performance

Garrett W. Moore, P.E.
Deputy Commissioner Chief Engineer

Dean H. Gustafson, P.E., PTOE
State Operations Engineer

Highway Performance - Hours of Delay

Choose Measure

Hours of Delay

Annual Hours of Delay per Traveler

Avg Annual Peak Travel Time Delay per Traveler

Data Year	Report Year	Washington DC (Proxy for NoVA)	Virginia Beach	Richmond
2011	2012	67	43	29
2010	2012	66	43	29
2009	2012	64	40	27
2008	2012	65	44	23
2007	2012	79	50	24
2006	2012	73	53	24
2005	2012	74	52	24
2004	2012	74	52	24
2003	2012	73	53	22
2002	2012	69	54	20

Hours of Delay

How to use this measure

The Texas Transportation Institute (TTI) publishes its annual Urban Mobility Report (UMR) tracking peak-travel-time congestion delays experienced in major urban areas. The grid to the left lists recent UMR 'annual hours of delay' results for Virginia's three major urban areas; Washington DC (proxy for Northern Virginia), Virginia Beach (including Hampton Roads metro area) and Richmond's metro area.

Note that there is a one to two year gap between a UMR's source-data dates and the date the UMR is released (i.e. the 2010 UMR, released in Jan 2011, is based on calendar year 2009 data). The line graph below provides historical Annual Hours of Delay results as reported in the recent UMR, showing trends by data-year for each metro area.

TTI allows access to an online version of the UMR report at: http://mobility.tamu.edu/ums/report/

Trends

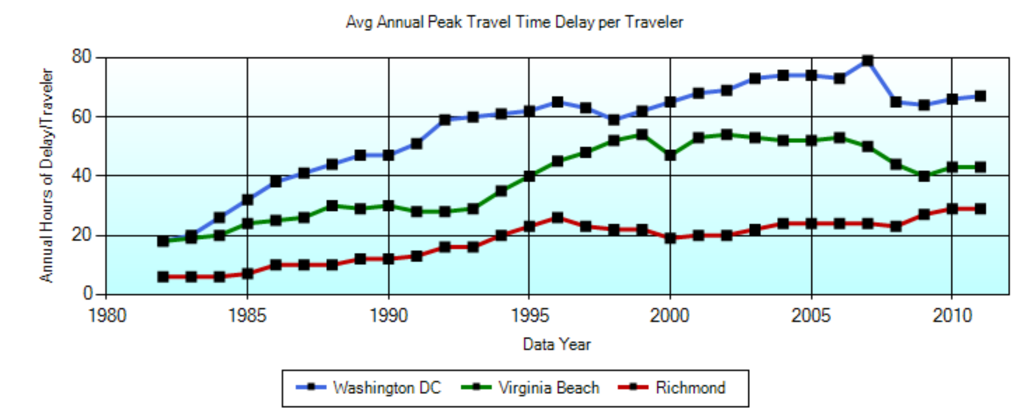
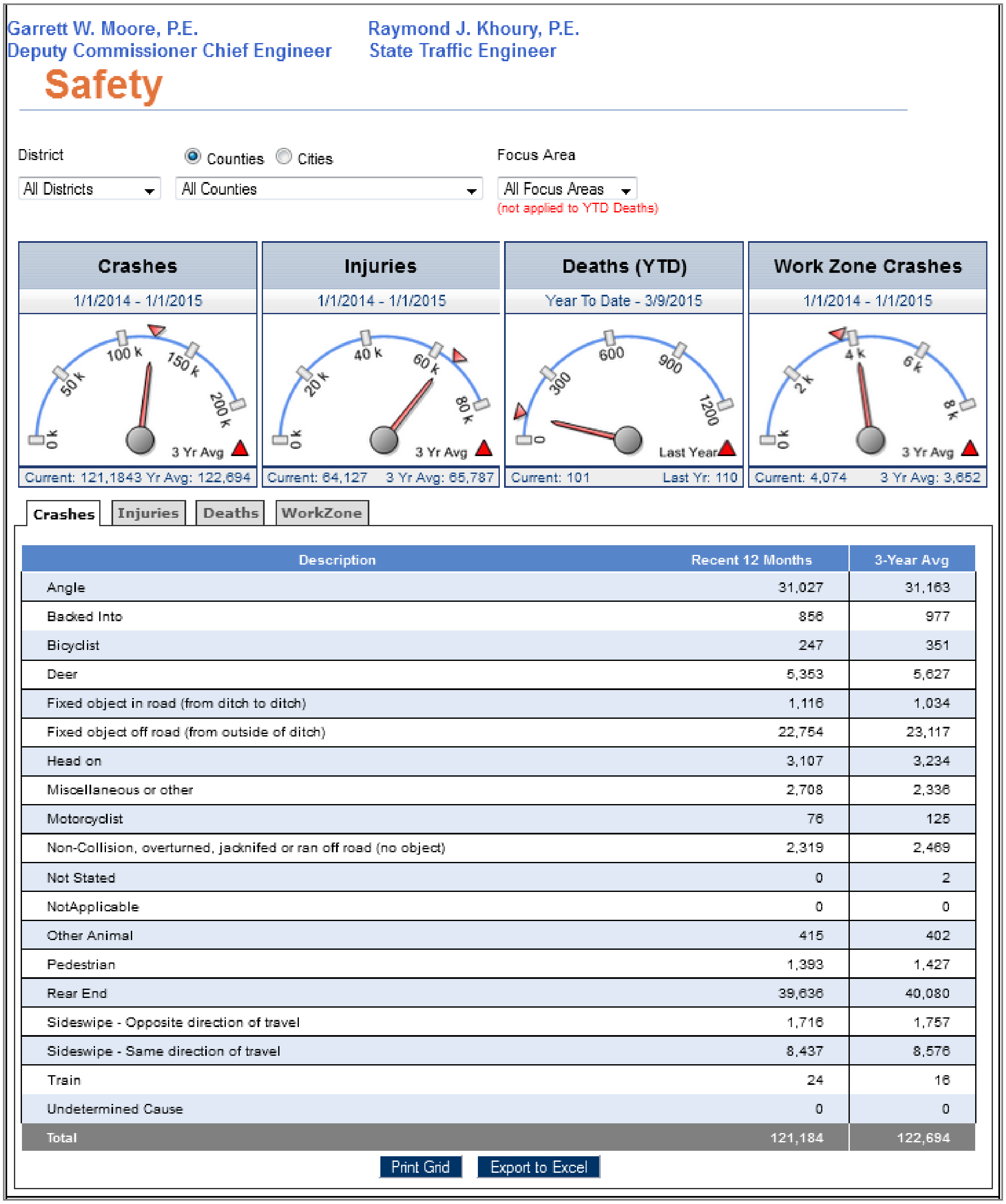


Figure 2-14 Hours of Delay of Virginia DOT Highway Performance


Garrett W. Moore, P.E.
Deputy Commissioner Chief Engineer

Raymond J. Khoury, P.E.
State Traffic Engineer

Safety

District: All Districts | Counties / Cities: All Counties | Focus Area: All Focus Areas (not applied to YTD Deaths)

Crashes	Injuries	Deaths (YTD)	Work Zone Crashes
1/1/2014 - 1/1/2015	1/1/2014 - 1/1/2015	Year To Date - 3/9/2015	1/1/2014 - 1/1/2015
Current: 121,1843 Yr Avg: 122,694	Current: 64,127 3 Yr Avg: 65,787	Current: 101 Last Yr: 110	Current: 4,074 3 Yr Avg: 3,652

Crashes | Injuries | Deaths | WorkZone

Description	Recent 12 Months	3-Year Avg
Angle	31,027	31,163
Backed Into	856	977
Bicyclist	247	351
Deer	5,353	5,627
Fixed object in road (from ditch to ditch)	1,116	1,034
Fixed object off road (from outside of ditch)	22,754	23,117
Head on	3,107	3,234
Miscellaneous or other	2,708	2,336
Motorcyclist	76	125
Non-Collision, overturned, jacknifed or ran off road (no object)	2,319	2,469
Not Stated	0	2
NotApplicable	0	0
Other Animal	415	402
Pedestrian	1,393	1,427
Rear End	39,636	40,080
Sideswipe - Opposite direction of travel	1,716	1,757
Sideswipe - Same direction of travel	8,437	8,576
Train	24	16
Undetermined Cause	0	0
Total	121,184	122,694

Print Grid | Export to Excel

Figure 2-15 Virginia DOT Safety Measures

2.2.8 *Wyoming Department of Transportation Dashboard*

Compared to the other state DOT dashboards, the dashboard offered by the Wyoming DOT is relatively simple (Wyoming DOT, 2015). Figure 2-16 shows a snapshot of this dashboard. The reported measures in this figure include:

- Customer satisfaction
- Number of fatalities
- Seat belt usage
- Road pavement conditions
- Airport pavement conditions

However, no additional options or filters are available for further analysis.

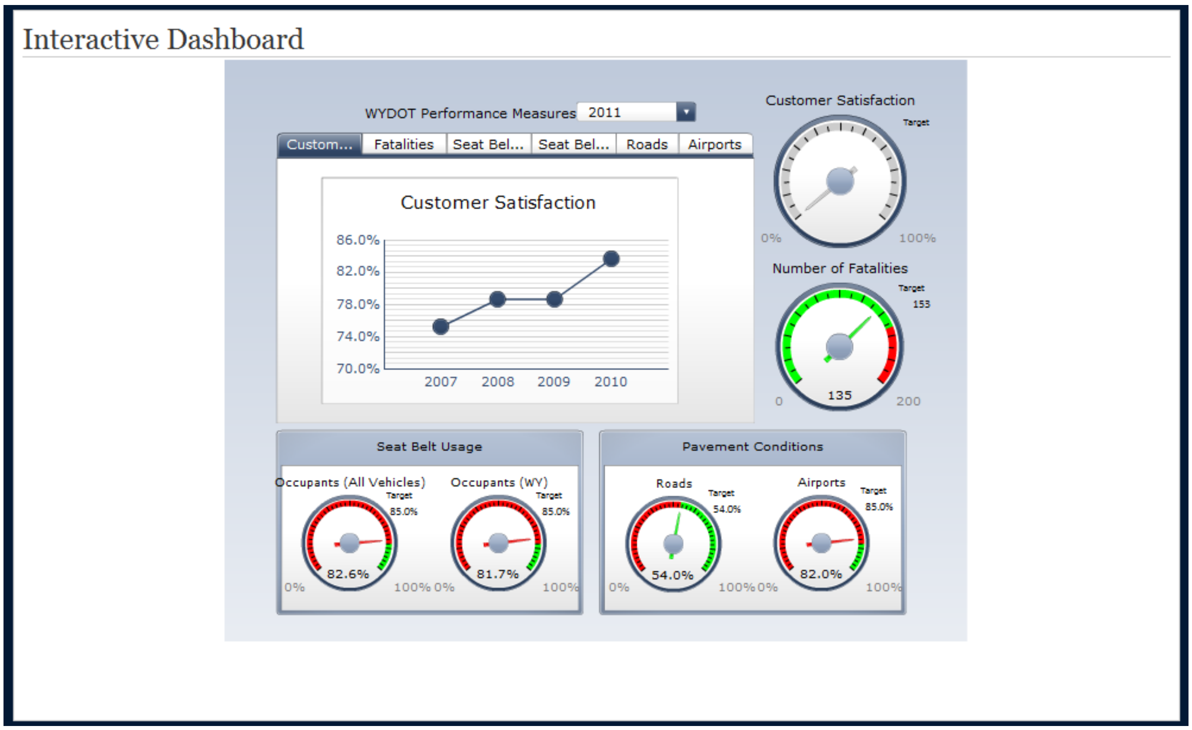


Figure 2-16 Wyoming DOT Dashboard

2.2.9 *Florida Department of Transportation Dashboard*

The performance dashboard developed by the Florida Department of Transportation (FDOT) is composed of five sets of performance measures, plus some information about the transportation system, system usage, work program plan, and major projects, as shown in Figure 2-17 (FDOT, 2015). Detailed information for each set of performance measures is presented on a separate webpage. For example, Figures 2-18 and 2-19 present the additional webpages for safety and mobility measures. As shown in these two figures, the safety measures include a 5-year average of the number of fatalities, serious injuries, pedestrian and bicyclist fatalities, pedestrian and bicyclist serious injuries, motorcycle fatalities, and motorcycle serious injuries. The mobility measures consist of capacity improvements, public transit ridership, and incident management in terms of average incident clearance time. Each measure provides a brief description, target value, current result, and a colored light indicating the level of satisfaction with the performance.

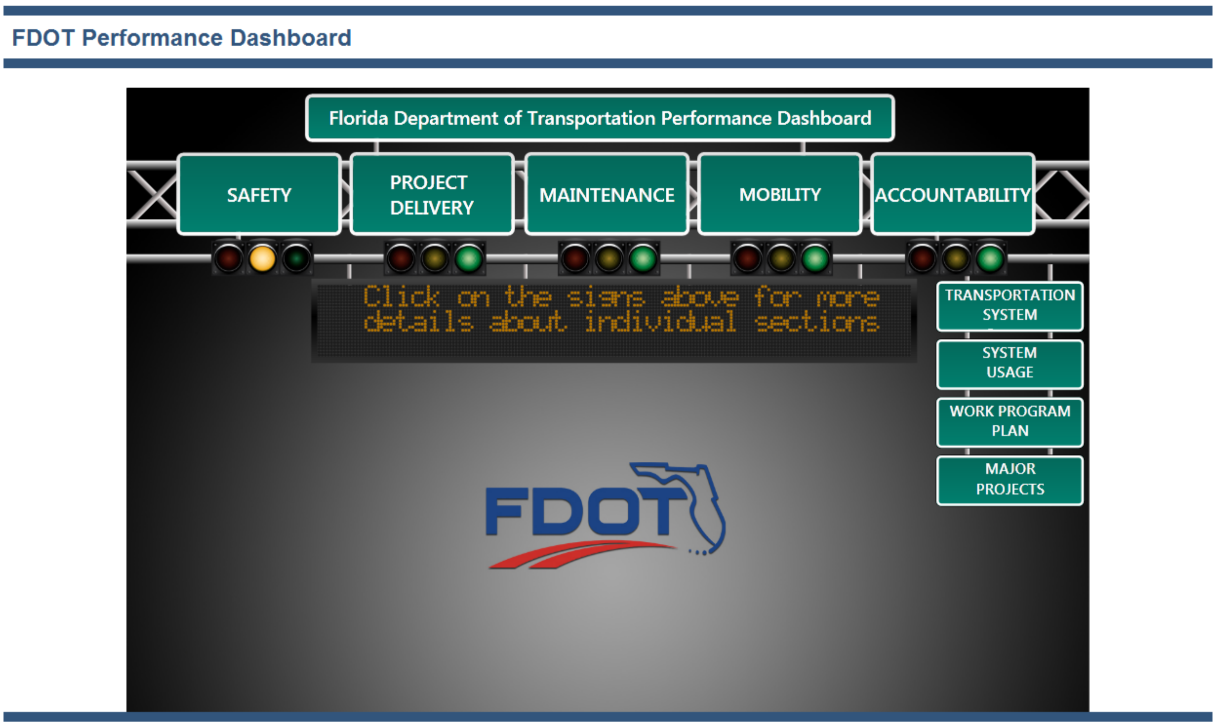


Figure 2-17 Florida DOT Performance Dashboard

FDOT Performance Dashboard

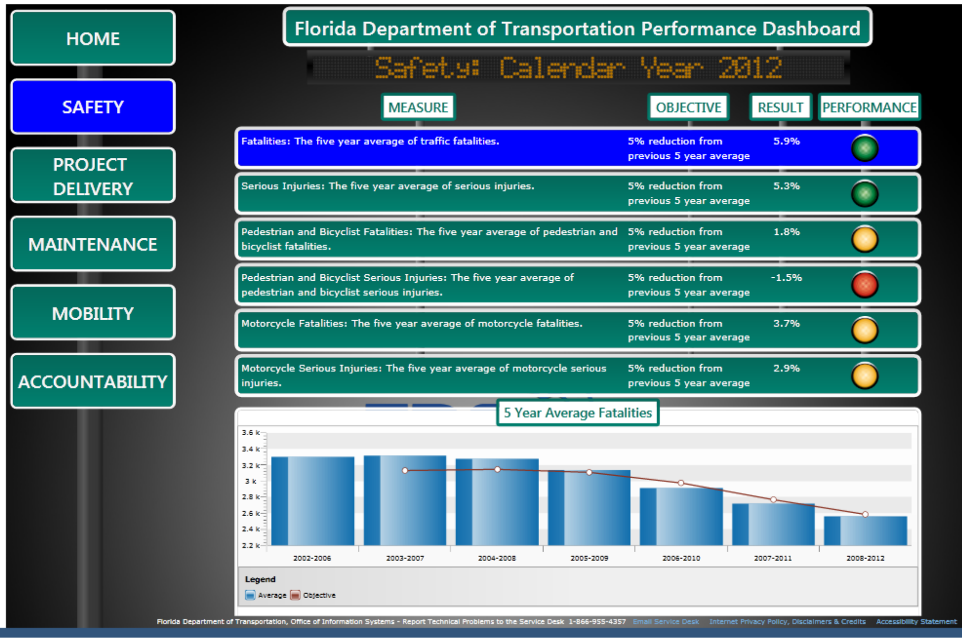


Figure 2-18 Florida DOT Performance Dashboard - Safety

FDOT Performance Dashboard

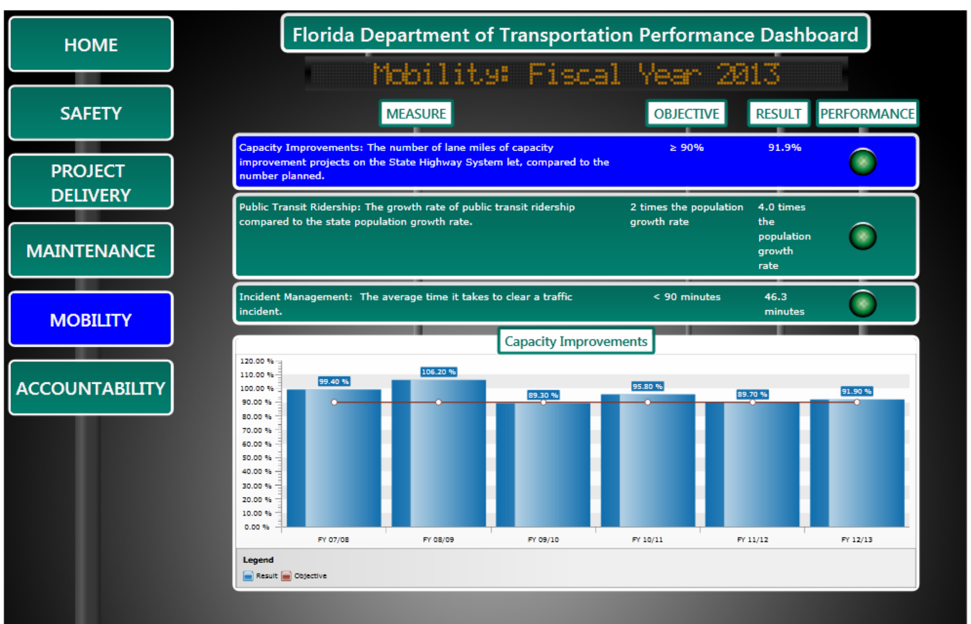


Figure 2-19 Florida DOT Performance Dashboard - Mobility

2.2.10 Regional Integrated Transportation Information System (RITIS) Dashboard

The Regional Integrated Transportation Information System (RITIS), developed by the University of Maryland, provides performance charts and performance summaries for predefined roadways (University of Maryland CATTI Lab, 2015). Figures 2-20 and 2-21 show a screenshot of these two functions. The performance measures that can be selected in the performance charts include the following:

- Speed
- Historical average speed
- Comparative speed (the measured speed defined as a percentage of the historical average speed)
- Congestion (the measured speed as a percentage of free-flow speed)
- Historical average congestion (defined as historic average speed as a percentage of free-flow speed)
- Buffer time

The reported measures in the performance summaries include:

- Buffer time
- Buffer index
- Planning time
- Planning time index
- Speed
- Travel time
- Travel time index

In addition, the RITIS website allows users to create their own dashboards. Figure 2-22 shows the dashboard options. Note that currently, only two options are available, speed and travel time table, and the ranked bottleneck table. Figures 2-23 and 2-24 illustrate the processes to generate these two types of dashboard.

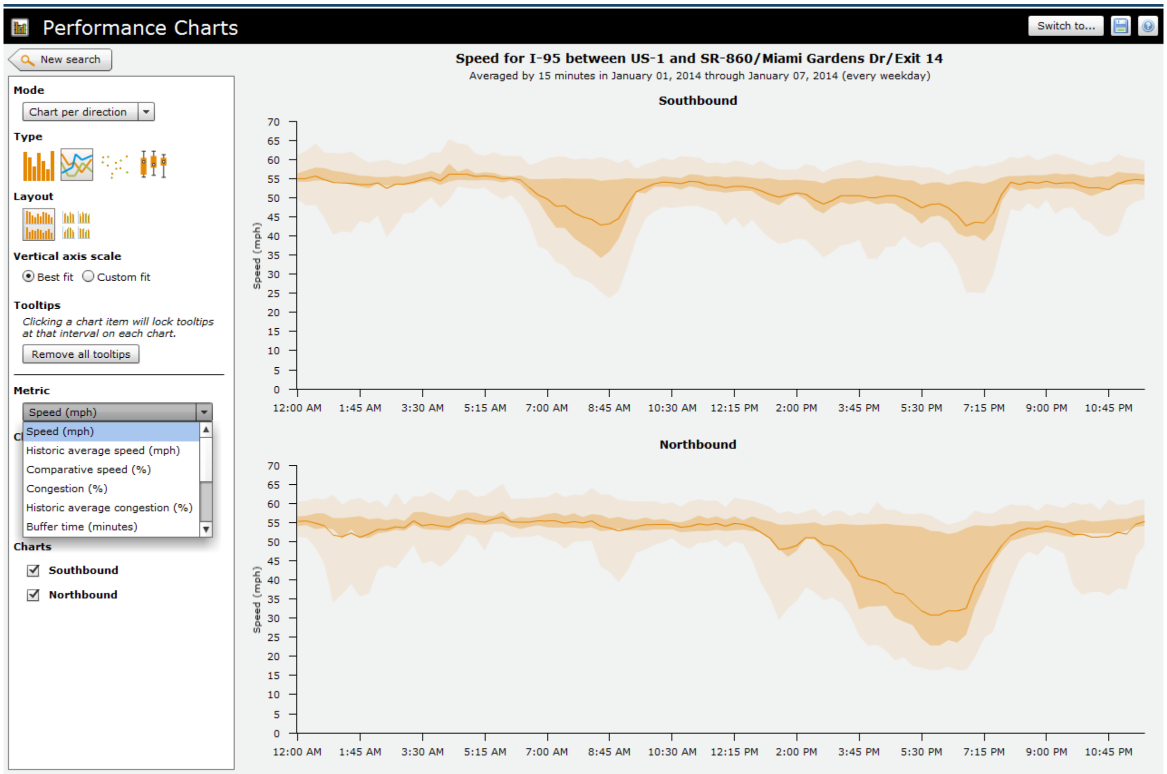


Figure 2-20 RITIS Performance Charts

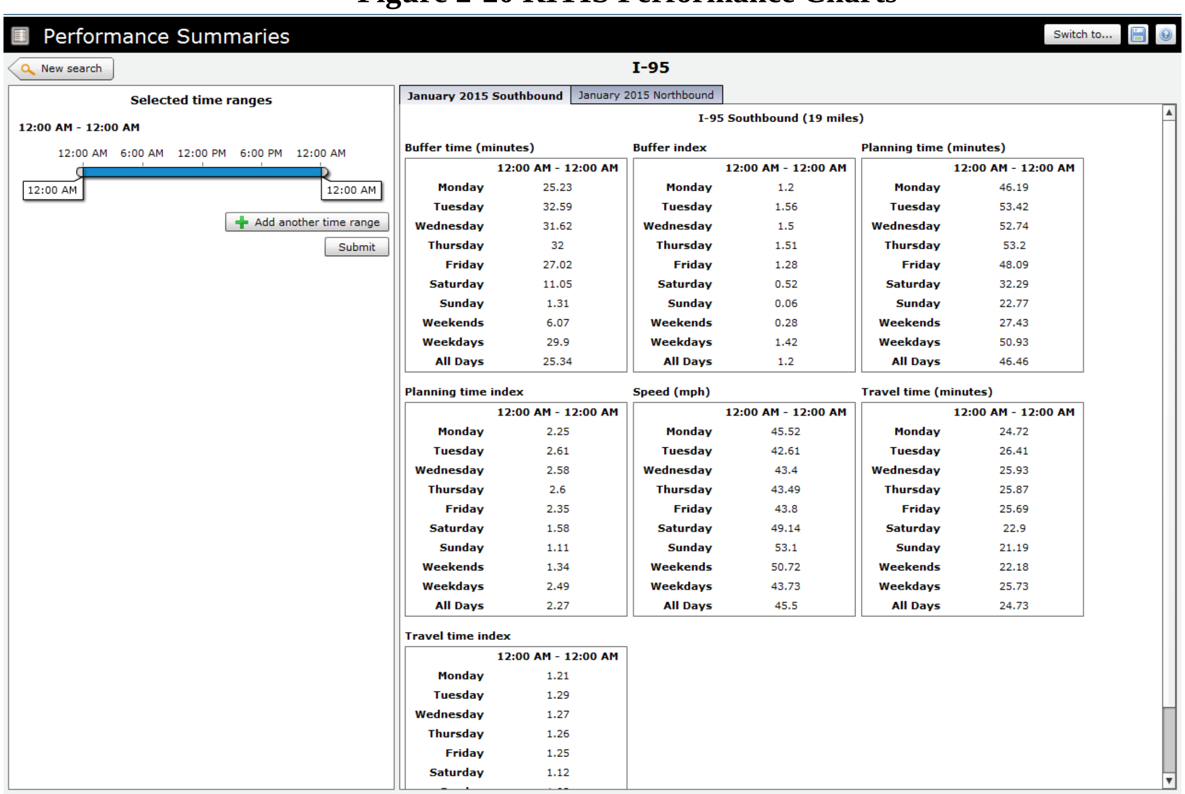


Figure 2-21 RITIS Performance Summaries

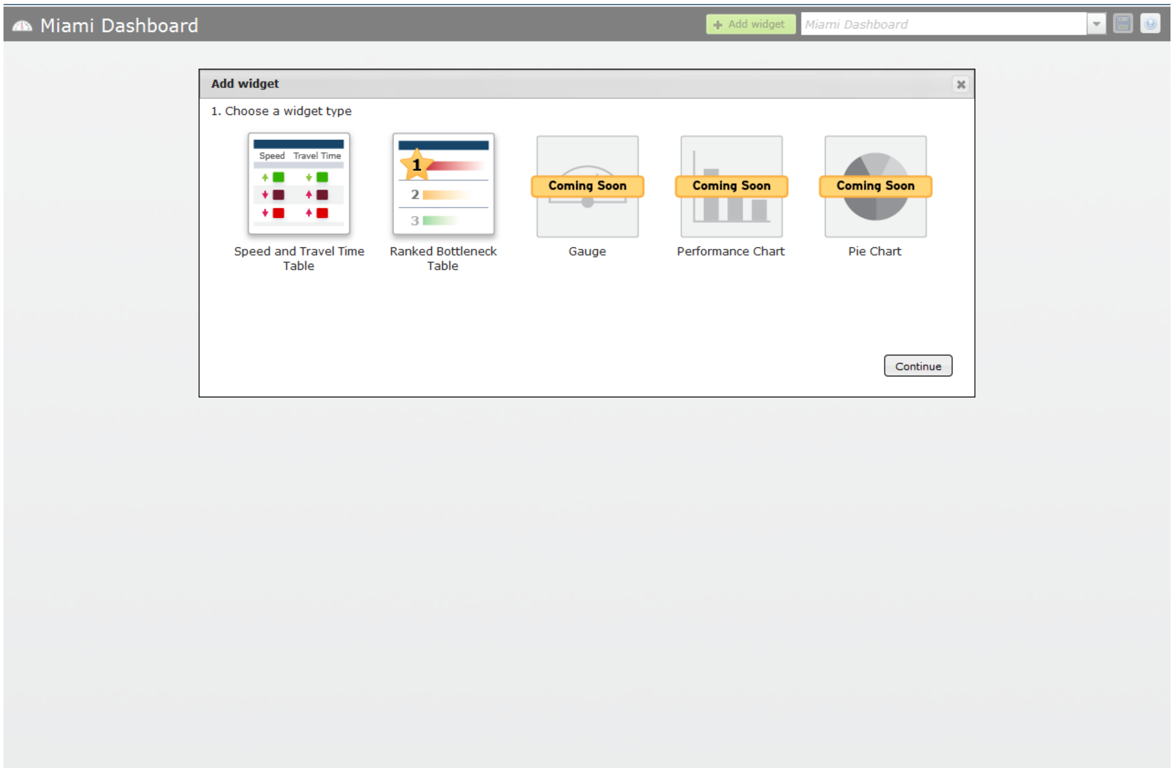


Figure 2-22 RITIS Dashboard Options

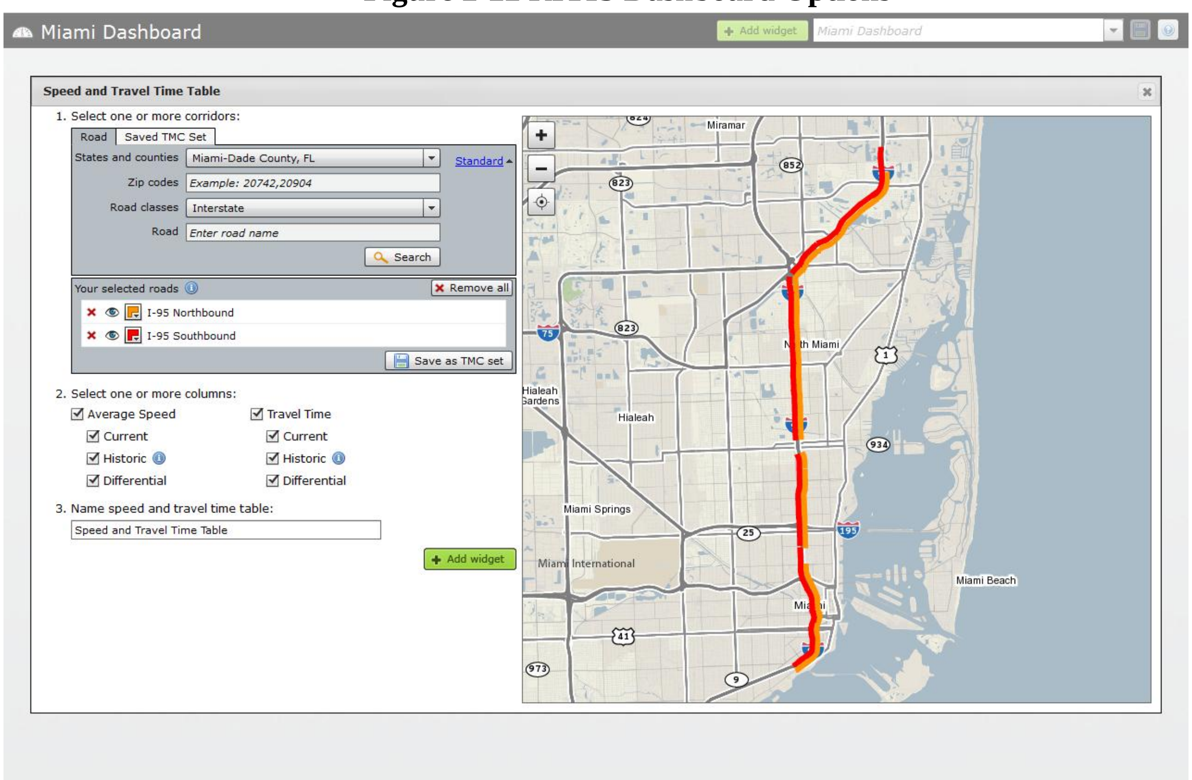


Figure 2-23 Example of Creating Speed and Travel Time Dashboard in RITIS

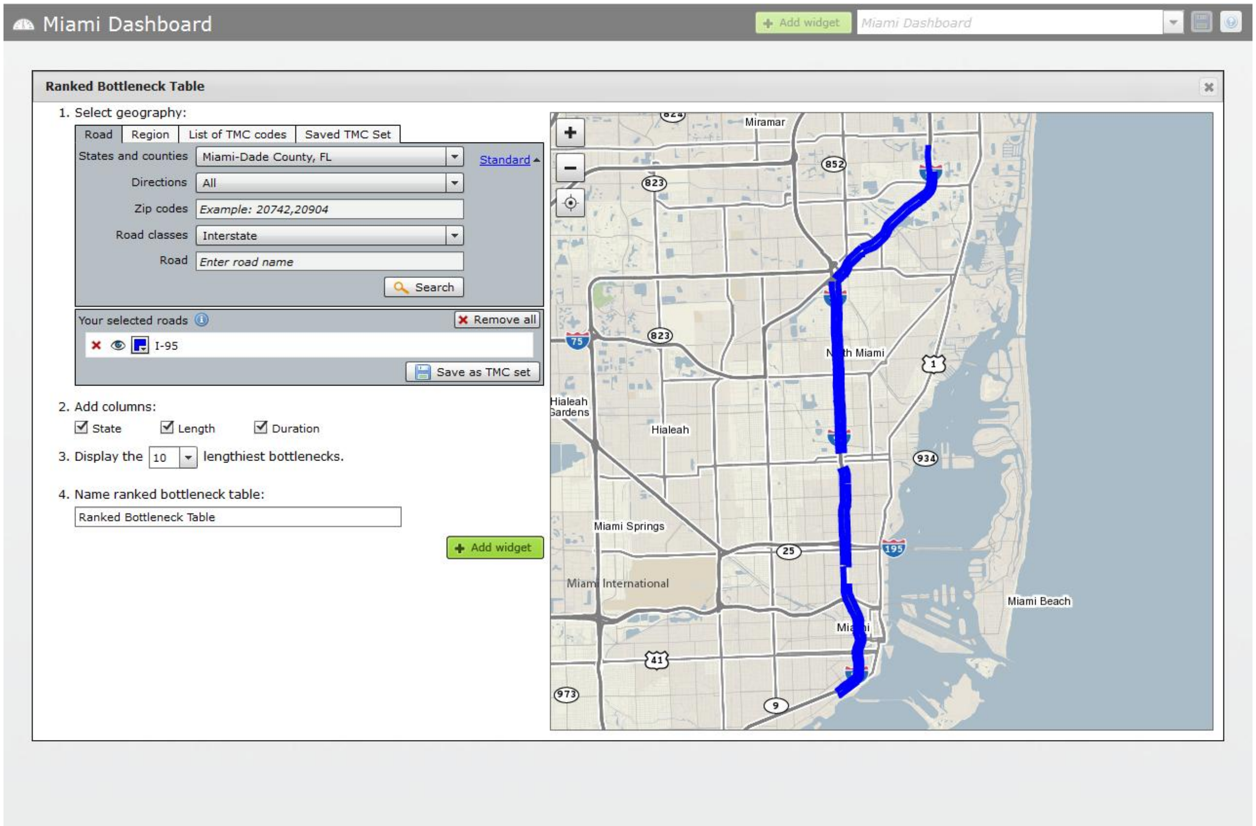


Figure 2-24 Example of Creating Ranked Bottleneck Dashboard in RITIS

2.2.11 Utah Department of Transportation Signal Performance Metrics

The Utah Department of Transportation provides a Web-based application for signal performance metrics (UDOT, 2015), which is shown in Figure 2-25. The listed metrics in this webpage include:

- Approach delay
- Approach volume
- Arrivals on red
- Purdue coordination diagram
- Speed
- Purdue phase termination
- Split monitor
- Turning movement counts

Among these metrics, the first five measures are calculated based on setback detectors. Turning movement counts are obtained from lane-by-lane stop bar detectors. The metrics of Purdue phase termination and split monitor are collected from the high resolution signal timing data (with a resolution of 1/10th seconds).

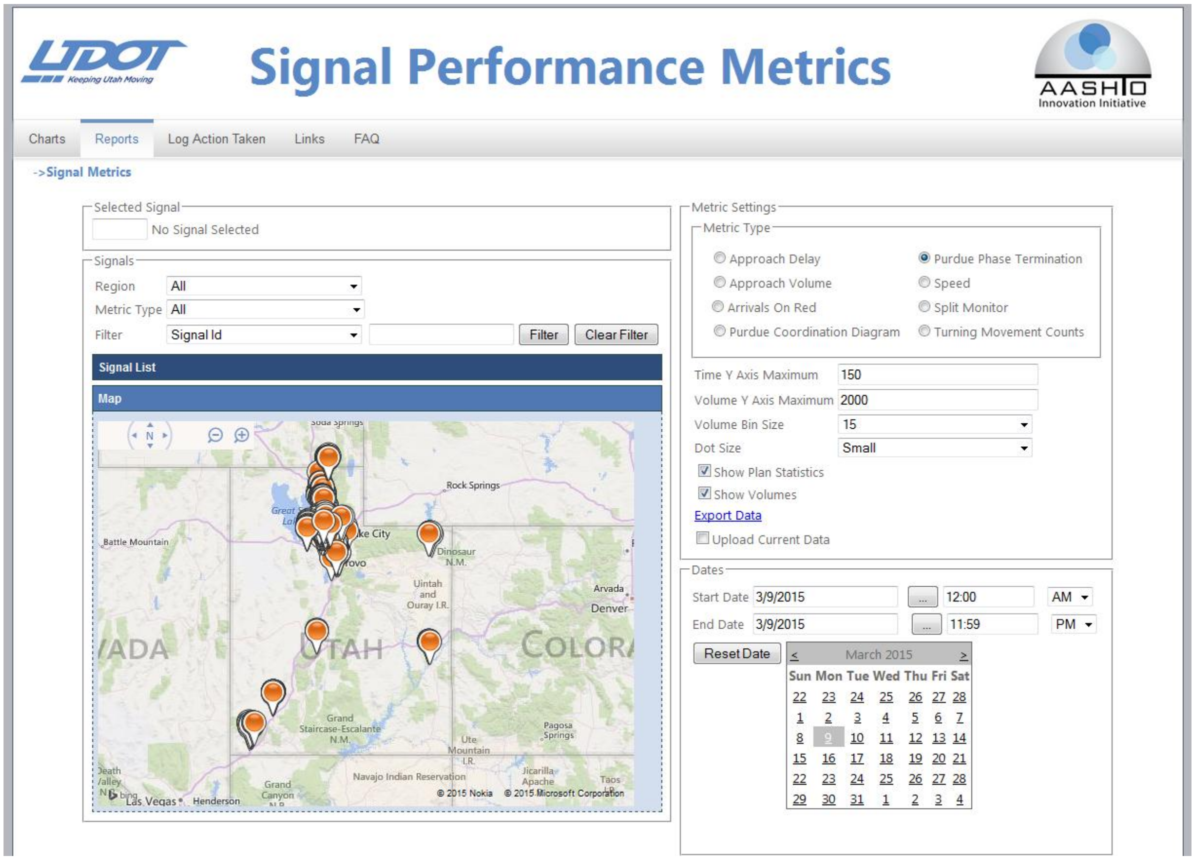


Figure 2-25 Utah DOT Signal Performance Metrics

2.2.12 City of Los Angeles Dashboard

A high-level performance dashboard is reported by the City of Los Angeles that covers measures related to the city's economy, livability and sustainability, safety, and government performance (City of Los Angeles, 2015), as shown in Figure 2-26. Among these measures, the three performance measures listed below are related to transportation engineering:

- Air quality: non-attainment days
- Street pavement conditions
- Mobility: daily vehicle-miles driven

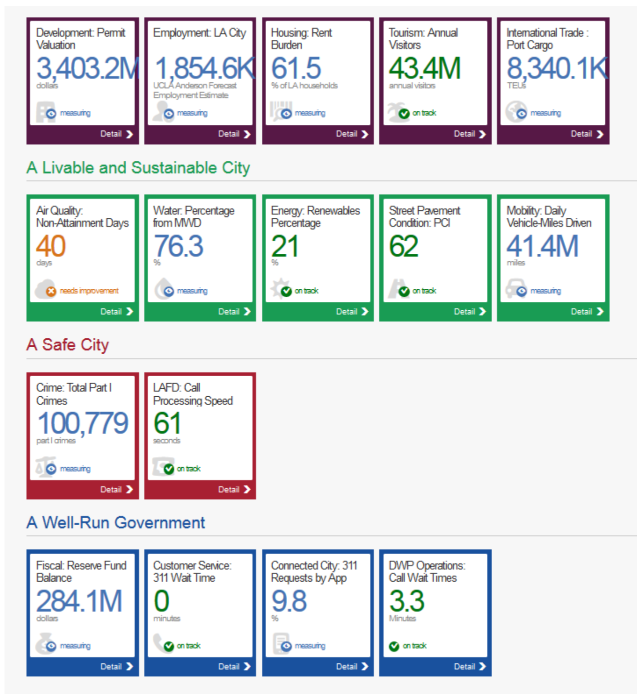


Figure 2-26 City of Los Angeles Performance Dashboard

By clicking the corresponding tab in Figure 2-26, more detailed information can be retrieved. For example, Figure 2-27 presents the obtained reports of daily vehicle-miles traveled. In addition to the explanations of the definition of mobility and daily vehicle-miles traveled, the information of transportation mode split percentages, walkscore for select LA neighborhoods, miles of bike lands and paths, percentage of new housing units permitted within 1,500 ft of rail, transitway, or rapid bus stop, and traffic counts are displayed as shown in Figure 2-27.

Figure 2-27 More Information about Mobility: Daily Vehicle-Miles Traveled (Continued on next page)

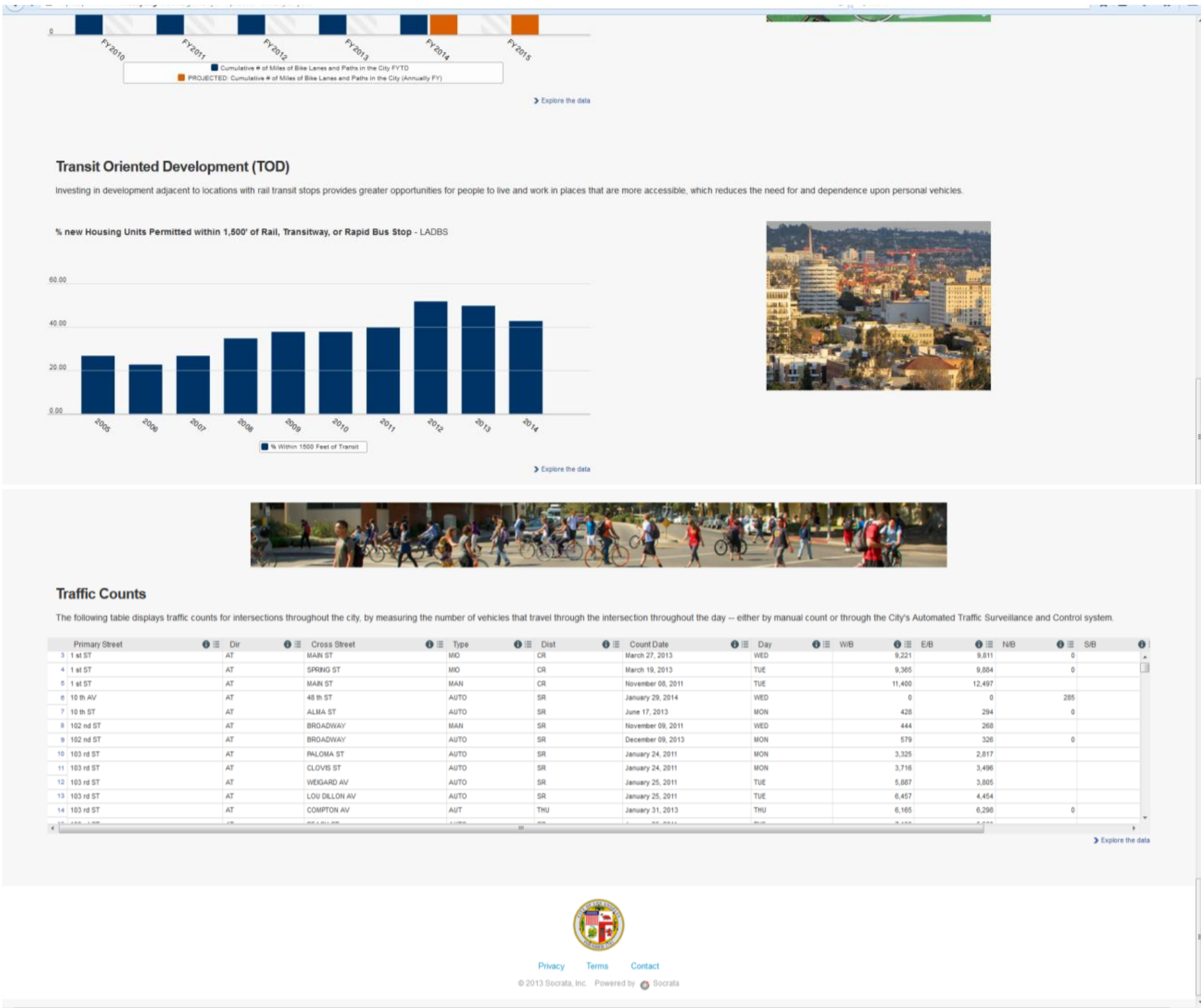


Figure 2-27 More Information about Mobility: Daily Vehicle-Miles Traveled

2.2.13 FAST Dashboard – Performance Monitoring and Measurement System

The Web-based FAST dashboard was originally designed for monitoring and displaying the performance measures of Las Vegas metropolitan freeways (RTC, 2015). This dashboard was later adopted by the Nevada DOT District II. A display of the I-15 Camera Snapshot Wall in California was also added to this web application. As shown in Figure 2-28, the performance measures presented in the FAST dashboard mainly consist of five categories: live traffic conditions, incidents, historical traffic conditions, ramp metering-related measures, and ITS device status and reported data. The following is a list of these type of performance measures:

- Average, 15th and 85th percentile of daily peak speeds during the past two weeks.
- Moving average, 15th and 85th percentile of speeds for each 15 minutes during the AM and PM peak period.

- Freeway average speeds in the past 30-60 days in the last year.
- Percentage of no-congestion, light, moderate, and heavy congestion during the previous day's AM and PM peaks.
- List of incidents for user-specified months, which include incident date and time, location, and lane blockage. The traffic conditions during the incidents can be visualized through the use of a map.
- The percentage of crashes by corridor, work zone, day type, day of week, time of day, peak period, and the top ten crash locations. Such information is only available to approved users.
- Demos of Congestion Storybook or a congestion slide show that presents the average monthly congestion during certain hours at each location of a predefined corridor.
- Time series of ramp volumes around the stop bar.
- Time series or time of day plots of speed, volume, occupancy, poll count, total volume, and truck volume from sensors. The speed-volume plot is also available for sensor data.
- Camera videos.
- Average, 15th and 85th percentile of travel times displayed on Dynamic Message Signs (DMSs).
- Time series of speed and travel time from Bluetooth data.
- Average wind or gust speed.

Additional functions, including monthly freeway performance reports, corridor travel times or speeds for predefined routes, 3D vehicle delay surface, and arterial travel times are still under development and/or testing.

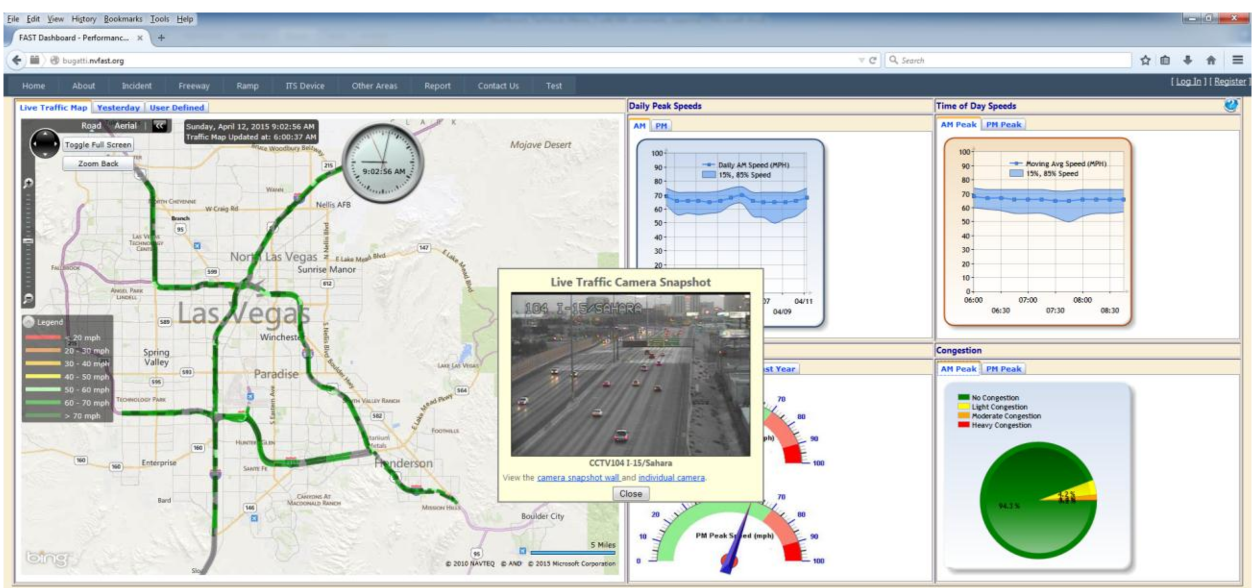


Figure 2-28 Snapshot of FAST Dashboard

2.2.14 Summary of Literature Review

Many agencies in different states are reporting their performance measures through interactive Web-based dashboards. Generally, these dashboards list the key performance indicators on the front page, with additional information for each performance measure on a separate webpage. The current value, together with a description and target value, is usually provided for each performance measure. However, the measures listed in these dashboards vary with the agency's goals and availability of data. Safety measures are commonly reported in the dashboards. Mobility measures such as travel time and speed are only provided by a few states, possibly due to lack of information. Travel time reliability metrics are usually missing in these dashboards. It should also be pointed out that most state DOTs focus on freeway traffic performance measures more than arterial measures. Currently, dashboards summarizing arterial performance measures are not common, and very few states are monitoring and reporting signal-related metrics. The approach that was used in the developed tool is to allow each agency to identify their dashboards based on their requirements. After defining the scope, the agency can work with the tool development and maintenance staff on implementing the dashboard in ITSDCAP. This will be discussed further in this document.

2.3 Performance Measurement in ITSDCAP

The performance measurement in ITSDCAP includes two modules: segment-based and intersection-based performance measurement modules. Segment-based performance measures are intended to be used for both freeway and arterial segments, while intersection-based measures are only for arterials. This section provides a discussion of these two modules.

2.3.1 Segment Performance Measurements

Data from multiple sources, such as the central data warehouses (STEWARD and RITIS), point detectors, travel times based on vehicle matching, safety databases (CARS and FHP crash reports), weather sensors, and other sources, provide a basis for the performance measure estimation in ITSDCAP. There is no specific requirement in data format for ITSDCAP. As long as the file format is readable, for example, csv or text format, the data can be read into the ITSDCAP database. Four sets of segment performance measures are available in the "Segment Performance Measurement" module. These sets are mobility, reliability, safety, and energy and environmental measures. The performance measures can be requested for freeway and/or arterial segments.

<u>Mobility Measures</u>

In ITSDCAP, nine key mobility performance measures can be estimated, as listed below:

- Average speed
- Volume
- Occupancy
- Travel Time
- Delay
- Free-flow speed-based congestion index
- Desired speed-based congestion index
- Vehicle-Miles Traveled (VMT)
- Vehicle-Hours Traveled (VHT)

Figure 2-29 displays a screen capture of the Mobility Tab in ITSDCAP. This figure shows that the mobility measures can be estimated for user-specified time periods and locations. The time periods can be a continuous period of time or discrete days that meet specific criteria, such as normal days or days with incidents or bad weather, or days with incidents and bad weather. The average values of speed, volume, and occupancy can be directly obtained from the aggregated point traffic detector measurements. However, it should be mentioned that the raw detector measurements of speed, volume, and occupancy are collected at a frequency of 20 or 30 seconds. The detector data go through a process of data filtering and cleaning before being aggregated into a certain time period (for example, 5 minutes) and imported into the database.

Two approaches are utilized in ITSDCAP to calculate travel time measures. When ITS devices can report the measurement of travel time (for example, Bluetooth-based travel time data) or when the estimated travel time data based on point detectors are available (for example, TVT data from the SunGuide software), the travel times reported by ITSDCAP are simply the average values for a given time period. However, when direct travel time information is not available, freeway segment travel times can be calculated in ITSDCAP from the speeds measured by point detectors using the mid-point methods, as shown in Equation 2-1.

$$TT_{1-2} = \frac{L_{1-2}/2}{S_1} + \frac{L_{1-2}/2}{S_2} \tag{2-1}$$

where TT_{1-2} is the travel time between the two adjacent detectors. L_{1-2} represents the corresponding distance between the two adjacent detectors. S_1 and S_2 are the measured speeds at the upstream and downstream detector locations, respectively. Note that, travel time estimates from point measurements of speed are usually not accurate along the arterials due to the existence of signals. In ITSDCAP, delay is calculated as the difference between the average travel time and travel time under the free-flow conditions.

In order to quantify the traffic congestion level, two congestion indices are used in ITSDCAP, one is the free-flow speed-based congestion index. and the other is the desired speed-based congestion index. Equation 2-2 presents the expressions for these two congestion indices.

$$CI_{FFS} = \frac{1}{N}\sum_{i,t}\left|\frac{S_{i,t} - S_{FFS}}{S_{FFS}}\right| \qquad \forall S_{i,t} < S_{FFS} \tag{2-2}$$

$$CI_{DS} = \frac{1}{N}\sum_{i,t}\left|\frac{S_{i,t} - S_{DS}}{S_{DS}}\right| \qquad \forall S_{i,t} < S_{DS} \tag{2-3}$$

As shown in these two equations, the congestion index, *CI*, is calculated as the average absolute speed difference from a predefined speed (free-flow speed S_{FFS} or desired operational speed S_{DS}). The symbol *N* indicates the total number of observations and the subscripts *i* and *t* refer to the station *i* and time interval *t*. Note that only the speeds that are lower than these predefined speeds are included in the calculation. The free-flow speed in Equation 2-2 can be estimated using the Highway Capacity Manual 2010 (TRB, 2010) procedures. The value of the congestion index is between zero and one with a zero, indicating that traffic is either under free-flow conditions or operating at the desired congestion level and a value of one corresponding to completely stopped traffic.

The calculation of vehicle-miles traveled and vehicle-hours traveled in ITSDCAP is straightforward and achieved by multiplying the volume counts with the corresponding distances and travel times, respectively.

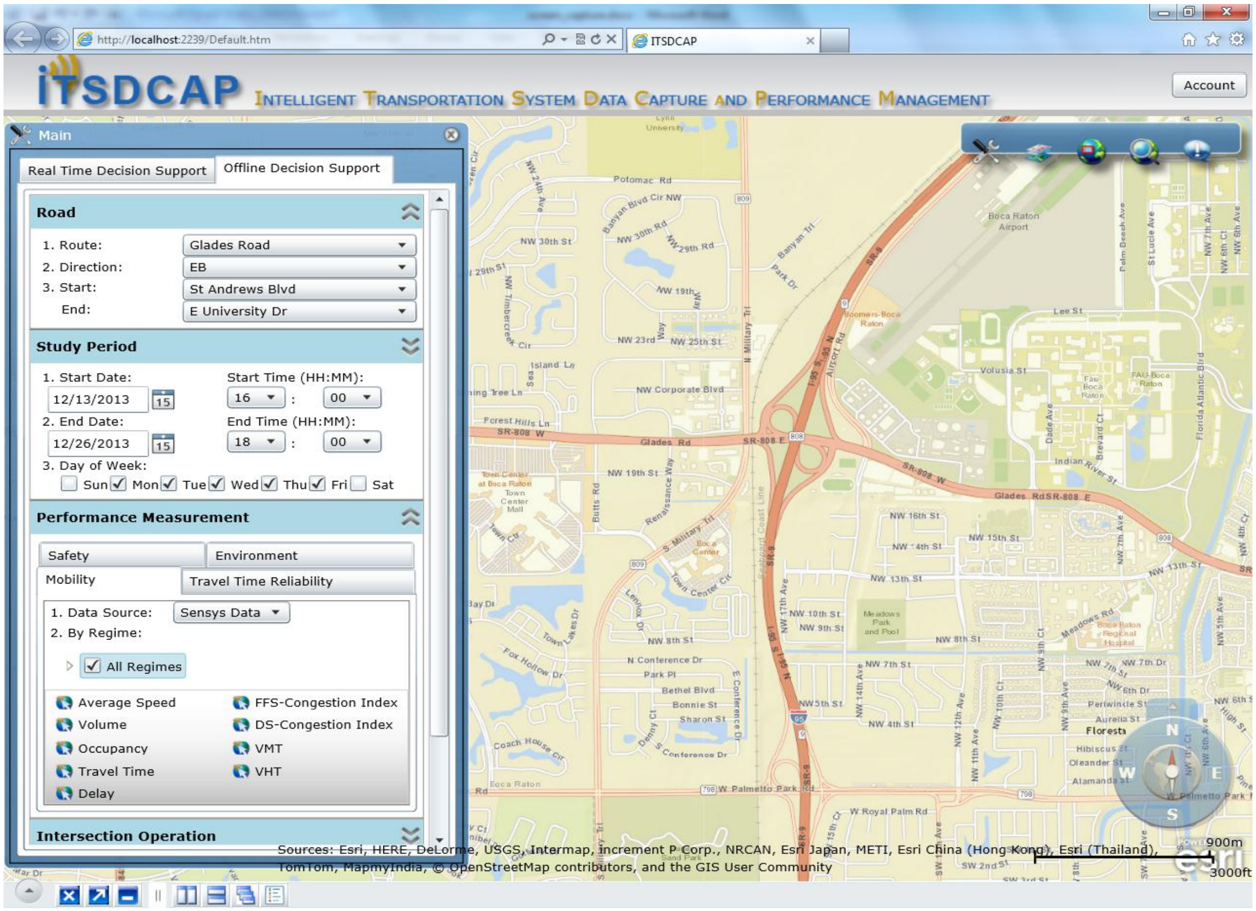


Figure 2-29 Mobility Performance Measure Estimation Interface in ITSDCAP

When a user clicks the buttons associated with the mobility performance measures, the corresponding point detector-based results (for point speed, volume, and occupancy, for example) and/or segment-based results (such as travel times) are shown on the map. Route measures from a specified starting location to an ending location can also be visualized in a pop-up window. For example, Figure 2-30 shows the average speed along one direction of Glades Road in Boca Raton, Florida on weekdays between December 13, 2013 and December 26, 2013. Figure 2-31 presents the corresponding delays relative to free-flow speed. The user can download the results by clicking the "Export" button in the pop-up window. The ITSDCAP tool also allows the user to play an animation of time-dependent variation of mobility performance measures by clicking the animation button on the tool bar, as shown in Figure 2-32. The user can change the play speed and the temporal data aggregation level by changing the options in the animation window.

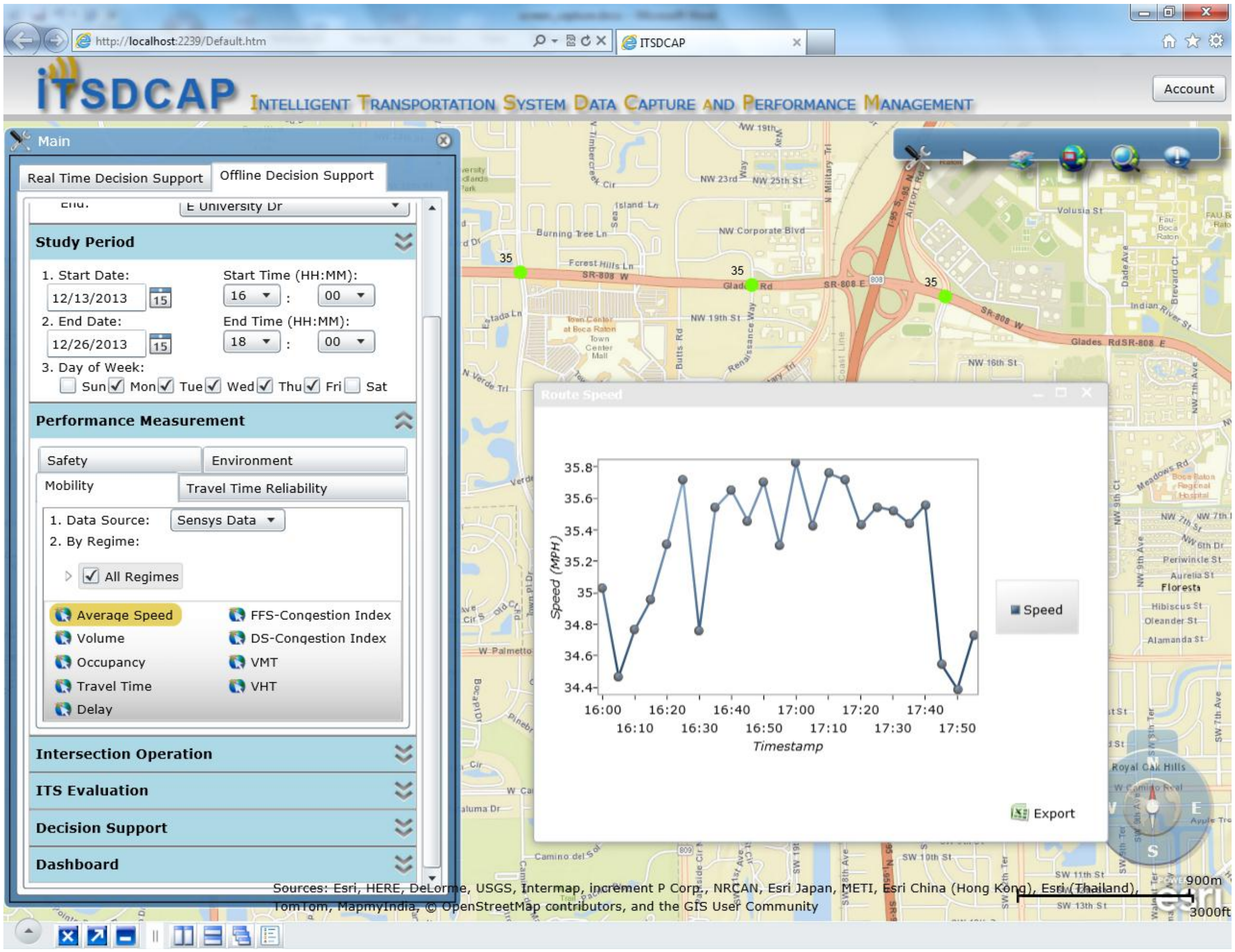


Figure 2-30 Example of Average Speed Output in ITSDCAP

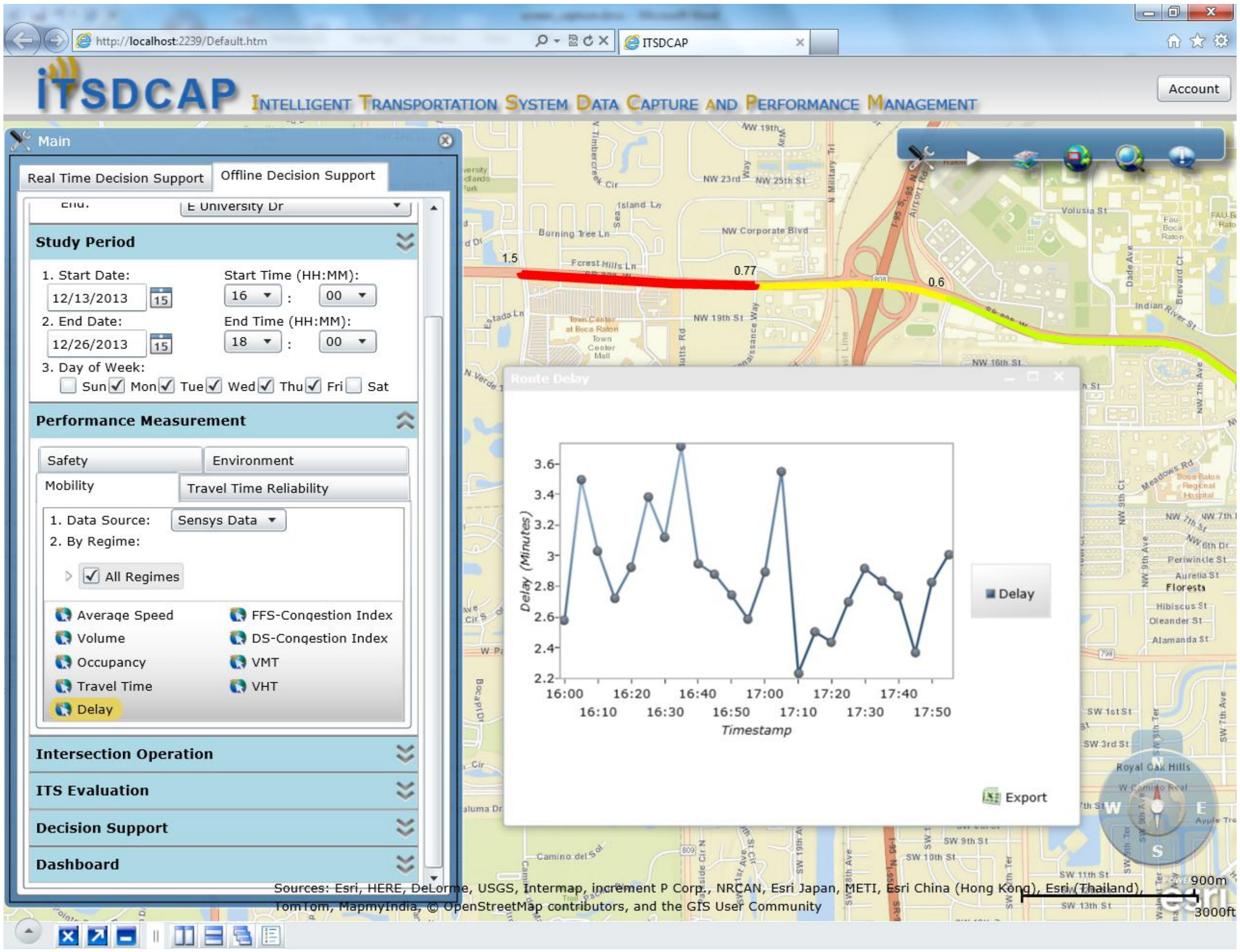


Figure 2-31 Example of Delay Output in ITSDCAP

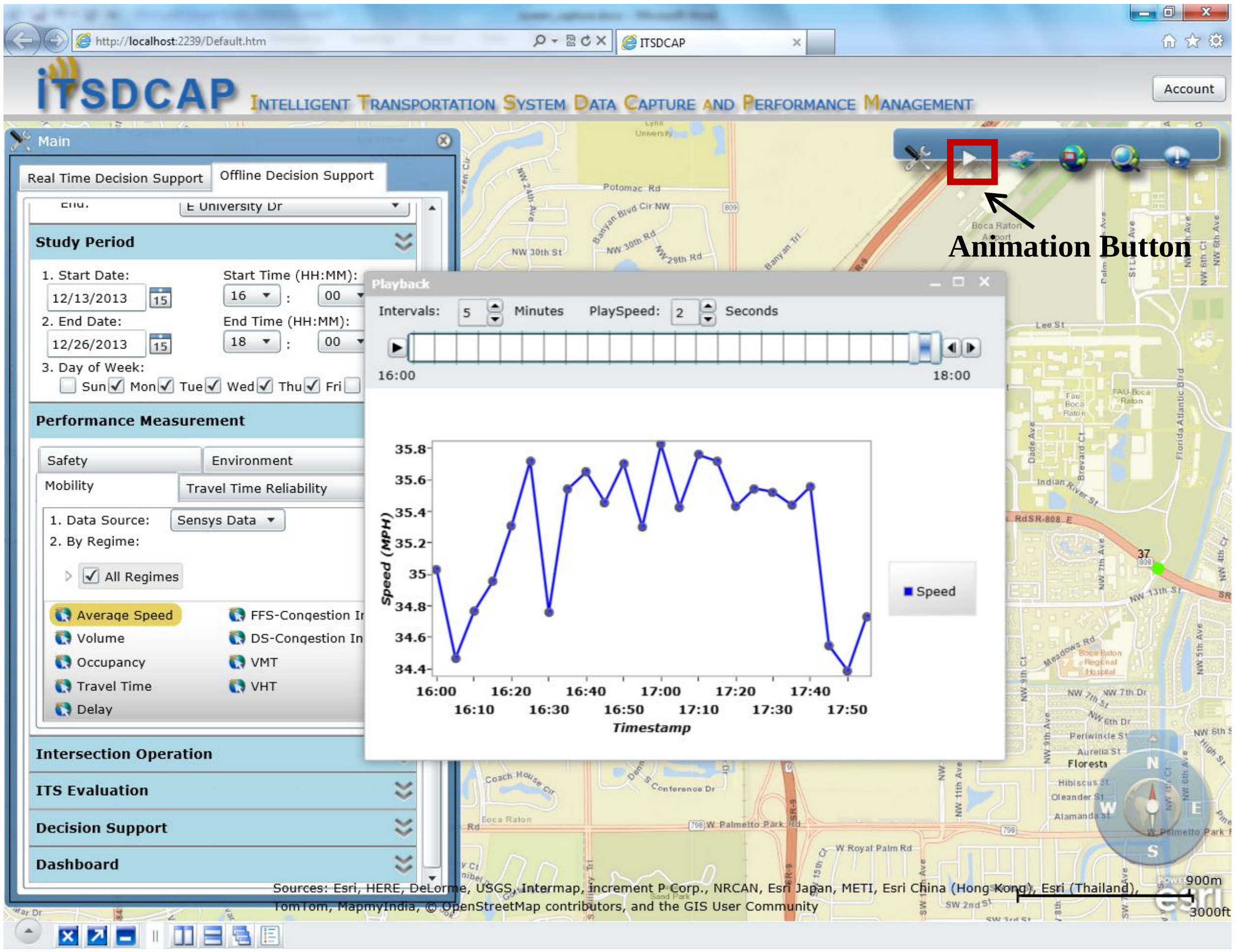


Figure 2-32 Example of Average Speed Animation

<u>Travel Time Reliability Measures</u>

Travel time reliability measures are increasingly being recognized as important measures, and agencies have started to include reliability in their performance measurement dashboards. In ITSDCAP, as shown in Figure 2-33, the following twelve travel time reliability metrics are reported:

- Cumulative density function (CDF) of travel time rate
- Probability density function (PDF) of travel time rate
- Unreliability contributions
- Percentage of occurrence by regime
- Percentage of severity by regime
- Standard deviation
- Buffer index

- Travel time index (including 95th percentile, 80th percentile, median, and mean travel time index)
- Policy index
- Failure/on-time
- Misery index
- Skew statistics

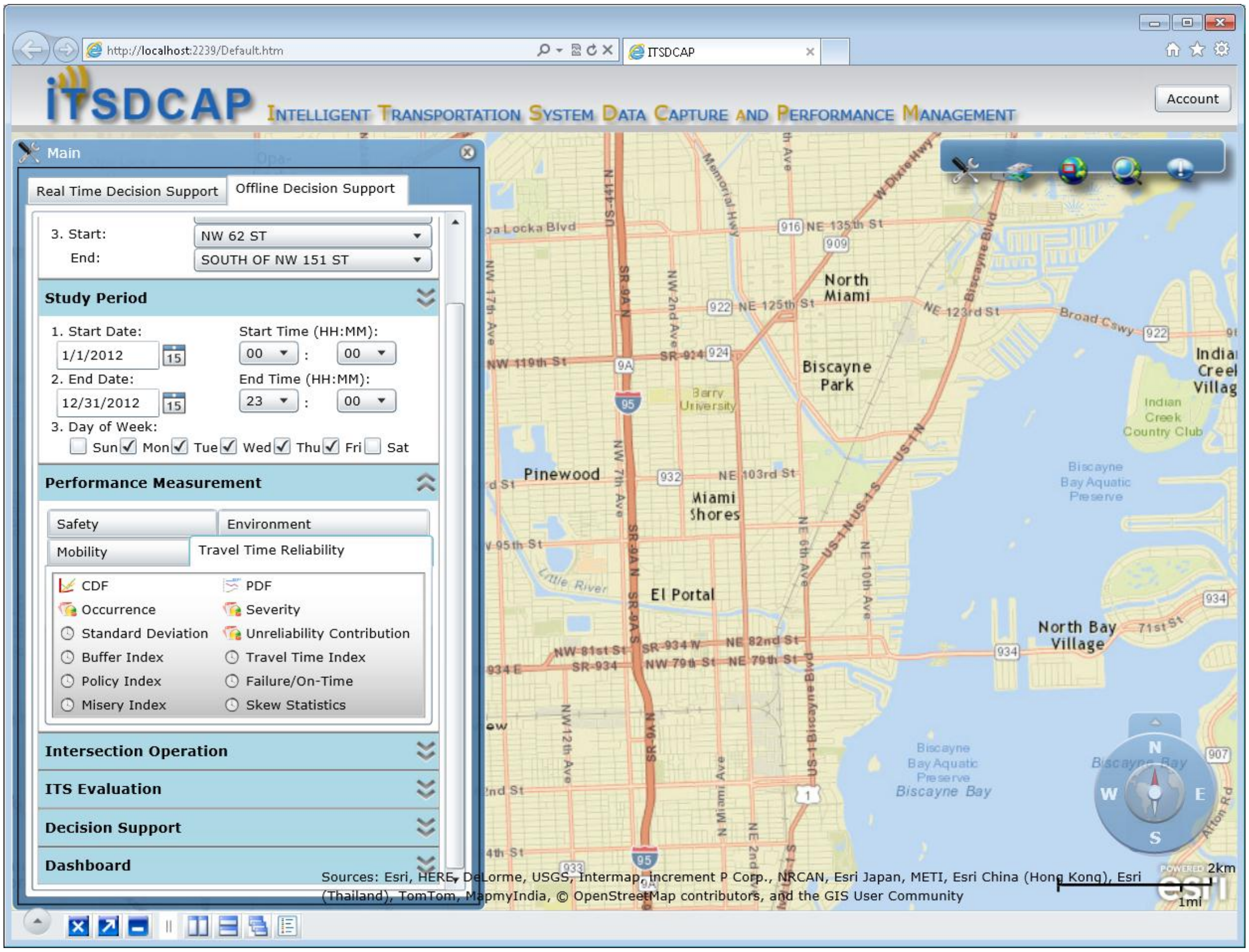


Figure 2-33 Interface of Travel Time Reliability in ITSDCAP

The first three measures originated from the SHRP 2 L02 project. The SHRP 2 L02 project developed methods for monitoring and evaluating travel time reliability based on data collected using traffic monitoring systems, such as those based on point traffic detectors, AVI, Automatic Vehicle Location (AVL), and private sector data. In the L02 procedure, traffic conditions are classified into different regimes using data from multiple sources, including normal, high demand, incident, weather event, and special event regimes; and are combined with low/medium/high congestion levels. New visualization and analysis methods such as travel time rate (defined as travel time along a unit distance), and probability density functions (PDFs) and their associated cumulative density functions (CDFs) by regimes were introduced in the L02 project. In addition, the percentage contribution of each regime to travel time unreliability is calculated using the following equation:

$$\%contribution_i = \frac{SV_i}{\sum_j SV_j \times n_j} * 100 \qquad (2\text{-}4)$$

where n_j is the occurrence frequency of regime j and SV_j is the semi-variance of travel time rate for regime j as defined below:

$$SV = \sqrt{\frac{1}{n}\sum_k (x_k - r)^2} \qquad \exists x_k > r \qquad (2\text{-}5)$$

where x_k is kth measurement of travel time rate and r is a reference value (for example, the free-flow travel time rate). For example, Figures 2-34 and 2-35 show the CDF and percentage of unreliability results for the I-95 southbound general-purpose lane (GPL) study route, in Miami, Florida using ITSDCAP. However, the percentage unreliability contribution metric listed in Equation 2-4 cannot differentiate whether the unreliability contribution is due to the frequent occurrence of the regime or because of its severe single-event impacts, even with less frequency. Therefore, two more measures are proposed in the SHRP 2 L38C project conducted by the Florida International University (FIU) research team. These measurements are the percentage of occurrence of each regime and the single-event severity of each regime. Note that the single-event severity is defined as the semi-standard deviation in travel time rate due to one single event. Figures 2-36 and 2-37 present the corresponding percentage of occurrence and single-event contributions for the I-95 southbound GPL route. The above reliability measures can help agencies identify and understand the causes of unreliability, and thus take the necessary mitigating actions.

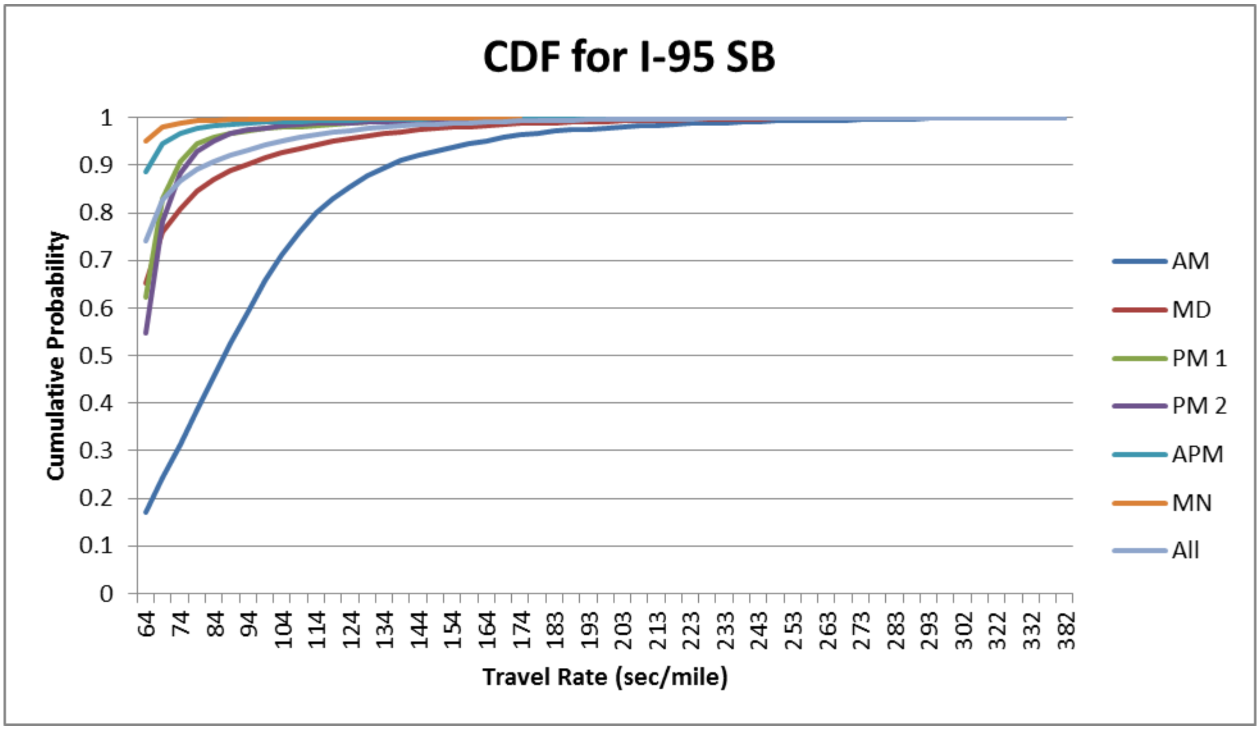


Figure 2-34 The CDF Results for I-95 Southbound GPL Using the ITSDCAP

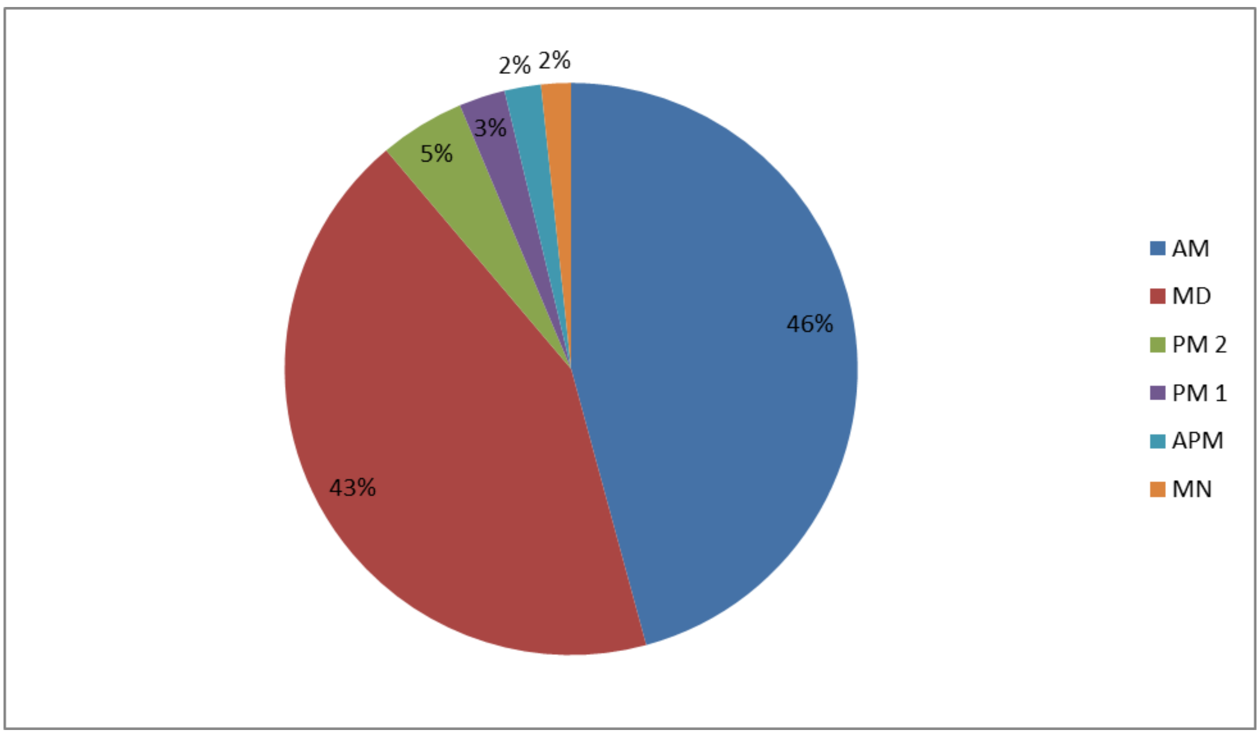


Figure 2-35 Percentage of Unreliability Contribution for I-95 Southbound GPL Using the ITSDCAP

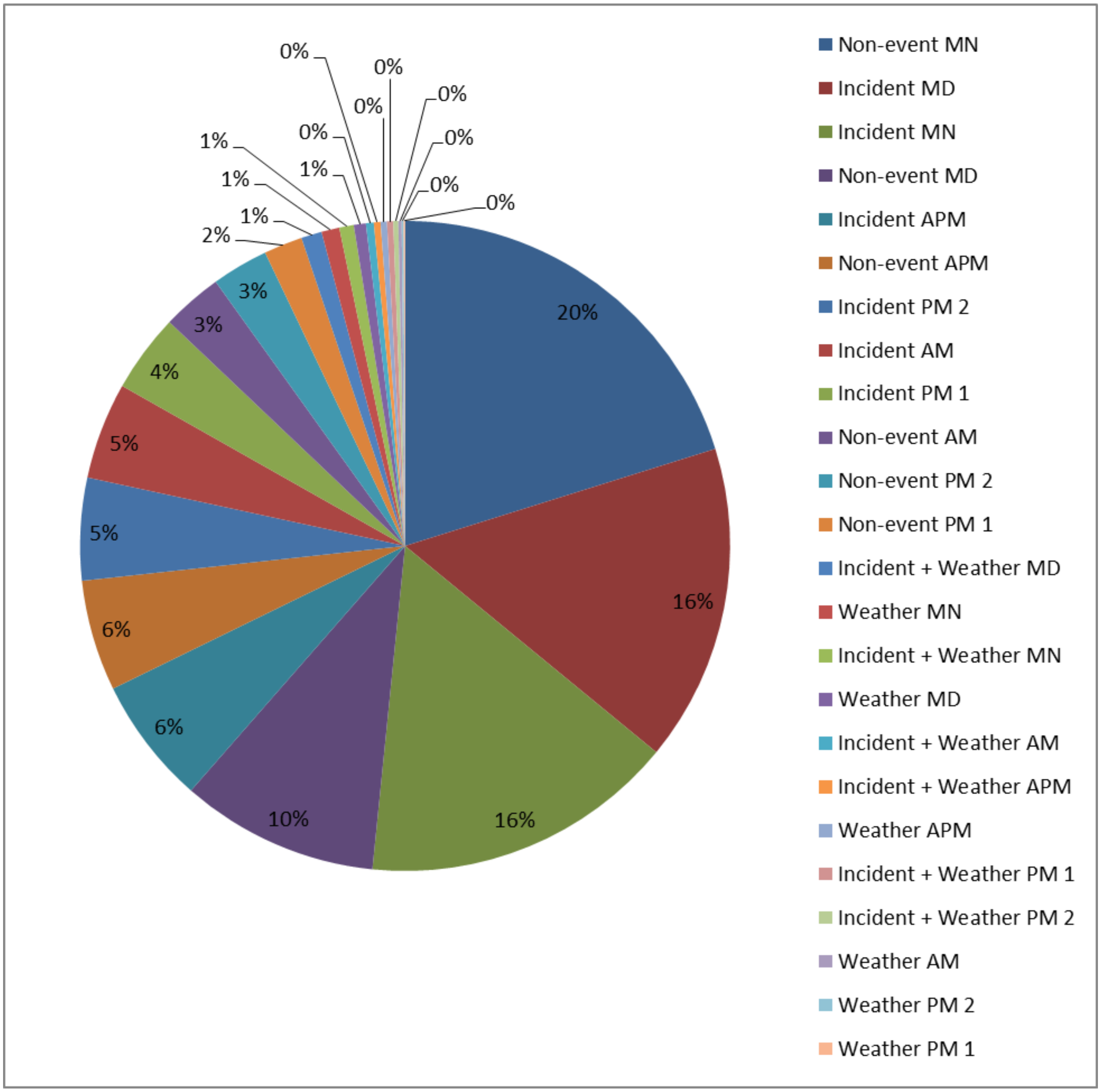


Figure 2-36 Percentage Occurrence of Each Regime for I-95 Southbound GPL Using the ITSDCAP

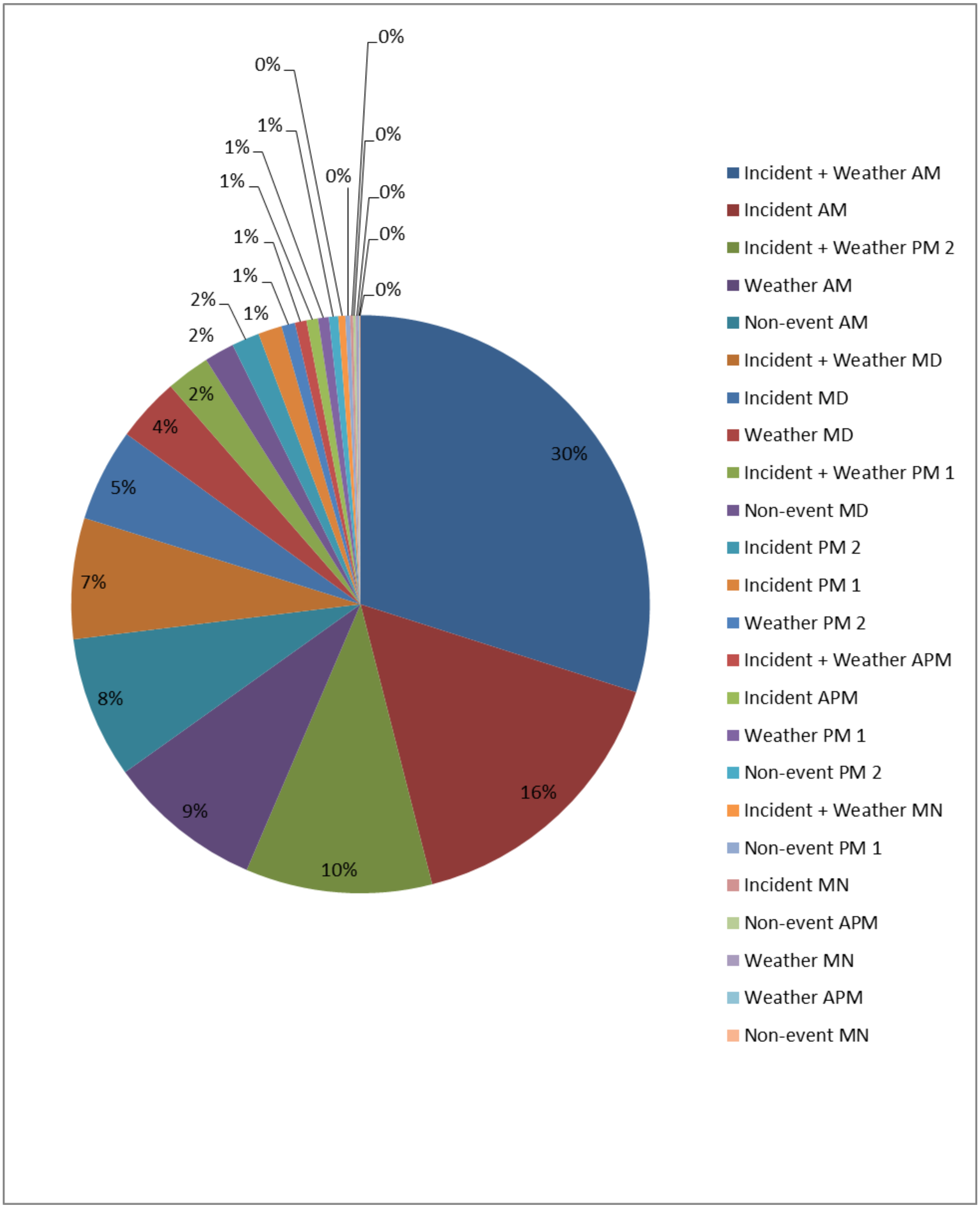


Figure 2-37 Single Event Unreliability Contribution for I-95 Southbound GPL Using the ITSDCAP

The remaining travel time reliability measures have also been used in practice. The definitions of these measures are explained in Table 2-1. These reliability metrics are reported at a time interval of five minutes in ITSDCAP. Figure 2-38 shows the variation of mean, median, 80th percentile and 95th percentile travel time index for the I-95 northbound GPL study route. As shown in this figure, the 80th and 95th travel time indexes start increasing between 2:00 and 3:00 p.m., which is earlier than the common definition of the PM peak period. Based on these results, the agency may want to activate congestion management strategies such as ramp metering and managed lane pricing strategies.

Table 2-1 Definitions of Travel Time Reliability Measures

Reliability Performance Metric	**Definition**
Standard Deviation	The standard deviation of travel time distribution.
Buffer Index (BI)	The difference between the 95th percentile travel time and the average travel time, normalized by the average travel time.
Mean Travel Time Index	Mean travel time divided by free-flow travel time.
Median Travel Time Index	Median travel time divided by free-flow travel time.
80th Percentile Travel Time Index	The 80th percentile travel time divided by the free-flow travel time.
95th Percentile Travel Time Index	The 95th percentile travel time divided by the free-flow travel time.
Policy Index	Mean travel time divided by travel time at target speed.
Failure/On-Time Performance	Percent of trips with travel times less than: • 1.1* median travel time • 1.25* median travel time
Skew Statistics	The ratio of 90th percentile travel time minus the median travel time, divided by the median travel time minus the 10th travel time percentile.
Misery Index	The average of the highest five percent of travel times divided by the free-flow travel time.

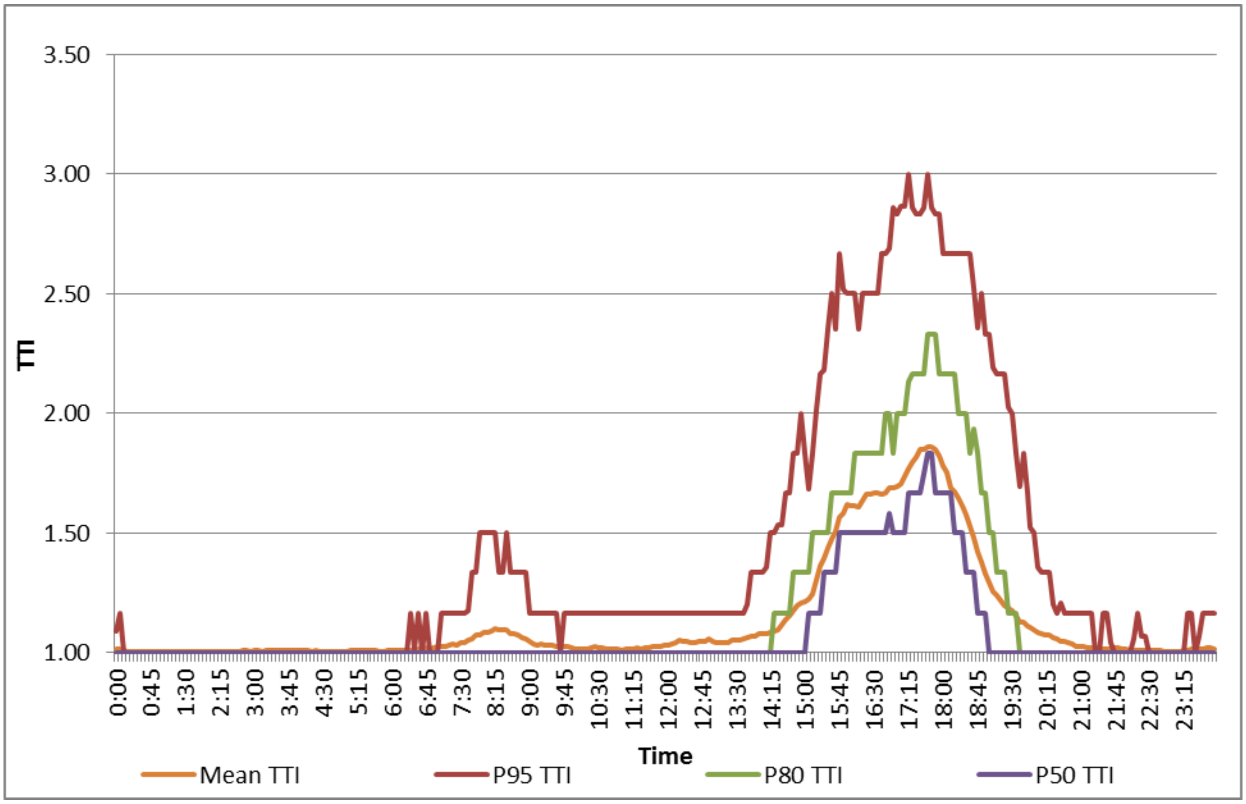


Figure 2-38 The Variations of Travel Time Index for I-95 Northbound GPL from ITSDCAP

Safety Performance Measures

Safety performance measures are among the most important indicators of system performance. Each state is required to track three categories of safety measures developed by the National Highway Traffic Safety Administration (NHTSA) and the Governors Highway Safety Association (GHSA), which include the core measures, behavior measures, and activity measures (Herbel et al., 2009). The core measures, or outcome measures, consist of the frequency of crashes, injuries and fatalities. The behavior measures associate the safety activities with behaviors. The activity measures focus on the actions taken by agencies to reduce crashes. Currently, the ITSDCAP tool mainly reports the core safety measures, which are listed below.

- Crash frequency by crash type
- Crash frequency by severity
- Total crash frequency
- Crash rate by type
- Crash rate by severity
- Total crash rate

The crash frequency by crash type is defined as the number of crashes for a given type of crash, such as rear-end, head-on, angle, sideswipe, and so on. The crash frequency by severity is the number of crashes for a certain severity level. Three severity levels are usually recorded in the crash databases, that is, the Property Damage Only (PDO), injury, and fatality. The total crash frequency is the total number of crashes, including all the crash types and severity levels. The corresponding crash rate by type, by severity, and total crash rate are defined in a similar way, except that these are calculated as the number of crashes per million vehicle miles traveled (MVMT) for roadway segments.

Figure 2-39 shows the safety performance measure interface in ITSDCAP. As shown in this figure, the user can select different data sources for safety performance measure calculation, such as the Freeway Highway Patrol (FHP) crash report and Florida Crash Analysis Reporting (CAR) System. The crash type or crash severity level can also be specified using pulldown menus. Once the user clicks either the crash rate or crash frequency button, the resulting safety performance measures will be displayed in the ITSDCAP interface. Figure 2-39 shows the crash frequency results for the rear-end crashes along Glades Road eastbound between St. Andrews Boulevard and East University Drive. The numbers shown on the map are the number of rear-end crashes occurring at those locations. The chart in the pop-up window shows the total number of crashes for each crash type, allowing the user to compare the occurrence of different types of crashes. Figure 2-40 presents similar results, but for property damage only crashes along Glades Road. The results in Figures 2-40 and 2-41 are based on FHP system data. The FHP data used for Glades Road does not have associated volume information required for the crash rate calculation. Thus, only crash frequency (not crash rate) is reported when using FHP data in the current version of ITSDCAP. The crash rates can be calculated when using the CARS data, as shown in Figure 2-41, for the I-95 northbound segment in Miami.

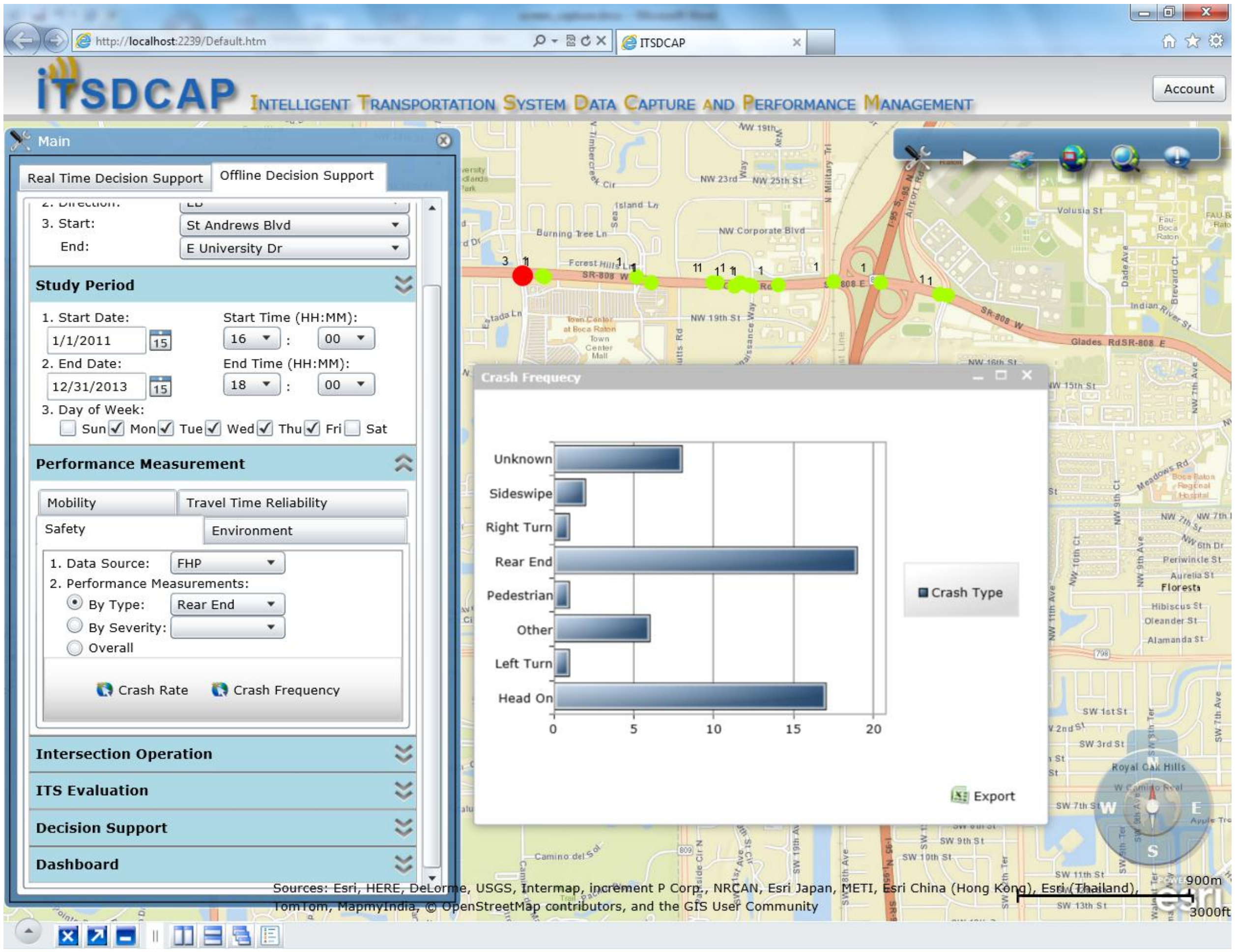


Figure 2-39 Rear-end Crash Frequency for Glades Road Eastbound in ITSDCAP

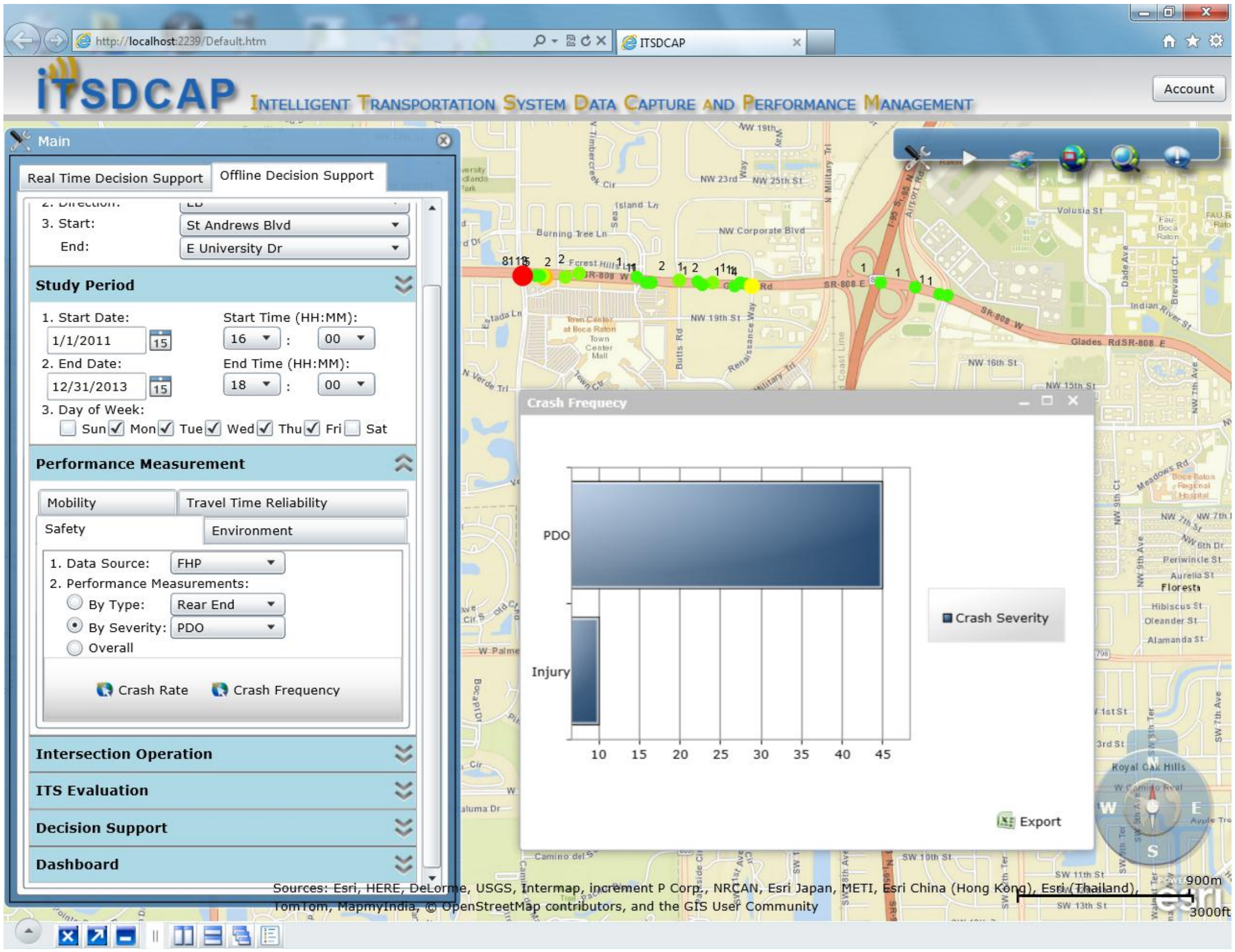


Figure 2-40 Frequency of Property Damage Only Crash for Glades Road Eastbound in ITSDCAP

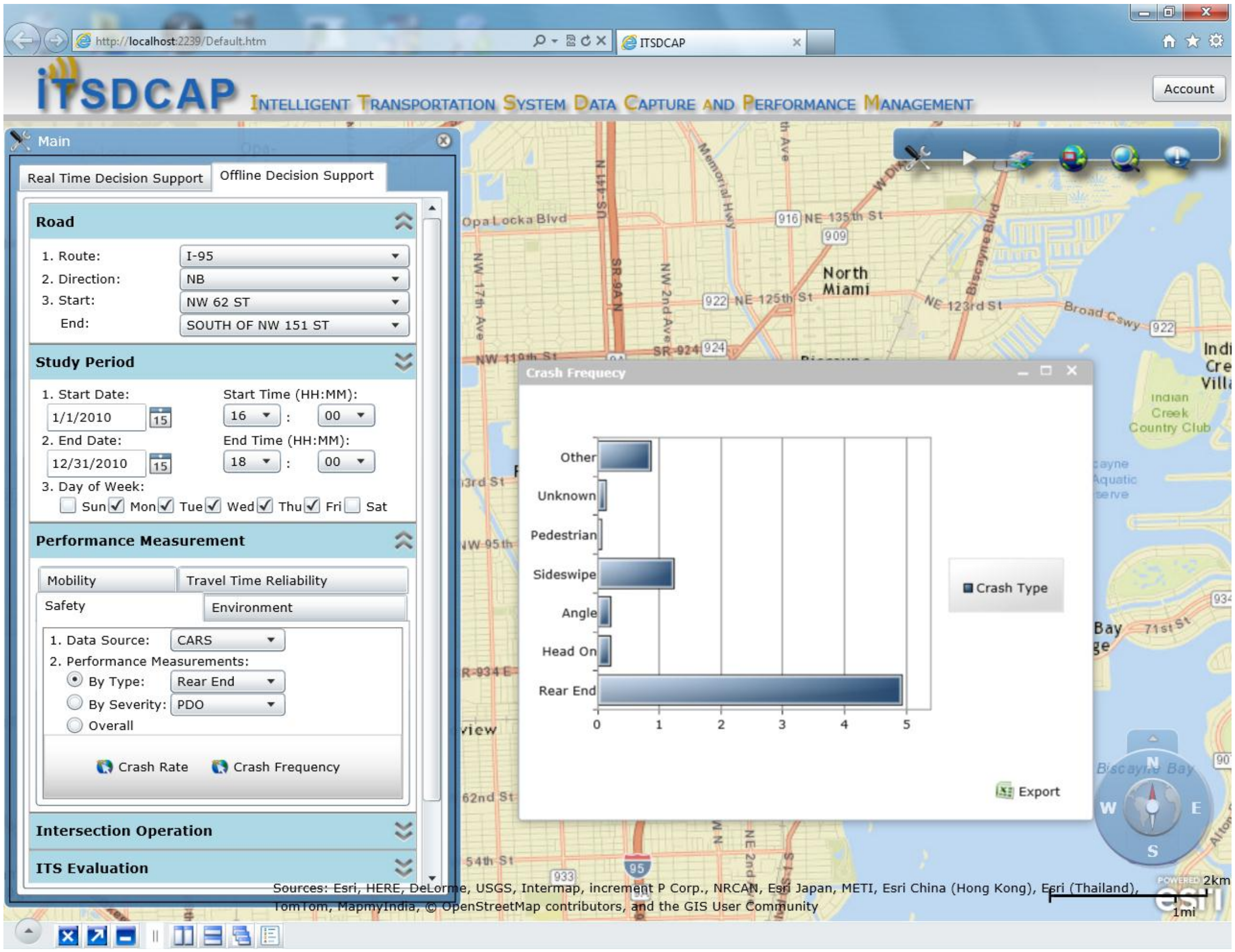


Figure 2-41 Results of Crash Rate by Crash Type for I-95 Northbound in ITSDCAP

Energy and Emission Measures

Fuel consumption and emissions due to traffic have gained more attention recently. In ITSDCAP, two types of fuel consumption are considered, gas and diesel, which are calculated based on vehicle-miles traveled and fuel consumption rate. The fuel consumption rate is a function of speed and vehicle type. The rates for freeway segments are also different from those along the arterials, as shown in Tables 2-2 and 2-3 (Cambridge Systematics, 2001).

Table 2-2 Freeway Fuel Consumption Rate

SPEED (MPH)	Gas (AUTO) (Gallons/VMT)	GAS (Truck) (Gallons/VMT)	Diesel (Truck) (Gallons/VMT)
0	0.540	0.650	0.450
5	0.182	0.310	0.696
10	0.123	0.181	0.489
15	0.089	0.135	0.297
20	0.068	0.118	0.185
25	0.054	0.120	0.131
30	0.044	0.133	0.110
35	0.037	0.156	0.112
40	0.034	0.185	0.122
45	0.033	0.223	0.136
50	0.033	0.264	0.153
55	0.034	0.310	0.170
60	0.037	0.374	0.187
65	0.043	0.439	0.204
70	0.052	0.511	0.221

Table 2-3 Arterial Fuel Consumption Rate

SPEED (MPH)	Gas (AUTO) (Gallons/VMT)	GAS (Truck) (Gallons/VMT)	Diesel (Truck) (Gallons/VMT)
5	0.144	0.275	0.383
10	0.091	0.174	0.241
15	0.073	0.140	0.194
20	0.064	0.123	0.171
25	0.059	0.113	0.157
30	0.056	0.106	0.147
35	0.053	0.101	0.140
40	0.051	0.097	0.135

The Motor Vehicles Emission Simulator (MOVES) is the latest emission modeling system developed by the U.S. Environmental Protection Agency (USEPA). Figure 2-42 illustrates the graphical user interface of MOVES. In MOVES, there are three different approaches to estimate emissions, the average speed approach, driving cycle approach, and operating mode distribution approach. The average speed approach requires inputting the average speeds, and based on these input values, a default driving cycle is applied to calculate the emissions. In the driving cycle approach, emissions are estimated based on a second-by-second speed profile that represents an average vehicle. Compared to the other two approaches, the operating mode distribution approach requires more detailed information about the amount of time that the vehicles spend in different operating modes. The operating modes are related to the bins defined by the values of vehicle specific power (VSP) and speed ranges, idling, braking, and so on. In the ITSDCAP tool, the estimation of emissions such as Carbon Monoxide (CO), Hydrocarbons (HC), and Nitrogen Oxides (NOx) can be achieved by running the MOVES in the background. Since detailed information about the driving cycles are not available from point detector data and AVI data (detailed AVL data such as GPS data at high resolution is needed), the average speed approach in MOVES is used at the current stage. Figure 2-43 presents the average CO results for the eastbound direction of the Glades Road study segment in Boca Raton, Florida, obtained using ITSDCAP. Similar to the other performance measures, the numbers on the map are the CO emissions along a sub-segment, and the curve in the chart shows the temporal variation of total CO emissions along the whole study segment.

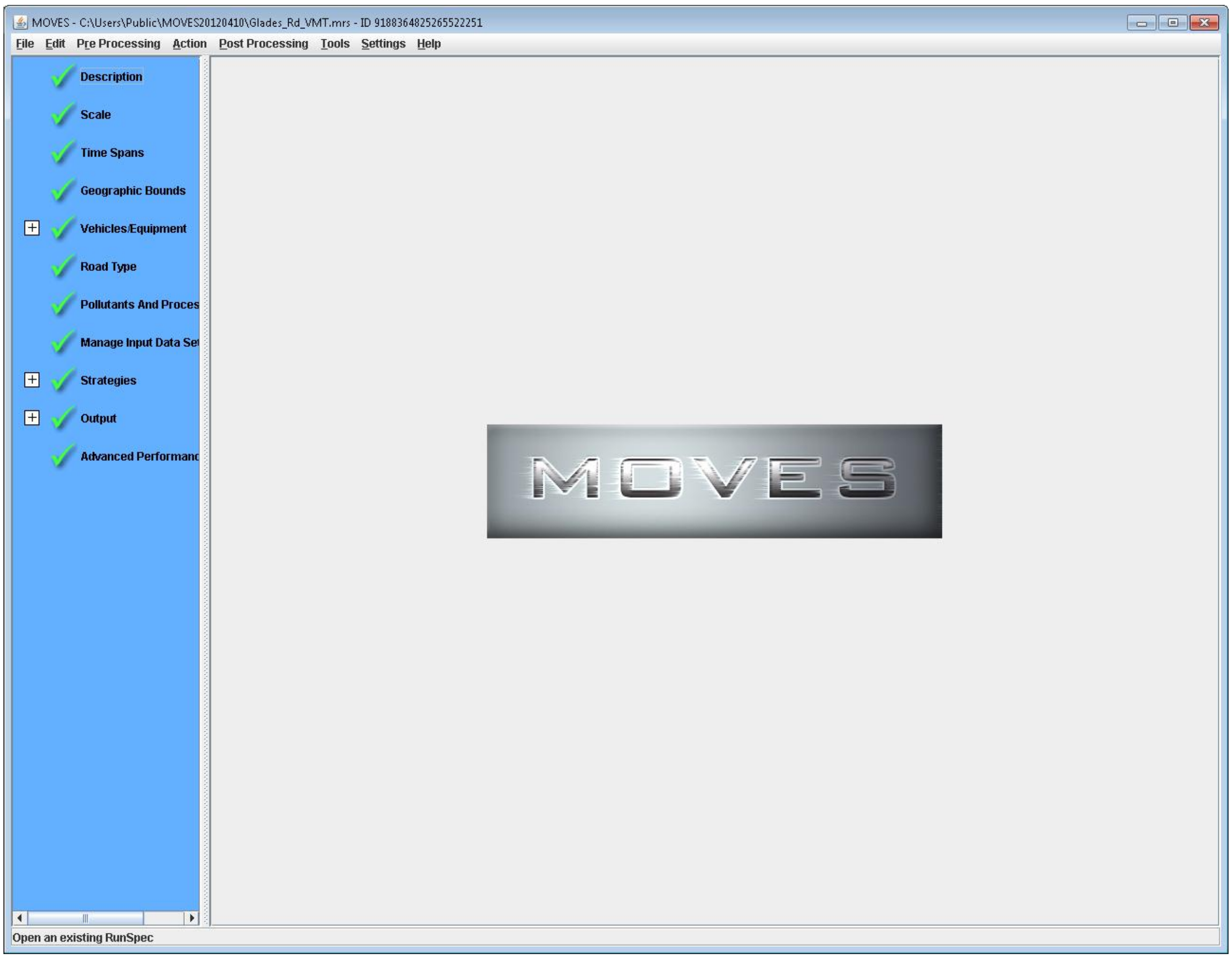


Figure 2-42 The Graphic User Interface of MOVES

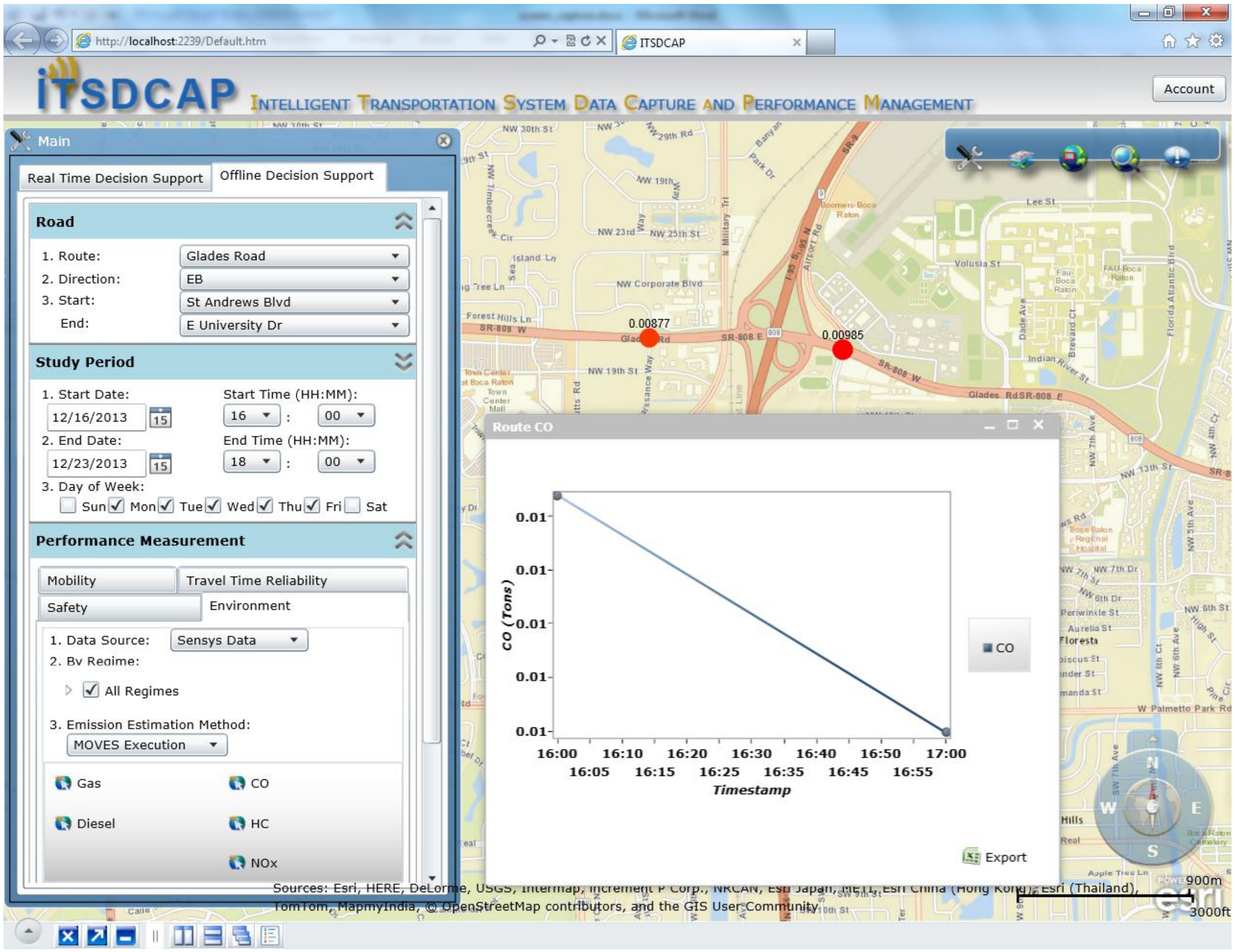


Figure 2-43 The User Interface for Energy and Emission Measure Calculation in ITSDCAP

There are certain limitations associated with the real-time execution of MOVES in ITSDCAP. First, a MOVES model needs to be created beforehand, although the ITSDCAP tool can dynamically change some of the attributes of this model, such as the calculation time period. Second, the temporal resolution in MOVES is one hour, instead of the five minutes usually used for the aggregation of traffic data. The last but the most important limitation of running MOVES in real time is that it requires a relatively long running time using the current server configuration. It may take a couple of minutes to run the MOVES model, even for the simple average speed approach and for a six-link system. Considering these limitations, a rate-based emission method is also provided in ITSDCAP. Similar to the calculation of fuel consumption, in the rate-based emission estimation method, the emissions are calculated based on the vehicle-miles traveled and predefined emission rates. The advantages of this method are that it requires much less running time, and it is able to provide the emission results at a five-minute interval. Figure 2-44 shows similar CO results as those in Figure 2-43, but uses the rate-based emission estimation method, instead of directly running the MOVES.

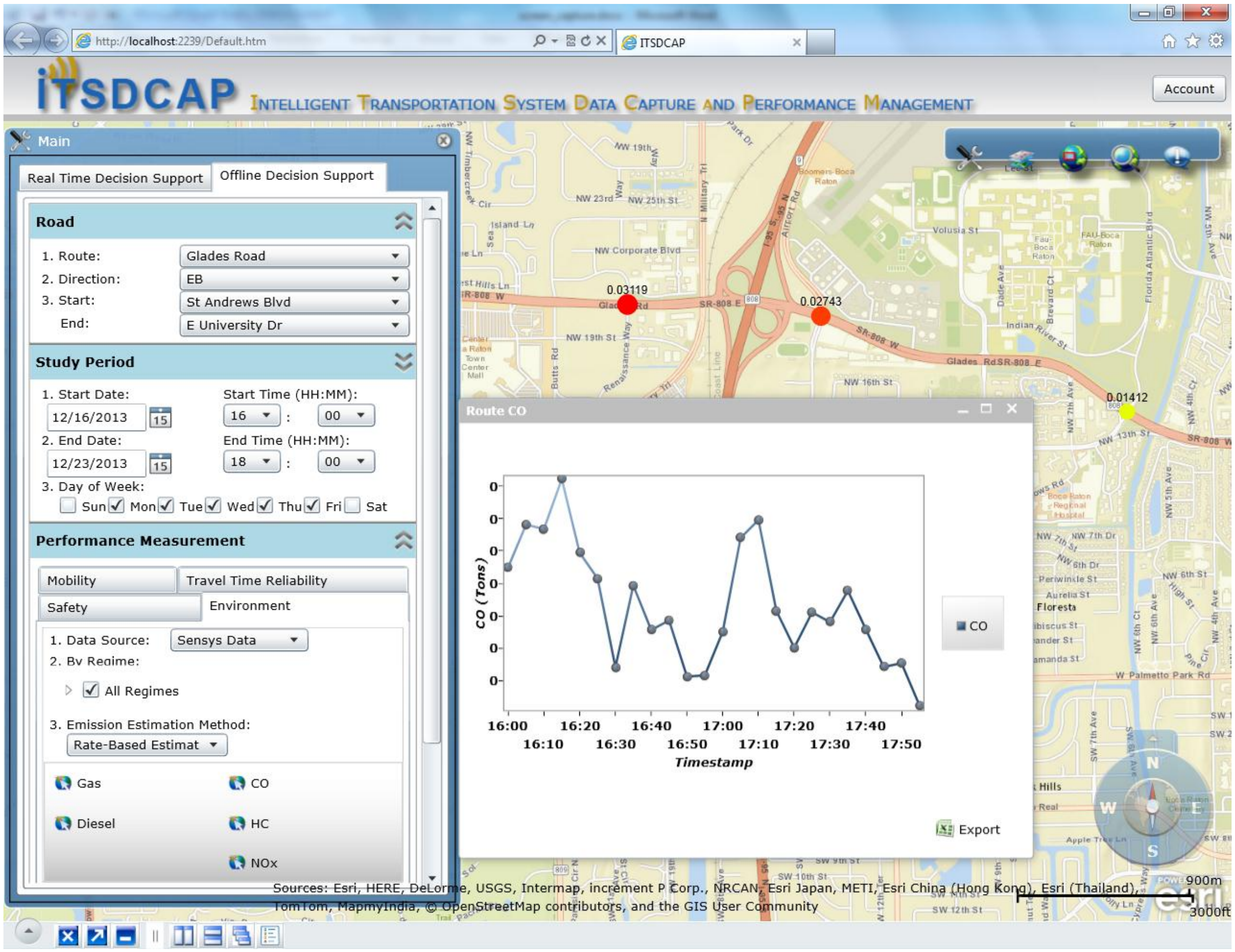


Figure 2-44 The Emission Rate-based CO Results

2.3.2 Intersection Performance Measurements

Compared to the wide deployment of data collection devices along freeways, the deployment of ITS devices along arterials lags behind. In recent years, the installation of microwave vehicle detection, Bluetooth, Wi-Fi, and magnetometer-based (e.g., Sensys devices) data collection systems along urban streets have increased. Data-based performance measures are gradually applied to monitor the performance of the arterial transportation system. Based on a thorough literature review of arterial performance measures used in the previous work (Balke et al., 2005; Smaglik et al., 2007; Day et al., 2009; Petty and Barkley, 2011; Li et al., 2013, and Bullock et al., 2014), a list of intersection-related performance metrics is proposed in ITSDCAP. Below are the descriptions of these performance measures. Some of these measures are not estimated in the current version of ITSDCAP due to the lack of data (high resolution controller and turning movement detector data). These measures are marked by a "*" in the list below and will be estimated in future efforts when the necessary data become available.

- Averages of occupancy, volume, and speed: These measures can be directly estimated from the point detector measurement installed at specific locations on the link. Speeds can also be obtained from Automatic Vehicle Identification (AVI) technologies such as Bluetooth or Wi-Fi. Average occupancy, volume and speed can be estimated for the whole approach in ITSDCAP if midblock detectors are available, however, these measures cannot be estimated for each movement due to the lack of movement detection.
- Standard deviation of occupancy, volume, and speed*: These measures refer to the standard deviations of occupancy, volume, and speed within each time interval (for example, every five minutes).
- Volume/capacity (v/c) ratio*, percentage of volume/capacity ratio greater than one*, and approach delay: These measures can be measured based on stop line detector data. Many detector settings and data availability from these detectors do not allow direct measurements of these parameters. Thus, in these cases, the capacity has to be estimated utilizing an equation based on the average green time and estimated saturation flow rate. The volumes can be estimated based on turning movement proportions multiplied by volumes measured by upstream detectors. The value of the saturation flow rate can be pre-calculated using the Highway Capacity Manual (HCM) procedure or measured in the field. The delays can also be calculated using HCM procedures.
- Green utilization: Green utilization is defined as the ratio of the time interval used to the total green time. The calculation of this measure requires high-resolution signal phase data and volume counts at the stop bars.
- Split failure percentage: A phase failure occurs when the traffic demand in a phase cannot be served by the phase green time. The split failure percentage measure can be derived based on the high resolution detector and signal data. A surrogate to this measure is the percentage of the phases that maximize out.
- Oversaturation severity index: This measure is defined as the ratio of unusable green time due to the discharge of residue queue or spillback from the downstream intersection to the total available green time in a cycle. It has a range between zero and one, with the zero value corresponding to perfect signal operation, and one indicating that all available green time is unusable. The calculation of this measure requires high-resolution vehicle actuation data and signal event data.
- Occupancy/green ratio: Phase occupancy/green ratio is the ratio of the detector occupancy during the green phase to the green time. This measure has been used as a surrogate measure to the volume/capacity ratio. It can be estimated based on signal phase data and occupancy data measured by sensors at the stop bars.
- Platoon ratio and percentage of arrival on green: These two measures are used to quantify the progression of platoon. They are related through the following equation:

$$R_p = P \times \frac{C}{g} \tag{2-6}$$

where R_P is platoon ratio and P is the percentage of arrivals on green. C denotes cycle length, and g is green time. This parameter can be measured based on high-resolution vehicle actuation data and signal event data. However, it can also be estimated based on platoon progression equations and/or AVI data.

- Green time percentage*: This measure is the percentage of time that the signal is green during a given time interval (for example, 5 minutes). The split history of signal controller can be used to calculate this measure.

Note that the performance measures that are currently available in ITSDCAP are indicated by a symbol "*". Those performance measures without the symbol "*" will be considered for implementation when related data become available.

Figure 2-45 illustrates the interface of the intersection-related performance measure module in ITSDCAP. The interface lists the potential performance measures that will be calculated for intersection operations. However, as stated earlier, the calculations of only a few measures are performed in this version of ITSDCAP due to the limited availability of high-resolution signal event data. As a proof-of-concept, intersection-related performance measures are calculated for Glades Road in Boca Raton, Florida in this project. Considering the limited data available for this corridor, the calculated intersection-related measures include the occupancy, speed, volume and their associated standard deviations, v/c ratio, and green time percentage. The results of the v/c ratio for this study corridor can also be visualized, as shown in Figure 2-45, in which the minimum, maximum, mean, median, and 95th percentile v/c ratio at the studied intersections are displayed. When the user clicks the colored point on the map, a pop-up window will be displayed, allowing the user to identify the performance of signal operations by visualizing the variations of the v/c ratio for a specific day.

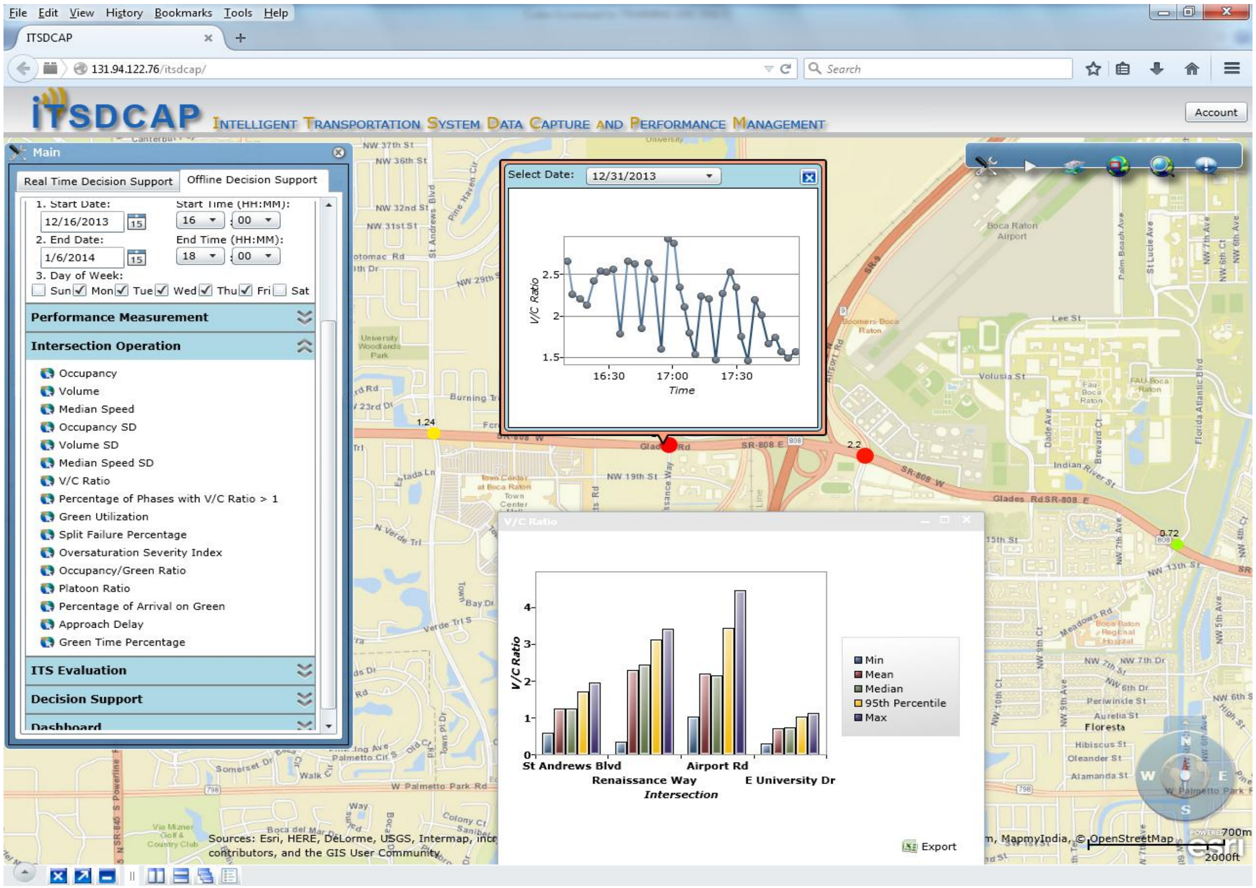


Figure 2-45 The Interface of Intersection-Related Performance Measure Module in ITSDCAP

2.4 Dashboard Module in ITSDCAP

The Dashboard Module in ITSDCAP is designed to help agencies monitor their facilities and generate performance measure reports. As stated earlier, these dashboards can be customized for each agency based on agency requirements and data availability. At the present time, the way that ITSDCAP works is that the agency can contact the developer and an agency-specific dashboard will be produced for the agency based on the requirement. However, we will also produce an excel file with data can be used externally to create the dashboard. An example of a customized dashboard is the performance measure dashboard that was created in ITSDCAP for the Broward County Arterial Management Program (AMP). To better manage arterial traffic, the FDOT D4 TSM&O program has heavily invested in deploying data collection devices along the major corridors in Broward County. The performance of the arterial transportation system is monitored through a monthly performance measure dashboard and reports. These reports were produced manually by the FDOT District 4 consultant. In this project, this process was automated and incorporated in the ITSDCAP tool.

2.4.1 Data Collection and Preprocessing

Two types of ITS devices are deployed along major arterials in Broward County, Florida: Bluetooth readers and Microwave Vehicle Detection System (MVDS). The icons in Figures 2-46 and 47 indicate the locations of the Bluetooth and MVDS devices, respectively. A total number of 53 Bluetooth readers are deployed at the intersection locations along major Broward County arterials. The locations of each reader expressed in latitude, longitude, roadway, and cross street are retrieved from the Broward County Regional Traffic Management Center (RTMC) databases. Data from these readers allow for the estimation of travel time and speed along predefined routes, as shown in Figure 2-48. It should be noted that the estimations are not reported for certain time intervals due to insufficient sample sizes (low market penetration of mobile devices with Bluetooth activated). In addition to the Bluetooth devices, a total number of 48 MVDS devices are installed at selected midblock locations of the same major arterials where the Bluetooth readers were installed. These MVDS devices measure traffic counts, point speeds, and occupancies at the detection locations. Figure 2-49 shows an example of the aggregated MVDS volume data file, retrieved from the Broward County RTMC. The hourly lane-by-lane traffic counts are reported in this file. The data collected by the Bluetooth readers and MVDS devices are imported into the Oracle database used by the ITSDCAP tool. Additional information, such as the reader link name and direction, MVDS device ID, and associated roadway information are added to the data table.

In addition to the two types of traffic data mentioned above, event data are also needed for producing the dashboard. Again, detailed event data are retrieved from the Broward County RTMC. Figure 2-50 displays an example of an event data file. As shown in this figure, the following event information is stored in the event data file:

- Event number
- Event type
- Report date, confirmed date, and closed date
- Event location
- Event duration
- Contact information
- Roadway conditions
- Whether the event involves rollover, fire or HAZMAT
- Number of lane blockages and durations
- Operator comments

The event information is extracted from the event data file and imported into the Oracle database by programming.

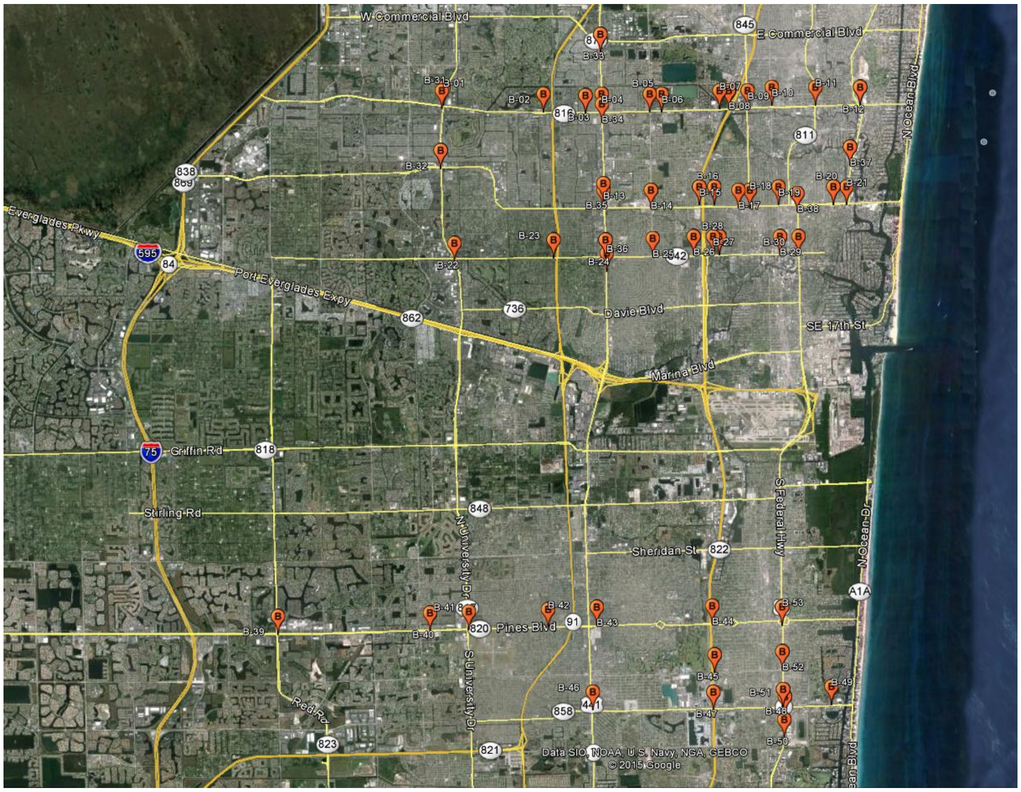


Figure 2-46 The Locations of the BlueToad Devices in Broward County, FL

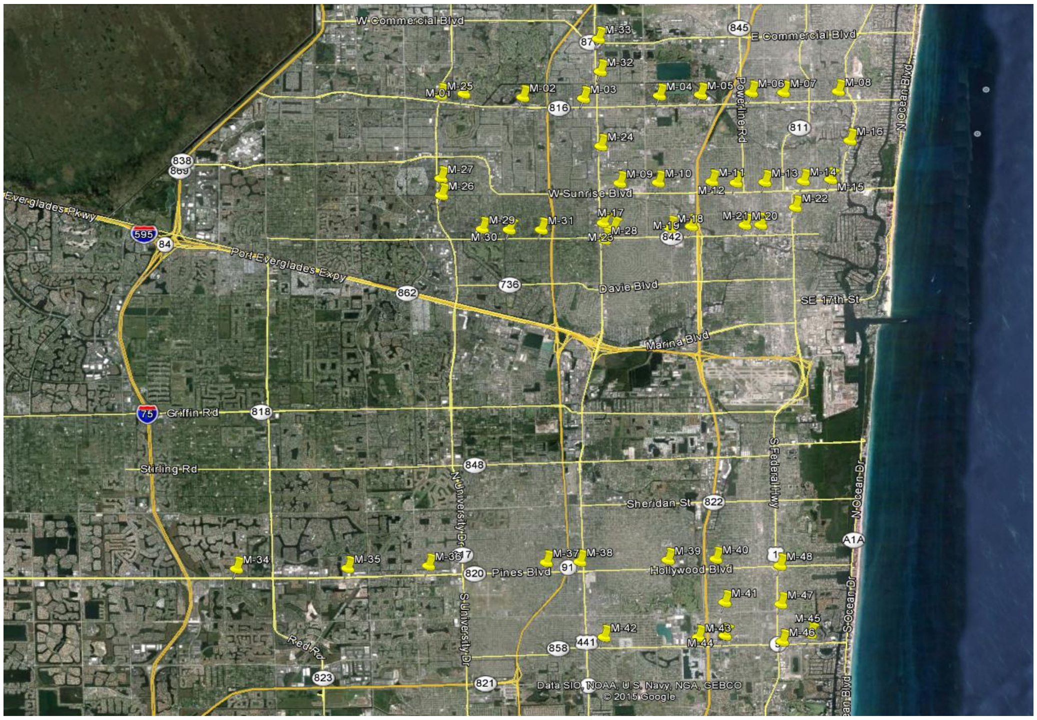
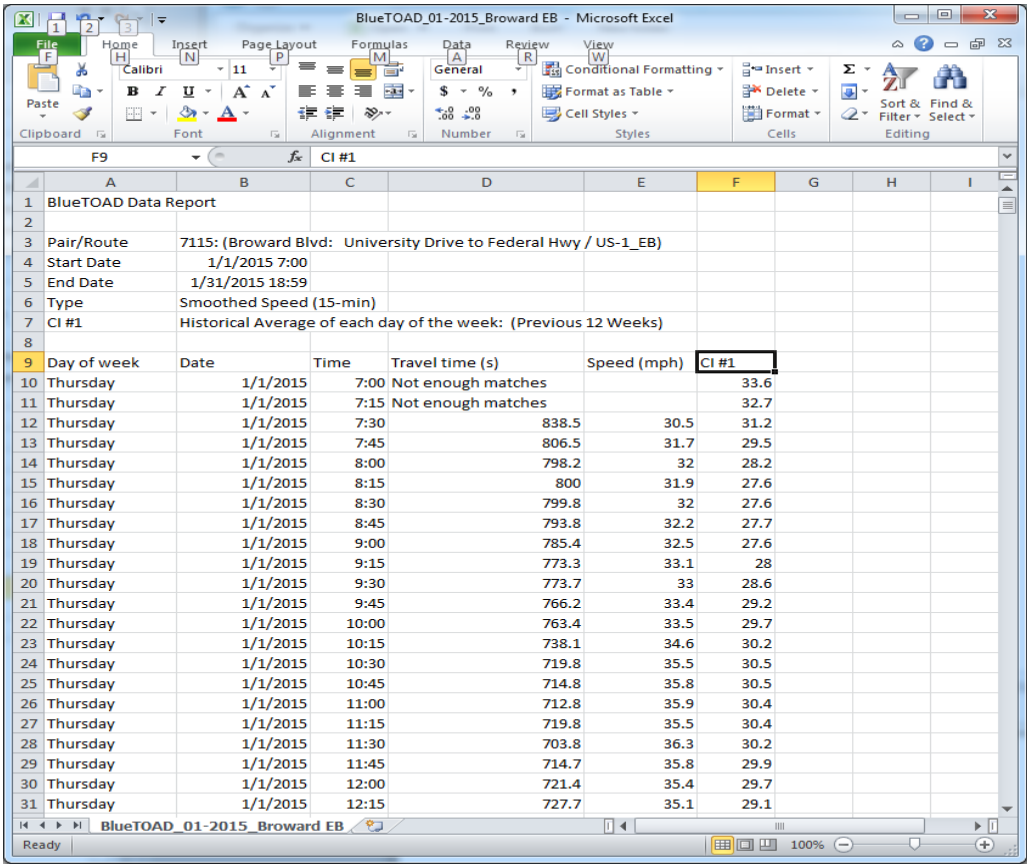
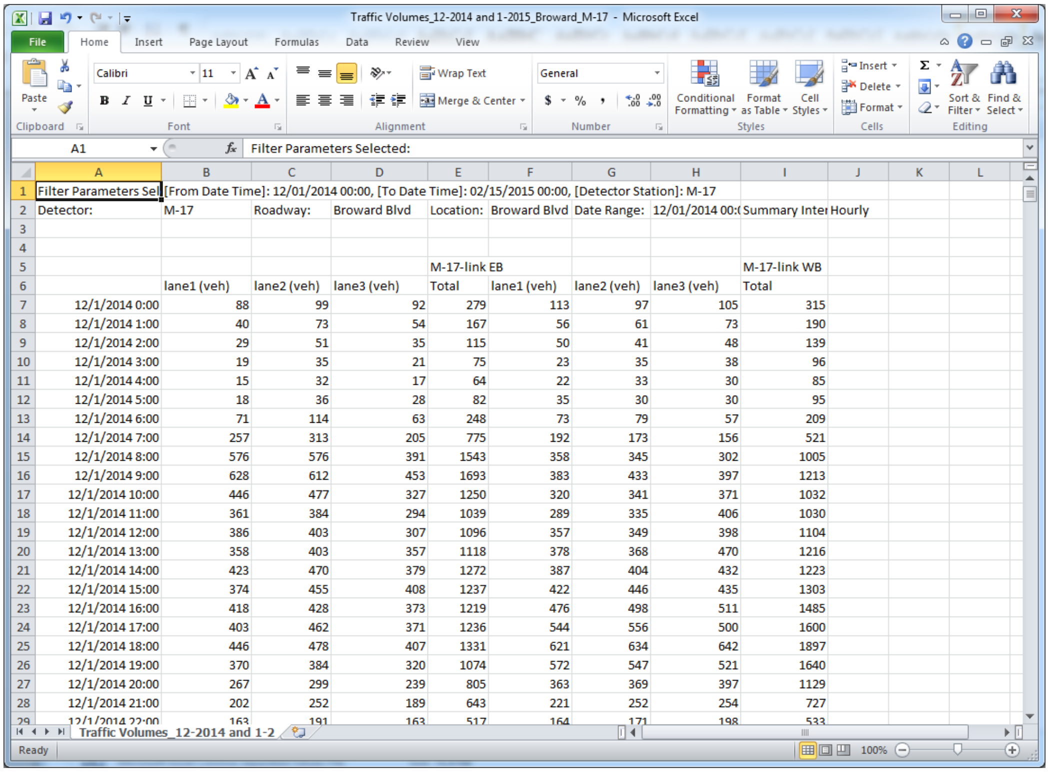


Figure 2-47 The Locations of the MVDS Devices in Broward County, FL



	A	B	C	D	E	F
1	BlueTOAD Data Report					
2						
3	Pair/Route	7115: (Broward Blvd: University Drive to Federal Hwy / US-1_EB)				
4	Start Date	1/1/2015 7:00				
5	End Date	1/31/2015 18:59				
6	Type	Smoothed Speed (15-min)				
7	CI #1	Historical Average of each day of the week: (Previous 12 Weeks)				
8						
9	Day of week	Date	Time	Travel time (s)	Speed (mph)	CI #1
10	Thursday	1/1/2015	7:00	Not enough matches		33.6
11	Thursday	1/1/2015	7:15	Not enough matches		32.7
12	Thursday	1/1/2015	7:30	838.5	30.5	31.2
13	Thursday	1/1/2015	7:45	806.5	31.7	29.5
14	Thursday	1/1/2015	8:00	798.2	32	28.2
15	Thursday	1/1/2015	8:15	800	31.9	27.6
16	Thursday	1/1/2015	8:30	799.8	32	27.6
17	Thursday	1/1/2015	8:45	793.8	32.2	27.7
18	Thursday	1/1/2015	9:00	785.4	32.5	27.6
19	Thursday	1/1/2015	9:15	773.3	33.1	28
20	Thursday	1/1/2015	9:30	773.7	33	28.6
21	Thursday	1/1/2015	9:45	766.2	33.4	29.2
22	Thursday	1/1/2015	10:00	763.4	33.5	29.7
23	Thursday	1/1/2015	10:15	738.1	34.6	30.2
24	Thursday	1/1/2015	10:30	719.8	35.5	30.5
25	Thursday	1/1/2015	10:45	714.8	35.8	30.5
26	Thursday	1/1/2015	11:00	712.8	35.9	30.4
27	Thursday	1/1/2015	11:15	719.8	35.5	30.4
28	Thursday	1/1/2015	11:30	703.8	36.3	30.2
29	Thursday	1/1/2015	11:45	714.7	35.8	29.9
30	Thursday	1/1/2015	12:00	721.4	35.4	29.7
31	Thursday	1/1/2015	12:15	727.7	35.1	29.1

Figure 2-48 An Example BlueToad Data File



	A	B	C	D	E	F	G	H	I	J
1	Filter Parameters Sel	[From Date Time]: 12/01/2014 00:00, [To Date Time]: 02/15/2015 00:00, [Detector Station]: M-17								
2	Detector:	M-17	Roadway:	Broward Blvd	Location:	Broward Blvd	Date Range:	12/01/2014 00:0	Summary Inter	Hourly
3										
4										
5					M-17-link EB				M-17-link WB	
6		lane1 (veh)	lane2 (veh)	lane3 (veh)	Total	lane1 (veh)	lane2 (veh)	lane3 (veh)	Total	
7	12/1/2014 0:00	88	99	92	279	113	97	105	315	
8	12/1/2014 1:00	40	73	54	167	56	61	73	190	
9	12/1/2014 2:00	29	51	35	115	50	41	48	139	
10	12/1/2014 3:00	19	35	21	75	23	35	38	96	
11	12/1/2014 4:00	15	32	17	64	22	33	30	85	
12	12/1/2014 5:00	18	36	28	82	35	30	30	95	
13	12/1/2014 6:00	71	114	63	248	73	79	57	209	
14	12/1/2014 7:00	257	313	205	775	192	173	156	521	
15	12/1/2014 8:00	576	576	391	1543	358	345	302	1005	
16	12/1/2014 9:00	628	612	453	1693	383	433	397	1213	
17	12/1/2014 10:00	446	477	327	1250	320	341	371	1032	
18	12/1/2014 11:00	361	384	294	1039	289	335	406	1030	
19	12/1/2014 12:00	386	403	307	1096	357	349	398	1104	
20	12/1/2014 13:00	358	403	357	1118	378	368	470	1216	
21	12/1/2014 14:00	423	470	379	1272	387	404	432	1223	
22	12/1/2014 15:00	374	455	408	1237	422	446	435	1303	
23	12/1/2014 16:00	418	428	373	1219	476	498	511	1485	
24	12/1/2014 17:00	403	462	371	1236	544	556	500	1600	
25	12/1/2014 18:00	446	478	407	1331	621	634	642	1897	
26	12/1/2014 19:00	370	384	320	1074	572	547	521	1640	
27	12/1/2014 20:00	267	299	239	805	363	369	397	1129	
28	12/1/2014 21:00	202	252	189	643	221	252	254	727	
29	12/1/2014 22:00	163	191	163	517	164	171	198	533	

Figure 2-49 An Example MVDS Data File

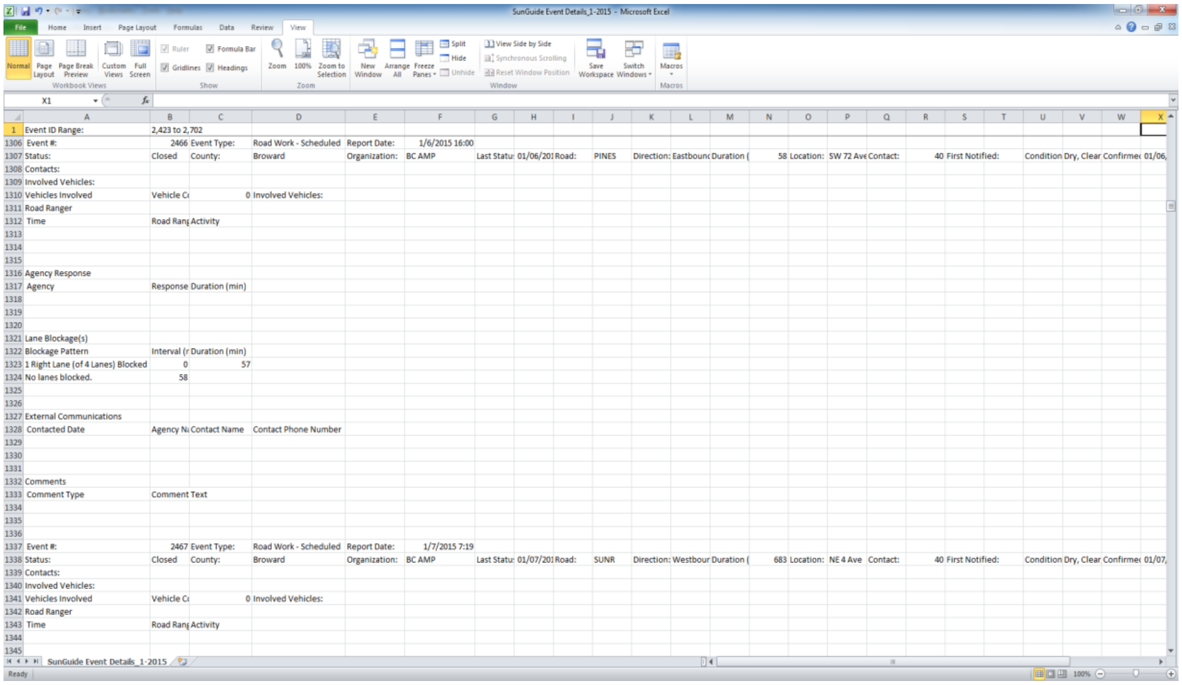


Figure 2-50 An Example Event Data File

2.4.2 Dashboard Implementation in ITSDCAP

The monthly performance measurement report for the Broward County Arterial Management Program (AMP) is implemented in ITSDCAP. Figure 2-51 shows a snapshot of the dashboard interface in ITSDCAP, as well as three types of Broward County dashboards can be generated using the ITSDCAP tool: corridor-level dashboard, county-level summary dashboard, and county-level dashboard.

In the corridor-level dashboard, the performance measures are reported for the AM and PM peaks. The maps at the top of this dashboard display the average speeds for both directions of the study corridors. These values are calculated from the link distance and the average link travel times based on Bluetooth data. Other performance measures calculated for the corridor include mean travel time, mean travel time index, average travel time variance defined as the difference between average travel time and historical travel time, average hourly volume counts from the MVDS data, and the frequency and duration of events that occurred along the study corridor. Figure 2-52 shows an example of the monthly performance measure dashboard for Oakland Park Boulevard in January 2015. Since this dashboard is long, only a partial dashboard can be captured from the ITSDCAP interface, as shown in Figure 2-52(a). A full corridor-level dashboard is presented in Figure 2-52(b).

Figure 53 shows a snapshot of the county-level summary dashboard, and the reported number of events and average event durations. The percentage of each type of event is shown in a pie chart. The total number of county-owned devices such as CCTV cameras, MVDS, AVI devices, arterial dynamic message signs, and fiber optic systems are summarized in a table.

An example of a county-level reliability dashboard for the Broward County AMP is shown in Figure 54. Similar to the corridor-level dashboard, the average speed based on Bluetooth readers is displayed on the map for both directions of the corridors equipped with the readers. Even though the speed values can also be obtained from the MVDS sensors, these speeds are point measurements and are not used in the dashboard. The travel times reported by the Bluetooth devices for the roadway segments can capture the travel time variations along the segments and thus are used as the source of travel time data. The performance measures displayed in the charts are the mean speed, 80th percentile speed, mean travel time index (TTI), 80th percentile TTI, total delay, and standard deviation of travel time. The mean and 80th percentile travel times are not reported in the dashboard, as these values vary with the route length and are not comparable among different routes. However, the mean and 80th percentile speeds are straightforward measurements that allow transportation agencies to easily understand traffic conditions along the study corridors; thus, these two speed measures are presented in the charts. The total delay displayed in this dashboard is calculated based on the traffic counts from the MVDS data and the average vehicle delays estimated from the difference between the Bluetooth-based travel time estimation and the free-flow travel time. In addition, the average values of the mean, 80th percentile, and 95th percentile TTI of all of the study corridors during the AM and PM peaks are listed at the bottom of the dashboard to show the overall travel time reliability of the Broward corridors.

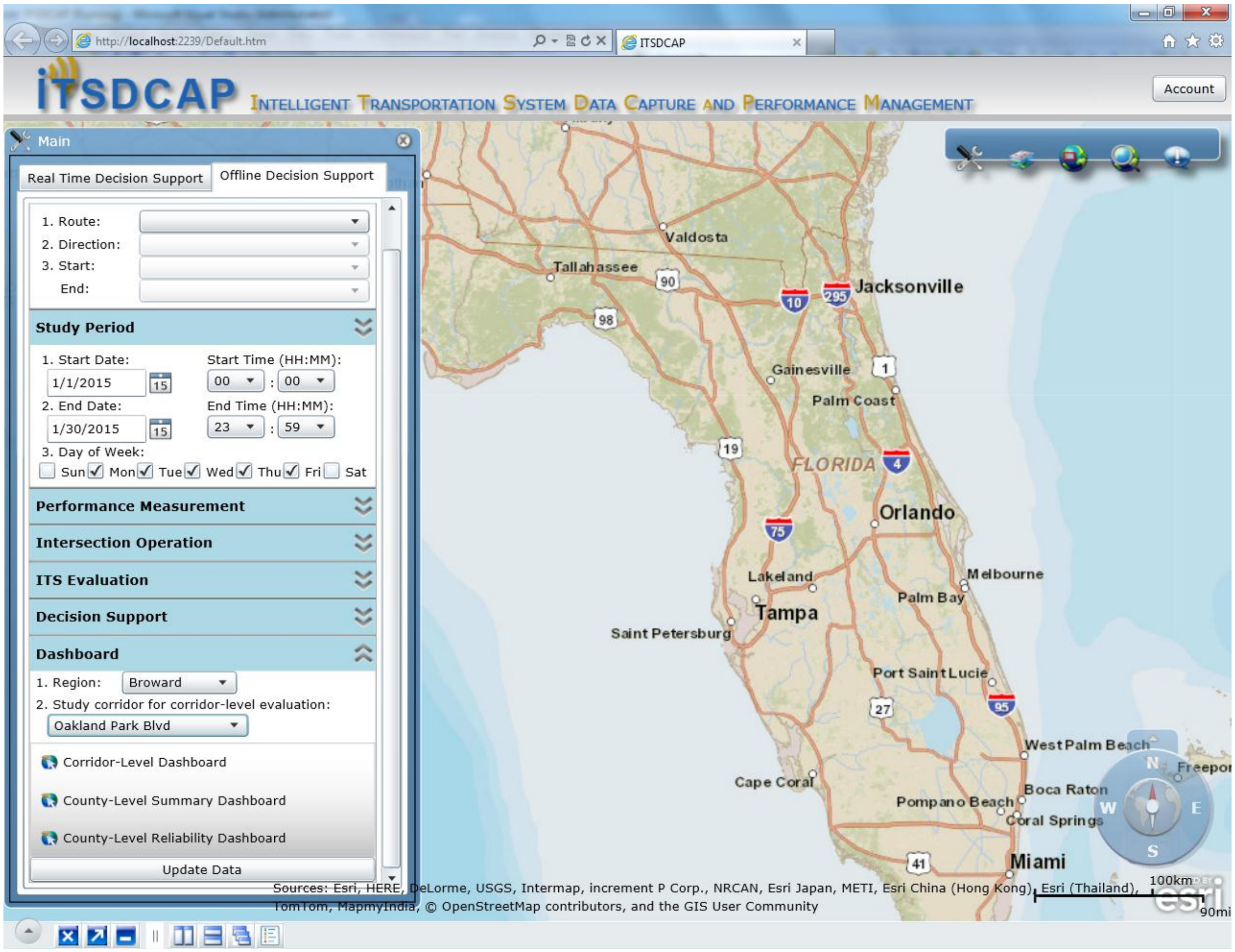
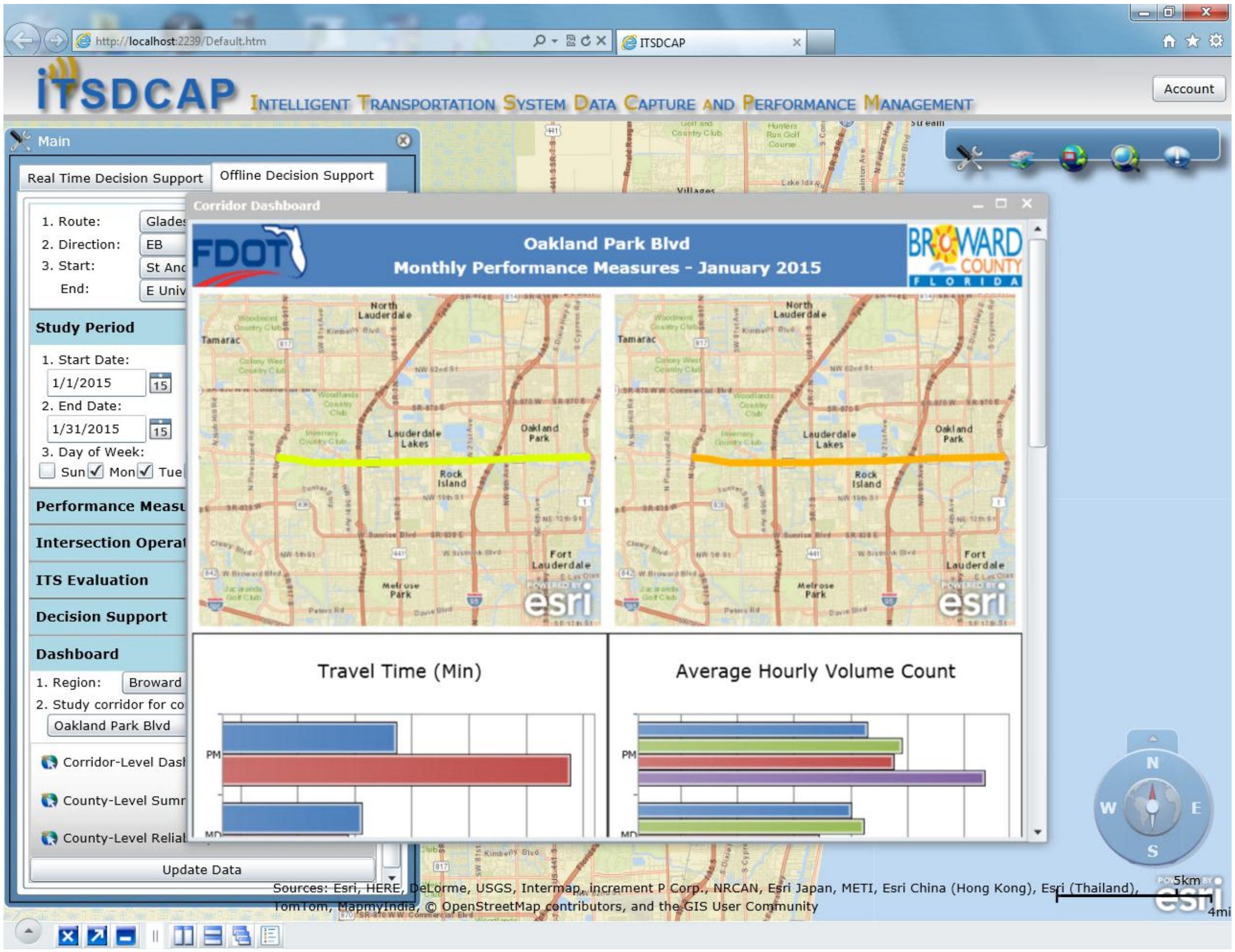
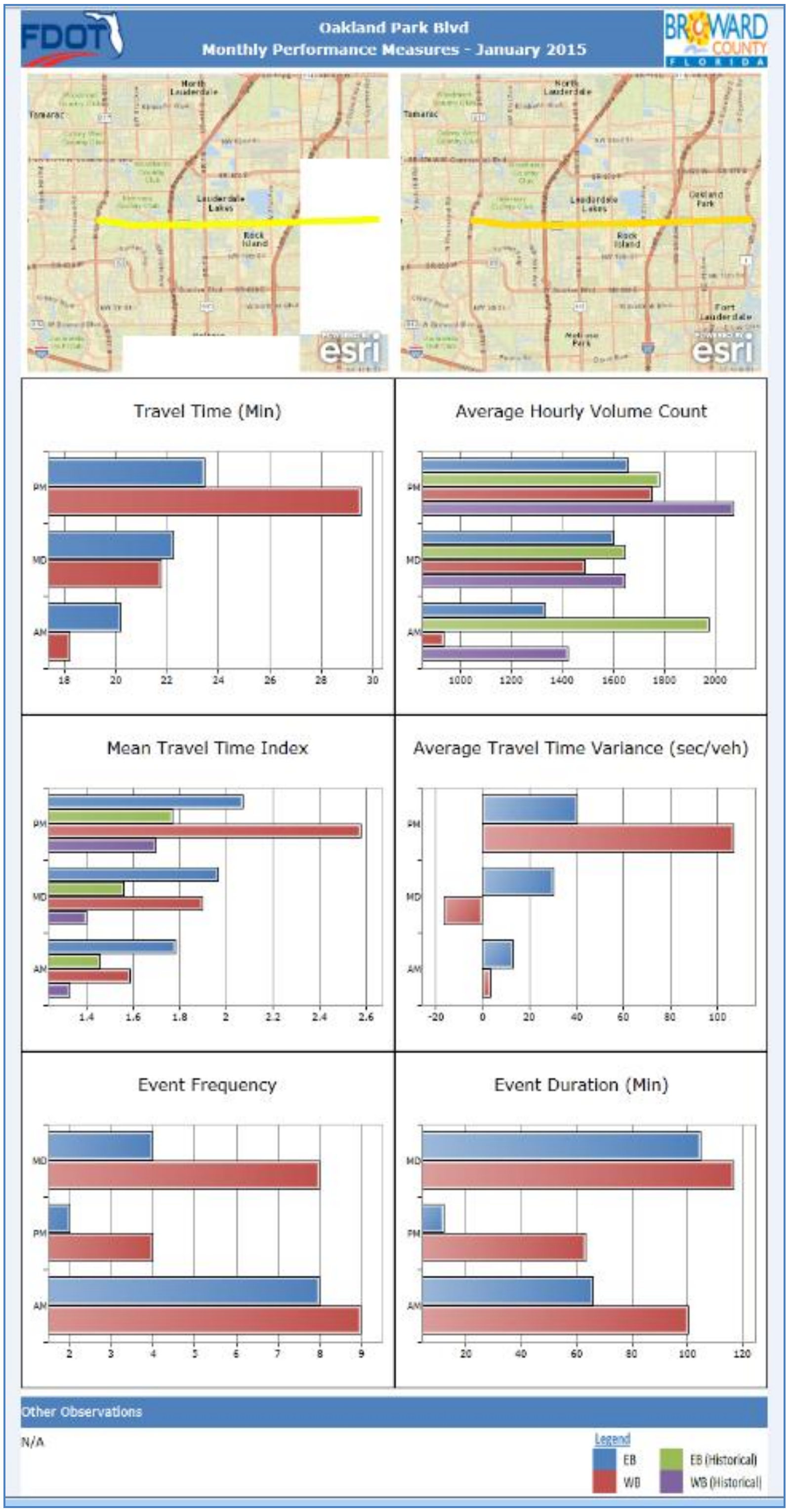


Figure 2-51 Snapshot of Dashboard Interface in ITSDCAP



(a) ITSDCAP Interface

Figure 2-52 Example Corridor-level Dashboard (Continued on next page)



(b) A Full Corridor-Level Dashboard
Figure 2-52 Example Corridor-Level Dashboard

(a) ITSDCAP Interface

Figure 2-53 Snapshot of the County-Level Summary Dashboard

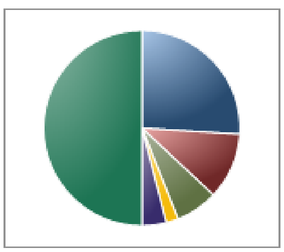
Broward Transportation Management Center
Active Arterial Traffic Management Program
Performance Measures - Monthly Report
January 2015

Number of Events	Number of Signal Timing Changes	Average Event Duration (Minutes)	Benefit-to-Cost (B/C)	Net Benefit Value
260	N/A	269	N/A	N/A

Estimated Monthly Benefits Summary

Performance Measure	Benefits/Costs (Dollars)
Travel Time/Delay Savings	N/A
Reduction in Emissions	N/A
Reduction in Fuel Consumption	N/A
Safety Benefits	N/A
Total Benefits	N/A
Estimated Monthly O&M Costs	N/A

Incident Summary



ITS Device Summary (County Maintained)

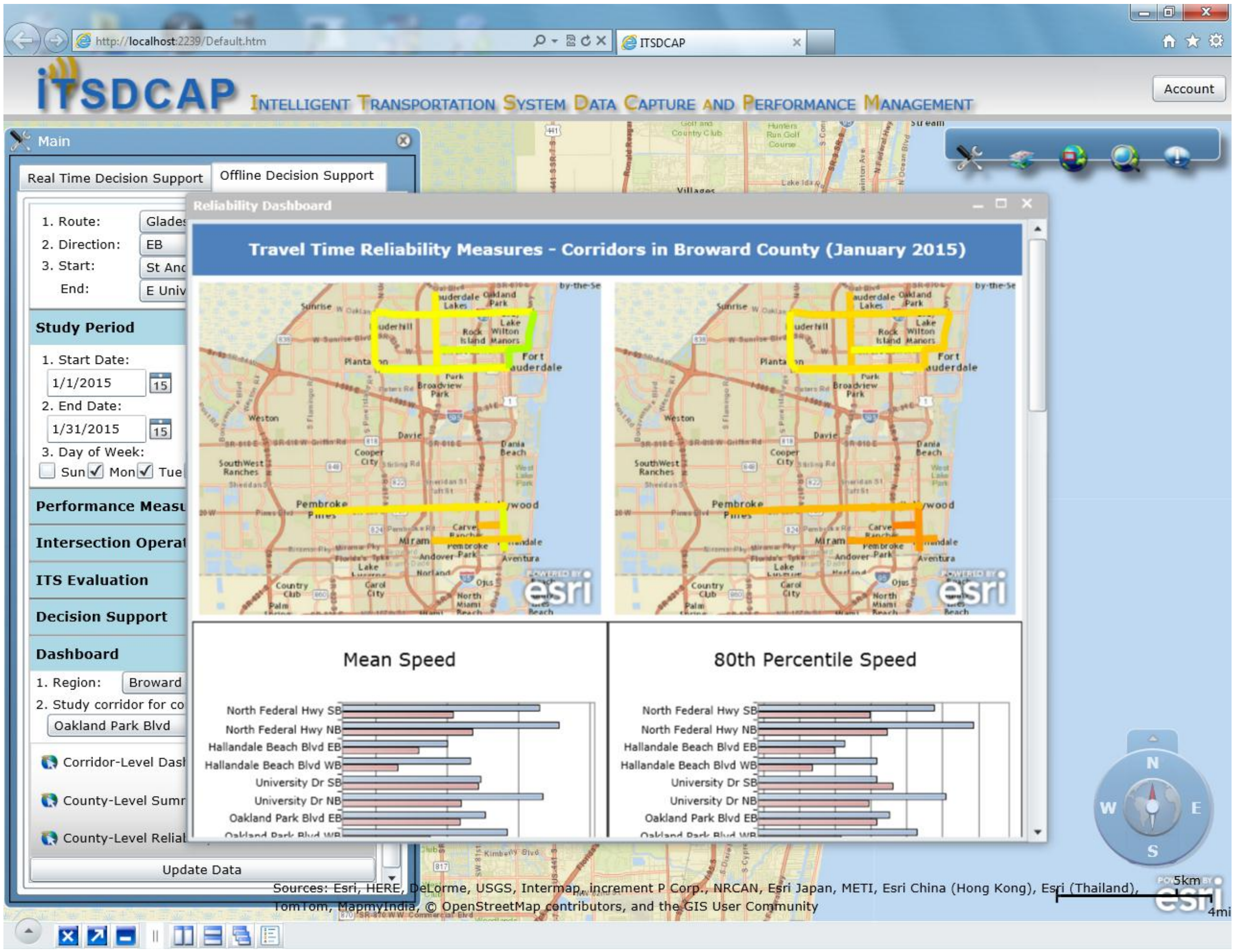
Device Type	Number of Devices
Online Traffic Signals (Percentage)	N/A
Signal System Health	N/A
Incident Management CCTV Cameras	63
Microwave Vehicle Detection System (MVDS)	48
AVI Devices	33
BlueTOAD Devices	53
Fiber Optic System (Miles)	22
Opticom Devices	N/A
Arterial Dynamic Message Signs (ADMS)	14

Broward County Arterial Management Program

FDOT District 4 in collaboration with Broward County started the Arterial Management Program in 2012 to actively monitor, manage, and improve arterial operations along six major corridors – Oakland Park Boulevard, Sunrise Boulevard, Broward Boulevard, University Dr., SR-7, and Federal Highway. As part of this initiative, FDOT District 4 installed several CCTV cameras and BlueTOAD vehicle detection devices along these corridors to monitor traffic conditions and collect travel times in real-time. In addition, FDOT District 4 provided staffing resources at the Broward County Traffic Management Center to monitor real-time traffic conditions and detect incidents. The hours of operation are Monday through Friday from 7AM to 7PM (reduced staffing during off-peak hours).

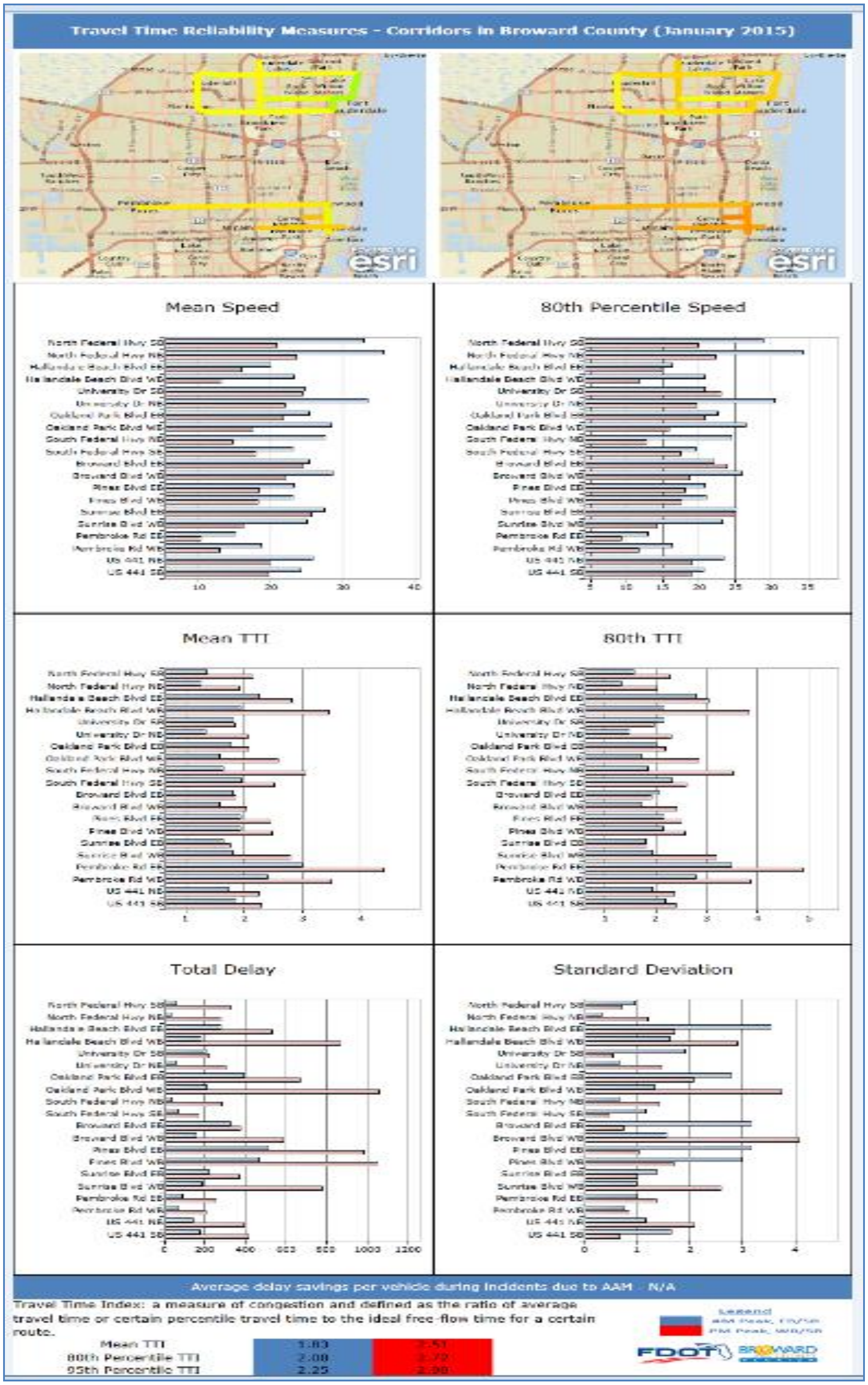
(b) A Full County-Level Summary Dashboard

Figure 2-53 Snapshot of the County-Level Summary Dashboard



(a) ITSDCAP Interface

Figure 2-54 Snapshot of the County-Level Reliability Dashboard



(b) A Full County-Level Reliability Dashboard

Figure 2-54 Snapshot of the County-Level Reliability Dashboard

3 INCORPORATING THE PROBABILITY OF BREAKDOWN

3.1 Introduction

When demand approaches a recurrent bottleneck's capacity, breakdown will occur. The term "breakdown" has been used to describe the point of transition to congestion. The occurrence of traffic breakdown not only increases travel time, but also reduces roadway throughput and thus increases the congestion impacts. Advanced traffic management strategies such as ramp metering, variable speed limit, congestion pricing, and advanced signal control strategies can be designed and implemented to reduce the probability of breakdown. For effective implementation of these strategies, accurate estimation and prediction of breakdown probability are needed to support the selection of these advanced strategies.

Studies have been conducted to explore the breakdown phenomenon along freeways. Based on these studies, breakdown is said to occur at a freeway location, when the speed is reduced by a predefined threshold or the speed is determined to be consistently lower than a given value for a certain time period. Examples of the reduction in speed criteria are 10 mph (Elefteriadou et al., 1995) and 6.22 mph (Brilon et al., 2005 and Brilon, 2005). Examples of speed thresholds are 43.5 mph (Brilon et al., 2005 and Brilon, 2005), 40 mph specified in the MUTCD (FHWA, 2000), 30 mph (Graves et al., 1998), and 25 mph (Okamura et al., 2000). The duration of such traffic conditions is usually required to be greater than 5 or 15 minutes before declaring that the breakdown has occurred (Graves et al., 1998; Persaud et al., 1998 and 2001; Okamura et al., 2000). The characteristics of freeway breakdown have been examined in terms of the duration of breakdown, average speed during breakdown, maximum pre-breakdown volume, and queue discharge rate during breakdown based on real-world traffic detector data (Elefteriadou and Lertworawanich, 2003; Geistefeldt, 2008; Cassidy and Bertini, 1999; Muñoz and Daganzo, 2003; Sarvi et al., 2007). Methods have also been developed to estimate the probability of freeway breakdown (Elefteriadou et al., 1995; Evans et al., 2001; Chow et al., 2009; Kondyli, 2009). However, no study has been found that addresses the arterial traffic breakdown.

One of the goals of this project is to investigate approaches to predict breakdown on arterial streets. Both the freeway and arterial breakdown prediction models will then be integrated in an Intelligent Transportation System Data Capture and Performance Management (ITSDCAP) tool developed by this research team for real-time prediction of probability of breakdown.

3.2 Probability of Breakdown for Freeway Facilities

Due to the stochastic characteristics of traffic, breakdown may or may not occur at the same bottleneck locations, even for the same combination of mainline and ramp demands. This has resulted in a significant interest in deriving the probability of breakdown along freeway segments based on measured parameters. Lorenz and Elefteriadou (2001) used the probability of breakdown to define freeway capacity. Evans et al. (2001) used the discrete-time Markov chain, a stochastic process with memoryless property, to develop the probability distribution of the time of freeway breakdown. Chow et al. (2009) conducted empirical analysis of traffic breakdown based on the use of the Weibull distribution in a bivariate form. Kondyli (2009) estimated the probability of breakdown based on speed or occupancy measurements at the bottleneck location using the Kaplan-Meier method, which is a product-limit method. Since the measurements of speed, volume and occupancy from point traffic detectors are the most widely available data for freeways, the method proposed by Kondyli (2009) is applied in this study to construct the probability of breakdown models for freeways.

3.2.1 Data Collection

The first step of this method involves collecting traffic, weather, and incident data for the recurrent bottleneck locations along the study corridor, where traffic demand is greater than roadway capacity. Point traffic detector data can be downloaded from the Regional Integrated Transportation Information System (RITIS) website. The downloaded data includes the volume, speed, and occupancy measurements at the detection stations located upstream and downstream of the bottleneck, as well as at the merging on-ramp. The temporal aggregation level of the downloaded detector data is one minute, as required by the methodology. The SunGuide incident database provides detailed incident information, including the timestamps that the incident is detected, responded to, and cleared, and various incident attributes and incident management parameters. Weather data, including 15-minute precipitation data, can be retrieved from the National Climatic Data Center. The incident and weather data allow traffic data to be further filtered to include only those measurements under normal traffic conditions without incidents and adverse weather for weekdays. This is important since the derived probability of breakdown is for recurrent congested conditions and does not address incident and weather events.

3.2.2 Development of Breakdown Probability Model

In accordance with the method developed by Kondyli (2009), the processed traffic data are fed into a speed-based algorithm to identify the occurrence of breakdown events at bottleneck locations. In this algorithm, sharp changes in speed are identified to determine the timestamp when breakdown occurs and the timestamp when traffic is recovered to the normal condition. Equations 3-1 to 3-3 list the criteria for the determination of breakdown occurrence.

$$\Delta S_i = S_{i+1} - S_i \tag{3-1}$$

$$Avg\ \{S_{i-4},\ \ldots,\ S_i\} > Avg\{S_{i+1},\ \ldots,\ S_{i+5}\} + X\ mi/h \tag{3-2}$$

$$Max\ \{S_{i+1}, \ldots, S_{i+Y}\} < S_i \tag{3-3}$$

where ΔS_i is the speed difference between two consecutive timestamps i and $i+1$. According to Equations 3-2 and 3-3, the breakdown occurs at timestamp i only when the average speed of the previous 5 minutes, including the speed at timestamp i, is greater than the average speed of the following 5 minutes by a predefined X mph threshold, and the maximum speed during the following Y minutes is less than the speed at time i. In this study, X is set at 10 mph, and Y is set to 5 minutes, according to Kondyli's (2009) recommendations.

The criteria to determine the timestamp of recovery are listed in Equations 3-4 and 3-5.

$$S_j - S_{j-1} > 0 \tag{3-4}$$

$$Min\ \{S_j, \ldots, S_{j+Y-1}\} > Avg\ \{S_{i+1}, S_i\} \tag{3-5}$$

These two criteria require that the speed at time j to be greater than the speed at the previous timestamp and the minimum speed of the following Y minutes to be greater than the average of the measured speeds before and after breakdown.

Once breakdown points are identified in the archived data, the processed data are classified into two sets, breakdown set and no-breakdown set. The breakdown set includes all of the measurements reported at the occurrence of breakdown, while the no-breakdown set consists of data before the breakdown and after the recovery. The non-parametric Product-Limit Method (PLM) proposed for use by Kondyli (2009) to estimate breakdown probability was then used. PLM is a method that estimates the survival function based on lifetime data. Equation 3-6 presents the mathematical expression of this distribution based on volume. Models based on occupancy can alternatively be derived in a similar manner.

$$F(q) = 1 - \prod_{i:q_j \le q} \frac{k_i - 1}{k_i}; i \in \{B\} \tag{3-6}$$

Where $F(q)$ represents the probability of breakdown with a traffic volume of q. For each interval i in breakdown set B, k_i denotes the number of intervals with the traffic volume of $q_i \le q$. A similar expression to Equation 3-6 can be applied to generate the probability model based on occupancy.

Once the breakdown probability models are developed based on utilizing the PLM method, these discrete models can be fitted with different statistical distributions, such as normal, Lognormal, Weibull, and Logistic distributions using the Maximum Likelihood Estimation (MLE) method.

3.2.3 Examples of Derived Breakdown Probability Models

This section presents the breakdown probability models developed in a previous project by Hadi et al. (2014), using the procedures discussed above. Figure 3-1 shows the distribution of breakdown based on downstream occupancy at the NW 103rd Street on-ramp merge point to the I-95 northbound (NB) facility in Miami, Florida. This merging location is one of the main recurrent I-95 bottlenecks in the northbound direction. Two distributions are presented Figure 3-1. The first is obtained using the PLM method, and the second using the Weibull distribution fitted to the detector data. The curves in this figure show that when the downstream occupancy is less than 20%, the probability of breakdown is less than 0.1, while the probability of breakdown increases to 1 as the downstream occupancy increases from 20% to about 35%.

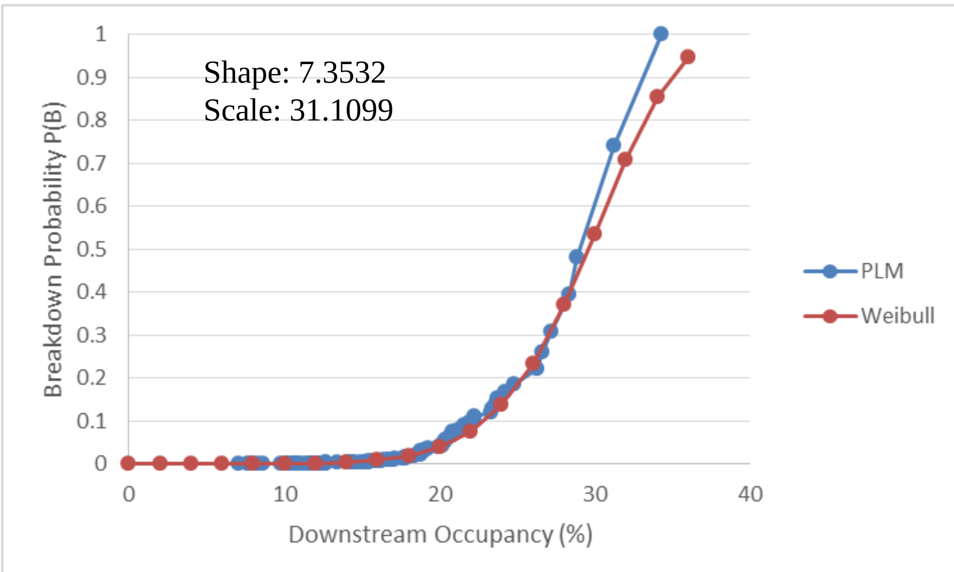


Figure 3-1 Breakdown Probability Model at NW 103rd Street Based on Downstream Detector Occupancy

3.3 Probability of Breakdown for Arterial Facilities

As mentioned in the previous section, most if not all of past research on the probability of breakdown has focused on freeways. No research was found in existing literature on estimating the probability of breakdown on urban arterial streets. Recent related research on urban arterial streets focuses on the performance measurement of arterial streets. These areas of research are of interest to this study since it may be possible to predict the probability of breakdown based on the estimated performance measures. Due to the significant variation in the types and locations of traffic sensors on urban streets, researchers have been able to use different types of data to estimate arterial performance. Smaglik et al. (2011) compared the use of volume to capacity ratio (v/c) and green to occupancy ratio (GOR) to measure saturation levels using high resolution stop bar data and signal data. In another paper, Smaglik et al. (2007) presented methods to estimate different measures of effectiveness (MOE), including v/c ratio, equivalent hourly volume, arrival type and delays, and from cycle-by-cycle signal and traffic data collected from stop bar and setback detectors. Sharma et al. (2007) compared the use of the input-output volume technique (use of advanced detector actuations, phase change data, and parametric data) with other techniques to estimate queue length and delay. They concluded that the input-output technique is more accurate in low-volume conditions, whereas a hybrid technique (which also uses stop bar detector actuations, along with the data used in the input-output method), works better in heavy conditions. Detector and signal data were used by Day et al. (2010a) to find the split failures and progression quality. Detector data, along with midblock Bluetooth vehicle re-identification data, were used by Day et al. (2010b) to determine vehicle arrival patterns. Hallenbeck et al. (2008) combined stop bar data with signal state data to determine traffic conditions. Wu et al. (2010) identified oversaturation and queue length using high-resolution traffic signal data and stop bar detector data.

One of the objectives of this research is to develop a methodology to determine the probability of breakdown on arterial streets in the immediate short-term future (10-15 minutes) based on the values of performance measures estimated utilizing traffic data collected from traffic sensors. Data mining techniques are used in this study to estimate the probability of breakdown. Data mining and statistical analysis have been used in various transportation engineering applications. For example, decision tree and regression analysis have been used extensively in traffic safety (e.g., Zeitouni and Chelghoum, 2001; Solomon et al., 2006; Chang and Chen, 2005; Chang and Wang, 2006; Chong et al., 2005; Chong et al., 2004) and travel choice modeling (e.g., Xie et al., 2003; Arentze and Timmermans, 2007). Bayes networks have been successfully used in traffic flow forecasting (e.g., Sun et al., 2006; Zheng et al., 2006], transportation modeling (Janssens et al., 2006), and safety (e.g., Zheng et al., 2008; Zhang and Taylor, 2006).

3.3.1 Methodology

This research proposes a methodology to determine the arterial breakdown probability on urban arterial streets utilizing performance measures estimated based on sensor data. This section describes the methodological steps and associated data used in this research.

Utilized Data

The settings of traffic sensor systems on urban streets vary significantly. Intersections with actuated and semi-actuated control have stop-line detectors. However, in many cases, the control systems are not set to upload traffic flow and occupancy data from these detectors to the central software in high resolution, so as to allow their archive and use in performance measurement and management. In addition, many locations with semi-actuated control do not have detectors on the through lanes of the main street. At some locations, system sensors were installed at midblock or upstream locations of a subset of system links, providing valuable information for performance measurements. More recently, agencies have started utilizing automatic vehicle identification (AVI) technologies, also referred to as vehicle re-identification technologies, to estimate travel times. Recent implementation of AVI technologies on arterial streets have utilized Bluetooth readers, Wi-Fi readers, and vehicle signature identification based on magnetometers, although electronic toll tag readers and license plate readers have also been used. When developing procedures to estimate the performance of the system, as is done in this study, it is important to consider the available detection technologies and the locations and configuration of the detection devices.

Breakdown Definition

The first step in predicting traffic breakdown is to have a proper definition of the term "breakdown." As discussed in the introduction section of this section, the term breakdown of flow on a freeway has been used to describe the transition from speeds in the vicinity of the posted speed limit to congested conditions. Breakdown on freeways is considered the condition when the speed drops below a certain threshold (e.g., 40 mph) and/or by a certain amount (e.g., 10 mph). These definitions are specifically used for freeways; no definition could be found in existing literature regarding traffic breakdown on arterials. In this study, breakdown occurrence on arterial roads is defined based on the Highway Capacity Manual's (TRB, 2010) threshold for level of service F on urban streets. According to the HCM, for urban street facilities, the level of service is F when the speed is less than 30% of the base free-flow speed. This means that the free-flow speed must be estimated and used as input to the prediction of traffic breakdown. Free-flow speed can be measured based on the measurements of travel time during low-flow conditions. The base free-flow speed can also be calculated using the following equation from the HCM 2010:

$$S_{f0} = S_o + f_{cs} + f_A \tag{3-7}$$

where,

S_{f0} = base free-flow speed (mi/h),

S_0 = speed constant (mi/h) according to Exhibit 17.11 of the HCM,

f_{cs} = adjustment for the cross section (mi/h) according to Exhibit 17.11 of the HCM, and

f_A = adjustment for the access points (mi/h) according to Exhibit 17.11 of the HCM.

Other definitions of breakdown may also be adopted by transportation agencies, depending on their policy considerations. For example, some agencies may implement the use of an alarm to signal a drop in the level of service below LOS D or E. Thus, in this case, the prediction will have to be made for these conditions.

Development of Breakdown Probability Model

The purpose of the model developed in this study is to determine the breakdown probability at a short time prior to the potential breakdown occurrence. The prediction period is the period between the time the prediction is made and the time for which the probability of breakdown is desired. A shorter prediction period does not give the user enough time to implement countermeasures to avoid or delay breakdown, and a longer prediction period results in a larger amount of errors and uncertainty in the prediction. This study utilizes ten minutes as the prediction period.

Past studies on freeway traffic breakdown (e.g., Chow et al., 2009; Evans et al., 2001; Kondyli, 2009; Lorenz and Elefteriadou, 2001) developed prediction models, where the probability of breakdown is a function of one or two traffic parameters. In the case of arterial streets, developing a prediction model is complicated by the existence of many parameters associated with traffic movements and signal control. In this study, the decision tree approach, combined with the binary logistic regression, is used to predict the breakdown probability. The developed model utilizes data from point detection and automatic vehicle identification matching technologies that are aggregated in space and time.

Development of the Decision Tree Model

The first step in growing a decision tree is to define a set (X) of possible instances. Each instance (x) within X is called a "feature vector." In traffic breakdown prediction, these feature vectors are traffic parameters collected from the roadway network.

$$X = \{x|x=\text{traffic parameters}\}$$

The second step is to define a target function, which utilizes the possible instances to predict breakdown (Y).

$$f: X \rightarrow Y$$

Depending on the given instances, there are sets of hypotheses H.

$$H = \{h|h: X \rightarrow Y\}$$

Each hypothesis h is a decision tree. The input to this model is a set of training examples $\{x^{(i)},y^{(i)}\}$ of the unknown target function f.

There are several algorithms that are used to find the hypothesis $h \in H$ that best approximates the target function f. Each algorithm has some type of limitation and works better with different data sets. In this study, a combination of two algorithms was selected for use: the top-down induction of decision trees (TDIDT) algorithm (Quinlan, 1986), and the Recursive Partitioning and Regression Trees (RPART) (Therneau and Atkinson, 1997), a version of the Classification and Regression tree (CART) approach. It was found that using the TDIDT to construct the top levels of the tree and PRART to construct the lower levels produced better performance, compared to solely using each of the two approaches to construct the entire tree. TDIDT is a greedy algorithm based on the identification of the best decision attribute (A) at each level to most effectively divide the decision tree to child nodes with the lowest impurity. The branching of the tree continues until the training examples are perfectly classified or no further improvements are possible.

The RPART algorithm, as implemented in R software (R Development Core Team, 2006), was used to derive the lower levels of the tree. This algorithm uses a virtually identical process to the TDIDT approach to find the appropriate decision tree. The main difference between the two algorithms is the technique used to find the best decision attributes (A). The combination of the two methods is used in this research in order to develop the best possible decision tree. At the top level of the decision tree, the number of data points with the breakdown condition is very small, compared to data points with the no-breakdown condition. It was found that using the RPART approach by itself at the top level may not produce the best decision tree. The use of the TDIDT algorithm at the top levels increases the proportion of the breakdown traffic condition points at decision nodes, permitting a more effective use of the RPART algorithm to further develop the decision tree.

Logistic Regression Model Development

After developing the decision tree, regression models are fitted to the data at the end nodes to allow for better classification of the breakdown based on the node attribute values. The equations were developed for the end nodes where sufficient data is available. Binary logistic regression is used to derive the equations. It should be mentioned that the regression tree approach was used in previous studies to derive a combination of decision tree with regression equations at the end nodes, which is similar to the approach used in this study. However, the regression tree approach was not used in this study since traffic breakdown is a binary output (yes/no) and the traditional regression tree analysis is only appropriate for continuous dependent variables. The derived binary logistic regression equations are displayed in the following format:

$$\pi_i = \Pr(Y_i = 1|X_i = x_i) = \frac{\exp(\beta_0 + \beta_1 x_i)}{1+\exp(\beta_0 + \beta_1 x_i)} \tag{3-8}$$

where,

π_i = probability of breakdown in observation i,

Y_i = 1 if breakdown occurs in observation i ,

Y_i = 0 if breakdown does not occur in observation i ,

x_i = observed value of the independent variables for observation I,

β_0 = intercept coefficient, and

β_1 = variable coefficient.

3.3.2 Method Application

The proposed methodology was applied to an arterial road utilized as a case study to demonstrate the breakdown prediction and test its performance. The case study is Glades Road, located in the city of Boca Raton, Florida. A decision tree model was originally developed in this study for a 0.64-mile link between the Renaissance Way and Airport Road intersection on this arterial, as shown in Figure 3-2 (see Link 1 in Figure 3-2). Subsequently, this model was applied to two other links (Link 2 and Link 3 in Figure 3-2) to test the transferability of this methodology, as described later in this section.

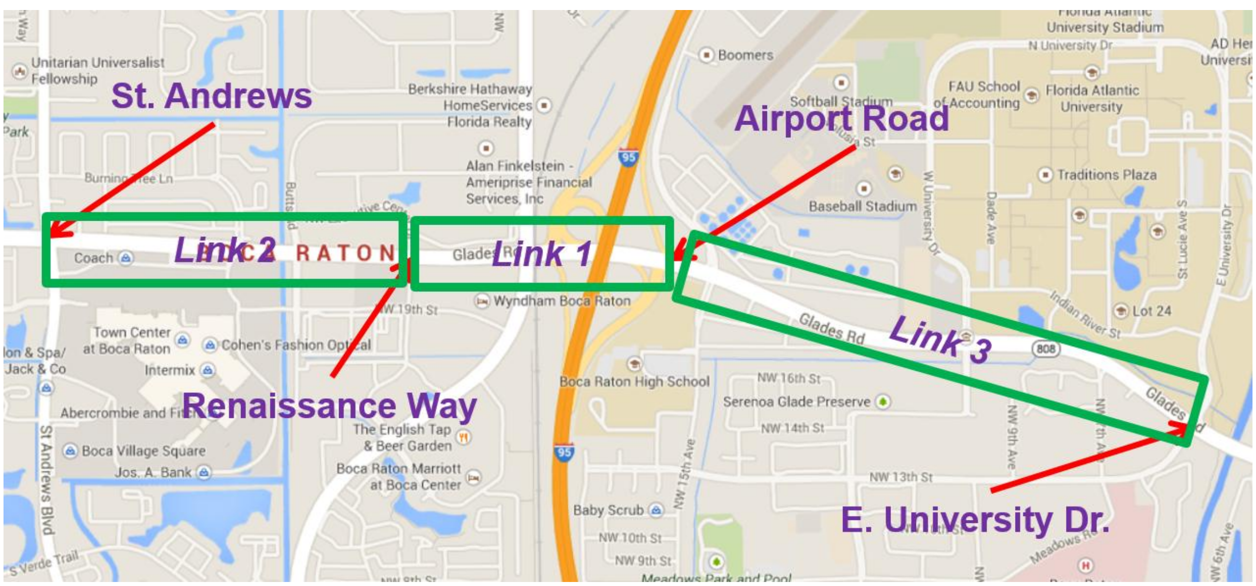


Figure 3-2 Location of the Arterial Segments Used in the Case Study

From January 2014 to July 2014, data were collected for the test segments. Stop-line detector data are not available from the central advanced traffic management system (ATMS) software. However, data from a sensor system is available and used for the purpose of this study. The sensor system utilizes the Sensys technology, which is based on magnetometers, and is installed upstream of each link to provide volume, speed, and occupancy measurements at detection points. In addition, the technology allows for travel time estimation utilizing vehicle re-identifications (based on vehicle signature), between upstream and downstream detectors. The volume, occupancy and speed measurements are archived at 30-second intervals, while the segment's travel times are archived at 5-minute intervals. In this research, all data are aggregated at the 5-minute interval level to allow for consistency in the analysis. The volume, speed, occupancy, and travel time data were downloaded, cleaned, aggregately fused and archived in a common database for analysis. The data were aggregated in space and time for use in different performance measurements and prediction applications.

Estimation of Potential Parameters of the Prediction Model

The collected sensor and signal timing data were used to estimate a number of performance measures that can be used as potential attributes in the developed prediction model. The calculated measures include:

- segment speeds based on vehicle re-identification technology; and
- five-minute averages and standard deviations of occupancy, volume, and spot speed measurements at the upstream and downstream point locations of traffic detectors.

A total of 15 attributes were estimated at the upstream and downstream detection locations, including the links that connect the locations. Further discussion of the estimation of attributes is presented below.

The sensor system provides travel times for each link based on matching vehicle signatures, aggregated at 5-minute intervals. The median, 90th percentile, and standard deviation of travel time measurements within five minutes were used as potential variables in the prediction. Link average speed is derived from the median travel time and link length.

Several variables are also estimated based on the point detection. The sensor system provides lane-by-lane occupancy data at the upstream and downstream detection stations. The occupancy at each station is calculated by taking the average of the lane-by-lane occupancy data across all lanes. The 30-second occupancy data is aggregated to represent the 5-minute occupancy by taking the average over ten 30-second intervals. Lane-by-lane volume data is converted to detection station volume data by summation. Five-minute volume estimates are calculated by adding the 30-second volume data during the 5-minute period. The median speed for the detection station is calculated based on the weighted average of individual lane speeds using the following equation:

$$S_{30} = \frac{\sum_{l=1}^{3} S_l V_l}{\sum_{l=1}^{3} V_l} \tag{3-8}$$

where,

S_{30} = median speed at the station for an individual 30 second interval,

l = lane identification,

S_l = measured speed for lane l, and

V_l = measured volume for lane l.

Then, the 30-second speed estimates are converted to 5-minute speed estimates as the weighted average by volume in each 30-second interval within the 5-minute period. The standard deviations of occupancy, volume and median speed within the 5-minute period are calculated using the following equation:

$$\sigma = \sqrt{\frac{1}{N}\sum_{i=1}^{N}(x_i - \mu)^2} \tag{3-9}$$

Where,

N = the sample size, which is the number of 30-second intervals within the 5-minute period,

x_i = individual 30-second estimates of the parameter under consideration (volume, speed, occupancy), and

μ = mean of the individual 30-second estimates within the 5-minute period.

Identification of Breakdown Conditions

In order to develop a breakdown prediction model based on archived real-world data, it is necessary to categorize the analyzed historical traffic conditions to breakdown conditions and non-breakdown conditions. This allows for the training and testing of the model based on the archived data. As stated earlier, this categorization was done in this study based on the level of service F threshold in the HCM Urban Street Facility procedure. The posted speed limit on the test section is 45 mph. Utilizing Equation 3-7, the free-flow speed is estimated at 44 mph. The HCM procedure defines the level of service to be F, when the travel speed is less than or equal to 30% of the free-flow speed. Thus, the breakdown conditions are defined as occurring when the link speed is 13.2 mph (44 mph multiplied by 0.3).

Data Cleaning and Filtering

Before utilizing the data in this study, the data is checked and filtered, including checking for completeness of the data, removing outliers, checking consistency, and data smoothing. If there is a missing attribute of a data point or the values of an attribute are illogical, then that data point is removed from the dataset. For example, data points with travel time equal to zero are taken out of the dataset. Similarly, data points with a high travel time resulting in illogical travel speeds (>100mph) are also removed. The link travel speeds are smoothed by taking a weighted average of the speeds in the last few time periods and the current time period.

After cleaning all of the data points, the remaining data points are 48,844 in the 6-month period. Within these data, 48,434 data points have no-breakdown, and 410 data points have a breakdown condition. Hence, without any model, the breakdown probability is 0.8% for the whole period.

Development of Breakdown Probability Models

As described in the previous section, the first step in growing a decision tree is to define a set (X) of possible instances. In this study, the set (X) is defined as follows:

$$X = \{T, LOS, T_L, S_L, SC_L, S_{up}, V_{up}, O_{up}, SD\_S_{up}, SD\_V_{up}, SD\_O_{up}, S_{down}, V_{down}, O_{down}, SD\_S_{down}, SD\_V_{down}, SD\_O_{down}\} \quad (3\text{-}10)$$

where,

T	= travel time,
LOS	= level of service,
T_L	= link travel time,
S_L	= link speed,
SC_L	= link speed change,
S_{up}	= upstream speed,
V_{up}	= upstream Volume,
O_{up}	= upstream occupancy,
SD_S_{up}	= standard deviation of upstream speed,
SD_V_{up}	= standard deviation of upstream volume,
SD_O_{up}	= standard deviation of upstream occupancy,
S_{down}	= downstream speed,
V_{down}	= downstream Volume,
O_{down}	= downstream occupancy,
SD_S_{down}	= standard deviation of downstream speed,
SD_V_{down}	= standard deviation of downstream volume, and

SD_O_{down} = standard deviation of downstream occupancy.

The target function uses these instances to predict breakdown occurrence. The decision tree that best approximates the target function is shown in Figure 3-3.

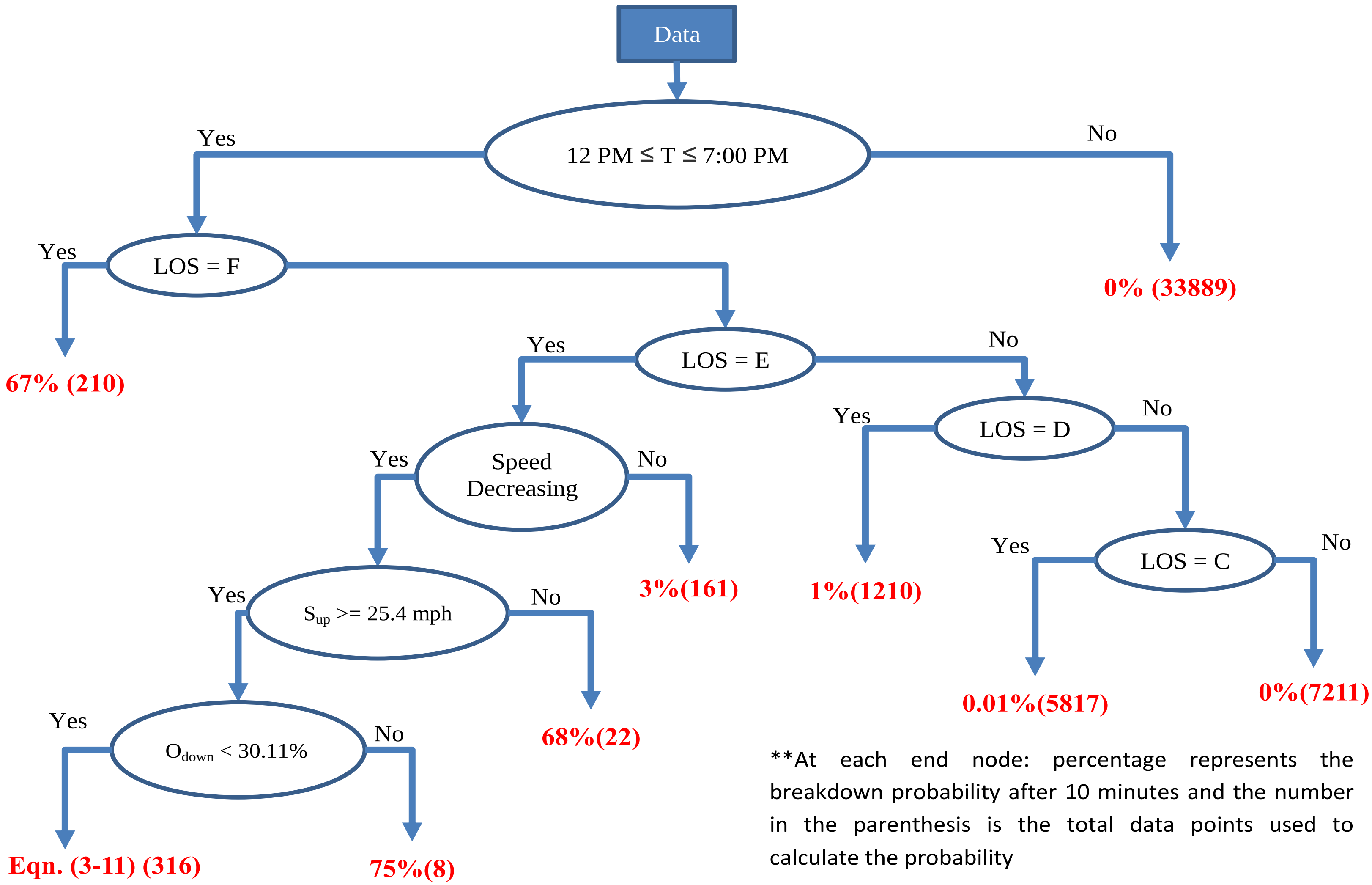
The number of data points with breakdown conditions is very small compared to data points with non-breakdown conditions. Hence, initially the TDIDT algorithm was used to reduce the data size by selecting attributes that most effectively divided to child nodes with the lowest possible impurity.

The breakdown event mostly occurs during the peak period when the traffic demand is high, so the attribute for the first decision node is the time of day. Dividing the data based on this attribute reduces the data dimension significantly and improves the classification results. The data showed that the entire breakdown of the test dataset occurs between 12:00 PM and 7:10 PM. Hence, other than this time period, the probability of breakdown is zero under recurrent conditions. The next node of the decision tree uses the level of service (LOS). The probability for breakdown occurrence in the next 10 minutes was found to be zero at LOS C or better, and around 1% at LOS D. Most breakdowns occur when the current level of service is E and when the level of service is F, which means that LOS F will continue in the next 10 minutes. When the LOS is E, the next node in the decision tree will determine whether or not the speed on the link has decreased in the previous 5 minutes. If the speed does not decrease, then the probability of breakdown is only 3%. If it decreases, then the next part of the decision tree will assess the attributes estimated based on point detections. Using the RPART algorithm (Therneau and Atkinson, 1997), it was found that the probability of breakdown is higher with lower upstream speed and higher downstream occupancy. If the downstream occupancy is high, which indicates that there is congestion at the downstream intersection that may spillback to the subject link, then the possibility of breakdown in the following 10 minutes is high. Also, if the upstream speed is lower, indicating that the test link is becoming congested, then the possibility of breakdown in the next 10 minutes is also high.

As stated in the Methodology section, the logistic regression equation was fitted to traffic data at the end nodes to further predict breakdown based on the end node attributes. The regression equations were derived only for those end nodes where enough data was available for developing the equations. There are a large number of data points (283 non-breakdown cases and 33 breakdown cases) at the end node, where the downstream occupancy is less than 30.11%). The other nodes do not have sufficient data, and thus regression equations with acceptable significant levels could not be developed. A logistic regression equation was only developed for the end node with a downstream occupancy less than 30.11%. The equation is as follows:

$$\pi_i = \Pr(Y_i = 1 | X_i = x_i) = \frac{\exp(-3.2936 + 0.0787\,x_i)}{1 + \exp(-3.2936 + 0.0787\,x_i)} \qquad (3\text{-}11)$$

This equation is significant at the 95% confidence level (p = 0.0183).



* Where, T = Time of Day, S_{up} = Upstream Speed, O_{down} = Downstream Occupancy

Figure 3-3 Derived Decision Tree for Link 1

3.3.3 Result Validation

Several techniques were used to validate the developed model shown in Figure 3-3. This section describes the validation procedure and results.

<u>Contribution Factors Validation</u>

As stated in the previous section, twelve attributes were used as potential explanatory variables in the decision tree based on point detectors at the upstream and downstream intersections. Among these variables, the decision tree used only two (downstream occupancy and upstream speed), in addition to time of day and link level of service as indicators of breakdown, as shown in Figure 3-3. To confirm that the selection of these two variables is justified a Random Forest analysis (Leo, 2001), another algorithm to develop a decision tree was applied to identify the most significant contributing factors that determine the probability of breakdown and was used to cross-check our model results. Random forest is an ensemble learning method for the classification or regression tree. The variable importance measure, as identified by this method, is shown in Table 3-1.

Table 3-1 Variable Importance as Identified by the Random Forest Method

Attribute	Variable Importance
Occupancy at downstream	5.936703
Speed at upstream	5.461463
Speed at downstream	4.633891
Occupancy at upstream	4.308245
Standard deviation of occupancy at downstream	3.236971
Standard deviation of speed at upstream	3.222469
Standard deviation of speed at downstream	2.751114
Standard deviation of volume at downstream	2.597553
Volume at upstream	2.477574
Standard deviation of occupancy at upstream	2.280323
Volume at downstream	2.220703
Standard deviation of volume at upstream	2.170719

Table 3-1 shows that the downstream occupancy and upstream speed are the most important compared to the other attributes, which matches the decision tree model selection. The decision tree only included these two attributes to avoid overfitting of the model. Selecting two factors gave the optimum error and higher prediction capability.

Validation Utilizing Additional Data

Validation of the model was further accomplished by performing the prediction with 1-month data, which was not included in the model's development. Two measures were calculated to determine the performance of the model with this data:

- Root Mean Square Error (RMSE)
- Mean Percentage Error (MPE)

The RMSE is defined as the average of the square of all of the differences between the model estimates of breakdown probability and measured breakdown probability. The estimated RMSE value is 13.6%. That means there is 13.6% variability between the estimated and measured breakdown probability.

Mean percentage error (MPE) is defined as the difference between model estimates and measured probability divided by model estimation. The MPE value is estimated as 11%, which means there is an 11% difference between model and measured probability, with respect to model estimation.

Cross-validation

Another method commonly used to validate the classification tree is cross-validation. In this study, k-fold cross-validation is used to validate model performance by checking the re-substitution error rate. In the k-fold cross-validation method, the test data is randomly sub-divided into k equal size subsamples. From these k subsamples, one subsample is used to validate the model, and the remaining k-1 subsamples are used to train the model. This process is repeated k number of times. The re-substitution error rate is calculated as the average of the errors from all iterations. Two most commonly used k-fold cross-validation methods are:

- the 2-fold cross-validation, and
- the 10-fold cross-validation.

The error rate for the 2-fold cross-validation was 12%, and the error rate for the 10-fold cross-validation was 18%. The reason for the difference in the results between the two methods is that in the 10-fold cross-validation, the validation data size is small, resulting in a large variation.

3.3.4 Model Transferability

The proposed methodology is applied to two other locations (Link 2 and Link 3 in Figure 3-2) to develop models to predict traffic breakdown on the links and test model transferability. Link 2 is a 0.76-mile long link between Renaissance Way and St. Andrews Boulevard, and Link 3 is a 1.09-mile long link between East University Drive and Airport Road.

Link 2 Prediction Model

Similar to Link 1, traffic data for Link 2 in the westbound direction was downloaded during a six-month period. The processed data is used throughout the same methodology to build a prediction model for this link. The prediction model for Link 2 is presented in Figure 3-4.

The results show that the breakdown occurs mainly between 1:00 p.m. and 6:30 p.m. For level of service D or better, the probability of breakdown in the next 10 minutes is zero. If the level of service is F, then there is a 52% probability to remain in this condition (LOS F) in the next 10 minutes. In the case of LOS E, if the speed increased in the past five minutes, then the probability of breakdown is low (5%), otherwise the model further classifies the data based on intersection attributes.

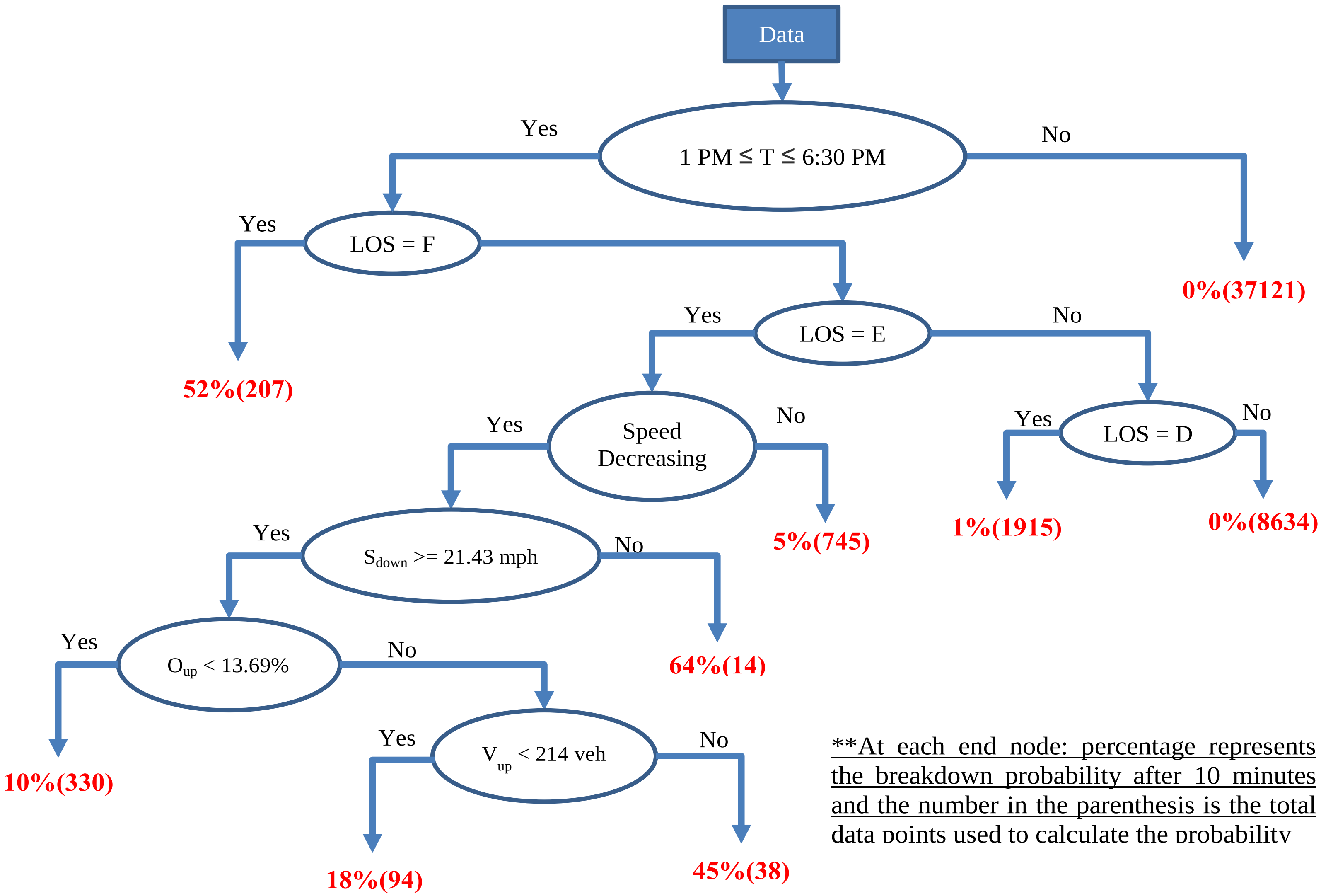
Up to this point, the contributing attributes to the prediction match those of the attributes derived in the model for Link 1. At this point, the RPART algorithm is used to find the remaining portion of the tree. Beyond this point, different classifying attributes are selected for different links. Different links have different characteristics and causes of congestion. So, the attributing factors are expected to be different for varying links. The process of finding these attributes, however, is the same. For Link 2, the probability of breakdown is higher when the downstream speed is low or when the downstream speed is moderate, but with high upstream occupancy and volume.

Link 3 Prediction Model

Link 3 is located between East University Drive and Airport Road. Data was downloaded for six months and processed, as with the other links. However, there was a high proportion of missing data in this location. Only 30 days of data could be used for the prediction model for this link, which resulted in a lower number of data points, compared to the other two locations. The model is presented in Figure 3-5. The results are quite similar to the other links. This link shows that the probability of breakdown is high when there is high upstream occupancy, along with a high downstream standard deviation (SD) of speed. For the end node with upstream speeds greater than 36.94 mph, the developed logistic regression equation is as follows:

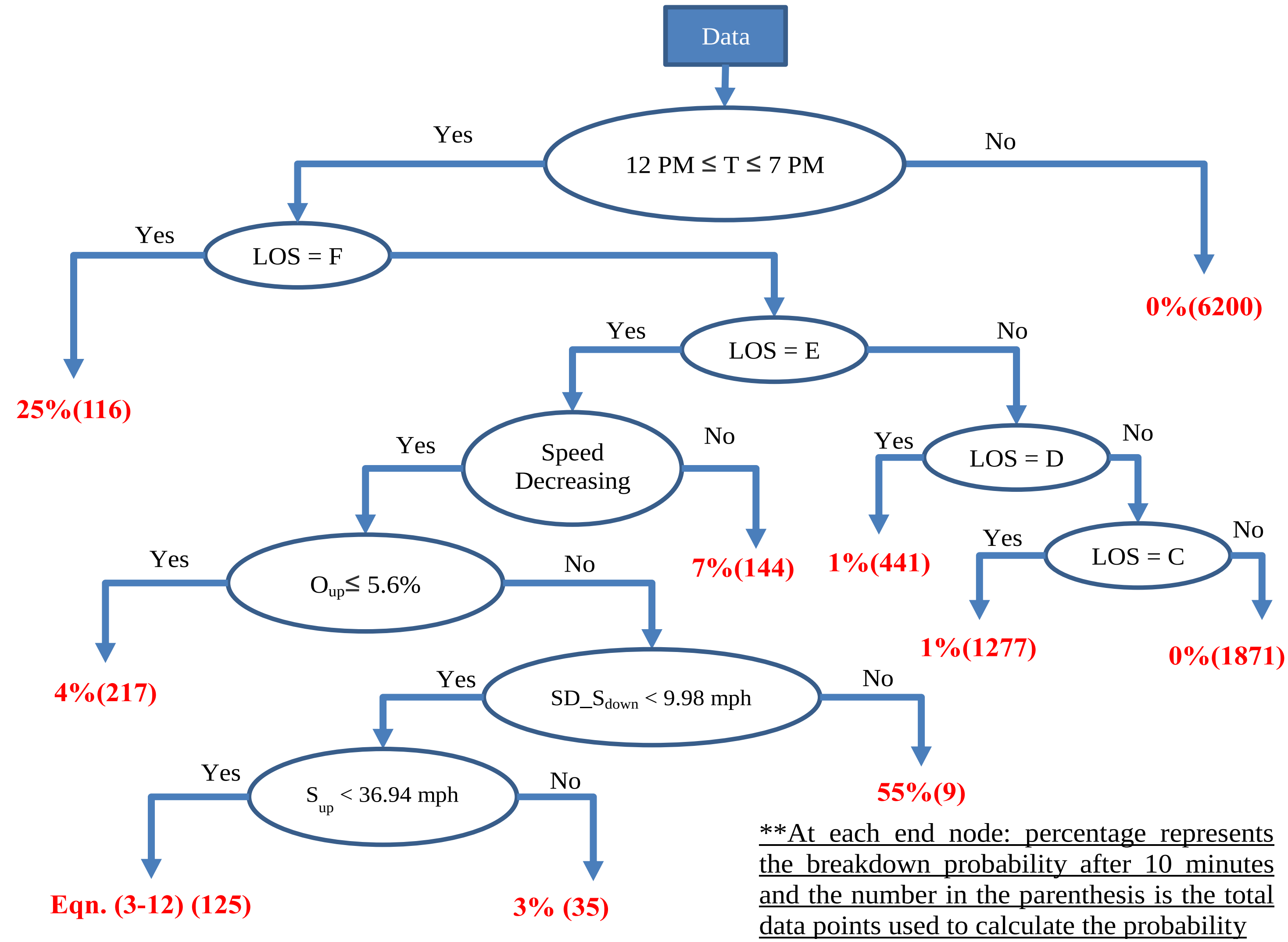
$$\pi_i = \Pr(Y_i = 1 | X_i = x_i) = \frac{\exp(-10.6340 + 0.2658 x_i)}{1 + \exp(-10.6340 + 0.2658 x_i)} \tag{3-12}$$

This equation has p value 0.0381(<.05). Hence, it is significant at the 95% confidence level.



*Where, T = Time of Day, S_{down} = Downstream Speed, O_{up} = Upstream Occupancy, V_{up} = Upstream Volume

Figure 3-4 Developed Decision Tree for Link 2



*Where, T = Time of Day, S_{up} = Upstream Speed, O_{up} = Upstream Occupancy, SD_S_{down} = Standard Deviation of Upstream Occupancy

Figure 3-5 Developed Decision Tree for Link 3

3.3.5 Summary

This study developed decision tree models to identify the breakdown probability of an arterial street segment. The models are able to identify conditions in which there are higher probabilities of traffic breakdown in the next 10 minutes (as high as 75% probability, which means that in three out of four cases, there will be a breakdown occurrence). Validation results show that the model performance is good. Further analysis shows that the best set of parameters used in the prediction model can be different for different links, due to the varying congestion causes and characteristics of different links. This is particularly true for the decrementing parameters in the lower parts of the decision trees. In the upper parts of the decision tree, the decrementing parameters are the time of day and the level of services, and in the lower level, the parameters are selected from upstream speed, downstream speed, upstream occupancy, downstream occupancy and upstream volume. Arterial breakdown prediction can be used to support signal control and other arterial active management strategies, thus allowing for the maintenance of acceptable levels of service. In summary, this research can be successfully applied to arterial traffic management and operation. Further research is recommended for additional arterial scenarios and data sets. Other data mining techniques can also be applied to predict breakdown occurrence and compare the results with the model developed in this study. Although data from the Sensys technology was used in this case study, any technology or combinations of technologies that provide point detection of measures and travel time estimates based on vehicle re-identification could have been used. For example, a number of cities in Florida have installed true presence microwave detectors for point detection and Bluetooth and/or Wi-Fi readers to allow travel estimation based on vehicle-re-identification on their urban streets. Such implementations are expected to increase in the future.

3.4 Implementation in ITSDCAP

The probability of breakdown models developed in the previous sections is then considered for implementation in the ITSDCAP tool. Currently, a real-time C2C connection to FDOT D6 Traffic Management Center (TMC) is available in ITSDCAP. Traffic data including speed, volume, and occupancy at each detector station in FDOT D6 can be retrieved from this connection, which can be in turn used as inputs to the developed probability of breakdown model to estimate the probability of breakdown in real time. Such breakdown information can alert ITSDCAP users of breakdown potentials and take corresponding actions. Figure 3-6 presents a snapshot of the probability of breakdown reported by ITSDCAP. As shown in this figure, when a user clicks the “Show” button under the detector module, the links in the map are displayed in different colors to reflect different detector speeds. A further click of a link produces a pop-up window, which displays the information of link ID, travel time, speed, volume, occupancy, and updated date reported from point detectors, as well as the probability of breakdown predicted using the procedure described above. A warning is displayed on the ITSDCAP user interface when the breakdown probability exceeds a certain limit specified by the user. The user can also request an e-mail alarm to be sent when the threshold is exceeded.

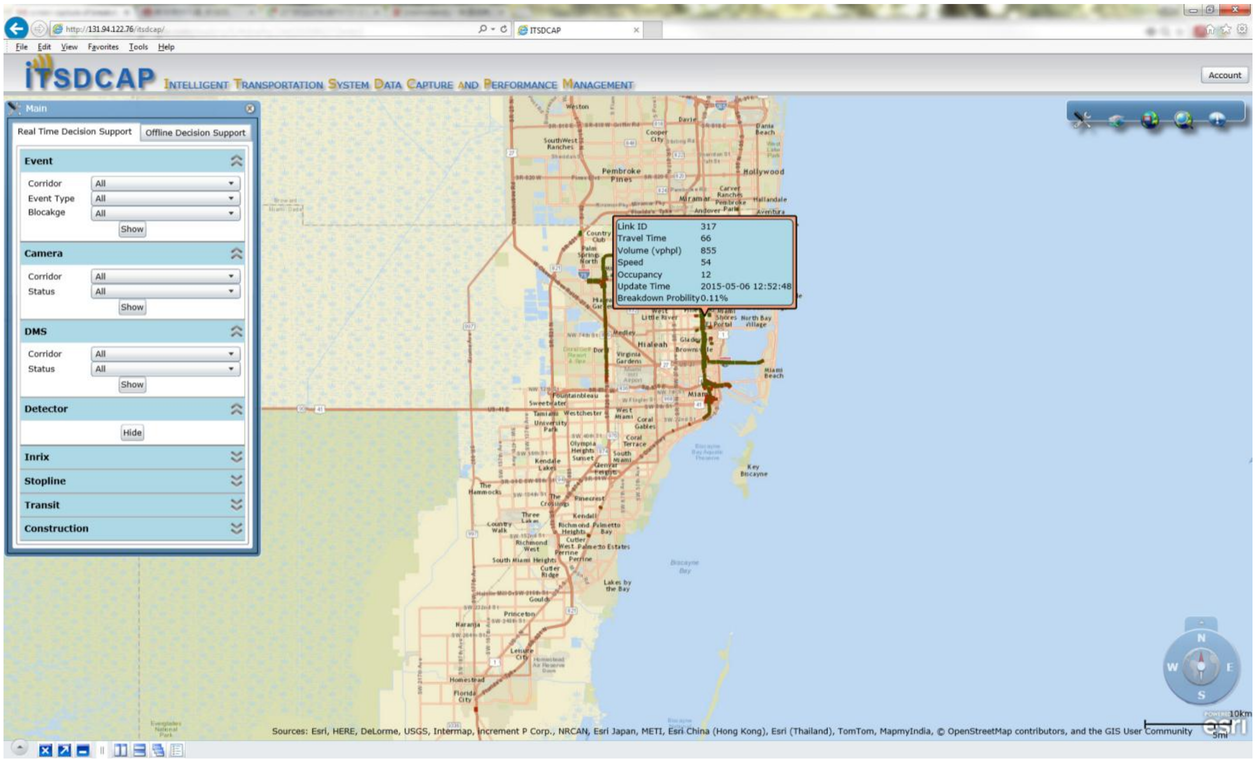


Figure 3-6 Snapshot of Probability of Breakdown Interface in ITSDCAP

Since real-time arterial traffic data are not currently available for ITSDCAP, the developed probability of breakdown prediction model was not implemented in the current version of ITSDCAP. However, this model can be easily implemented if such data becomes available in the future.

4 BENEFIT-COST MODULE EXTENSION AND IMPLEMENTATION IN ITSDCAP

4.1 Introduction

Evaluating the benefits of Intelligent Transportation Systems (ITS) implementation is necessary for both planning and operational purposes. With the availability of rich ITS data and wide implementations of ITS, it becomes feasible to evaluate the impacts of ITS based on real-world data. Furthermore, inputs required to existing ITS benefit-cost evaluation methods and tools can also be derived based on ITS data. The Intelligent Transportation System Data Capture and Performance Management (ITSDCAP) tool developed in this project includes a module to support agencies in their assessment of ITS benefits. This module in the Web-based version of ITSDCAP developed as part of this project focuses on two applications: incident management on freeways and incident management on arterials. It is possible, however, to extend this module to include the assessment of other ITS applications in future efforts. In fact, as part of an ongoing separate project, a module will be introduced for the assessment of adaptive signal control.

Previous studies mainly focused on the evaluation of active traffic management strategies for freeways. The utilized methods and tools include analytical models, sketch planning tool, and simulation-based analysis. Limited studies have been reported in the literature on the evaluation of active traffic management strategies for arterials. The purpose of this document is to present a review of existing related benefit evaluation methods and tools of ITS, with a focus on evaluating incident management on freeways and arterials, and to describe the benefit evaluation module implemented in ITSDCAP. This study developed a new method to assist agencies in estimating urban street incident impacts and thus assessing the benefits of the associated incident management.

4.2 Literature Review

Incident management is an important component of Transportation System Management and Operations (TSM&O), providing significant benefits in terms of travel time, travel time reliability, emission, fuel consumption, safety, and other performance measures of transportation systems. Estimating the impacts of incidents and incident management strategies allows traffic management agencies to determine the need for various incident management strategies and technologies, and to justify the decisions to invest in their programs. This justification is critical when requesting additional funds for future activities of the programs.

The impacts of incident management on mobility measures have been widely investigated for freeway facilities. Four types of methods have been used for this purpose: empirical analysis based on field data and data analytics when travel time before and after incident management are available, and queuing analysis, shock wave analysis, and simulation modeling.

Queuing and shockwave analysis methods have been applied successfully to the estimation of incident and bottleneck delays on freeway facilities (Hong et al., 2013; TRB, 2000). However, these methods are not easy to implement when estimating delays for incidents on signalized urban streets. This is due to interactions between the operations of traffic signals and the capacity drops due to queue spillbacks resulting from incidents with different attributes and locations relative to the locations of the adjacent signals. Earlier work by the research team of the current study (Yang et al., 2008) concluded, based on testing using simulation, that the simple queuing analysis equations underestimate incident delays on signalized urban networks due to the impacts on upstream intersections. Based on the results presented in that study, Xiao et al. (2010) used a factor of 1.25 to multiply the incident delays calculated based on queuing equations, when calculating urban street incident delays as part of the Florida ITS Evaluation tool (FITSEVAL) developed by the authors (McCandless, 2007).

With the increased focus on implementing TSM&O strategies on signalized urban streets in recent years, there has been an increasing interest in models that estimate incident and incident management delay impacts on these streets. For example, the TSM&O programs in Broward County and Palm Beach County in Florida are looking for simple methods to quantify the impacts of incident management processes that reduce incident duration and/or modify signal timing considering urban street incidents.

This section provides a brief review of the existing ITS evaluation tools, including the Florida ITS Evaluation Tool (FITSEVAL), TOPS-BC, and evaluation methodology used for the Palm Beach County Active Arterial Management strategy.

4.2.1 Overview of Utilized Methods to Assess Incident Management Benefits

In the absence of field measurements of the incident delays, queuing theory, shock wave, and simulation analyses have been used to assess incident delays. Microscopic traffic simulation is a powerful method to estimate the impacts of incident and incident management (McCandless, 2007; Gomes et al., 2004). However, the use of simulation models is expensive in terms of data collection, model input preparation, and calibration, particularly when the incident management systems need to be assessed at the regional levels and when the stochastic nature of incident attributes and locations need to be considered in the analysis. Queuing analysis has been more widely used to identify incident benefits than shock wave analysis (PTV Planning Transport, 2014; Hadi et al., 2007). A variety of examples of the use of queuing analysis (Yang et al., 2008; Xiao et al. 2010; McCandless, 2007; ; Gomes et al., 2004; PTV Planning Transport, 2014; Hadi et al., 2007; Knoop et al., 2008; Zhou and Feng, 2012) and shock wave theory analysis (Rakha and Zhang, 2005; May, 1990) for freeway incident impact assessments are available. Rakha and Zhang (2005) demonstrated the consistency in delay estimates that are derived from deterministic queuing theory and shock wave analyses. Thus, it was concluded that queuing theory provides a simple and accurate technique for estimating delays at highway bottlenecks.

In 2006, the authors of this study developed a benefit cost analysis procedure that utilizes incident and traffic data and automatically calculates the benefits and costs of the Florida Department of Transportation (FDOT) District 4 SMART SunGuide incident management for freeway facilities, using deterministic queuing analysis (Hadi et al., 2008a). This process is still in use successfully by the FDOT. The authors also used queuing analyses when assessing incident management, in their development of the incident management assessment module of the FITSEVAL sketch planning tool (Hadi et al., 2008b). This implementation is described further in the next section.

The main parameters required to estimate the impacts of a single incident using queuing analysis are the base capacity (with no incidents), incident impacts on capacity, and incident duration. For freeway segments, the 2010 Version of the Highway Capacity Manual (TRB, 2010) provides estimates of the drops in capacity due to incidents, as a function of the number of the blocked lanes and the total number of lanes of the freeway section. For example, the HCM 2010 suggests that for a three-lane freeway segment, these values are 17% for shoulder blockage incidents, 51% of a reduction in capacity for one-lane blockage incidents, and an 83% capacity drop for two-lane blockage incidents. The HCM 2010 does not address the capacity impacts of incidents on urban streets.

Knoop et al. (2008) found that if one of three lanes on a freeway is blocked, the maximum throughput due to incident is roughly 50% lower than the maximum throughput during normal conditions. Their results also indicate that the queue discharge rate of an unblocked lane is 30% lower than the normal queue discharge rate of the lane (Knoop et al., 2009).

As part of the SHRP 2 L08 Project (Kittelson & Associates et al., 2012), equations were developed to estimate the saturation flow rate adjustment factor for incidents present at the stop line of a traffic signal for use as part of the assessment of incident impacts on travel time reliability of urban street facilities. The equation estimates the saturation flow adjustment factor as a function of number of lanes, number of lanes blocked by the incident, and coefficients related to incident severity. The equations, however, do not address incident locations other than at the stop line.

Florida ITS Evaluation Tool (FITSEVAL)

FITSEVAL is a sketch planning-level ITS evaluation tool that was developed for the Florida Department of Transportation (FDOT) by FIU researchers. The tool works within the Florida

Standard Urban Transportation Modeling Structure (FSUTMS)/Cube environment. It can be used to estimate the benefits and costs of various types of ITS deployment, as listed below.

- Ramp Metering
- Incident Management Systems
- Highway Advisory Radio (HAR) and Dynamic Message Signs (DMS)
- Advanced Travel Information Systems (ATIS)
- Managed Lane
- Signal Control
- Emergency Vehicle Signal Preemption
- Smart Work Zone
- Road Weather Information Systems
- Transit Vehicle Signal Preemption
- Transit Security Systems
- Transit Information Systems
- Transit Electronic Payment Systems

The evaluation methodology implemented in FITSEVAL varies with the type of ITS deployments. The output of the FITSEVAL tool includes the impacts of ITS on performance measures including mobility, safety, fuel consumption, emission and other measures. FITSEVAL also outputs the benefits and costs in dollar values of ITS applications and the resulting benefit/cost ratios. These outputs can be used to assess the ITS deployment, prioritize alternatives, and support long-range plans. In a recent assessment by the University of Virginia, twelve different existing tools were evaluated, and FITSEVAL was recommended for use in Virginia (Ma and Demetsky, 2013).

In FITSEVAL, the deployment of incident management is assumed to reduce the incident duration and consequently, the incident delays, which are calculated based on queuing analysis with and without incident management. The time savings due to vehicle diversion during the incidents are considered in this evaluation methodology. The diversion rate is set as a function of the estimated saved delays. This methodology also assumes that 21% of fatalities are shifted to injuries due to faster incident detection, verification, and response of incident management systems. In addition, an additional reduction factor of 2.8 percent is used in the methodology for fatal, injury, and PDO accidents as a result of incident management. The emissions and fuel consumptions with and without incident management are estimated based on the speeds of queued and non-queued vehicles and the vehicle-miles in queue. Based on a previous study by the authors, the tool assumes that for the same incident and traffic conditions, the incident delays on the arterials are 1.25 higher on signalized arterials compared to uninterrupted facilities (Hadi et al., 2008b, Xiao et al., 2010).

Six types of signal control can be evaluated in FITSEVAL, and traffic signal retiming is one of them. Retiming traffic signals can slow down the deterioration of link travel time and therefore in FITSEVAL, its benefits are calculated in terms of a reduction in link travel times. A default value of 7.5% improvement in travel time is used in FITSEVAL, with a retiming of existing coordinated signals.

The assessments of the remaining types of ITS improvements are not reviewed here. The readers are referred to Reference 3 for detailed discussions.

<u>TOPS-BC</u>

The TOPS-BC is an Excel-based tool that is designed to support practitioners in conducting benefit and cost analyses. It has four main capabilities: 1) Investigate the impacts associated with prior deployments and Transportation System Management and Operations (TSM&O) strategies; 2) Include methods and tools at different analysis levels for benefit/cost analysis; 3) Estimate life-cycle costs, replacement costs, and annualized costs based on default cost data. The life-cycle costs include capital costs, as well as the soft costs required for the design, installation, operations and maintenance of the equipment. The replacement costs are the periodic cost of replacing/redeploying system equipment, and the annualized costs represents the average annual expenditure that would be expected in order to deploy, operate, and, maintain the TSM&O strategies; and 4) Estimate benefits for particular TSM&O strategies. Default impact values and parameters are recommended in this tool. The TSM&O strategies that can be evaluated in TOPS-BC are listed below:

- Traveler information
  a. Highway Advisory Radio (HAR)
  b. Dynamic Message Signs (DMS)
  c. Pre-Trip Travel Information
- Ramp Metering Systems
  a. Central Control
  b. Traffic Actuated
  c. Preset Timing
- Traveler information
  a. Highway Advisory Radio (HAR)
  b. Dynamic Message Signs (DMS)
  c. Pre-Trip Travel Information
- Ramp Metering Systems
  a. Central Control
  b. Traffic Actuated
  c. Preset Timing

- Incident Management Systems
- Signal Control
- Emergency Vehicle Signal Preemption
- ATDM Speed Harmonization
- Employer Based Traveler Demand Management
- ATDM Hard Shoulder Running
- ATDM High Occupancy Toll Lanes
- Road Weather Management
- Work Zone
- Supporting Strategies
    a. Traffic Management Center
    b. Loop Detection
    c. CCTV

In TOPS-BC, traffic incident management (TIM) is evaluated in terms of two main benefits: travel time reliability improvement and fatality crash reduction. The calculation of the additional TIM benefits, including the reductions in secondary crashes and fuel use, are optional. The improvement in travel time reliability is calculated as the reduction in incident-related delays, which is a function of the percentage reduction in incident response time. A certain percentage of fatality crashes are considered to be changed to injury crashes with the quick response of TIM in the safety benefit calculation. For the optional analysis of secondary crashes and fuel use, TOPS-BC requires the users to input the corresponding reduction factors.

FDOT District 4 SMART SunGuide Benefit-Cost Analysis

The FDOT District 4 has assessed the benefits of the SMART SunGuide incident management operations utilizing a method developed for the districts by FIU researchers in 2006. The method forms the basis for the incident management evaluation implemented in FITSEVAL, as described above. In this method, the incident delay reduction due to incident management is estimated using a deterministic queuing analysis based on the reduction in incident duration. The improvement in safety focuses on the reduction in the fatalities due to faster response and reduction in secondary crashes. The benefits in fuel consumption and emissions are determined based on the reduction in the vehicle-miles in the queue, which are also estimated using queuing theory. The route diversion resulting from DMS and traveler information systems are estimated based on the proportion of diverted motorists and the estimated differences between the route impacted by the incident and the alternative route.

Palm Beach County Active Traffic Management Evaluation Methodology

In order to minimize the impacts of incidents and improve traffic operations along the arterials, the FDOT District 4 worked with Palm Beach County and Broward County in developing active arterial management strategies, including installing Bluetooth readers, Closed Circuit Television (CCTV) cameras, and point detectors along major arterial corridors, as well as actively monitoring and managing incidents, and adjusting signal timing in real-time during incidents that occur on both arterials and adjacent freeways. Quantifying the benefits of these implemented active arterial traffic management strategies is necessary for providing decision supports for future agency investments and actions.

The evaluation methodology developed for Palm Beach County is in part based on the benefit-cost methodology developed for the FDOT District 4 SMART SunGuide benefit-cost methodology by FIU, described earlier. It considers the following four measures: reduction in travel time, fuel savings, reduction in emissions, and safety benefits. In this methodology, the incidents are classified into three classes: major, intermediate, and minor incident, based on the duration of the incident. The benefits are estimated using the two methods described below.

Method 1 is used to estimate the reduction in incident durations due to the implementation of incident management strategies on arterials. This method estimates the total delays based on incident duration, mean arrival rate (demand), and mean capacity under normal and incident condition. The total delay due to lane blockage is calculated using the queuing theory equations, as follows:

$$\text{Total Delay Saving (veh-hrs): } \frac{(t_R)*(t_Q)*(\lambda-\mu_R)}{2} \tag{4-1}$$

$$\text{Average time in queue} = t_Q = \frac{(t_R)*(\mu-\mu_R)}{(\mu-\lambda)} \tag{4-2}$$

Where t_Q is average time in queue, μ is mean capacity under normal conditions, μ_R is mean capacity under incident conditions, λ is mean arrival rate, and t_R is average incident duration. Incidents are classified into three categories based on their durations: major, intermediate, and minor for incident durations of 30 minutes or more, 15 minutes to 30 minutes, and less than 15 minutes; respectively. The method suggests a 20%, 12%, and 5% reduction of incident duration due to incident management for major, intermediate, and minor incident duration, respectively.

Method 2 estimates delay savings due to the adjustment of signal timings to better accommodating diverted traffic from adjacent freeways during freeway incidents. In this method, the total delay savings is calculated using the following equation:

$$\text{Total Delay Savings (veh-hrs)} = t_R^2\left(\left(\frac{Ci-C0}{2}\right) + Ci(A-1) + \frac{v(A-1)^2}{2}\right) \tag{4-3}$$

Where C_i is capacity with improved signal timing, C_0 is capacity with existing signal timing, and v is the flow under normal conditions. The parameter A is defined as $A=\frac{Ci-v}{C0-v}$, which is a function of capacities with or without adjustments in signal timing. The capacities in Equation 4-3 are functions of the effective green times (with and without adjustments) and cycle length of signalized intersection.

The capacities in this equation depend on the green time and cycle length before and after the signal timing adjustments. A ten percent reduction factor is applied to the capacity due to the rubbernecking effect due to heavy congestion. This factor may not be needed depending on the approach used to calculate capacity. Since the resulting total delay savings is in terms of vehicle-hours, it is converted into a person-hours delay savings by multiplying by a vehicle occupancy factor of 1.25 persons per vehicles.

The reductions in fuel consumption and emissions are calculated by multiplying the delay savings by the fuel consumption rate or emission rate for the estimated speed. In this methodology, it is assumed that the average speed during congestion is 20 mph.

The total number of crashes is calculated based on the vehicle miles traveled (VMT) and the arterial crash rates. The crash rates per million vehicle miles for injury crashes and property damage only (PDO) crashes used in this methodology are 1.715 and 2.394, respectively. The number of secondary crashes is assumed to be 3.6 percent of the total crashes. A crash reduction factor of 2.8% is applied to calculate the safety benefits, which are in turn converted to dollar values by using a crash cost of $6,300 for PDO crashes, and $229,775 for injury crashes.

The above evaluation methodology was implemented in a Microsoft Excel file by a Palm Beach County Traffic Management Center consultant. However, it is noted that in this version of the tool, only Method 2 is implemented for the delay savings calculation, and Method 1 is not used. Also, it is determined that the method does not consider the impact of the level of traffic diversion from freeways on the impacts calculated using Method 2. Furthermore, the method does not consider the impacts of changing signal timing at an upstream intersection in reducing the impacts of queue spillback from downstream incidents. For this reason, a new methodology was developed in this study to allow for the assessment of the benefits of incident management on urban streets, as described in the next section.

4.3 Developed Methodology

As mentioned in Section 2, a new methodology was developed in this study to assess the benefits of incident management on arterials, with consideration of the spillback effects of downstream incidents on upstream intersection capacity. The developed procedure was implemented in ITSDCAP. The required information includes signal timing data during normal operation and incidents; detailed incident information including the exact incident location, number of lane blockage, and duration; approach volume data, and turning movement counts. These data items should be provided in data files to be read by ITSDCAP. Figure 4-1 illustrates the study scenario. The description of the developed methodology is presented in this section.

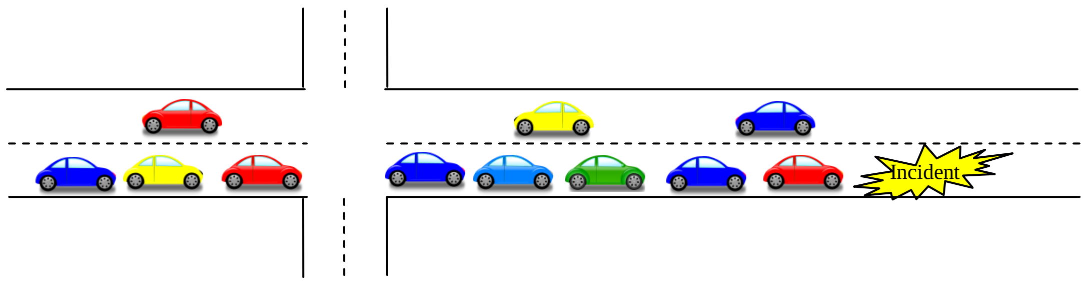


Figure 4-1 Illustration of Study Scenario

When an incident occurs at a location downstream of an upstream intersection, the throughput of the upstream links that feed the incident link can decrease if the queue from the incident spillbacks to the upstream links. For incidents that cause queuing due to a demand exceeding capacity at the incident location, when the signal phases serving the upstream feeding links are red, the downstream link queue starts decreasing due to the reduction in the arrivals at the back of the queue. This creates some queuing capacity that can accommodate flows from the upstream links when vehicles get the green signal. During the first parts of the upstream link green phases, the vehicles will be able to leave the stop lines of the feeding links at the saturation flow rates of these links until the queue due to the downstream incident spills back to the upstream signal again. From the moment this happens until the end of the green phase, the throughput of the upstream links will be controlled by the allowable throughput at the downstream incident location, commonly referred to as the capacity during incidents conditions. Thus, the upstream movement greens can be thought of as being divided into two parts. The first part, referred to as the unconstrained green in this study, is where vehicles from upstream links can leave at the saturation flow rates of these links due to the availability of queuing storage at the downstream link. In the second part, referred to as the constrained green in this study, the movements from the upstream links are controlled by the capacity at the incident location due to the spillback of the queues from the downstream incident. The result of having this constrained green is a reduction in the capacity of the upstream intersection feeding links, which causes an increase in the upstream movement delays. It should be noted that this only happens when there is a spillback from the incident location to the upstream signal.

The length of the unconstrained versus the constrained parts of the upstream movement greens and thus the reduction in the upstream intersection throughputs are expected to be a function of how far the incident is from the upstream intersection and the volume to capacity (V/C) ratio at the incident locations. There is no unconstrained green portion associated with incidents that is at the stop line of the upstream intersection. This portion, however, is expected to increase as the incident is located further downstream from the stop line. Higher reduction in capacity and higher demand at the incident location will result in an increase in the constrained green. This study develops a model to estimate the constrained green and thus the reduction in upstream intersection throughput due to incidents at different downstream locations and with different V/C ratios at the incident locations. The model is derived based on simulation modeling. The derived model is then used as part of an approach for the estimation of delays due to incidents on urban streets, as the summation of the delay at the incident link plus the increase in control delays at the upstream intersection due to queue spillback. The data required deriving the drop in the upstream link capacity estimation model and the testing of the model are performed using the VISSIM microscopic simulation tool (PTV Planning Transport, 2014). Ten replications of each simulation scenarios are run with different seed numbers to account for the model stochastic nature.

4.3.1 Saturation Flow Rate and Capacity during Incident Conditions

As mentioned in the literature review, the only study that could be found in the literature that estimates the drop in capacity due to incidents is the SHRP2 Program L08 project that uses the estimates to assess the reliability of arterial streets using the Urban Facility Procedure of the HCM (Kittelson & Associates, Inc. et al., 2012). The equation used in the estimation is as follows:

$$\mathrm{f}_{ic,int(i),n,m,ap,d} = \left(1 - \frac{N_{ic,int(i),n,m,ap,d}}{N_{n,int(i),n,m}}\right)\left(1 - \frac{b_{ic,int(i),n,ap,d}}{\sum_{m \in L,T,R} N_{n,int(i),n,m}}\right) \geq 0.1 \quad (4\text{-}4)$$

With

$$\mathrm{b}_{ic,int(i),n,ap,d} = 0.58\, I_{fi,int\,(i),n,ap,d} + 0.42 I_{\,pdo,int\,(i),n,ap,d} + 0.17 I_{other,int\,(i),n,ap,d} \quad (4\text{-}5)$$

Where $\mathrm{f}_{ic,int(i),n,m,ap,d}$ is the saturation flow adjustment factor for incident influence on movement m (m = L: left, T: through, R: right) at the subjected intersection during the analysis period (ap) and specific day (d). $N_{n,int(i),n,m}$ is the number of lanes serving movement (m) on the leg associated with phase (n) at the intersection, $N_{ic,int(i),n,m,ap,d}$ is the number of serving movement lanes (m) blocked by the incident on the leg associated with phase (n) at the intersection. $b_{ic,int(i),n,ap,d}$is the calibration coefficient, which is a function of incident severity. Noted, $I_{fi,int\,(i),n,ap,d}$, $I_{\,pdo,int\,(i),n,ap,d}$, $I_{other,int\,(i),n,ap,d}$ are indicator variables for fatal-or-injury, property damage only (PDO), and non-crash incident, respectively.

For segment-based incidents, the same adjustment factor as calculated in Equation 4-5 is applied to the segment speed and is used for the calculation of additional delays due to incidents. However, this methodology assumes that the segment is long compared to the length of incident impact area. Therefore, this study uses Equations 4-4 and 4-5 to estimate the throughput at an incident location, calibrate a microscopic simulation model to produce the capacity at the incident location, estimate the impacts on the throughput of upstream intersection link movements using simulation, then estimate delays based on these estimations using a combination of deterministic queuing analysis and the HCM signalized intersection procedure, as explained in the following subsections.

4.3.2 Calibration of Saturation Flow Rate in VISSIM for No-Incident Conditions

To estimate incident impacts on upstream intersection throughputs, incidents at different downstream link locations and with different V/C ratios had to be modeled using a microscopic simulation tool. Before using the model, however, it had to be calibrated to reflect the estimated capacity with and without incidents. Calibrating VISSIM for no-incident conditions has been addressed in the literature, and recommendations have been made regarding the adjustments of its car-following Wiedemann 74 parameters to achieve the desired capacity (Gomes et al., 2004; PTV Planning Transport, 2014; Hadi et al., 2007).

A virtual arterial network with three lanes in each direction was coded in the VISSIM software with no intersection signal control or cross street volume to calibrate for the saturation flow rate. The saturation flow rate of a link was defined as the maximum number of vehicles that can pass through the intersection during one hour. The traffic demand was increased until the network had enough traffic demands to allow the estimation of the saturation flow rate.

The VISSIM urban driver model (Wiedemann 74) parameters were investigated for potential fine-tuning to obtain the estimated saturation flow rate based on the HCM saturation flow estimation procedures. Urban driver behavior in VISSIM is defined by a set of parameters, such as average standstill distance, additive part of desired safety distance and multiplicative part of desired safety distance. Adjusting these parameters produced saturation flow rates of through movements of about 1854 veh/hr/lane, which is close to the value estimated by the HCM procedures.

4.3.3 Modeling Incidents in Microscopic Simulation

This section describes the calibration of microscopic simulation models for incident conditions in VISSIM. VISSIM does not allow the user to specify incidents in the model. Freeway incidents were emulated in VISSIM using buses with dwell time equal to incident duration on the lanes blocked by the incident combined with reduced speed area on the adjacent lanes to imitate a driver slowing down to observe the incident (Hadi et al., 2007; Zhou and Feng, 2012). Hadi et al. (2007) found that it is important to use the speed reduction area on the adjacent lanes; otherwise, the reduction in capacity due to an incident lane blockage in VISSIM is much lower than the HCM estimates of the capacity reductions. A speed of around 20 mph on the adjacent lanes to a blocked lane on a three-lane section resulted in a reduction in incident capacity near that reported by the HCM (52% reduction). Hong et al. (2000) set up a red signal at the incident lane to simulate the incident and used the reduced speed area to adjust the capacity drop in the adjacent open lanes. The signal turns red once the incident occurs, and turns green when the incident is cleared. Avetisyan et al. (2014) used the "Add vehicle" function within the VISSIM's COM interface to place a vehicle with zero speed at the time and location of the incident, and inserted a reduced speed area for the adjacent lanes. In the above studies, the length of the reduced speed area in the vicinity lanes of the incident was modified by trial and error to achieve the expected drop in capacity due to the freeway incident.

As discussed above, the incidents cannot be directly coded in VISSIM. In this study, incidents are modeled using buses with dwelling times equal to the lane blockage durations and reduced speed areas. This is the same approach used by Hadi et al. (2007). The drop in saturation flow rate at the incident location (midblock locations) was estimated using Equation 4-1. The VISSIM model was calibrated to produce the estimated drops in capacity at the incident locations by adjusting the speed limit in the reduced speed area.

4.3.4 Assessing the Impacts of Incidents on Upstream Intersection Throughputs

Once the capacity drops due to incidents calibrated in VISSIM, it is possible to assess the impacts of incidents at different distances from an upstream intersection and different V/C ratios on the upstream intersection. This was done by introducing incidents at different locations and with different capacity drops in a test network simulated in VISSIM. The network used in the testing is part of Glades Road in Boca Raton, Florida. The incidents were modeled on the link between the I-95 southbound ramp and the Glades Road eastbound downstream intersection. The intersection of Renaissance Way and Glades Road eastbound is referred to as the upstream intersection in this study.

The simulation started with a 15-minute warm-up period, followed by a one-hour analyzing period, and then a 30-minute cool-down period. The first scenario was a network without an incident, and then incidents with different attributes were introduced in the model, as described above. In all simulated scenarios, the incidents were assumed to occur 15 minutes after the simulation began. The simulation model was used to assess the impacts of incidents on upstream intersection throughputs. The incident location and the V/C ratio at the incident location were varied, and the maximum throughputs at the upstream intersection were assessed using the microscopic simulation. Table 4-1 presents the variation in the maximum upstream intersection throughput and unconstrained green as a function of the demand-to-capacity ratio at the incident location and the distance from the upstream intersection to the incident location. The unconstrained green duration was calculated using the following relationship:

$$MT = SF * \frac{UG}{C} + CI * \frac{(TG-UG)}{C} \tag{4-6}$$

Thus, the unconstrained green can be calculated as:

$$UG = \frac{(MT*C-CI*TG)}{(SF-CI)} \tag{4-7}$$

Where UG is the unconstrained green time, MT is Intersection Maximum Throughput (adjusted capacity in the control delay equation), C is cycle length, CI is capacity at incident location, and SF is saturation flow.

Figures 4-2 through 4-4 and Table 4-1 show the variation in the upstream intersection saturation flow rate with incident location and the V/C ratio at the incident location, considering the capacity drop at the incident location. In these figures and tables, the incident location references the distance from the upstream signal stop line. A distance of 200 ft indicates 200 ft from the upstream signal. It should be also noted that the capacity of the through movement at the upstream signal without incident is 5,562 veh/hr. Thus, the incidents with downstream locations and V/C ratios that produce this upstream intersection capacity in the simulation can be recognized as incidents that do not impact upstream signal operations. As can be seen from the figures and tables, there is a drop in saturation flow rate and capacity, and thus unconstrained green increases with the increase in the distance from the upstream intersection but decreases with the increase in the V/C ratio. Regression models were developed to estimate the intersection unconstrained green duration and reduction in the saturation flow rate based on the data in Figures 4-2 through 4 and Table 4-1. The developed regression models are presented in Table 4-2. The developed regression models show a significant relationship between the drops in capacity and these two variables, as indicated by the Coefficient of Determination (R-Squared) values and the t-test of the two independent variable coefficient significance.

Table 4-1 Variation in Upstream Intersection Throughput, Saturation Flow Rate, and Unconstrained Green in Terms of Location and Volume-to-Capacity (V/C) Ratio

V/C at Incident Location	Incident Location (ft)	Intersection Movement Throughput from Simulation (veh/hr)	Intersection Saturation Flow (veh/hr)	Unconstrained Green (sec)
1.1	0	1315	2794	0.00
	200	1941	4124	31.51
	400	2004	4259	36.06
	600	2199	4674	50.06
	800	2319	4929	58.65
	1000	2390	5078	63.69
	1200	2414	5129	65.41
	1400	2451	5209	68.11
	1600	2617	5562	80
1.3	0	1142	2426	0.00
	200	1800	3825	30.19
	400	1989	4226	41.68
	600	2031	4317	44.29
	800	2091	4444	47.95
	1000	2112	4488	49.20
	1200	2130	4526	50.30
	1400	2150	4569	51.52
	1600	2275	4834	59.14
	1800	2556	5432	76.26
	2000	2626	5581	80
1.5	0	1046	2222	0.00
	200	1481	3147	18.87
	400	1793	3810	35.66
	600	1778	3778	34.83
	800	1812	3851	36.67
	1000	1836	3902	37.96
	1200	1776	3774	34.73
	1400	1814	3854	36.76
	1600	2078	4415	50.97
	1800	2241	4761	59.75
	2000	2399	5096	68.25
	2200	2506	5325	74.01
	2400	2668	5670	80

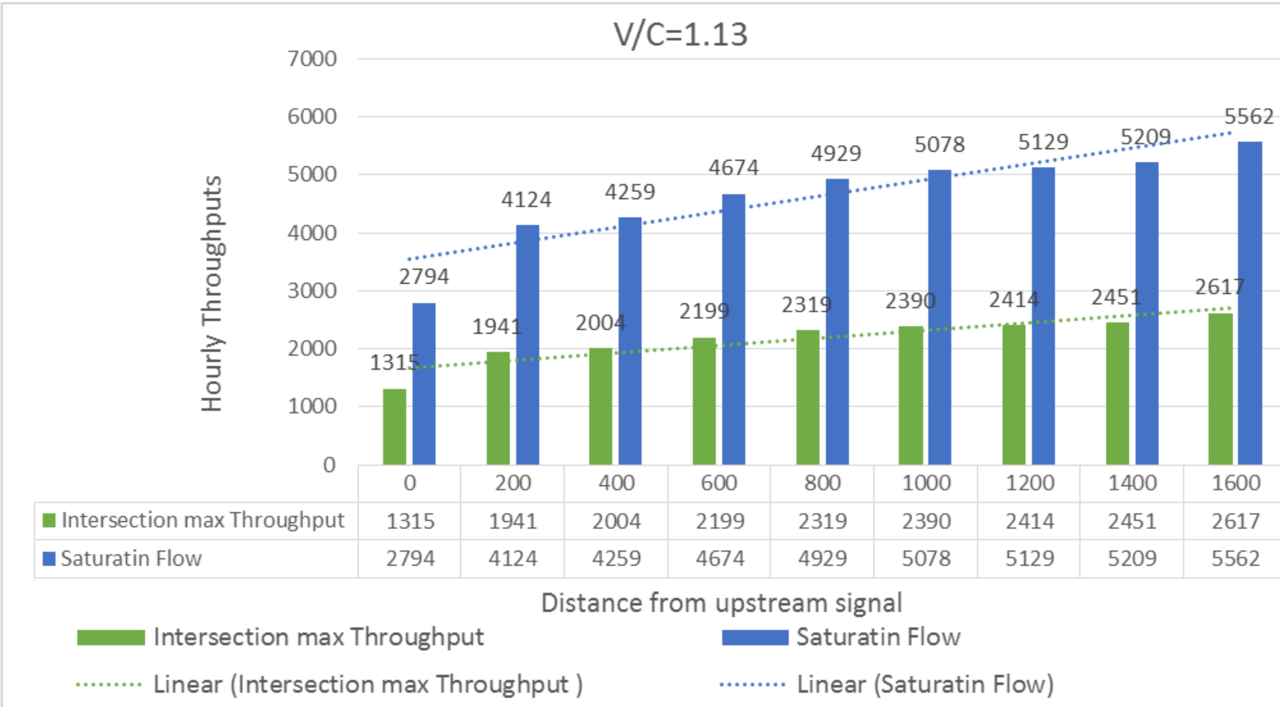


Figure 4-2 Upstream Intersection Saturation Flow Rate Variation with Incident Location with V/C Equal to 1.13 at the Incident Location

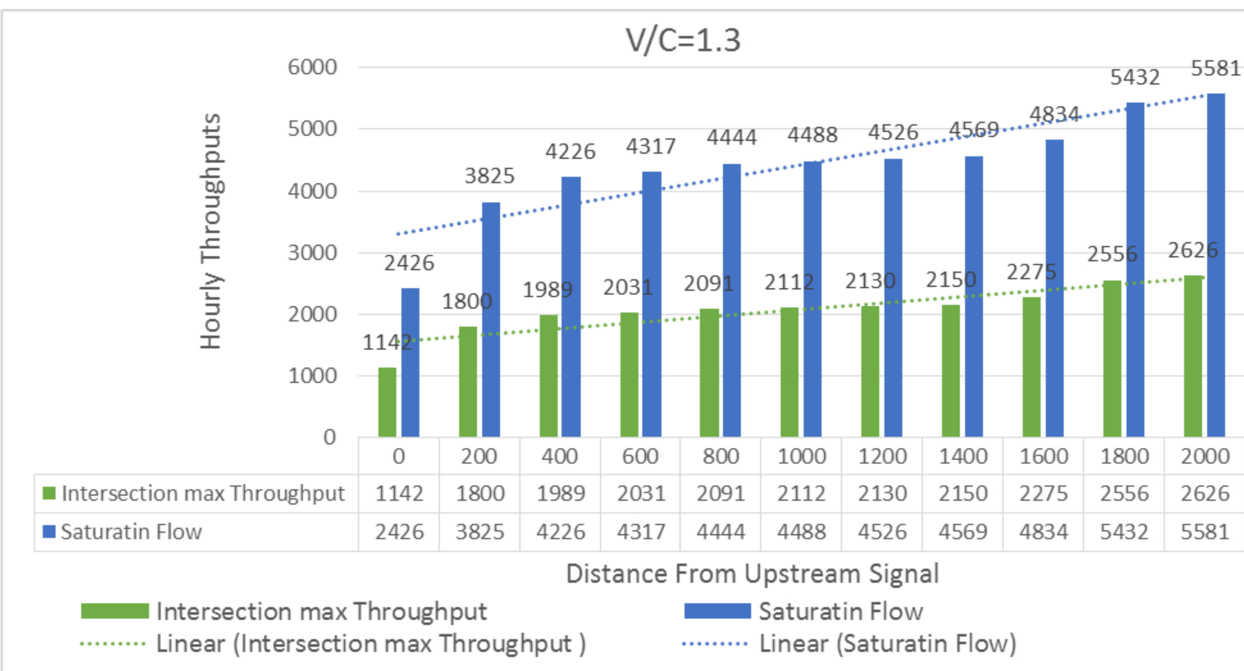


Figure 4-3 Upstream Intersection Saturation Flow Rate Variation with Incident Location with V/C Equal to 1.3 at the Incident Location

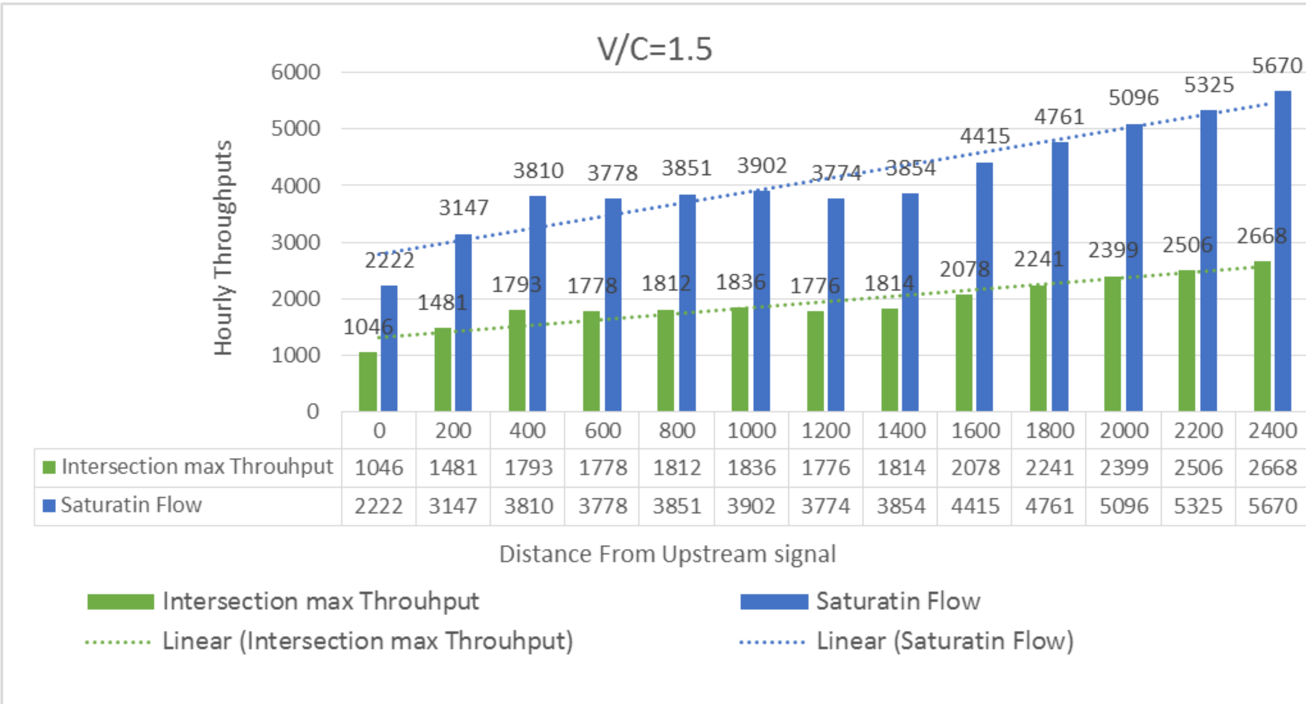


Figure 4-4 Upstream Intersection Saturation Flow Rate Variation with Incident Location with V/C Equal to 1.5 at the Incident Location

Table 4-2 Upstream Saturation Flow and Unconstrained Green Regression Models

V/C at Incident Location	**Intersection Movement Saturation Flow**	**R^2**	**Unconstrained Green**	**R^2**
1.13	$SF = 1.3728x + 3541.4$	0.821	$UG = 0.0418x + 16.916$	0.883
1.3	$SF = 1.1187x + 3305.6$	0.792	$UG = 0.0269x + 20.076$	0.701
1.5	$SF = 1.1195x + 2780.1$	0.880	$UG = 0.0271x + 11.149$	0.890

If the V/C ratio for an assessed situation is between two of the V/C ratios in Table 4-2, interpolation can be used to estimate the unconstrained green duration during incident conditions.

4.3.5 Estimating Incident Delays

Once the impact of the midblock incident on upstream intersection maximum throughput is determined, as described in the previous section, the delay due to the incident can be estimated. The incident delay can be estimated as a combination of the delay due to queuing on the incident link and the increase in upstream intersections delay due the reduction in the saturation flow rate or maximum throughput resulting from the queue spillback to the upstream intersection. The first component is calculated using the deterministic queueing analysis equation, as is used in estimating incident delays on freeways. This method estimates the total delays based on incident duration, mean arrival rate (demand), and mean capacity under incident condition. The total delay due to lane blockage is calculated, as follows:

$$\text{Total Delay Saving (veh-hrs): } \frac{(t_R)*(t_Q)*(\lambda-\mu_R)}{2} \tag{4-8}$$

$$\text{Average time in queue} = t_Q = \frac{(t_R)*(\mu-\mu_R)}{(\mu-\lambda)} \tag{4-9}$$

Where t_Q is average time in queue, μ is mean capacity under normal conditions; μ_R is mean capacity under incident conditions, λ is mean arrival rate, and t_R is average incident duration.

The increase in the upstream intersection delay is calculated using the signalized intersection control delay method presented in the HCM 2010 (TRB, 2010). The method calculates control delay as the sum of three components: uniform, incremental, and initial queue delays. An important parameter for calculating delay using this method is the capacity of the lane group, which is normally calculated as the multiplication of the saturation flow rate and effective green time divided by cycle length. To account for the spillback from the incident location, this capacity is recalculated based on the regression models, developed in this study as described above. The additional control delay due to the reduction in throughputs resulting from queue spillbacks from the incident locations can be calculated by simply using the equations in the HCM or by using commercially available tools such as the Highway Capacity Software (HCS).

4.3.6 Model Testing Results

The arterial incident delay estimation model developed in this study and described in the previous section was tested by comparing the results to the incident delays estimated using VISSIM. The incident delays were estimated as the sum of the incident link delays based on the queuing equations and the increase in upstream control delay due to spillback based on the regression models developed in this study, as described earlier. The HCS software was used for the calculation of the control delay with the adjusted saturation flow rates. The scenario used in this comparison was the scenario with V/C ratio at the incident location equal to1.13.

The network coded in the HCS was designed to correspond to the VISSIM network, as much as possible, to allow a valid comparison. This included traffic demands, network geometry, signal control, and saturation flow rates. The calibrated VISSIM model with no incidents produced a saturation flow rate of 1854 veh/hr/lane, which was used as the input saturation flow model in the HCS.

Incidents at different locations were modeled in VISSIM, and the additional delays due to the incidents were extracted based on the average of ten VISSIM model runs for the no incident conditions and ten runs for the incident conditions.

The base analysis period in the HCM, and thus the HCS analysis, is fixed at 15 minutes. However, the simulated incident duration in this study is 35 minutes. The saturation flow rate during incident conditions, which is used as input to the HCS, was calculated for the first two periods (30 minutes) after the incident occurrence, according to the regression equations presented in the previous section. For the third period (between minutes 30 and 45) after the incident occurrence, the saturation flow was calculated as a weighted average of the saturation rate during the incident in the first 5 minutes of the period, and the no-incident saturation flow rate during the last 10 minutes of the period. This accounted for the full 35-minute period of the incident. The incident delays in the HCS and VISSIM were calculated as the difference between the total delay with incident and the prevailing (no-incident) delay. A comparison was also made with the estimation of incident delay using the queuing equation by itself, as has been used for freeway incidents. The incident delay comparison results are presented Figure 4-5.

Figure 4-5 indicates that the incident delay decreased by moving the incident from the upstream signal toward the downstream, which is expected due to the reduction in the impact on the upstream intersection saturation flow rates due to queue spillbacks. The results show that the use of combination deterministic queuing and the HCM equations procedure to calculate incident delays produced results that are closer to the delays estimated by the microsimulation models, compared to the results obtained based on the deterministic queuing procedure by itself. This is particularly true for incidents located at distances up to 400-500 ft from the upstream intersection for the V/C ratio of this scenario, which is 1.13. Beyond this point, the VISSIM simulation shows much higher impacts of incidents on the delay of the upstream intersection when compared with the HCM procedure. This may be due to the stochastic nature of VISSIM, which better reflects the randomness of traffic arrivals and dissipations. HCM procedures do not fully account for this randomness, thus, it may underestimate the impacts under certain conditions.

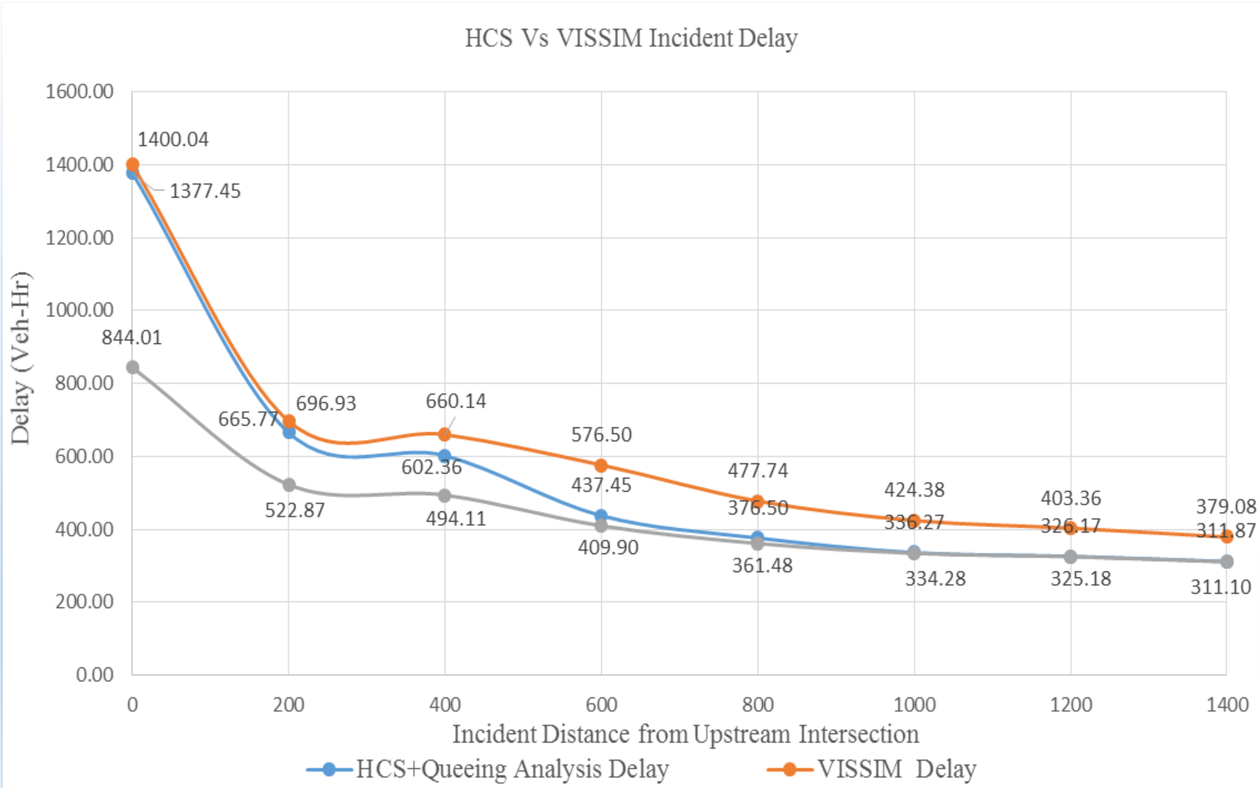


Figure 4-5 Comparison of Incident Delay Using Different Modeling Methods

4.3.7 Summary

The estimations of incident and incident management impacts on arterial streets have been a challenging issue for signalized networks due to the interactions between traffic control and the drop in capacity due to incidents. This study proposes a methodology to calculate the incident delays at signalized networks taking into consideration this interaction. Regression equations were developed to allow for the estimation of the drop in capacity at upstream intersections considering the distance to a downstream incident location and the V/C ratio at the incident location. The regression models show a significant relationship between the drops in capacity and these two variables. As expected, the drop in capacity increases as the incident location becomes closer to the upstream signal and as the V/C ratio at the incident location increases.

The incident delay impact was calculated as a combination of the traffic delay at the incident location using queuing equations plus the increase in control delay at the upstream intersection resulting from capacity drops caused by queue spillbacks due to the incidents. The increase in control delay was calculated using the HCM signalized intersection control delay equations. A comparison with microscopic simulation results showed that the delay estimated using this method produced better results than using the deterministic queuing procedure by itself.

The derived regression models are recommended to be used in sketch planning tools to assess the benefit-cost of incident management, macroscopic and mesoscopic simulation models to model incident and incident management impacts, and data analytics tools to supplement data from other sources to predict incident impacts in off-line and real-time environments.

4.4 Benefit/Cost Analysis Support in ITSDCAP

This section describes an off-line decision support module implemented in the version of ITSDCAP developed in this project to support the benefit analysis of ITS deployment and strategies. Two types of supports are provided in this module. The first type of support provides the input required for other ITS evaluation tools such as FITSEVAL and TOPS-BC. The second is to estimate the benefits directly based on data and modeling. As stated earlier, for this second type of benefit evaluation support, only incident management on arterials and freeways can be evaluated using the current version of ITSDCAP.

4.4.1 Data Support

The benefit-cost analysis results estimated from ITS evaluation tools such as FITSEVAL and BC-TOPS greatly depend on the quality of the input variables and modeling parameters. Table 4-3 presents a summary of the inputs and parameters required for FITSEVAL and TOPS-BC analysis of incident management. Even though default values were provided in these tools for these parameters, these values are usually based on national averages or values for specific regions. If possible, the estimation of these input parameters should be based on available local historical data. This will produce much more accurate benefit analysis results. Therefore, the first type of the benefit/cost analysis support module in ITSDCAP is to generate the required input parameters for the existing ITS evaluation tools using local traffic, incident and crash data. Figure 4-6 shows a snapshot of the ITSDCAP interface for the input data support function of the benefit evaluation module. As shown in this figure, the user can select one specific ITS evaluation tool and the associate inputs that need to be estimated by ITSDCAP. For example, incident rate and average incident duration, which are important inputs to incident management benefit evaluation in FITSEVAL, can be obtained using ITSDCAP and can be used in the FITSEVAL incident management evaluation.

Table 4-3 Summary of Inputs and Parameters Required by the ITS Evaluation Tools

Polices	Inputs/ Parameters	TOPS-BC	FITSEVAL
Incident Management	General Inputs	• Length of Analysis Period • Average Volume • Number of Lanes • Roadway Capacity • Free-Flow Speed • Link Length	• Incident rate • Link attributes (capacity, number of lanes, free-flow speed) • Incident duration • Diversion rate
	Analysis Parameters	• Travel Time Reliability input parameters (VMT, percentage reduction in incident time) • Crash inputs (crash rates by severity and VMT) • Secondary Crashes (Optional) or Reduction in Non-Fatality Crash Rate- no default value • Fuel Use (Optional)	• Percentage of fatalities shifted to injuries (a default value of 21%) • Percentage reduction factor for fatal, injury, and PDO (a default value of 2.8%)

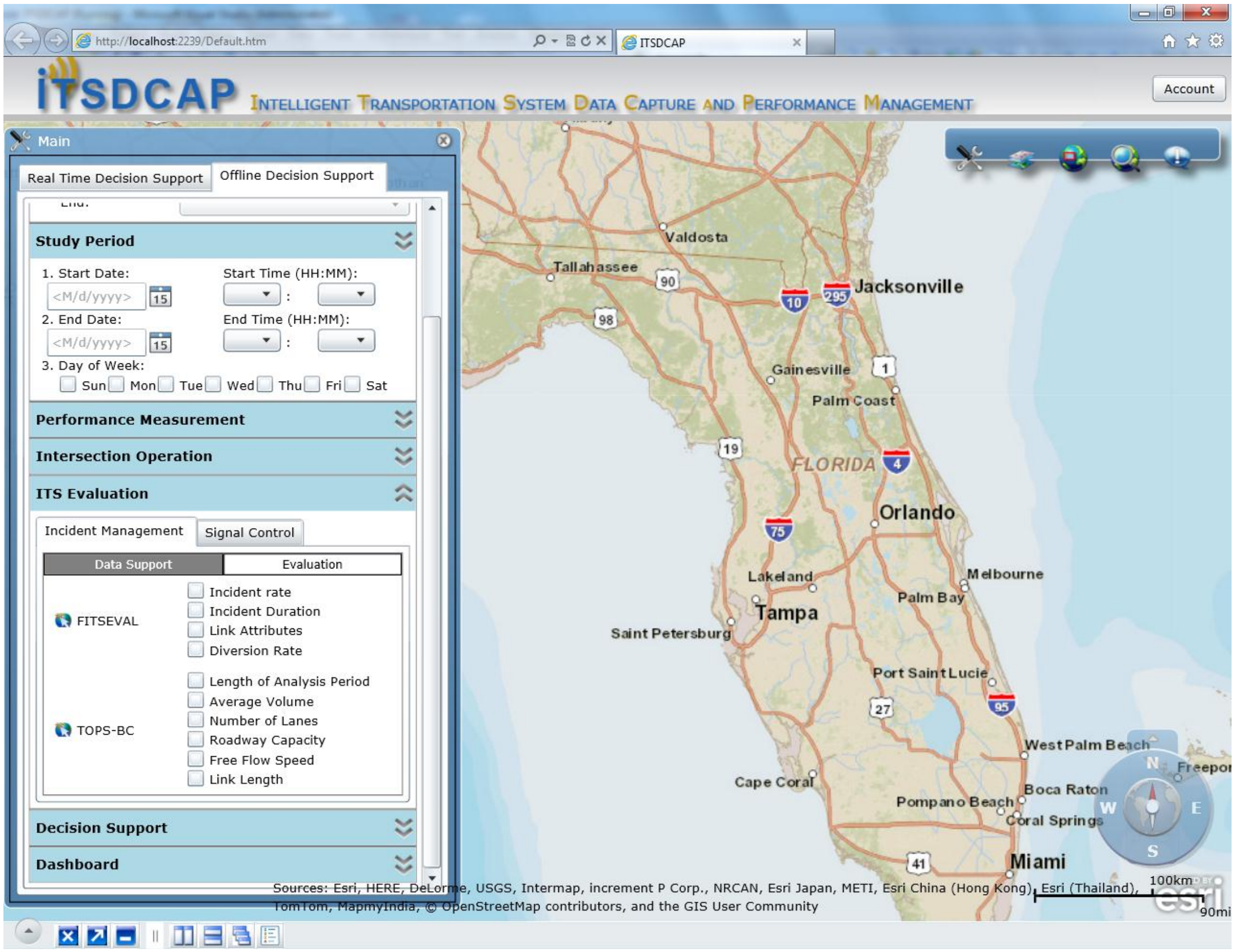


Figure 4-6 Snapshot of the Interface of the Data Support Function of the Benefit Evaluation Module of ITSDCAP

4.4.2 ITS Evaluation

In ITSDCAP, the benefits of incident management on freeways and arterials and signal control improvements during incidents have been incorporated in ITSDCAP. Depending on data availability, the evaluations can be done either based on data or based on analytical equations. Figure 4-7 shows the ITSDCAP interface for the evaluation function of the benefit evaluation module. When traffic and crash data are available, the system performance with or without the assessed strategy can be directly obtained by comparing the corresponding data for the before and after conditions. However, if such data is not available, queuing analysis-based evaluation methodology can be used for freeways in ITSDCAP, which is similar to the method implemented in FITSEVAL for incident management on freeways. For urban arterials, the method described in Section 3 has been implemented in ITSDCAP.

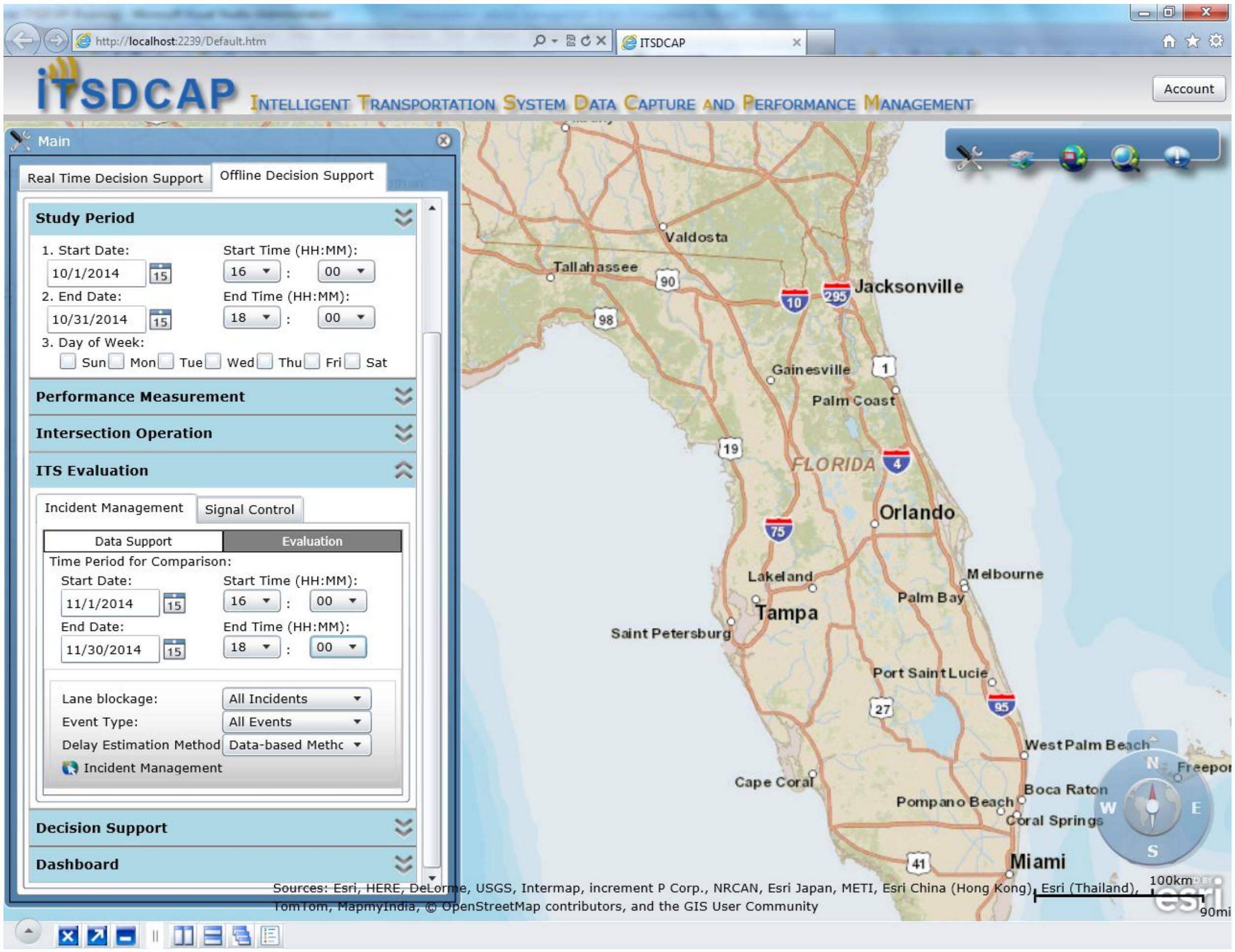


Figure 4-7 Snapshot of ITSDCAP Interface for the Evaluation Function

Data-Based Evaluation

As shown in Figure 4-7, the users can choose to evaluate incident management under the option titled "ITS Evaluation" Module. Other related inputs include the selection of the data source, study corridor, study time period, and types of impacts to be considered for before and after studies. The impacts that can be evaluated using ITSDCAP include the following:

- Incident statistics in terms of incident duration and frequency
- Incident rate
- Demand, queue length, and secondary incident probability for individual incidents
- Incident delay, safety, fuel consumption and emissions for benefit/cost analysis

The evaluation of these impacts is described below in detail.

Incident Statistics: Information of incident duration and incident frequency is useful for TMC and TSM&O operations to adjust their operations according to these statistics. In ITSDCAP, the incident frequency and average incident duration are summarized by time, location, and the number of blocked lanes, giving the user a picture of the temporal and spatial distribution of incidents.

Incident Rate: Incident rate is defined as the number of incidents per million vehicle-miles traveled (MVMT) by the lane blockage type. This is also an input that is required for the FITSEVAL tool that includes default values calculated based on FDOT District 4 data. In ITSDCAP, in order to calculate the MVMT, the total vehicle-miles traveled during the study period for the selected corridor is calculated based on the normal day traffic volumes for each period of the analysis, and estimated using the procedures described in previous sections. The incident rate estimates by the type of lane blockages are outputted by the ITSDCAP tool for each period of the analysis.

Demand, Mobility Impacts, and Secondary Incident Probability for Individual Incidents: Demands during incidents, queue lengths, and associated secondary incident probabilities are important factors that need to be evaluated for incident management assessment. Since the volume counts upstream of the incidents are not actual demand due to capacity constraints of incidents, in this project, the historical normal day volume count at the incident location is used to estimate the demands during the incidents. Travel time and queue length can be estimated based on the detector data using the speed threshold-based method. The maximum queue length associated with the incident is reported in the output.

An enhanced logistic regression model, developed in a previous effort by the research team (Zhan et al., 2009), was applied in this project to assess the potential for secondary crashes. This model was developed based on the FDOT District 4 incident database, and relates the probability of secondary incidents to factors that were found to have statistically significant influence on secondary incident occurrence including time of day, incident location, incident type, lane blockage duration, and queue length. Equation 4-10 shows the derived expression of the logistic regression model for the likelihood of a secondary crash.

$$\begin{aligned}\mathrm{Prob}(\mathit{SecondaryCrash}) = \exp(&-6.100 + 0.462 \times \ln(\mathrm{LaneBlockage}) + 0.170 \times \mathrm{QueueLength} \\ &+ 0.236 \times \mathrm{I95NB} + 0.702 \times \mathrm{PM} + 0.959 \times \mathrm{Midday} \\ &+ 1.397 \times \mathrm{AM} + 0.451 \times \mathrm{Accident})\end{aligned} \tag{4-10}$$

Where LaneBlockage represents the total length of lane blockage in minutes and QueueLength denotes the maximum queue length in miles caused by the incident. All of the other variables in Equation 4-10 are binary variables with a value of 0 or 1. The variable of I95NB indicates whether or not the incident occurred on I-95 northbound. The variables of AM, Midday, and PM have values of 1 if the incident occurred during the weekday AM peak period, midday period, or

PM peak period, respectively. If the incident type is crash, the variable of Accident has a value of 1.

Benefit/Cost Analysis: Four types of performance measures are reported in ITSDCAP: incident delay, fuel consumption, safety, and emissions. For data-based analysis, incident delay is calculated based on the incident day's vehicle-hour traveled compared to the normal day's vehicle-hour traveled for those timestamps with incident conditions, including the recovery time period. Note that the delays for those demands that cannot pass the incident location due to the reduced capacity are captured by considering the VHT changes during the incident recovery time period. The benefits of incident management between any two given periods are calculated by summing the delays caused by all incidents in each period and calculating the difference between the before and after period.

The safety benefits resulted from the implementation of incident management is calculated based on crash reduction factors since the user-specified evaluation period may not be long enough to overcome the well-known regression-to-the-mean problem. In ITSDCAP, a reduction of 2.8% in injury and PDO crash rate due to the quick incident detection, verification and response of incident management systems is assumed. Depending on the availability of on-scene safety management, a further calculation of 21% of fatalities shifted to injuries and 2.8% reduction in fatality crash rate can be applied.

In addition to incident delay and safety benefits, the fuel consumption and emission impacts of incidents are calculated as follows:

$$F_i = D \times e_{si} \tag{4-11}$$

where F_i represents either the fuel consumption or CO, HC, NOx emissions. D is the incident-induced delays, and e_{si} is the fuel consumption rate or emission rate at congested speed s. The advantage of this method is to better capture the fuel consumption and emissions under the stop-and-go conditions caused by incidents.

The abovementioned performance measures are converted to dollars by considering the value of time, safety, fuel costs and emission costs. The resulting benefits are then compared to the costs of implementing incident management to produce the benefit/cost ratio.

4.4.3 Analytical Evaluation of Incident Management

Incident management impacts can be evaluated based on real-world travel time, speed and traffic count data, if these data are available. However, when the data for before conditions or after conditions are not collected, an analytical evaluation can be used instead. This option is also desirable when traffic detector locations do not allow capturing the full lengths of queues due to incidents.

On freeway facilities, the analytical method to estimate mobility impacts can be based completely on a simple queuing theory. The application of incident management strategies on freeways reduces the lane blockage and total incident durations. The ITSDCAP analytical evaluation is similar to the one used in the FIRSEVAL tool but is based on input parameters measured using real-world data. The incident delay with and without incident management on freeways is modeled using the queuing theory, as shown below.

$$TD_i = \frac{t_R{}^2(\mu - \mu_R)(\lambda - \mu_R)}{2(\mu - \lambda)} \tag{4-12}$$

where t_R represents the average incident duration for the incident type under consideration (e.g., by lane blockage), which is retrieved from the incident database. λ is the mean arrival rate, μ is the mean capacity under the prevailing condition, and μ_R is the capacity during the incident. The mean arrival (demand) rate λ is estimated based on historical traffic counts from point detectors under normal conditions. This, however, can be reduced to account for diversion depending on a user-specified diversion rate or based on changes in off-ramp traffic counts during the incidents if such data is available. The evaluations of safety, energy, and emission benefits for incident management are similar to those used for the data-based analysis, that is, the safety benefits are estimated based on crash reduction factors, and the fuel consumption and emission benefits are also estimated using Equation 4-11.

For urban streets, the impacts on a signalized intersection operation from a downstream incident or increase in demand due to diversion can be estimated using the methodology developed as described in Section 3. However, the HCM intersection delay equations were simplified when implemented in ITSDCAP to allow for easier implementation. If it is desirable to perform the analysis without this simplification, a tool like the HCS or other tools that implement the HCM signalized intersection procedures can be used.

In addition to the delay savings, the benefits in safety, fuel consumption, and emissions resulting from the signal timing improvements are calculated using the same procedures as those used for estimating these parameters for incident management, as described earlier.

4.5 Benefit/Cost Analysis Case Study

4.5.1 Case Study 1: Incident Management

In the first case study, the impacts of incidents along the Sunrise Boulevard eastbound in Broward County, Florida during December, 2014 and January, 2015 were examined. Figure 4-8 shows the location of the study corridor. Figure 4-9 illustrates the inputs in the ITSDCAP interface. As shown in Figure 4-9, the user can specify the starting and ending locations of the study route and study period for specific days of the week and time of the day in the evaluation of incident management. The user is also allowed to select the lane blockage type (number of lane blockages) and event type by selecting from a pull-down menu. The lane blockage types that can be selected include all types of incidents; lane blockage only incidents; 1-lane, 2-lane, 3+ lane blockage incidents; or non-lane blockage incidents. The select event types can be all types of events, crash, disabled vehicle, abandoned vehicle, and so on. As mentioned in the methodology section, two methods can be applied to estimate incident delay, the data-based method, and the queuing-based method. In this case study, only the lane-blockage crashes occurring between 7:00 a.m. and 7:00 p.m. during the weekdays were included in the analysis. Since the BlueToad data and Microwave Vehicle Detection Sensors (MVDS) data are available along the study corridor for the study period, the data-based delay estimation method was applied in this case study.

Figure 4-10 presents the Case Study 1 analysis results. As shown in this figure, the incident frequency and incident duration at each location along the study corridor for both the before and after time periods are displayed in the map, which helps users to quickly identify the critical incident locations. The detailed incident impacts are displayed in the pop-up window. This incident management output window shows that 26 lane blockage crashes occurred in the before period (that is, December, 2014) with an average duration of 341 minutes, while 19 lane blockage crashes occurred during the after period (that is, January, 2015), and the average duration is 368 minutes. The lower frequency of crashes during the after period compared to the those in the before period results in reductions of 14,221 vehicle-hours in delays, 2,141 gallons in gas, and 5,446 gallons in diesel, and slightly decreases pollutant emissions. The corresponding dollar values are also listed in the output window. It is important to note that the before-after analysis conducted in this study was for illustration purposes and does not correspond to actual introduced improvement in incident management activities on the corridor. Also, it should be noted that the average incident duration is high, indicating that either the operator did not close the incidents in a timely manner in the SunGuide software, or that the average duration is biased by high incident durations of specific incident types that may need to be examined and isolated.

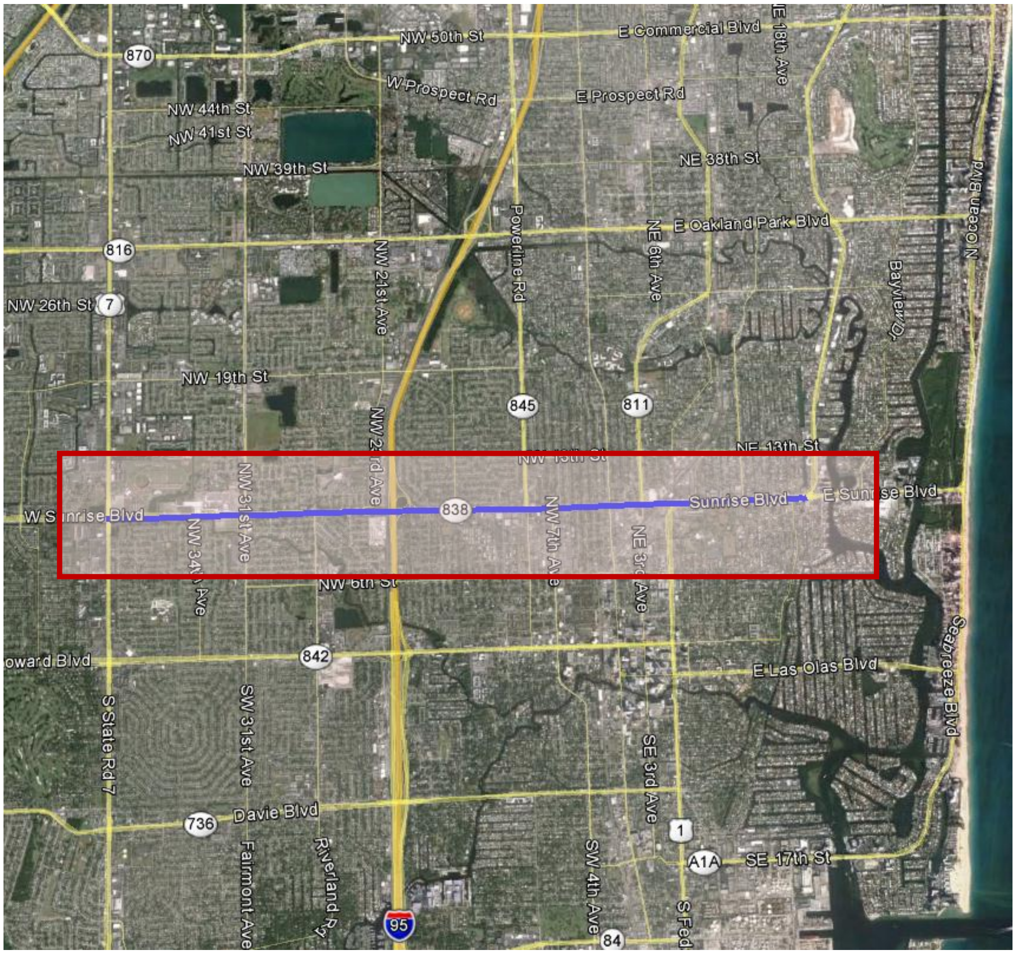


Figure 4-8 Study Corridor in Case Study 1

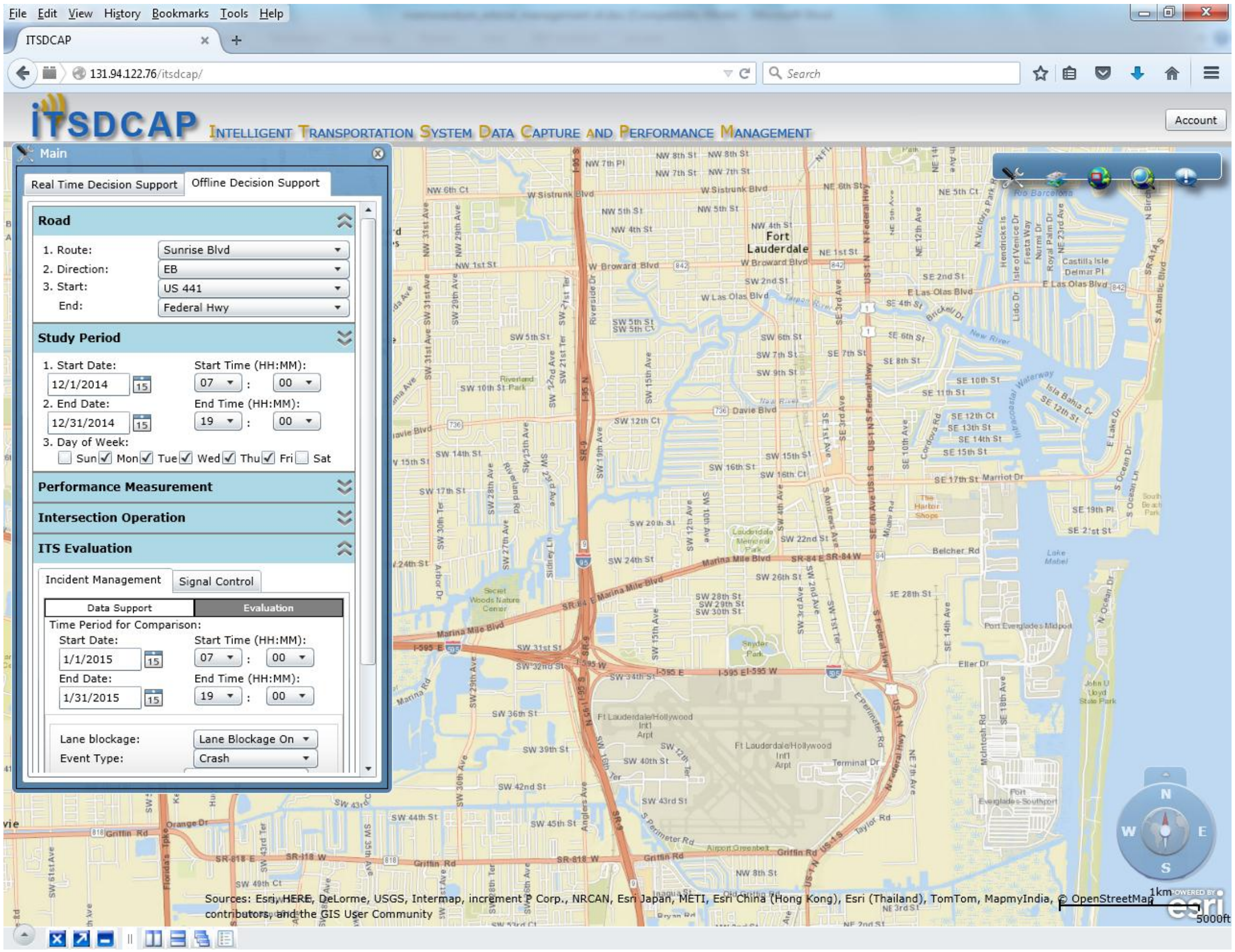


Figure 4-9 ITSDCAP Inputs for Case Study 1

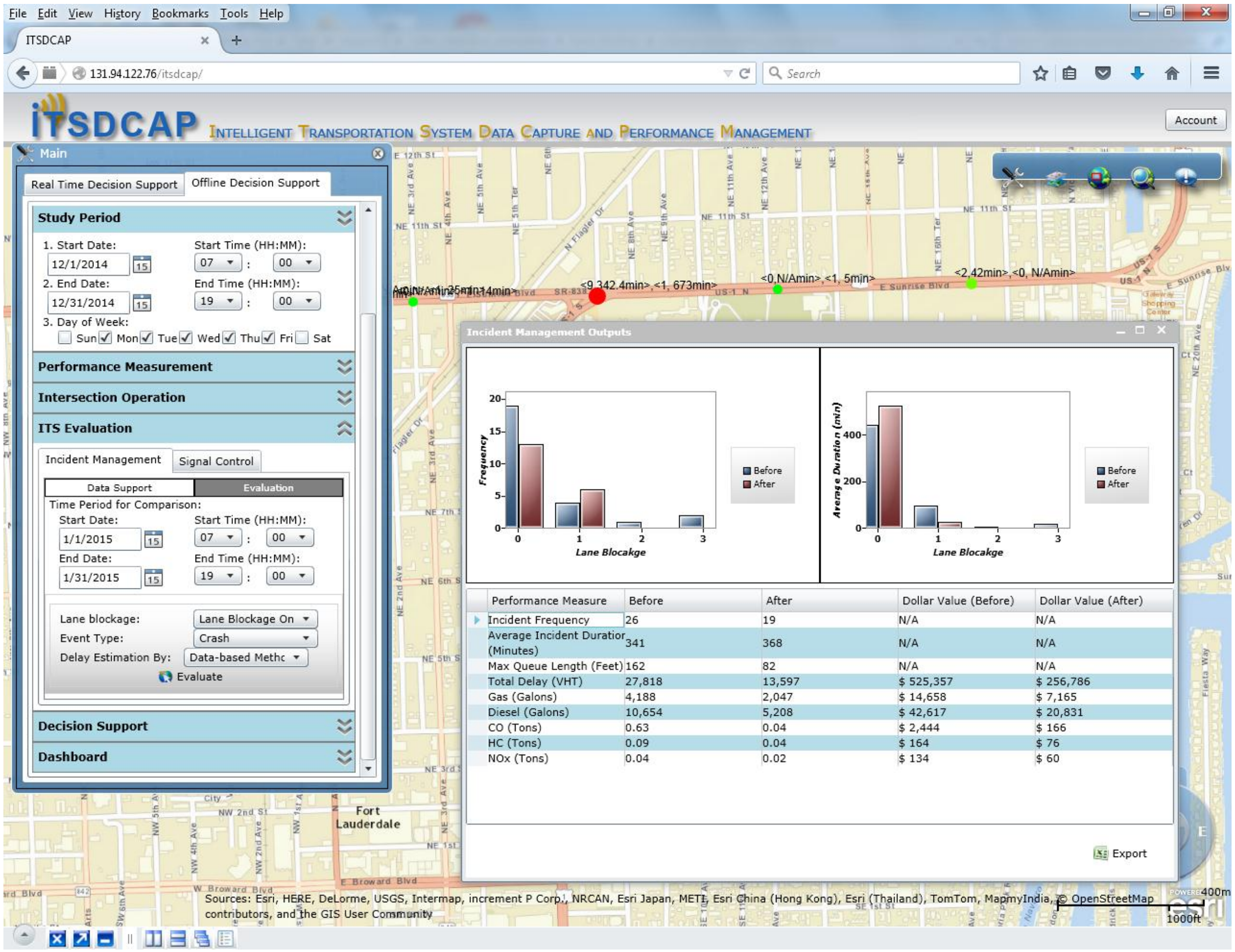


Figure 4-10 Incident Management Case Study Results

4.5.2 Case Study 2: Signal Control

In Case Study 2, the impacts of adjusting signal timing during an incident were investigated following the analysis procedure developed in this study and outlined in Section 3 above. In this case study, a one-lane blockage incident occurred westbound of Oakland Park Boulevard in Broward County, Florida, with a duration of 49 minutes. Table 4-4 lists the incident attributes, as recorded in SunGuide. During this incident, signal timing at the intersection of Oakland Park Boulevard and Powerline Road, which is located upstream of the incident, was adjusted to decrease the green time westbound through movement, shifting green time to movements that do not feed the incident locations, and reducing the cycle length. This resulted in reducing the arrival rate at the incident location, reducing the spillback to the upstream intersection, and utilizing the green time that would have been blocked by the queues from the downstream incident to serve other movements, reducing their delays.

Table 4-4 Summary of Signal Timing Adjustment Event in Case Study 2

Event ID	2239
Report Date	12/12/2014 09:18:18
Confirmed Date	2/12/2014 09:18:18
Last Status Update Time	12/12/2014 10:07:35
First Closed Date	12/12/2014 10:07:35
Road	SR-816
Event Duration	49
Contact	40
Rollover	FALSE
HAZMAT	FALSE
Road Name	Oakland Park Blvd
EVENT_TYPE	Crash
DIRECTION	Westbound
EVENT_LOCATION	I-95
CONDITION	Dry, Clear, Daylight
FIRE	FALSE
PERIOD	1
EVENT_LAT	26.166109
EVENT_LNG	-80.159373
MILEPOST	N/A

In order to analyze this event, various data were collected, including the MVDS detector data, I-95 off-ramp volume counts at Oakland Park Boulevard, historical signal timing data, and signal timing data on incident day. Table 4-5 presents the analysis results for Case Study 2. As shown in this table, two components of delay were calculated, according to the methodology presented in Section 4. The first is intersection delay using the HCM procedures, and the second is the queuing delay along the incident segment based on queuing analysis. Table 4-5 lists each component of delays, as well as total delays for the scenarios without incident and with incident, but no signal timing adjustment, and with incident and signal timing adjustment. The results show that if there is no signal timing adjustment, the incident delay will be 396.5 vehicle-hours, and this value can be decreased to 106.9 vehicle-hours with the adjustment in signal timing. The overall benefit is a delay savings of 289.6 vehicle-hours, which demonstrates the effectiveness of signal timing adjustment during the incident.

Table 4-5 Case Study 2 Analysis Results

Scenario	Signal Delay (Veh-Hr)	Queuing Analysis Delay (Veh-Hr)	Total Delay (Veh-Hr)	Incident Delay (Veh-Hr)	Delay Savings (Veh-Hr)
Without Incident	311.298	0	311.298		289.605
Incident without signal adjust	348.454	359.381	707.835	396.537	
Incident with signal adjust	288.015	130.214	418.230	106.932	

5 ESTIMATION OF CONSTRUCTION IMPACTS

5.1 Introduction

Proper assessment of work zone impacts is required at various stages of construction to support decisions regarding when, where, and how the work zone construction would be implemented. An important component of the decision-making process is to assess the work zone impacts. A report by Mallela and Sadasivam (2011) identified four main components of road user costs associated with the work zone impacts: mobility costs, safety costs, emission costs and other non-monetary costs. The level of details required in assessing the work zone impacts on system performance and the associated user costs depends on the stage of construction decision processes. During the early planning stage, simple analysis tools may be sufficient. In the design and implementation stage, more detailed analysis of work zone impacts is required at the corridor and possibly at the network levels, with the consideration of travel demand reduction, route diversion and so on. Highway capacity facility-based procedures and in some cases, simulation modeling, possibly combined with Dynamic Traffic Assignment (DTA), can be utilized at this stage to assess work zone impacts as well as the impacts of mitigation strategies for work zones. During the construction stage, data from point detectors, vehicle re-identification, or other technologies can be collected that can be directly analyzed to determine work zone impacts.

This study aims to develop a module within the Intelligent Transportation System Data Capture and Performance Management (ITSDCAP) environment to provide the data analysis and modeling support for impact analysis at multi-levels of details, depending on user analysis requirements. The available methods and tools to assess work zone impacts will be reviewed first. Based on a literature review, this study will identify applicable methods or tools for each level of the proposed multi-level construction impact analysis framework. The work zone evaluation based on real-world data will be directly implemented in ITSDCAP. For the external modeling tools, required inputs such as demand and capacity will also be provided by the ITSDCAP tool module.

5.2 Literature Review

This section will provide a detailed review of road user costs and their evaluation methods and tools that have been developed and discussed in the literature. The following section documents the literature review conducted as part of this task. A table at the end of this section (on Page 10) summarizes the literature review findings.

5.2.1 FHWA Road User Cost Estimation Procedures

The Federal Highway Administration (FHWA) report titled "Work Zone Road User Costs: Concepts and Applications" (Mallela and Sadasivam, 2011) classifies the road user costs into two categories, monetary and non-monetary user costs. The monetary road user costs are associated with the mobility, safety, and emission impacts that can be converted into dollar values. Non-monetary road user costs are defined by the FHWA report as social and environmental impacts such as noise resulting from construction. The proposed estimation method for each type of user cost in the FHWA report is reviewed below.

Travel delay costs: Work zone travel delay consists of five components: 1) Speed change delay as a vehicle approaches the work zone and departs from the work zone; 2) Reduced speed delay when a vehicle travels at a lower speed within the work zone; 3) Stopping delay; 4) Queue delay; and 5) Detour delay along the alternative routes. These delays can be converted into dollar values by multiplying by the monetary unit cost, as shown in Equation 5-1.

Travel Delay Cost = Average Delay Time (per vehicle)*Unit Cost*Number of Vehicles (5-1)

Note that the unit cost according to this procedure varies with the type of travels. In this procedure, three types of travels are considered: personal, business and truck travel.

Vehicle operating costs (VOC): VOC refers to the expenses that road users paid as a result of vehicle use, which includes speed change VOC, stopping VOC, queue idling VOC and detour VOC. The general formula is shown below:

VOC Cost=Additional Consumption*Unit Cost*Number of Vehicles (5-2)

The additional consumption in Equation 5-2 is related to the consumption cost of fuel, engine oil, tire-wear, repair and maintenance, and mileage-related depreciation. Three methods can be used to determine these consumptions. In the NCHRP Report 133 (Curry and Anderson, 1972), the consumptions are considered a function of initial speed. However, these parameters depend on vehicle speed, grade, and vehicle class in the Texas Research and Development Foundation (TRDF) method. In addition, a set of equations is used to calculate the VOC consumption in the FHWA's HERS-ST method (FHWA, 2005) based on the combination of consumption type, vehicle type, and influential factors such as vehicle speed, speed change, curvature, and grade.

Crash costs: The presence of work zone (WZ) can result in work zone-related crashes or detour-related crashes. Such costs can be determined based on the expected difference in the crash rate per million vehicle mile (MVMT) with or without a work zone. Equation 5-3 displays the calculation of the crash costs.

$$\text{Crash Cost} = (\text{Crash Rate with WZ} - \text{Crash rate pre-WZ}) * \text{MVMT} * \text{Unit Cost} \tag{5-3}$$

The difference in crash rate in Equation 5-3 can be estimated based on pre-work zone crash rate and crash modification factor (CMF) for work zone impacts, considering the impacts of safety improvement countermeasures (if they exist). The unit cost in this equation should also vary depending on the severity level of crashes.

Emission costs: The speed changes and stops when vehicles traveling through the work zones result in additional emissions. The corresponding emission costs can be calculated based on emission rate per MVMT, as follows:

$$\text{Emission Cost} = \sum(MVMT * Emission\ Rate * Unit\ Cost)_{\text{by pollutant type}} \tag{5-4}$$

$$\text{Work Zone Emission Cost} = \text{Emission Cost(WZ)} - \text{Emission Cost(Pre-WZ)} \tag{5-5}$$

The emission rate listed in Equation 5-4 can be estimated using either static emission factor models or dynamic instantaneous emission models. Examples of models that are based on static emission factor models include the Mobile 6.2 developed by the Environmental Protection Agency (EPA) (currently not supported) and the EMFAC model developed by the California Air Resource Board (CARB). The Motor Vehicle Emission Simulator (MOVES) developed by the EPA, the Comprehensive Model Emission Model (CMEM), and the Mobile Emission Assessment System for Urban and Regional Evaluation (MEASURES) are examples of dynamic instantaneous emission models, which are able to estimate emissions at a more detailed level. Note that the MOVES can be also used at a lower level of details using static factors.

Other Impacts: In addition to the above user costs, noise and business impacts are key non-monetary impacts of work zone. The FHWA Roadway Construction Noise Model (RCNM), a windows-based computer program, can be applied to predict noise levels during highway construction. However, there is no good method to estimate the damage caused by noise. For business and local community impacts, the procedure recommends conducting surveys with business managers and local residents to collect the impact information.

5.2.2 HCM-Based Method

A procedure is provided in the Highway Capacity Manual (TRB, 2010) to calculate the reduced freeway capacity due to short-term and long-term construction along a basic freeway segment. This procedure can be used in combination with other procedures to estimate work zone impacts on freeway segment operations. For short-term construction, the reduction in roadway capacity can be calculated from the number of available lanes, activity type and density, and the presence of adjacent on-ramps. However, for long-term construction, only a table that lists some values of long-term construction zone capacity as reported in previous studies is presented in the HCM.

In addition, the HCM 2010 provides macroscopic procedures to calculate the performance of freeways and urban streets. The corresponding computational engines are FREEVAL and STREETVAL, respectively. Recently, these two tools are further enhanced to model travel time reliability, which are called FREEVAL-RL and STREETVAL-RL. These tools can be calibrated to the existing conditions to allow estimates of work zone impacts.

In FREEVAL or FREEVAL-RL, the freeway facilities are divided into different types of segments, including basic, merge, diverge, and weaving segments. Different analysis approaches are used for undersaturated and oversaturated conditions. For undersaturated conditions, roadway segments are analyzed independently. Depending on segment type, the corresponding HCM procedure is applied to calculate the segment speed, capacity, and in turn, density and the level of service. When traffic is under oversaturated conditions, the freeway facility is analyzed as a node-link system and a cell transmission model-based algorithm is utilized to track queue accumulation and dissipation over multiple segments and periods.

Urban street facilities can be coded in STREETVAL or STREETVAL-RL as segments with boundary points that represent signalized and unsignalized intersections. The performance of a segment for the automobile mode is analyzed by first determining the segment running time, the through movement delay, and the stop rate in each 15 minutes based on the free-flow speed and the control types, and then calculating the segment travel speed, stop rate, and level of service. The level of service of signalized intersections is determined based on control delays. In the HCM procedure, this is a function of adjusted saturation flow rate and percentage of vehicles arriving on green.

The HCM work zone procedure has been updated in a new release of the manual that is scheduled for release later in 2015. A new version of FREEVAL (FREEVAL-2015E) has been developed in JAVA programming language that incorporates this updated work zone procedure. In FREEVAL-2015E, traffic demand and constructions are modeled deterministically, while the occurrence of incidents and weather are modeled using a stochastic approach. In addition to the work zone capacity for basic freeway segment, approaches to calculate work zone capacity for merging, diverging, weaving and crossover segment types are also proposed in the HCM 2015 and implemented in FREEVAL-2015E. The work zone impacts according to the procedure are functions of work zone configurations (normal and reduced number of lanes), segment type, ramp volumes, acceleration/deceleration lane length, among other factors. The output performance measures from FREEVAL-2015E include average speed, density, and LOS for each segment and each time interval.

The impacts of construction can be modeled using these HCM tools by reducing the number of available lanes and adjusting the speed limit and capacity in work zone. The output performance measures include travel time, delay, average speed, and so on.

5.2.3 Q-DAT

The Q-DAT tool developed by the Texas Transportation Institute is a simple Microsoft Excel spreadsheet-based tool for construction impact analysis. Two types of analysis can be conducted using this tool: Delay and Queue Estimation and Lane Closure Schedule, For the first type of analysis, with simple inputs consisting of travel demand and lane closure information, the tool can output the value of queue length by comparing traffic demand with reduced work zone capacity and delay due to a work zone based on a regression equation. In the Lane Closure Schedule analysis, the queue length and delay for every possible combination of construction hour and number of lanes blocked are calculated, and the scenarios with queue length and delays less than certain predefined thresholds are recommended to the user.

Q-DAT requires simple inputs and can produce estimates of queues and delays, which is applicable for planning purposes. However, only the mobility impacts due to work zone are assessed, and the outputs are not given in terms of road user costs directly.

5.2.4 RealCost

RealCost is a Visual Basic for Applications (VBA), a macro-enabled Microsoft Excel-based tool for life cycle cost analysis in pavement design, which was developed by the FHWA. In addition to traffic demand and work zone configuration, RealCost also needs the input of the pavement design alternatives and construction costs. RealCost can calculate the life cycle values for both user costs and agency costs. Agency costs have to be directly input by the users. User costs can be either a user-input or calculated by the RealCost tool based on the procedures recommended by the NCHRP 133 study. The cost analysis results from RealCost for multiple pavement alternatives can be used to prioritize alternatives.

RealCost can provide estimates for user costs and agency costs with simple traffic flow and project information, however, only mobility costs can be estimated using this tool. Safety and emission costs are not included in the analysis.

5.2.5 QuickZone

QuickZone is a tool developed by FHWA for analyzing work zone mobility impacts such as traffic delays, queue, and associated delay costs. It uses a node- and link-based network layout and estimates delays and queues based on a deterministic queuing model. The mobility impacts estimated by QuickZone can be used to compare alternative project phasing plans.

QuickZone is capable of modeling the entire network for work zone mobility impact analysis, and it can also be applied to evaluate traveler behaviors with the presence of work zone, such as route changes, peak-spreading, mode shifts, and trip losses. However, QuickZone mainly focuses on the mobility impacts for user costs.

5.2.6 SHRP 2 C11 Reliability Analysis

A sketch planning level to estimate reliability was used as part of an economic analysis tool developed for the SHRP 2 Project C11. This is a corridor spreadsheet tool based on SHRP 2 Reliability Project L03 research. It can be used to improve travel time reliability in the benefit/cost analysis. The Reliability Module involves minimal data development and model calibration. The tool requires simple inputs, including roadway capacity, annual average daily traffic (AADT), percent trucks, and growth rate.

5.2.7 SHRP 2 LO7 Reliability Analysis

The reliability evaluation tool developed by the SHRP 2 L07 project (Ingrid et al., 2013) was designed to analyze the effects of highway geometric design treatments on non-recurrent congestion using a reliability analysis framework. The tool has a Visual Basic for Applications (VBA) interface embedded in a Microsoft Excel spreadsheet.

The reliability evaluation tool allows the user to input data regarding site geometry, traffic demand, incident history, weather, special events, and work zones. Based on these data, the tool calculates base reliability conditions. The user can then analyze the effectiveness of a variety of treatments by providing fairly simple input data regarding the treatment effects and cost parameters. As outputs, the tool predicts cumulative Travel Time Index (TTI) curves for each hour of the day, from which other reliability variables are computed and displayed. The tool also calculates the cost-effectiveness of treatment alternatives by assigning monetary values to delay and reliability improvements, and compares these benefits with the expected cost over the life of each treatment.

Compared to the other evaluation tools, the L07 tool takes the travel time reliability into consideration and also provides the benefits and costs assessment for different mitigation designs. In addition, safety impacts are also estimated in the benefit/cost analysis.

5.2.8 WISE

The Work Zone Impacts and Strategies Estimator (WISE) is a product produced by the SHAP2 R11 Project. It is a decision-support tool to assist agencies in evaluating the impacts of work zone and work zone-related mitigation strategies along a given corridor or for a network (Lawrence et al., 2012).

WISE is able to evaluate renewal projects at both the planning and operation levels. When used as a planning tool, the user can evaluate the effectiveness of various travel demand and construction duration strategies for multiple projects by comparing two main measures: construction cost and traveler delay cost. When used at the operational level, time-dependent congestion and diversion caused by congestion can be captured by a simulation-based dynamic traffic assignment (DTA) tool. A more accurate estimation of the diversion due to the impacts of capacity reduction resulting from work zones can be obtained using the operation module based on the simulation outcomes. The user can model whether or not to change the sequence of projects, based on the diversion rate results.
However, WISE also has some limitations. It cannot be connected to a simulation-based DTA other than DynusT. It needs to be calibrated with a significant associated effort.

5.2.9 FITSEVAL

In a previous FDOT projects, the FIU research team investigated the development of tools and procedures to perform a sketch-planning evaluation of the costs and benefits of ITS alternatives within the FSTMUS Modeling environment (Mohammed H. et al., 2008). Based on the review of existing sketch-planning tools, such as IDAS and ITSOAM, this research team developed a Florida ITS Evaluation (FITSEVAL) tool to evaluate the various ITS deployments, which includes the smart work zone.

The FITSEVAL tool can be applied to evaluate the benefits of different types of smart work zone technologies, including systems providing congestion information and alternate route information, dynamic merging, speed advisory, and queue warning systems. The evaluation methodology varies with the technology considered. The value of work zone capacity in these evaluation methodologies are calculated based on the method included in the HCM 2000.

In order to calculate the benefits of a system that provides delay or alternative route information at smart work zones, a certain percentage of travelers are assumed to divert to alternative routes. Five percent of vehicles diverted to alternative routes when provided with delay information, and 15 percent are assumed in this tool when provided with alternative route information.

The benefits of the provision of a speed advisory message are calculated in this tool based on the assumption of a 10 percent reduction in speed variance. Similarly, a 7 percent reduction in crash rate is applied to assess the benefits of a queue warning system.

In addition, it is assumed that the work zone capacity will increase by 5 percent with the application of a dynamic lane merge system. This impacts the capacity results in terms of decreasing the travel time. The impact of a dynamic merge on safety is calculated by assuming an additional 40 percent reduction in the crash rate.

5.2.10 Reliability Analysis based on SHRP 2 L02 Project

The SHRP 2 L02 project developed methods for monitoring and evaluating travel time reliability based on data generated by traffic monitoring systems, such as those based on point traffic detectors, AVI, AVL, and private sector data. It provides guidelines for measuring, categorizing, identifying, and understanding the causes of unreliability necessary to identify possible mitigating actions.

The SHRP 2 L02 project provided recommendations to agencies regarding the establishment and use a Travel Time Reliability Monitoring System (TTRMS). Recommendations regarding three major components of the system – a data manager, a computational engine, and a report generator – are provided. The data manager assembles incoming information from traffic sensors and other systems, such as weather data feeds and incident and construction reporting systems, and places it in a database that is ready for analysis. The second component of the monitoring system, the computational engine, utilizes the collected, fused, and cleaned data to provide an assessment of the system's reliability and the contributing factors. New visualization and analysis methods such as travel time rate probability density functions (PDFs) and their associated cumulative density functions (CDFs) by regimes were introduced in the L02 project. The L02 project also provides recommendations regarding the third component of the monitoring system, the report generator, which presents results based on user requests.

Work zones with different congestion levels have been identified in Project L02 as one of the regimes for travel time reliability investigation. The impacts of work zone on reliability can be compared with other reliability influencing factors such as high demand, weather, incident, and special event using L02 procedures.

5.3 A Multi-Level Framework for Work Zone Impact Analysis

The literature review provided a list of existing work zone impact analysis tools. These tools can be applied to different levels of analysis according to user requirements. The type and level appropriate for work zone analysis may be different depending on the roadway project's phase in development or construction. Figure 5-1 shows a diagram of typical stages of required analysis. In addition, the level of analysis depends on the project characteristics and available resources for the analysis.

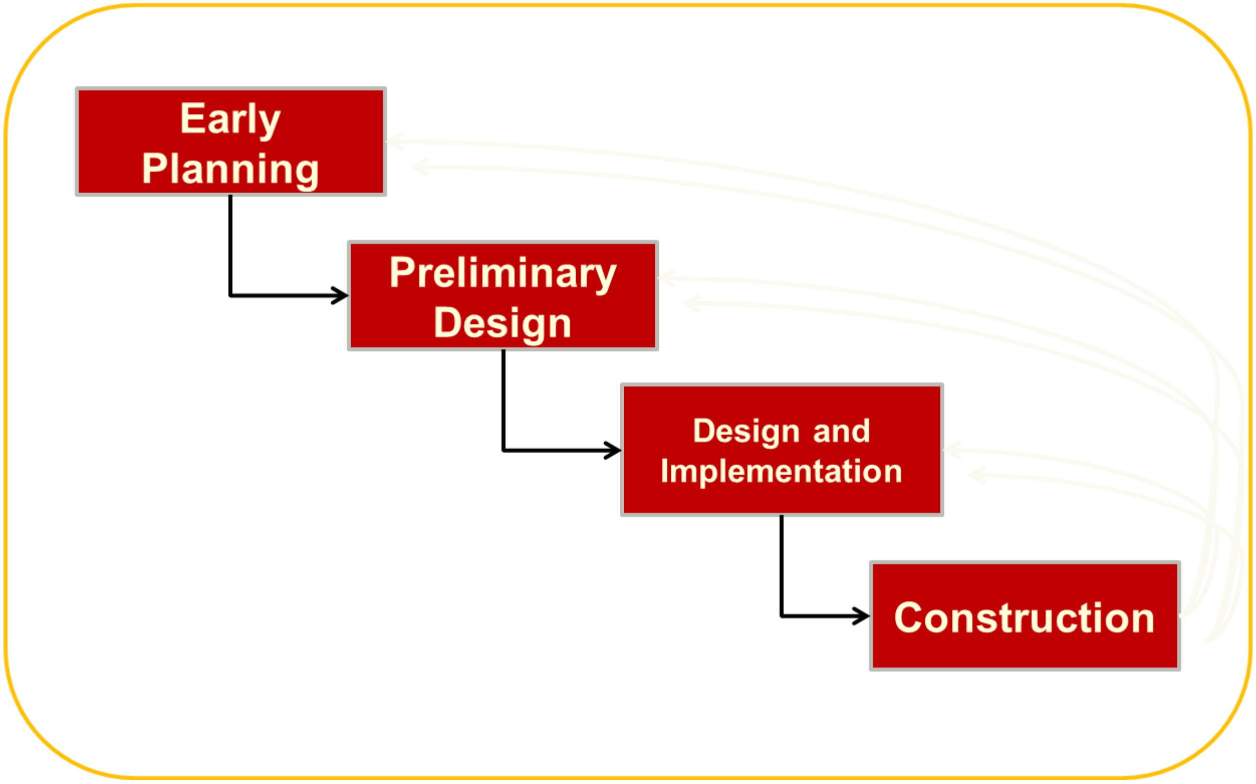


Figure 5-1 Diagram for Multi-Level Work Zone Impact Analysis Framework

As shown in Figure 5-1, the analysis may be conducted in four stages: early planning, preliminary design, design and implementation, and construction. In the first stage, the early planning stage, the analysis of work zone impacts may be conducted at the sketch-planning level, as there is very limited work zone information available. Available sketch-planning tools such as Q-DAT, SHRP 2 C11, and RealCost can be applied to evaluate work zone impacts with simple inputs. In the preliminary design stage, a combination of using the QuickZone and L07 tool is recommended, in addition to the FHWA procedures, as these tools can be utilized to analyze the work zone impacts at the corridor level and the impacts of mitigation strategies. At the design and implementation stage, more detailed analyses may be required to assess the impacts of the work zone. In this case, simulation tools, possibly combined with DTA, such as what is used in the WISE tool, can be used to model the work zone impacts, including traffic diversions. When lacking detailed data and the required resources to perform simulation-based DTA, HCM-based procedures and tools may be applied. During the construction and post-construction stages, real-world data may be available, therefore, a before and after study can be conducted based on the collected data to evaluate the impacts of construction. Table 5-1 summarizes the available tools for each analysis stage. The required inputs and outputs for each tool are also listed in this table.

Table 5-1 Summary of Available Tools

Stage	Tools	Costs	Platform	Input	Description	Output	Description	Note
Stage 1: Early planning	Q-DAT	FREE	Excel Spread-sheet	Lane closure information	No. of lanes, length, work zone capacity	Work Schedule	Lane closure time and construction plan	Simple input and output which is convenient for data-poor condition
				Travel Demand	AADT	Delay and queue	Delay and queue estimation	
	Realcost	FREE	Excel Spread-sheet	Project details	Construction costs, work activity	Costs estimation	User cost and agency cost	Simple input and output which is convenient for data-poor condition
				Traffic data	AADT, percentage of vehicles			
	SHRP 2 C11			Drop in Capacity				Reduced data requirement and convenience to obtain travel time reliability results
				Traffic data	Roadway capacity, annual average daily traffic (AADT), percent trucks			
Stage 2: Preliminary design	SHRP 2 L07	FREE	Excel Spread-sheet	Geometry features	Length, lane width, FFS	B/C analysis	B/C ratio for strategies	Require more inputs to perform benefit/cost analysis but provide estimates for reliability
				Travel Demand	Demand, percentage of truck	Travel reliability	Mean TTI, lateness Index, buffer index	
				Incident	Duration and No. of incidents, average costs			

Stage	Tools	Costs	Platform	Input	Description	Output	Description	Note
				Weather	Rainfall, snowfall			
				Events	Event frequency, hourly demand increase			
				Work zone	WZ capacity, No. of lanes closed			
	QuickZone	FREE	Excel Spread-sheet	Network	Node, links	Delay and queue	Delay, queue, travel behavior	Require the inputs of traffic network and more detailed data but diversion impacts are considered
				Demand	Daily traffic demand	Costs	Delay costs and agency costs	
				Project information	Date for project, mitigation strategy			
	FHWA Procedures			Travel Delay	Speed change delay, stop delay, reduced speed delay	Mobility costs	Travel delay costs and vehicle operating costs	Capable to estimate safety and environment costs in addition to mobility costs
				Travel information	Work zone speed, initial speed	Safety costs	Work zone crash costs	
				Incident	Crash rates, Incident changes	Emission costs	Work zone emission costs	
				Environment	Emission			
				Other	Business and			

Stage	Tools	Costs	Platform	Input	Description	Output	Description	Note
					local impacts			
Stage 3: Design and Implementation	Simulation-Based DTA Methods and Tools	FREE	DynusT	Network	Traffic network and travel demand	Monetary costs	Construction costs and travel delay costs	Utilize dynamic traffic assignment for modeling and require more detailed inputs
				Project information	Date for project, project strategies	Work plan	Optimized schedule for project	
	HCM-Facility Procedures and Tools	FREE	Excel Spread-sheet or JAVA	Project summary	Study period, geometry features	Travel reliability	TTI, PTI, probability distribution for reliability	Capable to estimate travel time reliability based on multiple data sources
				Travel Demand	Demand pattern, day and month demand			
				Incident	Incident type, incident rate			
				Weather	Probability of weather categories			
Stage 4: Construction and Post-Construction	Data Analytics			Demands	Measured	Impacts on travel time, reliability, emission, and safety		Appropriate for data rich condition
				Capacity	Measured			
				Crash rates	Measured			

5.4 ITSDCAP Support for Multi-Level Framework

ITSDCAP is designed to provide the data and modeling support for the abovementioned multi-level construction impact analysis framework. The analysis for Stage 1 to Stage 3 of this framework must be conducted using the modeling tools listed in Table 5-1. Therefore, the main support provided by ITSDCAP for these three stages is to produce the required inputs for these tools based on available historical construction and traffic data. The example output variables by ITSDCAP include traffic demands, crash rate, existing capacity, queue discharge rate, and free-flow speeds by segment and by time of day. The output format will be in a text format that can be easily used by these modeling tools. For Stage 4, if real-world data are collected before, during and after construction, ITSDCAP can directly estimate the impacts of the work zone based on the collected data. The results of the data analytics will include travel time rate distributions with and without consideration, as well as statistical hypothesis testing that the work zone has produced in changes in work zone performance.

Figure 5-2 shows a snapshot of the front-end design of the construction support module in ITSDCAP. Currently, this module is under development. An example of the output of ITSDCAP for a work zone impact analysis is shown in Figure 5-3. As shown in this figure, users can compare the travel time reliability without and with construction at different percentile levels.

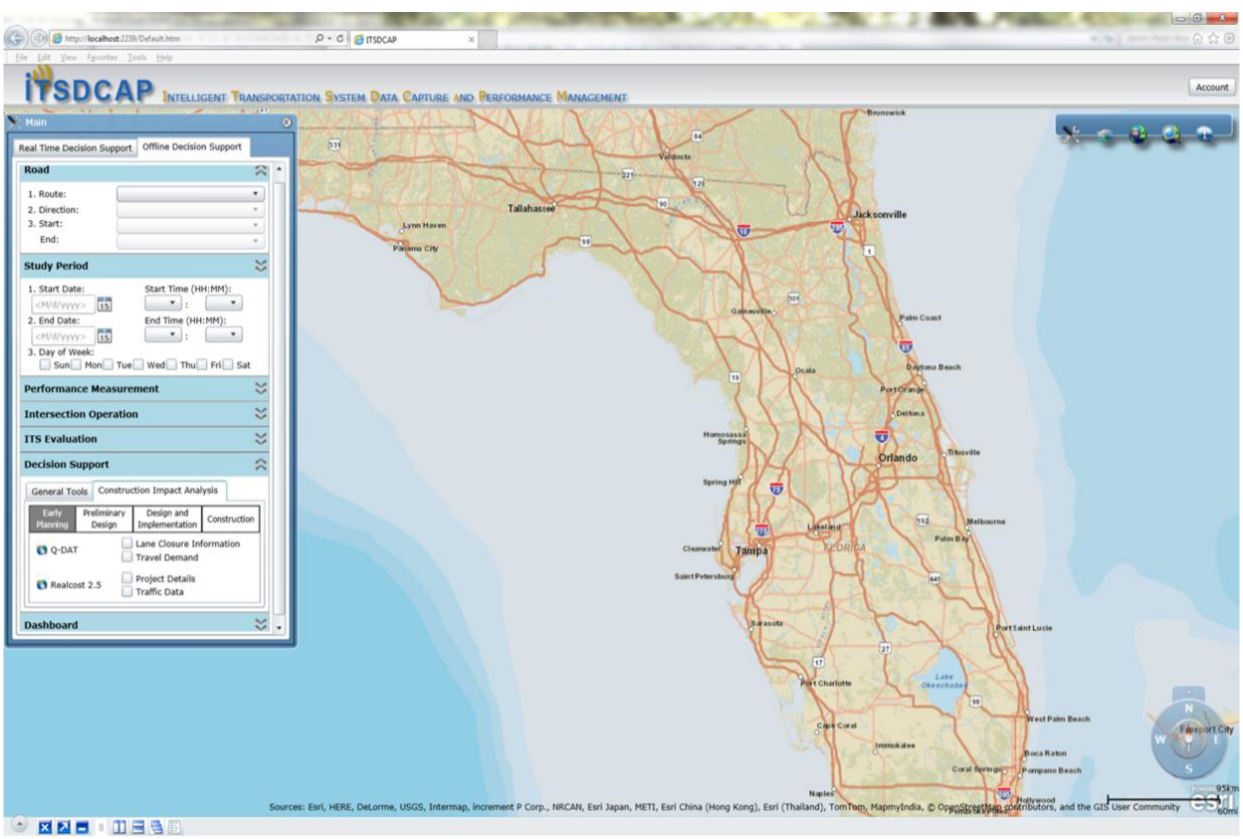


Figure 5-2 High Level Interface of Work Zone Impact Analysis in ITSDCAP

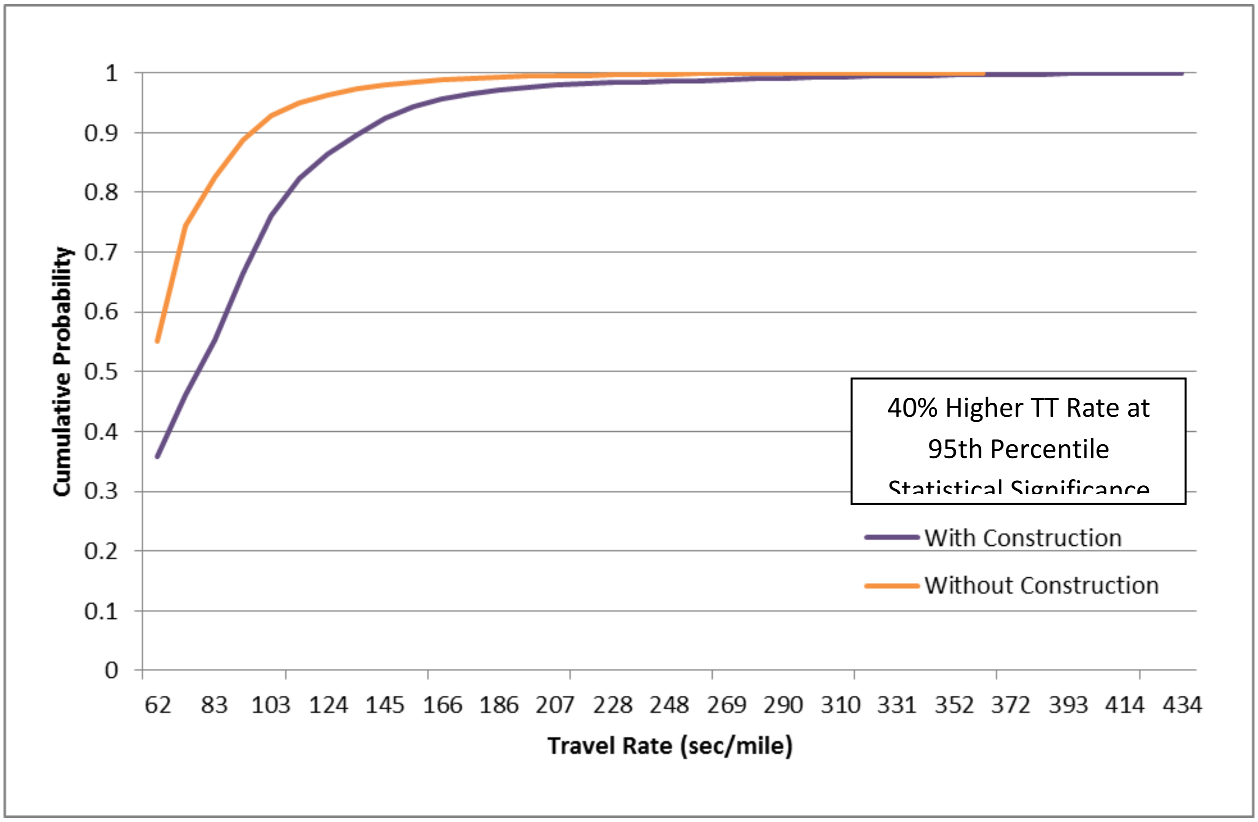


Figure 5-3 Example Output of ITSDCAP for Work Zone Impact Analysis

5.4.1 Construction Zone Case Study

A case study was conducted in this project to examine the impacts of the construction zone based on real-world data using ITSDCAP. In this case study, construction work along the SR 826 northbound was considered. Figure 5-4 shows the location of the construction, and Table 5-2 lists the construction information, including the time, location, lane blockage and descriptions. As shown in Table 5-2, the purpose of the construction is hydroblasting and restriping. This construction was conducted between 11:00 p.m. and 5:00 a.m. on four consecutive days with lanes closed during the night.

In ITSDCAP, when a user clicks the "Construction Support" button located under the Decision Support Module and selects the construction location, as shown in Figure 5-4, the panel of construction impacts will be shown. Figures 5-5 through 5-8 present the corresponding outputs from ITSDCAP. Figure 5-5 shows a snapshot of the basic construction information tab, which lists the construction location, duration, and lane blockage information. Figures 5-6 to 5-8 present the changes in traffic conditions 30 days before the construction, during the construction, and 30 days after the construction. As shown in Figure 5-6, the speeds during the night were decreased from more than 60 mph before the construction began, to around 50 mph during the construction. After the construction, the normal speed was recovered; however, the average value was about 2 or 3 mph lower than the value before the construction. The variation in the 5-minute volume count shown in Figure 5-7 reveals that the existence of construction during the night reduced the traffic throughput about 20 to 40 vehicles per 5 minutes, or, 240 to 480 vehicles per hour. It is also noted that the traffic volumes were almost the same for most of the day before, during, and after the construction. The corresponding changes in occupancy for this case study are presented in Figure 5-8. It can be seen from this figure that the occupancy during the construction was slightly increased; however, such increase is not significant due to the low volume at night. During the daytime, the occupancy values were consistent in the AM peak, with and without construction. There was a 2% increase in occupancy during the midday and PM peak periods during and after construction, compared to the before construction conditions.

Based on the above analysis, it can be concluded that this construction event causes a slight delay in traffic; however, its impacts are not severe. Such results can be used as a reference for agencies to plan for future construction activities. This construction zone case study also demonstrates the ability of ITSDCAP to support construction analysis.

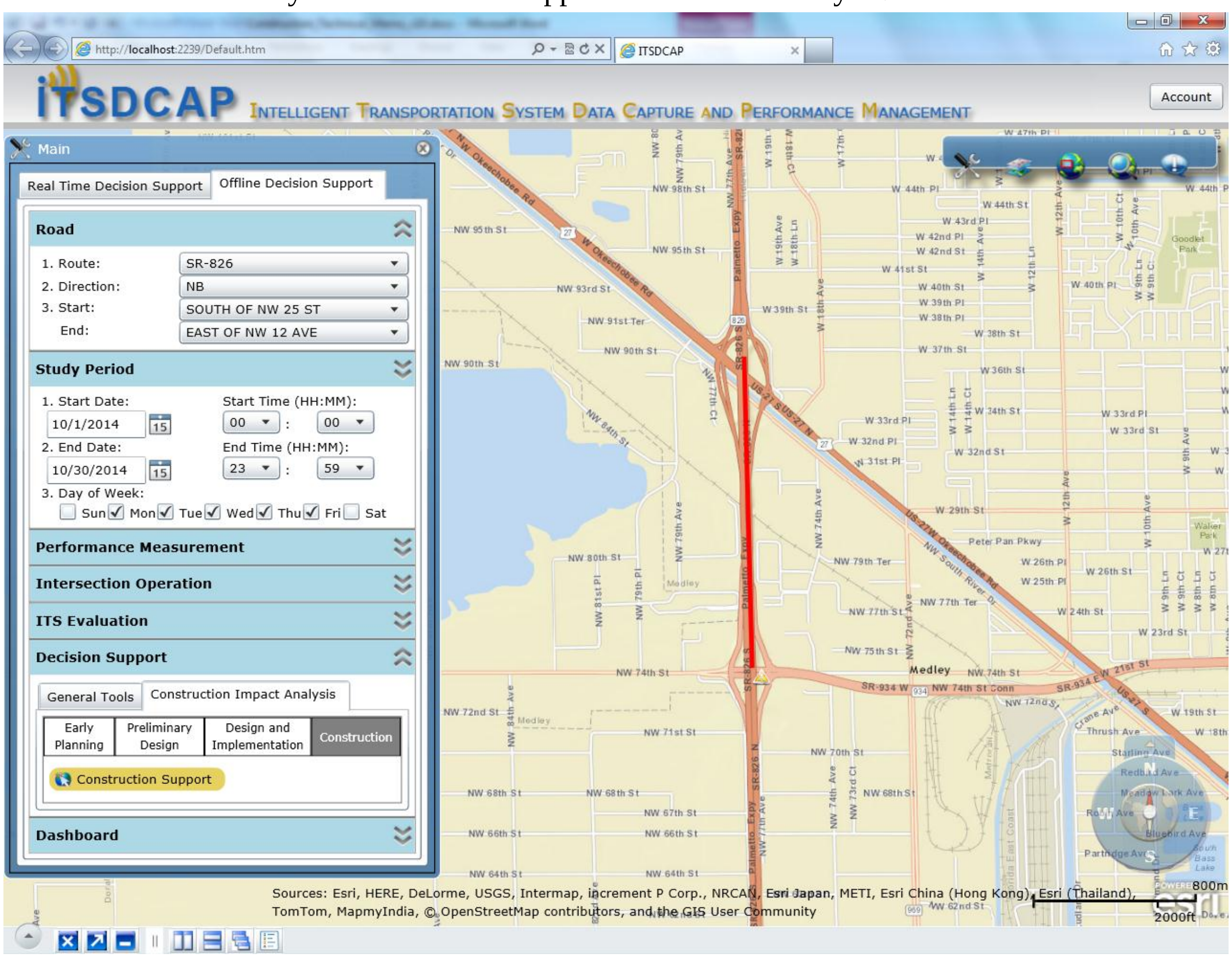


Figure 5-4 Location of Construction along SR 826 in Case Study

Table 5-2 Construction Information in Case Study

Construction ID	**7315**
Time	From 10/26/2014 11:00 PM To 10/29/2014 05:00 AM
Location	From NW 74th Street (25.840898,-80.322032) To Okeechobee Boulevard (25.854947,-80.322434)
Location Description	Complete Detour of northbound Palmetto from NW 74th Street to Okeechobee Boulevard in order to hydroblast and restripe for Phase 1A maintenance of traffic in this area.
Description	Hydroblasting and restriping of the Palmetto along the northbound lanes of Traffic from NW 74th Street to Okeechobee Blvd. to switch into Phase 1A MOT.

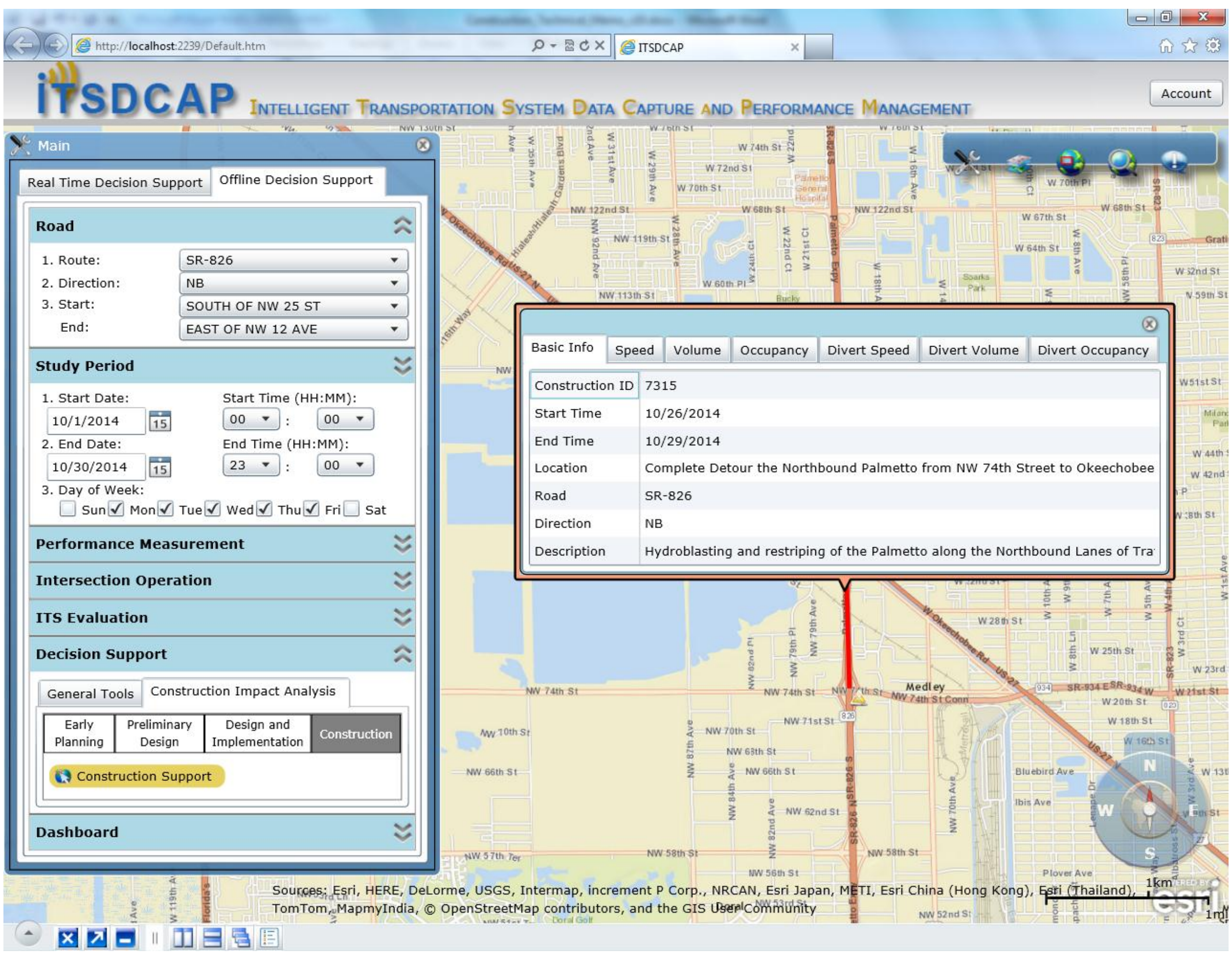


Figure 5-5 Snapshot of Basic Info Tab in ITSDCAP Construction Case Study

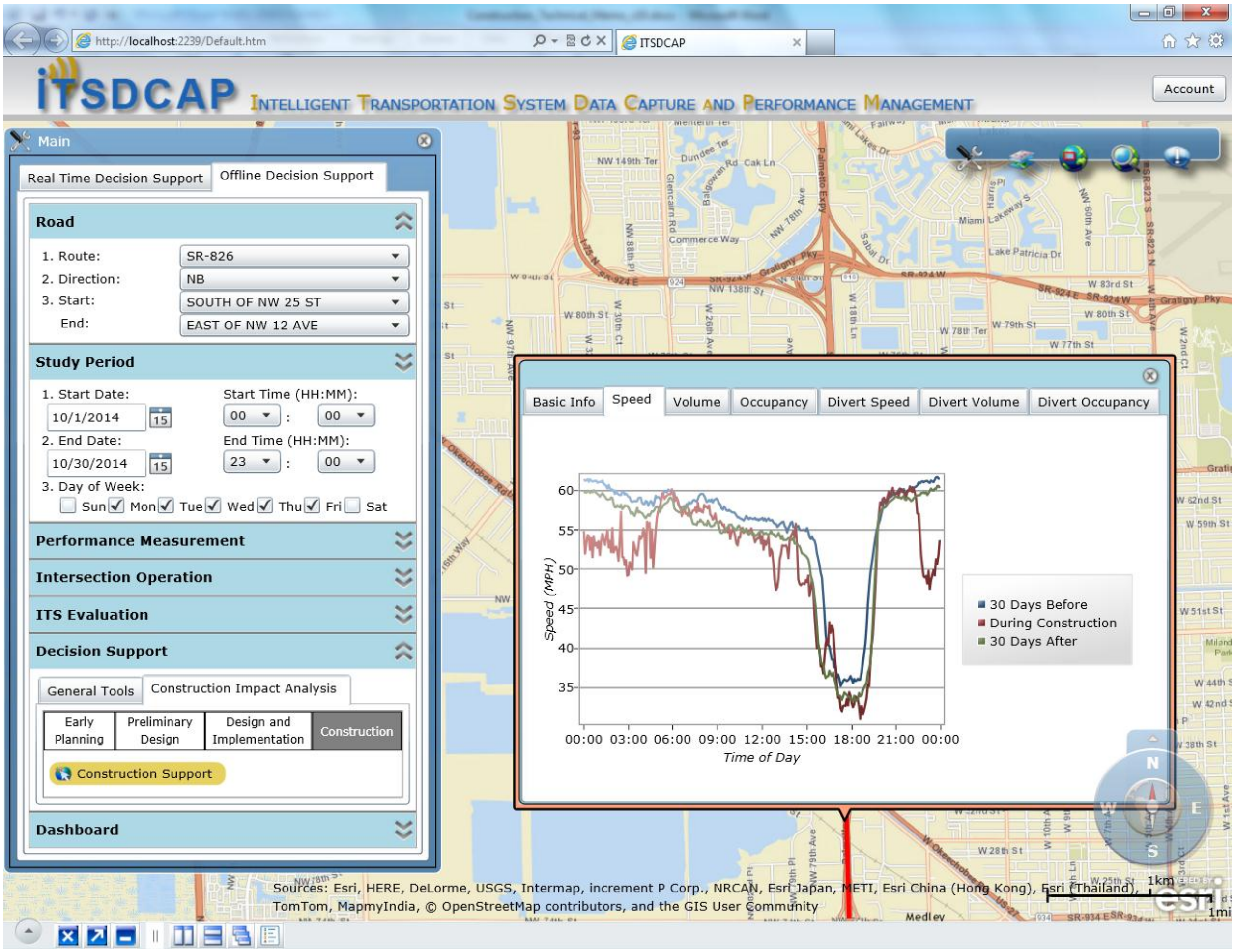


Figure 5-6 Snapshot of Speed Tab in ITSDCAP Construction Case Study

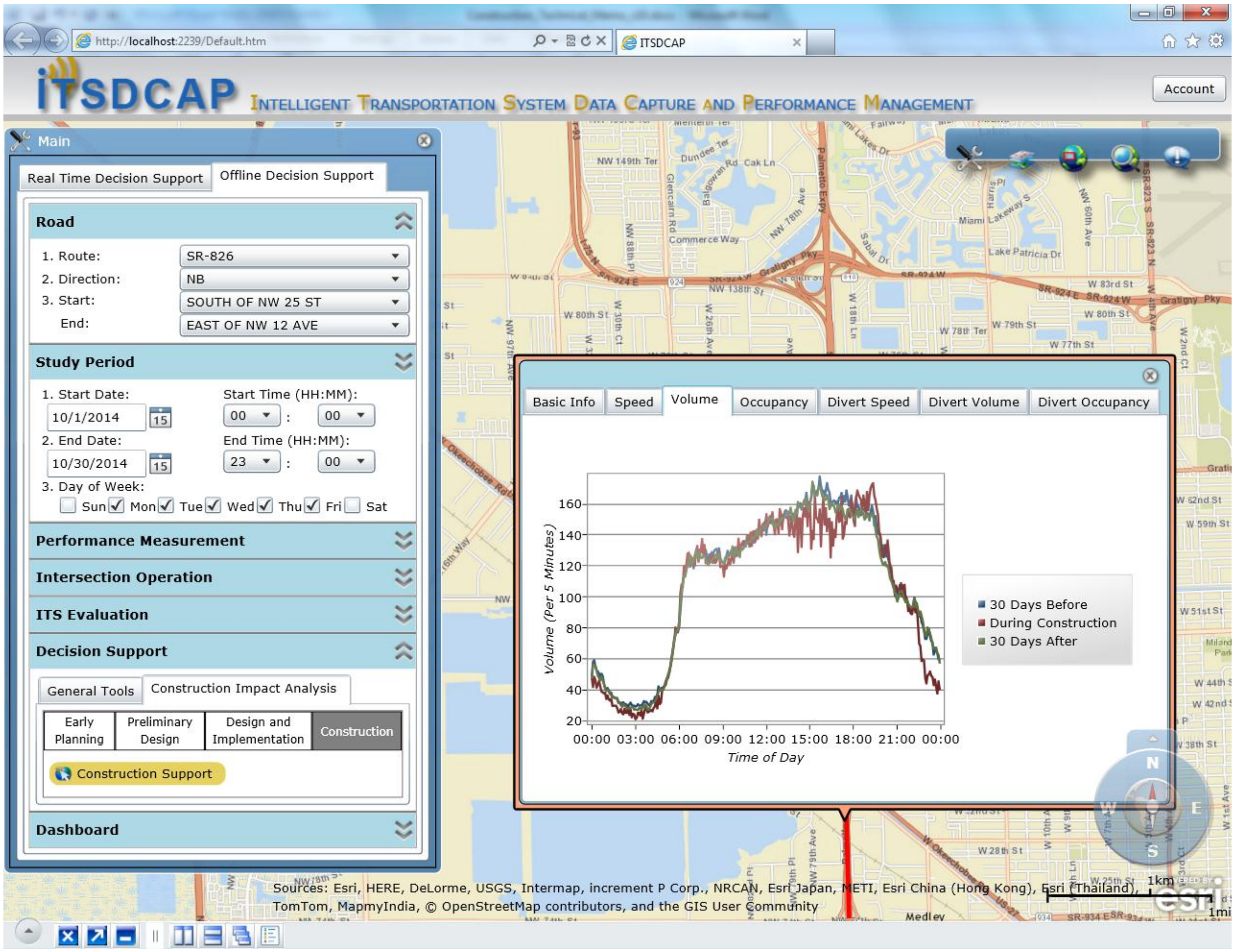


Figure 5-7 Snapshot of Volume Tab in ITSDCAP Construction Case Study

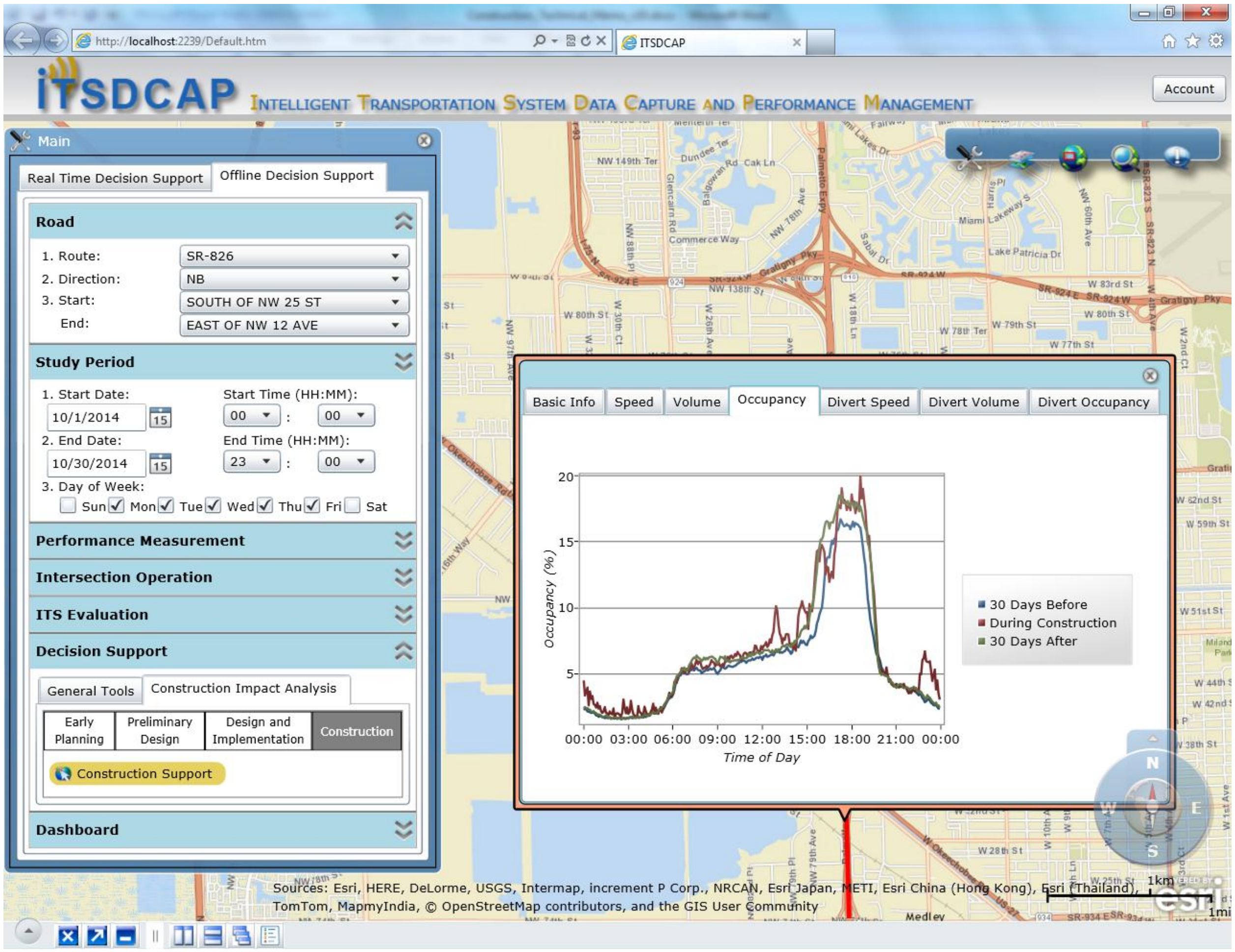


Figure 5-8 Snapshot of Occupancy Tab in ITSDCAP Construction Case Study

6 SIGNAL DIAGNOSIS

6.1 Introduction

Collecting detailed traffic data from multiple sources for signalized arterial streets are increasingly being considered and done by transportation agencies. However, there is a limited effort on the use of such data for better management of these streets in current traffic signal practices (National Transportation Operations Coalition, 2012). To retime a signal, agencies often use one to three days' turning movement counts combined with three to seven day's tube counts. These data are aggregated into 15-minute bins, adjusted, averaged, and input to signal timing optimization software. Developing signal timing plans based on such aggregated data for only a few days may lead to inaccurate or biased signal operations at the intersection (Bullock et al., 2014). On the other hand, with the emergence of ITS detection technologies on arterials, such as AVI technologies (e.g., Bluetooth readers, Wi-Fi readers, vehicle signature matching based on magnetometers), point detectors, and advanced control systems; detailed data are becoming available that can be utilized to support traffic control decisions but as stated above such utilization has been limited.

This section describes an initial effort conducted in this study to develop a signal timing diagnostic system that use a combination of existing relatively-low-cost data from Wi-Fi or Bluetooth readers combined with data from existing signal controllers to provide information for diagnosing signal operations. This initial development is discussed in Chapter 6 and will be extended in future efforts.

6.2 Literature Review

Even though automated traffic data collection, archiving, and utilization is not a new concept, most related efforts have been made for freeway systems rather than arterials. Several researches investigated the collection and analyzing of data to extract traffic signal performance measures. For instance, the arterial PeMS, or A-PeMS, (Petty and Barkley, 2011) adopted similar concepts to those for freeway performance measures, originally included in the PeMS system (Chen, 2002). A-PeMS is a Web-based system but was developed to automate processes for data collection and processing on the arterials (Petty and Barkley, 2011).

On the other hand, a system named Traffic Signal Performance Monitoring System (TSPMS) (Balke et al., 2005) was developed to assist Texas Department of Transportation (TxDOT) in automatically estimating performance measures including cycle time, time to service, queue service time, duration of the green, yellow, all-red and red interval for each phase, number of vehicles entering the intersection during each interval, yellow and all-red violation rates, and phase failure rate.

The TSPMS uses a Traffic Controller Interface Device (CID) to receive the electric signals from the traffic signal control system, which is, in this case, the Eagle® EPAC 300 controller. A Traffic Signal Event Recorder (TSER) is set up in TSPMS to record the status changes of various outputs from the traffic signal controller and the traffic detector according to the electric signals received by the CID. TSER also stores the time at which the changes occurred. Finally, a Performance Measure Report Generator (PMRG) is developed to calculate the mentioned performance measures based on the daily log files created by the TSER, which stores the changes in status and the time of change.

The American Association of State Highway and Transportation Officials (AASHTO) in cooperation with the Utah Department of Transportation (UDOT), the Indiana Department of Transportation (INDOT), and the Minnesota Department of Transportation (MNDOT) developed an automated traffic Signal Performance Measures (SPMs) system to provide automated signal performance metrics that show the real-time and historical performance measures at signalized intersections (UDOT, 2015). The metrics, including approach delay, approach volume, arrivals on red, Purdue coordination diagram (PCD), Purdue phase termination, speed, split monitor, and turning movement counts, are implemented in an online tool. The online tool evaluates the quality of signal control and progression of traffic, and identifies detector malfunctions, vehicle delays, speeds, and travel times. The development of the UDOT's Signal Performance Metrics is based on several previous studies, which have been conducted for INDOT to investigate new performance measures that can depict flow rates, quality of coordination, and split failures, with traffic signal controller vendors from three vendors: Econolite®, Siemens®, and Peek® (Smaglik et al., 2007; Li et al., 2013; Day et al., 2009).

One of the signal performance studies automated the required data collection by utilizing an enhanced NEMA controller which is able to record the detector records and phase state changes (Smaglik et al., 2007). These event-based data were used to provide quantitative graphs for the purpose of assessing progression and intersection delay. Another research used a system engineering approach to identify several objectives of signal control support including reliable communication to signal systems, good allocation of green times, and good progression (Li et al., 2013). As part of that research, the authors analyzed the information provided regarding phase force-offs and gap-outs to identify the potential opportunities for reallocating the split time. In addition, the values of offset in the coordinated traffic signal system were assessed using the PCD which can be constructed from the high-resolution data that record every vehicle's arrival time at the intersection (Day et al., 2009). The arrival pattern, e.g., platoon arrival on green, platoon arrival on red, random arrivals, etc., can be visually represented in the PCD. With the high-resolution data, the percent of vehicle arriving on green (POG) can be calculated simply by dividing the total number of vehicle arriving on green by the approach volume. The above study was able to improve the offset by finding the maximum value of the total number of vehicles arriving on green in the concurrent phases.

A systematic approach named SMART-SIGNAL (Systematic Monitoring of Arterial Road Traffic and Signals) was proposed for data collection and performance monitoring of closed loop signal control systems, with similar architecture as the TSPMS mentioned earlier (Ma, 2008; Liu et al., 2008). The approach also presented algorithms for queue length and turning movement proportion (TMP) estimations by combining mathematical models and the high resolution data collected by the SMART-SIGNAL system. Additionally, as part of the study, virtual probe vehicles were traced and one of three possible maneuvers: acceleration, deceleration, and no-speed-change, were predicted based on the current traffic states of the virtual probe with decision-tree technique. The aggregated statuses of virtual probe vehicles were used to estimate time-dependent arterial travel times and other performance measures, such as delay and number of stops.

In summary, a number of studies have been conducted on the estimation of performance measures utilizing high-resolution sensor and signal control data. However, collecting high resolution data requires hardware and software updates that are not always feasible for existing intersection control systems. Instead of utilizing this data, this study investigates the use of a combination of relatively-low-cost Wi-Fi or Bluetooth readers and current signal controllers to provide information for diagnosing signal operations.

6.3 Utilized Data

The developed method in this study uses the Acyclica® Wi-Fi readers but any AVI readers can be used. The used Wi-Fi readers were fit in signal controller cabinets, as shown in Figure 6-1. The readers record the Wi-Fi MAC addresses of the devices in its detection range radius and the associated reading time. Devices that have their Wi-Fi function on, such as, cell phones, laptops, and so on, are continuously detected as long as they are within the detection range of the Wi-Fi readers. The strengths of the received Wi-Fi signals are also recorded by the detectors. The strengths increase with the decrease of the distance between the devices and the Wi-Fi readers.

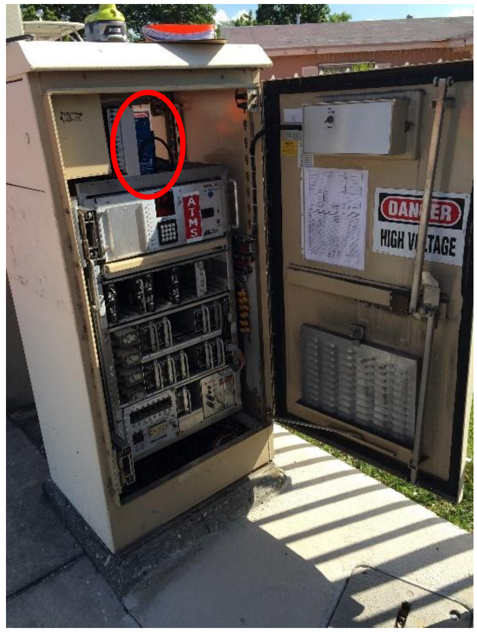


Figure 6-1 Wi-Fi Detectors Installed inside the Traffic Signal Controller Cabinet

The installed Wi-Fi readers are capable to detect the nearby activated Wi-Fi devices at a frequency of one detection per second. With the application of filtering and matching algorithms, each vehicle with an activated Wi-Fi device can be identified at different locations relative to the intersection, at which the device is installed, due to the uniqueness of the MAC addresses. An example is shown in Table 6-1. The Wi-Fi detectors assign an encrypted MAC address to each identified Wi-Fi device and log every detection with the strength of the received Wi-Fi signal. In this analysis, four stages of the Wi-Fi detection were identified and used in the analysis:

1) The first detection: the timestamp when the device was first detected;
2) The last detection: the timestamp when the device was last detected;
3) The maximum strength detection: the timestamp when the detection strength first reach the maximum value; and
4) Other detection: all timestamps other than the above three.

Combination of those four detection stages can be applied for different purposes. Details will be given in a later section.

Table 6-1 Wi-Fi Detection Results Example

Timestamp	MAC Hash	Strength	Serial
1436155201	58ea79aa3d32da928a66ee4e19eb8f78575ddbc469bae4ac5d05e5d3f5a58029	-75	265375
1436155202	39426c523a9ec6e4fd28a364be04c737bd46c1c87b80d14b2ce85a9481405890	-72	265375
1436155202	39426c523a9ec6e4fd28a364be04c737bd46c1c87b80d14b2ce85a9481405890	-73	265375
1436155204	20adde547afec4b25a6ff4162725668b49d05fd662db16416c8fc9a7d71dfe2b	-74	265375
1436155204	20adde547afec4b25a6ff4162725668b49d05fd662db16416c8fc9a7d71dfe2b	-74	265375

In this study, the intersection of Southwest 8th Street at Southwest 107th Avenue was selected as the target intersection for the initial development and testing of the proposed diagnosis system. Five Wi-Fi detectors were installed at that intersection and four adjacent signalized intersections: 1) Southwest 8th Street at Southwest 109th Avenue; 2) Southwest 8th Street at Southwest 102th Avenue; 3) Southwest 4th Street at Southwest 107th Avenue; and 4) Southwest 1100th Block at Southwest 107th Avenue. An illustration of the studied location is shown in Figure 6-2. The studied time periods were from 3:00 PM to 8:00 PM during weekdays.

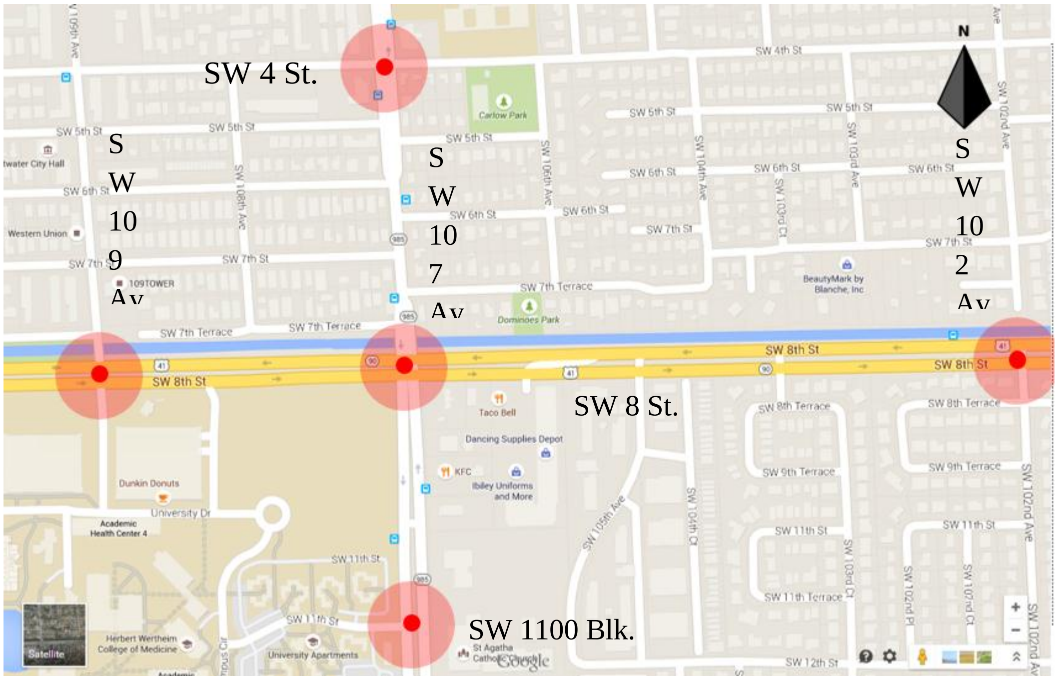


Figure 6-2 Wi-Fi Detector Locations at the Studied Intersection of Southwest 8th Street at Southwest 107th Avenue and Its Four Surrounding Intersections. (Background image source: Map data © 2015 Google)

Besides of the Wi-Fi detection data, historical intersection timing data were obtained from the Miami-Dade County signal system. The data were downloaded from the Integrated Transportation System (KITS®), which is the system used for signal control in the region. Although the data were not in high-resolution and no detector data were recorded, the times at which signal indications changed were logged in the archives data. An example is shown in Figure 6-3.

Historical Intersection Timing Report   *Int: SW 107 Av&SW 8 St (3709)*

Start Time: 4/02/15 07:00   *End Time: 4/02/15 09:00*

Print Date 04/02/2015   Print Time 2:40 PM

Time	Plan	Ring 1 Phase	Interval	Grn Dur	Ring 2 Phase	Interval	Grn Dur	Status	Poll Freq
4/02 07:00:52	3	2-WBT	Clear		6-EBT	Clear		Coord	
4/02 07:00:58		3-SBL	Green	11	7-NBL	Green	11		
4/02 07:01:09		3-SBL	Clear		7-NBL	Clear			
4/02 07:01:12		4-NBT	Green	68	8-SBT	Green	68		
4/02 07:02:20	3	4-NBT	Clear		8-SBT	Clear		Coord	
4/02 07:02:27		1-EBL	Green	21	5-WBL	Green	14		
4/02 07:02:41		1-EBL			5-WBL	Clear			
4/02 07:02:46		1-EBL			6-EBT	Green	66		

Figure 6-3 Example of KITS® Historical Intersection Timing Report

6.4 Developed Methodology

In order to obtain the travel time information, raw data in the format shown in Table 6-1 were first filtered and matched between the Wi-Fi detectors at the target intersection (Southwest 8th Street at Southwest 107th Avenue) and its four surrounding signalized intersections, as shown in Figure 6-4. A total of eight sets of matches were identified corresponding to the movements shown in the figure. The Wi-Fi central software has its own filtering method to ensure that the Wi-Fi signals included in the database are for vehicles, isolating out other Wi-Fi signals. An extra filtering rule was applied in this study that if a MAC address at the target intersection has the same MAC address at one of the nearby intersections, and the time difference between the detections at the two intersections was not longer than 10 minutes, then the two data points were matched and the associated trip was identified. The purpose of adding the 10 minute time threshold to the matching algorithm was to isolate out multiple trips conducted by the same vehicle within a certain time period.

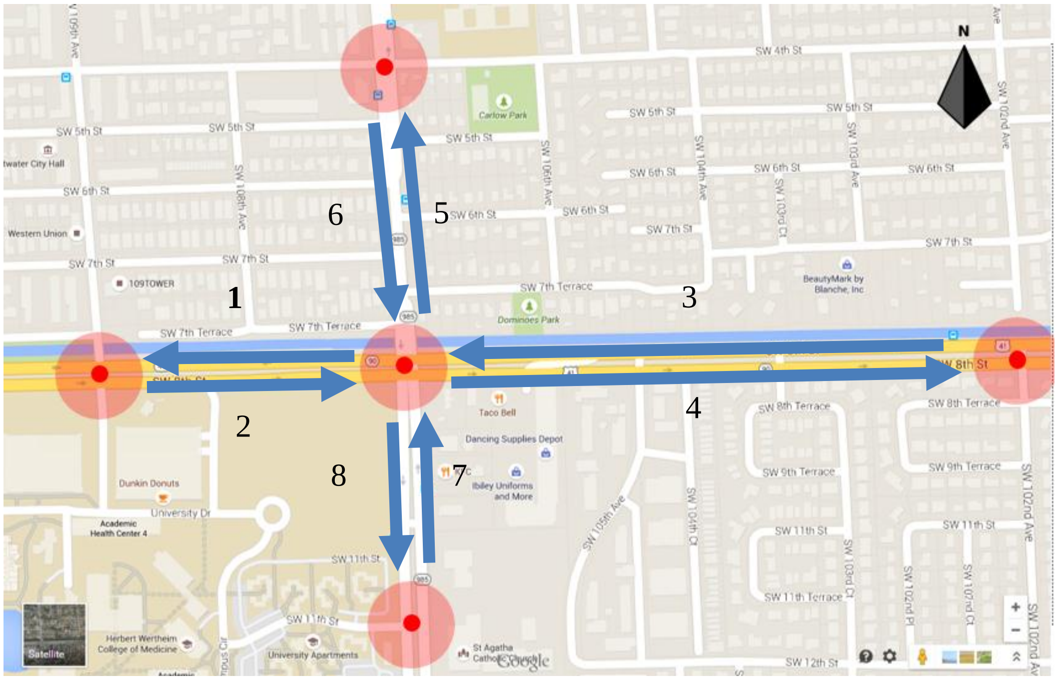


Figure 6-4 Data Matching between Target Intersection and Its Surrounding Intersections

As can be seen from Figure 6-4, all matched sets end or begin at the target intersection of Southwest 8th Street at Southwest 107th Avenue. Therefore, detected vehicles can be associated with turning movements at that intersection can be identified by matching the detected MAC addresses between an approaching link and a departing link of the target intersection. In this study, the maximum strength detection was utilized in the matching to identify the turning movements. For example, as shown in Figure 6-4, if a MAC address and the time stamp of its maximum strength detection at the intersection of Southwest 8th Street at Southwest 107th Avenue in the matching set 2 are exactly the same as those at the same intersection in the matching set 5, this information from the two data sets is matched indicating an eastbound left-turn movement set at the target intersection.

With the association of vehicles with turning movements, it was possible to calculate the travel time for each movement. The calculated travel time for each turning movement can be used to produce the Cumulative Density Function (CDF) plots, which is a visualization and analysis method extensively applied in the SHRP 2 L02 project for travel time reliability analysis. A set of CDF plots, which were produced with the February weekday data during the studied period, are presented in Figure 6-5 and Figure 6-6, for the purpose of evaluating vehicle travel times at the target intersection.

As a rule of thumb, the closer the CDF curves is to the top left corner, the better the travel time performance is. On the contrary, travel time performance worsens as the curve approaches the lower right corner. As an example, Figure 6-5 and Figure 6-6 show that the travel time performance was bad in the time period from 3:00 PM to 4:00 PM (black dots) for the eastbound left-turn, eastbound through, eastbound right-turn, westbound right-turn, and northbound through movements.

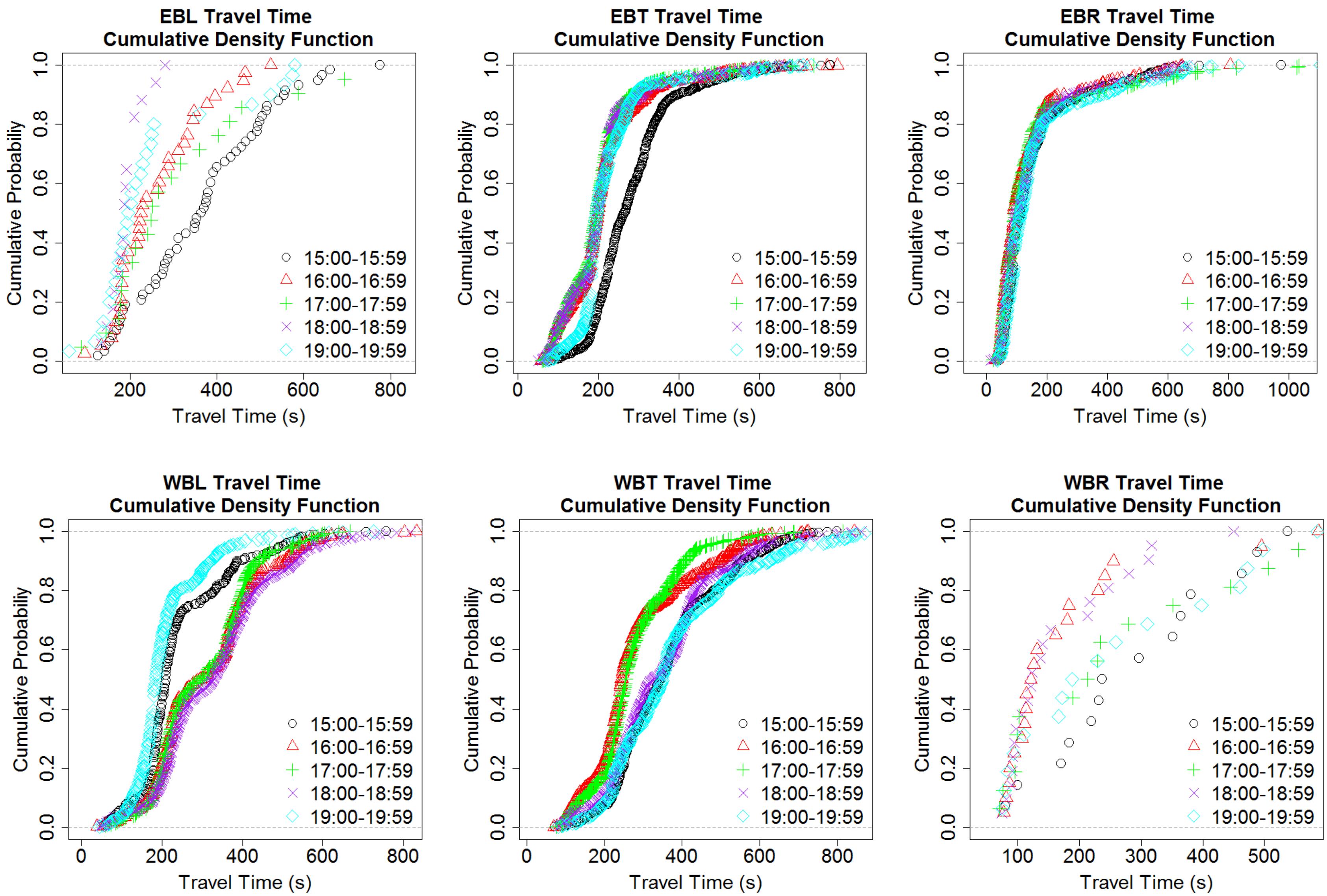


Figure 6-5 Cumulative Density Functions by Time Period for the Eastbound and Westbound Approaches

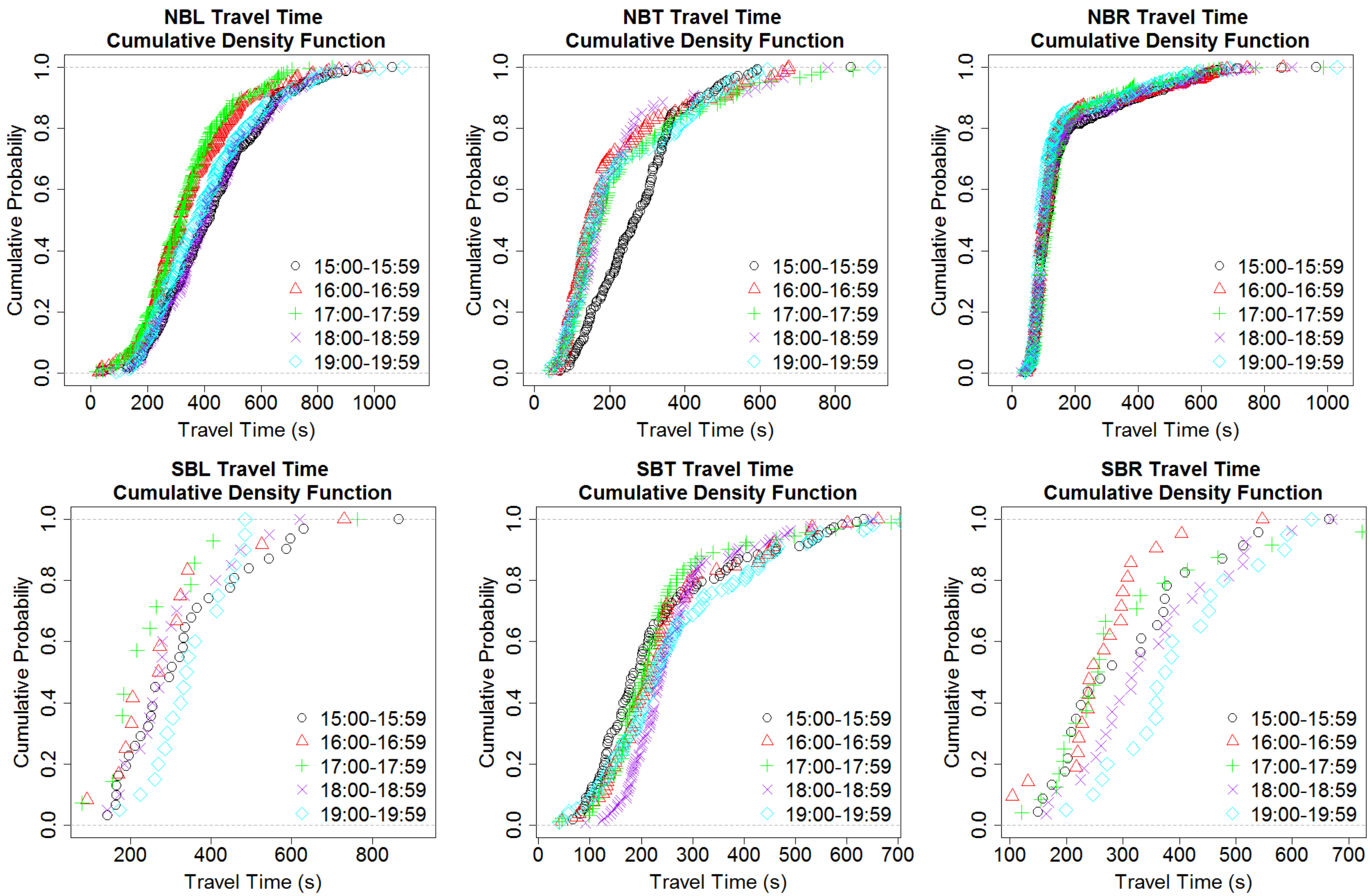


Figure 6-6 Cumulative Density Functions by Time Period for the Northbound and Southbound Approaches

To explore this further, the historical Time-of-Day (TOD) signal timing schedule at the target intersection is presented in

Current TOD Schedule	Plan	Cycle	Green Time 1 EBL	2 WBT	3 SBL	4 NBT	5 WBL	6 EBT	7 NBL	8 SBT	Ring Offset	Offset
	Free											
0500	Free											
0530	2	120	12	30	7	50	12	30	7	50	0	91
0630	3	180	28	52	11	68	19	61	22	57	0	50
0930	24	140	16	36	11	56	16	36	11	56	0	42
1400	9	150	18	41	14	56	18	41	20	50	0	130
1500	11	160	17	39	12	71	20	36	20	63	0	103
1600	12	180	18	50	12	79	15	53	24	67	0	103
1900	13	150	20	36	9	64	20	36	15	58	0	98
2030	14	135	23	26	10	55	18	31	10	55	0	134
2130	15	120	15	32	6	46	14	33	8	44	0	9
2230	16	110	14	34	5	36	15	33	5	36	0	9

Figure **6-7**. The figure shows that the time period from 3:00 PM to 4:00 PM during weekdays had a separate timing plan. Therefore, a possible inference can be made that there is a potential for improvements the signal plan assigned to the time period from 3:00 PM to 4:00 PM in the weekdays, particularly that this period is on the shoulder of the peak at which the intersection is not anticipated to be oversaturated.

Current TOD Schedule	Plan	Cycle	Green Time 1 EBL	2 WBT	3 SBL	4 NBT	5 WBL	6 EBT	7 NBL	8 SBT	Ring Offset	Offset
	Free											
0500	Free											
0530	2	120	12	30	7	50	12	30	7	50	0	91
0630	3	180	28	52	11	68	19	61	22	57	0	50
0930	24	140	16	36	11	56	16	36	11	56	0	42
1400	9	150	18	41	14	56	18	41	20	50	0	130
1500	11	160	17	39	12	71	20	36	20	63	0	103
1600	12	180	18	50	12	79	15	53	24	67	0	103
1900	13	150	20	36	9	64	20	36	15	58	0	98
2030	14	135	23	26	10	55	18	31	10	55	0	134
2130	15	120	15	32	6	46	14	33	8	44	0	9
2230	16	110	14	34	5	36	15	33	5	36	0	9

Figure 6-7 Time-of-Day Signal Timing Plan Schedule Report for the Intersection of Southwest 8th Street at Southwest 107th Avenue

In order to further analyze the data and check the potential of improvements for the signal timing plan from 3:00 PM to 4:00 PM, CDF plots for all movements during that time period were generated in Figure 6-8. Since this plot compares travel time of different movements, the travel times need to be normalized to account for vehicles traveling different distances. Instead of travel rate in seconds per mile, the Travel Time Index (*TTI*) is used as an input to the CDF plots. *TTI* is the ratio of the actual travel time to the travel time at free flow condition. In this study, the fifteen percentile of the matched travel times for each turning movement in all weekdays of February was used as the free flow travel time of that movement.

Figure 6-8 shows that there are differences in the *TTI* performances among different turning movements. For example, the northbound left-turn and northbound through movements had the worst *TTI* performance. Southbound right-turn movement also did not perform well according to the plots, but it may due to fact that the right-turn lane was shared with through movements. The westbound left-turn, eastbound left-turn, and eastbound through movements had significantly better *TTI* performance compared to the other non-right-turn movements. It is worth pointing out that the westbound through movement had much worse *TTI* performance compared to the eastbound through movement. The above discussion indicates the potential need for improving the assignment of splits in the plan implemented between 3:00 PM and 4:00 PM.

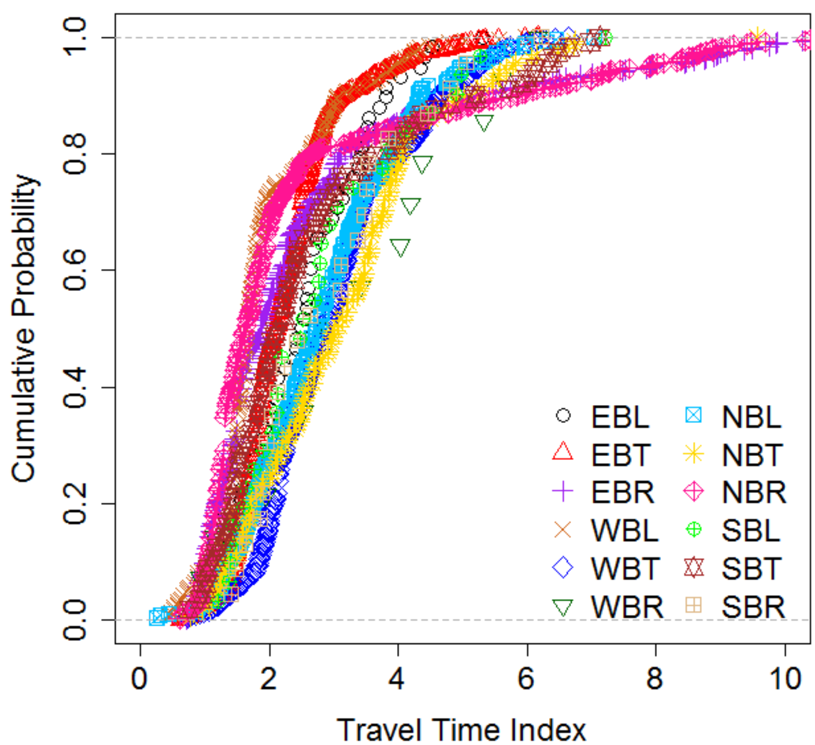


Figure 6-8 Cumulative Density Functions for All Movements between 15:00 to 15:59

Figure 6-9 shows the CDF plot for all movements between 4:00 PM to 5:00 PM. A comparison between the *TTI*s from 3:00 PM and 4:00 PM and the *TTI*s from 4:00 PM to 5:00 PM area in are presented in Figure 6-10. As can be seen from Figure 6-10, the area outlined by grey lines, representing the boundary of the *TTI* area between 4:00 PM to 5:00 PM, were closer to the top left corner of the chart, compared to the area depicted by the red lines representing the outline of the *TTI* area between 3:00 PM to 4:00 PM. Thus, the target intersection seems to operate better in the time period between 4:00 PM and 5:00 PM than in the time period of 3:00 PM to 4:00 PM. This is despite that the demands are higher between 4:00 PM and 5:00 PM, further indicating that the timing plan is inferior between 3:00 and 4:00 PM. The CDF plots for all movements in other study hours are presented from Figure 6-11 to Figure 6-13 respectively.

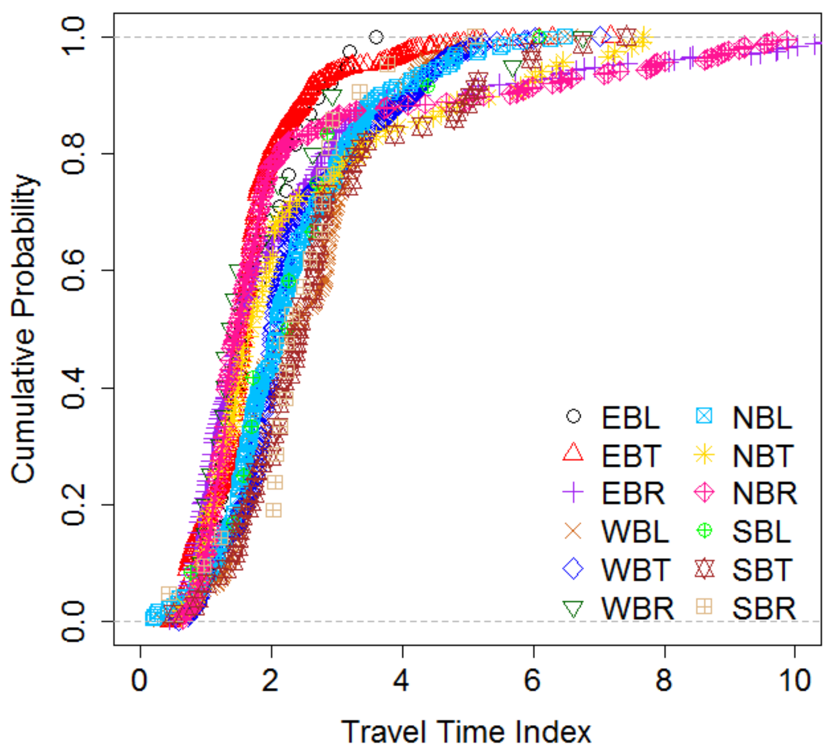


Figure 6-9 Cumulative Density Functions for All Movements between 16:00 to 16:59

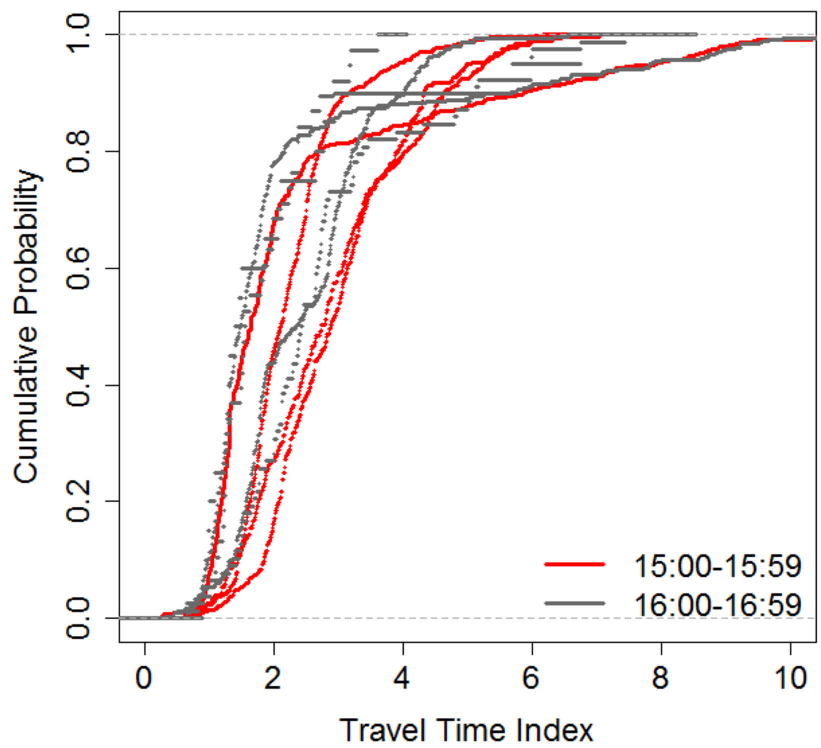


Figure 6-10 Comparison of Cumulative Density Functions between 15:00 to 15:59 and 16:00 to 16:59

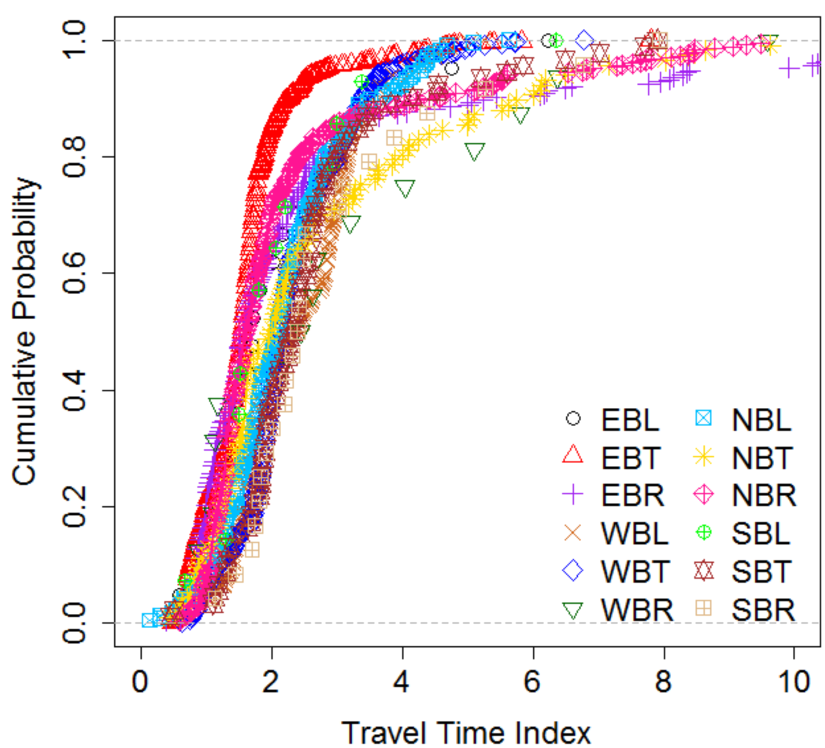


Figure 6-11 Cumulative Density Functions for All Movements between 17:00 to 17:59

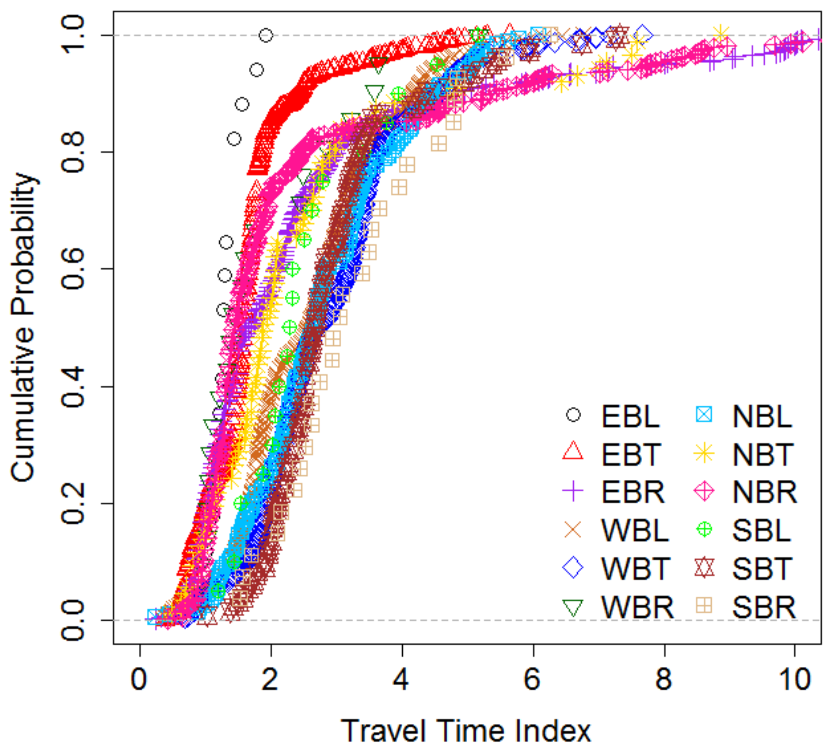


Figure 6-12 Cumulative Density Functions for All Movements between 18:00 to 18:59

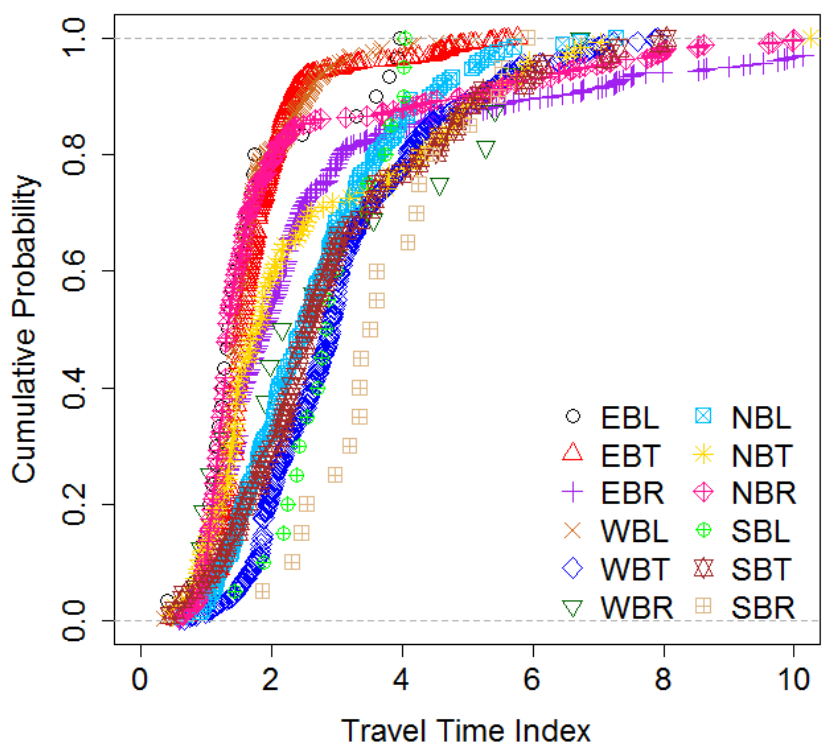


Figure 6-13 Cumulative Density Functions for All Movements between 19:00 to 19:59

It is useful to compare the *TTI* performances across various turning movements is to investigate the shape of the CDF functions shown in Figure 6-9 to Figure 6-13 for different movements. In terms of equity, the CDF curve for different movements are expected to cluster together, at least for the minor movements. The more a movement's CDF curves are distant from another, the more different the *TTI* performances are between these movements. For example, in Figure 6-11, all movements' CDF functions were relatively closer to each other compared to those in Figure 6-12 and Figure 6-13. It indicates that the *TTI* performances for different movements from 5:00 PM to 6:00 PM are closer together than the *TTI* performances from 6:00 PM to 8:00 PM, particularly from 6:00 PM to 7:00 PM.

Besides the figures discussed above, the *TTI* values by movements are presented in

Table **6-2**. Additional analyses can be done using the combinations of the 50th, 80th, and 95th percentiles of the *TTI*s. These analyses can be helpful in automating the process of investigating the patterns or relationships in the data. For example, if the 50th, 80th, and 95th percentile *TTI*s of a movement are higher than the corresponding values for other movements during an hour, it can be concluded that the performance of that movement is inferior to other movements all the time in that period. A different conclusion can be made, if only the 95^{th} percentile *TTI* of a movement is higher than those for other movements.

Table 6-2 50th, 80th, and 95th Percentile TTI by Movement in Different Time Periods

Hours	Percentiles	Movements											
		EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
15:00	50 Percentile	2.52	2.10	1.87	1.66	2.88	3.06	2.73	3.08	1.64	2.53	2.22	2.64
	80 Percentile	3.41	2.68	3.12	2.62	4.02	4.75	3.91	4.05	2.67	3.83	3.74	3.72
	95 Percentile	4.35	3.91	7.93	3.95	5.25	5.81	4.99	5.76	7.90	5.12	6.21	5.03
16:00	50 Percentile	1.56	1.57	1.50	2.34	2.03	1.42	2.09	1.68	1.48	2.26	2.43	2.32
	80 Percentile	2.36	1.98	2.84	3.21	3.13	2.66	3.08	3.35	2.16	2.82	3.34	2.88
	95 Percentile	3.16	3.23	7.10	4.34	4.53	5.73	4.66	6.37	7.83	5.15	5.97	3.78
17:00	50 Percentile	1.72	1.55	1.58	2.37	2.13	2.54	2.10	2.01	1.60	1.81	2.32	2.40
	80 Percentile	2.95	1.88	2.76	3.17	2.98	5.11	2.83	3.98	2.43	2.95	3.02	3.64
	95 Percentile	4.76	2.81	9.01	4.14	3.84	7.17	4.29	6.88	6.83	4.42	5.71	6.54
18:00	50 Percentile	1.28	1.58	1.67	2.58	2.73	1.41	2.66	1.89	1.45	2.30	2.68	3.06
	80 Percentile	1.44	1.86	3.00	3.40	3.63	2.83	3.82	2.96	2.46	3.49	3.34	4.46
	95 Percentile	1.80	3.22	7.98	4.60	5.01	3.64	5.07	7.08	7.80	4.57	5.40	5.37
19:00	50 Percentile	1.36	1.56	1.88	1.46	2.91	2.39	2.48	1.78	1.33	2.85	2.56	3.56
	80 Percentile	1.89	2.06	3.07	1.91	4.10	5.28	3.60	4.30	2.06	3.76	4.47	4.58
	95 Percentile	3.87	3.08	8.95	2.91	5.79	5.97	5.10	5.90	6.78	4.03	6.12	5.54

The above analysis is based solely on Wi-Fi data analyses that provide information about the vehicle travel time performance. However, additional data such as signal timing history and traffic volumes can be used to have a full picture of the system performance and influencing factors. Traffic volumes are not used in this version of the diagnostic method but will be included in a future version. Signal control history is included in the analysis as discussed next.

With the use of signal timing history, the relationship between signal timings and vehicle travel time performances can be identified, allowing better diagnosis of system performance. A study on the subject (Li et al., 2013) made an assumption that if one phase was forced-off (maxed-out) in three consecutive cycles and another phase was always gapped-out, potential changes to the maximum green times of those phases should be considered.

Combining Wi-Fi data with the corresponding historical signal timing data allows even better support of signal operation diagnosis. In this study, an additional decision support signal operation diagnosis scheme is developed for the target intersection of Southwest 8th Street at Southwest 107th Avenue. The proposed diagnosis scheme is presented in Figure 6-14. The scheme diagnoses the signal plan for the whole intersection first and then inspects the individual splits, as necessary. Before inspecting the individual splits, it is essential to investigate the signal plan for the whole intersection to determine if the whole plan is adequate to support the demands. If the plan is not adequate, there may be a need to increase the signal cycle length. If the cycle length is already high and the performance of all movements are bad with signal timing maxing outs on all phases, this may indicate the need for geometry (capacity) improvements.

If the of some movements are relatively good and others are bad, this may indicates adequate geometry and cycle length but a need for split adjustment. As shown in Figure 6-14**Error! Reference source not found.**, in addition to the movement *TTI*s discussed earlier, other variables derived based on the Wi-Fi data were also considered in the analysis including maxed-out versus gapped-out per movement, number of repeated hits per vehicle per movement (indicating delayed vehicles), and the arrival on green for the coordinated phases estimated based on combining timing data and the vehicle detection time stamps. It is important to note that to derive the arrival on green parameter, there is a need to synchronize the time clock between the Wi-Fi data and the signal timing data. This was not done yet in this study, due to the difficulty in coordinating with the Wi-Fi vendor.

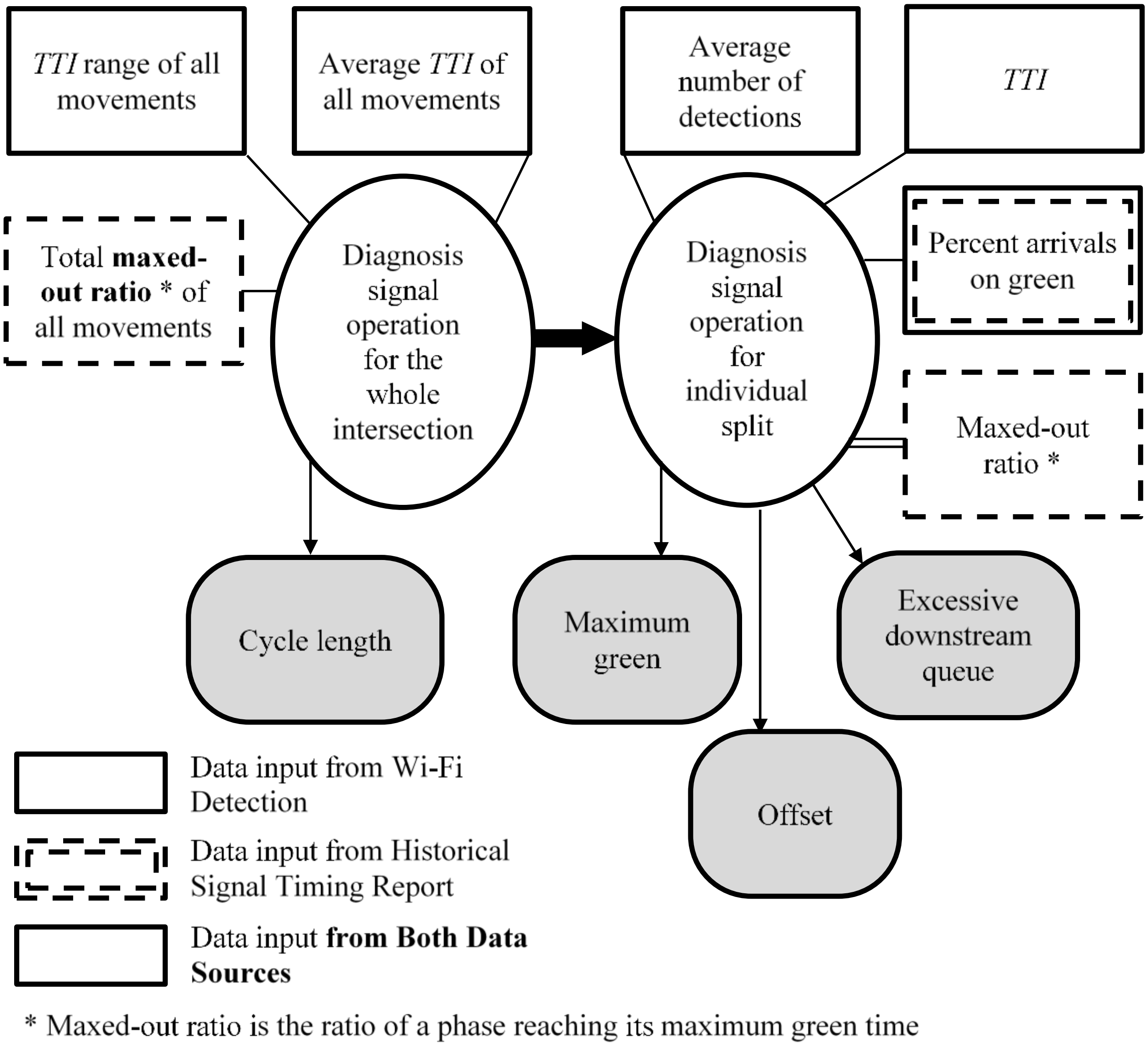


Figure 6-14 Proposed Decision Support Signal Operation Diagnosis Scheme

The first step in Figure 6-14 is to inspect the signal operation of the whole intersection. In this step, the scheme requires the *TTI* averaged by various critical movements, the range of *TTI* for these movements, and the total maxed-out ratio of the critical movements. A critical movement is defined as the movement that has the highest *TTI* among all movements served by a given phase group. For example, the target intersection in this study uses the dual-ring phasing sequence with leading left-turn phases as shown in Figure 6-15. Therefore, four critical movements are selected based on the following concurrent dual ring groups: 1) Phase 1 and Phase 5; 2) Phase 2 and Phase 6; 3) Phase 3 and Phase 7; and 4) Phase 4 and Phase 8. For example, if the eastbound left-turn movement has a higher *TTI* compared to the westbound left-turn movement, Phase 1 is considered as the critical movement in Group 1.

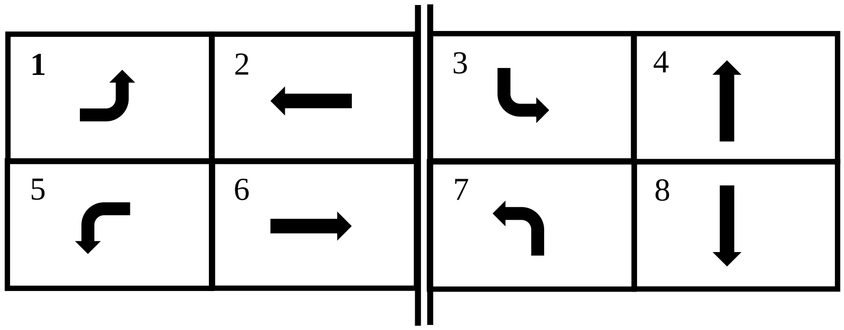


Figure 6-15 Dual-Ring-Barrier Diagram

The average *TTI* of all critical movements (*aTTI*) is the total *TTI* for all critical movements divided by the total number of movements. The *aTTI* of all critical movements reflects the general vehicle travel time performances for the whole intersection. The higher the *aTTI* is, the more congested the whole intersection is expected to be indicating the need for increasing the cycle length or improving geometry. The range of *TTI* for the critical movements (*rTTI*) is the difference between the minimum *TTI* and maximum *TTI* across these movements in the considered time period. The higher the *rTTI* is, the more different the travel times are among the critical movements in the time period, indicating potential benefits of reassigning the splits. The maxed-out ratio (*mRatio*) for a phase is the ratio of the number of the maxed-out instances of the phase in the period to the total number of the phase occurrences (which is the same as the number of cycles) in the period. For each phase, the value of *mRatio* falls between 0 and 1, where 0 indicates there is no maxed-out and 1 means the phase is always maxed-out. The total *mRatio* is the sum of the *mRatio* for all critical movements. The *aTTI* and *rTTI* were calculated based on the Wi-Fi data, as explained earlier. The *mRatio* was calculated based on the historical intersection signal timing report.

After inspecting the signal operation for the whole intersection, the diagnosis continues to inspect the individual phase splits, if the whole intersection analysis indicates that this is warranted based on the *rTTI* and *mRatio*. As shown in Figure 6-14, the individual-phase-split-level inspection requires the average number of detection, *TTI*, percent arrivals on green, and the *mRatio* as inputs. The average number of detection (*aDet*) is the sum of the detections of the MAC addresses over all identified trips during the considered time period divided by the total number of identified trips. As mentioned previously, the Wi-Fi detectors record the detection of each activated Wi-Fi device within its coverage range at a frequency of one detection per second. Thus, a higher number of detection implies a vehicle staying longer within the detectable range. In this study, the *aDet* is used as another surrogate of delay and will be discussed later.

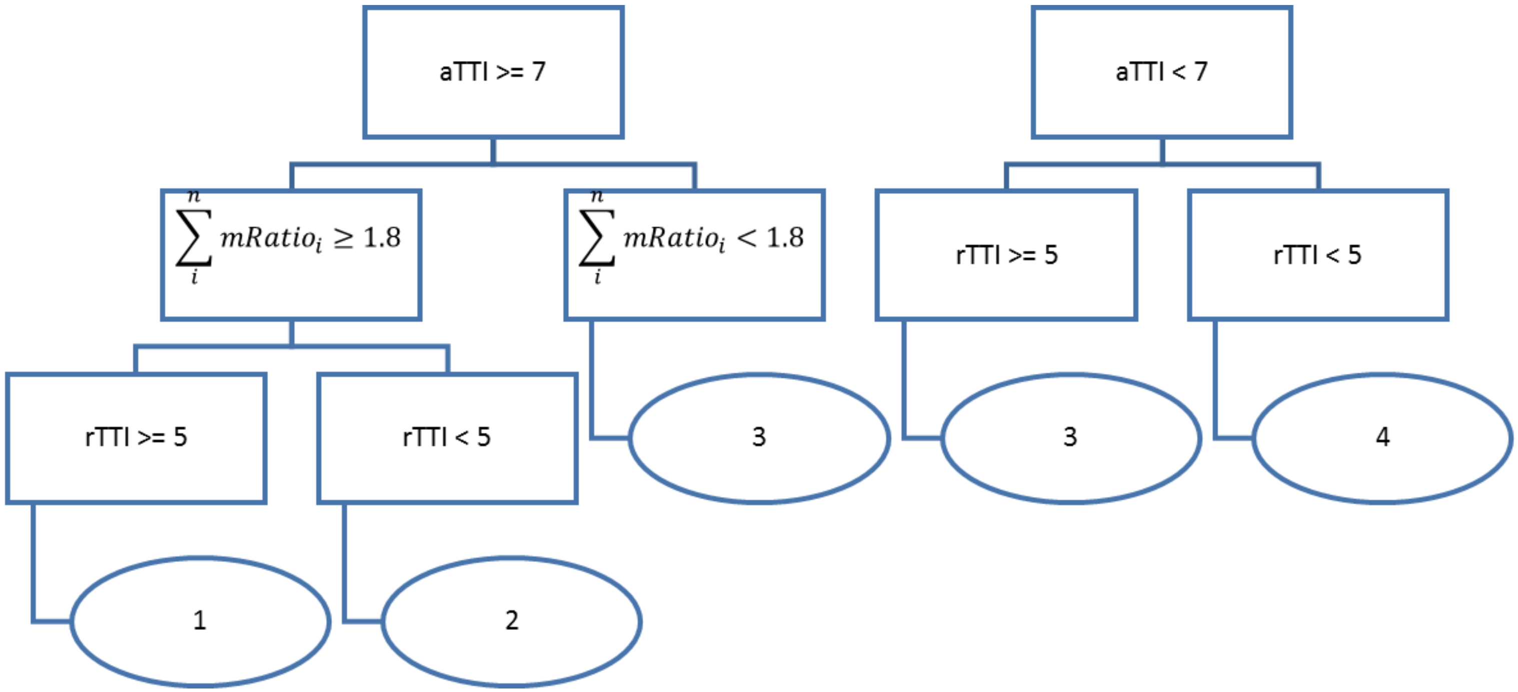
The percent arrivals on green (*POG*) parameter is calculated in this scheme as the ratio of the number of detected vehicle arrived at green time to the total numbers of detected vehicles. In this study, a vehicle is labeled as arrived on the green, if its first detection is within the green time interval of the corresponding phase for its movement. As previously mentioned, the *POG* requires a synchronization between Wi-Fi data and signal historical data. Unfortunately, this was not done in this project due to the difficulty coordinating with the Wi-Fi device manufacturer despite the many reports. Thus, the use of the *POG* in this report is to demonstrate the concept realizing that the POG calculation may not be accurate due to the synchronization problems. As with the analysis of the whole intersection, the data used for analyzing individual phase split are extracted from two sources. The *aDet* and *TTI* parameters were calculated based on the Wi-Fi reader data. The *mRatio* parameter was calculated based on the historical signal timing report. The *POG* parameter was calculated utilizing a combination of Wi-Fi data and historical signal timing data.

As illustrated in Figure 6-14, the diagnosis system proposed in this study first diagnoses the signal operation for the whole intersection. Based on this analysis, recommendations are given regarding the need for cycle length increase or geometry improvement. If the intersection-level diagnosis determines that it is necessary to inspect the individual phase splits, the diagnosis system will continue to investigate the need for fine-tuning the splits and/or offsets.

This study uses the intersection of Southwest 8th Street at Southwest 107th Avenue as a case study. The collected data included fifteen weekdays from July 6 to July 24, 2015; aggregated into 30 minute intervals. The 24 hours of the day were evenly divided into 48 time periods, each of which has 30 minutes. The data from the fifteen weekdays were categorized into these 48 time period bins. Thus, the 30 minute time periods become the basis of the diagnostic system.

As stated earlier, the first module of the diagnosis system is the inspection of the whole intersection's signal timing plan and its flowchart is presented in Figure 6-16. For each 30-minute time period, the module first reviews the average *TTI* of all critical movements (i.e., *aTTI*). The *TTI* used in this study is the 80th percentile *TTI*. If the *aTTI* has a value higher than or equal to 7, it can be inferred that either all critical movements had congestions or a few of them encountered severely long travel time to cause this high value. The next inference is made to differentiate between these conditions by checking the *rTTI*. If *rTTI* is high (i.e., higher than or equal to 5), it indicates that the differences between *TTI*s of the critical movements are large. Thus, it can be concluded that the high *aTTI* was a result of extreme high *TTI*s for a few movements. On the other hand, if *rTTI* is low (i.e., lower than 5), it indicates that the difference between *TTI*s on critical movements is relatively low. Consequently, inference can be made that all *TTI*s for the critical movements are high, which lead to the high *aTTI*.

However, before checking the *rTTI* to determine the cause of the high *aTTI*, the total *mRatio* needs to be investigated. The total *mRatio* is the sum of the *mRatio* of each critical movement. Higher total *mRatio* indicates more occurrences of phase maxed-out. The inspection of the total *mRatio* is intended to check the availability of green times that can be shifted from the not-so-busy phases to the congested phases. A higher total *mRatio* implies less opportunity to adjust the phase splits without increasing the cycle length or changing geometry, since at most of the time the majority of the phase splits reaches their maximum green time. The reason for checking the total *mRatio* before investigating the cause of the high *aTTI* (i.e., checking *rTTI*) is that if there are phases that do not frequently reach the maximum green time (i.e., low total *mRatio*), individual phase splits should be inspected to determine if shifts in green time is needed to mitigate the high *aTTI*.



where,

i **indicates the *i*[th] critical phase split.**

n **is total number of the phase splits corresponded to the critical movements.**

1. Recommend to increase cycle length (or improve geometry/capacity) and the diagnosis will continue to individual phase split.
2. Recommend to increase cycle length (or improve geometry/capacity).
3. Diagnosis will continue to individual phase split.
4. Recommend to keep the current signal plan.

Figure 6-16 Flowchart of the Intersection-Level Signal Inspection Module in the Decision Support Signal Operation Diagnosis System

It is also noted that the signal operation at the target intersection used coordination with fixed force-offs for most times of the day except at the night period at which the signal was running free operation (i.e., fully-actuated). The coordination with fixed force-off allows all of the following phases to inherit (part of) the unused green time from the previous uncoordinated phase(s). The amount of unused green time can be inherited by the following phase depends on the maximum green time set in an extra phase bank. For example, the TOD signal plan for the test intersection is presented in

			Green Time									
Current TOD Schedule	Plan	Cycle	1 EBL	2 WBT	3 SBL	4 NBT	5 WBL	6 EBT	7 NBL	8 SBT	Ring Offset	Offset
	Free											
0500	Free											
0530	2	120	12	30	7	50	12	30	7	50	0	91
0630	3	180	28	52	11	68	19	61	22	57	0	50
0930	24	140	16	36	11	56	16	36	11	56	0	42
1400	9	150	18	41	14	56	18	41	20	50	0	130
1500	11	160	17	39	12	71	20	36	20	63	0	103
1600	12	180	18	50	12	79	15	53	24	67	0	103
1900	13	150	20	36	9	64	20	36	15	58	0	98
2030	14	135	23	26	10	55	18	31	10	55	0	134
2130	15	120	15	32	6	46	14	33	8	44	0	9
2230	16	110	14	34	5	36	15	33	5	36	0	9

Figure **6-7** and the corresponding settings of the TOD function is shown in Figure 6-17. As shown in the figures, the maximum green time of the northbound through movement was set to be 79 seconds between 4:00 PM and 7:00 PM. However, according to the settings of the TOD function, Maximum Green 2 in phase bank 1 was adopted during this time period. Referring to Figure 6-18, the Maximum Green 2 in phase bank 1 for the northbound through movement was 81 seconds. As a result, Phase 4 for northbound through movement was capable to inherit 2 extra seconds (i.e., 81 seconds minus 79 seconds) unused green time from the previous phase(s).

Nonetheless, the coordinated phases, which is phase 2 and phase 6 at the target intersection, are always maxed-out in the coordination plans from 5:30 AM to 24:00 PM. Therefore, the inspection of total *mRatio* should not be applied to those two phases. The value of the total *mRatio* lies between 0 and 3 with 0 means no phase except phase 2 and/or phase 6 were maxed-out in the time period and 3 indicates that all phases corresponded to the critical movements were always maxed-out in that time period.

Local Time of Day Function

Time	Function	Settings *	Day of Week
0000	TOD OUTPUTS	-------1	M T W ThF
0000	TOD OUTPUTS	------2-	Su S
0100	TOD OUTPUTS	--------	Su S
0530	TOD OUTPUTS	------2-	M T W ThF
0600	TOD OUTPUTS	--------	Su S
0630	TOD OUTPUTS	--------	M T W ThF
1200	TOD OUTPUTS	--------	Su S
2100	TOD OUTPUTS	--------	Su S
2145	TOD OUTPUTS	------2-	M T W ThF

* Settings
Blank - FREE - Phase Bank 1, Max 1
Blank - Plan - Phase Bank 1, Max 2
1 - Phase Bank 2, Max 1
2 - Phase Bank 2, Max 2
3 - Phase Bank 3, Max 1
4 - Phase Bank 3, Max 2
5 - EXTERNAL PERMIT 1
6 - EXTERNAL PERMIT 2
7 - X-PED OMIT
8 - TBA

Figure 6-17 Settings of Time-of-day Functions for Signal Timing Plan of the Intersection of Southwest 8th Street at Southwest 107th Avenue

Phase Bank 1	Phase 1 EBL	Phase 2 WBT	Phase 3 SBL	Phase 4 NBT	Phase 5 WBL	Phase 6 EBT	Phase 7 NBL	Phase 8 SBT
Walk	0	0	0	2	0	0	0	2
Don't Walk	0	0	0	33	0	0	0	33
Min Initial	8	18	5	7	8	18	5	7
Type 3 Limit	0	0	0	0	0	0	0	0
Add Per Vehicle	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Veh Ext	2.0	2.5	2.0	2.5	2.0	2.5	2.0	2.5
Max Gap	2.0	2.5	2.0	2.5	2.0	2.5	2.0	2.5
Min Gap	2.0	2.5	2.0	2.5	2.0	2.5	2.0	2.5
Max Limit	18	33	6	45	15	33	6	45
Maximum 2	30	0	20	81	30	0	30	80
Adv/Dly Walk	0	0	0	0	0	0	0	0
Min Ped Clear	0	0	0	0	0	0	0	0
Cond Srv Min	0	0	0	0	0	0	0	0
Reduce Every	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Yellow	4.8	4.8	4.4	4.4	4.8	4.8	4.4	4.4
Red Clear	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0
Max Initial	0	0	0	0	0	0	0	0
Alt Walk	0	0	0	0	0	0	0	0
Alt Flash D/W	0	0	0	0	0	0	0	0
Alt Initial	0	0	0	0	0	0	0	0
Alt Exten	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0

Figure 6-18 Signal Timing Parameters in Phase Bank 1

In summary, the module shown in Figure 6-16 inspects the signal operation for the whole intersection. It first inspects the *aTTI* that is the average *TTI* for all critical movements to check whether the whole intersection was congested. If the whole intersection is congested, the module will execute the procedure on the left in Figure 6-16. It inspects the total *mRatio* that is the sum of the *mRatio* of all critical movements, for the purpose of finding whether there is a room to shift the maximum green times among signal phases. If the total *mRatio* is high, this indicates that there is not much room to adjust the phase splits, the module continues to inspect the *rTTI*, which is the range of the *TTI* of all critical movements. In the case of high *rTTI*, the diagnosis system recommends to increase the cycle length or improve geometry and will continue to the next module that inspects the individual phase splits, as discussed later. It is expected that even though there was not much room to adjust the phase splits with the current cycle lengths, there would be rooms for such adjustment with increasing the cycle length.

If the *rTTI* is low, the diagnosis system recommends to increase cycle length but does not suggest to reallocate the maximum green time proportion of the cycle. This means that the maximum green times of all phases can be increased proportionally when extending the cycle length. In the scenario of a low total *mRatio*, the diagnosis system will continue to inspect individual phase splits without recommendation of increasing cycle length because there are potential rooms for green time reallocation.

On the other hand, when *aTTI* is relatively low, indicating that the whole system is operating at a good level, the diagnosis continues to inspect *rTTI* in order to check the quality of the maximum green time allocation. If the *rTTI* is high, the diagnosis will continue to the next module that inspects the individual phase splits to determine opportunities to reallocate the maximum green time.

If the whole intersection analysis indicates that there may a potential for improvement with shifting the splits, , the diagnosis system executes the next module that inspects the individual phase splits. The flowcharts of that individual-phase-split-level inspection module are presented in Figure 6-19 and Figure 6-20 for uncoordinated phases and coordinated phases, respectively. The measurement, $aDet_i$, as shown in both flowcharts, is the average number of detection of all trips passing section i as illustrated in

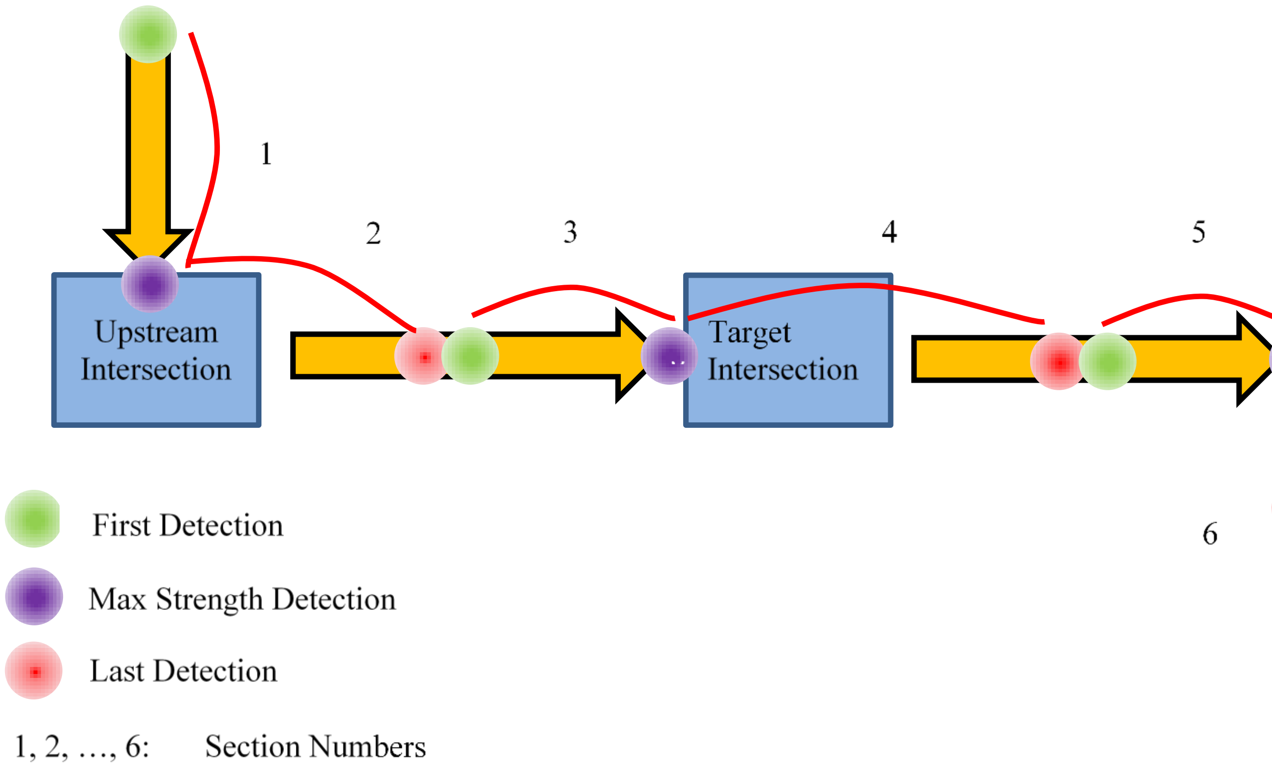


Figure **6-21**. For example, $aDet_3$ is calculated as the total number of records of a movement between the first detection and the maximum strength detection of the Wi-Fi detector at the target intersection divided by the total number of the identified trips for the movement. The $aDet_3$ is a function of the travel time of the vehicles in Section 3, which is directly impacted by the signal operation at the target intersection.

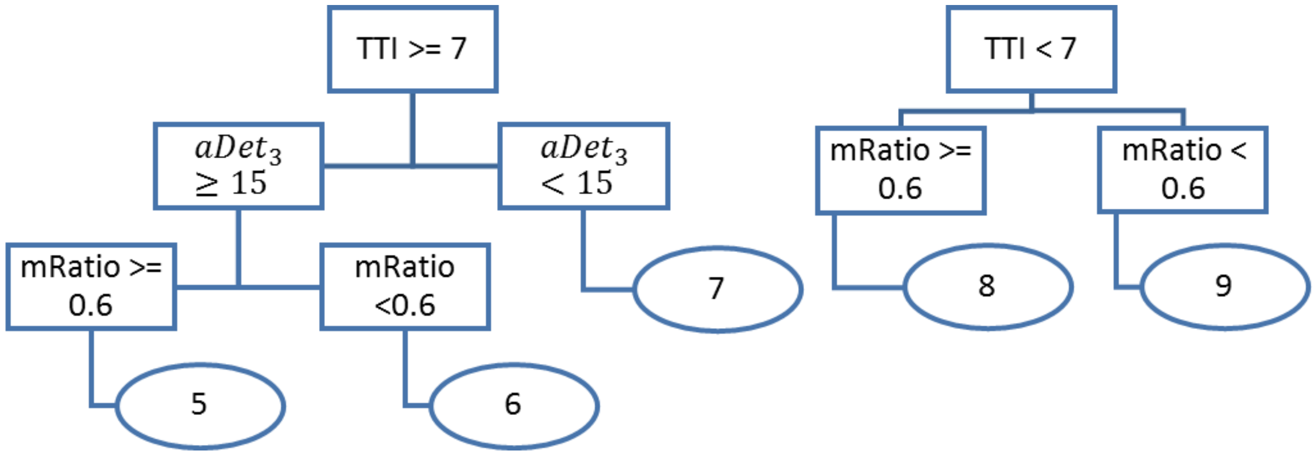
As discussed previously, Phase 2 and Phase 6 are the coordinated phases and there is a need to treat these phases separately using a different diagnosis module, which will be discussed later. Figure 6-19 shows the diagnosis flowchart for the uncoordinated signal phases. When the *TTI* is high indicating possible congestion, the diagnosis system examines the $aDet_3$ to determine whether the high *TTI* occurs likely as the vehicles approach the intersection or as they depart the intersection, possibly indicating downstream congestion. In this later case, the system also recommends further investigation of potential spillback from the downstream intersection.

In the next step, the diagnosis inspects the *mRatio* of the corresponding phase split. If the *mRatio* is high indicating that phase was maxed-out in a large proportion of the cycles, the diagnosis system recommends to increase the maximum green time for this individual phase split. If the *TTI* for a movement is high but the *mRatio* of the corresponding phase is low, the reason remains uncertain because it indicates vehicle experiences high delays without requesting the maximum green times. Potential reasons including low *POG*, spillovers, or spillbacks by other movements.

In the case that the *TTI* of the investigated movements is low as shown in right part of the flowchart in Figure 6-19, the diagnosis system inspects the *mRatio* of the corresponding phase split. It recommends to keep the current phase split parameters in the scenario with high *mRatio*. However, if the *mRatio* is low indicating that the movement does not need as much green time, the diagnosis system flags the corresponding phase split as a candidate to have its maximum green time decreased if necessary.

The examination of the *TTI* for the coordinated phases is the same as that for the uncoordinated movements. However, the inspection scheme also check the *POG* for the two directions of the main street traffic. The system recommends to change the offsets, if the *POG* is low indicating a low portion of vehicles arriving during the green time. In the case that the *TTI* values are low for the coordinated movements, the diagnosis system does not recommend to reduce the phase green time but a message is given to alert the user that the *TTI* of the coordinated phases are significantly lower than the other phases.

Note that the input performance measures required by the developed methodology in this study can be estimated in the current version of ITSDCAP. However, the developed methodology will be implemented in the ITSDCAP tool in the future work. The methodology can be implemented externally by the user using ITSDCAP outputs.



where,

$aDet_i$ indicates the average number of detection in section i as shown in

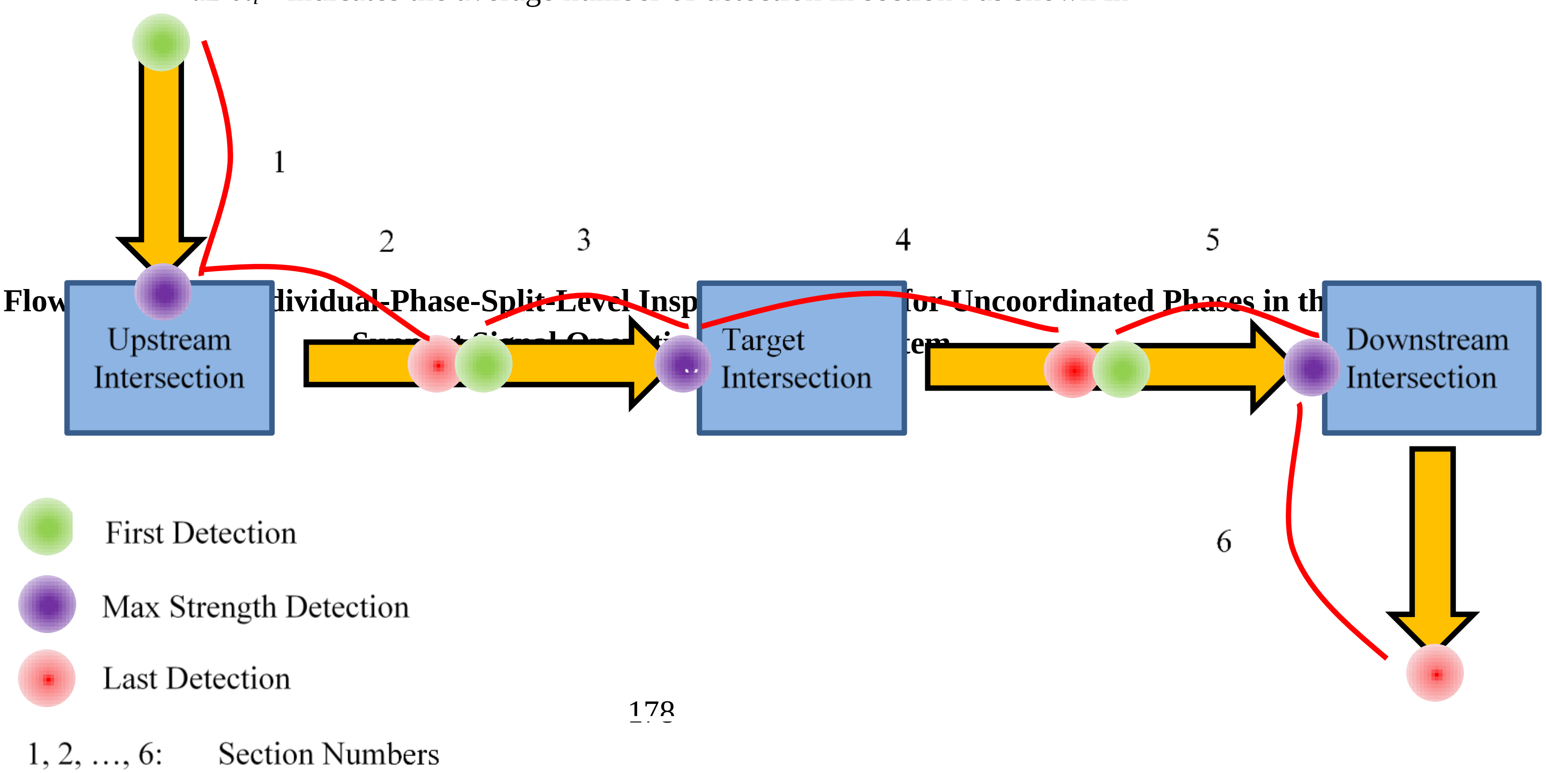
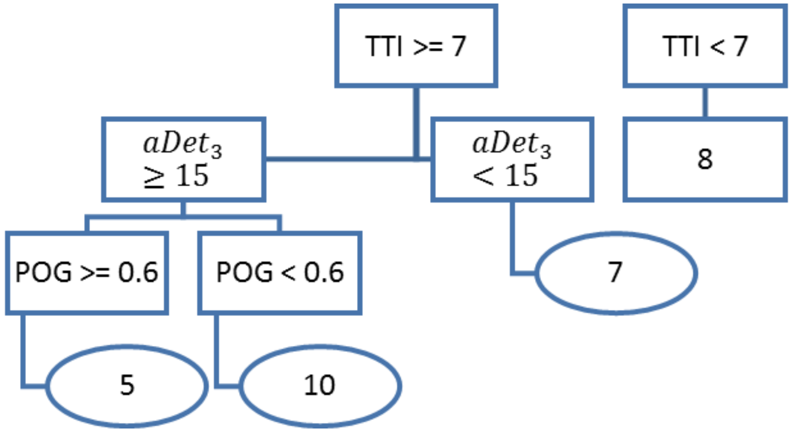


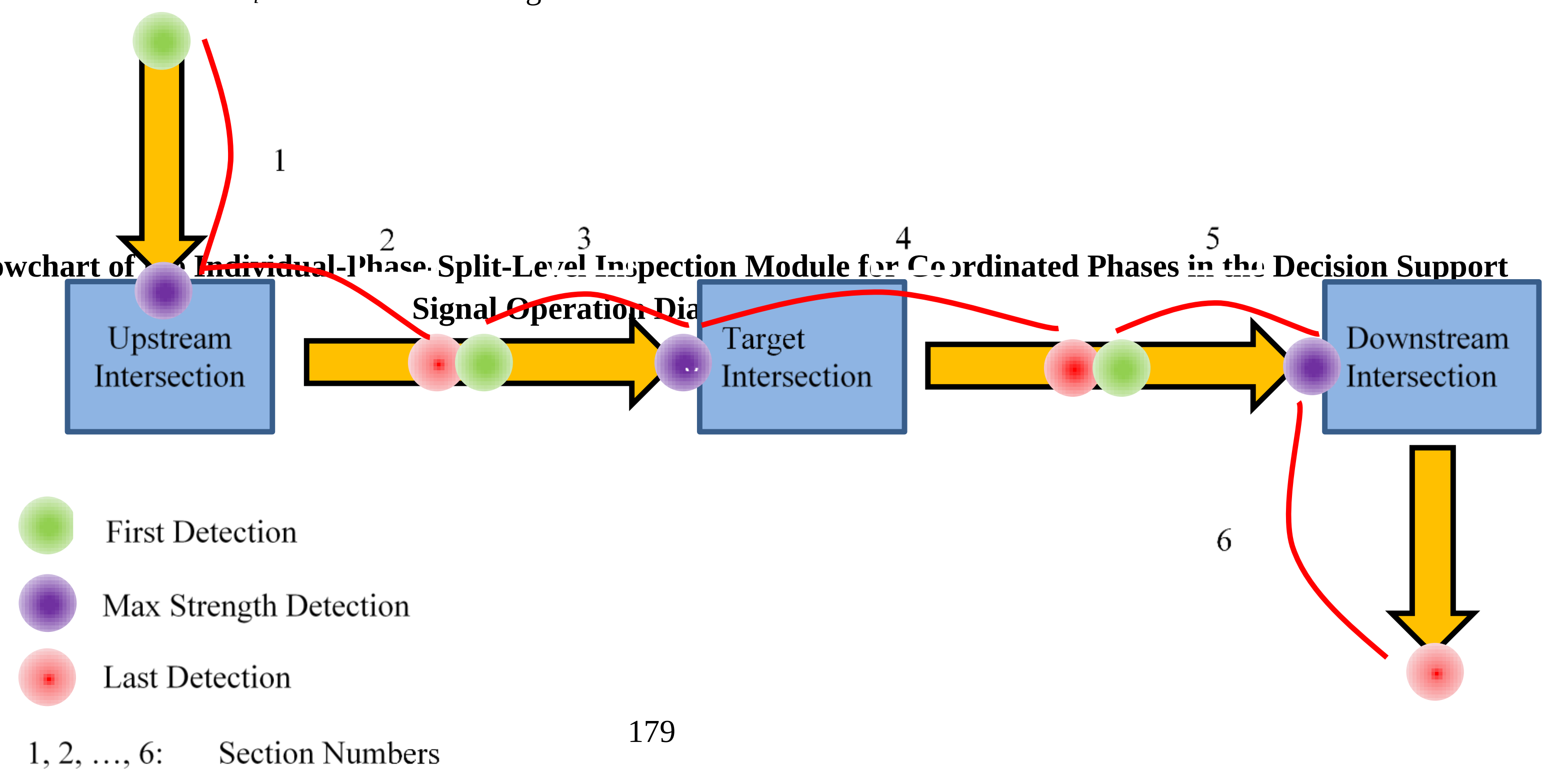
Figure 6-19 Flow dividual-Phase-Split-Level Insp for Uncoordinated Phases in th tem



where,

$aDet_i$ indicates the average number of detection in section i as shown in

Figure 6-20 Flowchart of the Individual-Phase Split-Level Inspection Module for Coordinated Phases in the Decision Support Signal Operation Dia



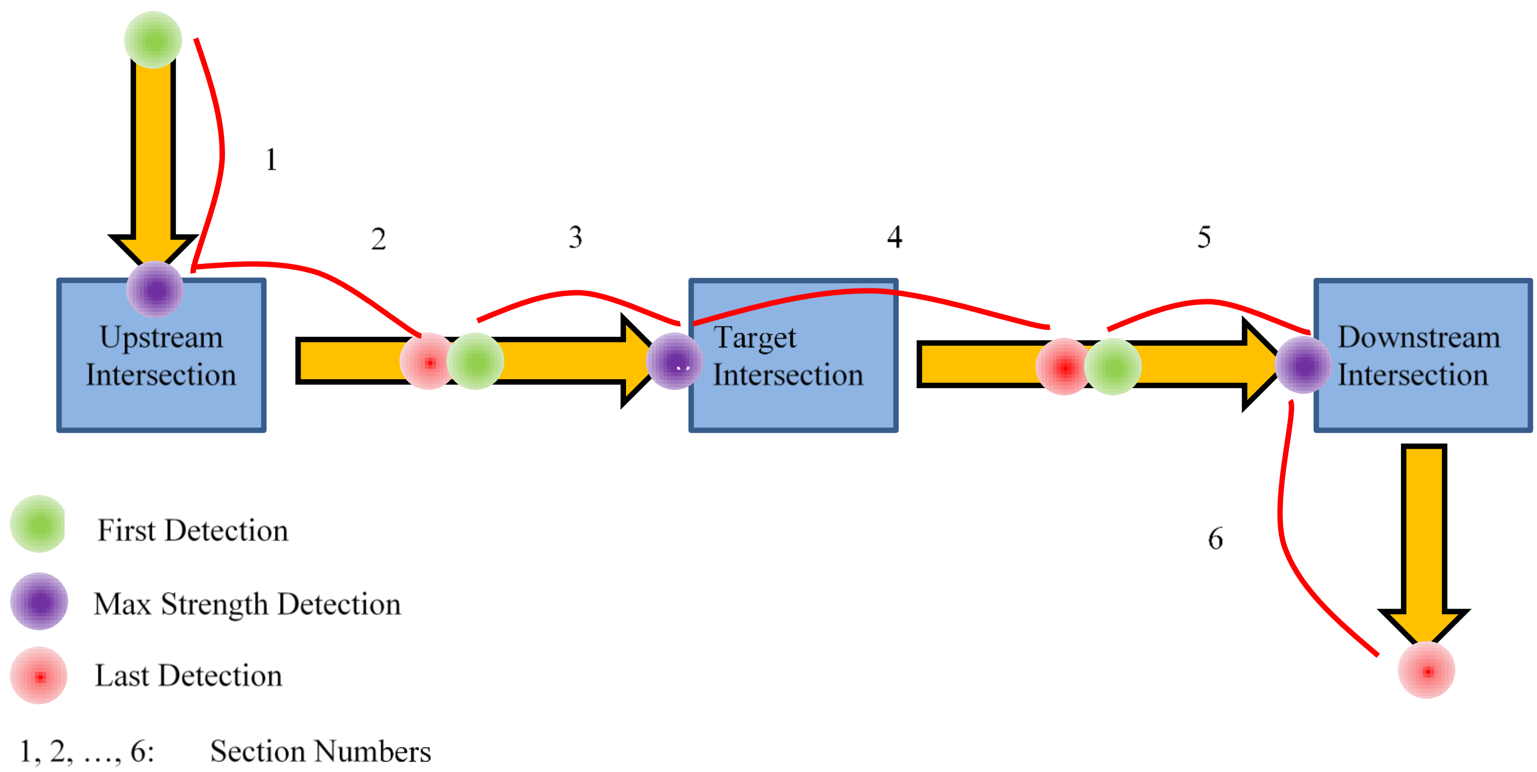


Figure 6-21 Illustration of Different Types of Detections with Section Numbers

6.5 Application of the Proposed Diagnosis System and Preliminary Results

The proposed decision support signal operation diagnosis system was applied to the target intersection of Southwest 8th Street at Southwest 107th Avenue. The Wi-Fi data and historical signal timing data from July 6 to July 24, 2015 were utilized and the goal of the application was to identify any potential problems existed in the current TOD signal timing plans.

The diagnosis system was started with the execution of the intersection-level signal inspection module as presented in Figure 6-16. The results of the intersection-level signal inspection are presented in Table 6-3. The results in Table 6-3 shows that the system recommends changing the split in most periods of the day. It also recommends increasing the cycle length during the noon peak period (12:00 PM to 2:00 PM). In addition, it points out to congested conditions between 4:30 PM and 6:30 PM, particularly between 4:30 PM and 5:00 PM.

The system recommends to increase the cycle length as well as to inspect the individual splits during the time period between 7:30 AM to 8:00 AM and 4:30 PM to 5:00 PM. Since the existing cycle length is already high (i.e., 180 seconds) according to

Current TOD Schedule	Plan	Cycle	Green Time 1 EBL	2 WBT	3 SBL	4 NBT	5 WBL	6 EBT	7 NBL	8 SBT	Ring Offset	Offset
	Free											
0500	Free											
0530	2	120	12	30	7	50	12	30	7	50	0	91
0630	3	180	28	52	11	68	19	61	22	57	0	50
0930	24	140	16	36	11	56	16	36	11	56	0	42
1400	9	150	18	41	14	56	18	41	20	50	0	130
1500	11	160	17	39	12	71	20	36	20	63	0	103
1600	12	180	18	50	12	79	15	53	24	67	0	103
1900	13	150	20	36	9	64	20	36	15	58	0	98
2030	14	135	23	26	10	55	18	31	10	55	0	134
2130	15	120	15	32	6	46	14	33	8	44	0	9
2230	16	110	14	34	5	36	15	33	5	36	0	9

Figure **6-7**, this may indicate that the existing capacity for those time periods is not sufficient and there is a need of geometry (capacity) improvement. It is worth mentioning that the diagnosis system also recommends to increase the cycle length for the time period from 3:00 PM to 3:30 PM that is the starting shoulder of the PM peak. This confirms with the visual identification of the potential problems as discussed on the CDF plots previously.

The results of the individual-phase-split-level inspection module are presented in Table 6-4. The shaded rows in the table represents the time periods in which the individual-phase-splits-level inspection was not recommended in the intersection-level signal inspection.

Table 6-3 Results of the Intersection-Level Signal Inspection for the Intersection of Southwest 8th Street and Southwest 107th Avenue

Time of day	Time period number	aTTI	rTTI	Total mRatio	Recommendation*
5:30-6:00	12	5.42	4.7	0.44	4
6:00-6:30	13	7.22	7.08	0.52	3
6:30-7:00	14	7.46	4.3	1.06	3
7:00-7:30	15	8.03	4.7	1.78	3
7:30-8:00	16	8.9	6.5	1.98	1
8:00-8:30	17	8.13	4.22	1.17	3
8:30-9:00	18	10	9.12	1.44	3
9:00-9:30	19	6.48	1.6	1.85	4
9:30-10:00	20	6.45	3.22	1.75	4
10:00-10:30	21	9.71	13.7	1.7	3
10:30-11:00	22	8.13	8.34	1.74	3
11:00-11:30	23	7.35	1.9	1.76	3
11:30-12:00	24	7.13	2.38	1.77	3
12:00-12:30	25	7.72	2.74	1.89	2
12:30-13:00	26	7.8	4.3	1.99	2
13:00-13:30	27	7.43	4.06	1.97	2
13:30-14:00	28	9.01	5.9	2.02	1
14:00-14:30	29	7.54	2.58	1.7	3
14:30-15:00	30	8.87	4.5	1.56	3
15:00-15:30	31	8.54	2.62	1.86	2
15:30-16:00	32	8.3	2.86	1.57	3
16:00-16:30	33	10.15	6.02	1.68	3
16:30-17:00	34	10.41	5.26	1.92	1
17:00-17:30	35	10.47	5.2	1.79	3
17:30-18:00	36	8.62	4.18	1.28	3
18:00-18:30	37	10.66	5.64	1.61	3
18:30-19:00	38	9.58	6.94	1.47	3
19:00-19:30	39	7.59	2.04	1.63	3
19:30-20:00	40	6.84	2.08	1.5	4
20:00-20:30	41	6.64	2.4	1.24	4
20:30-21:00	42	8.03	10.1	0.9	3
21:00-21:30	43	6.88	4.18	1.55	4
21:30-22:00	44	6.66	5.38	0.82	3
22:00-22:30	45	8.5	6.74	0.06	3
22:30-23:00	46	8.24	10.82	1.05	3
23:00-23:30	47	10.53	21.44	1.17	3
23:30-24:00	48	6.43	7.26	0.25	3

*

1. Recommend to increase cycle length (or improve geometry/capacity) and the diagnosis will continue to individual phase split.
2. Recommend to increase cycle length (or improve geometry/capacity).

3. Diagnosis will continue to individual phase split.
4. Recommend to keep the current signal plan.

Table 6-4 Results of the Individual-Phase-Split-Level Inspection for the Intersection of Southwest 8th Street and Southwest 107th Avenue

Time of day	Time Period Number	Recommendation[1]								
		Intersection	1 EBL	2 WBT	3 SBL	4 NBT	5 WBL	6 EBT	7 NBL	8 SBT
5:30-6:00	12	4								
6:00-6:30	13	3	9	8	8	6	9	8	9	6
6:30-7:00	14	3	9	8	8	8	6	8	6	5
7:00-7:30	15	3	9	8	8	5	6	8	9	5
7:30-8:00	16	1	9	8	5	5	6	8	6	7
8:00-8:30	17	3	9	8	8	5	6	8	6	5
8:30-9:00	18	3	9	8	8	5	6	8	6	5
9:00-9:30	19	4								
9:30-10:00	20	4								
10:00-10:30	21	3	9	10[2]	9	8	9	8	8	5
10:30-11:00	22	3	9	8	8	8	9	8	5	5
11:00-11:30	23	3	9	10[2]	5	8	9	8	7	8
11:30-12:00	24	3	9	8	8	8	9	8	5	5
12:00-12:30	25	2								
12:30-13:00	26	2								
13:00-13:30	27	2								
13:30-14:00	28	1	9	10[2]	8	8	9	8	5	5
14:00-14:30	29	3	9	10[2]	9	5	9	8	6	5
14:30-15:00	30	3	9	10[2]	8	8	6	8	6	5
15:00-15:30	31	2								
15:30-16:00	32	3	9	10[2]	5	8	9	10[2]	5	5
16:00-16:30	33	3	9	10[2]	8	5	9	10[2]	6	5
16:30-17:00	34	1	9	10[2]	5	5	6	10[2]	6	5
17:00-17:30	35	3	9	10[2]	8	9	6	10[2]	6	5
17:30-18:00	36	3	9	10[2]	5	6	9	10[2]	6	5
18:00-18:30	37	3	9	10[2]	8	6	6	10[2]	6	5
18:30-19:00	38	3	9	7	8	8	9	10[2]	6	5
19:00-19:30	39	3	9	10[2]	8	5	9	8	7	8
19:30-20:00	40	4								
20:00-20:30	41	4								
20:30-21:00	42	3	9	10[2]	8	8	9	8	8	5
21:00-21:30	43	4								
21:30-22:00	44	3	9	8	8	9	9	8	6	6
22:00-22:30	45	3	9	8	8	9	9	8	6	6
22:30-23:00	46	3	9	8	8	9	9	8	5	6
23:00-23:30	47	3	9	8	8	6	9	8	8	9
23:30-24:00	48	3	9	8	8	6	9	8	8	9

[1] Recommendations include:

5. Recommend to increase the maximum green time for this individual phase split.
6. High *TTI* due to uncertain reason (to be explored).
7. Recommend to increase the maximum green time for this individual phase split. However, excessive delays at the downstream link are suspected and need to be further explored.
8. Recommend to keep the current phase split parameters.
9. Recommend to flag this individual phase split as a candidate to have its maximum green time decreased.
10. Recommend to change the offsets.

[2] Synchronization between Wi-Fi data and Signal history data is required. This recommendation is based on the non-synchronized data only for the purpose of proofing the concept.

As can be seen from Table 6-4, it seems that the current signal timing plans provide surplus green times to the eastbound left-turn and westbound left-turn movements. Given the fact that the two left-turn phases are leading phases right before the coordinated phases (eastbound through and westbound through movements), the often unused green time from the two left-turn movements could only be inherited by those coordinated phases. Hence, the other uncoordinated movements, such as the northbound and southbound movements, were not able to utilize them even though they suffered with severe congestion. Based on the results, phase splits for northbound left-turn, northbound through, southbound left-turn, and southbound through movements require more green time than what they have in the existing timing plans. It indicates that these four movements experience inferior performance in general, especially during the PM peak that is confirmed by the CDF plots in the earlier section. Coordination of the north and south through movements with adjacent signals is also recommended if possible.

Recommendation 5 and 7 involves increasing the maximum green time for the specific phase splits. For this to be feasible, they should be accompanied by Recommendation 9 for one or more other phase splits in the same period, indicating that those phase splits are candidates for reduction. The results in Table 6-4 do not show many cases of unmatched recommendations except for the time period of 7:30 AM to 8:00 AM and 4:30 PM to 5:00 PM, in which there are three phase splits that are candidates for increasing their maximum green time extension but only one phase split that is candidate for decreasing its maximum green time. As stated earlier, for these two time periods, the diagnosis system also labeled them as candidates to have their cycle lengths extended or geometry (capacity) improved. Again, since the current cycle length in those two time periods are already long (i.e., 180 seconds), geometry (capacity) improvement is most likely needed. In summary, the results from the individual-phase-split-level inspection module confirms the findings from the intersection-level signal inspection module.

It is noteworthy that Recommendation 6 appears many times for Phase 7 corresponding to northbound left-turn movement during both the AM peak and PM peak. As mentioned earlier, recommendation 6 is a result of high *TTI* at the approach to the target intersection but the lack of utilization of the assigned maximum green times (i.e., low *mRatio*). The northbound left-turn movement was observed to be blocked by the northbound through movement.

For the coordinated phases, phase 2 and 6, recommendation 10 occurs in multiple divided time periods, which recommends to change the offsets due to the low *POG*. However, as discussed in the previous section, calculating accurate *POG* requires synchronization the clock between the Wi-Fi data and historical signal timing data that has not been done in this project due to the difficulty in coordinating with the Wi-Fi vendor. Thus, Recommendation 10 listed in Table 6-4 is provided only for demonstration of the concept and should not be accepted as an accurate value at this stage.

7 UTILIZATION OF THE HMC URBAN FACILITY PROCEDURES FOR THE ESTIMATION AND REAL-TIME PREDICTION OF TRAVEL TIME WITH CONSIDERATION OF RAIN IMPACTS

7.1 Introduction

Estimation and prediction of travel times under different operational and environmental conditions are critical to both the operation and planning of transportation systems (Haghani, 2013). Seven sources of congestion have been identified that directly impact travel time and travel time reliability: incidents, adverse weather, work zones, special events, signal control timing, demand fluctuations, and inadequate base capacity (Cambridge Systematics. Inc. and Texas Transportation Institute, 2005). Adverse weather causes about one billion hours of traffic delays in the United States (Rahman and Lownes, 2010). It has been reported that 15% of traffic congestion cases are due to adverse weather, which may include fog, rainfall and snowfall, icy or wet pavement, and high speed wind (Cambridge Systematics. Inc. and Texas Transportation Institute, 2005). The Highway Capacity Manual (TRB, 2010) provides adjustment factors for the capacity of freeway facilities under adverse weather conditions based on the event's level of intensity (Kittelson & Associates, Inc., 2014). No such consideration of the impact of adverse weather is included for the urban street facility in the HCM.

Rainfall events impact the saturation flow rates (SF) at signalized intersections and mid-segment free-flow speeds (Cambridge Systematics. Inc. and Texas Transportation Institute, 2005). Although not reported in the HCM 2010, saturation flow rates and free-flow speeds under rainfall have been identified in the version of the STREETVAL computational engine, which was used as part of the Strategic Highway Research Program (SHRP2) L08 project. STREETVAL was used as the computation engine of the SHRP2 L08 project procedure to determine the reliability of urban street facilities. The tool assesses reliability by generating and evaluating scenarios with different conditions that impact travel times. STREETEVAL generates the scenarios based on the probability of each weather event, which is calculated based on historical weather information from the nearest city, and collected as part of the SHRP2 L08 project (Kittelson & Associates, Inc., 2014).

This study focuses on real-time prediction of travel time on urban street facilities under rainy conditions utilizing the HCM urban street procedures. The study examines the use of the saturation flow rate and free-flow speed adjustment factors from the SHRP 2 L08 urban street facility procedure and other sources as inputs to the HCM procedures to estimate travel time. The travel time estimation is validated based on real-world measurements of traffic performance in conditions with different rain intensities. Once validated, this study examines the accuracy of using the HCM 2010 urban street facility procedure with these factors to predict weather impacts on travel time in real-time operations.

7.2 Literature Review

Adverse weather conditions such as precipitation and high speed wind can affect driver behaviors, vehicle performance, and thus, traffic flow characteristics, including capacity, speed, travel time, and safety (Federal Highway Administration, 2015). Travel times along 18 freeway segments and 15 arterials in the Washington D.C. area were studied under different levels of precipitation including none, light rain/snow, heavy rain, heavy snow/sleet, wind speed, visibility distance, and pavement conditions. The study results indicate that the average travel time under adverse weather conditions increased by 12% for two hours of the off-peak period (Mitretek, 2002).

Perrin and Martin (2002) assessed speed and flow rate reduction due to rain and snow at two signalized intersections. The results showed that the rain reduced the speeds and flow rates by 10% and 6%, respectively, while the reduction due to snow was 13% and 11%, respectively. Note that the study did not differentiate between various ranges of precipitation intensity. This research also found that signal retiming for adverse weather could improve travel time by as much as 18%.

Ibrahim and Hall (2002) investigated freeway speed reductions under adverse weather conditions and concluded that the speed was reduced by 3-5%, 14-15% and 30-40% for light precipitation (including both rain and snow), heavy rain, and heavy snow, respectively. The authors mentioned that these values could be different depending on the specific location characteristics and cannot be generalized for dissimilar regions.

An empirical study was done by the Federal Highway Administration (FHWA) (Cambridge Systematics, Inc. and Virginia Tech Transportation Institute, 2006) to examine the impact of adverse weather, including precipitation and visibility on freeway free-flow speed, speed at capacity, capacity, and jam density based on loop detector data from Baltimore, the Twin Cities, and Seattle. The results showed that the jam density is not impacted by weather conditions, while the free-flow speed and speed at capacity decreased with increasing rain intensity. However, the study found that the capacity reduction does not change with rain intensity and remains constant at a value of 10% to 11%. A 2% to 3.6% reduction in free-flow speed and an 8-10% reduction of speed at capacity were reported for light rain conditions (less than 0.0039 in/hr). The values of 6-9% reduction in free-flow speed and 8-14% reductions in speed at capacity were reported for heavy rain (0.63 in/hr).

Another study (FHWA, 2015) found that adverse weather results in a 10-25% speed reduction on signalized arterial routes with wet pavements, while snowy or slushy conditions can result in a

30-40% speed reduction. The study also found that saturation flow reduction due to weather events can vary between 2-21%, depending on the event intensity and time of day.

Agbolosu-Amison (2004) assessed traffic conditions under adverse weather events at signalized intersections and found that the saturation flow reduction varies between 2% and 16%, depending on the weather event intensity. Rahman and Lownes (2010) investigated rainfall impacts on speeds, travel times and average delays at one location in an urban arterial. The results showed about a 5% reduction in the average speed, and a 3.9% reduction in the free-flow speed. They also evaluated the effects of weather-responsive signal retiming and reported a 6.8% reduction in travel time as the benefit of signal retiming for the investigated corridor.

Seeherman et al. (2012) utilized historical data from 17 urban freeway corridors in California to estimate the proportion of delays related to rain. They found out that 3-25% of the total delays are due to rain. They also concluded that this proportion value is significantly dependent on the type of weather and the amount of recurring delays on the freeway segment.

Thakuriah and Tilahun (2011) examined incorporating real-time weather information to estimate the speed for a single corridor in Chicago. They used speed data from loop detectors and probe vehicles for 5-minute intervals and categorized the weather conditions into two categories: “good” and “bad.” Their empirical predictive model showed a 50% and 60% accuracy in estimating speed for light rain and heavy rain conditions, respectively.

Asamer and Van Zuylen (2011) assessed saturation flow rate reduction due to precipitation based on simulation modeling for three signalized intersections from video recorded data. They calibrated the VISSIM, a microscopic traffic simulation tool to reflect the reduced saturation flow rate due to rain, based on Perrin and Martin’s (2002) findings.

Van Stralen et al. (2014) investigated the impacts of adverse weather on the probability of traffic breakdown. They incorporated both the supply and demand aspects of adverse weather influence on traffic conditions using a panel mixed logit model. The average breakdown probability for dry weather was 50%, while the average breakdown probability for heavy rain condition was reported as 77.4%.

Yazici et al. (2013) studied the impacts of weather conditions on travel time and travel time variability in New York City, New York, using historical taxi GPS data. They explored the change in the travel time mean, mode and coefficient of variation. The results showed that the adverse weather conditions increased the travel time mean and mode; however, the amount of changes decreased as the congestion increased.

Li et al. (2014) assessed travel time reliability under rainfall in Florida. They calibrated the rainfall intensity distribution on a zip code basis and hourly precipitation, and evaluated the travel time reliability based on rainfall probability. The final results showed a 6% to 12% speed reduction for freeway and arterial facilities, depending on the rain intensity level. Table 7-1 represents a brief summary of the findings from previous studies regarding the rainfall impacts on freeway and urban street facilities operation parameters.

Table 7-1 Summary of Previous Studies on Rainfall Impact on Transportation Facilities Operation

Author(s)	Year	Location	Facility	Rainfall Impact on Traffic Operation
Ibrahim and Hall	1994	Canada	Freeway	• 3-5 % Speed Reduction for Light Rain • 14-15% Speed Reduction for Heavy Rain
Mitretek	2002	Washington D.C., USA	Freeway/ Urban Street	• 12% Reduction in Average Travel Time
Perrin and Martin	2002	Salt Lake Valley, UT, USA	Signalized Intersection	• 10% Speed Reduction, 6% Flow Rate Reduction
Agbolosu-Amison	2004	Burlington, Vermont, USA	Urban Street	• 2-16% SF Reduction Depending on Event Intensity
Chin et al.	2004	USA	Urban Street	• 6% Capacity Reduction • 10% Speed Reduction
FHWA		Twin Cities and Seattle, USA	Freeway	• 10-11% Capacity reduction • 2-3.6% FFS Reduction for Light Rain • 6-9% Speed at Capacity Reduction for Light Rain • 2-3.6% FFS Reduction for Heavy Rain • 8-14% Speed at Capacity Reduction for Heavy Rain
FHWA			Urban Street	• 30-40% Speed Reduction • 2-21% SF Reduction Depending on Time of Day and Event Intensity
Abdalla and Abdel-Aty	2006		Urban Street	• 9% Travel Time Increase for Light Rain • 17% Travel Time Increase for Heavy Rain
Rahman and Lownes	2010	Storrs Mansfield, CT,	Urban Street	• 5% Reduction in Average Speed • 3.9% Reduction in FFS

Author(s)	Year	Location	Facility	Rainfall Impact on Traffic Operation
		USA		
HCM	2010		Freeway	• 3-5% Speed Reduction for Light Rain • 14-15% Capacity Reduction for Heavy Rain • 2% Capacity Reduction for Light Rain • 7.2% Capacity Reduction for Medium Rain • 14.1% Capacity Reduction for Heavy Rain
Seeherman et al.	2012	California, USA	Freeway	• 3-25% of Total Delay is Due to The Rainfall Event, Dependent of Type of Weather
Li et al.	2014	Florida, USA	Freeway/ Urban Street	Freeway: • 6% Speed Reduction for Light Rain • 12% Speed Reduction for Heavy Rain Urban Street: • 10% Speed Reduction for Light Rain • 12% Speed Reduction for Heavy Rain

7.3 Case Study

Florida has an average total yearly precipitation of about 60 inches, and it is ranked as the fifth rainiest state among all 51 United States (2015). This study focuses on the estimation and prediction of travel time on urban street facilities under rainy conditions by utilizing the HCM's urban street procedures. To illustrate and test the methodology developed in this study, an urban street facility located in Boca Raton, Florida was used as a case study. The case study facility consists of nine coordinated signalized intersections along Glades Road in Boca Raton. The weather event data, including precipitation rate and duration, temperature, wind speed and direction, were collected from the National Climatic Data Center (NCDC) for each 15-minute interval. The nearest NCDC weather station is located at the Boca Raton Airport about a mile from the corridor. The data collection period started in June of 2014 until January of 2015, in order to fully cover the rainy season in Florida. Figure 7-1 illustrates the location of the study area and the weather station. Traffic parameters such as volume, speed and occupancy were collected from traffic detectors based on the magnetometer technology, which was installed for traffic management purposes by the City of Boca Raton. These detectors are located downstream of the signalized intersections on the facility. In addition, the utilized magnetometer detection technology allows for the estimation of travel time on facility segments, based on automatic vehicle re-matching by using identified vehicle signatures. Note that the turning movement percentages were estimated based on historical turning movement counts for the same time of day.

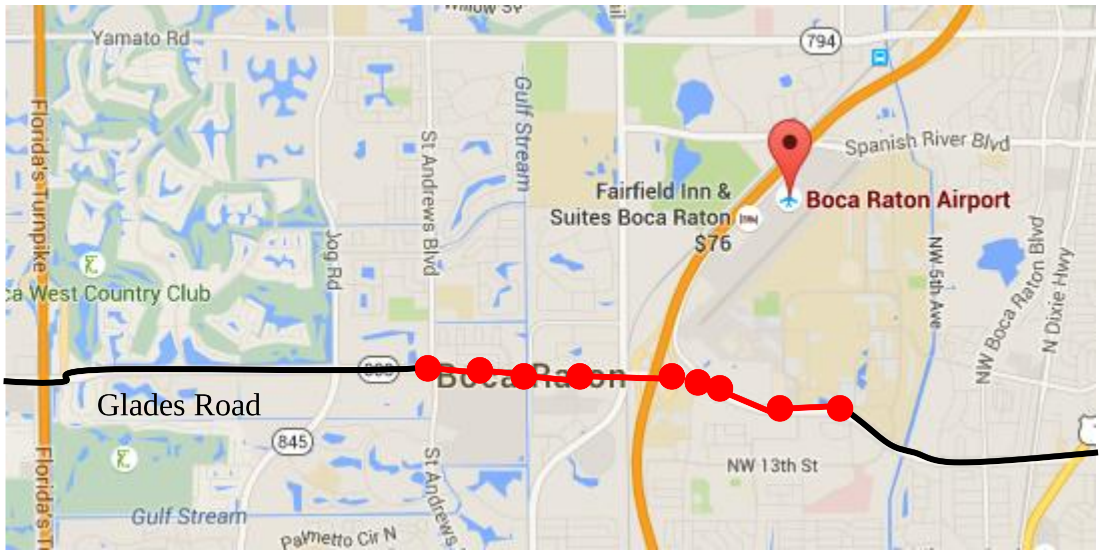


Figure 7-1 Location of the Study Area and Weather Station

7.3.1 Methodology

This section presents an overview of the methodology used to achieve the objectives of this study.

<u>Determination of Saturation Flow under Normal Conditions</u>

For normal conditions (no rain), the HCM 2010 provides a procedure to adjust the base saturation flow rate based on the physical attribute of the roadway and driver population. Zeeger et al. (2008) studied the default values in the HCM for different cities and reported that independent of weather conditions, SF may vary for different cities and populations.

The variation in the base saturation flow rate between cities suggests that it would be useful to fine-tune the SF in a narrow range around the values estimated using the HCM procedure to better predict the observed travel time in a calibration process. For this purpose, this study used travel time data for the weekday PM peak periods from a six-month period on the case study corridor. The travel time data were sorted according to congestion, and five days were randomly selected from each of the following three categories: median, congested and very congested days, representing the 50th, 80th and 95th percentile congestion on the corridor, respectively.

Each of the days was modeled in the urban street computational engine of the HCM 2010 (STREETVAL), with two values for SF (1800 vphpl and 1900 vphpl), then the travel times estimated by the model were compared to the real-world travel times based on vehicle re-matching using the magnetometer-based technology. In this study, the Mean Absolute Percentage Error (MAPE), Root Mean Square Error (RMSE), Normalized Root Mean Square Error (NRMSE), Mean Squared Percentage Error (MSPE) and Root Mean Squared Percentage Error (RMSPE) were used as the goodness-of-fit measures. According to FHWA's report on integrated corridor management, the MAPE of 15% is the maximum acceptable error for travel time calibration and prediction (Vassili, 2008).

<u>Saturation Flow Adjustment under Rainy Conditions</u>

In order to predict the travel times along urban facilities under rainfall utilizing the approach of this study, the rain impacts on the saturation flow rate at signalized intersections and free-flow speed of urban street segments need to be estimated (Kittelson & Associates, 2014). To examine whether the adjustment factors from SHRP2 L08 are appropriate or need to be fine-tuned, the Glades Road network mentioned earlier were modeled in STREETVAL software, and different scenarios with different SF and FFS combinations were input for different runs to determine how the use of different adjustment factors can impact travel time estimation under rainy conditions. First, rainfall events were grouped based on the rain intensity (precipitation rate in inch/hour). The group limits follow the HCM 2010 rain intensity categories for capacity drop on freeway facilities due to rainfall. Therefore, three groups of rainfall events were used: "Light Rain" (precipitation rate<0.1 inch/hr), "Medium Rain" (0.1 inch/hr <precipitation rate<0.25 inch/hr), and "Heavy Rain" (precipitation rate>0.25 inch/hr). Second, days with these event categories were modeled using STREETVAL, based on the actual volumes from detector data. Different scenarios were modeled to determine how different treatments of SF and FFS can affect travel time prediction. The travel time outputs from STREETVAL for each scenario were compared to the real-world travel time measurements for each 15-minute period of the rainfall event. Then, the results were used to identify the best combinations of SF and FFS adjustment factors.

In the first scenario, which can be considered as the base scenario, the SF and FFS were considered equal to the SF and FFS under normal conditions. In the second scenario, the SF is initially adjusted based on the SHRP2 L08 procedure, while the FFS was set equal to under normal conditions. In order to identify the best adjustment factor for the SF, a sensitivity analysis was performed using different values of the adjustment factor around those used in the SHRP 2 L08 by modifying the rain intensity parameter in Equation 1, presented later. In the third set of scenarios, the SF was considered equal to the best adjusted SF from the second set of scenarios, but with the FFS adjusted based on the SHRP2 L08 methodology (Kittelson & Associates, 2014). Sensitivity analysis was also done to determine the best adjustment factor for FFS, by modifying the rain intensity parameter in Equation 2, presented later. Note that all of the rainfall events are considered during daylight and clear visibility before rainfall.

As stated above, this research examines the use of adjustment factors for SF and FFS due to rainfall for urban facilities used in the SHRP2 L08 project. Equation 7-1 presents the SHRP2 L08 project adjustment factor calculation for SF under rainfall for urban street facilities (Kittelson & Associates, 2014).

$$f_r = \frac{1}{1+0.48\,R_r} \tag{7-1}$$

Where f_r is the saturation flow adjustment factor for rainfall and R_r is the rainfall rate during the analysis period, in/h.

Equation 7-2 presents the calculation of the FFS adjustment factor used in the SHRP2 L08 project.

$$f_s = \frac{1}{1+0.48\,R_r} \tag{7-2}$$

Where, f_s is the FFS adjustment factor for the rainfall with the intensity of R_r. SHRP2 L-08 also suggested using an adjustment factor of 0.95 for wet pavements without rain (Kittelson & Associates, 2014).

Travel Time Prediction under Rainfall

Real-time, short-term travel time prediction can benefit both road users and transportation system management. A main objective of this study is to investigate modeling-based travel time prediction with rainfall consideration using the HCM urban street procedures. Predictions for the next 15, 30, 45 and 60 minutes were considered, and goodness-of-fit measures were assessed for each case.

Travel time prediction using STREETVAL was performed utilizing three volume settings. The first is to input the “normal” day demands into the model. Second, the instantaneous measured demands were used as inputs. The third step involves using forecasted demands that are based on the combinations of current day instantaneous demands and the expected change in volumes based on historical trends.

The adjustment factors for SF and FFS are input to STREETEVAL to predict travel time for rainfall events based on the findings from the previous section. In order to evaluate the prediction results, the predicted travel times with rainfall consideration were compared to the actual travel times.

7.3.2 Results

<u>Saturation Flow under Normal Conditions</u>

As described in the methodology section, the no rain conditions on Glades Road in Boca Raton, Florida, were first modeled in STREETVAL to better calibrate the SF to reflect real-world travel time measurements. Table 7-2 illustrates the average results of modeling five days of the investigated three levels of congestion of the PM peak period. These three levels of congestion represent the 50^{th}, 80^{th} and 95^{th} percentiles of travel time, as discussed in the methodology section. Based on the results in Table 7-2, the 1900 vphpl was selected as the base saturation flow rate, since it produced the best correspondence between the model results and real-world travel times. This value also corresponds to that recommended by the HCM 2010.

Table 7-2 Goodness-of-Fit Measures for Saturation Flow Rates under Normal Conditions

Days	Saturation Flow (vphpl)	MAPE	RMSE	NRMSE	MSPE	RMSPE
50th Percentile Days	1800	0.142	19.208	0.174	0.030	0.173
	1900	0.081	13.304	0.121	0.010	0.010
80th Percentile Days	1800	0.173	24.295	0.184	0.050	0.223
	1900	0.107	17.569	0.133	0.019	0.138
90th Percentile Days	1800	0.126	22.572	0.146	0.026	0.161
	1900	0.123	23.552	0.153	0.023	0.152

<u>Saturation Flow under Rainfall</u>

The SF and FFS were then adjusted to account for the rainy conditions. Different scenarios with different adjustment factors for different rain intensities were run in STREETVAL, as described in the methodology section. This study categorized the rainfall events in three categories: light (R_r <0.1 in/hr), medium (0.1 in/hr< R_r > 0.25 in/hr) and heavy rain (R_r >0.25 in/hr). Table 7-3 illustrates the results of the sensitivity analysis of the SF and FFS for medium and heavy rain conditions. The highest rain intensity level in the light rain category leads to a 4.5% reduction in SF and FFS, which indicates a small difference between normal and light rain conditions, as confirmed by running the HCS procedure. In fact, not adjusting the SF and FFS produced slightly better results than those obtained with adjusting the parameters, as seen in Table 7-4.

For the SF adjustment under medium rain conditions, the R_r multiplier values of 0.28, 0.48 and 0.68 were considered for use in Equation 7-1 to estimate the adjustment factors. The SHRP2 L08 suggested that the Rr multiplier value is 0.48. Values higher than 0.68 were not considered because a value of 0.68 in all cases led to worse results compared to 0.48, as measured by the travel time goodness-of-fit measures. Regarding the FFS adjustment, 0.28 was not considered for the medium rain because it led to a very small difference, compared to 0.48. In summary, the R_r multiplier value of 0.48, used with the SSHRP 2 L08 project, led to the best results for SF and FFS adjustments in the case of medium rain condition, as seen in Table 7-3.

Table 7-3 Sensitivity Analysis Results for SF and FFS Adjustment Coefficient

Rain Category			R_r Multiplier Value	MAPE	RMSE	NRMSE	MSPE	RMSPE
Medium Rain	SF		0.28	0.138	25.876	0.225	0.030	0.173
			0.48	0.084	15.252	0.126	0.010	0.100
			0.68	0.098	18.987	0.158	0.018	0.134
	FFS		0.48	0.078	12.798	0.106	0.008	0.090
			0.68	0.084	13.927	0.115	0.010	0.101
Heavy Rain	SF	R_r<0.31	0.48	0.106	20.007	0.163	0.021	0.161
		R_r>0.31	SF= 1700	0.135	25.522	0.223	0.029	0.168
			SF=1650	0.094	18.975	0.159	0.016	0.126
			SF=1600	0.099	19.054	0.161	0.018	0.134
	FFS		0.18	0.083	17.764	0.147	0.014	0.118
			0.28	0.104	19.198	0.159	0.019	0.138
			0.48	0.132	21.825	0.185	0.028	0.167
			0.68	0.176	28.698	0.247	0.045	0.202

For heavy rain, according to Equation 7-1 with the SHRP L08 used parameters, the R_r multiplier value of 0.39 in/hr results in a saturation flow of 1600 vphpl, which is the least value that can be input into the STREETVAL software, as the saturation flow. However, there are some records of rainfall in the study area with an intensity of 0.75 inches per hour or more, which results in saturation flow of 1400 vphpl according to the SHRP2 L08 adjustment. Based on the sensitivity analysis results shown in Table 7-3, however, this reduction in SF is overestimated, and a value of 1650 vphpl, in fact, produced the best travel time goodness-of-fit results for all rain intensity more than 0.31 inches per hour. For rain intensity less than 0.31 inches per hour, the R_r multiplier value of 0.48 produced the best results. As mentioned, if the SHRP2 L08 used parameters for heavy rain is used in Equation 7-1, the drop in capacity would have been about 27% (1400 vphpl compared to 1900 vphpl). Using 1650 vphpl as the capacity instead is equivalent to a 13% drop in capacity, which is more in line with the values reported in Table 7-1, based on the literature review. Similar analysis was done for the FFS coefficients, and the R_r multiplier value of 0.18 was found to produce the best results for heavy rain conditions. Table 7-4 compares the final results of different scenarios after selecting the adjustment factors that produced the best results.

Table 7-4 Goodness-of-Fit Measures for Saturation Flow Rate under Rainfall

Rain Category	Scenario Description	MAPE	RMSE	NRMSE	MSPE	RMSPE
Light Rain	No Adjustment	0.110	17.432	0.138	0.019	0.138
	Adjusted SF	0.121	18.187	0.146	0.023	0.152
	Adjusted SF and FFS	0.158	21.337	0.175	0.037	0.192
Medium Rain	No Adjustment	0.086	18.313	0.152	0.010	0.100
	Adjusted SF	0.084	15.252	0.126	0.010	0.100
	Adjusted SF and FFS	0.078	12.798	0.106	0.008	0.089
Heavy Rain	No Adjustment	0.148	30.887	0.254	0.028	0.167
	Adjusted SF	0.103	20.007	0.159	0.017	0.130
	Adjusted SF and FFS	0.083	17.764	0.147	0.014	0.118

The best scenarios are highlighted in Table 7-4. As shown in this table, for light rain conditions, the first scenario with no adjustment to the SF and FFS led to the best results. This implies that under light rain scenarios, there is no need to adjust the SF and FFS for rain conditions. In the case of medium rain, the scenario with the adjustment to both SF and FFS utilizing SHRP 2 L08 parameters produced the best results. With the heavy rain scenario, the SF adjustment based on SHRP2 L08 seems to highly overestimate the impacts of the rainfall for intensity over 0.31 in/hr. According to Table 7-4, heavy rainfall, which mostly includes intensities over 0.31 in/hr, impacts traffic conditions less than what is estimated when using the SHRP2 L08 equation with no adjustments to its parameters. The value for the adjusted saturation flow that produced the best results for heavy rain conditions is 1650 vphpl. Similarly, in the case of free-flow speed, high rain intensities led overestimation of the reduction in free-flow speed based on Equation 2 with SHRP 2 L08 parameters. Real-world data showed a maximum reduction of 12.6% in free-flow speeds corresponding to the R_r multiplier value of 0.18 in Equation 7-2, which is sufficient to produce good results. This confirms the results of Li et al. (2014), who reported a 10% reduction in speed for arterials in Florida for the rain intensity of 1 in/hr.

Table 7-5 presents a summary of the R_r multiplier selected in this study for SF and FFS used in Equation 7-1 and 7-2 to calculate the SF and FFS. Table 7-5 also shows the range of percentage adjustment obtained for the SF and FFS in this study when the real-world rain intensity on the corridor was input into the equation.

Table 7-5 Final Adjustment Results for Rainfall Impact

	Parameter	Adjustment Factor	Adjustment Range (%)
Light Rain	SF	---	0
	FFS	---	0
Medium Rain	SF	$1/(1+0.48\ R_r)$	4.6-10.7%
	FFS	$1/(1+0.48\ R_r)$	4.6-10.7%
Heavy Rain	SF	Max $(1/(1+0.48\ R_r), 0.87)$	11-13.1%
	FFS	Max $(1/(1+0.18\ R_r), 0.87)$	5-12.6%

Prediction Results

Table 7-6 presents the results from the emulation of the real-time prediction of travel time for use in traffic management and traveler information applications. The prediction was performed using the HCM urban facility procedure and the SF and FFS adjustment factors, identified as described in the previous sections. The light rain category was excluded from the prediction procedure because the results showed that light rain conditions have minimal impacts on traffic flow.

As shown in Table 7-6 for both the medium and heavy rain conditions, the prediction of travel time with forecasted demands as inputs produced the best match to real-world measurements, when compared to no prediction, prediction with normal day demands as inputs, and prediction with instantaneous demands as inputs. For example, the prediction results when using the adjustment factors with instantaneous demands without volume forecasting achieved a MAPE of 1% to 8%, excluding one case with a MAPE of 14%. Demand forecasting improved the prediction results by up to 6%, with an average of a 2% improvement.

Table 7-6 Travel Time Prediction Results

Scenario	Medium Rain					
		MAPE	**RMSE**	**NRMSE**	**MSPE**	**RMSPE**
No Prediction	15 min	0.107	13.326	0.132	0.016	0.127
	30 min	0.117	18.668	0.192	0.012	0.108
	45 min	0.111	15.890	0.175	0.010	0.101
	60 min	0.210	43.012	0.391	0.050	0.223
Prediction using "Normal" Day Demands as Input	15 min	0.096	17.294	0.171	0.010	0.099
	30 min	0.103	23.187	0.239	0.013	0.115
	45 min	0.097	19.867	0.218	0.011	0.104
	60 min	0.219	46.868	0.426	0.050	0.223
Prediction using Instantaneous Demands as Input	15 min	0.059	12.111	0.125	0.004	0.063
	30 min	0.061	12.561	0.127	0.004	0.063
	45 min	0.043	8.513	0.094	0.002	0.045
	60 min	0.148	34.157	0.311	0.024	0.155
Prediction with Forecasted Demands as Input	15 min	0.048	10.700	0.106	0.003	0.055
	30 min	0.045	8.913	0.098	0.002	0.047
	45 min	0.045	6.087	0.072	0.004	0.061
	60 min	0.088	11.627	0.117	0.008	0.092
	Heavy Rain					
No Prediction	15 min	0.126	17.103	0.244	0.019	0.139
	30 min	0.208	32.016	0.508	0.051	0.227
	45 min	0.121	11.597	0.153	0.009	0.096
	60 min	0.160	21.840	0.240	0.019	0.138
Prediction using "Normal" Day Demands as Input	15 min	0.116	16.347	0.234	0.014	0.118
	30 min	0.108	16.523	0.262	0.013	0.116
	45 min	0.100	14.874	0.196	0.010	0.100
	60 min	0.146	26.217	0.288	0.022	0.149
Prediction using Instantaneous Demands as Input	15 min	0.015	2.948	0.042	0.000	0.017
	30 min	0.086	16.895	0.268	0.008	0.092
	45 min	0.028	3.619	0.048	0.001	0.031
	60 min	0.044	10.675	0.117	0.003	0.054
Prediction with Forecasted Demands as Input	15 min	0.015	2.948	0.042	0.000	0.017
	30 min	0.043	7.432	0.118	0.003	0.056
	45 min	0.020	2.658	0.035	0.000	0.021
	60 min	0.036	6.768	0.078	0.001	0.037

7.4 Conclusion

This study focuses on determining the accuracy of using the Highway Capacity Manual (HCM) urban street procedures to estimate and predict travel times under rainy conditions by adjusting the saturation flow (SF) and free-flow speed (FFS) inputs to the procedure. The study found that, under normal (no rain) conditions, using 1900 vphpl as the SF produced the best correspondence between the HCM model results and real-world measurements of travel times. For light rain conditions, the adjustments to the SF and FFS did not improve model estimation when compared to real-world estimates. Thus, such adjustments are not recommended for light rain conditions. Adjustments to the SF and FFS for medium rain conditions utilizing the procedure and parameters provided in the SHRP2 L08 project produced the best match to real-word measurements of travel times when compared to the other tested values of SF and FFS. The results indicate that using a 4.6% to 10.7% reduction in SF and FFS for medium rain, depending on the observed rain intensity, produced good results. However, heavy rain impacts on travel times were found to be overestimated when the SF and FFS adjustment parameters from the SHRP 2 L08 projects for these conditions were used. These adjustments had to be modified to constrain the impacts of heavy rains on SF and FFS to maximum values, resulting in a good matching to real-world conditions. In heavy rains, using a maximum of 13.1% for SF reduction and a maximum of 12.6% for reduction in FFS produced the best results.

This study also investigated real-time travel time prediction with rainfall consideration using the HCM urban street procedures. For both the medium and heavy rain conditions, prediction of travel time with forecasted demands as inputs produced the best match to real-world measurements, when compared to no prediction, prediction with normal day demands as inputs, and prediction with instantaneous demands as inputs. The prediction results are very promising and show the validity of the adjustment factor for SF and FFS, and also the potential for using the urban street methodology online in traffic management centers and traveler information applications. Note that the procedures developed in this section will be implemented in ITSDCAP in the future. The methodology can be implemented externally by the user using ITSDCAP outputs.

8 REVIEW OF PREVIOUS FDOT PROJECTS ON TRAFFIC MANAGEMENT

8.1 Introduction

The ITS Data Capture and Performance Management (ITSDCAP) tool was developed as a platform for the incorporation of support tools for both off-line and real-time TSM&O decisions. These tools are expected to take advantage of data from multiple sources, combined with modeling techniques as needed. The specific decision support tools must be developed based on TSM&O stakeholder requirements. As a starting point, it is useful to review previous FDOT research projects to determine related efforts that have developed products that may be useful for TSM&O decisions. The development of the ITSDCAP tool in this project provides an opportunity to incorporate decision support tools produced by these projects in a single environment. The review of the related FDOT research project also provides an indication of FDOT's needs in relation to decision support tools, since the research center projects are funded according to the prioritization of FDOT's requirements.

The review conducted in this study of the FDOT research center projects indicates that a number of FDOT projects, conducted by researchers from various universities, have delivered products and methods that can be used to support TSM&O project activities. Such projects are presented in this document. This technical memorandum is Deliverable 7 of the "Decision Support Systems for Transportation System Management and Operations (TSM&O)" project (FDOT Project BDV29 TWO 977-09). The technical memorandum reviews products and methods from selected FDOT research center projects that are related to TSM&O decision making, for possible inclusion in the developed environment in future efforts.

8.2 Review of Previous FDOT Research Projects

Final reports produced as part of FDOT research center projects were reviewed in this project, as mentioned above. Tables 8-1 through 8-8 present a summary of projects that produced products that are candidates for implementation in ITSDCAP.

Table 8-1 Effective and Efficient Deployment of Dynamic Message Signs to Display Travel Time Information

Project Title	Effective and Efficient Deployment of Dynamic Message Signs to Display Travel Time Information
Purpose	Support DMS placement decisions and the segments for which travel times should be displayed on DMS. The purpose is to support decisions regarding the locations of new DMSs or those to be relocated.
Summary	The project calculates the variability of travel time along 60 segments of I-95 and I-595 using existing data. Based on this variability, the research developed a benefit index and calculates this index for each investigated segment to support DMS installation and relocation decisions. Using a linear programming technique, the researchers then used benefit measures to determine the optimum destinations to display travel times.
ITSDCAP-Related Assessment	A similar approach can be incorporated in ITSDCAP. The approach can be extended to be based on travel time reliability, potential diversion routes, incident frequency, and incident severity. In addition, the approach can be extended to prioritize other devices such as CCTV cameras and detectors.

Table 8-2 Integrated Database and Analysis System for the Evaluation of Freeway Corridors for Potential Ramp Signaling

Project Title	Integrated Database and Analysis System for the Evaluation of Freeway Corridors for Potential Ramp Signaling
Purpose	The objectives of this study were to review existing ramp signal guidelines, and evaluate and select those considered to be suitable for Florida's use.
Summary	Seven guidelines were recommended for installing ramp signaling. These guidelines are grouped into three general categories in the form of warrants: traffic (warrants 1, 2, 3, and 4), geometric (warrants 5 and 6), and safety (warrant 7). Specifically, these warrants include: 1. Mainline peak hour volume > 1,200 vphpl. 2. Mainline peak hour speed < 50 mph. 3. For one-lane ramp, peak hour ramp volume is between 240 vph and 1,200 vph; and for multilane ramp, peak hour ramp volume is between 400 vph and 1,700 vph. 4. Total mainline volume and ramp volume is greater than the minimum threshold (depending on number of lanes) or the peak hour rightmost lane volume is greater than 2,050 vph. 5. Ramp storage distance is greater than the minimum requirement determined by the peak hour ramp volume. 6. Acceleration distance is greater than the minimum requirement determined by the freeway mainline prevailing speed. 7. Crash rate is greater than 80 per hundred million vehicle-miles.
ITSDCAP-Related Assessment	The guidelines can be incorporated in ITSDCAP. However, discussion with the FDOT indicates that there is a need to develop more advanced criteria for off-line and real-time decision making based on traffic flow dynamics, including bottleneck characteristics and system performance measures.

Table 8-3 Lifting HOV/HOT Lane Eligibility and Shoulder Use Restrictions for Traffic Incident Management

Project Title	Lifting HOV/HOT Lane Eligibility and Shoulder Use Restrictions for Traffic Incident Management
Purpose	Investigate the possibility of lifting HOV/HOT lane eligibility and shoulder use restrictions during major incidents on general-purpose (GP) lanes.
Summary	Using traffic data from FDOT Districts 4 and 6, the impacts of incidents of GP lanes on the operation of HOV/HOT lanes were investigated. A methodology was developed to determine the appropriateness of diverting the GP traffic to HOV/HOT lanes under different incident scenarios. The project also reviewed the regulations in Florida concerning the operations of HOV/HOT lanes and concluded that there was no legal obstacle or barrier that prevents opening HOV/HOT lanes to the GP traffic. Consequently, a two-stage decision-making procedure was proposed to implement a diversion plan. The feasibility of shoulder use for incident management as well as simultaneous use of other freeway management techniques such as variable speed limits and ramp metering were investigated. However, there are several maintenance and enforcement concerns that pertain to the shoulder lane use.
ITSDCAP-Related Assessment	The decision support for lifting managed lane eligibility can be implemented in ITSDCAP, if desired by FDOT.

Table 8-4 Decision Support Tools to Support the Operations of Traffic Management Centers (TMC)

Project Title	Decision Support Tools to Support the Operations of Traffic Management Centers (TMC)
Purpose	The goal of this project was to develop decision support tools to support traffic management operations based on collected intelligent transportation system (ITS) data.
Summary	The project developments included new models to estimate travel time based on point detectors for freeways. These models were compared with existing travel time estimation methods, including the one used in the SunGuide software. The results indicate that all of the tested methods perform at acceptable and comparable levels at low congestion levels. However, their performances vary with the increase in congestion levels. The comparison with other estimation methods shows that the developed models perform well in all cases. The developments of this study include a method to estimate traffic diversion based on the traffic detector and incident data. In addition, this study developed a method to determine the time lag between incident occurrence and the time it is recorded in the SunGuide database. This study also developed methods to estimate freeway secondary crashes, potential incident impacts on mobility, and a new method to allow incidents to be classified into categories based on primary incident attributes and impacts.
ITSDCAP-Related Assessment	Some of the methods developed in this research are already used in ITSDCAP. Two methods have the potential for implementation in a future version: estimating traffic diversion based on the traffic detector and incident data, and determining the time lag between incident occurrence and the time it is recorded in the SunGuide database.

Table 8-5 Demand-Based Signal Retiming

Project Title	Demand Based Signal Retiming
Purpose	The objective of this research was to develop methods to estimate demands based on field measurements, and to recommend thresholds to implement strategies to mitigate congestion problems.
Summary	Based on the results of this study, it was concluded that the methods and procedures developed in this research can be used to derive traffic demands from the available field sensors. For example, Bluetooth detection devices can supply travel times, whereas mid-block sensors can provide volume, speed, and occupancy data wherever such data are used in the developed procedures. The performance measures' thresholds can be used to identify different traffic conditions in the field and to implement strategies accordingly. The recommended strategies based on the thresholds can be classified as signal timing actions and information dissemination actions. The implementation of the strategies had a different impact on different scenarios.
ITSDCAP-Related Assessment	The developed methods to estimate demands and the thresholds to recommend strategies can be implemented in ITSDCAP. However, examination is needed of the transferability of the models between different locations (i.e., we need to determine if there is a need to calibrate based on simulation models for every location or not).

Table 8-6 Synthesis of the Advance in and Application of Fractal Characteristics of Traffic Flow

Project Name	Synthesis of the Advance in and Application of Fractal Characteristics of Traffic Flow
Purpose	Application of fractal theory to traffic management
Summary	The study used historical Florida traffic and crash data to detect fractal characteristics. Fractal behaviors in both annual and daily crash frequency trends were observed. It was found that the crash rates at specific intersections could be predicted using the fractal extrapolation method. Since fractal characteristics are evident in the explored trends, they can be used to support safety-related decisions. For example, a potential application of fractal theory in the three-year moving average trend analysis could be to predict whether a high-crash intersection would continue to be listed in the future high-crash location lists if no safety improvements have been made. The study concluded that the fractal theory is a candidate predicting short-term traffic flow, traffic pattern, identification of high-crash locations, and prediction of crash rates at specific locations.
ITSDCAP-Related Assessment	Short-term traffic prediction could be viewed as the main application of the fractal theory in the field of ITS, e.g., forecasting traffic flow in the next 15-minute period based on both the previous real-time data on the same day, as well as historical data for a three-week period, for example. This requires additional research, but fractal theory-based modeling has the potential to be implemented in a future research project in ITSDCAP.

Table 8-7 Traffic Management Simulation Development

Project Name	Traffic Management Simulation Development
Purpose	The goal of this project is to explore the development of methods and tools for the use of microscopic traffic simulation models to support the TMC software implementation, operation, and testing on one hand, and the use of Intelligent Transportation System (ITS) data to support the development and calibration of simulation models on the other.
Summary	The project produced software utilities that use the existing TMC databases and other available information for the preparation and calibration of microscopic simulation tools. In addition, the project produced utilities to support the testing of the TMC software modules and data archiving processes, as demonstrated by using cases of the tools developed in this study. Two software components have been developed in this project. The two components were referred to collectively as SunSim. The first component is the SunSim core simulation support environment, which supports the development of simulation models based on ITS data and user inputs. The second component is the SunSim TSS simulators, which are software utilities that allow for the exchange of data between the SunGuide software and virtual detectors in a simulation environment, for use in the SunGuide subsystem testing and operation evaluation. A number of use cases were designed in this study to demonstrate the use of the developed simulation environment in evaluating the SunGuide software modules and algorithms. These use cases include a software load test, travel time estimation based on point detectors, travel time estimation using Automatic Vehicle Identification (AVI) and/or License Plate Recognition (LPR) technologies, incident alarm threshold procedure testing, and ITS data warehousing process testing.
ITSDCAP-Related Assessment	The use of ITS data to support modeling has already been incorporated in ITSDCAP and is being further enhanced as part of the FDOT research center multi-resolution simulation project. The use of simulation to support off-line and real-time decision making as part of ITSDCAP has been explored, but still needs to be further developed.

Table 8-8 Real-Time Route Diversion Research Project

Project Name	Real-Time Route Diversion
Purpose	This project developed a modeling environment for routing recommendations in response to traffic incidents.
Summary	The project developed a computer-aided environment to help the TMC develop better diversion plans with the local authority. When a pre-planned alternate route cannot be used due to some unforeseen event, this system can automatically generate alternative plans and rank them to assist the TMC in the decision-making process. The system can be used if a pre-planned alternate route is not available for a given incident. The developed system is designed to cooperate with the SunGuide environment. It retrieves real-time traffic data from SunGuide, automatically generates alternate routes, and allows the user to disseminate route diversion information to dynamic message signs and highway advisory radio through SunGuide®, and to cellular phones. The system used the Dynasmart-P dynamic traffic assignment tool.
ITSDCAP-Related Assessment	Utilizing modeling and simulation in support of TSM&O operations is one of the techniques anticipated in the ITSDCAP development. In the initial work performed during the development of the IRISDS model (which is one of the parents of ITSDCAP), real-time modeling has been proposed and implemented. Current research on multi-resolution modeling in Florida (conducted as part of an FDOT research center project) and the FHWA Analysis, Modeling, and Simulation (AMS) effort is also related to the modeling in support of the TSM&O. It is recommended that modeling for both real-time and off-line applications be incorporated in ITSDCAP.

8.3 Conclusions

FDOT Districts are considering active, pro-active, and integrated strategies to manage urban corridors. As an example, FDOT districts and counties have started assigning dedicated personnel to monitor signalized arterial operations and recommend adjustments to signal timing parameters in real-time to accommodate congestion events such as over-saturated conditions, arterial incidents, and diversion from freeway incidents. Integrated corridor management strategies are also becoming a major consideration and ICM concepts of operation are being developed. Traditionally, detailed data were available only for freeways but in recent years such data have started to become available for urban arterials from multiple sources including advanced signal control system software, vehicle re-matching technologies (like Bluetooth and Wi-Fi), point detectors, FHP/police systems, weather agencies, and private sector data providers. However, this data has not been fully used to support the advanced strategies described earlier. ITSDCAP is recommended to be extended to expand its support of FDOT and local agency staff in real-time selection and implementation of active traffic management and integrated corridor management strategies utilizing data analytic tools, possibly combined with modeling of transportation systems.

The review presented in Tables 1 through 8 identified a number of methods that have been developed in previous FDOT research projects that can be considered for implementation in a decision support environment like ITSDCAP. The review confirms that the development and use of decision support tools to support TSM&O is a focus of FDOT interest and thus research in Florida. In particular, decision support tools have been developed related to active traffic management, incident management, managed lanes, signal control, and ITS strategy and technology deployment decisions. In addition to data-based decision support, off-line and real-time modeling is also very promising to support agency decisions. Incorporating modeling into ITSDCAP has been attempted and should be advanced further. TSM&O is a multi-modal and a multi-facility type oriented program. Thus, future activities related to performance measurements should include multi-modal decision-making processes similar to those used in the integrated corridor management (ICM) efforts. Thus, this would require the use of transit and freight data, in combination with traffic data. In the implementation of IRISDS (one of the parents of ITSDCAP), transit data was incorporated in the tool and a model to estimate arterial travel time based on transit automatic vehicle location (AVL) data was developed. That implementation showed the potential of combining transit and highway data to support agency decisions. In addition, the developed methodologies in this study such as signal diagnosis system, travel time prediction under rainy conditions, and arterial probability of breakdown model can be implemented in ITSDCAP in real time to provide alerts to agencies. Further estimation of arterial performance measures based on the trajectories constructed from the combination of point detector data and vehicle re-identification data can also be explored and implemented in

ITSDCAP. Currently, the data has to be downloaded from various data warehouses. It will be useful to incorporate module for automated exchange of data with data sources including the RITIS data warehouse. If determined to be useful by FDOT, a data warehouse module can be incorporated in ITSDCAP.

Related recent and on-going research and development activities performed as part of FDOT, FHWA, NCHRP, and SHRP2 research projects can provide an excellent basis for new modules in ITSDCAP for further support of the planning and transportation systems. Examples of related national efforts include:

- Operation of Traffic Signal Systems in Oversaturated Conditions - NCHRP Web Document 202 (Gettman, et al., 2012a and 2012b).
- FHWA Active Traffic Management (ATM) Feasibility and Screening – on-going FHWA project (http://ops.fhwa.dot.gov/atdm/research/#ttdm)
- Tools for Tactical Decision-Making/Advancing Methods for Predicting Performance – On going FHWA project (http://ops.fhwa.dot.gov/atdm/research/#ttdm)
- FHWA Integrated Corridor Management decision support tools (http://www.its.dot.gov/icms/) and the implementation in San Diego and Dallas
- FHWA project - Utah DOT Weather Responsive Traffic Signal Timing (www.its.dot.gov/index.htm)
- SHRP 2 project - Online Traffic Simulation Service for Highway Incident Management (Kurzhanskiy, 2013)

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