Before and After-Implementation Studies of Advanced Signal Control Technologies in Florida

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Submitted by Lily Elefteriadou, Ph.D.

Pruthvi Manjunatha Ph.D.

and

Sivaramakrishnan Srinivasan	Scott Washburn	Yafeng Yin
Xi Duan	Marian Ankomah	Yinan Zheng
Thomas Chase	Tyler Valila	Wei Sun
Zhuofei Li	Marilo Martin-Gasulla	Hope Sotolongo
Xiaotong Sun	Ria Kontou	Liteng Zha

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UF Transportation Institute
UNIVERSITY of FLORIDA

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16. Abstract

The Florida Department of Transportation (FDOT) has proposed the implementation of Adaptive Signal Control Technologies (ASCT) on eight corridors in Florida to overcome the limitations of traditional signal systems in cases of changes in traffic demand, weather, incidents, etc. The main objectives of this project are to evaluate the implementation of proposed ASCT traffic operations at several arterial corridors in Florida, before and after the installation of specific ASCT, document the advantages and disadvantages of different approaches and implementations, and provide recommendations for state-wide implementation of ASCT. The mobility and safety benefits of the ASCT implementation are assessed by comparing performance measures of time of the day (TOD) plans versus ASCT through field data collection. Two critical intersections are identified within each corridor and performance measures such as corridor travel time, intersection delay, major and minor street queues, turning movement etc. are collected. Crash data are collected over a period of fifty-nine months for safety analysis. A Benefit-Cost analysis is conducted by monetizing safety and mobility benefits. The summary field data are used to build regression models of performance measures as functions of site characteristics. Qualitative observations and institutional issues are obtained by interviewing local staff. Recommendations are made on the suitability of corridors for ASCT implementation, and guidelines are provided for effective field implementation.

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UNITS CONVERSION PAGE

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
		LENGTH		
In	inches	25.4	millimeters	mm
ft	feet	0.305	meters	m
mi	miles	1.61	kilometers	km
		VOLUME		
gal	gallons	3.785	liters	L
ft^3	cubic feet	0.028	cubic meters	m^3
		MASS		
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	metric ton	Mg

EXECUTIVE SUMMARY

The objective of this research was to evaluate the implementation of proposed Adaptive Signal Control Technologies (ASCT) traffic operations at eight arterial corridors in Florida, before and after the installation of specific ASCT (InSync and Synchro Green), document the effectiveness of these systems, their advantages and disadvantages, and provide recommendations for state-wide implementation of ASCT.

A literature review was first conducted to document the state of the industry and best practices for adaptive systems. Next, the impact of ASCT was evaluated by comparing traffic operational measures and crash statistics before and after their implementation at the eight corridors. Table 1 shows the detailed information on the corridor characteristics and data collection times.

Table 1 Data Collection Sites and Description

Site	*Route*	*Cross Section*	*Length (mile)*	*#Signalized Int. per mile*	*#Unsignalized Int. per mile*	*AADT*	*Speed Limit (mph)*	*Data Collection Time Periods*		
								AM Peak	*Off Peak*	*PM Peak*
Gainesville	Newberry Road	6-Lane Divided	1.45	8.3	13.8	45750	35	7am – 9am	1pm – 3 pm	4pm – 6pm
Deland	US 17/92	6-Lane Divided	2.28	2.2	6.6	31500	50 / 45	7am – 9am	1pm – 3 pm	4pm – 6pm
Panama City Beach	Beach Parkway	4/6-Lane Divided	8.5	1.1	5.6	28750	55	7am – 9am	1pm – 3 pm	4pm – 6pm
Sarasota and Manatee	University Parkway	4/6-Lane Divided	7.8	2.3	3.1	49110	40 / 45 / 50	7am – 9am	1pm – 3 pm	4pm – 6pm
Panama City	23rd Street	4-Lane Undivided	2	4.5	6.5	46875	45	7am – 9am	11am – 1 pm	4pm – 6pm
Pinellas	66th Street	6-Lane Divided	5	2.4	8	35585	45	7am – 9am	1pm – 3 pm	4pm – 6pm
Manatee	SR 70	6-Lane Divided	9.2	2.4	1.7	52185	40 / 50	7am – 9am	10am – 12pm	4pm – 6pm
Bartow	E.Van Fleet Drive & N. Broadway Ave.	6-Lane Divided	1.1	4.5	1.8	41180	45 / 35	7am – 9am	9.30am – 11.30am	4pm – 6pm

The staff at Traffic Management Centers (TMCs) and other agencies responsible for the installation and maintenance of the ASCT at these corridors were interviewed in order to obtain their perspective on the effectiveness of these systems. Based on the quantitative and qualitative information collected, the research team conducted a benefit-cost analysis, and developed recommendations and guidance for further implementation of ASCT.

Traffic Operations

To evaluate the impact of ASCT on traffic operations, two critical intersections and three critical time periods (AM, PM and Off Peak) were identified for each corridor. Five performance measures were obtained for the before and after study periods: Link/Route Travel Time, Delay at Intersections, Queue Length (at critical intersections), Queue to Lane Storage Ratio (at critical intersections), and Passenger Car Equivalent (PCE) flows (at critical intersections). For each performance measure, a comparison between the before and after data was conducted.

It was found that the implementation of ASCT led to an average overall reduction in travel time of 9.36%. All corridors show travel time reduction in at least one direction of travel and four corridors show reduction in both directions. US 17 in Deland and 66th Street in Pinellas showed the most improvement, whereas Newberry Road in Gainesville was adversely affected by the ASCT installation and the system was removed.

The ASCT generally helped increase major street throughput (6.96%) and reduce major street queues (15.57%). The minor street queues increased (16.98%) while the throughput remained almost the same (0.69%), i.e., ASCT is able to maintain the same levels of side street flows despite an increase in minor street queues.

Regression analysis showed that lower AADT, lower intersection density (signalized and unsignalized), and lower initial operating speed (before implementation of ASCT) resulted in higher traffic operational improvement. The sites that showed consistent improvements had minimal detection or construction issues, low-volume side streets, and simpler geometry (for example, no left turns as part of the main corridor).

Safety

The crash data along six of the eight corridors were obtained from the Signal Four Analytics System in order to evaluate the impact of ASCT on safety. Safety analysis was not conducted for the other two sites due to a lack of data for the "after" period. The data extracted from the Signal Four Analytics System was for the period January 2013 – November 2017.

The research team examined changes in total crashes, and changes in crashes by severity (fatal and injury crashes), crash type (such as rear end, intersection related), and time of day (Peak, Off Peak, Weekend). Estimates of traffic volume for the entire corridor (mainline) were obtained and these were used to compare annual crash rates. All crash data were collected over a period of 59 months (2013 to 2017). Depending on the date of ASCT implementation, the sites had varying data ranges for "before" and "after" crash data. We call this comparison as "long term". To have uniformity in comparison a 14 month window was used (7 before implementation and 7 after) for all sites. We call this comparison is as "short term".

US17 in Deland, 23rd Street in Panama City, and East Van fleet Drive in Bartow, showed reduction in crashes in both short and long term. University Parkway in Sarasota showed only short term improvement. Regarding the two corridors that showed an increase in crashes, SR 693 in Pinellas had a short "after" period for data collection. Beach Parkway in Panama City has had increasing tourist-related demand, which has led to higher seasonal traffic. This increase is potentially offsetting any safety benefits of signal coordination.

Interviews and Benefit-Cost (B/C)

The data collected from traffic engineering and safety analyses were used to compute the benefits for each site, while costs were obtained during interviews with operating agencies.

The interviews were conducted either through an on-site meeting or through video calls when in-person interviews could not be arranged. A questionnaire was developed and provided to the agency in advance of the interview, and it consisted of five sections and approximately 40 questions which focused on previous traffic control technologies used by the agency, their experience with ASCT, cost components, and institutional issues.

Based on the interviews, the level of staff satisfaction correlated with objective measurements such as travel time and queue improvement. Some of the key components for successful ASCT implementation identified through these interviews are:

- Regular maintenance and checks of the detection system and cameras
- ASCT software needs regular updating, and it is important to include maintenance funding
- Extensive training of 5 days or more is required, with providing additional staff for ASCT
- Sites where vendors installed the system and did the initial fine tuning performed better
- ASCT proved to be effective during the Off Peak periods at all sites and during Peak hours in some sites.

The benefit-cost analysis revealed overall net positive monetized benefits (12.8 considering safety, 5.4 without safety). The ASCT perform well for most of the corridors and for the overall program. The benefits are mainly attributed to reduced travel time along the corridors. The crashes are classified into five categories labeled KABCO: killed (K), incapacitating (A), non-incapacitating (B), possible injury (C), and property damage only (O). Since KABCO values weigh the fatalities heavily, safety benefits are extremely variable and could swing from net negative to net positive due to a single fatality.

Overall, the research team has concluded that ASCT generally yield better performance and a higher return on investment when implemented on corridors with lower intersection density, low-volume side streets, and high demand but not oversaturated traffic conditions. Based on interviews conducted, ASCT is not a “set it and forget it” system. Maintenance (especially detectors and cameras), training (at least 5 days), and appropriate staffing are some of the key factors contributing to their success.

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1. INTRODUCTION

Adaptive Signal Control Technologies (ASCT) conduct real-time optimization of traffic control using a variety of sensors and algorithms. The primary objective of implementing ASCT is to minimize travel time and decrease the number of stops through arterial corridors. Such systems are known to reduce traffic delays, crashes, and may result in fewer periodic re-timings of traffic signals.

1.1 Background and Objectives

The Florida Department of Transportation (FDOT) has been interested in evaluating the effectiveness of ASCT under various conditions, in order to develop guidelines for state-wide deployment. Previous implementations and evaluations of these technologies have found that ASCT may not be effective or warranted for all types of corridors and traffic conditions.

The main objectives of this project were to evaluate the effectiveness of ASCT at several arterial corridors in Florida, compare traffic operations and safety before and after their installation, and provide recommendations for state-wide implementation of ASCT. This evaluation is based on a quantitative and qualitative analysis, including equipment and personnel cost, and it concludes with a benefit-cost analysis for each corridor. A total of eight corridors are studied in this project.

1.2 Project Overview

The Florida Department of Transportation (FDOT) commissioned this project in May 2014 to evaluate the effectiveness of ASCT across Florida. InSync and Sychro Green are the two adaptive systems deployed at the eight corridors studied (Figure 1.1). Table 1.1 provides an overview of these corridors, including geometry features, speed limits, intersection density, and annual average daily traffic (AADT).

All corridors studied have similar cross-sections, but their total length varies from 1.1 to 9.2 miles. Most of the corridors have a relatively low density of signalized and unsignalized intersections (less than 0.5 and 1.6 intersections/mile, respectively), although, Newberry Rd (Gainesville) has by far the highest density for both types of intersections (5.7 and 9.5 intersections/mile). The AADT along these corridors are 41,000 veh/day on average, with the SR 70 corridor having the highest (52185 veh/day) and Panama City Beach Pkwy having the lowest (28750 veh/day).

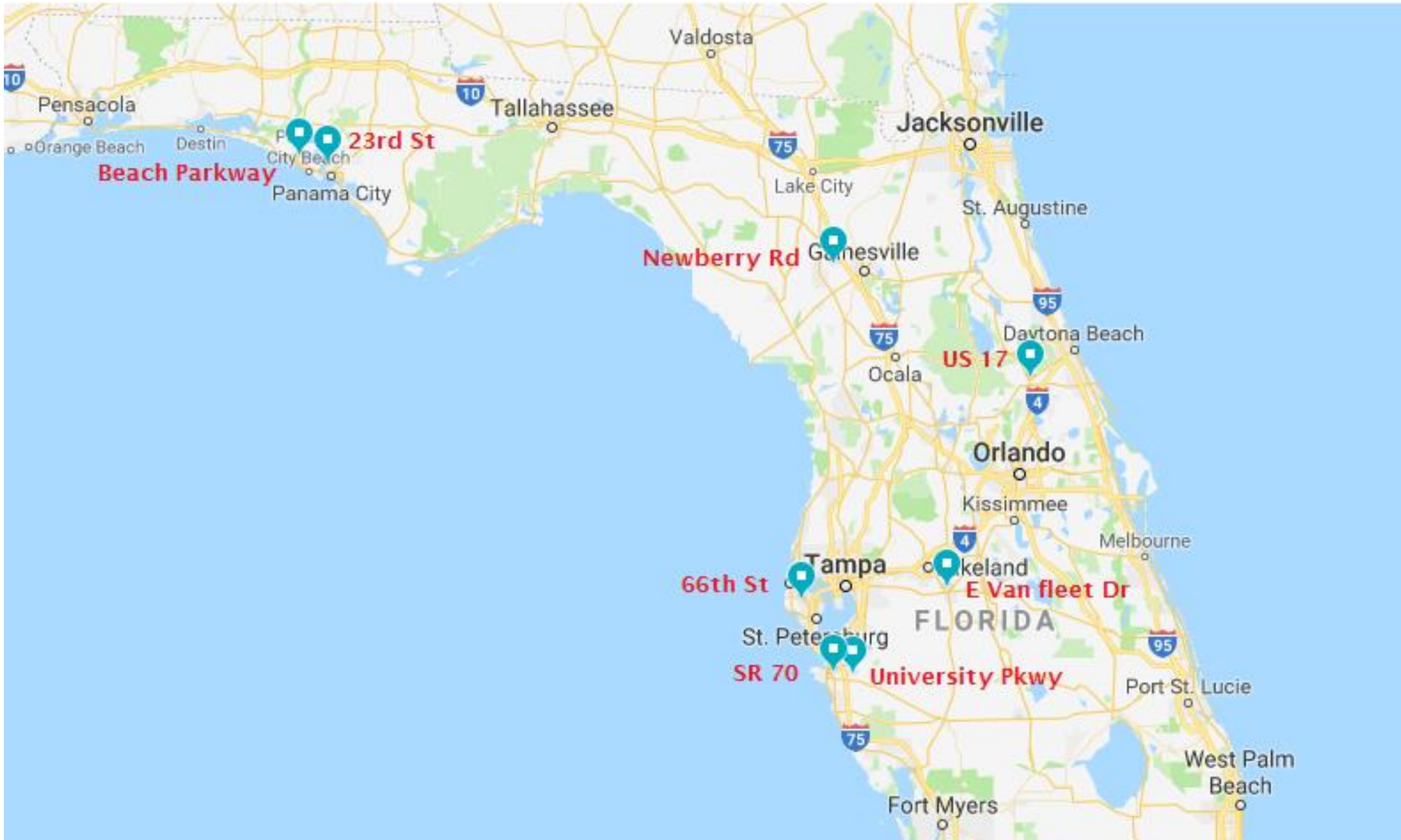


Figure 1.1 Locations Of The Study Corridors

Table 1.1 Data Collection Sites and Description

		Gainesville	Deland	Panama City Beach	Sarasota and Manatee	Panama City	Pinellas	Manatee	Bartow
Route		Newberry Road	US 17/92	Beach Parkway	University Parkway	23rd Street	66th Street	SR 70	E.Van Fleet Drive & N. Broadway Ave.
Cross Section		6-Lane Divided	6-Lane Divided	4/6-Lane Divided	4/6-Lane Divided	4-Lane Undivided	6-Lane Divided	6-Lane Divided	6-Lane Divided
Length (mile)		1.45	2.28	8.5	7.8	2.0	5.0	9.2	1.1
#Signalized Intersections per mile		8.3	2.2	1.1	2.3	4.5	2.4	2.4	4.5
#Unsignalized Intersections per mile		13.8	6.6	5.6	3.1	6.5	8	1.7	1.8
Pedestrian Signals		No	No	No	No	No	No	No	No
AADT		45750	31500	28750	49110	46875	35585	52185	41180
Speed Limit (mph)		35	50 / 45	55	40 / 45 / 50	45	45	40 / 50	45 / 35
Data Collection Time Periods	***AM Peak***	7am – 9am	7am – 9am	7am – 9am	7am – 9am	7am – 9am	7am – 9am	7am – 9am	7am – 9am
	Off Peak	1pm – 3 pm	1pm – 3 pm	1pm – 3 pm	1pm – 3 pm	11am – 1 pm	1pm – 3 pm	10am – 12pm	9.30am – 11.30am
	PM Peak	4pm – 6pm	4pm – 6pm	4pm – 6pm	4pm – 6pm	4pm – 6pm	4pm – 6pm	4pm – 6pm	4pm – 6pm

Prior to the data collection, the research team conducted a thorough literature review (Appendix A). The literature review focuses on traffic signal optimization approaches and provides an overview of existing products, their computational capability and functionality, and their approach to signal control optimization, along with their perceived advantages and disadvantages. It also includes an overview of industry best practices with regard to implementing ASCT, and the experience of other agencies. It also identifies pertinent performance measures that should be collected before and after installation of ASCT. These measures include arterial travel time, delay at each signal, turning movements, queue length, and quality of existing signal control and coordination.

The following chapters report the data, analyses, and results from this project. Chapter 2 provides an overview of the traffic engineering analysis conducted which considers travel time savings, reduction in queues, and other operational performance measures. The results of safety analysis are reported in Chapter 3. Chapter 4 summarizes the results from a series of discussions and interviews with local agencies responsible for installing the ASCT at each corridor. Chapter 5 provides the results of the benefit/cost analysis, which considers travel time savings, safety effects, and operational and maintenance costs of ASCT. The last chapter of this report provides recommendations and guidelines for implementation of ASCT.

2. LITERATURE REVIEW

The objective of this project is to evaluate the implementation of Advanced Signal Control Technologies (ASCT) for several corridors across Florida. This document summarizes the work conducted under Task 1 which reviewed and assessed existing ASCT, their computational capability and functionality and their approach to signal control optimization, along with their perceived advantages and disadvantages identified to-date.

The term Adaptive Signal Control Technology (ASCT) describes any system that collects data, evaluates traffic signal performance on the basis of one or more of the system's functional objectives and then updates signal timing in response to that evaluation [1]. These systems are expensive and take considerable time for deployment. Hence, it is necessary to evaluate the benefits and appropriateness of such a system for various corridor designs and demand conditions. Each ASCT varies in the extent and type of detection required, equipment deployed, as well as in the definition and algorithmic use of split, cycle, offsets and phase sequences.

SCOOT and SCATS are two of the first such systems and they have been the most popular (Figure 2.1). SCOOT was developed in the United Kingdom, while SCATS was developed in Australia. In an attempt to introduce this technology in the US, RHODES and OPAC were developed as part of the RT-TRACS program by the Federal Highway Administration (FHWA). The Los Angeles Department of Transportation (LA-DOT) independently developed its own ASCT system [2] while ACS-Lite was initially developed by the US DOT in partnership with Siemens, Purdue University, and the University of Arizona [3]. InSync and SynchroGreen are the latest adaptive signal control systems and these are being installed in several corridors across Florida and will be evaluated in this study.

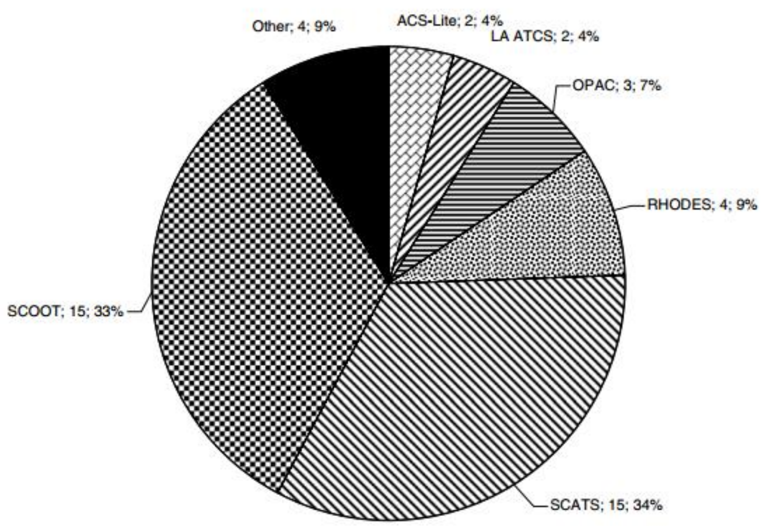


Figure 2.1 Market share of Adaptive Signal Systems in 2010 [4]

The next sections detail the ASCT methods and algorithms used in InSync and SynchroGreen followed by an overview of other ASCT that have been implemented elsewhere. The fourth section summarizes industry experiences with these systems as documented in the literature. The last section provides the conclusions of this task.

2.1 Insync Adaptive Traffic Signal Control System

2.1.1 Background

The InSync adaptive traffic control system is invented by Dr. Reggie Chandra, P.E., PTOE. The system is developed by Rhythm Engineering and was launched in 2008 after three years of research and development [5]. The company has been granted 4 patents for its unique design [6]. According to the company's website [6], this adaptive traffic signal control system has been deployed at more than 1,000 intersections all across the United States. The system is currently deployed at 22 states: Arizona, Arkansas, California, Colorado, Florida, Georgia, Iowa, Kansas, Kentucky, Michigan, Missouri, New Mexico, Oklahoma, Oregon, Pennsylvania, South Carolina, Tennessee, Texas, Vermont, Virginia, West Virginia and Wisconsin. There are scheduled deployments of the InSync adaptive signal control system in numerous sites for Idaho, Illinois, Florida, Ohio and South Dakota locations [6].

2.1.2 System Architecture & Hardware

Detection Methods

The InSync adaptive signal control is a distributed system of processors that need to be installed at each intersection that is part of the adaptive signal controlled corridor [5]. According to the company's website [6], the processor can work with any signal controller that the local agency has decided to implement, implying that there should not be any additional costs for controller upgrades. Furthermore, it is stated that InSync's processor is compatible with any signal control cabinet [6], and thus there is no need for upgrading the pre-installed ones to ensure functionality. Rhythm Engineering provides three options for detection: 1) implement their detection method, which consists of InSync cameras; 2) keep the agencies' detection devices, which could include inductive loop detectors, cameras, microwave techniques and radars; or 3) integrate all available devices and InSync cameras, an option that yields most accurate results [6]. Each of these three options is discussed in the following paragraphs

InSync Cameras

The InSync detection method consists of Samsung SNZ-5200 IP cameras with detection technology [6]. At each intersection up to 4 of those cameras can be installed, one for each approach, and there are options for installing additional cameras if needed. Those cameras have remote aim and focus and are used to monitor the intersection by detecting and measuring traffic demand at all intersection's approaches every second. The datasets that are acquired from this detection system include vehicles in queues. Figure 2.2 illustrates the queue detection through the InSync cameras. Each approach camera categorizes each lane as a detection lane; each detection lane is subdivided by vertical zones based on the average length of a car. Monitoring the intersection through these cameras can be done online from any web browser [6].

Figure 2.2 InSync cameras detection through image processing [6]

Using Existing Detection: InSync Tesla

For this detection option, the agency can keep their detection and monitoring devices, saving the InSync cameras installation costs. This way, if the corridor's agency has invested in installing their own detection method, they can still take advantage of it and and yet have the benefits of the adaptive traffic signal control that InSync yields [6]. The "Tesla system" is reported to accept most third party cameras, detectors and radars; in this system, stop bar detection is required for all lanes of the monitored intersection.

Using InSync and Existing Detection: InSync Fusion

This InSync detection option can be used to combine the benefits from installing InSync cameras and at the same time use other available detection methods. This way, the accuracy of the detection is increased since data sets are used from multiple sources.

InSync processor

In order for the system to be successfully deployed the company recommends that one InSync processor is located at every signal cabinet of each intersection adaptively controlled. Ethernet connections are required for every signal cabinet [5]. The processor weights approximately 7 lbs and works as follows: it gathers data from each intersection and vehicles monitoring sensor, pre-installed or newly installed, such as vehicle loop detectors, cameras and/or radars. The processor analyzes the datasets received and communicates with the upstream and downstream intersections to ensure data validity in corridor analysis [5] [6]. At every second the processor determines the traffic movement priority (additional information on phasing and traffic movement options is provided below). The processor communicates the results of the optimization to the controller and the changes appear on the network immediately.

Equipment panel

This panel is essentially the power supply channel and the Ethernet switch for the processor and the InSync cameras [6].

Detector card

This card supports the integration of the video detection and the traffic controller [6].

User's software

CentralSync is the software used for the initial and ongoing configuration of the system. The deployment of the 'traffic management variables and strategies' is conducted through this interface. This software is Windows-based, which might be an issue for any agency that has a different operating system. An advantageous option is that plans and strategies can be uploaded or downloaded remotely [6].

2.1.3 Signal Control Optimization Framework

According to Federal Highway's Administration report [7], adaptive signal control systems continuously adjust daily signal schedules to accommodate traffic accumulation, react promptly to changed traffic patterns and progressively improve travel time reliability along the implementation's corridor. The so-called real time adaptive signal control characteristics are usually listed as: effectively utilizing traffic flow models to predict vehicle arrivals and effectively adapt the signal timing in order to accommodate the progression of the vehicles in the corridor globally, and minimize delays locally (intersection level) [8]. The manufacturer of InSync indicates that the system is completely digital and operates without being restricted to the analog sequencing, splits, offsets and cycles. Instead, InSync introduces the concept of states: each state is a phase or a pair of phases that occur simultaneously without conflict [6]. The InSync adaptive signal control system achieves dynamic adaptation by altering signaling states, sequences or the green time so as to accommodate the current state of traffic. The state machine/processor can pick from any state or sequence that is allowable (these are pre-defined) in order to serve the current demand more efficiently. Figure 2.3 provides a series of 8 states and 16 sequences as an example of states and sequences that the digital model can choose from and implement in real-time [9]. InSync eliminates signal cycles and transition periods.

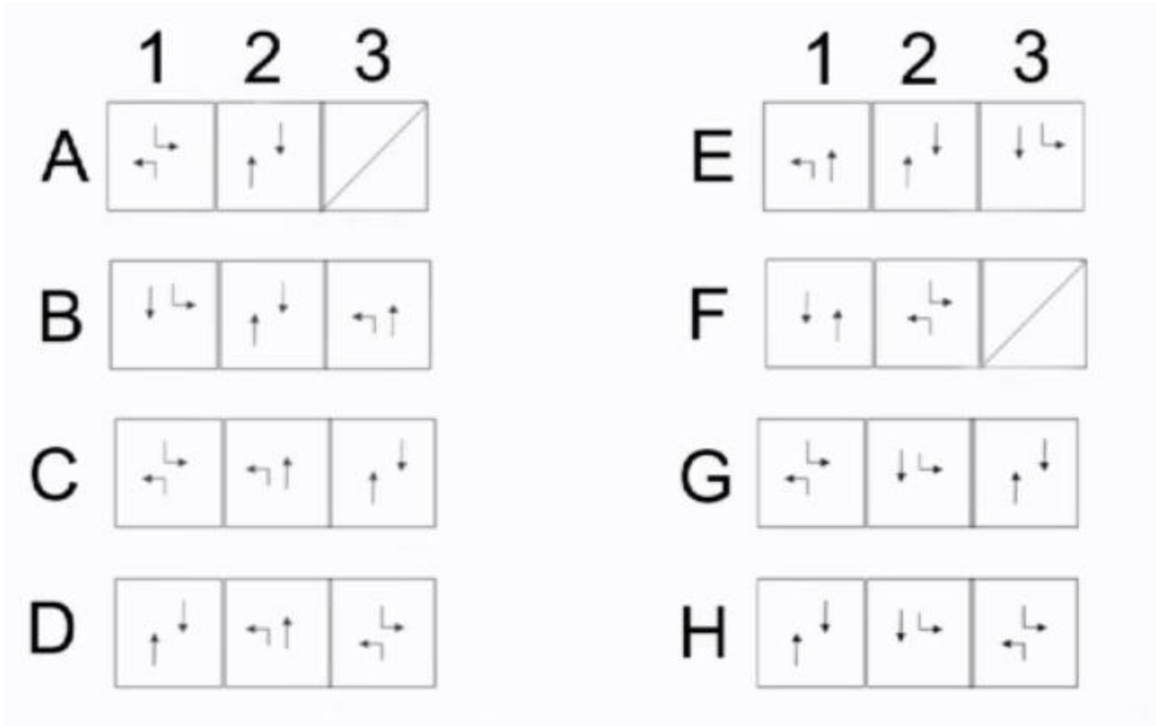


Figure 2.3 States and Sequences digitally called in InSync [6]

After gathering all available detection data, the logic of the “greedy” algorithm is implemented on the “local” intersection level in order to minimize delay. This algorithm works as follows: tokens are distributed to vehicles arriving at the intersection on red. Every 5 seconds that the vehicle is waiting on the intersection it gets a token (Figure 2.5); the algorithm aims to minimize the delay for the intersection’s approaches by minimizing the number of tokens handed out. The process is described as fully-actuated and the signal timing is optimized according to the number of cars waiting and the duration of their waiting time (delay).

Apart from optimizing traffic signal times locally, the InSync system’s objective is to progressively ensure the minimization of delay along the corridor [9]. This is achieved by creating speed lines through the corridor. Over time, speed lines are generated across the corridors so that a car travelling at the desired speed goes through the corridor without stopping. Note that not only though movements can be coordinated; any state including left turns can be part of the adaptive coordination [5]. The global optimizer guarantees progression of platoons of vehicles along the corridor; then control is turned over to the local optimizer at the intersection level and the optimal phase combination is served in order to satisfy the current demand [10]. The process doesn’t require deterministic cycle lengths or timing plans [10].

Figure 2.4 Token distribution at the intersection level [6]

2.1.4 Development Process

Rhythm Engineering provides the service of implementation to the sites; installation teams are sent to the corridor's location in order to proceed with the hardware installation and the cabinets' wiring [6]. According to the company's website, before integrating the system, the following information is needed: traffic counts, phases, SYNCHRO files and information on the controllers, the detection systems and the signal cabinets currently in use.

The system's company also provides complete documentation including manuals and guides for installation, operation, and maintenance. Training support is also included in the installation package, incorporating classroom sessions and hands-on training in the field. Technical support is provided during the operation process [5].

2.1.5 Monitoring

In addition to the CentralSync software that needs to be installed in the agency's computers, there is a web-user interface where information can be accessed online. The WebUI interface allows for monitoring and can also be used to re-conFigure Bameras and adjust detection areas [6]. Real-time information is provided at any time, following InSync's operation and showing statistics and diagnostics by calling the processor. The web-based user environment can also generate statistical reports or csv files for download after specifying the type of data needed to be exported.

2.1.6 Case Studies

The InSync adaptive signal control has been implemented in various corridor sites since its introduction in 2008. TJKM Transportation Consultants report the benefits of InSync's implementation along a 1.24 mile corridor in the city of Salinas, CA [11]. The results indicated that travel time along the corridor after the InSync deployment was reduced by 42%, the average speed increased by 69% and fuel savings were reported up to 33%. Kittelson and Associates, Inc. presented the results of an InSync implementation along a 1.7-mile corridor in Hillsboro, OR [12]. The consultants reported that the InSync system metered traffic along corridor entrances in order to facilitate the platoons' progressive movements. Average daily travel time was reduced by a range of 4-24% but average intersection delay was not improved. This report also brings into light small issues with the monitoring and detecting processes that were eventually addressed [12]. Kimley-Horn and Associates, Inc. conducted a study analyzing before and after deployment data at a 2.3-mile long corridor, in Pinellas County, Florida [13]. The results of the benefit cost analysis indicated benefits for the motorists from reduced travel times and fuel consumption [13]. HDR Engineering, Inc. reported improved travel times after the implementation of InSync along a 6 signalized intersection corridor at the Town of Mt. Pleasant, SC [14]. Atkins evaluated the implementation of InSync along a 4-mile corridor with 11 signalized intersections at the City of Greeley, CO [15]. Findings indicated that the implementation resulted in reducing average travel times along the corridor and led to higher average speeds. Also, they reported that there was overall reduction of the average delay along the corridor and fewer stops per vehicle that resulted in better level of service of the facility [15]. Missouri DOT reports on an adaptive traffic signal system installed in 2010 along a 12-signal and 2.5-mile arterial, located in Lee's Summit, MO [16]. The deployment results indicted savings in travel time ranging from 0-39%. However, the implementation yielded a slight increase in delay for minor street traffic [16].

2.2 Synchrogreen Adaptive Traffic Signal Control System

2.2.1 Introduction

SynchroGreen is a real-time Adaptive Signal Control Technology (ASCT) solution developed by Trafficware. It considers side-street, pedestrian traffic and mainline traffic in providing an adaptive solution to the varying traffic dynamics. Trafficware claims the following features for SynchroGreen [17]:

- "Adjusts traffic signal timing in real time based on traffic demand."
- "Utilizes three optimization engines to allocate green time and promote better traffic flow."
- "Compatible with existing traffic control infrastructure, including many common traffic controllers and various forms of detection."
- "Allows user selection of various strategies to facilitate balanced traffic flow, progression bandwidth, and critical movements."
- "Adaptive traffic control seamlessly integrates with Synchro and SimTraffic for modelling and evaluating different system settings before deployment."

2.2.2 System Compoents

SynchroGreen consists of three key components: the management system (server), local traffic controllers and the vehicle detection units [17]. According to Trafficware, the management system is also called the Signal System Master (SSM) with the SynchroGreen Central Server Software installed on the Window PC Server. The local traffic controllers are called Signal System Locals (SSLs). The SSM is responsible for processing and calculating the updated signal timing plans while the purpose of SSLs is to gather detector data and execute the commands from SSM. When in operation, communication between SSM and SSLs occurs every several seconds to guarantee the accuracy of the signal timing.

2.2.3 Signal Control Optimization frameworks

The primary goal of the SynchroGreen algorithm is to minimize the network delay while providing reasonable mainline progression bandwidth [17]. SynchroGreen also provides three different adaptive control modes to cater to potential needs. The Balanced Mode provides for an equitable distribution of green time with reasonable mainline bandwidth. The Progression mode gives priority to mainline progression. The Critical Movement Mode weights more heavily the identified critical movements. In summary, SynchroGreen follows a more traditional approach to signal control optimization, and utilizes the current traffic conditions to optimize the phase allocation (splits), period (cycle length) as well as start time (offsets) in real time.

Phase Allocation

Phase allocation determines the amount of green time each phase should receive. A targeted phase allocation time is initially calculated based on the green utilization, which is the duration of time that the current movement is served assuming saturation flow levels. The estimation of green utilization utilizes the stop-bar detectors after being calibrated considering their sizes, positions and the prevailing vehicle speed. However, the targeted phase allocation is not necessarily the final phase allocation that is sent to the controllers. The actual phase allocation will rely on the targeted period (or cycle length) of the intersections.

Period

Period is the adaptive counterpart of cycle length in a traditional coordination system. After the targeted phase allocation is determined, SynchroGreen establishes the targeted period of each intersection by constructing the respective ring-and-barrier diagrams. The intersection with the highest targeted period will serve as the critical intersection. This period of the critical intersection is assigned to all other intersections, and the actual phase allocation is enlarged proportionally to the previously targeted phase allocation.

Start Time

Start time consists of lag time and travel path. The lag time is similar to the concept of offset in traditional signal coordination. In SynchroGreen, lag time is dynamically modified based on detected traffic conditions, the determination of which often considers the existence of queuing and platoon arrival distribution which is extracted from advance detectors. The travel path is selected based on the predominant travel direction. It can vary by time-of-day and determines when each SSL should receive the updated timing plan. This maintains coordination in the order in which the platoon is expected to arrive at downstream intersections.

2.2.4 System Requirements

SynchroGreen can operate using existing infrastructure. SynchroGreen software supports 2070 and ATC-type traffic controllers and will operate on traffic controllers from various vendors. SynchroGreen requires advance detection on the mainline and stop-bar detection for each lane in order to better monitor and predict the traffic conditions. However, it supports any common detection technology such as loops, video and advance radar, and also allows for multiple detection methods to be used at the same time at an intersection.

2.2.5 Software Packages

Based on the software capability and the number of adaptive intersections that need to be served, SynchroGreen is available for three different levels [18].

SynchroGreen Lean – "includes the Local Intersection Software and Central Server Software, and provides a web-based interface for monitoring and controlling the system. This option is an economical way for a city to experience the benefits of adaptive traffic control." This package is used in the Newberry Road study corridor.

SynchroGreen Premium – "includes the local intersection software and enhanced central server software, and operates up to 150 intersections. It provides agencies with the ability to analyze real-time system performance, create detailed reports, log system calculations, and much more. This solution is designed to be easily integrated as part of federally-funded adaptive traffic control projects."

SynchroGreen Enterprise – "integrates directly with your ATMS.now central management system and also qualifies for federal funding. It allows agencies to operate up to 150 adaptive intersections and 9,999 total intersections."

2.2.6 Case Studies

Since the introduction of SynchroGreen, several field implementations have been conducted. According to Cheek et al. [19] , SynchroGreen was deployed along a 1.7 mile arterial in Seminole County, Florida in 2012. The corridor consists of 12 signals and features large fluctuation of day-to-day traffic flow. An over 35% reduction of delay on the arterial was observed after installing SynchroGreen; side-street traffic also benefited by an average of 19% delay reduction. Another SynchroGreen system was recently deployed at nine intersections along Glades Rd. in Boca Raton, Florida [20]. Results indicated that SynchoGreen efficiently handled the flow variation both at hourly and daily levels. Travel time was reduced by a range of 2.4% to 8.6% in three of the four study segments. However, not many of the improvements were found statistically significant.

2.3 Other Advanced Signal Control Technologies

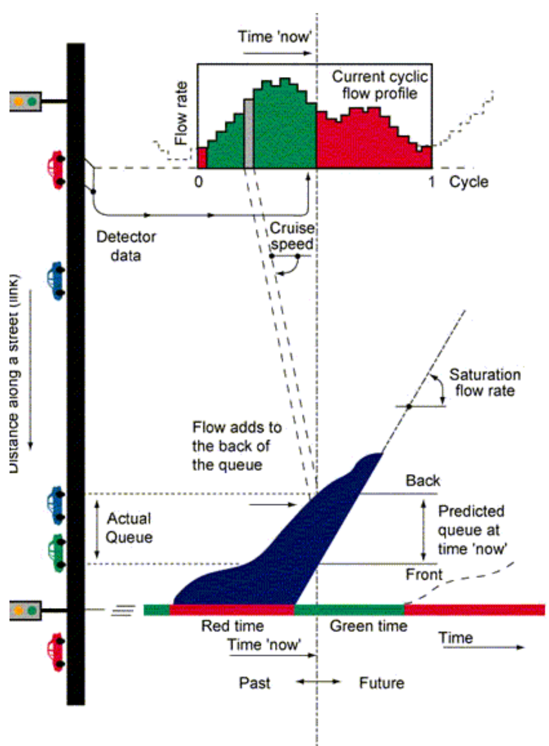
This section introduces and discusses other adaptive signal control methods found in use in the United States.

2.3.1 Split Cycle Offset Optimization Technique (SCOOT)

SCOOT is an adaptive traffic control system originally developed by the Transport Research Laboratory (TRL) [21]. Its algorithm is based on the TRANSYT optimization program (TRANSYT 7F is its US version.) SCOOT is installed in a central computer and has three optimizers that compute the best signal plan for the network based on detected traffic demand on all approaches to the system intersections. The optimizers are used to continuously adapt these parameters for all intersections in the SCOOT controlled area, aiming to minimize wasted green time at intersections as well as to reduce stops and delays by synchronizing adjacent sets of signals.

The operation of the SCOOT model is summarized in Figure 2.5. SCOOT obtains information on traffic flows from detectors, which are typically required on every link. Their location is important and they are usually positioned at the upstream end of the approach link. Inductive loops are used most often, but other methods are also feasible.

SCOOT receives traffic information and converts the data into its internal units and uses them to construct "Cyclic flow profiles" for each link as shown in the top left of Figure 2.5. The data from the model are then used by SCOOT in the three optimizers which are continuously adapting three key traffic control parameters - the cycle time, the splits, and the offsets. The cycle time optimizer computes an optimum cycle length for the critical intersection in the network. The split optimizer then assigns green splits for each intersection based on this cycle length and the offset optimizer calculates offsets. Phase sequence is also optimized.



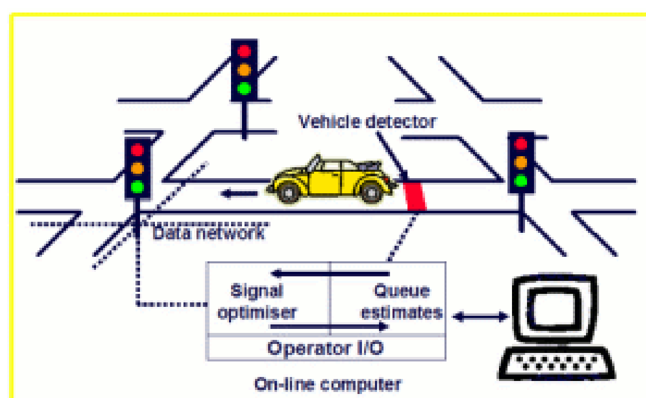


Figure 2.5 SCOOT Operation Procedure [22]

SCOOT was originally designed to control dense urban networks, such as large towns and cities. It has also been successful in small networks, especially for areas where traffic patterns are unpredictable. It is used extensively throughout the United Kingdom (England, Ireland, and Wales) as well as in other countries (including USA, China, Canada, Brazil, Thailand, Chile, etc.). In the US, SCOOT has been implemented in Ann Arbor, MI, Minneapolis, MN, Arlington, VA, Orange County, FL, Santa Barbara, CA, San Diego, CA, Oxnard, CA and Anaheim, CA.

Some of the advantages of SCOOT are that the cyclic flow profiles are created on-line and are updated every four seconds. It optimizes phase sequence, and estimates queue lengths based on flow-occupancy profiles from upstream detectors. It uses the profiles to determine splits and offsets for the next cycles as well. SCOOT has interfaces for CORSIM, S-Paramics, VISSIM, and Aimsun.

One of the drawbacks of SCOOT is its inability to handle closely-spaced signals. Due to its particular detection configuration requirements, it requires some time to detect vehicles and estimate arrivals from the upstream detectors. It is primarily designed to react to long-term, slow variations in traffic demand, and not to short-term random fluctuations.

2.3.2 Sydney Coordinated Adaptive Traffic System (SCATS)

SCATS is an intelligent transportation signal control strategy that automatically selects signal control plans from a background library in response to the detected traffic demands. Its objective is to achieve maximum throughput while minimizing stops and delay**.** It has been successfully deployed on arterial roads, downtown grid networks, and at small groups of intersections [4] [23] [24]. SCATS was developed by the Roads and Traffic Authority (RTA) of New South Wales, Australia (the former constituents of the Roads and Maritime Services) in the late 1970s. It is maintained by Roads and Maritime Services.

The input data for SCATS are collected by a system of traffic detectors. Two basic measures from detectors are used to adjust signal timings: degree of saturation and traffic flows. The system uses sensors at each traffic signal to detect vehicle presence in each lane and pedestrians waiting to cross at the local site. Sensors may be inductive loop detectors embedded in the pavement or video image devices mounted overhead on the signal strain poles. Pedestrian sensors are generally push buttons. SCATS is designed to automatically calibrate itself, and it is a cycle-by-cycle system that optimizes cycle length, split, and offset.

SCATS has been implemented in 27 countries worldwide, including Australia, Bangladesh, Brazil, Brunei, Chile, China, Ecuador, Fiji, Indonesia, Iran, Ireland, Jordan, Laos, Malaysia, Mexico, New Zealand, Pakistan, Philippines, Poland, Qatar, Saudi Arabia, Singapore, South Africa, Thailand, USA and Vietnam. In the US it has been implemented in Oakland County, MI and Newark, DE.

SCATS supports automatic reconfiguration of the subsystems based on predefined criteria. The developers indicate that SCATS replaces the manual collection of data which are required for road planning and provides a greater volume of original data with good accuracy level. It can interface with S-Paramics, VISSIM, and Aimsun.

Some of the limitations of SCATS are that it relies upon plan selection with some local adaptation. The cycle time and split are updated each cycle. It does not provide arrival prediction, queue estimation and phase sequence optimization so the system acts only reactively to an identified traffic pattern.

2.3.3 Los Angeles Adaptive Traffic Control System (LA-ATCS)

LA-ATCS is a computer-based traffic signal control system which provides fully automated traffic responsive signal control based on prevailing real-time traffic conditions [25]. LA-ATCS was first deployed as part of the Automated Traffic Surveillance and Control (ATSAC) Center in 1984 for the Los Angeles Olympic Games. The PC window-based system was completed in 1999. It has been implemented in over 3,000 intersections in the city of Los Angeles.

There are three operation modes in LA-ATCS: adaptive, time-of-day and operator control. In the adaptive mode it requires current flow conditions as input to determine a common section cycle time, splits, and offsets. In the time-of-day mode it operates on fixed-time plans as determined by the engineer. In the operator control mode it is used to handle traffic in special cases.

The input data (primarily flows) are collected by the detectors upstream of the stop bar for each system intersection. LA-ATCS automatically selects the intersection with the highest level of traffic, and the minimum and maximum cycle length and splits are determined by the engineers. Offset is also optimized at the end of the optimization process.

An advantage of this system is that cycle lengths can be different for each intersection. Any of its three optimized measures (cycle length, split and offset) can be disabled for selected links in a section. Also, it can interface with CORSIM (offline post-processing interface) and applies a set of logics to handle oversaturated traffic conditions in its network.

Its limitations are that it updates the cycle time and split each cycle which can lead to frequent transitions, and that it does not provide phase sequence optimization.

2.3.4 Real Time Hierarchical Optimized Distributed Effective System (RHODES)

RHODES is an adaptive traffic control system developed in the 1990s with FHWA support at the University of Arizona. The system is now managed by Siemens traffic solutions. RHODES uses input sensor data from detectors, AVLs, transponders, and so on. It produces real-time predictions of traffic flow and “optimally” controls the flow through the transportation network, using phase timing [26]. A schematic of the RHODES operation is shown in Figure 2.6.

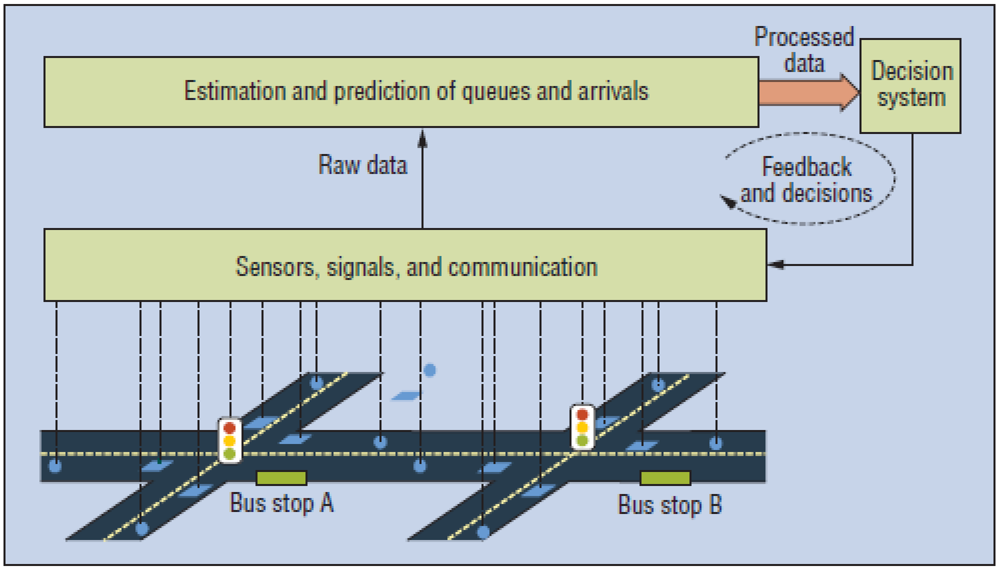


Figure 2.6 A simplified diagram of RHODES operation [26]

RHODES employs "proactive" traffic-adaptive signal control architecture, which decomposes the traffic control problem into sub-problems that are interconnected hierarchically (dynamic programming). Then, it predicts traffic flow at appropriate resolution levels (individual vehicles, platoons, transit vehicles, emergency response units, and trains) to enable proactive control. RHODES uses a data structure and computation and communication approaches that allow fast solution to solve those sub problems.

RHODES uses a three-level hierarchy for characterizing and managing traffic. It explicitly predicts traffic at these levels utilizing detector and other sensor information. It requires lane traffic data (e.g., through detectors- both stop line and upstream), real-time communication to/from processors, and PC-level computational capability.

The highest level in the hierarchy is a "**Dynamic network loading**" model which captures the slow-varying characteristics pertaining to the network geometry and the typical route selection of travelers.

Based on the traffic load on each particular link, RHODES allocates green time for each demand pattern and each phase. These decisions are made at the middle level of the hierarchy, referred to as "**Network flow control**."

Given the approximate green times, the "**Intersection control**" at the third level selects the appropriate phase change epochs based on observed and predicted arrivals of individual vehicles at each intersection as shown in Figure 2.7.

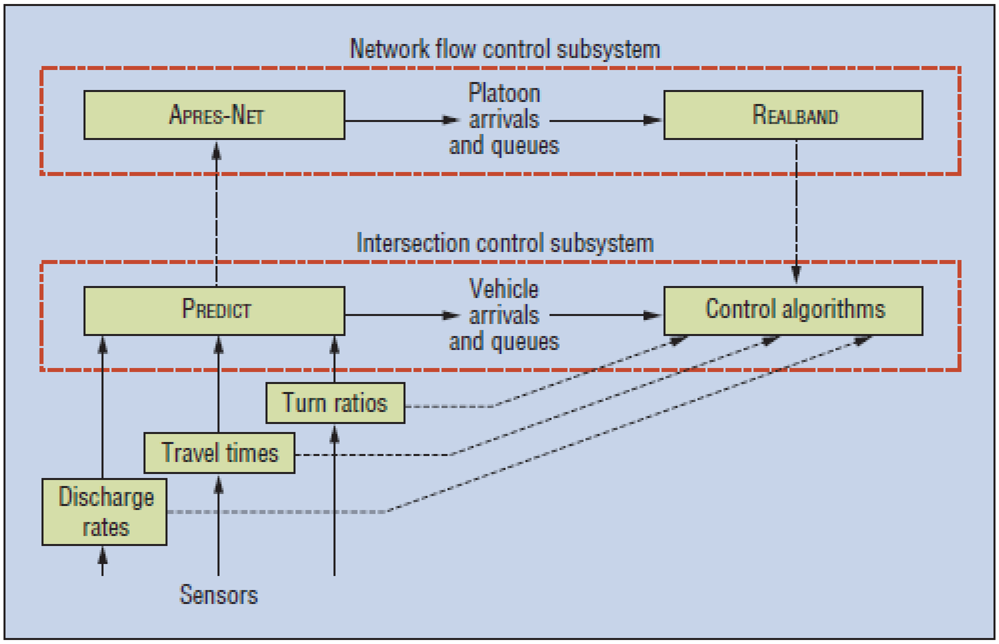


Figure 2.7 Middle and lower levels of RHODES architecture [26]

RHODES has been employed in the following locations in the US: Seattle, Santa Clara County (CA), Pinellas County (FL) and two locations in Arizona. Before and after studies conducted in the Pinellas County application showed a 14% improvement in travel times.

According to Mirchandani and Wang [26], the strengths of the RHODES system are automated setup, amenable to lab testing and consistency with traffic response objectives. RHODES is suitable for use in undersaturated arterials and widely-spaced grids. It can be applied as a signal control strategy for diamond interchanges.

2.3.5 Optimized Policies for Adaptive Control (OPAC)

The "Optimized Policies for Adaptive Control" (OPAC) strategy is implemented within the real-time adaptive control system (RT-TRACS), achieving the dual notion of "individual intersection control and coordinated control of intersection in a network" [27]. The University of Massachusetts at Lowell developed this strategy in the mid 1980's [28], aiming at introducing the first demand-responsive signal control. OPAC is designed for implementation on single intersections as well as arterials and networks. The OPAC strategy is essentially minimizing the objective function of the total intersections' delay and stops by adjusting the traffic signal timing through a dynamic optimization algorithm, over a pre-specified horizon [27].

According to the developers, the important advantages of the strategy is that it provides better results compared to the implementation of the TOD, off-line strategies and that it is truly demand-responsive, adapting to the real traffic conditions [29]. Four versions of OPAC strategy have been developed with each one minimizing intersections' performance measures, and constrained only by the minimum and maximum phase lengths. OPAC-1 (developed in 1979) introduced dynamic programming techniques for signal timing. However, this approach was not implemented in real time because processing real time data was not achievable at that point [28]. OPAC-2 (1980) applied an "optimal sequential constrained search methodology" in order to exhaustively search all the possible combinations of valid switching times and achieve OPAC objectives [29]. In OPAC-3 (1981), traffic patterns are projected based on upstream data, and the optimal switching times are optimized for the whole horizon. This new approach introduced the dynamic revision of decisions based on the most recent data [29]. OPAC-4 (1995) incorporates the signal coordination synchronization factor and was introduced within the RT-TRACS. Thus, the optimization control provided within this framework is continuous and on-line [29]. The fourth OPAC version developed flow profiles for each phase using a pre-specified horizon length with data from online upstream link detectors and projected data from smoothed volume counts [30]. These flow profiles are evaluated for each control setting and the decision is made in real time to extent the phase by 1-2 seconds or terminate it. This strategy allows for flexibility in the cycle length based on the virtual fixed cycle concept, a feature that is absent from adaptive systems with a fixed cycle length. The OPAC strategy can optimize up to eight phases within the dual ring configuration and all combinations of left turn lag/lead phasing [30].

The strategy was evaluated through a before-and-after study implemented by ITT Systems in the Reston Parkway test bed in Northern Virginia in 1997 and 1998 [29]. Results suggest that this type of control showed improvement on the order of 5% to 6% in average delays and stops compared to the time of day control used previously [27]. The implementation also revealed the effectiveness of the strategy during instances of loss of communication to the central monitoring system/control as well as instances when there is no communication between adjacent signals [27].

The OPAC strategy is capable of handling individual intersections as well as coordinated controlled arterials and networks. It allows prioritization of preemption control over OPAC, considering the movement of transit and emergency vehicles, recovering from preemption control immediately [30].

2.3.6 Adaptive Control Software Lite (ACS-Lite)

The "Adaptive Control Software Lite" is an initiative developed and funded by the FHWA in order to address the need to have widely-deployable, low-installation and operational cost adaptive control systems in the United States [31]. This "on-street" software is mainly used for monitoring traffic signal performance and adjusting signal timing for linear arterials [32]. The ACS-Lite software can control up to 16 consecutive intersections in a loop. The system is constantly updated with field data from upstream and approach detection, providing information on each intersection's performance. The main goal of the software is to be adaptive to the changes of the traffic but, at the same time, maintain the time-of-day (TOD) schedules that have been specified, complying with all the traffic engineering principles that have been set [32]. The software is successfully integrated with CORSIM simulation for testing purposes and adheres to the NTCIP communication protocol, incorporating the concepts of interoperability and interchangeability [29]. It operates in a closed-loop control system, as sensors monitor the system's outputs and feed the data into the controller in order to adjust the operation, if necessary. ACS-Lite deployment requires a low-cost upgrading to local intersection controllers, and agencies can retain the advantage of the familiarity with the controller software they currently use. ACS-Lite is flexible with the controller firmware update as well as the type of detectors used in the field. ACS-Lite needs stop line detectors for each phase to be installed in the field [32].

The adaptation logic of the ACS-Lite is based on making incremental adjustments to splits and offsets as often as every 5 to 10 minutes (which is the optimization step.) More specifically, during each phase, split adjustments are made based on the measure of utilization (v/c ratio) of each phase [31]. The same cycle length is maintained based on the time-of-day scheduler and the traffic engineer's judgement. Fully actuated gap-out and coordination logic are used by the controllers' software together with ACS-Lite during every cycle, so as to manage the duration of each split. Phase status data are obtained once per minute from the NTCIP detector (which can be loop or other point detection). The software detects the occupancy of each detector on coordinated approaches during the green and red intervals of each phase and the software determines whether traffic is using all the phases' split time. Optimization algorithms reallocate split times from phases that do not use their entire split time to phases where more split time is needed, attempting to balance the v/c ratio for all the phases [32]. This optimization is constrained by minimum and maximum green times and pedestrian intervals. With respect to the offset adjustment logic, the changes to the offset time values are quite small per cycle (from 2 to 5 seconds earlier or later). This type of adjustment is based on cycle flow profiles which are compiled by the monitoring of advance loops on progression approaches [31]. ACS-Lite develops a statistical flow profile using the data collected from the detectors and optimizes the offset, implementing it at the coordinated phase [33]. The frequency of the adjustments can also be controlled by the traffic engineer supervising the control process each time.

The ACS-Lite software has been independently evaluated in a simulation environment and in field studies. Using CORSIM, the controller was adjusted in order to handle multiple intersections' coordination. Results in this simulation test case indicate savings up to 4% in control delay and traffic travel time compared to suboptimal offset values. Compared to suboptimal split values, travel time ranged from 4.9% shorter to 6.8% longer [31]. Initial field tests were conducted evaluating the system's operation with different signal controller manufacturers. Test beds were deployed in 2006 at Gahanna, Ohio; Houston, Texas; Bradenton Florida, and El Cajon, California [31]. These tests indicated travel time savings up to 11%, as well as delay time reductions up to 35%, and fuel consumption reductions and cost benefits [31]. The Bradenton, FL location is unique in that it is an L-shaped corridor with two adjoined, perpendicular arterials. At this location, an 11% decrease in travel time and a 28% decrease in number of stops were found after implementation.

The ACS-Lite software was designed specifically to minimiize procurement, operations and maintenance costs. ACS-Lite can also respond to unexpected incidents and accommodate changes regarding the signal time monitoring scheme [31].

2.4 Best Practices in ASCT

Adaptive signal systems have been applied in many areas of the US, including Florida. Some of the implementations and the experiences reported are discussed below, followed by an overview of lessons learned and best practices observed in these studies.

2.4.1 Pinellas County, Florida (RHODES and OPAC)

Pinellas County, Florida has experience in multiple adaptive signal control systems. The RHODES system was implemented on a 17-intersection corridor with average intersection spacing of ¼ to ½ mile. According to an evaluation of the system [34], the RHODES configuration parameters had to be modified via text files and were found to be difficult to use for operators. Also the vendor support from University of Arizona was found to be inconsistent and the system was discontinued.

Pinellas County additionally tested OPAC on an 11-intersection corridor with average intersection spacing of 1 mile. 'Major' cycle and offset adjustments made by OPAC resulted in skipping of phases and there was a VME communication failure between the OPAC single board processor and the 2070 controller CPU. Eventually the system was upgraded to Econolite Centracts Adaptive (no specifics have been found in the literature on this approach).

In both cases, training of the operators and maintenance of detectors were found to be vital factors in achieving success with the system. The county currently uses In-sync, Econolite Centracts Adaptive and ACS-Lite.

2.4.2 City of Irvine, California (SWARM/OPAC)

A systematic evaluation of the performance and effectiveness of a Field Operational Test (FOT) of an integrated corridor-level adaptive control system was attempted from fall 1994 through spring 1999 in the City of Irvine, California. This test included OPAC as well as other new technologies for ramp metering, ITS and new 2070 ATCs. This study began developing software prior to the purchase of the new test controllers, and noted that basing the software on hardware that was still in development complicated the software porting tasks. The study concluded that lack of consistent communication and training prevented the arterial consultant from effectively deploying the test technology. [35]

2.4.3 VDOT Pilot Implementation (InSync)

The Virginia Department of Transportation (VDOT) employed the InSync system along thirteen corridors in Virginia between years 2011 to 2015 [36]. Corridors were selected on the basis on criteria like variability in traffic patterns, heavy side street flows, conflicts with other modes and support from local authorities.

Floating car probes, blue tooth and INRIX data were used. Travel times, speeds and number of stops were considered as the performance measures. A statistically significant 17% reduction in total intersection crashes and 71.8% reduction in total stops were found. About 70% of the corridors showed statistically significant reduction in travel time. The system generally showed improvement in the mainline performance but it was not significant when intersections were oversaturated and/or when the existing system (Time of Day) already performed well. Although corridors were retimed 3-5 years ago, no significant development occurred in the immediate vicinity of them, hence researchers assumed that it is safe to attribute the improvements to ASCT.

Widely-spaced intersections had problems with platoons breaking up and a break in communications was found to affect performance significantly. Side street delays generally increase (commonly between 5 to 10 s) when ASCT is deployed, although there is usually a net reduction in overall corridor delay. In quantifying improvements in operations, the agency reported that off-peak and queue data were found to be useful. INRIX data showed 15% to 20% improvement in travel time reliability.

The report noted that the public expectation was too high when the system was announced, and therefore advised to lower public expectations in urban areas, as the advantages of adaptive control are not aimed at the most highly congested time periods.

2.4.4 Park City, Utah (SCATS)

UDOT implemented ATCS in Park City as a pilot project on a 12=intersection corridor [37]. The following suggestions were made in the report after the evaluation of the project:

- There was no evaluation plan in advance of implementation and therefore the agency had to use existing limited "before" data. For such cases, the report recommends limiting such evaluations to comparing only individual intersections as opposed to comparisons of the performance of the entire system before and after SCATS is installed.
- The report recommended conducting a "with/without" evaluation instead of a "before/after" approach – this approach depends on the political decision to turn SCATS off for few weeks.

A VISSIM network simulation was developed and calibrated to get a larger data set in order to evaluate alternative geometric configurations. It also gave UDOT the opportunity to explore various traffic signal scenarios and future expansions of the system (not only in Park City, but on any other network), such as: "before/after" approach, system expansion and "freak" events.

2.4.5 City of Surrey, Canada (MAC)

The City of Surrey implemented their pilot ATSC project using "Multi-criteria Adaptive Control" system for 7 closely-spaced intersections on a single corridor [38]. Some of the important lessons that were reported are:

- To maximize the benefits of deploying ASCT, arterial corridors and signalized intersections with more highly variable and/or unpredictable traffic volumes should be selected as preferred locations.
- The length of the arterial corridor must be long enough to appreciate the travel time variations.
- Techniques to further fine-tune the configuration data and/or enhance the ASCT algorithms to improve the duration of the transition periods should be investigated. As a

minimum, the local traffic signal controllers should always be configured to use the "short-way" offset transition method, which shortens the cycle in order to make offset corrections.

- Robust and reliable communications between the Central Server and all MAC Adaptors in the field is a key consideration in the deployment of the ASCT system (as the ASCT algorithms cannot run until all the MAC Adaptors have reported data for the last completed cycle).

2.5 Summary of Best Practices and Lessons Learned

The following is a summary of best practices identified in previous studies:

- Robust communication and adaptability of controllers to any new adaptive signal control systems are both important factors in the effectiveness and functionality of any system.
- Functioning and maintenance of detectors is key from the operating agency's perspective.
- Transition from emergency events to regular patterns should be studied; any pre-emptive events during data collection would be useful in this regard.
- Since there are sometimes problems during pre- and post- congested periods, collecting off-peak and queue data is necessary to evaluate the effectiveness of the system.
- Infrastructure, such as communication cables, controllers and detection should be thoroughly tested before the data collection.
- The old systems must be re-calibrated and coordinated for the "before" data collection, so that the benefits derived can be measured more accurately.
- "With/without" evaluation instead of a "before/after" approach must also be considered.
- Whenever available, high resolution signal timing and detector data should be used and whenever necessary, simulation tools should be employed for rigorous evaluation of the system. Phase splits and other controller information can be useful in evaluating the access equity of the intersections [1].
- Travel demand can, and typically does change between before and after studies due to a variety of reasons such as site development and seasonal changes. This variability is often mitigated by collecting data on the same days of the week and within a given season.

2.6 Conclusions and Recommendations

The two adaptive systems of focus in this study, InSync and SynchroGreen, are at the leading edge of development among ASCTs. InSync approaches adaptive control in a different way than most other systems in allowing more possible phasing and patterns, while SynchroGreen optimizes currently used phasing based on changing traffic patterns. Both systems have existing user bases with positive experiences and target improvements in locations with large variability of traffic flows by time of day and day of the week.

The University of Florida will be evaluating the test corridors over the next two years with the principles mentioned in this document. Maintenance of the systems and selection of appropriate measures of effectiveness, time and points of data collection will be helpful in evaluating the systems.

3. TRAFFIC ENGINEERING ANALYSIS

The "before and after ASCT" data collection included geometric and signal control information, as well as demand data, turning movements, queuing patterns, the presence of preemption, and other information that may affect signal coordination (presence of pedestrians and bicycles, presence of heavy vehicles, speed limits, etc.).

Prior to the data collection, the research team identified all pertinent performance measures that should be collected before and after installation of ASCT. These measures included: arterial travel time, delay at each signal, mid-block delays, and quality of existing signal control and coordination. When previously collected information was available by FDOT and other agencies, the research team included such data in the analysis of those sites, and collected in the field the remaining data. Data were collected during three separate 2-hour time periods at each site: the morning Peak (AM), the evening Peak (PM), and an Off Peak period. The research team coordinated with the local agencies to identify suitable study periods for each corridor, and to obtain any relevant data in advance of the data collection.

Travel time data were primarily collected using our instrumented vehicle in a floating vehicle study. Travel time is estimated based on the time difference between arrivals at the first and last intersections for each corridor, when the vehicle has crossed the intersection at a speed approximately equal to the average operating speed. The number of runs along each corridor were a function of the time it took to traverse it during each period of analysis. A minimum of 5 runs were conducted at all corridors. In addition to travel time data collection, turning movement counts and queue lengths were collected at two critical intersections for each corridor. These were identified in consultation with the corresponding city, county, or managing agency. Based on these data, the following performance measures were obtained for the designated Peak and off-Peak study periods: queue length, queue to lane storage ratio, and passenger car equivalent (pce) flows.

Table 3.1 and 3.2 provide the summary of changes in performance measures for the corridors. As shown:

- Four of eight corridors (Panama City Beach Parkway, 23rd St, US 17 and 66th St) showed clear improvement in all performance measures on major streets, with increases in queues on minor streets
- Of the remaining four, SR 70 and University Parkway showed improvement upon further analysis:
    - SR 70 intersects with US 301, which carries heavy volumes and is a part of an ASCT implementation. Hence cross street travel time was also considered.
    - Intersections affected from DDI construction were removed from analysis for the University Parkway corridor.
- ASCT was removed from Newberry Rd, shortly after installation, as it was adversely affecting operations.
- E. Van Fleet Dr. in Bartow did not show clear operational improvement. This may be due to its unusual shape, which includes major left turns within the corridor.

The data and analysis for each site have been included in Appendices A through H. The next subsection presents a statistical comparison of selected traffic measures, while the last subsection presents the regression analysis conducted to show the relationship between site characteristics and operational improvements with ASCT implementation.

Table 3.1 Summary of Performance Measures for All Corridors (Direction of Change)

Corridor	Number of Signals	Length (mi)	ASCT Type	Performance Measures				
				Travel Time	Delay	Queue Length	Volume	Comment
Newberry Road-Gainesville	12	1.45	SynchroGreen	Increased	Increased	Increased	Increased	- Short links adversely affect detection
US 17/92- Deland	5	2.28	InSync	Decreased	Decreased	Decreased/ Increased	Increased	-Well-supported by the vendor
Panama City Beach Parkway-Bay County	10	8.5	InSync	Decreased	Decreased	Decreased	Increased	- Overall improved performance
University Parkway-Sarasota/Manatee County	19	7.8	InSync	Decreased /Increased	Decreased	Decreased /Increased	Decreased (Work Zone)	-While overall performance of the corridor slightly improved, operations at some intersections (close to DDI) deteriorated
23rd street-Panama City	9	2.0	InSync	Decreased	Decreased	Increased	Increased	- Travel time reduced - Side streets suffered significantly
66th Street- Pinellas	12	5.0	InSync	Decreased	Decreased/ Increased	Decreased	Increased	
SR 70- Manatee	21	9.2	SynchroGreen	Decreased /Increased	Increased	Increased/ No Change	Decreased	The system extends to cross street US 301, where the travel time and queues decreased
E. Van Fleet Drive & N. Broadway Ave.- Bartow	5	1.1	InSync	Decreased /Increased	Decreased /Increased	Increased	Increased	Corridor includes major left turns along the mainline

Table 3.2 Summary of Performance Measures for All Corridors (Magnitude of Change)

Corridor	Number of Signals	Length (mi)	ASCT Type	Performance Measures			
				Travel Time	Queue Length	Volume	Comment
Newberry Road-Gainesville	12	1.45	SynchroGreen	(D1) -22.70% (D2) -0.01%	Major Through 20.48% Minor Through 36.8%	Major Through 9.98% Minor Through -4.83%	- Would be more effective with advance detection
US 17/92- Deland	5	2.28	InSync	(D1) 10.57% (D2) 25.0%	Major Through -12.68% Minor Through 33.69%	Major Through 3.96% Minor Through 11.87%	-Well-supported by the supplier
Panama City Beach Parkway-Bay County	10	8.5	InSync	(D1) 5.7% (D2) 13.7%	Major Through -21.58% Minor Through 3.50%	Major Through 5.00% Minor Through 1.75%	- Overall improved performance
University Parkway-Sarasota/Manatee County	19	7.8	InSync	(D1) -11.22% (D2) 25.17%	Major Through -54.40% Minor Through 19.67%	Major Through -9.67% Minor Through 3.33%	-While overall performance of the corridor slightly improved, operations at some intersections deteriorated
23rd street- Panama City	9	2.0	InSync	(D1) 7.57% (D2) 13.07%	Major Through 3.58% Minor Through 16.33%	Major Through 10.08% Minor Through -1.83%	- Travel time reduced - Side streets suffered significantly
66th Street- Pinellas	12	5.0	InSync	(D1) 10.56% (D2) 20.31%	Major Through -35.58% Minor Through 25.83%	Major Through 7.50% Minor Through -16.42%	
SR 70- Manatee	21	9.2	SynchroGreen	(D1) -5.80% (D2) 4.78%	Major Through 1.00% Minor Through 4.00%	Major Through 3.00% Minor Through 0.00%	The system extends to cross street US 301, where the travel time and queues decreased
E. Van Fleet Drive & N. Broadway Ave.-Bartow	5	1.1	InSync	(D1) 12.82% (D2) -14.22%	Major Through 10.75% Minor Through 15.83%	Major Through 28.83% Minor Through -3.50%	Corridor has left turns in major street directions

3.1 Comparison of Traffic Performance Measures

In order to measure the effects of ASCT on the corridors' traffic operational conditions, several performance measures (PM) are evaluated. The change in travel time, major street throughput, and minor street throughput for each corridor are summarized in Table 3.3 and 3.4. The change rate of each measure is calculated as:

$$Change = \frac{PMi - PMi'}{PMi'} * 100\% \qquad (1)$$

The PM_i and PMi' represent the i^{th} PM after and before the implementation of ASCT respectively, where $i = 1,2,3,4,5,6,7$ and it represents the specific performance measure evaluated (travel time, main street throughput-through, main street queue length-through, main street throughput-left, main street queue length-left, minor street throughput-left, and minor street queue length left). The travel time changes are summarized in Table 3.3. D1 indicates the travel time change for direction 1 (EB/ NB) and D2 indicates the travel time for direction 2 (WB/SB). Green shading represents improved travel times with ASCT, while red shading represents worsening conditions with ASCT. Note that ASCT was not performing as expected along the Newberry Rd. (see Appendix B regarding the operational performance of this corridor), and therefore it is not considered in the calculation of the overall average.

All corridors show travel time improvement in at least one direction of travel and four corridors show improvement in both directions. Of the four, US 17 in Deland, 66th St in Pinellas, and 23rd St. in Panama City, showed statistically significant improvement in travel time in both directions. Overall, an average reduction of 9.36% in travel time (both directions) was observed across all study corridors. Detailed information on each of these is provided in Appendices A to H.

There were four instances where travel time increased after ASCT installation:

- Newberry Road EB, where the ASCT was eventually removed
- University Parkway EB, which had a new DDI at the east part of the corridor under construction during the after study
- E. Van fleet in Bartow, where the mainline includes left turns
- SR 70, where the ASCT favored the higher-volume cross street US 301. Upon further analysis, the cross street (US 301) which also had an ASCT installation but was not part of the study corridor for this project, showed a 20% reduction in travel time.

While travel time is important to analyze in order to evaluate the driver perception of the corridor level of service, the flows and queues collected at two critical intersections in each corridor indicate how ASCT is serving the major and minor streets. Due to the limited availability of time and resources to collect data and to have uniformity across sites, it was decided to collect data at only two "critical" intersections. These were identified for each corridor with the help of local agencies and FDOT district staff.

Table 3.3 Travel Time Changes for Each Corridor, By Direction

Corridor	Travel Time Change (D1)	Stat. Sig.	Travel Time Change (D2)	Stat. Sig.	Comments
US 17/92	-10.57%	***	-25.41%	**	
Newberry Road	22.70%	-	-0.06%	-	Detection and other issues, system currently not in use
Panama City Beach (PCB) Parkway	-5.39%	-	-12.08%	-	
23rd Street	-8.26%	**	-13.60%	*	
University Parkway	11.22%	-	-27.86%	-	DDI construction and Mall upgrades during the “after” study
66th street	-10.56%	**	-20.31%	***	
SR-70	5.80%	-	-4.78%	*	US 301 (High volume cross street)- 20% reduction in travel time
E. Van Fleet and N. Broadway Ave (Bartow)	-12.82%	-	14.22%	-	D2 had left turns along the mainline
Overall	**-9.36%** (Average change in travel time for both directions)				

*** indicates the travel time change is statistically significant with $P\ value \leq 0.01$
** indicates the travel time change is significant with $P\ value \leq 0.05$
* shows the travel time change is significant with $P\ value \leq 0.10$
- shows the travel time change is insignificant with $P\ value > 0.10$

The throughput change and queue length change of the major street/through), major street/left, and minor street/through are presented in Table 3.4. The values reported represent the average change for an approach and for the given performance measure. For example, “average percentage change in major street thru flows” is an average of twelve values: percentage changes in flow in two major street directions (2) x three time periods (3) x two critical intersections (2).

The corridors where the throughput increases and the queue length decreases at the same time, which are the best-performing cases, are marked in green. The corridors that incurred throughput reduction and queue length increase, which are the worst-performing cases, are marked in red. The instances where there was no clear trend in the changes observed (i.e. both throughput and queues either increase or decrease together) are marked in beige.

As indicated in Table 3.4, when noticeable improvements are observed for the major street there is typically a deterioration in the minor street traffic operations. The 66th Street corridor in Pinellas County achieved the best improvement with 7.50% throughput increase and 35.58% reduction in queues on the major street (through). However, its minor street (through) had a reduction of 16.42% in throughput and an increase of 25.83% in queues. Some of the percentages in queues seem large, however they often correspond to short queues consisting of only a few vehicles. Additional information regarding queue lengths for each corridor is provided in Appendices A to H.

Overall, the ASCT are generally effective as they leads to increases in throughput (6.96%) and reduction in queues (15.57%) for the major streets. Although they result in increases in queues (16.98%) on the minor streets, the throughput remains almost the same (-0.69%) i.e. ASCT is able to maintain the same levels of side street flows despite an increase in queues.

Table 3.4 Throughput and Queue Changes at Critical Intersections

Corridor	Major Street (Through)		Major Street (Left)		Minor Street (Through)		Comments
	Flow/Throughput	Queue	Flow/Throughput	Queue	Flow/Throughput	Queue	
US 17/92	3.96%	-12.68%	1.18%	2.88%	11.87%	33.69%	
Newberry Road	9.98%	20.48%	24.50%	10.43%	-4.83%	36.83%	
Panama City Beach (PCB) Parkway	5.00%	-21.58%	0.67%	-17.83%	1.75%	3.50%	
23rd Street	10.08%	3.58%	20.17%	21.83%	-1.83%	16.33%	
University Parkway	-9.67%	-54.50%	-17.67%	-33.00%	3.33%	19.67%	One of the critical intersections not considered due to vicinity to construction in after study
SR 693	7.50%	-35.58%	-20.92%	-11.33%	-16.42%	25.83%	
SR-70	3.00%	1.00%	17.00%	34.00%	0.00%	4.00%	24% reduction in queue on US 301
E. Van Fleet and N. Broadway Ave (Bartow)	28.83%	10.75%	10.92%	38.83%	-3.50%	15.83%	
Overall	**6.96%**	**-15.57%**	**1.62%**	**5.05%**	**-0.69%**	**16.98%**	

3.2 Regression Analysis and Findings

In order to understand the relationship between the characteristics of the study corridors and operational performance with the ASCT, linear regression analysis is performed and the results are reported in this subsection. The regression models developed quantify the ability of ASCT to improve performance as a function the corridors' characteristics. The resulting models can also be used to anticipate the effect of ASCT on other corridors, providing guidance on which corridors are best candidates for ASCT implementation.

Table 3.5 shows the correlation between explanatory variables, and the correlation between each response variable (PM) and the explanatory variables.

In the regression analysis, the response variables are:

- Percentage change in travel time (PTT_Diff)
- Difference in queue storage ratio (QS_Diff)
- Percentage change in major street throughput (PTR_Diff)
- Percentage change in minor street throughput (PTRMI_Diff)
- Percentage change in intersection delay (PDelay_Diff)

The corridor characteristics evaluated and used as explanatory variables are:

- Speed limit (Speed.limit) in mph
- Number of lanes (X.lanes)
- Annual Average Daily Traffic (AADT) in vehicles
- Average distance between intersections (Intersection.Distance) in feet
- Length of the corridor (Length) in miles
- Number of signalized intersections (X.Signals) per mile
- Number of unsignalized intersections (X.Unsignals) per mile

In addition, the following variables were created as a combination of the variables above:

- Number of access points (access.point = X.Signals+X.Unsignals) per mile
- Nominal variables based on the length of the corridor: short (short_dis) if it is less than 2 miles; long if it is more than 8 miles (long_dis); medium (med_dis) if it is 2-8 miles long
- Operating speed (before) to free flow speed (speed limit) ratio (Speed.FFS)

The correlation matrix provides helpful information in determining the combination of variables to be used in predicting each of the independent (response) variables. For example, with ASCT application (correlation coefficients provided in the parenthesis):

- Corridors with higher AADT, show less improvement in queues (0.46)
- Medium length corridors (2 to 8 miles) show relatively higher improvement (reduction) in travel time than short and long corridors (-0.36)
- Corridors with higher intersection density show less improvement in queues (0.47)

The correlation matrix is also helpful in identifying the exploratory variables which interact or have similar effects on the response variables. This information is helpful in reducing the number of exploratory variables or creating a new combination variable. For example:

- Longer corridors (0.76) with more lanes (0.31) show higher speed (before ASCT is implemented) to FFS ratio
- The number of signalized and unsignalized intersections per mile have a high correlation (0.63) and produce a similar effect on the response variables. Hence it might be better to use a combined variable for these.

Based on the information obtained from the correlation matrix a stepwise linear regression analysis was conducted. Other types of regression models (log-log, log-linear) yielded similar results. Linear models were chosen for simplicity and ease of comparison. Out of five, only two response variables resulted in meaningful and statistically significant models: Percentage Travel

Time Change and Queue Storage Ratio Change (Major Street). The following subsections discuss each of these models.

Table 3.5 Correlation Matrix

	Speed.Limit	X.Lanes	Divide	X.Signals	X.Unsignals	AADT	Intersection.Distance	medium_dis	long_dis	access.point	Length	Speed.FFS	PTT_Diff	PTR_Diff	QS_Diff	PTRMI_Diff	PDelay_Diff
Speed.Limit	1.00	-0.04	-0.04	-0.86	-0.39	-0.50	0.38	0.09	0.62	-0.62	0.74	0.57	-0.35	-0.13	-0.32	-0.16	0.18
X.Lanes	-0.04	1.00	1.00	-0.18	-0.06	-0.26	0.29	0.29	0.22	-0.12	0.42	0.31	0.14	-0.11	-0.20	0.12	0.05
Divide	-0.04	1.00	1.00	-0.18	-0.06	-0.26	0.29	0.29	0.22	-0.12	0.42	0.31	0.14	-0.11	-0.20	0.12	0.05
X.Signals	-0.86	-0.18	-0.18	1.00	0.63	0.40	-0.60	-0.42	-0.46	0.84	-0.78	-0.47	0.40	0.29	0.35	0.08	-0.03
X.Unsignals	-0.39	-0.06	-0.06	0.63	1.00	-0.18	-0.89	0.00	-0.35	0.95	-0.38	-0.15	0.09	0.37	0.47	-0.10	-0.13
AADT	-0.50	-0.26	-0.26	0.40	-0.18	1.00	0.31	-0.25	-0.06	0.03	-0.23	-0.14	0.30	-0.29	0.46	0.08	-0.07
Intersection.Distance	0.38	0.29	0.29	-0.60	-0.89	0.31	1.00	-0.05	0.61	-0.86	0.64	0.41	0.02	-0.46	-0.23	0.07	0.15
medium_dis	0.09	0.29	0.29	-0.42	0.00	-0.25	-0.05	1.00	-0.45	-0.17	0.12	-0.07	-0.36	-0.16	0.02	0.05	-0.34
long_dis	0.62	0.22	0.22	-0.46	-0.35	-0.06	0.61	-0.45	1.00	-0.43	0.83	0.73	0.06	-0.19	-0.04	-0.14	0.31
access.point	-0.62	-0.12	-0.12	0.84	0.95	0.03	-0.86	-0.17	-0.43	1.00	-0.58	-0.29	0.22	0.37	0.47	-0.04	-0.10
Length	0.74	0.42	0.42	-0.78	-0.38	-0.23	0.64	0.12	0.83	-0.58	1.00	0.76	-0.15	-0.31	-0.04	-0.13	0.13
Speed.FFS	0.57	0.31	0.31	-0.47	-0.15	-0.14	0.41	-0.07	0.73	-0.29	0.76	1.00	0.15	-0.25	-0.07	-0.13	0.07
PTT_Diff	-0.35	0.14	0.14	0.40	0.09	0.30	0.02	-0.36	0.06	0.22	-0.15	0.15	1.00	0.11	0.14	0.19	0.11
PTR_Diff	-0.13	-0.11	-0.11	0.29	0.37	-0.29	-0.46	-0.16	-0.19	0.37	-0.31	-0.25	0.11	1.00	0.03	0.32	0.00
QS_Diff	-0.32	-0.20	-0.20	0.35	0.47	0.46	-0.23	0.02	-0.04	0.47	-0.04	-0.07	0.14	0.03	1.00	-0.09	-0.19
PTRMI_Diff	-0.16	0.12	0.12	0.08	-0.10	0.08	0.07	0.05	-0.14	-0.04	-0.13	-0.13	0.19	0.32	-0.09	1.00	-0.07
PDelay_Diff	0.18	0.05	0.05	-0.03	-0.13	-0.07	0.15	-0.34	0.31	-0.10	0.13	0.07	0.11	0.00	-0.19	-0.07	1.00

3.2.1 Linear regression model for predicting percentage travel time change

Table 3.6 presents the model predicting percentage travel time change ((After TT-Before TT)/Before TT). A negative percentage travel time change indicates travel time improvement. The number of signalized intersections per mile represents the density of signalized intersections along the corridor. Higher density limits the capabilities of the signal system to provide efficient traffic progression.

Corridors with higher operating speed (before ASCT) to FFS ratio limits the capabilities of the signal system to improve travel time. Higher speed (before ASCT)-FFS ratio means lower congestion levels to begin with, hence not much opportunity for improvement.

Table 3.6 Percentage Travel Time Change Model Results

	Estimate	Std. Error	t value	Pr(>|t|)
(Intercept)	-0.45527	0.10321	-4.411	6.34E-05
#Signalized intersection	0.04562	0.01064	4.288	9.40E-05
Speed/FFS	0.47904	0.15474	3.096	0.00337
R^2	0.3069			
Adj. R^2	0.2761			

Based on the above analysis, the percentage travel time change can be predicted as:

$$\begin{aligned}\%Travel\ time\ change \\ &= -0.4553 + 0.0456\#signalized\ intersections\ per\ mile \\ &+ 0.4790\frac{Speed}{FFS}\end{aligned} \tag{1}$$

3.2.2 Linear regression model for predicting queue storage ratio change (major street)

Queue storage ratio is the ratio of queue length over the lane storage capacity. Table 3.7 presents the model predicting Queue Storage Ratio Change (After Q/S – Before Q/S) on the major street at the critical intersections along the corridors. Negative queue storage ratio change indicates improvement.

Table 3.7 Queue Storage Ratio Change (Major Street) Model Results

	Estimate	Std. Error	t value	Pr(>|t|)
(Intercept)	-0.66750	0.11180	-5.973	3.43E-07
#access.points	0.01542	0.00384	4.018	0.000221
AADT	9.900E-06	0.00000	3.899	0.000319
R^2	0.4185			
Adj. R^2	0.3927			

The number of access point (includes both signalized and unsignalized intersections) per mile represents the density of intersections along the corridor. Higher density limits the capabilities of the signal system to improve traffic progression, and results in higher queue storage ratios. The AADT positively affects queue storage change, i.e., higher AADT results in longer queues.

Based on the above analysis, the queue storage ratio change can be predicted as:

$$\begin{aligned} &Queue\ storage\ ratio\ change\ (Major\ Street) \\ &\quad = -0.6675 + 0.01542\#access\ point\ per\ mile + 9.9 \\ &\quad \times 10^{-6} AADT \end{aligned} \tag{2}$$

3.3 Travel Time Reliability

Travel time reliability is an important component when analyzing corridor performance. Travel Time index (TTI), as an indicator of travel time reliability, is calculated for all corridors and for each time period:

$$TTI = \frac{Peak\text{-}Period\ Travel\ Time}{Free\text{-}Flow\ Travel\ Time}$$

TTI index represents the average additional time required during peak times as compared to travel times of light traffic. The difference in TTI before and after system installation is shown in Table 3.8. The average change in TTI was -0.13 across all sites.

All corridors except 23rd Street have higher travel time and hence TTI in the PM peak. The last column in Table 3.8 shows the percentage difference in TTI between the least congested and most congested periods for a given site. A higher number indicates very congested peaks (usually in the PM Peak). The three sites that have minimal difference between Off Peak and Peak congestion levels are US 17 (16%), Panama City Beach Parkway (9%) and 66th Street (11%). All three show the most improvement in travel times. Hence, the least improvement in travel time reliability was observed on corridors with oversaturated peaks.

Table 3.8 Travel Time Index (TTI) For All Corridors in Both Directions

Site	Time Period	Direction 1			Direction 2			TTI (Max-Min) /Min %
		Before	After	Difference	Before	After	Difference	
Newberry Rd	AM	1.94	2.00	0.05	1.75	1.93	0.18	60%
	Off	2.11	2.43	0.33	2.00	2.15	0.16	
	PM	2.59	3.73	1.13	3.26	2.92	-0.34	
US 17 Deland	AM	1.90	1.73	-0.18	1.93	1.71	-0.22	16%
	Off	2.02	1.80	-0.23	2.19	1.62	-0.58	
	PM	2.24	1.99	-0.25	2.52	1.63	-0.89	
Beach Parkway	AM	1.21	1.11	-0.10	1.44	1.51	0.08	9%
	Off	1.09	1.14	0.05	1.83	1.51	-0.32	
	PM	1.24	1.10	-0.14	1.69	1.34	-0.35	
University Parkway	AM	1.59	1.52	-0.08	1.43	1.38	-0.06	57%
	Off	1.72	1.96	0.24	1.86	1.61	-0.25	
	PM	2.21	2.67	0.46	3.33	1.80	-1.54	
23rd Street	AM	2.30	2.01	-0.29	2.25	1.70	-0.54	27%
	Off	2.86	2.63	-0.23	2.97	2.53	-0.44	
	PM	2.52	2.40	-0.12	2.59	2.51	-0.08	
66th Street	AM	2.08	1.77	-0.31	1.94	1.59	-0.36	11%
	Off	1.96	1.79	-0.17	2.16	1.72	-0.43	
	PM	2.16	1.98	-0.18	2.23	1.73	-0.50	
SR 70	AM	1.72	1.75	0.03	1.66	1.51	-0.15	32%
	Off	1.28	1.37	0.08	1.28	1.27	-0.01	
	PM	1.61	1.76	0.15	1.75	1.68	-0.06	
E.Van Fleet Drive & N. Broadway Ave.	AM	2.66	2.55	-0.12	2.59	3.22	0.63	22%
	Off	2.56	2.62	0.05	2.82	3.58	0.76	
	PM	3.52	2.46	-1.06	3.41	3.27	-0.13	

3.4 Conclusions from Traffic Engineering Analysis

The traffic engineering analysis of the study corridors shows that the implementation of ASCT led to an average overall reduction in travel time of 9.36% (excluding the Newberry corridor, as indicated above). All corridors show travel time reduction in at least one direction of travel and four corridors show reduction in both directions. The travel time reliability improved overall.

The ASCT also helps to increase major street throughput (6.96%) and reduce major street queues at the same time (15.57%). The minor street queues increase (16.98%) while the throughput remains almost the same (0.69%), i.e., ASCT is able to maintain the same levels of side street flows despite an increase in minor street queues.

The linear regression analysis shows that higher AADT, higher intersection density (signalized and unsignalized), and higher initial operating speed (before implementation of ASCT) results in less traffic operational improvement. Qualitatively, the sites that showed consistent improvements had minimal detection or construction issues, low volume side streets, and simpler geometry (for example, no left turns as part of the main corridor).

In summary, ASCT is not suitable for implementation for all types of corridors and traffic conditions. The analysis shows that ASCT would yield better performance and a higher return on investment when implemented on corridors with low intersection density, low volume side streets, and high demand but not oversaturated traffic conditions. It is important to address any detection issues that stem from the corridor design (for example, short distances between intersections, where detection may not be effective) before ASCT is selected for implementation. Construction along the corridor reduces the effectiveness of ASCT.

4. SAFETY ANALYSIS

While it is essential to conduct traffic-engineering analysis to assess the improvements made by implementation of ASCT on traffic operations, it is also important to understand the impact of such technology on traffic safety. Often the introduction of new traffic patterns result in increase of rear-end crashes. In this chapter, we collect historical crash data for before and after the implementation of ASCT and analyze their effects. The next subsection provides an overview of the literature, followed by a description of the data assembled. The third subsection summarizes the analysis methodology, while the fourth subsection provides the results of the analysis by corridor. The last subsection summarizes the conclusions of the safety analysis.

4.1 Background on Safety Studies

In 2013, out of the 10.1 million motor vehicle crashes, about 10% of them occurred at signalized intersections [39]. This represents an increase from 9% out of the 6.3 million crashes reported in 2003 [10]. There is indeed an extensive body of literature on intersection safety. In this project, we focus our attention on the operational characteristics of traffic signals with particular emphasis on adaptive signal control and signal coordination.

Several operational characteristics of the signal can have impacts on safety. For example, increasing the yellow time and all red times could reduce red light running and hence improve safety [41-43]. Chin and Quddus [44] reported that crashes at signalized intersections increase with increasing number of phases and hypothesize that most crashes happen during phase changes. This result is also supported by Poch and Mannering [45].

Sunkari [46] reports that signal retiming, in general, has safety benefits as it reduces the number of stops and improves the smoothness of traffic flow which translates into reduced severe collisions. Also, the report by Hicks and Carter [47] states that ASCT reduce the number of stops by 28% to 41% reinforcing the point that ASCT improves coordination hence reducing the number of stops. Furthermore, a report from HRG, Inc.[1] supports the point that ASCT reduce the frequency of stops. Their report states that injury crashes decreased by half in Troy, Michigan and crashes reduced by 38% in West Des Moines, Iowa after the implementation of the adaptive control signals. In addition, FHWA[2] considers signal coordination a “proven” strategy for improving intersection safety and the rear-end crashes along the major street. Based on before-and-after studies, reductions in crash frequencies range from about 6% to 38%. Coordinated intersections have also been estimated to have 3-18% fewer total crashes and 14-43% fewer rear-end crashes compared to uncoordinated intersections[3]. The ITE toolbox

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[1] http://www.hrg-inc.com/adaptive-traffic-signals-reduce-delay-increase-safety-and-improve-public-satisfaction/?print=print

[2] http://safety.fhwa.dot.gov/intersection/other_topics/fhwasa08008/sa4_Signal_Coordination.pdf

[3] http://safety.fhwa.dot.gov/intersection/conventional/signalized/fhwasa13027/ch8.cfm#s833

on intersection safety design[4] also reports a 12% reduction in crashes during the peak periods based on a study in Illinois.

Based on before- and after- studies conducted in five corridors[5], adaptive signal control with coordination was shown to reduce crashes between 15% and 30% (Figure 4.1).

TAC Corridor Implementation Summary

Corridor and Agency	Scope of Corridor		Before Period		After Period		Change In Crashes		Signal Years Exposure		Annual Crash-Related Cost Saving
	Inter-sections	Length (miles)	Years	Ave. Annual Crashes	Years	Ave. Annual Crashes	Fre-quency	Percent-age	Before	After	
Washington Road Columbia County, GA	5	1	1.0	162	1.0	120	-42	-26%	5.0	5.0	$1,164,702
21st Street City of Topeka, KS	7	1	2.0	142	2.0	108	-34	-24%	14.0	14.0	$942,854
Missouri Highway 291, Lee's Summit (Missouri DOT)	12	2.5	3.0	262	1.0	217	-45	-17%	36.0	12.0	$1,247,895
Chipman Road City of Lee's Summit, MO	8	1	2.0	89	0.6	76	-13	-15%	16.0	4.7	$360,503
Thompson Road/ Hwy 71, City of Springdale, AR	8	3	0.9	63	0.9	44	-19	-30%	7.3	7.3	$526,889
Total	**40**	**8.5**	**8.9**	**718**	**5.5**	**565**	**-153**	**-22%**	**78.3**	**41.0**	**$4,242,843**

Figure 4.1 Impacts of Coordination on Safety[5]

Based on a before-and-after (2 years before and 2 years after) study of crashes along 6 corridors in Florida that were treated with signal retiming, Yauch[6] reports reductions in crashes based on both a simple comparison of observed crashes (Table 4.1) and using the Highway Safety Manual (HSM) approach which compared the observed crashes after the treatment with the expected number of crashes in the same corridor without the treatment (Table 4.2).

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[4] http://library.ite.org/pub/e1d08c51-2354-d714-51e9-f3967064dfb9

[5] http://rhythmtraffic.com/safety-benefits-associated-with-adaptive-traffic-signal-control/

[6] https://static1.squarespace.com/static/5771719ae4fcb57b45f0450e/t/5797882a725e25c408307ed5/1469548587406/2%29+Effects-of-Traffic-Signal-Retiming-on-Safety.pdf

Table 4.1 Comparison of Observed Crashes Before and After Retiming (Adapted From [6])

Corridor	Observed Before	Observed After	Difference	% Difference
1	1812	1532	-280	-15.45
2	1053	915	-138	-13.11
3	564	509	-55	-9.75
4	1180	850	-330	-27.97
5	2623	2151	-472	-17.99
6	802	723	-79	-9.85

Table 4.2 Comparison of Expected Crashes without Retiming with Observed Crashes with Retiming (adapted from [6])

Corridor	Observed After	Expected After	Difference	% Difference
1	1532	1670	-138	-9.01
2	915	1043	-128	-13.99
3	509	585	-76	-14.93
4	850	896	-46	-5.41
5	2151	NA	NA	NA
6	723	834	-111	-15.35

In contrast to the above study which has looked at the corridor as a whole, others have examined the crashes at individual intersections while considering its operational attributes such as coordination as an explanatory variable. Based on a study of intersections in Singapore, Chin and Quddus [44] report that intersections with adaptive signals are safer than those with pre-timed signals based on fewer estimated crashes (5% less). The authors argue that adaptive signals force traffic into a more regular discharge pattern and reduce long gaps in opposing flows leading to safety benefits. Such effects may also be observed in the case of signal coordination, although the effect of coordination was not the focus of that study.

Feng et al. [48] examined the safety of intersections using data from central Florida. Of the 170 intersections in their study, 50 were “isolated” and the rest were a part of a coordinated scheme. The model results showed that coordinated intersections on the mainline highways had more crashes than those that were isolated. The authors argue that this must not be interpreted as a causal effect (i.e., coordination increases crashes); rather, this could be a manifestation of endogeneity effects (unsafe corridors are more likely to get coordinated systems) and/or the impact of higher speeds in coordinated corridors.

Turner [49] examined intersections in New Zealand. Although the analysis focused on individual intersections, one of the factors considered was whether the intersection was coordinated with an upstream intersection. The effect of coordination was mixed. It was found to be associated with increased right-angle crashes (except in some metropolitan cities where the effects were negative) and right turning vehicle-pedestrian crashes. Coordination was also associated with fewer "other" vehicle crash types. There was no statistically significant effect of coordination on other crash types such as rear-end crashes.

In a study by Li and Tarko [50], crash patterns on six arterials with coordinated signals were studied. All data were collected after the signal coordination plan was implemented. The researchers studied the likelihood of rear-end and right turn crashes on every 15-minute interval. Their analysis indicates that traffic streams that are more likely to arrive at an intersection in periods without an existing queue (i.e., they arrive in the later part of a green phase) are less likely to have a rear-end crash. Since the intent of coordination is to eliminate such queues, the authors argue that coordination has net positive effect on reducing the incidence of rear-end crashes near intersections. As already indicated, this study did not look at crashes along the same corridors before the coordination.

In a most recent study on the safety benefit of ASCT, Osorio and Benekohal [51] examined six intersections on the Neil Street corridor in Champaign, IL through a before- and after- study utilizing the Empirical Bayes method for the study period of 2012-2016. Their study developed crash modification factors (CMFs) for different crash types and severity classifications. For multiple-vehicle fatal and injury crashes, they developed a CMF of 0.67 at all intersections and four-legged-only intersections. However, although the CMFs developed were not statistically significant at 95% confidence interval, they show a decreasing trend in multiple vehicles fatal and injury crashes. For Property Damage Only (PDO) and total crashes, the CMFs developed were 1.04 and 0.96 respectively. This finding also indicates that there is no change in PDO and total crashes as their CMFs developed were close to one.

Overall, the literature review shows that adaptive/coordinated signal systems can be expected to improve the safety of arterials. This is because improved progression leads to reduced number of stops and thus a reduction on rear-end crashes. Also, increased platooning allows for more gaps in opposing traffic streams, which results in reduced angle/turning crashes. Disaggregate modeling studies also imply that self-selection bias should be considered as well, i.e., unsafe corridors are perhaps more likely to warrant treatments in the first place.

4.2 Data Assembly

Table 4.3 summarizes the corridor data used for the safety analysis. The crash data along these corridors were obtained from the Signal Four Analytics System[7] . Although the Signal Four system receives data updates very night, there is a natural lag period between the date of the crash and the date on which this is included in the Signal Four system with a geolocation attached to it. Our analysis (for details see Appendix J) indicates that 95% of all crashes generally appear in Signal Four within one month of the crash in most corridors. This is true for crashes reported on both Long and Short Forms. A Long Form Report is an extended form which includes a narrative, diagram, and additional crash-related details. Long Form Reports are used for crashes that involve death or personal injury, property damages and driving under the influence. Short Form Reports are typically provided for all other types of traffic crashes.

Therefore, for this project we include only crashes that occurred at least one month prior to the date of the analysis. This will limit the possibility of the "after" period crashes being biased because they have not been processed and added into the system yet.

Table 4.3 Corridor Characteristics and Installation Dates of New Adaptive System for Each Site

Corridor	County	Length (mile)	Cross Section	# of Signalized Intersections per mile	New System Start Date
US 17/92	Volusia	2.28	6-Lane Divided	2.2	14-Nov
Newberry Road	Alachua	1.45	6-Lane Divided	8.3	14-Nov
Panama City Beach (PCB) Parkway	Bay	8.5	4/6-Lane Divided	1.1	1/9/2015
23rd Street	Bay	2	4-Land Undivided	4.5	4/24/2015
University Parkway	Sarasota	7.8	4/6-Lane Divided	2.3	16-Feb
E. Van Fleet and N. Broadway Ave (Bartow)	Polk	1.1	6-Lane Divided	4.5	3/8/2016
SR 693	Pinellas	5	6-Lane Divided	2.4	4/24/2017
SR-70	Manatee	9.2	6-Lane Divided	2.4	Sept 2018

[7] https://s4.geoplan.ufl.edu/

A second issue of concern is ensuring that crash reporting practices have not changed over the analysis period. While the recording of long-form crashes into electronic databases have generally remained consistent practice all across the state, there have also been recent initiatives to start recording all crashes (both long- and short- form). The recording of short-form crashes is also known to be varied across the state. Our analysis (see Appendix K) suggest that there is no systematic variation (specifically, an increase over time) in the proportions of short form crashes over time. Therefore, we use both long- and short- form crashes for analysis in this study, as this will not bias the “after” period crashes.

The data extracted from the Signal Four Analytics System was for the period January 2013 – November 2017. Note that the earliest installation of ASCT at the eight study corridors was November 2014 and the latest was April 2017. Initially, the research team extracted all crashes within 300 feet of the entire corridor. Therefore, crashes occurring within 300 feet upstream of the first ASCT intersection and those crashes occurring within 300 feet downstream of the last ASCT intersection were also included. Crashes occurring on side streets along the corridor (irrespective of whether the side-street intersection is signalized or not) were also included. Subsequently, the research team also created a subset of data comprising only the mainline crashes (i.e., crashes along the corridor in which the signals are coordinated)[8]. Broadly, mainline crashes represented 50-80% of all crashes along the corridors. Crashes occurring between 1 am and 6 am were filtered out since the vast majority of signals operate on flash during those times. Regardless, there were relatively few crashes recorded during these times.

The data assembled includes all crashes along these corridors, and not all of them are related to the signal system. The literature review presented previously indicates that certain types of crashes are more likely to be influenced by signal progression. Table 4.4 presents the various types of crashes examined (in addition to all crashes along the corridor) and the expected impact of signal improvements on these crash types.

Analyzing crash trends must also account for exposure, i.e., changes in traffic volumes in the study corridors over time. The best consistent estimates of traffic volumes over time were the AADT values obtained from FDOT (Florida Traffic Online[9]). The methodology for aggregating AADT estimated at different points along the corridor into a single estimate for each corridor is presented in Appendix L.

We also assembled crash data for the entire counties in which the corridors are located, during the same analyses period. This was done to examine whether there were any broader temporal trends in crashes over the analysis period, which are not related to ASCT.

[8] Feedback from engineers responsible for the individual corridors also suggested this to be a preferred approach. We show later in the analysis that including side-streets may be a better approach for the Bartow corridor as this is not a straight-line corridor.

[9] http://flto.dot.state.fl.us/website/FloridaTrafficOnline/viewer.html

Table 4.4 Description of Main Crash Attributes Used for Analysis

Crash Type	Potential Impact of Signal Progression
Rear-End crashes	A reduction in the number of stops because of the smoothening of the flow decreases the probability of a rear-end crash occurring
Peak and Off Peak Crashes / Weekend Crashes	Traffic volumes and platooning patterns differ between Peak and Off Peak periods. Increased platooning allows more gaps in the opposing traffic stream decreasing the chances of turning or angle crashes
Fatal and Injury Crashes	Smoothening of flow decreases the chance of injuries or fatalities because of decrease in rear-end crashes
Intersection-related/influenced Crashes	While signal coordination systems can be expected to impact safety along the entire corridor, it can potentially have a greater impact on crashes happing at or in the vicinity of intersections.

4.3 Methodology

The objective of this analysis is to determine the impact of ASCT deployment on the overall safety (crash patterns) of the entire corridor. The methods employed aim to accomplish this goal while recognizing (1) the analyses are being conducted right after the installation limiting the amount of “after” data, (2) estimates of traffic volume are available on a yearly basis (even though implementations may happen during the middle of a year), and (3) there may be other factors impacting the safety along the corridor .

The analysis results section presents the results by corridor followed by an overall summary. For each corridor, the following are presented and discussed:

- The temporal (monthly) profile of crashes along the corridor and for the entire county in which the corridor is located
- Crash frequencies per month before and after the implementation by crash type (all crashed, fatal and injury crashes, intersection-related crashes, rear-end crashes, weekday AM Peak crashes, Weekday PM Peak crashes, Weekday Off Peak crashes, and Weekend crashes) along the corridor (mainline and side streets)
- Crash frequencies per month before and after the implementation by crash type (all crashed, fatal and injury crashes, intersection-related crashes, rear-end crashes, weekday AM Peak crashes, Weekday PM Peak crashes, Weekday Off Peak crashes, and Weekend crashes) along the main line of the corridor
- Crash frequencies per month before and after the implementation by crash type (same types as before) for the entire county in which the corridor is located.

- Crash rate (Crashes / AADT) per year before and after the implementation by crash type (all crashes, fatal and injury crashes, rear-end crashes, and intersection-related crashes) along the corridor

The reader will note that crash types representing certain time of day periods (Peak / Off Peak) or day of the week (weekday / weekend) are not included in the crash rate analysis as the AADT will not be representative of traffic volumes during specific time-of-day or day-of-the week periods.

It is useful to note that the duration of the "before" and "after" periods for the sites are different as the deployments happened at different points in time. In order to examine the effects over the same time frame, we also present the crash frequencies for a seven month period before the deployment and a seven month period after deployment (see Appendix D). It is important to note that, while ensuring consistency in time frame of analysis, these represent short-term effects and potentially reflect different seasonal effects for the before- and after-periods (for example for the Panama City Beach corridor, the "after" 7 months capture the spring break and summer effects while the "before" 7 months capture the summer and winter periods.

4.4 Safety Analysis Results

In this section, the detailed analysis and results are presented for six of the eight total corridors. The new signal system was installed in the Newberry road corridor by November 2014. The system was run intermittently until June 2015. At this time, this was disabled and switched back to the manual co-ordination system already in place. There isn't adequate "after" data for the SR-70 corridor considering the late implementation date for this location.

4.4.1 US 17/92 Corridor

Figure 4.2 presents the temporal (monthly) profile of crashes along the corridor (mainline only and mainline + side streets) and Figure 4.3 presents the crash profile for Volusia County. The vertical line in the figure separates the before- and after- periods.

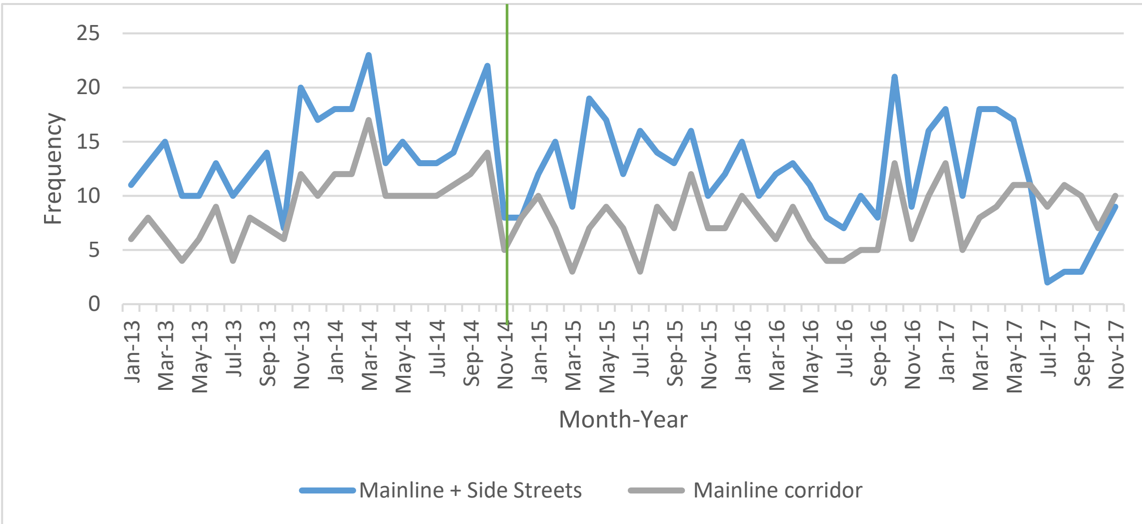

Figure 4.2 Temporal Profile of Total Crashes along US 17/92 Corridor

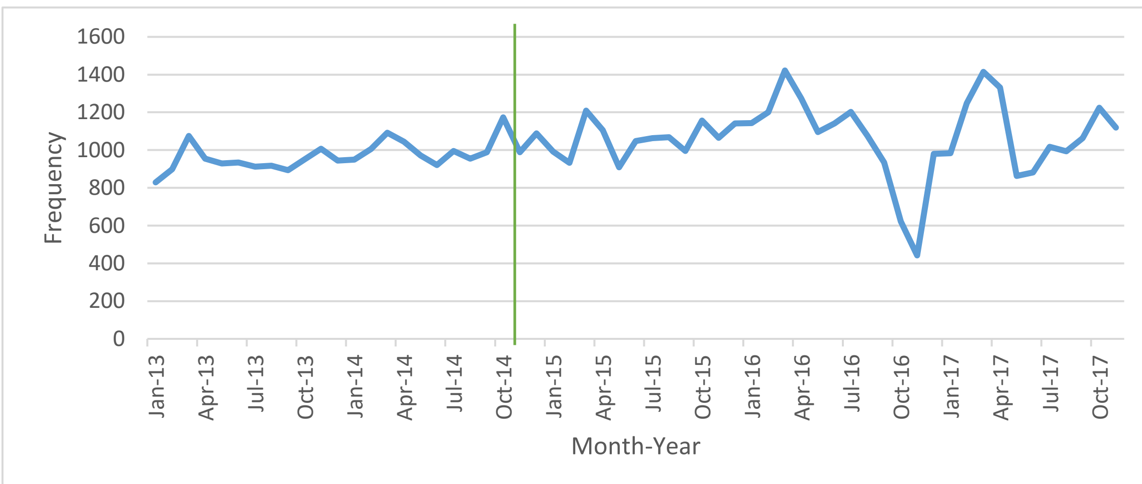

Figure 4.3 Temporal Profile of Total Crashes along Volusia County

Table 4.5 presents the aggregate before- and after- comparison of the monthly crashes on this corridor and its respective county, Volusia. The monthly crash frequencies decreased for all crash types (with the exception of weekday PM Peak) along the corridor after the ASCT implementation compared to the before-deployment period. However, the monthly crash frequencies increased for the entire county. The short-term (7-month) comparison also reflect these trends to a large extent (Table L1 in Appendix L).

Travel times for both directions on this corridor decreased by an average of 0.66 min (10.6%) for the NB and 1.67 min (25%) for the SB. Also, the intersection through movement delay decreased for almost all the intersections in both directions by an average of more than 11 seconds per intersection.

Table 4.5 Comparison of Crashes on US 17/92 and Volusia County Before (Jan 1, 2013 – Nov 14, 2014) and After (Nov 15, 2014 – Nov 30, 2017) the Implementation of ASCT

<table>
<tr><th colspan="3"></th><th>All crashes</th><th>Fatal and Injury</th><th>Intersection Related</th><th>Rear End</th><th>Weekday AM Peak</th><th>Weekday PM Peak</th><th>Weekday Off Peak</th><th>Weekend</th></tr>
<tr><td rowspan="5">US 17/92 Parkway (Mainline +Side Streets)</td><td rowspan="2">Before</td><td>Crash Freq.</td><td>325</td><td>83</td><td>66</td><td>88</td><td>32</td><td>68</td><td>154</td><td>71</td></tr>
<tr><td>Crash/ Month</td><td>14.44</td><td>3.69</td><td>2.93</td><td>3.91</td><td>1.42</td><td>3.02</td><td>6.84</td><td>3.16</td></tr>
<tr><td rowspan="2">After</td><td>Crash Freq.</td><td>430</td><td>98</td><td>85</td><td>122</td><td>44</td><td>106</td><td>194</td><td>86</td></tr>
<tr><td>Crash/ Month</td><td>11.78</td><td>2.68</td><td>2.33</td><td>3.34</td><td>1.21</td><td>2.90</td><td>5.32</td><td>2.36</td></tr>
<tr><td colspan="2">Diff (%)</td><td>-18.44</td><td>-27.22</td><td>-20.61</td><td>-14.54</td><td>-15.24</td><td>-3.91</td><td>-22.34</td><td>-25.33</td></tr>
<tr><td rowspan="5">US 17/92 Parkway (Mainline Corridor)</td><td rowspan="2">Before</td><td>Crash Freq.</td><td>207</td><td>48</td><td>32</td><td>62</td><td>18</td><td>40</td><td>102</td><td>47</td></tr>
<tr><td>Crash/ Month</td><td>9.23</td><td>2.14</td><td>1.43</td><td>2.76</td><td>0.80</td><td>1.78</td><td>4.55</td><td>2.10</td></tr>
<tr><td rowspan="2">After</td><td>Crash Freq.</td><td>288</td><td>58</td><td>21</td><td>72</td><td>26</td><td>68</td><td>135</td><td>59</td></tr>
<tr><td>Crash/ Month</td><td>7.88</td><td>1.59</td><td>0.57</td><td>1.97</td><td>0.71</td><td>1.86</td><td>3.69</td><td>1.61</td></tr>
<tr><td colspan="2">Diff (%)</td><td>-14.64</td><td>-25.87</td><td>-59.74</td><td>-28.76</td><td>-11.38</td><td>4.29</td><td>-18.80</td><td>-22.99</td></tr>
<tr><td rowspan="5">Volusia County</td><td rowspan="2">Before</td><td>Crash Freq.</td><td>21827</td><td>6367</td><td>4771</td><td>6431</td><td>2325</td><td>3900</td><td>10324</td><td>5278</td></tr>
<tr><td>Crash/ Month</td><td>970.09</td><td>282.98</td><td>212.04</td><td>285.82</td><td>103.33</td><td>173.33</td><td>458.84</td><td>234.58</td></tr>
<tr><td rowspan="2">After</td><td>Crash Freq.</td><td>38955</td><td>11486</td><td>8833</td><td>12457</td><td>4484</td><td>7031</td><td>18267</td><td>9173</td></tr>
<tr><td>Crash/ Month</td><td>1067.26</td><td>314.68</td><td>242.00</td><td>341.29</td><td>122.85</td><td>192.63</td><td>500.47</td><td>251.32</td></tr>
<tr><td colspan="2">Diff (%)</td><td>10.02</td><td>11.20</td><td>14.13</td><td>19.41</td><td>18.89</td><td>11.13</td><td>9.07</td><td>7.14</td></tr>
</table>

Table 4.6 presents the annual crash rates (crashes per AADT) on this corridor (mainline only) for each of the crash types not related to time of day[10]. As shown, the AADT values are fluctuating (increasing in some years and decreasing in others) over the analysis period. Meanwhile, the before and after traffic volume counts conducted showed a 4% and 11.9% increase in both the major street and minor street throughput respectively. It is useful to note that, the traffic volume changes were measured only at the critical signalized intersections along the corridor during peak hours.

The crash rates during the after years (2015-2017) do not appear substantially different from crash rates during the before years (2013-2014) with the exception of the rate of intersection-related crashes which decreased.

Table 4.6 Comparison of Cash Rates on US 17/92

Year	AADT	Rear End (RE)	RE /AADT	Fatalities & Injuries (FI)	FI /AADT	Intersection Related (IR)	IR /AADT	Total Crashes (TC)	TC /AADT
2013	38880	23	0.0006	20	0.0005	15	0.0004	86	0.0022
2014	37070	42	0.0011	31	0.0008	19	0.0005	131	0.0035
2015	37030	20	0.0005	15	0.0004	7	0.0002	88	0.0024
2016	42410	28	0.0007	20	0.0005	6	0.0001	86	0.0020
2017	41550	21	0.0005	20	0.0005	6	0.0001	104	0.0025

4.4.2 Panama City Beach Parkway Corridor

Figure 4.4 presents the temporal (monthly) profile of crashes along the corridor (mainline only and mainline + side streets) and Figure 4.5 presents the crash profile for Bay County. The vertical line in the figure separates the before- and after- periods. The profile shows peaking of crashes in March and July of each year, potentially coinciding with spring-break and summer traffic to the beaches. A steady increase in crashes over time is also noticeable. A report from the Bay County Tourist Development Council shows that there had been an increase in tourism and the development tax collection for July 2016 compared to July 2015[11]. Thus, an increase in

[10] As already discussed, peak-period crashes, weekend crashes etc. are not included in the crash rate analysis as we do not have the traffic volumes for these specific periods. Further traffic volumes are also not available for all cross streets and so the crash rate analysis is restricted to mainline crashes only.

[11] http://www.newsherald.com/business/20160906/update-july-breaks-all-time-tourism-record-in-panama-city-beach

the number of tourists over the past few years and the corresponding increase in traffic during specific months could explain the increase in the crash rates. Another reason could be due to changes in the crash reporting methodology.

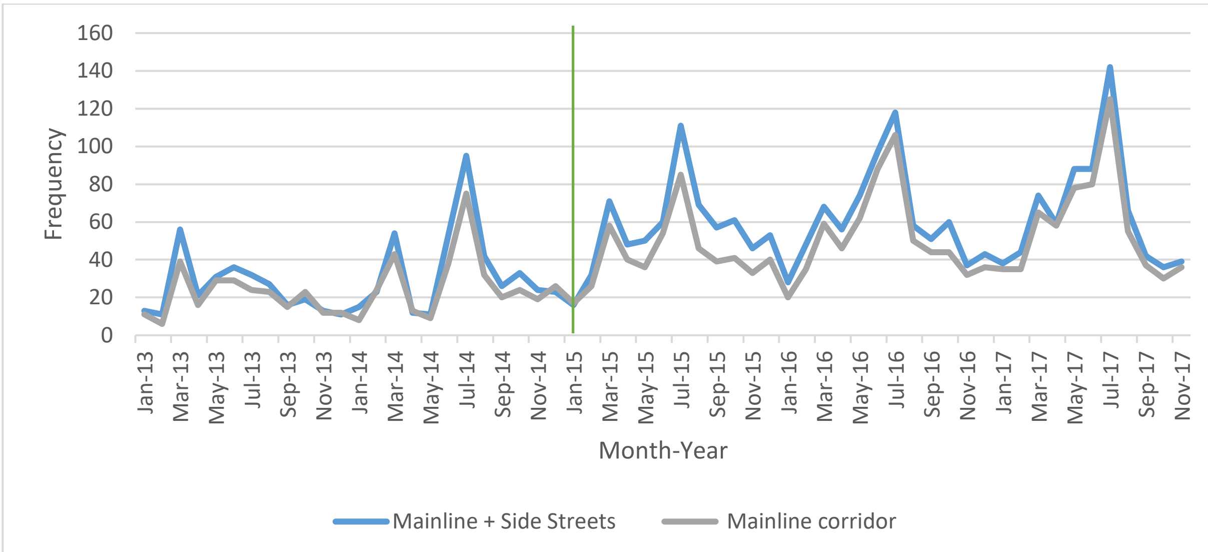

Figure 4.4 Temporal Profile of Total Crashes along PCB Parkway Corridor

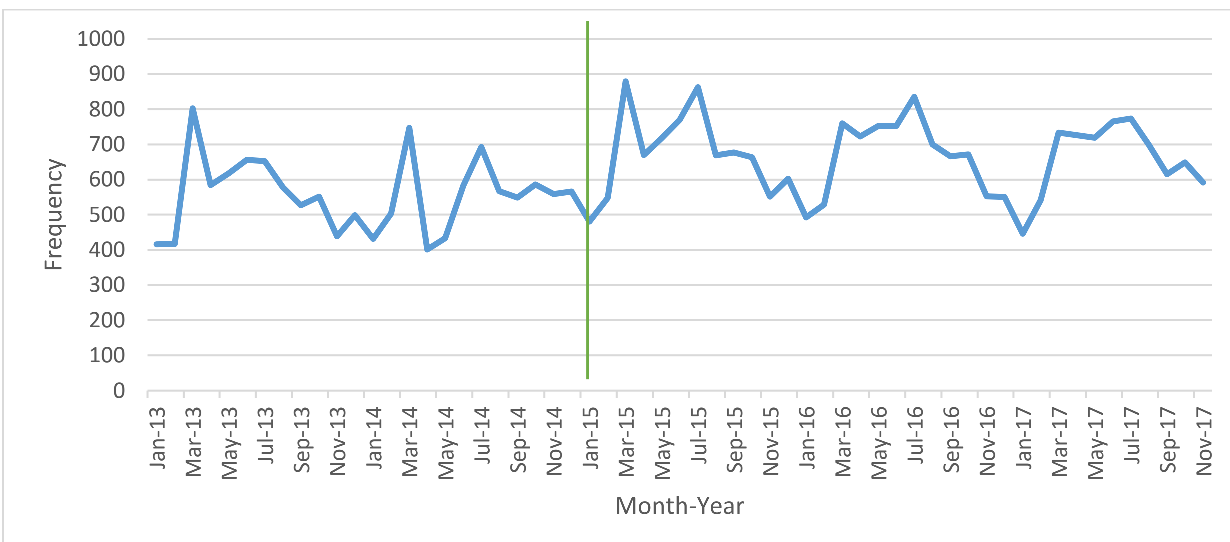

Figure 4.5 Temporal Profile of Total Crashes along Bay County

Table 4.7 Comparison of Crashes on PCB Parkway and Bay County Before (Jan 1, 2013 – Jan 8, 2015) and After (Jan 9, 2015 – Nov 30, 2017) the Implementation of ASCT

			All crashes	**Fatal and Injury**	**Intersection Related**	**Rear End**	**Weekday AM Peak**	**Weekday PM Peak**	**Weekday Off Peak**	**Weekend**
PCB Parkway (Mainline +Side Streets)	Before	Crash Freq.	698	129	46	161	66	121	287	224
		Crash /Month	28.77	5.32	1.90	6.64	2.72	4.99	11.83	9.23
	After	Crash Freq.	2127	287	110	513	194	370	918	645
		Crash /Month	61.22	8.26	3.17	14.77	5.58	10.65	26.42	18.57
	Diff (%)		112.77	55.34	66.97	122.48	105.24	113.51	123.34	101.05
PCB Parkway (Mainline Corridor)	Before	Crash Freq.	571	120	64	194	56	111	228	176
		Crash /Month	23.54	4.95	2.64	8.00	2.31	4.58	9.40	7.26
	After	Crash Freq.	1770	229	76	452	152	299	778	541
		Crash /Month	50.95	6.59	2.19	13.01	4.38	8.61	22.39	15.57
	Diff (%)		116.44	33.25	-17.08	62.68	89.52	88.08	138.26	114.63
Bay County	Before	Crash Freq.	13467	2545	2024	4189	1575	2586	6148	3158
		Crash /Month	555.16	104.91	83.44	172.68	64.93	106.60	253.44	130.18
	After	Crash Freq.	23216	4071	3562	7128	2811	4327	10730	5348
		Crash /Month	668.24	117.18	102.53	205.17	80.91	124.55	308.85	153.94
	Diff (%)		20.37	11.69	22.88	18.81	24.62	16.83	21.86	18.24

Table 4.7 presents the aggregate before- and after- comparison of the monthly crashes on this corridor and its respective county, Bay. The monthly crash frequencies increased for all crash types (with the exception of intersection-related crashes along the mainline, which decreased) compared to the before-deployment period. At the same time, the monthly crash frequencies increased for the entire county as well. The short-term (7 month) comparison also reflect these trends to a large extent (Table L2 in Appendix L). Also, the data quality for this corridor is problematic. There were relatively few crashes classified as "intersection related", and the corresponding data field in the original database has a high proportion of missing cases for reasons not readily apparent.

Generally, there was an overall improvement in travel time for both directions in this corridor after the installation of the ASCT system. Table 4.8 presents the annual crash rates (crashes per AADT) on this corridor for each of the crash types not related to time of day. As shown, the AADT values are systematically increasing over the analysis period. Similarly, the before and after traffic volume counts conducted showed a 5% and 1.8% increase in the major street and minor street throughput respectively. However, the traffic volume changes were measured only at the critical signalized intersections along the corridor during peak hours.

The rate of total crashes and rear-end crashes during the after years (2015-2017) do appear to be systematically higher than crash rates during the before years (2013-2014). The rate of intersection-related crashes decreased and there is no trend in the case of fatal and injury crashes.

Table 4.8 Comparison of Cash Rates on PCB Parkway

Year	AADT	Rear End (RE)	RE /AADT	Fatalities & Injuries (FI)	FI /AADT	Intersection Related (IR)	IR /AADT	Total Crashes (TC)	TC /AADT
2013	42268	83	0.0020	67	0.0016	34	0.0008	239	0.0057
2014	45076	110	0.0024	53	0.0012	30	0.0007	331	0.0073
2015	46155	116	0.0025	58	0.0013	25	0.0005	515	0.0112
2016	46745	162	0.0035	99	0.0021	25	0.0005	622	0.0133
2017	48202	175	0.0036	72	0.0015	26	0.0005	634	0.0132

4.4.3 23rd Street Corridor

Figure 4.6 presents the temporal (monthly) profile of crashes along the corridor (mainline only and mainline + side streets) and Figure 4.7 presents the crash profile for Bay County. The vertical line in the figure separates the before- and after- periods. Unlike the Panama City Beach Corridor which is in the same county, we do not see a systematic seasonal profile in the crashes as this corridor is not along the beach. Further, the temporal trend also shows a decrease in the number of crashes over time.

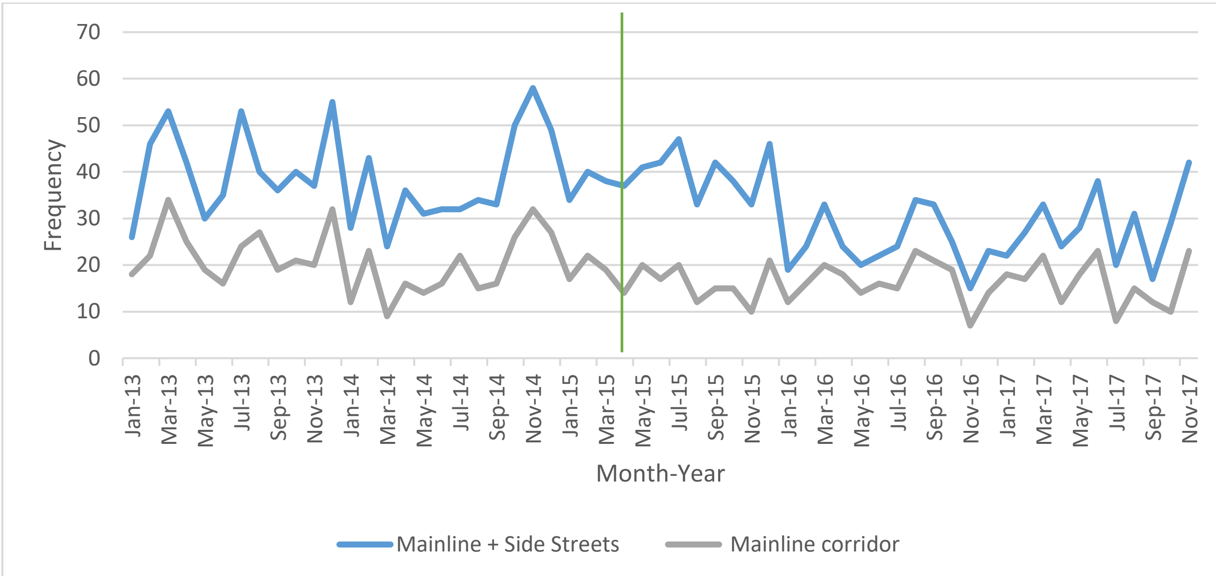

Figure 4.6 Temporal Profile of Total Crashes along 23rd St Corridor

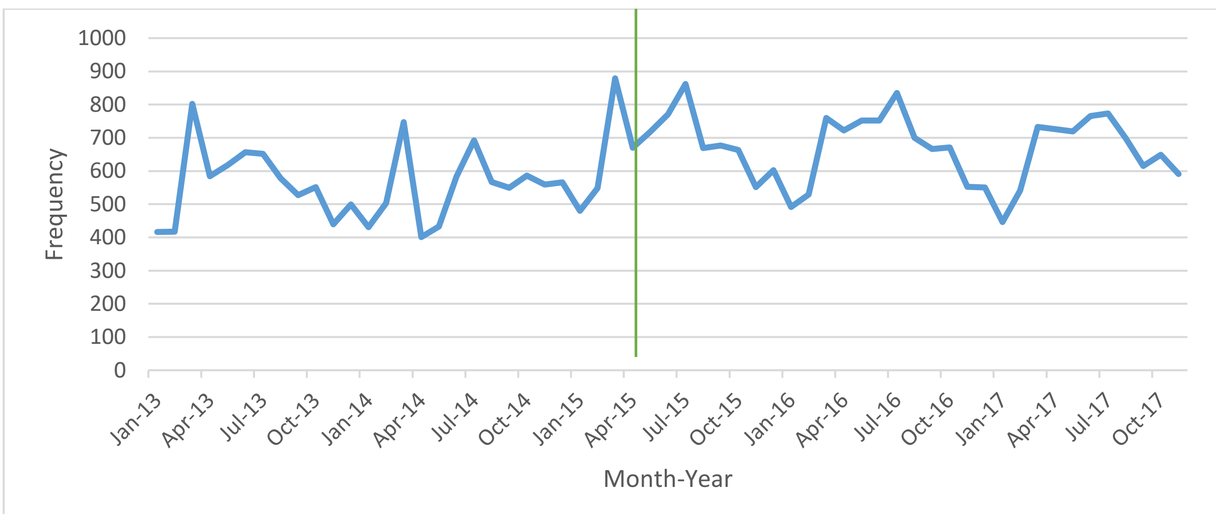

Figure 4.7 Temporal Profile of Total Crashes along Bay County

Table 4.9 presents the aggregate before- and after- comparison of the monthly crashes on this corridor and its respective county, Bay. The monthly crash frequencies decreased for all crash types (with the exception of intersection-related crashes, which increased) compared to the before-deployment period. At the same time, the monthly crash frequencies increased for the entire county as well. The short-term (7-month) comparison also reflect these trends (the intersection-related crashes also decreased in the short term; see Table L3 in Appendix L). There was a decrease in travel time in both directions during all time periods. However, the PM Peak had minimal improvements.

Table 4.9 Comparison of Crashes on 23rd St. and Bay County Before (Jan 1, 2013 – April 23, 2015) and After (April 24, 2015 – Nov 30, 2017) the Implementation of ASCT

			All crashes	**Fatal and Injury**	**Intersection Related**	**Rear End**	**Weekday AM Peak**	**Weekday PM Peak**	**Weekday Off Peak**	**Weekend**
23rd St. (Mainline +Side Streets)	Before	Crash Freq.	1086	185	95	428	91	189	608	198
		Crash/ Month	39.11	6.66	3.42	15.41	3.28	6.81	21.9	7.13
	After	Crash Freq.	935	152	182	427	94	172	514	155
		Crash/ Month	29.94	4.87	5.83	13.67	3.01	5.51	16.46	4.96
	Diff (%)		-23.46	-27	70.32	-11.3	-8.17	-19.1	-24.84	-30.41
23rd St. (Mainline Corridor)	Before	Crash Freq.	574	143	67	347	39	106	332	97
		Crash/ Month	20.67	5.15	2.41	12.5	1.4	3.82	11.96	3.49
	After	Crash Freq.	506	88	100	297	44	105	264	93
		Crash/ Month	16.2	2.82	3.2	9.51	1.41	3.36	8.45	2.98
	Diff (%)		-21.63	-45.29	32.69	-23.91	0.3	-11.94	-29.31	-14.77
Bay County	Before	Crash Freq.	15784	2955	2326	4861	1856	2989	7267	3672
		Crash/ Month	568.45	106.42	83.77	175.07	66.84	107.65	261.72	132.24
	After	Crash Freq.	20889	3661	3260	6456	2530	3924	9611	4834
		Crash/ Month	668.8	117.21	104.38	206.7	81	125.64	307.72	154.77
	Diff (%)		17.65	10.14	24.6	18.07	21.18	16.71	17.58	17.03

Table 4.10 presents the annual crash rates (crashes per AADT) on this corridor for each of the crash types not related to time of day. As shown, the AADT values are systematically increasing over the analysis period. Similarly, the before and after traffic volume counts conducted showed a 10.1% increase in the major street throughput although a 1.8% decrease in the minor street throughput. However, the traffic volume changes were measured only at the critical signalized intersections along the corridor during peak hours.

The rate of total crashes, fatal and injury crashes, and rear-end crashes during the after years (2015-2017) do appear to be systematically lower than crash rates during the before years (2013-2014). The rate of intersection-related crashes, however, increased.

Table 4.10 Comparison of Cash Rates on 23rd St.

Year	**AADT**	**Rear End (RE)**	**RE /AADT**	**Fatalities & Injuries (FI)**	**FI /AADT**	**Intersection Related (IR)**	**IR /AADT**	**Total Crashes (TC)**	TC /AADT
2013	21870	164	0.0075	75	0.0034	14	0.0006	86	0.0039
2014	29879	145	0.0049	52	0.0017	48	0.0016	131	0.0044
2015	31495	114	0.0036	44	0.0014	22	0.0007	88	0.0028
2016	33027	110	0.0033	31	0.0009	39	0.0012	86	0.0026
2017	33772	111	0.0033	29	0.0009	44	0.0013	104	0.0031

4.4.4 University Parkway Corridor

Figure 4.8 presents the temporal (monthly) profile of crashes along the corridor (mainline only and mainline + side streets) and Figure 4.9 presents the crash profile for Sarasota County. The vertical line in the figure separates the before- and after- periods. A gradual increase in crashes over time is observed for the corridor. During the deployment of the ASCT there was construction of a Diverging Diamond Interchange (DDI) at the interstate I-75 and University Parkway, which caused additional traffic issues.

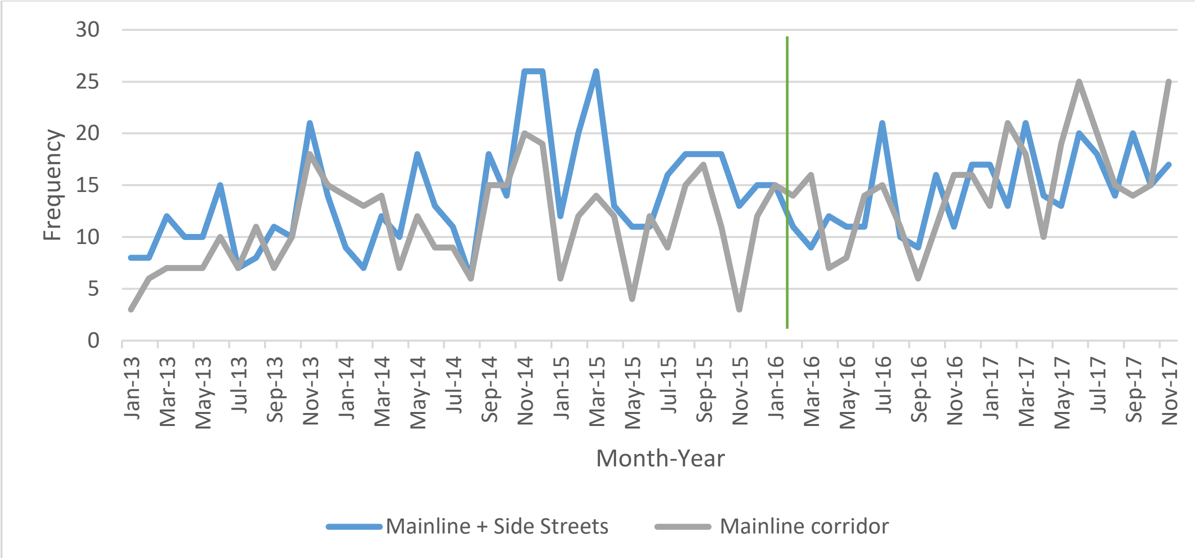

Figure 4.8 Temporal Profile of Total Crashes along University Parkway Corridor

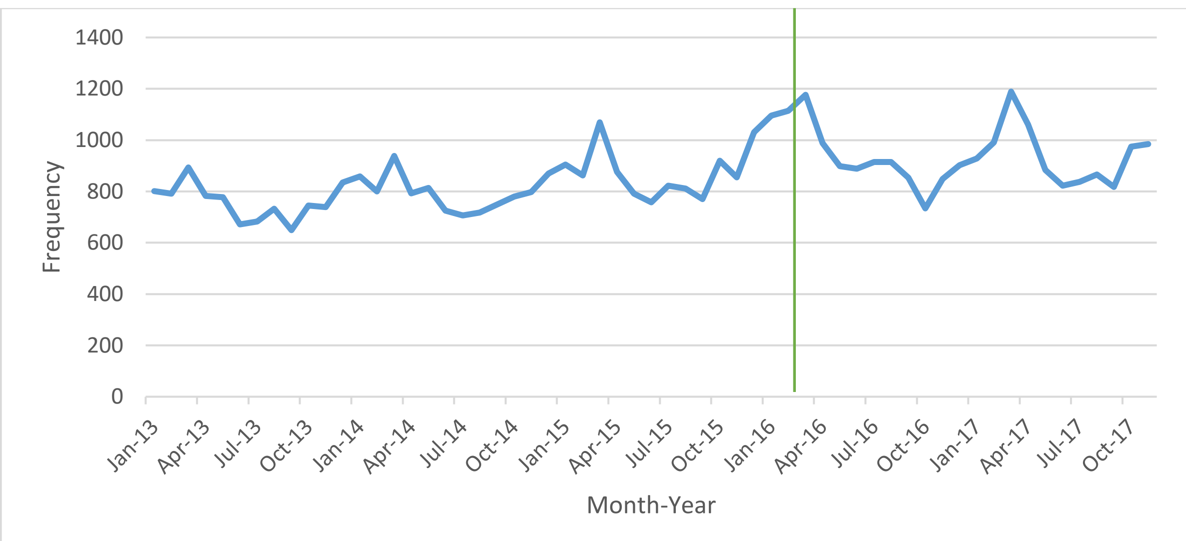

Figure 4.9 Temporal Profile of Total Crashes along Sarasota County

Table 4.11 presents the aggregate before- and after- comparison of the monthly crashes on this corridor and its respective county, Sarasota. The monthly crash frequencies increased for all crash types along the corridor compared to the before-deployment period. At the same time, the monthly crash frequencies increased for the entire county as well. The short-term (7-month) comparison indicate reductions in crash frequencies of various types; see Table L4 in Appendix L). There was an overall improvement in travel time and delay in this corridor after ASCT was installed.

Table 4.11 Comparison of Crashes on University Parkway Before (Jan 1, 2013 – Feb 14, 2016) and After (Feb 15, 2016 – Nov 30, 2017) the Implementation of ASCT

			All crashes	Fatal and Injury	Intersection Related	Rear End	Weekday AM Peak	Weekday PM Peak	Weekday Off Peak	Weekend
University Parkway (Mainline +Side Streets)	Before	Crash Freq.	517	138	240	326	57	119	222	119
		Crash/Month	13.78	3.68	6.40	8.69	1.52	3.17	5.92	3.17
	After	Crash Freq.	313	95	143	185	49	60	135	69
		Crash/Month	14.57	4.42	6.66	8.61	2.28	2.79	6.28	3.21
	Diff (%)		5.73	20.22	4.06	-0.90	50.13	-11.95	6.20	1.26
University Parkway (Mainline Corridor)	Before	Crash Freq.	410	134	175	275	56	98	172	84
		Crash/Month	10.93	3.57	4.66	7.33	1.49	2.61	4.58	2.24
	After	Crash Freq.	322	82	119	216	58	67	143	54
		Crash/Month	14.99	3.82	5.54	10.05	2.70	3.12	6.66	2.51
	Diff (%)		37.16	6.87	18.75	37.17	80.88	19.40	45.19	12.27
Sarasota County	Before	Crash Freq.	30774	7404	8829	11221	3750	6173	14621	6230
		Crash/Month	820.26	197.35	235.33	299.09	99.95	164.54	389.71	166.06
	After	Crash Freq.	20024	4567	5010	7404	2674	4037	9397	3916
		Crash/Month	932.10	212.59	233.21	344.65	124.47	187.92	437.42	182.29
	Diff (%)		13.63	7.72	-0.90	15.23	24.53	14.21	12.24	9.77

Table 4.12 presents the annual crash rates (crashes per AADT) on this corridor for each of the crash types not related to time of day. The AADT values are fluctuating over the analysis period (increases till 2016 and decreases). Meanwhile, the before and after traffic volume counts conducted showed a 9.7% decrease in the major street throughput and a 3.3% increase in the minor street throughput. However, the traffic volume changes were measured only at the critical signalized intersections along the corridor during peak hours.

In general, there was a systematic increase in crash rates in 2017 compared to the previous years.

Table 4.12 Comparison of Cash Rates on University Parkway

Year	AADT	Rear End (RE)	RE /AADT	Fatalities & Injuries (FI)	FI /AADT	Intersection Related (IR)	IR /AADT	Total Crashes (TC)	TC /AADT
2013	40864	65	0.0016	29	0.0007	44	0.0011	108	0.0026
2014	41331	99	0.0024	51	0.0012	67	0.0016	153	0.0037
2015	45011	95	0.0021	46	0.0010	53	0.0012	127	0.0028
2016	46552	97	0.0021	41	0.0009	54	0.0012	149	0.0032
2017	40373	135	0.0033	49	0.0012	76	0.0019	195	0.0048

4.4.5 Bartow Corridor

Figure 4.10 presents the temporal (monthly) profile of crashes along the corridor (mainline only and mainline + side streets) and Figure 4.11 presents the crash profile for Polk County. The vertical line in the figure separates the before- and after- periods.

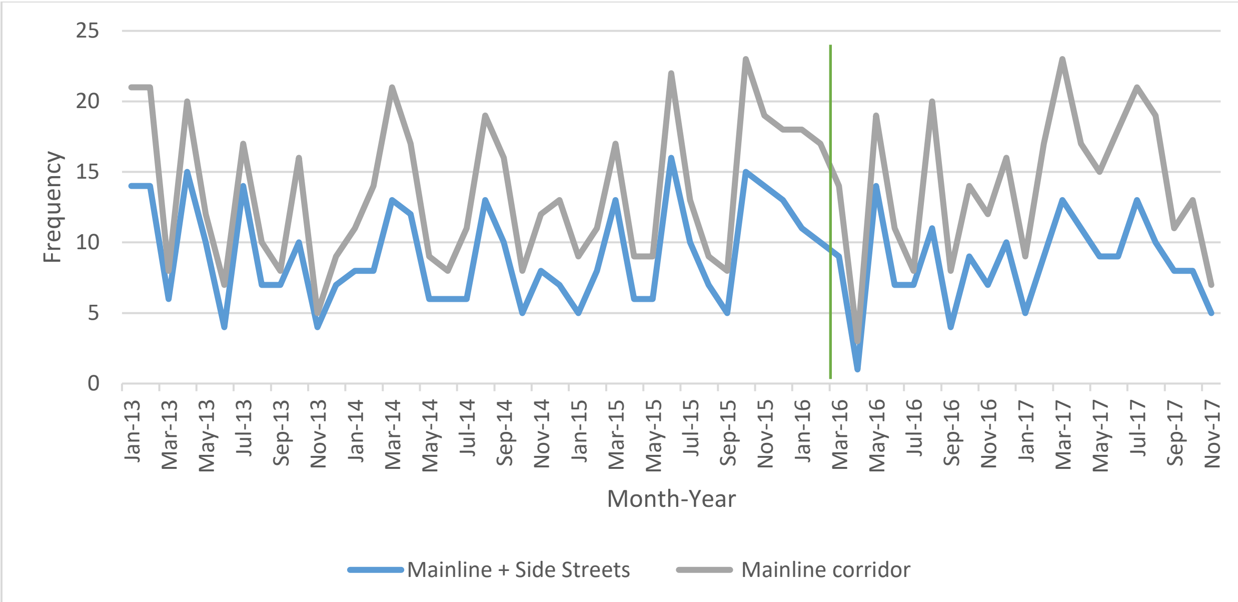

Figure 4.10 Temporal Profile of Total Crashes along Bartow Corridor

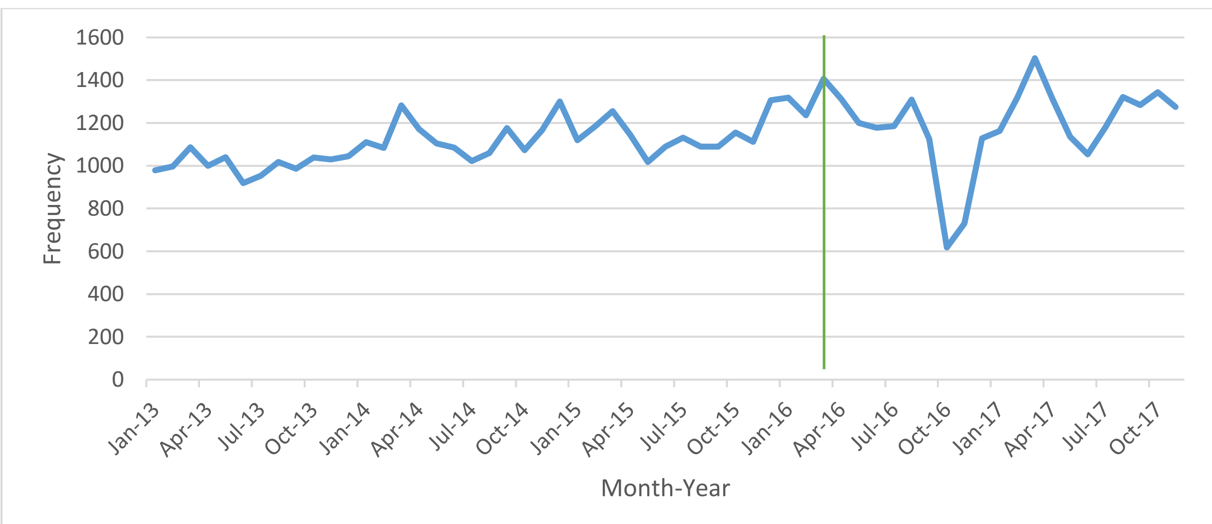

Figure 4.11 Temporal Profile of Total Crashes along Polk County

Table 4.13 presents the aggregate before- and after- comparison of the monthly crashes on this corridor and its respective county, Polk. The monthly crash frequencies increased for all crash types with the exception of weekday PM Peak and intersection-related crashes along the mainline of the corridor. However, when considering the side-streets along with the mainline, there is a reduction in the total crashes, intersection crashes, weekday Off Peak and PM Peak crashes, and weekend crashes. Unlike the other study corridors which are generally straight lines, the Bartow corridor is "L" shaped. Therefore, it might be more appropriate to consider both mainline and side streets in analyzing this corridor. The monthly crash frequencies increased for the entire county as well. The short-term (7-month) comparison, however, indicates reductions in crash frequencies of various types for both the mainline and mainline with side streets (see Table L5 in Appendix L).

The installation of the ASCT system generally shows improvement in travel time. Travel time reduced by 12.9% for the EB and increased by 14.2% for the WB.

Table 4.13 Comparison of Crashes on Bartow Corridor and Polk County Before (Jan 1, 2013 – March 7, 2016) and After (March 8, 2016 – Nov 30, 2017) the Implementation of ASCT

			All crashes	**Fatal and Injury**	**Intersection Related**	**Rear End**	**Weekday AM Peak**	**Weekday PM Peak**	**Weekday Off Peak**	**Weekend**
Bartow (Mainline +Side Streets)	Before	Crash Freq.	354	85	87	142	39	70	158	87
		Crash/ Month	9.26	2.22	2.28	3.71	1.02	1.83	4.13	2.28
	After	Crash Freq.	178	51	26	95	26	33	74	45
		Crash/ Month	8.57	2.45	1.25	4.57	1.25	1.59	3.56	2.17
	Diff (%)		-7.48	10.4	-45.01	23.1	22.67	-13.25	-13.82	-4.82
Bartow (Mainline Corridor)	Before	Crash Freq.	162	47	37	85	15	33	80	34
		Crash/ Month	4.24	1.23	0.97	2.22	0.39	0.86	2.09	0.89
	After	Crash Freq.	116	33	11	63	22	16	46	32
		Crash/ Month	5.58	1.59	0.53	3.03	1.06	0.77	2.21	1.54
	Diff (%)		31.76	29.2	-45.3	36.38	169.88	-10.78	5.8	73.18
Polk County	Before	Crash Freq.	42238	12961	10676	14485	5314	8332	19465	9127
		Crash/ Month	1104.96	339.06	279.29	378.93	139.02	217.97	509.21	238.77
	After	Crash Freq.	24824	7166	5943	8668	3305	4861	11371	5287
		Crash/ Month	1194.94	344.95	286.08	417.25	159.09	233.99	547.36	254.5
	Diff (%)		8.14	1.74	2.43	10.11	14.44	7.35	7.49	6.59

Table 4.14 presents the annual crash rates (crashes per AADT) on this corridor for each of the crash types not related to time of day. The AADT values are fluctuating over the analysis period. Meanwhile, the before and after traffic volume counts conducted showed a 28.8% increase in the major street throughput and a 3.5% decrease in the minor street throughput. It is useful to note that, the traffic volume changes were measured only at the critical signalized intersections along the corridor during peak hours. Correspondingly there is no clear pattern in crash rates in 2016-2017 compared to the previous years.

Table 4.14 Comparison of Cash Rates on Bartow

Year	AADT	Rear End (RE)	RE /AADT	Fatalities & Injuries (FI)	FI /AADT	Intersection Related (IR)	IR /AADT	Total Crashes (TC)	TC /AADT
2013	34595	17	0.0010	12	0.0005	6	0.0005	42	0.0032
2014	32706	32	0.0014	14	0.0008	12	0.0007	57	0.0031
2015	36405	29	0.0014	17	0.0010	12	0.0009	49	0.0032
2016	37773	33	0.0014	18	0.0007	12	0.0008	60	0.0026
2017	36015	37	0.0014	19	0.0007	6	0.0008	70	0.0026

4.4.6 SR 693 Corridor

Figure 4.12 presents the temporal (monthly) profile of crashes along the corridor (mainline only and mainline plus side streets) and Figure 4.13 presents the crash profile for Pinellas County. The vertical line in the figure separates the before- and after- periods.

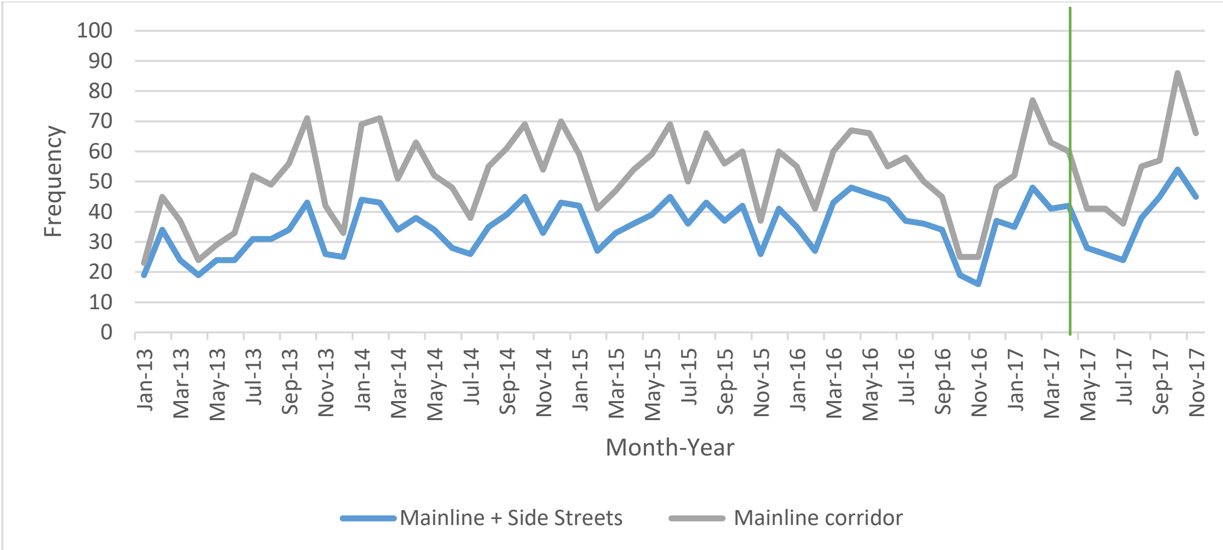

Figure 4.12 Temporal Profile of Total Crashes along SR 693 Corridor

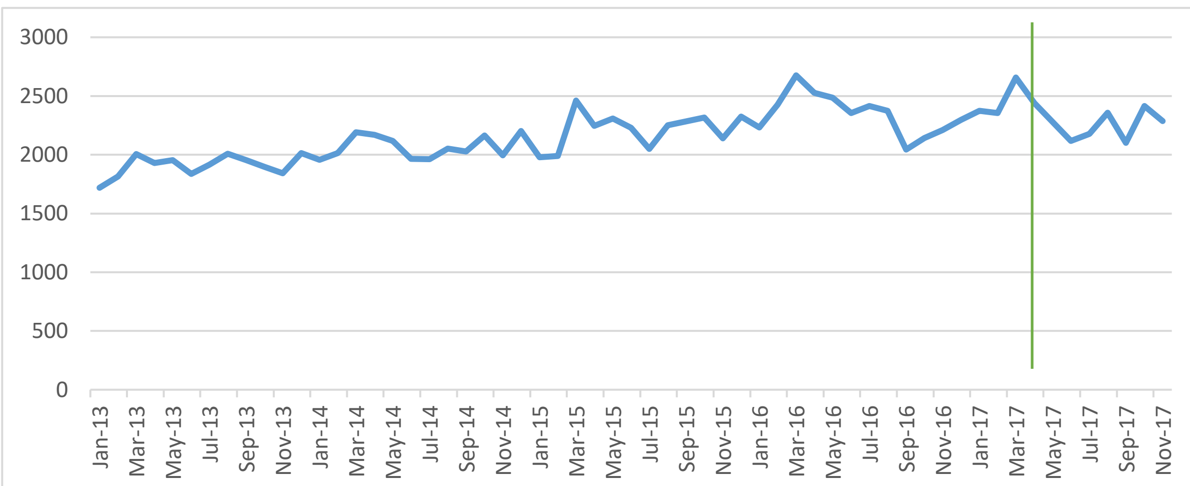

Figure 4.13 Temporal Profile of Total Crashes along Pinellas County

Table 4.15 presents the aggregate before- and after- comparison of the monthly crashes on this corridor and its respective county, Pinellas. The monthly crash frequencies decreased for fatal and injury crashes, intersection related crashes, weekday PM Peak crashes, and weekday Off Peak crashes along the mainline of this corridor. When considering the mainline and side streets, there was a reduction in only the fatal and injury crashes. The monthly crash frequencies also increased for the entire county (with the exception of fatal and injury crashes and weekend crashes). The short-term[12] (7-month) comparisons are largely consistent with this trend (see Table L6 in Appendix L).

The system shows an overall improvement in travel time and delay on this corridor. For all

[12] The long-term analysis uses the same 7 months of "after" data

travel periods, travel time decreased by 20.3% for the SB and 10.5% for the NB directions.

Table 4.15 Comparison of Crashes on SR 693 and Pinellas County Before (Jan 1, 2013 – April 23, 2017) and After (April 24, 2017 – Nov 30, 2017) the Implementation of ASCT

			All crashes	Fatal and Injury	Intersection Related	Rear End	Weekday AM Peak	Weekday PM Peak	Weekday Off Peak	Weekend
SR 693 (Mainline +Side Streets)	Before	Crash Freq.	1799	597	814	861	252	355	892	300
		Crash/Month	34.75	11.53	15.72	16.63	4.87	6.86	17.23	5.8
	After	Crash Freq.	272	73	129	142	43	58	126	45
		Crash/Month	37.6	10.09	17.83	19.63	5.94	8.02	17.42	6.22
	Diff (%)		8.21	-12.49	13.42	18.03	22.12	16.93	1.09	7.35
SR 693 (Mainline Corridor)	Before	Crash Freq.	884	333	414	445	124	181	437	142
		Crash/Month	17.08	6.43	8	8.6	2.4	3.5	8.44	2.74
	After	Crash Freq.	127	36	54	68	24	23	56	24
		Crash/Month	17.56	4.98	7.47	9.4	3.32	3.18	7.74	3.32
	Diff (%)		2.82	-22.63	-6.65	9.36	38.52	-9.06	-8.29	20.96
Pinellas County	Before	Crash Freq.	111740	29568	27547	36437	14564	22691	50036	24449
		Crash/Month	2158.53	571.18	532.14	703.87	281.34	438.33	966.57	472.29
	After	Crash Freq.	16294	4072	5527	5951	2182	3449	7317	3346
		Crash/Month	2252.63	562.95	764.1	822.72	301.66	476.82	1011.57	462.58
	Diff (%)		4.36	-1.44	43.59	16.89	7.22	8.78	4.66	-2.06

Table 4.16 presents the annual crash rates (crashes per AADT) on this corridor for each of the crash types not related to time of day. The AADT values are fluctuating over the analysis period. Meanwhile the before and after traffic volume counts conducted showed a 7.5% increase in the major street throughput and a 16.4% decrease in the minor street throughput. However, the traffic volume changes were measured only at the critical signalized intersections along the corridor during peak hours.

Table 4.16 Comparison of Cash Rates on SR 693 with AADT

Year	AADT	Rear End (RE)	RE /AADT	Fatalities & Injuries (FI)	FI /AADT	Intersection Related (IR)	IR /AADT	Total Crashes (TC)	TC /AADT
2013	35740	78	0.0022	77	0.0022	65	0.0018	160	0.0045
2014	36172	115	0.0032	106	0.0029	131	0.0036	259	0.0072
2015	37554	118	0.0031	63	0.0017	103	0.0027	211	0.0056
2016	35830	89	0.0025	62	0.0017	76	0.0021	173	0.0048
2017	36830	113	0.0031	61	0.0017	93	0.0025	208	0.0056

4.5 Summary and Conclusions from Safety Analysis

This task evaluated the impacts of ASCT deployments on the overall safety (crash patterns) of six corridors. Crash data from January 2013 to November 2017 were used for the analysis. Crash trends along the mainline and those along the mainline and side streets were analyzed. These were also compared to crash trends at the county level. Changes in total crashes were first examined followed by changes in crashes by severity (fatal and injury crashes), crash type (such as rear end, intersection related), and time of day (Peak, Off Peak, weekend). Estimates of traffic volume for the entire corridor (mainline) were obtained and these were used to examine patterns in annual crash rates. The duration of the “before” and “after” periods for the sites are different as the deployments were completed at different times. In order to examine the effects over the same time frame, we also compared the crash frequencies for a seven-month period before the deployment and a seven-month period after deployment for all sites (representing a short-term impact of the ASCT deployment). The results should be considered recognizing that (1) the analyses were conducted shortly after the ASCT installation limiting the amount of “after” data, (2) estimates of traffic volume are available on a yearly basis (even though implementations may be completed in the middle of a year) impacting crash-rate analyses, and (3) there may be other factors impacting the safety along the corridor.

Table 4.17 presents an overall summary of results. Although we examined multiple metrics we focus on total crashes and intersection-related crashes to present the major results.

Table 4.17 Summary of Safety Results

Site	Long Term				Short Term			
	Main Line and Side Streets		Main Line		Main Line and Side Streets		Main Line	
	All Crashes (%)	Intersection Crashes (%)	All Crashes (%)	Intersection Crashes (%)	All Crashes (%)	Intersection Crashes (%)	All Crashes (%)	Intersection Crashes (%)
US 17/92	-18.44	-20.61	-14.64	-59.74	-19.81	0.00	-36.00	-20.00
Bartow	-7.48	-45.01	31.76	-45.30	-28.00	-48.15	-13.89	-63.64
23rd Street	-23.46	70.32	-21.63	32.69	-11.11	-42.86	-29.94	-53.57
University Parkway	5.73	4.06	37.16	18.75	-24.32	-25.49	-4.71	-11.11
Panama City Beach Parkway	112.77	66.97	116.44	-17.08	40.96	23.81	43.72	0.00
SR 693	8.21	13.42	2.82	-6.65	13.19	15.74	12.61	15.22

Among all corridors, US17/92 in Deland showed consistent benefits in terms of crash reductions (both overall and intersection related crashes) and in both the short term and long term. The results hold for both mainline and mainline and side streets. In fact, there were reductions in nearly all crash types while the county as a whole saw increases in crashes over the same period of time. Bartow also showed decreases in crashes with the exception of all crashes along the mainline. However, as discussed earlier, it would be more appropriate to examine mainline and side street crashes in the context of Bartow as the corridor is “L” shaped.

Both 23rd Street and University Parkway showed short term benefits in terms of crash reductions. However, when examined over a longer term, University Parkway showed increases in crashes while 23rd Street showed increases in intersection –related crashes and reductions in all crashes. Regarding the University Parkway corridor, the construction of the interchange at I-75 could also have an effect on the overall crash patterns.

The corridor along Panama City Parkway showed systematic increases in crashes. It is possible that the effect of the increasing tourist demand over the years leading to higher seasonal traffic is potentially offsetting any safety benefits of signal coordination. It is also useful to note that this is the longest corridor with the least number of signalized intersections / mile, and therefore, the impact of signal improvements on the overall corridor may be limited. SR 693 also showed increases in crashes. This location had the shortest “after” period for analysis and as such the results could largely be reflecting short-term impacts.

In the overall, based on an examination of the temporal profiles of crash patterns in several corridors in which the advanced signal control technologies were deployed, this study finds that there are safety benefits to such deployments. Specifically, the reductions in crashes (crashes / month) along the deployment corridor could be up to 18% in locations which have reasonable intersection density (1.5/mile or more) and when there are no other major extraneous factors (such as construction or county-wide increases in crashes) which could have critically impacted safety. The corridor can experience a crash reduction post the signal coordination even though the crash rates may be increasing during the same period in the county in which the corridor is located. The study also demonstrated that the safety benefits can accrue along the main line and the side streets. Therefore it is important to consider the appropriate measure (crashes long main line only versus crashes along main line + side streets) depending on the corridor geometry and relative magnitudes of traffic along the main line and cross streets. This study also calculated crash reductions by crash type, crash severity, time of day, and day of the week.

As already acknowledged in the report, the analysis of crashes were undertaken fairly quickly after the final deployments and, therefore, the volume of “after” data are limited and varied across the corridors. By re-examining these corridors in the future by considering additional “after” years of data, it would be possible to develop Florida-specific crash modification factors (CMFs) to capture the benefits of advanced signal control technology deployments . Such CMFs would represent a more robust quantitative measure of safety benefits.

In the next chapter, the crash data from the safety analysis are used along with traffic operational measures to conduct benefit-cost analysis.

5. FINDINGS FROM STAFF INTERVIEWS

Appendices A through H provide the detailed information collected during interviews with TMC staff in a question-and-answer format. In this chapter, we describe the interview process followed by summaries of the interview data in four categories: Signal System; Performance and Satisfaction; Staff, training and cost; Institutional Issues. The last section summarizes the lessons learned from the interviews conducted.

5.1 Interview Process

The interviews were conducted at each site either through an in-person interview or through video calls when in-person interviews could not be arranged. The questionnaire was provided to the agency in advance of the interview. It consisted of five sections and approximately 40 questions which focused on previous traffic control technologies used by the agency, their experience with adaptive traffic control systems (ASCT), cost components, and institutional issues. In some cases questions were partially answered or not answered, and in those cases follow-up attempts were made whenever possible. A single interview was conducted at Bay County to discuss two corridors: Panama City Beach Parkway and Panama City 23rd Street, and the responses regarding these two corridors are identical. Therefore, the data are summarized below by agency, rather than by corridor.

5.2 Signal System

Table 5.1 shows the type of signal control applied at each corridor (before and after), and information regarding their maintenance and life expectancy. All sites studied had actuated coordinated control prior to ASCT installation. Of all the sites, only DeLand did not use NEMA TS-2. They used NEMA TS-1 instead, which was used in conjunction with NEMA TS-2 in other sites. Life expectancies and retiming frequencies had a large range of results for these controllers.

SynchroGreen was selected for installation at two of the eight corridors and InSync was used at the remaining six. InSync has an upgrade version called InSync Fusion which in addition to InSync cameras allows for the ASCT to connect with existing detectors, such as ground loop detectors. Most sites did not need to change controllers, with six of the eight corridors keeping the same controllers. The remaining two corridors had to replace the controllers with new ones. Maintenance was largely provided across the board on an as-needed or yearly basis. Often, it was found that the ASCT software needs updating on a regular basis.

It was found that most counties did not have a regular retiming schedule and the time of last retiming varied greatly from 1 year to 8 years before ASCT deployment. This could be a key factor in evaluating the effectiveness of the ASCT, especially in the areas where there has been significant land development and increases in traffic over the past several years.

Table 5.1 Signal System Details

Site/Question	**Gainesville**	**DeLand**	**Manatee**	**Sarasota**	**Bartow**	**Bay**	**Pinellas**
Traffic Control Type	Actuated coordinated control	Actuated coordinated control	Actuated coordinated control	Actuated coordinated control	Actuated coordinated control	Actuated coordinated control	Actuated coordinated control
Traffic Controller (before ASCT)	NEMA TS-1 NEMA TS-2	NEMA TS-1	NEMA TS-2	NEMA TS-2	NEMA TS-2	NEMA TS-1 NEMA TS-2	NEMA TS-2
Life expectancy of detectors (before ASCT)	Do not know	14 years	4 years	Do not know	Do not know	5 years	10 years
Last retime before data collection	3 years	5 years	4 years	1 year	8 years	3 years	3 years
Initial criteria for new detector	Extended cycle length	Adaptability to change	N/A	Mobility needs met	Complexity needs met	N/A	Simpler system
ASCT deployed	Synchro Green	InSync: Fusion	Synchro Green, ATMS 2.8	InSync, InTraffic	InSync, InTraffic	InSync: Fusion and Regular	InSync
Others considered?	InSync	ACS-Lite, SCOOT, Synchro Green	No	No	No	No	OPAC, RHODES
Traffic Controller (after ASCT)	Trafficware Nema 980/ATC	NEMA TS-2	NEMA TS-2	NEMA TS-2 with updates	NEMA TS-2 with updates	NEMA TS-1 NEMA TS-2	NEMA TS-2
Life expectancy of detectors (after ASCT)	N/A	6 years	4 years	4 years	1.5 years	5 years	10 years
Maintenance timing	As needed	As needed, twice a year	Once a year	Once a year	Once a year	Quarterly	As needed
Software update?	Yes	No	Yes	No	No	Yes	Yes

5.3 Performance and Satisfaction

This section summarizes the system performance and satisfaction of the local staff with the ASCT (Table 5.2). The most common challenges faced by TMC staff in each of the sites were with the camera detection and communication. Public perception was an issue only in Bartow and in Gainesville (understandably, since the system resulted in deterioration of operations at both locations). In the remaining six corridors, staff indicated that the complaints from the public went back to normal levels or lower within one month after ASCT deployment.

At failure, the ASCT primarily reverts to an off-line mode with a given time of day (TOD) plan. ASCT was found to be most effective during the Off Peak hours, and in some sites also during peak hours as indicated by local staff.

The management of saturation, the relatively low amount of complaints for a site, and especially sustained performance, contributed to staff satisfaction with ASCT. For example, the staff in Gainesville reported that ASCT did not perform well during oversaturated conditions and operations were often worse than TOD plans. This, in addition to 12 months of increasing complaints from the public, led to removal of ASCT on Newberry Road. Gainesville staff gave a "neutral" satisfaction rating to the ASCT product. The satisfaction of the staff also affected the likelihood of expansion to other corridors. One site with staff that indicated neutral satisfaction, Bartow, seemed hesitant to expand ASCT to other corridors. The staff in Gainesville indicated that they would expand the implementation under different circumstances. The staff who indicated they were very satisfied with the existing installation wanted to expand ASCT to other corridors, with one site, Bay, already expanding as of the date of the interview.

Table 5.2 Performance and Satisfaction

Site/Question	**Gainesville**	**DeLand**	**Manatee**	**Sarasota**	**Bartow**	**Bay**	**Pinellas**
Top Challenge/s	Communication	Camera Detection	Detection	Detection, Communication	Public Perception	Camera Detection	Camera Detection
ASCT response at failure	Nothing	Off-line mode with time of day plans	Off-line mode with time of day plans	Off-line mode with time of day plans	Other	Other	Historic data time splits
Performance evaluation	In house	In house	In house and FDOT	In house	In house and independent	In house	In house
When is it effective?	Off Peak	Peak and Off Peak	Peak, off, shoulders of peak	Off Peak	Off Peak	Peak and Off Peak	Off Peak
Manages oversaturation? 1- worst, 5-best	1	No oversaturation occurs	3	4	2	1	4
Months before complaints returned to normal levels	12	0.5	1	1	6	0	1
Sustained performance?	No, demand too high	Yes	Yes	Yes	Yes, improving	Yes	Yes
Level of Satisfaction 1- worst, 5- best	3	5	4	4	3	5	5
Expanding to other places?	Yes, to a less congested corridor	Yes	Yes, under right conditions	Yes, but not InSync	Maybe, dependent on results	Yes, one additional corridor	Yes

5.4 Staff, Training, and Cost

Table 5.3 highlights staff-related issues and the cost per intersection at each site. Staff training was conducted through the respective vendors. Except for Pinellas, no additional staff were used to monitor the ASCT performance. The number of training days per site varied, and for the most part correlated with the staff satisfaction (shown in Table 5.2): the more training days, the more satisfied with the ASCT the staff were.

Each county had a different type of contract with the vendors. Some paid a lump sum amount, whereas others paid separately by line item (procurement, installation, and maintenance). Since the ASCT packages, length of the corridor and type of contract vary, for ease of comparison, all costs are shown per intersection for each corridor (Table 5.3). As discussed earlier, the details of cost breakdown and type of contract are provided in the appendices. Except for Bartow, the cost per intersection varies from \$20k to \$45k. These costs are used in conducting benefit/cost analysis (summarized in Chapter 5 of this report).

Corridors that had lower performance and lower staff satisfaction generally had the following implementation characteristics (Table 5.2 and 5.3):

- The majority of the implementation was conducted in-house with little help from vendors
- The staff responsible for the corridor had fewer training days, and no specialized staff for ASCT
- There was no maintenance contract with the vendor.

Table 5.3 Staff, Training, and Cost

Site/Question	**Gainesville**	**DeLand**	**Manatee**	**Sarasota**	**Bartow**	**Bay**	**Pinellas**
Staff training	External	External	External	External	External	External	Initially external, mostly internal
Training days	1	10	5	2	5	5	5
Staff change	No	No	No	No	No	No	Yes, 1
Overall capital cost (per intersection)	$21,666	$30,000	$45,350	$36,500	$96,400	$42,500	$32,871

5.5 Institutional Issues

This section provides a brief overview of institutional issues encountered with the installation and implementation of the ASCT devices (details on these by site are provided in the Appendices A to H). These are categorized into: organization and management; regulatory and legal; human and facility resources; and financial.

5.5.1 Organization and Management

Most of the institutional issues identified during the interviews relate to the organization and management of ASCT. Gainesville staff indicated they had limited time to devote to correcting various issues with the system. In DeLand, the staff reported issues regarding the funding for maintenance of ASCT cameras. Sarasota staff indicated they had issues with the two different contractors. Bartow staff indicated they faced skepticism about the installation of the new ASCT system. Bay County staff indicated there was a significant amount of time that had to be devoted on learning the operations of ASCT.

5.5.2 Regulatory and Legal

Only Manatee faced institutional issues in terms of regulatory and legal bounds. The staff indicated there were legal issues with assigning the grant to an existing contract instead of bidding it out.

5.5.3 Human and Facility Resources

Several institutional issues identified were related to human and facility resources. Manatee staff indicated they faced difficulties in determining several aspects, such as changing traffic controllers, and complications with the involvement of multiple parties and contracts. Sarasota staff indicated they had issues with staffing, as employees were pulled away from their regular schedules to help complete the installation quickly. The public in both Sarasota and Bay initially found it difficult to adjust to the new system, which operated differently than they were used to.

5.5.4 Financial

There were also financial institutional issues. DeLand staff indicated they experienced issues in finding funding for the maintenance of the ASCTs installed. Sarasota staff indicated they experienced financial issues due to additional Ethernet ports that had to be installed at some of the intersections, extra maintenance costs, and expensive cameras.

5.6 Summary of Lessons Learnt from the Interviews

The following are concluded from the interviews conducted with staff at the agencies responsible for installing, operating and maintaining ASCT:

- Regular maintenance and checks of the detection system and cameras are essential for effective ASCT operation
- ASCT software needs regular updating, and it is important to include maintenance funding as part of the overall project
- Extensive training of 5 days or more is required for the successful operation of ASCT, which leads to more efficiency, functionality, and staff satisfaction
- The sites where the vendors installed the system and did the initial fine-tuning performed better
- Most counties were able to use their existing signal controllers (NEMA-TS2) for ASCT implementation
- The frequency of signal retiming prior to ASCT installation varies widely among corridors (1 to 8 years), and this could be an important factor in evaluating the effectiveness of the ASCT relative to "before" operations
- Issues such as accounting for additional staff hours for learning ASCT operations, arranging for funds for additional costs such as Ethernet ports, camera maintenance etc. need to be considered as part of the overall project
- ASCT proved to be effective during Off Peak hours at all sites and during peak hours in some sites
- The level of staff satisfaction was correlated with objective measurements such as travel time and queue improvement
- Staff at all sites (even when the system did not perform well at their corridor) were open to the idea of expanding the use of ASCT to other locations if the corridor characteristics were favorable for ASCT. One site (Pinellas) has already expanded to another corridor due to their satisfaction with an existing installation.

The next chapter utilizes the costs from the interviews and compares it with monetized benefits from Chapters 2 and 3 in conducting a benefit-cost analysis.

6. BENEFIT-COST ANALYSIS

The benefit-cost analysis is a systematic process for identifying, quantifying and comparing expected benefits and costs of the implementation of ASCT. The benefits include the economic value of positive outcomes as the result of the implementation of ASCT and may be experienced by users of the transportation systems or the public at- large. The costs include the spending on installation, equipment, and personnel costs.

6.1 Background on Benefits and Costs

The benefit components considered in this study include travel time savings, vehicle cost savings (gas consumption savings), air pollution reduction, and safety benefits. The cost part contains the equipment, installation, training and maintenance cost. While some sites paid the vendors separately for each line-item, others paid a lump-sum amount per intersection. Hence for the purposes of benefit-cost analysis, costs are listed as "total costs per site" as well as "cost per intersection".

The travel time savings consider the reduction of users' time spent traveling through the corridor into consideration. Since the personal trips and business trips have different monetary value, our analysis assumes that all trucks trips are for business purposes. As for passenger cars, it is assumed that 20% are personal trips and 80 % are business trips during the AM and PM peak hours, while 80% are personal trips and 20% are business trips during the Off Peak. We also assume that the average vehicle occupancy for trucks and passenger cars are 1 person/veh and 1.39 person/vehicle respectively [52]. The main vehicle cost savings included in this study is the reduced fuel consumption, which results from the improved average travel speed. The economic damages caused by exposure to air pollution represent externalities which are not considered here, because their impacts are borne by society, rather than by the travelers and operators whose activities generate those emissions. The air pollution reduction is associated with the reduced combustion of transportation fuel.

Implementation of ASCT may also affect the likelihood of signalization-related accidents. In this benefit-cost analysis we consider the intersection-related crashes. The crash data along the corridors were obtained from the Signal Four Analytics System. The crashes are classified into five categories labeled KABCO: killed (K), incapacitating (A), non-incapacitating (B), possible injury (C), and property damage only (O). Also, the analysis is conducted both with and without safety-related effects. This approach was followed due to the differences in crash reporting across the State. While all counties report the severe (long form) crashes, the short form crashes are not always available in the database for every county. In practice, this difference can result in including more crashes for some sites only because there are more crashes reported. At one of the sites (Panama City) the reporting changed from the before to the after period of our analysis, making the safety comparison uneven.

6.2 Calculation of Benefits

Four benefit components are included in the calculations: travel time savings, vehicle cost savings, air pollution savings, and safety savings. Each component is monetized as indicated below.

6.2.1 Time savings

Ideally, the "consumer's surplus" should be used to measure the benefit of travel time improvement. However, this method is not applicable to this project since it assumes by default that demand increases with the decreased travel time. This demand- travel time relationship is not consistent with the data we collected from this project. For example, for the University Parkway corridor (Sarasota and Manatee Counties, District 1), the travel time and demand decrease at the same time after ASCT was implemented. Therefore, the average traffic flow of before and after ASCT implementation is used, and the travel time saving is calculated as:

$$T_{personal_i} = T_i * (q_{pc_i} + q'_{pc_i})/2 * o_{pc} * (1 - B_i) \tag{3}$$

$$T_{business_i} = T_i * (q_{truck_i} + \ q'_{truck_i})/2 * o_{truck} + T_i * (q_{pc_i} + q'_{pc_i})/2 * o_{pc} * B_i \tag{4}$$

$$T'_{personal_i} = T'_i * (q_{pc_i} + q'_{pc_i})/2 * o_{pc} * (1 - B_i) \tag{5}$$

$$T'_{business_i} = T'_i * (q_{truck_i} + \ q'_{truck_i})/2 * o_{truck} + T'_i * (q_{pc_i} + q'_{pc_i})/2 * o_{pc} * B_i \tag{6}$$

Where:

$T_{personal_i}$ and $T'_{personal_i}$*(person*hr/hr)* are the hourly time consumption for personal trips on the corridor during period i before and after implementation respectively;

$T_{business_i}$ and $T'_{business_i}$ *(person*hr/hr)* are the hourly time consumption for business trips on the corridor during period i before and after implementation respectively;

i=1,2,3 represent AM Peak hours, Off Peak hours and PM Peak hours respectively;

T_i and T'_i (hr) are the average travel time during period i before and after implementation

q_{truck_i} and q'_{truck_i} (veh/ h) are the truck flow rates during period i before and after implementation;

q_{pc} and q'_{pc_i} (veh/ h) are the passenger car flow rates during period i before and after implementation;

B_i is the percentage of passenger cars traveling for business purposes during period i;

o_{pc} and o_{truck} (person/ veh) are the average occupancy for passenger cars and trucks respectively (in this study we use 1.39 and 1 for o_{pc} and o_{truck});

$m_{personal}$ and $m_{business}$ ($/ hr) are the unit monetized travel time cost;

h_i (hr/ day) is the number of hours considered for period i of a weekday, where $i = 1,2,3$ represent AM Peak hours, Off Peak hours and PM Peak hours respectively;

w is the number of weekdays considered for a year.

Based on these, $\mathrm{T_m}$ is the annually monetized time saving for the corridor and it is calculated as:

$$T_m = \sum_{i=1}^{3} \left(T_{personal_i} * m_{personal} + T_{bussinessl_i} * m_{business}\right) * h_i * w - \sum_{i=1}^{3} \left(T'_{personal_i} * m_{personal} + T'_{bussinessl_i} * m_{business}\right) * h_i * w \quad (7)$$

The calculation assumes the following:

- All truck trips are for business;
- The hourly monetized time saving of a personal trip ($m_{personal}$) is $14.20/h [52];
- The hourly monetized time saving of a business trip ($m_{business}$) is $26.50/h [52];
- There are 260 weekdays in a year (w);
- There are 2 AM Peak, 2 PM Peak and 7 Off Peak hours in a day (h_i)
- 80% of car trips during AM and PM and 20% of car trips in Off Peak are business trips (B).

6.2.2 Fuel Consumption Savings

There are several state-of-the-art emission estimation models which require use of second-by-second vehicle speed and acceleration, as well as grade, elevation, friction resistance and vehicle weight, drag coefficient, engine performance, and several other variables. Thus, these cannot be used in this project. Instead, a fuel consumption estimation model that is solely based on the average travel speed is used [53]. Table 6.1 shows the relationship between the average operating speed and the corresponding fuel consumption rate (gallons /mile). The fuel consumption rate for speed values in between those provided can be interpolated. The gas price is obtained through [54].

Table 6.1 Relationship between Fuel Consumption Rate and Operating Speed

Operating Speed (mph)	Auto (gallons/mile)
10	0.123
15	0.089
20	0.068
25	0.054
30	0.044
35	0.037
40	0.034
45	0.033
50	0.033
55	0.034
60	0.037

All trucks are assumed to burn diesel while all passenger cars consume gasoline only. The hourly diesel and gasoline consumption are calculated as a function of the fuel consumption rate, corridor length and traffic flow rate as follows:

$$Q_{gasoline_i} = F_i * q_{pc_i} * L \tag{8}$$

$$Q_{diesel_i} = F_i * q_{truck_i} * L \tag{9}$$

$$Q'_{gasoline_i} = F'_i * q'_{pc_i} * L \tag{10}$$

$$Q'_{diesel_i} = F'_i * q'_{truck_i} * L \tag{11}$$

Where:

$Q_{gasoline_i}$ and $Q'_{gasoline_i}$ (gallon-car/hr) are the hourly gasoline consumption for the corridor during period i before and after implementation;

Q_{diesel_i} and Q'_{diesel_i} (gallon-car/hr) are the hourly diesel consumption for the corridor during period i before and after implementation;

F_i and F'_i (gallons/ mile) are the fuel consumption rate during period i before and after implementation;

L is the length of the corridor (miles).

The annually monetized fuel saving is the difference between “before” and “after” fuel cost of vehicles on the corridors. They are calculated as follows:

$$Q_F = \sum_{i=1}^{3}(Q_{gasoline_i} * h_i * w * f_{gasoline} + Q_{diesel_i} * h_i * w * f_{diesel}) - \sum_{i=1}^{3}(Q'_{gasoline_i} * h_i * w * f_{gasoline} + Q'_{diesel_i} * h_i * w * f_{diesel}) \tag{12}$$

Where:

Q_F ($/ year) is the annually monetized fuel saving, $f_{gasoline}$ and f_{diesel} are the unit price ($/ gallon) of gasoline and diesel respectively.

It is assumed that all passenger cars use gasoline while trucks use diesel. The price of the gasoline and diesel are taken as $2.66/gal and $3.18/gal respectively [54].

6.2.3 Emission reduction

EPA methods to calculate emissions require detailed data sets. Hence, a simplified emission model [55] was chosen for use in this project, and the emissions are estimated according to the fuel type and consumption rate. E_{ij} and E'_{ij} (short ton/ hr) are the hourly emission production rate of substance j for period i, and are calculated as:

$$E_{ij} = Q_{gasoline_i} * EF_{gasoline_j} * \left(\frac{100 - ER_j}{100}\right) + Q_{diesel_i} * EF_{diesel_j} * \left(\frac{100 - ER_j}{100}\right) \tag{13}$$

$$E'_{ij} = Q'_{gasoline_i} * EF_{gasoline_j} * \left(\frac{100 - ER_j}{100}\right) + Q'_{diesel_i} * EF_{diesel} * \left(\frac{100 - ER_j}{100}\right) \tag{14}$$

Where:

$EF_{gasoline_j}$ and EF_{diesel_j}(short ton/ gallon) are the emission factors of substance j from gasoline and diesel respectively;

ER_j is the emission reduction efficiency for substance j [55]. If a specialized emission reduction equipment is installed on a vehicle, this parameter would have a value based on the extent to which the equipment reduces the emissions. We have made a conservative assumption that no vehicles in our study have any special emission reduction equipment, hence ERj= 0.

E_{p_i} is the hourly monetized air pollution cost ($/ hour);

e_j is the unit air pollution price for substance j.

Table 6.2 provides the emission factors for passenger cars and trucks [55]. The emission factor relates the quantity of a pollutant released to the atmosphere with an activity associated with the release of that pollutant.

Table 6.2 Emission Factors for Passenger Cars and Trucks [55]

Type of air pollution	**Emission factor for Gasoline Passenger Cars (Kg/m^3)**	**Emission factor for Gasoline Passenger Cars (short ton/gallon)**	**Emission Factor For Diesel Trucks(Kg/m3)**	**Emission Factor for Diesel Trucks(short ton/gallon)**
Carbon dioxide (CO_2)	/	/	/	/
Volatile Organic Compounds (VOCs)	2.5	1.04318E-05	1.8	7.51086E-06
Nitrogen oxides (NOx)	6.7	2.79571E-05	23	9.59721E-05
Particulate matter (PM)	0.129	5.38279E-07	3.5	1.46045E-05
Sulfur dioxide (SO2)	0.098	4.08925E-07	0.017	7.09359E-08

The recommended monetized value [52] for each type of air pollutant is shown in Table 6.3.

Table 6.3 Damage Costs for Pollutant Emission [52]

Type of air pollution	**cost ($/short ton $2017)**
Carbon dioxide (CO2)	/
Volatile Organic Compounds (VOCs)	1872
Nitrogen oxides (NOx)	7377
Particulate matter (PM)	337459
Sulfur dioxide (SO2)	43600

The yearly monetized air pollution saving E (\$/ year) is calculated as follows:

$$E = \sum_{i=1}^{3} \sum_{j} E_{ij} * e_j * h_i * w - \sum_{i=1}^{3} \sum_{j} E'_{ij} * e_j * h_i * w \tag{15}$$

Emission savings calculated by these simplified methods are likely to be small compared to other savings. Therefore, the research team conducted the benefit/cost analysis with and without the consideration of emissions.

6.2.4 Crash Reduction

Safety evaluation is conducted based on the estimated crash increase/reduction before and after the implementation of the ASCT. Only intersection-related crashes which do not involve direct human error are considered for a fair comparison. As indicated earlier, additional analysis was conducted with rear-end only crashes but the results are not reported here because it is based on sparse data.

The number of crashes for five different types of crash were collected from the corridor. The annually monetized safety savings S_c (\$ /year) are calculated as follows:

$$S_c = \sum_{k} C_k * CM_k - \sum_{k} C'_k * CM_k \tag{16}$$

Where:

C_k (crashes/ year) is the annual crash rate of crash type k, where $k = 1,2,3,4,5$ represents possible injury, non-incapacitating injury, incapacitating injury, fatality, and property damages;

CM_k (\$/ crash) is the unit monetized cost for crash type k.

Table 6.4 provides the suggested monetized value for each type of crash type [56].

Table 6.4 FDOT KABCO Crash Costs

KABCO Level	Monetized Value($)
C – Possible Injury	$97,650
B – Non-incapacitating	$157,170
A – Incapacitating	$580,320
K – Killed	$10,230,000
O- Property damage only crashes	$7,600

6.3 Calculation of Costs

Table 6.5 shows the total cost of ASCT for each corridor. The values shown include equipment, installation, training, and maintenance costs. While some sites paid the vendors by each of these as line-items, others paid a lump-sum amount per intersection. Hence, for the purposes of this analysis, costs are listed as "total costs per corridor" as well as "cost per intersection".

Table 6.5 Monetized Cost for Each Corridor

Corridors name	Location	Monetized Cost		
		Cost / intersection	Number of Intersections	Total cost
University Parkway	Sarasota and Manatee, District 1	$36,500	18	$657,000
East Van Fleet Drive and North	Bartow, District 1	$96,400	5	$482,000
Newberry Road	Gainesville, District 2			
23rd Street	Bay County, District 3	$40,000	9	$360,000
Panama City Beach Parkway	Bay County, District 3	$45,000	10	$450,000
SR 70 & US 301	Manatee, Districts 4 and 5	$45,350	22	$997,700
US 17 92	DeLand, District 5	$30,000	5	$150,000
66th Street	Pinellas, District 7	$32,871	12	$394,452
Total				**$3,491,152**

Note: ASCT was removed at Newberry Rd. Hence it is not considered in B/C calculations

6.4 Benefit-Cost Analysis and Findings

This section calculates the monetized benefit-cost of ASCT for each corridor according to the process described earlier. The before-after data collection dates and peak hours evaluated for each site are included in Table 6.6. The total monetized benefit value with and without including safety benefits are calculated for each corridor and the results are shown in Table 6.7a (with emission savings) and Table 6.7b (without emission savings). Also, the benefit-cost ratio with and without including safety benefits are calculated for each corridor and the results are shown in Table 6.8a (with emission savings) and Table 6.8b (without emission savings).

Table 6.6 Data Collection Dates and Peak Hours

Site Name	AM Peak	Off Peak	PM Peak	Before Dates	After Dates
University Parkway	7am—9am	1pm—3pm	4pm—6pm	March 18 & 19, 2015	March 14 & 15, 2017
East Van Fleet Drive and North	7am—9am	9:30am—11:30pm	4pm—6pm	February 22 & 23, 2017	March 28 & 29, 2017
Newberry Road	7am—9am	1pm—3pm	4pm—6pm	September 17 & 18, 2014	November 19 & 20, 2014
23rd Street	7am—9am	11am—1pm	4pm—6pm	January 27 & 28, 2015	September 22 & 23, 2015
Panama City Beach Parkway	7am—9am	11am—1pm	4pm—6pm	October 22 & 23, 2014	February 3 & 4, 2015
SR 70 & US 301	7am—9am	10am—12pm	4pm—6pm	December 8 & 9, 2016	October 3 & 4, 2018
US 17 92	7am—9am	11am—1pm	4pm—6pm	October 1 & 2, 2014	December 16 & 17, 2014
66th Street	7am—9am	1pm—3pm	4pm—6pm	November 9 & 10, 2016	May 16 & 17, 2017

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Table 6.7a Monetized Benefit Value for Each Corridor (Includes Emission Savings)

Corridors name	Location	Monetized Benefit					
		Travel Time Saving	Vehicles Cost Saving	Air Pollution Saving	Safety benefit	Total with safety	Total without safety
University Parkway	Sarasota and Manatee Counties, District 1	$4,373,802	$473,639	$30,469	$4,880,524	$9,758,434	$4,877,910
East Van Fleet Drive and North	Bartow, District 1	-$401,717	-$60,823	-$32,104	$472,349	-$22,295	-$494,644
Newberry Road	Gainesville, District 2						
23rd Street	Bay County, District 3	$1,588,365	$195,702	$39,451	-$189,008	$1,634,509	$1,823,518
Panama City Beach Parkway	Bay County, District 3	$3,016,459	$358,578	$74,906		$3,449,942	$3,449,942
SR 70 & US 301	Manatee, Districts 4 and 5	-$108,204	-$53,528	-$95,495		-$257,227	-$257,227
US 17 92	DeLand, District 5	$3,454,869	$489,110	$91,940	-$2,565,559	$1,470,361	$4,035,920
66th Street	Pinellas, District 7	$4,652,124	$665,447	$132,495	$23,316,234	$28,766,300	$5,450,066
Total		**$16,575,697**	**$2,068,125**	**$241,663**	**$25,914,540**	**$44,800,025**	**$18,885,485**

Table 6.7b Monetized Benefit Value for Each Corridor (Excludes Emission Savings)

Corridors name	Location	Monetized Benefit				
		Travel Time Saving	Vehicles Cost Saving	Safety benefit	Total	Total without safety
University Parkway	Sarasota and Manatee Counties, District 1	$4,373,802	$473,639	$4,880,524	$9,727,965	$4,847,441
East Van Fleet Drive and North	Bartow, District 1	-$401,717	-$60,823	$472,349	-$9,809	-$462,540
Newberry Road	Gainesville, District 2					
23rd Street	Bay County, District 3	$1,588,365	$195,702	-$189,008	$1,595,058	$1,784,067
Panama City Beach Parkway	Bay County, District 3	$3,016,459	$358,578		$3,375,037	$3,375,037
SR 70 & US 301	Manatee, Districts 4 and 5	-$108,204	-$53,528		-$161,731	-$161,731
US 17 92	DeLand, District 5	$3,454,869	$489,110	-$2,565,559	$1,378,421	$3,943,979
66th Street	Pinellas, District 7	$4,652,124	$665,447	$23,316,234	$28,633,805	$5,317,571
Total		**$16,575,697**	**$2,068,125**	**$25,914,540**	**$44,558,363**	**$18,643,823**

Table 6.8a Benefit and Cost Summary (Includes Emission Savings)

Corridors name	Location	Total cost	Total Benefits		B/C Ratio		Comments
			With Safety	Without safety	With safety	Without safety	
University Parkway	Sarasota and Manatee D1	$657,000	$9,758,434	$4,877,910	14.9	7.4	
East Van Fleet Drive and North	Bartow, D1	$482,000	-$22,295	-$494,644	0	-1	
Newberry Road	Gainesville, D2						ASCT was removed
23rd Street	Bay County, D3	$360,000	$1,634,509	$1,823,518	4.5	5.1	
Panama City Beach Parkway	Bay County, D3	$450,000	$3,449,942	$3,449,942		7.7	Crash reporting methodology changed
SR 70 & US 301	Manatee, D4 and D5	$997,700	-$257,227	-$257,227		-0.3	No safety data for after
US 17 92	DeLand, D5	$150,000	$1,470,361	$4,035,920	9.8	26.9	
66th Street	Pinellas, D7	$394,452	$28,766,300	$5,450,066	72.9	13.8	
Total		**$3,491,152**	**$44,800,025**	**$18,885,485**	**12.8**	**5.4**	

Table 6.8b Benefit and Cost Summary (Excludes Emission Savings)

Corridors name	Location	Total cost	Total Benefits		B/C Ratio		Comments
			Total	Total without safety	With safety	Without safety	
University Parkway	Sarasota and Manatee, D1	$657,000	$9,727,965	$4,847,441	14.8	7.4	
East Van Fleet Drive and North	Bartow, D1	$482,000	-$9,809	-$462,540	0	-1	
Newberry Road	Gainesville, D2						ASCT was removed
23rd Street	Bay County, D3	$360,000	$1,595,058	$1,784,067	4.4	5	
Panama City Beach Parkway	Bay County, D3	$450,000	$3,375,037	$3,375,037		7.5	Crash reporting methodology changed
SR 70 & US 301	Manatee, D4 and D5	$997,700	-$161,731	$161,731		-0.2	No safety data for after
US 17 92	DeLand, D5	$150,000	$1,378,421	$3,943,979	9.2	26.3	
66th Street	Pinellas, D7	$394,452	$28,633,805	$5,317,571	72.6	13.5	
Total		**$3,491,152**	**$44,558,363**	**$18,643,823**	**12.8**	**5.3**	

Since the ASCT was removed from Newberry Road (Gainesville), it is not considered in the benefit-cost analysis. For SR 70 (Manatee) there has not been adequate time after ASCT installation to collect crash-related data. An increase in the number of tourists over the past few years and the corresponding increase in traffic along with changes in crash reporting methodology resulted in significant increase in the reported number of crashes at Panama City Beach. Hence it was decided to include safety results for this site in benefit-cost Analysis.

As expected, emission savings are negligible compared to other benefits, and the overall B/C for the project does not change with the inclusion of emission savings. However, considering the safety benefits, the overall B/C increases from 5.4 to 12.8.

Without considering safety benefits, five of the seven corridors had positive outcomes by adopting the new system. The 66th Street (Pinellas) and University Parkway (Sarasota and Manatee) have the highest monetized benefit value of $5,450,066 and $4,877,910 respectively. The US 17 92 (Deland) had the best B/C: 26.9. Although the East Van Fleet Drive and North (Bartow) and SR 70 have negative benefit values, the value is close to zero (i.e. the introduction of ASCT does not produce significant changes to quality of service for these two corridors).

All the results above are for a 1-year analysis. To conduct a 10-year analysis, one would need to consider annual discounting for monetized benefits rate (i.e. how the commuters perceive the benefits from year 2 to year 10), recurring maintenance costs (how much additional dollars would it cost to maintain ASCT versus the system before), and fuel pricing scenarios (which needs to consider whether fuel prices remain same or change over time). Due to significant uncertainty in all these parameters, it was decided to present only the Year One benefit-cost analysis.

6.5 Benefit-Cost Findings and Conclusion

Overall, as indicated by the positive monetized benefits, the ASCT performs well for most of the corridors and for the overall program. The benefits are mainly attributed to reduced travel time along the corridors. Since KABCO values weigh the fatalities heavily, safety benefits are extremely variable and could swing from net negative to net positive due to a single fatality.

7. CONCLUSIONS AND RECOMMENDATIONS

This project evaluated the implementation of ASCT along eight corridors in Florida. Traffic engineering and safety analyses were conducted and several quantitative relationships between site characteristics and performance measures were developed.

Field data (before and after ASCT installations) were collected through floating car runs and turning movement counts along the study corridors during AM, PM and Off Peak periods. Changes in travel time, major street queue strorage ratio and traffic throughput show that ASCT generally resulted in an average overall reduction in travel time (9.36%) and major street queues (15.57%). The minor street queues increase (16.98%), however their throughput remains almost the same (0.69%). Regression analysis showed that higher AADT, higher intersection density (signalized and unsignalized), and higher initial operating speed (before implementation of ASCT) result in less traffic operational improvement.

US17 in Deland, 23rd street in Panama City and East Van fleet Drive in Bartow showed reduction in crashes when conducting a longer-term analysis, whereas University Parkway in Sarasota showed improvement only for the shorter-term analysis. Among the two corridors that showed increase in crashes, SR 693 in Pinellas had a short "after" period for data collection and Beach Parkway in Panama City had increasing tourist demand over the years leading to higher seasonal traffic, potentially offsetting any safety benefits of signal coordination.

Benefit-cost Analysis revealed overall net positive monetized benefits (12.8 considering safety, 5.4 without safety). Overall, ASCT performed well for most of the corridors and for the entire program.

The following are recommendations from this study:

- A high density of access points (i.e. both signalized and unsignalized intersections) along the corridor reduces the effectiveness of ASCT. This was very evident on the Newberry Rd. corridor where, on an average there was an access point every 1/22th of a mile and an unsignalized intersection every 1/14th of a mile. Some of these access points had heavy left turning traffic to/from major establishments such as the Oaks Mall and the North Florida Regional Medical Center. Also, many of these trips originate and end on the same side of Newberry Rd. but they have to use the mainline arterial because there is no connectivity between adjacent businesses.

- While an upper threshold on distance between signalized intersections (¾ miles, as specified in the FDOT guidance document [57]) is necessary to maintain the progression, high density of access points creates difficulties with detection and ASCT may not be effective at such locations. Access management steps such as closing non-essential access roads, linking internal roads along the same side of the corridor, increasing the distance between intersections, planning effective median openings and street connections, may be considered before ASCT is introduced.

- Detection and Communication were noted as top challenges by all the TMCs in their interviews. Regular maintenance of the detection and the ASCT system is necessary for a successful operation. Generally, the counties that had a maintenance contract with vendors had better ASCT performance. Only one county (Pinellas) had an internal training program for the staff and had an additional staff member to monitor the performance of ASCT. Pinellas showed the best overall improvement in terms of traffic operations (travel time, queues and throughput) among all sites. ASCT is not a "set it and forget it" system. Maintenance, training and appropriate staffing are key factors in the success of ASCT.

- ASCT is unlikely to show improvements at corridors with high volume cross streets (for example, SR 70 at Manatee County with cross street US 301). It simply redistributes the green to the cross street and the gains from improvement on the side streets are nullified by deterioration on the major street.

- Observation from the data, interviews and statistical analyses showed that ASCTs are not suitable for implementation for all types of corridors and traffic conditions. The analyses confirmed the empirical observations that ASCT would yield better performance and a higher return on investment when implemented on corridors with low intersection density, low volume side streets, and high demand but not oversaturated traffic conditions.

- Several corridors had minor to major ongoing construction projects during ASCT implementation. The installation and initial fine tuning periods have to be planned accordingly.

- Florida attracts heavy seasonal traffic (for example, snowbirds during winter at Sarasota and college students during spring break at Panama City beach). Counties should avoid introducing new and unfamiliar traffic patterns at the start of these seasons. Steps must be taken to educate travelers about ASCT and particularly unfamiliar phasing and timing patterns.

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APPENDIX A: Summary of Newberry Rd., Alachua County

A.1 EXECUTIVE SUMMARY

The objective of this research is to evaluate the implementation of proposed Adaptive Signal Control Technologies (ASCT) traffic operations at several arterial corridors in Florida, before and after the installation of specific ASCT, document the advantages and disadvantages of different approaches and implementations, and provide recommendations for state-wide implementation of ASCT. This Appendix summarizes the before and after field data collected along the Newberry Rd., Gainesville corridor from NW 76th Blvd to NW 8th Ave.

The SynchroGreen advanced signal control system was implemented along this corridor in December 2014. Floating car runs were conducted with the UFTI instrumented vehicle to collect vehicle travel times before the implementation of SynchroGreen, during three time periods (AM Peak, Off Peak and PM Peak). In addition, turning movement counts and queue lengths were collected at two critical intersections (NW 75th St. and I-75 N). Based on these, five performance measures were obtained for the before and after study periods: Link/Route Travel Time, Delay at Intersections, Queue Length (at critical intersections), Queue to Lane Storage Ratio (at critical intersections), and Passenger Car Equivalent (PCE) flows (at critical intersections). For each performance measure, a comparison between the before and after data is conducted and presented in this appendix.

The following were observed:

- The travel time in the EB direction increased during all study time periods (AM Peak, Off Peak and PM Peak) by an average of 1.25 min (22.6%). The travel time in the WB direction increased during the AM Peak and Off Peak by an average of 0.42 min (9.0%) and it decreased during the PM Peak by 0.84 min (10.4%).
- The intersection through movement delay in the EB direction increased for 9 out of the 12 intersections along the corridor by an average of 6.10 sec/intersection (43.6%). The intersection through movement delay in the WB direction increased at 8 out of 12 intersections by an average of 2.11 sec/intersection (14.6%).
- The average queue length for the I-75 NB & Newberry Rd. intersection decreased by an average of 0.32 vehicles (-7.2%) over all analysis periods. The average queue length for the I-75 SB & Newberry Rd. intersection increased during all analysis periods by an average of 1.38 vehicles (18.5%).
- The traffic volume at both intersections (I75N and 75 St.) increased during all three analysis periods by an average of 78 pce/h/ln (9.9%) and 30 pce/h/ln (8.4%) respectively.

City of Gainesville staff were interviewed regarding the effectiveness of the signal control along the corridor, and they also confirmed that the performance of the corridor deteriorated with the installation of SynchroGreen. They speculated that SynchroGreen could be more effective for less congested corridors as well as when advance detection is present (there is currently no advance detection.)

A.2 CORRIDOR INFORMATION

Gainesville is college-based, and Newberry Rd. is one of the major corridors in the city of Gainesville, which connects the University of Florida campus area (east-end) and the Interstate-75 for entering/leaving town (west-end). The adjacent land uses include malls, restaurants, hotels, gas stations, drug stores, automotive service shops and banks. Interstate-75 represents a major north-south freeway link for inter-city commuters throughout the entire North Central Florida region, and is thus expected to serve very high volumes of traffic entering/leaving the Gainesville area during the morning or afternoon peak period. A shopping mall is also located along the corridor, which is expected to generate/attract high traffic demands.

The dominant traffic during the morning peak is eastbound towards the city center, while the dominant traffic during the afternoon peak is westbound for leaving Gainesville. The traffic volume was quite high during the peak periods and congestions were often observed.

Figure A-1 provides a schematic of the Newberry Rd., Gainesville corridor. Table A-1 lists the intersections along the corridor. Two intersections (I-75 N and 75th St) were selected as the critical intersections along the corridor and detailed turning movement and queue counts were collected at these. Data collection was conducted during three time periods: 7 - 9 am, 1 pm – 3pm, and 4 - 6 pm. Figure A-2 provides the lane configuration of these two critical intersections.

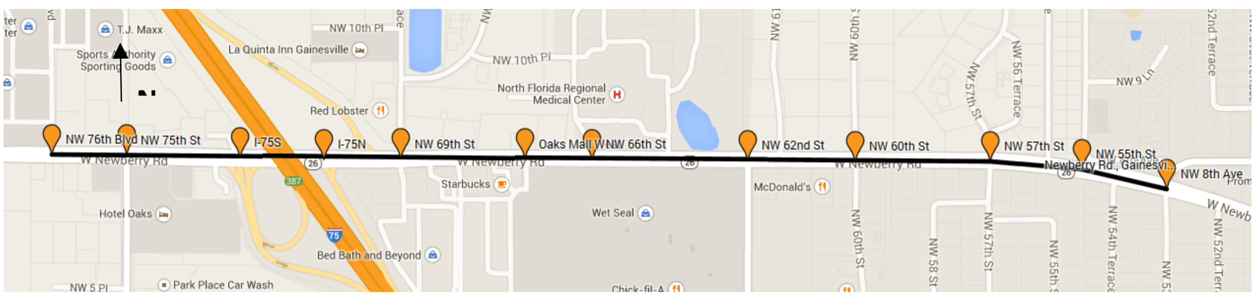
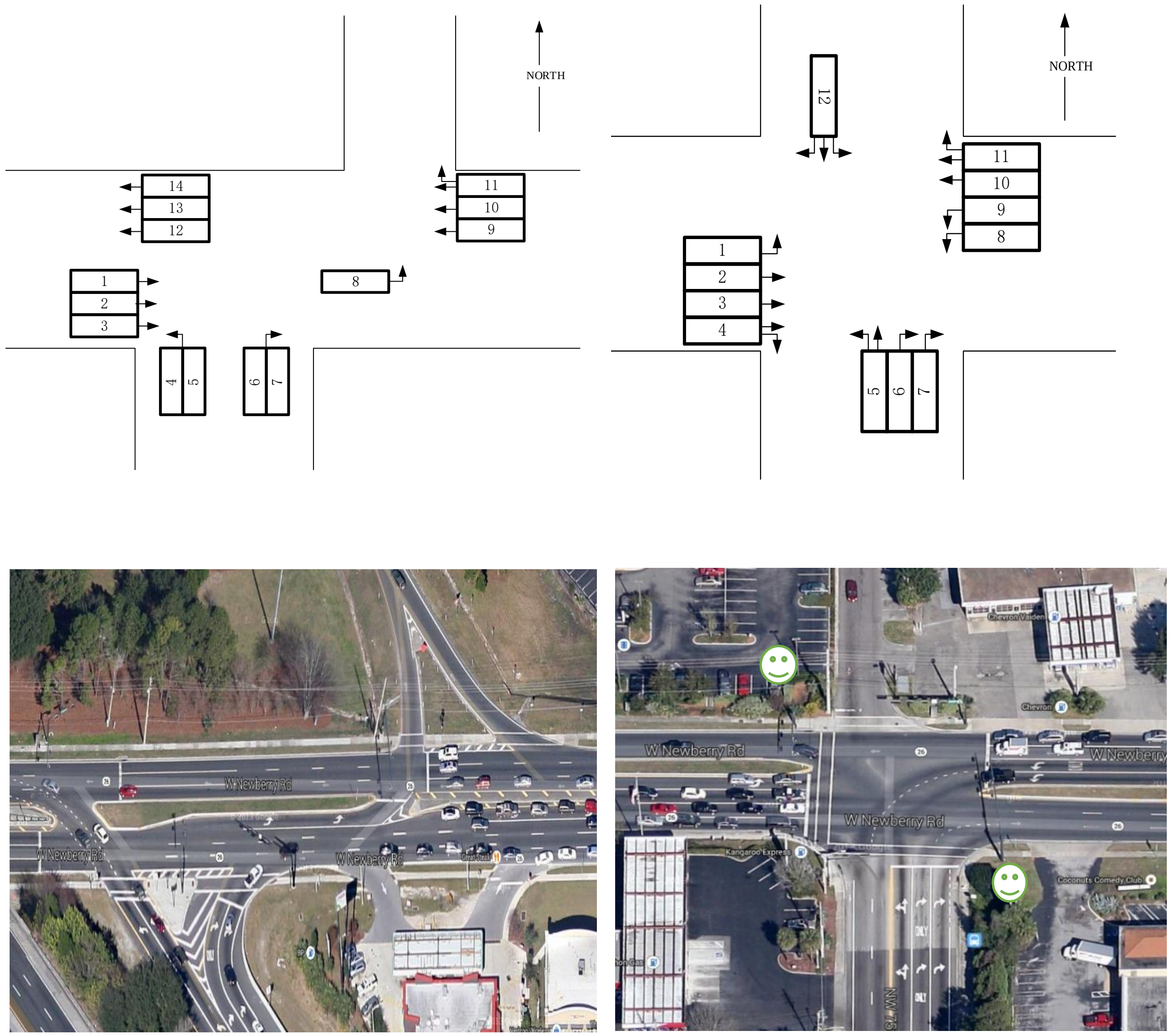


Figure A-1 Schematic of the Newberry Rd., Gainesville Corridor

Table A-1 Intersections along the Newberry Rd., Gainesville Corridor

	Intersection	Distance in Between (mile)	No. of Unsignalized Intersections in Between
1	NW 76th Blvd & Newberry Rd.		
2	**NW 75th St & Newberry Rd.**	**0.1**	**0**
3	I-75 S & Newberry Rd.	0.14	2
4	**I-75 N & Newberry Rd.**	**0.13**	**1**
5	NW 69th Terrace & Newberry Rd.	0.08	1
6	Oaks Mall West & Newberry Rd.	0.15	0
7	NW 66th St & Newberry Rd.	0.09	1
8	NW 62nd St & Newberry Rd.	0.2	1
9	NW 60th St & Newberry Rd.	0.14	1
10	NW 57th St & Newberry Rd.	0.18	0
11	NW 55th St & Newberry Rd.	0.12	1
12	NW 8th Ave & Newberry Rd.	0.12	1
Total	12 intersections	1.45	



(a) I75N & Newberry Rd.

(b) 75 ST & Newberry Rd.

Figure A-2 Schematic and Overview of Critical Intersections

A.3 PERFORMANCE MEASURES

Five performance measures were evaluated: Link/Route Travel Time, Delay at Intersections, Queue Length (critical intersections), Queue-to-Lane Storage Ratio (critical intersections), and PCE Flows (critical intersections). For each performance measure, a comparison between the before and after data is conducted and the results of the differences ("after data" – "before data") are presented.

A.3.1 ROUTE TRAVEL TIME

The average travel time (min) along the route was measured using a floating car. During the before data collection there was an incident on the I-75 ramp. The research team collected data during that time, and in the following tables we report separately any data collected when the incident was active. A total of 54 runs for EB and 51 runs for WB were conducted during the before study, and a total of 51 runs for EB and 51 runs for WB were conducted during the after study. The data are summarized below.

Before Study (Sept. 17 & Sept.18, 2014)

Table A-1.1 Route Travel Time (min)

Route TT (min)	AM Incident	AM No Incident	Off Peak	PM	Average
Newberry Rd. EB	7.7	4.83	5.24	6.45	**5.51**
Newberry Rd. WB	5.93	4.35	4.96	8.1	**5.8**

After Study (Nov. 19 & Nov. 20, 2014)

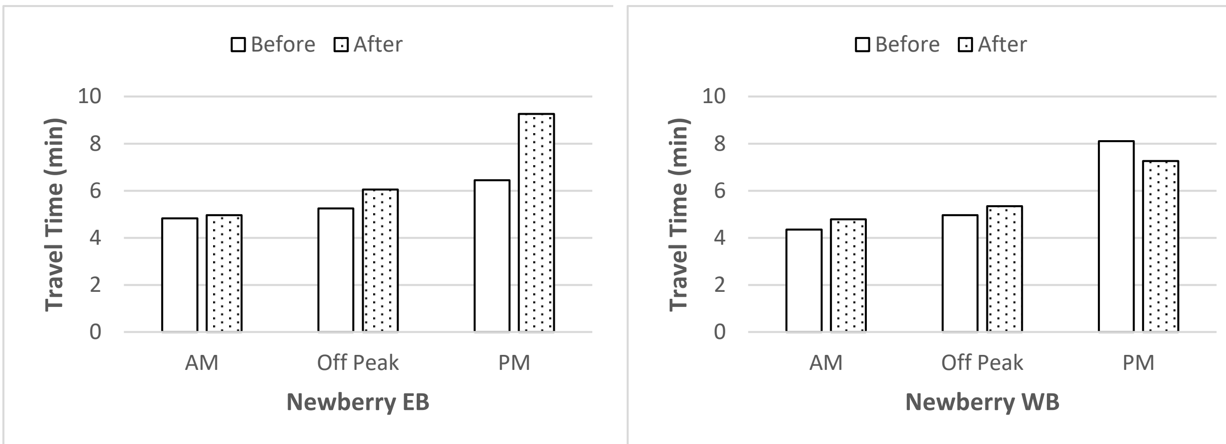
Table A-1.2 Route Travel Time (min)

Route TT (min)	AM	Off Peak	PM	Average
Newberry Rd. EB	4.96	6.05	9.26	6.76
Newberry Rd. WB	4.79	5.35	7.26	5.8

Comparisons of Before and After Travel Times

Table A-1.3 Change in Percentage of Route Travel Time (After – Before)

Route TT (min)	AM	Off Peak	PM	Average
Newberry Rd. EB	0.13 (2.7%)	0.80 (15.3%)	2.82 (43.7%)	**1.25 (22.7%)**
Newberry Rd. WB	0.44 (10.1%)	0.38 (7.7%)	-0.84 (-10.4%)	**-0.01 (-0.1%)**



(a) Newberry Rd. EB (b) Newberry Rd. WB

Figure A-1.1 Travel Times Along Newberry Rd., Gainesville

Discussion

The following can be concluded from the comparison of travel times:

- The travel time along Newberry Rd. in the EB increased in the AM Peak, Off Peak and PM Peak by an average of 1.25 min (22.7%). The EB direction carries the majority of the traffic during the AM peak. The highest increase occurred during the PM Peak when the demand is relatively lower, with an increase of 2.82 min (43.7%).
- The travel time along Newberry Rd. in the WB increased in the AM Peak and Off Peak by an average of 0.42 min (9.0%), while it decreased in the PM Peak by 0.84 min (10.4%). It is the WB direction that carries the majority of the traffic in the PM peak, and this shows improved travel times.
- In the before data, the AM peak travel time along Newberry in the EB direction is higher than that of the WB direction, which is reasonable as the majority of the traffic travels EB toward the University of Florida. The reverse occurs during the PM Peak, and it is the WB that has the higher travel time. However, in the after study, the heavily traveled WB direction has a lower travel time. It appears that the system favors the peak direction at a significant detriment to the opposing lower demand direction.

A.3.2 DELAY

Delay (sec) at each intersection along the corridor was also obtained using the floating car measurements.

Before Study (Sept. 17 & Sept.18, 2014)

Table A-2.1 Delay (sec) for each Intersection Through Movement Along the EB Direction

Delay by Intersection (sec)	AM Incident	AM No Incident	Off Peak	PM
NW 76th Blvd	78.4	18.6	35.28	50.52
NW 75th St	73.3	32.13	23.4	36.75
I-75 S	43.7	14.83	19.68	37.58
I-75 N	34.4	16.1	4.45	4.01
NW 69th Terrace	5	10.47	4.12	25.13
Oaks Mall W	12.4	15.07	5.24	13.69
NW 66th St	15.5	4.5	12.21	7.69
NW 62nd St	10.6	8.13	11.61	48.15
NW 60th St	13.9	2.73	3.68	5.55
NW 57th St	2.95	5.43	1.5	7.37
NW 55th St	9	1.67	1.45	7.54
NW 8th Ave	6.9	3.13	2.04	2.7

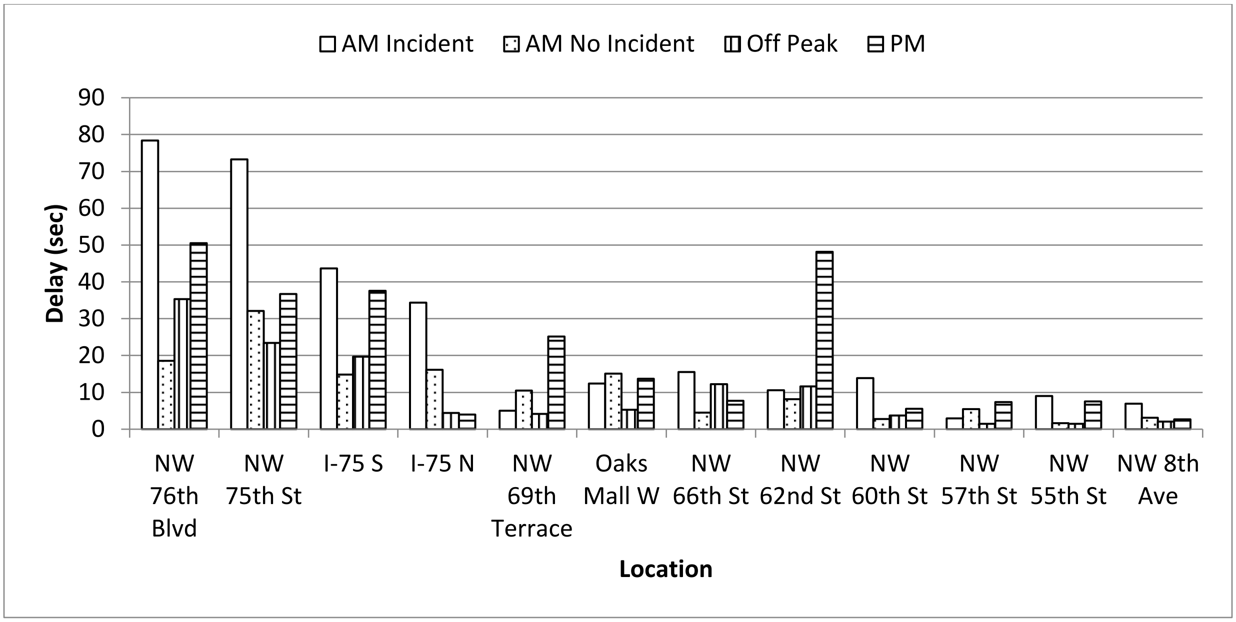


Figure A-2.1 Delay (sec) for each Intersection Through Movement Along the EB Direction

Table A-2.2 Delay (sec) for each Intersection Through Movement Along the WB Direction

Delay at Intersections (sec)	AM Incident	AM No Incident	Off Peak	PM
NW 8th Ave	9.8	4.27	11	43.38
NW 55th St	20.5	16.47	3.41	11.88
NW 57th St	2.7	1.6	1.39	4.34
NW 60th St	7.5	17.9	10.73	20.7
NW 62nd St	6.9	3.2	6.52	25.2
NW 66th St	3.6	3.8	23.78	58.76
Oaks Mall W	1.6	1.03	12.85	25.1
NW 69th Terrace	82	22.6	19.69	29.24
I-75 N	13.8	5.8	2.51	11.1
I-75 S	14.46	13	17.07	23.66
NW 75th St	11.94	3.67	22.39	23.66
NW 76th Blvd	8.16	11.33	4.2	3.35

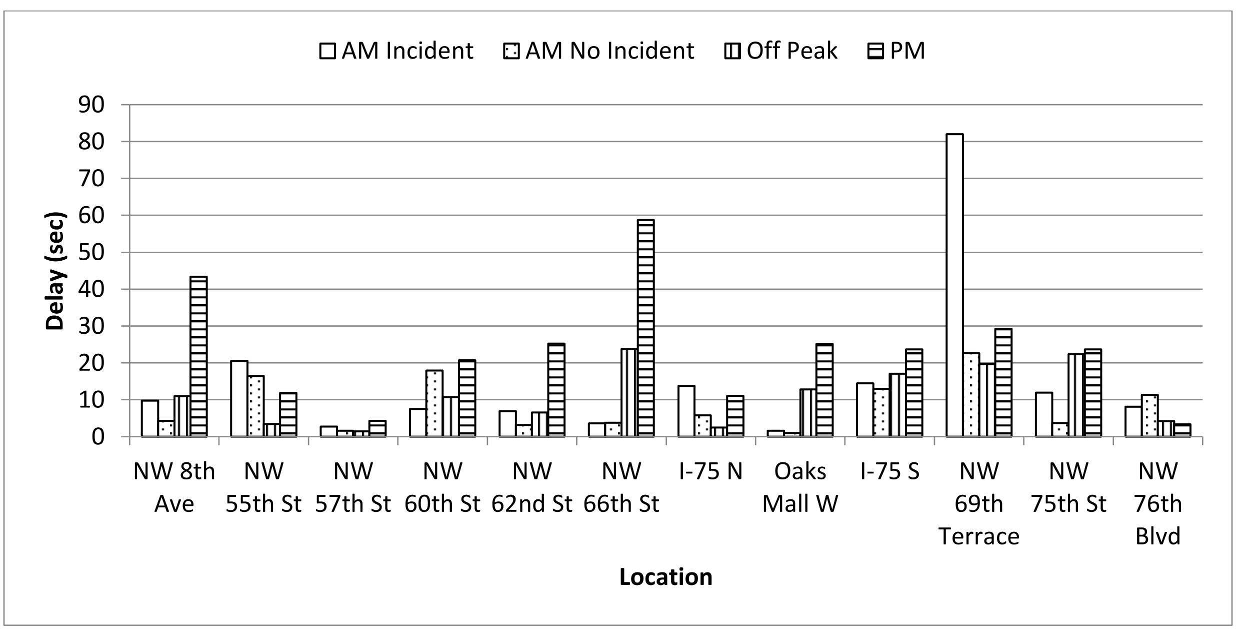


Figure A-2.2 Delay (sec) for each Intersection Through Movement Along the WB Direction

After Study (Nov. 19 & Nov. 20, 2014)

Table A-2.3 Delay (sec) for each Intersection Through Movement Along the EB Direction

Delay at Intersections (sec)	AM Peak	Off Peak	PM Peak
NW 76th Blvd	17.7	61.14	85.55
NW 75th St	31.8	26.46	71.5
I-75 S	27.16	49.05	84.7
I-75 N	18.84	3.82	6.75
NW 69th Terrace	2.75	6.03	1.5
Oaks Mall W	11.27	9.93	18.8
NW 66th St	1.85	11.46	7.4
NW 62nd St	21.16	19.47	71.81
NW 60th St	4.61	5.4	14.9
NW 57th St	1.03	3.06	5.48
NW 55th St	0.92	2.29	7.5
NW 8th Ave	1.36	6.16	3.2

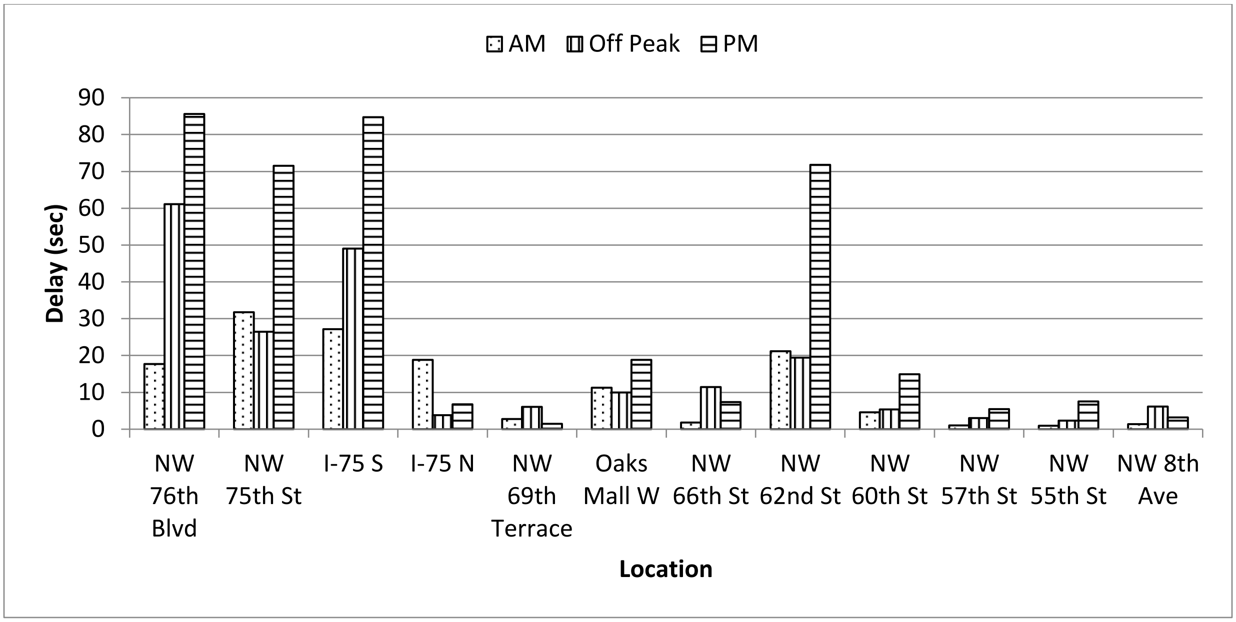


Figure A-2.3. Delay (sec) for each Intersection Through Movement Along the EB Direction

Table A-2.4 Delay (sec) for each Intersection Through Movement Along the WB Direction

Delay at Intersections (sec)	AM Peak	Off Peak	PM Peak
NW 8th Ave	10.55	30	36.15
NW 55th St	8.76	9.88	12
NW 57th St	1.37	2.65	10.4
NW 60th St	8.43	11.86	21.95
NW 62nd St	5.91	9.44	14.95
NW 66th St	11.58	20.62	45.15
Oaks Mall W	4.56	7.94	15.8
NW 69th Terrace	57.5	20.23	46.05
I-75 N	8.87	2.27	16.48
I-75 S	31.63	9.65	24.62
NW 75th St	13.6	27.33	16.7
NW 76th Blvd	5.5	8.16	8.17

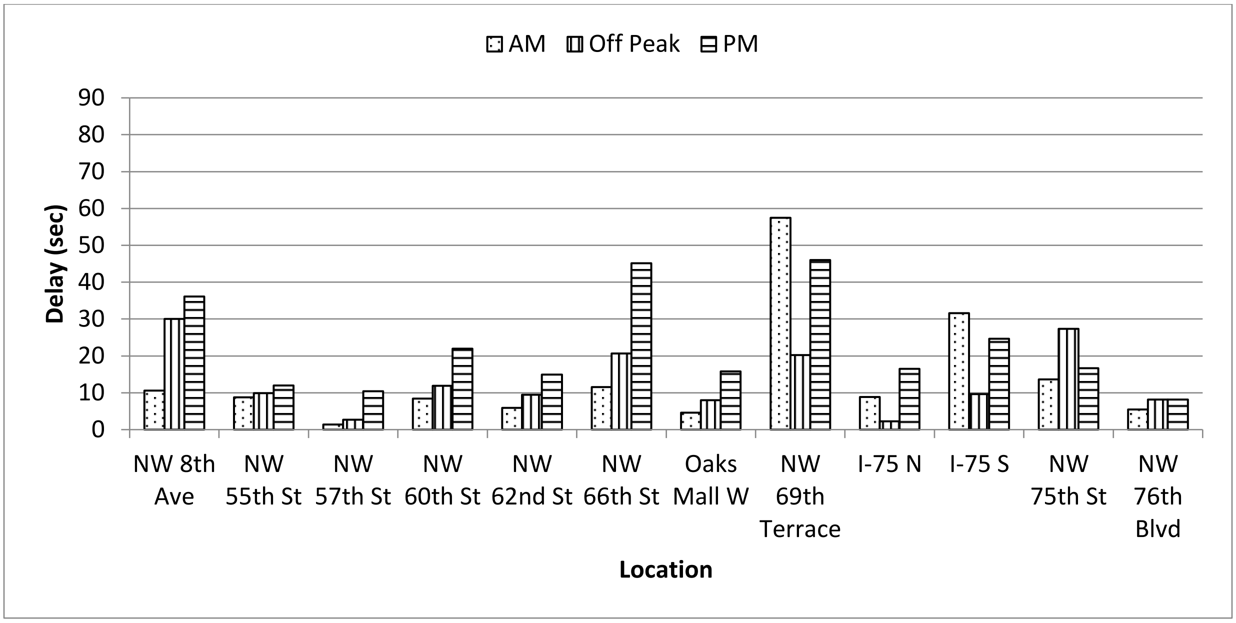


Figure A-2.4 Delay (sec) for each Intersection Through Movement Along the WB Direction

Comparisons of Before and After Intersection Delay Times

The differences in delay between the before and after study periods are shown in Table A-2.5 and Table A-2.6. The tables are color-coded as follows: green shows significant improvement, yellow shows modest change, and red shows significant deterioration in delay.

Table A-2.5 Difference in Delay (sec) for each Intersection Through Movement Along the EB Direction

Delay (sec)	AM Peak	Off Peak	PM Peak	Average
NW 76th Blvd	-0.9	25.86	35.04	20
NW 75th St	-0.33	3.06	34.75	12.49
I-75 S	12.33	29.37	47.12	29.61
I-75 N	2.73	-0.62	2.74	1.62
NW 69th Terrace	-7.72	1.91	-23.63	-9.81
Oaks Mall W	-3.79	4.69	5.11	2
NW 66th St	-2.65	-0.75	-0.29	-1.23
NW 62nd St	13.03	7.86	23.65	14.85

Delay (sec)	AM Peak	Off Peak	PM Peak	Average
NW 60th St	1.88	1.72	9.35	4.32
NW 57th St	-4.41	1.56	-1.89	-1.58
NW 55th St	-0.75	0.84	-0.04	0.02
NW 8th Ave	-1.77	4.12	0.5	0.95
Average	0.64	6.64	11.03	6.1

Table A-2.6 Difference in Delay (sec) for each Intersection Through Movement Along the WB Direction

Delay (sec)	AM Peak	Off Peak	PM Peak	Average
NW 8th Ave	6.28	18.99	-7.23	6.02
NW 55th St	-7.71	6.48	0.12	-0.37
NW 57th St	-0.24	1.26	6.06	2.36
NW 60th St	-9.47	1.13	1.25	-2.36
NW 62nd St	2.71	2.92	-10.25	-1.54
NW 66th St	7.78	-3.16	-13.61	-3
Oaks Mall W	3.53	-4.9	-9.3	-3.56
NW 69th Terrace	34.9	0.55	16.81	17.42
I-75 N	3.07	-0.25	5.38	2.73
I-75 S	18.63	-7.42	0.96	4.06
NW 75th St	9.93	4.95	-6.96	2.64
NW 76th Blvd	-5.83	3.96	4.82	0.98
Average	5.3	2.04	-1	2.11

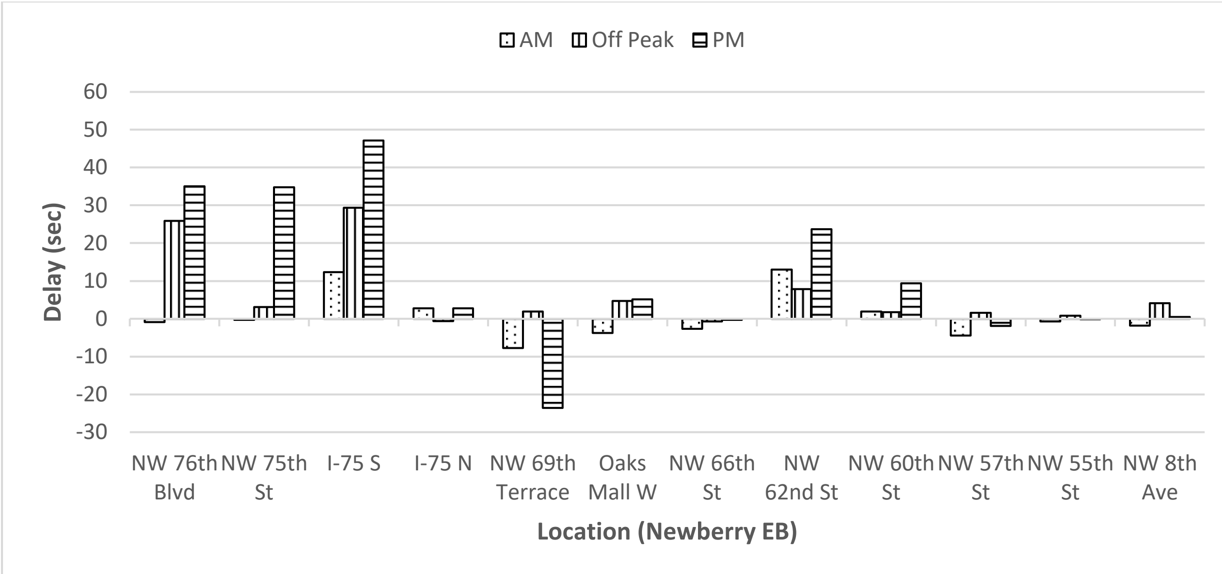


Figure A-2.5 Difference in Delay (sec) for each Intersection Through Movement Along the EB Direction

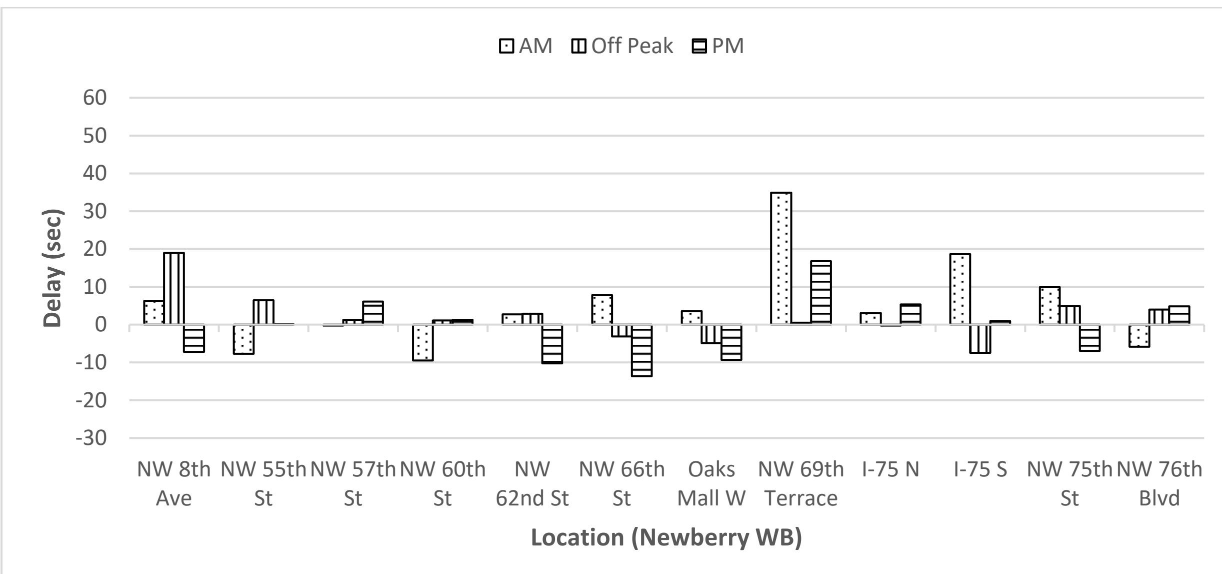


Figure A-2.6 Difference in Delay (sec) for each Intersection Through Movement Along the WB Direction

Discussion

The following can be concluded from the comparison of delays:

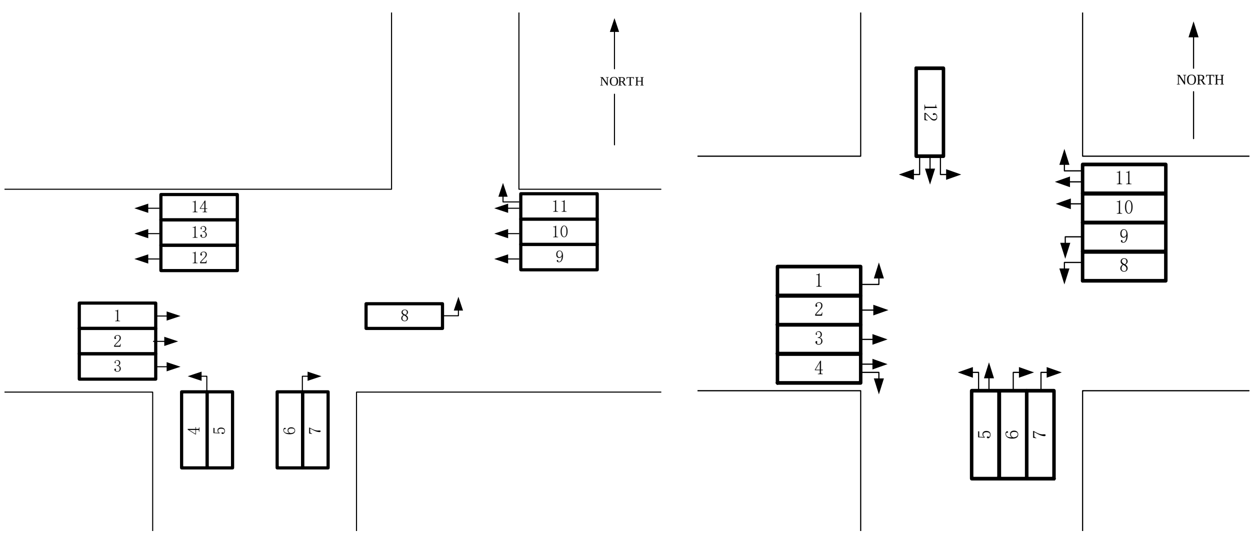
- The intersection delay in the EB direction increased at 9 out of the 12 intersections by an average of 6.10 sec/intersection. Delay increased significantly during the PM Peak, despite the fact that this is direction has a lower demand than the other during this time.

- The intersection delay in the WB direction increased at 8 out of the 12 intersections by an average of 2.11 sec/intersection. Delay increased the most during the AM Peak, which is not the predominant direction of travel at that time.
- The results of number of stops at the intersection approaches (EB and WB directions) were consistent with the intersection delay findings. Thus the detailed comparison of number of stops is not included in this report.

A.3.3 QUEUE LENGTH

Queue length (number of vehicles/lane) is presented by movement and by time period. This measure is used to evaluate oversaturated conditions at the critical intersections along the study corridors. Note that the queue length reported here is the observed maximum number of vehicles queued during each cycle, and does not represent the total number of vehicles that may have stopped during the cycle. During some time periods, because of cycle failure, vehicles need to stop multiple times before passing through the intersection.

Figure B-3.1 presents the schematic of the lane configurations at the two critical intersections. Queue length is reported for each of these lanes.



(a) I-75N & Newberry Rd.

(b) 75 St. & Newberry Rd.

Figure A-3.1 Lane Configuration of Critical Intersections

Before Study (Sept. 17 & Sept.18, 2014)

Table A-3.1 Average Queue Length by Lane (#vehs/lane) at I-75N & Newberry Rd

<table>
<tr><th colspan="2" rowspan="2">Time Period</th><th colspan="3">Eastbound</th><th colspan="4">NB</th><th>EB</th><th colspan="6">Westbound</th></tr>
<tr><th colspan="3">Thru</th><th colspan="2">Left</th><th colspan="2">Right</th><th>Left</th><th colspan="2">Thru</th><th>Right</th><th colspan="3">Thru</th></tr>
<tr><td colspan="2">Lane Number</td><td>1</td><td>2</td><td>3</td><td>4</td><td>5</td><td>6</td><td>7</td><td>8</td><td>9</td><td>10</td><td>11</td><td>12</td><td>13</td><td>14</td></tr>
<tr><td rowspan="5">AM Peak</td><td>1</td><td>8</td><td>8</td><td>7.1</td><td>5.3</td><td>5.7</td><td>7</td><td>6.9</td><td>N/A</td><td>3.4</td><td>4.3</td><td>4</td><td>0.9</td><td>0.7</td><td>1.6</td></tr>
<tr><td>2</td><td>8.4</td><td>8.4</td><td>6.6</td><td>5.3</td><td>7.6</td><td>6.6</td><td>7.4</td><td>N/A</td><td>3.3</td><td>3.8</td><td>2.7</td><td>1.8</td><td>1.8</td><td>1.2</td></tr>
<tr><td>3</td><td>9</td><td>9</td><td>9</td><td>12.9</td><td>15.3</td><td>10.6</td><td>12.4</td><td>N/A</td><td>4.3</td><td>6</td><td>7</td><td>4.8</td><td>4.5</td><td>3.3</td></tr>
<tr><td>4</td><td>9</td><td>9</td><td>9</td><td>4</td><td>7.2</td><td>3.7</td><td>3.8</td><td>N/A</td><td>3.2</td><td>3.7</td><td>2.2</td><td>1.3</td><td>1.8</td><td>1.3</td></tr>
<tr><td>Average</td><td>8.6</td><td>8.6</td><td>7.9</td><td>6.9</td><td>8.9</td><td>7</td><td>7.6</td><td>N/A</td><td>3.6</td><td>4.5</td><td>4</td><td>2.2</td><td>2.2</td><td>1.9</td></tr>
<tr><td rowspan="5">Off Peak</td><td>1</td><td>1</td><td>2</td><td>2</td><td>4.7</td><td>4.7</td><td>1.2</td><td>4.5</td><td>3</td><td>2.3</td><td>1</td><td>1.7</td><td>0.3</td><td>0.8</td><td>0.5</td></tr>
<tr><td>2</td><td>3.2</td><td>3.5</td><td>3.8</td><td>4.5</td><td>6</td><td>2.8</td><td>3.7</td><td>4.6</td><td>1.3</td><td>1.3</td><td>1.6</td><td>0</td><td>0.6</td><td>0.6</td></tr>
<tr><td>3</td><td>2.3</td><td>1.8</td><td>2.3</td><td>4.5</td><td>6.3</td><td>1.7</td><td>5.5</td><td>5</td><td>1.8</td><td>3.5</td><td>2.8</td><td>0</td><td>0.2</td><td>0.3</td></tr>
<tr><td>4</td><td>1.5</td><td>2.3</td><td>2.5</td><td>5.3</td><td>5</td><td>2.5</td><td>3.3</td><td>5.2</td><td>2.8</td><td>2.3</td><td>1.8</td><td>2.3</td><td>1.3</td><td>0.8</td></tr>
<tr><td>Average</td><td>2</td><td>2.4</td><td>2.7</td><td>4.8</td><td>5.5</td><td>2</td><td>4.3</td><td>4.4</td><td>2.1</td><td>2</td><td>2</td><td>0.7</td><td>0.7</td><td>0.6</td></tr>
<tr><td rowspan="5">PM Peak</td><td>1</td><td>1.3</td><td>0.5</td><td>1.8</td><td>8.2</td><td>10.7</td><td>3.3</td><td>2.5</td><td>3.3</td><td>5.8</td><td>5.7</td><td>2.8</td><td>7</td><td>4.5</td><td>2.8</td></tr>
<tr><td>2</td><td>1.5</td><td>1.8</td><td>1.5</td><td>11.2</td><td>10.3</td><td>1.7</td><td>3.3</td><td>6.5</td><td>5.2</td><td>3.8</td><td>4</td><td>5.3</td><td>2.8</td><td>0.8</td></tr>
<tr><td>3</td><td>1.7</td><td>1</td><td>1.2</td><td>23.8*</td><td>27.3*</td><td>2.2</td><td>3.2</td><td>7.8</td><td>5.8</td><td>6.2</td><td>3.8</td><td>4.5</td><td>4.2</td><td>2.2</td></tr>
<tr><td>4</td><td>1.5</td><td>1.3</td><td>1.8</td><td>23.3*</td><td>21.5*</td><td>2</td><td>4.7</td><td>7.5</td><td>7</td><td>6</td><td>2</td><td>5.2</td><td>4.2</td><td>2.7</td></tr>
<tr><td>Average</td><td>1.5</td><td>1.2</td><td>1.6</td><td>16.6</td><td>17.5</td><td>2.3</td><td>3.4</td><td>6.3</td><td>6</td><td>5.4</td><td>3.2</td><td>5.5</td><td>3.9</td><td>2.1</td></tr>
</table>

Note: * indicates cycle failure.

Note: Lane 8 was closed due to accident during AM Peak.

Table A-3.2 Average Queue Length (#vehs/lane) at I-75N & Newberry Rd.

Time Period	EB (Main)		WB (Main)			Side Street	
	Thru	Left	Thru(9,10,11)	Thru+Right	Thru(12,13,14)	Left	Right
AM Peak	8.4	N/A	4.01	3.96	2.1	7.9	7.29
Off Peak	2.4	4.4	2.05	1.98	0.65	5.13	3.15
PM Peak	1.4	6.3	5.69	3.17	3.85	17.04*	2.85

Note: * indicates cycle failure.

Note: Lane 8 was closed due to accident during morning peak.

Table A-3.3 Average Queue Length by Lane (#vehs/lane) at 75 St. & Newberry Rd.

Time Period		EB				NB			WB				SB
		Left	Thru		Right	Left/Thru	Right		Left		Thru	Thru/Right	All
Lane Number		**1**	**2**	**3**	**4**	**5***	**6***	**7***	**8**	**9**	**10**	**11**	**12**
AM Peak	1	1.4	15	16.4	16.6	8.2	12.6	12.6	14.4	14.2	2.8	3.4	1.6
	2	0.4	9.2	11	9.6	5	6.6	7.8	9.2	8.8	2	1.7	1.3
	3	0.1	6.7	6.3	5	7.3	4.6	4.6	8.3	7.6	1.1	1.6	2
	4	1	5.9	6.7	6.1	5	3.4	3.7	5.4	4.6	1.1	2.4	1.3
	Average	**0.7**	**9.2**	**10.1**	**9.3**	**6.4**	**6.8**	**7.2**	**9.3**	**8.8**	**1.8**	**2.3**	**1.6**
Off Peak	1	3.2	7.7	11.8	11.7	5.5	3.5	7.7	14.8	14.8	6.7	7.8	3.5
	2	0.8	4.3	5.3	6.3	5.2	4.7	7	15.0*	17.3*	6.3	6.3	2.7
	3	1.8	6.2	6.5	9.2	6	4.2	7.7	13.7*	12.2*	6.3	7.7	3.2
	4	2	6.8	7.3	9.5	6	4.2	7.3	14.3	15.8	5.8	6.7	2.5
	Average	**2**	**6.3**	**7.8**	**9.2**	**5.7**	**4.1**	**7.4**	**14.5**	**15**	**6.3**	**7.1**	**3**
PM Peak	1	2.3	11.8	15	15.7	8	5.7	6.2	9.8	8.3	4.3	5	2.3
	2	1.8	9.4	12	17.8	7.4	4.2	6.8	9.8	8.2	7.2	6.6	3.2
	3	2	13	15.4	19	8	5.8	6.8	9.8	8.8	10.2	12.8	3.2
	4	1.6	13	18.4	20	8	5.4	7.6	12.6	11.8	11.8	14.2	2.6
	Average	**1.9**	**11.8**	**15.2**	**18.1**	**7.9**	**5.3**	**6.8**	**10.5**	**9.3**	**8.4**	**9.7**	**2.8**

Note: *The queue length in Lanes 5,6, and 7 represents the maximum number of vehicles within the observers' sight distance.

Table A-3.4 Average Queue Length (#vehs/lane) at 75 St. & Newberry Rd.

Time Period	**EB (Main)**			**WB (Main)**			**NB**		**SB**
	Left	**Thru**	**Thru/Right**	**Left**	**Thru**	**Thru/Right**	**Left/Thru**	**Right**	**Left/Thru/Right**
AM Peak	0.74	9.65	9.34	9.06	1.77	2.27	6.37	6.99	1.55
Off Peak	1.96	7	9.17	14.75*	6.29	7.13	5.67	5.77	2.96
PM Peak	1.93	13.5	18.12	9.9	8.38	9.65	7.85	6.05	2.83

Note: * indicates cycle failure.

Note: The queue length of the NB movements represent the maximum number of vehicles within the observers' sight distance.

After Study (Nov. 19 & Nov. 20, 2014)

Table A-3.5 Average Queue Length by Lane (#vehs/lane) at I-75N & Newberry Rd.

Time Period		EB			NB				EB	WB					
		Thru			Left		Right		Left	Thru		Right	Thru		
Lane Number		**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**	**13**	**14**
AM Peak	1	9	9	8.6	6	6.4	7.8	10	3.2	7.6	7.6	2.6	2	0.8	2.2
	2	9	9	8	5.4	4.8	6	8.2	5.2	6.6	7.2	2.6	1.8	2.2	0.6
	3	5.4	5.4	3	4.4	5	3.4	3.4	4.4	2.4	2.9	1.1	0.3	0.5	0
	4	2.8	2.4	2	2.8	4.2	2	3.8	5	1	0.3	0.3	1.2	0	0.2
	Average	**6.6**	**6.5**	**5.4**	**4.7**	**5.1**	**4.8**	**6.4**	**4.5**	**4.4**	**4.5**	**1.7**	**1.3**	**0.9**	**0.7**
Off Peak	1	0.5	1	1.3	5.2	6.3	2.7	4.7	4.5	3.8	1.8	2.3	0.2	0.7	0.3
	2	1.5	1.7	2.2	3	5.7	1.8	4.5	5.3	3.5	2.5	2.7	0.3	0.5	0.3
	3	3.8	2.8	2.2	4	6.8	3.8	6	4.8	2.8	2.2	1.5	0.3	0	1
	4	1.2	1.5	1.7	4	6.2	1.7	5.3	7.5	4.7	3	2	2.8	1.5	1.3
	Average	**1.8**	**1.8**	**1.8**	**4**	**6.3**	**2.5**	**5.1**	**5.5**	**3.7**	**2.4**	**2.1**	**0.9**	**0.7**	**0.8**
PM Peak	1	3.8	2	2.6	11.2	10.8	2	5.2	9.6	8.4	7	5.8	9.6	5.8	5
	2	3	1.2	3	10.4	13.8	2.8	4	9.6	5.2	6.4	7.6	7.6	9.8	9
	3	1.2	1.2	2.4	9.4	10.4	2.6	4.8	7.8	6.6	4.4	3.8	7	5	4.4
	4	1.6	2	2.4	8.2	9	2.4	3.2	7.7	3.3	2	2.8	3.7	2	1.7
	Average	**2.4**	**1.6**	**2.6**	**9.8**	**11**	**2.5**	**4.3**	**8.7**	**5.9**	**5**	**5**	**7**	**5.7**	**5**

Table A-3.6 Average Queue Length (#vehs/lane) at I-75N & Newberry Rd.

Time Period	EB (Main)		WB (Main)			Side Street	
	Thru	Left	Thru(9,10,11)	Thru+Right	Thru(12,13,14)	Left	Right
AM Peak	6.13	4.46	4.45	1.67	0.98	4.88	5.58
Off Peak	1.78	5.54	3.04	2.13	0.78	5.15	3.81
PM Peak	2.2	8.67	5.42	5.01	5.88	10.4	3.38

Table A-3.7 Average Queue Length by Lane (#vehs/lane) at 75 ST & Newberry Rd.

Time Period		EB				NB			WB				SB
		Left	Thru		Right	Left/Thru	Right		Left		Thru	Thru/Right	All
Lane Number		**1**	**2**	**3**	**4**	**5***	**6***	**7***	**8**	**9**	**10**	**11**	**12**
AM Peak	1	0.6	16	16	15.4	6.8	11.4	11.8	13.8	11.5	9.5	7.7	2.3
	2	0.5	9.5	10.5	12.7	8.5	6.2	6.8	12.7	9.8	7.3	6.7	1.3
	3	0.4	5.2	7.2	9.6	6	6.6	8	8.2	9.2	1.2	2.6	0.6
	4	0.2	4.4	3.8	8.4	7.8	3.2	4.8	9.4	6	2	2.2	0.6
	Average	**0.4**	**8.8**	**9.4**	**11.5**	**7.3**	**6.8**	**7.9**	**11**	**9.1**	**5**	**4.8**	**1.2**
Off Peak	1	2.5	5.8	6.7	11.7	12	5.5	7.8	16.3*	23.8*	8	8.3	5.7
	2	0.7	5.3	5.5	11	11.2	5.7	10	15.7*	23.7*	7.8	9.3	5.2
	3	1.3	9.7	9.2	13.7	11.3	5.8	7.8	15.8*	24.2*	6.8	9	3
	4	1	6.7	5.7	11	11.3	3.7	4.8	15.2*	23.2*	6.2	9	3.3
	Average	**1.4**	**6.9**	**6.8**	**11.8**	**11.5**	**5.2**	**7.6**	**15.8**	**23.7**	**7.2**	**8.9**	**4.3**
PM Peak	1	1.3	13.2	13.2	15	12	4.7	7.2	12.6	11.8	12.2	12	4.2
	2	1.2	12	12.3	15	12	7.5	9.3	12.8	12	11.2	11.6	2
	3	1.7	14.2	15	15	12	11.5	11.7	12.2	11.4	15.4	14.4	4.4
	4	2	12.7	14.3	15	11	1.8	2.7	15.2	16	14	13.4	4.4
	Average	**1.5**	**13**	**13.7**	**15**	**11.8**	**6.4**	**7.7**	**13.2**	**12.8**	**13.2**	**12.9**	**3.8**

Note: * indicates cycle failure.

Note: The queue length in Lane 5,6,and 7 represents the maximum number of vehicles within the observers' sight distance.

Table A-3.8 Average Queue Length (#vehs/lane) at 75 ST & Newberry Rd.

Time Period	EB (Main)			WB (Main)			NB# (Side)		SB (Side)
	Left	**Thru**	**Thru+Right**	**Left**	**Thru**	**Thru+Right**	**Left+Thru**	**Right**	**Left+Thru+Right**
AM Peak	0.43	9.08	11.52	10.08	5.01	4.78	7.28	7.35	1.22
Off Peak	1.38	6.81	11.83	19.73*	7.21	8.92	11.46	6.4	4.29
PM Peak	1.54	13.35	15	13	13.2	12.85	11.75	7.04	3.75

Note: * indicates cycle failure.

Note: The queue length of the NB movements represents the maximum number of vehicles within the observers' sight distance.

Comparisons of Before and After Queue Lengths for Critical Intersections

The differences in queue length between the before and after measurements are shown in Table A-3.9 to Table A-3.12. The tables are color-coded as follows: green shows significant improvement, yellow shows modest change, and red shows significant increase in queue length.

Table A-3.9 Difference in Average Queue Length by Lane (#vehs/lane) at I-75N & Newberry Rd.

Time Period	Lane Number														Average Queue
	1	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**	**13**	**14**	
AM Peak	-2.06	-2.16	-2.53	-2.21	-3.83	-2.15	-1.29	N/A	0.84	0.05	-2.29	-0.9	-1.35	-1.11	**-1.18**
Off Peak	-0.25	-0.67	-0.83	-0.71	0.75	0.46	0.88	1.11	1.64	0.35	0.15	0.25	-0.06	0.19	**0.23**
PM Peak	0.9	0.43	1.02	-6.83	-6.46	0.16	0.88	2.38	-0.07	-0.47	1.84	1.47	1.73	2.89	**-0.01**

Note: Lane 8 was closed for part of the AM Peak in the before data collection.

Table A-3.10 Difference in Average Queue Length (#vehs/lane) at I-75N & Newberry Rd.

Time Period	EB (Main)		WB (Main)			NB (Side)	
	Thru	Left	Thru(9,10,11)	Thru+Right	Thru(12,13,14)	Left	Right
AM Peak	-2.25	N/A	0.45	-2.29	-1.12	-3.02	-1.72
Off Peak	-0.58	1.11	0.99	0.15	0.13	0.02	0.67
PM Peak	0.78	2.38	-0.27	1.84	2.03	-6.64	0.52

Note: The EB left turn lane was closed for part of the AM Peak in the before data collection.

Table A-3.11 Difference in Average Queue Length by Lane (#vehs/lane) at 75 St. & Newberry Rd.

Time Period	Lane Number												Average Queue
	1	2	3	4	5#	6#	7#	8#	9	10	11	12	
AM Peak	-0.31	-0.42	-0.73	2.18	0.9	0.04	0.69	1.7	0.34	3.24	2.52	-0.34	**0.82**
Off Peak	-0.58	0.63	-1	2.67	5.79	1.04	0.21	1.29*	8.67*	0.92	1.79	1.33	**1.9**
PM Peak	-0.39	1.19	-1.49	-3.12	3.9	1.11	0.87	2.69	3.52	4.82	3.2	0.92	**1.43**

Note: * indicates cycle failure.

Note: The queue length in Lanes 5,6, and 7 were the maximum number of vehicles within the observers' sight distance.

Table A-3.12 Difference in Average Queue Length (#vehs/lane)) at 75 St. & Newberry Rd.

Time Period	EB (Main)			WB (Main)			NB# (Side)		SB (Side)
	Left	Thru	Thru+Right	Left	Thru	Thru+Right	Left+Thru	Right	Left+Thru+Right
AM Peak	-0.31	-0.57	2.18	1.02	3.24	2.52	0.9	0.36	-0.34
Off Peak	-0.58	-0.19	2.67	4.98*	0.92	1.79	5.79	0.62	1.33
PM Peak	-0.39	-0.15	-3.12	3.1	4.82	3.2	3.9	0.99	0.92

Note: * indicates cycle failure.

Note: The queue length of the NB movements represent the maximum number of vehicles within the observers' sight distance.

Discussion

The following can be concluded from the comparison of queue length:

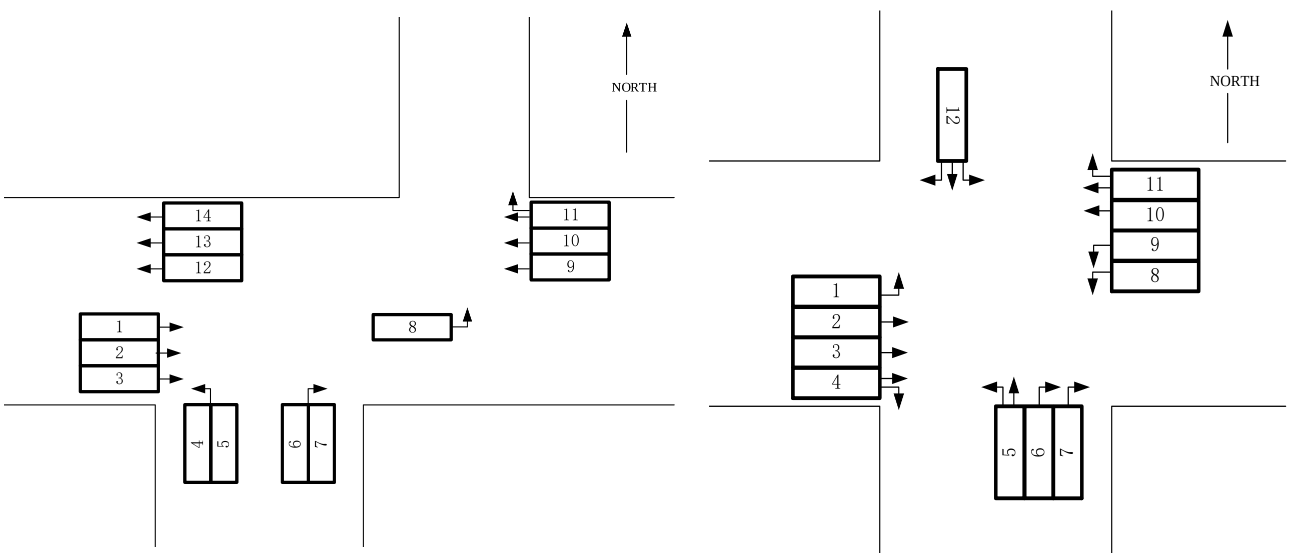
- For the I-75N & Newberry Rd intersection, the average queue length decreased by an average of 1.18 vehicles in the AM Peak, but increased by 0.23 vehicles in the Off Peak. The highest increase was observed in the EB left turn queue, which increased during both the Off Peak and PM Peak periods.
- For the 75 St. & Newberry Rd. intersection, the average queue increased during all three time periods by an average of 1.5 vehicles.

A.3.4 QUEUE TO LANE STORAGE RATIO

In addition to queue length, it is important to assess any impact to adjacent lanes or to upstream facilities. The queue to link/lane ratio is used to establish the likelihood of spillback, which is presented in this section by movement and by time period.

The following assumptions are used:

- The storage capacity is estimated as the maximum number of vehicles in the lane.
- The queue to link/lane storage ratio is estimated as 1 if the observer reported "spillback", and as 0.8 if reported as the maximum number of vehicles in the sight distance.
- Queue to lane storage ratios over 80% are highlighted in yellow, as they represent conditions with a high probability for spillback.



(a) I-75N & Newberry Rd.

(a) 75 St. & Newberry Rd.

Figure A-4.1 Lane Configuration of Critical Intersections

Before Study (Sept. 17 & Sept.18, 2014)

Table A-4.1 Average Queue Storage Ratio by Lane and by Period at I-75N & Newberry Rd.

Time Period		**Lane Number**													
		1	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**	**13**	**14**
AM Peak	1	0.8	0.8	0.71	0.2	0.22	0.27	0.26	N/A	0.34	0.43	0.4	0.09	0.07	0.16
	2	0.84	0.84	0.66	0.2	0.29	0.25	0.29	N/A	0.33	0.38	0.27	0.18	0.18	0.12
	3	0.9	0.9	0.9	0.56	0.69	0.47	0.54	N/A	0.43	0.6	0.7	0.48	0.45	0.33
	4	0.9	0.9	0.9	0.15	0.28	0.14	0.15	N/A	0.32	0.37	0.22	0.13	0.18	0.13
	Average	**0.86**	**0.86**	**0.79**	**0.28**	**0.37**	**0.28**	**0.31**	**N/A**	**0.36**	**0.44**	**0.4**	**0.22**	**0.22**	**0.19**
Off Peak	1	0.1	0.2	0.2	0.18	0.18	0.04	0.17	0.3	0.23	0.1	0.17	0.03	0.08	0.05
	2	0.32	0.35	0.38	0.17	0.23	0.11	0.14	0.49	0.13	0.13	0.16	0	0.06	0.06
	3	0.23	0.18	0.23	0.17	0.24	0.06	0.21	0.5	0.18	0.35	0.28	0	0.02	0.03
	4	0.15	0.23	0.25	0.21	0.19	0.1	0.13	0.55	0.28	0.23	0.18	0.23	0.13	0.08
	Average	**0.2**	**0.24**	**0.27**	**0.18**	**0.21**	**0.08**	**0.16**	**0.46**	**0.21**	**0.2**	**0.2**	**0.07**	**0.07**	**0.06**
PM Peak	1	0.13	0.05	0.18	0.31	0.41	0.13	0.1	0.33	0.68	0.6	0.28	0.73	0.48	0.28
	2	0.15	0.18	0.15	0.43	0.4	0.06	0.13	0.68	0.55	0.38	0.43	0.55	0.28	0.08
	3	0.17	0.1	0.12	0.85	0.94	0.08	0.12	0.88	0.6	0.63	0.38	0.48	0.42	0.22
	4	0.15	0.13	0.18	0.87	0.79	0.08	0.18	0.82	0.73	0.65	0.2	0.53	0.42	0.3
	Average	**0.15**	**0.12**	**0.16**	**0.62**	**0.63**	**0.09**	**0.13**	**0.68**	**0.64**	**0.57**	**0.33**	**0.58**	**0.4**	**0.22**

Note: Lane 8 was closed during part of the AM Peak.

Table A-4.2 Number of Cycles with Spillback at I-75N & Newberry Rd.

Time Period		#cycles in 15min	Lane Number													
			1	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**	**13**	**14**
AM Peak	1	**7**	5	5	3	0	0	0	0	N/A	0	0	0	0	0	0
	2	**6**	4	4	1	0	0	0	0	N/A	0	0	0	0	0	0
	3	**6**	6	6	6	1	2	1	1	N/A	0	2	2	1	0	1
	4	**6**	6	6	6	0	0	0	0	N/A	0	0	0	0	0	0
	Average		**5.25**	**5.25**	**4**	**0.25**	**0.5**	**0.25**	**0.25**	**N/A**	**0**	**0.5**	**0.5**	**0.25**	**0**	**0.25**
Off Peak	1	**6**	0	0	0	0	0	0	0	0	0	0	0	0	0	0
	2	**6**	0	0	0	0	0	0	0	1	0	0	0	0	0	0
	3	**6**	0	0	0	0	0	0	0	0	0	0	0	0	0	0
	4	**6**	0	0	0	0	0	0	0	1	0	0	0	0	0	0
	Average		**0**	**0**	**0**	**0**	**0**	**0**	**0**	**0.5**	**0**	**0**	**0**	**0**	**0**	**0**
PM Peak	1	**6**	0	0	0	0	0	0	0	0	3	1	0	2	1	0
	2	**6**	0	0	0	0	0	0	0	3	1	0	1	1	0	0
	3	**6**	0	0	0	2	4	0	0	4	1	1	0	1	0	0
	4	**6**	0	0	0	2	2	0	0	3	2	2	0	1	0	1
	Average		**0**	**0**	**0**	**1**	**1.5**	**0**	**0**	**2.5**	**1.75**	**1**	**0.25**	**1.25**	**0.25**	**0.25**

Note: Lane 8 was closed during part of the AM Peak.

Table A-4.3 Average Queue Storage Ratio by Lane and by Period at 75 St. & Newberry Rd.

Time Period		**Lane Number**											
		1	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**
AM Peak	1	0.04	0.8	0.85	0.85	0.56	0.8	0.8	0.7	0.6	0.09	0.16	0.16
	2	0.05	0.63	0.72	0.69	0.47	0.74	0.77	0.6	0.6	0.14	0.17	0.22
	3	0	0.34	0.33	0.28	0.53	0.32	0.38	0.33	0.3	0.06	0.08	0.16
	4	0.05	0.3	0.34	0.3	0.47	0.28	0.29	0.26	0.22	0.06	0.13	0.14
	Average	**0.04**	**0.52**	**0.56**	**0.53**	**0.51**	**0.54**	**0.56**	**0.47**	**0.43**	**0.09**	**0.13**	**0.17**
Off Peak	1	0.16	0.38	0.59	0.58	0.66	0.29	0.8	0.64	0.64	0.37	0.44	0.35
	2	0.04	0.22	0.27	0.32	0.53	0.41	0.67	0.65	0.72	0.35	0.35	0.27
	3	0.09	0.31	0.33	0.46	0.8	0.4	0.75	0.59	0.53	0.35	0.43	0.32
	4	0.1	0.34	0.37	0.48	0.8	0.45	0.7	0.62	0.69	0.32	0.37	0.25
	Average	**0.1**	**0.31**	**0.39**	**0.46**	**0.7**	**0.39**	**0.73**	**0.63**	**0.65**	**0.35**	**0.4**	**0.3**
PM Peak	1	0.12	0.72	0.75	0.78	0.8	0.52	0.58	0.43	0.36	0.24	0.28	0.23
	2	0.09	0.52	0.6	0.89	0.72	0.43	0.62	0.43	0.36	0.4	0.37	0.32
	3	0.1	0.75	0.77	0.95	0.8	0.56	0.67	0.43	0.38	0.57	0.71	0.32
	4	0.08	0.8	0.92	1	0.8	0.48	0.69	0.55	0.51	0.66	0.78	0.26
	Average	**0.1**	**0.7**	**0.76**	**0.91**	**0.78**	**0.5**	**0.64**	**0.46**	**0.4**	**0.47**	**0.53**	**0.28**

Table A-4.4 Number of Cycles with Spillback at 75 St. & Newberry Rd.

Time Period		**#cycles in 15min**	**Lane Number**											
			1	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**
AM Peak	1	**5**	1	1	1	1	0	0	0	1	0	0	0	0
	2	**5**	0	0	0	0	0	0	0	0	0	0	0	0
	3	**8**	0	0	0	0	0	0	0	0	0	0	0	0
	4	**8**	0	0	0	0	0	0	0	0	0	0	0	0
	Average		**0**	**0.25**	**0.25**	**0.25**	**0**	**0**	**0**	**0.25**	**0**	**0**	**0**	**0**
Off Peak	1	**6**	0	0	2	2	0	0	0	0	0	0	0	0
	2	**6**	0	0	0	1	0	0	0	1	0	0	0	0
	3	**6**	0	0	0	0	0	0	0	0	0	0	0	0
	4	**6**	0	0	0	0	0	0	0	0	0	0	0	0
	Average		**0**	**0**	**0.5**	**0.75**	**0**	**0**	**0**	**0.25**	**0**	**0**	**0**	**0**
PM Peak	1	**6**	0	3	3	3	0	0	0	0	0	0	0	0
	2	**5**	0	1	1	3	0	0	0	0	0	0	0	0
	3	**5**	0	2	2	4	0	0	0	0	0	0	0	0
	4	**5**	0	3	4	5	0	0	0	0	0	0	0	0
	Average		**0**	**2.25**	**2.5**	**3.75**	**0**	**0**	**0**	**0**	**0**	**0**	**0**	**0**

After Study (Nov. 19 & Nov. 20, 2014)

Table A-4.5 Average Queue Storage Ratio by Lane by Period at I-75N & Newberry Rd.

Time Period		Lane Number													
		1	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**	**13**	**14**
AM Peak	1	0.90	0.90	0.86	0.23	0.25	0.30	0.38	0.32	0.78	0.78	0.26	0.20	0.08	0.22
	2	0.90	0.90	0.80	0.21	0.18	0.23	0.32	0.52	0.66	0.74	0.26	0.18	0.22	0.06
	3	0.54	0.54	0.30	0.17	0.19	0.13	0.13	0.44	0.24	0.29	0.11	0.03	0.04	0.00
	4	0.28	0.24	0.20	0.11	0.16	0.08	0.15	0.50	0.10	0.03	0.03	0.12	0.00	0.02
	Average	**0.66**	**0.65**	**0.54**	**0.18**	**0.20**	**0.18**	**0.24**	**0.45**	**0.45**	**0.46**	**0.17**	**0.13**	**0.09**	**0.07**
Off Peak	1	0.05	0.10	0.13	0.20	0.24	0.10	0.18	0.45	0.38	0.18	0.23	0.02	0.07	0.03
	2	0.15	0.17	0.22	0.12	0.22	0.07	0.17	0.55	0.35	0.25	0.27	0.03	0.05	0.03
	3	0.40	0.28	0.22	0.15	0.26	0.15	0.23	0.50	0.28	0.22	0.15	0.03	0.00	0.10
	4	0.12	0.15	0.17	0.15	0.24	0.06	0.21	0.78	0.47	0.30	0.20	0.30	0.15	0.13
	Average	**0.18**	**0.18**	**0.18**	**0.16**	**0.24**	**0.10**	**0.20**	**0.57**	**0.37**	**0.24**	**0.21**	**0.10**	**0.07**	**0.08**
PM Peak	1	0.38	0.20	0.26	0.43	0.42	0.08	0.20	0.92	0.90	0.74	0.62	0.88	0.60	0.56
	2	0.30	0.12	0.30	0.40	0.53	0.11	0.15	1.00	0.56	0.66	0.80	0.74	0.96	0.90
	3	0.12	0.12	0.24	0.36	0.40	0.10	0.18	0.88	0.66	0.44	0.38	0.80	0.60	0.44
	4	0.15	0.18	0.23	0.30	0.34	0.11	0.13	0.86	0.30	0.22	0.30	0.52	0.22	0.20
	Average	**0.24**	**0.15**	**0.26**	**0.37**	**0.42**	**0.10**	**0.17**	**0.92**	**0.61**	**0.52**	**0.53**	**0.74**	**0.60**	**0.53**

Table A-4.6 Number of Cycles with Spillback at I-75N & Newberry Rd.

Time Period		#cycles in 15 mins	Lane Number 1	2	3	4	5	6	7	8	9	10	11	12	13	14
AM Peak	1	**5**	0	0	0	0	0	0	0	0	1	1	0	0	0	0
	2	**5**	0	0	0	0	0	0	0	0	0	1	0	0	0	0
	3	**5**	0	0	0	0	0	0	0	0	0	0	0	0	0	0
	4	**5**	0	0	0	0	0	0	0	0	0	0	0	0	0	0
	Average		**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.25**	**0.50**	**0.00**	**0.00**	**0.00**	**0.00**
Off Peak	1	**6**	0	0	0	0	0	0	0	0	0	0	0	0	0	0
	2	**6**	0	0	0	0	0	0	0	1	0	0	0	0	0	0
	3	**6**	1	0	0	0	0	0	0	1	0	0	0	0	0	0
	4	**6**	0	0	0	0	0	0	0	2	0	0	0	1	0	0
	Average		**0.25**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**1.00**	**0.00**	**0.00**	**0.00**	**0.25**	**0.00**	**0.00**
PM Peak	1	**5**	0	0	0	0	0	0	0	2	3	1	1	4	1	2
	2	**5**	0	0	0	0	0	0	0	5	1	1	3	1	3	2
	3	**5**	0	0	0	0	0	0	0	2	0	0	0	1	1	0
	4	**5**	0	0	0	0	0	0	0	2	0	0	0	1	0	0
	Average		**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**2.75**	**1.00**	**0.50**	**1.00**	**1.75**	**1.25**	**1.00**

Table A-4.7 Average Queue Storage Ratio by Lane by Period at 75 St. & Newberry Rd.

Time Period		Lane Number											
		1	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**
AM Peak	1	0.03	0.73	0.73	0.72	0.48	0.80	0.80	0.60	0.50	0.34	0.30	0.16
	2	0.04	0.68	0.71	0.87	0.71	0.60	0.68	0.60	0.50	0.53	0.39	0.20
	3	0.01	0.28	0.35	0.43	0.47	0.51	0.61	0.40	0.40	0.17	0.23	0.07
	4	0.02	0.28	0.28	0.51	0.58	0.30	0.42	0.43	0.30	0.13	0.12	0.02
	Average	**0.03**	**0.49**	**0.52**	**0.63**	**0.56**	**0.55**	**0.63**	**0.51**	**0.42**	**0.29**	**0.26**	**0.11**
Off Peak	1	0.13	0.29	0.33	0.67	0.80	0.46	0.59	0.93	0.96	0.44	0.46	0.57
	2	0.03	0.27	0.28	0.68	0.76	0.44	0.70	1.00	0.97	0.44	0.52	0.52
	3	0.07	0.48	0.46	0.85	0.81	0.45	0.59	1.00	0.99	0.38	0.50	0.30
	4	0.05	0.33	0.28	0.63	0.78	0.31	0.40	0.93	0.93	0.34	0.50	0.33
	Average	**0.07**	**0.34**	**0.34**	**0.71**	**0.79**	**0.41**	**0.57**	**0.97**	**0.97**	**0.40**	**0.50**	**0.43**
PM Peak	1	0.07	0.83	0.83	1.00	0.80	0.39	0.60	0.55	0.51	0.68	0.67	0.42
	2	0.06	0.68	0.70	1.00	0.80	0.59	0.74	0.56	0.52	0.62	0.64	0.20
	3	0.08	0.92	1.00	1.00	0.80	0.79	0.81	0.53	0.50	0.86	0.80	0.44
	4	0.10	0.72	0.88	1.00	0.82	0.15	0.22	0.66	0.70	0.78	0.74	0.44
	Average	**0.08**	**0.79**	**0.85**	**1.00**	**0.80**	**0.48**	**0.59**	**0.57**	**0.56**	**0.73**	**0.71**	**0.38**

Table A-4.8 Number of Cycles with Spillback at 75 St. & Newberry Rd.

Time Period		**#cycles in 15 mins**	**Lane Number**											
			1	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**
AM Peak	1	**5**	0	0	0	0	0	0	0	0	0	0	0	0
	2	**6**	2	2	4	0	0	0	0	0	0	0	0	0
	3	**6**	0	0	0	0	0	0	0	0	0	0	0	0
	4	**6**	0	0	0	1	0	0	0	0	0	0	0	0
	Average		**0.50**	**0.50**	**1.00**	**0.25**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**
Off Peak	1	**6**	0	0	0	2	0	0	0	4	4	0	0	0
	2	**6**	0	0	0	3	0	0	0	6	4	0	0	0
	3	**6**	0	0	0	3	0	0	0	6	4	0	0	0
	4	**6**	0	0	0	0	0	0	0	2	2	0	0	0
	Average		**0.00**	**0.00**	**0.00**	**2.00**	**0.00**	**0.00**	**0.00**	**4.50**	**3.50**	**0.00**	**0.00**	**0.00**
PM Peak	1	**5**	0	4	4	5	0	0	0	0	0	0	0	0
	2	**5**	0	2	2	5	0	0	0	0	0	0	0	0
	3	**5**	0	4	4	4	0	0	0	0	0	0	0	0
	4	**5**	0	2	4	5	0	0	0	0	0	0	0	0
	Average		**0.00**	**3.00**	**3.50**	**4.75**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**

Comparisons of Before and After Queue Storage Ratios

The differences in Queue Storage Ratio between before and after measurements are shown in Table A-4.9 to Table A-4.12. The tables are color-coded as follows: green shows significant improvement, yellow shows modest change, and red shows significant deterioration in queue storage ratios or spillback potential.

Table A-4.9 Difference in Average Queue Storage Ratios at I-75N & Newberry Rd.

Time Period	Lane Number														Average
	1	2	3	4	5	6	7	8	9	10	11	12	13	14	
AM Peak	- 0.21	- 0.22	- 0.25	- 0.10	- 0.17	- 0.10	- 0.07	N/A	0.09	0.02	- 0.23	- 0.09	- 0.14	- 0.11	-0.08
Off Peak	- 0.02	- 0.07	- 0.08	- 0.03	0.03	0.02	0.03	0.11	0.16	0.03	0.01	0.03	- 0.01	0.02	0.02
PM Peak	0.09	0.04	0.10	- 0.24	- 0.21	0.01	0.04	0.24	- 0.04	- 0.05	0.20	0.16	0.20	0.30	0.06

Note: Lane 8 was closed during part of the AM Peak Period in the Before data collection.

Table A-4.10 Difference in Percent of Cycles with Spillback at I-75N & Newberry Rd.

Time Period	Lane Number														Average
	1	2	3	4	5	6	7	8	9	10	11	12	13	14	
AM Peak	- 85%	- 85%	- 65%	-4%	-8%	-4%	-4%	N/A	4%	-1%	-8%	-4%	0%	- 4%	-19%
Off Peak	4%	0%	0%	0%	0%	0%	0%	8%	0%	0%	0%	4%	0%	0%	1%
PM Peak	0%	0%	0%	- 17%	- 25%	0%	0%	4%	- 13%	-8%	13%	8%	17%	13 %	-1%

Note: Lane 8 was closed during part of the AM Peak Period in the Before data collection.

Table A-4.11 Difference in Average Queue Storage Ratio at 75 St. & Newberry Rd.

Time Period	Lane Number												Average
	1	2	3	4	5	6	7	8	9	10	11	12	
AM Peak	-0.01	-0.03	-0.04	0.10	0.06	0.02	0.07	0.03	-0.01	0.20	0.13	-0.06	0.04
Off Peak	-0.03	0.03	-0.05	0.25	0.09	0.03	-0.16	0.34	0.32	0.05	0.10	0.13	0.09
PM Peak	-0.02	0.09	0.09	0.09	0.02	-0.01	-0.05	0.12	0.15	0.27	0.18	0.09	0.09

Table A-4.12 Difference in Percent of Cycles with Spillback at 75 St. & Newberry Rd.

Time Period	Lane Number												Average
	1	2	3	4	5	6	7	8	9	10	11	12	
AM Peak	10%	5%	15%	-2%	0%	0%	0%	-5%	0%	0%	0%	0%	2%
Off Peak	0%	0%	-8%	21%	0%	0%	0%	71%	58%	0%	0%	0%	12%
PM Peak	0%	14%	19%	18%	0%	0%	0%	0%	0%	0%	0%	0%	4%

Discussion

The following were concluded regarding queue storage ratios:

- Queue storage ratios were fairly high during the AM peak in the before data collection for the I-75N intersection. These were substantially improved with the new system.
- Queue storage ratios increased for the 75 St. intersection, with increased probability for spillback.

A.3.5 EQUIVALENT PCE FLOWS

Traffic flows were counted manually and converted to equivalent PCE flows (pce/hour) by considering the percentage of heavy vehicles. It was assumed that the PCE for trucks is 2. Note that the truck percentage in the "before study" was not observed. The following tables provide truck percentages by movement, as well as traffic volumes in units of pce/hr.

Truck Percentage Observations

Table A-5.1 Truck Percentages at I-75N& Newberry Rd.

Period	Interval	SB			WB			NB			EB			Total
		Left	Thru	Right	Left	Thru	Right	Left	Thru	Right	Left	Thru	Right	
AM Peak	1					2.46%	4.55%	5.00%		2.06%	5.00%	0.85%		3.32%
	2					2.68%	1.96%	5.22%		3.55%	0.00%	0.99%		2.40%
	3					3.45%	0.00%	3.85%		2.75%	3.13%	0.92%		2.35%
	4					3.93%	1.43%	7.23%		4.10%	6.45%	2.12%		4.21%
	Average					**3.13%**	**1.98%**	**5.32%**		**3.11%**	**3.64%**	**1.22%**		**3.07%**
Off Peak	1					1.64%	0.00%	5.63%		3.00%	9.09%	3.22%		3.76%
	2					0.29%	0.00%	4.65%		4.76%	3.45%	2.02%		2.53%
	3					0.00%	0.00%	5.94%		0.00%	0.00%	0.97%		1.15%
	4					0.79%	0.00%	2.11%		1.02%	2.56%	1.53%		1.33%
	Average					**0.68%**	**0.00%**	**4.58%**		**2.20%**	**3.78%**	**1.93%**		**2.19%**
PM Peak	1					1.64%	0.00%	2.92%		0.00%	9.09%	1.59%		2.54%

Period	Interval	SB			WB			NB			EB			Total
		Left	Thru	Right	Left	Thru	Right	Left	Thru	Right	Left	Thru	Right	
	2					0.29%	0.00%	1.80%		0.00%	3.45%	0.43%		0.99%
	3					0.00%	0.00%	0.59%		0.00%	0.00%	0.76%		0.22%
	4					0.79%	0.00%	0.67%		1.03%	2.56%	1.06%		1.02%
	Average					**0.68%**	**0.00%**	**1.49%**		**0.26%**	**3.78%**	**0.96%**		**1.19%**

Table A-5.2 Truck Percentages at 75 St. & Newberry Rd.

Period	Interval	SB			WB			NB			EB			Total
		Left	Thru	Right	Left	Thru	Right	Left	Thru	Right	Left	Thru	Right	
AM Peak	1	0.00%	0.00%	0.00%	7.04%	6.11%	0.00%	0.00%	0.00%	1.20%	5.56%	1.52%	2.99%	2.03%
	2	0.00%	20.00%	0.00%	2.43%	5.05%	0.00%	0.00%	0.00%	1.86%	0.00%	1.91%	5.88%	3.09%
	3	0.00%	0.00%	0.00%	1.09%	5.50%	0.00%	6.06%	0.00%	2.30%	5.88%	1.85%	5.13%	2.32%
	4	0.00%	0.00%	0.00%	3.53%	2.30%	16.67%	2.38%	0.00%	2.74%	2.86%	3.14%	2.08%	2.98%
	Average	**0.00%**	**5.00%**	**0.00%**	**3.52%**	**4.74%**	**4.17%**	**2.11%**	**0.00%**	**2.02%**	**3.57%**	**2.11%**	**4.02%**	**2.61%**
Off Peak	1	0.00%	6.25%	0.00%	1.36%	1.77%	0.00%	1.41%	0.00%	1.13%	11.11%	1.53%	3.08%	2.30%
	2	0.00%	0.00%	0.00%	0.00%	1.51%	0.00%	1.43%	0.00%	2.09%	0.00%	2.30%	1.59%	0.74%
	3	0.00%	0.00%	0.00%	4.55%	3.97%	0.00%	1.11%	0.00%	1.15%	0.00%	2.64%	1.35%	1.23%
	4	0.00%	0.00%	0.00%	3.00%	0.97%	0.00%	0.00%	0.00%	0.00%	0.00%	1.95%	3.03%	0.75%
	Average	**0.00%**	**1.56%**	**0.00%**	**2.23%**	**2.05%**	**0.00%**	**0.99%**	**0.00%**	**1.09%**	**2.78%**	**2.11%**	**2.26%**	**1.26%**
PM Peak	1	0.00%	0.00%	25.00%	4.29%	3.99%	0.00%	3.13%	14.29%	0.99%	0.00%	1.90%	0.00%	4.46%
	2	7.14%	0.00%	0.00%	1.23%	3.31%	0.00%	1.54%	0.00%	2.35%	0.00%	0.52%	0.00%	1.34%
	3	22.22%	9.09%	0.00%	2.85%	4.74%	9.09%	0.00%	0.00%	1.41%	0.00%	2.02%	0.00%	4.29%
	4	0.00%	0.00%	0.00%	3.27%	1.64%	0.00%	0.85%	0.00%	1.05%	0.00%	0.46%	0.00%	0.61%
	Average	**7.34%**	**2.27%**	**6.25%**	**2.91%**	**3.42%**	**2.27%**	**1.38%**	**0.00%**	**1.45%**	**0.00%**	**1.23%**	**0.00%**	**2.38%**

Before Study (Sept. 17 & Sept.18, 2014)

Table A-5.3 Traffic Volume (pce/15 min) and Traffic Flow (pce/hour) at I-75N & Newberry Rd.

Time Period		SB			WB			NB			EB		
		Left	**Thru**	**Right**	**Left**	**Thru**	**Right**	**Left**	**Thru**	**Right**	**Left**	**Thru**	**Right**
AM Peak	1					195	N/A	69		97	N/A	520	
	2					340	N/A	141		129	N/A	627	
	3					282	N/A	78		90	N/A	626	
	4					317	N/A	183		238	N/A	487	
	Flow Rate					**1134**	**N/A**	**471**		**554**	**N/A**	**2260**	
Off Peak	1					379	62	78		84	27	448	
	2					386	105	89		68	34	420	
	3					349	78	74		77	33	492	
	4					409	88	70		90	30	398	
	Flow Rate					**1524**	**333**	**311**		**319**	**125**	**1758**	
PM Peak	1					504	120	117		77	56	407	
	2					483	132	133		85	43	417	
	3					426	137	110		107	45	439	
	4					499	96	125		87	35	360	
	Flow Rate					**1912**	**485**	**485**		**356**	**178**	**1623**	

Note: No traffic was recorded for the WB right and EB left during the AM peak due to an incident.

Table A-5.4 Traffic Volume (pce/15 min) and Traffic Flow (pce/hour) at 75 St. & Newberry Rd.

Time Period		SB			WB			NB			EB		
		Left	Thru	Right	Left	Thru	Right	Left	Thru	Right	Left	Thru	Right
AM Peak	1	6	5	1	176	157	13	44	7	282	3	377	26
	2	5	8	0	177	164	17	23	21	288	9	347	31
	3	6	3	2	135	196	8	51	8	210	9	306	41
	4	5	3	1	156	175	14	67	6	190	5	218	49
	Flow Rate	**22**	**19**	**4**	**645**	**692**	**52**	**185**	**42**	**970**	**26**	**1248**	**147**
Off Peak	1	10	3	3	136	245	8	65	0	174	5	216	56
	2	8	7	3	155	274	12	50	5	177	6	224	74
	3	9	5	3	166	289	12	48	4	181	7	231	61
	4	10	7	0	209	289	14	75	3	187	9	226	88
	Flow Rate	**37**	**22**	**9**	**666**	**1097**	**46**	**238**	**12**	**719**	**27**	**897**	**279**
PM Peak	1	7	6	8	215	370	10	56	0	115	5	234	59
	2	4	2	1	247	420	13	55	3	189	11	251	68
	3	7	11	1	211	430	2	88	1	231	11	238	99
	4	7	6	1	222	427	2	84	8	155	10	189	56
	Flow Rate	**26**	**25**	**11**	**895**	**1648**	**27**	**283**	**12**	**690**	**37**	**912**	**282**

After Study (Nov. 19 & Nov. 20, 2014)

Table A-5.5 Traffic Volume (pce/15 min) and Traffic Flow (pce/hour) at I-75N& Newberry Rd.

Time Period		Southbound			Westbound			Northbound			Eastbound		
		Left	Thru	Right	Left	Thru	Right	Left	Thru	Right	Left	Thru	Right
AM Peak	1					203	44	120		97	40	819	
	2					261	51	115		141	43	806	
	3					261	53	78		109	32	543	
	4					331	70	83		122	31	613	
	Flow Rate					**1056**	**218**	**396**		**469**	**146**	**2781**	
Off Peak	1					427	76	71		100	33	497	
	2					339	79	86		105	29	495	
	3					406	86	101		85	44	414	
	4					507	121	95		98	39	392	
	Flow Rate					**1679**	**362**	**353**		**388**	**145**	**1798**	
PM Peak	1					509	143	137		76	51	504	
	2					503	161	167		90	53	468	
	3					539	163	170		63	62	528	
	4					441	125	149		97	53	379	
	Flow Rate					**1993**	**592**	**623**		**326**	**219**	**1879**	

Table A-5.6 Traffic Volume (pce/15 min) and Traffic Flow (pce/hour) at 75 St. & Newberry Rd.

Time Period		**Southbound**			**Westbound**			**Northbound**			**Eastbound**		
		Left	**Thru**	**Right**	**Left**	**Thru**	**Right**	**Left**	**Thru**	**Right**	**Left**	**Thru**	**Right**
AM Peak	1	7	3	0	142	131	4	36	1	251	18	396	67
	2	3	5	1	206	198	5	36	6	323	10	366	34
	3	4	4	0	184	200	4	66	5	261	17	324	39
	4	5	4	1	170	217	6	42	4	219	35	318	48
	Flow Rate	**19**	**16**	**2**	**702**	**746**	**19**	**180**	**16**	**1054**	**80**	**1404**	**188**
Off Peak	1	10	16	4	147	282	9	71	1	177	9	261	65
	2	6	6	5	137	331	11	70	0	191	3	261	63
	3	6	13	8	154	378	12	90	0	174	7	265	74
	4	12	8	5	100	414	17	61	2	208	3	256	66
	Flow Rate	**34**	**43**	**22**	**538**	**1405**	**49**	**292**	**3**	**750**	**22**	**1043**	**268**
PM Peak	1	8	6	4	219	409	6	64	7	203	3	210	65
	2	14	10	1	261	494	14	65	0	170	2	194	59
	3	9	11	2	239	474	11	80	2	213	9	247	43
	4	6	12	2	233	448	6	117	0	191	9	219	73
	Flow Rate	**34**	**43**	**22**	**538**	**1405**	**49**	**292**	**3**	**750**	**22**	**1043**	**268**

Comparisons of Before and After Flows

The differences in traffic flow between the before and after measurements are shown in Table A-5.7 and Table A-5.8. The tables are color-coded as follows: green shows significant decrease, yellow shows modest change, and red shows significant increase in flow.

Table A-5.7 Difference in Traffic Flow Rate (pce/hr) at I-75N & Newberry Rd.

Period	Southbound			Westbound			Northbound			Eastbound		
	Left	Thru	Right	Left	Thru	Right	Left	Thru	Right	Left	Thru	Right
AM Peak					-78	N/A	-75		-85	N/A	521	
Off Peak					155	29	42		69	20	40	
PM Peak					82	107	138		-30	41	256	

Note: No traffic of WB right and EB left during AM peak of Before Study due to accidents.

Table A-5.8 Difference in Traffic Flow Rate (pce/hr) at 75 St. & Newberry Rd.

Period	Southbound			Westbound			Northbound			Eastbound		
	Left	Thru	Right	Left	Thru	Right	Left	Thru	Right	Left	Thru	Right
AM Peak	-3	-3	-2	57	54	-33	-5	-26	84	54	156	41
Off Peak	-3	21	13	-128	308	3	54	-9	31	-5	146	-11
PM Peak	11	14	-2	57	178	10	43	-3	87	-14	-42	-42

Discussion

The following can be concluded regarding traffic volumes at this corridor:

- The traffic volume at both critical intersections (I-75N & Newberry Rd. and 75 St. & Newberry Rd.) increased during all three time periods by an average of 9.9% and 8.59% respectively.
- The increased travel times and queues may be at least partially due to the increase in flows between the before and after data collection periods.

A.3.6 CONSIDERATION OF TRAFFIC FLOWS JOINTLY WITH QUEUE LENGTH

The differences in “Traffic Flow” and “Queue Length” between before and after measurements are shown in Table A-6.1 and Table A-6.3. The tables are color-coded as follows: green indicates that “Queue” decreases and “Traffic Flow” increases, Red indicates “Queue” increases and “Traffic Flow” decreases, No Color indicates “Traffic Flow” and “Queue” increase or decrease at the same time. Generally, green indicates improvement despite an increase in flow.

Table A-6.1 Differences in Traffic Flow (TF) and Queue Length (Q) at I-75N& Newberry Rd.

Time Period	Westbound (Main)					Eastbound (Main)					Northbound (Side)					Southbound (Side)		
	Left	Thru		Right		Left		Thru		Right	Left		Thru	Right		Left	Thru	Right
		TF	Q	TF	Q	TF	Q	TF	Q		TF	Q		TF	Q			
AM Peak		-78	-1.53	N/A	N/A	N/A	N/A	521	-2.25		-75	-3.02		-85	-1.72			
Off Peak		155	0.94	29	0.2	20	1.11	40	-0.58		42	0.02		69	0.67			
PM Peak		82	1.21	107	0.36	41	2.38	256	0.78		138	-6.64		-30	0.52			

Note: No traffic of WB right and EB left during AM Peak of Before Study due to accidents.

Table A-6.2 Differences (%) in Traffic Flow (TF) and Queue Length (Q) at I-75N& Newberry Rd.

Time Period	**Westbound (Main)**					**Eastbound (Main)**					**Northbound (Side)**					**Southbound (Side)**		
	Left	**Thru**		**Right**		**Left**		**Thru**		**Right**	**Left**		**Thru**	**Right**		**Left**	**Thru**	**Right**
		TF	**Q**	**TF**	**Q**	**TF**	**Q**	**TF**	**Q**		**TF**	**Q**		**TF**	**Q**			
AM Peak		-6.90%	-23.10%	N/A	N/A	N/A	N/A	23.10%	-26.80%		-15.90%	-38.30%		-15.30%	-23.60%			
Off Peak		10.20%	28.30%	8.70%	28.30%	16.10%	25.00%	2.30%	-24.70%		13.50%	0.40%		21.60%	21.20%			
PM Peak		4.30%	17.70%	22.10%	17.70%	22.90%	37.70%	15.80%	55.30%		28.50%	-39.00%		-8.40%	18.20%			

Note: No traffic of WB right and EB left during AM Peak of Before Study due to accidents.

Table A-6.3 Differences in Traffic Flow and Queue Length at 75 St. & Newberry Rd.

Time Period	Westbound (Main)						Eastbound (Main)						Northbound (Side)						Southbound (Side)					
	Left		Thru		Right		Left		Thru		Right		Left		Thru		Right		Left		Thru		Right	
	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q
AM Peak	57	1.02	54	5.61	-33	0.14	54	-0.31	156	1.42	41	0.19	-5	0.83	-26	0.07	84	0.36	-3	-0.02	-3	-0.28	-2	-0.04
Off Peak	-128	4.98	308	2.62	3	0.09	-5	-0.58	146	1.97	-11	0.51	54	5.73	-9	0.06	31	0.62	-3	0.02	21	1.04	13	0.27
PM Peak	57	3.1	178	7.86	10	0.16	-14	-0.39	-42	-2.56	-42	-0.71	43	3.8	-3	0.1	87	0.99	11	0.02	14	0.7	-2	0.19

Table A-6.4 Differences (%) in Traffic Flow and Queue Length at 75 St. & Newberry Rd.

Time Period	Westbound (Main)						Eastbound (Main)						Northbound (Side)						Southbound (Side)					
	Left		Thru		Right		Left		Thru		Right		Left		Thru		Right		Left		Thru		Right	
	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q
AM Peak	9%	11%	8%	142%	-63%	142%	208%	-3%	13%	8%	28%	8%	-3%	14%	-62%	14%	9%	5%	-14%	-22%	-18%	-22%	-50%	23%
Off Peak	-19%	34%	28%	20%	7%	20%	-19%	-8%	16%	15%	-4%	15%	23%	102%	-75%	102%	4%	11%	-8%	45%	94%	45%	144%	21%
PM Peak	6%	31%	11%	44%	36%	44%	-38%	-3%	-5%	-10%	-15%	-10%	15%	50%	-25%	50%	13%	16%	44%	32%	57%	32%	-14%	35%

Discussion

The following can be concluded from the comparison of traffic flows jointly with queue length:

- There does not appear to be a high correlation between increasing traffic and increasing queue at these two critical intersections.
- It is shown from the colored cells in the tables that generally speaking, traffic conditions at the 75 St. & Newberry Rd. intersection have deteriorated during several time periods after the Syncrhro Green installation.

Conditions at the I-75 N intersection have improved despite an increase in flow.

BEFORE AND AFTER-IMPLEMENTATION STUDIES OF

ADVANCED SIGNAL CONTROL TECHNOLOGIES IN FLORIDA

The main objective of this project is to evaluate the implementation of proposed adaptive signal control technology (ASCT) traffic operations at several arterial corridors in Florida, before and after the installation of the ASCT, document the advantages and the disadvantages of different approaches and implementations, and provide recommendations for statewide implementation of ASCT.

A.4 QUESTIONNAIRE

SECTION 1: RESPONDENT'S INFORMATION

Name **Max Elliott**

Organization City of Gainesville Public Works Department Traffic Management

Position ITS Operations Engineer

Address 405 NW 39th Ave

Phone 352-393-7960

Fax

E-mail ElliottSM@CityofGainesville.org

SECTION 2: PREVIOUS TRAFFIC CONTROL TECHNOLOGY

1. Please specify the type of traffic control used before Adaptive Signal Control Technology (ASCT) was installed for your site. (Please check all that apply.)

a. Fixed time coordinated control **b. Actuated coordinated control**

c. Fixed-time isolated control d. Actuated isolated control

e. Other, please specify: ____

2. What type of traffic controller was used before ASCT was implemented?

a. NEMA TS-1 b. NEMA TS-2

c. 170 d. 2070

e. Other, please specify: ____

How many detectors were used before ASCT was implemented for a typical intersection? Please also specify the location where the detectors were typically placed for each specific detector type (video detection, inductive loops, radars, etc.).

One Detector per lane, 30' modified type C detectors, at the stopbar.

3. What was the frequency of retiming the signal timing plan for the corridor before ASCT was implemented?

Last full retiming was over 3 years ago. Timings have been continously modified.

4. How often were maintenance and updates performed to your previous control system in terms of the following components?

a. Detection: **Within 60 days of failure**
b. Control Hardware: **Controllers swapped out with firmware updates.**
c. Software: **Updated for major Trafficware firmware changes.**

5. What was the annual maintenance cost in terms of hardware, software and personnel of the previous control system for the whole corridor where the ASCT is now deployed?
Annual preventative maintenance, updates and trouble calls run approximately $3,000 per intersection.

6. What was the life expectancy of your traffic control before the deployment of ASCT? (Please refer to the hardware and software.)

The controllers were scheduled to be replaced.

7. How many crashes of each category were reported during the past 4 years before the ASCT implementation? (2011-2014)

	K (Killed)	A (Disabling Injury)	B (Evident Injury)	C (Possible Injury)	O (No Apparent Injury)
2011					
2012					
2013					
2014					

SECTION 3: ADAPTIVE TRAFFIC CONTROL SYSTEM (ASCT)

9. Which Adaptive Traffic Control System (ASCT) does your agency deploy?

   a. InSync; Version:____________

   b. **Synchro Green**; Version:________________

   c. Other; please specify: ______________________

10. Did you consider any other ASCT before you selected this one for installation? Why did you reject the other(s) in favor of this one?

It was chosen to compare to the InSync system that will be going along NW 8th Ave

11. What were your major criteria for choosing the ASCT for the selected corridor?

Extended cycle lengths.

12. What is the life expectancy of your ASCT system? (Consider both hardware and software.)

A few months for the software. Some signals will be rebuilt soon.

13. What type of traffic controller is currently in use after the ASCT implementation?

   a. Same as before the ASCT implementation
   b. **New, please specify type: Trafficware Nema 980/ATC**
   c. Other, specify:

14. How many and what type of detectors are used after the ASCT implementation on an intersection level? Please note any updates that the previous detection system needed so as to work with your ASCT.

No changes yet. Video detection coming with resurfacing.

15. Was there a need to update your previous software operating system at your Transportation Management Center in order to get your current ASCT software working?

a. Yes b. No

16. How often do you plan to perform physical maintenance or updates on your new ASCT in terms of the following components?

   a. Detection: **Annual Preventative Maintenance**
   b. ASCT Hardware: **Annual Preventative Maintenance**
   c. Software: **As updates are made**

17. Was there a need for training your personnel in order to operate the new ASCT?

a. **Yes, Num. of employees trained <u>8</u>. Hours of training per employee <u>4</u>.** b. No

18. Was there any change on the number of staff required to operate/maintain the effective signal ASCT operations? (Comparison of staff needed before and after the deployment.)

No

19. Where did expertise come from for your personnel's training, the ASCT installation, operation and the system's maintenance? Please check all that apply. **From marilo**

	In-House	Vendor	Contractor
Employee Training		**Yes**	
ASCT Installation	**Yes**		
ASCT Operation	**Yes**		
ASCT Maintenance	**Yes**		

20. How many crashes of each category were reported after ASCT was implemented?

K (Killed)	A (Disabling Injury)	B (Evident Injury)	C (Possible Injury)	O (No Apparent Injury)

SECTION 4: COST COMPONENTS

21. What is the overall capital cost of purchasing the ASCT (in terms of the software and licenses of the ASCT) for the corridor? If the purchase includes any service of implementation, personnel training or maintenance, please also specify here.

Purchasing $254,500 in total; $11,500 SynchroGreen per Arterial (installation, initial settings, implementation), $14750* intersection license (processor firmware and support), $75,000 central server license

22. What is the implementation cost (considering the installation on site, any updates in software and hardware, design needs and contract hours) for the ASCT corridor? Please specify if this cost is partially or totally included in previous cost items.

included in the per Arterial cost noted above

23. What was total cost for training your personnel to operate and manage the new ASCT? Please specify if this cost is partially or totally included in previous cost items.

$2,500 for 8-hour on site training (included in total cost -in $254,500-)

24. What is the expected maintenance cost (in terms of hardware, software and personnel) for the ASCT corridor on a yearly basis? Please specify if this cost is partially or totally included in previous cost items.

Approximately $3,000 per intersection (FDOT average spending per intersection).

25. Can you specify the costs for the following ASCT components?

a. Firmware $____________ b. Software $_____________

c. Equipment d. Maintenance of Traffic Cost $ **0.00**

e. Design Needs $__________ f. Contract Help/Agency Hours Cost $ **0.00**

SECTION 5: INSTITUTIONAL ISSUES

26. Were there any institutional issues that you had to overcome while implementing and operating the ASCT project? Please categorize and explain those issues below.

a. Organization and Management Institutional Issues:

Management of time to put toward correcting issues.

b. Regulatory and Legal Institutional Issues:

c. Human and Facility Resources Institutional Issues:

d. Financial Institutional Issues:

27. Which component (e.g. detection, communication, hardware, software, licensing or other) of your ASCT deployment is the most challenging to maintain and why?

Communication

28. What happens to your ASCT operations when some of your detectors fail on the corridor level?

a. ASCT triggers an alarm and notifies operators

b. ASCT switches to an “off-line” mode by implementing Time-Of-Day plans

c. Combination of the above. d. **Other (please describe): Nothing**

29. How is your ASCT performance evaluated?

a. **In-house:**

b. By an independent evaluator, please describe: ______________________________

c. Not applicable—there is no evaluation

d. Combination:

30. If the corridor on which the ASCT operates experiences over saturation, how would you rate the operation of the ASCT in response to these traffic conditions?

a. ASCT prevents or eliminates oversaturation

b. ASCT eliminates or reduces the extent of the periods of oversaturation

c. **ASCT adversely affects the traffic conditions during periods of oversaturation**

d. Other: Doesn’t prevent or eliminate but helps to manage oversaturation

e. Oversaturation is very rare on the corridors operated by our ASCT

31. Based on your opinion and the up-to-date operation, when are ASCT operations proven to be the most effective?

a. Peak periods

b. Off-peak periods

c. Shoulders of peak periods

d. Other, please specify: ______________

32. Has the level of performance of ASCT been sustained since its installation?

a. Yes **b. No; why not? Not originally programmed for demands.**

33. What was the public reaction to the ASCT operation? Have you received any complaints about long delays and queues?

Numerous complaints

34. How satisfied are you, in general, with your ASCT deployment?

a. Very satisfied b. Somewhat satisfied

c. **Neutral** d. Somewhat dissatisfied

e. Not satisfied at all

35. Are there any other costs or benefits related to ASCT deployment that you would like to report?

a. Benefits:

b. Costs: **<u>Extensive time programming.</u>**

36. Would you consider expanding the ASCT program to any other corridors? Why or why not? If yes, would you use the same technology/firmware or have any suggestions for other systems? If so, why?

Yes. Along a less congested corrdior with advanced detection.

SECTION 6: SAFETY ISSUES

37. Do you have any anecdotal / qualitative data on safety benefits / disbenefits of the signal improvement along this corridor?

____

____

38. Has there been other changes along this corridor over the analysis period that could affect the safety of this corridor either positively or negatively?

____

____

39. Are you aware of any changes to the crash data reporting procedures over the analysis period? If yes, what are the changes?

____

____

____

____

40. What are your overall reactions to the trends presented in the report?

____

____

41. Is there another comparable corridor in which signal improvements have not been made to which the results of this corridor can be compared to?

____

____

42. Other thoughts / comments for us?

____

____

Thank you for your help in completing this survey. Your responses will help us with evaluating the deployment and benefits of your current ASCT.

If you have any questions regarding the survey, please contact Ria Kontou, email: ekontou@ufl.edu or Liteng Zha, email: litengzha@ufl.edu who work under the direct supervision of the faculty advisor Dr. Yafeng Yin.

A.5 BENEFIT COST ANALYSIS

Note: No Benefit Cost Analysis for this site

APPENDIX B: Summary of US 17/92, Volusia County

B.1 EXECUTIVE SUMMARY

The objective of this research is to evaluate the implementation of proposed Adaptive Signal Control Technologies (ASCT) traffic operations at several arterial corridors in Florida, before and after the installation of specific ASCT, document the advantages and disadvantages of different approaches and implementations, and provide recommendations for state-wide implementation of ASCT. This appendix summarizes the before and after field data along the US 17/92, Deland corridor, from W Beresford Ave to Firehouse Rd. The InSync system was installed in October and November of 2014 on all 5 intersections in the corridor.

Floating car runs were conducted to obtain travel times through the corridor, and traffic counts and queue counts were collected at two critical intersections (Taylor Rd and Orange Camp Rd) over three time periods (AM Peak, Off Peak and PM Peak).

Both the before and after studies were conducted over two days. Five performance measures were evaluated: Link/Route Travel Time, Delay at Intersections, Queue Length (critical intersections), Queue to Lane Storage Ratio (critical intersections), and Equivalent PCE Flows (critical intersections. For each performance measure, a comparison between the before and after data is conducted and presented in this report.

The following were observed:

- Oversaturated conditions were not observed in the before or after study.
- The travel time along US 17/92 NB decreased during all study periods by an average of 0.66 min (10.6%). The link travel time along US 17/92 SB decreased during all periods by an average of 1.67 min (25%).
- The intersection through movement delay of US 17/92 NB decreased at 4 out of 5 intersections by an average of 11.8 sec/intersection (49.6%). The intersection through movement delay of US 17/92 SB decreased for all 5 intersections by an average of 16.9 sec/intersection (49.5%).
- In general, the new signal control strategy does not greatly change the overall queue length at the two critical intersections studied. The average queue length for the US 17/92 & Taylor Rd. intersection decreased by an average of 0.41 vehicles, while the average queue length for the US 17/92 & Orange Camp Rd. intersection increased during all three time periods by an average of 0.09 vehicles.
- The traffic volume at both intersections (Taylor and Orange Camp) increased during all three time periods by an average of 229 pcu/h/ln (6.2%), except for the off peak period which had a decrease by an average of 54 pcu/h/ln (1.5%).

Part of the decreases in delays and travel times may be due to the change in coordination from two coordination groups to a single coordination group, but approaches that were coordinated in the earlier system still showed lower delays. In general, the In-sync signal system has shown clear operational improvement. A detailed interview with District 5 indicates that the system was performing up to expectations and deployment and operation of the system was well-supported by the supplier.

B.2 CORRIDOR INFORMATION

Figure B-1 provides a schematic of the US 17/92 Deland corridor. Table B-2 lists the intersections along the corridor. During the before study, there are two separated coordination groups in the studied corridor, first coordination group includes intersection W Beresford Ave, W New Hampshire Ave and Taylor Rd, the second coordination group includes intersection Orange Camp Rd and Firehouse Rd while all intersections are coordinated in the after study. Since through trucks are not permitted from intersection Taylor Rd northwards, so trucks have to make a detour, thus there are obviously large northbound left turn volumes on intersection Taylor Rd & US 17/92. For the studied corridor, oversaturation condition is very rare.

Two intersections (Taylor Rd and Beresford Ave) were selected as the critical intersections along the corridor and detailed turning movement and queue counts were collected at these. The data collection time period for US 17/92 Corridor is identified as Table B-1.

Table B-1 Field Data Collection Time Period (US 17/92 Corridor, Deland)

	AM Peak	**Off Peak**	**PM Peak**
Time Period	7am-9am	11am-1pm	4pm-6pm

Figure B-2 provides the lane configuration of these two critical intersections. The red circles are the data collection spot for turning movement and queue counts for the intersections.

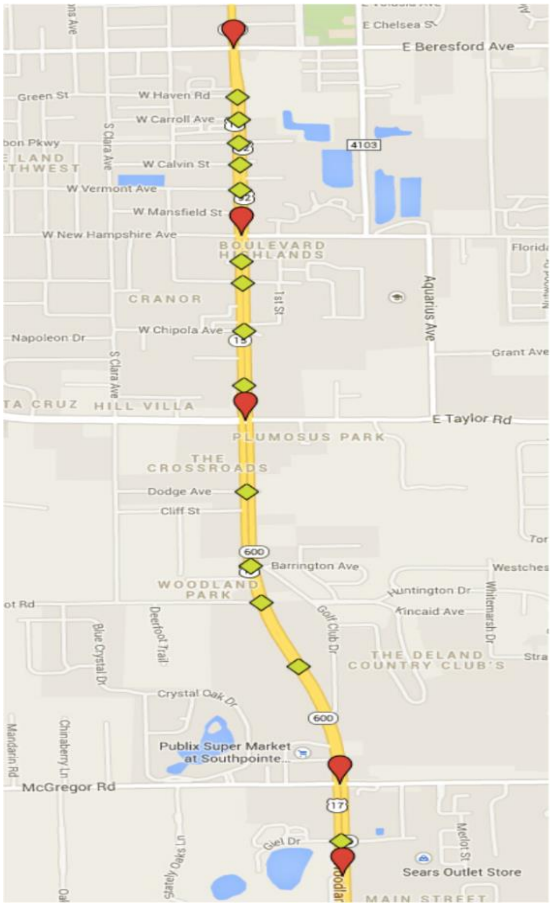
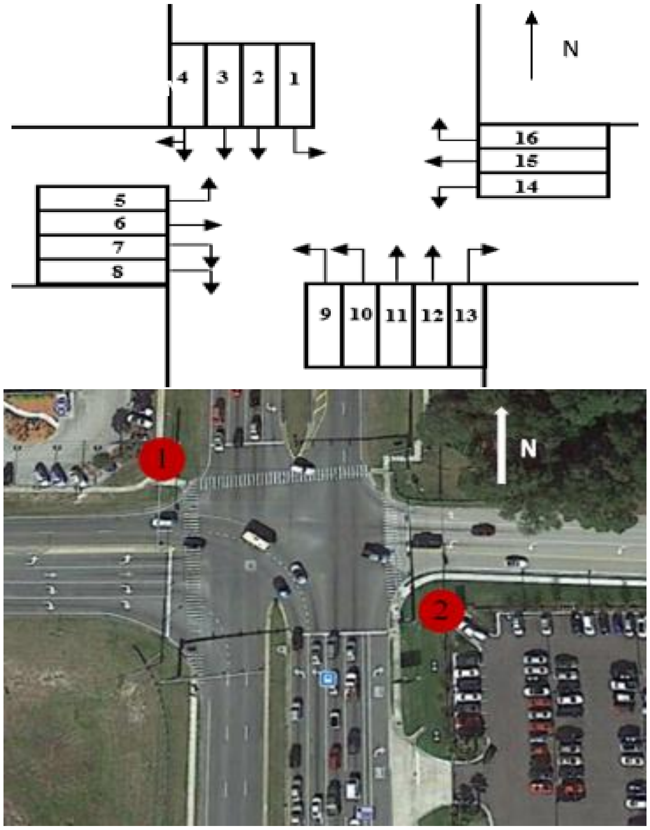


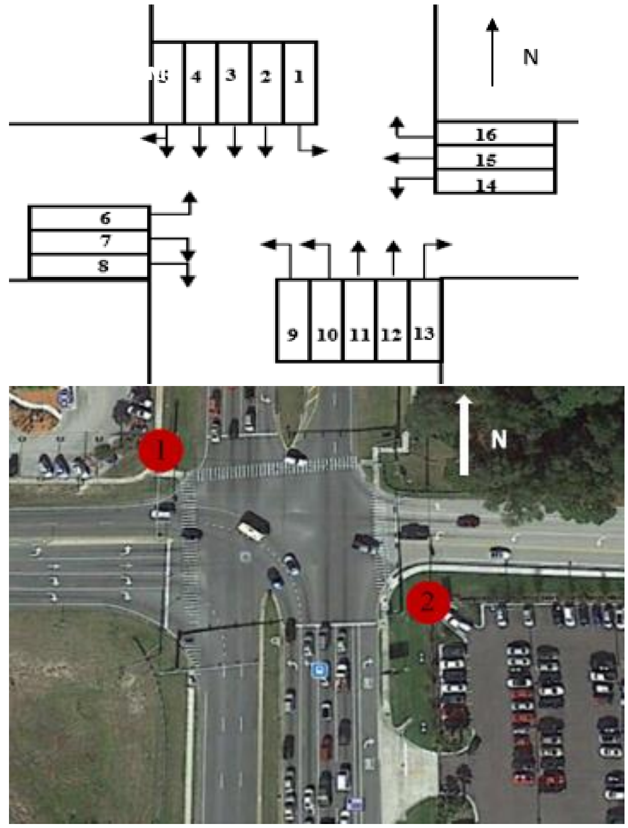
Figure B-1 Schematic of the US 17/92, Deland Corridor

Table B-2 Intersections along the US 17/92, Deland Corridor

EB	**Signalized Intersection**
1	W Beresford Ave & US 17/92
2	W New Hampshire Ave & US 17/92
3	**Taylor Rd & US 17/92**
4	**Orange Camp Rd & US 17/92**
5	Firehouse Rd & US 17/92



(a) Taylor Rd & US 17/92



(b) Orange Camp Rd & US 17/92

Figure B-2 Schematic and Overview of Critical Intersections

B.3 PERFORMANCE MEASURES

Five performance measures were evaluated: Link/Route Travel Time, Delay at Intersections, Queue Length (critical intersections), Queue-to-Lane Storage Ratio (critical intersections), and PCE Flows (critical intersections). For each performance measure, a comparison between the before and after data is conducted and the results of the differences ("after data" – "before data") are presented.

B.3.1 ROUTE TRAVEL TIME

The average travel time (min) along the route was measured using a floating car. We were able to cover 18 before and 18 after runs during the AM peak, 17 before and 18 after runs during the OFF peak and 16 before and 17 after runs during the PM peak. Table B-1.1 and Table B-1.2 provide the route travel time for before and after study, since the distance between Taylor Rd and Orange Camp Rd is over 1 mile, we reported the travel time data separately along the corridor. NB1 means the travel time from intersection Firehouse Rd to Orange Camp Rd, NB2 is from Taylor Rd to W Beresford Ave, SB1 is from W Beresford Ave to Taylor Rd, and SB2 is from Orange Camp Rd to Firehouse Rd.

Before Study (Oct. 1 & 2, 2014)

Table B-1.1 Route Travel Time (min)

Route TT (min)	AM Peak	Off Peak	PM Peak	Average
NB1	2.04	2.84	2.79	2.54
NB2	3.75	3.31	4.01	3.7
SB1	2.49	2.62	3.66	2.9
SB2	3.36	4.04	4.01	3.79
NB	5.79	6.15	6.8	6.24
SB	5.86	6.66	7.67	6.69

After Study (Dec. 16 & 17, 2014)

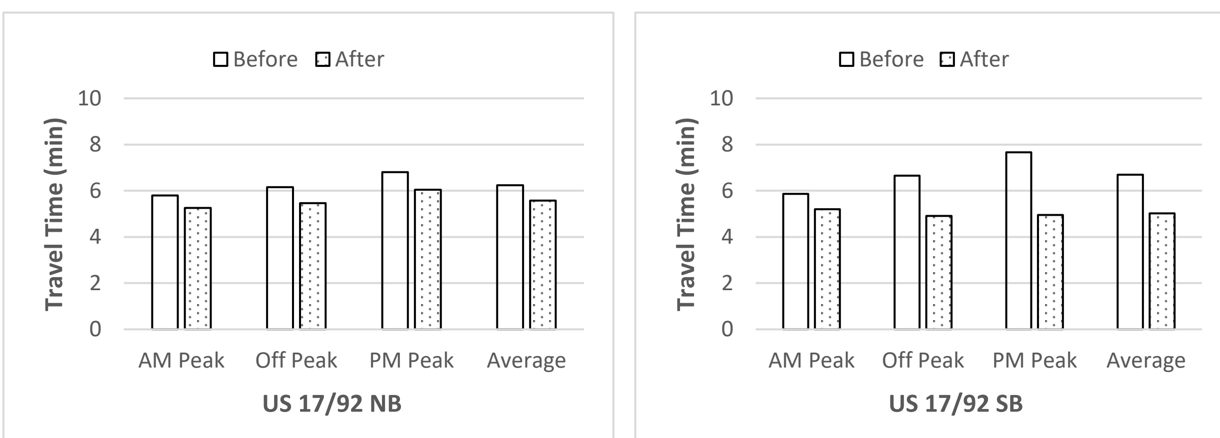
Table B-1.2 Route Travel Time (min)

Route TT (min)	AM Peak	Off Peak	PM Peak	Average
NB1	2.30	2.55	3.02	2.62
NB2	2.95	2.91	3.04	2.96
SB1	1.94	1.98	2.06	1.99
SB2	3.25	2.93	2.89	3.03
NB Total	5.25	5.46	6.05	5.58
SB Total	5.20	4.91	4.95	5.02

Comparisons of Before and After Travel Times

Table B-1.3 Change in Percentage of Route Travel Time (After – Before)

Route TT (min)	AM Peak	Off Peak	PM Peak	Average
NB	-0.54 (-9.3%)	-0.69 (-11.2%)	-0.75 (-11.0%)	-0.66 (-10.6%)
SB	-0.66 (-11.3%)	-1.75 (-26.3%)	-2.72 (-35.5%)	-1.67 (-25.0%)



(a) US 17/92 NB

(b) US 17/92 SB

Figure B-1.1 Travel Times Along US 17/92, Deland

Discussion

The following can be concluded from the comparison of travel times:

- The travel time along US 17/92 decreased during all three periods by an average of 0.66 min (10.6%) in the NB and 1.66 min (25%) in the SB.
- In the before data, travel time in the SB was higher than in the NB during all three study periods, while in the after study, the NB has a higher travel time. Traffic patterns changed slightly between the before and after studies, which would account for some of the larger decreases in southbound travel time. Detailed flow rates are found in Section 5.

B.3.2 DELAY

Delay (sec) at each intersection along the corridor was also obtained using the floating car measurements.

Before Study (Oct. 1 & 2, 2014)

Table B-2.1 Delay (sec) for each Intersection Through Movement Along the NB Direction

Delay by Intersections (sec)	AM Peak	Off Peak	PM Peak
Beresford Ave	44.92	27.45	24.5
Firehouse	6.39	17.05	15.88
New Hampshire	9.43	10.14	48.74
Orange Camp	18.83	50.24	49.45
Taylor	28.6	20.88	27.87

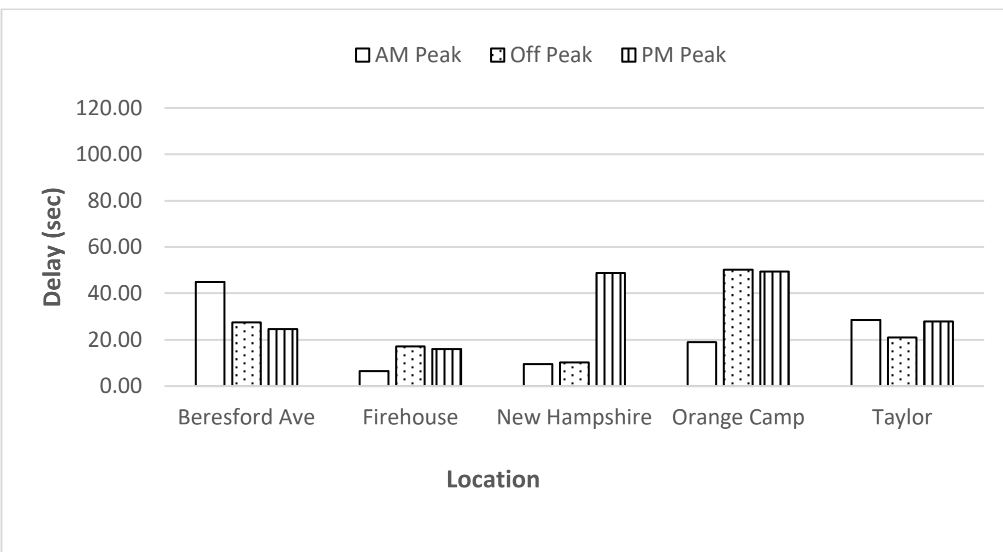


Figure B-2.1 Delay (sec) for each Intersection Through Movement Along the NB Direction

Table B-2.2 Delay (sec) for each Intersection Through Movement Along the SB Direction

Delay by Intersections (sec)	AM Peak	Off Peak	PM Peak
Beresford Ave	14.13	17.59	19.29
Firehouse	23.25	10.43	13.9
New Hampshire	7.61	25.57	14.68
Orange Camp	30.68	49.77	108.88
Taylor	33.25	42.46	57.18

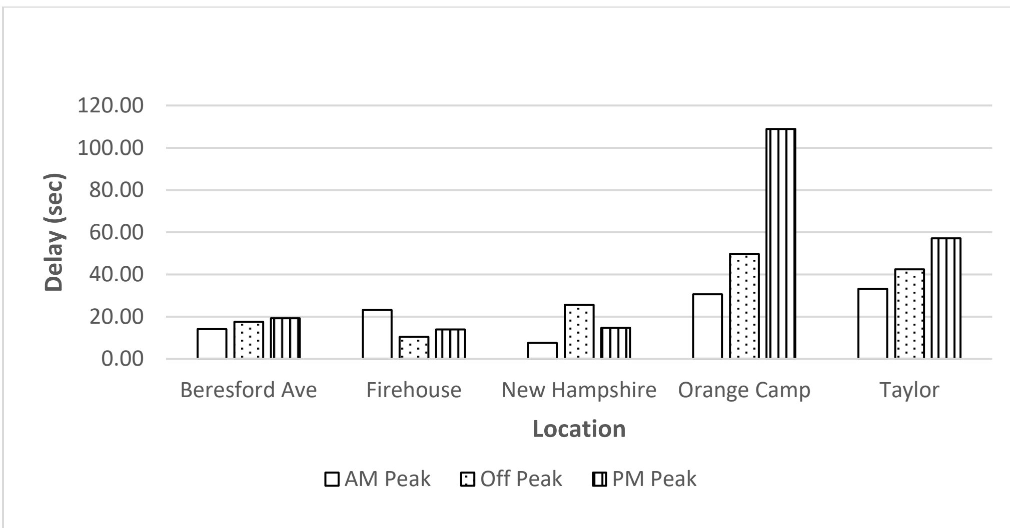


Figure B-2.2 Delay (sec) for each Intersection Through Movement Along the SB Direction

After Study (Dec. 16 & 17, 2014)

Table B-2.3 Delay (sec) for each Intersection Through Movement Along the NB Direction

Delay at Intersections (sec)	AM Peak	Off Peak	PM Peak
Beresford Ave	18.30	16.61	22.69
Firehouse	7.17	4.06	14.16
New Hampshire	10.13	3.39	3.02
Orange Camp	31.33	49.63	67.33
Taylor	11.56	16.91	12.71

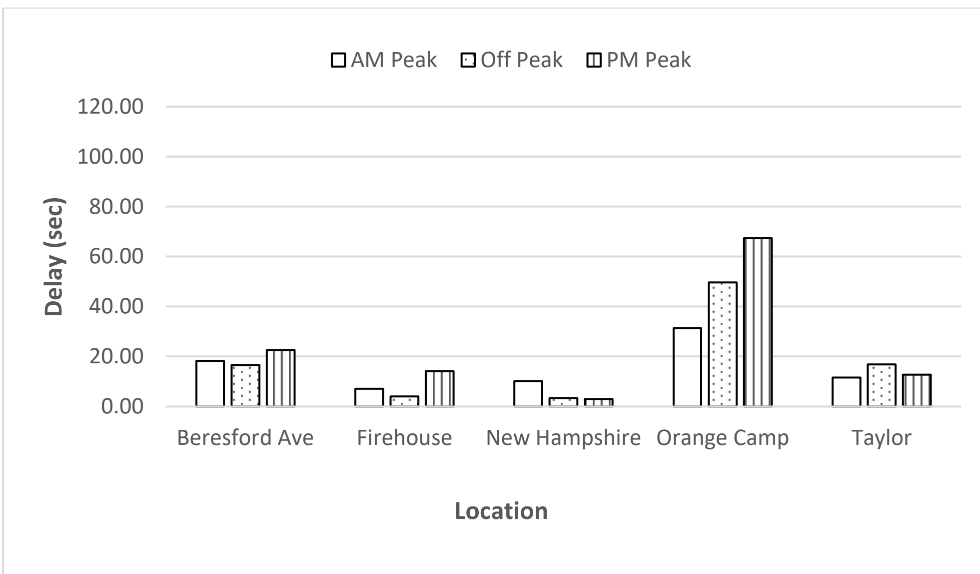


Figure B-2.3 Delay (sec) for each Intersection Through Movement Along the NB Direction

Table B-2.4 Delay (sec) for each Intersection Through Movement Along the SB Direction

Delay at Intersections (sec)	AM Peak	Off Peak	PM Peak
Beresford Ave	12.74	19.20	11.21
Firehouse	4.53	3.34	7.81
New Hampshire	11.24	6.47	5.62
Orange Camp	16.63	20.81	22.71
Taylor	25.30	26.20	20.92

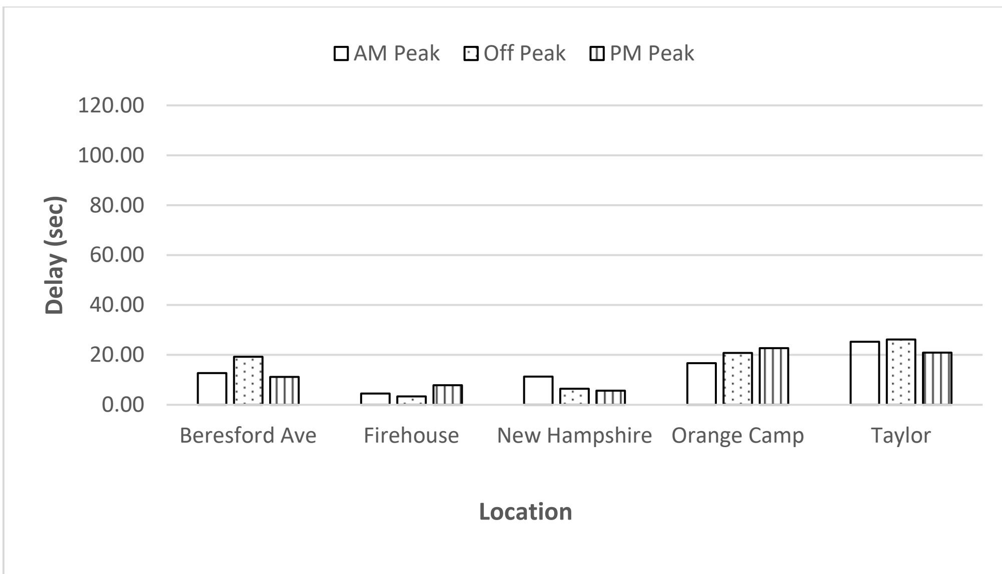


Figure B-2.4 Delay (sec) for each Intersection Through Movement Along the SB Direction

Comparisons of Before and After Intersection Delay Times

The differences in delay between before and after study are shown in Table B-2.5 and Table B-2.6. The tables are color-coded as follows: green shows improvement, yellow shows modest change, and red shows deterioration in delay.

Table B-2.5 Difference in Delay (sec) for each Intersection Through Movement Along the NB Direction

Delay at Intersections (sec)	AM Peak	Off Peak	PM Peak
Beresford Ave	-26.62	-10.84	-1.81
Firehouse	0.79	-12.99	-1.73
New Hampshire	0.70	-6.75	-45.72
Orange Camp	12.50	-0.60	17.88
Taylor	-17.04	-3.97	-15.16

Table B-2.6 Difference in Delay (sec) for each Intersection Through Movement Along the SB Direction

Delay at Intersections (sec)	AM Peak	Off Peak	PM Peak
Beresford Ave	-1.39	1.61	-8.07
Firehouse	-18.73	-7.09	-6.10
New Hampshire	3.63	-19.11	-9.06
Orange Camp	-14.05	-28.95	-86.18
Taylor	-7.96	-16.26	-36.26

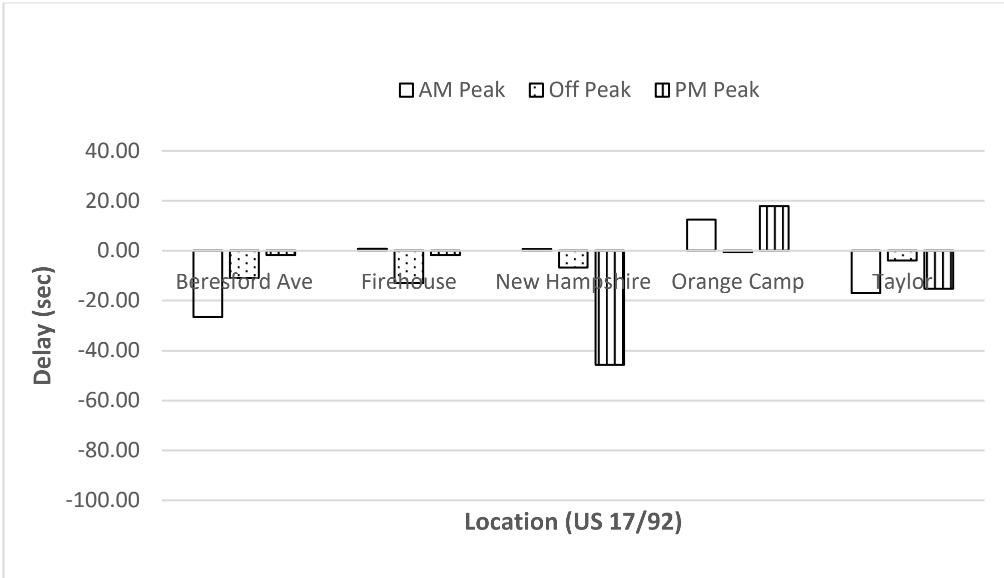


Figure B-2.5 Difference in Delay (sec) for each Intersection Through Movement Along the NB Direction

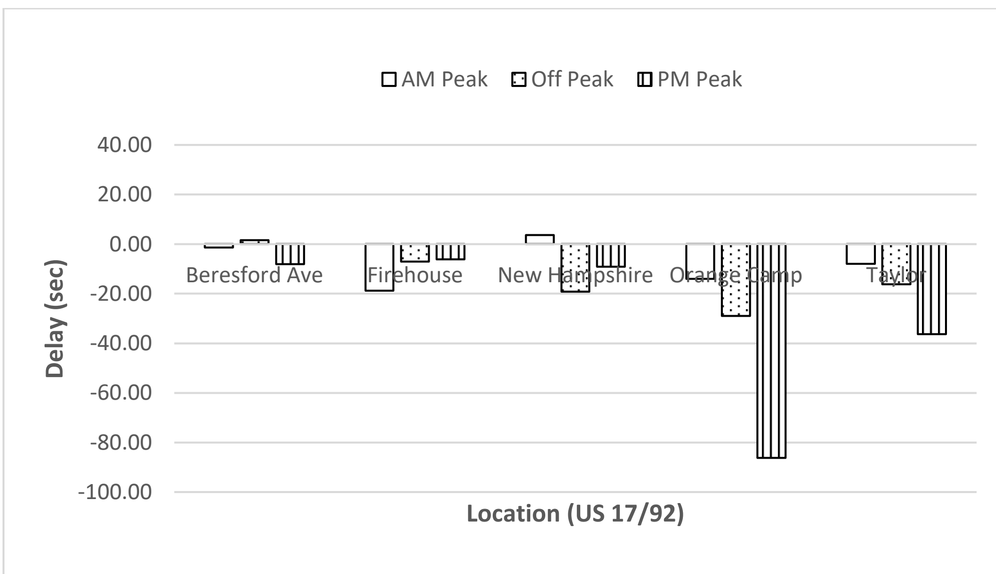


Figure B-2.6 Difference in Delay (sec) for each Intersection Through Movement Along the SB Direction

Discussion

The following can be concluded from the comparison of delays:

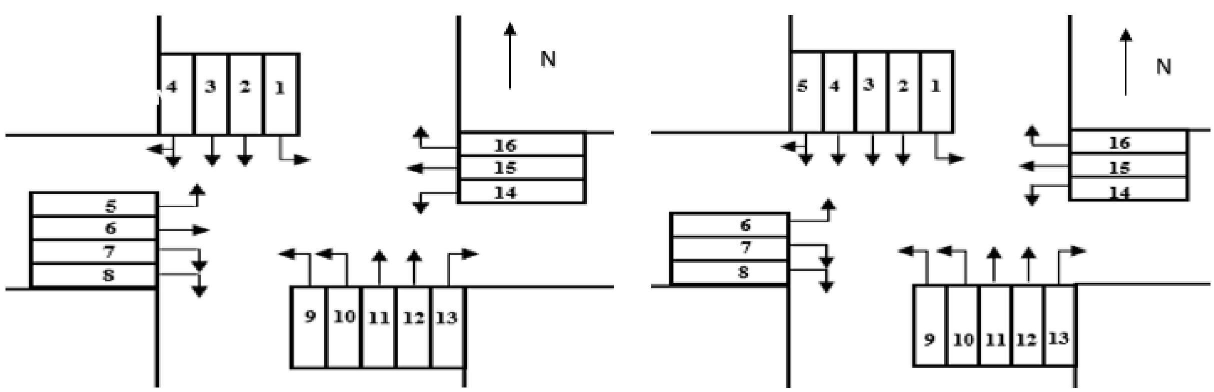
- The intersection delay decreased at most of the intersections and periods, there are only 2 intersections have relatively big increase in delay (12.50% and 17.88%), the rest of the intersections and periods show great improvement in delay, and there are 18 intersections (NB and SB, all periods) whose delay decrease over 10%.
- In the AM peak period, the intersection delay at Orange Camp NB increased greatly; the intersection delay increased slightly at Firehouse NB and New Hampshire in both directions.
- In the off peak period, the intersection delay at Beresford Ave SB increased slightly.
- In the PM peak period, the intersection delay at Orange Camp NB increased greatly.

The increases in delay for NB Orange Camp were not justified based on the volume observed, and may be due to the priorities of the optimization algorithm. The algorithm may be adjusted to reallocate the delay within the intersection based on agency preferences.

B.3.3 QUEUE LENGTH

Queue length (number of vehicles/lane) is presented by movement and by time period. This measure is used to evaluate oversaturated conditions at the critical intersections along the study corridors. Note that the queue length reported here is the observed maximum number of vehicles queued during each cycle, and does not represent the total number of vehicles that may have stopped during the cycle.

Figure B-3.1 presents the schematic of the lane configurations at the two critical intersections. Queue length is reported for each of these lanes.



(a) US 17/92 & Taylor Rd. (b) US 17/92 & Orange Camp Rd.

Figure B-3.1 Lane Configuration of Critical Intersections

Before Study (Oct. 1 & 2, 2014)

Table B-3.1 Average Queue Length by Lane (#vehs/lane) at US 17/92 & Taylor Rd

Time Period		Lane Number															
		1	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**	**13**	**14**	**15**	**16**
AM Peak	**1**	1	3.29	2.71	2.86	1.14	2	4.43	5.29	13.43	13.29	11.43	12.43	0	3.43	7.29	1.43
	2	1.14	4.86	4.29	4.43	1.57	3.57	4.43	7.29	10.71	10.57	15.86	15.43	0.14	2.29	3	0.86
	3	1.29	5.57	4.71	3.86	2.43	1.71	4.14	5	8.57	9.57	7.29	7.71	0.14	3.29	4.71	2.29
	4	0.57	4.57	4.86	2.29	1.71	1.86	3	6.71	7.29	6.57	5.29	5.43	0.14	1.86	2	0.57
	Average	**1**	**4.57**	**4.14**	**3.36**	**1.71**	**2.29**	**4**	**6.07**	**10**	**10**	**9.96**	**10.25**	**0.11**	**2.71**	**4.25**	**1.29**
Off Peak	**1**	1.43	6.57	3.71	3.71	2.86	1.29	1.43	4.14	5.43	6.14	10.14	9.57	0	1.86	1.14	0.14
	2	1	5.71	5.86	4.71	3.14	1	1.86	5.29	6	7.57	8.43	8.71	0	2.29	1.57	0.71
	3	1.14	6.29	6.14	4.57	2.57	1.86	1.71	4.29	6.29	6.86	9.43	9.71	0.14	2.14	2.29	0.14
	4	2	5.86	5.29	3.57	3.14	2.43	1.71	4.14	7	6.57	8.71	10.29	0.14	2.86	2.14	0.14
	Average	**1.39**	**6.11**	**5.25**	**4.14**	**2.93**	**1.64**	**1.68**	**4.46**	**6.18**	**6.79**	**9.18**	**9.57**	**0.07**	**2.29**	**1.79**	**0.29**
PM Peak	**1**	2	10.5	10.5	7.33	2.83	3.67	3	6.33	12.33	12.67	14.83	16.83	0.17	2.5	3.5	0
	2	2	12.17	10.83	12.17	3.67	3.67	4.5	9.17	7.33	8.17	6	9.17	0.17	2.33	3	1.33
	3	1.67	12.17	12.5	9.33	1.83	3.67	3	6	9.33	10	5.33	5.83	0.17	2	3.83	0
	4	3	8.33	9	6.67	2.33	3.83	2.83	4.17	7.83	11.83	7.5	8.83	0	2.5	2	0.17
	Average	**2.17**	**10.79**	**10.71**	**8.88**	**2.67**	**3.71**	**3.33**	**6.42**	**9.21**	**10.67**	**8.42**	**10.17**	**0.13**	**2.33**	**3.08**	**0.38**

Table B-3.2 Average Queue Length (#vehs/lane) at US 17/92 & Taylor Rd

Time Period	SB(Main)		EB			NB(Main)			WB		
	Left	Thru+Right	Left	Thru	Right	Left	Thru	Right	Left	Thru	Right
AM Peak	1	4.02	1.71	2.29	5.04	10	10.11	0.11	2.71	4.25	1.29
Off Peak	1.39	5.17	2.93	1.64	3.07	6.48	9.38	0.07	2.29	1.79	0.29
PM Peak	2.17	10.13	2.67	3.71	4.88	9.94	9.29	0.13	2.33	3.08	0.38

Table B-3.3 Average Queue Length by Lane (#vehs/lane) at US 17/92 & Orange Camp Rd.

Time Period		Lane															
		1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
AM Peak	1	3.67	4.83	6.83	10	0	2.67	2	1.83	4.83	12.5	12.5	9.33	0.67	2.33	1.33	3.5
	2	5.17	7	9.17	8.83	0	1.17	3.5	1.67	3	11.17	10.67	8	0	3.33	2.17	1
	3	4	6.33	7.17	9.17	0	1.83	2.17	1.5	3	9	9	4.5	1.33	2.17	2.33	2
	4	4.17	3.67	4.67	7.5	0	2.33	2.33	1.33	3.17	6	3.83	2.67	0.17	2	1.33	1
	Average	4.25	5.46	6.96	8.88	0	2	2.5	1.58	3.5	9.67	9	6.13	0.54	2.46	1.79	1.88
Off Peak	1	7.33	4.5	5.5	8.5	0	2	2.5	3	4.5	6.67	6.83	4.17	0.33	1.83	4.33	1.83
	2	8.33	8.67	10.17	10.67	0	4.17	3.33	1.5	6.5	13.33	11.33	7	1	2.67	2.33	3.17
	3	7.83	10.83	10.67	13.17	0	3.83	2.5	2.17	5.83	6.67	4.67	3.17	0.5	3	1.67	2.17

Time Period		Lane															
		1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
	4	7.33	7.5	8.67	8.5	0.17	4.5	1.83	1.83	5.83	8.5	6	2.5	0	2.83	3	2.67
	Average	7.71	7.88	8.75	10.21	0.04	3.63	2.54	2.13	5.67	8.79	7.21	4.21	0.46	2.58	2.83	2.46
PM Peak	**1**	8.8	15	14	8.8	0.8	5	8.6	2	8.8	13.2	15.8	17	0	4	3.8	3.2
	2	10.4	11.2	10.2	5.4	0	2.2	4.2	1.4	7.8	13.6	16.2	19	0	3.2	3.2	3.6
	3	8.2	18.4	16.4	12.2	0	3.4	5	3.2	10	21.4	27	31.4	0	2.2	3.2	3.6
	4	7.6	8	6.8	5	0	4.2	2.4	3	7.6	8.6	11.2	13	0	3.4	7	2.4
	Average	8.75	13.15	11.85	7.85	0.2	3.7	5.05	2.4	8.55	14.2	17.55	20.1	0	3.2	4.3	3.2

Table B-3.4 Average Queue Length (#vehs/lane) at US 17/92 & Orange Camp Rd.

Time	SB(Main)			EB			NB(Main)			WB		
	Left	Thru	Right	Left	Thru	Right	Left	Thru	Right	Left	Thru	Right
AM Peak	4.25	7.1	0	2	2.5	1.58	3.5	8.26	0.54	2.46	1.79	1.88
Off Peak	7.71	8.94	0.04	3.63	2.54	2.13	5.67	6.74	0.46	2.58	2.83	2.46
PM Peak	8.75	10.95	0.2	3.7	5.05	2.4	8.55	17.28	0	3.2	4.3	3.2

After Study (Nov. 19 & Nov. 20, 2014)

Table B-3.5 Average Queue Length by Lane (#vehs/lane) at US 17/92 & Taylor Rd.

Time	Period	Lane															
		1	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**	**13**	**14**	**15**	**16**
AM Peak	**1**	1.33	5.67	4.83	3.33	1.50	2.00	3.50	6.50	6.83	6.50	4.67	4.00	0.00	4.00	5.33	1.33
	2	1.33	6.33	5.83	4.17	2.33	2.50	1.67	3.17	5.33	5.17	4.33	4.50	0.00	1.50	4.50	1.67
	3	1.17	8.83	7.67	4.17	2.00	2.33	1.83	3.17	3.33	3.17	4.33	4.00	0.17	4.33	3.50	1.00
	4	0.67	5.67	6.17	3.67	3.00	1.50	1.00	1.83	3.00	3.33	3.33	4.00	0.33	0.67	1.67	1.00
	Average	**1.13**	**6.63**	**6.13**	**3.83**	**2.21**	**2.08**	**2.00**	**3.67**	**4.63**	**4.54**	**4.17**	**4.13**	**0.13**	**2.63**	**3.75**	**1.25**
Off Peak	**1**	1.67	9.33	7.17	5.33	3.50	2.33	2.67	5.67	5.50	5.50	3.00	3.00	0.17	1.83	2.50	0.83
	2	2.33	11.83	7.33	4.50	3.67	2.67	2.83	6.17	7.00	6.83	4.17	4.33	0.83	2.83	1.83	1.67
	3	2.00	11.00	9.33	4.83	2.00	3.50	2.50	7.17	7.17	6.83	2.67	3.83	0.67	1.83	3.50	1.17
	4	1.33	10.33	8.50	6.00	3.33	4.00	3.50	8.33	8.17	7.17	2.50	3.67	0.83	3.17	2.50	1.00
	Average	**1.83**	**10.63**	**8.08**	**5.17**	**3.13**	**3.13**	**2.88**	**6.83**	**6.96**	**6.58**	**3.08**	**3.71**	**0.63**	**2.42**	**2.58**	**1.17**
PM Peak	**1**	1.00	11.20	8.20	8.40	4.20	6.60	5.20	17.60	11.80	10.80	2.20	3.80	0.80	3.60	4.80	1.00
	2	1.60	9.20	7.60	6.20	4.20	4.00	8.00	12.20	8.20	8.40	3.60	3.60	0.00	3.60	3.80	0.80
	3	1.20	8.60	7.80	4.80	3.00	3.60	5.60	9.00	15.20	15.20	4.80	5.40	0.00	3.60	5.60	1.20
	4	1.80	11.40	10.00	7.60	4.80	3.80	4.20	7.40	7.80	8.00	2.40	2.80	0.40	5.80	4.20	1.80

Time	Period	Lane															
		1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
	Average	**1.40**	**10.10**	**8.40**	**6.75**	**4.05**	**4.50**	**5.75**	**11.55**	**10.75**	**10.60**	**3.25**	**3.90**	**0.30**	**4.15**	**4.60**	**1.20**

Table B-3.6 Average Queue Length (#vehs/lane) at US 17/92 & Taylor Rd.

Time	SB(Main)		EB			NB(Main)			WB		
	Left	Thru+Right	Left	Thru	Right	Left	Thru	Right	Left	Thru	Right
AM Peak	1.13	5.53	2.21	2.08	2.83	4.58	4.15	0.13	2.63	3.75	1.25
Off Peak	1.83	7.96	3.13	3.13	4.85	6.77	3.40	0.63	2.42	2.58	1.17
PM Peak	1.40	8.42	4.05	4.50	8.65	10.68	3.58	0.30	4.15	4.60	1.20

Table B-3.7 Average Queue Length by Lane (#vehs/lane) at US 17/92 & Orange Camp Rd.

Time	Period	Lane															
		1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
AM Peak	1	4.67	7.50	7.50	12.33	1.00	2.67	2.50	2.83	3.00	11.00	7.83	6.50	0.50	2.33	2.33	6.00
	2	5.17	4.50	4.83	7.00	0.17	1.67	1.83	2.83	3.67	11.17	9.33	6.50	0.17	3.17	2.50	4.50
	3	9.50	4.50	6.67	6.83	0.00	2.67	2.50	3.00	3.83	10.50	9.33	8.00	0.17	4.17	1.33	5.17
	4	3.50	3.33	4.83	5.83	0.00	1.83	2.17	3.83	2.00	6.50	5.50	2.83	0.50	1.67	2.83	2.50
	Average	**5.71**	**4.96**	**5.96**	**8.00**	**0.29**	**2.21**	**2.25**	**3.13**	**3.13**	**9.79**	**8.00**	**5.96**	**0.33**	**2.83**	**2.25**	**4.54**
Off Peak	1	10.00	7.20	7.60	7.80	0.20	3.40	4.20	3.20	6.20	9.00	7.60	4.20	0.60	2.00	5.00	1.20

Time	Period	Lane															
		1	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**	**13**	**14**	**15**	**16**
	2	7.60	7.60	9.60	10.80	0.00	1.80	3.80	2.60	4.80	8.60	7.00	3.40	1.00	2.80	4.00	1.60
	3	8.20	8.40	8.40	10.40	0.20	4.60	4.20	4.60	8.40	12.60	8.60	5.40	1.00	2.00	1.80	1.00
	4	11.00	5.40	7.80	8.60	0.20	3.60	4.80	2.00	5.60	12.40	8.40	6.80	0.80	2.80	2.80	2.20
	Average	**9.20**	**7.15**	**8.35**	**9.40**	**0.15**	**3.35**	**4.25**	**3.10**	**6.25**	**10.65**	**7.90**	**4.95**	**0.85**	**2.40**	**3.40**	**1.50**
PM Peak	**1**	9.60	8.00	7.80	10.80	0.40	6.20	10.60	3.20	7.80	13.20	10.60	8.00	1.20	7.40	15.00	5.40
	2	12.40	12.80	14.00	14.80	0.20	4.80	8.00	4.60	8.00	14.60	13.20	8.80	1.60	6.20	3.20	3.20
	3	10.60	11.00	12.20	14.00	0.20	7.40	7.00	4.20	9.40	14.40	13.40	8.60	1.40	4.00	6.40	3.80
	4	7.80	8.00	7.80	11.20	0.60	3.80	4.80	4.40	8.80	11.20	9.20	5.60	1.00	2.00	4.00	5.00
	Average	**10.10**	**9.95**	**10.45**	**12.70**	**0.35**	**5.55**	**7.60**	**4.10**	**8.50**	**13.35**	**11.60**	**7.75**	**1.30**	**4.90**	**7.15**	**4.35**

Table B-3.8 Average Queue Length (#vehs/lane) at US 17/92 & Orange Camp Rd.

Time	SB(Main)			EB				NB(Main)			WB	
	Left	**Thru**	**Right**	**Left**	**Left**	**Thru**	**Right**	**Left**	**Left**	**Thru**	**Right**	**Left**
AM Peak	5.71	6.31	0.29	2.21	2.25	3.13	3.13	7.92	0.33	2.83	2.25	4.54
Off Peak	9.20	8.30	0.15	3.35	4.25	3.10	6.25	7.83	0.85	2.40	3.40	1.50
PM Peak	10.10	11.03	0.35	5.55	7.60	4.10	8.50	10.90	1.30	4.90	7.15	4.35

Comparisons of Before and After Queue Lengths for Critical Intersections

The differences in queue length between the before and after measurements are shown in Table B-3.9 to Table B-3.12. The tables are color-coded as follows: green shows significant improvement, yellow shows modest change, and red shows significant increase in queue length.

Table B-3.9 Difference in Average Queue Length by Lane (#vehs/lane) at US 17/92 & Taylor Rd.

Time Period	**Lane Number**																**Average Queue**
	1	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**	**13**	**14**	**15**	**16**	
AM Peak	0.13	2.05	1.98	0.48	0.49	-0.20	-2.00	-2.40	-5.38	-5.46	-5.80	-6.13	0.02	-0.09	-0.50	-0.04	**-1.43**
Off Peak	0.44	4.52	2.83	1.02	0.20	1.48	1.20	2.37	0.78	-0.20	-6.10	-5.86	0.55	0.13	0.80	0.88	**0.32**
PM Peak	-0.77	-0.69	-2.31	-2.13	1.38	0.79	2.42	5.13	1.54	-0.07	-5.17	-6.27	0.18	1.82	1.52	0.83	**-0.11**

Table B-3.10 Difference in Average Queue Length (#vehs/lane) at US 17/92 & Taylor Rd.

Time Period	**SB (Main)**		**EB (Main)**			**NB (Main)**			**WB (Main)**		
	Left	**Thru+Right**	**Left**	**Thru**	**Right**	**Left**	**Thru**	**Right**	**Left**	**Thru**	**Right**
AM Peak	0.13	1.50	0.49	-0.20	-2.20	-5.42	-5.96	0.02	-0.09	-0.50	-0.04
Off Peak	0.44	2.79	0.20	1.48	1.78	0.29	-5.98	0.55	0.13	0.80	0.88
PM Peak	-0.77	-1.71	1.38	0.79	3.78	0.74	-5.72	0.18	1.82	1.52	0.83

Table B-3.11 Difference in Average Queue Length by Lane (#vehs/lane) at US 17/92 & Orange Camp Rd.

Time Period	Lane Number																Average Queue
	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	
AM Peak	1.46	-0.50	-1.00	-0.88	0.29	0.21	-0.25	1.54	-0.38	0.13	-1.00	-0.17	-0.21	0.38	0.46	2.67	0.17
Off Peak	1.49	-0.73	-0.40	-0.81	0.11	-0.28	1.71	0.98	0.58	1.86	0.69	0.74	0.39	-0.18	0.57	-0.96	0.36
PM Peak	1.35	-3.20	-1.40	4.85	0.15	1.85	2.55	1.70	-0.05	-0.85	-5.95	-12.35	1.30	1.70	2.85	1.15	-0.27

Table B-3.12 Difference in Average Queue Length (#vehs/lane)) at US 17/92 & Orange

Time Period	SB (Main)			EB (Main)			NB (Main)			WB (Main)		
	Left	Thru	Right	Left	Thru	Right	Left	Thru	Right	Left	Thru	Right
AM Peak	1.46	-0.79	0.29	0.21	-0.25	1.54	-0.38	-0.35	-0.21	0.38	0.46	2.67
Off Peak	1.49	-0.64	0.11	-0.28	1.71	0.98	0.58	1.10	0.39	-0.18	0.57	-0.96
PM Peak	1.35	0.08	0.15	1.85	2.55	1.70	-0.05	-6.38	1.30	1.70	2.85	1.15

Discussion

The following can be concluded from the comparison of queue length:

- Generally, the difference in average queue length between the before and after data does not exceed 2 vehicles, which shows no large improvement or deterioration in queue length at both critical intersections.

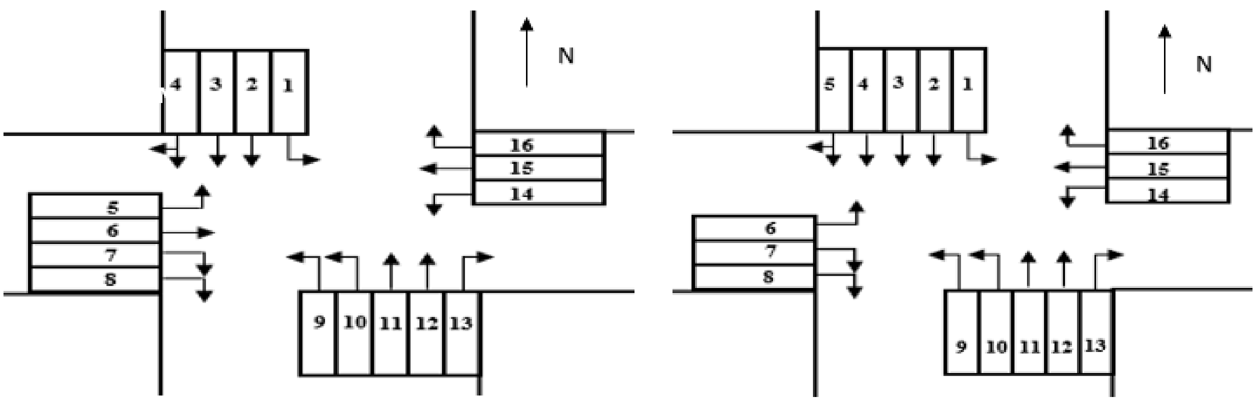
- For US 17/92 & Taylor Rd intersection, the average queue length decreased by an average of 1.43 vehicles in the a.m. peak period and 0.11 vehicles in the p.m. peak period; while it increased by an average of 0.32 vehicles in the off peak period. Especially, the queue of north bound through traffic decreased greatly.
- For US 17/92 & Orange Camp Rd intersection, the average queue increased in the AM peak period by an average of 0.17 vehicles and in the off peak period by an average of 0.36; while it decreased in the PM peak period by an average of 0.27 vehicles.

B.3.4 QUEUE TO LANE STORAGE RATIO

In addition to queue length, it is important to assess any impact to adjacent lanes or to upstream facilities. The queue to link/lane ratio is used to establish the likelihood of spillback, which is presented in this section by movement and by time period.

The following assumptions are used:

- The storage capacity is estimated as the maximum number of vehicles in the lane.
- The queue to link/lane storage ratio is estimated as 1 if the observer reported "spillback", while it is estimated as 0.8 if reported as the maximum number of vehicles in the sight distance.
- Queue to lane storage ratios over 80% are highlighted in yellow, as they represent conditions with a high probability for spillback.



(a) US 17/92 & Taylor Rd.

(b) US 17/92 & Orange Camp Rd.

Figure B-4.1 Lane Configuration of Critical Intersections

Before Study (Oct. 1 & 2, 2014)

Table B-4.1 Average Queue Storage Ratio by Lane and by Period at US 17/92 & Taylor Rd.

Time Period		Lane Number															
		1	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**	**13**	**14**	**15**	**16**
AM Peak	**1**	0.06	0.18	0.15	0.16	0.07	0.13	0.28	0.33	0.67	0.66	0.57	0.62	0	0.16	0.35	0.07
	2	0.06	0.27	0.24	0.25	0.1	0.22	0.28	0.46	0.54	0.53	0.79	0.77	0.01	0.11	0.14	0.04
	3	0.07	0.31	0.26	0.21	0.15	0.11	0.26	0.31	0.43	0.48	0.36	0.39	0.01	0.16	0.22	0.11
	4	0.03	0.25	0.27	0.13	0.11	0.12	0.19	0.42	0.36	0.33	0.26	0.27	0.01	0.09	0.1	0.03
	Average	**0.06**	**0.25**	**0.23**	**0.19**	**0.11**	**0.14**	**0.25**	**0.38**	**0.5**	**0.5**	**0.5**	**0.51**	**0.01**	**0.13**	**0.2**	**0.06**
Off Peak	**1**	0.08	0.37	0.21	0.21	0.18	0.08	0.09	0.26	0.27	0.31	0.51	0.48	0	0.09	0.05	0.01
	2	0.06	0.32	0.33	0.26	0.2	0.06	0.12	0.33	0.3	0.38	0.42	0.44	0	0.11	0.07	0.03
	3	0.06	0.35	0.34	0.25	0.16	0.12	0.11	0.27	0.31	0.34	0.47	0.49	0.01	0.1	0.11	0.01
	4	0.11	0.33	0.29	0.2	0.2	0.15	0.11	0.26	0.35	0.33	0.44	0.51	0.01	0.14	0.1	0.01
	Average	**0.08**	**0.34**	**0.29**	**0.23**	**0.18**	**0.1**	**0.1**	**0.28**	**0.31**	**0.34**	**0.46**	**0.48**	**0**	**0.11**	**0.09**	**0.01**
PM Peak	**1**	0.11	0.58	0.58	0.41	0.18	0.23	0.19	0.4	0.62	0.63	0.74	0.84	0.01	0.12	0.17	0
	2	0.11	0.68	0.6	0.68	0.23	0.23	0.28	0.57	0.37	0.41	0.3	0.46	0.01	0.11	0.14	0.06
	3	0.09	0.68	0.69	0.52	0.11	0.23	0.19	0.38	0.47	0.5	0.27	0.29	0.01	0.1	0.18	0
	4	0.17	0.46	0.5	0.37	0.15	0.24	0.18	0.26	0.39	0.59	0.38	0.44	0	0.12	0.1	0.01
	Average	**0.12**	**0.6**	**0.59**	**0.49**	**0.17**	**0.23**	**0.21**	**0.4**	**0.46**	**0.53**	**0.42**	**0.51**	**0.01**	**0.11**	**0.15**	**0.02**

Table B-4.2 Average Queue Storage Ratio by Lane and by Period at US 17/92 & Orange Camp Rd.

Time Period		**Lane Number**															
		1	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**	**13**	**14**	**15**	**16**
AM Peak	**1**	0.18	0.24	0.34	0.5	0	0.13	0.1	0.09	0.28	0.45	0.45	0.33	0.02	0.12	0.07	0.18
	2	0.26	0.35	0.46	0.44	0	0.06	0.18	0.08	0.18	0.4	0.38	0.29	0	0.17	0.11	0.05
	3	0.2	0.32	0.36	0.46	0	0.09	0.11	0.08	0.18	0.32	0.32	0.16	0.05	0.11	0.12	0.1
	4	0.21	0.18	0.23	0.38	0	0.12	0.12	0.07	0.19	0.21	0.14	0.1	0.01	0.1	0.07	0.05
	Average	**0.21**	**0.27**	**0.35**	**0.44**	**0**	**0.1**	**0.13**	**0.08**	**0.21**	**0.35**	**0.32**	**0.22**	**0.02**	**0.12**	**0.09**	**0.09**
Off Peak	**1**	0.37	0.23	0.28	0.43	0	0.1	0.13	0.15	0.26	0.24	0.24	0.15	0.01	0.09	0.22	0.09
	2	0.42	0.43	0.51	0.53	0	0.21	0.17	0.08	0.38	0.48	0.4	0.25	0.04	0.13	0.12	0.16
	3	0.39	0.54	0.53	0.66	0	0.19	0.13	0.11	0.34	0.24	0.17	0.11	0.02	0.15	0.08	0.11
	4	0.37	0.38	0.43	0.43	0.01	0.23	0.09	0.09	0.34	0.3	0.21	0.09	0	0.14	0.15	0.13
	Average	**0.39**	**0.39**	**0.44**	**0.51**	**0**	**0.18**	**0.13**	**0.11**	**0.33**	**0.31**	**0.26**	**0.15**	**0.02**	**0.13**	**0.14**	**0.12**
PM Peak	**1**	0.44	0.75	0.7	0.44	0.04	0.25	0.43	0.1	0.52	0.47	0.56	0.61	0	0.2	0.19	0.16
	2	0.52	0.56	0.51	0.27	0	0.11	0.21	0.07	0.46	0.49	0.58	0.68	0	0.16	0.16	0.18
	3	0.41	0.92	0.82	0.61	0	0.17	0.25	0.16	0.59	0.76	0.96	1	0	0.11	0.16	0.18
	4	0.38	0.4	0.34	0.25	0	0.21	0.12	0.15	0.45	0.31	0.4	0.46	0	0.17	0.35	0.12
	Average	**0.44**	**0.66**	**0.59**	**0.39**	**0.01**	**0.19**	**0.25**	**0.12**	**0.5**	**0.51**	**0.63**	**0.72**	**0**	**0.16**	**0.22**	**0.16**

After Study (Dec. 16 & 17, 2014)

Table B-4.3 Average Queue Storage Ratio by Lane by Period at US 17/92 & Taylor Rd.

Time Period		Lane															
		1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
AM Peak	1	0.07	0.31	0.27	0.19	0.09	0.13	0.22	0.41	0.34	0.33	0.23	0.20	0.00	0.19	0.25	0.06
	2	0.07	0.35	0.32	0.23	0.15	0.16	0.10	0.20	0.27	0.26	0.22	0.23	0.00	0.07	0.21	0.08
	3	0.06	0.49	0.43	0.23	0.13	0.15	0.11	0.20	0.17	0.16	0.22	0.20	0.01	0.21	0.17	0.05
	4	0.04	0.31	0.34	0.20	0.19	0.09	0.06	0.11	0.15	0.17	0.17	0.20	0.02	0.03	0.08	0.05
	Average	0.06	0.37	0.34	0.21	0.14	0.13	0.13	0.23	0.23	0.23	0.21	0.21	0.01	0.13	0.18	0.06
Off Peak	1	0.09	0.52	0.40	0.30	0.22	0.15	0.17	0.35	0.28	0.28	0.15	0.15	0.01	0.09	0.12	0.04
	2	0.13	0.66	0.41	0.25	0.23	0.17	0.18	0.39	0.35	0.34	0.21	0.22	0.04	0.13	0.09	0.08
	3	0.11	0.61	0.52	0.27	0.13	0.22	0.16	0.45	0.36	0.34	0.13	0.19	0.03	0.09	0.17	0.06
	4	0.07	0.57	0.47	0.33	0.21	0.25	0.22	0.52	0.41	0.36	0.13	0.18	0.04	0.15	0.12	0.05
	Average	0.10	0.59	0.45	0.29	0.20	0.20	0.18	0.43	0.35	0.33	0.15	0.19	0.03	0.12	0.12	0.06
PM Peak	1	0.06	0.62	0.46	0.47	0.26	0.41	0.33	1.00	0.59	0.54	0.11	0.19	0.04	0.17	0.23	0.05
	2	0.09	0.51	0.42	0.34	0.26	0.25	0.50	0.76	0.41	0.42	0.18	0.18	0.00	0.17	0.18	0.04
	3	0.07	0.48	0.43	0.27	0.19	0.23	0.35	0.56	0.76	0.76	0.24	0.27	0.00	0.17	0.27	0.06
	4	0.10	0.63	0.56	0.42	0.30	0.24	0.26	0.46	0.39	0.40	0.12	0.14	0.02	0.28	0.20	0.09
	Average	0.14	0.56	0.47	0.38	0.25	0.28	0.36	0.72	0.54	0.53	0.16	0.20	0.02	0.20	0.22	0.06

Table B-4.4 Average Queue Storage Ratio by Lane by Period at US 17/92 & Orange Rd.

Time Period		**Lane**															
		1	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**	**13**	**14**	**15**	**16**
AM Peak	**1**	0.23	0.38	0.38	0.62	0.05	0.13	0.13	0.14	0.18	0.39	0.28	0.23	0.02	0.12	0.12	0.30
	2	0.26	0.23	0.24	0.35	0.01	0.08	0.09	0.14	0.22	0.40	0.33	0.23	0.01	0.16	0.13	0.23
	3	0.48	0.23	0.33	0.34	0.00	0.13	0.13	0.15	0.23	0.38	0.33	0.29	0.01	0.21	0.07	0.26
	4	0.18	0.17	0.24	0.29	0.00	0.09	0.11	0.19	0.12	0.23	0.20	0.10	0.02	0.08	0.14	0.13
	Average	0.29	0.25	0.30	0.40	0.01	0.11	0.11	0.16	0.18	0.35	0.29	0.21	0.01	0.14	0.11	0.23
Off Peak	**1**	0.50	0.36	0.38	0.39	0.01	0.17	0.21	0.16	0.36	0.32	0.27	0.15	0.02	0.10	0.25	0.06
	2	0.38	0.38	0.48	0.54	0.00	0.09	0.19	0.13	0.28	0.31	0.25	0.12	0.04	0.14	0.20	0.08
	3	0.41	0.42	0.42	0.52	0.01	0.23	0.21	0.23	0.49	0.45	0.31	0.19	0.04	0.10	0.09	0.05
	4	0.55	0.27	0.39	0.43	0.01	0.18	0.24	0.10	0.33	0.44	0.30	0.24	0.03	0.14	0.14	0.11
	Average	0.46	0.36	0.42	0.47	0.01	0.17	0.21	0.16	0.37	0.38	0.28	0.18	0.03	0.12	0.17	0.08
PM Peak	**1**	0.48	0.40	0.39	0.54	0.02	0.31	0.53	0.16	0.46	0.47	0.38	0.29	0.04	0.37	0.75	0.27
	2	0.62	0.64	0.70	0.74	0.01	0.24	0.40	0.23	0.47	0.52	0.47	0.31	0.06	0.31	0.16	0.16
	3	0.53	0.55	0.61	0.70	0.01	0.37	0.35	0.21	0.55	0.51	0.48	0.31	0.05	0.20	0.32	0.19
	4	0.39	0.40	0.39	0.56	0.03	0.19	0.24	0.22	0.52	0.40	0.33	0.20	0.04	0.10	0.20	0.25
	Average	0.51	0.50	0.52	0.64	0.02	0.28	0.38	0.21	0.50	0.48	0.41	0.28	0.05	0.25	0.36	0.22

Comparisons of Before and After Queue Storage Ratios

The differences in Queue Storage Ratio between before and after measurements are shown in Table B-4.5 and Table B-4.6. The tables are color-coded as follows: green shows improvement, yellow shows modest change, and red shows deterioration in queue storage ratios or spillback potential.

Table B-4.5 Difference in Average Queue Storage Ratios at US 17/92 & Taylor Rd.

Time Period	Lane Number															
	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
AM Peak	0	0.12	0.11	0.02	0.03	-0.01	-0.12	-0.15	-0.27	-0.27	-0.29	-0.3	0	0	-0.02	0
Off Peak	0.02	0.25	0.16	0.06	0.02	0.1	0.08	0.15	0.04	-0.01	-0.31	-0.29	0.03	0.01	0.03	0.05
PM Peak	0.02	-0.04	-0.12	-0.11	0.08	0.05	0.15	0.32	0.08	0	-0.26	-0.31	0.01	0.09	0.07	0.04

Table B-4.6 Difference in Average Queue Storage Ratio at US 17/92 & Orange Camp Rd.

Time Period	Lane Number															
	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
AM Peak	0.08	-0.02	-0.05	-0.04	0.01	0.01	-0.02	0.08	-0.03	0	-0.03	-0.01	-0.01	0.02	0.02	0.14
Off Peak	0.07	-0.03	-0.02	-0.04	0.01	-0.01	0.08	0.05	0.04	0.07	0.02	0.03	0.01	-0.01	0.03	-0.04
PM Peak	0.07	-0.16	-0.07	0.25	0.01	0.09	0.13	0.09	0	-0.03	-0.22	-0.44	0.05	0.09	0.14	0.06

Discussion

The following are concluded from the comparison of queue storage ratios:

- Left turn lane spillback rarely happened in either before study or after study, left turn spillback only happened 4 times in before study and at Taylor Rd intersection, due to the large left turn truck volume on northbound left, no left turn lane spillback happened in the after study.
- The queue storage ratios improved for the NB direction at the Taylor Rd. intersection, especially during the AM peak.
- The queue storage ratios show modest improvement for the US 17/92 & Orange Camp Rd. intersection for the new system.

B.3.5 EQUIVALENT PCE FLOWS

Traffic flows were counted manually and converted to equivalent PCE flows (pce/hour) by considering the presence of heavy vehicles. It was assumed that the PCE for trucks is 2.

Truck Percentage Observations

Table B-5.1 Truck Percentages at US 17/92 & Taylor Rd.

Period	Interval	SB			WB			NB			EB			Total
		Left	Thru	Right	Left	Thru	Right	Left	Thru	Right	Left	Thru	Right	
AM Peak	1	0.00%	0.78%	0.00%	9.52%	0.00%	5.56%	5.00%	1.37%	0.00%	0.00%	0.00%	2.87%	2.42%
	2	0.00%	2.70%	0.00%	0.00%	2.56%	10.00%	4.82%	1.08%	10.00%	6.25%	0.00%	6.38%	3.21%
	3	0.00%	1.29%	0.00%	0.00%	0.00%	0.00%	9.90%	1.69%	0.00%	0.00%	0.00%	3.09%	2.58%
	4	0.00%	1.23%	0.00%	2.44%	0.00%	0.00%	9.28%	3.97%	5.88%	0.00%	0.00%	6.67%	4.06%
	Average	**0.00%**	**1.42%**	**0.00%**	**2.68%**	**0.86%**	**3.80%**	**6.40%**	**1.92%**	**3.13%**	**1.75%**	**0.00%**	**4.53%**	**3.00%**
Off Peak	1	14.29%	3.08%	0.00%	0.00%	0.00%	8.33%	10.99%	2.63%	0.00%	4.55%	16.67%	2.94%	4.07%
	2	0.00%	1.38%	13.33%	6.25%	15.00%	0.00%	9.28%	4.29%	0.00%	8.33%	8.33%	6.73%	4.79%
	3	0.00%	2.75%	0.00%	6.25%	6.25%	25.00%	9.43%	2.39%	20.00%	0.00%	4.00%	6.41%	4.55%
	4	0.00%	2.25%	8.70%	4.76%	0.00%	0.00%	5.47%	1.47%	10.00%	0.00%	0.00%	3.05%	2.83%

Period	Interval	SB			WB			NB			EB			Total
		Left	Thru	Right	Left	Thru	Right	Left	Thru	Right	Left	Thru	Right	
	Average	**1.79%**	**2.35%**	**5.56%**	**4.41%**	**5.13%**	**6.12%**	**7.86%**	**2.68%**	**6.35%**	**3.06%**	**4.94%**	**4.44%**	**3.87%**
PM Peak	1	0.00%	1.06%	0.00%	0.00%	4.55%	0.00%	4.05%	1.10%	0.00%	0.00%	0.00%	4.19%	2.12%
	2	0.00%	2.11%	0.00%	0.00%	3.03%	0.00%	6.34%	1.49%	5.56%	0.00%	0.00%	4.52%	2.85%
	3	0.00%	1.01%	0.00%	0.00%	2.56%	6.67%	2.96%	0.55%	0.00%	0.00%	0.00%	1.79%	1.35%
	4	0.00%	0.56%	0.00%	0.00%	3.45%	0.00%	1.96%	0.00%	0.00%	0.00%	0.00%	2.84%	1.05%
	Average	**0.00%**	**1.16%**	**0.00%**	**0.00%**	**3.15%**	**2.08%**	**3.67%**	**0.77%**	**1.14%**	**0.00%**	**0.00%**	**3.18%**	**1.79%**

Table B-5.2 Truck Percentages at US 17/92 & Orange Camp Rd.

Period	Interval	SB			WB			NB			EB			Total
		Left	Thru	Right	Left	Thru	Right	Left	Thru	Right	Left	Thru	Right	
AM Peak	1	0.00%	5.13%	0.00%	0.00%	10.53%	5.17%	3.70%	2.19%	3.70%	0.00%	0.00%	6.06%	3.56%
	2	7.41%	6.21%	0.00%	13.04%	7.14%	1.79%	3.33%	3.07%	4.55%	0.00%	0.00%	0.00%	4.16%
	3	5.88%	6.85%	0.00%	8.70%	4.76%	4.65%	0.00%	2.75%	7.14%	5.26%	0.00%	3.70%	4.50%
	4	4.35%	5.70%	0.00%	0.00%	0.00%	5.00%	5.00%	6.67%	0.00%	5.88%	0.00%	6.67%	5.60%
	Average	**4.59%**	**5.73%**	**0.00%**	**6.67%**	**6.06%**	**3.96%**	**3.00%**	**3.43%**	**3.66%**	**3.45%**	**0.00%**	**4.20%**	**4.25%**
Off Peak	1	3.45%	4.04%	7.69%	0.00%	0.00%	0.00%	3.45%	2.46%	0.00%	10.53%	0.00%	0.00%	3.08%
	2	5.56%	3.94%	9.09%	4.35%	0.00%	0.00%	0.00%	2.05%	0.00%	0.00%	0.00%	0.00%	2.44%
	3	4.00%	4.14%	0.00%	0.00%	0.00%	2.50%	0.00%	2.48%	0.00%	2.86%	0.00%	0.00%	2.59%
	4	5.26%	3.86%	0.00%	0.00%	0.00%	0.00%	2.17%	1.58%	0.00%	2.38%	3.45%	1.85%	2.51%
	Average	**4.29%**	**3.83%**	**4.41%**	**1.32%**	**0.00%**	**0.68%**	**1.13%**	**2.06%**	**0.00%**	**3.17%**	**1.19%**	**0.65%**	**2.58%**
PM Peak	1	0.00%	2.88%	0.00%	6.06%	0.00%	2.27%	0.00%	1.33%	10.34%	4.76%	1.14%	5.00%	2.15%
	2	2.88%	4.14%	0.00%	2.70%	2.38%	5.77%	4.08%	2.17%	0.00%	0.00%	5.71%	3.45%	3.16%
	3	0.00%	0.90%	0.00%	0.00%	0.00%	0.00%	3.13%	1.14%	3.70%	0.00%	0.00%	0.00%	0.92%
	4	1.80%	0.73%	0.00%	3.85%	0.00%	0.00%	0.00%	2.29%	5.56%	0.00%	0.00%	2.08%	1.30%
	Average	**1.40%**	**1.99%**	**0.00%**	**3.01%**	**0.63%**	**2.14%**	**1.80%**	**1.67%**	**4.67%**	**0.83%**	**1.38%**	**2.37%**	**1.83%**

Before Study (Oct. 1 & 2, 2014)

Table B-5.3 Traffic Volume (pce/15 min) and Traffic Flow (pce/hour) at US 17/92 & Taylor Rd.

Time Period		**SB**			**WB**			**NB**			**EB**		
		Left	**Thru**	**Right**	**Left**	**Thru**	**Right**	**Left**	**Thru**	**Right**	**Left**	**Thru**	**Right**
AM Peak	1	8	132	13	18	24	9	114	163	6	7	13	184
	2	9	111	15	16	41	21	146	396	17	23	18	215
	3	16	161	19	28	28	16	125	279	20	19	18	145
	4	11	176	16	16	20	11	110	205	6	11	14	136
	Flow Rate	**44**	**580**	**63**	**78**	**113**	**57**	**495**	**1043**	**49**	**60**	**63**	**680**
Off Peak	1	10	213	14	10	18	19	114	212	13	26	17	113
	2	11	247	17	16	12	15	102	238	14	22	25	132
	3	13	262	27	12	20	13	112	234	16	21	13	120
	4	13	207	32	19	16	12	108	202	17	37	16	109
	Flow Rate	**47**	**929**	**90**	**57**	**66**	**59**	**436**	**886**	**60**	**106**	**71**	**474**
PM Peak	1	11	229	15	20	19	11	164	181	13	20	24	196
	2	10	271	11	21	25	20	150	256	25	21	30	182
	3	12	323	19	22	25	11	152	198	26	24	36	167
	4	14	230	15	21	32	16	124	190	27	16	27	139
	Flow Rate	**47**	**1053**	**60**	**84**	**101**	**58**	**590**	**825**	**91**	**81**	**117**	**684**

Table B-5.4 Traffic Volume (pce/15 min) and Traffic Flow (pce/hour) at US 17/92 & Orange Camp Rd.

Time Period		SB			WB			NB			EB		
		Left	**Thru**	**Right**	**Left**	**Thru**	**Right**	**Left**	**Thru**	**Right**	**Left**	**Thru**	**Right**
AM Peak	1	26	268	18	18	15	39	27	337	14	7	20	19
	2	36	285	13	31	13	64	23	457	16	9	17	44
	3	38	289	14	16	7	48	27	338	15	14	10	23
	4	36	262	14	23	11	51	36	304	31	8	16	16
	Flow Rate	**136**	**1104**	**59**	**88**	**46**	**202**	**113**	**1436**	**76**	**38**	**63**	**102**
Off Peak	1	49	286	15	33	28	51	34	294	20	21	26	25
	2	54	260	11	19	30	46	33	299	21	20	37	38
	3	36	291	8	21	20	34	39	267	26	41	49	23
	4	71	349	13	19	26	52	33	310	23	27	23	30
	Flow Rate	**210**	**1186**	**47**	**92**	**104**	**183**	**139**	**1170**	**90**	**109**	**135**	**116**
PM Peak	1	74	358	41	32	30	67	54	297	23	23	27	36
	2	73	392	27	23	26	44	63	400	24	26	36	32
	3	64	450	16	31	23	52	58	330	30	38	43	35
	4	82	455	21	28	36	49	38	325	24	38	46	38
	Flow Rate	**293**	**1655**	**105**	**114**	**115**	**212**	**213**	**1352**	**101**	**125**	**152**	**141**

After Study (Dec. 16 & 17, 2014)

Table B-5.5 Traffic Volume (pce/15 min) and Traffic Flow (pce/hour) at US 17/92 & Taylor Rd.

Time Period		Southbound			Westbound			Northbound			Eastbound		
		Left	**Thru**	**Right**	**Left**	**Thru**	**Right**	**Left**	**Thru**	**Right**	**Left**	**Thru**	**Right**
AM Peak	1	4	129	11	21	28	18	140	219	7	12	23	174
	2	7	114	15	29	40	22	174	374	11	17	25	200
	3	9	157	19	20	23	19	111	240	28	21	20	167
	4	4	164	15	42	25	20	106	262	18	7	14	144
	Flow Rate	**24**	**564**	**60**	**112**	**116**	**79**	**531**	**1095**	**64**	**57**	**82**	**685**
Off Peak	1	8	268	12	12	21	13	101	195	11	23	14	140
	2	22	220	17	17	23	15	106	243	18	26	13	111
	3	13	262	18	17	17	10	116	214	12	26	26	166
	4	13	273	25	22	17	11	135	207	22	23	28	169
	Flow Rate	**56**	**1023**	**72**	**68**	**78**	**49**	**458**	**859**	**63**	**98**	**81**	**586**
PM Peak	1	9	286	12	23	23	14	154	183	20	32	33	224
	2	10	338	20	19	34	9	151	205	19	32	24	185
	3	17	400	16	17	40	16	139	184	22	21	30	228
	4	13	356	11	21	30	9	156	204	27	22	31	181
	Flow Rate	**49**	**1380**	**59**	**80**	**127**	**48**	**600**	**776**	**88**	**107**	**118**	**818**

Table B-5.6 Traffic Volume (pce/15 min) and Traffic Flow (pce/hour) at US 17/92 & Orange Camp Rd.

Time Period		Southbound			Westbound			Northbound			Eastbound		
		Left	**Thru**	**Right**	**Left**	**Thru**	**Right**	**Left**	**Thru**	**Right**	**Left**	**Thru**	**Right**
AM Peak	1	20	273	8	12	19	58	27	366	27	11	17	33
	2	29	325	9	26	15	57	31	571	23	9	6	26
	3	36	312	20	25	22	45	21	373	15	20	12	28
	4	24	241	12	12	10	42	21	352	17	18	11	32
	Flow Rate	**109**	**1151**	**49**	**75**	**66**	**202**	**100**	**1662**	**82**	**58**	**46**	**119**
Off Peak	1	60	309	28	16	26	36	30	250	16	21	13	33
	2	38	264	12	24	17	28	57	298	23	26	18	33
	3	52	277	11	18	28	41	43	248	22	36	23	32
	4	60	350	17	18	22	43	47	321	14	43	30	55
	Flow Rate	**210**	**1200**	**68**	**76**	**93**	**148**	**177**	**1117**	**75**	**126**	**84**	**153**
PM Peak	1	68	322	16	35	33	45	56	381	32	22	89	42
	2	107	377	8	38	43	55	51	330	28	38	37	30
	3	70	448	9	33	33	39	66	356	28	29	49	48
	4	113	412	9	27	51	48	49	313	19	32	43	49
	Flow Rate	**358**	**1559**	**42**	**133**	**160**	**187**	**222**	**1380**	**107**	**121**	**218**	**169**

Comparisons of Before and After Flows

The differences in traffic flow between the before and after measurements are shown in Table B-5.7 and B-5.8. The tables are color-coded as follows: green shows significant decrease, yellow shows modest change, and red shows significant increase in flow.

Table B-5.7 Difference in Traffic Flow Rate (pce/hr) at US 17/92 & Talor Rd.

Period	Southbound			Westbound			Northbound			Eastbound		
	Left	Thru	Right	Left	Thru	Right	Left	Thru	Right	Left	Thru	Right
AM Peak	-20	-16	-3	34	3	22	36	52	15	-3	19	5
Off Peak	9	94	-18	11	12	-10	22	-27	3	-8	10	112
PM Peak	2	327	-1	-4	26	-10	10	-49	-3	26	1	134

Table B-5.8 Difference in Traffic Flow Rate (pce/hr) at US 17/92 & Orange Camp Rd.

Period	Southbound			Westbound			Northbound			Eastbound		
	Left	Thru	Right	Left	Thru	Right	Left	Thru	Right	Left	Thru	Right
AM Peak	-27	47	-10	-13	20	0	-13	226	6	20	-17	17
Off Peak	0	14	21	-16	-11	-35	38	-53	-15	17	-51	37
PM Peak	65	-96	-63	19	45	-25	9	28	6	-4	66	28

Discussion

The following can be concluded from the comparison of PCE flows:

- The traffic volume at both intersections (Taylor and Orange Camp) increased during all three time periods except for the off peak period at the Orange Camp Rd intersection.

B.3.6 CONSIDERATION OF TRAFFIC FLOWS JOINTLY WITH QUEUE LENGTH

The differences in “Traffic Flow” and “Queue Length” between before and after measurements are shown in Table B-6.1 and Table B-6.3. The tables are color-coded as follows: green indicates that “Queue” decreases and “Traffic Flow” increases, Red indicates “Queue” increases and “Traffic Flow” decreases, No Color indicates “Traffic Flow” and “Queue” increase or decrease at the same time. Generally, green indicates improvement despite an increase in flow.

Table B-6.1 Differences in Traffic Flow (TF) and Queue Length (Q) at US 17/92 & Taylor Rd.

Period	MOEs	Southbound			Westbound			Northbound			Eastbound		
		Left	Thru	Right	Left	Thru	Right	Left	Thru	Right	Left	Thru	Right
AM Peak	Flow	-20	-16		34	3	22	36	52	15	-3	19	5
	Queue	0.13	1.50		-0.09	-0.50	-0.04	-5.42	-5.96	0.02	0.49	-0.20	-2.20
Off Peak	Flow	9	94		11	12	-10	22	-27	3	-8	10	112
	Queue	0.44	2.79		0.13	0.80	0.88	0.29	-5.98	0.55	0.20	1.48	1.78
PM Peak	Flow	2	327		-4	26	-10	10	-49	-3	26	1	134
	Queue	-0.77	-1.71		1.82	1.52	0.83	0.74	-5.72	0.18	1.38	0.79	3.78

Table B-6.2 Differences (%) in Traffic Flow (TF) and Queue Length (Q) at US 17/92 & Taylor Rd.

Period	MOEs	Southbound			Westbound			Northbound			Eastbound		
		Left	Thru	Right	Left	Thru	Right	Left	Thru	Right	Left	Thru	Right
AM Peak	Flow	-45.5%	-2.8%		43.6%	2.7%	38.6%	7.3%	5.0%	30.6%	-5.0%	30.2%	0.7%
	Queue	12.5%	37.4%		-3.3%	-11.8%	-2.8%	-54.2%	-59.0%	16.7%	28.8%	-8.9%	-43.7%
Off Peak	Flow	19.1%	10.1%		19.3%	18.2%	-16.9%	5.0%	-3.0%	5.0%	-7.5%	14.1%	23.6%
	Queue	31.6%	54.0%		5.7%	44.7%	308.3%	4.5%	-63.8%	775.0%	6.7%	90.2%	58.0%
PM Peak	Flow	4.3%	31.1%		-4.8%	25.7%	-17.2%	1.7%	-5.9%	-3.3%	32.1%	0.9%	19.6%
	Queue	-35.4%	-16.9%		77.9%	49.2%	220.0%	7.4%	-61.5%	140.0%	51.9%	21.3%	77.4%

Table B-6.3 Differences in Traffic Flow and Queue Length at US 17/92 & Orange Camp Rd.

Period	MOEs	Southbound			Westbound			Northbound			Eastbound		
		Left	Thru	Right	Left	Thru	Right	Left	Thru	Right	Left	Thru	Right
AM Peak	Flow	-27	47	-10	-13	20	0	-13	226	6	20	-17	17
	Queue	1.46	-0.79	0.29	0.38	0.46	2.67	-0.38	-0.35	-0.21	0.21	-0.25	1.54
Off Peak	Flow	0	14	21	-16	-11	-35	38	-53	-15	17	-51	37
	Queue	1.49	-0.64	0.11	-0.18	0.57	-0.96	0.58	1.10	0.39	-0.28	1.71	0.98
PM Peak	Flow	65	-96	-63	19	45	-25	9	28	6	-4	66	28
	Queue	1.35	0.08	0.15	1.70	2.85	1.15	-0.05	-6.38	1.30	1.85	2.55	1.70

Table B-6.4 Differences (%) in Traffic Flow and Queue Length at US 17/92 & Orange Camp Rd.

Period	MOEs	Southbound			Westbound			Northbound			Eastbound		
		Left	Thru	Right	Left	Thru	Right	Left	Thru	Right	Left	Thru	Right
AM Peak	Flow	-19.9%	4.3%	-16.9%	-14.8%	43.5%	0.0%	-11.5%	15.7%	7.9%	52.6%	-27.0%	16.7%
	Queue	34.3%	-11.2%	0.0%	15.3%	25.6%	142.2%	-10.7%	-4.2%	-38.5%	10.4%	-10.0%	97.4%
Off Peak	Flow	0.0%	1.2%	44.7%	-17.4%	-10.6%	-19.1%	27.3%	-4.5%	-16.7%	15.6%	-37.8%	31.9%
	Queue	19.4%	-7.2%	260.0%	-7.1%	20.0%	-39.0%	10.3%	16.3%	85.5%	-7.6%	67.2%	45.9%
PM Peak	Flow	22.2%	-5.8%	-60.0%	16.7%	39.1%	-11.8%	4.2%	2.1%	5.9%	-3.2%	43.4%	19.9%
	Queue	15.4%	0.8%	75.0%	53.1%	66.3%	35.9%	-0.6%	-36.9%	0.0%	50.0%	50.5%	70.8%

Discussion

The following can be concluded from the comparison of traffic flows jointly with queue length:

- From the comparison above, we cannot conclude that there is a high correlation between increasing traffic with increasing queue at these two critical intersections.
- From the colored cells in the tables, traffic conditions at Orange Camp Rd have deteriorated during more time periods after ASCT installment than at intersection US 17/92 & Taylor Rd. Regarding Operational Performance Measures

Based on the field data collection and the comparison of operational performance measures we conclude the following:

The travel time and delay have greatly improved after installing the InSync system along the coordinated through movements of this corridor. However, the combined comparison of queue and flow rate shows that other movements at critical intersections did not experience large changes.

BEFORE AND AFTER-IMPLEMENTATION STUDIES OF

ADVANCED SIGNAL CONTROL TECHNOLOGIES IN FLORIDA

The main objective of this project is to evaluate the implementation of proposed adaptive signal control technology (ASCT) traffic operations at several arterial corridors in Florida, before and after the installation of the ASCT, document the advantages and the disadvantages of different approaches and implementations, and provide recommendations for statewide implementation of ASCT.

B.4 QUESTIONNAIRE

SECTION 1: RESPONDENT'S INFORMATION

Name Manny Rodriguez

Organization FDOT District 5

Position Traffic Engineer at FDOT District 5

Address

Phone

Fax

E-mail Manny.Rodriguez@dot.state.fl.us

SECTION 2: PREVIOUS TRAFFIC CONTROL TECHNOLOGY

1. Please specify the type of traffic control used before Adaptive Signal Control Technology (ASCT) was installed for your site. (Please check all that apply.)

a. Fixed time coordinated control **b. Actuated coordinated control**

c. Fixed-time isolated control d. Actuated isolated control

e. Other, please specify: ____

2. What type of traffic controller was used before ASCT was implemented?

a. NEMA TS-1 b. NEMA TS-2

c. 170 d. 2070

e. Other, please specify: ____

3. How many detectors were used before ASCT was implemented for a typical intersection? Please also specify the location where the detectors were typically placed for each specific detector type (video detection, inductive loops, radars, etc.).

Mainline (2 loop detectors for left turns), Side streets (presence detectors), Dilemma zone detection (essentially for green extension)

4. What was the frequency of retiming the signal timing plan for the corridor before ASCT was implemented?

Latest retiming at 2009; every 3-5 years the signal retiming process was taking place

How often were maintenance and updates performed to your previous control system in terms of the following components?

a. Detection: **twice per year maintenance and loop update**
b. Control Hardware: **last year update switch**
c. Software: **when updates are provided by the vendor**

5. What was the annual maintenance cost in terms of hardware, software and personnel of the previous control system for the whole corridor where the ASCT is now deployed? **$2,700-$3000 per intersection repair; average provided by FDOT**

6. What was the life expectancy of your traffic control before the deployment of ASCT? (Please refer to the hardware and software.)

Anywhere between 7-20 years depending on the conditions and the environment

7. How many crashes of each category were reported during the past 4 years before the ASCT implementation? (2011-2014)

	K (Killed)	A (Disabling Injury)	B (Evident Injury)	C (Possible Injury)	O (No Apparent Injury)
2011					
2012					
2013					
2014					

SECTION 3: ADAPTIVE TRAFFIC CONTROL SYSTEM (ASCT)

9. Which Adaptive Traffic Control System (ASCT) does your agency deploy?

d. **InSync; Version: Fusion (local detection plus cameras)**

e. Synchro Green; Version:________________

f. Other; please specify: ______________________

16. Did you consider any other ASCT before you selected this one for installation? Why did you reject the other(s) in favor of this one?

ACS-Lite was a no go due to its lagging functions, SCOOT not considered and SynchroGreen was already tried out

17. What were your major criteria for choosing the ASCT for the selected corridor?

Adaptability of the system and differences in the Deland Corridor (accommodate changes in speed from 50 to 35 and changes in lanes from 6 to 2)

18. What is the life expectancy of your ASCT system? (Consider both hardware and software.)

5-7 years depending on the components (cameras need replacement sooner than the rest).

19. What type of traffic controller is currently in use after the ASCT implementation?

a. Same as before the ASCT implementation
b. **New, please specify type: NEMA TS-2**
c. Other, specify:

20. How many and what type of detectors are used after the ASCT implementation on an intersection level? Please note any updates that the previous detection system needed so as to work with your ASCT.

Camera detection in place (W Beresford Avenue and US17/92 intersection as well as New Hampshire and US17/92 will be only operating with video detection); easier to fix cameras as you don't have to cut operations

21. Was there a need to update your previous software operating system at your Transportation Management Center in order to get your current ASCT software working?

a. Yes **b. No**

21. How often do you plan to perform physical maintenance or updates on your new ASCT in terms of the following components?

a. Detection: **Yearly**
b. ASCT Hardware: **Checked yearly**
c. Software: **As updates are made**

22. Was there a need for training your personnel in order to operate the new ASCT?

a. **Yes, Num. of employees trained <u>1</u>. Hours of training per employee <u>20 (2 weeks)</u>.** b. No

23. Was there any change on the number of staff required to operate/maintain the effective signal ASCT operations? (Comparison of staff needed before and after the deployment.)

No

24. Where did expertise come from for your personnel's training, the ASCT installation, operation and the system's maintenance? Please check all that apply. **From marilo**

	In-House	Vendor	Contractor
Employee Training		**Yes**	
ASCT Installation		**Yes**	
ASCT Operation	**Yes**	**Initially**	
ASCT Maintenance		**Yes**	

25. How many crashes of each category were reported after ASCT was implemented?

K (Killed)	A (Disabling Injury)	B (Evident Injury)	C (Possible Injury)	O (No Apparent Injury)

SECTION 4: COST COMPONENTS

29. What is the overall capital cost of purchasing the ASCT (in terms of the software and licenses of the ASCT) for the corridor? If the purchase includes any service of implementation, personnel training or maintenance, please also specify here.

InSync offers complete package; Fusion $30,000/intersection plus $2,000 for installation, cables etc. extension of warranty is available in $900 per intersection (from 2 to 3 years)

30. What is the implementation cost (considering the installation on site, any updates in software and hardware, design needs and contract hours) for the ASCT corridor? Please specify if this cost is partially or totally included in previous cost items.

included in package

31. What was total cost for training your personnel to operate and manage the new ASCT? Please specify if this cost is partially or totally included in previous cost items.

Included in package

32. What is the expected maintenance cost (in terms of hardware, software and personnel) for the ASCT corridor on a yearly basis? Please specify if this cost is partially or totally included in previous cost items.

Included in package

33. Can you specify the costs for the following ASCT components?

a. Firmware $_____________ b. Software $______________

c. Equipment d. Maintenance of Traffic Cost $ **0.00**

e. Design Needs $___________ f. Contract Help/Agency Hours Cost $ **0.00**

SECTION 5: INSTITUTIONAL ISSUES

34. Were there any institutional issues that you had to overcome while implementing and operating the ASCT project? Please categorize and explain those issues below.

a. Organization and Management Institutional Issues:

Local agency raised concerns on maintenance components of the new system (considering funding for maintenance of cameras etc.)

b. Regulatory and Legal Institutional Issues:

c. Human and Facility Resources Institutional Issues:

d. Financial Institutional Issues:

35. Which component (e.g. detection, communication, hardware, software, licensing or other) of your ASCT deployment is the most challenging to maintain and why?

Cameras (due to ethernet extensions and noise issues); however if camera stops detecting historical data are used instead in order to proceed with the adaptive control)

36. What happens to your ASCT operations when some of your detectors fail on the corridor level?

d. ASCT triggers an alarm and notifies operators

e. **ASCT switches to an “off-line” mode by implementing Time-Of-Day plans**

f. Combination of the above. d. Other (please describe): Nothing

29. How is your ASCT performance evaluated?

e. **In-house:**

f. By an independent evaluator, please describe: ______________________________

g. Not applicable—there is no evaluation

h. Combination:

30. If the corridor on which the ASCT operates experiences over saturation, how would you rate the operation of the ASCT in response to these traffic conditions?

f. ASCT prevents or eliminates oversaturation

g. ASCT eliminates or reduces the extent of the periods of oversaturation

h. ASCT adversely affects the traffic conditions during periods of oversaturation

i. Other: Doesn’t prevent or eliminate but helps to manage oversaturation

j. **Oversaturation is very rare on the corridors operated by our ASCT**

31. Based on your opinion and the up-to-date operation, when are ASCT operations proven to be the most effective?

a. Peak periods

b. Off-peak periods

c. Shoulders of peak periods

d. Other, please specify: ______________

32. Has the level of performance of ASCT been sustained since its installation?

a. Yes b. No; why not?

35. What was the public reaction to the ASCT operation? Have you received any complaints about long delays and queues?

Complaints while installing cameras because people thought that those were red light cameras) no complaints on operation

36. How satisfied are you, in general, with your ASCT deployment?

a. **Very satisfied** b. Somewhat satisfied

c. Neutral d. Somewhat dissatisfied

e. Not satisfied at all

8. Are there any other costs or benefits related to ASCT deployment that you would like to report?

a. Benefits: **<u>Additional benefits if radar technology is to be implemented</u>**

b. Costs:

9. Would you consider expanding the ASCT program to any other corridors? Why or why not? If yes, would you use the same technology/firmware or have any suggestions for other systems? If so, why?

2 projects: 1) in State Rd 46: 10 signals and 2) in US92: 22 signals

SECTION 6: SAFETY ISSUES

37. Do you have any anecdotal / qualitative data on safety benefits / disbenefits of the signal improvement along this corridor?

____

____

38. Has there been other changes along this corridor over the analysis period that could affect the safety of this corridor either positively or negatively?

____

____

39. Are you aware of any changes to the crash data reporting procedures over the analysis period? If yes, what are the changes?

____

____

____

____

40. What are your overall reactions to the trends presented in the report?

____

____

41. Is there another comparable corridor in which signal improvements have not been made to which the results of this corridor can be compared to?

____

____

42. Other thoughts / comments for us?

____

____

Thank you for your help in completing this survey. Your responses will help us with evaluating the deployment and benefits of your current ASCT.

If you have any questions regarding the survey, please contact Ria Kontou, email: ekontou@ufl.edu or Liteng Zha, email: litengzha@ufl.edu who work under the direct supervision of the faculty advisor Dr. Yafeng Yin.

B.5 BENEFIT COST ANALYSIS

Table B-7.1 Monetized Saving

Monetized Saving	
Time saving	$3,454,869.30
Fuel Consumption saving	$489,110.08
Air Pollution Saving	$91,940.32
Safety Saving	-$2,565,558.78
Total Saving without Safety	$4,035,919.70
Total Saving	$1,470,360.91
Total Cost	$150,000.00
B/C Ratio without safety	26.90613132
B/C Ratio	9.802406096

Table B-7.2 Monetized Saving with No Emissions

Monetized Saving	
Time saving	$3,454,869.30
Fuel Consumption saving	$489,110.08
Safety Saving	-$2,565,558.78
Total Saving without Safety	$3,943,979.38
Total Saving	$1,378,420.59
Total Cost	$150,000.00
B/C Ratio without safety	26.29319585
B/C Ratio	9.189470627

APPENDIX C: Summary of Panama City Beach Parkway, Bay County

C.1 EXECUTIVE SUMMARY

The objective of this research is to evaluate the implementation of proposed Adaptive Signal Control Technologies (ASCT) traffic operations at several arterial corridors in Florida, before and after the installation of specific ASCT, document the advantages and disadvantages of different approaches and implementations, and provide recommendations for state-wide implementation of ASCT. This Appendix summarizes the before and after field data collected along the Panama City Beach Parkway corridor from SR-79 to Woodlawn Dr., and provides some observations and initial conclusions.

The InSync advanced signal control system was implemented along this corridor in January 2015. Floating car runs were conducted with the UFTI instrumented vehicle to collect vehicle travel times before and after the implementation of InSync, during three time periods (AM Peak, Off-Peak, and PM Peak). In addition, turning movement counts and queue lengths were collected at two critical intersections (Richard Jackson Blvd. and SR-79). Based on these, five performance measures were obtained for the before study period: Link/Route Travel Time, Delay at Intersections, Queue Length (at critical intersections), Queue to Lane Storage Ratio (at critical intersections), and Passenger Car Equivalent (PCE) flows (at critical intersections. For each performance measure, a comparison between the before and after data is conducted and presented in this appendix.

The following were concluded:

- The travel time along Panama City Beach Pkwy, EB direction, decreased by an average of 0.59 min (5.7%), with the highest decrease in the PM Peak (1.30 min, or 12.7%).
- The travel time along Panama City Beach Pkwy in the WB decreased overall by an average of 1.85 min (13.7%). It decreased in the PM Peak by an average of 3.28 min (26.5%), while it increased in the AM Peak by an average of 0.71 min (5.1%). The decrease in PM peak travel time is consistent with a decrease observed in queues on both the Richard Jackson Blvd. and SR-79 intersections during the same time period.
- Apart from the Richard Jackson Blvd. and Powell Adams intersections, all others show a decrease in average delay for the EB. Apart from the Richard Jackson and Pier Park intersections, all others show a decrease in average delay for the WB.
- For the PCB Pkwy and Richard Jackson Blvd. intersection, the average queue length decreased by an average of 0.7, 0.8 and 0.6 vehicles each in the AM, Off Peak and PM respectively.
- For the PCB Pkwy and SR-79 intersection, the average queue length decreased by an average of 0.3, 0.5, and 1.4 vehicles each in the AM, Off Peak and PM respectively.
- The total traffic volume at both critical intersections increased during all three time periods by 135 (1.26% at Richard Jackson Blvd.) and 527 (5.9% at SR-79) vehicles. This can be due to an increase in throughput or demand or both.
- The SR-79 and Pier Park intersections were coordinated separately before the In-Sync implementation. After the implementation, they are coordinated together with

the rest of the corridor. Hence some of the improvements observed may be due to this change.

An interview with Bay County officials indicated that the traffic volume (demand) was heavier this year which made a one-to-one comparison with the previous system in the oversaturated cases less straightforward. Despite this increase, Bay County staff reported that public perception about the new system is positive, which is consistent with our overall quantitative operational assessment.

C.2 CORRIDOR INFORMATION

Panama City Beach is a city in Bay County, Florida. Although it has a small population of about 12,018 as per 2010 census, the traffic in the city is higher during the tourist season. It is a popular destination during spring break and summer seasons. The Panama City Beach Parkway corridor runs parallel to the beach and carries most of this traffic. There are commercial establishments on either side of the corridor between the Pier Park and Richard Jackson intersections. Traffic demands on both the eastbound and westbound directions were observed to be high and nearly equal. The data collection for this project was conducted during the low tourism season.

Figure C.1 provides a schematic of the Panama City Beach Pkwy Corridor. Table C.1 lists the intersections along the corridor. Two intersections (Richard Jackson Blvd. and SR-79) were selected as the critical intersections along the corridor and detailed turning movement and queue counts were collected at these. Figure C.2 provides the lane configuration of these two critical intersections. For the EB direction travel time was measured as our instrumented vehicle travelled using the fly-over, whereas for the WB direction the vehicle travelled through the intersections at Woodlawn Dr. and Thomas Dr. Data collection was conducted during three time periods: 7 - 9 am, 11 am – 1 pm, and 4 - 6 pm.

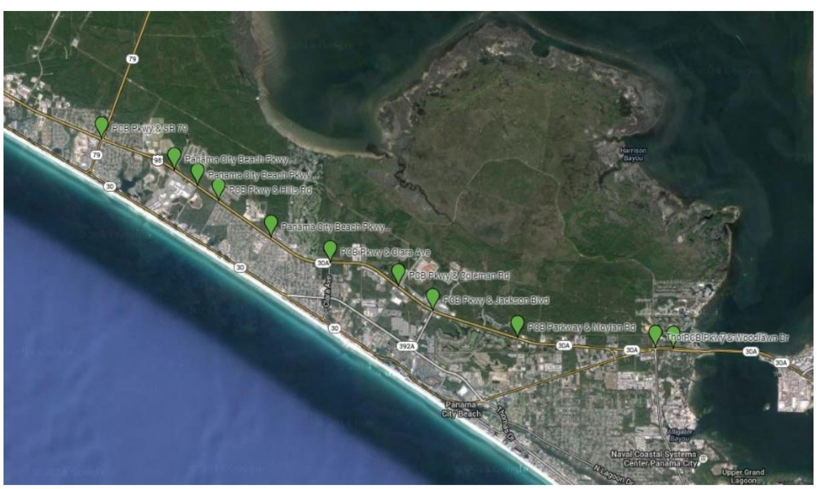
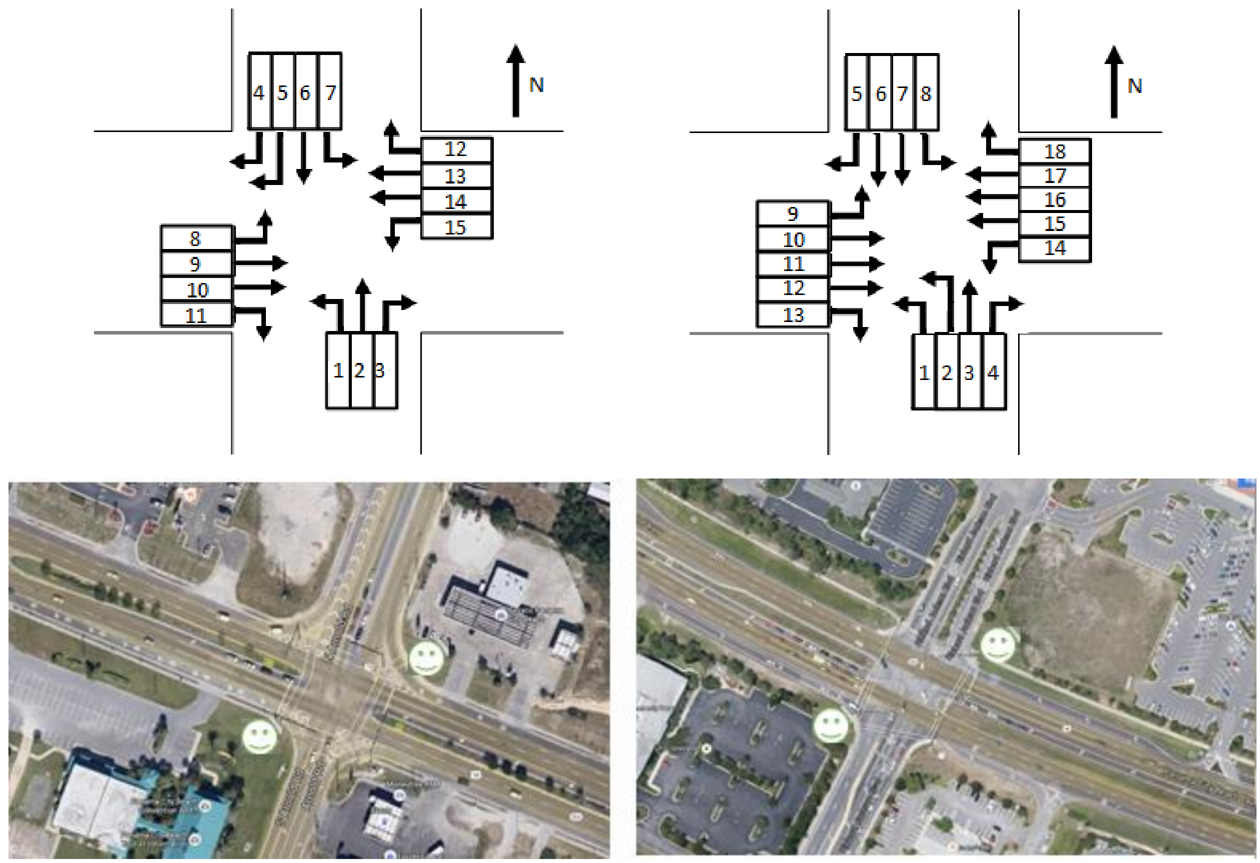


Figure C-1 Schematic of the Panama City Beach Pkwy Corridor

Table C-1 Intersections along the PCB Pkwy Corridor

	Intersection	**Comments**	**No. of Unsignalized Intersections**	**Distance**
1	**SR-79**		**-**	**-**
2	Pier Park Dr		10	1.1 miles
3	Powell Adams Rd		1	0.4 miles
4	Nautilus St		4	1.2 miles
5	Clara Ave	T-Intersection	10	0.9 mile
6	Alf Coleman Rd		8	1 mile
7	**Richard Jackson Blvd**		**4**	**0.6 mile**
8	Moylan Rd	T-Intersection	5	1.2 miles
9	Thomas Dr	Off-ramp	7	1.9 miles
10	Woodlawn Dr	Fly-over	0	0.2 miles



(a) SR-79. (b) Richard Jackson

Figure C-2 Schematic and Overview of Critical Intersections

C.3 PERFORMANCE MEASURES

Five performance measures were evaluated: Link/Route Travel Time, Delay at Intersections, Queue Length (critical intersections), Queue-to-Lane Storage Ratio (critical intersections), and PCE Flows (critical intersections). For each performance measure, a comparison between the before and after data is conducted and the results of the differences ("after data" – "before data") are presented.

C.3.1 ROUTE TRAVEL TIME

The average travel time (min) along the route was measured using a floating car. The travel times recorded are reported in the tables below. During the PM peak of the day of data collection it was raining. Hence, additional data were obtained from District 3 from Bluetooth devices available to FDOT, and these were used to obtain the PM travel time (data were obtained for Sep 29, 2015). For comparison, the travel time during rain has been included in parentheses.

Before Study (Oct.22 & Oct.23, 2014)

Table C-1.1 Route Travel Time (min)

Route TT (min)	AM	Off Peak	PM	Average
Panama City Beach Pkwy EB	11.207	10.107	11.537	**10.95**
Panama City Beach Pkwy WB	13.317	17.003	15.672	**15.331**

After Study (Feb. 3 & Feb. 4, 2015)

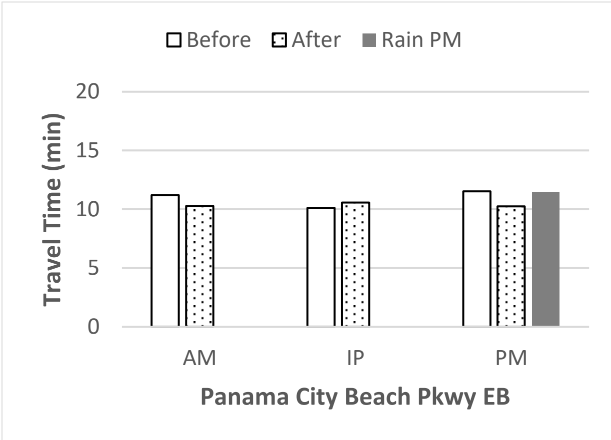
Table C-1.2 Route Travel Time (min)

Route TT (min)	AM	Off Peak	PM	Average
Panama City Beach Pkwy EB	10.273	10.572	10.234 (Rain 11.5)	**10.360**
Panama City Beach Pkwy WB	14.026	14.018	12.391 (Rain 15.5)	**13.478**

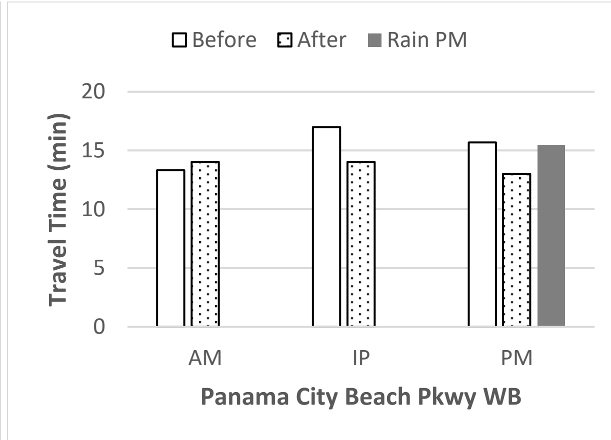
Comparisons of Before and After Travel Times

Table C-1.3 Change in Travel time in Min and as a Percentage (After – Before)

Route TT (min)	AM	Off Peak	PM	Average
Panama City Beach Pkwy EB	-0.934 (-9.1%)	0.465 (4.4%)	-1.303 (-12.7%)	**-0.590 (-5.7%)**
Panama City Beach Pkwy WB	0.709 (5.1%)	-2.985 (-21.3%)	-3.281 (-26.5%)	**-1.852 (-13.7%)**



(a) Panama City Beach Pkwy EB



(b) Panama City Beach Pkwy WB

Figure C-1.1 Travel Times Along Panama City Beach Pkwy

C.3.1.4. Discussion

The following can be concluded from the comparison of travel times:

- The travel time along Panama City Beach Pkwy in the EB decreased by an average of 0.59 min (5.7%). The highest decrease occurred during the PM Peak with a decrease of 1.30 min (12.7%).
- The travel time along Panama City Beach Pkwy in the WB decreased overall by an average of 1.85 min (13.7%). It decreased in the PM Peak by an average of 3.28 min (26.5%).
- WB travel times are higher than those in the EB for both the before and after data collection. One of the reasons for this is that the EB direction includes the fly-over above Thomas Drive, while the WB direction incurs additional delay as it goes through the signalized intersection at Thomas Dr.
- As expected, travel times during rainy conditions are higher (by 12% to 19%) than during fair weather conditions.
- Bluetooth travel times were compared with travel times from the vehicle for the AM Peak (data were obtained on different days), and Bluetooth travel times were lower than those measured by our instrumented vehicle by 0.3 min and 1.7 min for the EB and WB directions respectively.
- The travel times during rainy conditions were measured using the instrumented vehicle, while the PM Peak travel times for the after conditions were obtained through Bluetooth devices. Given that Bluetooth provided lower travel time estimate for the data analyzed, the differences due to rain may not be as pronounced when comparing travel times obtained with the same data collection method.

C.3.2 DELAY

Delay (sec) at each intersection along the corridor was also obtained using the floating car measurements. As indicated earlier, the after PM peak data collection was undertaken during rainy conditions. However, the Bluetooth data cannot provide delay by intersection. Therefore, the delays reported here for the PM Peak (after data) represent rainy conditions.

Before Study (Oct. 22 & Oct.23, 2014)

Table C-2.1 Delay (sec) for each Intersection Through Movement Along the EB Direction

Delay by Intersection (sec)	AM	Off Peak	PM
SR79	16	47	50
Pier Park	17	21	34
Powell Adams	5	3	2
Nautilus St	11	0	6
Clara Ave	4	0	1
Alf Coleman	31	19	24
Richard Jackson	32	13	17
Moylan	0	1	0

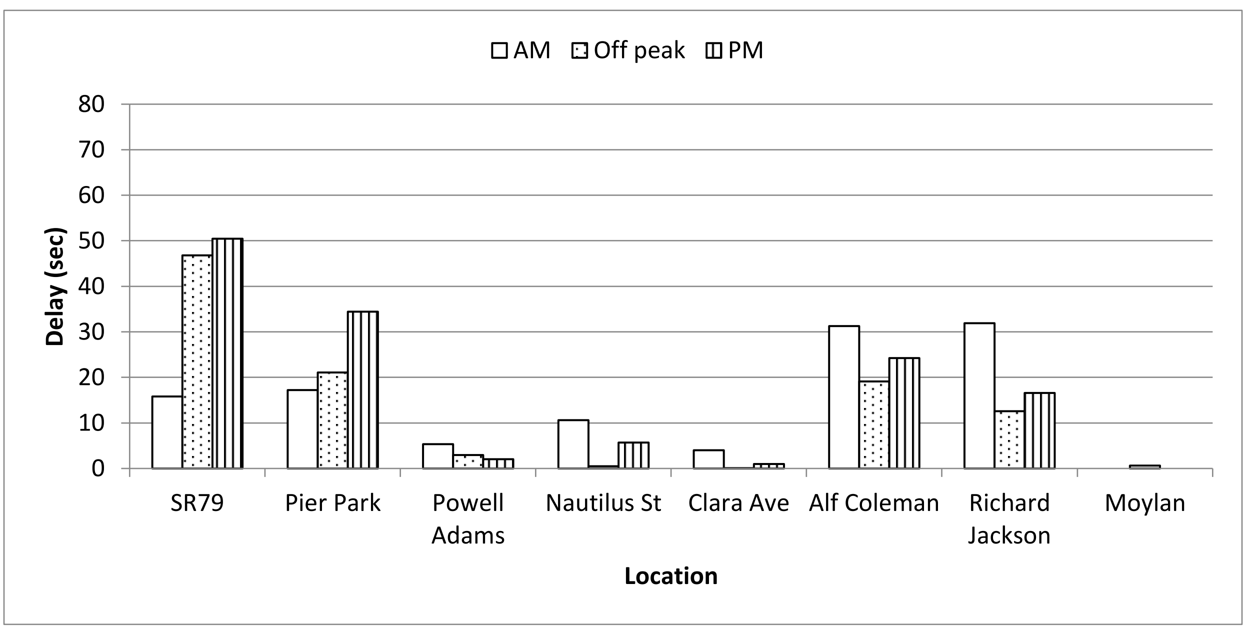


Figure C-2.1 Delay (sec) for each Intersection Through Movement Along the EB Direction

Table C-2.2 Delay (sec) for each Intersection Through Movement Along the WB Direction

Delay by Intersection (sec)	AM	Off Peak	PM
Woodlawn	6	8	5
Thomas Dr	60	29	72
Moylan	0	3	4
Richard Jackson	8	21	26
Alf Coleman	2	13	28
Clara Ave	0	1	2
Nautilus St	1	3	22
Powell Adams	1	24	18
Pier Park	1	3	1
SR79	25	48	52

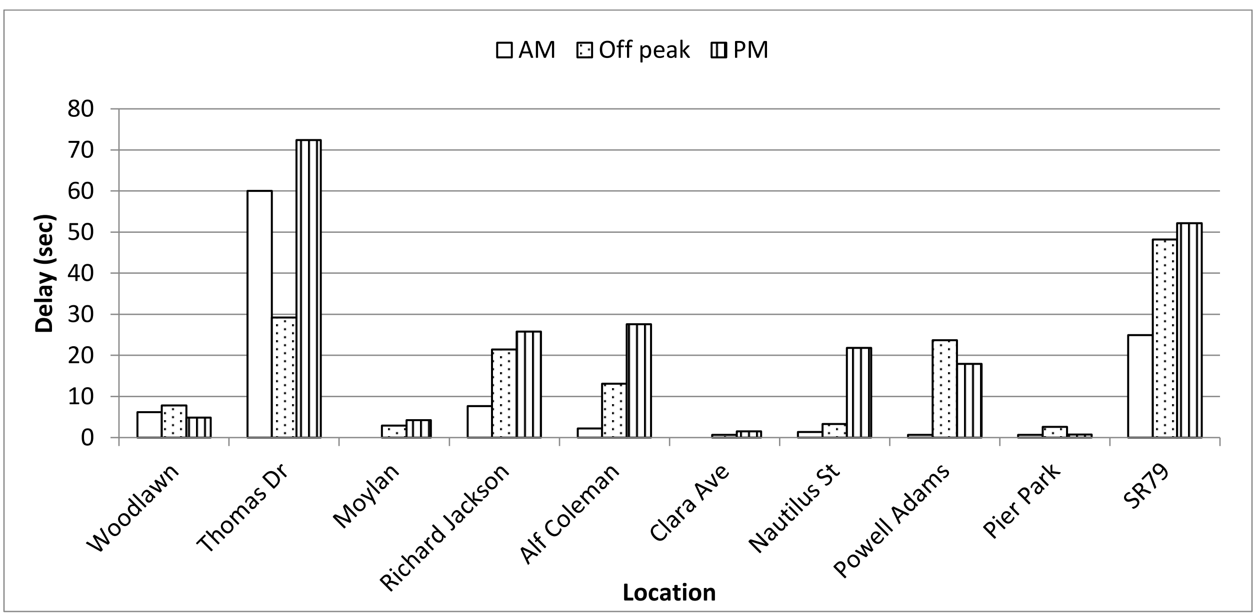


Figure C-2.2 Delay (sec) for each Intersection Through Movement Along the WB Direction

After Study (Feb. 3 & Feb. 4, 2015)

Table C-2.3 Delay (sec) for each Intersection Through Movement Along the EB Direction

Delay at Intersections (sec)	AM Peak	Off Peak	PM Peak (Rain)
SR79	40	19	19
Pier Park	5	2	8
Powell Adams	4	15	17
Nautilus St	0	13	1
Clara Ave	1	4	0
Alf Coleman	4	11	12
Richard Jackson	22	22	70
Moylan	0	1	5

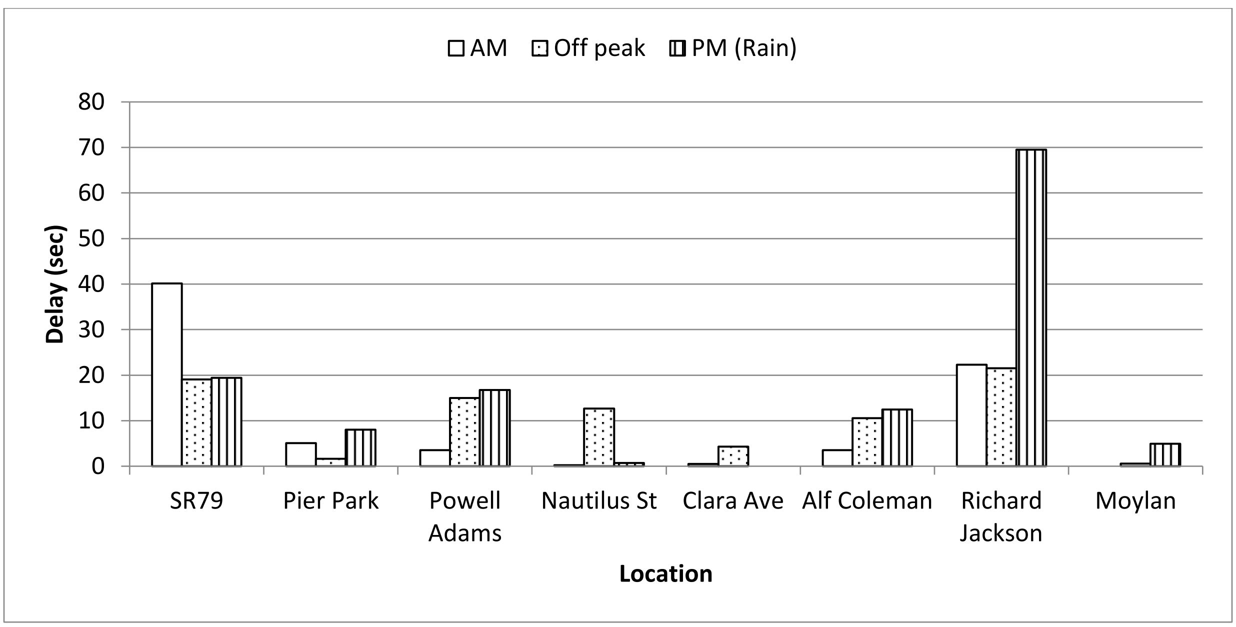


Figure C-2.3 Delay (sec) for each Intersection Through Movement Along the EB Direction

Table C-2.4 Delay (sec) for each Intersection Through Movement Along the WB Direction

Delay at Intersections (sec)	AM Peak	Off Peak	PM Peak (Rain)
Woodlawn	4	5	12
Thomas Dr	23	35	17
Moylan	0	0	0
Richard Jackson	63	29	16
Alf Coleman	36	1	7
Clara Ave	0	4	0
Nautilus St	2	1	8
Powell Adams	0	8	9
Pier Park	1	38	1
SR79	11	17	17

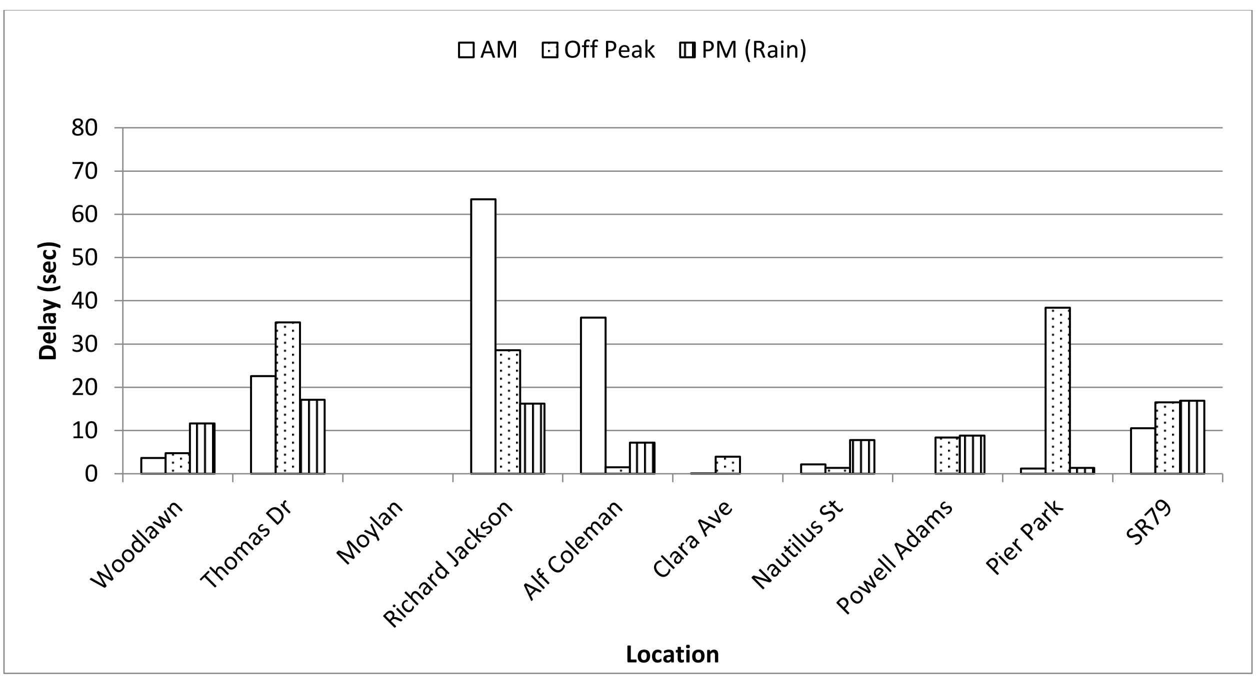


Figure C-2.4 Delay (sec) for each Intersection Through Movement Along the WB Direction

Comparisons of Before and After Intersection Delay Times

The differences in delay between the before and after study periods are shown in Table C-2.5 and Table C-2.6. The tables are color-coded as follows: green shows significant improvement, yellow shows modest change, and red shows significant deterioration in delay.

Table C-2.5 Difference in Delay (sec) for each Intersection Through Movement Along the EB Direction

Delay (sec)	AM Peak	Off Peak	PM Peak	Average
SR79	24.0	-28.0	-31.0	-11.0
Pier Park	-12.0	-19.0	-26.0	-19.0
Powell Adams	-2.0	12.0	15.0	8.0
Nautilus St	-10.0	12.0	-5.0	-1.0
Clara Ave	-3.0	4.0	-1.0	0.0
Alf Coleman	-28.0	-9.0	-12.0	-16.0
Richard Jackson	-10.0	9.0	53.0	17.0
Moylan	0.0	0.0	5.0	2.0

Table C-2.6 Difference in Delay (sec) for each Intersection Through Movement Along the WB Direction

Delay (sec)	AM Peak	Off Peak	PM Peak	Average
Woodlawn	-3.0	-3.0	7.0	0.0
Thomas Dr	-37.0	6.0	-55.0	-29.0
Moylan	0.0	-3.0	-4.0	-2.0
Richard Jackson	56.0	7.0	-10.0	18.0
Alf Coleman	34.0	-12.0	-20.0	1.0
Clara Ave	0.0	3.0	-2.0	1.0
Nautilus St	1.0	-2.0	-14.0	-5.0
Powell Adams	-1.0	-15.0	-9.0	-8.0
Pier Park	1.0	36.0	1.0	12.0
SR79	-14.0	-32.0	-35.0	-27.0

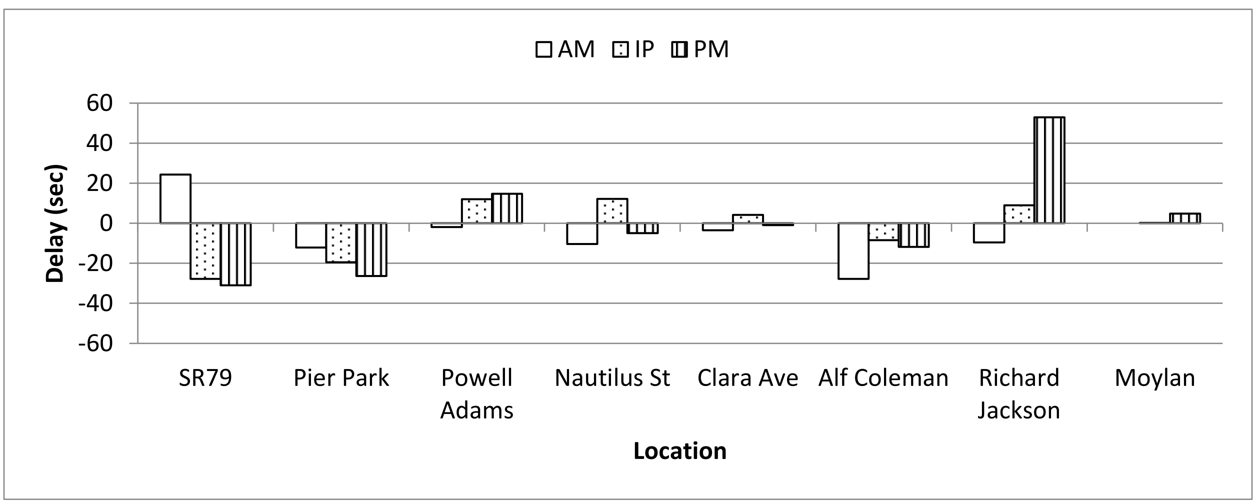


Figure C-2.5 Difference in Delay (sec) for each Intersection Through Movement Along the EB Direction

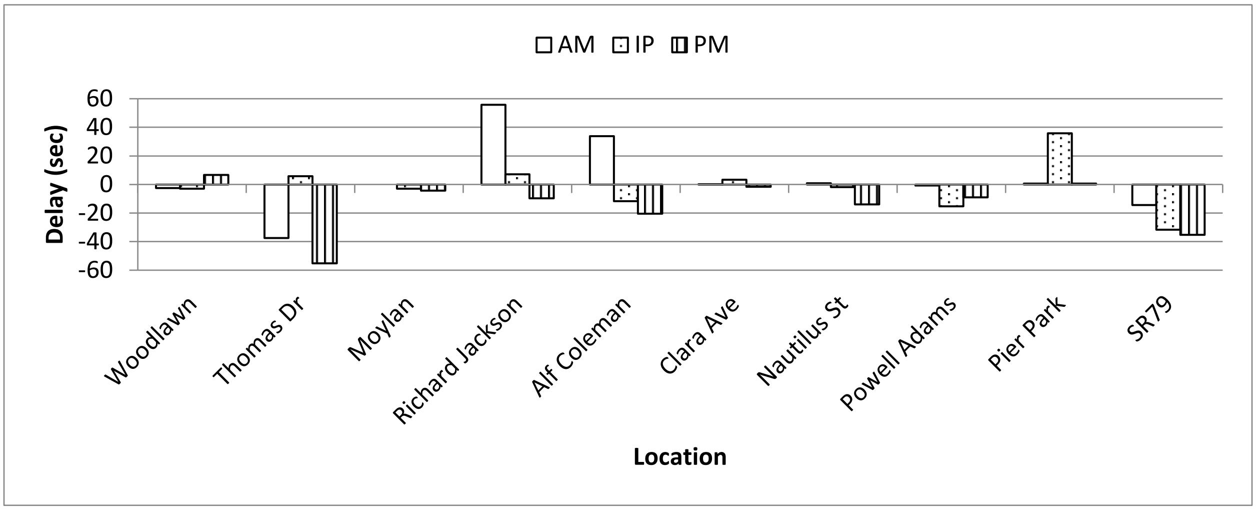


Figure C-2.6 Difference in Delay (sec) for each Intersection Through Movement Along the WB Direction

Discussion

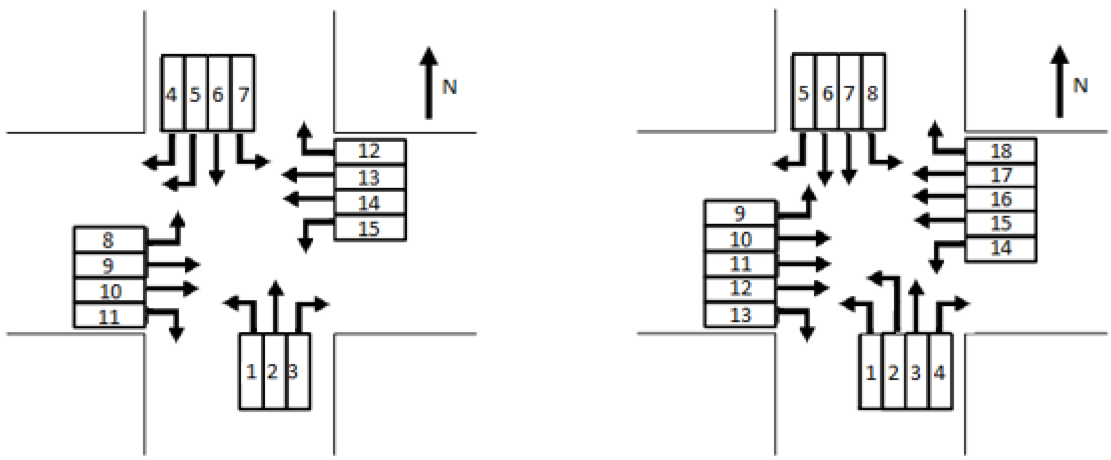
The following can be concluded from the comparison of delays:

- Apart from the Richard Jackson Blvd. and Powell Adams, all the other intersections show a decrease in average delay for the EB direction. Delay increased at the Richard Jackson intersection by 53 s for the EB direction. This is likely due to the rain on the day of data collection. While travel time for the corridor under good weather conditions was obtained from Bluetooth data, it was not possible to obtain intersection delays.
- Apart from the Richard Jackson Blvd. and Pier Park Dr, all the other intersections show a decrease in average delay for the WB direction.
- During the before data collection, SR 79 and Pier Park were coordinated separately, while during the after data collection they were coordinated as part of the entire corridor. This modification alone may have improved overall travel time.

C.3.3 QUEUE LENGTH

Queue length (number of vehicles/lane) is presented by movement and by time period. This measure is used to evaluate oversaturated conditions at the critical intersections along the study corridors. Note that the queue length reported here is the observed maximum number of vehicles queued during each cycle, and does not represent the total number of vehicles that may have stopped during the cycle. During some time periods, because of cycle failure, vehicles need to stop multiple times before passing through the intersection.

Figure C-3.1 presents the schematic of the lane configurations at the two critical intersections. Queue length is reported for each of these lanes.



(a) SR-79. (b) Richard Jackson

Figure C-3.1 Lane Configuration of Critical Intersections

Before Study (Oct. 22 & Oct.23, 2014)

Table C-3.1 Average Queue Length by Lane (#vehs/lane) at PCB Pkwy and Richard Jackson

Time Period		NB				SB				EB					WB				
		Left		**Through**	**Right**	**Left**	**Through**		**Right**	**Left**	**Through**			**Right**	**Left**	**Through**			**Right**
Lane Number		**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**	**13**	**14**	**15**	**16**	**17**	**18**
AM Peak	1	3.2	3.4	9.8	3.2	15.8	6.8	4	1.3	6.8	7	7.5	1.3	1.8	8	18	18.4	9.6	6.2
	2	5.4	6.2	3.6	2.4	4.6	3.8	1	1	2	7.8	6.4	0.6	1.2	5	20	20	7.8	0.2
	3	5.6	6.4	2.6	2	4.8	2.3	1	1.8	3.3	7	5.5	0.8	0.8	4.6	14	14.6	4.2	1.8
	4	4.6	3.2	4	2	4	2.5	0.8	0.5	2.8	9.8	8.3	2	0	12.8	12.8	12.8	6.8	2.2
	Average	**4.7**	**4.8**	**5**	**2.4**	**7.3**	**3.8**	**1.7**	**1.1**	**3.7**	**7.9**	**6.9**	**1.2**	**0.9**	**7.6**	**16.2**	**16.5**	**7.1**	**2.6**
Off Peak	1	3	3.6	3.4	1	11.6	6.2	4.4	1.2	4.4	7.6	4.8	0.8	0.4	4.6	10.8	9	4.8	0.8
	2	5.8	6.8	4.6	1.4	8.8	5	2.8	1.4	4	5.6	4.4	0.8	1	4.8	13.4	11.2	7.2	2.2
	3	6	6.8	5.6	1.2	5.8	4.2	1.6	0.4	2.6	6.8	6.6	1.6	1.6	4.8	15	12.2	6	1
	4	4.4	5	4.2	1.8	5.8	3.8	2.8	1.3	4	9	9.8	0.5	0.5	5.8	15.6	12.8	7.4	1.2
	Average	**4.8**	**5.6**	**4.5**	**1.4**	**8**	**4.8**	**2.9**	**1.1**	**3.8**	**7.3**	**6.4**	**0.9**	**0.9**	**5**	**13.7**	**11.3**	**6.4**	**1.3**
PM Peak	1	5.5	6.3	5	2	6	4.8	1.3	1.3	8.3	5.3	7.3	1.3	0.8	5.8	12.3	10.5	3.8	1
	2	5.5	6.8	4.5	3.3	7.5	4.8	2.3	0.8	6.5	7.5	8.3	1	0.8	7	16	12.5	6.5	0.5

Time Period		NB				SB				EB					WB				
		Left		Through	Right	Left	Through		Right	Left	Through			Right	Left	Through			Right
	3	4.8	5.8	6	1.8	8	1.8	1.8	1.5	3.5	4.3	4.5	0.5	0.3	10.3	16.5	16	3.3	1
	4	3.3	2.8	2.5	3.3	4.5	3.8	2	0.5	3.5	8.5	6.5	1.5	0.5	6	13.3	12.8	4.3	0.5
	Average	**4.8**	**5.4**	**4.5**	**2.6**	**6.5**	**3.8**	**1.8**	**1**	**5.4**	**6.4**	**6.6**	**1.1**	**0.6**	**7.3**	**14.5**	**12.9**	**4.4**	**0.8**

Table C-3.2 Average Queue Length (#vehs/lane) at PCB Pkwy and Richard Jackson

Time	NB			SB			EB			WB		
Period	**Left**	**Through**	**Right**	**Left**	**Through**	**Right**	**Left**	**Through**	**Right**	**Left**	**Through**	**Right**
AM Peak	4.8	5	2.4	7.3	2.8	1.1	3.7	5.3	0.9	7.6	13.3	2.6
Off Peak	5.2	4.5	1.4	8	3.8	1.1	3.8	4.9	0.9	5	10.5	1.3
PM Peak	5.1	4.5	2.6	6.5	2.8	1	5.4	4.7	0.6	7.3	10.6	0.8

Table C-3.3 Average Queue Length by Lane (#vehs/lane) at PCB Pkwy and SR-79

Time Period		NB			SB				EB				WB			
		Left	Through	Right	Left		Through	Right	Left	Through		Right	Left	Through		Right
Lane Number		**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**	**13**	**14**	**15**
AM peak	1	2.1	2	0.6	4.4	4.4	2	0.6	2	11.6	12	1	3.7	6.1	6.3	0

Time Period		NB			SB				EB				WB			
		Left	Through	Right	Left		Through	Right	Left	Through		Right	Left	Through		Right
	2	1.7	1	0.2	4.8	4.8	1.4	0.6	3.2	9.8	10.4	1	2.5	8.5	8	0.3
	3	2	1.5	1	3.4	3.8	1.2	0.2	3.4	8.8	10.4	0.2	2.9	11.9	10.3	1.6
	4	2.5	1.5	0.7	4.2	4	1.3	0.3	4.2	10.5	10.2	1	2.6	11.3	11.7	0.1
	Average	**2.1**	**1.5**	**0.6**	**4.2**	**4.3**	**1.5**	**0.4**	**3.2**	**10.2**	**10.7**	**0.8**	**2.9**	**9.4**	**9.1**	**0.5**
Off peak	1	3.6	0.6	0.7	1.4	2.9	1.6	0.7	3.4	7.6	11.3	1	4.3	7.3	8	0.6
	2	5	3	1.6	5.2	6	2	0.2	4.4	10.6	13.8	1	5.8	10.4	8.2	0.4
	3	2.6	1.2	1.4	2	3.6	3	1.4	3.4	8.2	10.8	1.2	5.6	6	8.4	0.2
	4	4.5	1.7	0.3	2.3	3.8	1.8	0.3	4.2	10.7	15	1	6.2	10	11.3	0.5
	Average	**3.9**	**1.6**	**1**	**2.7**	**4.1**	**2.1**	**0.7**	**3.8**	**9.3**	**12.7**	**1.1**	**5.5**	**8.4**	**9**	**0.4**
PM Peak	1	4.3	4.8	1	4.8	5.8	1.8	0.3	6.5	13.8	18.3	0.8	7.8	11.3	11.3	0.8
	2	2.3	2.7	0.7	4	5.4	2.4	0.2	6.8	12.3	13.7	0.8	7.3	12.5	12.3	0.7
	3	3.2	4	1.4	6.8	6.8	0.5	0	7.5	15	16	0.8	6.4	13.6	14	1.8
	4	2.8	1.2	0.3	2.3	2.8	1.5	0.3	5.8	11	12.5	0.5	5	10.2	11.2	0.8
	Average	**3.2**	**3.1**	**0.9**	**4.5**	**5.2**	**1.5**	**0.2**	**6.7**	**13**	**15.1**	**0.7**	**6.6**	**11.9**	**12.2**	**1**

Table C-3.4 Average Queue Length (#vehs/lane) at PCB Pkwy and SR-79

Time	**NB**			**SB**			**EB**			**WB**		
Period	**Left**	**Through**	**Right**	**Left**	**Through**	**Right**	**Left**	**Through**	**Right**	**Left**	**Through**	**Right**
AM Peak	2.1	1.5	0.6	4.3	1.5	0.4	3.2	10.4	0.8	2.9	9.3	0.5
Off Peak	3.9	1.6	1	3.4	2.1	0.7	3.8	11	1.1	5.5	8.7	0.4
PM Peak	3.2	3.1	0.9	4.8	1.5	0.2	6.7	14.1	0.7	6.6	12.1	1

After Study (Feb. 3 & Feb.4, 2015 for AM and OFF PEAK and Sep 29, 2015 for PM)

Table C-3.5 Average Queue Length by Lane (#vehs/lane) at PCB Pkwy and Richard Jackson

Time Period		NB				SB				EB					WB				
		Left		Through	Right	Left	Through		Right	Left	Through			Right	Left	Through			Right
Lane Number		**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**	**13**	**14**	**15**	**16**	**17**	**18**
AM Peak	1	1.6	3.0	12.6	1.8	11.6	4.8	2.0	1.2	8.8	4.8	5.8	0.8	0.0	4.8	10.0	10.2	7.6	2.0
	2	2.4	4.4	4.6	2.4	10.8	5.0	3.0	2.4	3.2	15.4	13.2	3.6	0.0	8.4	8.4	9.2	9.2	1.0
	3	4.6	5.8	3.8	0.8	3.8	3.2	3.0	0.6	3.0	5.2	4.8	1.4	0.0	9.8	16.0	14.8	16.2	0.0
	4	4.0	4.0	4.6	0.8	4.0	2.2	2.2	0.6	2.8	6.2	6.6	1.8	0.0	4.0	7.0	4.8	4.6	0.4
	Average	**3.2**	**4.3**	**6.4**	**1.5**	**7.6**	**3.8**	**2.6**	**1.2**	**4.5**	**7.9**	**7.6**	**1.9**	**0.0**	**6.8**	**10.4**	**9.8**	**9.4**	**0.9**
Off Peak	1	4.4	5.8	5.0	2.6	4.4	5.6	1.8	1.0	3.6	7.0	6.4	0.6	0.0	3.0	9.4	7.8	2.0	1.0
	2	4.6	6.8	7.2	0.0	5.4	3.8	1.6	1.0	3.4	6.4	7.4	1.0	0.0	5.2	8.2	6.4	2.8	1.6
	3	6.8	8.2	6.0	2.2	5.6	5.2	2.6	1.0	3.6	7.2	5.8	2.6	0.0	4.6	7.8	6.4	1.8	0.4
	4	4.6	5.8	6.0	1.8	5.0	5.0	2.8	3.0	4.0	4.4	4.2	4.2	0.0	6.6	8.8	10.2	2.4	0.8
	Average	**5.1**	**6.7**	**6.1**	**1.7**	**5.1**	**4.9**	**2.2**	**1.5**	**3.7**	**6.3**	**6.0**	**2.1**	**0.0**	**4.9**	**8.6**	**7.7**	**2.3**	**1.0**
PM Peak	1	3.8	4.4	3.0	1.8	6.8	2.4	2.4	1.2	2.4	9.8	9.6	2.6	0.0	5.2	11.0	10.0	4.0	0.2
	2	3.2	4.2	3.8	3.8	6.0	3.8	2.4	1.0	0.8	7.8	6.6	3.0	0.0	5.2	8.0	8.6	3.2	0.0
	3	4.6	4.4	3.0	5.0	7.4	4.0	2.6	2.0	3.8	8.2	8.4	2.0	0.0	4.6	9.2	8.8	4.8	0.0

Time Period		NB				SB				EB					WB				
		Left		Through	Right	Left	Through		Right	Left	Through			Right	Left	Through			Right
	4	4.0	4.6	2.4	1.8	6.8	3.2	1.2	2.0	1.6	11.6	11.4	3.4	0.0	5.6	7.6	8.4	4.6	0.0
	Average	**3.9**	**4.4**	**3.1**	**3.1**	**6.8**	**3.4**	**2.2**	**1.6**	**2.2**	**9.4**	**9.0**	**2.8**	**0.0**	**5.2**	**9.0**	**9.0**	**4.2**	**0.1**

Table C-3.6 Average Queue Length (#vehs/lane) at PCB Pkwy and Richard Jackson

Time	NB			SB			EB			WB		
Period	Left	Through	Right	Left	Through	Right	Left	Through	Right	Left	Through	Right
AM Peak	3.8	6.4	1.5	7.6	3.2	1.2	4.5	5.8	0.0	6.8	9.9	0.9
Off Peak	5.9	6.1	1.7	5.1	3.6	1.5	3.7	4.8	0.0	4.9	6.2	1.0
PM Peak	4.2	3.1	3.1	6.8	2.8	1.6	2.2	7.1	0.0	5.2	7.4	0.7

Table C-3.7 Average Queue Length by Lane (#vehs/lane) at PCB Pkwy and SR-79

Time Period		NB			SB				EB				WB			
		Left	Through	Right	Left		Through	Right	Left	Through		Right	Left	Through		Right
Lane Number		**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**	**13**	**14**	**15**
AM peak	1	2.4	3.2	0.4	6.2	5.6	2.2	0.4	3.0	12.6	12.4	0.0	3.0	6.0	6.6	0.0
	2	2.6	1.6	0.4	7.2	5.6	2.8	0.0	2.8	9.4	9.6	0.4	3.6	2.8	3.8	0.0

Time Period		NB			SB				EB				WB			
		Left	Through	Right	Left		Through	Right	Left	Through		Right	Left	Through		Right
Lane Number		**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**	**13**	**14**	**15**
	3	4.2	2.2	0.8	4.0	4.4	1.8	0.0	4.0	10.4	10.0	0.2	3.8	4.2	4.8	0.0
	4	3.2	1.2	0.8	4.8	4.4	2.0	0.0	3.4	9.0	8.8	0.4	4.0	6.6	6.8	0.0
	Average	**3.1**	**2.1**	**0.6**	**5.6**	**5.0**	**2.2**	**0.1**	**3.3**	**10.4**	**10.2**	**0.3**	**3.6**	**4.9**	**5.5**	**0.0**
Off peak	1	2.2	2.0	0.0	2.0	3.6	1.4	0.0	2.6	11.4	15.2	0.8	6.0	4.4	5.4	0.0
	2	3.8	2.6	0.0	3.8	4.2	1.4	0.0	3.4	10.6	11.8	1.0	4.0	4.0	5.2	0.0
	3	0.0	0.8	6.0	4.0	4.4	1.8	0.0	3.6	11.8	15.4	0.6	7.2	4.0	5.4	0.0
	4	0.0	1.4	4.8	2.6	3.6	0.8	0.0	3.0	10.6	12.4	2.2	6.0	4.0	4.4	0.0
	Average	**1.5**	**1.7**	**2.7**	**3.1**	**4.0**	**1.4**	**0.0**	**3.2**	**11.1**	**13.7**	**1.2**	**5.8**	**4.1**	**5.1**	**0.0**
PM Peak	1	2.6	3.0	0.6	6.2	5.4	0.2	0.0	5.6	9.0	9.2	0.6	5.6	5.2	7.8	0.0
	2	4.2	2.6	0.8	4.2	5.0	3.6	0.0	4.8	15.0	14.0	1.6	6.4	3.6	4.4	0.0
	3	3.2	2.8	0.4	3.2	4.6	1.6	0.0	5.6	11.0	9.6	0.4	6.8	3.8	4.0	0.0
	4	2.2	3.8	0.0	2.4	3.4	1.2	0.0	2.8	9.6	26.6	0.0	6.4	5.8	5.8	0.0
	Average	**3.1**	**3.1**	**0.5**	**4.0**	**4.6**	**1.7**	**0.0**	**4.7**	**11.2**	**14.9**	**0.7**	**6.3**	**4.6**	**5.5**	**0.0**

Table C-3.8 Average Queue Length (#vehs/lane) at PCB Pkwy and SR-79.

Time	NB			SB			EB			WB		
Period	Left	Through	Right	Left	Through	Right	Left	Through	Right	Left	Through	Right
AM Peak	3.1	2.1	0.6	5.3	2.2	0.1	3.3	10.3	0.3	3.6	5.3	0.0
Off Peak	1.5	1.7	2.7	3.6	1.4	0.0	3.2	12.4	1.2	5.8	4.6	0.0
PM Peak	3.1	3.1	0.5	4.3	1.7	0.0	4.7	13.1	0.7	6.3	5.0	0.0

Comparisons of Before and After Queue Lengths for Critical Intersections

The differences in queue length between the before and after measurements are shown in Table C-3.9 to Table C-9.12. The tables are color-coded as follows: green shows significant improvement, yellow shows modest change, and red shows significant increase in queue length.

Table C-3.9 Difference in Average Queue Length by Lane (#vehs/lane) at PCB Pkwy and Richard Jackson

Time Period	NB				SB				EB (Main)					WB (Main)					Average
	Left		Through	Right	Left	Through		Right	Left	Through			Right	Left	Through			Right	
Lane Number	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	
AM	-1.6	-0.5	1.4	-1.0	0.3	0.0	0.9	0.1	0.8	0.0	0.7	0.8	-0.9	-0.9	-5.9	-6.7	2.3	-1.8	-0.7
Off Peak	0.3	1.1	1.6	0.3	-2.9	0.1	-0.7	0.4	-0.1	-1.0	-0.4	1.2	-0.9	-0.2	-5.2	-3.6	-4.1	-0.4	-0.8
PM	-0.9	-1.0	-1.5	0.5	0.3	-0.4	0.3	0.6	-3.3	3.0	2.4	1.7	-0.6	-2.1	-5.6	-4.0	-0.3	-0.7	-0.6

Table C-3.10 Difference in Average Queue Length (#vehs/lane) at PCB Pkwy and Richard Jackson

Time Period	NB			SB			EB (Main)			WB (Main)		
	Left	Through	Right	Left	Through	Right	Left	Through	Right	Left	Through	Right
AM	-1.0	1.4	-0.9	0.3	0.4	0.1	0.8	0.5	-0.9	-0.8	-3.4	-1.7
Off Peak	0.7	1.6	0.3	-2.9	-0.2	0.4	-0.1	-0.1	-0.9	-0.1	-4.3	-0.3
PM	-0.9	-1.4	0.5	0.3	0.0	0.6	-3.2	2.4	-0.6	-2.1	-3.2	-0.1

Table C-3.11 Difference in Average Queue Length by Lane (#vehs/lane) at PCB Pkwy and SR-79.

Time	**NB**			**SB**				**EB (Main)**				**WB (Main)**				**Average**
Period	**Left**	**Through**	**Right**	**Left**		**Through**	**Right**	**Left**	**Through**		**Right**	**Left**	**Through**		**Right**	
Lane Number	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**	**13**	**14**	**15**	
AM	1.0	0.6	0.0	1.4	0.8	0.7	-0.3	0.1	0.2	-0.5	-0.6	0.7	-4.5	-3.6	-0.5	-0.3
Off Peak	-2.4	0.1	1.7	0.4	-0.1	-0.8	-0.7	-0.7	1.8	1.0	0.1	0.3	-4.3	-3.9	-0.4	-0.5
PM	-0.1	-0.1	-0.4	-0.5	-0.6	0.1	-0.2	-2.0	-1.9	-0.3	-0.1	-0.3	-7.3	-6.7	-1.0	-1.4

Table C-3.12 Difference in Average Queue Length (#vehs/lane) at PCB Pkwy and SR-79.

Time	**NB**			**SB**			**EB (Main)**			**WB (Main)**		
Period	**Left**	**Through**	**Right**	**Left**	**Through**	**Right**	**Left**	**Through**	**Right**	**Left**	**Through**	**Right**
AM	1.0	0.6	0.0	1.0	0.7	-0.3	0.1	-0.1	-0.5	0.7	-4.0	-0.5
Off Peak	-2.4	0.1	1.7	0.2	-0.7	-0.7	-0.6	1.4	0.1	0.3	-4.1	-0.4
PM	-0.1	0.0	-0.4	-0.5	0.2	-0.2	-2.0	-1.0	0.0	-0.3	-7.1	-1.0

Discussion

The following can be concluded from the comparison of queue length:

- For the PCB Pkwy and Richard Jackson Blvd. intersection, the average queue length decreased by an average of 0.7, 0.8 and 0.6 vehicles each in the AM, Off Peak and PM respectively.
- For the PCB Pkwy and SR-79 intersection, the average queue length decreased by an average of 0.3, 0.5, and 1.4 vehicles each in the AM, Off Peak and PM respectively.

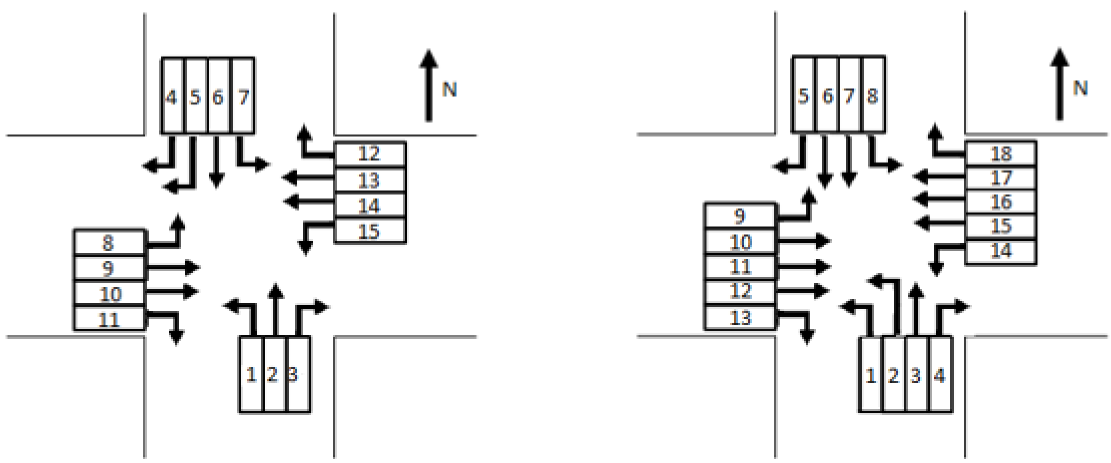
C.3.4 QUEUE TO LANE STORAGE RATIO

In addition to queue length, it is important to assess any impact to adjacent lanes or to upstream facilities. The queue to link/lane ratio is used to establish the likelihood of spillback, which is presented in this section by movement and by time period.

The following assumptions are used:

- The storage capacity is estimated as the maximum number of vehicles in the lane.
- The queue to link/lane storage ratio is estimated as 1 if the observer reported "spillback", and as 0.8 if reported as the maximum number of vehicles in the sight distance.
- Queue to lane storage ratios over 80% are highlighted in yellow in the tables of this section, as they represent conditions with a high probability for spillback.

Note: For Richard Jackson PM Peak, data collection was not possible because of the rain. Instead data from a different day was collected through video recording.



(a) SR-79. (b) Richard Jackson

Figure C-4.1 Lane Configuration of Critical Intersections

Before Study (Oct. 22 & Oct.23, 2014)

Table C-4.1 Average Queue Storage Ratio by Lane and by Period at PCB Pkwy and Richard Jackson

Time	Time	NB				SB				EB					WB				
Period	**Segment**	**Left**		**Through**	**Right**	**Left**	**Through**		**Right**	**Left**	**Through**			**Right**	**Left**	**Through**			**Right**
Lane Number		**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**	**13**	**14**	**15**	**16**	**17**	**18**
AM Peak	1	0.3	0.3	0.8	0.5	0.8	0.7	0.4	0.2	0.5	0.4	0.4	0.3	0.4	0.4	0.8	0.8	0.9	0.5
	2	0.5	0.6	0.3	0.3	0.2	0.4	0.1	0.2	0.1	0.5	0.4	0.2	0.3	0.3	0.9	0.9	0.7	0
	3	0.6	0.6	0.2	0.3	0.2	0.2	0.1	0.3	0.2	0.4	0.3	0.2	0.2	0.2	0.6	0.7	0.4	0.2
	4	0.5	0.3	0.3	0.3	0.2	0.3	0.1	0.1	0.2	0.6	0.5	0.5	0	0.6	0.6	0.6	0.6	0.2
	Average	**0.5**	**0.5**	**0.4**	**0.3**	**0.4**	**0.4**	**0.2**	**0.2**	**0.2**	**0.5**	**0.4**	**0.3**	**0.2**	**0.4**	**0.7**	**0.7**	**0.6**	**0.2**
Off Peak	1	0.3	0.4	0.3	0.1	0.6	0.6	0.5	0.2	0.3	0.5	0.3	0.2	0.1	0.2	0.5	0.4	0.4	0.1
	2	0.6	0.7	0.4	0.2	0.4	0.5	0.3	0.2	0.3	0.4	0.3	0.2	0.3	0.2	0.6	0.5	0.7	0.2
	3	0.6	0.7	0.5	0.2	0.3	0.4	0.2	0.1	0.2	0.4	0.4	0.4	0.4	0.2	0.7	0.6	0.5	0.1
	4	0.4	0.5	0.4	0.3	0.3	0.4	0.3	0.2	0.3	0.6	0.6	0.1	0.1	0.3	0.7	0.6	0.7	0.1
	Average	**0.5**	**0.6**	**0.4**	**0.2**	**0.4**	**0.5**	**0.3**	**0.2**	**0.3**	**0.5**	**0.4**	**0.2**	**0.2**	**0.3**	**0.6**	**0.5**	**0.6**	**0.1**
PM Peak	1	0.6	0.6	0.4	0.3	0.3	0.5	0.1	0.2	0.6	0.3	0.4	0.3	0.2	0.3	0.6	0.5	0.3	0.1
	2	0.6	0.7	0.4	0.5	0.4	0.5	0.3	0.1	0.4	0.5	0.5	0.3	0.2	0.4	0.7	0.6	0.6	0
	3	0.5	0.6	0.5	0.3	0.4	0.2	0.2	0.3	0.2	0.3	0.3	0.1	0.1	0.5	0.8	0.7	0.3	0.1

Time	Time	NB				SB				EB					WB				
Period	**Segment**	**Left**		**Through**	**Right**	**Left**	**Through**		**Right**	**Left**	**Through**			**Right**	**Left**	**Through**			**Right**
	4	0.3	0.3	0.2	0.5	0.2	0.4	0.2	0.1	0.2	0.5	0.4	0.4	0.1	0.3	0.6	0.6	0.4	0
	Average	**0.5**	**0.5**	**0.4**	**0.4**	**0.3**	**0.4**	**0.2**	**0.2**	**0.4**	**0.4**	**0.4**	**0.3**	**0.1**	**0.4**	**0.7**	**0.6**	**0.4**	**0.1**

Table C-4.2 Average Queue Storage Ratio by Lane and by Period at PCB Pkwy and SR-79

Time	Time	NB			SB				EB				WB			
Period	**Segment**	**Left**	**Through**	**Right**	**Left**		**Through**	**Right**	**Left**	**Through**		**Right**	**Left**	**Through**		**Right**
Lane Number		**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**	**13**	**14**	**15**
AM Peak	1	0.3	0.3	0.1	0.4	0.5	0.5	0.2	0.2	0.6	0.5	0.3	0.3	0.3	0.3	0
	2	0.2	0.1	0	0.5	0.5	0.4	0.2	0.3	0.5	0.4	0.3	0.2	0.4	0.4	0.1
	3	0.3	0.2	0.3	0.3	0.4	0.3	0.1	0.3	0.5	0.4	0.1	0.2	0.5	0.5	0.4
	4	0.3	0.2	0.2	0.4	0.4	0.3	0.1	0.4	0.6	0.4	0.3	0.2	0.5	0.5	0
	Average	**0.3**	**0.2**	**0.2**	**0.4**	**0.5**	**0.4**	**0.1**	**0.3**	**0.5**	**0.4**	**0.2**	**0.2**	**0.4**	**0.4**	**0.1**
Off Peak	1	0.5	0.1	0.2	0.1	0.3	0.4	0.2	0.3	0.4	0.5	0.3	0.4	0.3	0.4	0.1
	2	0.6	0.4	0.4	0.5	0.7	0.5	0.1	0.4	0.6	0.6	0.3	0.5	0.5	0.4	0.1
	3	0.3	0.2	0.4	0.2	0.4	0.8	0.5	0.3	0.4	0.4	0.3	0.5	0.3	0.4	0.1
	4	0.6	0.2	0.1	0.2	0.4	0.5	0.1	0.4	0.6	0.6	0.3	0.5	0.4	0.5	0.1

Time	Time	NB			SB				EB				WB			
Period	**Segment**	**Left**	**Through**	**Right**	**Left**		**Through**	**Right**	**Left**	**Through**		**Right**	**Left**	**Through**		**Right**
	Average	**0.5**	**0.2**	**0.3**	**0.3**	**0.5**	**0.5**	**0.2**	**0.3**	**0.5**	**0.5**	**0.3**	**0.5**	**0.4**	**0.4**	**0.1**
PM Peak	1	0.5	0.7	0.3	0.5	0.6	0.4	0.1	0.5	0.7	0.7	0.2	0.7	0.5	0.5	0.2
	2	0.3	0.4	0.2	0.4	0.6	0.6	0.1	0.6	0.7	0.6	0.2	0.6	0.5	0.6	0.2
	3	0.4	0.6	0.4	0.7	0.8	0.1	0	0.6	0.8	0.6	0.2	0.5	0.6	0.6	0.5
	4	0.4	0.2	0.1	0.2	0.3	0.4	0.1	0.5	0.6	0.5	0.1	0.4	0.4	0.5	0.2
	Average	**0.4**	**0.5**	**0.2**	**0.5**	**0.6**	**0.4**	**0.1**	**0.6**	**0.7**	**0.6**	**0.2**	**0.6**	**0.5**	**0.6**	**0.3**

After Study (Feb. 3 & Feb.4, 2015 for AM and OFF PEAK and Sep 29, 2015 for PM)

Table C-4.3 Average Queue Storage Ratio by Lane and by Period at PCB Pkwy and Richard Jackson

Time	Time	NB				SB				EB					WB				
Period	**Segment**	**Left**		**Through**	**Right**	**Left**	**Through**		**Right**	**Left**	**Through**			**Right**	**Left**	**Through**			**Right**
Lane Number		**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**	**13**	**14**	**15**	**16**	**17**	**18**
AM	1	0.2	0.3	0.8	0.4	0.6	0.4	0.3	0.2	0.6	0.2	0.2	0.1	0.0	0.4	0.5	0.5	0.3	0.4
Peak	2	0.2	0.4	0.3	0.5	0.6	0.5	0.5	0.5	0.2	0.6	0.5	0.5	0.0	0.7	0.4	0.5	0.3	0.2
	3	0.5	0.5	0.3	0.2	0.2	0.3	0.5	0.1	0.2	0.2	0.2	0.2	0.0	0.8	0.8	0.7	0.6	0.0
	4	0.4	0.4	0.3	0.2	0.2	0.2	0.4	0.1	0.2	0.3	0.3	0.3	0.0	0.3	0.4	0.2	0.2	0.1

Time Period	**Time Segment**	**NB**				**SB**				**EB**					**WB**				
		Left		**Through**	**Right**	**Left**	**Through**		**Right**	**Left**	**Through**			**Right**	**Left**	**Through**			**Right**
	Average	**0.3**	**0.4**	**0.4**	**0.3**	**0.4**	**0.4**	**0.4**	**0.2**	**0.3**	**0.3**	**0.3**	**0.3**	**0.0**	**0.6**	**0.5**	**0.5**	**0.3**	**0.2**
Off Peak	1	0.4	0.5	0.3	0.5	0.2	0.5	0.3	0.2	0.3	0.3	0.3	0.1	0.0	0.3	0.5	0.4	0.1	0.2
	2	0.5	0.6	0.5	0.0	0.3	0.4	0.3	0.2	0.2	0.3	0.3	0.1	0.0	0.4	0.4	0.3	0.1	0.3
	3	0.7	0.8	0.4	0.4	0.3	0.5	0.4	0.2	0.3	0.3	0.2	0.4	0.0	0.4	0.4	0.3	0.1	0.1
	4	0.5	0.5	0.4	0.4	0.3	0.5	0.5	0.6	0.3	0.2	0.2	0.6	0.0	0.6	0.4	0.5	0.1	0.2
	Average	**0.5**	**0.6**	**0.4**	**0.3**	**0.3**	**0.5**	**0.4**	**0.3**	**0.3**	**0.3**	**0.2**	**0.3**	**0.0**	**0.4**	**0.4**	**0.4**	**0.1**	**0.2**
PM Peak	1	0.4	0.4	0.2	0.2	0.4	0.2	0.4	0.2	0.2	0.4	0.4	0.4	0.0	0.4	0.6	0.5	0.1	0.0
	2	0.3	0.4	0.3	0.4	0.3	0.3	0.4	0.2	0.1	0.3	0.3	0.4	0.0	0.4	0.4	0.4	0.1	0.0
	3	0.5	0.4	0.2	0.6	0.4	0.4	0.4	0.4	0.3	0.3	0.3	0.3	0.0	0.4	0.5	0.4	0.2	0.0
	4	0.4	0.4	0.2	0.2	0.4	0.3	0.2	0.4	0.1	0.5	0.5	0.5	0.0	0.5	0.4	0.4	0.2	0.0
	Average	**0.4**	**0.4**	**0.2**	**0.3**	**0.4**	**0.3**	**0.4**	**0.3**	**0.2**	**0.4**	**0.4**	**0.4**	**0.0**	**0.4**	**0.4**	**0.4**	**0.1**	**0.0**

Table C-4.4 Average Queue Storage Ratio by Lane by Period at PCB Pkwy and SR-79

Time Period	**Time Segment**	**NB**			**SB**				**EB**				**WB**			
		Left	**Through**	**Right**	**Left**		**Through**	**Right**	**Left**	**Through**		**Right**	**Left**	**Through**		**Right**
Lane Number		**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**	**13**	**14**	**15**
AM Peak	1	0.3	0.5	0.1	0.6	0.7	0.3	0.2	0.3	0.7	0.7	0.0	0.3	0.4	0.5	0.0
	2	0.3	0.2	0.1	0.7	0.7	0.4	0.0	0.3	0.5	0.5	0.1	0.4	0.2	0.3	0.0
	3	0.5	0.3	0.1	0.4	0.6	0.3	0.0	0.4	0.6	0.5	0.0	0.4	0.3	0.3	0.0
	4	0.4	0.2	0.1	0.5	0.6	0.3	0.0	0.4	0.5	0.5	0.1	0.4	0.5	0.5	0.0
	Average	**0.4**	**0.3**	**0.1**	**0.6**	**0.6**	**0.3**	**0.1**	**0.4**	**0.6**	**0.5**	**0.0**	**0.4**	**0.4**	**0.4**	**0.0**
Off Peak	1	0.3	0.3	0.0	0.2	0.5	0.2	0.0	0.3	0.6	0.8	0.1	0.6	0.3	0.4	0.0
	2	0.5	0.4	0.0	0.4	0.5	0.2	0.0	0.4	0.6	0.6	0.1	0.4	0.3	0.4	0.0
	3	0.0	0.1	0.8	0.4	0.6	0.3	0.0	0.4	0.7	0.8	0.1	0.7	0.3	0.4	0.0
	4	0.0	0.2	0.6	0.3	0.5	0.1	0.0	0.3	0.6	0.7	0.3	0.6	0.3	0.3	0.0
	Average	**0.2**	**0.2**	**0.3**	**0.3**	**0.5**	**0.2**	**0.0**	**0.4**	**0.6**	**0.7**	**0.2**	**0.6**	**0.3**	**0.4**	**0.0**
PM	1	0.3	0.4	0.1	0.6	0.7	0.0	0.0	0.6	0.5	0.5	0.1	0.6	0.4	0.6	0.0
	2	0.5	0.4	0.1	0.4	0.6	0.5	0.0	0.5	0.8	0.7	0.2	0.6	0.3	0.3	0.0
	3	0.4	0.4	0.1	0.3	0.6	0.2	0.0	0.6	0.6	0.5	0.1	0.7	0.3	0.3	0.0

Time Period	Time Segment	NB			SB				EB				WB			
		Left	**Through**	**Right**	**Left**		**Through**	**Right**	**Left**	**Through**		**Right**	**Left**	**Through**		**Right**
Lane Number		**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**	**13**	**14**	**15**
Peak	4	0.3	0.5	0.0	0.2	0.4	0.2	0.0	0.3	0.5	0.5	0.0	0.6	0.4	0.4	0.0
	Average	**0.4**	**0.4**	**0.1**	**0.4**	**0.6**	**0.2**	**0.0**	**0.5**	**0.6**	**0.5**	**0.1**	**0.6**	**0.3**	**0.4**	**0.0**

Comparisons of Before and After Queue Storage Ratios

The differences in Queue Storage Ratio between before and after measurements are shown in Tables C-4.5 and C-4.6. The tables are color-coded as follows: green shows significant improvement, yellow shows modest change, and red shows significant deterioration in queue storage ratios or spillback potential.

Table C-4.5 Difference in Average Queue Storage Ratios at PCB Pkwy and Richard Jackson

Time	NB				SB				EB					WB					Average
Period	Left		Through	Right	Left	Through		Right	Left	Through			Right	Left	Through			Right	
Lane Number	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	
AM Peak	-0.2	-0.1	0.0	-0.1	0.1	0.0	0.2	0.1	0.1	-0.2	-0.1	0.0	-0.2	0.2	-0.2	-0.3	-0.3	0.0	0.0
Off Peak	0.0	0.0	0.0	0.1	-0.1	0.0	0.0	0.1	0.0	-0.2	-0.1	0.1	-0.2	0.2	-0.2	-0.1	-0.5	0.1	-0.1
PM Peak	-0.1	-0.1	-0.2	0.0	0.1	-0.1	0.2	0.1	-0.2	0.0	0.0	0.1	-0.1	0.1	-0.2	-0.1	-0.3	-0.1	-0.1

Table C-4.6 Difference in Average Queue Storage Ratio at PCB Pkwy and SR-79

Time	NB			SB				EB				WB			
Segment	**Left**	**Through**	**Right**	**Left**		**Through**	**Right**	**Left**	**Through**		**Right**	**Left**	**Through**		**Right**
Lane	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**	**13**	**14**	**15**
AM Peak	0.1	0.1	-0.1	0.1	0.2	-0.1	-0.1	0.1	0.0	0.1	-0.2	0.1	-0.1	0.0	-0.1
Off Peak	-0.3	0.0	0.1	0.0	0.0	-0.3	-0.2	0.0	0.1	0.2	-0.1	0.1	-0.1	0.0	-0.1
PM Peak	0.0	0.0	-0.2	0.0	0.0	-0.1	-0.1	0.0	-0.1	-0.1	-0.1	0.1	-0.2	-0.2	-0.3

Discussion

The following were concluded regarding queue storage ratios:

- Queue storage ratios were fairly high for major street movements for the Richard Jackson Blvd. intersection in the before study, and these were substantially improved with the new system.
- There was no change in the overall average queue storage ratios at Richard Jackson (due to increase in queues on side street), whereas the ratio decreased by 0.1 for SR-79.

C.3.5 EQUIVALENT PCE FLOWS

Traffic flows were counted manually and converted to equivalent PCE flows (pce/hour) by considering the presence of heavy vehicles. It was assumed that the PCE for trucks is 2

Truck Percentage Observations

Table C-5.1 Truck Percentages at PCB Pkwy and Richard Jackson

Period	Interval	SB			WB			NB			EB		
		Left	**Thru**	**Right**	**Left**	**Thru**	**Right**	**Left**	**Thru**	**Right**	**Left**	**Thru**	**Right**
AM Peak	1	0.00%	11.76%	5.56%	16.00%	1.57%	16.00%	0.00%	0.00%	0.00%	0.00%	0.61%	0.00%
	2	5.26%	1.56%	4.26%	0.00%	2.04%	0.00%	0.00%	0.00%	0.00%	12.90%	3.12%	0.75%
	3	2.17%	0.00%	0.00%	0.00%	1.69%	0.00%	0.00%	3.85%	11.11%	1.85%	2.85%	1.89%
	4	2.04%	0.00%	5.00%	0.00%	3.27%	0.00%	3.23%	0.00%	0.00%	5.26%	3.97%	2.63%
	Average	**2.58%**	**3.40%**	**3.39%**	**2.96%**	**2.14%**	**2.96%**	**0.49%**	**0.88%**	**1.19%**	**5.22%**	**2.88%**	**1.04%**
Off Peak	1	4.17%	0.00%	0.00%	0.00%	5.68%	0.00%	4.00%	0.00%	0.00%	3.57%	3.98%	0.00%
	2	0.00%	4.17%	0.00%	6.45%	3.69%	0.00%	0.00%	2.78%	0.00%	0.00%	1.72%	0.00%
	3	0.00%	0.00%	0.00%	0.00%	4.10%	0.00%	0.00%	0.00%	0.00%	3.70%	3.50%	0.00%
	4	0.00%	0.00%	0.00%	0.00%	2.99%	0.00%	0.00%	2.94%	0.00%	3.70%	6.02%	0.00%
	Average	**0.70%**	**1.23%**	**0.00%**	**1.52%**	**4.07%**	**0.00%**	**0.84%**	**1.38%**	**0.00%**	**2.91%**	**3.84%**	**0.00%**
PM	1	2.27%	10.53%	6.67%	0.00%	3.33%	0.00%	0.00%	4.11%	0.00%	0.00%	1.42%	0.00%

Period	Interval	SB			WB			NB			EB		
		Left	Thru	Right	Left	Thru	Right	Left	Thru	Right	Left	Thru	Right
Peak	2	0.00%	0.00%	0.00%	2.56%	0.00%	0.00%	0.00%	3.16%	0.00%	0.00%	1.12%	0.00%
	3	0.00%	7.69%	0.00%	0.00%	6.06%	0.00%	3.85%	2.49%	3.92%	0.00%	2.75%	0.00%
	4	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	2.85%	1.89%	0.00%	1.67%	0.00%
	Average	**0.54%**	**4.55%**	**1.32%**	**0.61%**	**2.54%**	**0.00%**	**1.27%**	**3.13%**	**1.37%**	**0.00%**	**1.75%**	**0.00%**

Table C-5.2 Truck Percentages at PCB Pkwy and SR-79

Period	Interval	SB			WB			NB			EB		
		Left	Thru	Right	Left	Thru	Right	Left	Thru	Right	Left	Thru	Right
AM Peak	1	2.82%	0.00%	15.56%	5.26%	6.01%	11.76%	0.00%	3.51%	0.00%	9.09%	3.30%	12.50%
	2	9.72%	0.00%	9.62%	0.00%	5.36%	22.86%	0.00%	0.00%	0.00%	6.67%	3.41%	0.00%
	3	8.00%	0.00%	15.38%	0.00%	2.87%	12.82%	0.00%	0.00%	0.00%	4.00%	2.69%	5.56%
	4	8.20%	9.09%	20.00%	4.55%	9.45%	10.00%	0.00%	0.00%	0.00%	2.86%	1.16%	0.00%
	Average	**7.17%**	**1.85%**	**14.56%**	**2.30%**	**6.01%**	**14.49%**	**0.00%**	**1.96%**	**0.00%**	**4.65%**	**2.79%**	**2.60%**
Off Peak	1	15.22%	10.00%	15.38%	3.77%	3.98%	20.00%	2.86%	16.67%	0.00%	25.00%	2.46%	0.00%
	2	11.76%	6.67%	15.00%	0.00%	5.00%	13.33%	0.00%	7.69%	0.00%	0.00%	4.10%	4.17%
	3	11.36%	0.00%	24.00%	2.38%	3.30%	5.41%	3.70%	20.83%	0.00%	0.00%	2.16%	2.38%

Period	Interval	SB			WB			NB			EB		
		Left	Thru	Right	Left	Thru	Right	Left	Thru	Right	Left	Thru	Right
	4	14.81%	16.67%	15.38%	3.03%	4.33%	13.33%	2.44%	7.69%	2.38%	13.64%	2.12%	0.00%
	Average	**13.21%**	**6.82%**	**18.31%**	**2.52%**	**4.15%**	**11.97%**	**2.67%**	**14.52%**	**0.55%**	**11.67%**	**2.76%**	**1.85%**
PM Peak	1	4.41%	0.00%	0.00%	0.00%	2.45%	3.90%	0.00%	0.00%	0.00%	2.56%	2.41%	0.00%
	2	1.54%	0.00%	0.00%	0.00%	2.39%	4.62%	3.03%	7.14%	0.00%	2.56%	2.80%	2.94%
	3	0.00%	0.00%	25.00%	0.00%	0.00%	2.60%	0.00%	5.00%	2.86%	11.54%	3.40%	0.00%
	4	0.00%	0.00%	0.00%	0.00%	0.45%	0.00%	0.00%	0.00%	0.00%	17.65%	3.87%	0.00%
	Average	**1.77%**	**0.00%**	**2.63%**	**0.00%**	**1.35%**	**3.15%**	**0.92%**	**2.20%**	**0.61%**	**7.97%**	**3.05%**	**0.81%**

Before Study (Oct. 22 & Oct.23, 2014)

Table C-5.3 Traffic Volume (pce/15 min) and Traffic Flow (pce/hour) at PCB Pkwy and Richard Jackson

Time Period		NB			SB			EB			WB		
		Left	Through	Right	Left	Through	Right	Left	Through	Right	Left	Through	Right
AM Peak	1	25	90	77	65	68	26	97	265	10	21	314	113
	2	50	43	17	65	49	42	58	308	44	40	344	42
	3	50	28	23	32	38	6	34	310	61	33	321	29
	4	19	37	47	24	26	12	32	192	32	18	148	31

Time Period		NB			SB			EB			WB		
		Left	Through	Right	Left	Through	Right	Left	Through	Right	Left	Through	Right
	Flow Rate	**144**	**198**	**164**	**186**	**181**	**86**	**221**	**1075**	**147**	**112**	**1127**	**215**
Off Peak	1	58	29	22	24	23	11	18	213	36	19	236	32
	2	54	47	22	52	17	10	26	193	33	30	188	36
	3	51	20	28	34	19	7	20	247	41	27	283	29
	4	45	34	17	29	26	5	33	240	41	14	248	23
	Flow Rate	**208**	**130**	**89**	**139**	**85**	**33**	**97**	**893**	**151**	**90**	**955**	**120**
PM Peak	1	43	44	31	29	40	8	21	289	39	31	321	18
	2	69	38	51	48	42	20	24	294	42	33	402	24
	3	71	35	38	55	40	17	27	309	57	30	340	13
	4	34	21	18	46	31	13	21	284	23	27	312	18
	Flow Rate	**217**	**138**	**138**	**178**	**153**	**58**	**93**	**1176**	**161**	**121**	**1375**	**73**

Table C-5.4 Traffic Volume (pce/15 min) and Traffic Flow (pce/hour) at PCB Pkwy and SR-79

Time Period		NB			SB			EB			WB		
		Left	**Through**	**Right**	**Left**	**Through**	**Right**	**Left**	**Through**	**Right**	**Left**	**Through**	**Right**
AM Peak	1	26	13	13	92	15	37	16	217	12	12	211	35
	2	21	20	20	60	15	43	24	165	14	37	204	32
	3	27	10	16	65	9	29	30	182	19	20	277	33
	4	23	26	14	37	14	16	25	183	15	14	145	34
	Flow Rate	**97**	**69**	**63**	**254**	**53**	**125**	**95**	**747**	**60**	**83**	**837**	**134**
Off Peak	1	24	6	17	52	11	27	33	240	22	28	199	22
	2	21	17	22	71	15	60	49	204	35	24	195	29
	3	24	26	28	61	13	47	57	243	45	25	212	29
	4	31	21	30	34	16	34	31	241	33	32	222	31
	Flow Rate	**100**	**70**	**97**	**218**	**55**	**168**	**170**	**928**	**135**	**109**	**828**	**111**
PM Peak	1	19	18	33	62	22	48	60	264	45	38	189	54
	2	27	25	17	72	20	32	51	279	43	31	197	43
	3	27	18	31	59	22	35	61	295	39	38	187	50
	4	30	13	35	57	17	29	45	232	34	32	201	49
	Flow Rate	**103**	**74**	**116**	**250**	**81**	**144**	**217**	**1070**	**161**	**139**	**774**	**196**

After Study (Feb 3 and Feb 4, 2015)

Table C-5.5 Traffic Volume (pce/15 min) and Traffic Flow (pce/hour) at PCB Pkwy and Richard Jackson

Time Period			NB			SB			EB			WB	
		Left	Through	Right	Left	Through	Right	Left	Through	Right	Left	Through	Right
AM Peak	1	36	6	9	22	38	19	11	165	64	29	258	14
	2	103	62	46	40	65	49	35	331	134	70	350	13
	3	33	27	10	47	24	33	55	325	54	18	301	40
	4	32	20	20	50	25	21	40	288	39	22	284	49
	Flow Rate	**204**	**115**	**85**	**159**	**152**	**122**	**141**	**1109**	**291**	**139**	**1193**	**116**
Off Peak	1	52	30	30	25	28	11	29	235	44	32	242	25
	2	55	37	24	37	50	21	21	236	39	33	225	29
	3	59	45	29	35	34	11	28	266	34	36	279	27
	4	74	35	20	47	53	19	28	264	50	33	276	22
	Flow Rate	**240**	**147**	**103**	**144**	**165**	**62**	**106**	**1001**	**167**	**134**	**1022**	**103**
PM Peak	1	45	21	32	43	31	21	18	304	65	26	286	19
	2	41	25	38	40	30	21	14	261	50	32	272	33
	3	50	28	55	44	35	26	27	329	53	44	299	23
	4	51	18	29	37	25	17	21	325	54	30	305	21

Time Period		NB			SB			EB			WB		
		Left	Through	Right	Left	Through	Right	Left	Through	Right	Left	Through	Right
	Flow Rate	**187**	**92**	**154**	**164**	**121**	**85**	**80**	**1219**	**222**	**132**	**1162**	**96**

Table C-5.6 Traffic Volume (pce/15 min) and Traffic Flow (pce/hour) at PCB Pkwy and SR-79

Time Period		NB			SB			EB			WB		
		Left	Through	Right	Left	Through	Right	Left	Through	Right	Left	Through	Right
AM Peak	1	11	59	24	73	6	52	12	282	9	20	194	38
	2	13	23	24	79	22	57	16	273	16	25	236	43
	3	14	11	17	81	15	30	26	191	19	21	179	44
	4	29	11	32	66	12	42	36	174	35	23	220	33
	Flow Rate	**67**	**104**	**97**	**299**	**55**	**181**	**90**	**920**	**79**	**89**	**829**	**158**
Off Peak	1	36	14	48	53	11	15	20	208	23	55	183	24
	2	20	14	50	76	16	23	8	254	50	31	210	34
	3	56	29	43	49	13	31	14	189	43	43	219	39
	4	42	14	43	62	7	15	25	241	49	34	217	34
	Flow Rate	**154**	**71**	**184**	**240**	**47**	**84**	**67**	**892**	**165**	**163**	**829**	**131**
PM Peak	1	23	17	49	71	13	13	40	298	35	38	251	80

Time Period		Left	NB Through	Right	Left	SB Through	Right	Left	EB Through	Right	Left	WB Through	Right
	2	34	15	38	66	13	17	40	294	35	35	257	68
	3	20	21	36	50	18	5	29	335	33	45	243	79
	4	33	40	43	43	8	4	40	188	21	32	222	35
	Flow Rate	**110**	**93**	**166**	**230**	**52**	**39**	**149**	**1115**	**124**	**150**	**973**	**262**

Comparisons of Before and After Flows

The differences in traffic flow between the before and after measurements are shown in Table C-5.7 and Table C-5.8. The tables are color-coded as follows: green shows significant decrease, yellow shows modest change, and red shows significant increase in flow.

Table C-5.7 Difference in Traffic Flow Rate (pce/hr) at PCB Pkwy and Richard Jackson

Time	NB			SB			EB			WB		
Period	Left	Through	Right	Left	Through	Right	Left	Through	Right	Left	Through	Right
AM Peak	60	-83	-79	-27	-29	36	-80	34	144	27	66	-99
Off Peak	32	17	14	5	80	29	9	108	16	44	67	-17
PM Peak	-30	-46	16	-14	-32	27	-13	43	61	11	-213	23

Table C-5.8 Difference in Traffic Flow Rate (pce/hr) at PCB Pkwy and SR-79

Time Period	NB			SB			EB			WB		
	Left	Through	Right	Left	Through	Right	Left	Through	Right	Left	Through	Right
AM Peak	-30	35	34	45	2	56	-5	173	19	6	-8	24
Off Peak	54	1	87	22	-8	-84	-103	-36	30	54	1	20
PM Peak	7	19	50	-20	-29	-105	-68	45	-37	11	199	66

Discussion

It can be concluded that the traffic volume at both critical intersections increased during all three time periods by 135 (1.3%) and 527 (5.9%) vehicles at Richard Jackson and SR-79 and respectively. Despite this moderate increase, delays and queues decreased with the installation of the new system.

C.3.6 CONSIDERATION OF TRAFFIC FLOWS JOINTLY WITH QUEUE LENGTH

The differences in “Traffic Flow” and “Queue Length” between before and after measurements are shown in Table C-6.1 and Table C-6.3. The tables are color-coded as follows: green indicates that “Queue” decreases and “Traffic Flow” increases, Red indicates “Queue” increases and “Traffic Flow” decreases, No Color indicates “Traffic Flow” and “Queue” increase or decrease at the same time. Generally, green indicates improvement despite an increase in flow.

Table C-6.1 Differences in Traffic Flow (TF) and Queue Length (Q) at PCB Pkwy and Richard Jackson.

Time Period	Northbound (Side)						Southbound (Side)						Eastbound (Main)						Westbound (Main)					
	Left		Thru		Right		Left		Thru		Right		Left		Thru		Right		Left		Thru		Right	
	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q
AM Peak	60	-1.0	-83	1.4	-79	-1.0	-27	0.3	-29	0.4	36	0.1	-80	0.8	34	0.5	144	-0.9	27	-0.9	66	-3.4	-99	-1.8
OFF PEAK	32	0.7	17	1.6	14	0.3	5	-2.9	80	-0.3	29	0.4	9	-0.1	108	-0.1	16	-0.9	44	-0.2	67	-4.3	-17	-0.4
PM Peak	-31	-0.9	-50	-1.5	14	0.5	-15	0.3	-35	0.0	27	0.6	-14	-3.3	6	2.3	58	-0.6	11	-2.1	-233	-3.3	23	-0.7

Table C-6.2 Differences (%) in Traffic Flow (TF) and Queue Length (Q) at PCB Pkwy and Richard Jackson.

Time Period	Northbound (Side)						Southbound (Side)						Eastbound (Main)						Westbound (Main)					
	Left		Thru		Right		Left		Thru		Right		Left		Thru		Right		Left		Thru		Right	
	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q
AM	42%	-22%	-42%	28%	-48%	-40%	-15%	4%	-16%	25%	42%	7%	-36%	21%	3%	25%	98%	-100%	24%	-11%	6%	-15%	-46%	-67%
OFF PEAK	15%	13%	13%	36%	16%	22%	4%	-36%	94%	-11%	88%	41%	9%	-3%	12%	35%	11%	-100%	49%	-3%	7%	-45%	-14%	-27%
PM	-14%	-18%	-36%	-32%	10%	21%	-8%	4%	-23%	4%	47%	55%	-15%	-60%	1%	80%	36%	-100%	9%	-29%	-17%	-25%	32%	-93%

Table C-6.3 Differences in Traffic Flow and Queue Length at PCB Pkwy and SR-79.

Time Period	Northbound (Side)						Southbound (Side)						Eastbound (Main)						Westbound (Main)					
	Left		Thru		Right		Left		Thru		Right		Left		Thru		Right		Left		Thru		Right	
	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q
AM Peak	-30	0.8	35	0.0	34	1.4	45	0.8	2	0.2	56	0.1	-5	0.2	173	-0.1	19	-4.5	6	-3.6	-8	-0.2	24	0.0
OFF PEAK	54	-1.2	1	1.7	87	0.4	22	-0.1	-8	-0.7	-84	-0.7	-103	1.8	-36	0.5	30	-4.3	54	-3.9	1	-0.1	20	0.0
PM Peak	7	-0.1	19	-0.4	50	-0.5	-20	-0.6	-29	0.0	-105	-2.0	-68	-1.9	45	-0.2	-37	-7.3	11	-6.7	199	-0.3	66	0.0

Table C-6.4 Differences (%) in Traffic Flow and Queue Length at PCB Pkwy and SR-79.

Time Period	Northbound (Side)						Southbound (Side)						Eastbound (Main)						Westbound (Main)					
	Left		Thru		Right		Left		Thru		Right		Left		Thru		Right		Left		Thru		Right	
	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q
AM Peak	-31%	43%	51%	0%	54%	32%	18%	18%	4%	-14%	45%	3%	-5%	2%	23%	-17%	32%	-48%	7%	-39%	-1%	-100%	18%	0%
OFF PEAK	54%	-28%	1%	167%	90%	13%	10%	-3%	-15%	-68%	-50%	-18%	-61%	20%	-4%	8%	22%	-51%	50%	-43%	0%	-100%	18%	0%
PM Peak	7%	-3%	26%	-47%	43%	-10%	-8%	-11%	-36%	-46%	-73%	-30%	-31%	-14%	4%	-5%	-23%	-61%	8%	-55%	26%	-100%	34%	0%

Discussion

The following can be concluded from the comparison of traffic flows jointly with queue length:

- There does not seem to be a high correlation between increasing traffic and increasing queues at the two critical intersections.

Generally, traffic conditions at both intersections have improved. This is particularly true with the WB traffic.

BEFORE AND AFTER-IMPLEMENTATION STUDIES OF

ADVANCED SIGNAL CONTROL TECHNOLOGIES IN FLORIDA

The main objective of this project is to evaluate the implementation of proposed adaptive signal control technology (ASCT) traffic operations at several arterial corridors in Florida, before and after the installation of the ASCT, document the advantages and the disadvantages of different approaches and implementations, and provide recommendations for statewide implementation of ASCT.

This questionnaire will allow us to access the appropriateness and effectiveness of the ASCT implementation at each subject corridor by: conducting benefit cost analyses, comparing staffing requirements before and after the deployment, comparing the resource needs in terms of hardware, software and personnel, pinpointing any institutional issues and generally evaluating the ASCT implementation and maintenance requirements by incorporating the agency's perspective.

C.4 QUESTIONNAIRE

SECTION 1: RESPONDENT'S INFORMATION

Name **Marc R. Mackey**

Organization **Bay County Public Works**

Position **Traffic Operations Engineer**

Address

Phone

Fax ____

E-mail **mmackey@baycountyfl.gov**

SECTION 2: PREVIOUS TRAFFIC CONTROL TECHNOLOGY

1. Please specify the type of traffic control used before Adaptive Signal Control Technology (ASCT) was installed for your site. (Please check all that apply.)

a. Fixed time coordinated control

b. Actuated coordinated control

c. Fixed-time isolated control

d. Actuated isolated control

e. Other, please specify: ____

2. What type of traffic controller was used before ASCT was implemented?

a. NEMA TS-1 **b. NEMA TS-2**

c. 170 d. 2070

e. Other, please specify: ____

3. How many detectors were used before ASCT was implemented for a typical intersection (e.g. *Newberry Rd & NW 75th St*)? Please also specify the location where the detectors were typically placed for each specific detector type (video detection, inductive loops, radars, etc.).

   Two advance detectors at each main through movement. Left turning detectors for all approaches; Stop bar detector for side-streets.

4. What was the frequency of retiming the signal timing plan for the corridor before ASCT was implemented?

   2-3 years

5. How often were maintenance and updates performed to your previous control system in terms of the following components?

a. Detection: **Regular quarterly check usually accompanied by an annual check**

b. Control Hardware:

6. Software:

7. What was the annual maintenance cost in terms of hardware, software and personnel of the previous control system for the whole corridor where the ASCT is now deployed?

8. What was the life expectancy of your traffic control before the deployment of ASCT? (Please refer to the hardware and software.)

It went well (for 6 years) before being replaced. The old system is quite simple and durable.

9. How many crashes of each category were reported during the past 4 years before the ASCT implementation? (2011-2014)

	K (Killed)	A (Disabling Injury)	B (Evident Injury)	C (Possible Injury)	O (No Apparent Injury)
2011					
2012					
2013					
2014					

SECTION 3: ADAPTIVE TRAFFIC CONTROL SYSTEM (ASCT)

10. Which Adaptive Traffic Control System (ASCT) does your agency deploy?

    a. InSync; Version: **Fusion**

    b. Synchro Green; Version: ____________________

    c. Other; please specify: _______________________

11. Did you consider any other ASCT before you selected this one for installation? Why did you reject the other(s) in favor of this one?

    No, state picked the software

12. What were your major criteria for choosing the ASCT for the selected corridor?

    N/A

13. What is the life expectancy of your ASCT system? (Consider both hardware and software.)

    5 years warranty purchased.

14. What type of traffic controller is currently in use after the ASCT implementation?

a. Same as before the ASCT implementation

b. New, please specify type:

c. Same, but the following updates were needed: ______________________________

15. How many and what type of detectors are used after the ASCT implementation on an intersection level (e.g. *Newberry Rd & NW 75th St*)? Please note any updates that the previous detection system needed so as to work with your ASCT.

4 cameras (one each approach); All left-turn detectors; Side street stop bar detectors. Fusion provides additional back-up data counts. However, the operation of the system depends on the video detection

16. Was there a need to update your previous software operating system at your Transportation Management Center in order to get your current ASCT software working?

a. Yes b. No

17. How often do you plan to perform physical maintenance or updates on your new ASCT in terms of the following components?

a. Detection:

b. ASCT Hardware:

c. Software:

18. Was there a need for training your personnel in order to operate the new ASCT?

a. **Yes,** Num. of employees trained **4 ,** extensive training that covers intro, software, hardware and system setting, implementation.

b. No

19. Was there any change on the number of staff required to operate/maintain the effective signal ASCT operations? (Comparison of staff needed before and after the deployment.)

No

20. Where did expertise come from for your personnel's training, the ASCT installation, operation and the system's maintenance? Please check all that apply.

	In-House	Vendor	Contractor
Employee Training		**Yes**	
ASCT Installation		**Yes**	
ASCT Operation		**Yes**	
ASCT Maintenance		**Yes**	

21. How many crashes of each category were reported after ASCT was implemented?

K (Killed)	A (Disabling Injury)	B (Evident Injury)	C (Possible Injury)	O (No Apparent Injury)

SECTION 4: COST COMPONENTS

22. What is the overall capital cost of purchasing the ASCT (in terms of the software and licenses of the ASCT) for the corridor? If the purchase includes any service of implementation, personnel training or maintenance, please also specify here.

    $45,000 per intersection

23. What is the implementation cost (considering the installation on site, any updates in software and hardware, design needs and contract hours) for the ASCT corridor? Please specify if this cost is partially or totally included in previous cost items.

24. What was total cost for training your personnel to operate and manage the new ASCT? Please specify if this cost is partially or totally included in previous cost items.

25. What is the expected maintenance cost (in terms of hardware, software and personnel) for the ASCT corridor on a yearly basis? Please specify if this cost is partially or totally included in previous cost items.

26. Can you specify the costs for the following ASCT components analytically?

a. Firmware $_____________ b. Software $______________

c. Equipment $____________ d. Maintenance of Traffic Cost $

e. Design Needs $___________ f. Contract Help/Agency Hours Cost $

SECTION 5: INSTITUTIONAL ISSUES

27. Were there any institutional issues that you had to overcome while implementing and operating the ASCT project? Please categorize and explain those issues below.

a. Organization and Management Institutional Issues:

N/A

b. Regulatory and Legal Institutional Issues:

____

c. Human and Facility Resources Institutional Issues:

____

d. Financial Institutional Issues:

____

28. Which component (e.g. detection, communication, hardware, software, licensing or other) of your ASCT deployment is the most challenging to maintain and why?

Camera; can get damaged in lightning

29. What happens to your ASCT operations when some of your detectors fail on the corridor level?

a. ASCT triggers an alarm and notifies operators

b. ASCT switches to an “off-line” mode by implementing Time-Of-Day plans

c. Combination of the above.

d. Other (please describe): _ Run off line data in the most recent two weeks

30. How is your ASCT performance evaluated?

a. In-house

1. from the preliminary analysis:

There is some benefits in the reduction of the rear-end crashes;

2. Majority of the benefits are in the travel time saving. Generally 7% travel time reduction is seen; in certain case, the reduction is even closer to 20% yet somewhat hurts the side-street;

3. The benefits are mainly seen in the light or moderate traffic volumes. In the over-saturated case, the performance is "equally bad" to the previous system.

b. By an independent evaluator, please describe: ______________________________

c. Not applicable—there is no evaluation

31. If the corridor on which the ASCT operates experiences over saturation, how would you rate the operation of the ASCT in response to these traffic conditions?

a. ASCT prevents or eliminates oversaturation

b. ASCT eliminates or reduces the extent of the periods of oversaturation

c. ASCT adversely affects the traffic conditions during periods of oversaturation

d. Other: ____

e. Oversaturation is very rare on the corridors operated by our ASCT

32. Based on your opinion and the up-to-date operation, when are ASCT operations proven to be the most effective?

a. Peak periods (Comment: with no saturation) b. Off-peak periods

c. Shoulders of peak periods d. Other, please specify: ______________

33. Has the level of performance of ASCT been sustained since its installation?

a. Yes b. No; why not?

34. What was the public reaction to the ASCT operation? Have you received any complaints about long delays and queues?

The public generally are happy with the new system

35. How satisfied are you, in general, with your ASCT deployment?

a. Very satisfied b. Somewhat satisfied

c. Neutral d. Somewhat dissatisfied

e. Not satisfied at all

36. Are there any other costs or benefits related to ASCT deployment that you would like to report?

a. Benefits **synchronizes and enriches the data from varying aspects. Can have straight access to the database with evidence if things went wrong.**

b. Costs

36. Would you consider expanding the ASCT program in any other of your other sites? Why or why not? If yes, would you use the same technology/firmware or have any suggestions for others? If so, why?

Yes, one additional corridor (Tyndall Parkway) will also deploy InSync

SECTION 6: SAFETY ISSUES

37. Do you have any anecdotal / qualitative data on safety benefits / disbenefits of the signal improvement along this corridor?

____

____

38. Has there been other changes along this corridor over the analysis period that could affect the safety of this corridor either positively or negatively?

____

____

39. Are you aware of any changes to the crash data reporting procedures over the analysis period? If yes, what are the changes?

____

____

____

____

40. What are your overall reactions to the trends presented in the report?

____

____

41. Is there another comparable corridor in which signal improvements have not been made to which the results of this corridor can be compared to?

____

____

42. Other thoughts / comments for us?

____

____

Thank you for your help in completing this survey. Your responses will help us with evaluating the deployment and benefits of your current ASCT. If you have any questions regarding the survey, please contact Ria Kontou, email: ekontou@ufl.edu or Liteng Zha, email: litengzha@ufl.edu who work under the direct supervision of the faculty advisor Dr. Yafeng Yin

C.5 BENEFIT COST ANALYSIS

Table C-7.1 Monetized Saving

Monetized Saving	
Time saving	$3,016,458.72
Fuel Consumption saving	$358,577.82
Air Pollution Saving	$74,905.53
Safety Saving	-$8,014,851.23
Total Saving without Safety	$3,449,942.07
Total Saving	-$4,564,909.16
Total Cost	$450,000.00
B/C Ratio without safety	7.666537932
B/C Ratio	-10.14424257

Table C-7.2 Monetized Saving with No Emissions

Monetized Saving	
Time saving	$3,016,458.72
Fuel Consumption saving	$358,577.82
Safety Saving	-$8,014,851.23
Total Saving without Safety	$3,375,036.54
Total Saving	-$4,639,814.69
Total Cost	$450,000.00
B/C Ratio without safety	7.500081196
B/C Ratio	-10.31069931

APPENDIX D: Summary of University Parkway, Sarasota County

D.1 EXECUTIVE SUMMARY

The objective of this research is to evaluate the implementation of proposed Adaptive Signal Control Technologies (ASCT) traffic operations at several arterial corridors in Florida, before and after the installation of specific ASCT, document the advantages and disadvantages of different approaches and implementations, and provide recommendations for state-wide implementation of ASCT. This Appendix summarizes the before and after field data collected along the University Parkway, Sarasota corridor from Airport Circle to Lakewood Ranch Blvd.

Two data collection methods were used to collect the desired information. Floating car runs were conducted with the UFTI instrumented vehicle to collect vehicle travel times before the implementation of InSync, during three time periods (AM Peak, Off Peak and PM Peak). In addition, turning movement counts and queue lengths were collected at two critical intersections (US 301 & North Cattleman/Cooper Creek). Based on these, five performance measures were obtained for the before study periods: Link/Route Travel Time, Delay at Intersections, Queue Length (at critical intersections), Queue to Lane Storage Ratio (at critical intersections), and Passenger Car Equivalent (PCE) flows (at critical intersections). For each performance measure, a comparison between the before and after data is conducted and presented in this appendix.

Both data collection process were developed while there was a construction in the interstate I75 (a diverging diamond interchange, DDI) affecting the corridor. Moreover, the after data collection was carried out when also some mall upgrades were being done on the southwest part of the North Cattleman intersection. Since, delays and volumes were affected on the eastern part of the corridor, the analysis has been carried out by dividing the corridor into two stretches and the comparison is presented for the western part of the corridor as free from the influence of the construction area).

The following were concluded:

- The average delay for the entire corridor decreased for both directions of travel for all the periods studied. Comparison of the before and after conditions shows that, while the overall performance of the corridor has improved, operations for some intersections (N. Cattleman for example) deteriorated.
- The average travel time in the EB direction had a slight decrease in the AM peak (0.24 min or 4.77%), while it increased for both the Off Peak and PM peak (2.88 min or 3.95%, and 5.77 min or 20.60% respectively). In the WB direction, the average travel time had a minimal increase in the AM peak by 0.19 min (1.36%), while it decreased for Off Peak and decreased significantly for PM peak (1.91 min or 10.25%, and 15.02 min or 44.58%, respectively).
- The intersection through movement delay in the EB direction decreased for 14 out of the 17 intersections by an average of 9.05 sec/intersection. The intersection through movement delay in the WB direction decreased at 14 out of the 17 intersections by an average of 13.43 sec/intersection.

- The average queue length for the University Parkway and US 301 intersection decreased during the AM peak by 6.6 vehicles, with negligible variation for the other periods. The average queue length for the University Parkway and North Cattlemen Rd intersection increased during all three periods as follows: 0.73 vehicles in the AM peak, 3.27 vehicles in the Off Peak, and 2.57 vehicles in the PM peak. The delay at this intersection is considered mainly due to the construction site.
- Generally, the percentage of trucks is low (1.75% and 3.90% at University Pkwy & North Cattlemen Rd and University Pkwy & US 301, respectively), however, the high percentage observed along the EB at the University Pkwy & US 301 intersection could have resulted in an increase in average travel time observed for that direction of travel.
- The overall reduction of traffic flow at both critical intersections is not high enough to be the main reason for the observed performance improvement (reduction of less than 78 pce/h for the University Pkwy & North Cattlemen Rd intersection, and 35 pce/h for the University Pkwy & US 301 intersection).
- While the results for the entire corridor had some issues due construction, the evaluation parameters (i.e delay, queues etc.) showed improvement across the board for western part (West of Medici Ct.) of the corridor.

D.2 CORRIDOR INFORMATION

Figure D-1 provides a schematic of the University Parkway, Sarasota corridor. Table D-1 lists the intersections along the corridor. Two intersections (North Cattlemen Rd and US 301) were selected as the critical intersections along the corridor and detailed turning movement and queue counts were collected at these. Figure D-2 provides the lane configuration of these two critical intersections.

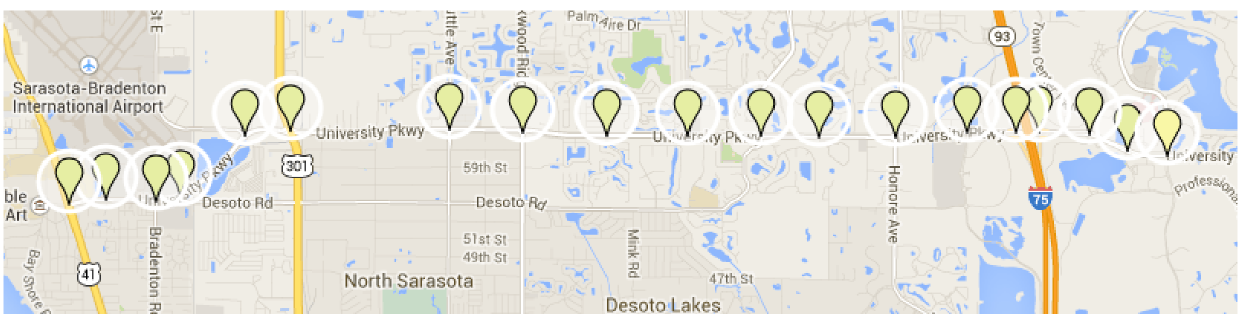
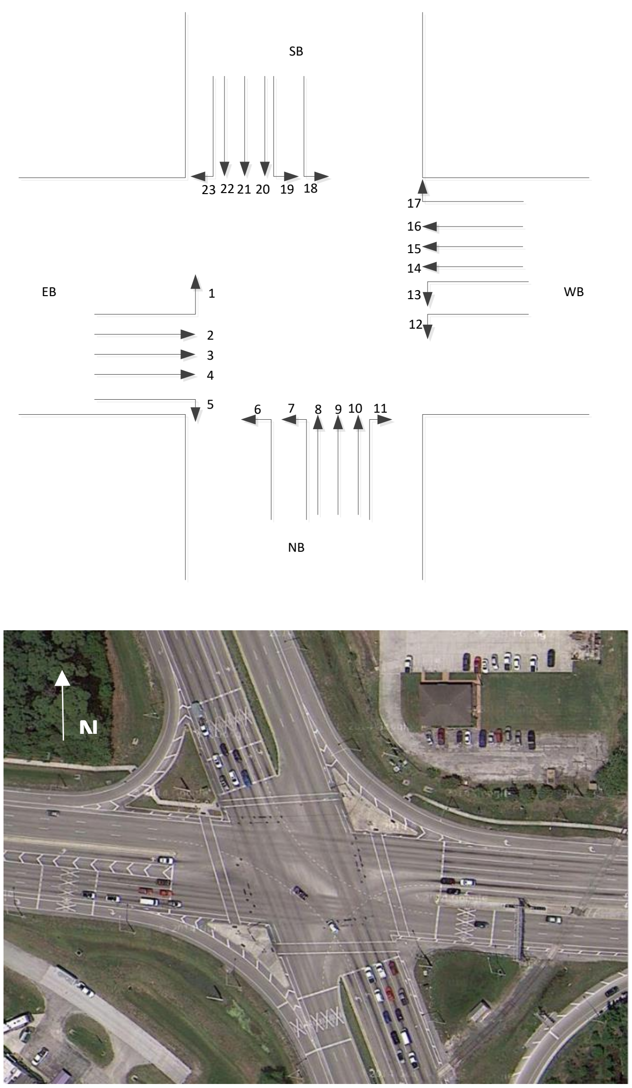


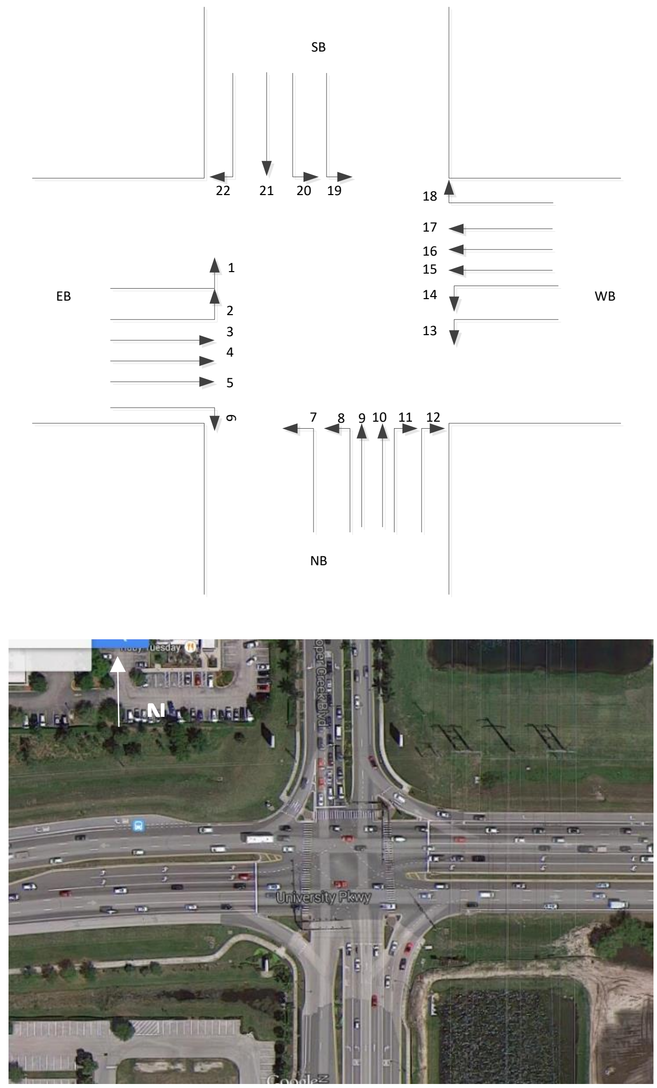
Figure D-1 Schematic of the University Parkway, Sarasota Corridor.

Table D-1 Intersections along the University Parkway, Sarasota Corridor

	Signalized Intersection	**Distance in Between**	**Unsignalized Intersections in Between**
1	Lakewood Ranch Blvd		
2	Town Center Pkwy	0.27 miles	0
3	Market St	0.30 miles	1
4	I-75 NB	0.34 miles	0
5	I-75 SB	828 ft	0
6	**North Cattlemen Rd. / Cooper Creek B.**	**0.32 miles**	**0**
7	Honore Ave	0.50 miles	4
8	Medici Ct	0.52 miles	3
9	Longwood Run	0.40 miles	0
10	Whitfield Ave	0.51 miles	1
11	Country Park Way	0.58 miles	0
12	Lockwood Ridg Rd	0.58 miles	1
13	Tuttle Ave	0.50 miles	1
14	**US 301**	**1.09 miles**	**10**
15	County Line Rd	0.33 miles	1
16	Desoto Road	0.61 miles	1
17	Bradenton Road	935 ft	0
18	Airport Cir	0.35 miles	1
Total	19 Intersections	7.8 miles	



(a) University Pkwy & US 301



(b) University Pkwy & North Cattlemen Rd.

Cooper Creek B.

Figure D-2 Schematic and Overview of Critical Intersections

As mentioned above, due to the influence that the construction sites had on the eastern part of the corridor, apart from the standard comparisons, additional analysis has been done considering only the western part of the corridor (West of Medici Ct.) not affected by the construction sites (I-75 and mall at N Cattleman Rd). The following figure shows the stretch of the corridor which has been separated for a better comparison.

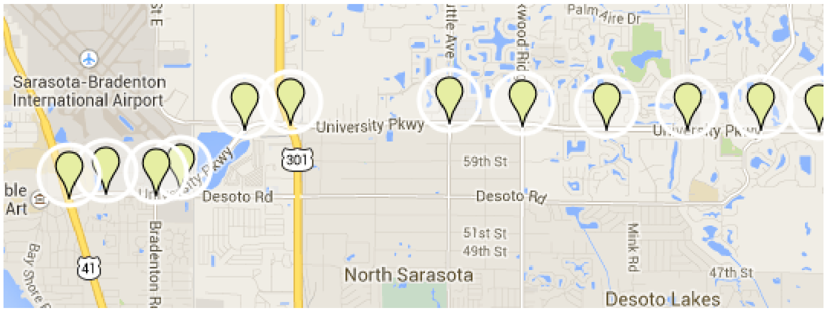


Figure D-3 Schematic of the Western part of the University Parkway Corridor considered for delay comparison. Sarasota

D.3 PERFORMANCE MEASURES

Five performance measures were evaluated: Link/Route Travel Time, Delay at Intersections, Queue Length (critical intersections), Queue-to-Lane Storage Ratio (critical intersections), and PCE Flows (critical intersections).

D.3.1 ROUTE TRAVEL TIME

The average travel time (min) along the route was measured using a floating car. During the before data collection 11 runs for the AM peak, 12 for the OFF peak and 8 for the PM peak were carried out. Table D-1.1 provides the route travel time for the before data.

Before Study (Mar.18 & Mar.19, 2015)

Table D-1.1 Route Travel Time (min)

Route TT (min)	AM Peak	Off Peak	PM Peak	Average
University Parkway, EB	16.57	17.92	23.01	**19.16**
University Parkway, WB	14.91	19.33	34.65	**22.96**

After Study (Mar.14 & Mar.15, 2017)

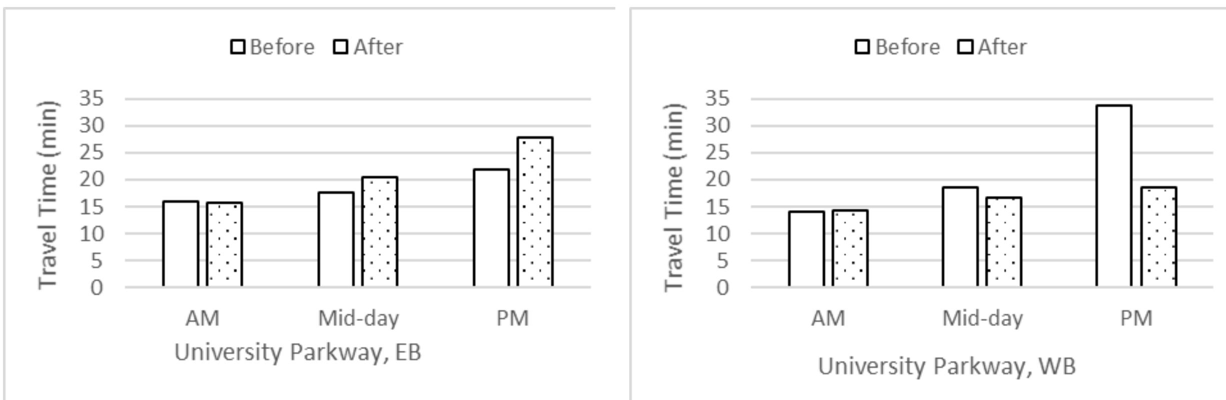
Table D-1.2 Route Travel Time (min)

Route TT (min)	AM Peak	Off Peak	PM Peak	Average
University Parkway, EB	15.78	20.42	27.75	21.31
University Parkway, WB	14.30	16.73	18.67	16.57

Comparisons of Before and After Travel Times

Table D-1.3 Change in Percentage of Route Travel Time (After – Before)

Route TT (min)	AM Peak	Off Peak	PM Peak	Average
University Parkway, EB	-0.24 (-4.77%)	2.88 (13.95%)	5.77 (20.60%)	**2.80 (11.22%)**
University Parkway, WB	0.19 (1.36%)	-1.91 (-10.25%)	-15.02 (-44.58%)	**-5.57 (-25.17%)**



(a) University Parkway, EB (b) University Parkway, WB

Figure D-1.1 Travel Times Along University Parkway, Sarasota

Discussion

The following can be concluded from the comparison of travel times:

- The travel time along University Parkway in the EB decreased in the AM peak by 0.24 min (4.77%), while the EB direction carried the majority of the traffic during the AM peak. Travel time increased during both the Off Peak and PM peak. The highest increase occurred during the PM peak when the demand was relatively higher, with an increase of 5.77 min (20.60%).
- The travel time along University Parkway in the WB increased in the AM peak by 0.19 min (1.36%), when the demand was relatively lower. Travel time decreased during both the Off Peak and PM peak. The highest decrease occurred during the PM peak when the demand was relatively higher, with a decrease of 15.02 min (44.58%).
- In the before data, the AM peak travel time along University Parkway in the EB direction was higher than that of the WB direction. The reverse occurred during the Off Peak and PM peaks, with WB the direction with the higher traffic and higher travel time. However, in the after study, the heavily traveled WB direction had a

lower travel time. It appears that the system favored the peak direction while for the opposing lower-demand direction travel time increased by 2.80 min (11.22%).

- Three incidents occurred during the data collection, which may have slightly affected travel times: 3/14/17, AM Peak to early PM Peak, EB direction, Lockwood Ridge Rd. to Country Park, one lane closure 200 ft downstream of the study site; 3/15/2017, PM Peak (5:42 PM), EB, Accident after the Medici Ct. intersection (3 cars); 3/15/2017, PM Peak (5:31 PM), WB, Airport Circle accident, rear-end, severity: 2 (possible injury). During these incidents, the research team did not experience significant changes in delay.

D.3.2 DELAY

Delay (sec) at each intersection along the corridor was also obtained using the floating car measurements. As indicated earlier, the after study excluded the Lakewood Ranch Intersection.

Before Study (Mar.18 & Mar.19, 2015)

Table D-2.1 Delay (sec) for each Intersection Through Movement Along the EB Direction

Delay at Intersections (sec)	AM Peak	Off Peak	PM Peak
Airport Cir	11.72	22.47	1.65
Bradenton Rd	11.44	20.93	1.65
Desoto Rd	1.32	5.3	19.8
County Line Rd	1.2	5.8	29.85
US 301	79.6	60.8	80.2
Tuttle Ave	58.07	55.91	52.31
Lockwood Ridge Rd	6.7	57.38	56.67
Country Park Way	2.19	7.69	3.79
Whitfield Ave	27.6	15.1	16.65
Longwood Run	17.62	14.82	20.17
Medici Ct	7.71	6.97	16.47
Honore Ave	55.6	58.06	114.91
N Cattleman Rd	30.69	53.63	141.78
I-75 SB	33.6	69.3	144.3

Delay at Intersections (sec)	AM Peak	Off Peak	PM Peak
I-75 NB	30.87	34.77	6.87
Market St	49.98	21.85	23.49
Town Center Pkwy	6.58	14.32	61.27
Lakewood Ranch Blvd	15.33	4.93	44.28

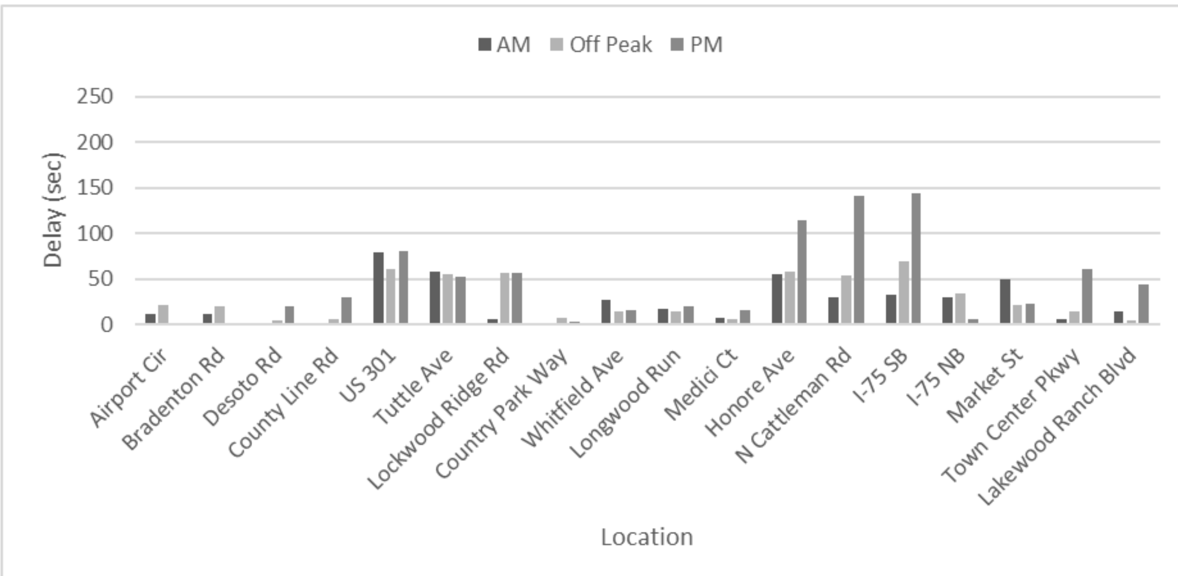


Figure D-2.1 Delay (sec) for each Intersection Through Movement Along the EB Direction

Table D-2.2 Delay (sec) for each Intersection Through Movement Along the WB Direction

Delay at Intersections (sec)	AM Peak	Off Peak	PM Peak
Lakewood Ranch Blvd	30.52	23.82	40.22
Town Center Pkwy	13.53	30.03	6.03
Market St	5.82	28.52	83.01
I-75 NB	65.15	85.24	161.75
I-75 SB	23.07	24.27	18.12
N Cattleman Rd	9.3	31.4	48.15
Honore Ave	24.23	41.13	50.13
Medici Ct	11.76	12.46	4.81
Longwood Run	9.37	44.67	10.34
Whitfield Ave	13.22	28.82	4.72
Country Park Way	3.4	10.7	12.3
Lockwood Ridge Rd	25.59	55.89	34.54
Tuttle Ave	7.87	45.77	31.17
US 301	57.7	42.8	68.55
County Line Rd	16.1	32.1	22.15
Desoto Rd	1.5	11.6	0.9
Bradenton Rd	1.1	4	17.55
Airport Cir	11.77	36.3	55.6

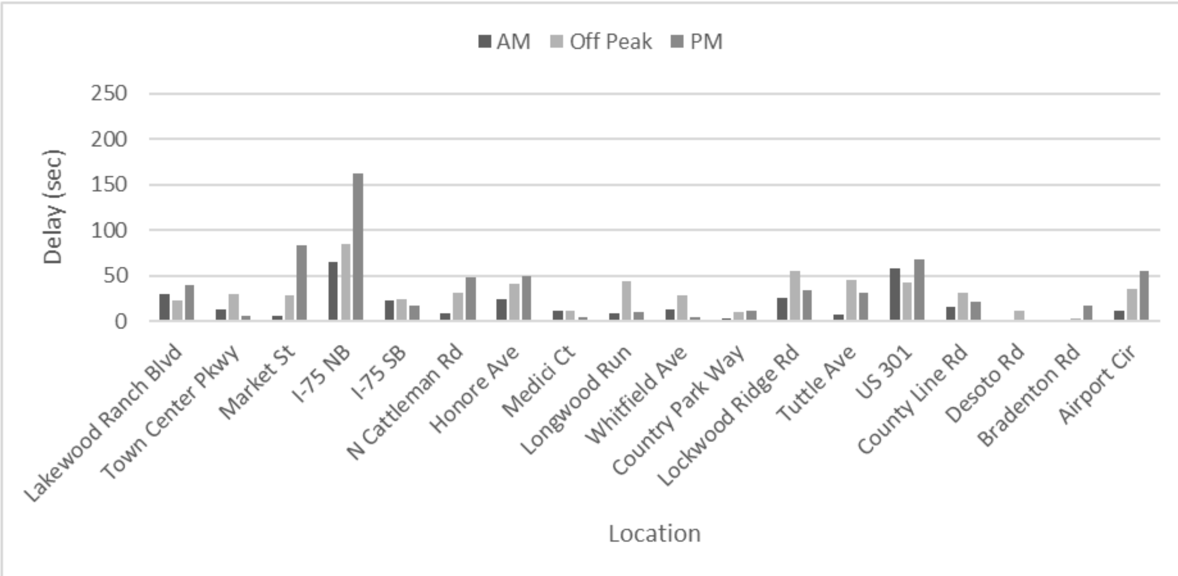


Figure D-2.2 Delay (sec) for each Intersection Through Movement Along the WB Direction

After Study (Mar.14 & Mar.15, 2017)

Table D-2.3 Delay (sec) for each Intersection Through Movement Along the EB Direction - After

Delay at Intersections (sec)	AM Peak	Off Peak	PM Peak
Airport Cir	0.75	3.67	17.40
Bradenton Rd	1.88	2.67	3.00
Desoto Rd	1.75	0.00	0.00
County Line Rd	16.00	8.33	29.60
US 301	**18.38**	**47.17**	**16.00**
Tuttle Ave	0.50	15.00	59.60
Lockwood Ridge Rd	10.38	17.33	59.40
Country Park Way	0.00	0.00	0.00
Whitfield Ave	3.50	0.00	0.00
Longwood Run	19.88	17.50	1.40
Medici Ct	5.63	0.00	36.20
Honore Ave	17.88	40.00	161.40

Delay at Intersections (sec)	AM Peak	Off Peak	PM Peak
N Cattleman Rd	**39.00**	**108.67**	**227.50**
I-75 SB	58.13	82.43	48.00
I-75 NB	7.75	0.00	13.20
Market St	11.13	18.43	30.60
Town Center Pkwy	0.00	6.71	4.40

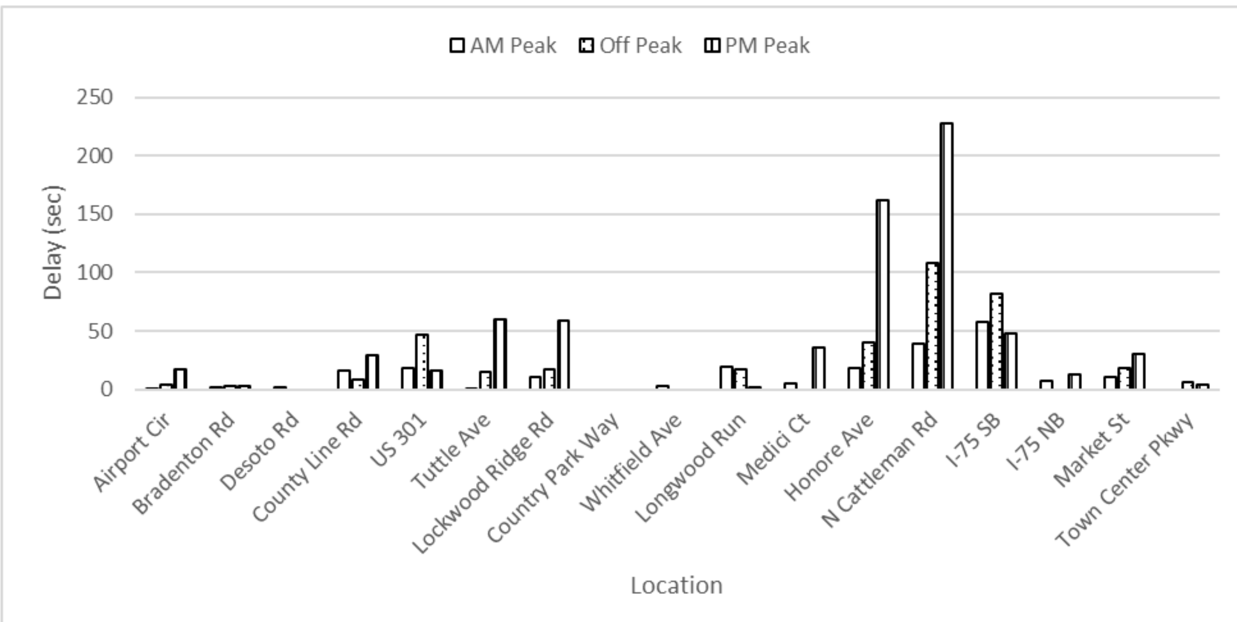


Figure D-2.3 Delay (sec) for each Intersection Through Movement Along the EB Direction - After

Table D-2.4 Delay (sec) for each Intersection Through Movement Along the WB Direction - After

Delay at Intersections (sec)	AM Peak	Off Peak	PM Peak
Town Center Pkwy	17.63	9.67	0.40
Market St	10.50	16.33	29.40
I-75 NB	10.88	0.00	94.80
I-75 SB	0.00	0.00	0.00
N Cattleman Rd	**23.38**	**29.17**	**27.00**

Delay at Intersections (sec)	AM Peak	Off Peak	PM Peak
Honore Ave	18.13	0.00	4.00
Medici Ct	4.38	15.50	0.00
Longwood Run	7.38	0.00	0.00
Whitfield Ave	7.13	17.67	3.60
Country Park Way	2.50	18.00	23.80
Lockwood Ridge Rd	15.50	47.00	32.00
Tuttle Ave	12.88	84.57	38.00
US 301	**6.75**	**31.67**	**25.75**
County Line Rd	0.00	3.17	0.00
Desoto Rd	9.71	75.50	20.60
Bradenton Rd	4.57	0.00	0.00
Airport Cir	1.86	5.80	4.40

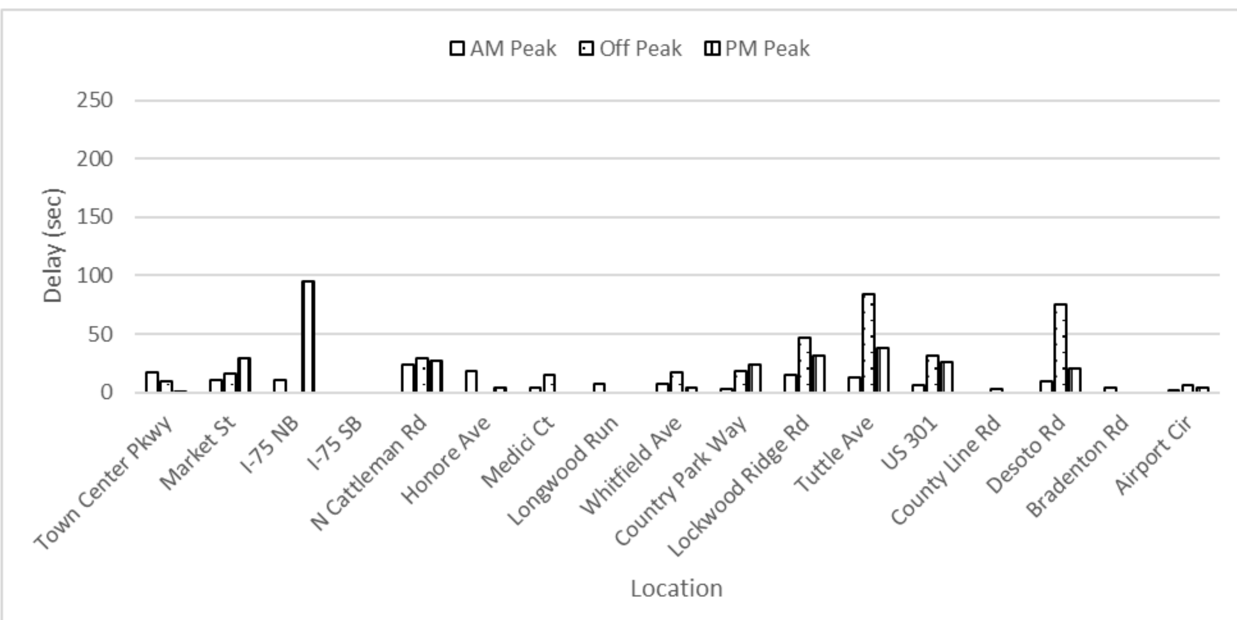


Figure D-2.4 Delay (sec) for each Intersection Through Movement Along the WB Direction - After

Comparisons of Before and After Intersection Delay Times

The differences in delay between the before and after study are shown in Table A-2.5 and Table D-2.6. The tables are color-coded as follows: green shows significant improvement, yellow shows modest change (either improvement or deterioration), and red shows significant deterioration in delay. Several gradations of each color are used to represent different variations within each classification.

Table D-2.5 Difference in Delay (sec) for each Intersection Through Movement Along the EB Direction

Delay at Intersections (sec)	AM Peak	Off Peak	PM Peak	Average
Airport Cir	-10.97	-18.80	15.75	**-4.67**
Bradenton Rd	-9.57	-18.26	1.35	**-8.83**
Desoto Rd	0.43	-5.30	-19.80	**-8.22**
County Line Rd	14.80	2.53	-0.25	**5.69**
US 301	**-61.23**	**-13.63**	**-64.20**	**-46.35**
Tuttle Ave	-57.57	-40.91	7.29	**-30.40**
Lockwood Ridge Rd	3.68	-40.05	2.73	**-11.21**
Country Park Way	-2.19	-7.69	-3.79	**-4.56**
Whitfield Ave	-24.10	-15.10	-16.65	**-18.62**
Longwood Run	2.26	2.68	-18.77	**-4.61**
Medici Ct	-2.09	-6.97	19.73	**3.56**
Honore Ave	-37.73	-18.06	46.49	**-3.10**
N Cattleman Rd	**8.31**	**55.04**	**85.72**	**49.69**
I-75 SB	24.53	13.13	-96.30	**-19.55**
I-75 NB	-23.12	-34.77	6.33	**-17.19**
Market St	-38.86	-3.42	7.11	**-11.72**
Town Center Pkwy	-6.58	-7.61	-56.87	**-23.69**
Average	**-12.94**	**-9.25**	**-4.95**	**-9.05**

Table D-2.6 Difference in Delay (sec) for each Intersection Through Movement Along the WB Direction

Delay at Intersections (sec)	AM Peak	Off Peak	PM Peak	Average
Town Center Pkwy	4.10	-20.36	-5.63	**-7.30**
Market St	4.68	-12.19	-53.61	**-20.37**
I-75 NB	-54.28	-85.24	-66.95	**-68.82**
I-75 SB	-23.07	-24.27	-18.12	**-21.82**
N Cattleman Rd	**14.08**	**-2.23**	**-21.15**	**-3.10**
Honore Ave	-6.11	-41.13	-46.13	**-31.12**
Medici Ct	-7.39	3.04	-4.81	**-3.05**
Longwood Run	-2.00	-44.67	-10.34	**-19.00**
Whitfield Ave	-6.10	-11.15	-1.12	**-6.12**
Country Park Way	-0.90	7.30	11.50	**5.97**
Lockwood Ridge Rd	-10.09	-8.89	-2.54	**-7.17**
Tuttle Ave	5.01	38.80	6.83	**16.88**
US 301	**-50.95**	**-11.13**	**-42.80**	**-34.96**
County Line Rd	-16.10	-28.93	-22.15	**-22.39**
Desoto Rd	8.21	63.90	19.70	**30.60**
Bradenton Rd	3.47	-4.00	-17.55	**-6.03**
Airport Cir	-9.91	-30.50	-51.20	**-30.54**
Average	**-8.67**	**-12.45**	**-19.18**	**-13.43**

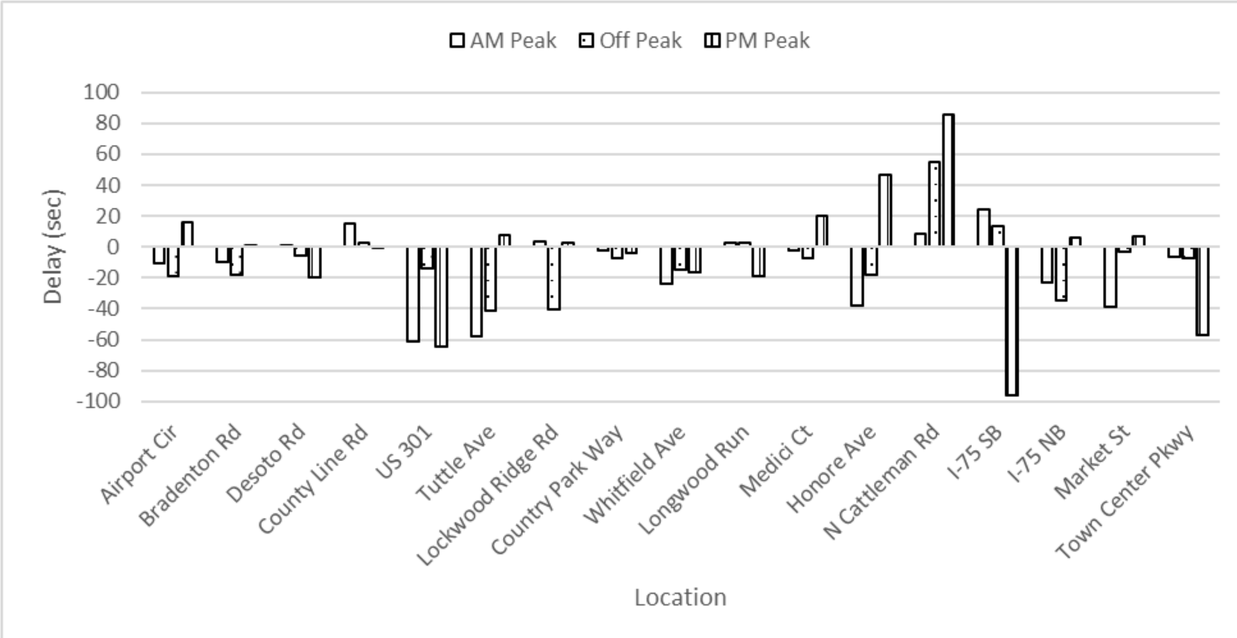


Figure D-2.5 Difference in Delay (sec) for each Intersection Through Movement Along the EB Direction

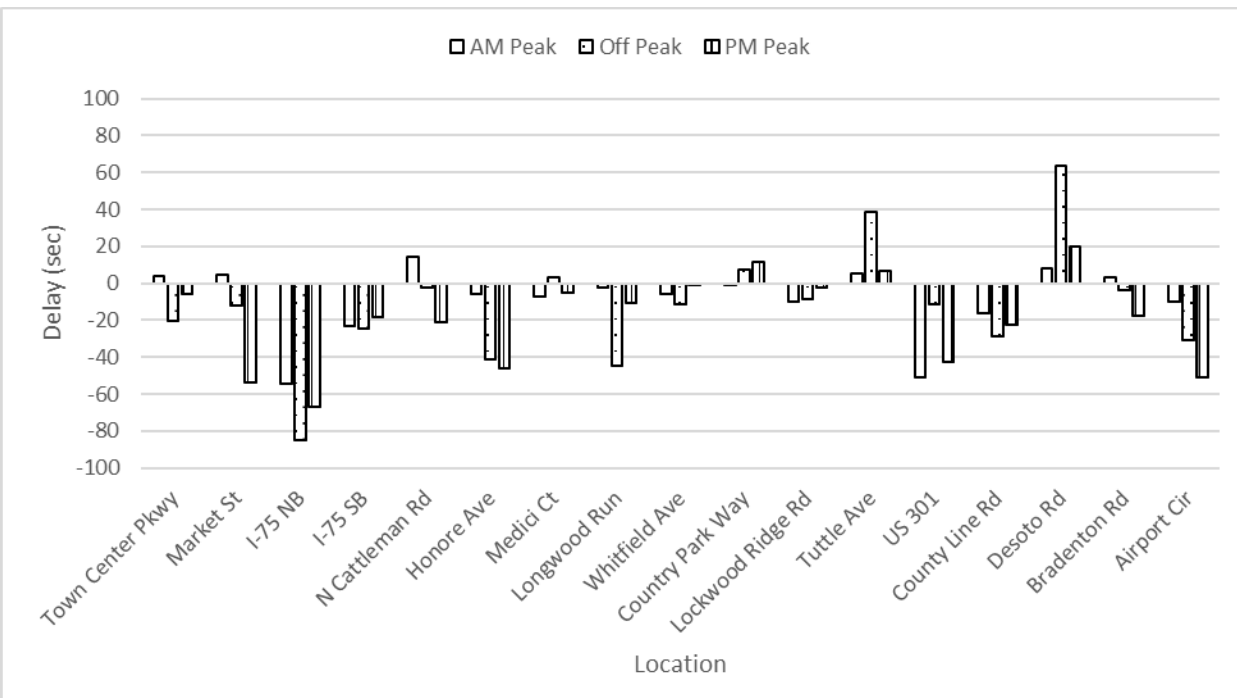


Figure D-2.6 Difference in Delay (sec) for each Intersection Through Movement Along the WB Direction

Since the influence of construction on the eastern side of the corridor can veil the effectiveness of the adaptive system, western side (West of Medici Ct.) of the corridor has been considered separately. The differences in delay for the rest of intersections in the through movement of the instrumented vehicle along both directions are represented in the following figures:

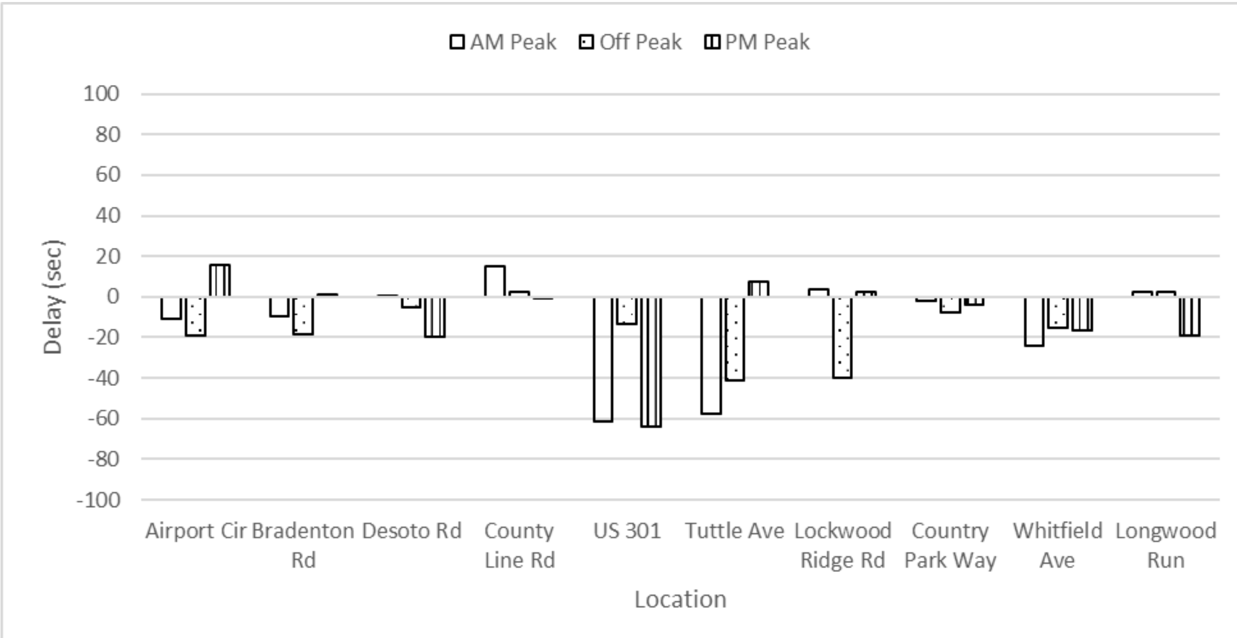


Figure D-2.7 Difference in Delay (sec) for each Intersection on Western part of the corridor Along the EB Direction

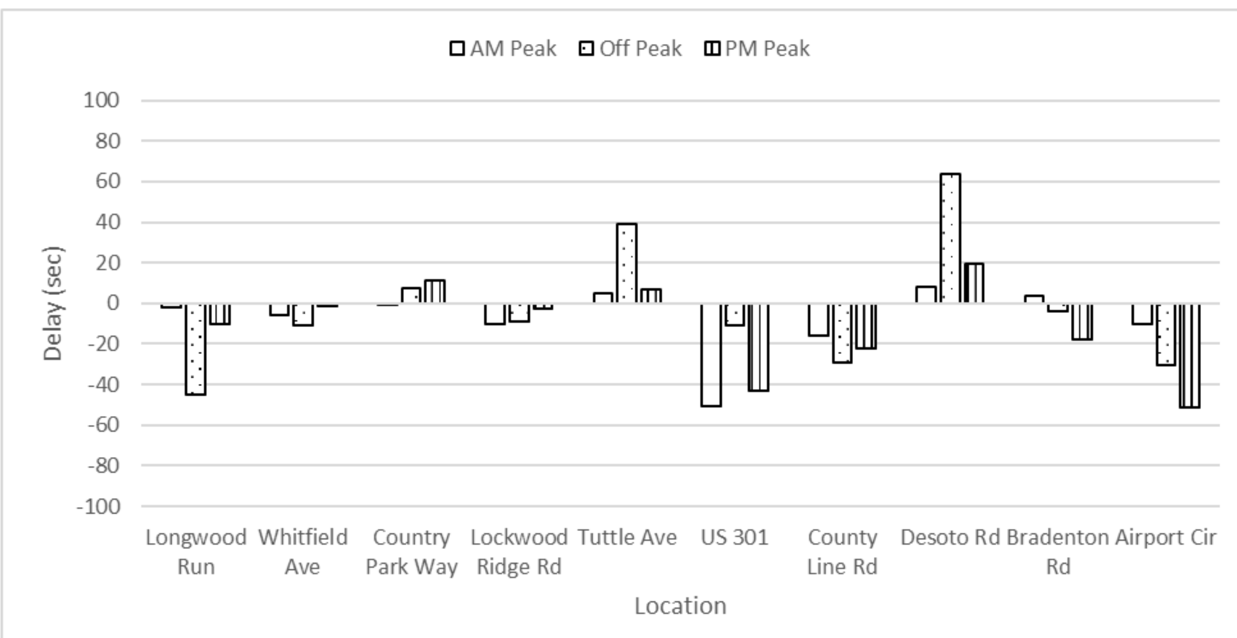


Figure D-2.8 Difference in Delay (sec) for each Intersection on Western part of the corridor Along the WB Direction

Discussion

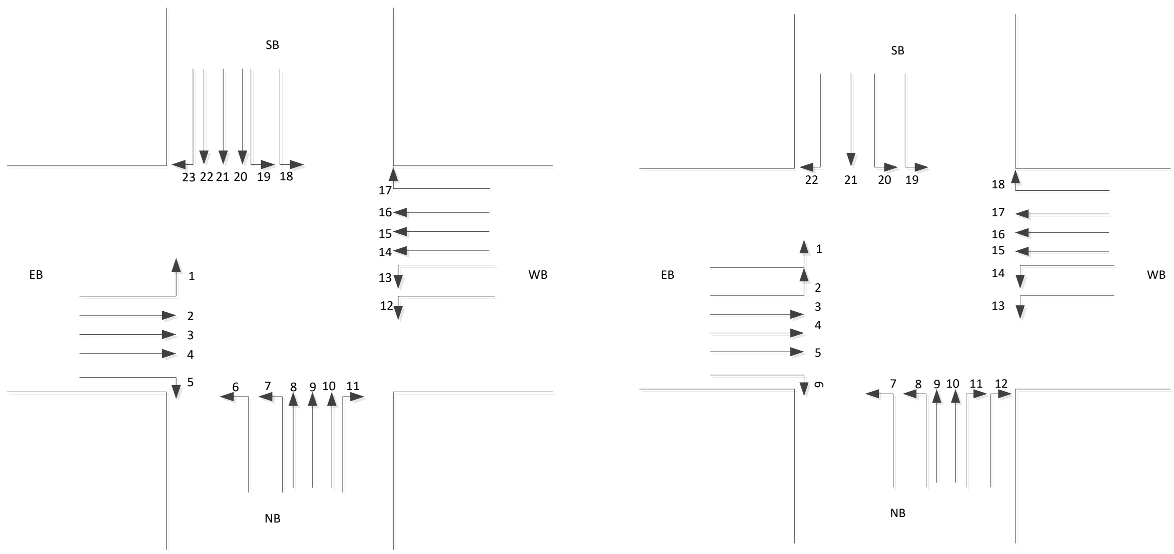
The following can be concluded from the comparison of delays:

- The average delay across the entire corridor decreased for both directions of travel for all the periods studied.
- The intersection delay in the EB direction decreased at 14 out of the 17 intersections by an average of 9.05 sec/intersection. Despite the overall decrease of delay in the EB direction, delay increased at a few intersections (N Cattleman Rd, Honore Ave, and Medici Ct) at the western part of the corridor, where the demand is higher in both directions, especially during the PM peak. Moreover, during that period, queues in the EB through movement reached around 40 vehicles.
- The intersection delay in the WB direction decreased at 14 out of the 17 intersections by an average of 13.43 sec/intersection.
- The intersection with I75 SB in the WB direction, the fact that the delay is null is considered due to the coordination with the previous intersection (I75 NB).
- Once the construction effects have been removed from the comparison and, therefore, having only one critical intersection in the analysis of delay, it can be seen that especially for the EB there is an overall improvement at all intersections. For the WB, only Desoto Road and Tuttle Avenue intersections, both during the off-peak period, show an increase in delay of 69.9 and 38.8 sec, respectively.

D.3.3 QUEUE LENGTH

Queue length (number of vehicles/lane) is presented by movement and by time period. This measure is used to evaluate oversaturated conditions at the critical intersections along the study corridors. Note that the queue length reported here is the observed maximum number of vehicles queued during each cycle, and does not represent the total number of vehicles that may have stopped during the cycle. During some time periods, because of cycle failure, vehicles need to stop multiple times before passing through the intersection.

Figure D 3.1 presents the schematic of the lane configurations at the two critical intersections. Queue length is reported for each of these lanes.



(a) University Pkwy & US 301

(b) University Pkwy & North Cattlemen Rd. / Cooper Creek B.

Figure D-3.1 Lane Configuration of Critical Intersections

Before Study (Mar.18 & Mar.19, 2015)

Table D-3.1 Average Queue Length by Lane (#vehs/lane) at University Pkwy & US 301

Time Period	Time Segment	EB					NB						WB						SB					
		Left	Through			Right	Left		Through			Right	Left		Through			Right	Left		Through			Right
Lane Number		**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**	**13**	**14**	**15**	**16**	**17**	**18**	**19**	**20**	**21**	**22**	**23**
AM Peak	1	4.6	7.4	8.4	9.4	0	3.8	4.4	8.4	8.8	2.6	0	16.8	15.4	14.2	14	13.4	0.2	9.4	9	13.6	13.2	12.2	0.6
	2	3.2	7.8	8.4	8.4	0	2.2	3.2	9.6	9.6	1.8	0	11.6	10.8	12.2	10.2	11.8	0	7.8	7.4	11.8	10.6	10.8	0.4
	3	5	11	12	12	0	4	4	5	6	0	0	9.4	9.6	13.4	14.2	14	0	6.6	7.6	9.4	10.2	10.2	0
	4	6	10	11	12	0	5	5	10	10	3	0	9.2	8.8	10.8	10.8	10.8	0	7	7	8.6	8.4	9	0.2
	Average	**4.7**	**9.05**	**9.95**	**10.45**	**0**	**3.75**	**4.15**	**8.25**	**8.6**	**1.85**	**0**	**11.75**	**11.15**	**12.65**	**12.3**	**12.5**	**0.05**	**7.7**	**7.75**	**10.85**	**10.6**	**10.55**	**0.3**
Off Peak	1	7	6	9	10	0.17	2	4	10	11	5	1.17	3.5	3.33	7.17	7.17	7.83	1.5	6.33	5.67	5.33	6	6.5	2.5
	2	1	5	5	7	1.6	5	6	5	5	1	0.8	6	5.4	10.6	10.4	11.8	0.8	5.6	4.8	5.6	6.2	6.4	1
	3	5	11	12	12	0.6	4	4	5	6	0	2.8	6.8	6.4	10.4	10.2	11	0.4	5.6	5.4	6.8	6.8	6.4	0.6
	4	6	10	11	12	1.8	5	5	10	10	3	3	7	6.2	11.2	10	9.2	0.6	7.6	6	6.8	8.2	6.6	0.4

Time Period	Time Segment	EB					NB						WB						SB					
		Left	Through			Right	Left		Through			Right	Left		Through			Right	Left		Through			Right
Lane Number		1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23
	Average	**4.75**	**8**	**9.25**	**10.25**	**1.04**	**4**	**4.75**	**7.5**	**8**	**2.25**	**1.94**	**5.83**	**5.33**	**9.84**	**9.44**	**9.96**	**0.83**	**6.28**	**5.47**	**6.13**	**6.8**	**6.48**	**1.13**
PM Peak	1	7	6	9	10	0.2	2	4	10	11	5	2.8	5.4	6.4	7	10.4	8	1.4	9	9.4	10.2	12.6	10.2	0.2
	2	1	5	5	7	0.8	5	6	5	5	1	0.8	4	5.6	8.8	11	10.6	3.8	9	10.6	9.6	10.2	9	1.2
	3	5	11	12	12	1.4	4	4	5	6	0	5.6	16	4	5.6	4.6	9.2	13.2	5.8	10.4	12	10.8	11.2	9.4
	4	6	10	11	12	0	5	5	10	10	3	2	18	5.4	6.2	3.8	10.2	10.2	2.8	8.4	8.6	6.8	8.2	6.8
	Average	**4.75**	**8**	**9.25**	**10.25**	**0.6**	**4**	**4.75**	**7.5**	**8**	**2.25**	**2.8**	**10.85**	**5.35**	**6.9**	**7.45**	**9.5**	**7.15**	**6.65**	**9.7**	**10.1**	**10.1**	**9.65**	**4.4**

Table D-3.2 Average Queue Length (#vehs/lane) at University Pkwy & US 301

Time Period	EB(Main)			NB			WB(Main)			SB		
	Left	Through	Right	Left	Through	Right	Left	Through	Right	Left	Through	Right
AM Peak	4.7	9.82	0	7.9	6.23	0	11.45	12.48	0.05	7.73	10.67	0.3
Off Peak	4.75	9.17	1.04	8.75	5.92	1.94	5.58	9.75	0.83	5.88	6.47	1.13
PM Peak	4.75	9.17	0.6	8.75	5.92	2.8	8.1	7.95	7.15	8.18	9.95	4.4

Table D-3.3 Average Queue Length by Lane (#vehs/lane) at University Pkwy & North Cattlemen Rd. / Cooper Creek B.

Time Period	Time Segment	EB						NB						WB						SB			
		Left		Through			Right	Left		Through		Right		Left		Through			Right	Left		Through	Right
Lane Number		**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**	**13**	**14**	**15**	**16**	**17**	**18**	**19**	**20**	**21**	**22**
AM Peak	1	2	1.33	9	9	12.5	0.17	1.5	0.5	0.33	0	2.33	1.33	5.33	4.83	10.5	10.83	10.5	0.83	5.33	4.67	2.5	2.33
	2	3	2.33	10	8.5	11.17	0	1.17	0.5	0.67	0.17	2.33	2.83	5.17	5	8.17	7.5	6	1.33	5.33	4.67	2	2.17
	3	3.33	2.67	5.67	3.83	7.5	0	1.5	1.5	1	0.5	2.83	1.67	4.67	3.83	11.5	11.83	11.67	0	5.67	5.83	2	0.17
	4	2.43	2.29	5.29	5.14	7.57	0.29	1	0.86	0.86	1	1.71	2.43	3.5	3	11	11	10.67	0.5	6.17	5.33	1.5	0.5
	Average	**2.69**	**2.15**	**7.49**	**6.62**	**9.68**	**0.11**	**1.29**	**0.84**	**0.71**	**0.42**	**2.3**	**2.07**	**4.67**	**4.17**	**10.29**	**10.29**	**9.71**	**0.67**	**5.63**	**5.13**	**2**	**1.29**
Off Peak	1	7.67	6.83	13.5	13.83	15.83	2.5	4	7.17	4.67	5.33	4.33	5	13.17	12.17	13.33	12.83	13	4.67	8.17	7.67	8.83	3.17
	2	9.29	8.86	18	18	18	2.71	5.14	9.71	4.29	5.57	8.57	9	15.83	15.33	18.17	17.83	17.17	9.5	13.67	12.5	11.5	4.17
	3	8.5	7.17	18	18	18	3	3.67	6.83	4.33	6.17	7	7.33	15.67	15.5	15.83	15.5	14.33	5.5	16.67	12.5	11.17	5.17
	4	8	7.5	15.33	14.5	15.33	2.17	5	7.17	5.17	6	7.5	8.33	8.17	7.67	13.83	13.67	11.83	4.67	9.83	9.17	10.67	4.17
	Average	**8.36**	**7.59**	**16.21**	**16.08**	**16.79**	**2.6**	**4.45**	**7.72**	**4.61**	**5.77**	**6.85**	**7.42**	**13.21**	**12.67**	**15.29**	**14.96**	**14.08**	**6.08**	**12.08**	**10.46**	**10.54**	**4.17**

Time Period	Time Segment	EB						NB						WB						SB			
		Left		Through			Right	Left		Through		Right		Left		Through			Right	Left		Through	Right
Lane Number		**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**	**13**	**14**	**15**	**16**	**17**	**18**	**19**	**20**	**21**	**22**
PM Peak	1	4.17	8.33	25.00*	25.00*	25.00*	0.83	4.67	7.67	4.5	4.33	13.5	10.67	10	10	13	13	12.83	5.5	12	18.33	6.67	3.33
	2	7.8	8.8	25.00*	25.00*	25.00*	1.4	4	7.4	3.4	8	10.4	13.6	8.33	7.5	13.33	13.33	13.17	3.83	11.67	15.5	6.17	3
	3	4.5	6	25.00*	25.00*	25.00*	0.5	8.33	6.5	5.33	4.33	12.33	9	8.17	8	11.5	10.83	10.5	3.33	12	18.33	8.83	2.67
	4	4.86	5.57	25.00*	25.00*	25.00*	1.5	4.5	5.17	4.83	5.5	10	9.5	9.17	9.17	11.33	11.33	11.17	5.17	8.83	8.67	6.17	2.33
	Average	**5.33**	**7.18**	**25**	**25**	**25**	**1.06**	**5.38**	**6.68**	**4.52**	**5.54**	**11.56**	**10.69**	**8.92**	**8.67**	**12.29**	**12.13**	**11.92**	**4.46**	**11.13**	**15.21**	**6.96**	**2.83**

Note: * indicates cycle failure

Table D-3.4 Average Queue Length (#vehs/lane) at University Pkwy & North Cattlemen Rd. / Cooper Creek B.

Time Period	EB(Main)			NB			WB(Main)			SB		
	Left	**Through**	**Right**	**Left**	**Through**	**Right**	**Left**	**Through**	**Right**	**Left**	**Through**	**Right**
AM Peak	2.42	7.93	0.11	1.07	0.57	2.18	4.42	10.1	0.67	5.38	2	1.29
Off Peak	7.98	16.36	2.6	6.09	5.19	7.13	12.94	14.78	6.08	11.27	10.54	4.17
PM Peak	6.25	25	1.06	6.03	5.03	11.13	8.79	12.11	4.46	13.17	6.96	2.83

After Study (Mar. 14 & Mar. 15, 2017)

Table D-3.5 Average Queue Length by Lane (#vehs/lane) at University Pkwy & US 301 – After Study

Time Period	**Time Segment**	**EB (Main)**					**NB**						**WB (Main)**						**SB**					
		Left	**Through**			**Right**	**Left**		**Through**			**Right**	**Left**		**Through**			**Right**	**Left**		**Through**			**Right**
Lane Number		**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**	**13**	**14**	**15**	**16**	**17**	**18**	**19**	**20**	**21**	**22**	**23**
AM Peak	1	2.50	4.83	4.67	3.67	0.00	3.00	3.00	4.00	3.00	1.00	0.00	7.20	6.20	2.60	2.00	1.60	0.00	5.00	5.00	7.40	8.00	8.00	0.00
	2	3.57	3.71	4.57	4.57	0.00	2.14	2.43	6.86	4.71	3.86	0.00	8.67	7.50	2.67	3.33	2.67	0.00	4.33	4.71	8.29	9.00	9.00	0.00
	3	3.50	3.38	4.88	4.00	0.00	1.86	4.57	4.86	6.00	3.43	0.00	6.86	7.57	2.29	2.86	2.29	0.00	4.57	4.71	7.71	8.29	8.29	0.00
	4	3.33	3.17	3.67	4.00	0.00	3.67	3.17	6.17	6.17	3.00	0.00	7.33	7.00	1.33	3.00	2.00	0.00	9.33	8.83	13.33	13.33	13.17	0.00
	Average	**0.36**	**0.53**	**0.39**	**0.26**	**0.00**	**0.67**	**0.64**	**1.04**	**1.11**	**0.91**	**0.00**	**0.58**	**0.47**	**0.44**	**0.40**	**0.34**	**0.00**	**1.76**	**1.51**	**2.08**	**1.84**	**1.78**	**0.00**
Off Peak	1	5.83	7.17	6.67	8.17	0.00	4.20	6.60	15.20	12.20	6.20	0.00	5.20	4.80	3.00	4.20	4.40	0.00	6.83	7.17	4.83	7.33	6.83	0.00
	2	4.60	4.83	7.83	8.17	0.00	5.33	4.33	6.33	7.00	2.60	0.00	4.17	5.00	3.67	5.00	2.67	0.00	5.83	6.00	6.33	8.50	7.50	0.00
	3	3.17	5.50	7.17	6.50	0.00	2.40	3.20	15.40	14.60	7.80	0.00	4.67	5.33	2.83	4.83	5.00	0.00	5.80	5.60	8.60	9.40	8.80	0.00
	4	5.80	7.00	7.00	6.20	0.00	5.00	6.60	16.20	13.20	8.60	0.00	5.67	6.17	1.83	3.67	4.17	0.00	6.00	6.50	7.50	7.83	7.50	0.00

Time Period	**Time Segment**	**EB (Main)**					**NB**						**WB (Main)**						**SB**					
		Left	**Through**			**Right**	**Left**		**Through**			**Right**	**Left**		**Through**			**Right**	**Left**		**Through**			**Right**
Lane Number		**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**	**13**	**14**	**15**	**16**	**17**	**18**	**19**	**20**	**21**	**22**	**23**
	Average	**4.85**	**6.13**	**7.17**	**7.26**	**0.00**	**4.23**	**5.18**	**13.28**	**11.75**	**6.30**	**0.00**	**4.93**	**5.33**	**2.83**	**4.43**	**4.06**	**0.00**	**6.12**	**6.32**	**6.82**	**8.27**	**7.66**	**0.00**
PM Peak	1	6.17	8.17	7.83	7.17	0.00	8.50	8.33	16.00	17.67	10.80	0.00	3.33	4.83	1.33	2.67	2.17	0.00	10.00	11.00	8.17	8.33	7.00	0.00
	2	6.83	10.67	11.33	8.67	0.00	2.83	5.17	14.00	11.50	9.83	0.00	3.50	4.67	0.50	3.67	5.17	0.00	6.67	8.50	9.17	9.50	8.50	0.00
	3	10.00	16.50	17.50	19.00	0.00	6.50	6.00	33.00	33.00	18.00	0.00	2.67	3.33	0.67	2.83	4.17	0.00	12.83	14.83	9.83	10.17	9.83	0.00
	4	5.33	6.67	7.83	7.17	0.00	7.20	11.00	17.60	15.40	12.00	0.00	3.33	4.17	0.83	4.17	4.50	0.00	10.67	11.50	7.33	8.83	7.33	0.00
	Average	**7.08**	**10.50**	**11.13**	**10.50**	**0.00**	**6.26**	**7.63**	**20.15**	**19.39**	**12.66**	**0.00**	**3.21**	**4.25**	**0.83**	**3.33**	**4.00**	**0.00**	**10.04**	**11.46**	**8.63**	**9.21**	**8.17**	**0.00**

Table D-3.6 Average Queue Length (#vehs/lane) at University Pkwy & US 301 – After Study

Time Period	EB(Main)			NB			WB(Main)			SB		
	Left	**Through**	**Right**	**Left**	**Through**	**Right**	**Left**	**Through**	**Right**	**Left**	**Through**	**Right**
AM Peak	0.36	0.39	0.00	0.65	1.02	0.00	0.52	0.39	0.00	1.64	1.90	0.00
Off Peak	4.85	6.85	0.00	4.71	10.44	0.00	5.13	3.77	0.00	6.22	7.58	0.00
PM Peak	7.08	10.71	0.00	6.94	17.40	0.00	3.73	2.72	0.00	10.75	8.67	0.00

Table D-3.7 Average Queue Length by Lane (#vehs/lane) at University Pkwy & North Cattlemen Rd. / Cooper Creek B – After Study

Time Period	**Time Segment**	**EB (Main)**						**NB**							**WB (Main)**					**SB**			
		Left		**Through**			**Right**	**Left**		**Through**		**Right**			**Left**	**Through**			**Right**	**Left**		**Through**	**Right**
Lane Number		**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**	**13**	**14**	**15**	**16**	**17**	**18**	**19**	**20**	**21**	**22**
AM Peak	1	0.50	2.50	6.17	4.83	9.50	0.17	0.43	0.57	0.29	0.00	0.00	0.00	2.43	1.86	5.29	4.57	4.29	0.00	3.17	3.50	2.20	0.00
	2	0.29	2.43	12.57	16.00	21.14	0.00	0.29	1.29	1.29	0.71	0.86	0.57	3.43	5.00	8.29	7.00	7.14	0.00	6.14	5.57	3.43	0.00
	3	0.83	2.33	20.83	20.83	23.00	0.00	0.86	1.86	0.86	0.29	1.00	0.57	2.71	3.29	10.14	8.57	8.71	0.00	6.14	4.86	2.43	0.00
	4	3.00	4.20	28.33	28.33	31.67	0.00	0.86	1.14	1.00	0.71	0.29	0.14	4.43	5.14	9.71	9.71	9.29	0.00	3.83	4.33	2.83	0.00
	Average	**1.15**	**2.87**	**16.98**	**17.50**	**21.33**	**0.04**	**0.61**	**1.21**	**0.86**	**0.43**	**0.54**	**0.32**	**3.25**	**3.82**	**8.36**	**7.46**	**7.36**	**0.00**	**4.82**	**4.57**	**2.72**	**0.00**
Off Peak	1	6.33	8.83	31.67	31.67	31.67	0.00	8.67	10.67	11.33	11.50	7.33	7.33	24.17	24.17	20.00	20.00	20.00	0.00	10.67	9.83	9.00	0.00
	2	5.33	11.00	30.00	30.00	30.00	0.00	6.60	9.00	9.20	9.20	10.00	10.00	23.00	23.00	18.00	18.00	18.00	0.00	10.00	10.00	10.00	0.00
	3	4.00	7.67	30.00	30.00	30.00	0.00	8.60	11.40	13.40	13.40	9.60	8.60	16.60	17.00	16.00	16.00	16.00	0.00	10.00	10.00	10.00	0.00
	4	2.40	4.80	24.00	24.00	28.00	0.00	12.00	13.80	13.00	12.60	11.00	11.00	23.00	23.00	18.00	18.00	17.50	0.00	10.00	10.00	10.00	0.00

Time Period	Time Segment	EB (Main)						NB						WB (Main)						SB			
		Left		Through			Right	Left		Through		Right		Left		Through			Right	Left		Through	Right
	Average	**4.52**	**8.08**	**28.92**	**28.92**	**29.92**	**0.00**	**8.97**	**11.22**	**11.73**	**11.68**	**9.48**	**9.23**	**21.69**	**21.79**	**18.00**	**18.00**	**17.88**	**0.00**	**10.17**	**9.96**	**9.75**	**0.00**
PM Peak	1	2.40	2.60	30.00	30.00	30.00	0.00	14.00	15.00	15.00	15.00	11.00	11.00	24.00	24.00	20.00	20.00	20.00	0.00	10.00	10.00	10.00	0.00
	2	1.60	3.25	30.00	30.00	30.00	0.00	14.00	15.00	15.00	15.00	15.00	15.00	25.00	25.00	20.00	20.00	20.00	0.00	10.00	10.00	10.00	0.00
	3	1.60	2.80	30.00	30.00	30.00	0.00	9.80	11.20	6.40	6.60	13.20	13.80	18.60	18.60	15.60	15.00	15.00	0.00	10.00	10.00	10.00	0.00
	4	1.50	4.25	30.00	30.00	30.00	0.00	9.20	12.20	6.00	7.80	7.80	7.60	7.60	8.00	15.00	14.40	14.40	0.00	8.40	7.40	8.40	0.00
	Average	**1.78**	**3.23**	**30.00**	**30.00**	**30.00**	**0.00**	**11.75**	**13.35**	**10.60**	**11.10**	**11.75**	**11.85**	**18.80**	**18.90**	**17.65**	**17.35**	**17.35**	**0.00**	**9.60**	**9.35**	**9.60**	**0.00**

Note: * indicates cycle failure

Table D-3.8 Average Queue Length (#vehs/lane) at University Pkwy & North Cattlemen Rd. / Cooper Creek B – After Study

Time Period	EB(Main)			NB			WB(Main)			SB		
	Left	Through	Right	Left	Through	Right	Left	Through	Right	Left	Through	Right
AM Peak	2.01	18.60	0.04	0.91	0.64	0.43	3.54	7.73	0.00	4.69	2.72	0.00
Off Peak	6.30	29.25	0.00	10.09	11.70	9.36	21.74	17.96	0.00	10.06	9.75	0.00
PM Peak	2.50	30.00	0.00	12.55	10.85	11.80	18.85	17.45	0.00	9.48	9.60	0.00

Comparisons of Before and After Queue Lengths for Critical Intersections

The differences in queue length between the before and after measurements are shown in Tables D-3.9 to D-3.12. The tables are color-coded as follows: green shows significant improvement, yellow shows modest change (either improvement or deterioration), and red shows significant deterioration in delay. Several gradations of each color are used to represent different variations within each classification.

Table D-3.9 Difference in Average Queue Length by Lane (#vehs/lane) at University Pkwy & US 301

Time	EB (Main)					NB						WB (Main)						SB						Ave
Period	Left	Through			Right	Left		Through			Right	Left		Through			Right	Left		Through			Right	
Lane Number	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	
AM Peak	-4.34	-8.52	-9.56	-10.19	0.00	-3.08	-3.51	-7.21	-7.49	-0.94	0.00	-11.17	-10.68	-12.21	-11.90	-12.16	-0.05	-5.94	-6.24	-8.78	-8.76	-8.77	-0.30	-6.60
Off Peak	0.10	-1.88	-2.08	-2.99	-1.04	0.23	0.43	5.78	3.75	4.05	-1.94	-0.90	0.00	-7.01	-5.02	-5.90	-0.83	-0.16	0.85	0.69	1.47	1.18	-1.13	-0.54
PM Peak	2.33	2.50	1.88	0.62	-0.60	2.26	2.88	12.65	11.39	12.16	-2.80	-7.64	-1.10	-6.07	-4.12	-5.50	-7.15	3.39	1.76	-1.48	-0.89	-1.48	-4.40	0.46

Table D-3.10 Difference in Average Queue Length (#vehs/lane) at University Pkwy & US 301

Time Period	EB(Main)			NB			WB(Main)			SB		
	Left	Through	Right	Left	Through	Right	Left	Through	Right	Left	Through	Right
AM Peak	-4.34	-9.43	0.00	-7.25	-5.21	0.00	-10.93	-12.09	-0.05	-6.09	-8.77	-0.30
Off Peak	0.10	-2.32	-1.04	-4.04	4.52	-1.94	-0.46	-5.98	-0.83	0.34	1.11	-1.13
PM Peak	2.33	1.66	-0.60	-1.81	12.06	-2.80	-4.37	-5.23	-7.15	2.57	-1.28	-4.40

Table D-3.11 Difference in Average Queue Length by Lane (#vehs/lane) at University Pkwy & North Cattlemen Rd. / Cooper Creek B

Time Period	EB (Main)						NB						WB (Main)						SB				Average
	Left		Through			Right	Left		Through		Right		Left		Through			Right	Left		Through	Right	
Lane Number	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	
AM Peak	-1.54	0.72	9.49	10.88	11.65	-0.07	-0.68	0.37	0.15	0.01	-1.76	-1.75	-1.42	-0.35	-1.93	-2.83	-2.35	-0.67	-0.81	-0.56	0.72	-1.29	0.73
Off Peak	-3.84	0.49	12.71	12.84	13.13	-2.60	4.52	3.50	7.12	5.91	2.63	1.81	8.48	9.12	2.71	3.04	3.80	-6.08	-1.91	-0.50	-0.79	-4.17	3.27
PM Peak	-3.56	-3.96	5.00	5.00	5.00	-1.06	6.37	6.67	6.08	5.56	0.19	1.16	9.88	10.23	5.36	5.22	5.43	-4.46	-1.53	-5.86	2.64	-2.83	2.57

Table D-3.12 Difference in Average Queue Length (#vehs/lane) at University Pkwy & North Cattlemen Rd. / Cooper Creek B

Time Period	EB(Main)			NB			WB(Main)			SB		
	Left	Through	Right	Left	Through	Right	Left	Through	Right	Left	Through	Right
AM Peak	-0.41	10.67	-0.07	-0.16	0.07	-1.75	-0.88	-2.37	-0.67	-0.69	0.72	-1.29
Off Peak	-1.68	12.89	-2.60	4.00	6.51	2.23	8.80	3.18	-6.08	-1.21	-0.79	-4.17
PM Peak	-3.75	5.00	-1.06	6.52	5.82	0.67	10.06	5.34	-4.46	-3.70	2.64	-2.83

Discussion

The following can be concluded from the comparison of queue length:

- For the University Parkway and US 301 intersection, the average queue length decreased by 6.6 vehicles in the AM peak. The average queue length decreased slightly (0.54 vehicles) during the Off Peak, and increased slightly (0.46 vehicles) during the PM peak.
- For the University Parkway and North Cattlemen Rd intersection, the average queue length increased by an average of 0.73 vehicles in the AM peak, 3.27 vehicles in the Off Peak, and 2.57 vehicles in the PM peak. This increase in queue length could be mainly due to the construction area where some lanes were narrowed. Specially, queues in the EB direction reached around 40 vehicles, during some cycles in the Off Peak and almost the PM peak period.
- US 301 is the critical intersection on the western part of the corridor, except for PM peak NBT (side street) movement, majority of the movements show decrease in queue levels at all times of the day.

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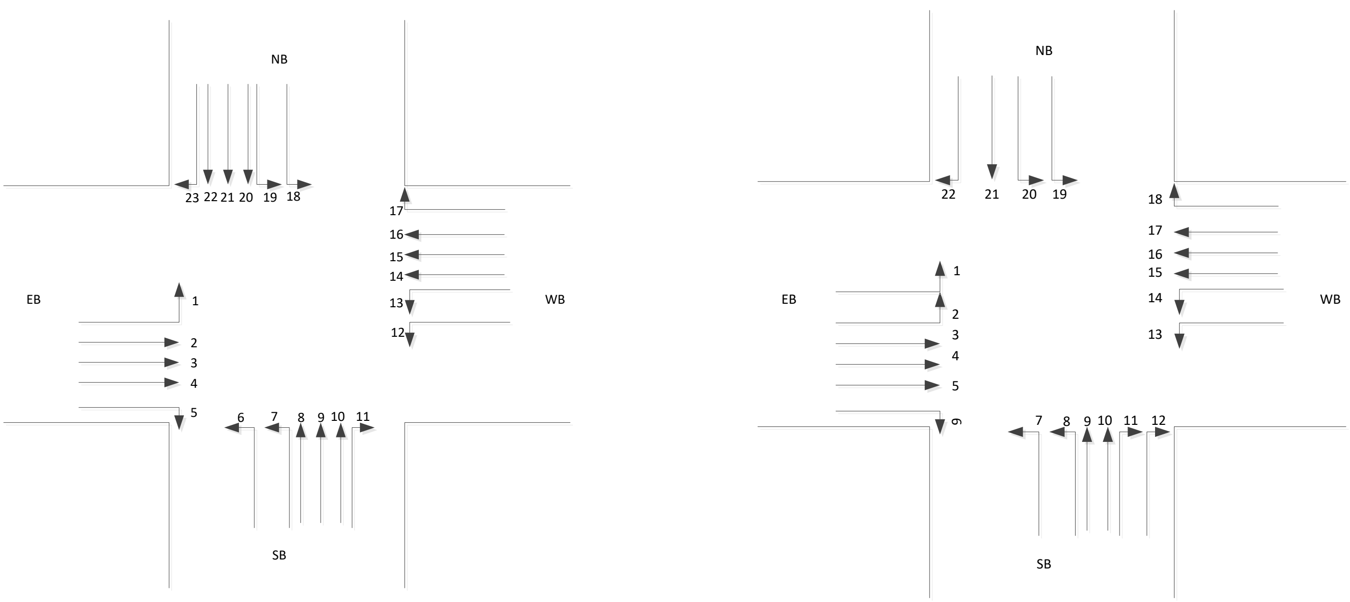
D.3.4 QUEUE TO LANE STORAGE RATIO

In addition to queue length, it is important to assess any impact to adjacent lanes or to upstream facilities. The queue to link/lane ratio is used to establish the likelihood of spillback, which is presented in this section by movement and by time period.

The following assumptions are used:

- The storage capacity is estimated as the maximum number of vehicles in the lane.
- The queue to link/lane storage ratio is estimated as 1 if the observer reported "spillback", and as 0.8 if reported as the maximum number of vehicles in the sight distance.
- Queue to lane storage ratios over 80% are highlighted in yellow, as they represent conditions with a high probability for spillback.



(a) University Pkwy & US 301

(b) University Pkwy & N. Cattlemen Rd. /Cooper Creek B.

Figure D-4.1 Lane Configuration of Critical Intersections

Before Study (Mar.18 & Mar.19, 2015)

Table D-4.1 Average Queue Storage Ratio by Lane and by Period at University Pkwy & US 301

Time Period	Time Segment	EB (Main)					NB						WB (Main)						SB					
		Left	Through			Right	Left		Through			Right	Left		Through			Right	Left		Through			Right
Lane Number		**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**	**13**	**14**	**15**	**16**	**17**	**18**	**19**	**20**	**21**	**22**	**23**
AM Peak	1	0.27	0.44	0.47	0.49	0	0.38	0.37	0.24	0.25	0.14	0	0.93	0.96	0.75	0.64	0.64	0.02	0.49	0.5	0.8	0.78	0.72	0.05
	2	0.19	0.46	0.47	0.44	0	0.22	0.27	0.27	0.27	0.1	0	0.64	0.68	0.64	0.46	0.56	0	0.41	0.41	0.69	0.62	0.64	0.03
	3	0.29	0.65	0.67	0.63	0	0.4	0.33	0.14	0.17	0	0	0.52	0.6	0.71	0.65	0.67	0	0.35	0.42	0.55	0.6	0.6	0
	4	0.35	0.59	0.61	0.63	0	0.5	0.42	0.29	0.29	0.17	0	0.51	0.55	0.57	0.49	0.51	0	0.37	0.39	0.51	0.49	0.53	0.02
	Average	**0.28**	**0.53**	**0.55**	**0.55**	**0**	**0.38**	**0.35**	**0.24**	**0.25**	**0.1**	**0**	**0.65**	**0.7**	**0.67**	**0.56**	**0.6**	**0.01**	**0.41**	**0.43**	**0.64**	**0.62**	**0.62**	**0.03**
Off Peak	1	0.41	0.35	0.5	0.53	0.02	0.2	0.33	0.29	0.31	0.28	0.12	0.19	0.21	0.38	0.33	0.37	0.15	0.33	0.31	0.31	0.35	0.38	0.21
	2	0.06	0.29	0.28	0.37	0.23	0.5	0.5	0.14	0.14	0.06	0.08	0.33	0.34	0.56	0.47	0.56	0.08	0.29	0.27	0.33	0.36	0.38	0.08
	3	0.29	0.65	0.67	0.63	0.09	0.4	0.33	0.14	0.17	0	0.28	0.38	0.4	0.55	0.46	0.52	0.04	0.29	0.3	0.4	0.4	0.38	0.05
	4	0.35	0.59	0.61	0.63	0.26	0.5	0.42	0.29	0.29	0.17	0.3	0.39	0.39	0.59	0.45	0.44	0.06	0.4	0.33	0.4	0.48	0.39	0.03

Time Period	Time Segment	EB (Main)					NB						WB (Main)						SB					
		Left	Through			Right	Left		Through			Right	Left		Through			Right	Left		Through			Right
	Average	**0.28**	**0.47**	**0.51**	**0.54**	**0.15**	**0.4**	**0.4**	**0.21**	**0.23**	**0.13**	**0.19**	**0.32**	**0.33**	**0.52**	**0.43**	**0.47**	**0.08**	**0.33**	**0.3**	**0.36**	**0.4**	**0.38**	**0.09**
PM Peak	1	0.41	0.35	0.5	0.53	0.03	0.2	0.33	0.29	0.31	0.28	0.28	0.3	0.4	0.37	0.47	0.38	0.14	0.47	0.52	0.6	0.74	0.6	0.02
	2	0.06	0.29	0.28	0.37	0.11	0.5	0.5	0.14	0.14	0.06	0.08	0.22	0.35	0.46	0.5	0.5	0.38	0.47	0.59	0.56	0.6	0.53	0.1
	3	0.29	0.65	0.67	0.63	0.2	0.4	0.33	0.14	0.17	0	0.56	0.22	0.35	0.24	0.42	0.63	0.58	0.55	0.67	0.64	0.66	0.55	0.25
	4	0.35	0.59	0.61	0.63	0	0.5	0.42	0.29	0.29	0.17	0.2	0.3	0.39	0.2	0.46	0.49	0.28	0.44	0.48	0.4	0.48	0.4	0.15
	Average	**0.28**	**0.47**	**0.51**	**0.54**	**0.09**	**0.4**	**0.4**	**0.21**	**0.23**	**0.13**	**0.28**	**0.26**	**0.37**	**0.32**	**0.46**	**0.5**	**0.35**	**0.48**	**0.56**	**0.55**	**0.62**	**0.52**	**0.13**

Table D-4.2 Number of Cycles with Spillback at University Pkwy & US 301.

Time Period	Time Segment	#Cycles in 15 min	EB (Main)					NB						WB (Main)						SB					
			Left	Through			Right	Left		Through			Right	Left		Through			Right	Left		Through			Right
Lane Number			**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**	**13**	**14**	**15**	**16**	**17**	**18**	**19**	**20**	**21**	**22**	**23**
AM Peak	1	5	0	0	0	0	0	0	0	0	0	0	0	1	5	5	0	0	0	0	0	0	3	3	2
	2	5	0	1	1	1	0	0	0	0	0	0	0	1	0	1	1	0	1	0	0	0	1	0	0
	3	5	0	0	0	0	0	0	0	0	0	0	0	1	0	1	1	1	1	0	0	0	1	1	1
	4	5	0	0	0	0	0	0	0	0	0	0	0	1	0	0	0	0	0	0	0	0	0	0	0
	Average		**0**	**0.25**	**0.25**	**0.25**	**0**	**0**	**0**	**0**	**0**	**0**	**0**	**1**	**1.25**	**1.75**	**0.5**	**0.25**	**0.5**	**0**	**0**	**0**	**1.25**	**1**	**0.75**
Off Peak	1	6	0	0	0	0	0	0	0	0	0	0	0	2	0	0	0	0	0	0	0	0	0	0	0
	2	5	0	0	0	1	0	0	0	0	0	0	0	1	0	0	0	0	0	0	0	0	0	0	0
	3	5	0	0	0	0	0	0	0	0	0	0	1	1	0	0	0	0	0	0	0	0	0	0	0
	4	5	0	0	0	0	1	0	0	0	0	0	1	1	0	0	0	0	0	0	0	0	0	0	0
	Average		**0**	**0**	**0**	**0.25**	**0.25**	**0**	**0**	**0**	**0**	**0**	**0.5**	**1.25**	**0**	**0**	**0**	**0**	**0**	**0**	**0**	**0**	**0**	**0**	**0**
PM Peak	1	6	3	1	1	0	0	1	1	0	0	2	0	1	0	0	0	0	0	0	0	1	1	2	0
	2	5	0	1	1	1	0	1	2	0	0	0	0	1	0	0	0	0	0	1	0	1	0	1	0

Time Period	Time Segment	#Cycles in 15 min	EB (Main)					NB						WB (Main)						SB					
			Left	Through			Right	Left		Through			Right	Left		Through			Right	Left		Through			Right
Lane Number			1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23
	3	5	2	3	2	2	0	2	3	1	1	3	2	1	0	0	0	0	1	1	1	1	1	1	1
	4	5	0	0	0	0	0	1	2	0	0	0	0	1	0	0	0	0	0	0	0	0	0	0	0
	Average		**1.25**	**1.25**	**1**	**0.75**	**0**	**1.25**	**2**	**0.25**	**0.25**	**1.25**	**0.5**	**1**	**0**	**0**	**0**	**0**	**0.25**	**0.5**	**0.25**	**0.75**	**0.5**	**1**	**0.25**

Table D-4.3 Average Queue Storage Ratio by Lane and by Period at University Pkwy & North Cattlemen Rd. /Creek Cooper

Time Period	Time Segment	EB (Main)						NB						WB (Main)						SB			
		Left		Through			Right	Left		Through		Right		Left		Through			Right	Left		Thru	Right
Lane Number		1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22
AM Peak	1	0.15	0.08	0.36	0.36	0.50	0.03	0.11	0.04	0.04	0.00	0.14	0.07	0.30	0.27	0.53	0.60	0.58	0.04	0.27	0.23	0.21	0.12
	2	0.23	0.14	0.4	0.34	0.45	0.00	0.08	0.04	0.08	0.01	0.14	0.16	0.29	0.28	0.41	0.42	0.33	0.07	0.27	0.23	0.17	0.11
	3	0.26	0.16	0.23	0.15	0.30	0.00	0.11	0.11	0.13	0.04	0.17	0.09	0.26	0.21	0.58	0.66	0.65	0.00	0.28	0.29	0.17	0.01
	4	0.19	0.13	0.21	0.21	0.30	0.06	0.07	0.06	0.11	0.08	0.10	0.13	0.19	0.17	0.55	0.61	0.59	0.03	0.31	0.27	0.13	0.03

Time Period	Time Segment	EB (Main)						NB						WB (Main)						SB			
		Left		Through			Right	Left		Through		Right		Left		Through			Right	Left		Thru	Right
Lane Number		1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22
	Average	0.21	0.13	0.3	0.26	0.39	0.02	0.09	0.06	0.09	0.03	0.14	0.11	0.26	0.23	0.51	0.57	0.54	0.03	0.28	0.26	0.17	0.06
Off Peak	1	0.59	0.4	0.54	0.55	0.63	0.50	0.29	0.51	0.58	0.41	0.25	0.28	0.73	0.68	0.67	0.71	0.72	0.23	0.41	0.38	0.74	0.16
	2	0.71	0.52	0.72	0.72	0.72	0.54	0.37	0.69	0.54	0.43	0.50	0.50	0.88	0.85	0.91	0.99	0.95	0.48	0.68	0.63	0.96	0.21
	3	0.65	0.42	0.72	0.72	0.72	0.6	0.26	0.49	0.54	0.47	0.41	0.41	0.87	0.86	0.79	0.86	0.8	0.28	0.83	0.63	0.93	0.26
	4	0.62	0.44	0.61	0.58	0.61	0.43	0.36	0.51	0.65	0.46	0.44	0.46	0.45	0.43	0.69	0.76	0.66	0.23	0.49	0.46	0.89	0.21
	Average	0.64	0.45	0.65	0.64	0.67	0.52	0.32	0.55	0.58	0.44	0.40	0.41	0.73	0.70	0.76	0.83	0.78	0.30	0.60	0.52	0.88	0.21
PM Peak	1	0.32	0.49	1.00	1.00	1.00	0.17	0.33	0.55	0.56	0.33	0.79	0.59	0.56	0.56	0.65	0.72	0.71	0.28	0.60	0.92	0.56	0.17
	2	0.60	0.52	1.00	1.00	1.00	0.28	0.29	0.53	0.43	0.62	0.61	0.76	0.46	0.42	0.67	0.74	0.73	0.19	0.58	0.78	0.51	0.15
	3	0.35	0.35	1.00	1.00	1.00	0.10	0.6	0.46	0.67	0.33	0.73	0.50	0.45	0.44	0.58	0.60	0.58	0.17	0.60	0.92	0.74	0.13

Time Period	Time Segment	EB (Main)						NB						WB (Main)						SB			
		Left		Through			Right	Left		Through		Right		Left		Through			Right	Left		Thru	Right
Lane Number		**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**	**13**	**14**	**15**	**16**	**17**	**18**	**19**	**20**	**21**	**22**
	4	0.37	0.33	1.00	1.00	1.00	0.30	0.32	0.37	0.6	0.42	0.59	0.53	0.51	0.51	0.57	0.63	0.62	0.26	0.44	0.43	0.51	0.12
	Average	0.41	0.42	1.00	1.00	1.00	0.21	0.38	0.48	0.56	0.43	0.68	0.59	0.50	0.48	0.61	0.67	0.66	0.22	0.56	0.76	0.58	0.14

Table D-4.4 Number of Cycles with Spillback at University Pkwy & North Cattlemen Rd. /Creek Cooper

Time Period	**Time Segment**	**# cycles in 15 min**	**EB (Main)**						**NB**						**WB (Main)**						**SB**			
			Left		**Through**			**Right**	**Left**		**Through**		**Right**		**Left**		**Through**			**Right**	**Left**		**Thru**	**Right**
Lane Number			**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**	**13**	**14**	**15**	**16**	**17**	**18**	**19**	**20**	**21**	**22**
AM Peak	**1**	6	0	0	0	0	0	0	0	0	0	0	0	2	0	0	0	0	0	0	0	0	1	2
	2	6	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1
	3	6	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
	4	6	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	1	1	0	0	0	0	0
	Average		0	0	0	0	0	0	0	0	0	0	0	0	0.5	0	0	0.25	0.25	0.25	0	0	0	0.25
Off Peak	**1**	6	0	0	0	0	0	0	0	0	2	0	0	0	1	1	0	0	0	1	0	0	0	0
	2	6	2	0	0	0	0	1	0	2	0	0	0	0	5	5	4	4	4	3	1	1	3	0
	3	6	1	0	0	0	0	1	0	0	0	0	0	0	5	5	2	3	1	0	3	0	3	0
	4	6	1	0	0	0	0	0	0	0	1	0	0	0	0	0	0	0	0	0	0	0	1	0
	Average		0	1	0	0	0	0	0.5	0	0.5	0.75	0	0	0	2.75	2.75	1.5	1.75	1.25	1	1	0.25	1.75
PM Peak	**1**	6	0	1	6*	6*	6*	0	0	0	0	0	3	0	0	0	0	0	0	1	0	5	0	0
	2	5	0	0	5*	5*	5*	0	0	0	0	1	0	2	0	0	0	0	0	0	0	1	0	0
	3	6	0	0	6*	6*	6*	0	1	0	2	0	2	0	0	0	0	0	0	0	0	1	0	0

Time Period	Time Segment	# cycles in 15 min	EB (Main)						NB						WB (Main)						SB			
			Left		Through			Right	Left		Through		Right		Left		Through			Right	Left		Thru	Right
Lane Number			1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22
	4	6	0	0	6*	6*	6*	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	0	0
	Average		0	0	0.25	5.75	5.75	5.75	0	0.25	0	0.5	0.25	1.25	0.5	0	0	0	0	0	0.25	0	2	0

Note: * indicates cycle failure

After Study (Nov. 19 & Nov. 20, 2014)

Table D-4.5 Average Queue Storage Ratio by Lane by Period at University Pkwy & US 301 – After Study

Time Period		EB (Main)					NB						WB (Main)						SB					
		Left	Through			Right	Left		Through			Right	Left		Through			Right	Left		Through			Right
Lane Number		1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23
AM Peak	1	0.15	0.28	0.26	0.19	0.00	0.30	0.25	0.11	0.09	0.06	0.00	0.40	0.39	0.14	0.09	0.08	0.00	0.26	0.28	0.44	0.47	0.47	0.00
	2	0.21	0.22	0.25	0.24	0.00	0.21	0.20	0.20	0.13	0.21	0.00	0.48	0.47	0.14	0.15	0.13	0.00	0.23	0.26	0.49	0.53	0.53	0.00
	3	0.21	0.20	0.27	0.21	0.00	0.19	0.38	0.14	0.17	0.19	0.00	0.38	0.47	0.12	0.13	0.11	0.00	0.24	0.26	0.45	0.49	0.49	0.00
	4	0.20	0.19	0.20	0.21	0.00	0.37	0.26	0.18	0.18	0.17	0.00	0.41	0.44	0.07	0.14	0.10	0.00	0.49	0.49	0.78	0.78	0.77	0.00

Time Period		EB (Main)					NB						WB (Main)						SB					
		Left	Through			Right	Left		Through			Right	Left		Through			Right	Left		Through			Right
Lane Number		1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23
	Average	**0.02**	**0.03**	**0.02**	**0.01**	**0.00**	**0.07**	**0.05**	**0.03**	**0.03**	**0.05**	**0.00**	**0.03**	**0.03**	**0.02**	**0.02**	**0.02**	**0.00**	**0.09**	**0.08**	**0.12**	**0.11**	**0.10**	**0.00**
Off Peak	1	0.34	0.42	0.37	0.43	0.00	0.42	0.55	0.43	0.35	0.34	0.00	0.29	0.30	0.16	0.19	0.21	0.00	0.36	0.40	0.28	0.43	0.40	0.00
	2	0.27	0.28	0.44	0.43	0.00	0.53	0.36	0.18	0.20	0.14	0.00	0.23	0.31	0.19	0.23	0.13	0.00	0.31	0.33	0.37	0.50	0.44	0.00
	3	0.19	0.32	0.40	0.34	0.00	0.24	0.27	0.44	0.42	0.43	0.00	0.26	0.33	0.15	0.22	0.24	0.00	0.31	0.31	0.51	0.55	0.52	0.00
	4	0.34	0.41	0.39	0.33	0.00	0.50	0.55	0.46	0.38	0.48	0.00	0.31	0.39	0.10	0.17	0.20	0.00	0.32	0.36	0.44	0.46	0.44	0.00
	Average	**0.29**	**0.36**	**0.40**	**0.38**	**0.00**	**0.42**	**0.43**	**0.38**	**0.34**	**0.35**	**0.00**	**0.27**	**0.33**	**0.15**	**0.20**	**0.19**	**0.00**	**0.32**	**0.35**	**0.40**	**0.49**	**0.45**	**0.00**
PM Peak	1	0.36	0.48	0.44	0.38	0.00	0.85	0.69	0.46	0.50	0.60	0.00	0.19	0.30	0.07	0.12	0.10	0.00	0.53	0.61	0.48	0.49	0.41	0.00
	2	0.40	0.63	0.63	0.46	0.00	0.28	0.43	0.40	0.33	0.55	0.00	0.19	0.29	0.03	0.17	0.25	0.00	0.35	0.47	0.54	0.56	0.50	0.00
	3	0.59	0.97	0.97	1.00	0.00	0.65	0.50	0.94	0.94	1.00	0.00	0.15	0.21	0.04	0.13	0.20	0.00	0.68	0.82	0.58	0.60	0.58	0.00
	4	0.31	0.39	0.44	0.38	0.00	0.72	0.92	0.50	0.44	0.67	0.00	0.19	0.26	0.04	0.19	0.21	0.00	0.56	0.64	0.43	0.52	0.43	0.00

Time Period		EB (Main)					NB						WB (Main)						SB					
		Left	Through			Right	Left		Through			Right	Left		Through			Right	Left		Through			Right
Lane Number		1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23
	Average	0.42	0.62	0.62	0.55	0.00	0.63	0.64	0.58	0.55	0.70	0.00	0.18	0.27	0.04	0.15	0.19	0.00	0.53	0.64	0.51	0.54	0.48	0.00

Table D-4.6 Number of Cycles with Spillback at University Pkwy & US 301 – After Study

Time Period	Time Segment	# Cycles in 15 mins	EB (Main)					NB						WB (Main)						SB					
			Left	Through			Right	Left		Through			Right	Left		Through			Right	Left		Through			Right
Lane Number			1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23
AM Peak	1	5	0	0	0	0	0	0	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
	2	6	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
	3	7	0	0	0	0	0	0	0	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0
	4	6	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	4	5	4	0
	Average		0	0	0	0	0	0	0	0.25	0.25	0	0	0	0	0	0	0	0	0	0	1	1.25	1	0
Off Peak	1	5	0	0	0	0	0	0	0	5	4	0	0	0	0	0	0	0	0	0	0	0	0	0	0
	2	6	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
	3	6	0	0	0	0	0	0	0	4	3	1	0	0	0	0	0	0	0	0	0	1	1	0	0

Time Period	Time Segment	# Cycles in 15 mins	EB (Main)					NB						WB (Main)						SB					
			Left	Through			Right	Left		Through			Right	Left		Through			Right	Left		Through			Right
Lane Number			**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**	**13**	**14**	**15**	**16**	**17**	**18**	**19**	**20**	**21**	**22**	**23**
	4	6	0	0	0	0	0	0	0	5	4	0	0	0	0	0	0	0	0	0	0	0	0	0	0
	Average		**0**	**0**	**0**	**0**	**0**	**0**	**0**	**3.5**	**2.75**	**0.25**	**0**	**0**	**0**	**0**	**0**	**0**	**0**	**0**	**0**	**0.25**	**0.25**	**0**	**0**
PM Peak	1	6	0	1	0	0	0	0	0	5	6*	0	0	0	0	0	0	0	0	1	1	0	0	0	0
	2	6	0	2	1	0	0	0	0	2	2	0	0	0	0	0	0	0	0	0	0	0	0	0	0
	3	6	0	1	1	1	0	0	0	3	4	3	0	0	0	0	0	0	0	3	4	0	0	0	0
	4	6	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	1	0	0	0	0
	Average		**0**	**1**	**0.5**	**0.25**	**0**	**0**	**0**	**2.5**	**3**	**0.75**	**0**	**0**	**0**	**0**	**0**	**0**	**0**	**1.25**	**1.5**	**0**	**0**	**0**	**0**

Note: * indicates cycle failure

Table D-4.7 Average Queue Storage Ratio by Lane by Period at University Pkwy & North Cattlemen Rd. /Creek Cooper – After Study

Time		EB (Main)						NB						WB (Main)						SB			
		Left		Through			Right	Left		Through		Right		Left		Through			Right	Left		Through	Right
Lane Number		**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**	**13**	**14**	**15**	**16**	**17**	**18**	**19**	**20**	**21**	**22**
AM Peak	1	0.04	0.15	0.19	0.15	0.30	0.03	0.03	0.04	0.02	0.00	0.00	0.00	0.10	0.07	0.26	0.23	0.21	0.00	0.29	0.18	0.16	0.00

Time		EB (Main)						NB						WB (Main)						SB			
		Left		Through			Right	Left		Through		Right		Left		Through			Right	Left		Through	Right
Lane Number		**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**	**13**	**14**	**15**	**16**	**17**	**18**	**19**	**20**	**21**	**22**
	2	0.02	0.14	0.39	0.50	0.66	0.00	0.02	0.09	0.09	0.05	0.06	0.03	0.14	0.20	0.41	0.35	0.36	0.00	0.56	0.28	0.24	0.00
	3	0.06	0.14	0.65	0.65	0.72	0.00	0.06	0.12	0.06	0.02	0.07	0.03	0.11	0.13	0.51	0.43	0.44	0.00	0.56	0.24	0.17	0.00
	4	0.23	0.25	0.89	0.89	0.99	0.00	0.06	0.08	0.07	0.05	0.02	0.01	0.18	0.21	0.49	0.49	0.46	0.00	0.35	0.22	0.20	0.00
	Average	**0.09**	**0.17**	**0.53**	**0.55**	**0.67**	**0.01**	**0.04**	**0.08**	**0.06**	**0.03**	**0.04**	**0.02**	**0.13**	**0.15**	**0.42**	**0.37**	**0.37**	**0.00**	**0.44**	**0.23**	**0.19**	**0.00**
Off Peak	1	0.49	0.52	0.99	0.99	0.99	0.00	0.62	0.71	0.76	0.77	0.49	0.41	0.97	0.97	1.00	1.00	1.00	0.00	0.97	0.49	0.64	0.00
	2	0.41	0.65	0.94	0.94	0.94	0.00	0.47	0.60	0.61	0.61	0.67	0.56	0.92	0.92	0.90	0.90	0.90	0.00	0.91	0.50	0.71	0.00
	3	0.31	0.45	0.94	0.94	0.94	0.00	0.61	0.76	0.89	0.89	0.64	0.48	0.66	0.68	0.80	0.80	0.80	0.00	0.91	0.50	0.71	0.00
	4	0.18	0.28	0.75	0.75	0.88	0.00	0.86	0.92	0.87	0.84	0.73	0.61	0.92	0.92	0.90	0.90	0.88	0.00	0.91	0.50	0.71	0.00
	Average	**0.35**	**0.48**	**0.90**	**0.90**	**0.93**	**0.00**	**0.64**	**0.75**	**0.78**	**0.78**	**0.63**	**0.51**	**0.87**	**0.87**	**0.90**	**0.90**	**0.89**	**0.00**	**0.92**	**0.50**	**0.70**	**0.00**
PM Peak	1	0.18	0.15	0.94	0.94	0.94	0.00	1.00	1.00	1.00	1.00	0.73	0.61	0.96	0.96	1.00	1.00	1.00	0.00	0.91	0.50	0.71	0.00

Time		EB (Main)						NB						WB (Main)						SB			
		Left		Through			Right	Left		Through		Right		Left		Through			Right	Left		Through	Right
Lane Number		**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**	**13**	**14**	**15**	**16**	**17**	**18**	**19**	**20**	**21**	**22**
	2	0.12	0.19	0.94	0.94	0.94	0.00	1.00	1.00	1.00	1.00	1.00	0.83	1.00	1.00	1.00	1.00	1.00	0.00	0.91	0.50	0.71	0.00
	3	0.12	0.16	0.94	0.94	0.94	0.00	0.70	0.75	0.43	0.44	0.88	0.77	0.74	0.74	0.78	0.75	0.75	0.00	0.91	0.50	0.71	0.00
	4	0.12	0.25	0.94	0.94	0.94	0.00	0.66	0.81	0.40	0.52	0.52	0.42	0.30	0.32	0.75	0.72	0.72	0.00	0.76	0.37	0.60	0.00
	Average	**0.14**	**0.19**	**0.94**	**0.94**	**0.94**	**0.00**	**0.84**	**0.89**	**0.71**	**0.74**	**0.78**	**0.66**	**0.75**	**0.76**	**0.88**	**0.87**	**0.87**	**0.00**	**0.87**	**0.47**	**0.69**	**0.00**

Table D-4.8 Number of Cycles with Spillback at University Pkwy & North Cattlemen Rd. /Creek Cooper – After Study

Time Period	Time Segment	# cycles in 15 mins	EB (Main)						NB						WB (Main)						SB			
			Left		Through			Right	Left		Through		Right		Left		Through			Right	Left		Thru	Right
Lane Number			**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**	**13**	**14**	**15**	**16**	**17**	**18**	**19**	**20**	**21**	**22**
AM	**1**	7	0	0	0	0	2	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
	2	7	0	0	4	5	6	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Peak	**3**	7	0	0	7*	7*	7*	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
	4	7	0	0	7*	7*	7*	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

Time Period	Time Segment	# cycles in 15 mins	EB (Main)						NB						WB (Main)						SB			
			Left		Through			Right	Left		Through		Right		Left		Through			Right	Left		Thru	Right
	Average		**0**	**0**	**4.5**	**4.75**	**5.5**	**0**	**0**	**0**	**0**	**0**	**0**	**0**	**0**	**0**	**0**	**0**	**0**	**0**	**0**	**0**	**0**	**0**
Off Peak	**1**	6	0	1	6*	6*	6*	0	0	0	2	2	0	0	6*	6*	6*	6*	6*	0	0	0	0	0
	2	5	0	1	5*	5*	5*	0	0	0	0	0	0	0	5*	5*	5*	5*	5*	0	0	0	0	0
	3	6	0	0	6*	6*	6*	0	0	0	3	3	1	1	5*	5*	6*	6*	6*	0	0	0	0	0
	4	5	0	0	5*	5*	5*	0	0	3	3	3	0	0	5*	5*	5*	5*	5*	0	0	0	0	0
	Average		**0**	**0.5**	**5.5**	**5.5**	**5.5**	**0**	**0**	**0.75**	**2**	**2**	**0.25**	**0.25**	**5.25**	**5.25**	**5.5**	**5.5**	**5.5**	**0**	**0**	**0**	**0**	**0**
PM	**1**	5	0	0	5*	5*	5*	0	5*	5*	5*	5*	2	2	5*	5*	5*	5*	5*	0	0	0	0	0
	2	5	0	0	5*	5*	5*	0	5*	5*	5*	5*	5*	5*	5*	5*	5*	5*	5*	0	0	0	0	0
Peak	**3**	5	0	0	5*	5*	5*	0	0	1	0	0	1	1	5*	5*	5*	5*	5*	0	0	0	0	0
	4	5	0	0	5*	5*	5*	0	0	1	0	0	0	0	0	0	5*	5*	5*	0	0	0	0	0
	Average		**0**	**0**	**5**	**5**	**5**	**0**	**2.5**	**3**	**2.5**	**2.5**	**2**	**2**	**3.75**	**3.75**	**5**	**5**	**5**	**0**	**0**	**0**	**0**	**0**

Note: * indicates cycle failure

Comparisons of Before and After Queue Storage Ratios

The differences in Queue Storage Ratio between before and after measurements are shown in Table D-4.9 to Table D-4.12. The tables are color-coded as follows: green shows significant improvement, yellow shows modest change (either improvement or deterioration), and red shows significant deterioration in delay. Several gradations of each color are used to represent different variations within each classification.

Table D-4.9 Difference in Average Queue Storage Ratios at University Pkwy & US 301.

Time Period	EB (Main)					NB						WB (Main)						SB						Average
	Left	Through			Right	Left		Through			Right	Left		Through			Right	Left		Through			Right	
Lane Number	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	
AM Peak	-0.26	-0.50	-0.53	-0.54	0.00	-0.31	-0.30	-0.21	-0.22	-0.05	0.00	-0.62	-0.67	-0.65	-0.54	-0.58	-0.01	-0.32	-0.35	-0.52	-0.51	-0.52	-0.03	-0.36
Off Peak	0.01	-0.11	-0.11	-0.16	-0.15	0.02	0.03	0.17	0.11	0.22	-0.19	-0.05	0.00	-0.37	-0.23	-0.28	-0.08	-0.01	0.05	0.04	0.09	0.07	-0.09	-0.04
PM Peak	0.14	0.15	0.11	0.01	-0.09	0.23	0.24	0.37	0.32	0.57	-0.28	-0.08	-0.11	-0.27	-0.31	-0.31	-0.35	0.04	0.07	-0.04	-0.08	-0.04	-0.13	0.01

Table D-4.10 Difference in Percent of Cycles with Spillback at University Pkwy & US 301.

Time	EB (Main)					NB						WB (Main)							SB					Average
Period	Left	Through			Right	Left		Through			Right	Left		Through			Right		Left	Through			Right	
Lane Number	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	
AM Peak	0%	-5%	-5%	-5%	0%	0%	0%	4%	4%	0%	0%	-20%	-25%	-35%	-10%	-5%	-10%	0%	0%	17%	-4%	-3%	-15%	-5%
Off Peak	0%	0%	0%	-5%	-5%	0%	0%	61%	48%	4%	-10%	-24%	0%	0%	0%	0%	0%	0%	0%	4%	4%	0%	0%	3%
PM Peak	-24%	-7%	-11%	-10%	0%	-24%	-38%	37%	45%	-11%	-10%	-19%	0%	0%	0%	0%	-5%	11%	20%	-14%	-10%	-19%	-5%	-4%

Table D-4.11 Difference in Average Queue Storage Ratio at University Pkwy & North Cattlemen Rd. /Creek Cooper.

Time Period	EB (Main)						NB						WB (Main)						SB				Average
	Left		Through			Right	Left		Through		Right		Left		Through			Right	Left		Through	Right	
Lane Number	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	
AM Peak	-0.12	0.04	0.23	0.29	0.28	-0.01	-0.05	0.02	-0.03	0.00	-0.10	-0.09	-0.13	-0.08	-0.09	-0.20	-0.17	-0.03	0.16	-0.03	0.02	-0.06	-0.01
Off Peak	-0.29	0.03	0.25	0.26	0.26	-0.52	0.32	0.20	0.20	0.34	0.23	0.10	0.14	0.17	0.14	0.07	0.11	-0.30	0.32	-0.02	-0.18	-0.21	0.07
PM Peak	-0.27	-0.23	-0.06	-0.06	-0.06	-0.21	0.46	0.41	0.15	0.31	0.10	0.07	0.25	0.28	0.27	0.20	0.21	-0.22	0.31	-0.29	0.11	-0.14	0.07

Table D-4.12 Difference in Percent of Cycles with Spillback at University Pkwy & North Cattlemen Rd. /Creek Cooper.

Time Period	EB (Main)						NB						WB (Main)						SB				Average
	Left		Through			Right	Left		Through		Right		Left		Through			Right	Left		Through	Right	
Lane Number	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	
AM Peak	0%	0%	64%	68%	79%	0%	0%	0%	0%	0%	0%	-8%	0%	0%	-4%	-4%	-4%	0%	0%	0%	-4%	-13%	8%

Time Period	EB (Main)						NB						WB (Main)						SB				Average
	Left		Through			Right	Left		Through		Right		Left		Through			Right	Left		Through	Right	
Lane Number	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	
Off Peak	-17%	9%	100%	100%	100%	-8%	0%	5%	24%	36%	5%	5%	50%	50%	75%	71%	79%	-17%	-17%	-4%	-29%	0%	28%
PM Peak	0%	-4%	0%	0%	0%	0%	46%	60%	41%	46%	18%	31%	75%	75%	100%	100%	100%	-4%	0%	-35%	0%	0%	29%

Discussion

The following are concluded from the comparison of queue storage ratios:

- Queue storage ratios show higher variation than other variables. On average, the levels of improvement were higher than the deterioration reached at both critical intersections. A higher percentage of movements improved was observed in the WB at the University Parkway & US 301 intersection where the traffic flows decreased; at the same time, the EB and SB directions are slightly improved.
- At the University Parkway & US 301 intersection, the average queue storage ratios improved in the AM peak and slightly improved in the Off Peak, while deteriorated in the PM peak. However, on average, this variable is improved for all three study periods, while the Off Peak shows a slight deterioration in the percent of cycles with spillback (3%).
- Average queue storage ratios deteriorated somewhat during Off Peak and PM peak periods at the University Parkway & North Cattlemen Rd intersection. The percent of cycles with spillback were higher in this intersection and during all three time periods.

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D.3.5 EQUIVALENT PCE FLOWS

Traffic flows were counted manually and converted to equivalent PCE flows (pce/hour) by considering the percentage of heavy vehicles. It was assumed that the PCE for trucks is 2.

Truck Percentage Observations

Table D-5.1 Truck Percentages at University Pkwy & US 301.

Period	Interval	EB			NB			WB			SB			Ave
		Left	Thru	Right	Left	Thru	Right	Left	Thru	Right	Left	Thru	Right	
AM Peak	1	0.00%	1.71%	33.33%	10.53%	10.13%	14.81%	5.69%	3.72%	3.39%	9.68%	2.50%	0.00%	5.47%
	2	6.67%	0.65%	0.00%	10.77%	9.55%	3.03%	6.41%	2.93%	7.46%	2.38%	2.01%	0.00%	5.12%
	3	3.57%	2.15%	0.00%	8.06%	4.23%	0.00%	2.41%	3.75%	6.98%	9.43%	4.05%	0.00%	4.27%
	4	6.67%	0.70%	0.00%	4.44%	2.87%	4.08%	4.23%	3.24%	4.05%	1.67%	10.27%	0.00%	4.03%
	Average	**4.27%**	**1.37%**	**33.33%**	**8.17%**	**6.76%**	**5.34%**	**4.60%**	**3.39%**	**5.22%**	**5.38%**	**4.33%**	**0.00%**	**4.71%**
Off Peak	1	5.56%	4.11%	8.57%	6.15%	5.13%	0.00%	0.86%	1.41%	3.33%	1.67%	9.29%	10.77%	4.62%
	2	2.86%	4.90%	12.07%	1.63%	4.93%	4.35%	7.30%	3.03%	9.46%	2.00%	8.11%	5.32%	5.19%
	3	0.00%	2.02%	11.11%	7.44%	1.40%	0.00%	11.00%	1.18%	3.28%	5.26%	3.93%	5.15%	3.90%
	4	2.38%	2.33%	10.64%	3.70%	3.43%	0.00%	5.77%	1.80%	8.16%	3.77%	4.43%	2.90%	3.55%
	Average	**2.84%**	**3.16%**	**10.84%**	**4.62%**	**3.89%**	**0.93%**	**6.09%**	**1.91%**	**6.15%**	**3.18%**	**6.45%**	**5.85%**	**4.31%**
	1	2.78%	0.43%	7.50%	2.63%	4.02%	0.00%	4.35%	3.88%	12.82%	4.76%	3.07%	3.19%	3.56%

Period	Interval	EB			NB			WB			SB			Ave
		Left	Thru	Right	Left	Thru	Right	Left	Thru	Right	Left	Thru	Right	
PM Peak	2	1.82%	1.79%	2.27%	1.44%	3.41%	5.88%	1.67%	2.65%	6.50%	4.29%	2.61%	4.39%	3.01%
	3	2.27%	0.00%	0.00%	3.65%	2.36%	2.00%	3.17%	4.29%	4.12%	3.53%	2.79%	2.50%	2.49%
	4	0.00%	1.24%	0.00%	1.72%	1.04%	0.00%	1.54%	1.08%	4.21%	1.79%	1.32%	1.28%	1.35%
	Average	**1.60%**	**0.82%**	**1.95%**	**2.37%**	**2.68%**	**1.74%**	**2.72%**	**3.06%**	**6.62%**	**3.33%**	**2.34%**	**2.69%**	**2.54%**

Table D-5.2 Truck Percentages at University Pkwy & North Cattlemen Rd. /Creek Cooper

Period	Interval	EB			SB			WB			NB			Ave
		Left	Thru	Right	Left	Thru	Right	Left	Thru	Right	Left	Thru	Right	
AM Peak	1	0.00%	1.83%	5.56%	1.25%	0.00%	0.00%	0.00%	3.28%	0.00%	0.00%	0.00%	0.00%	2.17%
	2	8.33%	3.05%	8.00%	1.16%	0.00%	11.11%	0.00%	5.17%	2.30%	16.67%	0.00%	3.33%	3.75%
	3	4.17%	4.43%	6.90%	2.17%	0.00%	0.00%	5.00%	3.68%	1.54%	0.00%	0.00%	0.00%	3.71%
	4	0.00%	5.36%	6.06%	1.39%	0.00%	0.00%	0.83%	2.24%	4.60%	6.25%	0.00%	2.86%	3.27%
	Average	**2.52%**	**3.63%**	**6.67%**	**1.52%**	**0.00%**	**3.23%**	**1.50%**	**3.66%**	**2.50%**	**6.98%**	**0.00%**	**1.87%**	**3.26%**
Off Peak	1	1.03%	3.02%	2.38%	0.79%	0.00%	0.00%	4.47%	3.86%	3.42%	1.52%	0.00%	0.97%	2.68%
	2	0.00%	3.66%	2.70%	1.34%	0.00%	0.00%	1.30%	0.98%	1.97%	0.00%	0.00%	1.15%	1.36%
	3	2.06%	3.37%	1.41%	1.61%	0.00%	0.00%	2.75%	4.22%	2.76%	1.00%	0.00%	2.97%	2.70%

Period	Interval	EB			SB			WB			NB			Ave
		Left	Thru	Right	Left	Thru	Right	Left	Thru	Right	Left	Thru	Right	
	4	0.00%	2.97%	2.17%	0.68%	0.00%	0.00%	0.46%	3.43%	1.27%	1.20%	0.00%	0.80%	1.86%
	Average	**0.78%**	**3.23%**	**2.04%**	**1.15%**	**0.00%**	**0.00%**	**2.10%**	**3.15%**	**2.36%**	**0.83%**	**0.00%**	**1.44%**	**2.16%**
PM Peak	1	1.47%	2.76%	2.00%	0.62%	0.00%	0.00%	0.00%	1.62%	1.06%	0.97%	0.00%	0.00%	1.36%
	2	0.00%	2.38%	3.70%	1.12%	0.00%	0.00%	0.80%	2.16%	0.00%	0.99%	0.00%	0.51%	1.40%
	3	0.00%	3.18%	4.35%	0.00%	0.00%	0.00%	0.00%	1.76%	0.00%	1.69%	0.00%	0.49%	1.39%
	4	0.00%	1.56%	0.00%	0.60%	0.00%	0.00%	0.66%	1.58%	0.00%	2.20%	1.54%	0.98%	1.10%
	Average	**0.48%**	**2.45%**	**2.26%**	**0.58%**	**0.00%**	**0.00%**	**0.36%**	**1.77%**	**0.19%**	**1.45%**	**0.42%**	**0.51%**	**1.31%**

Before Study (Mar.18 & Mar.19, 2015)

Table D-5.3 Traffic Volume (pce/15 min) and Traffic Flow (pce/hour) at University Pkwy & US 301

Time Period		EB			NB			WB			SB		
		Left	Thru	Right	Left	Thru	Right	Left	Thru	Right	Left	Thru	Right
AM Peak	1	29	178	4	84	261	31	130	195	122	34	205	0
	2	32	156	0	144	218	34	166	246	72	43	203	0
	3	29	190	0	134	222	22	170	249	92	58	154	0
	4	32	143	0	141	215	51	148	287	77	61	161	0
	Flow Rate	**122**	**667**	**4**	**503**	**916**	**138**	**614**	**977**	**363**	**196**	**723**	**0**
Off Peak	1	38	152	38	69	205	37	117	216	62	61	200	72
	2	36	214	65	125	213	24	147	306	81	51	160	99
	3	28	253	70	130	145	32	111	258	63	60	185	102
	4	43	263	52	84	211	16	165	339	53	55	165	71
	Flow Rate	**145**	**882**	**225**	**408**	**774**	**109**	**540**	**1119**	**259**	**227**	**710**	**344**
PM Peak	1	37	234	43	117	181	41	72	214	88	66	235	97
	2	56	228	45	141	212	36	61	155	131	73	354	119
	3	45	279	62	142	217	51	65	219	101	88	295	123
	4	53	245	59	118	195	47	66	187	99	114	385	158
	Flow Rate	**191**	**986**	**209**	**518**	**805**	**175**	**264**	**775**	**419**	**341**	**1269**	**497**

Table D-5.4 Traffic Volume (pce/15 min) and Traffic Flow (pce/hour) at University Pkwy & North Cattlemen Rd. /Creek Cooper

Time Period		**EB**			**NB**			**WB**			**SB**		
		Left	**Thru**	**Right**	**Left**	**Thru**	**Right**	**Left**	**Thru**	**Right**	**Left**	**Thru**	**Right**
AM Peak	1	20	445	19	81	12	5	27	409	41	3	6	17
	2	26	473	27	87	15	10	59	407	89	14	7	31
	3	25	495	31	94	8	8	63	479	66	12	7	25
	4	51	413	35	73	20	9	122	320	91	17	13	36
	Flow Rate	**122**	**1826**	**112**	**335**	**55**	**32**	**271**	**1615**	**287**	**46**	**33**	**109**
Off Peak	1	98	341	43	127	48	53	187	431	151	67	37	104
	2	108	283	38	151	54	86	234	412	155	113	66	88
	3	99	521	72	189	78	74	187	445	186	101	37	104
	4	84	451	47	148	61	53	220	483	159	84	52	126
	Flow Rate	**389**	**1596**	**200**	**615**	**241**	**266**	**828**	**1771**	**651**	**365**	**192**	**422**
PM Peak	1	69	484	51	162	44	57	132	439	95	104	46	176
	2	48	473	28	181	40	34	126	378	149	102	54	198
	3	40	486	24	183	50	38	151	463	141	120	73	206
	4	52	520	33	169	47	49	152	386	150	93	66	207
	Flow Rate	**209**	**1963**	**136**	**695**	**181**	**178**	**561**	**1666**	**535**	**419**	**239**	**787**

After Study (Mar.14 & Mar.15, 2017)

Table D-5.5 Traffic Volume (pce/15 min) and Traffic Flow (pce/hour) at University Pkwy & US 301 – After Study

Time Period		**NB**			**SB**			**EB (Main)**			**WB (Main)**		
		Left	**Thru**	**Right**	**Left**	**Thru**	**Right**	**Left**	**Thru**	**Right**	**Left**	**Thru**	**Right**
AM Peak	1	42	148	34	69	246	26	28	166	48	92	143	110
	2	39	179	40	60	213	17	13	141	76	114	151	84
	3	47	227	52	66	168	25	16	184	57	73	183	72
	4	41	143	58	108	240	39	19	168	63	130	233	83
	Flow Rate	**169**	**697**	**184**	**303**	**867**	**107**	**76**	**659**	**244**	**409**	**710**	**349**
Off Peak	1												
	2	41	163	71	66	192	17	27	162	40	111	214	96
	3	81	216	48	93	211	24	30	198	39	118	184	97
	4	56	271	81	101	214	9	20	218	50	105	201	96
	Flow Rate	**237**	**867**	**267**	**347**	**823**	**67**	**103**	**771**	**172**	**445**	**799**	**385**
PM Peak	1	91	262	78	130	192	10	43	262	44	87	216	86
	2	101	312	112	107	222	14	42	243	55	74	198	77
	3	93	329	74	88	234	19	36	261	45	89	219	71
	4	105	343	71	100	180	18	39	208	38	89	246	54
	Flow Rate	**390**	**1246**	**335**	**425**	**828**	**61**	**160**	**974**	**182**	**339**	**879**	**288**

Table D-5.6 Traffic Volume (pce/15 min) and Traffic Flow (pce/hour) at University Pkwy & North Cattlemen Rd. /Creek Cooper – After Study

Time Period		**NB**			**SB**			**EB (Main)**			**WB (Main)**		
		Left	**Thru**	**Right**	**Left**	**Thru**	**Right**	**Left**	**Thru**	**Right**	**Left**	**Thru**	**Right**
AM Peak	1	9	1	1	28	4	3	15	190	6	32	197	25
	2	12	4	26	81	10	6	18	427	10	60	458	61
	3	16	28	37	74	18	5	9	270	16	89	437	67
	4	112	523	90	63	14	15	35	372	17	15	11	14
	Flow Rate	**149**	**556**	**154**	**246**	**46**	**29**	**77**	**1259**	**49**	**196**	**1103**	**167**
Off Peak	1	67	57	53	118	61	42	76	406	42	220	355	136
	2	78	57	151	197	70	60	65	363	28	186	401	123
	3	86	72	163	160	78	45	46	324	42	137	338	105
	4	79	70	167	140	71	40	61	396	47	143	394	121
	Flow Rate	**310**	**256**	**534**	**615**	**280**	**187**	**248**	**1489**	**159**	**686**	**1488**	**485**
PM Peak	1	80	66	110	139	63	38	42	318	36	101	356	81
	2	65	44	164	110	51	29	34	302	17	143	419	93
	3	78	68	188	146	48	27	29	469	16	226	385	84
	4	82	74	186	122	46	25	44	478	32	115	414	102
	Flow Rate	**305**	**252**	**648**	**517**	**208**	**119**	**149**	**1567**	**101**	**585**	**1574**	**360**

Comparisons of Before and After Flows

The differences in traffic flow between the before and after measurements are shown in Table D-5.7 and Table D-5.8. The tables are color-coded as follows: green shows significant decrease, yellow shows modest change (either improvement or deterioration), and red shows significant deterioration in delay. Several gradations of each color are used to represent different variations within each classification.

Table D-5.7 Difference in Traffic Flow Rate (pce/hr) at University Pkwy & US 301

Period	EB (Main)			NB			WB (Main)			SB		
	Left	Thru	Right	Left	Thru	Right	Left	Thru	Right	Left	Thru	Right
AM Peak	-46	-8	240	-27	-26	184	-205	-267	-14	-200	-49	-31
Off Peak	-42	-111	-53	10	157	-77	-95	-320	126	-61	49	-42
PM Peak	-31	-12	-27	49	-23	-162	75	104	-131	-93	23	-114

Table D-5.8 Difference in Traffic Flow Rate (pce/hr) at University Pkwy & North Cattlemen Rd. /Creek Cooper

Period	EB (Main)			NB			WB (Main)			SB		
	Left	Thru	Right	Left	Thru	Right	Left	Thru	Right	Left	Thru	Right
AM Peak	-45	-567	-63	103	523	45	-75	-512	-120	-89	-9	-3
Off Peak	-141	-107	-41	-55	64	112	-142	-283	-166	0	39	-79
PM Peak	-60	-396	-35	-114	13	-139	24	-92	-175	-178	27	-59

Discussion

The following can be concluded regarding traffic volumes at this corridor:

- The traffic volume at both critical intersections (University Pkwy & US 301 and University Pkwy & North Cattlemen Rd. /Creek Cooper) decreased during all three time periods by an average of 7.3% and 14.3% respectively.

D.3.6 CONSIDERATION OF TRAFFIC FLOWS JOINTLY WITH QUEUE LENGTH

The differences in "Traffic Flow" and "Queue Length" between before and after measurements are shown in Table D-6.1 and Table D-6.3. The tables are color-coded as follows: green indicates that "Queue" decreases and "Traffic Flow" increases, Red indicates "Queue" increases and "Traffic Flow" decreases, No Color indicates "Traffic Flow" and "Queue" increase or decrease at the same time. Generally, green indicates improvement despite an increase in flow.

Table D-6.1 Differences in Traffic Flow (TF) and Queue Length (Q) at University Pkwy & US 301

Time Period	EB (Main)						NB						WB (Main)						SB					
	Left		Thru		Right		Left		Thru		Right		Left		Thru		Right		Left		Thru		Right	
	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q
AM Peak	-46	-4.34	-8	-9.43	240	0.00	-27	-7.25	-26	-5.21	184	0.00	-205	-10.93	-267	-12.09	-14	-0.05	-200	-6.09	-49	-8.77	-31	-0.30
Off Peak	-42	0.10	-111	-2.32	-53	-1.04	10	-4.04	157	4.52	-77	-1.94	-95	-0.46	-320	-5.98	126	-0.83	-61	0.34	49	1.11	-42	-1.13
PM Peak	-31	2.33	-12	1.66	-27	-0.60	49	-1.81	-23	12.06	-162	-2.80	75	-4.37	104	-5.23	-131	-7.15	-93	2.57	23	-1.28	-114	-4.40

Table D-6.2 Differences (%) in Traffic Flow (TF) and Queue Length (Q) at University Pkwy & US 301

Time Period	EB (Main)						NB						WB (Main)						SB					
	Left		Thru		Right		Left		Thru		Right		Left		Thru		Right		Left		Thru		Right	
	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q
AM Peak	-38%	-92%	-1%	-96%	6000%	N/A	-14%	-92%	-4%	-84%	N/A	0%	-33%	-95%	-27%	-97%	-4%	-100%	-40%	-79%	-5%	-82%	-22%	-100%
Off Peak	-29%	2%	-13%	-25%	-24%	-100%	5%	-46%	22%	76%	-22%	-100%	-18%	-8%	-29%	-61%	49%	-100%	-15%	6%	6%	17%	-39%	-100%
PM Peak	-16%	49%	-1%	18%	-13%	-100%	14%	-21%	-2%	204%	-33%	-100%	28%	-54%	13%	-66%	-31%	-100%	-18%	31%	3%	-13%	-65%	-100%

Table D-6.3 Differences in Traffic Flow and Queue Length at University Pkwy & North Cattlemen Rd. /Creek Cooper

Time Period	EB (Main)						NB						WB (Main)						SB					
	Left		Thru		Right		Left		Thru		Right		Left		Thru		Right		Left		Thru		Right	
	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q
AM Peak	-45	-0.41	-567	10.67	-63	-0.07	103	-0.16	523	0.07	45	-1.75	-75	-0.88	-512	-2.37	-120	-0.67	-89	-0.69	-9	0.72	-3	-1.29

Time Period	EB (Main)						NB						WB (Main)						SB					
	Left		Thru		Right		Left		Thru		Right		Left		Thru		Right		Left		Thru		Right	
	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q
Off Peak	-141	-1.68	-107	12.89	-41	-2.60	-55	4.00	64	6.51	112	2.23	-142	8.80	-283	3.18	-166	-6.08	0	-1.21	39	-0.79	-79	-4.17
PM Peak	-60	-3.75	-396	5.00	-35	-1.06	-114	6.52	13	5.82	-139	0.67	24	10.06	-92	5.34	-175	-4.46	-178	-3.70	27	2.64	-59	-2.83

Table D-6.4 Differences (%) in Traffic Flow and Queue Length at University Pkwy & North Cattlemen Rd. /Creek Cooper

Time Period	EB (Main)						NB						WB (Main)						SB					
	Left		Thru		Right		Left		Thru		Right		Left		Thru		Right		Left		Thru		Right	
	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q
AM Peak	-37%	-17%	-31%	135%	-56%	-62%	224%	-15%	1585%	13%	41%	0%	-28%	-20%	-32%	-24%	-42%	-100%	-27%	-13%	-16%	36%	-9%	-100%
Off Peak	-36%	-21%	-7%	79%	-21%	-100%	-15%	66%	33%	126%	27%	31%	-17%	68%	-16%	22%	-25%	-100%	0%	-11%	16%	-7%	-30%	-100%
PM Peak	-29%	-60%	-20%	20%	-26%	-100%	-27%	108%	5%	116%	-18%	6%	4%	114%	-6%	44%	-33%	-100%	-26%	-28%	15%	38%	-33%	-100%

Discussion

The following can be concluded from the comparison of traffic flows jointly with queue length:

- From the colored cells one could distinguish the significant differences in performance between US 301 and N.Cattleman Rd.
- Significant number of white or green colored cells in US 301, indicate the improvement or neutral condition on the western part of the corridor, whereas for N. Cattleman road, a significant number of red cells are due to large queues despite the reduction in traffic flows. This indicates overall deterioration performance on the eastern part of the corridor.

D.4 QUESTIONNAIRE

SECTION 1: RESPONDENT'S INFORMATION

Name ____

Organization ____

Position ____

Address ____

Phone ____

Fax ____

E-mail ____

SECTION 2: PREVIOUS TRAFFIC CONTROL TECHNOLOGY

1. Please specify the type of traffic control used before Adaptive Signal Control Technology (ASCT) was installed for your site. (Please check all that apply.)

a. Fixed time coordinated control **b. Actuated coordinated control**

c. Fixed-time isolated control d. Actuated isolated control

e. Other, please specify: ____

2. What type of traffic controller was used before ASCT was implemented?

a. NEMA TS-1 **b. NEMA TS-2**

c. 170 d. 2070

e. Other, please specify: ____

8. How many detectors were used before ASCT was implemented for a typical intersection (e.g. *Newberry Rd & NW 75th St*)? Please also specify the location where the detectors were typically placed for each specific detector type (video detection, inductive loops, radars, etc.).

Video detection in each movement

9. What was the frequency of retiming the signal timing plan for the corridor before ASCT was implemented?

Last retiming for Honore Ave. through I-75 was October 2014. Fine tuning and controller updates for Airport Cir. through Honore Ave. was 6 months ago

10. How often were maintenance and updates performed to your previous control system in terms of the following components?

a. Detection: **Annual Maintenance Schedule as needed**
b. Control Hardware: **Annual Maintenance Schedule as needed**
c. Sofware: **Annual Maintenance Schedule as needed**

11. What was the annual maintenance cost in terms of hardware, software and personnel of the previous control system for the whole corridor where the ASCT is now deployed?

____

____

12. What was the life expectancy of your traffic control before the deployment of ASCT? (Please refer to the hardware and software.)

Traffic controllers haven't changed. InSync modules/processes have been added to the ASCT

13. How many crashes of each category were reported during the past 4 years before the ASCT implementation? (2011-2014)

	K (Killed)	A (Disabling Injury)	B (Evident Injury)	C (Possible Injury)	O (No Apparent Injury)
2011					
2012					
2013					
2014					

SECTION 3: ADAPTIVE TRAFFIC CONTROL SYSTEM (ASCT)

9. Which Adaptive Traffic Control System (ASCT) does your agency deploy?

g. **InSync; Version: latest – InTraffic**
h. Synchro Green; Version: ____________________

i. Other; please specify: ______________________

22. Did you consider any other ASCT before you selected this one for installation? Why did you reject the other(s) in favor of this one?

No. There were not many options and there was the interest to try this out

23. What were your major criteria for choosing the ASCT for the selected corridor?

To see how InSync reacts to the varying traffic conditions. The mobility needs of the corridor

24. What is the life expectancy of your ASCT system? (Consider both hardware and software.)

3-5 years

25. What type of traffic controller is currently in use after the ASCT implementation?

a. Same as before the ASCT implementation
b. New, please specify type: ____
c. **Same, but the following updates were needed: 980 controllers**

26. How many and what type of detectors are used after the ASCT implementation on an intersection level? Please note any updates that the previous detection system needed so as to work with your ASCT.

Replaced old cameras with InSync cameras

27. Was there a need to update your previous software operating system at your Transportation Management Center in order to get your current ASCT software working?

a. Yes **b. No**

26. How often do you plan to perform physical maintenance or updates on your new ASCT in terms of the following components?

a. Detection: **Yearly, and as needed. Maintenance needs increases with detection cameras**
b. ASCT Hardware: **Yearly, and as needed**
c. Software: **Yearly, and as needed**

27. Was there a need for training your personnel in order to operate the new ASCT?

a. **Yes,** Num. of employees trained **10**, Hours of training per employee **8-16** b. No

28. Was there any change on the number of staff required to operate/maintain the effective signal ASCT operations? (Comparison of staff needed before and after the deployment.)

No

29. Where did expertise come from for your personnel's training, the ASCT installation, operation and the system's maintenance? Please check all that apply.

	In-House	Vendor	Contractor
Employee Training		**yes**	
ASCT Installation	**yes**	**yes**	
ASCT Operation	**yes**		
ASCT Maintenance	**yes**		

30. How many crashes of each category were reported after ASCT was implemented?

K (Killed)	A (Disabling Injury)	B (Evident Injury)	C (Possible Injury)	O (No Apparent Injury)

SECTION 4: COST COMPONENTS

37. What is the overall capital cost of purchasing the ASCT (in terms of the software and licenses of the ASCT) for the corridor? If the purchase includes any service of implementation, personnel training or maintenance, please also specify here.

Total cost InSync (Manatee): $129,650, total cost InSync (Sarasota): $496,700, total cost BlueTOAD (Manatee); $30,690, total cost BlueTOAD (Sarasota): $16,410

38. What is the implementation cost (considering the installation on site, any updates in software and hardware, design needs and contract hours) for the ASCT corridor? Please specify if this cost is partially or totally included in previous cost items.

____

____

39. What was total cost for training your personnel to operate and manage the new ASCT? Please specify if this cost is partially or totally included in previous cost items.

____

____

40. What is the expected maintenance cost (in terms of hardware, software and personnel) for the ASCT corridor on a yearly basis? Please specify if this cost is partially or totally included in previous cost items.

____

____

41. Can you specify the costs for the following ASCT components?

a. Firmware $_____________ b. Software $______________

c. Equipment $____________ d. Maintenance of Traffic Cost $______________

e. Design Needs $___________ f. Contract Help/Agency Hours Cost $__________

SECTION 5: INSTITUTIONAL ISSUES

42. Were there any institutional issues that you had to overcome while implementing and operating the ASCT project? Please categorize and explain those issues below.

a. Organization and Management Institutional Issues:

Issues with using two different contractors

b. Regulatory and Legal Institutional Issues:

____

____

c. Human and Facility Resources Institutional Issues:

Installation: Employees were pulled away from their regular day-to-day schedule to help complete the installation as they were pressed with time. Public finding it difficult to adjust to new system

d. Financial Institutional Issues:

Additional Ethernets had to installed at some intersections, extra maintenance cost, expensive camera

43. Which component (e.g. detection, communication, hardware, software, licensing or other) of your ASCT deployment is the most challenging to maintain and why?

i. **Detection: expensive cameras to maintain and camera failing to work**

ii. Communication

iii. Hardware/Software

44. What happens to your ASCT operations when some of your detectors fail on the corridor level?

g. ASCT triggers an alarm and notifies operators

h. **ASCT switches to an "off-line" mode by implementing Time-Of-Day plans (for detection cameras)**

i. Combination of the above. d. Other (please describe): __________

29. How is your ASCT performance evaluated?

i. **In-house**

j. By an independent evaluator, please describe: ______________________________

k. Not applicable—there is no evaluation

30. If the corridor on which the ASCT operates experiences over saturation, how would you rate the operation of the ASCT in response to these traffic conditions?

k. ASCT prevents or eliminates oversaturation

l. **ASCT** eliminates or **reduces the extent of the periods of oversaturation**

m. ASCT adversely affects the traffic conditions during periods of oversaturation

n. Other: ____

o. Oversaturation is very rare on the corridors operated by our ASCT

31. Based on your opinion and the up-to-date operation, when are ASCT operations proven to be the most effective?

a. Peak periods **b. Off-peak periods**

c. Shoulders of peak periods d. Other, please specify: ______________

32. Has the level of performance of ASCT been sustained since its installation?

a. Yes b. No; why not? ______________________________________

37. What was the public reaction to the ASCT operation? Have you received any complaints about long delays and queues?

Violated drivers' expectation. Complains have reduced

38. How satisfied are you, in general, with your ASCT deployment?

a. Very satisfied **b. Somewhat satisfied**

c. Neutral d. Somewhat dissatisfied

e. Not satisfied at all

37. Are there any other costs or benefits related to ASCT deployment that you would like to report?

a. Benefits: **very responsive**

b. Costs: **maintenance cost of camera**

38. Would you consider expanding the ASCT program to any other corridors? Why or why not? If yes, would you use the same technology/firmware or have any suggestions for other systems? If so, why?

ASCT in general : Yes

InSync: Maybe not, May rather want to deploy Synchro Green

SECTION 6: SAFETY ISSUES

37. Do you have any anecdotal / qualitative data on safety benefits / disbenefits of the signal improvement along this corridor?

____

____

38. Has there been other changes along this corridor over the analysis period that could affect the safety of this corridor either positively or negatively?

____

____

39. Are you aware of any changes to the crash data reporting procedures over the analysis period? If yes, what are the changes?

____

____

____

____

40. What are your overall reactions to the trends presented in the report?

____

____

41. Is there another comparable corridor in which signal improvements have not been made to which the results of this corridor can be compared to?

____

____

42. Other thoughts / comments for us?

____

____

Thank you for your help in completing this survey. Your responses will help us with evaluating the deployment and benefits of your current ASCT.

If you have any questions regarding the survey, please contact Marian Ankomah, email: moankomah@ufl.edu or Tyler Valila, email: tvalila67@ufl.edu who work under the direct supervision of the faculty advisor Dr. Lily Elefteriadou.

D.5 BENEFIT COST ANALYSIS

Table D-7.1 Monetized Saving

Monetized Saving	
Time saving	$4,373,801.65
Fuel Consumption saving	$473,638.96
Air Pollution Saving	$30,469.40
Safety Saving	$4,880,524.32
Total Saving without Safety	$4,877,910.02
Total Saving	$9,758,434.34
Total Cost	$657,000.00
B/C Ratio without safety	7.424520574
B/C Ratio	14.8530203

Table D-7.2 Monetized Saving with No Emissions

Monetized Saving	
Time saving	$4,373,801.65
Fuel Consumption saving	$473,638.96
Safety Saving	$4,880,524.32
Total Saving without Safety	$4,847,440.61
Total Saving	$9,727,964.94
Total Cost	$657,000.00
B/C Ratio without safety	7.378144009
B/C Ratio	14.80664374

APPENDIX E: Summary of 23rd Street, Bay County

E.1 EXECUTIVE SUMMARY

The objective of this research is to evaluate the implementation of proposed Adaptive Signal Control Technologies (ASCT) traffic operations at several arterial corridors in Florida, before and after the installation of specific ASCT, document the advantages and disadvantages of different approaches and implementations, and provide recommendations for state-wide implementation of ASCT. This Appendix summarizes the before and after field data collected along the 23rd Street, Panama City corridor from Lisenby Ave to SR 77, and provides some observations and initial conclusions.

Floating car runs were conducted with the UFTI instrumented vehicle to collect vehicle travel times before the implementation of InSync, during three time periods (AM Peak, Off Peak and PM Peak). In addition, turning movement counts and queue lengths were collected at two critical intersections (Jenks Ave & SR 77). Based on these, five performance measures were obtained for the before study periods: Link/Route Travel Time, Delay at Intersections, Queue Length (at critical intersections), Queue to Lane Storage Ratio (at critical intersections), and Passenger Car Equivalent (PCE) flows (at critical intersections). For each performance measure, a comparison between the before and after data is conducted and presented in this appendix.

The following were concluded:

- The travel time in the EB direction decreased during all study time periods (AM Peak, Off Peak and PM Peak) by an average of 0.51 min (7.57%). The travel time in the WB direction decreased during all study time periods (AM Peak, Off Peak and PM Peak) by an average of 0.9 min (13.07%).
- The intersection through movement delay in the EB direction decreased at 5 out of the 9 intersections along the corridor by an average of 1.07 sec/intersection (4.46%). The intersection through movement delay in the WB direction decreased at 6 out of 9 intersections by an average of 5.78 sec/intersection (21.77%).
- The average queue length for the Jenks Ave. & 23rd Street intersection increased by an average of 0.11 vehicles (11.96%) during the AM peak and Off Peak analysis periods. The average queue length for the SR 77 & 23rd Street intersection increased by an average of 0.89 vehicles (143.30%) during the Off Peak and PM peak analysis periods.
- The traffic volume for the Jenks Ave. & 23rd Street intersection increased during Off Peak and PM peak analysis periods by an average of 88 pce/h/ln (2.03%); for the SR 77 & 23rd Street intersection it increased during all analysis periods by an average of 259 pce/h/ln (6.98%).

An interview with Bay County Public Works was conducted regarding both Parkway corridor and 23rd St. corridor, certain comments of the interview apply to both corridors. The interview indicated that InSync operates most effectively when conditions are undersaturated. The traffic operations engineer of Bay County Public Works also reported an average of 7% reduction on the travel time after the implementation of the InSync system (in certain cases, the reduction was even close to 20%, however, the side streets suffered significant delay).

E.2 CORRIDOR INFORMATION

Panama City Beach is a city in Bay County, Florida. Although it has a small population of about 12,018 as per 2010 census, the traffic in the city is higher during the tourist season. It is a popular destination during spring break and summer seasons. The 23rd Street, Panama City corridor, passing by the Panama City Mall as well as many supermarkets and restaurants on both sides of the street, attracting large amount of traffic every day. Traffic demand on both the eastbound and westbound directions were observed to be high, and eastbound demand was observed to be higher than westbound demand. The Off Peak and PM Peak were observed to have the heavier traffic for both the eastbound and westbound directions. Figure E-1 provides a schematic of the 23rd Street, Panama City corridor. Table E-1 lists the intersections along the corridor. Two intersections (Jenks Ave and SR 77) were selected as the critical intersections along the corridor and detailed turning movement and queue counts were collected at these. Figure E-2 provides the lane configuration of these two critical intersections. The data collection for this project was conducted during the low tourism season.

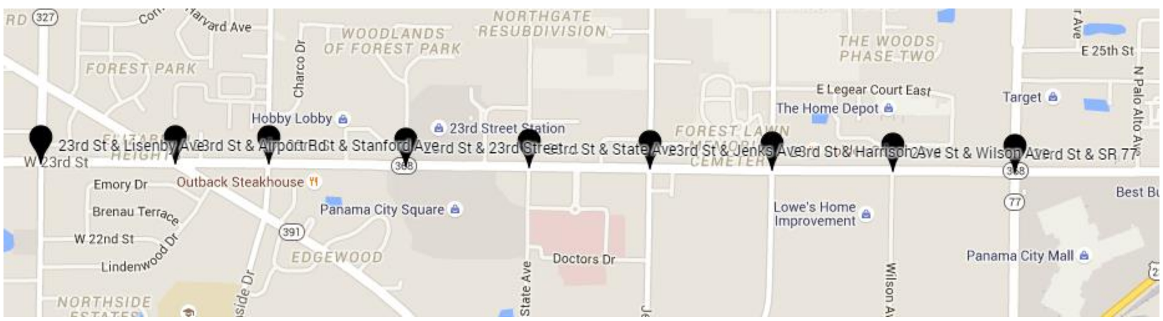
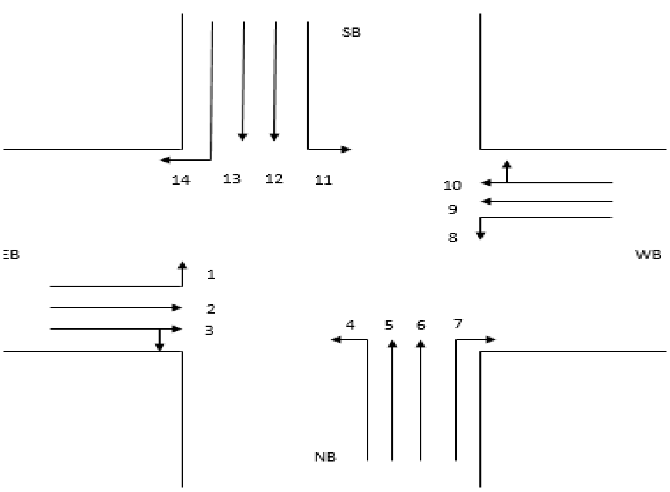
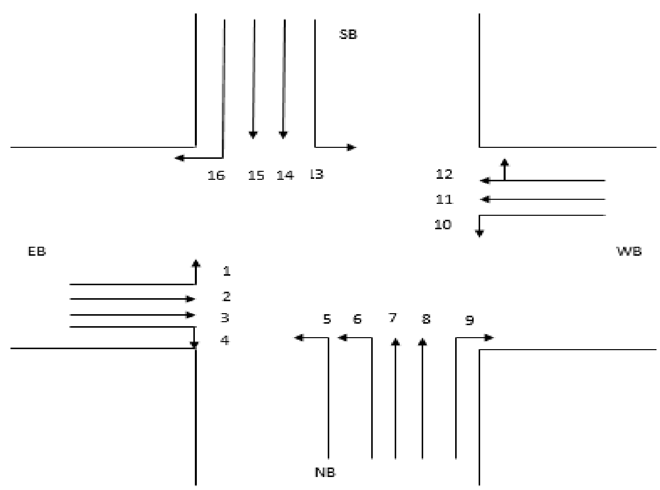


Figure E-1 Schematic of the 23rd Street, Panama City Corridor.

Table E-1 Intersections along the 23rd Street, Panama City Corridor

	Signalized Intersection	Distance in Between	Unsignalized Intersections in Between
1	Lisenby Ave		
2	Airport Rd	0.27 miles	1
3	Stanford Ave	0.19 miles	1
4	23rd Street Plaza	0.28 miles	2
5	State Ave	0.25 miles	4
6	**Jenks Ave**	0.25 miles	2
7	Harrison Ave	0.25 miles	2
8	Wilson Ave	0.25 miles	1
9	**SR 77**	0.25 miles	0





(a) Jenks Ave. & 23rd Street

(b) SR 77 & 23rd Street

Figure E-2 Schematic and Overview of Critical Intersections

E.3 PERFORMANCE MEASURES

Five performance measures were evaluated: Link/Route Travel Time, Delay at Intersections, Queue Length (critical intersections), Queue-to-Lane Storage Ratio (critical intersections), and PCE Flows (critical intersections). For each performance measure, a comparison between the before and after data is conducted and the results of the differences (“after data” – “before data”) are presented.

E.3.1 ROUTE TRAVEL TIME

The average travel time (min) along the route was measured using a floating car. During the AM peak of the after data collection there was an incident nearby, around 0.2 mile from the studied corridor, which could affect the traffic flow of the Lisenby Ave. and 23rd St. intersection. Figure E-1.1 provides a schematic of the incident location. According to the traffic engineer there who informed us about this incident, the incident may reduce the traffic flow through the corridor. The research team collected data during that time, also, Bluetooth data collected on 9/22/2015 was obtained as the travel time for AM peak without incident. In the following tables we report separately any data collected when the incident was active. A total of 41 runs for EB and 40 runs for WB were conducted during the before study, and a total of 42 runs for EB and 41 runs for WB were conducted during the after study.

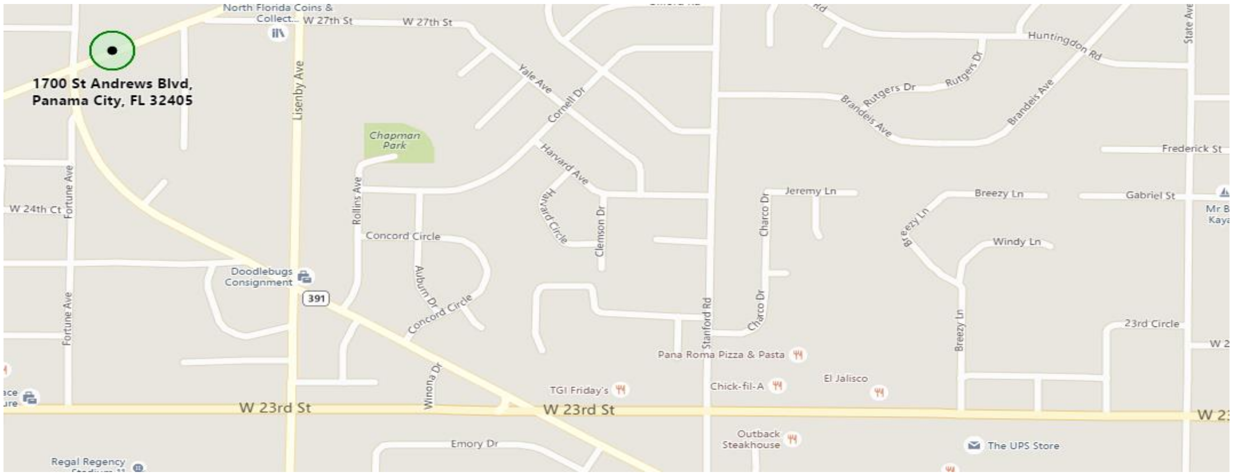


Figure E-1.1 Schematic Incident Location

Before Study (Jan. 27 & Jan.28, 2015)

Table E-1.1 Route Travel Time (min)

Route TT (min)	AM	Off	PM	Average
23rd Street EB	6.14	7.62	6.71	**6.77**
23rd Street WB	5.99	7.92	6.9	**6.89**

After Study (Sep. 22 & Sep. 23, 2015)

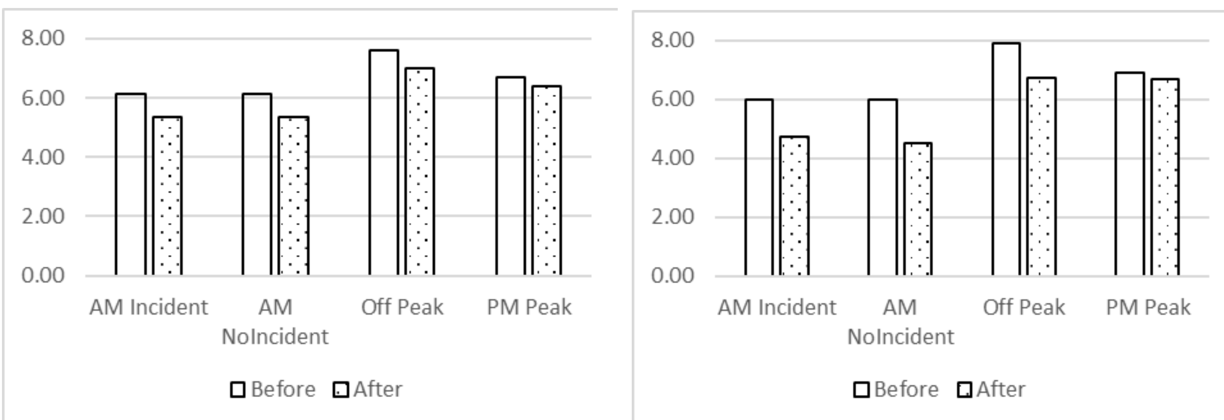
Table E-1.2 Route Travel Time (min)

Route TT (min)	AM (Incident)	AM (No Incident)	Off	PM	Average
23rd Street EB	5.35	5.37	7.01	6.40	6.26
23rd Street WB	4.75	4.54	6.74	6.70	5.99

Comparisons of Before and After Travel Times

Table E-1.3 Change in Percentage of Route Travel Time (After – Before)

Route TT (min)	AM (Incident)	AM (No Incident)	Off	PM	Average
23rd Street EB	-0.79 (-12.83%)	-0.77 (-12.59%)	-0.61 (-8.02%)	-0.31 (-4.69%)	**-0.51 (-7.57%)**
23rd Street WB	-1.25 (-20.84)	-1.46 (-24.32%)	-1.18 (-14.90%)	-0.20 (-2.95%)	**-0.90 (-13.07%)**



(a) 23rd Street. EB (b) 23rd Street WB

Figure E-1.2 Travel Times Along 23rd Street., Panama City

Discussion

The following can be concluded from the comparison of travel times:

- The travel time along 23rd Street in the EB decreased by an average of 0.51 min (7.57%). The EB direction carries the majority of the traffic during the PM peak. The highest decrease occurred during the AM Peak when the demand is relatively lower, with a decrease of 0.79 min (12.83%) with incident and 0.77 min (12.59%) without incident. The incident does not seem to affect the EB travel time.
- The travel time along 23rd Street in the WB decreased by an average of 0.90 min (13.07%). The WB direction carries the majority of the traffic in the AM peak, which has significant decrease of travel time, the highest decrease occurred during the AM Peak with a decrease of 1.25 min (20.84%) with incident and 1.46 min (24.32%) without incident.

- For both directions, the AM Peak and Off Peak have the most decrease in travel time, while the PM peak, which has the heaviest traffic, shows relatively small improvement.

Overall, travel time along the main street decreased after the installation of the system.

E.3.2 DELAY

Delay (sec) at each intersection along the corridor was also obtained using the floating car measurements. For the AM peak during the after study, we use the delay obtained while the nearby incident was active, since the Bluetooth data obtained (data without an incident) for the AM peak do not provide the intersection delay.

Before Study (Jan. 27 & Jan.28, 2015)

Table E-2.1 Delay (sec) for each Intersection Through Movement Along the EB Direction

Delay at Intersections (sec)	AM Peak	Off Peak	PM Peak
Lisenby Ave	11.74	10	28.34
Airport Ave	32.44	30.05	10.52
Stanford Ave	16.16	10.75	10.18
23rd Street Plaza	5.95	39.5	20.57
State Ave	13.34	52.25	12.17
Jenks Ave	38.48	29	42.42
Harrison Ave	9.64	23.15	24.21
Wilson Ave	10.09	26.05	13.29
SR 77	44.28	43.15	48.17

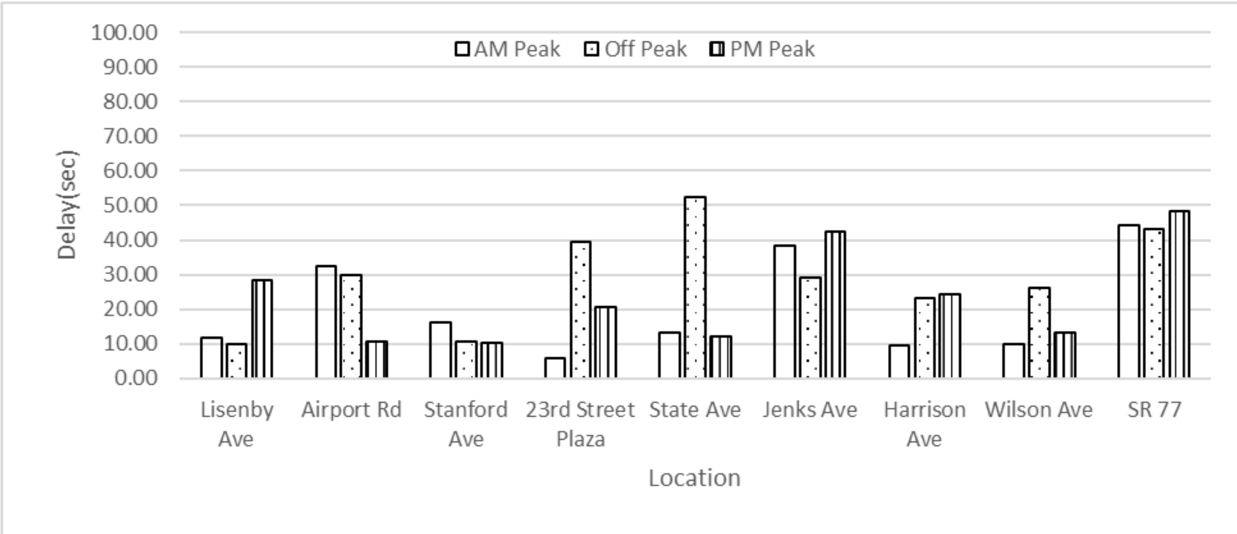


Figure E-2.1 Delay (sec) for each Intersection Through Movement Along the EB Direction

Table E-2.2 Delay (sec) for each Intersection Through Movement Along the WB Direction

Delay at Intersections (sec)	AM Peak	Off Peak	PM Peak
Lisenby Ave	93.4	95.33	53.36
Airport Ave	12.32	18.83	26.14
Stanford Ave	5.08	8.83	15.93
23rd Street Plaza	27.49	71.88	19.93
State Ave	16.56	15.88	49.47
Jenks Ave	3.97	25.55	24.3
Harrison Ave	4.81	22.42	31.5
Wilson Ave	2.49	29.34	6.8
SR 77	7.88	28.61	3.75

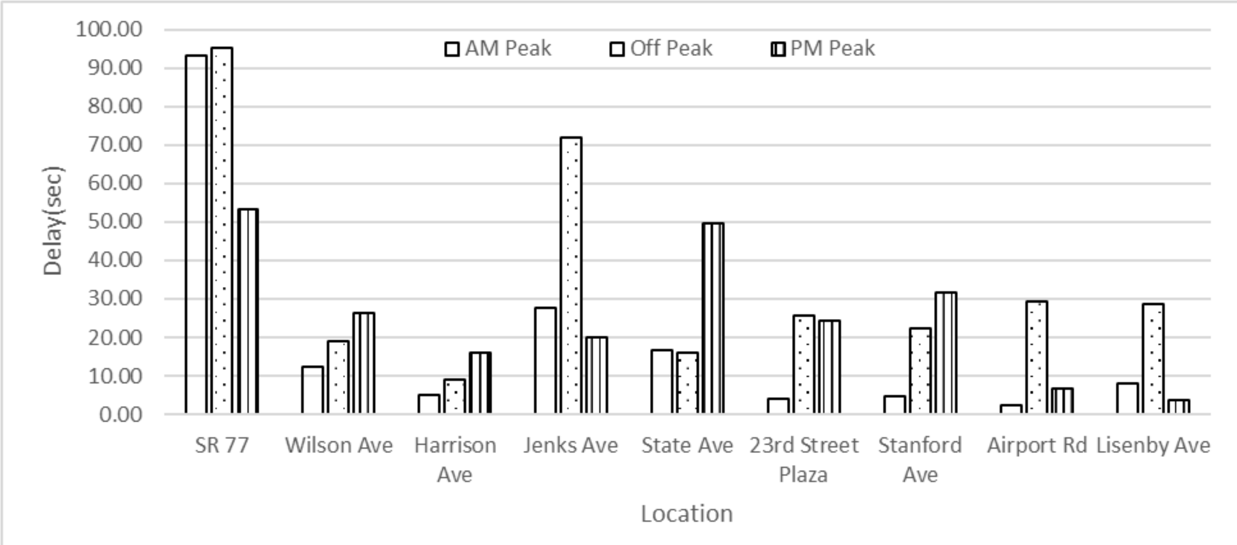


Figure E-2.2 Delay (sec) for each Intersection Through Movement Along the WB Direction

After Study (Sep. 22 & Sep. 23, 2015)

Table E-2.3 Delay (sec) for each Intersection Through Movement Along the EB Direction

Delay by Intersection (sec)	AM Peak (Incident)	Off Peak	PM Peak
Lisenby Ave	13.47	17.34	26.14
Airport Ave	14.11	41.25	24.96
Stanford Ave	12.41	8.19	9.04
23rd Street Plaza	9.31	22.06	19.69
State Ave	15.70	17.64	10.44
Jenks Ave	17.80	51.89	60.24
Harrison Ave	10.63	16.89	8.05
Wilson Ave	4.88	39.34	25.71
SR 77	60.45	38.53	31.09

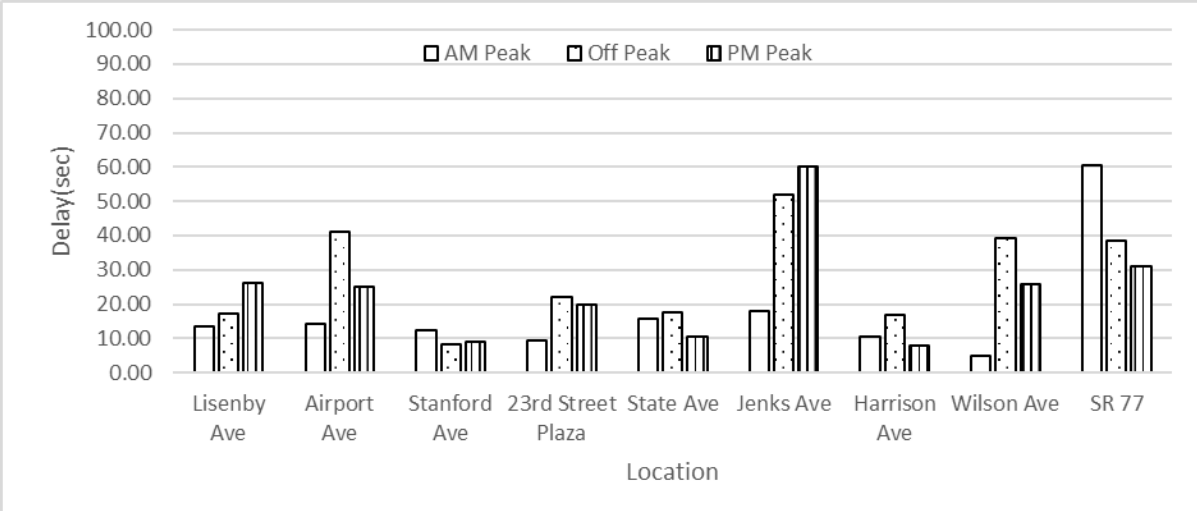


Figure E-2.3 Delay (sec) for each Intersection Through Movement Along the EB Direction

Table E-2.4 Delay (sec) for each Intersection Through Movement Along the WB Direction

Delay at Intersections (sec)	**AM Peak (Incident)**	**Off Peak**	**PM Peak**
SR 77	44.70	50.33	40.25
Wilson Ave	4.51	6.45	6.13
Harrison Ave	13.88	23.05	27.07
Jenks Ave	16.36	41.03	34.39
State Ave	5.60	61.77	19.00
23rd Street Plaza	4.51	15.93	32.72
Stanford Ave	6.52	26.91	39.60
Airport Ave	2.72	4.86	21.20
Lisenby Ave	5.47	9.27	15.12

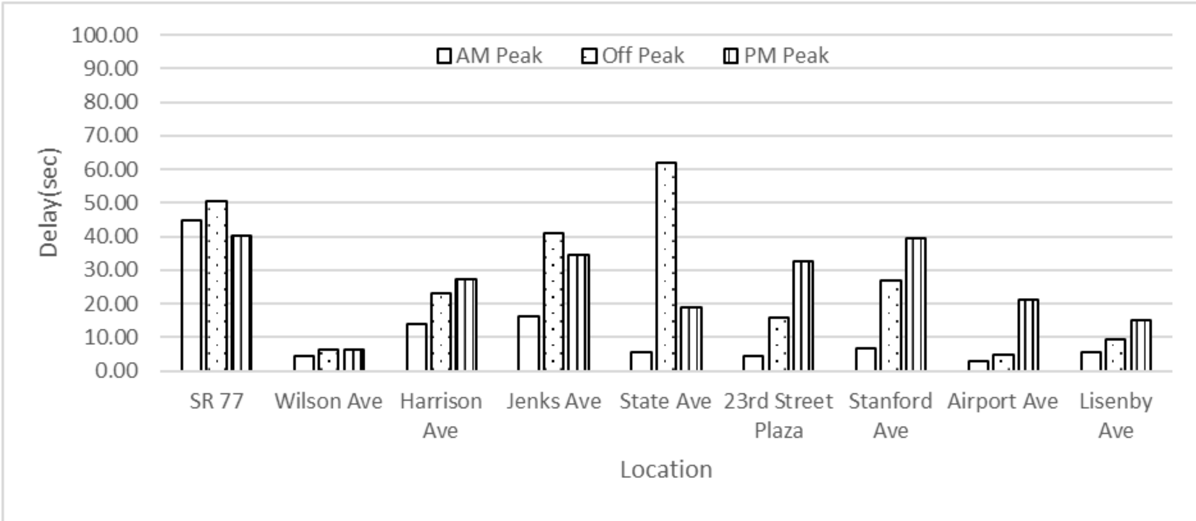


Figure E-2.4 Delay (sec) for each Intersection Through Movement Along the WB Direction

Comparisons of Before and After Intersection Delay Times

The differences in delay between the before and after study are shown in Table E-2.5 and Table E-2.6. The tables are color-coded as follows: green shows significant improvement, yellow shows modest change, and red shows significant deterioration in delay.

Table E-2.5 Difference in Delay (sec) for each Intersection Through Movement Along the EB Direction

Delay (sec)	AM Peak (Incident)	Off Peak	PM Peak	Average
Lisenby Ave	1.73	7.34	-2.20	2.07
Airport Rd	-18.33	11.20	14.44	1.37
Stanford Ave	-3.75	-2.56	-1.14	-2.62
23rd Street Plaza	3.35	-17.44	-0.88	-3.94
State Ave	2.37	-34.60	-1.73	-9.68
Jenks Ave	-20.67	22.89	17.83	4.54
Harrison Ave	0.99	-6.26	-16.16	-6.66
Wilson Ave	-5.20	13.29	12.42	6.22
SR 77	16.17	-4.62	-17.08	-0.87
Average	-2.59	-1.20	0.61	-1.07

Table E-2.6 Difference in Delay (sec) for each Intersection Through Movement Along the WB Direction

Delay (sec)	AM Peak (Incident)	Off Peak	PM Peak	Average
SR 77	-48.69	-45.00	-13.12	-37.87
Wilson Ave	-7.81	-12.37	-20.01	-12.77
Harrison Ave	8.80	14.22	11.15	11.39
Jenks Ave	-11.13	-30.85	14.46	-11.62
State Ave	-10.96	45.89	-30.48	1.60
23rd Street Plaza	0.53	-9.62	8.42	-0.40
Stanford Ave	1.71	4.50	8.10	4.43
Airport Rd	0.23	-24.48	14.40	-3.14
Lisenby Ave	-2.41	-19.35	11.37	-3.63
Average	-7.75	-8.56	0.48	-5.78

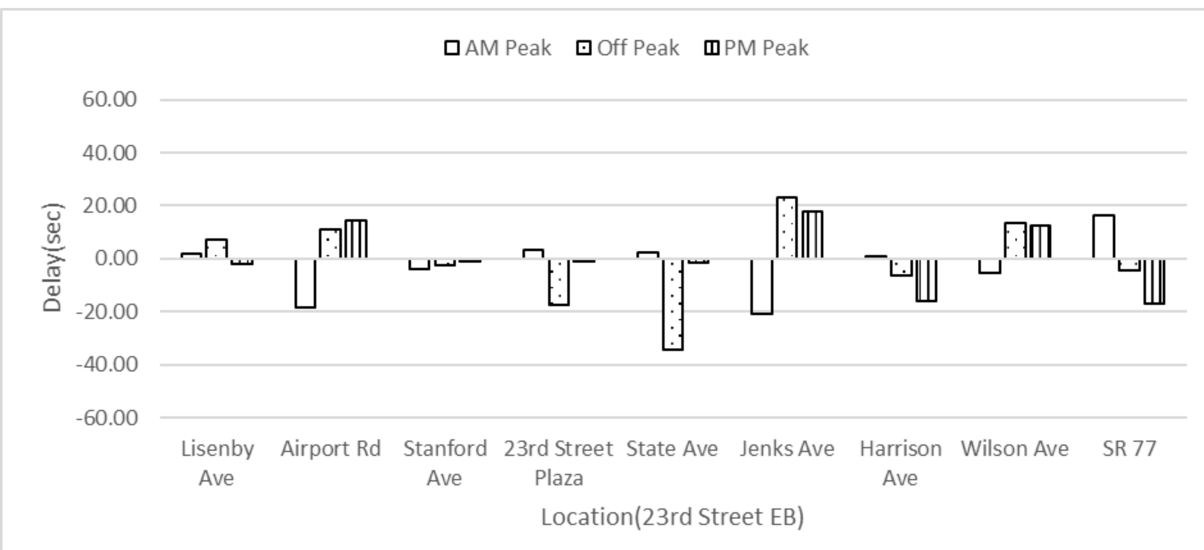


Figure E-2.5 Difference in Delay (sec) for each Intersection Through Movement Along the EB Direction

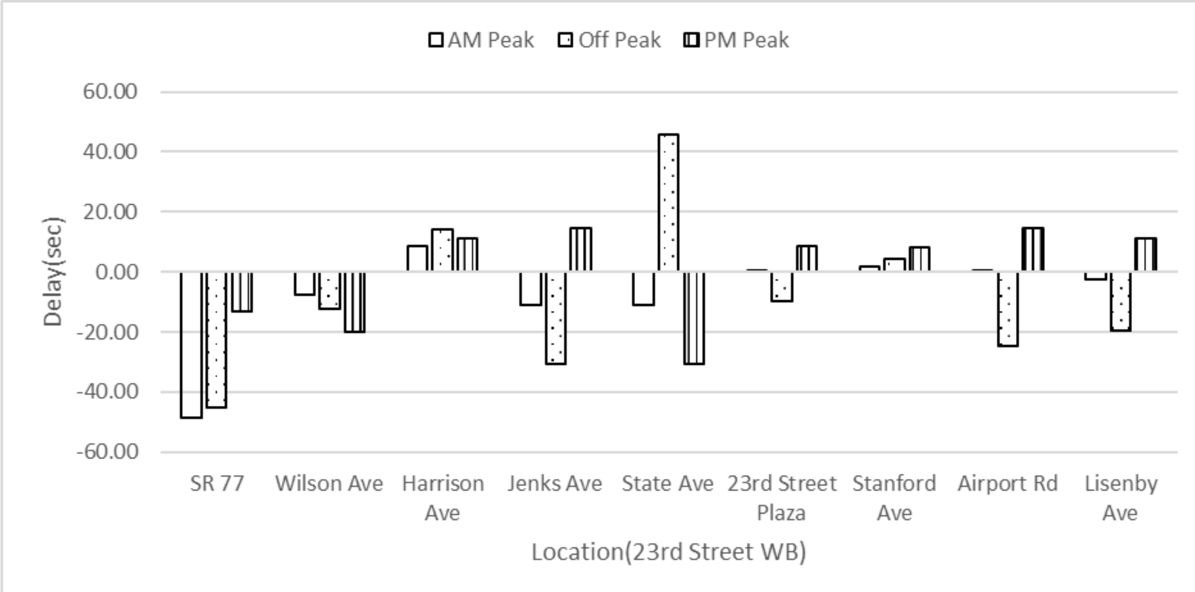


Figure E-2.6 Difference in Delay (sec) for each Intersection Through Movement Along the WB Direction

Discussion

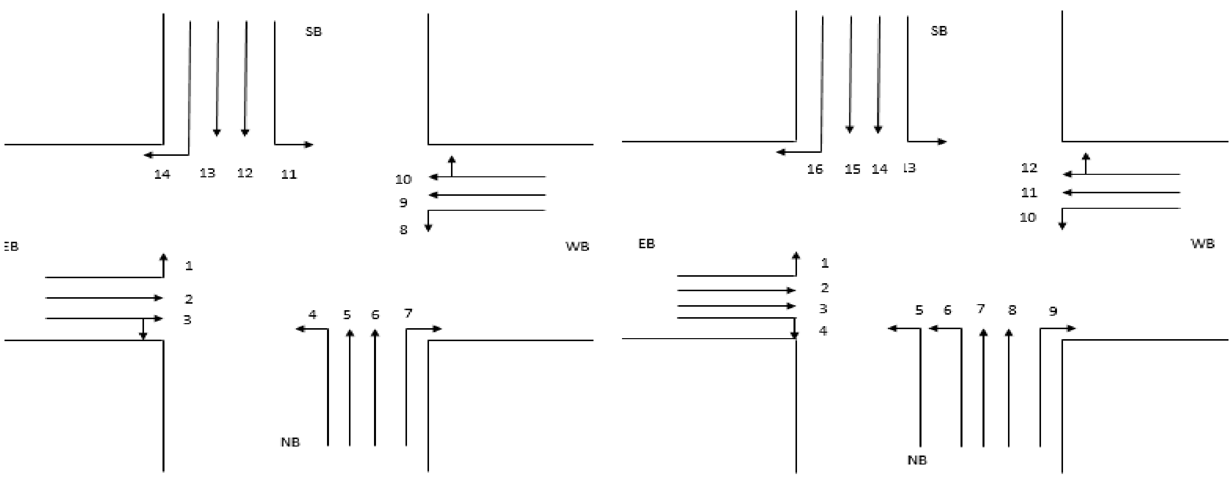
The following can be concluded from the comparison of delays:

- The intersection delay in the EB direction decreased at 5 out of the 9 intersections by an average of 1.07 sec/intersection (4.46%). Delay decreased for the AM Peak and Off Peak, but increased during the PM Peak, when traffic is heaviest in the EB.
- The intersection delay in the WB direction decreased at 6 out of the 9 intersections by an average of 5.78 sec/intersection (21.77%). Delay decreased for the AM and Off Peak, but increased during the PM Peak, despite the fact that the WB has lower demand than the EB during this time.
- Even though the PM peak delay increased, with the high decrease of delay for the AM peak and Off Peak, the average delay decreased overall for both directions.

E.3.3 QUEUE LENGTH

Queue length (number of vehicles/lane) is presented by movement and by time period. This measure is used to evaluate oversaturated conditions at the critical intersections along the study corridors. Note that the queue length reported here is the observed maximum number of vehicles queued during each cycle, and does not represent the total number of vehicles that may have stopped during the cycle. During some time periods, because of cycle failure, vehicles need to stop multiple times before passing through the intersection. For the AM peak after study, the research team collected the SR 77 and 23rd St. intersection traffic counts on 9/22/2015 when the incident was active, as the “AM Peak Incident”, and we use the queue length collected from video during a weekday (9/29/2015, Tuesday) for the SR 77 and 23rd St. intersection as the “AM Peak No Incident”.

Figure E-3.1 presents the schematic of the lane configurations at the two critical intersections. Queue length is reported for each of these lanes.



(a) Jenks Ave. & 23rd Street (b) SR 77 & 23rd Street

Figure E-3.1 Lane Configuration of Critical Intersections

Before Study (Jan. 27 & Jan.28, 2015)c

Table E-3.1 Average Queue Length by Lane (#vehs/lane) at Jenk Ave & 23rd Street

Time Period		EB (Main)			NB				WB (Main)			SB			
		Left	Thru	Thru/Right	Left	Thru		Right	Left	Thru	Thru/Right	Left	Thru		Right
		1	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**	**13**	**14**
AM Peak	1	3.71	6.43	4.86	3.86	4.14	1	0.71	1.5	5.17	4.5	2	6.67	4.33	2.67
	2	5.78	8.33	7.44	1.56	3.67	1.33	0.67	2.33	6.89	7.44	4.11	6.89	5.22	2.33
	3	3.43	5.43	6.71	1	5.14	1.14	1.14	1.13	2.75	3.75	1.13	4.38	3.63	2.63
	4	2.63	5.63	6.75	1.63	3.88	1.5	0.75	1.63	4.38	5.13	3	3.25	3.13	2.63
	Average	**3.89**	**6.45**	**6.44**	**2.01**	**4.21**	**1.24**	**0.82**	**1.65**	**4.8**	**5.2**	**2.56**	**5.3**	**4.08**	**2.56**
Off Peak	1	4	4.43	6.57	4.43	4	1.29	2.57	1.57	13	15.43	6.57	3.43	2.57	3.57
	2	4.29	8.86	10	7.86	5	2.14	3	2.57	9	10.86	4.86	4	3.43	2.57
	3	4.29	9	9.71	12.43	7.14	2.29	2.57	1.57	9.86	12.71	9.57	5	2.57	4.86
	4	5.43	10.43	12.14	3.71	5.43	1.29	2.43	3.14	8.86	14.43	2.71	5.14	3.57	3
	Average	**4.5**	**8.18**	**9.61**	**7.11**	**5.39**	**1.75**	**2.64**	**2.21**	**10.18**	**13.36**	**5.93**	**4.39**	**3.04**	**3.5**
PM Peak	1	5.33	10.83	8.33	4.17	12.67	4.5	3.17	1.5	10	11.17	2.83	3.83	2.33	3.83
	2	6.67	10.5	9.33	4	12.5	4.83	3.83	2.67	8	11.5	8.5	6	2.83	3.67
	3	4.67	9.67	8.83	4.17	7.83	3.5	2	0.67	5.67	7.83	5	4.17	3.17	2.67

Time Period		EB (Main)			NB				WB (Main)			SB			
		Left	Thru	Thru/Right	Left	Thru		Right	Left	Thru	Thru/Right	Left	Thru		Right
		1	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**	**13**	**14**
	4	3.67	6.83	7.17	5	4.5	1.17	1	1.67	7.33	8.5	2.5	3	2.33	2.5
	Average	**5.08**	**9.46**	**8.42**	**4.33**	**9.38**	**3.5**	**2.5**	**1.63**	**7.75**	**9.75**	**4.71**	**4.25**	**2.67**	**3.17**

Table E-3.2 Average Queue Length (#vehs/lane) at Jenk Ave & 23rd Street

Time Period	EB (Main)			NB			WB (Main)			SB		
	Left	Thru	Thru/Right	Left	Thru	Right	Left	Thru	Thru/Right	Left	Thru	Right
AM Peak	3.89	6.45	6.44	2.01	2.73	0.82	1.65	4.8	5.2	2.56	4.69	2.56
Off-Peak	4.5	8.18	9.61	7.11	3.57	2.64	2.21	10.18	13.36	5.93	3.71	3.5
PM Peak	5.08	9.46	8.42	4.33	6.44	2.5	1.63	7.75	9.75	4.71	3.46	3.17

Table E-3.3 Average Queue Length by Lane (#vehs/lane) at SR77 & 23rd Street.

Time Period		Eastbound				Northbound					Westbound			Southbound			
		Left	Thru		Right	Left		Thru		Right	Left	Thru	Thru/Right	Left	Thru		Right
		1	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**	**13**	**14**	**15**	**16**
	1	2.71	4.86	3	2.57	3.71	4.43	8.43	9.14	0.14	0.83	8.5	11	1.57	6.57	6	1.14

Time Period		Eastbound				Northbound					Westbound			Southbound			
		Left	Thru		Right	Left		Thru		Right	Left	Thru	Thru/Right	Left	Thru		Right
		1	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**	**13**	**14**	**15**	**16**
AM Peak	2	3.43	4	2.29	1.43	4.57	6.14	8.14	8.71	0.43	1	12.29	13.14	1.71	8.14	6.29	1.14
	3	5	4.43	2.14	3	3.57	4.57	6.86	8	0.14	1.43	7.43	7.14	1.14	4.43	4.43	1.57
	4	5.14	4.14	3.57	1.29	4.14	4.57	4.71	6	0	1.43	7.29	7	0.71	2.86	3.57	1
	Average	**4.07**	**4.36**	**2.75**	**2.07**	**4**	**4.93**	**7.04**	**7.96**	**0.18**	**1.17**	**8.88**	**9.57**	**1.29**	**5.5**	**5.07**	**1.21**
Off Peak	1	4.71	9.86	7.86	5.71	4.29	6.86	5.86	6.86	0.14	1.86	6.71	8.71	2.57	3.14	3.71	1.43
	2	6.71	9	9.43	7.14	4.71	7.86	9	9	0.14	2.14	5.29	7.43	3.57	3.86	5	2.29
	3	9.14	11.43	9.86	6	6.14	10.14	8	9	0.71	3.29	9.14	14.29	4.29	3.86	4.71	2.57
	4	7.57	9.43	8.14	5.71	5.71	8	6.57	5.71	0.14	1.71	6.86	7.86	4.43	4.57	5.86	2.43
	Average	**7.04**	**9.93**	**8.82**	**6.14**	**5.21**	**8.21**	**7.36**	**7.64**	**0.29**	**2.25**	**7**	**9.57**	**3.71**	**3.86**	**4.82**	**2.18**
PM Peak	1	7.29	9.14	6.29	2.29	5.14	9.57	12.14	10.14	0	1.14	7.86	9.29	4.14	5.29	5.43	2
	2	8.5	7.5	4	1.83	4.5	6.33	11.83	9.33	0.67	1.83	6.33	9.83	6.33	4.33	5.17	1.83
	3	6	8.86	5.86	3	3.14	5.57	16.43	14.43	0.29	2.5	7.83	9.67	5.33	3.17	4.33	1.5
	4	3.43	8.14	6.71	3.43	3.86	5.43	4.86	5	0.29	1.67	5.5	9.17	2.17	3.33	3.33	2
	Average	**6.3**	**8.41**	**5.71**	**2.64**	**4.16**	**6.73**	**11.32**	**9.73**	**0.31**	**1.79**	**6.88**	**9.49**	**4.49**	**4.03**	**4.57**	**1.83**

Table E-3.4 Average Queue Length (#vehs/lane) at SR77 & 23rd Street.

Time Period	EB (Main)			NB			WB (Main)			SB		
	Left	Thru	Right	Left	Thru	Right	Left	Thru	Thru/Right	Left	Thru	Right
AM Peak	4.07	3.55	2.07	4.46	7.5	0.18	1.17	8.88	9.57	1.29	5.29	1.21
Off Peak	7.04	9.38	6.14	6.71	7.5	0.29	2.25	7	9.57	3.71	4.34	2.18
PM Peak	6.3	7.06	2.64	5.44	10.52	0.31	1.79	6.88	9.49	4.49	4.3	1.83

After Study (Sep. 22 & Sep. 23, 2015)

Table E-3.5 Average Queue Length by Lane (#vehs/lane) at Jenks Ave & 23rd Street

Time Period		Eastbound			Northbound				Westbound			Southbound			
		Left	Thru	Thru+ Right	Left	Thru		Right	Left	Thru	Thru+ Right	Left	Thru		Right
		1	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**	**13**	**14**
AM Peak	1	0.71	2.57	1.43	1.71	2.86	1.29	0.43	1.86	9.29	10.14	2.33	7.00	5.83	2.83
	2	4.43	2.57	4.14	3.71	1.86	4.43	1.29	0.71	9.14	10.29	2.57	8.71	8.43	3.00
	3	1.71	2.14	2.43	1.29	4.86	0.29	1.29	1.71	7.29	9.00	2.57	4.43	4.00	1.57
	4	2.00	2.57	3.29	2.29	4.29	1.86	0.71	1.71	8.57	9.57	3.00	6.43	4.71	2.14
	Average	**2.21**	**2.46**	**2.82**	**2.25**	**3.46**	**1.96**	**0.93**	**1.50**	**8.57**	**9.75**	**2.62**	**6.64**	**5.74**	**2.39**

Time Period		Eastbound			Northbound				Westbound			Southbound			
		Left	Thru	Thru+ Right	Left	Thru		Right	Left	Thru	Thru+ Right	Left	Thru		Right
		1	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**	**13**	**14**
Off Peak	1	3.83	4.67	4.50	3.33	3.83	4.33	4.50	2.50	7.50	11.17	4.83	2.83	2.00	5.67
	2	4.00	9.67	7.83	2.50	3.00	4.83	4.67	1.83	5.83	8.33	3.67	4.00	2.83	5.50
	3	6.83	11.67	12.17	3.50	3.00	7.00	8.00	3.17	13.67	15.17	9.67	4.33	4.17	4.83
	4	6.67	11.83	12.17	4.17	2.67	7.00	4.83	5.33	16.00	16.00	5.17	4.83	4.00	4.17
	Average	**5.33**	**9.46**	**9.17**	**3.38**	**3.13**	**5.79**	**5.50**	**3.21**	**10.75**	**12.67**	**5.83**	**4.00**	**3.25**	**5.04**
PM Peak	1	4.33	8.33	7.67	3.83	9.67	5.33	4.17	2.00	6.83	8.83	5.83	15.50	2.50	4.50
	2	3.17	7.83	5.33	4.00	8.00	2.67	2.83	2.17	8.50	10.50	3.67	3.83	3.17	3.33
	3	5.83	13.67	11.33	4.33	8.33	6.33	2.83	2.00	7.50	11.50	5.67	4.50	2.83	3.50
	4	1.83	5.67	4.83	3.50	7.17	2.00	1.33	0.83	5.00	7.00	2.83	3.33	3.00	1.17
	Average	**3.79**	**8.88**	**7.29**	**3.92**	**8.29**	**4.08**	**2.79**	**1.75**	**6.96**	**9.46**	**4.50**	**6.79**	**2.88**	**3.13**

Table E-3.6 Average Queue Length (#vehs/lane) at Jenks Ave & 23rd Street

Time Period	EB(Main)			NB(Side)			WB(Main)			SB(Side)		
	Left	Thru	Thru+Right	Left	Thru	Right	Left	Thru	Thru+Right	Left	Thru	Right
AM Peak	2.21	2.46	2.82	2.25	2.71	0.93	1.50	8.57	9.75	2.62	6.19	2.39
Off Peak	5.33	9.46	9.17	3.38	4.46	5.50	3.21	10.75	12.67	5.83	3.63	5.04
PM Peak	3.79	8.88	7.29	3.92	6.19	2.79	1.75	6.96	9.46	4.50	4.83	3.13

Table E-3.7 Average Queue Length by Lane (#vehs/lane) at SR77 & 23rd Street

Time Period		**Eastbound**				**Northbound**					**Westbound**			**Southbound**			
		Left	**Thru**		**Right**	**Left**		**Thru**		**Right**	**Left**	**Thru**	**Thru+Right**	**Left**	**Thru**		**Right**
		1	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**	**13**	**14**	**15**	**16**
AM Peak (Incident)	**1**	5.83	5.67	4.17	1.67	2.33	3.33	6.67	7.33	0.33	0.86	9.29	8.57	2.57	10.29	9.14	4.00
	2	7.17	6.00	2.67	2.50	2.00	4.00	3.00	4.17	0.33	1.43	7.71	8.00	2.14	10.29	10.00	3.57
	3	5.14	5.71	3.57	1.71	4.00	3.14	6.14	6.43	0.14	1.00	7.43	6.14	1.86	6.14	6.29	2.14
	4	9.33	7.00	5.50	1.33	3.83	5.00	4.67	6.00	0.00	0.71	7.57	7.29	3.00	8.29	7.00	2.57
	Average	**6.87**	**6.10**	**3.98**	**1.80**	**3.04**	**3.87**	**5.12**	**5.98**	**0.20**	**1.00**	**8.00**	**7.50**	**2.39**	**8.75**	**8.11**	**3.07**
AM Peak (No Incident)	1	4.29	4.14	2.57	0.71	3.33	4.50	4.33	4.50	0.17	1.00	6.86	6.57	1.71	11.29	9.00	4.29
	2	5.71	6.43	2.29	1.00	5.29	6.71	4.43	5.71	0.00	1.14	12.71	13.14	1.86	10.43	9.71	4.43
	3	5.57	3.14	2.57	0.14	2.71	3.43	4.57	5.29	0.00	0.71	5.43	5.86	1.71	6.71	6.86	2.43
	4	5.00	3.71	2.43	0.43	2.14	3.57	4.29	5.57	0.00	1.00	5.71	7.00	1.71	6.71	6.71	2.29

Time Period		Eastbound				Northbound					Westbound			Southbound			
		Left	**Thru**		**Right**	**Left**		**Thru**		**Right**	**Left**	**Thru**	**Thru+Right**	**Left**	**Thru**		**Right**
		1	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**	**13**	**14**	**15**	**16**
	Average	**5.14**	**4.36**	**2.46**	**0.57**	**3.37**	**4.55**	**4.40**	**5.27**	**0.04**	**0.96**	**7.68**	**8.14**	**1.75**	**8.79**	**8.07**	**3.36**
Off Peak	1	11.43	6.86	4.57	1.71	6.86	6.57	7.71	6.14	1.43	1.43	10.71	11.00	3.57	7.43	8.29	2.86
	2	13.17	7.17	5.17	1.17	4.67	7.33	7.33	8.67	1.33	3.33	12.17	12.33	4.57	8.14	8.86	4.29
	3	12.50	7.83	6.50	0.83	4.33	6.50	9.83	10.67	0.67	2.33	12.00	15.00	8.00	11.50	8.50	2.17
	4	10.50	4.67	3.67	2.17	6.00	6.67	7.33	8.67	1.17	2.67	9.50	11.17	5.83	10.17	10.00	6.83
	Average	**11.90**	**6.63**	**4.98**	**1.47**	**5.46**	**6.77**	**8.05**	**8.54**	**1.15**	**2.44**	**11.10**	**12.38**	**5.49**	**9.31**	**8.91**	**4.04**
PM Peak	1	14.67	12.83	4.67	1.17	4.00	7.00	13.33	12.50	1.00	3.83	10.17	10.83	7.83	9.17	9.50	4.17
	2	12.00	7.83	3.50	0.67	4.80	7.80	16.60	16.40	1.00	2.50	7.00	13.33	5.33	7.50	7.33	5.50
	3	15.60	9.60	3.80	2.00	3.40	6.00	20.00*	20.00*	1.00	3.67	8.33	10.67	6.33	9.67	8.83	6.33
	4	9.83	6.17	3.50	1.33	4.40	5.60	5.80	6.40	1.60	2.50	8.50	11.33	5.33	6.83	6.33	3.67
	Average	**13.03**	**9.11**	**3.87**	**1.29**	**4.15**	**6.60**	**13.93**	**13.83**	**1.15**	**3.13**	**8.50**	**11.54**	**6.21**	**8.29**	**8.00**	**4.92**

Note: * indicates cycle failure.

Table E-3.8 Average Queue Length (#vehs/lane) at SR77 & 23rd Street

Time Period	EB (Main)			NB (Side)			WB (Main)			SB (Side)		
	Left	Thru	Right	Left	Thru	Right	Left	Thru	Thru+Right	Left	Thru	Right
AM Peak (Incident)	5.14	3.41	0.57	3.96	4.84	0.04	0.96	7.68	8.14	1.75	8.43	3.36
AM Peak (No Incident)	6.87	5.04	1.80	3.46	5.55	0.20	1.00	8.00	7.50	2.39	8.43	3.07
Off Peak	11.90	5.80	1.47	6.12	8.29	1.15	2.44	11.10	12.38	5.49	9.11	4.04
PM Peak	13.03	6.49	1.29	5.38	16.75	1.15	3.13	8.50	11.54	6.21	8.15	4.92

Comparisons of Before and After Queue Lengths for Critical Intersections

The differences in queue length between the before and after measurements are shown in Table E-3.9 to Table E-3.12. The tables are color-coded as follows: green shows significant improvement, yellow shows modest change, and red shows significant increase in queue length.

Table E-3.9 Difference in Average Queue Length by Lane (#vehs/lane) at Jenks Ave & 23rd Street

Time Period	**Lane**														**Average**
	1	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**	**13**	**14**	
AM Peak	-1.67	-3.99	-3.62	0.24	-0.74	0.72	0.11	-0.15	3.78	4.55	0.06	1.35	1.67	-0.18	0.15
Off Peak	0.83	1.28	-0.44	-3.73	-2.27	4.04	2.86	0.99	0.57	-0.69	-0.10	-0.39	0.21	1.54	0.34
PM Peak	-1.29	-0.58	-1.13	-0.42	-1.08	0.58	0.29	0.13	-0.79	-0.29	-0.21	2.54	0.21	-0.04	-0.15

Table E-3.10 Difference in Average Queue Length (#vehs/lane) at Jenks Ave & 23rd Street

Time Period	EB (Main)			NB (Side)			WB (Main)			SB (Side)		
	Left	Thru	Thru+Right	Left	Thru	Right	Left	Thru	Thru+Right	Left	Thru	Right
AM Peak	-1.67	-3.99	-3.62	0.24	-0.01	0.11	-0.15	3.78	4.55	0.06	1.51	-0.18
Off Peak	0.83	1.28	-0.44	-3.73	0.89	2.86	0.99	0.57	-0.69	-0.10	-0.09	1.54
PM Peak	-1.29	-0.58	-1.13	-0.42	-0.25	0.29	0.13	-0.79	-0.29	-0.21	1.38	-0.04

Table E-3.11 Difference in Average Queue Length by Lane (#vehs/lane) at SR77 & 23rd Street

Time Period	Lane																Average
	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	
AM Peak (Incident)	2.80	1.74	1.23	-0.27	-0.96	-1.06	-1.92	-1.98	0.02	-0.17	-0.88	-2.07	1.11	3.25	3.04	1.86	0.36
AM Peak (No Incident)	1.07	0.00	-0.29	-1.50	-0.63	-0.38	-2.63	-2.70	-0.14	-0.21	-1.20	-1.43	0.46	3.29	3.00	2.14	-0.07
Off Peak	4.86	-3.30	-3.85	-4.67	0.25	-1.45	0.70	0.89	0.86	0.19	4.10	2.80	1.78	5.45	4.09	1.86	0.91
PM Peak	6.72	0.70	-1.85	-1.35	-0.01	-0.13	2.62	4.10	0.84	1.34	1.62	2.05	1.71	4.26	3.43	3.08	1.82

Table E-3.12 Difference in Average Queue Length (#vehs/lane) at SR77 & 23rd Street

Time Period	EB (Main)			NB (Side)			WB (Main)			SB (Side)	
	Left	Thru	Right	Left	Thru	Right	Left	Thru	Thru +Right	Left	Thru	Right
AM Peak (Incident)	2.80	1.48	-0.27	-1.01	-1.95	0.02	-0.17	-0.88	-2.07	1.11	3.14	1.86
AM Peak (No Incident)	1.07	-0.14	-1.50	-0.50	-2.66	-0.14	-0.21	-1.20	-1.43	0.46	3.14	2.14
Off Peak	4.86	-3.57	-4.67	-0.60	0.79	0.86	0.19	4.10	2.80	1.78	4.77	1.86
PM Peak	6.72	-0.58	-1.35	-0.07	3.36	0.84	1.34	1.62	2.05	1.71	3.85	3.08

Discussion

For the Jenks Ave. & 23rd Street intersection, the average queue length decreased by an average of 0.15 vehicles in the PM Peak, but increased by 0.15 and 0.34 vehicles in the AM and Off Peak, respectively. The highest increase was observed in the WB through and right turn queue, which increased during the AM Peak by 4.55 vehs/lane (it is the predominant direction of travel at that time).

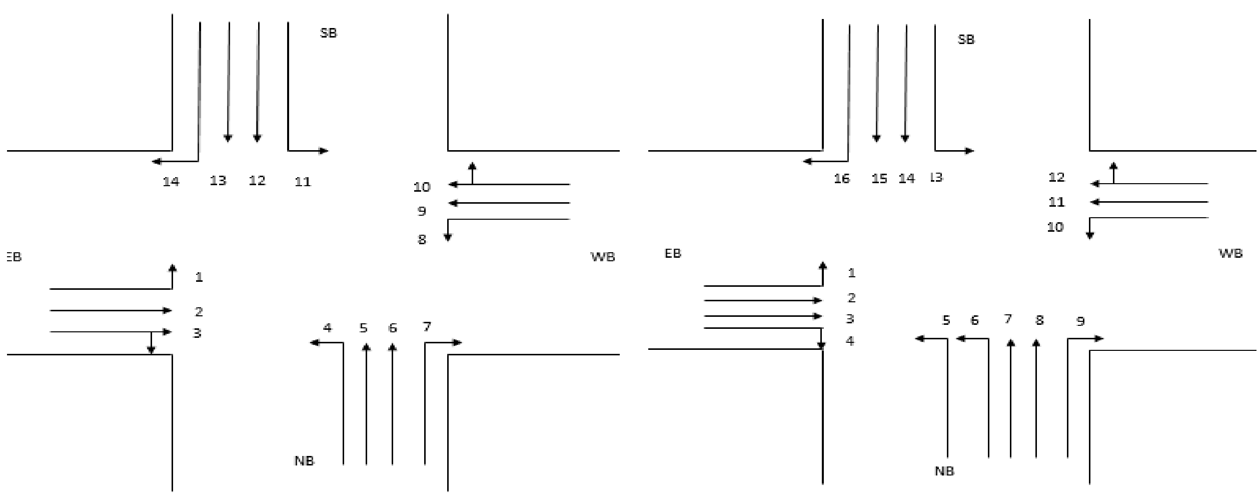
For the SR 77 & 23rd Street intersection, for AM peak no incident, the average queue length decreased by an average of 0.07 vehicles, for AM peak with incident, the average queue length increased by an average of 0.36 vehicles. From the queue length data, the influence of incident on queue length at SR77 & 23rd Street intersection is not significant. The Off peak and PM peak increased by 0.91 and 1.82 vehicles, respectively. The highest increase was observed in the EB left turn queue (main line movements), which increased during all time periods by 1.07 (2.80), 4.86 and 6.72 vehicles, respectively (it is the predominant direction of travel at that time). Also, for all turn movements of the SB (the side street movements), queue length increased.

E.3.4 QUEUE TO LANE STORAGE RATIO

In addition to queue length, it is important to assess any impact to adjacent lanes or to upstream facilities. The queue to link/lane ratio is used to establish the likelihood of spillback, which is presented in this section by movement and by time period. For the AM peak after study, the research team collected the SR 77 and 23rd St. intersection traffic counts on 9/22/2015 when the incident was active, as the "AM Peak Incident", and we use the queue length collected from video during a weekday (9/29/2015, Tuesday) for the SR 77 and 23rd St. intersection as the "AM Peak No Incident".

The following assumptions are used:

- The storage capacity is estimated as the maximum number of vehicles in the lane.
- The queue to link/lane storage ratio is estimated as 1 if the observer reported "spillback", and as 0.8 if reported as the maximum number of vehicles in the sight distance.
- Queue to lane storage ratios over 80% are highlighted in yellow, as they represent conditions with a high probability for spillback.



(a) Jenks Ave. & 23rd Street (b) SR 77 & 23rd Street

Figure E-4.1 Lane Configuration of Critical Intersections

Before Study (Jan. 27 & Jan.28, 2015)

Table E-4.1 Average Queue Storage Ratio by Lane and by Period at Jenks Ave. & 23rd Street

Time Period		EB (Main)			NB				WB (Main)			SB			
		Left	**Thru**	**Thru/Right**	**Left**	**Left**		**Thru**	**Thru/Right**	**Left**	**Left**	**Thru**	**Thru/Right**		**Left**
		1	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**	**13**	**14**
AM Peak	1	0.46	0.23	0.17	0.43	0.14	0.03	0.07	0.19	0.2	0.17	0.18	0.24	0.15	0.24
	2	0.72	0.3	0.27	0.17	0.12	0.04	0.07	0.29	0.26	0.29	0.37	0.25	0.19	0.21
	3	0.43	0.19	0.24	0.11	0.17	0.04	0.11	0.14	0.11	0.14	0.1	0.16	0.13	0.24
	4	0.33	0.2	0.24	0.18	0.13	0.05	0.08	0.2	0.17	0.2	0.27	0.12	0.11	0.24
	Average	**0.49**	**0.23**	**0.23**	**0.22**	**0.14**	**0.04**	**0.08**	**0.21**	**0.18**	**0.2**	**0.23**	**0.19**	**0.15**	**0.23**
Off peak	1	0.5	0.16	0.23	0.49	0.13	0.04	0.26	0.2	0.5	0.59	0.6	0.12	0.09	0.32
	2	0.54	0.32	0.36	0.87	0.17	0.07	0.3	0.32	0.35	0.42	0.44	0.14	0.12	0.23
	3	0.54	0.32	0.35	1.00	0.24	0.08	0.26	0.2	0.38	0.49	0.87	0.18	0.09	0.44
	4	0.68	0.37	0.43	0.41	0.18	0.04	0.24	0.39	0.34	0.55	0.25	0.18	0.13	0.27
	Average	**0.56**	**0.29**	**0.34**	**0.79**	**0.18**	**0.06**	**0.26**	**0.28**	**0.39**	**0.51**	**0.54**	**0.16**	**0.11**	**0.32**
PM Peak	1	0.67	0.39	0.3	0.46	0.42	0.15	0.32	0.19	0.38	0.43	0.26	0.14	0.08	0.35
	2	0.83	0.38	0.33	0.44	0.42	0.16	0.38	0.33	0.31	0.44	0.77	0.21	0.1	0.33
	3	0.58	0.35	0.32	0.46	0.26	0.12	0.2	0.08	0.22	0.3	0.45	0.15	0.11	0.24

Time Period		EB (Main)			NB				WB (Main)			SB			
		Left	Thru	Thru/Right	Left	Left		Thru	Thru/Right	Left	Left	Thru	Thru/Right		Left
		1	2	3	4	5	6	7	8	9	10	11	12	13	14
	4	0.46	0.24	0.26	0.56	0.15	0.04	0.1	0.21	0.28	0.33	0.23	0.11	0.08	0.23
	Average	**0.64**	**0.34**	**0.3**	**0.48**	**0.31**	**0.12**	**0.25**	**0.2**	**0.3**	**0.38**	**0.43**	**0.15**	**0.1**	**0.29**

Table E-4.2 Average Queue Storage Ratio by Lane and by Period at SR 77 & 23rd Street

Time Period		EB				NB					WB			SB			
		Left	Thru		Right	Left		Thru		Right	Left	Thru	Thru/Right	Left	Thru		Right
		1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
AM Peak	1	0.12	0.17	0.11	0.12	0.25	0.3	0.3	0.33	0.01	0.12	0.39	0.5	0.1	0.27	0.25	0.05
	2	0.16	0.14	0.08	0.06	0.3	0.41	0.29	0.31	0.03	0.14	0.56	0.6	0.11	0.34	0.26	0.05
	3	0.23	0.16	0.08	0.14	0.24	0.3	0.24	0.29	0.01	0.2	0.34	0.32	0.07	0.18	0.18	0.07
	4	0.23	0.15	0.13	0.06	0.28	0.3	0.17	0.21	0	0.2	0.33	0.32	0.04	0.12	0.15	0.04
	Average	**0.19**	**0.16**	**0.1**	**0.09**	**0.27**	**0.33**	**0.25**	**0.28**	**0.01**	**0.17**	**0.4**	**0.44**	**0.08**	**0.23**	**0.21**	**0.05**
Off Peak	1	0.21	0.35	0.28	0.26	0.29	0.46	0.21	0.24	0.01	0.27	0.31	0.4	0.16	0.13	0.15	0.06
	2	0.31	0.32	0.34	0.32	0.31	0.52	0.32	0.32	0.01	0.31	0.24	0.34	0.22	0.16	0.21	0.1

Time Period		EB				NB					WB			SB			
		Left	Thru		Right	Left		Thru		Right	Left	Thru	Thru/Right	Left	Thru		Right
		1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
	3	0.42	0.41	0.35	0.27	0.41	0.68	0.29	0.32	0.05	0.47	0.42	0.65	0.27	0.16	0.2	0.11
	4	0.34	0.34	0.29	0.26	0.38	0.53	0.23	0.2	0.01	0.24	0.31	0.36	0.28	0.19	0.24	0.1
	Average	**0.32**	**0.35**	**0.32**	**0.28**	**0.35**	**0.55**	**0.26**	**0.27**	**0.02**	**0.32**	**0.32**	**0.44**	**0.23**	**0.16**	**0.2**	**0.09**
PM Peak	1	0.33	0.33	0.22	0.1	0.34	0.64	0.43	0.36	0	0.16	0.36	0.42	0.26	0.22	0.23	0.08
	2	0.39	0.27	0.14	0.08	0.3	0.42	0.42	0.33	0.04	0.26	0.29	0.45	0.4	0.18	0.22	0.08
	3	0.27	0.32	0.21	0.14	0.21	0.37	0.59	0.52	0.02	0.36	0.36	0.44	0.33	0.13	0.18	0.06
	4	0.16	0.29	0.24	0.16	0.26	0.36	0.17	0.18	0.02	0.24	0.25	0.42	0.14	0.14	0.14	0.08
	Average	**0.29**	**0.3**	**0.2**	**0.12**	**0.28**	**0.45**	**0.4**	**0.35**	**0.02**	**0.26**	**0.31**	**0.43**	**0.28**	**0.17**	**0.19**	**0.08**

After Study (Sep. 22 & Sep. 23, 2015)

Table E-4.3 Average Queue Storage Ratio by Lane by Period at Jenks Ave. & 23rd Street

Time Period		Eastbound			Northbound				Westbound			Southbound			
		Left	**Thru**	**Thru+ Right**	**Left**	**Thru**		**Right**	**Left**	**Thru**	**Thru+ Right**	**Left**	**Thru**		**Right**
		1	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**	**13**	**14**
AM Peak	1	0.09	0.09	0.05	0.19	0.10	0.04	0.04	0.23	0.36	0.39	0.21	0.25	0.21	0.26
	2	0.55	0.09	0.15	0.41	0.06	0.15	0.13	0.09	0.35	0.40	0.23	0.31	0.30	0.27
	3	0.21	0.08	0.09	0.14	0.16	0.01	0.13	0.21	0.28	0.35	0.23	0.16	0.14	0.14
	4	0.25	0.09	0.12	0.25	0.14	0.06	0.07	0.21	0.33	0.37	0.27	0.23	0.17	0.19
	Average	0.28	0.09	0.10	0.25	0.12	0.07	0.09	0.19	0.33	0.38	0.24	0.24	0.21	0.22
Off Peak	1	0.48	0.17	0.16	0.37	0.13	0.14	0.45	0.31	0.29	0.43	0.44	0.10	0.07	0.52
	2	0.50	0.35	0.28	0.28	0.10	0.16	0.47	0.23	0.22	0.32	0.33	0.14	0.10	0.50
	3	0.85	0.42	0.43	0.39	0.10	0.23	0.80	0.40	0.53	0.58	0.88	0.15	0.15	0.44
	4	0.83	0.42	0.43	0.46	0.09	0.23	0.48	0.67	0.62	0.62	0.47	0.17	0.14	0.38
	Average	0.67	0.34	0.33	0.38	0.10	0.19	0.55	0.40	0.41	0.49	0.53	0.14	0.12	0.46
PM Peak	1	0.54	0.30	0.27	0.43	0.32	0.18	0.42	0.25	0.26	0.34	0.53	0.55	0.09	0.41
	2	0.40	0.28	0.19	0.44	0.27	0.09	0.28	0.27	0.33	0.40	0.33	0.14	0.11	0.30

Time Period		Eastbound			Northbound				Westbound			Southbound			
		Left	Thru	Thru+ Right	Left	Thru		Right	Left	Thru	Thru+ Right	Left	Thru		Right
		1	2	3	4	5	6	7	8	9	10	11	12	13	14
	3	0.73	0.49	0.40	0.48	0.28	0.21	0.28	0.25	0.29	0.44	0.52	0.16	0.10	0.32
	4	0.23	0.20	0.17	0.39	0.24	0.07	0.13	0.10	0.19	0.27	0.26	0.12	0.11	0.11
	Average	0.47	0.32	0.26	0.44	0.28	0.14	0.28	0.22	0.27	0.36	0.41	0.24	0.10	0.28

Table E-4.4 Average Queue Storage Ratio by Lane by Period at SR 77 & 23rd Street

Time Period		Eastbound				Northbound					Westbound			Southbound			
		Left	Thru		Right	Left		Thru		Right	Left	Thru	Thru+ Right	Left	Thru		Right
		1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
AM Peak (Incident)	1	0.27	0.20	0.15	0.08	0.16	0.22	0.24	0.26	0.02	0.12	0.42	0.39	0.16	0.43	0.38	0.17
	2	0.33	0.21	0.10	0.11	0.13	0.27	0.11	0.15	0.02	0.20	0.35	0.36	0.13	0.43	0.42	0.15
	3	0.23	0.20	0.13	0.08	0.27	0.21	0.22	0.23	0.01	0.14	0.34	0.28	0.12	0.26	0.26	0.09
	4	0.42	0.25	0.20	0.06	0.26	0.33	0.17	0.21	0.00	0.10	0.34	0.33	0.19	0.35	0.29	0.11
	Average	0.31	0.22	0.14	0.08	0.20	0.26	0.18	0.21	0.01	0.14	0.36	0.34	0.15	0.36	0.34	0.13

Time Period		Eastbound				Northbound					Westbound			Southbound			
		Left	Thru		Right	Left		Thru		Right	Left	Thru	Thru+ Right	Left	Thru		Right
		1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
AM Peak (No Incident)	1	0.19	0.15	0.09	0.03	0.22	0.30	0.15	0.16	0.01	0.14	0.31	0.30	0.11	0.47	0.38	0.18
	2	0.26	0.23	0.08	0.05	0.35	0.45	0.16	0.20	0.00	0.16	0.58	0.60	0.12	0.43	0.40	0.18
	3	0.25	0.11	0.09	0.01	0.18	0.23	0.16	0.19	0.00	0.10	0.25	0.27	0.11	0.28	0.29	0.10
	4	0.23	0.13	0.09	0.02	0.14	0.24	0.15	0.20	0.00	0.14	0.26	0.32	0.11	0.28	0.28	0.10
	Average	0.23	0.16	0.09	0.03	0.22	0.30	0.16	0.19	0.00	0.14	0.35	0.37	0.11	0.37	0.34	0.14
Off Peak	1	0.52	0.24	0.16	0.08	0.46	0.44	0.28	0.22	0.10	0.20	0.49	0.50	0.22	0.31	0.35	0.12
	2	0.60	0.26	0.18	0.05	0.31	0.49	0.26	0.31	0.09	0.48	0.55	0.56	0.29	0.34	0.37	0.18
	3	0.57	0.28	0.23	0.04	0.29	0.43	0.35	0.38	0.04	0.33	0.55	0.68	0.50	0.48	0.35	0.09
	4	0.48	0.17	0.13	0.10	0.40	0.44	0.26	0.31	0.08	0.38	0.43	0.51	0.36	0.42	0.42	0.28
	Average	0.54	0.24	0.18	0.07	0.36	0.45	0.29	0.30	0.08	0.35	0.50	0.56	0.34	0.39	0.37	0.17
PM Peak	1	0.67	0.46	0.17	0.05	0.27	0.47	0.48	0.45	0.07	0.55	0.46	0.49	0.49	0.38	0.40	0.17
	2	0.55	0.28	0.13	0.03	0.32	0.52	0.59	0.59	0.07	0.36	0.32	0.61	0.33	0.31	0.31	0.23
	3	0.71	0.34	0.14	0.09	0.23	0.40	0.71	0.71	0.07	0.52	0.38	0.48	0.40	0.40	0.37	0.26

Time Period		Eastbound				Northbound					Westbound			Southbound			
		Left	Thru		Right	Left		Thru		Right	Left	Thru	Thru+ Right	Left	Thru		Right
		1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
	4	0.45	0.22	0.13	0.06	0.29	0.37	0.21	0.23	0.11	0.36	0.39	0.52	0.33	0.28	0.26	0.15
	Average	0.59	0.33	0.14	0.06	0.28	0.44	0.50	0.49	0.08	0.45	0.39	0.52	0.39	0.35	0.33	0.20

Comparisons of Before and After Queue Storage Ratios

The differences in Queue Storage Ratio between before and after measurements are shown in Table E-4.5 to Table E-4.8. The tables are color-coded as follows: green shows significant improvement, yellow shows modest change, and red shows significant deterioration in queue storage ratios or spillback potential.

Table E-4.5 Difference in Average Queue Storage Ratios at Jenks Ave. & 23rd Street

Time Period	Lane														Average
	Eastbound			Northbound				Westbound			Southbound				
	1	2	3	4	5	6	7	8	9	10	11	12	13	14	
AM Peak	-0.21	-0.14	-0.13	0.03	-0.02	0.02	0.01	-0.02	0.15	0.17	0.01	0.05	0.06	-0.02	0.00
Off Peak	0.10	0.05	-0.02	-0.41	-0.08	0.13	0.29	0.12	0.02	-0.03	-0.01	-0.01	0.01	0.14	0.02
PM Peak	-0.16	-0.02	-0.04	-0.05	-0.04	0.02	0.03	0.02	-0.03	-0.01	-0.02	0.09	0.01	0.00	-0.01

Table E-4.6 Difference in Percent of Average Queue Storage Ratios at Jenks Ave. & 23rd Street

Time Period	Lane														Average
	Eastbound			Northbound				Westbound			Southbound				
	1	2	3	4	5	6	7	8	9	10	11	12	13	14	
AM Peak	-43.02%	-61.82%	-56.20%	11.97%	-17.65%	57.89%	13.45%	-8.86%	78.75%	87.32%	2.35%	25.45%	40.91%	-6.85%	8.84%
Off Peak	18.52%	15.65%	-4.58%	-52.51%	-42.05%	230.95%	108.11%	44.89%	5.61%	-5.17%	-1.61%	-8.94%	7.06%	44.05%	25.71%
PM Peak	-25.41%	-6.17%	-13.37%	-9.62%	-11.56%	16.67%	11.67%	7.69%	-10.22%	-2.99%	-4.42%	59.80%	7.81%	-1.32%	1.33%

Table E-4.7 Difference in Average Queue Storage Ratio at SR 77 & 23rd Street

Time Period	Lane																Average
	Eastbound				Northbound					Westbound			Southbound				
	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	
AM Peak (Incident)	0.13	0.06	0.04	-0.01	-0.06	-0.07	-0.07	-0.07	0.00	-0.02	-0.04	-0.09	0.07	0.14	0.13	0.08	0.01
AM Peak (No Incident)	0.05	0.00	-0.01	-0.07	-0.04	-0.03	-0.09	-0.10	-0.01	-0.03	-0.05	-0.06	0.03	0.14	0.13	0.09	0.00

Time Period	Lane																Average
	Eastbound				Northbound					Westbound			Southbound				
	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	
Off Peak	0.22	-0.12	-0.14	-0.21	0.02	-0.10	0.02	0.03	0.06	0.03	0.19	0.13	0.11	0.23	0.17	0.08	0.04
PM Peak	0.31	0.02	-0.07	-0.06	0.00	-0.01	0.09	0.15	0.06	0.19	0.07	0.09	0.11	0.18	0.14	0.13	0.09

Table E-4.8 Difference in Percent of Average Queue Storage Ratio at SR 77 & 23rd Street

Time Period	Lane																Ave
	Eastbound					Northbound				Westbound			Southbound				
	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	
AM Peak (Incident)	68.71%	39.89%	44.59%	-12.93%	-23.96%	-21.50%	-27.24%	-24.89%	13.33%	-14.72%	-9.86%	-21.64%	86.11%	59.09%	59.86%	152.94%	22.99%
AM Peak (No Incident)	26.32%	0.00%	-10.39%	-72.41%	-15.77%	-7.61%	-37.39%	-33.86%	-76.67%	-17.77%	-13.48%	-14.93%	36.11%	59.74%	59.15%	176.47%	3.59%
Off Peak	69.12%	-33.21%	-43.59%	-76.07%	4.79%	-17.61%	9.47%	11.68%	302.08%	8.47%	58.50%	29.29%	47.92%	141.36%	84.81%	85.25%	42.64%
PM Peak	106.63%	8.29%	-32.33%	-51.02%	-0.26%	-1.88%	23.14%	42.14%	271.54%	75.00%	23.53%	21.64%	38.15%	105.76%	75.23%	168.18%	54.61%

Discussion

- For the Jenks Ave. intersection, during the before study there were three queue storage ratios that were high. Two of them (NB left during the Off peak and EB left during the PM peak) were significantly reduced in the after data. However, the storage ratios for the EB left movement increased during the Off peak period. Generally, queue storage ratios increased for the AM Peak and Off Peak, and they decreased for the PM Peak.
- For the SR 77 intersection there are no critical queue storage ratios for either of the two analysis periods. However, queue storage ratios increased for all time periods.
- The highest increase of queue storage ratio for Jenks Ave. & 23rd Street intersection was observed for the NB Off Peak (0.29), and the highest increase of queue storage ratio for SR 77 & 23rd Street intersection was observed for the EB PM Peak (0.31); in terms of percentages, the highest increase was observed for the NB Off Peak (302.08% for Jenks Ave. & 23rd St., 230.95% for SR 77 & 23rd St.).
- For both intersections, the highest increases of queue storage ratios were for the NB and SB directions (side streets movements), while the EB queue storage ratios generally decreased (main line movements). The new system seems to favor the main street traffic.

E.3.5 EQUIVALENT PCE FLOWS

Traffic flows were counted manually and converted to equivalent PCE flows (pce/hour) by considering the percentage of heavy vehicles. It was assumed that the PCE for trucks is 2. For the AM peak after study, the research team collected the SR 77 and 23rd St. intersection traffic counts on 9/22/2015 when the incident was active, as the "AM Peak Incident", and we use the traffic flow collected from video during a weekday (9/29/2015, Tuesday) for the SR 77 and 23rd St. intersection as the "AM Peak No Incident".

Truck Percentage Observations

Table E-5.1 Truck Percentages at Jenks Ave. & 23rd Street.

Period	Interval	EB			WB			NB			SB			Total
		Left	Thru	Right	Left	Thru	Right	Left	Thru	Right	Left	Thru	Right	
AM Peak	1	0.00%	1.84%	0.00%	0.00%	2.58%	2.94%	5.00%	3.70%	8.70%	0.00%	0.65%	0.00%	1.94%
	2	0.00%	2.16%	0.00%	1.85%	2.23%	0.00%	0.00%	0.00%	0.00%	2.70%	0.00%	1.15%	1.35%
	3	0.00%	2.05%	3.03%	0.00%	3.46%	1.47%	0.00%	0.94%	0.00%	2.38%	1.23%	0.00%	1.87%
	4	0.00%	1.02%	2.44%	0.00%	3.49%	10.53%	0.00%	2.20%	2.17%	2.17%	1.75%	1.14%	2.29%
	Average	**0.00%**	**0.56%**	**0.73%**	**0.62%**	**1.94%**	**1.44%**	**0.85%**	**0.80%**	**0.68%**	**1.94%**	**0.31%**	**0.00%**	**1.03%**
Off Peak	1	3.45%	1.00%	0.00%	3.57%	1.46%	13.33%	0.00%	0.00%	0.00%	0.00%	0.93%	1.32%	1.40%
	2	3.77%	1.77%	0.00%	2.56%	1.07%	5.13%	2.27%	1.63%	0.00%	0.00%	0.00%	2.75%	1.48%
	3	0.00%	0.94%	0.00%	2.94%	0.55%	3.85%	0.00%	4.00%	0.00%	0.00%	0.73%	0.00%	0.86%
	4	0.82%	1.74%	1.47%	0.00%	0.59%	2.50%	0.00%	2.44%	0.00%	4.23%	0.70%	0.00%	1.20%
	Average	**1.13%**	**0.96%**	**0.53%**	**2.38%**	**1.26%**	**6.43%**	**1.05%**	**1.94%**	**0.00%**	**0.97%**	**0.61%**	**0.60%**	**1.18%**

Period	Interval	EB			WB			NB			SB			Total
		Left	**Thru**	**Right**	**Left**	**Thru**	**Right**	**Left**	**Thru**	**Right**	**Left**	**Thru**	**Right**	
PM Peak	1	0.00%	0.74%	0.00%	0.00%	1.41%	4.76%	1.72%	2.03%	0.00%	1.41%	1.36%	1.18%	1.18%
	2	1.32%	0.71%	2.27%	0.00%	1.00%	0.00%	1.67%	0.53%	0.00%	0.00%	0.00%	1.08%	0.75%
	3	2.33%	0.87%	0.00%	4.35%	0.60%	2.33%	0.00%	1.00%	0.00%	0.00%	0.00%	0.00%	0.73%
	4	0.00%	0.81%	0.00%	0.00%	1.31%	4.76%	1.96%	0.00%	2.33%	3.39%	0.00%	0.00%	0.96%
	Average	**0.98%**	**0.78%**	**0.65%**	**0.93%**	**1.07%**	**2.96%**	**1.32%**	**0.88%**	**0.41%**	**1.06%**	**0.38%**	**0.59%**	**0.90%**

Table E–5.2 Truck Percentages at SR 77 & 23rd Street

Period	Interval	EB			WB			NB			SB			Ave
		Left	**Thru**	**Right**	**Left**	**Thru**	**Right**	**Left**	**Thru**	**Right**	**Left**	**Thru**	**Right**	
AM Peak	1	1.27%	4.57%	0.00%	4.76%	1.68%	0.00%	0.88%	1.89%	0.00%	2.04%	1.86%	1.95%	1.93%
	2	2.86%	3.38%	2.78%	1.89%	1.83%	0.91%	3.43%	3.90%	0.00%	2.44%	0.82%	1.37%	2.18%
	3	1.50%	3.73%	3.36%	0.00%	2.22%	1.47%	0.00%	4.13%	3.70%	2.13%	3.59%	2.89%	2.75%
	4	0.71%	7.88%	0.00%	0.00%	2.05%	8.00%	1.70%	3.70%	0.00%	8.47%	3.61%	0.00%	3.23%
	Average	**1.53%**	**4.78%**	**1.65%**	**1.89%**	**1.94%**	**2.52%**	**1.68%**	**3.42%**	**1.06%**	**3.80%**	**2.32%**	**1.56%**	**2.51%**
Off Peak	1	0.79%	1.18%	1.72%	0.00%	2.02%	3.03%	0.68%	2.64%	3.45%	0.00%	2.55%	1.82%	1.67%
	2	0.00%	1.85%	2.52%	2.78%	0.69%	3.70%	3.23%	1.91%	0.00%	0.00%	2.91%	2.38%	1.83%

Period	Interval	EB			WB			NB			SB			Ave
		Left	Thru	Right	Left	Thru	Right	Left	Thru	Right	Left	Thru	Right	
	3	0.59%	2.63%	0.71%	0.00%	2.18%	2.38%	1.22%	3.51%	0.00%	1.19%	3.31%	1.23%	2.07%
	4	0.68%	1.81%	0.00%	0.00%	2.53%	5.10%	1.49%	1.92%	0.00%	3.85%	3.63%	2.78%	2.18%
	Average	**0.16%**	**0.89%**	**0.35%**	**0.00%**	**0.90%**	**1.83%**	**0.62%**	**1.11%**	**0.00%**	**0.29%**	**1.51%**	**1.22%**	**0.86%**
PM Peak	1	1.66%	0.98%	1.53%	5.56%	1.29%	4.44%	1.42%	2.72%	0.00%	1.37%	1.94%	2.06%	1.81%
	2	0.53%	0.64%	0.70%	0.00%	1.01%	2.17%	0.68%	1.82%	0.00%	3.70%	1.48%	2.68%	1.20%
	3	0.47%	0.52%	0.66%	3.70%	0.75%	0.00%	0.61%	0.70%	0.00%	1.92%	2.61%	0.82%	0.95%
	4	1.51%	0.00%	1.35%	0.00%	0.38%	5.00%	0.00%	0.26%	3.23%	0.00%	1.09%	0.00%	0.60%
	Average	**0.90%**	**0.54%**	**1.05%**	**2.17%**	**0.75%**	**2.82%**	**0.69%**	**1.26%**	**0.00%**	**1.79%**	**1.74%**	**1.39%**	**1.07%**

Before Study (Jan. 27 & Jan.28, 2015)

Table E-5.3 Traffic Volume (pce/15 min) and Traffic Flow (pce/hour) at Jenks Ave. & 23rd Street

Period	Interval	EB			WB			NB			SB		
		Left	Thru	Right	Left	Thru	Right	Left	Thru	Right	Left	Thru	Right
AM Peak	1	16	104	7	18	212	22	13	18	15	15	87	34
	2	29	151	26	22	270	42	14	67	15	19	96	57
	3	34	143	17	22	187	34	22	53	24	16	95	47
	4	16	155	28	20	227	22	23	41	16	21	64	55

Period	Interval	EB			WB			NB			SB		
		Left	**Thru**	**Right**	**Left**	**Thru**	**Right**	**Left**	**Thru**	**Right**	**Left**	**Thru**	**Right**
	Flow Rate	**95**	**553**	**78**	**82**	**896**	**120**	**72**	**179**	**70**	**71**	**342**	**193**
Off Peak	1	30	217	26	16	251	29	42	42	34	46	53	40
	2	28	234	21	20	244	16	48	63	31	42	52	51
	3	22	271	24	16	274	12	47	66	33	34	63	59
	4	49	295	42	19	251	20	40	68	27	38	67	37
	Flow Rate	**129**	**1017**	**113**	**71**	**1020**	**77**	**177**	**239**	**125**	**160**	**235**	**187**
PM Peak	1	35	269	18	15	219	18	30	70	26	33	84	51
	2	48	276	25	16	247	15	35	91	35	32	71	52
	3	55	298	18	10	232	25	28	110	29	38	68	48
	4	30	245	18	9	218	9	26	76	16	31	42	41
	Flow Rate	**168**	**1088**	**79**	**50**	**916**	**67**	**119**	**347**	**106**	**134**	**265**	**192**

Table E-5.4 Traffic Volume (pce/15 min) and Traffic Flow (pce/hour) at SR 77 & 23rd Street

Period	Interval	EB			WB			NB			SB		
		Left	Thru	Right	Left	Thru	Right	Left	Thru	Right	Left	Thru	Right
AM Peak	1	29	65	31	11	154	9	42	102	3	12	167	64
	2	26	73	35	18	223	7	90	120	1	27	206	74
	3	42	71	25	6	174	9	38	104	5	19	167	66
	4	39	68	20	5	145	8	60	89	7	28	158	63
	Flow Rate	**136**	**277**	**111**	**40**	**696**	**33**	**230**	**415**	**16**	**86**	**698**	**267**
Off Peak	1	57	125	67	15	117	13	65	108	12	57	92	94
	2	53	128	88	16	147	20	76	136	12	39	111	98
	3	74	143	78	15	138	19	78	183	13	42	129	75
	4	68	169	68	30	152	19	93	112	13	54	173	95
	Flow Rate	**252**	**565**	**301**	**76**	**554**	**71**	**312**	**539**	**50**	**192**	**505**	**362**
PM Peak	1	77	188	55	20	112	24	72	200	12	37	96	40
	2	89	232	80	18	143	19	72	172	14	42	104	50
	3	108	191	67	12	123	22	77	240	16	46	129	58
	4	60	170	56	14	107	19	73	161	9	30	96	48
	Flow Rate	**334**	**781**	**258**	**64**	**485**	**84**	**294**	**773**	**51**	**155**	**425**	**196**

After Study (Sep. 22 & Sep. 23, 2015)

Table E-5.5 Traffic Volume (pce/15 min) and Hourly Volume (pce/hour) at Jenks& 23rd Street

Period	Interval	Eastbound			Westbound			Northbound			Southbound		
		Left	Thru	Right	Left	Thru	Right	Left	Thru	Right	Left	Thru	Right
AM Peak	1	22	117	11	18	185	13	8	38	10	15	69	35
	2	21	133	19	33	281	26	13	56	17	19	121	31
	3	23	155	17	17	202	35	15	54	21	27	70	31
	4	26	142	14	13	188	20	10	52	31	26	52	34
	Flow Rate	**92**	**547**	**61**	**81**	**856**	**94**	**46**	**200**	**79**	**87**	**312**	**131**
Off Peak	1	30	290	23	13	305	22	31	42	26	37	56	37
	2	27	284	29	20	322	25	42	62	43	41	46	61
	3	44	268	26	19	277	15	43	64	38	42	75	38
	4	74	290	27	27	259	21	45	58	37	36	77	39
	Flow Rate	**175**	**1132**	**105**	**79**	**1163**	**83**	**161**	**226**	**144**	**156**	**254**	**175**
PM Peak	1	40	272	20	15	286	26	29	81	24	39	65	35
	2	29	288	20	13	260	14	26	99	40	39	68	42
	3	33	282	21	14	273	19	30	92	44	45	74	48
	4	38	255	16	16	245	13	26	67	28	30	51	26

Period	Interval	Eastbound			Westbound			Northbound			Southbound		
		Left	Thru	Right	Left	Thru	Right	Left	Thru	Right	Left	Thru	Right
	Flow Rate	**140**	**1097**	**77**	**58**	**1064**	**72**	**111**	**339**	**136**	**153**	**258**	**151**

Table E-5.6 Traffic Volume (pce/15 min) and Hourly Volume (pce/hour) at SR 77 & 23rd Street

Period	Interval	Eastbound			Westbound			Northbound			Southbound			Total
		Left	Thru	Right	Left	Thru	Right	Left	Thru	Right	Left	Thru	Right	
AM Peak (Incident)	1	14	76	30	11	148	10	36	114	4	21	200	41	705
	2	42	89	35	18	167	7	75	145	11	26	241	73	929
	3	32	90	46	14	133	8	96	103	8	12	179	46	767
	4	57	76	33	15	118	13	68	115	8	13	138	55	709
	Flow Rate	**145**	**331**	**144**	**58**	**566**	**38**	**275**	**477**	**31**	**72**	**758**	**215**	**3110**
AM Peak (No Incident)	1	37	88	42	22	122	11	36	108	8	17	127	52	670
	2	40	83	41	18	202	7	76	135	15	31	169	75	892
	3	61	89	52	8	218	11	57	121	15	17	144	66	858
	4	46	75	40	16	155	17	51	104	10	23	134	39	710
	Flow Rate	**184**	**335**	**175**	**64**	**697**	**46**	**220**	**468**	**48**	**88**	**574**	**232**	**3130**
Off Peak	1	71	133	51	10	136	21	82	125	18	48	109	74	878

Period	Interval	Eastbound			Westbound			Northbound			Southbound			Total
		Left	**Thru**	**Right**	**Left**	**Thru**	**Right**	**Left**	**Thru**	**Right**	**Left**	**Thru**	**Right**	
	2	98	147	75	21	145	36	84	131	23	33	101	74	968
	3	97	208	64	18	143	24	88	112	24	43	121	89	1031
	4	103	206	83	29	158	16	85	108	12	31	123	72	1026
	Flow Rate	**369**	**694**	**273**	**78**	**582**	**97**	**339**	**476**	**77**	**155**	**454**	**309**	**3903**
PM Peak	1	107	223	78	18	123	23	71	177	11	37	114	59	1041
	2	100	242	63	23	158	28	75	163	10	42	102	65	1071
	3	105	198	85	16	147	24	89	194	19	60	146	65	1148
	4	142	230	94	20	160	23	58	226	23	48	90	53	1167
	Flow Rate	**454**	**893**	**320**	**77**	**588**	**98**	**293**	**760**	**63**	**187**	**452**	**242**	**4427**

Comparisons of Before and After Flows

The differences in traffic flow between the before and after measurements are shown in Table E-5.7 and Table E-5.8. The tables are color-coded as follows: green shows significant decrease, yellow shows modest change, and red shows significant increase in flow.

Table E-5.7 Difference in Traffic Flow Rate (pce/hr) at Jenks Ave. & 23rd Street

Period	Eastbound			Westbound			Northbound			Southbound		
	Left	Thru	Thru+Right	Left	Thru	Thru+Right	Left	Thru	Right	Left	Thru	Right
AM Peak	-3	-6	-17	-1	-40	-26	-26	21	9	16	-30	-62
Off Peak	46	115	-8	8	143	6	-16	-13	19	-4	19	-12
PM Peak	-28	9	-2	8	148	5	-8	-8	30	19	-7	-41

Table E-5.8 Difference in Traffic Flow Rate (pce/hr) at SR 77 & 23rd Street

Period	Eastbound			Westbound			Northbound			Southbound		
	Left	Thru	Right	Left	Thru	Thru+Right	Left	Thru	Right	Left	Thru	Right
AM Peak (Incident)	9	54	33	18	-130	5	45	62	15	-14	60	-52
AM Peak (No incident)	48	58	64	24	1	13	-10	53	32	2	-124	-35
Off Peak	117	129	-28	2	28	26	27	-63	27	-37	-51	-53
PM Peak	120	112	62	13	103	14	-1	-13	12	32	27	46

Discussion

- The traffic volume for the SR 77 & 23rd Street intersection increased during all three time periods by an average of 6.98%
- For the Jenks Ave. & 23rd Street intersection volume increased for the Off Peak and PM peak, while it decreased for the AM peak by 6.00%.

The increased queues may be at least partially due to the increase in flows between the before and after data collection periods.

E.3.6 CONSIDERATION OF TRAFFIC FLOWS JOINTLY WITH QUEUE LENGTH

The differences in "Traffic Flow" and "Queue Length" between before and after measurements are shown in Table E-6.1 and Table E-6.3. The tables are color-coded as follows: green indicates that "Queue" decreases and "Traffic Flow" increases, Red indicates "Queue" increases and "Traffic Flow" decreases, No Color indicates "Traffic Flow" and "Queue" increase or decrease at the same time. Generally, green indicates improvement despite an increase in flow. For the AM peak after study, the research team collected the SR 77 and 23rd St. intersection traffic counts on 9/22/2015 when the incident was active, as the "AM Peak Incident", and we use the flow and queue length collected from video during a weekday (9/29/2015, Tuesday) for the SR 77 and 23rd St. intersection as the "AM Peak No Incident".

Table E-6.1 Differences in Traffic Flow (TF) and Queue Length (Q) at Jenks Ave. & 23rd Street

Period	EB(Main)						NB(Side)						WB(Main)						SB(Side)					
	Left		Thru		Thru+Right		Left		Thru		Right		Left		Thru		Thru+Right		Left		Thru		Right	
	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q
AM Peak	-3	-1.67	-6	-3.99	-17	-3.62	-26	0.24	21.00	-2.74	9	0.11	-1	-0.15	-40	3.78	-26	4.55	16	0.06	-30	1.51	-62	-0.18
Off Peak	46	0.83	115	1.28	-8	-0.44	-16	-3.73	-13.00	-2.68	19	2.86	8	0.99	143	0.57	6	-0.69	-4	-0.10	19	-0.09	-12	1.54
PM Peak	-28	-1.29	9	-0.58	-2	-1.13	-8	-0.42	-8.00	-6.69	30	0.29	8	0.13	148	-0.79	5	-0.29	19	-0.21	-7	1.38	-41	-0.04

Table E-6.2 Differences (%) in Traffic Flow (TF) and Queue Length (Q) at Jenks Ave. & 23rd Street

Period	EB(Main)						NB(Side)						WB(Main)						SB(Side)					
	Left		Thru		Thru+Right		Left		Thru		Right		Left		Thru		Thru+Right		Left		Thru		Right	
	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q
AM Peak	-3%	-43%	-1%	-62%	-22%	-56%	-36%	12%	12%	-50%	13%	13%	-1%	-9%	-4%	79%	-22%	87%	23%	2%	-9%	32%	-32%	-7%
Off Peak	36%	19%	11%	16%	-7%	-5%	-9%	-53%	-5%	-38%	15%	108%	11%	45%	14%	6%	8%	-5%	-3%	-2%	8%	-2%	-6%	44%
PM Peak	-17%	-25%	1%	-6%	-3%	-13%	-7%	-10%	-2%	-52%	28%	12%	16%	8%	16%	-10%	7%	-3%	14%	-4%	-3%	40%	-21%	-1%

Table E-6.3 Differences in Traffic Flow and Queue Length at SR 77 & 23rd Street

Period	EB(Main)						NB(Side)						WB(Main)						SB(Side)					
	Left		Thru		Right		Left		Thru		Right		Left		Thru		Thru+Right		Left		Thru		Right	
	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q
AM Peak (Incident)	9	2.80	54	1.48	33	-0.27	18	-1.01	-130	-1.95	5	0.02	45	-0.17	62	-0.88	15	-2.07	-14	1.11	60	3.14	-52	1.86
AM Peak (No Incident)	48	1.07	58	-0.14	64	-1.50	-10	-0.50	53.00	-2.66	32	-0.14	24	-0.21	1	-1.20	13	-1.43	2	0.46	-124	3.14	-35	2.14

Period	EB(Main)						NB(Side)						WB(Main)						SB(Side)					
	Left		Thru		Right		Left		Thru		Right		Left		Thru		Thru+Right		Left		Thru		Right	
	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q
Off Peak	117	4.86	129	-3.57	-28	-4.67	27	-0.60	-63.00	0.79	27	0.86	2	0.19	28	4.10	26	2.80	-37	1.78	-51	4.77	-53	1.86
PM Peak	120	6.72	112	-0.58	62	-1.35	-1	-0.07	-13.00	3.36	12	0.84	13	1.34	103	1.62	14	2.05	32	1.71	27	3.85	46	3.08

Table E-6.4 Differences (%) in Traffic Flow and Queue Length at SR 77 & Street

Period	EB(Main)						NB(Side)						WB(Main)						SB(Side)					
	Left		Thru		Right		Left		Thru		Right		Left		Thru		Thru+Right		Left		Thru		Right	
	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q
AM Peak (Incident)	7%	69%	19%	42%	30%	-13%	45%	-23%	-19%	-26%	15%	13%	20%	-15%	94%	-10%	-16%	-22%	-16%	86%	9%	59%	-19%	153%
AM Peak (No Incident)	35%	26%	21%	-4%	58%	-72%	-4%	-11%	13%	-36%	200%	-77%	60%	-18%	0%	-13%	39%	-15%	2%	36%	-18%	59%	-13%	176%
Off Peak	46%	69%	23%	-38%	-9%	-76%	9%	-9%	-12%	11%	54%	302%	3%	8%	5%	59%	37%	29%	-19%	48%	-10%	110%	-15%	85%
PM Peak	36%	107%	14%	-8%	24%	-51%	0%	-1%	-2%	32%	24%	272%	20%	75%	21%	24%	17%	22%	21%	38%	6%	90%	23%	168%

Discussion

The following can be concluded from the comparison of traffic flows jointly with queue length:

We cannot conclude that there is a high correlation between increasing traffic with increasing queue at these two critical intersections. One of the reasons for this may be that generally the changes in either of these measures are not significant. A second reason may be that improved signalization patterns resulted in higher throughputs for certain movements, which also showed shorter queues.

It is shown from the colored cells in the tables that the traffic conditions for the SR 77. & 23rd Street intersection EB and WB (Main) have improved during several time periods after the InSync installation. However, the traffic conditions for the NB and SB (Side) have somewhat deteriorated during several time periods.

BEFORE AND AFTER-IMPLEMENTATION STUDIES OF

ADVANCED SIGNAL CONTROL TECHNOLOGIES IN FLORIDA

The main objective of this project is to evaluate the implementation of proposed adaptive signal control technology (ASCT) traffic operations at several arterial corridors in Florida, before and after the installation of the ASCT, document the advantages and the disadvantages of different approaches and implementations, and provide recommendations for statewide implementation of ASCT.

This questionnaire will allow us to access the appropriateness and effectiveness of the ASCT implementation at each subject corridor by: conducting benefit cost analyses, comparing staffing requirements before and after the deployment, comparing the resource needs in terms of hardware, software and personnel, pinpointing any institutional issues and generally evaluating the ASCT implementation and maintenance requirements by incorporating the agency's perspective.

E.4 QUESTIONNAIRE

SECTION 1: RESPONDENT'S INFORMATION

Name **Marc R. Mackey**

Organization **Bay County Public Works**

Position **Traffic Operations Engineer**

Address

Phone

Fax ____

E-mail **mmackey@baycountyfl.gov**

NOTE: The interview in Appendix B and in this Appendix are the same interview. Both this site and the site in Appendix B are situated in Bay County, and only one interview was conducted for the county.

SECTION 2: PREVIOUS TRAFFIC CONTROL TECHNOLOGY

37. Please specify the type of traffic control used before Adaptive Signal Control Technology (ASCT) was installed for your site. (Please check all that apply.)

a. Fixed time coordinated control

b. Actuated coordinated control

c. Fixed-time isolated control

d. Actuated isolated control

e. Other, please specify: ____

38. What type of traffic controller was used before ASCT was implemented?

a. NEMA TS-1

b. NEMA TS-2

c. 170

d. 2070

e. Other, please specify: ____

39. How many detectors were used before ASCT was implemented for a typical intersection (e.g. *Newberry Rd & NW 75th St*)? Please also specify the location where the detectors were typically placed for each specific detector type (video detection, inductive loops, radars, etc.).

    Two advance detectors at each main through movement. Left turning detectors for all approaches; Stop bar detector for side-streets.

40. What was the frequency of retiming the signal timing plan for the corridor before ASCT was implemented?

    2-3 years

41. How often were maintenance and updates performed to your previous control system in terms of the following components?

c. Detection: **Regular quarterly check usually accompanied by an annual check**

d. Control Hardware:

42. Software:

43. What was the annual maintenance cost in terms of hardware, software and personnel of the previous control system for the whole corridor where the ASCT is now deployed?

44. What was the life expectancy of your traffic control before the deployment of ASCT? (Please refer to the hardware and software.)

It went well (for 6 years) before being replaced. The old system is quite simple and durable.

45. How many crashes of each category were reported during the past 4 years before the ASCT implementation? (2011-2014)

	K (Killed)	A (Disabling Injury)	B (Evident Injury)	C (Possible Injury)	O (No Apparent Injury)
2011					
2012					
2013					
2014					

SECTION 3: ADAPTIVE TRAFFIC CONTROL SYSTEM (ASCT)

46. Which Adaptive Traffic Control System (ASCT) does your agency deploy?

    a. InSync; Version: **Fusion**

    b. Synchro Green; Version: ____________________

    c. Other; please specify: _______________________

47. Did you consider any other ASCT before you selected this one for installation? Why did you reject the other(s) in favor of this one?

    No, state picked the software

48. What were your major criteria for choosing the ASCT for the selected corridor?

    N/A

49. What is the life expectancy of your ASCT system? (Consider both hardware and software.)

    5 years warranty purchased.

50. What type of traffic controller is currently in use after the ASCT implementation?

d. Same as before the ASCT implementation

e. New, please specify type:

f. Same, but the following updates were needed: ______________________________

51. How many and what type of detectors are used after the ASCT implementation on an intersection level (e.g. *Newberry Rd & NW 75th St*)? Please note any updates that the previous detection system needed so as to work with your ASCT.

    4 cameras (one each approach); All left-turn detectors; Side street stop bar detectors. Fusion provides additional back-up data counts. However, the operation of the system depends on the video detection

52. Was there a need to update your previous software operating system at your Transportation Management Center in order to get your current ASCT software working?

    a. Yes b. No

53. How often do you plan to perform physical maintenance or updates on your new ASCT in terms of the following components?

c. Detection:

d. ASCT Hardware:

c. Software:

54. Was there a need for training your personnel in order to operate the new ASCT?

c. **Yes,** Num. of employees trained **4 ,** extensive training that covers intro, software, hardware and system setting, implementation.

d. No

55. Was there any change on the number of staff required to operate/maintain the effective signal ASCT operations? (Comparison of staff needed before and after the deployment.)

No

56. Where did expertise come from for your personnel's training, the ASCT installation, operation and the system's maintenance? Please check all that apply.

	In-House	Vendor	Contractor
Employee Training		**Yes**	
ASCT Installation		**Yes**	
ASCT Operation		**Yes**	
ASCT Maintenance		**Yes**	

57. How many crashes of each category were reported after ASCT was implemented?

K (Killed)	A (Disabling Injury)	B (Evident Injury)	C (Possible Injury)	O (No Apparent Injury)

SECTION 4: COST COMPONENTS

58. What is the overall capital cost of purchasing the ASCT (in terms of the software and licenses of the ASCT) for the corridor? If the purchase includes any service of implementation, personnel training or maintenance, please also specify here.

    $45,000 per intersection

59. What is the implementation cost (considering the installation on site, any updates in software and hardware, design needs and contract hours) for the ASCT corridor? Please specify if this cost is partially or totally included in previous cost items.

60. What was total cost for training your personnel to operate and manage the new ASCT? Please specify if this cost is partially or totally included in previous cost items.

61. What is the expected maintenance cost (in terms of hardware, software and personnel) for the ASCT corridor on a yearly basis? Please specify if this cost is partially or totally included in previous cost items.

62. Can you specify the costs for the following ASCT components analytically?

b. Firmware $_____________ b. Software $______________

c. Equipment $____________ d. Maintenance of Traffic Cost $

e. Design Needs $___________ f. Contract Help/Agency Hours Cost $

SECTION 5: INSTITUTIONAL ISSUES

63. Were there any institutional issues that you had to overcome while implementing and operating the ASCT project? Please categorize and explain those issues below.

e. Organization and Management Institutional Issues:

N/A

f. Regulatory and Legal Institutional Issues:

____

g. Human and Facility Resources Institutional Issues:

____

h. Financial Institutional Issues:

____

64. Which component (e.g. detection, communication, hardware, software, licensing or other) of your ASCT deployment is the most challenging to maintain and why?

Camera; can get damaged in lightning

65. What happens to your ASCT operations when some of your detectors fail on the corridor level?

a. ASCT triggers an alarm and notifies operators

b. ASCT switches to an “off-line” mode by implementing Time-Of-Day plans

c. Combination of the above.

d. Other (please describe): _ Run off line data in the most recent two weeks

66. How is your ASCT performance evaluated?

a. In-house

1. from the preliminary analysis:

There is some benefits in the reduction of the rear-end crashes;

2. Majority of the benefits are in the travel time saving. Generally 7% travel time reduction is seen; in certain case, the reduction is even closer to 20% yet somewhat hurts the side-street;

3. The benefits are mainly seen in the light or moderate traffic volumes. In the over-saturated case, the performance is "equally bad" to the previous system.

b. By an independent evaluator, please describe: ________________________________

c. Not applicable—there is no evaluation

67. If the corridor on which the ASCT operates experiences over saturation, how would you rate the operation of the ASCT in response to these traffic conditions?

a. ASCT prevents or eliminates oversaturation

b. ASCT eliminates or reduces the extent of the periods of oversaturation

c. ASCT adversely affects the traffic conditions during periods of oversaturation

d. Other: ____

e. Oversaturation is very rare on the corridors operated by our ASCT

68. Based on your opinion and the up-to-date operation, when are ASCT operations proven to be the most effective?

a. Peak periods (Comment: with no saturation) b. Off-peak periods

c. Shoulders of peak periods d. Other, please specify: _______________

69. Has the level of performance of ASCT been sustained since its installation?

a. Yes b. No; why not?

70. What was the public reaction to the ASCT operation? Have you received any complaints about long delays and queues?

The public generally are happy with the new system

71. How satisfied are you, in general, with your ASCT deployment?

a. Very satisfied b. Somewhat satisfied

c. Neutral d. Somewhat dissatisfied

e. Not satisfied at all

72. Are there any other costs or benefits related to ASCT deployment that you would like to report?

a. Benefits **synchronizes and enriches the data from varying aspects. Can have straight access to the database with evidence if things went wrong.**

b. Costs

36. Would you consider expanding the ASCT program in any other of your other sites? Why or why not? If yes, would you use the same technology/firmware or have any suggestions for others? If so, why?

Yes, one additional corridor (Tyndall Parkway) will also deploy InSync

SECTION 6: SAFETY ISSUES

37. Do you have any anecdotal / qualitative data on safety benefits / disbenefits of the signal improvement along this corridor?

____

____

38. Has there been other changes along this corridor over the analysis period that could affect the safety of this corridor either positively or negatively?

____

____

39. Are you aware of any changes to the crash data reporting procedures over the analysis period? If yes, what are the changes?

____

____

____

____

40. What are your overall reactions to the trends presented in the report?

____

____

41. Is there another comparable corridor in which signal improvements have not been made to which the results of this corridor can be compared to?

____

____

42. Other thoughts / comments for us?

____

____

Thank you for your help in completing this survey. Your responses will help us with evaluating the deployment and benefits of your current ASCT. If you have any questions regarding the survey, please contact Ria Kontou, email: ekontou@ufl.edu or Liteng Zha, email: litengzha@ufl.edu who work under the direct supervision of the faculty advisor Dr. Yafeng Yin

E.5 BENEFIT COST ANALYSIS

Table E-7.1 Monetized Saving

Monetized Saving	
Time saving	$1,588,364.71
Fuel Consumption saving	$195,701.98
Air Pollution Saving	$39,451.13
Safety Saving	-$189,008.41
Total Saving without Safety	$1,823,517.82
Total Saving	$1,634,509.41
Total Cost	$360,000.00
B/C Ratio without safety	5.065327264
B/C Ratio	4.540303914

Table E-7.2 Monetized Saving with No Emissions

Monetized Saving	
Time saving	$1,588,364.71
Fuel Consumption saving	$195,701.98
Safety Saving	-$189,008.41
Total Saving without Safety	$1,784,066.69
Total Saving	$1,595,058.28
Total Cost	$360,000.00
B/C Ratio without safety	4.955740795
B/C Ratio	4.430717445

APPENDIX F: Summary of 66th St, Pinellas County

F.1 EXECUTIVE SUMMARY

The objective of this research is to evaluate traffic operations at several arterial corridors in Florida, before and after the implementation of proposed Adaptive Signal Control Technologies (ASCT), document the advantages and disadvantages of different ASCT approaches and implementations, and provide recommendations for state-wide implementation of ASCT.

This appendix summarizes the before and after field data collected along the 66th Street, Pinellas County corridor from Ulmerton Road to 54th Avenue. This corridor has a different type of configuration compared to the other arterial corridors analyzed in this project, since the InSync adaptive signal control system has been implemented along a side street rather than along the mainline arterial. The system has been implemented along 66th Street (which runs North-South), while the mainline arterials are Ulmerton Road and Park Boulevard (which run East-West). These two arterials carry high levels of commuter traffic to Tampa via I-275 and to Clearwater via SR-19. The intersections of these arterials with the 66th Street are considered as the two critical intersections in the analysis of this corridor. In total, the corridor spans 5 miles in length with 40 unsignalized sections spread over either sides of the corridor.

The InSync adaptive signal control system was implemented along this corridor in April 24, 2017. Two data collection methods were used to collect the desired information. Floating car runs were conducted with the UFTI instrumented vehicle to collect vehicle travel times during three time periods – AM Peak (7-9AM), Off Peak (1-3 PM) and PM Peak (4-6PM.) In addition, turning movement counts and queue lengths were collected at the two critical intersections (Ulmerton Road and Park Boulevard). Based on these, five performance measures were obtained for the before and after study period: Link/Route Travel Time, Delay at Intersections, Queue Length (at critical intersections), Queue to Lane Storage Ratio (at critical intersections), and Passenger Car Equivalent (PCE) flows (at critical intersections). For each performance measure, a comparison between the before and after data is conducted and presented in this appendix.

The following were observed:

- The InSync resulted in an overall decrease in travel time in both directions (20.3% for the SB, and 10.5% for the NB) and during all time periods studied. Conditions improved more significantly for the SB direction. This corridor has the highest percentage decrease in travel time among all the corridors studied in this project so far.
- During the before study, the intersection delay for the two critical intersections – Ulmerton Rd and Park Boulevard – were the highest compared to all other intersections along the corridor, particularly for the SB. After the InSync installation there was an overall decrease in delay in both directions, especially for the SB.
- The average queue length at Ulmerton Road decreased for all approaches and time periods except for the WB through movements during PM peak and Off peak. The Park Boulevard intersection average queue length decreased for the NB and SB

approaches, but the EB and WB approaches had longer queues during the Off and PM peak periods.

- As expected, the traffic volumes at both the critical intersections are higher for the EB and WB directions during both before and after studies. The PM peak has higher volumes than the AM and Off peak periods.

F.2 CORRIDOR INFORMATION

Figure F-1 provides a schematic of the 66th Street, Pinellas County Corridor (North-South). Table F-1 lists the intersections along the corridor. The adjacent land use is mostly industrial and commercial with restaurants, hotels, gas stations, drug stores, automotive service shops and banks. Commuting and recreational traffic coming mainly from Tampa towards St. Pete Beach is carried by I-275, toward the NE of this corridor. This traffic crosses the corridor through two intersections (Ulmerton Rd and Park Blvd) which were selected by the County as the critical intersections for this corridor. These two roadways are actually the mainline arterials, while the study corridor (66th St) is a side street. Ulmerton Road mainly carries traffic to and from Clearwater and Tampa, whereas Park Boulevard carries traffic mainly to and from Tampa, feeding I-275. For the analysis of the two critical intersections detailed turning movement and queue counts were collected. Figure F-2 provides the lane configuration of these two critical intersections.

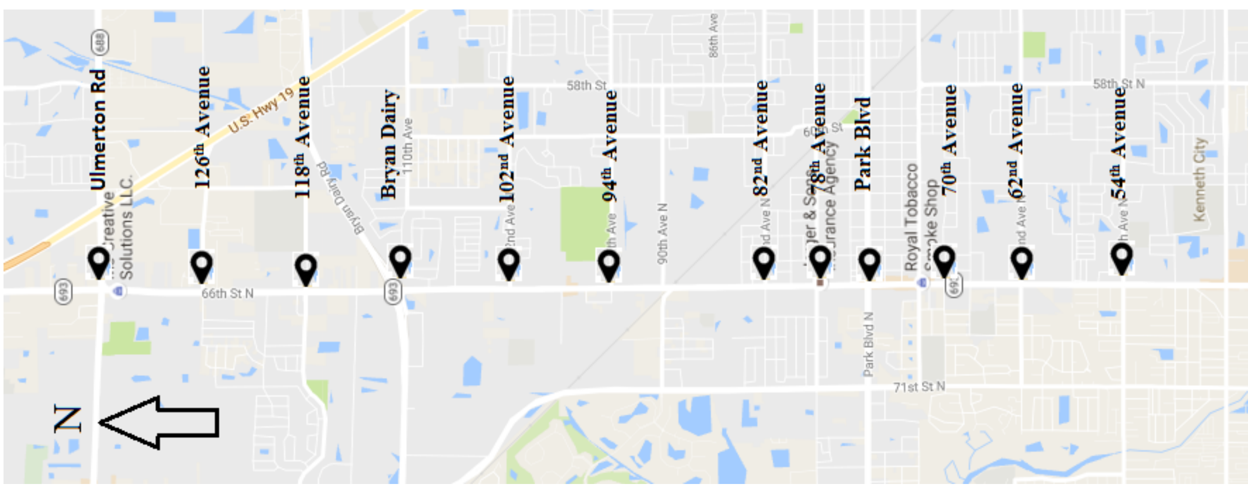
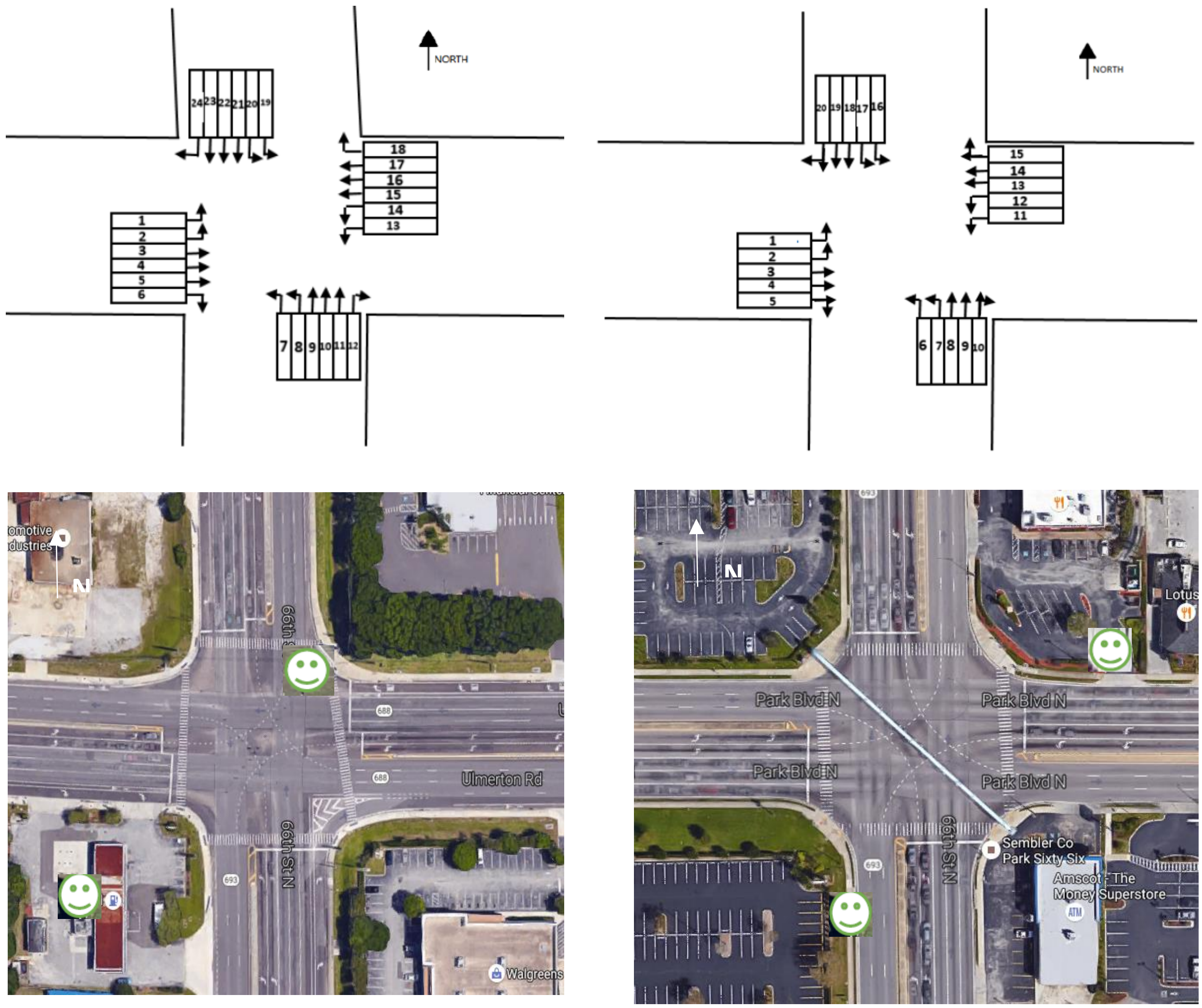


Figure F-1 Schematic of 66th Street, Pinellas County Corridor

Table F-1 List of Intersections along SR 693 (66^{th} Street) Corridor

	Intersection	No. of Unsignalized Intersections	Distance
1	**Ulmerton Rd**	-	-
2	126^{th} Avenue	2	0.5 Miles
3	118^{th} Avenue	6	0.5 Miles
4	Bryan Dairy	3	0.45 Miles
5	102^{nd} Avenue	2	0.55 Miles
6	94^{th} Avenue	5	0.5 Miles
7	82^{nd} Avenue	8	0.75 Miles
8	78^{th} Avenue	1	0.25 Miles
9	**Park Blvd**	2	0.25 Miles
10	70^{th} Avenue	2	0.25 Miles
11	62^{nd} Avenue	5	0.5 Miles
12	54^{th} Avenue	4	0.5 Miles



(a) SR 693 (66th Street) and SR 688 (Ulmerton Rd) (b) SR 693 (66th Street) and SR 694 (Park Boulevard)

Figure F-2 Lane Configuration Schematic and Overview Aerial Photo of Critical Intersections

F.3 PERFORMANCE MEASURES

Five performance measures were evaluated: Link/Route Travel Time, Delay at Intersections, Queue Length (critical intersections), Queue-to-Lane Storage Ratio (critical intersections), and PCE Flows (critical intersections). For each performance measure, a comparison between the before and after data is conducted and the results of the differences ("after data" – "before data") are presented.

F.3.1 ROUTE TRAVEL TIME

The average travel time (min) along the route was measured through a floating car study. During the before data collection we collected data for a total of 8 runs for each time period. During the after data collection we were able to perform during the first day a total of 10 runs for each time period, and during the second day 11 runs for the AM peak, 9 for the OFF peak and 9 for the PM peak. Tables F-1.1 and F-1.2 provide the route travel time for the before and after data. The travel time comparison is presented in Table F-1.3.

Before Study (Nov. 9 & Nov. 10, 2016)

Table F-1.1 Route Travel Time (min)

Route TT (min)	AM Peak	Off Peak	PM Peak	Average
66th Street, SB	12.94	14.37	14.84	**14.05**
66th Street, NB	13.88	13.06	14.43	**13.78**

After Study (May 16 & May 17, 2017)

Table F-1.2 Route Travel Time (min)

Route TT (min)	AM	Off Peak	PM	Average
66th Street, SB	10.57	11.49	11.53	**11.20**
66th Street, NB	11.82	11.96	13.22	**12.33**

Comparison of Before and After Travel Times

Table F-1.3 Change in Percentage of Route Travel Time (After – Before-no rain)

Route TT (min)	AM	Off Peak	PM	Average
66th Street, SB	-2.37 (-18.3%)	-2.88 (-20%)	-3.31 (-22.3%)	**-2.85 (-20.3%)**
66th Street, NB	-2.06 (-14.8%)	-1.1 (-8.4%)	-1.21 (-8.4%)	**-1.45 (-10.5%)**

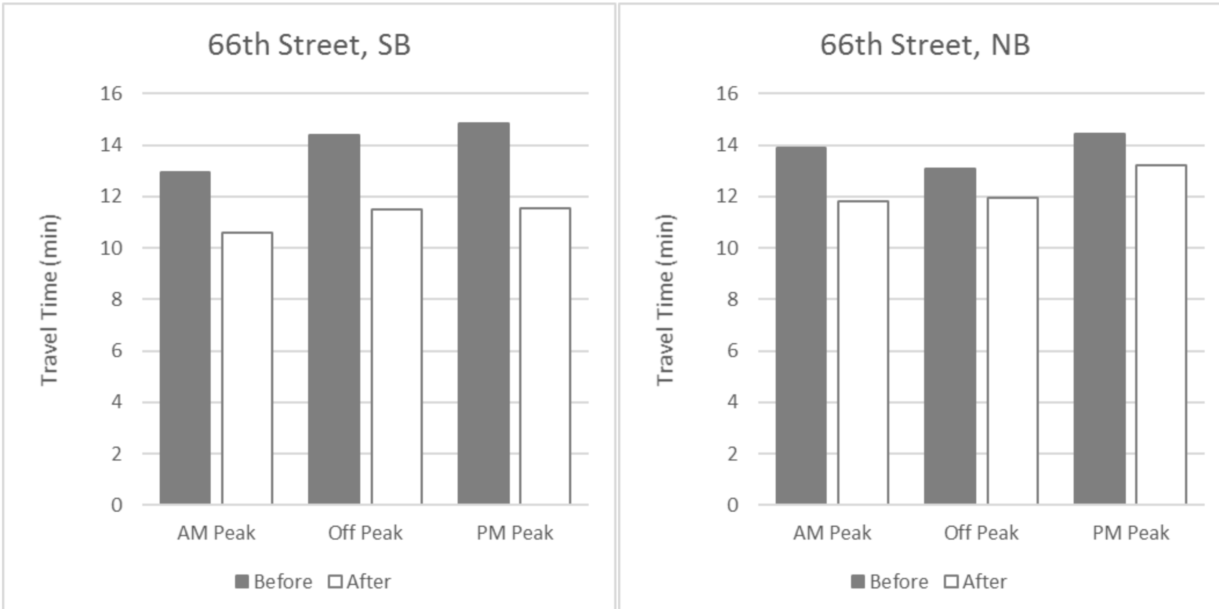


Figure F-1.1 Travel Times along 66th Street, Pinellas County

Discussion

The following can be concluded from the comparison of travel times:

- Overall, there was a reduction in travel time for both directions (20.3% for the SB, and 10.5% for the NB) and during all time periods. The reduction is generally higher for the SB direction. This corridor has the highest percentage decrease in travel time among all the corridors evaluated to-date for this project. This may be because this corridor is not a main arterial, and thus may not have had optimal signal timings.
- The greatest reduction in travel time occurs during the PM peak for the SB direction (22.3% or 3.31 min.)
- The relationship of travel times between the three study periods is similar for the before and after data collection, i.e., the travel time for the PM peak period is the highest for both the SB and NB directions along the 66th Street. In the SB, the Off Peak travel time is higher than the AM Peak for both the before and after studies. In the NB, the Off Peak is less than the AM Peak and the PM Peak for the before, and nearly the same for the after study.

F.3.2 DELAY

Delay (sec) at each intersection along the corridor was also obtained using the floating car measurements.

Before Study (Nov. 9 & Nov. 10, 2016)

Table F-2.1 and Figure F-2.1 show the delay at each intersection for the through movement in the SB in tabular and graphical view. The intersection delay for the two critical intersections (Ulmerton Rd and Park Blvd.) is the highest during all study time periods (AM Peak, Off Peak and PM Peak), particularly for the SB. At other intersections, the delay is significantly less.

Table F-2.1 Delay (s) for each Intersection Through Movement Along the SB Direction

Delay at Intersections (sec)	AM Peak	Off Peak	PM Peak
Ulmerton Rd	**121.13**	**112.00**	**150.00**
126th Avenue	0.00	16.21	0.00
118th Avenue	0.00	0.00	0.00
Bryan Dairy	9.50	35.92	0.00
102nd Avenue	2.75	27.29	0.00
94th Avenue	1.63	6.58	0.88
82nd Avenue	3.75	11.79	2.38
78th Avenue	26.63	0.00	12.75
Park Blvd	**86.75**	**102.92**	**112.75**
70th Avenue	7.38	1.50	0.00
62nd Avenue	14.63	6.79	15.88
54th Avenue	45.25	47.00	58.38

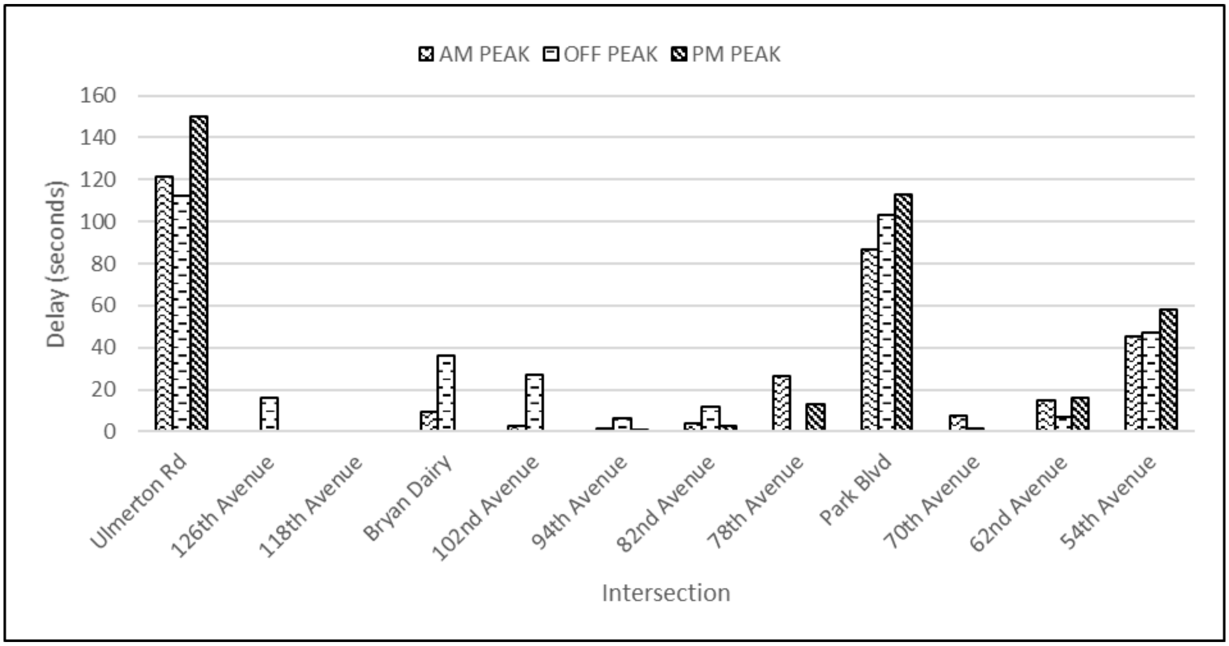


Figure F-2.1 Delay (s) for each Intersection Through Movement Along the SB Direction-Before Study

Table F-2.2 and Figure F-2.2 show the delays obtained by intersection for the NB. Figure F-2.2 shows the delay at each intersection for the through movement in the NB direction. The intersection delay in the NB direction is highest at Park Blvd during all study time periods (AM Peak, Off Peak and PM Peak). Delay at Ulmerton Rd is highest during the PM Peak period. The Off-Peak delays are higher than the PM Peak delays at six different intersections.

Table F-2.2 Delay (s) for each Intersection Through Movement Along the NB Direction – Before Study

Delay at Intersections (sec)	AM Peak	Off Peak	PM Peak
Ulmerton Rd	**31.25**	**35.63**	**69.00**
126th Avenue	35.88	13.00	2.13
118th Avenue	14.00	15.88	36.80
Bryan Dairy	39.00	32.38	20.25
102nd Avenue	23.75	23.75	20.50
94th Avenue	2.63	12.13	0.00
82nd Avenue	12.00	25.13	31.38
78th Avenue	13.88	14.13	0.00
Park Blvd	**88.50**	**67.13**	**70.63**
70th Avenue	3.38	8.38	5.63
62nd Avenue	18.00	14.88	23.00
54th Avenue	20.25	35.25	52.38

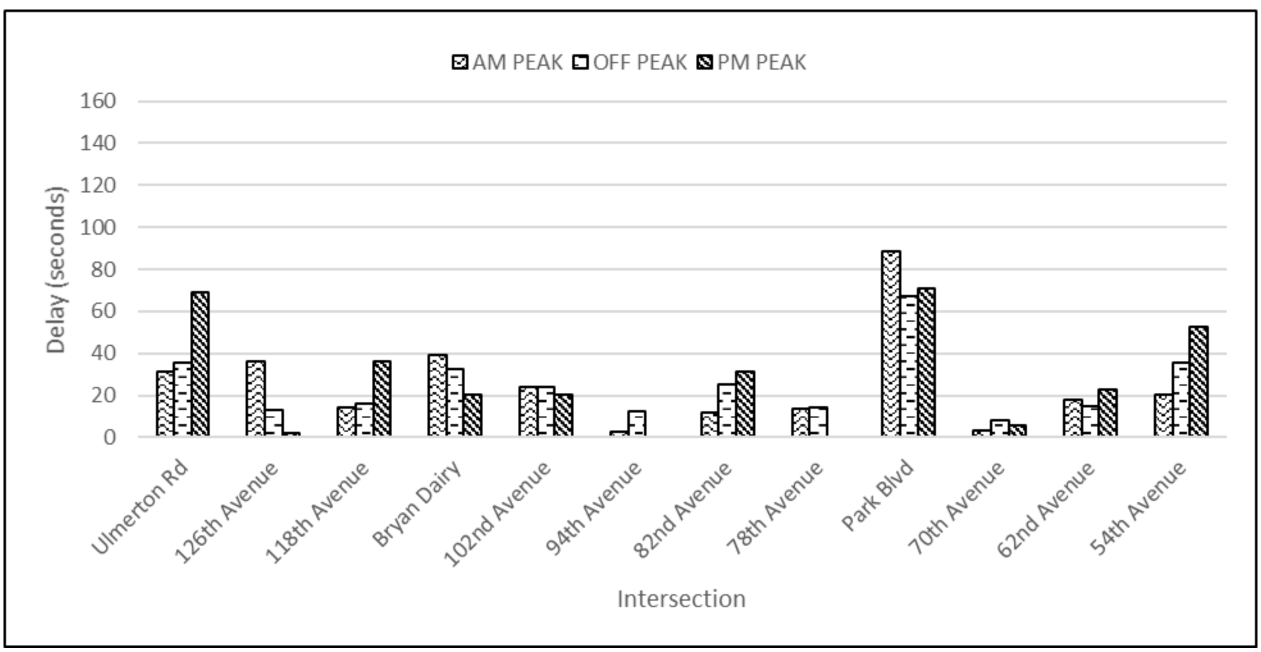


Figure F-2.2 Delay (s) for each Intersection Through Movement Along the NB Direction – Before Study

After Study (May 16 & May 17, 2017)

Table F-2.3 and Figure F-2.3 summarize the delay data for the after study in the SB direction. Again, the data shows that the highest intersection delay is reached at the two critical intersections. At other intersections, the delay is significantly less. The intersection with 118th Avenue remained with zero delay for all the runs conducted by floating car. Also, during the AM and PM peak periods some intersections had zero or very low delay for all runs.

Table F-2.3 Delay (s) for each Intersection Through Movement Along the SB Direction –After Study

Delay at Intersections (sec)	AM Peak	Off Peak	PM Peak
Ulmerton Rd	**81.91**	**57.00**	**42.44**
126th Avenue	0.09	2.22	0.00
118th Avenue	0.00	0.00	0.00
Bryan Dairy	0.00	21.56	0.00
102nd Avenue	10.00	21.44	21.33
94th Avenue	2.09	3.00	0.67
82nd Avenue	0.00	7.56	10.00
78th Avenue	2.45	6.78	7.33
Park Blvd	**35.82**	**36.78**	**65.78**
70th Avenue	0.27	8.44	5.56
62nd Avenue	0.00	3.78	6.33
54th Avenue	0.00	11.67	17.56

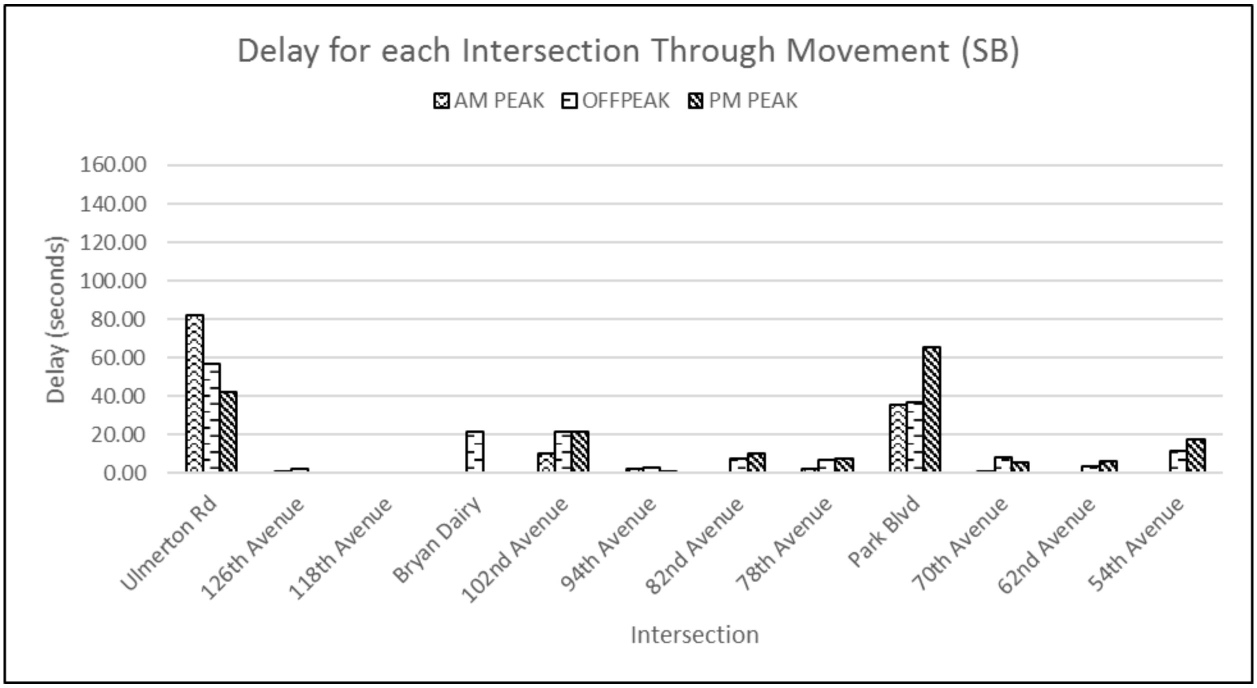


Figure F-2.3 Delay (sec) for each Intersection Through Movement Along the SB Direction – After Study

Table F-2.4 and Figure F-2.4 provide the delay data for the NB direction. As shown, the highest delay is reached at the intersections of Ulmerton Rd and 54th Avenue (approximately 1 minute of delay during the Off-peak and PM peak). Some intersections show zero or very low delay.

Table F-2.4 Delay (s) for each Intersection Through Movement Along the NB Direction– After Study

Delay at Intersections (sec)	AM Peak	Off Peak	PM Peak
Ulmerton Rd	**35.40**	**73.44**	**51.67**
126th Avenue	0.00	3.11	2.00
118th Avenue	20.00	1.56	0.67
Bryan Dairy	29.70	14.67	50.89
102nd Avenue	19.00	21.78	29.22
94th Avenue	1.60	0.00	0.00
82nd Avenue	7.20	0.00	0.00
78th Avenue	0.00	0.00	0.00
Park Blvd	**16.30**	**14.22**	**0.89**
70th Avenue	15.80	1.78	15.67
62nd Avenue	8.70	0.78	12.11
54th Avenue	25.80	62.89	64.00

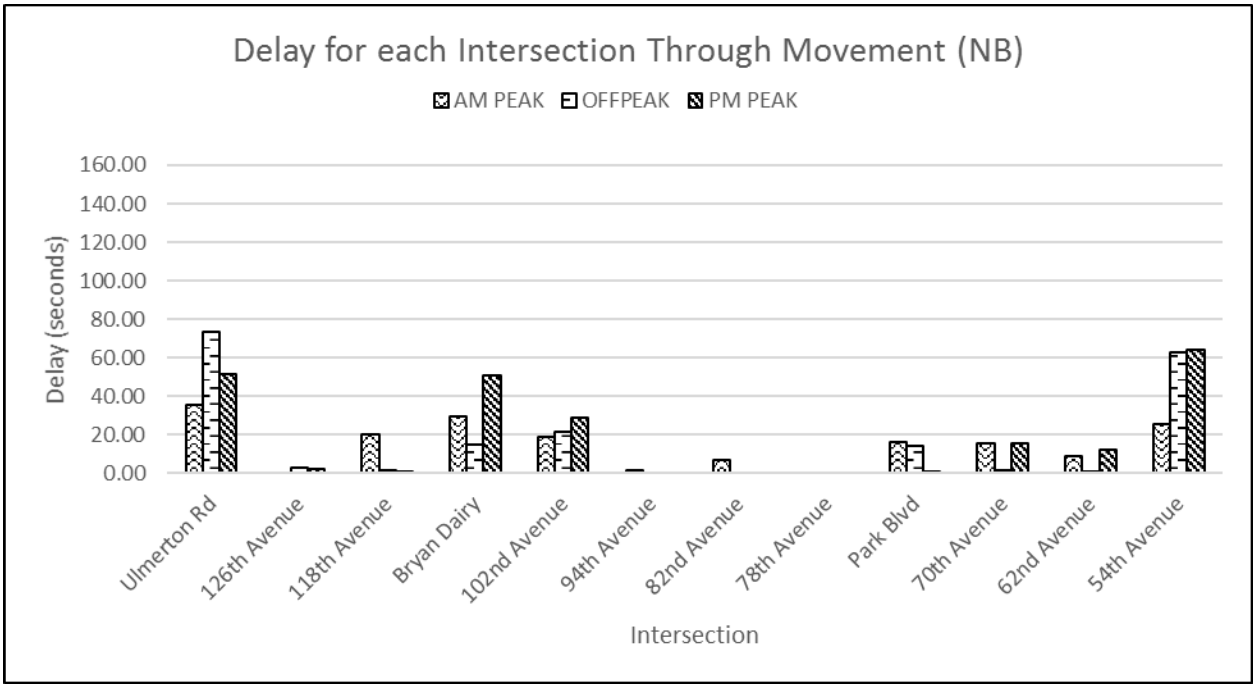


Figure F-2.4 Delay (sec) for each Intersection Through Movement Along the NB Direction – After Study

Comparisons of Before and After Intersection Delay Times

The differences in delay between the before and after studies are shown in Table F-2.5 and Table F-2.6. The tables are color-coded as follows: green shows significant improvement, yellow shows modest change (either improvement or deterioration), and red shows significant deterioration in delay. Several gradations of each color are used to represent different variations within each classification.

Table F-2.5 Difference in Delay (sec) for each Intersection Through Movement Along the SB Direction

Delay at Intersections (sec)	AM Peak	Off Peak	PM Peak
Ulmerton Rd	**-39.22**	**-55.00**	**-107.56**
126th Avenue	0.09	-13.99	0.00
118th Avenue	0.00	0.00	0.00
Bryan Dairy	-9.50	-14.36	0.00
102nd Avenue	7.25	-5.85	21.33
94th Avenue	0.46	-3.58	-0.21
82nd Avenue	-3.75	-4.23	7.62
78th Avenue	-24.18	6.78	-5.42
Park Blvd	**-50.93**	**-66.14**	**-46.97**
70th Avenue	-7.11	6.94	5.56
62nd Avenue	-14.63	-3.01	-9.55
54th Avenue	-45.25	-35.33	-40.82
Average	**-15.56**	**-15.65**	**-14.67**

Table F-2.6 Difference in Delay (sec) for each Intersection Through Movement Along the NB Direction

Delay at Intersections (sec)	AM Peak	Off Peak	PM Peak
Ulmerton Rd	**4.15**	**37.81**	**-17.33**
126th Avenue	-35.88	-9.89	-0.13
118th Avenue	6.00	-14.32	-36.13
Bryan Dairy	-9.30	-17.71	30.64
102nd Avenue	-4.75	-1.97	8.72
94th Avenue	-1.03	-12.13	0.00
82nd Avenue	-4.80	-25.13	-31.38
78th Avenue	-13.88	-14.13	0.00
Park Blvd	**-72.20**	**-52.91**	**-69.74**
70th Avenue	12.42	-6.60	10.04
62nd Avenue	-9.30	-14.10	-10.89
54th Avenue	5.55	27.64	11.62
Average	**-10.25**	**-8.62**	**-8.72**

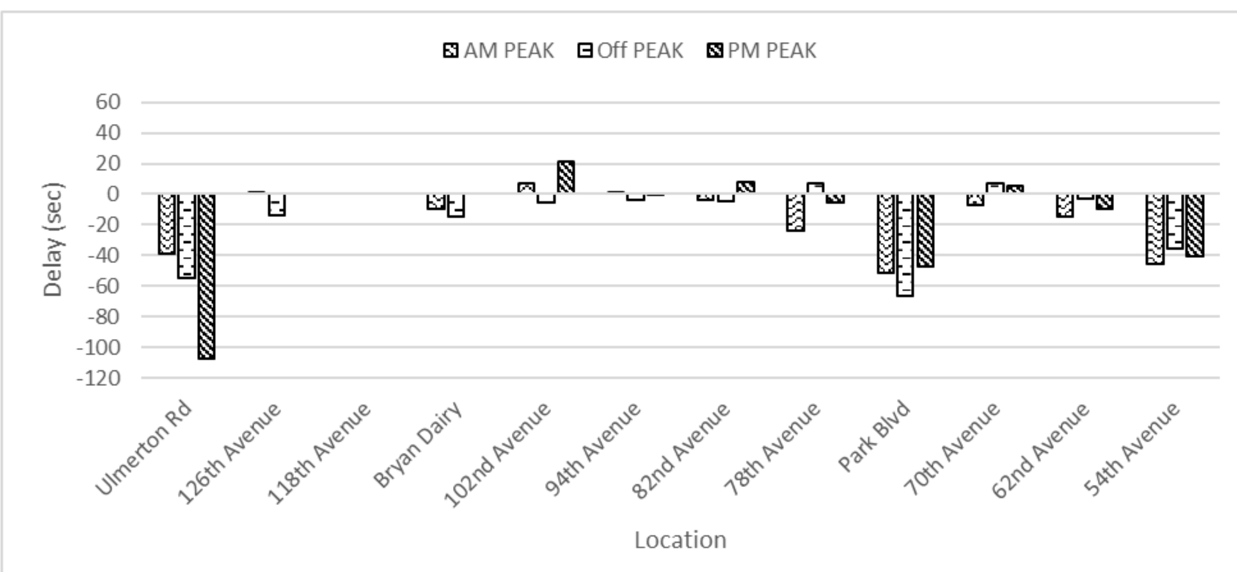


Figure F-2.5 Difference in Delay (sec) for each Intersection Through Movement Along the SB Direction

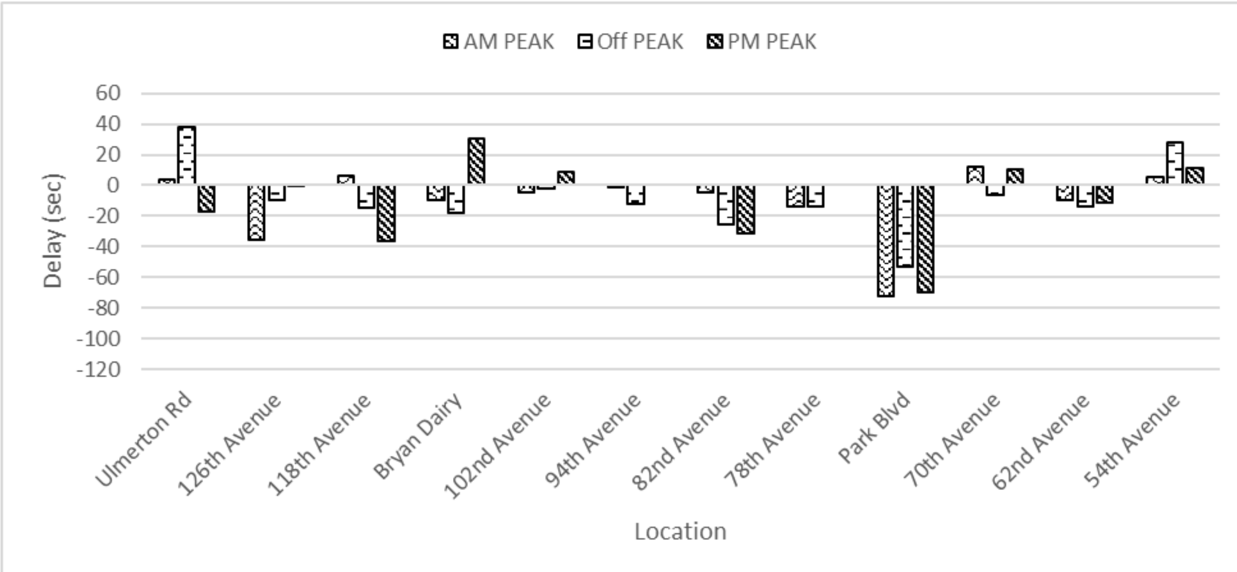


Figure F-2.6 Difference in Delay (sec) for each Intersection Through Movement Along the NB Direction

Discussion

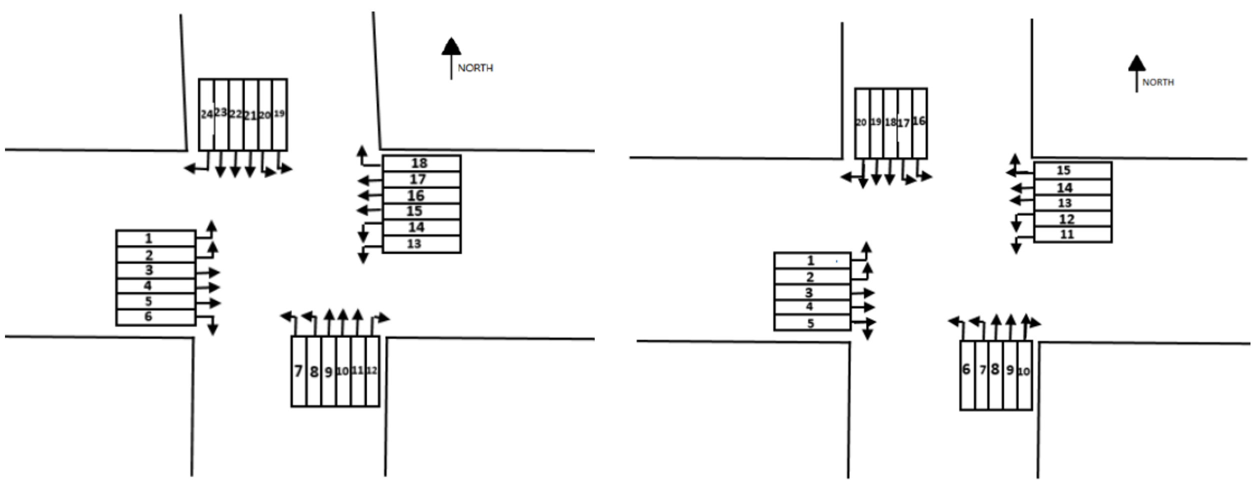
The following can be concluded from the comparison of delays:

- The data shows that after installation of the InSync system there was an overall decrease in delay in both directions.
- One of the critical intersections, Park Boulevard, had a reduction in delay during all three time periods in both directions. The most significant delay reduction was 107.56 seconds and was observed at the other critical intersection, Ulmerton Road, during the PM peak in the SB direction.
- An increase in delay was observed for some intersections. In the SB direction, the maximum increase was 21.3 seconds and was observed at the 102nd Avenue intersection during the PM peak. In the NB direction, four intersections show a significant increase in delay (mostly during the PM peak).

F.3.3 QUEUE LENGTH

Queue length (number of vehicles/lane) is presented by movement and by time period. This measure is used to evaluate oversaturated conditions at the critical intersections along the study corridors. Note that the queue length reported here is the observed maximum number of vehicles queued during each cycle, and does not represent the total number of vehicles that may have stopped during the cycle. During some time periods, because of cycle failure, vehicles need to stop multiple times before passing through the intersection.

Figure F-3.1 presents the schematic of the lane configurations at the two critical intersections. The queue length is reported for each of these lanes.



(a) SR 693 (66th Street) and SR 688 (Ulmerton Rd) (b) SR 693 (66th Street) and SR 694 (Park Blvd)

Figure F-3.1 Lane Configuration of Critical Intersections

Before Study (Nov. 9 & Nov.10, 2016)

Table F-3.1 Average Queue Length by Lane (#vehs/lane) at 66th Street & Ulmerton Rd – Before Study.

Time Period	Time Segment (15 min)	EB						NB						WB						SB					
		Left		Through			Right	Left		Through			Right	Left		Through			Right	Left		Through			Right
Lane Number		**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**	**13**	**14**	**15**	**16**	**17**	**18**	**19**	**20**	**21**	**22**	**23**	**24**
AM Peak	1	22	22	22	22	22	0	4	5	7	8	7	0	5	5	11	10	9	0	3	7	15	17	8	2
	2	22	22	22	22	22	0	6	6	13	13	13	0	6	6	9	8	8	0	3	7	14	11	11	3
	3	22	22	22	22	22	0	4	6	16	15	14	0	7	7	8	9	8	1	3	5	20	18	17	2
	4	22	22	22	22	22	0	6	6	13	13	13	0	5	7	9	10	11	0	2	3	13	12	10	2
	Average	**22**	**22**	**22**	**22**	**22**	**0**	**5**	**6**	**12**	**12**	**11**	**0**	**6**	**6**	**9**	**9**	**9**	**0**	**3**	**5**	**15**	**14**	**12**	**2**
Off Peak	1	15	15	16	15	15	0	5	4	10	10	8	0	5	6	9	11	10	0	2	5	11	6	9	6
	2	15	15	18	18	18	0	8	9	11	10	10	3	7	7	6	5	6	2	0	2	13	11	9	6
	3	11	10	14	13	12	0	7	7	13	12	10	0	5	6	5	6	8	2	3	4	16	9	9	4
	4	7	7	18	19	19	0	7	8	17	17	16	0	5	6	8	7	6	0	2	3	14	7	6	6
	Average	**12**	**12**	**16**	**16**	**16**	**0**	**7**	**7**	**12**	**12**	**11**	**1**	**5**	**6**	**7**	**7**	**7**	**1**	**2**	**3**	**13**	**8**	**8**	**5**
PM Peak	1	9	10	14	15	16	4	7	9	16	16	16	5	7	7	8	8	8	1	2	3	8	7	7	2
	2	10	10	19	19	20	4	9	10	14	14	13	5	5	6	9	12	8	2	1	3	7	6	7	4
	3	11	10	14	12	13	5	10	10	18	17	17	4	8	7	12	11	12	1	2	1	18	14	11	5
	4	8	8	8	9	7	4	8	10	15	15	15	4	4	6	9	9	10	1	1	3	12	9	10	4

Time Period	Time Segment (15 min)	EB						NB						WB						SB					
		Left		Through			Right	Left		Through			Right	Left		Through			Right	Left		Through			Right
	Average	10	10	14	14	14	4	8	10	16	16	15	5	6	6	9	10	9	1	1	3	11	9	8	4

Table F-3.2 Average Queue Length (#vehs/lane) at 66th Street & Ulmerton Rd – Before Study.

Time Period	EB			NB			WB			SB		
	Left	Through	Right	Left	Through	Right	Left	Through	Right	Left	Through	Right
AM Peak	22	22	0	5	12	0	6	9	0	4	14	2
Off Peak	12	16	0	7	12	1	6	7	1	3	10	5
PM Peak	10	14	4	9	15	5	6	10	1	2	9	4

Table F-3.3 Average Queue Length by Lane (#vehs/lane) at 66th Street & Park Blvd – Before Study.

Time Period	Time Segment	EB					NB					WB					SB				
		Left		Through		T/R	Left		Through		T/R	Left		Through		T/R	Left		Through		T/R
		1	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**	**13**	**14**	**15**	**16**	**17**	**18**	**19**	**20**
AM Peak	1	2	4	15	17	14	5	6	19	20	0	5	5	10	10	9	5	7	11	12	12
	2	2	4	17	18	17	6	9	19	19	0	13	15	16	7	10	3	7	18	19	20
	3	4	6	21	21	20	12	14	20	20	0	5	8	12	11	10	6	14	20	20	20
	4	4	4	16	16	16	5	6	15	17	1	5	7	10	8	9	6	10	17	18	16
	Average	**3**	**4**	**17**	**18**	**17**	**7**	**9**	**18**	**19**	**0**	**7**	**9**	**12**	**9**	**10**	**5**	**9**	**16**	**17**	**17**
Off Peak	1	4	5	6	7	11	6	8	17	14	18	6	7	11	12	4	7	8	18	18	16
	2	3	5	7	7	14	8	15	12	9	16	6	7	7	8	3	10	10	18	18	19
	3	4	5	7	9	15	9	15	18	14	19	6	6	9	10	2	10	11	20	20	20
	4	4	5	7	8	14	9	19	20	18	19	7	8	10	10	5	8	7	20	20	20
	Average	**4**	**5**	**7**	**8**	**13**	**8**	**14**	**16**	**14**	**18**	**6**	**7**	**9**	**10**	**3**	**9**	**9**	**19**	**19**	**19**
PM Peak	1	7	10	12	12	13	6	13	13	13	13	5	10	18	19	20	9	16	20	20	20
	2	7	10	8	8	8	7	13	19	19	19	5	5	15	16	18	8	9	19	20	19
	3	6	9	10	10	9	6	10	16	15	13	3	6	17	17	20	7	9	20	20	20
	4	6	8	10	10	11	8	12	10	10	10	5	5	16	17	18	7	8	18	20	19
	Average	**6**	**9**	**10**	**10**	**10**	**7**	**12**	**14**	**14**	**14**	**4**	**6**	**16**	**17**	**19**	**7**	**10**	**19**	**20**	**20**

Table F-3.4 Average Queue Length (#vehs/lane) at 66th Street & Park Blvd – Before Study.

Time Period	EB			NB			WB			SB		
	Left	Through	T/R	Left	Through	T/R	Left	Through	T/R	Left	Through	T/R
AM Peak	4	18	17	8	18	0	8	10	10	7	17	17
Off Peak	4	7	13	11	15	18	7	9	3	9	19	19
PM Peak	8	10	10	9	14	14	5	17	19	9	20	20

As shown above, the longest queues were observed for the 66th Street and Ulmerton Rd intersection, for the EB left and through movements, with queues reaching 22 vehicles.

For the 66th Street and Park Blvd. intersection, the longest queue observed was 21 vehicles for the EB through movement. Also, long queues were observed in the other approaches, with up to 20 vehicles during certain periods, mostly for the through and right movements.

After Study (May 16 & May 17, 2017)

Tables F-3.5 to F-3.8 show the average queue length by lane and by movement for both critical intersections from the after study.

Table F-3.5 Average Queue Length by Lane (#vehs/lane) at 66th Street & Ulmerton Rd – After Study

Time Period	**Time Segment (15 min)**	**EB**						**NB**						**WB**						**SB**					
		Left		**Through**			**Right**	**Left**		**Through**			**Right**	**Left**		**Through**			**Right**	**Left**		**Through**			**Right**
Lane Number		**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**	**13**	**14**	**15**	**16**	**17**	**18**	**19**	**20**	**21**	**22**	**23**	**24**
AM Peak	1	8	8	19	19	19	1	6	5	10	10	8	1	4	4	4	4	4	4	4	4	4	4	4	4
	2	8	7	18	18	17	1	4	6	15	14	12	2	6	6	6	6	6	6	6	6	6	6	6	6
	3	7	6	14	15	15	3	5	6	12	11	10	2	4	4	4	4	4	4	4	4	4	4	4	4
	4	7	7	15	15	16	3	4	6	11	11	8	2	4	4	4	4	4	4	4	4	4	4	4	4
	Average	**7**	**7**	**17**	**17**	**17**	**2**	**5**	**6**	**12**	**11**	**9**	**2**	**4**	**4**	**4**	**4**	**4**	**4**	**4**	**4**	**4**	**4**	**4**	**4**
Off Peak	1	5	5	8	9	10	2	6	5	5	5	3	1	6	7	13	14	14	3	3	3	11	10	10	3
	2	5	6	7	7	9	1	4	5	3	3	1	1	7	6	15	16	15	3	1	2	13	10	9	2
	3	7	8	10	10	11	3	6	7	7	5	5	1	4	4	16	16	16	3	2	3	11	10	10	4
	4	7	8	13	12	12	3	7	8	13	9	8	1	3	4	11	11	12	2	2	4	7	7	7	3
	Average	**6**	**7**	**9**	**10**	**10**	**2**	**6**	**6**	**7**	**5**	**4**	**1**	**5**	**5**	**14**	**14**	**14**	**3**	**2**	**3**	**10**	**9**	**9**	**3**
PM Peak	1	11	10	14	14	14	3	13	12	14	14	12	1	3	4	7	8	8	1	2	3	7	7	8	5
	2	11	13	16	17	18	3	7	9	17	15	13	1	5	6	12	13	13	3	1	2	14	13	13	5
	3	8	7	14	13	17	4	9	9	12	12	10	1	5	5	23	23	23	3	1	1	16	17	15	6
	4	7	7	10	10	13	2	7	10	11	11	9	1	5	5	10	11	12	1	1	2	15	12	11	5

Time Period	Time Segment (15 min)	EB						NB						WB						SB					
		Left		Through			Right	Left		Through			Right	Left		Through			Right	Left		Through			Right
Lane Number		1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24
	Average	9	9	13	14	16	3	9	10	14	13	11	1	4	5	13	14	14	2	1	2	13	12	12	5

Table F-3.6 Average Queue Length (#vehs/lane) at 66th Street & Ulmerton Rd – After Study.

Time Period	EB			NB			WB			SB		
	Left	Through	Right	Left	Through	Right	Left	Through	Right	Left	Through	Right
AM Peak	7	17	2	5	11	2	4	4	4	4	4	4
Off Peak	6	10	2	6	5	1	5	14	3	2	9	3
PM Peak	9	14	3	10	12	1	5	13	2	2	12	5

Table F-3.7 Average Queue Length by Lane (#vehs/lane) at 66th Street & Park Blvd – After Study.

Time Period	**Time Segment**	**EB**					**NB**					**WB**					**SB**				
		Left		**Through**		**T/R**	**Left**		**Through**		**T/R**	**Left**		**Through**		**T/R**	**Left**		**Through**		**T/R**
		1	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**	**13**	**14**	**15**	**16**	**17**	**18**	**19**	**20**
AM Peak	1	3	3	13	13	12	3	6	10	9	9	7	8	9	9	8	5	5	6	6	7
	2	2	4	17	17	17	7	8	11	12	10	9	10	13	13	14	6	8	8	8	8
	3	4	7	15	15	17	7	9	8	8	6	5	8	10	10	9	6	6	8	8	7
	4	3	5	15	16	18	4	6	8	8	8	5	8	14	15	15	5	5	11	11	12
	Average	**3**	**5**	**15**	**15**	**16**	**5**	**7**	**9**	**9**	**8**	**6**	**8**	**11**	**11**	**11**	**5**	**6**	**8**	**8**	**8**
Off Peak	1	5	5	15	15	20	7	8	4	3	4	8	7	16	17	15	8	9	11	11	7
	2	4	4	12	12	16	6	7	5	6	8	7	7	15	14	12	5	6	9	10	8
	3	4	5	20	20	20	8	11	9	9	9	7	6	13	13	11	5	7	9	9	8
	4	4	6	15	16	20	8	10	6	5	6	5	6	19	20	19	6	7	9	8	10
	Average	**4**	**5**	**16**	**16**	**19**	**7**	**9**	**6**	**6**	**7**	**7**	**6**	**16**	**16**	**14**	**6**	**7**	**9**	**9**	**8**
PM Peak	1	3	4	18	17	20	7	8	7	9	10	6	9	19	18	17	9	9	9	9	8
	2	4	6	14	15	19	8	14	7	6	5	8	9	16	15	13	8	9	8	8	7
	3	4	4	20	20	20	7	9	6	5	4	8	10	17	18	14	10	10	15	15	15
	4	5	10	9	6	12	6	8	6	5	7	7	8	12	12	9	11	11	14	14	13

Time Period	Time Segment	EB					NB					WB					SB				
		Left		Through		T/R	Left		Through		T/R	Left		Through		T/R	Left		Through		T/R
		1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20
	Average	4	6	15	14	18	7	10	6	6	6	7	9	16	16	13	9	10	12	11	11

Table F-3.8 Average Queue Length (#vehs/lane) at 66th Street & Park Blvd – After Study.

Time Period	EB			NB			WB			SB		
	Left	Through	T/R	Left	Through	T/R	Left	Through	T/R	Left	Through	T/R
AM Peak	4	15	16	6	9	8	7	11	11	6	8	8
Off Peak	4	16	19	8	6	7	7	16	14	7	9	8
PM Peak	5	15	18	8	6	6	8	16	13	10	11	11

For the after study, at the 66th Street and Ulmerton Rd intersection the longest queues observed were at the EB through movement during the AM peak, with 19 vehicles. For the 66th Street and Park Blvd. intersection, the longest queue observed was of 20 vehicles during the Off peak and the PM peak for the through-right shared lane.

Comparison of Before and After Queue Lengths for Critical Intersections

The differences in queue length between the before and after measurements are shown in Table F-3.9 to Table F-3.12. The tables are color-coded as follows: green shows significant improvement, yellow shows modest change (either improvement or deterioration), and red shows significant deterioration in delay. Several gradations of each color are used to represent different variations within each classification.

Table F-3.9 Difference in Avg. Queue Length by Lane (#vehs/lane) at 66th Street & Ulmerton Rd

Time Period	EB						NB						WB						SB						Avg.
	Left		Through			Right	Left		Through			Right	Left		Through			Right	Left		Through			Right	
Lane Number	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	
AM Peak	-14.73	-15.05	-5.41	-5.16	-5.43	1.94	-0.09	0.03	-0.24	-0.71	-2.06	1.53	-1.46	-1.61	-0.25	-0.25	-1.44	-0.19	-0.25	-1.61	-1.79	-2.61	0.39	-2.06	-2.44
Off Peak	-5.65	-4.60	-6.97	-6.67	-5.76	2.18	-0.78	-0.88	-5.58	-6.64	-6.69	0.10	-0.09	-1.04	6.89	7.28	6.90	1.71	0.01	-0.10	-2.84	0.99	0.68	-2.43	-1.25
PM Peak	-0.28	-0.54	-0.23	-0.04	1.64	-0.88	0.98	0.28	-1.83	-2.75	-4.45	-3.46	-1.49	-1.46	3.74	3.63	4.53	0.75	-0.09	-0.34	1.74	3.40	3.54	1.75	0.34

Table F-3.10 Difference in Avg. Queue Length (#vehs/lane) at 66th Street & Ulmerton Rd

Time Period	EB			NB			WB			SB		
	Left	Through	Right	Left	Through	Right	Left	Through	Right	Left	Through	Right
AM Peak	-14.89	-5.33	1.94	-0.03	-1.00	1.53	-1.54	-0.65	-0.19	-0.93	-1.34	-2.06
Off Peak	-5.13	-6.47	2.18	-0.83	-6.30	0.10	-0.56	7.02	1.71	-0.04	-0.39	-2.43
PM Peak	-0.41	0.46	-0.88	0.63	-3.01	-3.46	-1.48	3.96	0.75	-0.21	2.89	1.75

Table F-3.11 Difference in Avg. Queue Length by Lane (#vehs/lane) at 66th Street & Park Blvd

Time Period	EB					NB					WB					SB					Average
	Left		Through		Right	Left		Through		Right	Left		Through		Right	Left		Through		Right	
Lane Number	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	
AM Peak	0.09	0.25	-2.54	-2.69	-0.43	-1.69	-1.50	-9.25	-9.44	7.81	-0.54	-0.19	-0.49	2.65	1.58	0.31	-3.31	-7.94	-8.81	-8.56	-2.23
Off Peak	0.13	0.06	8.75	8.00	5.75	-0.69	-5.29	-10.67	-8.13	-11.19	0.38	-0.63	6.56	6.13	10.56	-2.75	-1.75	-9.38	-9.63	-10.44	-1.21
PM Peak	-2.34	-3.44	5.30	4.44	7.81	0.13	-2.06	-8.00	-8.06	-7.50	2.61	2.34	-0.64	-1.39	-5.66	2.06	-0.38	-7.63	-8.56	-8.63	-1.98

Table F-3.12 Difference in Average Queue Length (#vehs/lane) at 66th Street & Park Blvd

Time Period	EB			NB			WB			SB		
	Left	Through	Right	Left	Through	Right	Left	Through	Right	Left	Through	Right
AM Peak	0.17	-2.61	-0.43	-1.59	-9.34	7.81	-0.36	1.08	1.58	-1.50	-8.38	-8.56
Off Peak	0.09	8.38	5.75	-2.99	-9.40	-11.19	-0.13	6.34	10.56	-2.25	-9.50	-10.44
PM Peak	-2.89	4.87	7.81	-0.97	-8.03	-7.50	2.48	-1.01	-5.66	0.84	-8.09	-8.63

Discussion

The following can be concluded from the comparison of queue lengths:

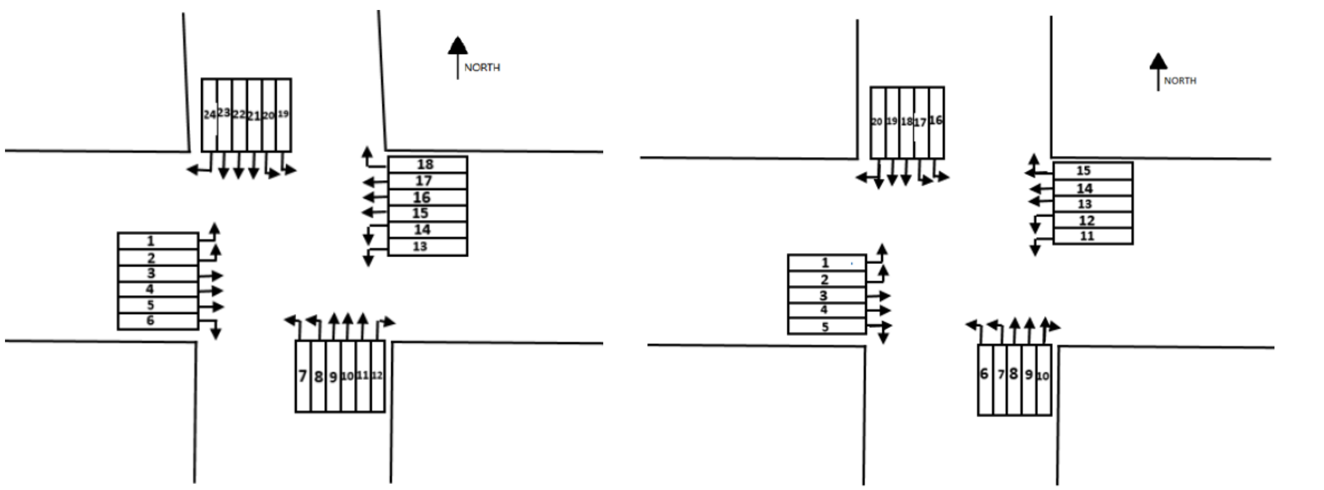
- The average queue length mostly improved for all approaches at the Ulmerton Rd intersection, especially for the EB approach where the reduction reaches almost 15 vehicles per lane. Through lanes on the WB approach have an increase in average queue length of 7 vehicles/lane. This is consistent with other implementations of InSync, where queues along the coordinated arterial are reduced while the queues on the side streets increase.
- The 66th Street and Park Blvd intersection average queue length mostly improved for the main approaches (NB and SB). The greatest improvement is of 11 vehicles/lane and it occurs for the NB through movement. However, there is an increase in queue lengths for both the EB and WB approaches (maximum increase is 11 vehicles for the WB approach.)
- On average, at the Ulmerton Rd intersection, the queue length decreased for the AM peak and Off peak periods (2.06 and 2.43 veh/lane, respectively) and increased for the PM peak (1.75 veh/lane). At the Park Blvd intersection, the average queue length decreased during all time periods (reductions of 8.56, 10.44 and 8.63 veh/lane during the AM, Off and PM peak, respectively.)

F.3.4 QUEUE TO LANE STORAGE RATIO

In addition to queue length, it is important to assess any impact to adjacent lanes or to upstream facilities. The queue to link/lane ratio is used to establish the likelihood of spillback, which is presented in this section by movement and by time period.

The following assumptions are used:

- The storage capacity is estimated as the maximum number of vehicles in the lane.
- The queue to link/lane storage ratio is estimated as 1 if the observer reported "spillback", and as 0.8 if reported as the maximum number of vehicles visible to the observer.
- Queue to lane storage ratios over 80% are highlighted in yellow, as they represent conditions with a high probability for spillback.



(a) SR 693 (66th Street) and SR 688 (Ulmerton Rd) (b) SR 693 (66th Street) and SR 694 (Park Boulevard)

Figure F-4.1 Lane Configuration of Critical Intersections

Before Study (Nov. 9 & Nov.10, 2016)

Table F-4.1 Average Queue Storage Ratio by Lane and by Period at 66th Street & Ulmerton Rd – Before Study.

Time period	Time segment	EB						NB						WB						SB					
		Left		Through			Right	Left		Through			Right	Left		Through			Right	Left		Through			Right
Lane Number		**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**	**13**	**14**	**15**	**16**	**17**	**18**	**19**	**20**	**21**	**22**	**23**	**24**
AM PEAK	**1**	1.00	1.00	1.00	1.00	1.00	0.00	0.35	0.40	0.32	0.34	0.31	0.00	0.44	0.50	0.58	0.61	0.44	0.00	0.75	0.81	0.60	0.71	0.44	0.10
	2	1.00	1.00	1.00	1.00	1.00	0.00	0.48	0.46	0.59	0.58	0.57	0.00	0.48	0.63	0.47	0.50	0.38	0.06	0.63	0.81	0.56	0.45	0.63	0.17
	3	1.00	1.00	1.00	1.00	1.00	0.00	0.31	0.48	0.69	0.67	0.61	0.00	0.54	0.70	0.42	0.55	0.38	0.13	0.63	0.59	0.84	0.76	0.94	0.13
	4	1.00	1.00	1.00	1.00	1.00	0.00	0.48	0.46	0.59	0.58	0.57	0.00	0.42	0.70	0.51	0.63	0.52	0.00	0.50	0.31	0.55	0.48	0.57	0.15
	Average	**1.00**	**1.00**	**1.00**	**1.00**	**1.00**	**0.00**	**0.41**	**0.45**	**0.55**	**0.54**	**0.51**	**0.00**	**0.47**	**0.63**	**0.50**	**0.57**	**0.43**	**0.05**	**0.63**	**0.63**	**0.64**	**0.60**	**0.65**	**0.14**
OFF PEAK	**1**	0.66	0.66	0.73	0.69	0.67	0.00	0.38	0.33	0.43	0.47	0.38	0.00	0.38	0.58	0.47	0.69	0.46	0.00	0.56	0.56	0.44	0.26	0.47	0.38
	2	0.66	0.67	0.82	0.82	0.83	0.00	0.65	0.71	0.49	0.44	0.43	0.25	0.54	0.68	0.31	0.28	0.27	0.38	0.06	0.19	0.53	0.45	0.51	0.42
	3	0.48	0.45	0.61	0.60	0.56	0.00	0.56	0.54	0.58	0.52	0.43	0.00	0.40	0.60	0.28	0.38	0.36	0.50	0.69	0.44	0.68	0.36	0.50	0.23
	4	0.23	0.24	0.63	0.64	0.65	0.00	0.44	0.46	0.59	0.59	0.57	0.00	0.42	0.58	0.44	0.42	0.27	0.00	0.56	0.38	0.56	0.28	0.35	0.37

Time period	Time segment	EB						NB						WB						SB					
		Left		Through			Right	Left		Through			Right	Left		Through			Right	Left		Through			Right
Lane Number		**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**	**13**	**14**	**15**	**16**	**17**	**18**	**19**	**20**	**21**	**22**	**23**	**24**
	Average	**0.51**	**0.51**	**0.70**	**0.69**	**0.68**	**0.00**	**0.51**	**0.51**	**0.52**	**0.51**	**0.45**	**0.06**	**0.43**	**0.61**	**0.38**	**0.44**	**0.34**	**0.22**	**0.47**	**0.39**	**0.55**	**0.34**	**0.46**	**0.35**
PM PEAK	**1**	0.41	0.44	0.61	0.66	0.70	0.42	0.56	0.65	0.73	0.72	0.73	0.40	0.56	0.70	0.43	0.50	0.39	0.31	0.56	0.38	0.32	0.27	0.38	0.10
	2	0.45	0.47	0.80	0.81	0.64	0.53	0.75	0.88	0.63	0.63	0.55	0.42	0.44	0.55	0.47	0.72	0.38	0.50	0.31	0.38	0.30	0.25	0.36	0.27
	3	0.51	0.47	0.59	0.51	0.55	0.53	0.79	0.79	0.81	0.78	0.78	0.35	0.59	0.59	0.55	0.63	0.43	0.25	0.34	0.16	0.82	0.69	0.65	0.33
	4	0.35	0.38	0.36	0.40	0.33	0.42	0.63	0.75	0.67	0.68	0.67	0.35	0.33	0.63	0.50	0.58	0.46	0.13	0.19	0.41	0.49	0.38	0.53	0.28
	Average	**0.43**	**0.44**	**0.59**	**0.59**	**0.55**	**0.47**	**0.68**	**0.77**	**0.71**	**0.70**	**0.68**	**0.38**	**0.48**	**0.62**	**0.49**	**0.61**	**0.42**	**0.30**	**0.35**	**0.33**	**0.48**	**0.40**	**0.48**	**0.24**

Table F-4.2 Number of Cycles with Spillback at 66th Street & Ulmerton Rd – Before Study.

Time period	**Time segment**	**# cycles in 15 min**	**EB**						**NB**						**WB**						**SB**					
			Left		**Through**			**Right**	**Left**		**Through**			**Right**	**Left**		**Through**			**Right**	**Left**		**Through**			**Right**
Lane Number			**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**	**13**	**14**	**15**	**16**	**17**	**18**	**19**	**20**	**21**	**22**	**23**	**24**
AM PEAK	**1**	4	4	4	4	4	4	0	0	0	0	0	0	0	0	0	0	0	0	0	1	2	0	0	0	0
	2	4	4	4	4	4	4	0	0	0	0	0	0	0	0	0	0	0	0	0	1	2	0	0	0	0
	3	4	4	4	4	4	4	0	0	0	1	1	1	0	0	1	0	0	0	0	0	0	1	0	3	0
	4	4	4	4	4	4	4	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
	Average		**4**	**4**	**4**	**4**	**4**	**0**	**0**	**0**	**0.25**	**0.25**	**0.25**	**0**	**0**	**0.25**	**0**	**0**	**0**	**0**	**0.5**	**1**	**0.25**	**0**	**0.75**	**0**
OFF PEAK	**1**	4	2	2	2	2	2	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
	2	4	1	1	2	2	2	0	0	1	0	0	0	1	1	0	0	0	0	1	0	0	0	0	0	1
	3	4	0	0	0	0	0	0	0	0	0	0	0	0	0	1	0	0	0	0	1	0	0	0	0	0
	4	4	0	0	0	1	0	0	0	0	1	1	1	0	0	0	0	0	0	0	1	0	0	0	0	0
	Average		**0.8**	**0.8**	**1**	**1.3**	**1**	**0**	**0**	**0.3**	**0.3**	**0.3**	**0.3**	**0.3**	**0.3**	**0.3**	**0**	**0**	**0**	**0.3**	**0.5**	**0**	**0**	**0**	**0**	**0.3**
PM PEAK	**1**	4	0	0	1	1	1	1	0	0	2	2	2	0	0	0	0	0	0	0	0	0	0	0	0	0
	2	4	0	0	3	3	2	0	1	0	1	1	1	0	0	0	0	1	0	0	0	0	0	0	0	0

Time period	Time segment	# cycles in 15 min	EB						NB						WB						SB					
			Left		Through			Right	Left		Through			Right	Left		Through			Right	Left		Through			Right
Lane Number			1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24
	3	4	0	0	1	1	1	0	0	0	3	3	3	0	0	0	0	0	0	0	0	0	1	1	0	0
	4	4	0	0	0	0	0	0	0	0	2	2	2	0	0	0	0	0	0	0	0	0	0	0	0	0
	Average		**0**	**0**	**1.25**	**1.25**	**1**	**0.25**	**0.25**	**0**	**2**	**2**	**2**	**0**	**0**	**0**	**0**	**0.25**	**0**	**0**	**0**	**0**	**0.25**	**0.25**	**0**	**0**

Table F-4.3 Average Queue Storage Ratio by Lane and by Period at 66th Street & Park Blvd – Before Study.

Time period	Time segment	EB					NB					WB					SB				
		Left		Through		T/R	Left		Through		T/R	Left		Through		T/R	Left		Through		T/R
Lane Number		1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20
AM PEAK	1	0.25	0.31	0.68	0.77	0.63	0.38	0.28	0.95	0.98	0.00	0.35	0.28	0.51	0.48	0.45	0.40	0.33	0.54	0.61	0.58
	2	0.22	0.29	0.78	0.81	0.76	0.48	0.40	0.94	0.93	0.00	0.87	0.83	0.79	0.33	0.51	0.21	0.35	0.89	0.93	0.98
	3	0.47	0.48	0.93	0.95	0.91	0.94	0.61	1.00	1.00	0.00	0.30	0.43	0.60	0.55	0.51	0.46	0.68	1.00	1.00	1.00
	4	0.53	0.35	0.74	0.73	0.70	0.35	0.28	0.75	0.83	0.06	0.32	0.38	0.48	0.40	0.44	0.46	0.49	0.84	0.89	0.81

Time period	Time segment	EB					NB					WB					SB				
		Left		Through		T/R	Left		Through		T/R	Left		Through		T/R	Left		Through		T/R
Lane Number		**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**	**13**	**14**	**15**	**16**	**17**	**18**	**19**	**20**
	Average	**0.37**	**0.36**	**0.78**	**0.82**	**0.75**	**0.54**	**0.39**	**0.91**	**0.93**	**0.02**	**0.46**	**0.48**	**0.59**	**0.44**	**0.48**	**0.38**	**0.46**	**0.82**	**0.86**	**0.84**
OFF PEAK	**1**	0.44	0.42	0.28	0.33	0.49	0.46	0.36	0.85	0.70	0.90	0.42	0.38	0.53	0.58	0.21	0.54	0.39	0.88	0.90	0.79
	2	0.41	0.42	0.32	0.30	0.63	0.60	0.69	0.58	0.44	0.78	0.42	0.38	0.36	0.39	0.14	0.77	0.49	0.88	0.89	0.93
	3	0.50	0.38	0.30	0.42	0.66	0.69	0.69	0.89	0.71	0.95	0.38	0.35	0.45	0.49	0.10	0.77	0.56	1.00	1.00	1.00
	4	0.53	0.40	0.33	0.36	0.63	0.67	0.84	0.98	0.91	0.95	0.47	0.44	0.48	0.48	0.24	0.62	0.36	1.00	1.00	1.00
	Average	**0.47**	**0.40**	**0.31**	**0.35**	**0.60**	**0.61**	**0.65**	**0.82**	**0.69**	**0.89**	**0.42**	**0.39**	**0.45**	**0.48**	**0.17**	**0.67**	**0.45**	**0.94**	**0.95**	**0.93**
PM PEAK	**1**	0.84	0.85	0.55	0.55	0.57	0.46	0.58	0.65	0.65	0.65	0.32	0.53	0.89	0.93	0.98	0.67	0.79	1.00	1.00	1.00
	2	0.88	0.79	0.35	0.35	0.35	0.50	0.57	0.94	0.94	0.94	0.32	0.28	0.74	0.79	0.91	0.58	0.43	0.94	1.00	0.96
	3	0.78	0.77	0.45	0.43	0.40	0.48	0.44	0.78	0.76	0.65	0.22	0.33	0.85	0.85	1.00	0.50	0.45	1.00	1.00	1.00
	4	0.72	0.65	0.45	0.47	0.49	0.58	0.53	0.49	0.51	0.51	0.33	0.29	0.81	0.86	0.88	0.52	0.38	0.90	0.98	0.94

Time period	Time segment	EB					NB					WB					SB				
		Left		Through		T/R	Left		Through		T/R	Left		Through		T/R	Left		Through		T/R
Lane Number		**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**	**13**	**14**	**15**	**16**	**17**	**18**	**19**	**20**
	Average	**0.80**	**0.77**	**0.45**	**0.45**	**0.45**	**0.50**	**0.53**	**0.71**	**0.72**	**0.69**	**0.30**	**0.36**	**0.82**	**0.86**	**0.94**	**0.57**	**0.51**	**0.96**	**0.99**	**0.98**

Table F-4.4 Number of Cycles with Spillback at 66th Street & Park Blvd – Before Study.

Time period	Time segment	# cycles in 15 min	EB					NB					WB					SB				
			Left		Through		T/R	Left		Through		T/R	Left		Through		T/R	Left		Through		T/R
Lane Number			**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**	**13**	**14**	**15**	**16**	**17**	**18**	**19**	**20**
AM PEAK	**1**	4	0	0	1	1	1	0	0	3	0	0	0	0	0	0	0	0	0	0	0	0
	2	4	0	0	0	0	0	0	0	3	3	0	1	1	10	0	0	0	0	2	2	3
	3	4	1	0	3	3	3	3	0	4	4	0	0	0	0	0	0	0	2	4	4	4
	4	4	0	0	0	0	0	0	0	2	2	0	0	0	0	0	0	0	0	2	2	1
	Average		**0.25**	**0**	**1**	**1**	**1**	**0.75**	**0**	**3**	**2.25**	**0**	**0.25**	**0.25**	**2.5**	**0**	**0**	**0**	**0.5**	**2**	**2**	**2**
OFF PEAK	**1**	4	0	0	0	0	0	0	0	2	2	2	0	0	0	0	0	0	0	3	3	1
	2	4	0	0	0	0	0	0	1	0	0	0	0	0	0	0	0	0	0	3	3	3
	3	4	0	0	0	0	0	0	0	1	0	3	0	0	0	0	0	0	0	4	4	4
	4	4	1	0	0	0	0	0	0	3	2	2	0	0	0	0	0	0	0	4	4	4
	Average		**0.3**	**0**	**0**	**0**	**0**	**0**	**0.3**	**1.5**	**1**	**1.8**	**0**	**0**	**0**	**0**	**0**	**0**	**0**	**3.5**	**3.5**	**3**

Time period	Time segment	# cycles in 15 min	EB					NB					WB					SB				
			Left		Through		T/R	Left		Through		T/R	Left			Through	T/R	Left		Through		T/R
Lane Number			1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20
PM PEAK	1	4	1	2	0	0	0	0	0	0	0	0	0	0	2	2	2	0	0	4	4	4
	2	4	0	0	0	0	0	0	0	3	3	3	0	0	1	2	2	0	0	3	3	3
	3	4	1	1	0	0	0	0	0	0	0	0	0	0	0	0	4	0	0	4	4	4
	4	4	0	0	0	0	0	0	0	0	0	0	0	0	1	3	0	0	0	3	3	3
	Average		**0.5**	**0.75**	**0**	**0**	**0**	**0**	**0**	**0.75**	**0.75**	**0.75**	**0**	**0**	**1**	**1.75**	**2**	**0**	**0**	**3.5**	**3.5**	**3.5**

After Study (May 16 & May 17, 2017)

Table F-4.5 Average Queue Storage Ratio by Lane and by Period at 66th Street & Ulmerton Rd – After Study

Time Period		EB						NB						WB						SB					
		Left		Through			Right	Left		Through			Right	Left		Through			Right	Left		Through			Right
Lane Number		1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24
AM Peak	1	0.36	0.35	0.86	0.88	0.86	0.13	0.50	0.40	0.49	0.48	0.40	0.08	0.30	0.34	0.43	0.53	0.28	0.00	0.45	0.30	0.55	0.58	0.73	0.00
	2	0.35	0.31	0.84	0.84	0.76	0.07	0.33	0.44	0.74	0.70	0.60	0.17	0.48	0.54	0.46	0.48	0.34	0.00	0.69	0.47	0.61	0.58	0.81	0.00
	3	0.31	0.28	0.65	0.67	0.68	0.36	0.40	0.48	0.61	0.55	0.48	0.13	0.31	0.58	0.54	0.63	0.39	0.00	0.55	0.55	0.52	0.42	0.53	0.00

Time Period		EB						NB						WB						SB					
		Left		Through			Right	Left		Through			Right	Left		Through			Right	Left		Through			Right
Lane Number		**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**	**13**	**14**	**15**	**16**	**17**	**18**	**19**	**20**	**21**	**22**	**23**	**24**
	4	0.30	0.32	0.67	0.67	0.70	0.31	0.37	0.49	0.55	0.54	0.40	0.13	0.29	0.43	0.50	0.59	0.44	0.00	0.56	0.41	0.58	0.39	0.60	0.00
	Average	**0.33**	**0.32**	**0.75**	**0.77**	**0.75**	**0.22**	**0.40**	**0.45**	**0.60**	**0.57**	**0.47**	**0.13**	**0.35**	**0.47**	**0.48**	**0.55**	**0.36**	**0.00**	**0.56**	**0.43**	**0.57**	**0.49**	**0.67**	**0.00**
Off Peak	**1**	0.22	0.25	0.35	0.39	0.46	0.22	0.50	0.35	0.23	0.23	0.13	0.05	0.53	0.66	0.71	0.85	0.65	0.65	0.63	0.41	0.46	0.40	0.57	0.22
	2	0.23	0.28	0.31	0.33	0.39	0.06	0.37	0.42	0.16	0.14	0.06	0.05	0.56	0.63	0.85	1.00	0.73	0.81	0.31	0.28	0.52	0.42	0.49	0.10
	3	0.31	0.36	0.45	0.47	0.48	0.33	0.46	0.54	0.34	0.25	0.23	0.08	0.33	0.35	0.86	1.00	0.75	0.63	0.50	0.38	0.45	0.41	0.53	0.25
	4	0.32	0.37	0.61	0.56	0.55	0.36	0.58	0.62	0.64	0.46	0.41	0.10	0.27	0.38	0.61	0.70	0.56	0.50	0.45	0.45	0.31	0.30	0.40	0.19
	Average	**0.27**	**0.32**	**0.43**	**0.44**	**0.47**	**0.24**	**0.48**	**0.48**	**0.34**	**0.27**	**0.21**	**0.07**	**0.43**	**0.50**	**0.76**	**0.90**	**0.67**	**0.65**	**0.47**	**0.38**	**0.43**	**0.38**	**0.50**	**0.19**
PM Peak	**1**	0.52	0.45	0.62	0.63	0.65	0.36	1.00	0.90	0.71	0.68	0.58	0.06	0.25	0.36	0.40	0.50	0.36	0.15	0.56	0.41	0.30	0.29	0.43	0.33
	2	0.48	0.58	0.73	0.77	0.83	0.31	0.62	0.66	0.86	0.75	0.66	0.12	0.42	0.60	0.67	0.78	0.63	0.69	0.25	0.28	0.57	0.55	0.72	0.35
	3	0.35	0.32	0.62	0.61	0.77	0.49	0.77	0.71	0.61	0.59	0.51	0.10	0.40	0.48	1.00	1.00	1.00	0.85	0.19	0.16	0.66	0.70	0.85	0.42

Time Period		EB						NB						WB						SB					
		Left		Through			Right	Left		Through			Right	Left		Through			Right	Left		Through			Right
Lane Number		1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24
	4	0.33	0.30	0.47	0.47	0.57	0.27	0.60	0.77	0.56	0.54	0.43	0.08	0.40	0.50	0.53	0.69	0.56	0.25	0.35	0.28	0.61	0.52	0.63	0.32
	Average	**0.42**	**0.41**	**0.61**	**0.62**	**0.71**	**0.35**	**0.76**	**0.76**	**0.69**	**0.64**	**0.54**	**0.09**	**0.37**	**0.49**	**0.72**	**0.85**	**0.66**	**0.48**	**0.34**	**0.28**	**0.53**	**0.51**	**0.66**	**0.35**

Table F-4.6 Number of Cycles with Spillback at 66th Street & Ulmerton Rd – After Study

Time Period	Time Segment	# Cycles in 15 mins	EB						NB						WB						SB					
			Left		Through			Right	Left		Through			Right	Left		Through			Right	Left		Through			Right
Lane Number			1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24
AM Peak	1	5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	0
	2	5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	0	0	0	1	0
	3	5	0	0	0	0	0	0	0	0	1	0	0	0	0	0	0	1	0	0	0	0	0	0	0	0
	4	5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	0	0	0	0	0
	Average		**0**	**0**	**0**	**0**	**0**	**0**	**0**	**0**	**0.25**	**0**	**0**	**0**	**0**	**0**	**0**	**0.25**	**0**	**0**	**0.5**	**0**	**0**	**0**	**0.5**	**0**
Off Peak	1	5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	0	2	2	0	0	0	0	0
	2	5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	2	1	2	0	0	0	0	0	0

Time Period	Time Segment	# Cycles in 15 mins	EB						NB						WB						SB					
			Left		Through			Right	Left		Through			Right	Left		Through			Right	Left		Through			Right
Lane Number			1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24
	3	4	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	2	1	1	1	0	0	0	0	0
	4	5	0	0	0	0	0	0	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
	Average		**0**	**0**	**0**	**0**	**0**	**0**	**0**	**0.25**	**0**	**0**	**0**	**0**	**0**	**0**	**1**	**1.25**	**0.5**	**1.25**	**0.75**	**0**	**0**	**0**	**0**	**0**
PM Peak	1	5	0	0	0	0	0	0	3	2	1	0	0	0	0	0	0	1	0	0	0	0	0	0	0	0
	2	5	0	0	0	0	0	0	0	0	2	1	0	0	0	0	0	1	0	1	0	0	0	0	1	0
	3	5	0	0	0	0	0	1	1	0	0	0	0	0	0	0	5	5	5	3	0	0	0	0	2	0
	4	5	0	0	0	0	0	0	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
	Average		**0**	**0**	**0**	**0**	**0**	**0.25**	**1**	**0.75**	**0.75**	**0.25**	**0**	**0**	**0**	**0**	**1.25**	**1.75**	**1.25**	**1**	**0**	**0**	**0**	**0**	**0.75**	**0**

Table F-4.7 Average Queue Storage Ratio by Lane and by Period at 66th Street & Park Blvd – After Study

Time Period		EB					NB					WB					SB				
		Left		Through		Right	Left		Through		Right	Left		Through		Right	Left		Through		Right
Lane Number		1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20
AM Peak	1	0.38	0.23	0.58	0.60	0.56	0.25	0.28	0.48	0.46	0.43	0.45	0.43	0.34	0.43	0.39	0.35	0.25	0.31	0.31	0.33

Time Period		EB					NB					WB					SB				
		Left		Through		Right	Left		Through		Right	Left		Through		Right	Left		Through		Right
Lane Number		**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**	**13**	**14**	**15**	**16**	**17**	**18**	**19**	**20**
	2	0.28	0.33	0.75	0.78	0.76	0.52	0.36	0.54	0.61	0.51	0.60	0.54	0.45	0.65	0.68	0.44	0.41	0.41	0.41	0.39
	3	0.44	0.54	0.66	0.67	0.78	0.56	0.40	0.40	0.39	0.30	0.31	0.44	0.23	0.48	0.44	0.48	0.29	0.41	0.40	0.36
	4	0.43	0.42	0.68	0.72	0.82	0.31	0.26	0.38	0.38	0.39	0.33	0.46	0.25	0.73	0.73	0.37	0.23	0.54	0.54	0.58
	Average	**0.38**	**0.38**	**0.67**	**0.69**	**0.73**	**0.41**	**0.33**	**0.45**	**0.46**	**0.41**	**0.42**	**0.47**	**0.32**	**0.57**	**0.56**	**0.41**	**0.29**	**0.42**	**0.42**	**0.41**
Off Peak	**1**	0.56	0.40	0.67	0.69	0.91	0.52	0.35	0.18	0.14	0.19	0.52	0.39	0.39	0.83	0.74	0.60	0.45	0.55	0.53	0.36
	2	0.47	0.33	0.56	0.56	0.74	0.46	0.33	0.27	0.30	0.40	0.48	0.38	0.36	0.71	0.58	0.40	0.31	0.45	0.49	0.40
	3	0.44	0.44	0.91	0.91	0.90	0.63	0.50	0.43	0.45	0.46	0.43	0.32	0.33	0.64	0.56	0.40	0.35	0.45	0.45	0.39
	4	0.47	0.46	0.68	0.70	0.90	0.60	0.44	0.29	0.25	0.29	0.35	0.32	0.26	0.98	0.93	0.44	0.34	0.43	0.40	0.48
	Average	**0.48**	**0.41**	**0.70**	**0.72**	**0.86**	**0.55**	**0.41**	**0.29**	**0.28**	**0.33**	**0.45**	**0.35**	**0.33**	**0.79**	**0.70**	**0.46**	**0.36**	**0.47**	**0.47**	**0.41**
PM Peak	**1**	0.43	0.33	0.83	0.77	0.91	0.52	0.38	0.35	0.45	0.48	0.41	0.48	0.31	0.91	0.83	0.65	0.44	0.46	0.43	0.40

Time Period		EB					NB					WB					SB				
		Left		Through		Right	Left		Through		Right	Left		Through		Right	Left		Through		Right
Lane Number		1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20
	2	0.53	0.46	0.63	0.66	0.86	0.60	0.65	0.35	0.31	0.23	0.50	0.49	0.38	0.76	0.66	0.63	0.46	0.40	0.39	0.36
	3	0.53	0.33	0.91	0.91	0.91	0.52	0.39	0.28	0.23	0.20	0.53	0.56	0.40	0.88	0.70	0.79	0.50	0.75	0.75	0.75
	4	0.56	0.79	0.41	0.26	0.55	0.42	0.34	0.28	0.26	0.35	0.43	0.43	0.33	0.60	0.44	0.83	0.56	0.70	0.70	0.66
	Average	**0.51**	**0.48**	**0.69**	**0.65**	**0.81**	**0.51**	**0.44**	**0.31**	**0.31**	**0.31**	**0.47**	**0.49**	**0.79**	**0.79**	**0.66**	**0.73**	**0.49**	**0.58**	**0.57**	**0.54**

Table F-4.8 Number of Cycles with Spillback at 66th Street & Park Blvd – After Study

Time Period	Time Segment	# Cycles in 15 mins	EB					NB					WB					SB				
			Left		Through		Right	Left		Through		Right	Left		Through		Right	Left		Through		Right
Lane Number			1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20
AM Peak	1	4	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
	2	4	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
	3	5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
	4	4	0	0	0	0	0	0	0	0	0	0	0	0	1	1	1	0	0	0	0	0

Time Period	Time Segment	# Cycles in 15 mins	EB					NB					WB					SB				
			Left		Through		Right	Left		Through		Right	Left		Through		Right	Left		Through		Right
Lane Number			**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**	**13**	**14**	**15**	**16**	**17**	**18**	**19**	**20**
	Average		**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.25**	**0.25**	**0.25**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**
Off Peak	**1**	**4**	0	0	0	0	0	0	0	0	0	0	0	0	1	1	1	0	0	0	0	0
	2	**4**	0	0	0	0	0	0	0	0	0	0	0	0	1	1	1	0	0	0	0	0
	3	**4**	0	0	0	0	0	0	0	0	0	0	0	0	1	1	1	0	0	0	0	0
	4	**4**	0	0	0	0	0	0	0	0	0	0	0	0	3	3	3	0	0	0	0	0
	Average		**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**1.50**	**1.50**	**1.50**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**
PM Peak	**1**	**5**	0	0	0	0	0	0	0	0	0	0	0	0	4	3	2	0	0	0	0	0
	2	**5**	0	0	0	0	0	0	0	0	0	0	0	0	1	1	0	0	0	0	0	0
	3	**4**	0	0	0	0	0	0	0	0	0	0	0	0	2	2	1	1	0	0	0	0
	4	**4**	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0	1	0	0	0	0
	Average		**0.00**	**0.25**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**1.75**	**1.50**	**0.75**	**0.50**	**0.00**	**0.00**	**0.00**	**0.00**

Comparisons of Before and After Queue Storage Ratios

The differences in Queue Storage Ratio between before and after measurements are shown in Table F-4.9 and F-4.10. The tables are color-coded as follows: green shows significant improvement, yellow shows modest change (either improvement or deterioration), and red shows significant deterioration in delay. Several gradations of each color are used to represent different variations within each classification.

Table F-4.9 Difference in Avg. Queue Storage Ratios at 66th Street & Ulmerton Rd

Time Period	EB						NB						WB						SB						Avg.
	Left		Through			Right	Left		Through			Right	Left		Through			Right	Left		Through			Right	
Lane Number	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	
AM Peak	-0.67	-0.68	-0.25	-0.23	-0.25	0.22	-0.01	0.00	-0.01	-0.04	-0.10	0.13	-0.12	-0.16	-0.01	-0.02	-0.07	-0.05	-0.06	-0.20	-0.07	-0.11	0.02	-0.14	-0.12
Off Peak	-0.26	-0.21	-0.32	-0.30	-0.26	0.24	-0.06	-0.07	-0.28	-0.33	-0.33	0.01	-0.01	-0.10	0.38	0.45	0.33	0.43	0.00	-0.01	-0.12	0.04	0.04	-0.16	-0.04
PM Peak	-0.01	-0.02	-0.01	0.00	0.07	-0.10	0.08	0.02	-0.09	-0.14	-0.22	-0.29	-0.12	-0.15	0.21	0.23	0.22	0.19	-0.02	-0.04	0.07	0.14	0.20	0.12	0.01

Table F-4.10 Difference in Avg. Queue Storage Ratios at 66th Street & Park Blvd

Time Period	EB					NB					WB					SB					Avg
	Left		Through		Right	Left		Through		Right	Left		Through		Right	Left		Through		Right	
Lane Number	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	
AM Peak	0.10	-0.15	0.12	0.14	0.73	-0.05	0.10	0.22	0.16	0.32	0.47	-0.33	-0.13	-0.11	-0.15	-0.31	0.41	0.01	-0.02	-0.64	0.04
Off Peak	0.20	-0.06	0.19	0.18	0.71	0.01	-0.11	0.07	0.10	0.32	0.16	0.01	0.46	0.18	0.03	-0.11	0.39	0.14	0.11	-0.36	0.13
PM Peak	0.23	0.01	0.18	0.11	0.72	0.04	-0.09	0.10	0.08	0.34	0.21	0.19	0.46	0.30	0.39	0.04	-0.14	0.22	0.00	-0.59	0.14

Discussion

The following are concluded from the comparison of queue storage ratios:

- Overall, the queue storage ratios have improved for the two critical intersections.
- The queue storage ratios for the EB left at Ulmerton Road significantly improved, particularly for the AM peak. However, the WB through and right movements have an increase in the queue storage ratios, especially for the Off-peak period.
- The Park Boulevard intersection shows moderate changes during all periods from all approaches, but there is a significant improvement on queue storage ratios for the SB right movement during all periods, and a deterioration for the EB right during all periods.
- There is reduction in the number of cycles with spillback at both critical intersections, particularly for Ulmerton Road at the EB approach and, for Park Boulevard at the main approaches (NB and SB) throughout the day. Since the Insync adaptive system does not use traditional cycle lengths, the approaches with spillbacks or longer queues could be served on multiple occasions.

F.3.5 EQUIVALENT PCE FLOWS

Traffic flows were counted manually and converted to equivalent PCE flows (pce/hour) by considering the percentage of heavy vehicles. It was assumed that the PCE for trucks is 2.

Truck Percentage Observations

Table F-5.1 Truck Percentages at 66th Street & Ulmerton Rd.

Period	Interval	NB			SB			EB			WB			Ave.
		Left	Thru	Right	Left	Thru	Right	Left	Thru	Right	Left	Thru	Right	
AM Peak	1	4.41%	5.00%	6.56%	0.00%	0.43%	5.83%	7.02%	3.87%	1.79%	7.69%	5.53%	18.18%	5.53%
	2	8.62%	3.35%	1.89%	0.00%	1.23%	3.85%	2.50%	3.28%	3.03%	0.00%	4.14%	0.00%	2.66%
	3	12.00%	6.06%	2.63%	0.00%	2.13%	0.00%	5.19%	5.96%	1.67%	2.33%	6.37%	11.11%	4.62%
	4	2.90%	4.91%	0.00%	5.13%	0.99%	2.88%	0.00%	5.64%	13.04%	4.35%	4.86%	6.25%	4.25%
	Average	**6.98%**	**4.83%**	**2.77%**	**1.28%**	**1.19%**	**3.14%**	**3.68%**	**4.69%**	**4.88%**	**3.59%**	**5.23%**	**8.89%**	**4.26%**
Off Peak	1	4.48%	2.03%	7.69%	5.26%	3.60%	4.05%	1.49%	8.31%	1.72%	6.25%	5.74%	3.85%	4.54%
	2	1.30%	2.82%	0.00%	0.00%	1.95%	3.75%	9.09%	8.75%	1.79%	1.72%	7.25%	5.26%	3.64%
	3	7.02%	3.87%	8.33%	5.00%	3.42%	2.99%	7.32%	9.47%	5.71%	0.00%	5.13%	0.00%	4.85%
	4	3.33%	3.39%	10.42%	4.00%	3.05%	8.47%	2.70%	6.82%	3.85%	3.45%	4.70%	5.88%	5.01%
	Average	**4.03%**	**3.03%**	**6.61%**	**3.57%**	**3.00%**	**4.82%**	**5.15%**	**8.34%**	**3.27%**	**2.86%**	**5.71%**	**3.75%**	**4.51%**
PM Peak	1	4.35%	2.22%	5.56%	0.00%	0.00%	4.65%	0.94%	3.57%	1.41%	1.92%	2.67%	0.00%	2.27%
	2	1.19%	1.22%	0.00%	0.00%	1.12%	4.21%	1.64%	2.47%	1.28%	2.94%	1.70%	0.00%	1.48%

Period	Interval	NB			SB			EB			WB			Ave.
		Left	Thru	Right	Left	Thru	Right	Left	Thru	Right	Left	Thru	Right	
	3	0.00%	0.40%	2.27%	0.00%	1.97%	0.00%	0.00%	2.18%	0.00%	4.41%	2.11%	0.00%	1.11%
	4	3.03%	0.98%	0.00%	0.00%	1.63%	0.00%	0.84%	2.58%	0.00%	2.08%	1.61%	2.33%	1.26%
	Average	**2.14%**	**1.21%**	**1.96%**	**0.00%**	**1.18%**	**2.22%**	**0.86%**	**2.70%**	**0.67%**	**2.84%**	**2.02%**	**0.58%**	**1.53%**

Table F-5.2 Truck Percentages at 66th Street & Park Blvd.

Period	Interval	NB			SB			EB			WB			Ave.
		Left	Thru	Right	Left	Thru	Right	Left	Thru	Right	Left	Thru	Right	
AM Peak	1	0.00%	0.93%	0.00%	3.64%	0.00%	2.70%	0.00%	1.78%	8.11%	2.50%	6.22%	4.00%	2.49%
	2	0.00%	1.09%	3.85%	2.82%	4.93%	5.13%	0.00%	2.58%	6.98%	3.92%	3.21%	6.25%	3.40%
	3	2.44%	1.78%	1.75%	4.29%	3.05%	3.23%	2.17%	1.16%	7.14%	2.04%	3.77%	2.63%	2.95%
	4	0.00%	3.30%	6.25%	2.74%	2.67%	6.67%	7.14%	1.09%	7.02%	1.85%	6.85%	2.78%	4.03%
	Average	**0.61%**	**1.77%**	**2.96%**	**3.37%**	**2.66%**	**4.43%**	**2.33%**	**1.65%**	**7.31%**	**2.58%**	**5.01%**	**3.91%**	**3.22%**
Off Peak	1	0.00%	3.55%	2.78%	2.86%	0.00%	0.00%	1.75%	2.76%	4.17%	1.59%	2.73%	2.63%	2.07%
	2	1.69%	2.70%	13.04%	3.57%	2.29%	2.00%	1.75%	5.41%	0.00%	0.00%	1.03%	2.63%	3.01%
	3	0.00%	0.00%	4.65%	0.00%	0.00%	0.00%	3.92%	1.35%	1.82%	0.00%	3.10%	1.61%	1.37%
	4	3.41%	2.13%	4.00%	1.65%	3.21%	0.00%	2.33%	5.26%	0.00%	4.92%	0.78%	6.67%	2.86%
	Average	**1.28%**	**2.10%**	**6.12%**	**2.02%**	**1.37%**	**0.50%**	**2.44%**	**3.69%**	**1.50%**	**1.63%**	**1.91%**	**3.39%**	**2.33%**

Period	Interval	NB			SB			EB			WB			Ave.
		Left	Thru	Right	Left	Thru	Right	Left	Thru	Right	Left	Thru	Right	
PM Peak	1	1.18%	2.42%	4.65%	0.00%	2.33%	0.00%	1.69%	3.63%	0.00%	0.00%	1.53%	16.13%	2.80%
	2	1.56%	0.53%	0.00%	4.00%	0.92%	0.00%	1.69%	3.33%	0.00%	1.37%	1.11%	3.85%	1.53%
	3	0.00%	1.38%	2.13%	0.00%	0.43%	0.00%	0.00%	1.58%	1.43%	0.00%	0.68%	2.44%	0.84%
	4	3.30%	1.86%	4.88%	0.00%	1.16%	0.00%	0.00%	1.86%	1.35%	0.00%	0.23%	4.88%	1.63%
	Average	**1.51%**	**1.55%**	**2.91%**	**1.00%**	**1.21%**	**0.00%**	**0.85%**	**2.60%**	**0.69%**	**0.34%**	**0.89%**	**6.82%**	**1.70%**

Before Study (Nov. 9 & Nov. 10, 2016)

Table F-5.3 PCE Flow Rates (pce/15 min and pce/hour) at 66th Street & Ulmerton Rd.

Time Period (15 min)		NB			SB			EB			WB		
		Left	Thru	Right	Left	Thru	Right	Left	Thru	Right	Left	Thru	Right
AM Peak	1	71	168	65	38	234	109	61	403	57	42	267	13
	2	63	216	54	37	247	108	82	409	68	67	352	8
	3	28	210	39	15	144	66	81	391	61	44	217	10
	4	71	171	64	41	204	107	125	356	78	48	259	17
	Flow Rate	**233**	**765**	**222**	**131**	**829**	**390**	**349**	**1559**	**264**	**201**	**1095**	**48**
Off Peak	1	70	151	42	20	144	77	68	352	59	51	221	27

Time Period (15 min)		NB			SB			EB			WB		
		Left	Thru	Right	Left	Thru	Right	Left	Thru	Right	Left	Thru	Right
	2	78	146	33	23	157	83	60	323	57	59	281	20
	3	61	161	39	21	151	69	88	370	74	53	287	30
	4	62	183	53	26	169	64	76	407	54	30	334	18
	Flow Rate	**271**	**641**	**167**	**90**	**621**	**293**	**292**	**1452**	**244**	**193**	**1123**	**95**
PM Peak	1	72	230	57	30	173	90	107	319	72	53	385	25
	2	85	248	52	20	181	99	124	374	79	70	419	38
	3	91	251	90	21	207	55	105	375	68	71	532	25
	4	102	207	62	24	187	61	120	278	46	49	378	44
	Flow Rate	**350**	**936**	**261**	**95**	**748**	**305**	**456**	**1346**	**265**	**243**	**1714**	**132**

Table F-5.4 PCE Flow Rates (pce/15 min and pce/hour) at 66th Street & Park Blvd.

Time Period (15 min)		**NB**			**SB**			**EB**			**WB**		
		Left	**Thru**	**Right**	**Left**	**Thru**	**Right**	**Left**	**Thru**	**Right**	**Left**	**Thru**	**Right**
AM Peak	1	62	218	38	57	177	38	41	400	40	41	222	26
	2	71	278	54	73	319	41	33	437	46	53	257	34
	3	42	229	58	73	270	32	47	350	45	50	220	39
	4	64	282	51	75	308	16	45	371	61	55	234	37
	Flow Rate	**239**	**1007**	**201**	**278**	**1074**	**127**	**166**	**1558**	**192**	**199**	**933**	**136**
Off Peak	1	75	175	37	72	184	28	58	298	75	64	263	39
	2	60	190	52	87	179	51	58	312	58	26	293	39
	3	62	182	45	101	188	36	53	301	56	44	333	63
	4	91	240	26	123	225	31	44	200	68	64	389	32
	Flow Rate	**288**	**787**	**160**	**383**	**776**	**146**	**213**	**1111**	**257**	**198**	**1278**	**173**
PM Peak	1	86	212	45	69	220	27	60	343	79	71	398	36
	2	65	190	39	104	220	42	60	372	75	74	454	27
	3	69	220	48	83	235	34	74	385	71	79	447	42
	4	94	219	43	54	261	18	57	383	75	93	434	43
	Flow Rate	**314**	**841**	**175**	**310**	**936**	**121**	**251**	**1483**	**300**	**317**	**1733**	**148**

The traffic volumes in Eastbound and Westbound movements are highest during all study time periods (AM Peak, Off Peak and PM Peak) at 66th Street & Ulmerton Rd (Table F-5.3). This is because the major street is Ulmerton Rd which connects to I-275 in the East and the minor street is 66th Street. Similarly the traffic volumes in Eastbound and Westbound movements are higher during the Off Peak and PM Peak at 66th Street & Park Blvd (Table F-5.4). This is because the major street is Park Blvd which connects to I-275 and SR-92 in the East and the minor street is 66th Street. During AM Peak at Park Blvd, NB and SB volumes are more than the WB volume.

Truck Percentage Observations After Study (May 16 & May 17, 2017)

Table F-5.5 Truck Percentages at 66th Street & Ulmerton Rd – After Study

Period	Interval	EB			NB			WB			SB			Ave.
		Left	Thru	Right	Left	Thru	Right	Left	Thru	Right	Left	Thru	Right	
AM Peak	1	1.79%	0.68%	2.27%	0.00%	1.76%	2.94%	13.79%	8.59%	0.00%	0.00%	2.09%	8.86%	3.56%
	2	0.00%	2.82%	1.41%	2.60%	4.37%	10.00%	10.17%	6.21%	0.00%	8.00%	2.80%	13.33%	5.14%
	3	0.00%	1.37%	1.54%	2.27%	9.66%	10.64%	7.69%	9.48%	10.00%	8.00%	3.85%	0.00%	5.37%
	4	0.00%	1.32%	1.61%	4.00%	11.72%	23.40%	4.65%	10.00%	6.67%	7.69%	3.91%	5.68%	6.72%
	Avg	**0.45%**	**1.55%**	**1.71%**	**2.22%**	**6.88%**	**11.75%**	**9.08%**	**8.57%**	**4.17%**	**5.92%**	**3.16%**	**6.97%**	**5.20%**
Off Peak	1	0.00%	3.15%	4.05%	0.00%	1.85%	0.00%	3.41%	4.33%	0.00%	3.13%	2.05%	5.08%	2.26%
	2	0.63%	2.24%	3.49%	1.20%	1.37%	4.55%	4.35%	5.99%	6.67%	3.57%	3.80%	8.82%	3.89%
	3	0.96%	2.73%	0.00%	0.00%	0.61%	5.41%	2.17%	3.64%	0.00%	2.86%	1.93%	9.43%	2.48%
	4	0.00%	2.55%	3.13%	0.00%	1.46%	3.45%	3.13%	4.12%	25.00%	0.00%	0.96%	17.78%	5.13%
	Avg	**0.40%**	**2.67%**	**2.67%**	**0.30%**	**1.32%**	**3.35%**	**3.26%**	**4.52%**	**7.92%**	**2.39%**	**2.19%**	**10.28%**	**3.44%**
	1	0.71%	1.79%	0.00%	0.00%	1.29%	0.00%	20.00%	2.81%	5.26%	4.76%	5.75%	13.33%	4.64%

Period	Interval	EB			NB			WB			SB			Ave.
		Left	Thru	Right	Left	Thru	Right	Left	Thru	Right	Left	Thru	Right	
PM Peak	2	0.00%	0.00%	0.00%	0.00%	3.45%	2.63%	0.00%	0.00%	0.00%	0.00%	1.98%	5.48%	1.13%
	3	0.00%	0.00%	0.00%	1.27%	0.00%	0.00%	3.39%	3.28%	2.17%	5.26%	4.48%	4.17%	2.00%
	4	0.00%	0.00%	0.00%	0.00%	4.94%	0.00%	0.00%	2.01%	0.00%	0.00%	1.98%	6.14%	1.26%
	Avg	**0.18%**	**0.45%**	**0.00%**	**0.32%**	**2.42%**	**0.66%**	**5.85%**	**2.03%**	**1.86%**	**2.51%**	**3.55%**	**7.28%**	**2.26%**

Table F-5.6 Truck Percentages at 66th Street & Park Blvd – After Study.

Period	Interval	EB			NB			WB			SB			Ave.
		Left	Thru	Right	Left	Thru	Right	Left	Thru	Right	Left	Thru	Right	
AM Peak	1	0.00%	1.52%	2.27%	1.89%	0.94%	3.57%	9.80%	8.59%	0.00%	10.81%	7.55%	5.56%	4.38%
	2	0.00%	0.60%	0.00%	0.00%	1.13%	0.00%	6.00%	6.21%	0.00%	4.44%	5.49%	10.00%	2.82%
	3	0.00%	1.24%	1.39%	2.56%	0.33%	1.22%	8.11%	9.48%	10.00%	7.84%	3.91%	0.00%	3.84%
	4	0.00%	0.36%	5.93%	4.00%	0.00%	2.94%	7.41%	10.00%	6.67%	3.39%	4.72%	11.76%	4.77%
	Avg	**0.00%**	**0.93%**	**2.40%**	**2.11%**	**0.60%**	**1.93%**	**7.83%**	**8.57%**	**4.17%**	**6.62%**	**5.42%**	**6.83%**	**3.95%**
Off Peak	1	0.00%	0.92%	3.45%	0.87%	1.63%	1.18%	13.33%	2.53%	6.67%	7.55%	6.64%	4.88%	4.14%
	2	24.92%	2.70%	1.17%	0.87%	0.55%	1.17%	1.79%	2.02%	9.38%	8.70%	4.75%	8.33%	5.53%
	3	0.00%	0.93%	0.00%	0.00%	0.55%	0.00%	5.80%	2.69%	5.45%	4.62%	2.38%	1.75%	2.01%
	4	0.00%	0.84%	1.08%	0.00%	0.00%	0.00%	0.00%	2.51%	2.08%	9.59%	2.47%	0.00%	1.55%

Period	Interval	EB			NB			WB			SB			Ave.
		Left	Thru	Right	Left	Thru	Right	Left	Thru	Right	Left	Thru	Right	
	Avg	**6.23%**	**1.35%**	**1.42%**	**0.43%**	**0.68%**	**0.59%**	**5.23%**	**2.44%**	**5.89%**	**7.61%**	**4.06%**	**3.74%**	**3.31%**
PM Peak	1	2.60%	0.45%	0.00%	0.00%	0.32%	2.44%	6.38%	2.61%	11.11%	9.23%	1.22%	3.85%	3.35%
	2	0.00%	0.00%	0.00%	0.00%	0.58%	0.79%	0.00%	0.00%	0.00%	12.70%	2.54%	3.64%	1.69%
	3	0.00%	0.58%	0.00%	0.00%	0.00%	1.92%	8.22%	4.89%	0.00%	5.08%	3.72%	7.14%	2.63%
	4	0.00%	0.00%	0.00%	3.03%	0.80%	1.49%	2.99%	1.81%	0.00%	4.11%	5.74%	0.00%	1.66%
	Avg	**0.65%**	**0.26%**	**0.00%**	**0.76%**	**0.43%**	**1.66%**	**4.40%**	**2.33%**	**2.78%**	**7.78%**	**3.30%**	**3.66%**	**2.33%**

PCE Flow Rates After Study (May 16 & May 17, 2017)

Table F-5.7 PCE Flow Rates (pce/15 min and pce/hour) at 66th Street & Ulmerton Rd – After Study.

Time Period		**EB**			**NB**			**WB**			**SB**		
		Left	**Thru**	**Right**	**Left**	**Thru**	**Right**	**Left**	**Thru**	**Right**	**Left**	**Thru**	**Right**
AM Peak	1	57	296	45	47	173	35	33	215	9	19	195	86
	2	64	328	72	79	191	33	65	325	9	27	220	102
	3	48	222	66	45	227	52	42	231	22	27	162	75
	4	97	308	63	26	143	58	45	242	16	28	186	93
	Flow Rate	**266**	**1154**	**246**	**197**	**734**	**178**	**185**	**1013**	**56**	**101**	**763**	**356**
Off Peak	1	60	262	77	49	110	16	91	289	19	33	149	62
	2	160	501	89	84	371	115	48	301	16	29	191	37
	3	105	263	16	36	166	78	47	313	14	36	211	58
	4	113	241	33	34	209	60	33	354	10	51	211	53
	Flow Rate	**438**	**1267**	**215**	**203**	**856**	**269**	**219**	**1257**	**59**	**149**	**762**	**210**
PM Peak	1	142	397	67	80	157	46	42	366	20	44	184	51
	2	111	375	149	65	150	39	47	402	28	16	206	77
	3	46	392	117	80	169	37	61	504	47	20	303	100
	4	84	188	45	57	85	47	69	709	51	38	257	121
	Flow Rate	**383**	**1352**	**378**	**282**	**561**	**169**	**219**	**1981**	**146**	**118**	**950**	**349**

Table F-5.8 PCE Flow Rates (pce/15 min and pce/hour) at 66th Street & Park Blvd – After Study

Time Period		**EB**			**NB**			**WB**			**SB**		
		Left	**Thru**	**Right**	**Left**	**Thru**	**Right**	**Left**	**Thru**	**Right**	**Left**	**Thru**	**Right**
AM Peak	1	30	200	45	54	214	29	56	215	9	41	171	38
	2	97	334	64	20	268	74	53	325	9	47	288	44
	3	83	245	73	40	301	83	40	231	22	55	239	33
	4	72	277	125	26	132	35	58	242	16	61	244	38
	Flow Rate	**282**	**1056**	**307**	**140**	**915**	**221**	**207**	**1013**	**56**	**204**	**942**	**153**
Off Peak	1	6	219	90	116	187	86	68	445	48	114	305	43
	2	10	228	86	117	184	86	57	405	35	75	397	39
	3	6	218	84	114	182	84	73	382	58	68	431	58
	4	7	239	94	125	198	92	44	327	49	80	374	59
	Flow Rate	**29**	**904**	**354**	**471**	**751**	**348**	**242**	**1559**	**190**	**337**	**1507**	**199**
PM Peak	1	79	224	68	68	314	126	50	275	30	71	249	27
	2	67	191	55	16	346	127	69	334	38	71	242	57
	3	123	345	112	60	308	106	79	322	19	62	307	60
	4	96	191	51	34	376	68	69	338	25	76	258	40
	Flow Rate	**365**	**951**	**286**	**178**	**1344**	**427**	**267**	**1269**	**112**	**280**	**1056**	**184**

Comparisons of Before and After Flows

The differences in traffic flow between the before and after measurements are shown in Table F-5.9 and Table F-5.10. The tables are color-coded as follows: green shows significant decrease, yellow shows modest change (either improvement or deterioration), and red shows significant deterioration in delay. Several gradations of each color are used to represent different variations within each classification.

Table F-5.9 Difference in Traffic Flow Rate (pce/hr) at 66th Street & Ulmerton Rd

Period	EB			NB			WB			SB		
	Left	Thru	Right	Left	Thru	Right	Left	Thru	Right	Left	Thru	Right
AM Peak	-83	-405	-18	-36	-31	-44	-16	-82	8	-30	-66	-34
Off Peak	146	-185	-29	-68	215	102	26	134	-36	59	141	-83
PM Peak	-73	6	113	-68	-375	-92	-24	267	14	23	202	44

Table F-5.10 Difference in Traffic Flow Rate (pce/hr) at 66th Street & Park Blvd

Period	EB			NB			WB			SB		
	Left	Thru	Right	Left	Thru	Right	Left	Thru	Right	Left	Thru	Right
AM Peak	116	-502	115	-99	-92	20	8	80	-80	-74	-132	26
Off Peak	-184	-207	97	183	-36	188	44	281	17	-46	731	53
PM Peak	114	-532	-14	-136	503	252	-50	-464	-36	-30	120	63

Discussion

The following are concluded from the comparison of traffic flows:

- At Ulmerton Rd, volumes during the after study were found to be lower during the AM peak for all the approaches, which may be due to seasonal variation. During the Off-peak, the volumes are higher on all approaches with the exception of the EB approach. During the PM peak the volumes decrease for the NB and increase for the other approaches.
- At Park Boulevard, the EB through movement has a reduction in volumes across all time periods, while the WB has a reduction during the PM peak. The NB has a significant increase in volume during the PM peak and the SB during the Off peak. This might be due to the prioritization of North-South movements by the adaptive system.

F.3.6 CONSIDERATION OF TRAFFIC FLOWS JOINTLY WITH QUEUE LENGTH

The differences in “Traffic Flow” and “Queue Length” between before and after measurements are shown in Table F-6.1 and Table F-6.3. The tables are color-coded as follows: green indicates that “Queue” decreases and “Traffic Flow” increases, red indicates “Queue” increases and “Traffic Flow” decreases, no color indicates “Traffic Flow” and “Queue” increase or decrease at the same time. Generally, green indicates improvement despite an increase in flow.

Table F-6.1 Differences in Traffic Flow (TF) and Queue Length (Q) at 66th Street & Ulmerton Rd

Time Period	EB						NB						WB						SB					
	Left		Thru		Right		Left		Thru		Right		Left		Thru		Right		Left		Thru		Right	
	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q
AM Peak	-83	-14.89	-405	-5.33	-18	1.94	-36	-0.03	-31	-1.00	-44	1.53	-16	-1.54	-82	-0.65	8	-0.19	-30	-0.93	-66	-1.34	-34	-2.06
Off peak	146	-5.13	-185	-6.47	-29	2.18	-68	-0.83	215	-6.30	102	0.10	26	-0.56	134	7.02	-36	1.71	59	-0.04	141	-0.39	-83	-2.43
PM Peak	-73	-0.41	6	0.46	113	-0.88	-68	0.63	-375	-3.01	-92	-3.46	-24	-1.48	267	3.96	14	0.75	23	-0.21	202	2.89	44	1.75

Table F-6.2 Difference (%) in Traffic Flow (TF) and Queue Length (Q) at 66th Street & Ulmerton Rd

Time Period	EB						NB						WB						SB					
	Left		Thru		Right		Left		Thru		Right		Left		Thru		Right		Left		Thru		Right	
	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q
AM Peak	-36%	-68%	-53%	-24%	-8%	N/A	-27%	-1%	-4%	-8%	-11%	N/A	-5%	-26%	-5%	-7%	3%	N/A	-15%	-23%	-6%	-10%	-71%	-103%
Off Peak	54%	-43%	-29%	-40%	-17%	N/A	-76%	-12%	35%	-53%	35%	10%	9%	-9%	9%	100%	-15%	171%	31%	-1%	13%	-4%	-87%	-49%
PM Peak	-21%	-4%	1%	3%	43%	-22%	-72%	7%	-50%	-20%	-30%	-69%	-5%	-25%	20%	40%	5%	75%	9%	-11%	12%	32%	33%	44%

Table F-6.3 Differences in Traffic Flow (TF) and Queue Length (Q) at 66th Street & Park Blvd

Time Period	EB						NB						WB						SB					
	Left		Thru		Right		Left		Thru		Right		Left		Thru		Right		Left		Thru		Right	
	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q
AM Peak	116	0.17	-502	-2.61	115	-0.43	-99	-1.59	-92	-9.34	20	7.81	8	-0.36	80	1.08	-80	1.58	-74	-1.50	-132	-8.38	26	-8.56
Off-peak	-184	0.09	-207	8.38	97	5.75	183	-2.99	-36	-9.40	188	-11.19	44	-0.13	281	6.34	17	10.56	-46	-2.25	731	-9.50	53	-10.44

Time Period	EB						NB						WB						SB					
	Left		Thru		Right		Left		Thru		Right		Left		Thru		Right		Left		Thru		Right	
	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q
PM Peak	114	-2.89	-532	4.87	-14	7.81	-136	-0.97	503	-8.03	252	-7.50	-50	2.48	-464	-1.01	-36	-5.66	-30	0.84	120	-8.09	63	-8.63

Table F-6.4 Difference (%) in Traffic Flow (TF) and Queue Length (Q) at 66th Street & Park Blvd

Time Period	EB						NB						WB						SB					
	Left		Thru		Right		Left		Thru		Right		Left		Thru		Right		Left		Thru		Right	
	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q
AM Peak	49%	4%	-50%	-17%	57%	-7%	-36%	-20%	-9%	-78%	16%	N/A	5%	-5%	5%	12%	-42%	53%	-37%	-21%	-14%	-56%	19%	-143%
Off-peak	-64%	2%	-26%	105%	61%	115%	48%	-27%	-5%	-67%	129%	-186%	21%	-2%	25%	91%	7%	1056%	-23%	-25%	57%	-56%	31%	-209%
PM Peak	36%	-36%	-63%	54%	-8%	195%	-44%	-11%	54%	-62%	208%	-188%	-20%	50%	-31%	-7%	-12%	-81%	-9%	9%	7%	-45%	43%	-216%

Discussion

The following can be concluded from the comparison of traffic flows jointly with queue length:

- At Ulmerton Rd, all approaches, except a few right turn movements, showed either a moderate improvement or remained as before.
- At Park Blvd. conditions either improved or remained the same for all approaches except for the EB. The SB during Off peak and the NB during PM peak showed the highest percentage improvements. This can be due to the adaptive system prioritizing the North-South direction.

BEFORE AND AFTER-IMPLEMENTATION STUDIES OF

ADVANCED SIGNAL CONTROL TECHNOLOGIES IN FLORIDA

The main objective of this project is to evaluate the implementation of proposed adaptive signal control technology (ASCT) traffic operations at several arterial corridors in Florida, before and after the installation of the ASCT, document the advantages and the disadvantages of different approaches and implementations, and provide recommendations for statewide implementation of ASCT.

This questionnaire will allow us to access the appropriateness and effectiveness of the ASCT implementation at each subject corridor by: conducting benefit cost analyses, comparing staffing requirements before and after the deployment, comparing the resource needs in terms of hardware, software and personnel, pinpointing any institutional issues and generally evaluating the ASCT implementation and maintenance requirements by incorporating the agency's perspective.

F.4 QUESTIONNAIRE

SECTION 1: RESPONDENT'S INFORMATION

Name: Norman Jester

Organization: Pinellas County Public Works Transportation

Position: Signal System Supervisor

Address: 22211 U.S. Highway 19/Bldg. 10 Clearwater, Florida 33765

Phone (727)464-8908

Fax (727)4648858

E-mail njester@pinellascounty.org

SECTION 2: PREVIOUS TRAFFIC CONTROL TECHNOLOGY

1. Please specify the type of traffic control used before Adaptive Signal Control Technology (ASCT) was installed for your site. (Please check all that apply.)

a. Fixed time coordinated control b. Actuated coordinated control

c. Fixed-time isolated control d. Actuated isolated control

e. Other, please specify: ____

2. What type of traffic controller was used before ASCT was implemented?

a. NEMA TS-1 b. NEMA TS-2

c. 170 d. 2070

e. Other, please specify: ____

14. How many detectors were used before ASCT was implemented for a typical intersection (e.g. *Newberry Rd & NW 75th St*)? Please also specify the location where the detectors were typically placed for each specific detector type (video detection, inductive loops, radars, etc.).

Loops or video, at every intersection and every approach. On main street, they were placed 25-30 feet back while at minor roads they were located at the stop bar.

15. What was the frequency of retiming the signal timing plan for the corridor before ASCT was implemented?

It was retimed about two to three years before the study, with minor adjustments when necessary.

16. How often were maintenance and updates performed to your previous control system in terms of the following components?

a. Detection: As needed
b. Control Hardware: As needed
c. Sofware: whenever there are updates

TS1-TS2: 1 year before the year before ASCT.

17. What was the annual maintenance cost in terms of hardware, software and personnel of the previous control system for the whole corridor where the ASCT is now deployed?

Will look into it.

18. What was the life expectancy of your traffic control before the deployment of ASCT? (Please refer to the hardware and software.)

Ten years for control hardware. For the coordination system, the same technology is still in use. The estimated life time is 10 years.

19. How many crashes of each category were reported during the past 4 years before the ASCT implementation? (2011-2014)

	K (Killed)	A (Disabling Injury)	B (Evident Injury)	C (Possible Injury)	O (No Apparent Injury)
2011					
2012					
2013					
2014					

SECTION 3: ADAPTIVE TRAFFIC CONTROL SYSTEM (ASCT)

9. Which Adaptive Traffic Control System (ASCT) does your agency deploy?

j. InSync; Version:1.7 ___________________________

k. Synchro Green; Version: ____________________

l. Other; please specify: _______________________

28. Did you consider any other ASCT before you selected this one for installation? Why did you reject the other(s) in favor of this one?

Yes, considerer using InSync, OPAC and RHODES at the same time.

29. What were your major criteria for choosing the ASCT for the selected corridor?

InSync was considered simpler, while OPAC and RHODES had hardware/support issues.

30. What is the life expectancy of your ASCT system? (Consider both hardware and software.)

10 years.

31. What type of traffic controller is currently in use after the ASCT implementation?

a. Same as before the ASCT implementation XX
b. New, please specify type: ____
c. Same, but the following updates were needed:
____

32. How many and what type of detectors are used after the ASCT implementation on an intersection level (e.g. *Newberry Rd & NW 75th St*)? Please note any updates that the previous detection system needed so as to work with your ASCT.

All video detection (cameras) changed with InSync.

33. Was there a need to update your previous software operating system at your Transportation Management Center in order to get your current ASCT software working?

a. Yes XX b. No

31. How often do you plan to perform physical maintenance or updates on your new ASCT in terms of the following components?

a. Detection: As needed____
b. ASCT Hardware: As needed ____
c. Software: As needed____

32. Was there a need for training your personnel in order to operate the new ASCT?

a. Yes, Num. of employees trained _3_, Hours of training per employee _a week_ b. No

Many employees were already trained, and are still learning about the system.

33. Was there any change on the number of staff required to operate/maintain the effective signal ASCT operations? (Comparison of staff needed before and after the deployment.)

There was a change – 1 person was added.

34. Where did expertise come from for your personnel's training, the ASCT installation, operation and the system's maintenance? Please check all that apply.

	In-House	Vendor	Contractor
Employee Training	Currently manages this	Initial Training	
ASCT Installation	Finished installation	Started the Process	
ASCT Operation	90%	10%	
ASCT Maintenance	All done in house		

35. How many crashes of each category were reported after ASCT was implemented?

K (Killed)	A (Disabling Injury)	B (Evident Injury)	C (Possible Injury)	O (No Apparent Injury)

SECTION 4: COST COMPONENTS

45. What is the overall capital cost of purchasing the ASCT (in terms of the software and licenses of the ASCT) for the corridor? If the purchase includes any service of implementation, personnel training or maintenance, please also specify here.

US$ 40,000 per intersection that covers hardware/equipment, two years of support and engineering behind it. Does not include the installation, which is done in house.

46. What is the implementation cost (considering the installation on site, any updates in software and hardware, design needs and contract hours) for the ASCT corridor? Please specify if this cost is partially or totally included in previous cost items.

Will look into it.

47. What was total cost for training your personnel to operate and manage the new ASCT? Please specify if this cost is partially or totally included in previous cost items.

Will look into it.

48. What is the expected maintenance cost (in terms of hardware, software and personnel) for the ASCT corridor on a yearly basis? Please specify if this cost is partially or totally included in previous cost items.

____

____

Will look into it.

49. Can you specify the costs for the following ASCT components? (will look into it)

a. Firmware $_____________ b. Software $______________

c. Equipment $____________ d. Maintenance of Traffic Cost $______________

e. Design Needs $___________ f. Contract Help/Agency Hours Cost $_________

SECTION 5: INSTITUTIONAL ISSUES

50. Were there any institutional issues that you had to overcome while implementing and operating the ASCT project? Please categorize and explain those issues below.

a. Organization and Management Institutional Issues:

The learning curve of different signal operation systems.

b. Regulatory and Legal Institutional Issues:

NA

c. Human and Facility Resources Institutional Issues:

Public perception, since InSync operates differently from what people were used to before.

d. Financial Institutional Issues:

NA

51. Which component (e.g. detection, communication, hardware, software, licensing or other) of your ASCT deployment is the most challenging to maintain and why?

The hardware components related to detection, because it is video based and maintenance intensive.

52. What happens to your ASCT operations when some of your detectors fail on the corridor level?

j. ASCT triggers an alarm and notifies operators

k. ASCT switches to an "off-line" mode by implementing Time-Of-Day plans

l. Combination of the above. d. Other (please describe): historic data time splits, using the period of previous months.

29. How is your ASCT performance evaluated?

l. In-house XX

m. By an independent evaluator, please describe: _UFTI study and previous consultancy

n. Not applicable—there is no evaluation

30. If the corridor on which the ASCT operates experiences over saturation, how would you rate the operation of the ASCT in response to these traffic conditions?

p. ASCT prevents or eliminates oversaturation: helps prevent oversaturation to an extent.

q. ASCT eliminates or reduces the extent of the periods of oversaturation: handles better than a TOD plan.

r. ASCT adversely affects the traffic conditions during periods of oversaturation

s. Other:

t. Oversaturation is very rare on the corridors operated by our ASCT

31. Based on your opinion and the up-to-date operation, when are ASCT operations proven to be the most effective?

a. Peak periods b. Off-peak periods XX

c. Shoulders of peak periods d. Other, please specify: _______________

32. Has the level of performance of ASCT been sustained since its installation?

a. Yes XX b. No; why not? ____

39. What was the public reaction to the ASCT operation? Have you received any complaints about long delays and queues?

Mostly complains on delays. Drivers on side streets complain about the cycle length.

40. How satisfied are you, in general, with your ASCT deployment?

a. Very satisfied XX b. Somewhat satisfied

c. Neutral d. Somewhat dissatisfied

e. Not satisfied at all

39. Are there any other costs or benefits related to ASCT deployment that you would like to report?

a.Benefits: The flexibility

b.Costs: NA

40. Would you consider expanding the ASCT program to any other corridors? Why or why not? If yes, would you use the same technology/firmware or have any suggestions for other systems? If so, why?

Yes, because the ones that are in place show good performance. They are implementing new ones in the moment. InSync will be kept. Centracs may be tried.

SECTION 6: SAFETY ISSUES

37. Do you have any anecdotal / qualitative data on safety benefits / disbenefits of the signal improvement along this corridor?

____

____

38. Has there been other changes along this corridor over the analysis period that could affect the safety of this corridor either positively or negatively?

____

____

39. Are you aware of any changes to the crash data reporting procedures over the analysis period? If yes, what are the changes?

____

____

____

____

40. What are your overall reactions to the trends presented in the report?

____

____

41. Is there another comparable corridor in which signal improvements have not been made to which the results of this corridor can be compared to?

____

____

42. Other thoughts / comments for us?

____

Thank you for your help in completing this survey. Your responses will help us with evaluating the deployment and benefits of your current ASCT.

If you have any questions regarding the survey, please contact Marian Ankomah, email: moankomah@ufl.edu or Tyler Valila, email: tvalila67@ufl.edu who work under the direct supervision of the faculty advisor Dr. Lily Elefteriadou.

F.5 BENEFIT COST ANALYSIS

Table F-7.1 Monetized Saving

Monetized Saving	
Time saving	$4,652,123.70
Fuel Consumption saving	$665,446.95
Air Pollution Saving	$132,495.44
Safety Saving	$23,316,234.28
Total Saving without Safety	$5,450,066.08
Total Saving	$28,766,300.37
Total Cost	$394,452.00
B/C Ratio without safety	13.81680429
B/C Ratio	72.92725191

Table F-7.2 Monetized Saving with No Emissions

Monetized Saving	
Time saving	$4,652,123.70
Fuel Consumption saving	$665,446.95
Safety Saving	$23,316,234.28
Total Saving without Safety	$5,317,570.64
Total Saving	$28,633,804.93
Total Cost	$394,452.00
B/C Ratio without safety	13.48090679
B/C Ratio	72.5913544

APPENDIX G: SUMMARY OF SR 70, MANATEE COUNTY

G.1 EXECUTIVE SUMMARY

The objective of this research is to evaluate the implementation of proposed Adaptive Signal Control Technologies (ASCT) traffic operations at several arterial corridors in Florida before and after the installation of specific ASCT, document the advantages and disadvantages of different approaches and implementations, and provide recommendations for state-wide implementation of ASCT. This appendix summarizes the before and after field data collected along the SR 70, Manatee County corridor from 5th St W to Lakewood Ranch Rd.

Two data collection methods were employed to collect the desired information. Floating car runs were conducted with the UFTI instrumented vehicle to obtain vehicle travel times before and after the implementation of SynchroGreen, during three time periods (AM Peak (7-9 AM), Off Peak (10AM-Noon) and PM Peak (4-6 PM)). The floating car method involves driving with the flow of traffic, while the driver passes as many vehicles as pass the driver to obtain an average speed. In addition, turning movement counts and queue lengths were collected at two critical intersections (US 301 and Lockwood Ridge Rd). Based on these, five performance measures were obtained for the before and after study periods: Link/Route Travel Time, Delay at Intersections, Queue Length (at critical intersections), Queue to Lane Storage Ratio (at critical intersections), and Passenger Car Equivalent (PCE) flows (at critical intersections).

The following were observed:

- While travel time reduces in the WB (-4.8%), it increases in the EB (5.8%)
- Most of the increase in travel time was due to increasing delay at the intersections east of I-75, possibly due to detection issues at 87th St E.
- Upon further investigation, it was observed that the SynchroGreen prioritizes operations along US 301, which intersects the SR 70 corridor and carries relatively higher traffic volume.
- WB through volumes at both intersections decreased significantly. This may be due to seasonal variation and the opening of a new roadway parallel to SR 70.
- The decrease in WB volumes had different effects at the two critical intersections. The US 301 intersection had high side street demands and showed significant increase in WB queues and drop in SB queues. This could be due to SynchroGreen prioritizing the high demand (NB and SB) movements. The Lockwood Ridge Rd. intersection had low side street demands and had a drop in the WB queues.
- Though there is an overall increase in EB travel time (5.8%), the US 301 shows significant improvement in travel time (SB: -20.9%; NB: -17.2%) as well as queues (SB: -3.2; NB: -2.3).

G.2 CORRIDOR INFORMATION

Figure G-1 provides a schematic of the SR 70, Manatee County Corridor (West-East). Table G-1 lists the intersections along the corridor. The adjacent land uses include malls, restaurants, hotels, gas stations, drug stores, automotive service shops and banks. Two intersections (US 301 and Lockwood Ridge Rd, denoted in bold within Table G-1) were selected as the critical intersections along the corridor and detailed turning movement and queue counts were collected at these. Figure G-2 provides the lane configuration of these two critical intersections. Due to high volumes on US 301, which intersects SR-70 in the middle of the corridor, the US 301/51st Ave intersection, north of US301/SR70 was included in the SynchroGreen implementation.

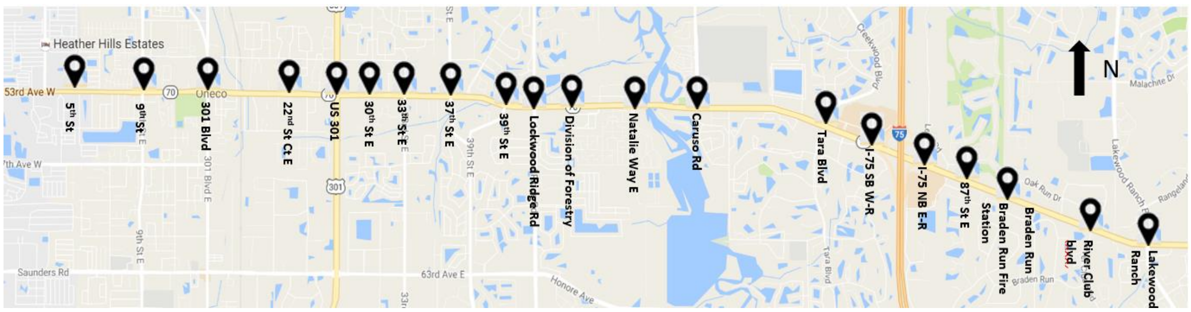
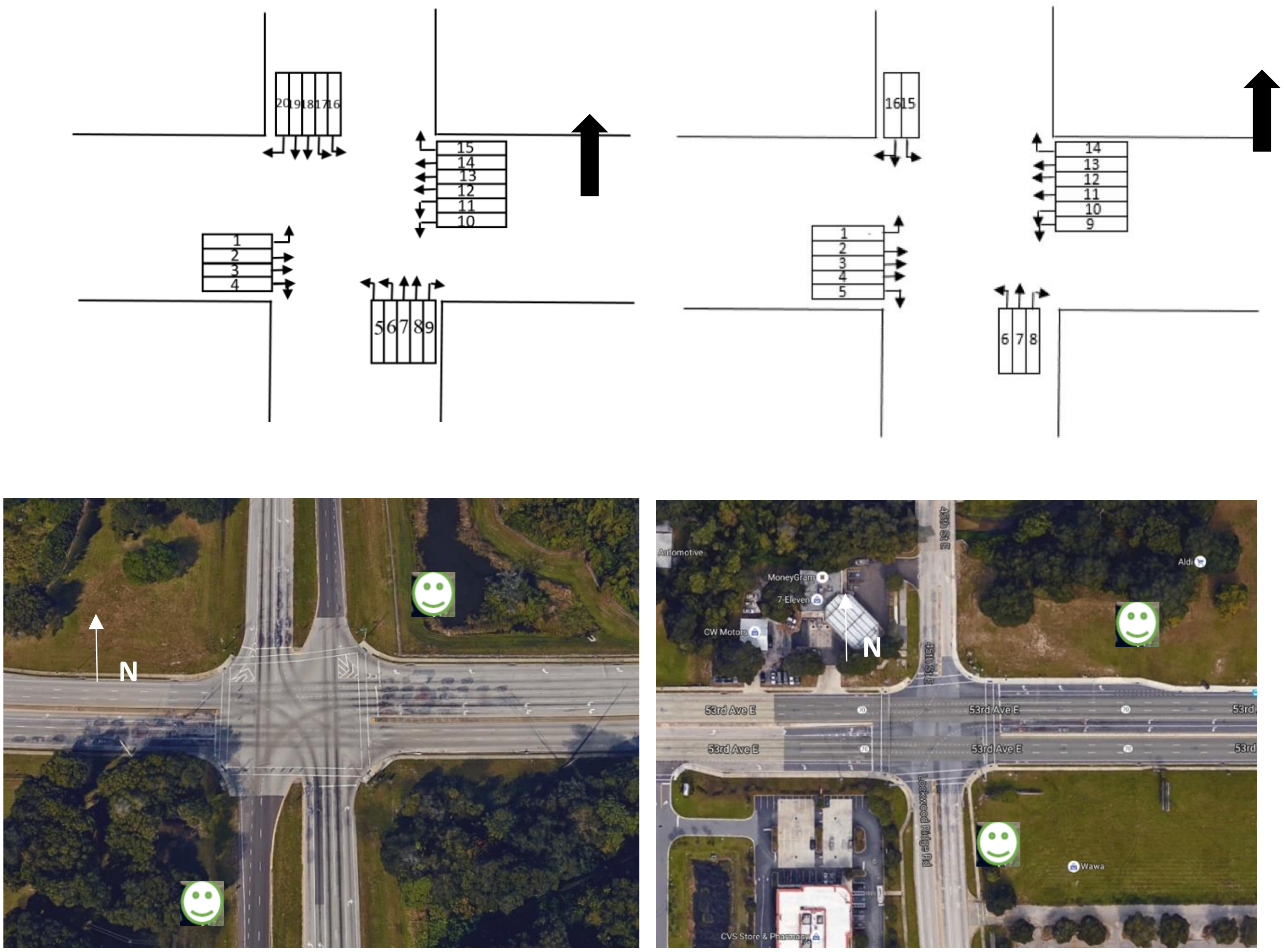


Figure G-1. Schematic of SR 70, Manatee County Corridor

Table G-1 List of Intersections along SR 70 Corridor

	Intersection	No. of Unsignalized Intersections in Between	Distance (Miles)
1	5th St W	-	-
2	9th St E	3	0.76
3	**301 Blvd E**	1	0.5
4	22nd St Ct E.	4	0.65
5	US 301.	1	0.36
6	30th St E	0	0.25
7	33rd St E	0	0.28
8	37th St E	0	0.35
9	39th St E	0	0.44
10	**Lockwood Ridge Rd**	0	0.23
11	Division of Forestry (Emergency Signal)	1	0.29
12	Natalie Way E	0	0.48
13	Caruso Rd	0	0.36
14	Tara Blvd	2	1.01
15	I-75 SB W-R	1	0.45
16	I-75 NB E-R	0	0.29
17	87th St E	1	0.41
18	Braden Run Fire Station (Emergency Signal)	0	0.27
19	Braden Run	0	0.08
20	Forest Run E/River Club Blvd	0	0.68

	Intersection	No. of Unsignalized Intersections in Between	Distance (Miles)
21	Lakewood Ranch Rd	1	0.44

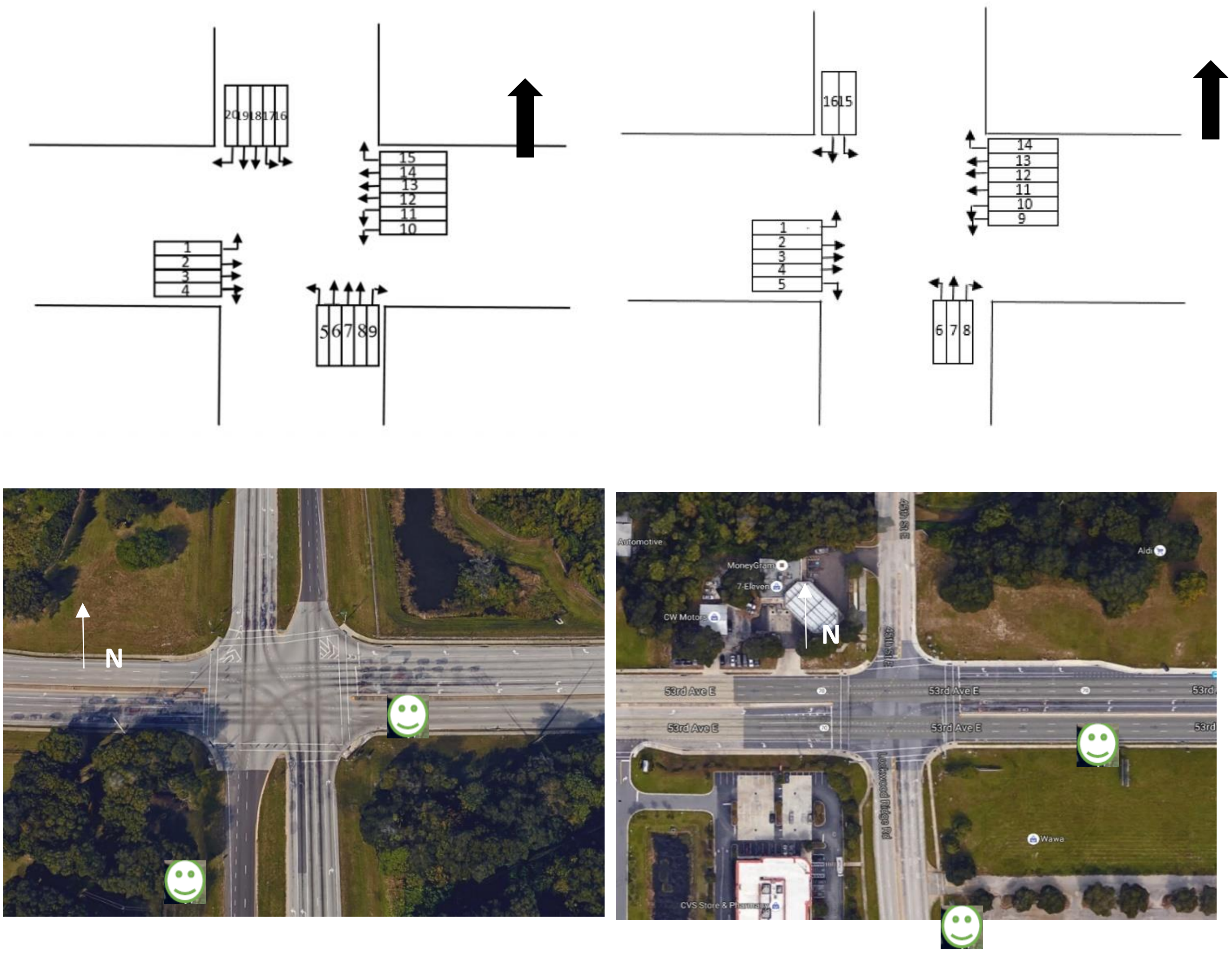


(a) US 301. & 53rd Ave (SR 70). (b) Lockwood Ridge Rd & 53rd Ave (SR 70)

Figure G-2 Schematic and Overview of Critical Intersections

(The ‘smiley’ faces are observer locations) – Before Study

G.3 PERFORMANCE MEASURES



(a) US 301. & 53rd Ave (SR 70).

(b) Lockwood Ridge Rd & 53rd Ave (SR 70)

Figure G-3 Schematic and Overview of Critical Intersections

(The ‘smiley’ faces are observer locations) – After Study

G.3 PERFORMANCE MEASURES

Six performance measures are evaluated: Link/Route Travel Time, Delay at Intersections, Number of Stops, Queue Length (critical intersections), Queue-to-Lane Storage Ratio (critical intersections), and PCE Flows (critical intersections). For each performance measure, a comparison between the before and after data is conducted and the results of the differences ("after data" – "before data") are presented.

G.3.1 ROUTE TRAVEL TIME

The average travel time (min) along SR 70 is measured using a floating car. During the before data collection period, we obtained data for 4 runs for the AM Peak, 6 runs for the OFF Peak and 4 runs for the PM Peak. During the after data collection period, we were able to perform a total of 4 runs for the AM Peak, 5 runs for the OFF Peak, and 3 runs for the PM Peak. During the second day of the after-data collection period, we amassed data for 3 runs in the AM Peak, 6 runs for the OFF Peak, and 4 runs for the PM Peak. Tables G-1.1 and G-1.2 provide the route travel time for the before and after data. The travel time comparison is presented in Table G-4.

Before Study SR 70 (Dec. 8 & Dec. 9, 2016)

Table G-1.1 *Route Travel Time (min)*

Route TT (min)	AM Peak	Off Peak	PM Peak	Average
SR 70, EB	21.11	15.74	19.72	18.85
SR 70, WB	20.40	15.70	21.41	19.17

After Study SR 70 (Oct. 3 & Oct. 4, 2018)

Table G-1.2 *Route Travel Time (min)*

Route TT (min)	AM Peak	Off Peak	PM Peak	Average
SR 70, EB	21.48	16.76	21.61	19.95
SR 70, WB	18.53	15.57	20.66	18.26

Comparison of Before and After Travel Time SR 70

Table G-1.3 *Change in Amount and Percentage of Route Travel Time (After – Before)*

Route TT (min)	AM Peak	Off Peak	PM Peak	Average
SR 70, EB	0.38 (1.8%)	1.02 (6.50%)	1.89 (9.6%)	1.10 (5.8%)
SR 70, WB	-1.87 (-9.2%)	-0.12 (-0.8%)	-0.75 (-3.5%)	-0.91 (-4.8%)

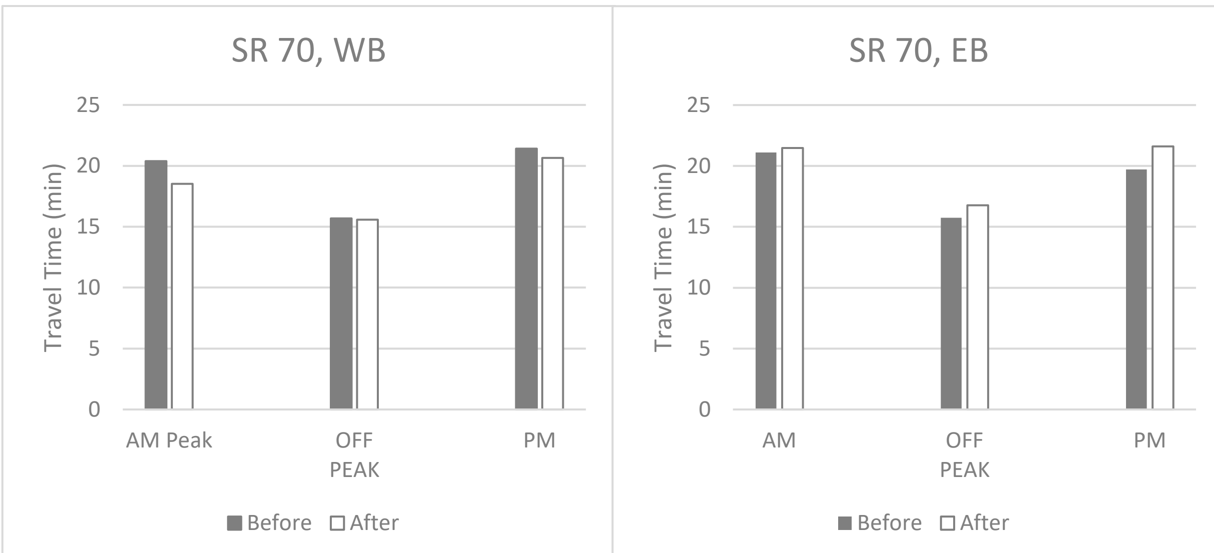


Figure G-1.1 Travel Times along SR 70, Manatee County

Discussion of SR 70

The following can be concluded from the comparison of travel times:

- Overall, there was a reduction in travel time for the WB direction (-4.8%) and an increase in travel time for the EB direction (5.8%) during all time periods.
- The greatest increase in travel time occurs during the PM Peak for the EB direction (9.6% or 1.89 min.)
- The greatest reduction in travel time occurs during the AM Peak for the WB direction (-9.2% or 1.87 min.)
- The SR 70 corridor intersects with US 301, which is a part of Synchrogreen implementation (US 301/ 51st Ave) carries relatively higher traffic volume. Hence to get a better understanding, travel times along US 301 were obtained from different data

sources (floating car and BlueTOAD) and compared. The results of this comparison are shown below.

US 301 Travel Time

The before travel time along US 301 was measured using a floating car by the local TMC in March 2016 and the after travel time was obtained from BlueTOAD for October 2018. The before data included 6 runs during each time period, and the after data is an average of Tuesdays, Wednesdays, and Thursdays during the month of October 2018. The lengths of the sections used in the two data sources were different. Hence, the shorter section was used for the comparison. The US 301 route of 0.813 miles is shown in Figure G-. Table G-1.4 and Table G-1.5 provide the route travel time for the before and after data. The travel time comparison is presented in Table G-1.6.

Figure G-1.2 Control Points for US 301 Travel Time

Before Study US 301 (Mar. 17, 2016)

Table G-1.4 *Route Travel Time (min)*

Route TT (min)	AM	PM	Average
US 301, SB	2.32	1.58	1.95
US 301 NB	1.90	1.92	1.91

After Study US 301 (Tues/Wed/Thu: October 1, 2018 to October 31, 2018)

Table G-1.5 *Route Travel Time (min)*

Route TT (min)	AM	PM	Average
US 301, SB	1.61	1.47	1.54
US 301, NB	1.40	1.76	1.58

Comparison of Before and After Travel Times along US 301

Table G-1.6 *Change in Percentage of Route Travel Time (After – Before)*

Route TT (min)	AM	PM	Average
US 301, SB	-0.70 (-30.2 %)	-0.11 (-7.0%)	**-0.41 (-20.9%)**
US 301, NB	-0.50 (-26.1%)	-0.16 (-8.4%)	**-0.33 (-17.2%)**

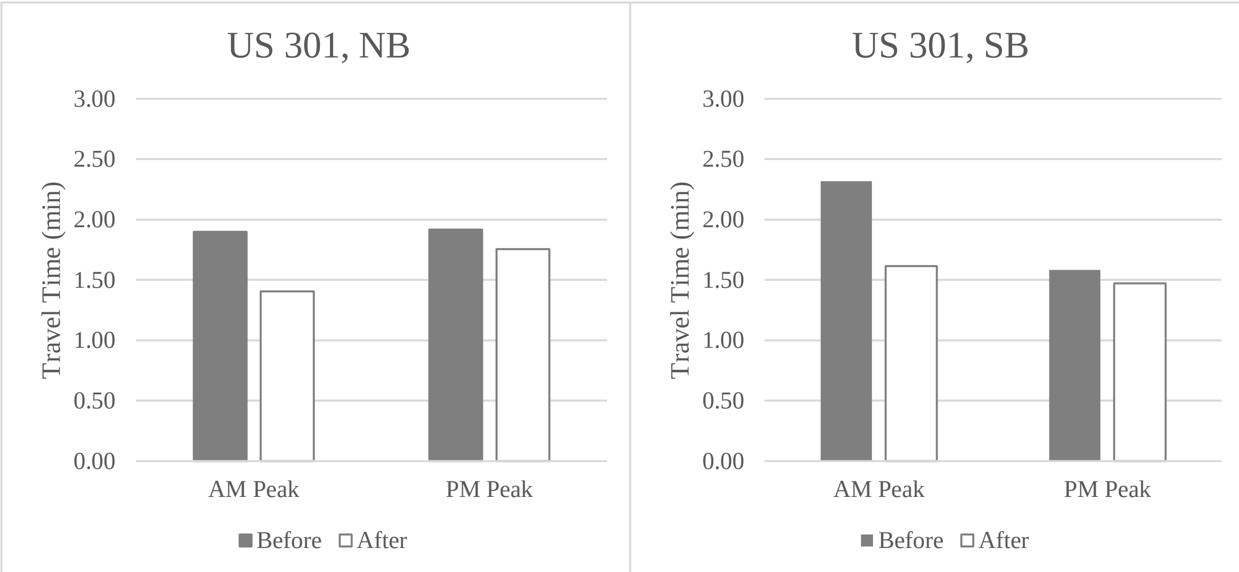


Figure G-1.3 Travel Times along US 301

Discussion of US 301

The following can be concluded from the comparison of travel times along US 301:

- Overall, there was a significant decrease in travel time for both directions (-20.9% for the SB, and -17.2% for the NB) and during both time periods.
- US 301 carries relatively higher volumes compared to SR 70 and SynchroGreen could be prioritizing US 301 over SR 70. This issue is discussed further in Section 6, considering both flows and queues at the SR70/US301 intersection.

G.3.2 DELAY

Delay (sec) at each intersection along the corridor was also obtained using the floating car measurements.

Before Study (Dec. 8 & Dec. 9, 2016)

Table G-2.1 Delay (s) for each Intersection Through Movement Along the EB Direction – Before Study

Delay at Intersections (sec)	AM Peak	Off Peak	PM Peak
5th St W	0.00	0.00	0.33
9th St E	11.33	2.57	51.33
301 Blvd E	3.83	20.57	24.33
22nd St Ct E.	14.50	0.00	0.00
US 301	**57.33**	**12.43**	**48.33**
30th St E	12.00	6.57	10.83
33rd St E	4.40	0.00	8.67
37th St E	0.00	4.00	2.83
39th St E	13.40	5.57	25.75
Lockwood Ridge Rd	**75.25**	**23.50**	**36.25**
Division of Forestry (Emergency)	0.00	0.00	0.00
Natalie Way E	0.00	0.00	10.50
Caruso Rd	27.75	23.83	9.75
Tara Blvd	4.75	39.50	29.00
I-75 SB W-R	2.75	0.00	3.50
I-75 NB E-R	0.00	0.00	0.00
87th St E	0.00	2.00	8.50
Braden Run Fire Station (Emergency)	0.00	0.00	0.00
Braden Run	0.00	0.00	6.75
Forest Run E/River Club Blvd	20.25	0.00	0.00
Lakewood Ranch Rd	34.00	4.67	0.00

Figure G-2.1 graphs the delay at each intersection for the through movement in the EB direction. The intersection delay for the two critical sections – US 301 and Lockwood Ridge Rd are the highest during all study time periods (AM Peak, Off Peak and PM Peak) in the EB direction. At other intersections, the delay is significantly less. A few intersections had zero delay for all the runs conducted during the floating car runs.

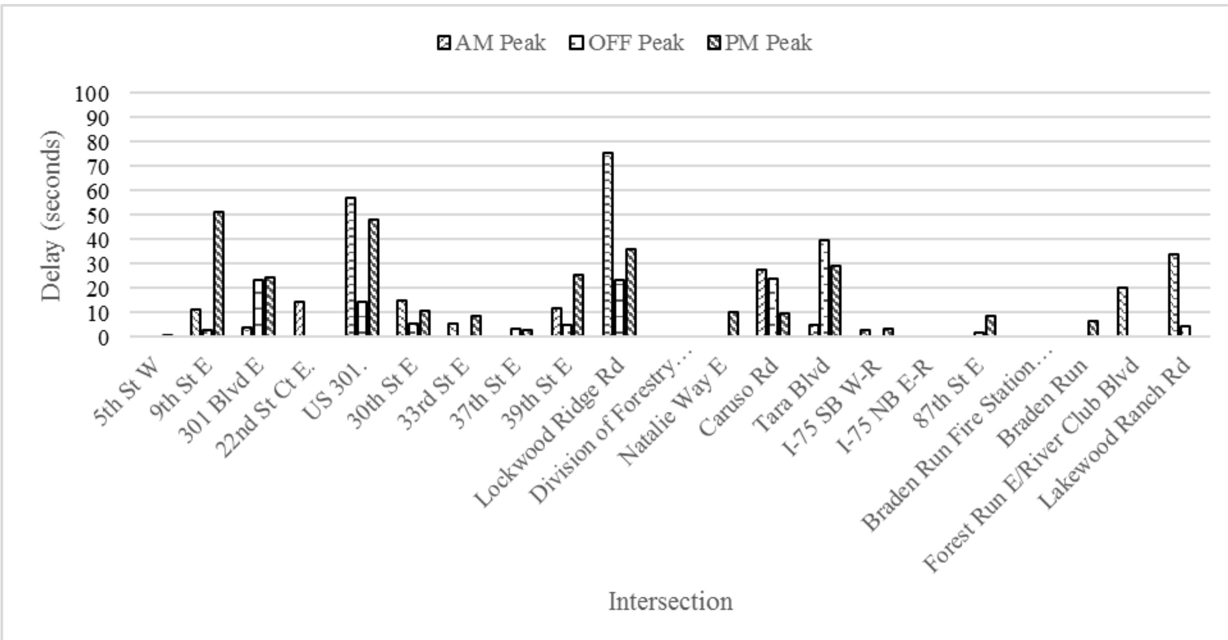


Figure G-2.1 Delay (s) for each Intersection Through Movement Along the EB Direction – Before Study

Table G-2.2 and Figure G-2.2 show the delays obtained by intersection for the WB. Figure G-2.2 shows the delay at each intersection for the through movement in the WB direction. The intersection delay in the WB direction is highest at Lakewood Ranch Rd during all time periods. The Off-Peak delays are less than either AM Peak or PM Peak delays at most intersections.

Table G-2.2 Delay (s) for each Intersection Through Movement Along the WB Direction – Before Study

Delay at Intersections (sec)	AM Peak	Off Peak	PM Peak
Lakewood Ranch Rd	80.50	58.14	65.25
Forest Run E/River Club Blvd	15.75	0.00	0.00
Braden Run	6.25	0.71	0.00
Braden Run Fire Station (Emergency Signal)	0.00	0.00	0.00
87th St E	1.50	0.00	7.50
I-75 NB E-R	0.00	0.00	8.25
I-75 SB W-R	8.50	0.00	13.75
Tara Blvd	14.75	26.57	52.50
Caruso Rd	20.75	13.14	42.50
Natalie Way E	0.00	0.00	3.25
Division of Forestry (Emergency Signal)	0.00	0.00	0.00
Lockwood Ridge Rd	**12.25**	**11.62**	**34.00**
39th St E	3.25	1.29	7.75
37th St E	0.00	1.14	3.25
33rd St E	14.60	4.86	7.80
30th St E	26.40	12.71	10.00
US 301.	**30.20**	**12.88**	**21.50**
22nd St Ct E.	24.00	0.00	0.00
301 Blvd E	25.67	0.00	16.00
9th St E	4.83	0.00	19.50
5th St W	13.33	0.00	13.33

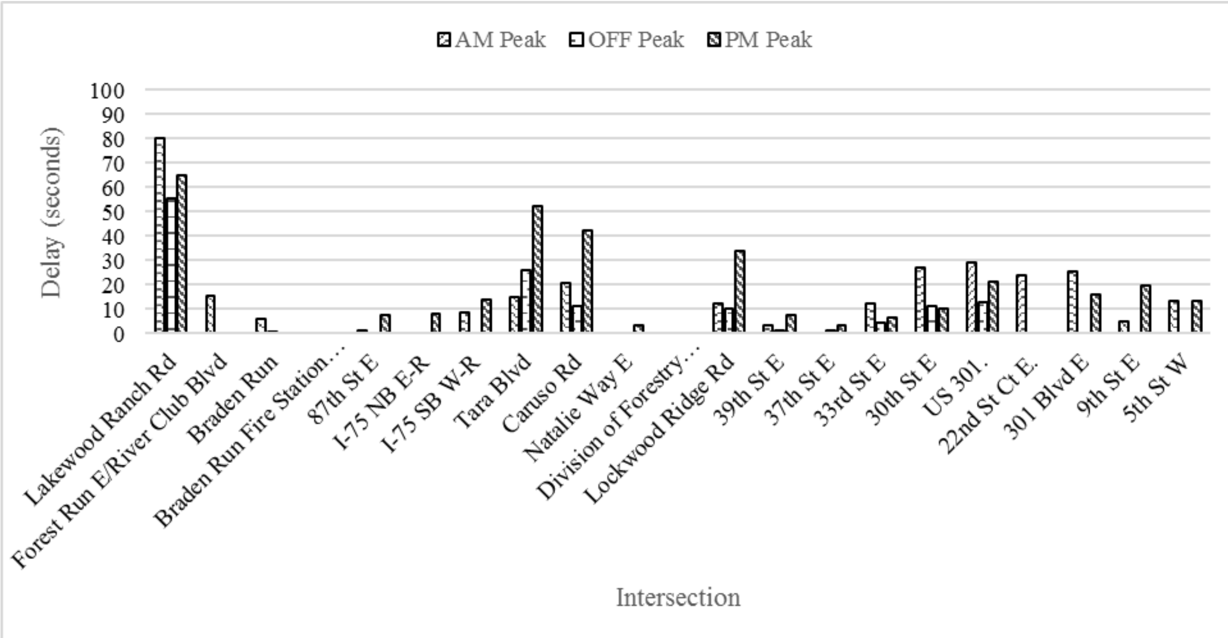


Figure G-2.2 Delay (s) for each Intersection Through Movement Along the WB Direction – Before Study

After Study (Oct. 3 & Oct. 4, 2018)

Table G-2.3 and Figure G-2.3 summarize the delay data for the after study in the EB direction. The data show that the critical intersections along with Tara Blvd and Lakewood Ranch have the highest intersection delays. A few intersections had zero delay for all the floating car runs.

Table G-2.3 Delay (s) for each Intersection Through Movement Along the EB Direction – After Study

Delay at Intersections (sec)	AM Peak	Off Peak	PM Peak
5th St W	3.50	0.83	4.75
9th St E	20.60	16.67	16.25
301 Blvd E	62.00	29.67	14.00
22nd St Ct E.	4.20	0.00	9.25
US 301	**55.40**	**17.83**	**32.25**
30th St E	0.00	9.00	0.00
33rd St E	0.00	0.00	0.25
37th St E	0.00	0.00	0.00
39th St E	7.60	2.17	6.00
Lockwood Ridge Rd	**27.00**	**29.67**	**91.50**
Division of Forestry (Emergency)	0.50	0.00	0.00
Natalie Way E	0.00	0.00	7.75
Caruso Rd	8.25	34.00	0.00
Tara Blvd	11.25	20.50	8.50
I-75 SB W-R	0.00	0.00	14.25
I-75 NB E-R	0.00	0.00	0.00
87th St E	12.25	6.67	47.25
Braden Run Fire Station (Emergency)	5.50	0.00	0.00
Braden Run	25.00	0.00	22.75
Forest Run E/River Club Blvd	31.25	0.00	31.00
Lakewood Ranch Rd	18.75	35.50	24.25

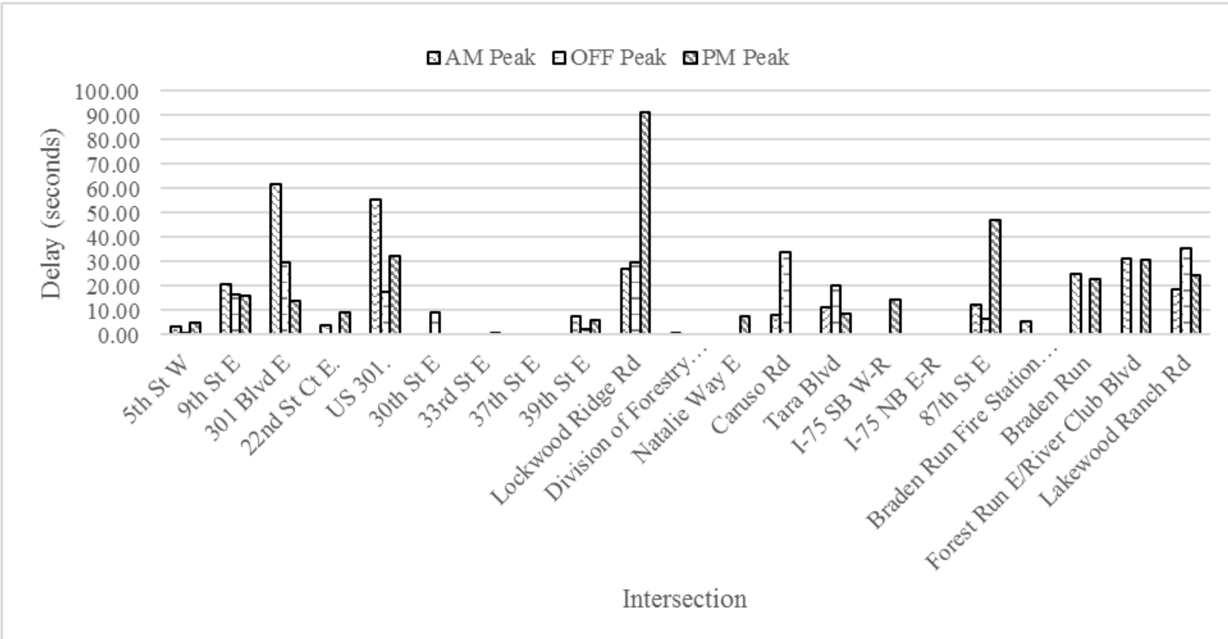


Figure G-2.3 Delay (s) for each Intersection Through Movement Along the EB Direction – After Study

Table G-2.4 and Figure G-2.4 provide the delay data for the NB direction. As shown, the highest consistent delay was observed at the intersection of Lakewood Ranch. The delay at the Lakewood Ranch Rd. ranges from half a minute to a minute for all three periods. Some intersections had zero or very low delay during our data collection.

Table G-2.4 Delay (s) for each Intersection Through Movement Along the WB Direction – After Study

Delay at Intersections (sec)	AM Peak	Off Peak	PM Peak
Lakewood Ranch Rd	54.75	32.17	66.50
Forest Run E/River Club Blvd	0.00	0.00	0.00
Braden Run	0.00	0.00	0.00
Braden Run Fire Station (Emergency Signal)	0.00	0.00	0.00
87th St E	0.00	0.00	1.25
I-75 NB E-R	0.00	0.00	0.00
I-75 SB W-R	6.50	0.00	2.00
Tara Blvd	91.50	31.60	12.75
Caruso Rd	3.25	14.20	1.50
Natalie Way E	0.00	0.00	0.00
Division of Forestry (Emergency Signal)	0.00	0.00	0.00
Lockwood Ridge Rd	**9.25**	**3.20**	**21.25**
39th St E	0.00	0.00	4.40
37th St E	5.25	13.00	40.20
33rd St E	15.00	0.00	5.80
30th St E	16.25	9.60	44.00
US 301.	**31.75**	**10.60**	**46.60**
22nd St Ct E.	0.00	0.00	6.60
301 Blvd E	17.00	7.60	15.00
9th St E	0.00	2.20	15.00
5th St W	0.00	1.80	9.00

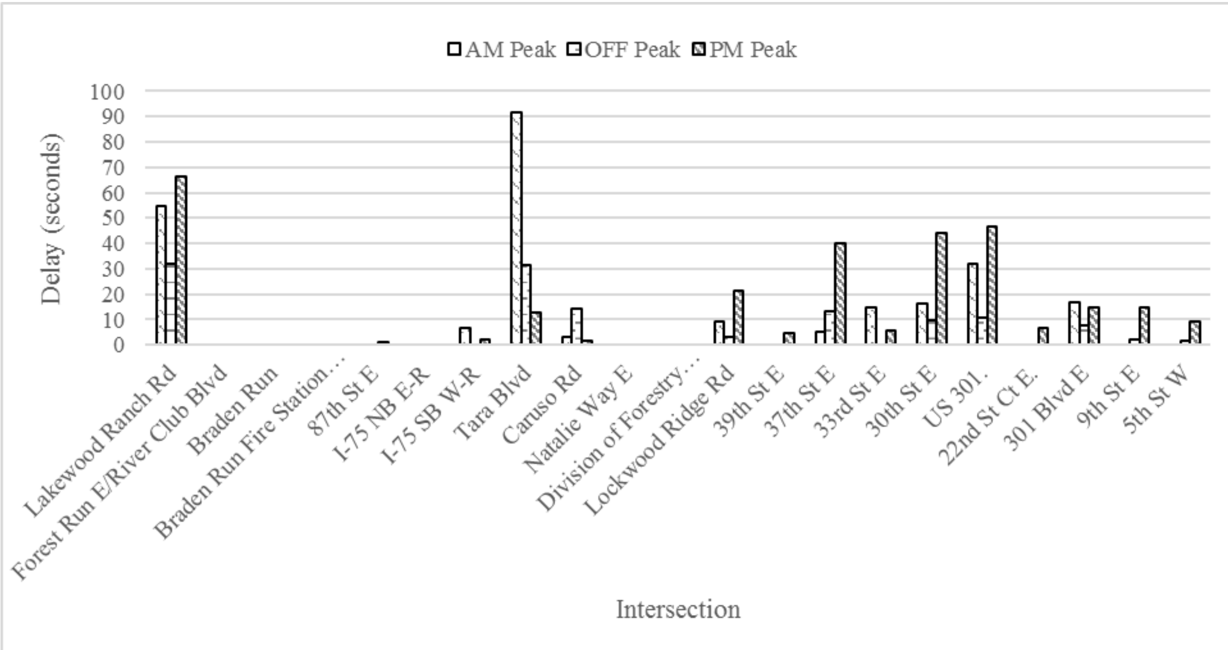


Figure G-2.4 Delay (s) for each Intersection Through Movement Along the WB Direction – After Study

Comparisons of Before and After Intersection Delay Times

The differences in delay between the before and after studies are shown in Table G-2.5 and Table G-2.6. The tables are color-coded as follows: green shows significant improvement, yellow shows modest change (either improvement or deterioration), and red shows significant deterioration in delay. Several gradations of each color are used to represent different variations within each classification.

Table G-2.5 Difference in Delay (sec) for each Intersection Through Movement Along the EB Direction

Intersections	AM Peak	Off Peak	PM Peak
5th St W	3.50	0.83	4.42
9th St E	9.27	14.10	-35.08
301 Blvd E	58.17	9.10	-10.33
22nd St Ct E.	-10.30	0.00	9.25
US 301.	**-1.93**	**5.40**	**-16.08**
30th St E	-12.00	2.43	-10.83
33rd St E	-4.40	0.00	-8.42
37th St E	0.00	-4.00	-2.83
39th St E	-5.80	-3.40	-19.75
Lockwood Ridge Rd	**-48.25**	**6.17**	**55.25**
Division of Forestry (Emergency)	0.50	0.00	0.00
Natalie Way E	0.00	0.00	-2.75
Caruso Rd	-19.50	10.17	-9.75
Tara Blvd	6.50	-19.00	-20.50
I-75 SB W-R	-2.75	0.00	10.75
I-75 NB E-R	0.00	0.00	0.00
87th St E	12.25	4.67	38.75
Braden Run Fire Station (Emergency)	5.50	0.00	0.00
Braden Run	25.00	0.00	16.00
Forest Run E/River Club Blvd	11.00	0.00	31.00
Lakewood Ranch Rd	-15.25	30.83	24.25

Table G-2.6 Difference in Delay (sec) for each Intersection Through Movement Along the WB Direction

Intersections	AM Peak	Off Peak	PM Peak
Lakewood Ranch Rd	-25.75	-25.98	1.25
Forest Run E/River Club Blvd	-15.75	0.00	0.00
Braden Run	-6.25	-0.71	0.00
Braden Run Fire Station (Emergency)	0.00	0.00	0.00
87th St E	-1.50	0.00	-6.25
I-75 NB E-R	0.00	0.00	-8.25
I-75 SB W-R	-2.00	0.00	-11.75
Tara Blvd	76.75	5.03	-39.75
Caruso Rd	-17.50	1.06	-41.00
Natalie Way E	0.00	0.00	-3.25
Division of Forestry (Emergency)	0.00	0.00	0.00
Lockwood Ridge Rd	-3.00	-8.42	-12.75
39th St E	-3.25	-1.29	-3.35
37th St E	5.25	11.86	36.95
33rd St E	0.40	-4.86	-2.00
30th St E	-10.15	-3.11	34.00
US 301.	1.55	-2.28	25.10
22nd St Ct E.	-24.00	0.00	6.60
301 Blvd E	-8.67	7.60	-1.00
9th St E	-4.83	2.20	-4.50
5th St W	-13.33	1.80	-4.33

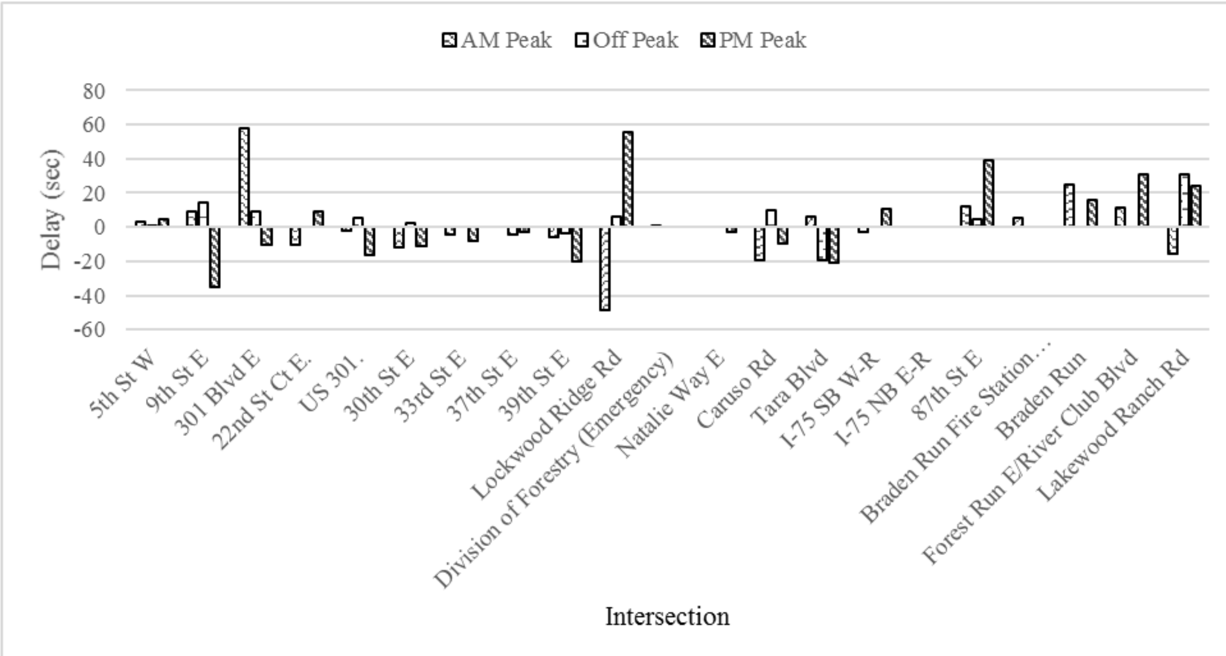


Figure G-2.5 Difference in Delay (sec) for each Intersection Through Movement Along the EB Direction

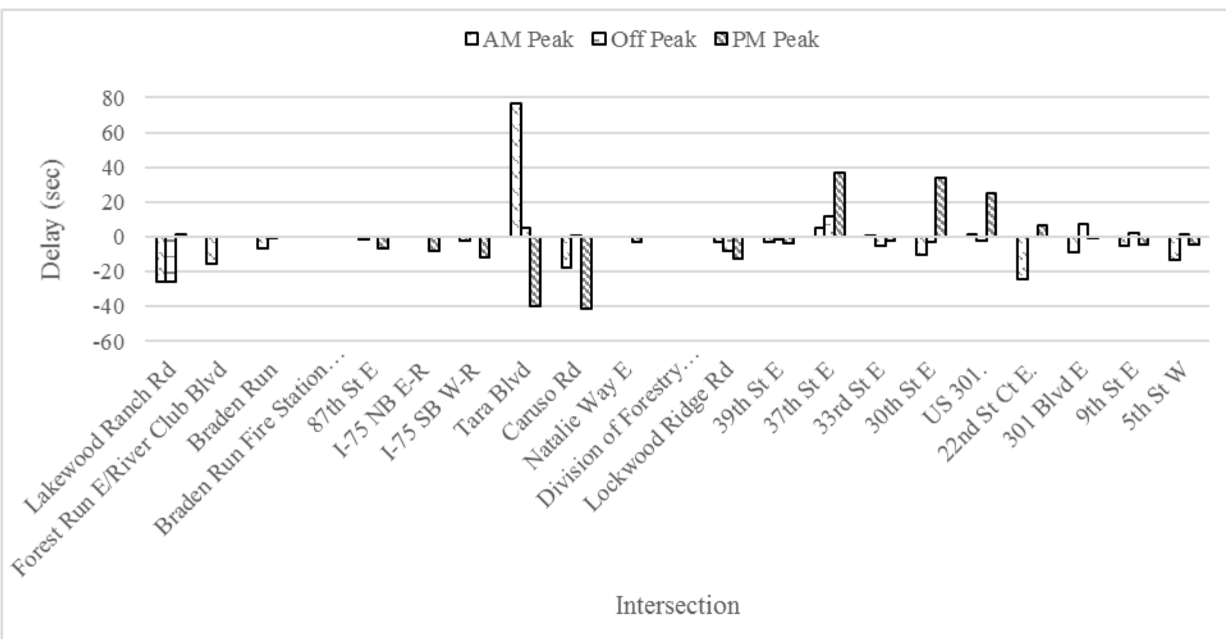


Figure G-2.6 Difference in Delay (sec) for each Intersection Through Movement Along the WB Direction

Discussion

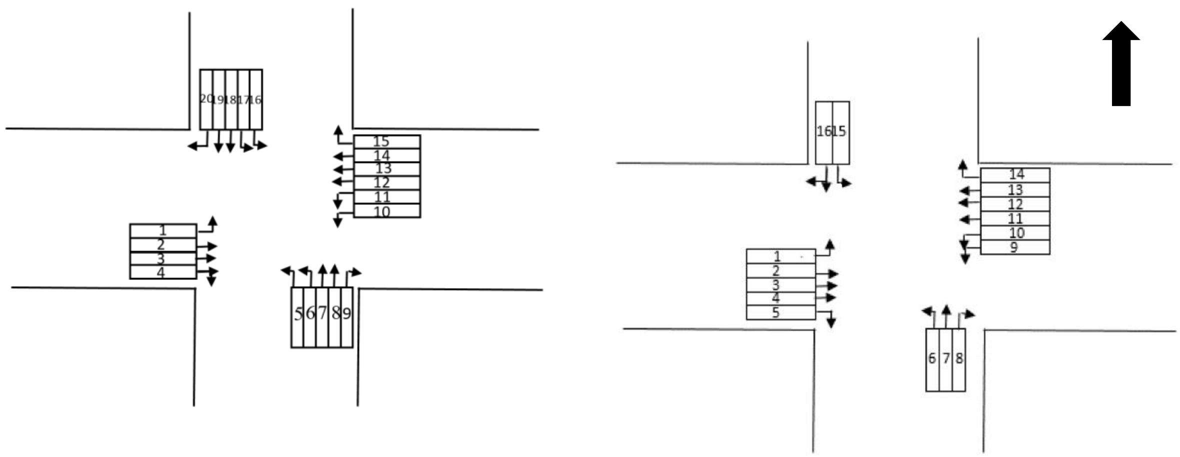
The following can be concluded from the comparison of delays:

- The differences in travel time between the EB and WB directions are primarily due to the differences in delay for the intersections on the east side of I-75 (87^{th} St to Lakewood Ranch Rd). Increased delay was observed in the EB direction for all the intersections east of I-75, while reduced delay was observed in the WB direction for these intersections. Based on discussions with local traffic management center, this increase in delay could be due to detection issues at 87^{th} E St.
- One of the critical intersections, Lockwood Ridge Rd., had low demands on the side streets and therefore had a decrease in delay in all periods in the WB direction
- US301, the second critical intersection, had high side street demands. It had an increase in delay for EB in Off Peak and for WB in AM and PM Peaks.

G.3.3 QUEUE LENGTH

Queue length (number of vehicles/lane) is presented by movement and by time period. This measure is used to evaluate oversaturated conditions at the critical intersections along the study corridors. Note that the queue length reported here is the observed maximum number of vehicles queued during each cycle, and does not represent the total number of vehicles that may have stopped during the cycle. During some time periods, because of cycle failure, vehicles need to stop multiple times before passing through the intersection.

Figure G-3.1 presents the schematic of the lane configurations at the two critical intersections. Queue length is reported for each of these lanes.



(a) SR 70 and US 301

(b) SR 70 and Lockwood Ridge Rd

Figure G-3.1 Lane Configuration of Critical Intersections

Before Study (Dec. 8 & Dec.9, 2016)

Table G-3.1 Average Queue Length by Lane (#vehs/lane) at SR 70 & US 301 – Before Study

Time Period	Time Segment	Eastbound				Northbound					Westbound						Southbound				
		Left	Through		T/R	Left		Through		Right	Left		Through			Right	Left		Through		Right
Lane Number		**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**	**13**	**14**	**15**	**16**	**17**	**18**	**19**	**20**
AM Peak	1	4	15	15	15	4	4	11	10	2	12	11	7	6	4	0	15	15	15	15	3
	2	3	15	15	15	5	5	12	13	2	15	15	13	13	12	6	15	15	15	15	2
	3	3	11	11	11	5	5	14	11	2	9	8	6	6	4	0	6	9	13	12	0
	4	3	10	11	10	4	4	12	11	1	10	10	9	8	8	1	3	10	9	7	0
	Average	**3**	**13**	**13**	**13**	**4**	**5**	**12**	**11**	**2**	**11**	**11**	**9**	**9**	**7**	**2**	**10**	**12**	**13**	**12**	**1**
Off Peak	1	2	6	6	8	4	4	8	8	3	6	5	7	7	5	2	5	5	13	11	0
	2	2	6	6	7	4	3	8	7	2	6	6	9	8	7	2	4	5	9	10	0
	3	2	5	5	6	3	3	9	9	1	5	5	7	7	6	2	4	5	9	9	0
	4	2	7	7	9	2	3	14	12	1	6	6	8	8	6	1	5	5	12	12	0
	Average	**2**	**6**	**6**	**7**	**3**	**3**	**9**	**9**	**2**	**5**	**5**	**8**	**7**	**6**	**2**	**4**	**5**	**11**	**11**	**0**
PM Peak	1	5	12	13	13	6	5	15	15	6	14	14	11	10	7	1	4	6	8	8	1
	2	5	12	12	12	6	6	15	15	8	7	7	10	10	7	1	7	9	10	9	1
	3	2	13	13	14	5	6	15	15	9	6	5	12	12	12	2	6	8	9	9	1
	4	5	9	10	10	5	6	15	15	9	7	5	15	15	15	2	3	5	7	6	1

Time Period	Time Segment	Eastbound				Northbound					Westbound						Southbound				
		Left	Through		T/R	Left		Through		Right	Left		Through			Right	Left		Through		Right
Lane Number		1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20
	Average	5	12	12	12	5	6	15	15	8	8	8	12	12	10	1	5	7	8	8	1

Table G-3.2 Average Queue Length (#vehs/lane) at SR 70 & US 301 – Before Study

Time Period	Eastbound			Northbound			Westbound			Southbound		
	Left	Through	Right	Left	Through	Right	Left	Through	Right	Left	Through	Right
AM Peak	3	11	5	4	12	2	11	8	2	11	13	1
Off Peak	2	6	2	3	9	2	5	7	2	5	11	0
PM Peak	5	11	3	6	15	8	8	11	1	6	8	1

Table G-3.3 Average Queue Length by Lane (#vehs/lane) at SR 70 & Lockwood Ridge Rd – Before Study

Time Period	Time Segment	Eastbound					Northbound			Westbound						Southbound	
		Left	Through			Right	Left	Through	Right	Left		Through			Right	L	T/R
Lane Number		**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**	**13**	**14**	**15**	**16**
AM Peak	1	0	15	15	15	0	8	3	9	12	12	12	12	15	15	15	11
	2	0	15	15	15	0	6	2	12	11	12	12	11	11	4	15	7
	3	0	15	15	15	0	10	2	12	14	14	12	12	11	0	7	7
	4	1	10	11	9	1	6	2	2	7	7	10	9	7	0	7	4
	Average	**1**	**14**	**14**	**13**	**0**	**7**	**2**	**9**	**11**	**12**	**12**	**11**	**11**	**5**	**11**	**7**
Off Peak	1	3	9	8	8	1	4	2	6	8	7	9	8	7	1	5	5
	2	2	5	5	5	1	4	3	8	9	8	6	6	6	1	4	5
	3	2	3	3	3	0	6	3	8	7	7	7	7	6	1	4	7
	4	4	5	4	6	0	6	2	6	8	9	8	9	9	1	5	6
	Average	**3**	**6**	**5**	**5**	**1**	**5**	**3**	**7**	**8**	**8**	**7**	**7**	**7**	**1**	**5**	**6**
PM Peak	1	3	12	11	10	0	7	4	15	4	13	0	13	14	14	15	15
	2	5	15	15	14	0	10	6	15	8	9	0	14	14	14	14	14
	3	2	15	15	15	0	12	11	15	2	15	1	15	15	15	15	15
	4	5	15	15	15	0	10	9	15	5	15	1	14	15	15	15	15
	Average	**4**	**14**	**14**	**14**	**0**	**10**	**8**	**15**	**5**	**13**	**0**	**14**	**15**	**15**	**15**	**15**

Table G-3.4 Average Queue Length (#vehs/lane) at SR 70 & Lockwood Ridge Rd – Before Study

Time Period	Eastbound			Northbound			Westbound			Southbound		
	Left	Through	Right	Left	Through	Right	Left	Through	Right	Left	Through	Right
AM Peak	1	14	0	7	2	9	11	11	5	11	6	2
Off Peak	3	5	1	5	3	7	8	7	1	5	4	2
PM Peak	4	14	0	10	8	15	9	10	15	15	12	2

As shown above, the queues were longest for the SR 70 and Lockwood Ridge Rd. intersection in the PM Peak, for the EB and SB through movements which had a length of 15 cars or greater. Both critical intersections had high queues reaching or surpassing a queue length of 15 cars in the AM Peak in the EB direction.

After Study (Oct. 3 & Oct. 4, 2018)

Tables G-3.5 to G-3.8 show the average queue length by lane and by movement for both critical intersections from the after study.

Table G-3.5 Average Queue Length by Lane (#vehs/lane) at SR 70 & US 301 – After Study

Time Period	Time Segment	Eastbound				Northbound					Westbound						Southbound				
		Left	**Through**		**T/R**	**Left**	**Through**			**Right**	**Left**		**Through**			**Right**	**Left**		**Through**		**Right**
Lane Number		**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**	**13**	**14**	**15**	**16**	**17**	**18**	**19**	**20**
AM Peak	1	4	7	8	4	4	10	12	12	2	4	4	12	12	12	1	9	10	10	8	0
	2	6	9	9	7	5	11	11	10	1	7	7	14	14	14	2	10	11	11	8	0
	3	5	13	14	4	5	15	14	15	3	6	6	15	15	15	1	8	9	9	8	0
	4	0	0	0	0	3	11	12	9	3	4	4	15	14	13	1	9	10	10	7	0
	Average	**4**	**7**	**8**	**4**	**4**	**12**	**12**	**11**	**2**	**5**	**5**	**14**	14	**13**	**1**	**9**	**10**	**10**	**8**	**0**
Off Peak	1	3	7	8	1	1	4	4	6	0	4	2	9	8	8	1	5	5	5	5	1
	2	4	6	7	1	2	5	6	8	0	3	2	9	8	8	1	5	6	5	4	0
	3	3	12	13	1	1	2	4	6	0	2	2	11	10	9	1	5	7	7	5	1
	4	5	12	12	1	2	7	7	8	0	3	3	10	10	10	0	6	8	8	4	2
	Average	**4**	**9**	**10**	**1**	**1**	**4**	**6**	**7**	**0**	**3**	**2**	**10**	9	**9**	**1**	**5**	**6**	**6**	**5**	**1**
PM Peak	1	4	10	9	0	5	12	12	12	1	6	7	15	15	14	1	5	7	7	5	2
	2	14	11	14	0	8	15	15	15	2	9	8	15	15	15	4	7	8	7	6	2
	3	8	13	13	0	4	12	12	12	0	8	6	15	15	15	3	9	10	10	9	0
	4	6	7	7	0	4	10	12	0	6	6	13	13	13	13	3	7	9	8	7	0
	Average	**8**	**10**	**11**	**0**	**5**	**12**	**13**	**10**	**2**	**7**	**9**	**15**	14	**14**	**3**	**7**	**8**	**8**	**7**	**1**

Table G-3.6 Average Queue Length (#vehs/lane) at SR 70 & US 301 – After Study

Time Period	Eastbound			Northbound			Westbound			Southbound		
	Left	Through	Right	Left	Through	Right	Left	Through	Right	Left	Through	Right
AM Peak	4	7	1	4	12	2	4	11	1	9	9	0
Off Peak	4	7	0	1	6	0	3	7	1	6	6	1
PM Peak	8	10	0	5	11	2	7	12	3	8	8	1

Table G-3.7 Average Queue Length by Lane (#vehs/lane) at SR 70 & Lockwood Ridge Rd – After Study

Time Period	Time Segment	Eastbound					Northbound			Westbound						Southbound	
		Left	Through			Right	Left	Through	Right	Left		Through			Right	L	T/R
Lane Number		**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**	**13**	**14**	**15**	**16**
AM Peak	1	4	12	12	14	0	9	6	5	13	13	15	15	14	0	14	3
	2	4	14	14	12	0	7	9	8	15	15	12	13	11	0	15	3
	3	1	14	14	14	1	9	6	8	13	13	9	10	6	0	7	2
	4	0	10	10	10	0	9	6	7	8	10	7	8	7	0	11	3
	Average	**3**	**13**	**13**	**12**	**0**	**8**	**7**	**7**	**12**	**13**	**11**	**11**	**10**	**0**	**12**	**3**
Off Peak	1	2	4	3	5	0	8	7	4	8	9	4	4	5	0	7	3
	2	1	5	5	6	0	6	5	2	8	7	5	4	4	0	4	4
	3	2	5	5	4	0	9	4	4	8	8	3	4	4	0	7	3

Time Period	Time Segment	Eastbound					Northbound			Westbound						Southbound	
		Left	Through			Right	Left	Through	Right	Left		Through			Right	L	T/R
Lane Number		**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**	**13**	**14**	**15**	**16**
	4	1	8	8	8	0	6	5	3	8	7	7	6	5	0	4	3
	Average	**1**	**5**	**5**	**6**	**0**	**7**	**5**	**3**	**8**	**8**	**5**	**4**	**4**	**0**	**6**	**3**
PM Peak	1	5	15	15	15	0	11	12	7	10	10	9	8	9	0	12	2
	2	3	15	15	15	0	12	13	7	10	10	7	7	6	0	12	4
	3	1	14	14	13	0	12	10	6	8	9	7	7	8	0	15	4
	4	0	15	15	15	0	13	14	14	12	12	6	5	6	0	15	2
	Average	**2**	**15**	**15**	**15**	**0**	**12**	**12**	**9**	**10**	**10**	**7**	**7**	**7**	**0**	**14**	**3**

Table G-3.8 Average Queue Length (#vehs/lane) at SR 70 & Lockwood Ridge Rd – After Study

Time Period	Eastbound			Northbound			Westbound			Southbound		
	Left	Through	Right	Left	Through	Right	Left	Through	Right	Left	Through	Right
AM Peak	3	12	0	8	7	7	12	11	0	12	3	0
Off Peak	1	6	0	7	5	3	8	4	0	6	2	1
PM Peak	2	15	0	12	12	9	10	7	0	14	3	0

For the after study, SR 70 & Lockwood Ridge Rd. had the longest queues during the PM Peak in the EB direction. The long queues on EB at Lockwood ridge correspond with the longest delay (91.5 s) and longest travel time (24.9 min) in the PM Peak. For SR 70 & US 301, the longest through movement queues were found in the EB and WB directions and shorter queues were observed on SB and NB for relatively higher traffic flow. The corresponding NB and SB travel times also decreased significantly on US 301.

Comparison of Before and After Queue Lengths for Critical Intersections

The differences in queue length between the before and after measurements are shown in Tables G-3.9 to G-3.12. The tables are color-coded as follows: green shows significant improvement, yellow shows modest change (either improvement or deterioration), and red shows significant deterioration in delay. Several gradations of each color are used to represent different variations within each classification.

Table G-3.9 Difference in Avg. Queue Length by Lane (#vehs/lane) at US301 & SR 70

Time Period	Eastbound				Northbound					Westbound						Southbound				
	Left	Through		T/R	Left		Through		Right	Left		Through			Right	Left		Through		Right
Lane Number	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20
AM Peak	0.7	-5.3	-5.4	-9.1	0.0	7.4	0.1	0.0	0.7	-6.6	-5.9	5.3	5.3	6.5	-0.3	-0.6	-2.3	-3.3	-4.7	-1.2
Off Peak	1.9	3.3	3.5	-6.3	-1.5	1.3	-3.9	-2.2	-1.5	-2.4	-3.1	2.0	1.7	2.5	-1.2	1.1	1.5	-4.2	-5.8	1.1
PM Peak	3.3	-1.5	-1.4	-12.2	0.0	6.4	-2.2	-5.5	-5.6	-1.2	0.7	2.5	2.7	4.1	1.3	2.1	1.7	-0.2	-0.8	0.4

Note: Lane 6 changed from a left movement to a through movement, which may explain why Lane 6 increases in queue length

Table G-3.10 Difference in Avg. Queue Length (#vehs/lane) at US301 & SR 70

Time Period	EB			NB			WB			SB		
	Left	Through	Right	Left	Through	Right	Left	Through	Right	Left	Through	Right
AM Peak	0.7	-5.2	-4.1	-0.1	0.1	0.7	-6.3	5.7	-0.3	-1.4	-4.0	-1.2
Off Peak	1.9	0.7	-1.7	-1.6	-3.1	-1.5	-2.8	2.1	-1.2	1.3	-5.0	1.1
PM Peak	3.3	-4.0	-3.2	-0.2	-3.8	-5.6	-0.3	3.1	1.3	1.9	-0.5	0.4

Table G-3.11 Difference in Avg. Queue Length by Lane (#vehs/lane) at Lockwood Ridge & SR 70

Time Period	EB					NB			WB						SB	
	Left	Through			Right	Left	Through	Right	Left		Through			Right	L	T/R
Lane Number	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
AM Peak	2.0	-1.2	-1.4	-1.0	0.0	1.2	4.5	-1.8	1.1	1.0	-0.8	0.4	-1.5	-4.8	0.7	-4.5
Off Peak	-1.3	-0.1	0.4	0.5	-0.4	2.5	2.6	-3.9	0.2	-0.2	-2.7	-2.8	-2.3	-0.9	1.1	-2.5
PM Peak	-2.0	0.6	0.8	1.1	0.0	2.5	4.5	-6.4	5.0	-2.6	6.8	-7.2	-7.6	-14.6	-0.9	-11.4

Table G-3.12 Difference in Avg. Queue Length (#vehs/lane) at Lockwood Ridge & SR 70

Time Period	EB			NB			WB			SB		
	Left	Through	Right	Left	Through	Right	Left	Through	Right	Left	Through	Right
AM Peak	2.0	-1.2	0.0	1.2	4.5	-1.8	1.1	-0.6	-4.8	0.7	-3.2	-1.3
Off Peak	-1.3	0.3	-0.4	2.5	2.6	-3.9	0.0	-2.6	-0.9	1.1	-1.8	-0.6
PM Peak	-2.0	0.8	0.0	2.5	4.5	-6.4	1.2	-2.7	-14.6	-0.9	-9.6	-1.8

Discussion

The following can be concluded from the comparison of queue lengths:

- US 301 intersection: Except for the WB through movements, all other queues either reduced or remained approximately the same. The queues reduce significantly at the high volume side streets (SB and NB). Consistent with the decrease in travel time along US 301, this further shows that SynchroGreen is prioritizing N-S movements on US 301 over E-W movements along SR 70, most likely due to the relatively higher traffic volumes.
- Lockwood Ridge Rd. intersection: Except for the NB through and left movements, all other queues either reduced or remained approximately the same. Unlike US 301, WB queues were reduced. The reduced WB queue could be due to a new roadway (44th Avenue E) that opened up to the public in 2017 (see Figure G-3.2), as traffic may be diverted from SR 70 to the new facility. This new roadway runs parallel to SR 70 between 19th St E and 45th St E (shown in green in Figure G-3.2). The decrease in the SB queue could be due to the roadway-widening project along this segment (marked in red in Figure G-3.2).

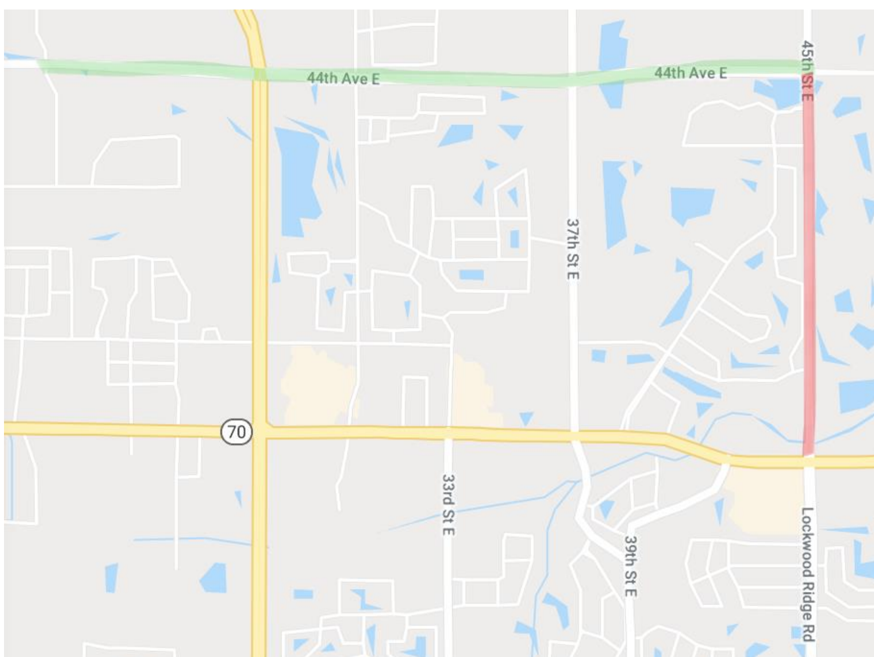


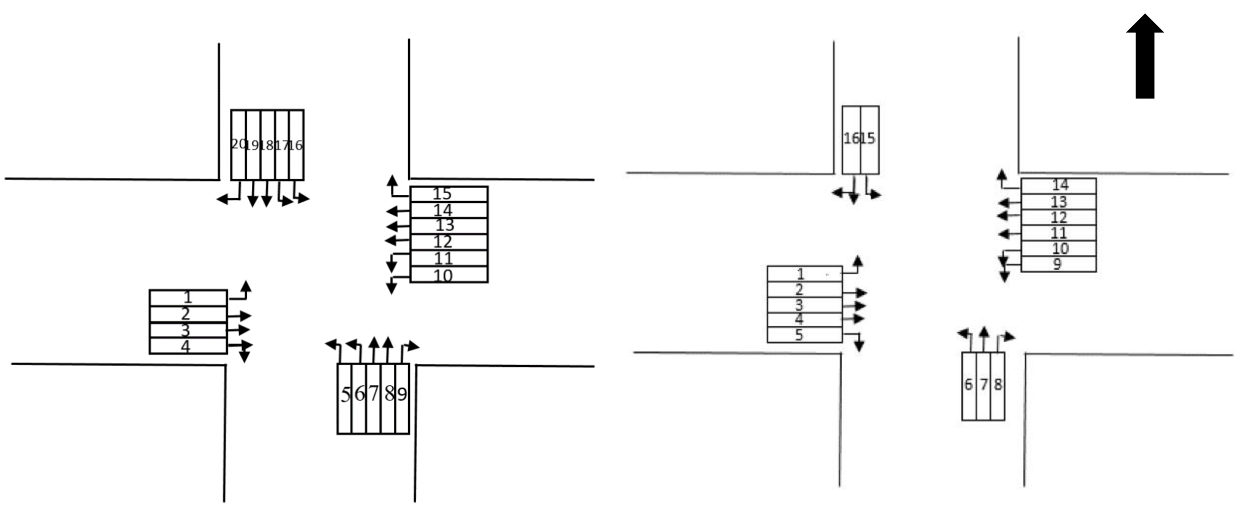
Figure G-3.2 Changes near Lockwood Ridge intersection of SR-70 corridor

G.3.4 QUEUE TO LANE STORAGE RATIO

In addition to queue length, it is important to assess any impact to adjacent lanes or to upstream facilities. The queue to link/lane ratio is used to establish the likelihood of spillback, which is presented in this section by movement and by time period.

The following assumptions are used:

- The storage capacity is estimated as the maximum number of vehicles in the lane.
- The queue to link/lane storage ratio is estimated as 1 if the observer reported "spillback", and as 0.8 if reported as the maximum number of vehicles in the sight distance.
- Queue to lane storage ratios over 80% are highlighted in yellow, as they represent conditions with a high probability for spillback.



(a) SR 70 and US 301 (b) SR 70 and Lockwood Ridge Rd

Figure G-4. 1 Lane Configuration of Critical Intersections

Before Study (Dec.8 & Dec.9, 2016)

Table G-4.1 Average Queue Storage Ratio by Lane and by Period at SR 70 & US 301– Before Study

Time period	Time segment	Eastbound				Northbound					Westbound						Southbound				
		Left	Through		T/R	Left		Through		Right	Left		Through			Right	Left		Through		Right
Lane Number		1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20
AM Peak	1	0.24	1	1	1	0.29	0.33	0.72	0.68	0.16	0.82	0.78	0.45	0.43	0.31	0	0.83	0.83	0.83	0.83	0.13
	2	0.19	0.83	0.83	0.83	0.28	0.29	0.69	0.72	0.1	0.98	0.98	0.89	0.89	0.81	0.4	0.83	0.83	0.83	0.83	0.11
	3	0.19	0.7	0.74	0.71	0.31	0.32	0.9	0.76	0.1	0.61	0.52	0.42	0.42	0.27	0.01	0.37	0.57	0.89	0.8	0
	4	0.19	0.67	0.71	0.68	0.27	0.24	0.77	0.73	0.07	0.66	0.63	0.61	0.56	0.5	0.04	0.2	0.67	0.61	0.49	0
	Average	0.2	0.8	0.82	0.81	0.29	0.3	0.77	0.72	0.11	0.77	0.73	0.59	0.58	0.47	0.11	0.56	0.73	0.79	0.74	0.06
Off Peak	1	0.14	0.4	0.41	0.5	0.23	0.23	0.5	0.52	0.17	0.37	0.34	0.47	0.47	0.34	0.12	0.3	0.34	0.83	0.72	0
	2	0.14	0.38	0.41	0.49	0.26	0.2	0.5	0.49	0.11	0.38	0.39	0.58	0.53	0.47	0.16	0.26	0.36	0.62	0.63	0
	3	0.13	0.32	0.33	0.41	0.18	0.2	0.62	0.58	0.07	0.32	0.31	0.44	0.46	0.39	0.12	0.24	0.3	0.57	0.62	0
	4	0.1	0.47	0.49	0.57	0.13	0.19	0.9	0.81	0.07	0.37	0.4	0.54	0.52	0.42	0.09	0.33	0.33	0.82	0.81	0
	Average	0.13	0.39	0.41	0.49	0.2	0.21	0.63	0.6	0.1	0.36	0.36	0.51	0.49	0.41	0.12	0.28	0.33	0.71	0.7	0
PM Peak	1	0.3	0.67	0.71	0.74	0.31	0.3	0.83	0.83	0.32	0.77	0.77	0.61	0.58	0.38	0.06	0.23	0.32	0.42	0.42	0.04
	2	0.3	0.67	0.66	0.66	0.33	0.36	0.83	0.83	0.42	0.38	0.39	0.56	0.53	0.4	0.04	0.39	0.48	0.56	0.5	0.03
	3	0.13	0.74	0.73	0.76	0.26	0.33	0.83	0.83	0.49	0.4	0.33	0.8	0.79	0.79	0.1	0.34	0.43	0.52	0.48	0.06

Time period	Time segment	Eastbound				Northbound					Westbound						Southbound				
		Left	Through		T/R	Left		Through		Right	Left		Through			Right	Left		Through		Right
Lane Number		1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20
	4	0.29	0.51	0.56	0.58	0.28	0.31	0.83	0.83	0.48	0.4	0.3	0.83	0.82	0.83	0.12	0.17	0.28	0.37	0.31	0.03
	Average	0.26	0.65	0.66	0.68	0.29	0.33	0.83	0.83	0.43	0.49	0.45	0.7	0.68	0.6	0.08	0.28	0.38	0.47	0.43	0.04

Table G-4.2 Average Queue Storage Ratio by Lane and by Period at SR 70 & Lockwood Ridge Rd – Before Study

Time period	Time segment	Eastbound					Northbound			Westbound						Southbound	
		Left	Through			Right	Left	Through	Right	Left		Through			Right	L	T/R
Lane Number		1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
AM Peak	1	0.01	1.00	1.00	1.00	0.00	0.53	0.17	0.57	0.79	0.83	0.81	0.83	0.99	0.99	1.00	0.76
	2	0.02	1.00	1.00	1.00	0.00	0.39	0.10	0.82	0.78	0.90	0.83	0.72	0.78	0.33	1.00	0.42
	3	0.01	0.99	1.00	0.97	0.00	0.64	0.16	0.79	0.95	0.95	0.83	0.79	0.73	0.00	0.47	0.47
	4	0.09	0.67	0.73	0.60	0.08	0.38	0.16	0.16	0.49	0.49	0.68	0.59	0.48	0.02	0.49	0.29
	Average	**0.03**	**0.91**	**0.93**	**0.89**	**0.02**	**0.49**	**0.14**	**0.58**	**0.75**	**0.79**	**0.79**	**0.73**	**0.75**	**0.34**	**0.74**	**0.48**
Off Peak	1	0.18	0.60	0.54	0.50	0.06	0.26	0.16	0.40	0.51	0.48	0.57	0.50	0.47	0.06	0.31	0.34
	2	0.16	0.36	0.31	0.32	0.04	0.26	0.19	0.53	0.60	0.54	0.39	0.39	0.37	0.03	0.27	0.36
	3	0.12	0.21	0.22	0.22	0.01	0.37	0.18	0.56	0.44	0.49	0.43	0.43	0.37	0.06	0.29	0.48
	4	0.28	0.31	0.27	0.38	0.02	0.37	0.16	0.39	0.52	0.61	0.54	0.60	0.57	0.09	0.36	0.38

Time period	Time segment	Eastbound					Northbound			Westbound						Southbound	
		Left	Through			Right	Left	Through	Right	Left		Through			Right	L	T/R
Lane Number		1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
	Average	**0.18**	**0.37**	**0.34**	**0.36**	**0.03**	**0.31**	**0.17**	**0.47**	**0.52**	**0.53**	**0.48**	**0.48**	**0.44**	**0.06**	**0.31**	**0.39**
PM Peak	**1**	0.23	0.79	0.73	0.67	0.00	0.47	0.27	1.00	0.29	0.86	0.01	0.86	0.94	0.94	0.97	1.00
	2	0.43	1.00	1.00	0.92	0.00	0.62	0.43	1.00	0.52	0.60	0.02	0.90	0.95	0.95	0.90	0.90
	3	0.16	1.00	1.00	1.00	0.00	0.77	0.71	1.00	0.13	1.00	0.05	0.99	1.00	1.00	1.00	1.00
	4	0.36	1.00	1.00	1.00	0.00	0.65	0.61	1.00	0.36	1.00	0.04	0.94	1.00	1.00	1.00	1.00
	Average	**0.30**	**0.95**	**0.93**	**0.90**	**0.00**	**0.63**	**0.51**	**1.00**	**0.32**	**0.86**	**0.03**	**0.92**	**0.97**	**0.97**	**0.97**	**0.98**

After Study (Oct. 3 & Oct.4, 2018)

Table G-4.3 Average Queue Storage Ratio by Lane and by Period at SR 70 & US 301 – After Study

	Time segment	Eastbound				Northbound					Westbound						Southbound				
		Left	Through		T/R	Left		Through		Right	Left		Through			Right	Left		Through		Right
Lane Number		1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20
AM Peak	1	0.39	0.64	0.67	0.27	0.19	0.57	0.64	0.51	0.08	0.18	0.19	0.68	0.63	0.06	0.60	0.65	0.66	0.51	0.00	0.39
	2	0.41	0.53	0.55	0.41	0.33	0.76	0.73	0.64	0.09	0.44	0.44	0.96	0.96	0.16	0.68	0.72	0.75	0.51	0.00	0.41
	3	0.36	0.74	0.78	0.24	0.36	0.98	0.96	1	0.20	0.38	0.38	1	1	0.07	0.56	0.61	0.59	0.53	0.00	0.36
	4	0.41	0.66	0.69	0.16	0.2	0.75	0.77	0.6	0.20	0.23	0.23	0.98	0.87	0.07	0.58	0.63	0.63	0.48	0.00	0.41
	Average	0.39	0.64	0.67	0.27	0.27	0.76	0.77	0.69	0.14	0.31	0.31	0.9	0.86	0.09	0.60	0.65	0.66	0.51	0.00	0.39
Off Peak	1	0.22	0.47	0.51	0.08	0.08	0.23	0.29	0.38	0.01	0.23	0.13	0.57	0.54	0.04	0.34	0.36	0.35	0.36	0.07	0.22
	2	0.27	0.4	0.44	0.08	0.13	0.31	0.42	0.5	0.01	0.18	0.16	0.59	0.51	0.03	0.35	0.4	0.36	0.27	0.00	0.27
	3	0.19	0.78	0.84	0.07	0.04	0.13	0.23	0.33	0.00	0.12	0.12	0.6	0.49	0.04	0.33	0.46	0.48	0.34	0.10	0.19
	4	0.34	0.63	0.62	0.04	0.07	0.24	0.26	0.28	0.00	0.16	0.1	0.47	0.41	0.00	0.43	0.53	0.56	0.3	0.15	0.34
	Average	0.26	0.57	0.6	0.07	0.08	0.23	0.3	0.37	0.01	0.17	0.13	0.56	0.49	0.03	0.36	0.44	0.44	0.32	0.08	0.26
PM Peak	1	0.21	0.51	0.48	0.00	0.31	0.81	0.83	0.77	0.08	0.41	0.45	1	0.95	0.05	0.28	0.39	0.39	0.28	0.11	0.21
	2	0.91	0.76	0.91	0.00	0.55	0.99	1	0.99	0.11	0.61	0.53	1	1	0.29	0.45	0.53	0.49	0.4	0.15	0.91
	3	0.51	0.83	0.83	0.00	0.29	0.77	0.81	0.79	0.00	0.53	0.43	1	1	0.19	0.63	0.67	0.65	0.63	0.03	0.51

	4	0.35	0.51	0.45	0.03	0.28	0.69	0.77	0.00	0.39	0.37	0.85	0.87	0.87	0.19	0.49	0.57	0.56	0.45	0.00	0.35
	Average	0.49	0.65	0.67	0.01	0.36	0.82	0.85	0.64	0.14	0.48	0.57	0.97	0.95	0.18	0.46	0.54	0.52	0.44	0.07	0.49

Table G-4.4 Average Queue Storage Ratio by Lane and by Period at SR 70 & Lockwood Ridge Rd – After Study

Time period	**Time segment**	**Eastbound**					**Northbound**			**Westbound**						**Southbound**	
		Left	**Through**			**Right**	**Left**	**Through**	**Right**	**Left**		**Through**			**Right**	**L**	**T/R**
Lane Number		**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**	**13**	**14**	**15**	**16**
AM Peak	**1**	0.24	0.68	0.69	0.78	0.00	0.40	0.24	0.23	0.84	0.84	1.00	1.00	0.95	0.00	0.95	0.20
	2	0.24	0.78	0.79	0.66	0.00	0.41	0.50	0.42	1.00	1.00	0.88	0.90	0.83	0.00	1.00	0.27
	3	0.06	0.79	0.77	0.77	0.04	0.49	0.33	0.42	0.87	0.89	0.63	0.68	0.39	0.00	0.44	0.16
	4	0.02	0.64	0.66	0.67	0.02	0.48	0.33	0.40	0.47	0.53	0.40	0.42	0.39	0.00	0.63	0.14
	Average	0.14	0.72	0.73	0.72	0.02	0.44	0.35	0.37	0.79	0.82	0.73	0.75	0.64	0.00	0.76	0.19
Off Peak	**1**	0.11	0.29	0.22	0.31	0.02	0.43	0.37	0.22	0.42	0.49	0.20	0.23	0.26	0.00	0.38	0.19
	2	0.06	0.26	0.29	0.33	0.00	0.36	0.28	0.12	0.44	0.41	0.26	0.21	0.24	0.00	0.23	0.20
	3	0.10	0.28	0.29	0.24	0.00	0.48	0.24	0.21	0.43	0.43	0.19	0.20	0.20	0.00	0.40	0.17
	4	0.08	0.44	0.44	0.47	0.00	0.27	0.20	0.12	0.47	0.39	0.37	0.34	0.26	0.00	0.24	0.19
	Average	0.09	0.32	0.31	0.34	0.01	0.38	0.27	0.17	0.44	0.43	0.25	0.25	0.24	0.00	0.31	0.19
	1	0.31	1.00	1.00	1.00	0.00	0.71	0.77	0.44	0.53	0.53	0.52	0.47	0.50	0.00	0.68	0.15

Time period	Time segment	Eastbound					Northbound			Westbound						Southbound	
		Left	Through			Right	Left	Through	Right	Left		Through			Right	L	T/R
Lane Number		1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
PM Peak	2	0.20	1.00	1.00	1.00	0.00	0.78	0.83	0.50	0.70	0.67	0.52	0.52	0.42	0.00	0.78	0.30
	3	0.05	0.93	0.95	0.88	0.00	0.83	0.68	0.41	0.52	0.61	0.44	0.45	0.52	0.00	1.00	0.29
	4	0.00	1.00	1.00	1.00	0.00	0.88	0.91	0.96	0.69	0.69	0.33	0.28	0.31	0.00	0.83	0.17
	Average	0.14	0.98	0.99	0.97	0.00	0.80	0.80	0.58	0.61	0.63	0.45	0.43	0.44	0.00	0.82	0.23

Comparisons of Before and After Queue Storage Ratios

The differences in Queue Storage Ratio between before and after measurements are shown in Table G-4.5 and Table G-4.6. The tables are color-coded as follows: green shows significant improvement, yellow shows modest change (either improvement or deterioration), and red shows significant deterioration in delay. Several gradations of each color are used to represent different variations within each classification.

Table G-4.5 Difference in Avg. Queue Storage Ratios at SR 70 & US 301

Time Period	EB				NB					WB						SB				
	Left	Through		T/R	Left		Through		Right	Left		Through			Right	Left		Through		Right
Lane Number	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20
AM Peak	0.2	-0.2	-0.2	-0.5	0.0	0.5	0.0	0.0	0.0	-0.5	-0.4	0.3	0.3	0.4	0.0	0.0	-0.1	-0.1	-0.2	-0.1
Off Peak	0.1	0.2	0.2	-0.4	-0.1	0.0	-0.3	-0.2	-0.1	-0.2	-0.2	0.0	0.0	0.1	-0.1	0.1	0.1	-0.3	-0.4	0.1
PM Peak	0.2	0.0	0.0	-0.7	0.1	0.5	0.0	-0.2	-0.3	0.0	0.1	0.3	0.3	0.4	0.1	0.2	0.2	0.1	0.0	0.0

Note: Lane 6 changed from a left movement to a through movement.

Table G-4.6 Difference in Avg. Queue Storage Ratios at SR 70 & Lockwood Ridge

Time Period	EB					NB			WB						SB	
	Left	Through			Right	Left	Through	Right	Left		Through			Right	L	T/R
Lane Number	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
AM Peak	0.1	-0.1	-0.1	-0.1	0.0	0.1	0.3	-0.1	0.1	0.0	-0.1	0.0	-0.1	-0.3	0.0	-0.3
Off Peak	-0.1	0.0	0.0	0.0	0.0	0.2	0.2	-0.3	0.0	0.0	-0.2	-0.2	-0.2	-0.1	0.1	-0.2
PM Peak	-0.2	0.0	0.1	0.1	0.0	0.2	0.3	-0.4	0.3	-0.2	0.5	-0.5	-0.5	-1.0	-0.1	-0.8

Discussion

The queues reaching capacity/spillover are marked in yellow in Tables G-4.1 to G-4.4. The following are concluded from the comparison of queue storage ratios:

- US 301 intersection: Compared to the before data collection, the EB and SB have fewer or no instances of queues reaching capacity (Q/S>0.8). While there was no change in the NB queues, the WB has more instances of queues about to spillover during the PM Peak.
- Lockwood Ridge Rd. intersection: During the before study all approaches had some instances of queues reaching capacity. However, after SynchroGreen implementation, except for WB in the AM Peak and EB in the PM Peak, all other approaches have little to no instances of queue spillover.

G.3.5 EQUIVALENT PCE FLOWS

Traffic flows were counted manually and converted to equivalent PCE flows (pce/hour) by considering the percentage of heavy vehicles. It was assumed that the PCE for trucks is 2.

Truck Percentage Observations Before Study (Dec.8 & Dec.9, 2016)

Table G-5.1 Truck Percentages at SR 70 & US 301 – Before Study

Period	Interval	Northbound			Southbound			Eastbound			Westbound			Ave.
		Left	Thru	Right	Left	Thru	Right	Left	Thru	Right	Left	Thru	Right	
AM Peak	1	9.52%	7.55%	0.00%	4.08%	0.00%	0.00%	3.57%	3.08%	4.00%	5.56%	2.95%	1.79%	3.51%
	2	0.00%	6.64%	23.40%	2.99%	2.67%	0.00%	5.56%	4.49%	2.78%	13.38%	9.51%	16.36%	7.31%
	3	9.62%	4.66%	14.55%	1.01%	2.86%	0.00%	8.33%	3.60%	1.89%	6.35%	5.75%	6.90%	5.46%
	4	12.20%	8.57%	14.75%	4.88%	0.55%	6.25%	0.00%	5.15%	9.09%	12.84%	5.96%	8.00%	7.35%
	Average	**7.83%**	**6.85%**	**13.18%**	**3.24%**	**1.52%**	**1.56%**	**4.37%**	**4.08%**	**4.44%**	**9.53%**	**6.04%**	**8.26%**	**5.91%**
Off Peak	1	5.77%	9.72%	14.04%	8.16%	0.00%	8.33%	0.00%	5.31%	15.63%	22.89%	4.30%	8.00%	8.51%
	2	5.88%	6.29%	19.30%	10.91%	7.64%	14.29%	0.00%	4.13%	11.76%	11.90%	3.66%	12.77%	9.04%
	3	1.49%	0.00%	14.29%	4.17%	0.00%	12.50%	0.00%	4.41%	6.06%	19.54%	3.67%	5.45%	5.97%
	4	1.89%	6.45%	6.96%	8.11%	12.22%	0.00%	0.00%	6.77%	15.63%	13.43%	2.68%	5.41%	6.63%
	Average	**3.76%**	**5.61%**	**13.64%**	**7.84%**	**4.97%**	**8.78%**	**0.00%**	**5.16%**	**12.27%**	**16.94%**	**3.58%**	**7.91%**	**7.54%**
PM Peak	1	1.11%	4.14%	5.88%	2.06%	4.76%	5.00%	10.34%	1.36%	13.16%	9.30%	5.54%	7.46%	5.84%
	2	5.19%	2.99%	8.33%	0.74%	2.35%	0.00%	2.63%	0.28%	2.33%	9.02%	3.14%	3.17%	3.35%
	3	2.27%	2.01%	5.13%	2.80%	2.43%	0.00%	0.00%	1.88%	2.86%	5.74%	1.34%	3.39%	2.49%
	4	4.76%	2.94%	1.75%	1.18%	1.91%	0.00%	0.00%	2.11%	2.50%	9.17%	1.38%	2.63%	2.53%
	Average	**3.34%**	**3.02%**	**5.27%**	**1.69%**	**2.86%**	**1.25%**	**3.24%**	**1.41%**	**5.21%**	**8.31%**	**2.85%**	**4.16%**	**3.55%**

Table G-5.2 Truck Percentages at SR 70 & Lockwood Ridge Rd – Before Study

Period	Interval	Northbound			Southbound			Eastbound			Westbound			Ave.
		Left	Thru	Right	Left	Thru	Right	Left	Thru	Right	Left	Thru	Right	
AM Peak	1	0.00%	0.00%	4.76%	4.35%	8.00%	11.11%	6.00%	5.72%	9.52%	2.38%	4.18%	3.57%	4.97%
	2	1.49%	0.00%	1.83%	1.05%	0.00%	0.00%	10.00%	6.53%	6.67%	3.91%	5.01%	9.09%	3.80%
	3	5.17%	0.00%	3.21%	2.25%	2.63%	0.00%	11.11%	6.35%	6.38%	6.77%	7.79%	17.39%	5.75%
	4	6.85%	0.00%	8.20%	6.15%	0.00%	0.00%	0.00%	6.59%	8.70%	9.00%	7.49%	12.50%	5.46%
	Average	**3.38%**	**0.00%**	**4.50%**	**3.45%**	**2.66%**	**2.78%**	**6.78%**	**6.30%**	**7.82%**	**5.51%**	**6.12%**	**10.64%**	**4.99%**
Off Peak	1	7.84%	0.00%	7.41%	2.38%	3.45%	0.00%	4.55%	12.26%	12.82%	2.78%	5.34%	0.00%	4.90%
	2	4.08%	12.50%	9.88%	2.13%	2.94%	0.00%	3.33%	10.10%	2.50%	3.70%	5.26%	4.00%	5.04%
	3	3.77%	0.00%	6.67%	2.44%	0.00%	7.69%	3.57%	7.69%	2.27%	1.36%	5.10%	0.00%	3.38%
	4	1.75%	0.00%	3.77%	1.96%	0.00%	7.69%	0.00%	8.70%	1.72%	2.45%	6.15%	0.00%	2.85%
	Average	**4.36%**	**3.13%**	**6.93%**	**2.23%**	**1.60%**	**3.85%**	**2.86%**	**9.69%**	**4.83%**	**2.57%**	**5.46%**	**1.00%**	**4.04%**
PM Peak	1	4.84%	4.88%	5.00%	2.56%	0.00%	8.33%	9.68%	4.68%	1.54%	3.85%	4.46%	0.00%	4.15%
	2	1.43%	4.00%	1.96%	0.00%	1.96%	0.00%	4.17%	4.48%	6.38%	2.63%	7.31%	3.70%	3.17%
	3	0.00%	5.88%	4.90%	1.37%	0.00%	0.00%	0.00%	3.07%	0.00%	0.78%	3.03%	1.69%	1.73%
	4	5.48%	3.66%	0.63%	0.00%	0.00%	0.00%	0.00%	2.17%	2.94%	3.77%	2.49%	0.00%	1.76%
	Average	**2.94%**	**4.60%**	**3.12%**	**0.98%**	**0.49%**	**2.08%**	**3.46%**	**3.60%**	**2.72%**	**2.76%**	**4.32%**	**1.35%**	**2.70%**

PCE Flow Rates Before Study (Dec. 8 & Dec. 9, 2016)

Table G-5.3 Traffic Volume (pce/15 min) and Traffic Flow (pce/hour) at SR 70 & US 301 – Before Study

Period	Interval	Northbound			Southbound			Eastbound			Westbound		
		Left	Thru	Right	Left	Thru	Right	Left	Thru	Right	Left	Thru	Right
AM Peak	1	21	159	59	49	309	17	28	260	50	108	271	56
	2	45	211	47	67	375	20	36	245	72	142	347	55
	3	52	236	55	99	384	16	24	250	53	189	400	58
	4	41	140	61	41	181	16	35	272	33	109	319	50
	Average	**159**	**746**	**222**	**256**	**1249**	**69**	**123**	**1027**	**208**	**548**	**1337**	**219**
Off Peak	1	52	144	57	49	125	12	17	226	32	83	256	50
	2	51	175	57	55	157	14	19	242	34	84	273	47
	3	67	188	63	48	138	8	22	272	33	87	245	55
	4	53	186	115	74	180	9	14	266	32	67	224	37
	Average	**223**	**693**	**292**	**226**	**600**	**43**	**72**	**1006**	**131**	**321**	**998**	**189**
PM Peak	1	90	290	85	97	231	20	29	220	38	129	289	67
	2	77	301	84	135	213	35	38	359	43	122	382	63
	3	88	349	117	143	206	27	39	426	35	122	374	59
	4	63	340	114	170	262	37	41	332	40	109	435	76
	Average	**318**	**1280**	**400**	**545**	**912**	**119**	**147**	**1337**	**156**	**482**	**1480**	**265**

Table G-5.4 Traffic Volume (pce/15 min) and Traffic Flow (pce/hour) at SR 70 & Lockwood Ridge Rd – Before Study

Period	Interval	Northbound			Southbound			Eastbound			Westbound		
		Left	Thru	Right	Left	Thru	Right	Left	Thru	Right	Left	Thru	Right
AM Peak	1	41	20	168	46	25	9	50	367	21	42	287	56
	2	67	24	164	95	35	4	20	429	30	128	459	22
	3	58	13	156	89	38	5	27	362	47	133	398	23
	4	73	23	122	65	44	20	19	349	46	100	414	24
	Average	**239**	**80**	**610**	**295**	**142**	**38**	**116**	**1507**	**144**	**403**	**1558**	**125**
Off Peak	1	51	20	81	42	29	12	22	310	39	108	393	17
	2	49	24	81	47	34	11	30	287	40	135	342	25
	3	53	14	90	41	32	13	28	325	44	147	392	20
	4	57	26	106	51	39	13	26	345	58	163	439	28
	Average	**210**	**84**	**358**	**181**	**134**	**49**	**106**	**1267**	**181**	**553**	**1566**	**90**
PM Peak	1	62	41	120	39	41	12	31	534	65	78	404	13
	2	70	50	153	80	51	10	24	580	47	114	438	27
	3	60	51	143	73	57	5	29	521	45	128	396	59
	4	73	82	160	89	58	10	22	508	34	106	441	60
	Average	**265**	**224**	**576**	**281**	**207**	**37**	**106**	**2143**	**191**	**426**	**1679**	**159**

Truck Percentage Observations After Study (Oct. 3 & Dec. 4, 2018)

Table G-5.5 Truck Percentages at SR 70 & US 301 – After Study

Period	Interval	Northbound			Southbound			Eastbound			Westbound			Ave.
		Left	Thru	Right	Left	Thru	Right	Left	Thru	Right	Left	Thru	Right	
AM Peak	1	0.00%	1.47%	0.00%	7.23%	0.00%	22.22%	14.12%	6.67%	20.00%	0.00%	3.96%	43.90%	9.96%
	2	0.00%	3.01%	2.00%	15.28%	6.83%	25.00%	16.67%	2.95%	20.00%	0.00%	6.18%	24.14%	10.17%
	3	0.00%	2.84%	3.70%	4.72%	2.75%	14.29%	0.00%	7.84%	4.76%	0.00%	1.86%	11.36%	4.51%
	4	0.00%	3.68%	2.56%	7.23%	7.32%	7.81%	15.56%	5.15%	0.00%	6.52%	6.51%	7.25%	5.80%
	Average	**0.00%**	**2.75%**	**2.07%**	**8.61%**	**4.22%**	**17.33%**	**11.58%**	**5.65%**	**11.19%**	**1.63%**	**4.63%**	**21.66%**	**7.61%**
Off Peak	1	0.00%	1.25%	9.09%	13.04%	0.00%	0.00%	12.90%	7.41%	7.89%	4.55%	6.14%	16.22%	6.54%
	2	0.00%	0.58%	3.85%	4.69%	10.27%	11.11%	31.71%	8.45%	8.06%	5.71%	6.02%	15.09%	8.80%
	3	7.14%	0.00%	6.45%	14.63%	0.00%	0.00%	20.00%	3.29%	14.29%	9.09%	4.76%	9.84%	7.46%
	4	0.00%	2.25%	3.45%	10.31%	6.69%	11.76%	10.89%	5.84%	4.55%	8.57%	5.43%	10.00%	6.65%
	Average	**1.79%**	**1.02%**	**5.71%**	**10.67%**	**4.24%**	**5.72%**	**18.88%**	**6.25%**	**8.70%**	**6.98%**	**5.59%**	**12.79%**	**7.36%**
PM Peak	1	4.17%	1.56%	0.00%	9.78%	5.42%	9.52%	10.84%	5.35%	11.76%	0.00%	2.10%	7.07%	5.63%
	2	4.76%	1.09%	4.00%	8.70%	5.20%	15.00%	8.21%	3.48%	5.43%	0.00%	2.49%	6.45%	5.40%
	3	0.00%	1.29%	8.33%	3.17%	3.30%	100.00%	9.26%	2.33%	3.41%	0.00%	1.81%	4.07%	11.41%
	4	0.00%	2.06%	0.00%	3.88%	5.80%	18.75%	3.42%	3.77%	2.25%	1.56%	1.34%	5.19%	4.00%
	Average	**2.23%**	**1.50%**	**3.08%**	**6.38%**	**4.93%**	**35.82%**	**7.93%**	**3.73%**	**5.71%**	**0.39%**	**1.93%**	**5.70%**	**6.61%**

Table G-5.6 Truck Percentages at SR 70 & Lockwood Ridge Rd – After Study

Period	Interval	Northbound			Southbound			Eastbound			Westbound			Ave.
		Left	Thru	Right	Left	Thru	Right	Left	Thru	Right	Left	Thru	Right	
AM Peak	1	9.38%	23.53%	6.72%	6.25%	0.00%	0.00%	42.86%	7.29%	17.39%	0.00%	4.88%	0.00%	9.86%
	2	3.68%	6.25%	6.42%	3.70%	10.00%	0.00%	0.00%	6.63%	5.00%	1.42%	7.28%	0.00%	4.20%
	3	5.97%	11.36%	9.09%	0.00%	0.00%	0.00%	40.00%	9.41%	6.52%	3.60%	5.85%	0.00%	7.65%
	4	2.25%	7.69%	5.04%	0.00%	4.17%	0.00%	0.00%	9.06%	5.56%	2.33%	5.72%	2.86%	3.72%
	Average	**5.32%**	**12.21%**	**6.82%**	**2.49%**	**3.54%**	**0.00%**	**20.71%**	**8.10%**	**8.62%**	**1.84%**	**5.93%**	**0.71%**	**6.36%**
Off Peak	1	4.92%	6.56%	7.94%	37.14%	58.33%	83.33%	0.00%	10.37%	2.78%	31.94%	19.00%	60.00%	26.86%
	2	1.56%	6.45%	5.81%	0.00%	0.00%	0.00%	0.00%	11.15%	8.11%	2.50%	8.89%	8.70%	4.43%
	3	4.76%	0.00%	3.23%	2.50%	0.00%	0.00%	0.00%	5.72%	0.00%	4.42%	5.77%	0.00%	2.20%
	4	6.56%	6.25%	5.38%	0.00%	0.00%	0.00%	0.00%	7.61%	2.04%	4.35%	8.77%	4.55%	3.79%
	Average	**4.45%**	**4.81%**	**5.59%**	**9.91%**	**14.58%**	**20.83%**	**0.00%**	**8.71%**	**3.23%**	**10.80%**	**10.61%**	**18.31%**	**9.32%**
PM Peak	1	3.66%	4.40%	4.71%	2.38%	4.55%	0.00%	2.86%	8.77%	5.56%	5.77%	4.62%	9.09%	4.70%
	2	1.89%	1.47%	4.38%	2.17%	0.00%	20.00%	6.25%	3.94%	4.35%	1.38%	6.93%	5.08%	4.82%
	3	0.00%	0.00%	0.99%	0.00%	0.00%	0.00%	0.00%	5.53%	0.00%	3.31%	5.88%	6.90%	1.88%
	4	2.25%	4.84%	1.94%	1.52%	1.52%	6.67%	0.00%	2.18%	0.00%	3.48%	5.81%	7.02%	2.12%
	Average	**1.95%**	**2.68%**	**3.00%**	**1.52%**	**1.52%**	**6.67%**	**2.28%**	**5.11%**	**2.48%**	**3.48%**	**5.81%**	**7.02%**	**3.63%**

PCE Flow Rates After Study (Oct. 3 & Oct. 4, 2016)

Table G-5.7 Traffic Volume (pce/15 min) and Traffic Flow (pce/hour) at SR 70 & US 301 – After Study

Period	Interval	Northbound			Southbound			Eastbound			Westbound		
		Left	Thru	Right	Left	Thru	Right	Left	Thru	Right	Left	Thru	Right
AM Peak	1	15	204	23	83	227	81	85	315	35	21	101	41
	2	22	166	50	72	249	60	30	237	25	38	178	29
	3	16	141	27	106	327	49	34	268	21	52	161	44
	4	9	163	39	83	328	64	45	272	5	46	169	69
	Average	**62**	**674**	**139**	**344**	**1131**	**254**	**194**	**1092**	**86**	**157**	**609**	**183**
Off Peak	1	14	160	22	23	120	3	31	162	38	22	114	37
	2	15	172	26	64	224	18	41	213	62	35	133	53
	3	14	219	31	41	126	12	25	152	28	33	105	61
	4	12	222	29	97	254	68	101	274	22	35	129	80
	Average	**55**	**773**	**108**	**225**	**724**	**101**	**198**	**801**	**150**	**125**	**481**	**231**
PM Peak	1	24	192	25	92	277	21	83	243	51	57	238	99
	2	21	183	25	161	269	20	134	345	92	58	241	124
	3	34	232	24	189	364	2	108	472	88	78	277	123
	4	25	194	21	129	207	16	117	292	89	64	224	77
	Average	**104**	**801**	**95**	**571**	**1117**	**59**	**442**	**1352**	**320**	**257**	**980**	**423**

Table G-5.8 Traffic Volume (pce/15 min) and Traffic Flow (pce/hour) at SR 70 & Lockwood Ridge Rd – After Study

Period	Interval	Northbound			Southbound			Eastbound			Westbound		
		Left	**Thru**	**Right**	**Left**	**Thru**	**Right**	**Left**	**Thru**	**Right**	**Left**	**Thru**	**Right**
AM Peak	1	32	17	119	16	9	0	7	192	23	21	82	12
	2	136	16	187	54	20	4	14	543	80	141	261	50
	3	67	44	242	63	23	1	10	563	46	139	359	29
	4	89	26	119	64	24	3	15	287	36	129	332	35
	Average	**324**	**103**	**667**	**197**	**76**	**8**	**46**	**1585**	**185**	**430**	**1034**	**126**
Off Peak	1	61	61	63	35	12	6	27	270	36	72	221	10
	2	64	31	86	41	20	8	16	260	37	80	315	23
	3	84	28	124	40	23	6	10	332	42	113	260	20
	4	61	48	93	43	17	7	8	355	49	92	285	22
	Average	**270**	**168**	**366**	**159**	**72**	**27**	**61**	**1217**	**164**	**357**	**1081**	**75**
PM Peak	1	82	91	170	42	22	2	35	399	90	104	303	44
	2	106	68	160	46	26	5	48	558	69	145	361	59
	3	122	46	303	45	21	3	58	561	48	121	323	58
	4	89	62	258	0	0	0	34	504	44	0	0	0
	Average	**399**	**267**	**891**	**133**	**69**	**10**	**175**	**2022**	**251**	**370**	**987**	**161**

Comparisons of Before and After Flows

The differences in traffic flow between the before and after measurements are shown in Tables H-5.9 and H-5.10. The tables are color-coded as follows: green shows significant decrease, yellow shows modest change (either improvement or deterioration), and red shows significant deterioration in delay. Several gradations of each color are used to represent different variations within each classification.

Table G-5.9 Difference in Traffic Flow Rate (pce/hr) at SR 70 & US 301

Period	NB			SB			EB			WB		
	Left	Thru	Right	Left	Thru	Right	Left	Thru	Right	Left	Thru	Right
AM Peak	-97	-72	-83	88	118	185	71	65	-122	-391	-728	-36
Off Peak	-168	80	-184	-1	-124	58	126	-205	19	-196	-517	-42
PM Peak	-214	-479	-305	26	205	-60	295	15	164	-225	-500	158

Table G-5.10 Difference in Traffic Flow Rate (pce/hr) at SR 70 & Lockwood Ridge Rd

Period	NB			SB			EB			WB		
	Left	Thru	Right	Left	Thru	Right	Left	Thru	Right	Left	Thru	Right
AM Peak	85	23	57	-98	-66	-30	-70	78	41	27	-524	1
Off Peak	60	84	8	-22	-62	-22	-45	-50	-17	-196	-485	-15
PM Peak	134	43	315	-104	-115	-24	69	-121	60	67	-363	56

Discussion

- US 301 intersection: Compared to before, EB left and SB through have increased volumes, while WB through and left have significantly reduced volumes. This could be due to decreased demand in WB direction or due to SynchroGreen prioritizing the high-volume SB direction.
- Lockwood Ridge Rd. intersection: Except for the WB through, there are no significant changes in volume in the other approaches. Unlike US 301, the side street volumes at Lockwood Ridge Rd. are very low. Therefore, the low volume in WB could be a direct result of decrease in demand due to seasonal variations or due to a new public roadway that opened in 2017 (44th Avenue E), that runs parallel to SR 70 (Figure G-3.2).
- The decrease in SB at Lockwood Ridge Road could be due to the roadway-widening project on 45th Street East, as commuters might be using alternate routes/parallel facilities to avoid construction (Figure G-3.2).

G.3.6 CONSIDERATION OF TRAFFIC FLOWS JOINTLY WITH QUEUE LENGTH

The differences in “Traffic Flow” and “Queue Length” between before and after measurements are shown in Tables G-6.1 and G-6.3. The tables are color-coded as follows: green indicates that “Queue” decreases and “Traffic Flow” increases, red indicates “Queue” increases and “Traffic Flow” decreases, no color indicates “Traffic Flow” and “Queue” increase or decrease at the same time. Generally, green indicates improvement despite an increase in flow.

Table G-6.1 Differences in Traffic Flow (TF) and Queue Length (Q) at SR 70 & US 301

Time Period	NB						SB						EB						WB					
	Left		Thru		Right		Left		Thru		Right		Left		Thru		Right		Left		Thru		Right	
	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q
AM Peak	-97	-0.1	-72	0.1	-83	0.7	88	-1.4	-118	-4.0	185	-1.2	71	0.7	65	-5.2	-122	-4.1	-391	-6.3	-728	5.7	-36	-0.3

Time Period	NB						SB						EB						WB					
	Left		Thru		Right		Left		Thru		Right		Left		Thru		Right		Left		Thru		Right	
	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q
Off Peak	-168	-1.6	80	-3.1	-184	-1.5	-1	1.3	124	-5.0	58	1.1	126	1.9	-205	0.7	19	-1.7	-196	-2.8	-517	2.1	42	-1.2
PM Peak	-214	-0.2	-479	-3.8	-305	-5.6	26	1.9	205	-0.5	-60	0.4	295	3.3	15	-4.0	164	-3.2	-225	-0.3	-500	3.1	158	1.3

Table G-6.2 Difference (%) in Traffic Flow (TF) and Queue Length (Q) at SR 70 & US 301

Time Period	NB						SB						EB						WB					
	Left		Thru		Right		Left		Thru		Right		Left		Thru		Right		Left		Thru		Right	
	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q
AM Peak	-61%	-3%	-10%	1%	-37%	44%	34%	-13%	-9%	-32%	268%	-100%	58%	20%	6%	-47%	-59%	-86%	-71%	-56%	-54%	70%	-16%	-15%
Off Peak	-75%	-51%	12%	-33%	-63%	-95%	0%	28%	21%	-47%	135%	N/A	175%	96%	-20%	13%	15%	-81%	-61%	-52%	-52%	29%	22%	-68%
PM Peak	-67%	-4%	-37%	-26%	-76%	-72%	5%	32%	22%	-6%	-50%	53%	201%	71%	1%	-37%	105%	-99%	-47%	-3%	-34%	28%	60%	96%

Table G-6.3 Differences in Traffic Flow (TF) and Queue Length (Q) at SR 70 & Lockwood Ridge Rd

Time Period	**NB**						**SB**						**EB**						**WB**					
	Left		**Thru**		**Right**		**Left**		**Thru**		**Right**		**Left**		**Thru**		**Right**		**Left**		**Thru**		**Right**	
	TF	**Q**	**TF**	**Q**	**TF**	**Q**	**TF**	**Q**	**TF**	**Q**	**TF**	**Q**	**TF**	**Q**	**TF**	**Q**	**TF**	**Q**	**TF**	**Q**	**TF**	**Q**	**TF**	**Q**
AM Peak	85	1.2	23	4.5	57	-1.8	-98	0.7	-66	-3.2	-30	-1.3	-70	2	78	-1.2	41	0.0	27	1.1	-524	-0.6	1	-4.8
Off Peak	60	2.5	84	2.6	8	-3.9	-22	1.1	-62	-1.8	-22	-0.6	-45	-1.3	-50	0.3	-17	-0.4	-196	0.0	-485	-2.6	-15	-0.9
PM Peak	134	2.5	43	4.5	315	-6.4	-104	-0.9	-115	-9.6	-24	-1.8	69	-2	-121	0.8	60	0.0	67	1.2	-363	-2.7	56	-14.6

Table G-6.4 Difference (%) in Traffic Flow (TF) and Queue Length (Q) at SR 70 & Lockwood Ridge Rd.

Time Period	**NB**						**SB**						**EB**						**WB**					
	Left		**Thru**		**Right**		**Left**		**Thru**		**Right**		**Left**		**Thru**		**Right**		**Left**		**Thru**		**Right**	
	TF	**Q**	**TF**	**Q**	**TF**	**Q**	**TF**	**Q**	**TF**	**Q**	**TF**	**Q**	**TF**	**Q**	**TF**	**Q**	**TF**	**Q**	**TF**	**Q**	**TF**	**Q**	**TF**	**Q**
AM Peak	36%	16%	29%	208%	9%	-21%	-33%	6%	-46%	-55%	-79%	-84%	-60%	400%	5%	-9%	28%	0%	7%	10%	-34%	-5%	1%	-100%

Time Period	NB						SB						EB						WB					
	Left		Thru		Right		Left		Thru		Right		Left		Thru		Right		Left		Thru		Right	
	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q
Off Peak	29%	54%	100%	102%	2%	-55%	-12%	24%	-46%	-47%	-45%	-26%	-42%	-47%	-4%	6%	-9%	-80%	-35%	0%	-31%	-37%	-17%	-103%
PM Peak	51%	26%	19%	60%	55%	-43%	-37%	-1%	-56%	-82%	-65%	-59%	65%	-48%	-6%	6%	31%	0%	16%	13%	-22%	-28%	35%	-100%

Discussion

The green boxes indicate clear improvement and the red boxes indicate clear deterioration. The following can be concluded from the comparison of traffic flows jointly with queue length:

- US 301 intersection: SB and EB movements show clear improvement, and the volumes increase while queues decrease. WB through queues significantly increase, despite having reduced volumes. The SB improvement is consistent with the decrease in travel time on US 301 (i.e. SynchroGreen prioritizing the high volume along US 301).
- Lockwood Ridge Rd. intersection: Unlike US 301, there are no significant changes at Lockwood Ridge Rd. While there is a drop in traffic volume in the WB through direction, the queues along this movement also decreased. The drop in volume at WBT may be due to people using the newly opened parallel road, 44th Ave E, as an alternative route (Figure G-3.2).

BEFORE AND AFTER-IMPLEMENTATION STUDIES OF

ADVANCED SIGNAL CONTROL TECHNOLOGIES IN FLORIDA

The main objective of this project is to evaluate the implementation of proposed adaptive signal control technology (ASCT) traffic operations at several arterial corridors in Florida, before and after the installation of the ASCT, document the advantages and the disadvantages of different approaches and implementations, and provide recommendations for statewide implementation of ASCT.

G.4 QUESTIONNAIRE

SECTION 1: RESPONDENT'S INFORMATION

Name **Renjan Joseph**

Organization FDOT

Position TSM&O Engineer

Address FDOT District 1 – Traffic Operations 801 N Broadway Ave, P.O. Box -1249

Phone (863) – 519 - 2746

Fax

E-mail Renjan.Joseph@dot.state.fl.us

SECTION 2: PREVIOUS TRAFFIC CONTROL TECHNOLOGY

1. Please specify the type of traffic control used before Adaptive Signal Control Technology (ASCT) was installed for your site. (Please check all that apply.)

a. Fixed time coordinated control **b. Actuated coordinated control**

c. Fixed-time isolated control d. Actuated isolated control

e. Other, please specify: ____

2. What type of traffic controller was used before ASCT was implemented?

a. NEMA TS-1 **b. NEMA TS-2**

c. 170 d. 2070

e. Other, please specify: ____

20. How many detectors were used before ASCT was implemented for a typical intersection? Please also specify the location where the detectors were typically placed for each specific detector type (video detection, inductive loops, radars, etc.).

Loops (one per lane)

21. What was the frequency of retiming the signal timing plan for the corridor before ASCT was implemented?

Retiming in 2012.

Fine-tuning in 2016 based on observations.

22. How often were maintenance and updates performed to your previous control system in terms of the following components?

Everything is checked annually.

a. Detection: **as needed basis**
b. Control Hardware:
c. Software:

23. What was the annual maintenance cost in terms of hardware, software and personnel of the previous control system for the whole corridor where the ASCT is now deployed?

24. What was the life expectancy of your traffic control before the deployment of ASCT? (Please refer to the hardware and software.)

Controllers never had to be changed.

25. How many crashes of each category were reported during the past 4 years before the ASCT implementation? (2011-2014)

	K (Killed)	A (Disabling Injury)	B (Evident Injury)	C (Possible Injury)	O (No Apparent Injury)
2011					
2012					
2013					
2014					

SECTION 3: ADAPTIVE TRAFFIC CONTROL SYSTEM (ASCT)

9. Which Adaptive Traffic Control System (ASCT) does your agency deploy?

m. InSync; Version:____________

n. **Synchro Green**; Version: **ATMS 2.8**

o. Other; please specify: ______________________

34. Did you consider any other ASCT before you selected this one for installation? Why did you reject the other(s) in favor of this one?

No, it was a federally funded grant and had to be open bid.

35. What were your major criteria for choosing the ASCT for the selected corridor?

N/A

36. What is the life expectancy of your ASCT system? (Consider both hardware and software.)

Software maintenance agreement has warranty for 3 years. Hoping for at least 5 years.

37. What type of traffic controller is currently in use after the ASCT implementation?

a. **Same as before the ASCT implementation**
b. New, please specify type: ____
c. Other, specify:

38. How many and what type of detectors are used after the ASCT implementation on an intersection level? Please note any updates that the previous detection system needed so as to work with your ASCT.

39. Was there a need to update your previous software operating system at your Transportation Management Center in order to get your current ASCT software working?

a. Yes b. No

36. How often do you plan to perform physical maintenance or updates on your new ASCT in terms of the following components?

Once a year, the intersections are all checked

a. Detection: **as needed**
b. ASCT Hardware:
c. Software: **Covered by software maintenance agreement for first three years. Vendor will do it as necessary when updates are released.**

37. Was there a need for training your personnel in order to operate the new ASCT?

a. **Yes** b. No

5 days

38. Was there any change on the number of staff required to operate/maintain the effective signal ASCT operations? (Comparison of staff needed before and after the deployment.)

Staff number is the same.

39. Where did expertise come from for your personnel's training, the ASCT installation, operation and the system's maintenance? Please check all that apply.

	In-House	Vendor	Contractor
Employee Training			
ASCT Installation			
ASCT Operation			
ASCT Maintenance		**Yes**	

40. How many crashes of each category were reported after ASCT was implemented?

K (Killed)	A (Disabling Injury)	B (Evident Injury)	C (Possible Injury)	O (No Apparent Injury)

SECTION 4: COST COMPONENTS

53. What is the overall capital cost of purchasing the ASCT (in terms of the software and licenses of the ASCT) for the corridor? If the purchase includes any service of implementation, personnel training or maintenance, please also specify here.

54. What is the implementation cost (considering the installation on site, any updates in software and hardware, design needs and contract hours) for the ASCT corridor? Please specify if this cost is partially or totally included in previous cost items.

55. What was total cost for training your personnel to operate and manage the new ASCT? Please specify if this cost is partially or totally included in previous cost items.

56. What is the expected maintenance cost (in terms of hardware, software and personnel) for the ASCT corridor on a yearly basis? Please specify if this cost is partially or totally included in previous cost items.

____

____

57. Can you specify the costs for the following ASCT components?

a. Firmware $____________ b. Software $_____________

c. Equipment d. Maintenance of Traffic Cost $_____________

e. Design Needs $__________ f. Contract Help/Agency Hours Cost $_________

SECTION 5: INSTITUTIONAL ISSUES

58. Were there any institutional issues that you had to overcome while implementing and operating the ASCT project? Please categorize and explain those issues below.

Issues in all of the following categories

a. Organization and Management Institutional Issues:

Matching grant issues

b. Regulatory and Legal Institutional Issues:

Legal issues with assigning grant to an existing contract instead of bidding it out

c. Human and Facility Resources Institutional Issues:

Difficulty figuring a lot out such as changing out traffic controllers, complication with lots of parties involved and contracts

d. Financial Institutional Issues:

59. Which component (e.g. detection, communication, hardware, software, licensing or other) of your ASCT deployment is the most challenging to maintain and why?

Detection: Vendor preferred magnetometers while county wanted video detection, so had to add and buy more detection. Issue is solved now.

60. What happens to your ASCT operations when some of your detectors fail on the corridor level?

m. ASCT triggers an alarm and notifies operators

n. **ASCT switches to an "off-line" mode by implementing Time-Of-Day plans**

o. Combination of the above. d. Other (please describe):

29. How is your ASCT performance evaluated?

o. In-house:

p. By an independent evaluator, please describe: ______________________________

q. Not applicable—there is no evaluation

r. **Combination:**

In-house: Bluetooth travel time

FDOT does evaluation on state roads, reporting quarterly for strategic plan

30. If the corridor on which the ASCT operates experiences over saturation, how would you rate the operation of the ASCT in response to these traffic conditions?

u. ASCT prevents or eliminates oversaturation

v. ASCT eliminates or reduces the extent of the periods of oversaturation

w. ASCT adversely affects the traffic conditions during periods of oversaturation

x. **Other: Doesn't prevent or eliminate but helps to manage oversaturation**

y. Oversaturation is very rare on the corridors operated by our ASCT

31. Based on your opinion and the up-to-date operation, when are ASCT operations proven to be the most effective?

Depends

a. Peak periods - when looking at saturated conditions

b. Off-peak periods

c. Shoulders of peak periods

d. Other, please specify: ______________

32. Has the level of performance of ASCT been sustained since its installation?

a. Yes b. No; why not?

41. What was the public reaction to the ASCT operation? Have you received any complaints about long delays and queues?

They do not notice it. First two weeks were bad, but after parameter adjustments, it was the same with number of calls received.

42. How satisfied are you, in general, with your ASCT deployment?

a. Very satisfied b. **Somewhat satisfied**

c. Neutral d. Somewhat dissatisfied

e. Not satisfied at all

41. Are there any other costs or benefits related to ASCT deployment that you would like to report?

a. Benefits: **Helps with incident management and lowered frequency of free timing**

b. Costs

42. Would you consider expanding the ASCT program to any other corridors? Why or why not? If yes, would you use the same technology/firmware or have any suggestions for other systems? If so, why?

Yes, with the right timing, funding, and corridor.

Technology depends on funding and ability to choose

SECTION 6: SAFETY ISSUES

37. Do you have any anecdotal / qualitative data on safety benefits / disbenefits of the signal improvement along this corridor?

No.

38. Has there been other changes along this corridor over the analysis period that could affect the safety of this corridor either positively or negatively?

Construction on I-75 interchange and 45th Street E. No prediction of either negatively affecting safety.

39. Are you aware of any changes to the crash data reporting procedures over the analysis period? If yes, what are the changes?

No.

40. What are your overall reactions to the trends presented in the report?

More explanation can come from information on construction, new openings, and US301 travel time

41. Is there another comparable corridor in which signal improvements have not been made to which the results of this corridor can be compared to?

Yes and no. SR64 was just retimed, and it was opened to 6 lanes from 4 lanes recently. So, it might not be a fair comparison even though they both have six lane corridors. It is also under construction.

42. Other thoughts / comments for us?

None

Thank you for your help in completing this survey. Your responses will help us with evaluating the deployment and benefits of your current ASCT.

If you have any questions regarding the survey, please contact Dr. Pruthvi Manjunatha, email: pruthvim@ufl.edu who works with Dr. Lily Elefteriadou.

G.5 BENEFIT COST ANALYSIS

Table G-7.1 Monetized Saving

Monetized Saving	
Time saving	-$108,203.84
Fuel Consumption saving	-$53,527.64
Air Pollution Saving	-$95,495.15
Safety Saving	$0.00
Total Saving without Safety	-$257,226.62
Total Saving	-$257,226.62
Total Cost	$997,700.00
B/C Ratio without safety	-0.257819609
B/C Ratio	-0.257819609

Table G-7.2 Monetized Saving with No Emissions

Monetized Saving	
Time saving	-$108,203.84
Fuel Consumption saving	-$53,527.64
Safety Saving	$0.00
Total Saving without Safety	-$161,731.48
Total Saving	-$161,731.48
Total Cost	$997,700.00
B/C Ratio without safety	-0.16210432
B/C Ratio	-0.16210432

APPENDIX H: Summary of E. Van Fleet Dr. and N. Broadway Ave., Polk County

H.1 EXECUTIVE SUMMARY

The objective of this research is to evaluate traffic operations at several arterial corridors in Florida, before and after the implementation of proposed Adaptive Signal Control Technologies (ASCT), document the advantages and disadvantages of different ASCT approaches and implementations, and provide recommendations for state-wide implementation of ASCT.

This appendix summarizes the before and after field data collected along the Van Fleet Drive and North Broadway Ave, Bartow corridor from Manor Drive to N. Holland Pkwy.

The InSync adaptive signal control system was implemented along this corridor in March 2016. Two data collection methods were used to collect the desired information. Floating car runs were conducted with the UFTI instrumented vehicle to collect vehicle travel times during three time periods (AM Peak (7-9 AM), Off Peak (9:30 AM -11:30 AM) and PM Peak (4-6 PM)). The floating car method involves driving with traffic and the driver passes as many vehicles that pass the driver to obtain an average speed. In addition, turning movement counts and queue lengths were collected at two critical intersections (N. Broadway Ave. and E. Van Fleet. and Walmart Access Rd. and E. Van Fleet). Based on these measurements, five performance measures were obtained for the before study period: Link/Route Travel Time, Delay at Intersections, Queue Length (at critical intersections), Queue to Lane Storage Ratio (at critical intersections), and Passenger Car Equivalent (PCE) flows (at critical intersections). For each performance measure, a comparison between the before and after data is conducted and presented in this appendix.

All eastbound delays rise throughout the day with the exception of Manor Drive (Rain), Walmart access, and E. Van Fleet. Westbound delays rise throughout the day, and follow a rising trend at every intersection with the exception of Walmart. Based on delay, the most critical intersection in each of the time periods is:

- AM Peak: E. Van Fleet (rain and no rain) (EB) and N. Holland Pkwy (rain and no rain) (WB). The EB travel time does not change between rain and no rain.
- Off Peak: E. Van Fleet (rain) and N. Broadway Ave (no rain) (EB) and Walmart (rain and no rain) (WB). Truck percentages rise during this period across most approaches at both critical intersections.
- PM Peak: N. Broadway Ave. (no rain) and E. Van Fleet (rain) (EB) and N. Holland Pkwy (rain and no rain) (WB). The left-turning traffic heading NB at Walmart has high queue-storage ratios throughout the peak time. The travel time EB during rain versus WB during rain rises by more than 1 minute.

The before study was conducted after the implementation of the InSync in the corridor, while the system was turned off. During the first day of the before study (Feb 22nd, 2017), light rain persisted through the morning and off peak times with severe rain during the afternoon peak times. Due to the heavy rain, data were not collected for approximately 27 minutes (from 4.48 to 5.15 PM) for the N. Broadway Ave and E. Van Fleet intersection. The missing queue and volume data for this intersection were obtained through interpolation using the data collected immediately before and immediately after this time period. The first 12-minute period (4.48-5.00 PM) corresponded to queue length data collection and the remaining 15-minute period to volume data collection. For the before study there is one day of travel time data collection without rain, and one with rain. Delay and travel times for the day with rain are included in this report. However, most of the before-after comparisons were made using the data obtained during good weather.

The following were concluded:

- Based on the comparison of each performance measure, we observe that the quality of service for most of the movements has generally improved. Overall, there was a reduction in travel time for the EB (-12.9%), and an increase for the WB (14.2%). Prior to the InSync installation, the average travel time for the two directions was similar, but it seems the new system favors the EB.
- The two movements with the most significant increase in delay are left turns: the left turn from N. Broadway onto E. Van Fleet in the EB direction, and the left turn from N. Holland onto E. Van Fleet in the WB direction. Again, this may be a function of priorities set up within InSync to favor through movements.
- The SB left at N. Broadway Ave. and E. Van Fleet Drive intersection, which had one of the longest queues during the AM and PM peak periods in the before study, had a reduction in queue length for those time periods. The queues at the NB left at the Walmart Access Rd. and E. Van Fleet intersection, which also had long queues in the before study, were also reduced, particularly during the PM peak period.
- The queue storage ratios for the NB left at the Walmart Access and E. Van Fleet Drive were significantly improved, particularly for the PM peak. The NB through at the N. Broadway Ave and East Van Fleet continues to have longer queues. This movement had a 15-min period with spillback for two of the lanes during the PM peak. Overall, the queue storage ratios were improved for the two critical intersections.
- At N. Broadway Ave. and E. Van Fleet Drive, with the InSync installation, the volumes traveling through the intersection are lower during the AM peak and much higher during the Off-peak, especially the WB through, WB right, and SB left movements.
- At the Walmart Access and E. Van Fleet Drive, volumes are similar between the before and after study, except for reductions in the EB through and right movements and an increase in the WB through movement during the AM peak.

H.2 CORRIDOR INFORMATION

Figure H-1 provides a schematic of the N. Broadway Ave and E. Van Fleet Drive, at Bartow. As shown, the corridor starts in the SB direction along N. Broadway, and turns eastbound at E. Van Fleet Drive. Table H-1 lists the intersections along the corridor. The land use from Manor Drive to the Walmart Access Road is mostly industrial and commercial with abundant parking. Two intersections (N. Broadway Ave. and E. Van Fleet Drive, and Walmart Access Road and E. Van Fleet Drive) were selected as the critical intersections where detailed turning movement and queue counts were collected. Figure H-2 provides the lane configuration of these two critical intersections.

The before study was conducted after the implementation of InSync in the corridor, while the system was turned off. During the first day of the before study (Feb 22nd, 2017), there was light rain through the AM and Off-peak times with severe rain during the PM peak. Due to the heavy rain, the observers were not able to collect data for approximately 27 minutes (from 4.48 to 5.15 PM) for the N. Broadway Ave and E. Van Fleet intersection. The missing queue and volume data for this intersection were obtained through interpolation using the data collected immediately before and immediately after this time period. The first 12-minute period (4.48-5.00 PM) corresponded to queue length data collection and the remaining 15-minute period to volume data collection. For the before study there is one day of travel time data collection without rain, and one with rain. Delay and travel times for the day with rain are included in this report. However, most of the before-after comparisons were made using the data obtained during good weather.

The EB direction of the corridor begins north of Manor Drive, proceeds southbound, and turns left on E. Van Fleet Drive until the N. Holland Pkwy intersection, where the instrumented vehicle makes a left turn. The EB direction ends shortly thereafter. The WB direction of the corridor begins south of the N. Holland Pkwy. intersection, heads north, turns left at E. Van Fleet Drive, flows westbound, turns right at the N. Broadway Ave. intersection, and clears Manor Drive. Therefore, the EB travel time is affected by two left turns, meanwhile the WB travel time has one left and one right turn movement.

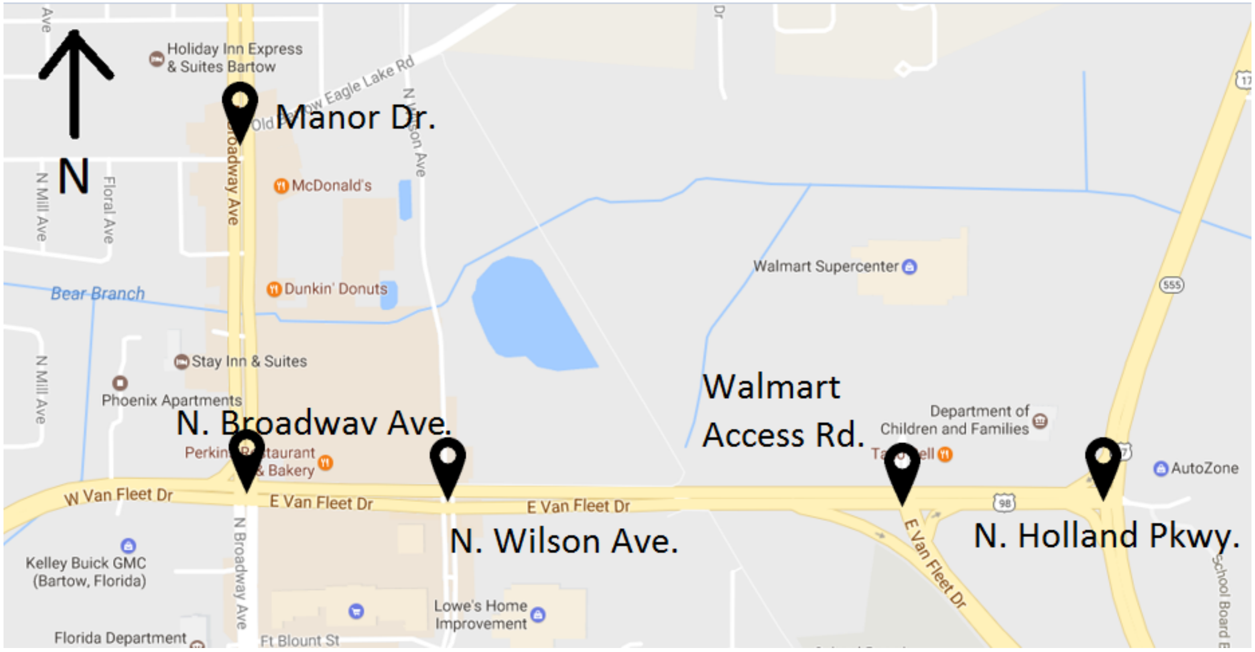
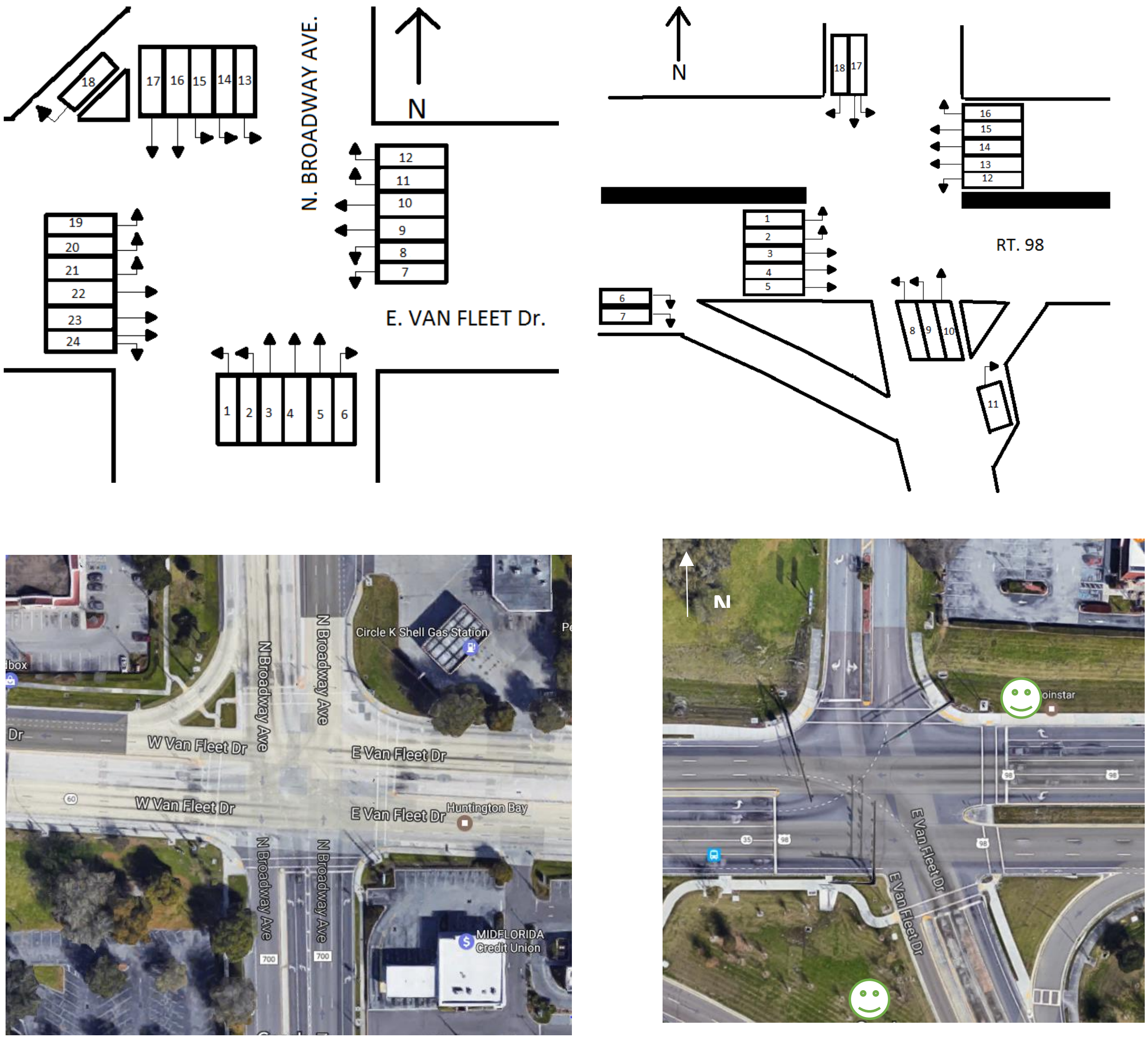


Figure H-1 Schematic of the E. Van Fleet and N. Broadway Ave, Bartow Corridor

Table H-1 Intersections along the E. Van Fleet and N. Broadway Ave, Bartow Corridor

EB	**Intersection**
1	Manor Drive and N. Broadway Ave
2	**N. Broadway Ave. and E. Van Fleet**
3	N. Wilson Ave. and E. Van Fleet
4	**Walmart Access Rd. and E. Van Fleet**
5	N Holland Pkwy and E. Van Fleet



(a) N. Broadway Ave. and E. Van Fleet

(b) Walmart Access Rd. and E. Van Fleet

Figure H-2 Schematic and Overview of Critical Intersections

Note: The smily faces denote the location of data collectors at each site

H.3 PERFORMANCE MEASURES

Five performance measures were evaluated: Link/Route Travel Time, Delay at Intersections, Queue Length (critical intersections), Queue-to-Lane Storage Ratio (critical intersections), and PCE Flows (critical intersections). For each performance measure, a comparison between the before and after data is conducted and the results of the differences (“after data” – “before data”) are presented.

H.3.1 ROUTE TRAVEL TIME

The average travel time (min) along the route was measured using a floating car. Rain fell throughout the first day of data collection (Feb 22) during the before study, with heavy rain during part of the afternoon data collection. Rainfall data (usclimatedata.com) confirms this event having over a half inch of rain. Table H-1.1 provides the route travel time for the before data provided separately for rain and no rain data collection periods.

Before Study (February 22nd & 23rd, 2017)

Table H-1.1 Route Travel Time (min)

Route TT (min)	AM Rain	AM No Rain	Off Peak Rain	Off Peak No Rain	PM Rain	PM No Rain	Average
E. Van Fleet Drive and N. Broadway Ave, EB	4.44	4.39	4.5	4.23	6.18	5.81	**4.81**
E. Van Fleet Drive and N. Broadway Ave, WB	4.7	4.28	4.58	4.66	4.86	5.62	**4.85**

After Study (March 28 & March 29, 2017)

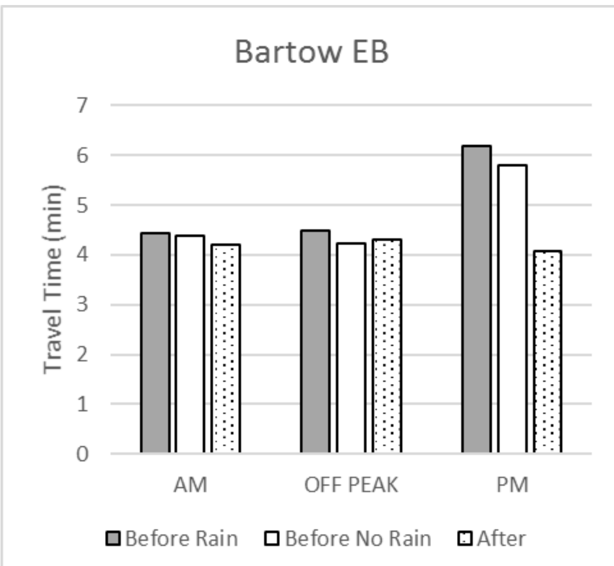
Table H-1.2 Route Travel Time (min)

Route TT (min)	AM	Off Peak	PM	Average
E. Van Fleet Drive and N. Broadway Ave, EB	4.20	4.32	4.06	**4.19**
E. Van Fleet Drive and N. Broadway Ave, WB	5.32	5.91	5.40	**5.54**

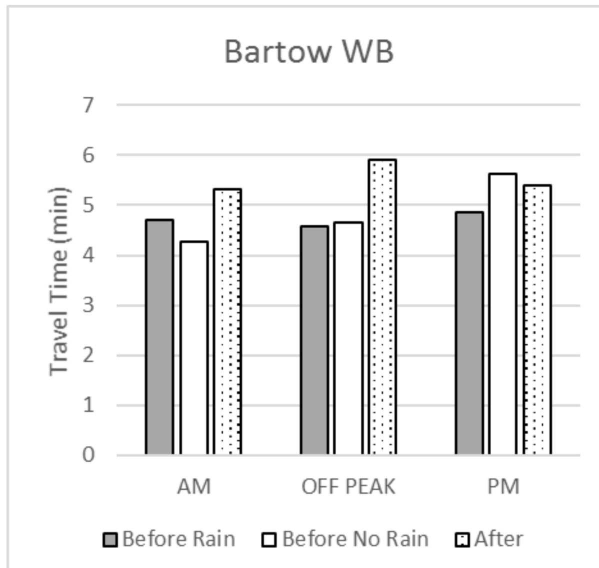
Comparison of Before and After Travel Times

Table H-1.3 Change in Percentage of Route Travel Time (After – Before-no rain)

Route TT (min)	AM	Off Peak	PM	Average
E. Van Fleet Drive and N. Broadway Ave, EB	-0.19 (-4.3%)	0.09 (2.1%)	-1.75 (-30.1%)	**-0.62 (-12.9%)**
E. Van Fleet Drive and N. Broadway Ave, WB	1.04 (24.3%)	1.25 (26.8%)	-0.22 (-3.9%)	**0.69 (14.2%)**



(a) E. Van Fleet Dr. and N. Broadway Ave., EB



(b) E. Van Fleet Dr. and N. Broadway Ave., WB

Figure H-1.1 Travel Times Along E. Van Fleet Dr. and N. Broadway Ave., Bartow

Discussion

The following can be concluded from the comparison of travel times:

- Overall, there was a reduction in travel time for the EB (-12.9%), and an increase for the WB (14.2%). Prior to the InSync installation, the average travel time for the two directions was similar, but it seems the new system favors the EB.
- The most significant improvement in travel time was for the EB direction during the PM peak (-30.1%).
- The most significant deterioration in travel time was for the WB direction during the Off-peak (26.8%).
- Travel times during the rain where somewhat higher, particularly in the EB, but overall rain effects were not dramatic.

H.3.2 DELAY

Delay (sec) at each intersection along the corridor was also obtained using the floating car measurements. Since the corridor includes turning movements, the graphs below indicate the type of the movement along which delays were obtained.

As shown in Table H-2.1 and Figure H-2.1, for the EB, the delay during the rain was sometimes higher and sometimes lower than the delay during good weather conditions. Generally, the delay was much higher during the rain at the N. Holland Pkwy intersection. For example, the delay during the PM peak was nearly three times more than the delay without rain. This may occur because this movement is a left turn, and it is possible that left turns are more affected by rain than the through movements.

Before Study (February 22nd & 23rd, 2017)

Table H-2.1 Delay (sec) for each Intersection Movement Along the EB Direction

Delay at Intersections (sec)	AM Rain	AM No Rain	Off Peak Rain	Off Peak No Rain	PM Rain	PM No Rain
Manor Drive and N. Broadway Ave	18.9	19.9	12.1	13.1	14.8	24.3
N. Broadway Ave. and E. Van Fleet	12.3	15.9	27.1	24.6	57.5	66.1
N. Wilson Ave. and E. Van Fleet	5.6	1.6	0.5	1.4	7.2	8
Walmart Access Rd. and E. Van Fleet	0	0	0.4	6	1.8	3.6
N. Holland Pkwy and E. Van Fleet	27.1	23.4	29.1	11	62	21.7

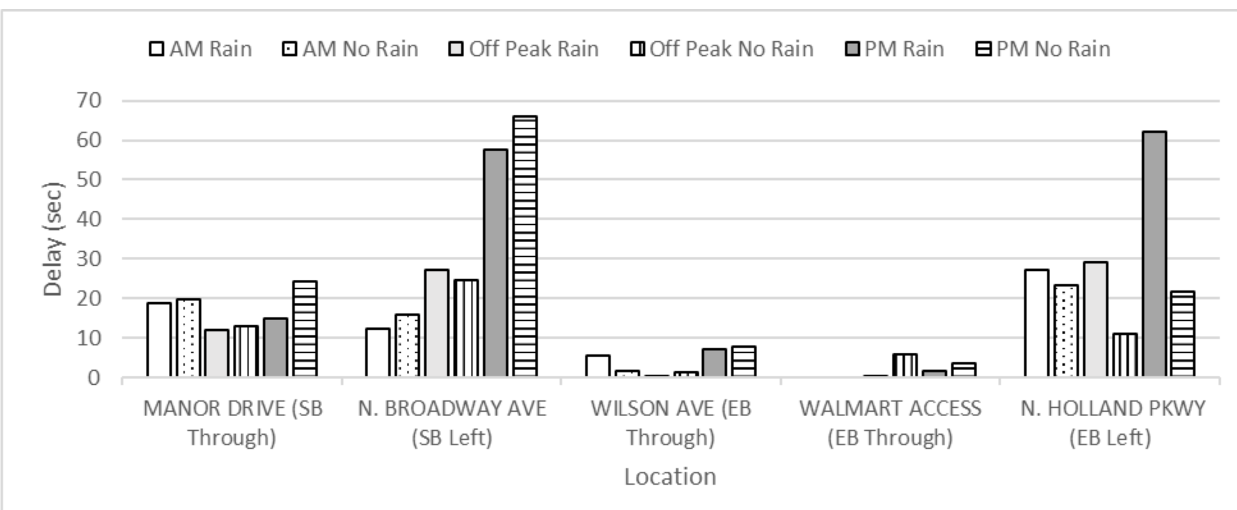


Figure H-2.1 Delay (sec) for each Intersection Movement Along the EB Direction

As shown in Table H-2.1 and Figure H-2.1, for the EB, the delay during the rain was sometimes higher and sometimes lower than the delay during good weather conditions. Generally, the delay was much higher during the rain at the N. Holland Pkwy intersection. For example, the delay during the PM peak was nearly three times more than the delay without rain. This may occur because this movement is a left turn, and it is possible that left turns are more affected by rain than the through movements.

Table H-2.2 Delay (sec) for each Intersection Movement Along the WB Direction

Delay at Intersections (sec)	AM Rain	AM No Rain	Off Peak Rain	Off Peak No Rain	PM Rain	PM No Rain
N. Holland Pkwy and E. Van Fleet	51.9	28.8	13.3	11.6	48.8	58.7
Walmart Access Rd. and E. Van Fleet	8.9	8.1	28	31	0	1.3
N. Wilson Ave. and E. Van Fleet	8	3.3	9.8	4.1	20	21.3
N. Broadway Ave. and E. Van Fleet	13.8	7.2	8	14.6	2.6	22.7
Manor Drive and N. Broadway Ave	0	4.9	10.3	4	1.6	5.2

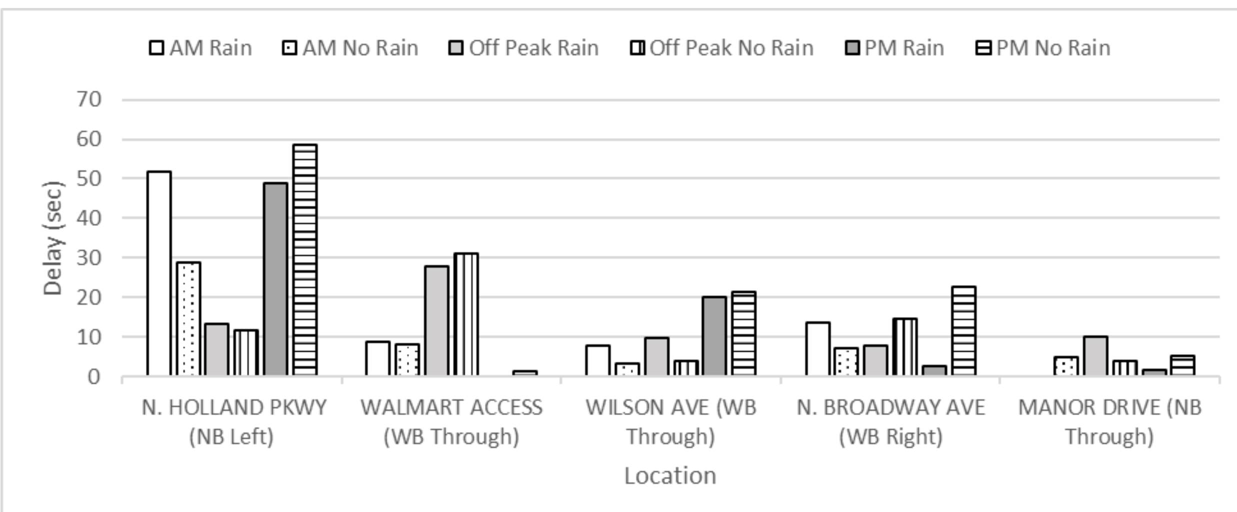


Figure H-2.2 Delay (sec) for each Intersection Movement Along the WB Direction

Table H-2.2 and Figure H-2.2 show the delays obtained by intersection for the WB. Similarly to the WB, delay during rain was sometimes higher and sometimes lower. For the N. Holland Pkwy and E. Van Fleet Drive, the delay during the AM was significantly higher when it rained (51.9 sec with rain vs. 28.8 sec without rain). Again, this movement is a left turn, and it is possible that left turns are more affected by rain than through movement.

After Study (March 28 & March 29, 2017)

Table H-2.3 and Figure H-2.3 summarize the delay data for the after study in the EB direction. As shown, there is little to negligible delay heading in the EB direction at the Wilson, Walmart, and N. Holland Parkway intersections. The N. Broadway Ave. and E. Van Fleet intersection has the highest overall delay of any period with the AM Peak being the worst.

Table H-2.3 Delay (sec) for each Intersection Movement Along the EB Direction – After Study

Delay at Intersections (sec)	AM	Off Peak	PM
Manor Drive and N. Broadway Ave	8.6	20.3	11.2
N. Broadway Ave. and E. Van Fleet	41.1	26.5	29.6
N. Wilson Ave. and E. Van Fleet	3.9	2.5	2.8
Walmart Access Rd. and E. Van Fleet	0.0	4.1	0.0
N. Holland Pkwy and E. Van Fleet	1.1	3.0	1.4

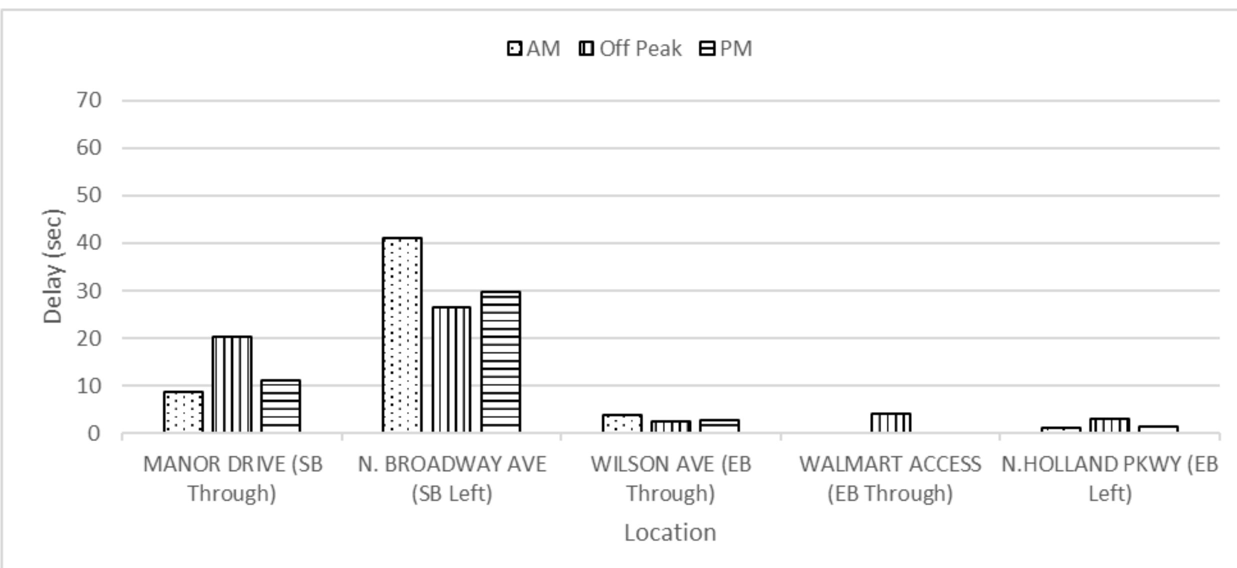


Figure H-2.3 Delay (sec) for each Intersection Movement Along the EB Direction – After Study

Table H-2.4 and Figure H-2.4 provide the delay data for the WB direction. As shown, the N. Holland Parkway intersection has the highest delays for all three analysis periods.

Table H-2.4 Delay (sec) for each Intersection Movement Along the WB Direction – After Study

Delay at Intersections (sec)	AM	Off Peak	PM
N. Holland Pkwy and E. Van Fleet	57.3	66.9	42.9
Walmart Access Rd. and E. Van Fleet	4.4	9.5	7.9
N. Wilson Ave. and E. Van Fleet	18.9	30.8	33.1
N. Broadway Ave. and E. Van Fleet	17.3	13.1	15.3
Manor Drive and N. Broadway Ave	7.4	4.8	10.4

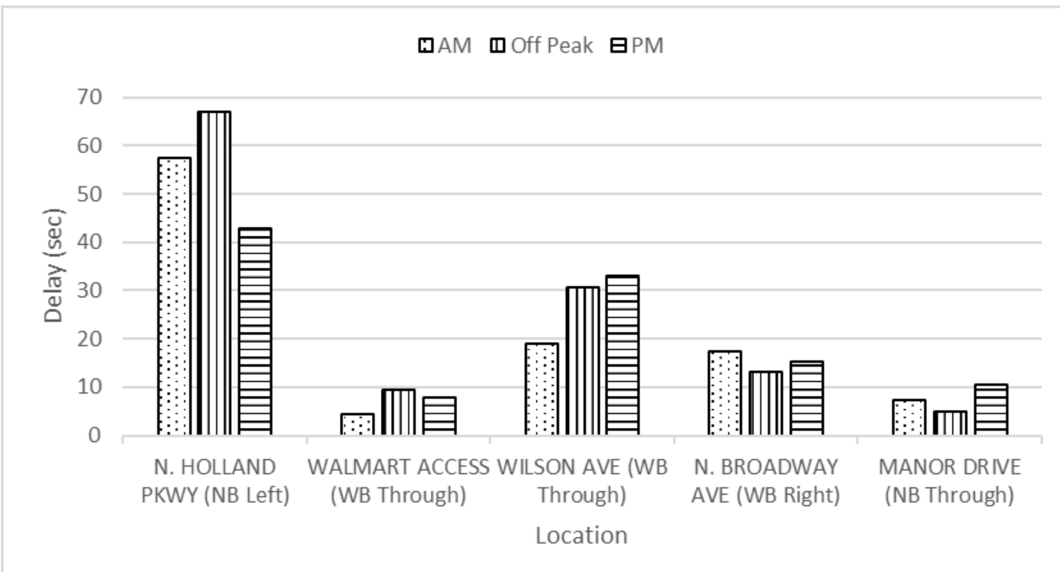


Figure H-2.4 Delay (sec) for each Intersection Movement Along the WB Direction – After Study

Comparisons of Before and After Intersection Delay Times

The differences in delay between before and after studies (considering only no-rain conditions) are shown in Table H-2.5 and Table H-2.6. The tables are color-coded as follows: green shows significant improvement, yellow shows modest change (either improvement or deterioration), and red shows significant deterioration in delay. Several gradations of each color are used to represent different variations within each classification.

Table H-2.5 Difference in Delay (sec) for Each Intersection Movement Along the EB Direction

Delay (sec)	AM Peak	Off Peak	PM Peak	Average
Manor Drive and N. Broadway Ave	-11.26	7.19	-13.11	**-5.73**
N. Broadway Ave. and E. Van Fleet	25.17	1.88	-36.50	**-3.15**
N. Wilson Ave. and E. Van Fleet	2.38	1.13	-5.18	**-0.56**
Walmart Access Rd. and E. Van Fleet	0.00	-1.94	-3.57	**-1.84**
N. Holland Pkwy and E. Van Fleet	-22.38	-8.00	-20.30	**-16.89**
Average	**-1.22**	**0.05**	**-15.73**	**-5.63**

Table H-2.6 Difference in Delay (sec) for Each Intersection Movement Along the WB Direction

Delay (sec)	AM Peak	Off Peak	PM Peak	Average
N. Holland Pkwy and E. Van Fleet	28.56	55.31	-15.73	**22.71**
Walmart Access Rd. and E. Van Fleet	-3.72	-21.50	6.55	**-6.22**
N. Wilson Ave. and E. Van Fleet	15.61	26.63	11.73	**17.99**
N. Broadway Ave. and E. Van Fleet	10.11	-1.50	-7.37	**0.41**
Manor Drive and N. Broadway Ave	2.56	0.81	5.25	**2.87**
Average	**10.62**	**11.95**	**0.08**	**7.55**

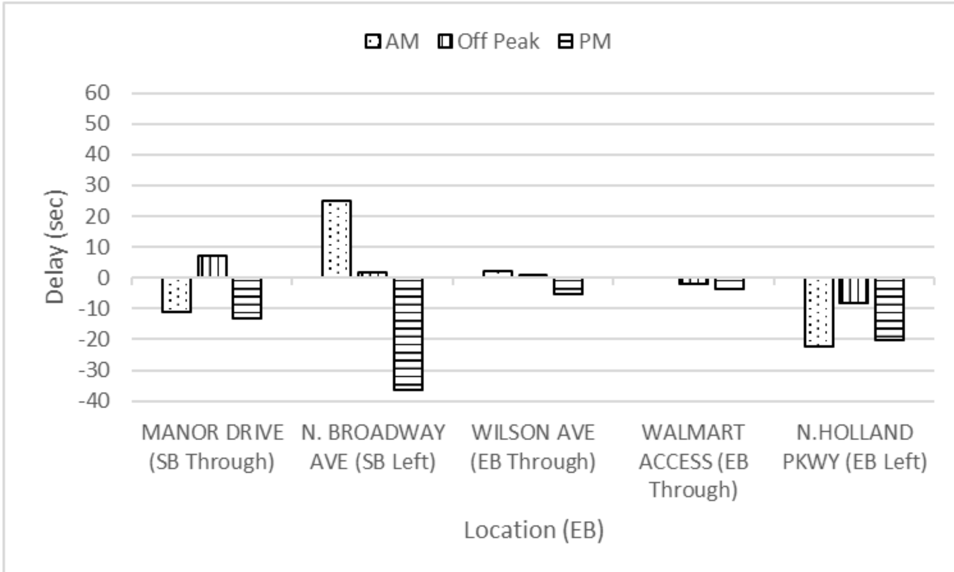


Figure H-2.5 Difference in Delay (sec) for Each Intersection Movement Along the EB Direction

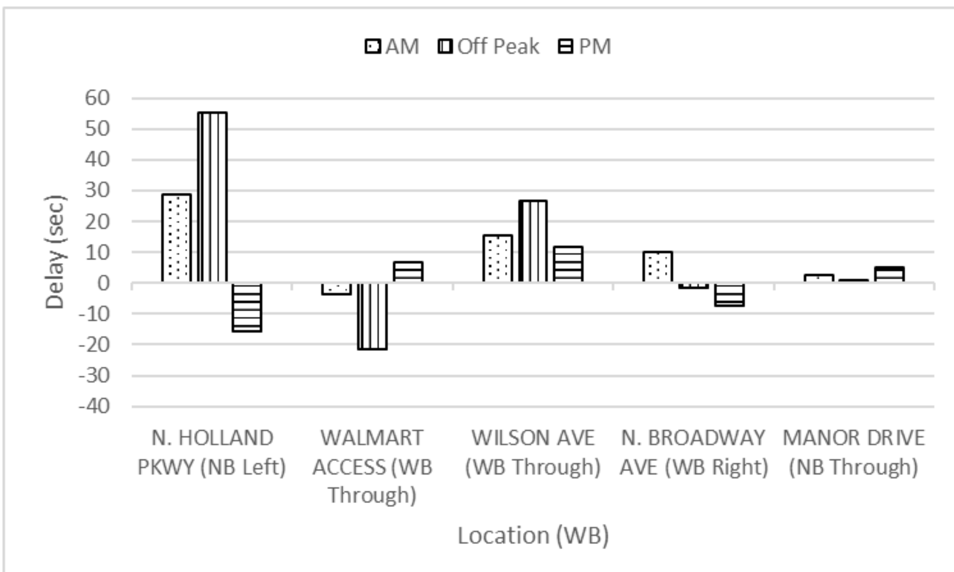


Figure H-2.6 Difference in Delay (sec) for Each Intersection Movement Along the WB Direction

Discussion

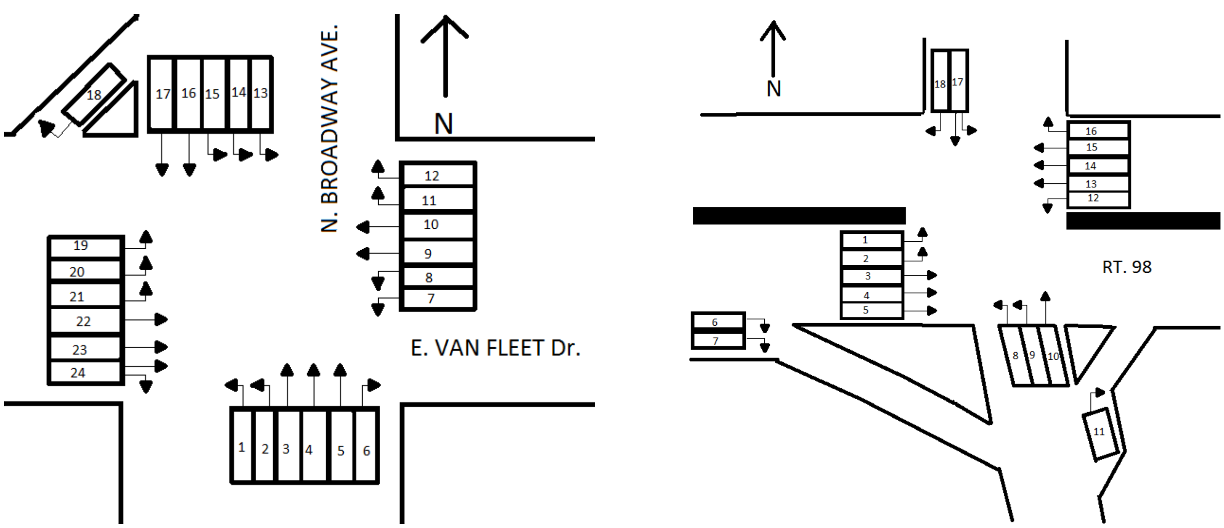
The following can be concluded from the comparison of delays:

- The InSync resulted in an overall decrease in travel time in the EB direction, and an increase in the WB direction. This may be a function of the priorities set up within the system, to favor the EB direction.
- The two movements with the most significant increase in delay are left turns: the left turn from N. Broadway onto E. Van Fleet in the EB direction, and the left turn from N. Holland onto E. Van Fleet in the WB direction. Again, this may be a function of priorities set up within InSync to favor through movements.

H.3.3 QUEUE LENGTH

Queue length (number of vehicles/lane) is presented by movement and by time period. This measure is used to evaluate oversaturated conditions at the critical intersections along the study corridors. Note that the queue length reported here is the observed maximum number of vehicles queued during each cycle, and does not represent the total number of vehicles that may have stopped during the cycle. During some time periods, because of cycle failure, vehicles need to stop multiple times before passing through the intersection.

Figure H-3.1 presents the schematic of the lane configurations at the two critical intersections. Queue length is reported for each of these lanes.



(a) N. Broadway Ave. and E. Van Fleet (b) Walmart Access Rd. and E. Van Fleet

Figure H-3.1 Lane Configuration of Critical Intersections

Before Study (February 22 & February 23, 2017)

As shown below, for the N. Broadway and E. Van Fleet intersection, the longest queues were observed in the SB, with queues reaching up to 11 vehicles for the SB left turn movement. For the Walmart Access Rd. intersection, the longest queue observed was 15 vehicles, for the NB left movement. Note that the observers did not have visibility beyond roughly 15 vehicles, thus queues during some of these intervals may have been longer.

Table H-3.1 Average Queue Length by Lane (#vehs/lane) at N. Broadway Ave. and E. Van Fleet Drive – Before Study

Time period	Time segment	NB						WB (Main)						SB						EB (Main)					
		Left		Through			Right	Left		Through		Right		Left			Through		Right	Left			Through		Through /Right
LANE NUMBER		**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**	**13**	**14**	**15**	**16**	**17**	**18**	**19**	**20**	**21**	**22**	**23**	**24**
AM PEAK	**1**	0	2	6	6	2	1	8	6	6	5	3	3	9	11	9	8	4	2	1	2	3	5	5	4
	2	1	0	4	4	2	0	6	7	3	4	2	4	6	8	9	10	8	1	1	1	2	4	3	3
	3	0	1	2	1	2	0	2	3	4	4	2	3	5	7	9	8	8	0	1	2	2	4	3	3
	4	0	1	4	5	2	0	2	3	3	4	3	5	5	5	5	5	4	0	1	4	2	3	3	2
	Average	**0**	**1**	**4**	**4**	**2**	**1**	**5**	**5**	**4**	**4**	**3**	**3**	**6**	**8**	**8**	**8**	**6**	**1**	**1**	**2**	**3**	**4**	**3**	**3**
OFF PEAK	**1**	0	1	4	3	2	0	1	3	2	2	1	1	2	3	3	3	3	1	1	2	2	2	3	3
	2	1	1	4	3	2	0	1	3	2	2	2	3	3	5	6	2	2	0	1	1	2	3	3	2
	3	1	1	4	3	3	0	1	1	1	2	2	2	2	3	4	3	2	0	1	1	2	3	2	3
	4	1	2	4	4	3	0	1	3	2	2	2	2	2	3	5	2	2	1	1	1	2	2	3	3
	Average	**0**	**1**	**4**	**3**	**2**	**0**	**1**	**2**	**2**	**2**	**2**	**2**	**2**	**3**	**4**	**2**	**2**	**0**	**1**	**1**	**2**	**3**	**3**	**3**
PM PEAK	**1**	1	2	8	4	4	1	1	2	4	5	3	4	5	7	9	6	6	1	2	3	3	4	3	3
	2	1	2	8	5	5	1	1	3	4	4	3	4	6	8	10	6	6	1	2	4	4	5	4	5
	3	1	2	7	6	7	1	2	4	4	3	4	4	8	8	11	5	5	1	3	4	5	6	5	8
	4	1	1	6	4	3	0	1	3	2	3	2	3	5	8	10	4	4	0	1	3	3	4	3	4

Time period	Time segment	NB						WB (Main)						SB						EB (Main)					
		Left		Through			Right	Left		Through		Right		Left			Through		Right	Left			Through		Through /Right
LANE NUMBER		**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**	**13**	**14**	**15**	**16**	**17**	**18**	**19**	**20**	**21**	**22**	**23**	**24**
	Average	**1**	**2**	**7**	**5**	**5**	**1**	**1**	**3**	**3**	**4**	**3**	**4**	**6**	**8**	**10**	**5**	**5**	**1**	**2**	**3**	**4**	**5**	**4**	**5**

Table H-3.2 Average Queue Length (#vehs/lane) at N. Broadway Ave. and E. Van Fleet Drive – Before Study

Time Period	NB			WB (Main)			SB			EB (Main)		
	Left	**Through**	**Right**	**Left**	**Through**	**Right**	**Left**	**Through**	**Right**	**Left**	**Through**	**Right**
AM Peak	1	3	1	5	4	3	7	7	1	2	3	0
OFF Peak	1	3	0	2	2	2	3	2	0	1	3	0
PM Peak	1	6	1	2	3	3	8	5	1	3	4	0

Note: Due to severe rain, all data collection was suspended for the third time segment of the PM peak period. Missing data of queues and volumes were interpolated between bordering data sets.

Table H-3.3 Average Queue Length by Lane (#vehs/lane) at Walmart Access Rd. and E. Van Fleet Drive – Before Study

Time period	Time segment	EB (Main)							NB				WB (Main)					SB	
		Left		Through			Right		Left		Through	Right	Left	Through			Right	Through/Left	Right
LANE NUMBER		**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**	**13**	**14**	**15**	**16**	**17**	**18**
AM PEAK	**1**	1	1	2	3	7	0	0	12	12	1	1	0	4	3	1	0	2	2
	2	2	1	0	2	4	0	0	14	14	1	0	0	9	5	2	0	1	1
	3	2	3	1	2	5	0	0	8	10	0	0	0	7	3	2	0	2	1
	4	1	2	1	2	4	0	0	9	9	1	0	1	4	2	2	0	3	0
	Average	**1**	**2**	**1**	**2**	**5**	**0**	**0**	**11**	**11**	**1**	**0**	**0**	**6**	**3**	**2**	**0**	**2**	**1**
OFF PEAK	**1**	2	2	3	3	5	0	0	9	9	1	0	1	3	2	2	0	1	1
	2	3	2	1	3	4	0	0	9	9	0	0	0	2	3	2	0	3	3
	3	3	3	3	3	4	0	0	10	11	1	0	1	3	3	4	0	3	2
	4	3	4	3	4	6	0	0	11	12	0	0	1	4	3	3	0	4	3
	Average	**3**	**3**	**2**	**3**	**5**	**0**	**0**	**10**	**10**	**1**	**0**	**1**	**3**	**3**	**3**	**0**	**3**	**2**
PM PEAK	**1**	3	3	5	5	5	0	0	14	14	1	0	1	5	3	4	0	4	7
	2	3	4	4	4	5	0	0	13	12	0	0	1	4	4	4	0	7	3
	3	4	5	6	6	5	0	0	15	14	1	0	2	7	7	4	0	4	5
	4	3	3	4	4	7	0	0	12	10	1	0	1	7	6	6	0	7	5
	Average	**3**	**4**	**4**	**5**	**5**	**0**	**0**	**13**	**13**	**1**	**0**	**1**	**6**	**5**	**4**	**0**	**6**	**5**

Note: The queue lengths of 15 vehicles for the NB left represent the maximum number of vehicles within the observers' sight distance

Table H-3.4 Average Queue Length (#vehs/lane) at Walmart Access Rd. and E. Van Fleet Drive – Before Study

Time Period	EB (Main)			NB			WB (Main)			SB		
	Left	Through	Right	Left	Through	Right	Left	Through	Right	Left	Through	Right
AM Peak	2	3	0	11	1	0	0	4	0	1	1	1
OFF Peak	3	4	0	10	1	0	1	3	0	2	1	2
PM Peak	4	5	0	13	1	0	1	5	0	4	2	5

Note: The queue lengths of 15 vehicles for the NB left represent the maximum number of vehicles within the observers' sight distance

After Study (March 28 & March 29, 2017)

Table H-3.5 Average Queue Length by Lane (#vehs/lane) at N. Broadway Ave. and E. Van Fleet Drive – After Study

Time period	Time segment	NB						WB (Main)						SB						EB (Main)					
		Left		Through			Right	Left		Through		Right		Left			Through		Right	Left			Through		Through/Right
LANE NUMBER		**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**	**13**	**14**	**15**	**16**	**17**	**18**	**19**	**20**	**21**	**22**	**23**	**24**
AM PEAK	**1**	1	1	5	4	3	0	1	5	6	7	5	4	5	10	10	10	9	1	3	2	3	5	3	4
	2	1	2	4	3	4	0	5	10	6	5	2	2	4	5	8	10	9	0	2	1	3	5	4	5
	3	0	3	4	2	4	1	3	6	4	5	1	1	3	5	6	9	10	0	2	2	2	4	4	4
	4	1	2	4	3	3	0	1	4	4	4	1	1	2	4	6	6	6	0	1	2	2	3	3	5
	Average	**1**	**2**	**4**	**3**	**4**	**0**	**3**	**6**	**5**	**5**	**2**	**2**	**3**	**6**	**8**	**9**	**9**	**0**	**2**	**2**	**2**	**4**	**3**	**5**
	1	2	0	5	4	4	0	2	3	7	7	9	8	7	9	9	5	5	1	2	2	1	5	4	4

Time period	Time segment	NB						WB (Main)						SB						EB (Main)					
		Left		Through			Right	Left		Through		Right		Left			Through		Right	Left			Through		Through/Right
LANE NUMBER		**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**	**13**	**14**	**15**	**16**	**17**	**18**	**19**	**20**	**21**	**22**	**23**	**24**
OFF PEAK	**2**	2	2	3	3	3	1	1	2	4	5	6	6	9	11	10	3	3	0	2	2	0	2	2	2
	3	1	1	3	4	3	0	1	2	4	4	3	2	7	8	7	3	2	0	0	1	1	4	3	4
	4	1	0	4	3	5	0	2	3	4	4	4	3	6	9	8	2	2	0	1	1	1	2	3	3
	Average	**1**	**1**	**4**	**3**	**4**	**0**	**2**	**2**	**5**	**5**	**5**	**5**	**7**	**9**	**8**	**3**	**3**	**0**	**1**	**2**	**1**	**3**	**3**	**3**
PM PEAK	**1**	2	2	11	7	7	0	1	3	4	4	2	3	5	8	9	6	5	0	2	2	3	8	7	6
	2	0	3	11	8	6	1	1	2	3	3	2	3	6	7	10	4	4	0	3	3	4	4	3	5
	3	1	2	17	15	13	0	1	2	4	5	3	4	4	7	9	4	3	0	3	3	4	6	5	6
	4	1	2	5	3	3	0	1	2	4	5	3	4	4	6	9	3	4	0	3	3	4	7	5	6
	Average	**1**	**2**	**11**	**8**	**7**	**0**	**1**	**2**	**4**	**4**	**3**	**3**	**5**	**7**	**9**	**4**	**4**	**0**	**3**	**2**	**4**	**6**	**5**	**6**

Table H-3.6 Average Queue Length (#vehs/lane) at N. Broadway Ave. and E. Van Fleet Drive – After Study

Time Period	NB			WB (Main)			SB			EB (Main)		
	Left	Through	Right	Left	Through	Right	Left	Through	Right	Left	Through	Right
AM Peak	1	4	0	4	5	2	6	9	0	2	4	0
OFF Peak	1	4	0	2	5	5	8	3	0	1	3	0
PM Peak	1	9	0	2	4	3	7	4	0	3	6	0

Table H-3.7 Average Queue Length by Lane (#vehs/lane) at Walmart Access Rd. and E. Van Fleet – After Study

Time period	Time segment	EB (Main)							NB				WB (Main)					SB	
		Left		Through			Right		Left		Through	Right	Left	Through			Right	Through/Left	Right
LANE NUMBER		1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18
AM PEAK	1	1	2	2	2	4	0	0	13	13	1	0	1	6	4	4	0	2	1
	2	1	1	1	1	2	0	0	12	10	0	0	2	9	3	2	0	1	0
	3	1	1	1	1	3	0	0	10	8	0	0	2	6	3	4	0	2	0
	4	2	2	1	2	2	0	0	8	8	1	0	1	3	2	2	0	1	0
	Average	**1**	**1**	**1**	**1**	**3**	**0**	**0**	**11**	**10**	**0**	**0**	**1**	**6**	**3**	**3**	**0**	**2**	**0**
OFF PEAK	1	2	2	0	1	3	0	0	9	8	0	0	1	4	2	2	0	2	0
	2	3	2	1	2	3	0	0	9	7	1	0	1	5	3	3	0	2	1
	3	3	3	2	2	3	0	0	8	7	2	0	1	3	2	2	0	3	0
	4	2	2	1	1	4	0	0	11	10	0	0	2	3	4	4	0	3	0
	Average	**2**	**2**	**1**	**2**	**3**	**0**	**0**	**9**	**8**	**1**	**0**	**1**	**4**	**3**	**3**	**0**	**3**	**0**
PM PEAK	1	3	3	2	3	3	0	0	9	7	2	0	1	6	7	6	0	4	1
	2	4	5	3	2	2	0	0	12	11	2	0	0	4	5	5	0	5	3
	3	4	3	3	3	4	0	0	10	10	2	0	1	4	4	6	0	4	2
	4	3	3	1	2	2	0	0	9	8	2	0	1	4	4	4	0	5	1
	Average	**3**	**4**	**2**	**2**	**3**	**0**	**0**	**10**	**9**	**2**	**0**	**1**	**4**	**5**	**5**	**0**	**4**	**1**

Note: The queue lengths of 15 vehicles for the NB left represent the maximum number of vehicles within the observers' sight distance

Table H-3.8 Average Queue Length (#vehs/lane) at Walmart Access Rd. and E. Van Fleet – After Study

Time Period	**EB (Main)**			**NB**			**WB (Main)**			**SB**		
	Left	**Through**	**Right**	**Left**	**Through**	**Right**	**Left**	**Through**	**Right**	**Left**	**Through**	**Right**
AM Peak	1	2	0	10	0	0	1	4	0	1	0	0
OFF Peak	2	2	0	9	1	0	1	3	0	2	1	0
PM Peak	3	2	0	9	2	0	1	5	0	3	1	1

Note: The queue lengths of 15 vehicles for the NB left represent the maximum number of vehicles within the observers' sight distance

Comparison of Before and After Queue Lengths for Critical Intersections

The differences in queue length between the before and after measurements are shown in Table H-3.9 to Table H-3.12. The tables are color-coded as follows: green shows significant improvement, yellow shows modest change (either improvement or deterioration), and red shows significant deterioration in delay. Several gradations of each color are used to represent different variations within each classification.

Table H-3.9 Difference in Avg. Queue Length by Lane (#vehs/lane) at N. Broadway Ave. & E. Van Fleet Drive

Time Period	NB						WB (Main)						SB							EB (Main)					Average Queue
	Left		Through			Right	Left	Through			Right		Left		Through			Right		Left		Through		Through / Right	
Lane Number	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	
AM Peak	0.10	0.98	0.44	-0.84	1.48	-0.21	-2.06	1.25	1.08	0.83	-0.62	-1.74	-2.62	-1.64	-0.41	1.09	2.49	-0.55	0.65	-0.71	-0.14	0.07	0.05	1.56	0.02
Off Peak	0.87	-0.58	-0.10	0.25	1.16	0.22	0.41	-0.25	3.10	3.01	3.60	2.99	4.91	5.70	3.98	0.78	0.73	0.02	0.61	0.15	-1.03	0.58	0.04	0.78	1.33
PM Peak	-0.17	0.37	3.50	3.51	2.43	-0.61	-0.41	-0.39	0.52	0.27	-0.23	-0.18	-1.44	-0.85	-0.76	-1.05	-1.05	-0.44	0.65	-0.82	0.09	1.27	1.33	0.59	0.26

Table H-3.10 Difference in Avg. Queue Length (#vehs/lane) at N. Broadway Ave. & E. Van Fleet Drive

Time Period	NB			WB (Main)			SB			EB (Main)		
	Left	Through	Right	Left	Through	Right	Left	Through	Right	Left	Through	Right
AM Peak	0.54	0.36	-0.21	-0.41	0.96	-1.18	-1.56	1.79	-0.55	-0.07	0.48	0.24
OFF Peak	0.15	0.44	0.22	0.08	3.05	3.29	4.86	0.76	0.02	-0.09	0.44	0.08
PM Peak	0.10	3.15	-0.61	-0.40	0.40	-0.20	-1.02	-1.05	-0.44	-0.03	1.03	0.10

Table H-3.11 Difference in Avg. Queue Length by Lane (#vehs/lane) at Walmart Access Rd and E. Van Fleet Drive

Time Period	EB (Main)							NB				WB (Main)					SB		Average Queue
	Left		Through			Right		Left		Through	Right	Left	Through			Right	Through/Left	Right	
Lane Number	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**	**13**	**14**	**15**	**16**	**17**	**18**	
AM Peak	-0.12	-0.56	0.04	-0.55	-2.16	0.00	0.00	0.08	-1.76	-0.46	-0.20	1.05	-0.02	-0.21	0.92	0.11	-0.23	-0.85	-0.27
Off Peak	-0.19	-0.75	-1.31	-1.70	-2.01	0.00	0.00	-0.63	-2.15	0.36	0.00	0.61	0.61	-0.15	-0.08	0.05	-0.36	-2.04	-0.54
PM Peak	0.08	-0.17	-2.20	-2.45	-2.62	0.00	0.00	-3.58	-3.54	1.02	-0.04	-0.09	-1.55	0.22	1.04	-0.08	-1.24	-3.24	-1.03

Table H-3.12 Difference in Average Queue Length (#vehs/lane) at Walmart Access Rd. and E. Van Fleet Drive

Time Period	EB (Main)			NB			WB (Main)			SB		
	Left	Through	Right	Left	Through	Right	Left	Through	Right	Left	Through	Right
AM Peak	-0.34	-0.89	0.00	-0.84	-0.46	-0.20	1.05	0.23	0.11	0.06	-0.29	-0.85
OFF Peak	-0.47	-1.67	0.00	-1.39	0.36	0.00	0.61	0.13	0.05	-0.14	-0.22	-2.04
PM Peak	-0.04	-2.42	0.00	-3.56	1.02	-0.04	-0.09	-0.10	-0.08	-0.75	-0.49	-3.24

Discussion

The following can be concluded from the comparison of queue length:

- The SB left at N. Broadway Ave. and E. Van Fleet Drive intersection, which had one of the longest queues during the AM and PM peak periods in the before study, had a reduction in queue length for those time periods. The queues at the NB left at the Walmart Access Rd. and E. Van Fleet intersection, which also had long queues in the before study, were also reduced, particularly during PM peak period.
- The NB through at N. Broadway Ave. and E. Van Fleet Drive intersection had a significant increase in queues, particularly for the PM peak period.

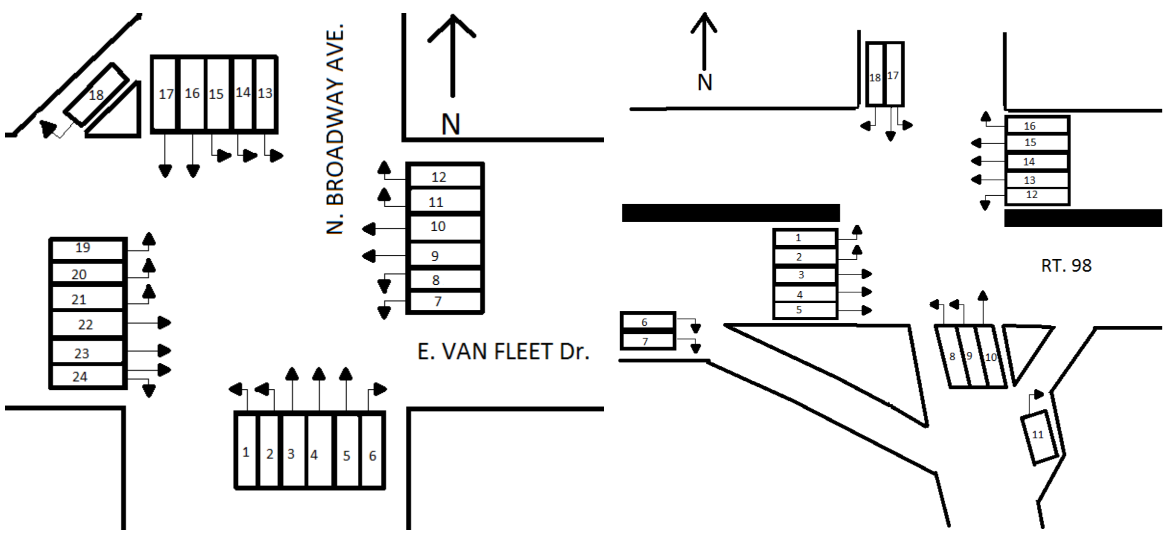
Some of the queue increases observed were for movements that initially had relatively low queue lengths (for example, some of the WB lanes at the Walmart Access Rd. and E. Van Fleet intersection). In those cases, a large percent increase reflects small absolute change in terms of number of vehicles.

H.3.4 QUEUE TO LANE STORAGE RATIO

In addition to queue length, it is important to assess any impact to adjacent lanes or to upstream facilities. The queue to link/lane ratio is used to establish the likelihood of spillback, which is presented in this section by movement and by time period.

The following assumptions are used:

- The storage capacity is estimated as the maximum number of vehicles in the lane.
- The queue to link/lane storage ratio is estimated as 1 if the observer reported "spillback", and as 0.8 if reported as the maximum number of vehicles in the sight distance.
- Queue to lane storage ratios over 80% are highlighted in yellow, as they represent conditions with a high probability for spillback.



(a) N. Broadway Ave. and E. Van Fleet (b) Walmart Access Rd. and E. Van Fleet

Figure H-4.1 Lane Configuration of Critical Intersections

Before Study (February 22nd & 23rd, 2017)

Table H-4.1 Average Queue Storage Ratio by Lane and by Period at N. Broadway Ave. and E. Van Fleet Drive – Before Study

Time period	Time segment	NB						WB (Main)						SB						EB (Main)					
		Left		Through			Right	Left		Through		Right		Left			Through		Right	Left			Through		Through/ Right
LANE NUMBER		1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24
AM Peak	1	0.03	0.1	0.33	0.33	0.13	0.11	0.38	0.32	0.31	0.26	0.16	0.13	0.51	0.6	0.49	0.44	0.22	0.08	0.12	0.18	0.19	0.24	0.26	0.21
	2	0.05	0.02	0.21	0.23	0.12	0.01	0.31	0.35	0.15	0.21	0.11	0.18	0.32	0.43	0.49	0.55	0.47	0.07	0.07	0.11	0.13	0.19	0.15	0.16
	3	0.02	0.07	0.12	0.06	0.11	0.04	0.12	0.15	0.2	0.19	0.09	0.13	0.25	0.37	0.51	0.43	0.46	0.02	0.11	0.15	0.12	0.21	0.14	0.17
	4	0.01	0.03	0.26	0.26	0.13	0.04	0.11	0.16	0.16	0.21	0.16	0.25	0.27	0.28	0.29	0.25	0.21	0.01	0.1	0.29	0.13	0.14	0.13	0.11
	Average	**0.03**	**0.05**	**0.23**	**0.22**	**0.12**	**0.05**	**0.23**	**0.25**	**0.21**	**0.22**	**0.13**	**0.17**	**0.34**	**0.42**	**0.44**	**0.42**	**0.34**	**0.05**	**0.1**	**0.18**	**0.14**	**0.2**	**0.17**	**0.16**
OFF Peak	1	0.02	0.07	0.21	0.2	0.13	0.02	0.06	0.13	0.11	0.09	0.03	0.06	0.12	0.15	0.17	0.14	0.16	0.03	0.05	0.13	0.13	0.12	0.16	0.14
	2	0.03	0.08	0.24	0.15	0.12	0.01	0.04	0.13	0.08	0.09	0.08	0.14	0.17	0.26	0.31	0.13	0.13	0.01	0.06	0.11	0.11	0.14	0.17	0.09
	3	0.03	0.06	0.21	0.18	0.18	0.01	0.05	0.06	0.04	0.08	0.1	0.08	0.12	0.16	0.22	0.14	0.12	0.02	0.06	0.09	0.09	0.15	0.11	0.14

Time period	Time segment	NB						WB (Main)						SB						EB (Main)					
		Left		Through			Right	Left		Through		Right		Left			Through		Right	Left			Through		Through/ Right
LANE NUMBER		**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**	**13**	**14**	**15**	**16**	**17**	**18**	**19**	**20**	**21**	**22**	**23**	**24**
	4	0.03	0.1	0.21	0.23	0.15	0	0.06	0.17	0.1	0.08	0.11	0.12	0.13	0.18	0.26	0.13	0.13	0.03	0.06	0.11	0.12	0.09	0.13	0.14
	Average	**0.03**	**0.08**	**0.22**	**0.19**	**0.14**	**0.01**	**0.05**	**0.12**	**0.08**	**0.09**	**0.08**	**0.1**	**0.13**	**0.19**	**0.24**	**0.13**	**0.13**	**0.02**	**0.06**	**0.11**	**0.11**	**0.13**	**0.14**	**0.13**
PM Peak	**1**	0.08	0.12	0.49	0.24	0.25	0.1	0.06	0.11	0.18	0.23	0.13	0.22	0.29	0.4	0.49	0.32	0.32	0.03	0.15	0.22	0.17	0.2	0.16	0.16
	2	0.06	0.11	0.46	0.29	0.31	0.1	0.07	0.14	0.18	0.19	0.17	0.2	0.36	0.43	0.55	0.31	0.31	0.04	0.2	0.27	0.24	0.26	0.2	0.27
	3	0.04	0.11	0.42	0.35	0.38	0.1	0.08	0.18	0.18	0.16	0.2	0.18	0.43	0.45	0.6	0.29	0.29	0.05	0.26	0.31	0.29	0.31	0.23	0.38
	4	0.06	0.08	0.33	0.24	0.2	0.04	0.06	0.14	0.09	0.17	0.08	0.14	0.28	0.46	0.58	0.24	0.24	0.01	0.07	0.2	0.16	0.19	0.16	0.21
	Average	**0.06**	**0.1**	**0.42**	**0.28**	**0.29**	**0.09**	**0.06**	**0.14**	**0.16**	**0.19**	**0.14**	**0.18**	**0.34**	**0.43**	**0.55**	**0.29**	**0.29**	**0.03**	**0.17**	**0.25**	**0.21**	**0.24**	**0.19**	**0.25**

Table H-4.2 Number of Cycles with Spillback at N. Broadway Ave. and E. Van Fleet Drive – Before Study

Time period	Time segment	# cycles in 15 min	NB						WB (Main)						SB						EB (Main)					
			Left		Through			Right	Left		Through		Right		Left			Through		Right	Left			Through		Through/ Right
LANE NUMBER			**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**	**13**	**14**	**15**	**16**	**17**	**18**	**19**	**20**	**21**	**22**	**23**	**24**
AM Peak	**1**	7	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
	2	7	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
	3	7	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
	4	7	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
	Average		**0**	**0**	**0**	**0**	**0**	**0**	**0**	**0**	**0**	**0**	**0**	**0**	**0**	**0**	**0**	**0**	**0**	**0**	**0**	**0**	**0**	**0**	**0**	**0**
OFF Peak	**1**	8	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
	2	8	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
	3	8	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
	4	8	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
	Average		**0**	**0.25**	**0**	**0**	**0**	**0**	**0**	**0**	**0**	**0**	**0**	**0**	**0**	**0**	**0**	**0**	**0**	**0**	**0**	**0**	**0**	**0**	**0**	**0**
PM Peak	**1**	6	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
	2	6	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
	3	6	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
	4	7	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

Time period	Time segment	# cycles in 15 min	NB						WB (Main)						SB						EB (Main)					
			Left		Through			Right	Left		Through		Right		Left			Through		Right	Left			Through		Through/ Right
LANE NUMBER			1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24
	Average		**0**	**0**	**0**	**0**	**0**	**0**	**0**	**0**	**0**	**0**	**0**	**0**	**0**	**0**	**0**	**0**	**0**	**0**	**0**	**0**	**0**	**0**	**0**	**0**

Table H-4.3 Average Queue Storage Ratio by Lane and by Period at Walmart Access Rd. and E. Van Fleet – Before Study

Time period	Time segment	EB (Main)							NB				WB (Main)					SB	
		Left		Through			Right		Left		Through	Right	Left	Through			Right	Through/ Left	Right
LANE NUMBER		1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18
AM PEAK	1	0.07	0.06	0.11	0.13	0.33	0.00	0.00	0.77	0.82	0.08	0.03	0.01	0.18	0.15	0.07	0.00	0.08	0.11
	2	0.11	0.07	0.02	0.09	0.22	0.00	0.00	0.94	0.93	0.06	0.02	0.02	0.41	0.20	0.09	0.00	0.06	0.05
	3	0.09	0.13	0.05	0.10	0.27	0.00	0.00	0.55	0.68	0.02	0.00	0.03	0.34	0.17	0.11	0.01	0.09	0.03
	4	0.06	0.10	0.05	0.08	0.21	0.00	0.00	0.62	0.61	0.05	0.00	0.04	0.18	0.11	0.09	0.00	0.13	0.02
	Average	**0.08**	**0.09**	**0.06**	**0.10**	**0.26**	**0.00**	**0.00**	**0.72**	**0.76**	**0.05**	**0.01**	**0.02**	**0.28**	**0.16**	**0.09**	**0.00**	**0.09**	**0.05**
OFF PEAK	1	0.11	0.12	0.15	0.16	0.26	0.00	0.00	0.58	0.62	0.06	0.00	0.05	0.14	0.09	0.10	0.02	0.07	0.06
	2	0.16	0.12	0.06	0.16	0.20	0.00	0.00	0.61	0.58	0.03	0.00	0.03	0.11	0.14	0.11	0.01	0.13	0.16
	3	0.17	0.17	0.13	0.17	0.22	0.00	0.00	0.68	0.73	0.05	0.00	0.08	0.14	0.17	0.20	0.00	0.17	0.11
	4	0.15	0.21	0.13	0.19	0.32	0.00	0.00	0.76	0.80	0.00	0.00	0.04	0.22	0.17	0.15	0.03	0.22	0.15

Time period	Time segment	EB (Main)							NB				WB (Main)					SB	
		Left		Through			Right		Left		Through	Right	Left	Through			Right	Through/ Left	Right
LANE NUMBER		1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18
	Average	**0.15**	**0.15**	**0.12**	**0.17**	**0.25**	**0.00**	**0.00**	**0.65**	**0.68**	**0.03**	**0.00**	**0.05**	**0.15**	**0.14**	**0.14**	**0.02**	**0.15**	**0.12**
PM PEAK	1	0.17	0.15	0.26	0.26	0.26	0.00	0.00	0.96	0.91	0.10	0.01	0.04	0.23	0.16	0.20	0.03	0.21	0.33
	2	0.18	0.20	0.18	0.19	0.24	0.00	0.00	0.86	0.80	0.02	0.00	0.08	0.22	0.20	0.18	0.00	0.34	0.16
	3	0.23	0.23	0.29	0.30	0.23	0.00	0.00	0.98	0.96	0.05	0.00	0.13	0.34	0.33	0.20	0.00	0.19	0.23
	4	0.16	0.17	0.18	0.22	0.33	0.00	0.00	0.77	0.67	0.06	0.00	0.05	0.36	0.28	0.28	0.01	0.37	0.23
	Average	**0.18**	**0.19**	**0.22**	**0.24**	**0.27**	**0.00**	**0.00**	**0.89**	**0.84**	**0.05**	**0.00**	**0.07**	**0.29**	**0.24**	**0.21**	**0.01**	**0.28**	**0.24**

Table H-4.4 Number of Cycles with Spillback at Walmart Access Rd. and E. Van Fleet Drive – Before Study

Time period	Time segment	# cycles in 15 min	EB (Main)							NB				WB (Main)					SB	
			Left		Through			Right		Left		Through	Right	Left	Through			Right	Through/ Left	Right
LANE NUMBER			1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18
AM PEAK	1	8	0	0	0	0	0	0	0	3	3	0	0	0	0	0	0	0	0	0
	2	7	0	0	0	0	0	0	0	4	4	0	0	0	0	0	0	0	0	0
	3	7	0	0	0	0	0	0	0	0	1	0	0	0	0	0	0	0	0	0
	4	7	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
	Average		**0**	**0**	**0**	**0**	**0**	**0**	**0**	**0**	**1.75**	**2**	**0**	**0**	**0**	**0**	**0**	**0**	**0**	**0**

Time period	Time segment	# cycles in 15 min	EB (Main)							NB				WB (Main)					SB	
			Left		Through			Right		Left		Through	Right	Left	Through			Right	Through/ Left	Right
LANE NUMBER			**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**	**13**	**14**	**15**	**16**	**17**	**18**
OFF PEAK	**1**	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
	2	0	0	0	0	0	0	0	1	1	0	0	0	0	0	0	0	0	0	0
	3	0	0	0	0	0	0	0	1	1	0	0	0	0	0	0	0	0	0	0
	4	0	0	0	0	0	0	0	3	4	0	0	0	0	0	0	0	0	0	0
	Average		**0**	**0**	**0**	**0**	**0**	**0**	**0**	**0**	**1.25**	**1.5**	**0**	**0**	**0**	**0**	**0**	**0**	**0**	**0**
PM PEAK	**1**	0	0	0	0	0	0	0	5	4	0	0	0	0	0	0	0	0	0	0
	2	0	0	0	0	0	0	0	3	3	0	0	0	0	0	0	0	0	0	0
	3	0	0	0	0	0	0	0	5	5	0	0	0	0	0	0	0	0	0	0
	4	0	0	0	0	0	0	0	2	1	0	0	0	0	0	0	0	0	0	0
	Average		**0**	**0**	**0**	**0**	**0**	**0**	**0**	**0**	**3.75**	**3.25**	**0**	**0**	**0**	**0**	**0**	**0**	**0**	**0**

After Study (March 28 & March 29, 2017)

Table H-4.5 Average Queue Storage Ratio by Lane and by Period at N. Broadway Ave. and E. Van Fleet Drive – After Study

Time period	Time segment	NB						WB (Main)						SB						EB (Main)					
		Left		Through			Right	Left		Through		Right		Left			Through		Right	Left			Through		Through/ Right
LANE NUMBER		1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24
AM PEAK	1	0.03	0.06	0.29	0.24	0.20	0.00	0.07	0.26	0.31	0.34	0.24	0.18	0.30	0.53	0.56	0.55	0.52	0.05	0.22	0.18	0.15	0.24	0.16	0.22
	2	0.04	0.10	0.23	0.18	0.24	0.01	0.25	0.49	0.31	0.26	0.08	0.10	0.20	0.28	0.44	0.56	0.52	0.02	0.18	0.09	0.15	0.24	0.19	0.26
	3	0.02	0.15	0.26	0.10	0.21	0.07	0.13	0.29	0.22	0.23	0.04	0.03	0.16	0.26	0.36	0.49	0.54	0.00	0.14	0.12	0.12	0.18	0.20	0.21
	4	0.04	0.14	0.24	0.16	0.19	0.04	0.06	0.20	0.20	0.20	0.03	0.03	0.12	0.24	0.33	0.31	0.33	0.00	0.08	0.12	0.13	0.15	0.14	0.26
	Average	**0.03**	**0.11**	**0.25**	**0.17**	**0.21**	**0.03**	**0.13**	**0.31**	**0.26**	**0.26**	**0.10**	**0.08**	**0.19**	**0.33**	**0.42**	**0.48**	**0.48**	**0.02**	**0.15**	**0.13**	**0.14**	**0.20**	**0.17**	**0.24**
OFF PEAK	1	0.09	0.00	0.26	0.25	0.25	0.03	0.10	0.13	0.33	0.33	0.43	0.40	0.40	0.47	0.47	0.29	0.28	0.04	0.13	0.17	0.06	0.23	0.18	0.22
	2	0.12	0.09	0.15	0.16	0.15	0.09	0.05	0.08	0.22	0.23	0.28	0.30	0.51	0.59	0.54	0.15	0.16	0.02	0.19	0.12	0.02	0.11	0.11	0.10
	3	0.06	0.05	0.20	0.21	0.18	0.00	0.06	0.09	0.19	0.18	0.13	0.12	0.38	0.47	0.40	0.15	0.13	0.02	0.01	0.11	0.06	0.18	0.17	0.22

Time period	Time segment	NB						WB (Main)						SB						EB (Main)					
		Left		Through			Right	Left		Through		Right		Left			Through		Right	Left			Through		Through/ Right
LANE NUMBER		1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24
	4	0.05	0.02	0.22	0.19	0.27	0.01	0.09	0.13	0.22	0.21	0.19	0.16	0.33	0.48	0.44	0.11	0.13	0.02	0.11	0.08	0.06	0.11	0.13	0.14
	Average	**0.08**	**0.04**	**0.21**	**0.21**	**0.21**	**0.03**	**0.08**	**0.11**	**0.24**	**0.24**	**0.26**	**0.25**	**0.40**	**0.50**	**0.46**	**0.18**	**0.17**	**0.02**	**0.11**	**0.12**	**0.05**	**0.15**	**0.15**	**0.17**
PM PEAK	1	0.10	0.13	0.64	0.41	0.41	0.00	0.05	0.16	0.18	0.20	0.10	0.14	0.26	0.43	0.50	0.32	0.26	0.01	0.18	0.14	0.18	0.40	0.34	0.30
	2	0.01	0.15	0.65	0.45	0.35	0.07	0.04	0.12	0.16	0.15	0.11	0.16	0.33	0.41	0.54	0.22	0.24	0.00	0.21	0.20	0.23	0.20	0.17	0.26
	3	0.03	0.09	0.97	0.90	0.76	0.00	0.04	0.11	0.20	0.23	0.15	0.18	0.22	0.36	0.49	0.19	0.19	0.01	0.26	0.21	0.23	0.28	0.27	0.29
	4	0.06	0.14	0.26	0.19	0.19	0.03	0.04	0.10	0.21	0.23	0.17	0.22	0.21	0.35	0.52	0.19	0.24	0.01	0.25	0.20	0.24	0.34	0.23	0.28
	Average	**0.05**	**0.13**	**0.63**	**0.49**	**0.43**	**0.03**	**0.04**	**0.12**	**0.19**	**0.20**	**0.13**	**0.17**	**0.26**	**0.39**	**0.51**	**0.23**	**0.23**	**0.01**	**0.23**	**0.19**	**0.22**	**0.30**	**0.25**	**0.28**

Table H-4.6 Number of Cycles with Spillback at N. Broadway Ave. and E. Van Fleet Drive – After Study

Time period	Time segment	# cycles in 15 min	NB						WB (Main)						SB						EB (Main)					
			Left		Through			Right	Left		Through		Right		Left			Through		Right	Left			Through		Through / Right
LANE NUMBER			1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24
AM PEAK	1	7	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
	2	7	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
	3	7	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
	4	7	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
	Average		**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**
OFF PEAK	1	8	0	0	0	0	0	0	0	0	0	0	1	1	0	0	0	0	0	0	0	0	0	0	0	1
	2	8	0	0	0	0	0	0	0	0	0	0	0	0	1	1	1	0	0	0	0	0	0	0	0	0
	3	8	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1
	4	8	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
	Average		**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.25**	**0.25**	**0.25**	**0.25**	**0.25**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.50**
PM	1	6	0	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
	2	6	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

Time period	Time segment	# cycles in 15 min	NB						WB (Main)						SB						EB (Main)					
			Left		Through			Right	Left		Through		Right		Left			Through		Right	Left			Through		Through / Right
LANE NUMBER			1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24
PEAK	3	6	0	0	5	2	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
	4	6	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1
	Average		0.00	0.00	1.50	0.50	0.25	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.25

Table H-4.7 Average Queue Storage Ratio by Lane and by Period at Walmart Access Rd. and E. Van Fleet Drive – After Study

Time period	Time segment	EB (Main)							NB				WB (Main)					SB	
		Left		Through			Right		Left		Through	Right	Left	Through			Right	Through/Left	Right
LANE NUMBER		1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18
AM PEAK	1	0.07	0.08	0.12	0.08	0.18	0.00	0.00	0.88	0.85	0.04	0.00	0.08	0.28	0.21	0.18	0.04	0.08	0.03
	2	0.06	0.03	0.04	0.07	0.12	0.00	0.00	0.81	0.64	0.00	0.00	0.15	0.44	0.16	0.11	0.00	0.06	0.00
	3	0.07	0.06	0.04	0.03	0.17	0.00	0.00	0.70	0.56	0.01	0.00	0.12	0.31	0.14	0.19	0.00	0.11	0.01
	4	0.09	0.08	0.04	0.11	0.12	0.00	0.00	0.52	0.51	0.04	0.00	0.08	0.13	0.11	0.08	0.01	0.07	0.00

Time period	Time segment	EB (Main)							NB				WB (Main)					SB	
		Left		Through			Right		Left		Through	Right	Left	Through			Right	Through/Left	Right
LANE NUMBER		1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18
	Average	**0.07**	**0.06**	**0.06**	**0.07**	**0.15**	**0.00**	**0.00**	**0.73**	**0.64**	**0.02**	**0.00**	**0.11**	**0.29**	**0.15**	**0.14**	**0.01**	**0.08**	**0.01**
OFF PEAK	**1**	0.10	0.10	0.01	0.07	0.14	0.00	0.00	0.61	0.55	0.01	0.00	0.10	0.19	0.12	0.12	0.01	0.11	0.02
	2	0.14	0.12	0.04	0.10	0.13	0.00	0.00	0.60	0.48	0.05	0.00	0.08	0.24	0.16	0.16	0.02	0.11	0.03
	3	0.19	0.14	0.08	0.11	0.14	0.00	0.00	0.53	0.48	0.14	0.00	0.08	0.14	0.09	0.09	0.01	0.14	0.01
	4	0.11	0.10	0.07	0.06	0.19	0.00	0.00	0.70	0.65	0.03	0.00	0.12	0.15	0.18	0.18	0.03	0.16	0.02
	Average	**0.14**	**0.12**	**0.05**	**0.09**	**0.15**	**0.00**	**0.00**	**0.61**	**0.54**	**0.06**	**0.00**	**0.09**	**0.18**	**0.13**	**0.14**	**0.02**	**0.13**	**0.02**
PM PEAK	**1**	0.16	0.15	0.12	0.13	0.14	0.00	0.00	0.57	0.49	0.16	0.00	0.08	0.28	0.33	0.28	0.00	0.20	0.06
	2	0.22	0.23	0.13	0.10	0.12	0.00	0.00	0.77	0.73	0.12	0.00	0.03	0.20	0.24	0.27	0.01	0.24	0.13
	3	0.21	0.17	0.15	0.16	0.18	0.00	0.00	0.69	0.63	0.10	0.00	0.10	0.19	0.22	0.29	0.00	0.18	0.08

Time period	Time segment	EB (Main)							NB				WB (Main)					SB	
		Left		Through			Right		Left		Through	Right	Left	Through			Right	Through/Left	Right
LANE NUMBER		1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18
	4	0.16	0.17	0.07	0.08	0.10	0.00	0.00	0.59	0.54	0.11	0.00	0.07	0.18	0.21	0.22	0.00	0.24	0.04
	Average	**0.19**	**0.18**	**0.11**	**0.12**	**0.14**	**0.00**	**0.00**	**0.65**	**0.60**	**0.12**	**0.00**	**0.07**	**0.21**	**0.25**	**0.26**	**0.00**	**0.21**	**0.07**

Table H-4.8 Number of Cycles with Spillback at Walmart Access Rd. and E. Van Fleet Drive – After Study

Time period	Time segment	# cycles in 15 min	EB (Main)							NB				WB (Main)					SB	
			Left		Through			Right		Left		Through	Right	Left	Through			Right	Through/Left	Right
LANE NUMBER			1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18
AM PEAK	1	6	0	0	0	0	0	0	0	2	1	0	0	0	0	0	0	0	0	0
	2	6	0	0	0	0	0	0	0	2	0	0	0	0	0	0	0	0	0	0
	3	6	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
	4	6	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
	Average		**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**1.00**	**0.25**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**
	1	7	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

Time period	Time segment	# cycles in 15 min	EB (Main)							NB				WB (Main)					SB	
			Left		Through			Right		Left		Through	Right	Left	Through			Right	Through/Left	Right
LANE NUMBER			1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18
OFF PEAK	2	7	0	0	0	0	0	0	1	0	0	0	0	0	0	0	0	0	0	0
	3	7	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
	4	7	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
	Average		**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.25**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**
PM PEAK	1	6	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
	2	6	0	0	0	0	0	0	2	1	0	0	0	0	0	0	0	0	0	0
	3	6	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
	4	6	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
	Average		**0.50**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.50**	**0.25**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**

Comparisons of Before and After Queue Storage Ratios

The differences in Queue Storage Ratio between before and after measurements are shown in Table H-4.9 and I-4.10. The tables are color-coded as follows: green shows significant improvement, yellow shows modest change (either improvement or deterioration), and red shows significant deterioration in delay. Several gradations of each color are used to represent different variations within each classification.

Table H-4.9 Difference in Avg. Queue Storage Ratios at N. Broadway Ave. & E. Van Fleet

Time Period	NB						WB (Main)						SB						EB (Main)						Ave.
	Left		Through			Right	Left		Through		Right		Left			Through		Right	Left			Through		Through/ Right	
Lane N	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	
AM Peak	0.01	0.06	0.03	-0.05	0.09	-0.02	-0.10	0.06	0.05	0.04	-0.03	-0.09	-0.15	-0.09	-0.02	0.06	0.14	-0.03	0.05	-0.05	-0.01	0.00	0.00	0.08	0.00
Off Peak	0.05	-0.03	-0.01	0.01	0.07	0.02	0.02	-0.01	0.15	0.15	0.18	0.15	0.27	0.32	0.22	0.04	0.04	0.00	0.05	0.01	-0.06	0.03	0.00	0.04	0.07
PM Peak	-0.01	0.02	0.21	0.21	0.14	-0.06	-0.02	-0.02	0.03	0.01	-0.01	-0.01	-0.08	-0.05	-0.04	-0.06	-0.06	-0.02	0.05	-0.06	0.00	0.06	0.07	0.03	0.01

Table H-4.10 Difference in Avg. Queue Storage Ratios at Walmart Access Rd, E. Van Fleet

Time Period	EB (main)							NB				WB (Main)					SB		Average
	Left		Through			Right		Left		Through	Right	Left	Through			Right	Through/Left	Right	
Lane N	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	
AM Peak	-0.01	-0.03	0.00	-0.03	-0.11	0.00	0.00	0.01	-0.12	-0.03	-0.01	0.08	0.01	0.00	0.05	0.01	-0.01	-0.04	-0.01
Off Peak	-0.01	-0.04	-0.07	-0.08	-0.10	0.00	0.00	-0.04	-0.14	0.02	0.00	0.05	0.03	-0.01	0.00	0.00	-0.02	-0.10	-0.03
PM Peak	0.00	-0.01	-0.11	-0.12	-0.13	0.00	0.00	-0.24	-0.24	0.07	0.00	-0.01	-0.08	0.01	0.05	-0.01	-0.06	-0.16	-0.06

Discussion

The following are concluded from the comparison of queue storage ratios:

- Overall, the queue storage ratios were improved for the two critical intersections.
- The queue storage ratios for the NB left at the Walmart Access and E. Van Fleet Drive were significantly improved, particularly for the PM peak.
- There is a reduction in the number of cycles with spillback at Walmart Access Rd. and E. Van Fleet Drive intersection, especially for the NB left and through movements, throughout the day.

H.3.5 EQUIVALENT PCE FLOWS

Traffic flows were counted manually and converted to equivalent passenger car flows (PCE, in pce/hour) by considering the percentage of heavy vehicles. It was assumed that the PCE for trucks is 2.

Truck Percentage Observations

Table H-5.1 Truck Percentages at N. Broadway Ave. and E. Van Fleet

Period	Interval	NB			SB			EB			WB			Ave
		Left	Thru	Right	Left	Thru	Right	Left	Thru	Right	Left	Thru	Right	
AM Peak	1	0.00%	4.00%	0.00%	8.24%	3.59%	2.22%	10.26%	13.04%	0.00%	1.59%	8.33%	6.54%	4.82%
	2	0.00%	2.99%	0.00%	7.41%	2.83%	4.23%	25.49%	15.15%	0.00%	0.96%	9.91%	7.45%	6.37%
	3	0.00%	2.97%	6.67%	10.00%	0.74%	0.00%	6.67%	16.28%	0.00%	0.00%	11.43%	12.12%	5.57%
	4	0.00%	2.50%	0.00%	6.61%	0.00%	2.78%	0.00%	18.81%	0.00%	4.65%	9.90%	11.26%	4.71%
	Average	**0.00%**	**3.11%**	**1.67%**	**8.07%**	**1.79%**	**2.31%**	**10.60%**	**15.82%**	**0.00%**	**1.80%**	**9.89%**	**9.34%**	**5.37%**
Off Peak	1	0.00%	3.09%	0.00%	10.67%	0.00%	6.52%	5.71%	16.87%	0.00%	7.32%	15.70%	11.61%	6.46%
	2	8.33%	2.78%	0.00%	13.04%	1.92%	0.00%	12.20%	20.69%	12.50%	5.41%	16.05%	15.44%	9.03%
	3	0.00%	2.83%	3.33%	10.91%	0.00%	5.26%	10.71%	22.89%	100.00%	9.38%	18.18%	9.52%	16.09%
	4	0.00%	2.27%	5.26%	17.93%	1.33%	9.43%	22.73%	22.99%	0.00%	12.50%	20.48%	12.93%	10.66%
	Average	**2.08%**	**2.74%**	**2.15%**	**13.14%**	**0.81%**	**5.30%**	**12.84%**	**20.86%**	**28.13%**	**8.65%**	**17.60%**	**12.38%**	**10.56%**
PM Peak	1	4.55%	1.60%	0.00%	5.11%	0.85%	6.38%	5.71%	6.56%	0.00%	0.00%	9.26%	9.09%	4.09%
	2	0.00%	0.00%	2.86%	3.95%	1.71%	2.04%	11.27%	1.02%	0.00%	3.23%	8.11%	7.29%	3.46%

Period	Interval	NB			SB			EB			WB			Ave
		Left	Thru	Right	Left	Thru	Right	Left	Thru	Right	Left	Thru	Right	
	3	0.00%	0.26%	1.96%	6.03%	0.97%	2.44%	6.37%	3.65%	0.00%	1.96%	4.76%	7.30%	2.98%
	4	0.00%	0.68%	0.00%	8.19%	0.00%	3.03%	2.08%	4.84%	0.00%	0.00%	1.37%	7.32%	2.29%
	Average	**1.14%**	**0.64%**	**1.20%**	**5.82%**	**0.88%**	**3.47%**	**6.36%**	**4.02%**	**0.00%**	**1.30%**	**5.87%**	**7.75%**	**3.20%**

Table H-5.2 Truck Percentages at Walmart Access Rd. and E. Van Fleet

Period	Interval	NB			SB			EB			WB			Ave
		Left	Thru	Right	Left	Thru	Right	Left	Thru	Right	Left	Thru	Right	
AM Peak	1	8.50%	17.65%	50.00%	0.00%	0.00%	13.64%	0.00%	7.69%	11.66%	0.00%	10.85%	0.00%	10.00%
	2	6.61%	0.00%	0.00%	0.00%	0.00%	5.13%	5.26%	6.09%	7.96%	0.00%	2.39%	7.14%	3.38%
	3	6.72%	0.00%	20.00%	25.00%	0.00%	0.00%	3.70%	7.04%	13.13%	71.43%	7.51%	10.00%	13.71%
	4	12.83%	0.00%	0.00%	0.00%	11.11%	3.70%	3.45%	10.53%	9.25%	16.67%	11.02%	8.33%	7.24%
	Average	**8.66%**	**4.41%**	**17.50%**	**6.25%**	**2.78%**	**5.62%**	**3.10%**	**7.84%**	**10.50%**	**22.02%**	**7.95%**	**6.37%**	**8.58%**
Off Peak	1	16.94%	0.00%	20.00%	0.00%	0.00%	3.70%	8.82%	7.19%	17.65%	0.00%	6.96%	0.00%	6.77%
	2	17.02%	14.29%	100.00%	0.00%	7.14%	0.00%	1.92%	5.00%	12.94%	0.00%	11.49%	0.00%	14.15%
	3	17.32%	7.69%	100.00%	0.00%	0.00%	3.23%	2.63%	10.64%	18.75%	100.00%	8.21%	0.00%	22.37%
	4	14.81%	11.11%	0.00%	0.00%	0.00%	0.00%	6.56%	12.62%	13.66%	0.00%	5.00%	0.00%	5.31%
	Average	**16.52%**	**8.27%**	**55.00%**	**0.00%**	**1.79%**	**1.73%**	**4.98%**	**8.86%**	**15.75%**	**25.00%**	**7.91%**	**0.00%**	**12.15%**

Period	Interval	NB			SB			EB			WB			Ave
		Left	**Thru**	**Right**	**Left**	**Thru**	**Right**	**Left**	**Thru**	**Right**	**Left**	**Thru**	**Right**	
PM Peak	1	8.96%	0.00%	0.00%	4.35%	0.00%	0.00%	0.00%	2.44%	0.00%	16.67%	5.43%	7.41%	3.77%
	2	7.64%	11.11%	0.00%	0.00%	0.00%	2.27%	1.08%	3.02%	5.31%	75.00%	5.13%	0.00%	9.21%
	3	7.41%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	4.35%	4.71%	0.00%	2.37%	0.00%	1.57%
	4	10.75%	0.00%	16.67%	0.00%	0.00%	0.00%	1.49%	2.49%	3.20%	100.00%	2.25%	0.00%	11.40%
	Average	**8.69%**	**2.78%**	**4.17%**	**1.09%**	**0.00%**	**0.57%**	**0.64%**	**3.07%**	**3.31%**	**47.92%**	**3.80%**	**1.85%**	**6.49%**

Before Study (February 22 & February 23, 2017)

Table H-5.3 Truck Percentages at N. Broadway Ave. and E. Van Fleet Drive – Before Study

Period	Interval	NB			SB			EB			WB			Ave
		Left	Thru	Right	Left	Thru	Right	Left	Thru	Right	Left	Thru	Right	
AM Peak	1	0.00%	4.00%	0.00%	8.24%	3.59%	2.22%	10.26%	13.04%	0.00%	1.59%	8.33%	6.54%	4.82%
	2	0.00%	2.99%	0.00%	7.41%	2.83%	4.23%	25.49%	15.15%	0.00%	0.96%	9.91%	7.45%	6.37%
	3	0.00%	2.97%	6.67%	10.00%	0.74%	0.00%	6.67%	16.28%	0.00%	0.00%	11.43%	12.12%	5.57%
	4	0.00%	2.50%	0.00%	6.61%	0.00%	2.78%	0.00%	18.81%	0.00%	4.65%	9.90%	11.26%	4.71%
	Average	**0.00%**	**3.11%**	**1.67%**	**8.07%**	**1.79%**	**2.31%**	**10.60%**	**15.82%**	**0.00%**	**1.80%**	**9.89%**	**9.34%**	**5.37%**
Off Peak	1	0.00%	3.09%	0.00%	10.67%	0.00%	6.52%	5.71%	16.87%	0.00%	7.32%	15.70%	11.61%	6.46%
	2	8.33%	2.78%	0.00%	13.04%	1.92%	0.00%	12.20%	20.69%	12.50%	5.41%	16.05%	15.44%	9.03%
	3	0.00%	2.83%	3.33%	10.91%	0.00%	5.26%	10.71%	22.89%	100.00%	9.38%	18.18%	9.52%	16.09%
	4	0.00%	2.27%	5.26%	17.93%	1.33%	9.43%	22.73%	22.99%	0.00%	12.50%	20.48%	12.93%	10.66%
	Average	**2.08%**	**2.74%**	**2.15%**	**13.14%**	**0.81%**	**5.30%**	**12.84%**	**20.86%**	**28.13%**	**8.65%**	**17.60%**	**12.38%**	**10.56%**
PM Peak	1	4.55%	1.60%	0.00%	5.11%	0.85%	6.38%	5.71%	6.56%	0.00%	0.00%	9.26%	9.09%	4.09%
	2	0.00%	0.00%	2.86%	3.95%	1.71%	2.04%	11.27%	1.02%	0.00%	3.23%	8.11%	7.29%	3.46%
	3	0.00%	0.26%	1.96%	6.03%	0.97%	2.44%	6.37%	3.65%	0.00%	1.96%	4.76%	7.30%	2.98%
	4	0.00%	0.68%	0.00%	8.19%	0.00%	3.03%	2.08%	4.84%	0.00%	0.00%	1.37%	7.32%	2.29%
	Average	**1.14%**	**0.64%**	**1.20%**	**5.82%**	**0.88%**	**3.47%**	**6.36%**	**4.02%**	**0.00%**	**1.30%**	**5.87%**	**7.75%**	**3.20%**

Table H-5.4 Truck Percentages at Walmart Access Rd. and E. Van Fleet Drive – Before Study

Period	Interval	NB			SB			EB			WB			Ave
		Left	Thru	Right	Left	Thru	Right	Left	Thru	Right	Left	Thru	Right	
AM Peak	1	8.50%	17.65%	50.00%	0.00%	0.00%	13.64%	0.00%	7.69%	11.66%	0.00%	10.85%	0.00%	10.00%
	2	6.61%	0.00%	0.00%	0.00%	0.00%	5.13%	5.26%	6.09%	7.96%	0.00%	2.39%	7.14%	3.38%
	3	6.72%	0.00%	20.00%	25.00%	0.00%	0.00%	3.70%	7.04%	13.13%	71.43%	7.51%	10.00%	13.71%
	4	12.83%	0.00%	0.00%	0.00%	11.11%	3.70%	3.45%	10.53%	9.25%	16.67%	11.02%	8.33%	7.24%
	Average	**8.66%**	**4.41%**	**17.50%**	**6.25%**	**2.78%**	**5.62%**	**3.10%**	**7.84%**	**10.50%**	**22.02%**	**7.95%**	**6.37%**	**8.58%**
Off Peak	1	16.94%	0.00%	20.00%	0.00%	0.00%	3.70%	8.82%	7.19%	17.65%	0.00%	6.96%	0.00%	6.77%
	2	17.02%	14.29%	100.00%	0.00%	7.14%	0.00%	1.92%	5.00%	12.94%	0.00%	11.49%	0.00%	14.15%
	3	17.32%	7.69%	100.00%	0.00%	0.00%	3.23%	2.63%	10.64%	18.75%	100.00%	8.21%	0.00%	22.37%
	4	14.81%	11.11%	0.00%	0.00%	0.00%	0.00%	6.56%	12.62%	13.66%	0.00%	5.00%	0.00%	5.31%
	Average	**16.52%**	**8.27%**	**55.00%**	**0.00%**	**1.79%**	**1.73%**	**4.98%**	**8.86%**	**15.75%**	**25.00%**	**7.91%**	**0.00%**	**12.15%**
PM Peak	1	8.96%	0.00%	0.00%	4.35%	0.00%	0.00%	0.00%	2.44%	0.00%	16.67%	5.43%	7.41%	3.77%
	2	7.64%	11.11%	0.00%	0.00%	0.00%	2.27%	1.08%	3.02%	5.31%	75.00%	5.13%	0.00%	9.21%
	3	7.41%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	4.35%	4.71%	0.00%	2.37%	0.00%	1.57%
	4	10.75%	0.00%	16.67%	0.00%	0.00%	0.00%	1.49%	2.49%	3.20%	100.00%	2.25%	0.00%	11.40%
	Average	**8.69%**	**2.78%**	**4.17%**	**1.09%**	**0.00%**	**0.57%**	**0.64%**	**3.07%**	**3.31%**	**47.92%**	**3.80%**	**1.85%**	**6.49%**

At the N. Broadway intersection, the highest average queue lengths by period and approach are AM – Southbound through and left, Off Peak – Northbound and Eastbound through movements along with Southbound lefts, and PM – Southbound lefts. At the Walmart intersection, the highest average queue lengths by period and approach are Northbound lefts across all time periods. Northbound left turns at Walmart are the only turning movements measured on the corridor that have PCE hourly flows over 1000 vehicles per hour (AM and PM). Walmart Northbound left turns have the highest consistent 0.8 or greater values for average queue storage ratios and highest average number of cycles experiencing spillback.

The project location experiences high percentages of heavy vehicle movements. At North Broadway, the Off Peak time period experienced the highest heavy vehicle percentage of 10.56%. Almost 21% of all vehicles heading Eastbound through the intersection were heavy vehicles during Off Peak times. The highest percentage of heavy vehicles at Walmart was 12.15% during Off Peak. More than 16% of all vehicles turning left on to West Van Fleet were heavy vehicles during Off Peak. The time of day is correlated with heavy vehicle percentages. During Off Peak times, the percentage of heavy vehicles increases in almost every direction of each intersection, suggesting that deliveries of goods may be done during this time.

On average, rain increased corridor travel time by 0.2 minutes Eastbound, but reduced corridor travel time by 0.2 minutes Westbound. Rain affected Eastbound left turns at East Van fleet with higher delays in Off Peak and PM times. Westbound delay at Northbound left turns at North Holland Pkwy increased during rain in the AM but decreased in the PM. Westbound right turns at West Van fleet experienced decreased delay with rain.

After Study (March 28 & March 29, 2017)

Table H-5.5 Truck Percentages at N. Broadway Ave. and E. Van Fleet Drive – After Study

Period	Interval	NB			SB			EB			WB			Average
		Left	Thru	Right	Left	Thru	Right	Left	Thru	Right	Left	Thru	Right	
AM Peak	1	0.00%	0.00%	0.00%	6.45%	14.66%	0.00%	0.00%	14.52%	0.00%	0.00%	9.02%	9.09%	4.48%
	2	0.00%	0.85%	5.00%	8.59%	8.59%	10.00%	3.45%	15.69%	0.00%	0.00%	11.35%	9.02%	6.04%
	3	0.00%	0.00%	20.00%	17.89%	1.56%	0.00%	6.15%	13.04%	9.09%	8.00%	10.24%	6.62%	7.72%
	4	0.00%	2.75%	10.00%	19.59%	1.89%	7.41%	0.00%	13.83%	0.00%	2.63%	13.85%	10.91%	6.90%

Period	Interval	NB			SB			EB			WB			Average
		Left	Thru	Right	Left	Thru	Right	Left	Thru	Right	Left	Thru	Right	
	Average	**0.00%**	**0.90%**	**8.75%**	**13.13%**	**6.67%**	**4.35%**	**2.40%**	**14.27%**	**2.27%**	**2.66%**	**11.11%**	**8.91%**	**6.29%**
Off Peak	1	4.55%	2.99%	0.00%	0.00%	3.64%	12.50%	3.70%	8.93%	0.00%	0.00%	26.12%	15.60%	6.50%
	2	6.67%	4.08%	10.34%	16.67%	0.00%	5.41%	5.00%	20.63%	0.00%	4.35%	28.43%	19.26%	10.07%
	3	0.00%	0.81%	0.00%	16.55%	5.19%	8.33%	20.00%	20.54%	12.50%	0.00%	18.64%	15.89%	9.87%
	4	4.55%	1.82%	0.00%	9.59%	5.15%	20.00%	2.22%	13.04%	0.00%	4.55%	15.12%	13.77%	7.48%
	Average	**3.94%**	**2.42%**	**2.59%**	**10.70%**	**3.50%**	**11.56%**	**7.73%**	**15.79%**	**3.13%**	**2.22%**	**22.08%**	**16.13%**	**8.48%**
PM Peak	1	0.00%	0.00%	0.00%	5.79%	1.72%	8.20%	8.20%	7.69%	14.29%	4.35%	7.76%	8.04%	5.50%
	2	0.00%	0.00%	2.13%	5.29%	0.92%	1.15%	5.26%	6.72%	0.00%	2.78%	10.61%	12.04%	3.91%
	3	0.00%	0.00%	3.70%	3.11%	0.00%	5.36%	3.70%	5.75%	0.00%	0.00%	7.01%	5.56%	2.85%
	4	0.00%	0.00%	0.00%	1.02%	0.94%	2.90%	0.00%	3.70%	0.00%	0.00%	8.55%	5.69%	1.90%
	Average	**0.00%**	**0.00%**	**1.46%**	**3.80%**	**0.90%**	**4.40%**	**4.29%**	**5.97%**	**3.57%**	**1.78%**	**8.48%**	**7.83%**	**3.54%**

Table H-5.6 Truck Percentages at Walmart Access Rd. and E. Van Fleet Drive – After Study

Period	Interval	NB			SB			EB			WB			Average
		Left	**Thru**	**Right**	**Left**	**Thru**	**Right**	**Left**	**Thru**	**Right**	**Left**	**Thru**	**Right**	
AM Peak	1	8.00%	0.00%	58.33%	0.00%	0.00%	0.00%	8.33%	7.14%	15.12%	0.00%	3.15%	0.00%	8.34%
	2	5.77%	0.00%	37.50%	0.00%	0.00%	0.00%	0.00%	8.47%	10.50%	100.00%	6.60%	0.00%	14.07%
	3	9.41%	0.00%	25.00%	0.00%	0.00%	0.00%	0.00%	5.26%	8.38%	0.00%	4.38%	0.00%	4.37%
	4	9.95%	0.00%	20.00%	0.00%	0.00%	0.00%	0.00%	7.01%	17.39%	0.00%	5.74%	0.00%	5.01%
	Average	**8.28%**	**0.00%**	**35.21%**	**0.00%**	**0.00%**	**0.00%**	**2.08%**	**6.97%**	**12.85%**	**25.00%**	**4.97%**	**0.00%**	**7.95%**
Off Peak	1	13.59%	0.00%	33.33%	0.00%	0.00%	0.00%	0.00%	12.27%	17.59%	0.00%	9.09%	5.88%	7.65%
	2	14.12%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	11.27%	15.65%	0.00%	6.92%	0.00%	4.00%
	3	17.65%	10.00%	33.33%	0.00%	0.00%	0.00%	0.00%	10.17%	15.91%	0.00%	9.63%	7.14%	8.65%
	4	14.01%	0.00%	28.57%	0.00%	0.00%	1.96%	2.08%	14.29%	21.71%	25.00%	8.21%	0.00%	9.65%
	Average	**14.84%**	**2.50%**	**23.81%**	**0.00%**	**0.00%**	**0.49%**	**0.52%**	**12.00%**	**17.71%**	**6.25%**	**8.46%**	**3.26%**	**7.49%**
PM Peak	1	17.46%	6.67%	0.00%	0.00%	0.00%	0.00%	1.67%	7.05%	12.50%	0.00%	4.35%	0.00%	4.14%
	2	11.89%	0.00%	0.00%	3.57%	0.00%	0.00%	1.64%	6.12%	9.05%	100.00%	5.00%	5.26%	11.88%
	3	9.04%	0.00%	50.00%	0.00%	0.00%	0.00%	0.00%	3.63%	8.91%	0.00%	1.18%	0.00%	6.06%
	4	7.73%	0.00%	20.00%	0.00%	0.00%	0.00%	0.00%	3.02%	7.41%	40.00%	7.75%	0.00%	7.16%
	Average	**11.53%**	**1.67%**	**17.50%**	**0.89%**	**0.00%**	**0.00%**	**0.83%**	**4.95%**	**9.47%**	**35.00%**	**4.57%**	**1.32%**	**7.31%**

Before Study (February 22nd & 23rd, 2017)

Table H-5.7 Traffic Vol. (pce/15 min) and Traffic Flow (pce/hour) at N. Broadway Ave. & E. Van Fleet Drive – Before Study

Time Period		NB			SB			EB			WB		
		Left	**Through**	**Right**	**Left**	**Through**	**Right**	**Left**	**Through**	**Right**	**Left**	**Through**	**Right**
AM Peak	1	19	104	6	197	173	46	43	156	5	64	143	228
	2	9	138	12	232	254	74	64	114	2	105	122	173
	3	10	104	16	165	136	50	48	100	3	57	117	148
	4	6	82	11	129	88	37	51	120	6	45	111	168
	Flow Rate	**44**	**428**	**45**	**723**	**651**	**207**	**206**	**490**	**16**	**271**	**493**	**717**
Off Peak	1	11	100	23	197	102	49	37	97	3	44	140	173
	2	13	111	25	130	53	26	46	105	9	39	94	157
	3	4	109	31	122	62	20	31	102	2	35	130	138
	4	14	135	20	171	76	58	54	107	4	27	100	131
	Flow Rate	**42**	**455**	**99**	**620**	**293**	**153**	**168**	**411**	**18**	**145**	**464**	**599**
PM Peak	1	23	190	35	185	118	50	74	130	5	55	177	144
	2	32	236	36	184	119	50	79	99	3	32	160	206
	3	27	191	26	185	105	42	84	142	3	26	154	191
	4	21	147	17	185	90	34	98	195	3	20	148	176
	Flow Rate	**103**	**764**	**114**	**739**	**432**	**176**	**335**	**566**	**14**	**133**	**639**	**717**

Table H-5.8 Traffic Vol. (pce/15 min) and Traffic Flow (pce/hour) at Walmart Access Rd. & E. Van Fleet Drive – Before Study

Time Period		NB			SB			EB			WB		
		Left	**Through**	**Right**	**Left**	**Through**	**Right**	**Left**	**Through**	**Right**	**Left**	**Through**	**Right**
AM Peak	1	217	20	18	4	4	25	16	140	182	0	143	14
	2	258	4	4	10	3	41	20	244	244	2	214	15
	3	286	9	6	5	7	22	28	213	293	12	186	11
	4	255	4	0	14	10	28	30	189	248	7	141	13
	Flow Rate	**1016**	**37**	**28**	**33**	**24**	**116**	**94**	**786**	**967**	**21**	**684**	**53**
Off Peak	1	214	4	6	21	10	28	37	164	240	1	123	12
	2	220	8	2	12	15	25	53	189	192	2	165	15
	3	210	14	2	11	10	32	39	156	171	4	145	20
	4	155	10	2	17	12	51	65	116	208	5	147	24
	Flow Rate	**799**	**36**	**12**	**61**	**47**	**136**	**194**	**625**	**811**	**12**	**580**	**71**
PM Peak	1	231	12	14	24	17	41	65	168	1	7	194	29
	2	310	10	20	26	16	45	94	205	337	7	205	33
	3	261	8	7	28	14	53	55	288	267	3	216	26
	4	237	4	7	32	8	56	68	206	290	2	227	22
	Flow Rate	**1039**	**34**	**48**	**110**	**55**	**195**	**282**	**867**	**895**	**19**	**842**	**110**

After Study (March 28 & March 29, 2017)

Table H-5.9 Traffic Vol. (pce/15 min) and Traffic Flow (pce/hour) at N. Broadway Ave. & E. Van Fleet Drive – After Study

Time Period		NB			SB			EB			WB		
		Left	Through	Right	Left	Through	Right	Left	Through	Right	Left	Through	Right
AM Peak	1	10	131	10	33	133	7	45	142	7	19	133	120
	2	8	118	21	139	177	44	60	118	14	35	157	133
	3	8	108	6	112	130	48	69	104	12	54	140	145
	4	13	112	11	116	108	29	53	107	1	39	74	122
	Flow Rate	**39**	**469**	**48**	**400**	**548**	**128**	**227**	**471**	**34**	**147**	**504**	**520**
Off Peak	1	23	138	38	21	342	27	56	122	6	84	507	378
	2	16	102	32	266	144	78	42	152	6	48	262	322
	3	17	124	30	324	162	52	42	135	9	66	140	350
	4	23	112	22	320	204	60	46	104	8	46	198	314
	Flow Rate	**79**	**476**	**122**	**931**	**852**	**217**	**186**	**513**	**29**	**244**	**1107**	**1364**
PM Peak	1	14	184	27	256	118	66	66	140	8	24	125	215
	2	16	210	48	219	110	88	80	127	6	37	198	242
	3	20	252	28	199	81	59	112	184	2	21	168	247
	4	34	181	27	198	107	71	99	168	11	33	165	223
	Flow Rate	**84**	**827**	**130**	**872**	**416**	**284**	**357**	**619**	**27**	**115**	**656**	**927**

Table H-5.10 Traffic Vol. (pce/15 min) and Traffic Flow (pce/hour) at Walmart Access Rd. & E. Van Fleet Drive – After Study

Time Period		NB			SB			EB			WB		
		Left	**Through**	**Right**	**Left**	**Through**	**Right**	**Left**	**Through**	**Right**	**Left**	**Through**	**Right**
AM Peak	1	243	0	19	16	8	28	13	150	198	4	262	12
	2	275	6	11	14	6	20	10	192	200	4	452	16
	3	221	0	10	0	2	18	26	140	207	10	286	14
	4	232	3	6	18	4	32	25	168	189	12	258	18
	Flow Rate	**971**	**9**	**46**	**48**	**20**	**98**	**74**	**650**	**794**	**30**	**1258**	**60**
Off Peak	1	209	1	16	4	1	19	30	183	234	3	120	18
	2	194	7	5	13	7	29	32	158	170	1	139	18
	3	200	11	4	10	6	30	39	130	204	7	148	15
	4	236	10	9	17	16	52	49	128	185	10	145	14
	Flow Rate	**839**	**29**	**34**	**44**	**30**	**130**	**150**	**599**	**793**	**21**	**552**	**65**
PM Peak	1	222	16	12	25	11	30	61	167	189	5	144	12
	2	160	17	8	29	15	51	62	208	217	8	168	20
	3	193	15	6	30	11	61	58	257	220	5	171	18
	4	195	9	6	20	11	44	85	205	261	7	153	24
	Flow Rate	**770**	**57**	**32**	**104**	**48**	**186**	**266**	**837**	**887**	**25**	**636**	**74**

Comparisons of Before and After Flows

The differences in traffic flow between the before and after measurements are shown in Table H-5.11 and Table H-5.12. The tables are color-coded as follows: green shows significant decrease, yellow shows modest change (either improvement or deterioration), and red shows significant deterioration in delay. Several gradations of each color are used to represent different variations within each classification.

Table H-5.11 Difference in Traffic Flow Rate (pce/hr) at N. Broadway Ave. & E. Van Fleet Drive

Time Period	NB			SB			EB			WB		
	Left	Through	Right	Left	Through	Right	Left	Through	Right	Left	Through	Right
AM Peak	-5	41	3	-323	-103	-79	21	-19	18	-124	11	-197
Off Peak	37	21	23	311	559	64	18	102	11	99	643	765
PM Peak	-19	63	16	134	-16	108	23	53	13	-18	17	210

Note: The volumes recorded at N. Broadway during the before trip were during an all-day rain event

Table H-5.12 Difference in Traffic Flow Rate (pce/hr) at Walmart Access Rd. & E. Van Fleet Drive

Time Period	NB			SB			EB			WB		
	Left	Throug h	Righ t	Lef t	Throug h	Righ t	Lef t	Throug h	Righ t	Lef t	Throug h	Righ t
AM Peak	-45	-28	18	15	-4	-18	-20	-136	-173	9	574	7
Off Peak	40	-7	22	-17	-17	-6	-44	-26	-18	9	-28	-6
PM Peak	- 269	23	-16	-6	-7	-9	-16	-30	-8	6	-206	-36

Discussion

The following are concluded from the comparison of traffic flows:

- At N. Broadway Ave. and E. Van Fleet Drive, the comparison is between a rainy day with no InSync and a sunny day with InSync. With the InSync installation, the volumes traveling through the intersection are lower during the AM peak and much higher during the Off-peak, especially the WB through, WB right, and SB left movements.
- At the Walmart Access and E. Van Fleet Drive, volumes are similar between the before and after study, except for reductions in the EB through and right movements and an increase in the WB through movement during the AM peak.

H.3.6 CONSIDERATION OF TRAFFIC FLOWS JOINTLY WITH QUEUE LENGTH

The differences in “Traffic Flow” and “Queue Length” between before and after measurements are shown in Table H-6.1 and Table H-6.3. The tables are color-coded as follows: green indicates that “Queue” decreases and “Traffic Flow” increases, red indicates “Queue” increases and “Traffic Flow” decreases, no color indicates “Traffic Flow” and “Queue” increase or decrease at the same time. Generally, green indicates improvement despite an increase in flow.

Table H-6.1 Differences in Traffic Flow (TF) and Queue Length (Q) at N. Broadway Ave. & E. Van Fleet Drive

Time Period	Northbound (Side)						Southbound (Main)						Eastbound (Side)						Westbound (Main)					
	Left		Thru		Right		Left		Thru		Right		Left		Thru		Right		Left		Thru		Right	
	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q
AM Peak	-5	0.5	41	0.4	3	-0.2	-323	-1.6	-103	1.8	-79	-0.5	21	-0.1	-19	0.5	18	0.2	-124	-0.4	11	1.0	-197	-1.2
Off Peak	37	0.1	21	0.4	23	0.2	311	4.9	559	0.8	64	0.0	18	-0.1	102	0.4	11	0.1	99	0.1	643	3.1	765	3.3
PM Peak	-19	0.1	63	3.1	16	-0.6	134	-1.0	-16	-1.1	108	-0.4	23	0.0	53	1.0	13	0.1	-18	-0.4	17	0.4	210	-0.2

Table H-6.2 Difference (%) in Traffic Flow (TF) and Queue Length (Q), N. Broadway Ave. & E. Van Fleet Drive

Time Period	NB (Side)						SB (Main)						EB (Side)						WB (Main)					
	Left		Thru		Right		Left		Thru		Right		Left		Thru		Right		Left		Thru		Right	
	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q
AM Peak	-11%	78%	10%	11%	7%	-39%	-45%	-22%	-16%	26%	-38%	-65%	10%	-3%	-4%	14%	113%	202%	-46%	-9%	2%	23%	-27%	-39%
Off Peak	88%	17%	5%	14%	23%	186%	50%	145%	191%	32%	42%	5%	11%	-7%	25%	17%	61%	72%	68%	4%	139%	178%	128%	187%
PM Peak	-18%	2%	8%	57%	14%	-68%	18%	-13%	-4%	-20%	61%	-78%	7%	4%	9%	22%	93%	67%	-14%	-16%	3%	10%	29%	-8%

Table H-6.3 Difference in Traffic Flow (TF) and Queue Length (Q) at Walmart Access Rd. & E. Van Fleet Drive

Time Period	NB (Side)						SB (Side)						EB (Main)						WB (Main)					
	Left		Thru		Right		Left		Thru		Right		Left		Thru		Right		Left		Thru		Right	
	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q
AM Peak	-45	-0.8	-28	-0.5	18	0.0	15	0.1	-4	-0.3	-18	-0.9	-20	-0.3	-136	-0.9	-173	0.0	9	1.1	574	0.2	7	0.1
Off Peak	40	-1.4	-7	0.4	22	0.0	-17	-0.1	-17	-0.2	-6	-2.0	-44	-0.5	-26	-1.7	-18	0.0	9	0.6	-28	0.1	-6	0.0

Time Period	NB (Side)						SB (Side)						EB (Main)						WB (Main)					
	Left		Thru		Right		Left		Thru		Right		Left		Thru		Right		Left		Thru		Right	
	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q
PM Peak	-269	-3.6	23	1.0	-16	0.0	-6	-0.8	-7	-0.5	-9	-3.2	-16	0.0	-30	-2.4	-8	0.0	6	-0.1	-206	-0.1	-36	-0.1

Table H-6.4 Difference (%) in Traffic Flow (TF) and Queue Length (Q) at Walmart Access Rd. & E. Van Fleet Drive

Time Period	NB (Side)						SB (Side)						EB (Main)						WB (Main)					
	Left		Thru		Right		Left		Thru		Right		Left		Thru		Right		Left		Thru		Right	
	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q	TF	Q
AM Peak	-4%	-8%	-76%	-59%	64%	0%	45%	6%	-17%	-39%	-16%	-80%	-21%	-21%	-17%	-32%	-18%	0%	43%	328%	84%	6%	13%	343%
Off Peak	5%	-14%	-19%	71%	183%	0%	-28%	-8%	-36%	-17%	-4%	-85%	-23%	-17%	-4%	-46%	-2%	0%	75%	97%	-5%	4%	-8%	26%
PM Peak	-26%	-27%	68%	125%	-33%	0%	-5%	-20%	-13%	-26%	-5%	-69%	-6%	-1%	-3%	-50%	-1%	0%	32%	-9%	-24%	-2%	-33%	-67%

Discussion

The following can be concluded from the comparison of traffic flows jointly with queue length:

- Generally, queues decreased at several movements at the N. Broadway Ave. and E. Van Fleet Drive intersection, despite increases in volume.
- The intersection of Walmart Access and E. Van Fleet Drive has queue changes correlating with volume changes, with a few exceptions. The NB left turn shows the most improvement, with an increase in volume (5%) and a decrease in queuing (14%).

The main objective of this project is to evaluate the implementation of proposed adaptive signal control technology (ASCT) traffic operations at several arterial corridors in Florida, before and after the installation of the ASCT, document the advantages and the disadvantages of different approaches and implementations, and provide recommendations for statewide implementation of ASCT.

H.4 QUESTIONNAIRE

SECTION 1: RESPONDENT'S INFORMATION

Name **Renjan Joseph**

Organization FDOT

Position Arterial Management System Engineer

Address FDOT District 1 – Traffic Operations 801 N Broadway Ave, P.O. Box -1249

Phone (863) – 519 - 2746

Fax

E-mail Renjan.Joseph@dot.state.fl.us

SECTION 2: PREVIOUS TRAFFIC CONTROL TECHNOLOGY

1. Please specify the type of traffic control used before Adaptive Signal Control Technology (ASCT) was installed for your site. (Please check all that apply.)

a. Fixed time coordinated control **<u>b. Actuated coordinated control</u>**

c. Fixed-time isolated control d. Actuated isolated control

e. Other, please specify: ____

2. What type of traffic controller was used before ASCT was implemented?

a. NEMA TS-1 **<u>b. NEMA TS-2</u>**

c. 170 d. 2070

e. Other, please specify: ____

26. How many detectors were used before ASCT was implemented for a typical intersection (e.g. *Newberry Rd & NW 75th St*)? Please also specify the location where the detectors were typically placed for each specific detector type (video detection, inductive loops, radars, etc.).

Detectors at all intersections. The detectors used were radars, videos, and loops.

27. What was the frequency of retiming the signal timing plan for the corridor before ASCT was implemented?

Last full time retiming was 2015 after it was done in 2007. Timings not done frequently because some systems do not need it.

28. How often were maintenance and updates performed to your previous control system in terms of the following components?

a. Detection: **Once a year**
b. Control Hardware: **Once a year**
c. Sofware: **Once a year**

29. What was the annual maintenance cost in terms of hardware, software and personnel of the previous control system for the whole corridor where the ASCT is now deployed?

The maintenance cost for the 5 intersections in the Bartow system was approximately $6025 in 2015

30. What was the life expectancy of your traffic control before the deployment of ASCT? (Please refer to the hardware and software.)

The controllers are usually not replaced based on a new project

31. How many crashes of each category were reported during the past 4 years before the ASCT implementation? (2011-2014)

	K (Killed)	A (Disabling Injury)	B (Evident Injury)	C (Possible Injury)	O (No Apparent Injury)
2011					
2012					
2013					
2014					

SECTION 3: ADAPTIVE TRAFFIC CONTROL SYSTEM (ASCT)

9. Which Adaptive Traffic Control System (ASCT) does your agency deploy?

p. <u>**InSync; InSync was initially used, then updated to InTraffic in January 2017**</u>

q. Synchro Green; Version: ____________________

r. Other; please specify: _______________________

40. Did you consider any other ASCT before you selected this one for installation? Why did you reject the other(s) in favor of this one?

There weren't many ASCT so the most common type, InSync, was chosen

41. What were your major criteria for choosing the ASCT for the selected corridor?

Complexity of the corridor. Just a test on ASCT because it wasn't popular

42. What is the life expectancy of your ASCT system? (Consider both hardware and software.)

1.5 years

43. What type of traffic controller is currently in use after the ASCT implementation?

a. Same as before the ASCT implementation

b. New, please specify type: ____

c. <u>**Same, but the following updates were needed: added backup detection everywhere**</u>

44. How many and what type of detectors are used after the ASCT implementation on an intersection level? Please note any updates that the previous detection system needed so as to work with your ASCT.

Radar(electronics) at all locations. They all InSync camera for queue detection. The outer two intersections have loops and the three intersections in the middle have electronics.

45. Was there a need to update your previous software operating system at your Transportation Management Center in order to get your current ASCT software working?

a. Yes **b. No**

41. How often do you plan to perform physical maintenance or updates on your new ASCT in terms of the following components?

a. Detection: **Annual maintenance program**
b. ASCT Hardware: **Annual maintenance program**
c. Software: **Annual maintenance program**

42. Was there a need for training your personnel in order to operate the new ASCT?

a. **Yes,** Num. of employees trained **9**, Hours of training per employee **8** b. No

43. Was there any change on the number of staff required to operate/maintain the effective signal ASCT operations? (Comparison of staff needed before and after the deployment.)

No

44. Where did expertise come from for your personnel's training, the ASCT installation, operation and the system's maintenance? Please check all that apply.

	In-House	Vendor	Contractor
Employee Training		**yes**	
ASCT Installation			**yes**
ASCT Operation	**yes**		
ASCT Maintenance	**yes**		

45. How many crashes of each category were reported after ASCT was implemented?

K (Killed)	A (Disabling Injury)	B (Evident Injury)	C (Possible Injury)	O (No Apparent Injury)

SECTION 4: COST COMPONENTS

61. What is the overall capital cost of purchasing the ASCT (in terms of the software and licenses of the ASCT) for the corridor? If the purchase includes any service of implementation, personnel training or maintenance, please also specify here.

InSync cost: $186,950, Equipment (Blue Toad): $51,535, Installation (Blue Toad): $4867.60, Initial Install (TransCore): $136,739.43, Wavetronix: $81,097.58, Add'l equipment: $60,270.89, Second Install (Transcore): $136,739.43

62. What is the implementation cost (considering the installation on site, any updates in software and hardware, design needs and contract hours) for the ASCT corridor? Please specify if this cost is partially or totally included in previous cost items.

Installation cost for BlueToad and TransCore: 278,336.46

63. What was total cost for training your personnel to operate and manage the new ASCT? Please specify if this cost is partially or totally included in previous cost items.

Cost of training was included in the total package

64. What is the expected maintenance cost (in terms of hardware, software and personnel) for the ASCT corridor on a yearly basis? Please specify if this cost is partially or totally included in previous cost items.

____

____

65. Can you specify the costs for the following ASCT components?

a. Firmware $____________ b. Software $_____________

c. Equipment **$379,852.89** d. Maintenance of Traffic Cost $_____________

e. Design Needs $__________ f. Contract Help/Agency Hours Cost $_________

SECTION 5: INSTITUTIONAL ISSUES

66. Were there any institutional issues that you had to overcome while implementing and operating the ASCT project? Please categorize and explain those issues below.

a. Organization and Management Institutional Issues:

Skeptical about the installation of this new ASCT

b. Regulatory and Legal Institutional Issues:

____

____

c. Human and Facility Resources Institutional Issues:

____

____

d. Financial Institutional Issues:

____

____

67. Which component (e.g. detection, communication, hardware, software, licensing or other) of your ASCT deployment is the most challenging to maintain and why?

Public perception. Adjusting to the adaptiveness of the signal

68. What happens to your ASCT operations when some of your detectors fail on the corridor level?

p. ASCT triggers an alarm and notifies operators

q. ASCT switches to an "off-line" mode by implementing Time-Of-Day plans

r. Combination of the above. **d. Other (please describe):**

29. How is your ASCT performance evaluated?

s. **In-house: done by multiple people**

t. **By an independent evaluator**, please describe: ________________________________

u. Not applicable—there is no evaluation

30. If the corridor on which the ASCT operates experiences over saturation, how would you rate the operation of the ASCT in response to these traffic conditions?

z. ASCT prevents or eliminates oversaturation aa. ASCT eliminates or reduces the extent of the periods of oversaturation

bb. ASCT adversely affects the traffic conditions during periods of oversaturation

cc. **Other: Oversaturation occurs virtually every day in the PM Peak**

dd. Oversaturation is very rare on the corridors operated by our ASCT

31. Based on your opinion and the up-to-date operation, when are ASCT operations proven to be the most effective?

a. Peak periods **b. Off-peak periods**

c. Shoulders of peak periods d. Other, please specify: _______________

32. Has the level of performance of ASCT been sustained since its installation?

a. **Yes b. No; why not? It appears inconsistent, however, it has become more consistent after the software update**

43. What was the public reaction to the ASCT operation? Have you received any complaints about long delays and queues?

Frequent complains after installation initially but complains have reduced in the last 6 months

44. How satisfied are you, in general, with your ASCT deployment?

a. Very satisfied b. Somewhat satisfied

c. Neutral d. Somewhat dissatisfied

e. Not satisfied at all

43. Are there any other costs or benefits related to ASCT deployment that you would like to report?

a.Benefits____

____

b.Costs____

____

44. Would you consider expanding the ASCT program to any other corridors? Why or why not? If yes, would you use the same technology/firmware or have any suggestions for other systems? If so, why?

Yes/No. The results from the study will help decide. However, Synchro Green is being tried on a different corridor

SECTION 6: SAFETY ISSUES

37. Do you have any anecdotal / qualitative data on safety benefits / disbenefits of the signal improvement along this corridor?

____

____

38. Has there been other changes along this corridor over the analysis period that could affect the safety of this corridor either positively or negatively?

____

____

39. Are you aware of any changes to the crash data reporting procedures over the analysis period? If yes, what are the changes?

____

____

____

____

40. What are your overall reactions to the trends presented in the report?

____

____

41. Is there another comparable corridor in which signal improvements have not been made to which the results of this corridor can be compared to?

____

____

42. Other thoughts / comments for us?

____

Thank you for your help in completing this survey. Your responses will help us with evaluating the deployment and benefits of your current ASCT.

If you have any questions regarding the survey, please contact Marian Ankomah, email: moankomah@ufl.edu or Tyler Valila, email: tvalila67@ufl.edu who work under the direct supervision of the faculty advisor Dr. Lily Elefteriadou.

H.5 BENEFIT COST ANALYSIS

Table H-7.1 Monetized Saving

Monetized Saving	
Time saving	-$401,717.09
Fuel Consumption saving	-$60,822.76
Air Pollution Saving	-$32,104.06
Safety Saving	$472,348.77
Total Saving without Safety	-$494,643.91
Total Saving	-$22,295.14
Total Cost	$482,000.00
B/C Ratio without safety	-1.026232173
B/C Ratio	-0.046255477

Table H-7.2 Monetized Saving with No Emissions

Monetized Saving	
Time saving	-$401,717.09
Fuel Consumption saving	-$60,822.76
Safety Saving	$472,348.77

Monetized Saving	
Total Saving without Safety Total Saving	-$462,539.85 $9,808.92
Total Cost	$482,000.00
B/C Ratio without safety B/C Ratio	-0.959626236 0.020350459

APPENDIX I: Time Taken to Add Crash Data

To examine the time taken for most crashes to get added to the system, we examined the data between January and November 2017 for all the corridors and for the counties to which these corridors belong. Tables I-1 and I-2 present the number of days taken to incorporate 90% and 95% of all crashes into the system. The time taken is calculated as the number of days between the date the crash shows up in Signal Four and the date of the crash. Results are presented separately for long-form and short-form crashes.

Table I-1 Days to Incorporate 90% and 95% of crashes on the corridors

Corridor	Long Form			Short Form		
	Total	90%	95%	Total	90%	95%
US 17/92	62	27	39	56	37	39
Newberry Road	356	20	27	37	16	18
Panama City Beach (PCB) Parkway	219	15	27	535	12	24
23rd Street	164	14	26	153	21	28
University Parkway	137	13	15	54	10	11
Bartow Corridor	66	28	33	43	13	19
SR 693	259	18	30	196	15	20
SR-70	649	12	19	276	12	23

Table I-2 Days to Incorporate 90% and 95% of crashes in the counties

County	Long Form			Short Form		
	Total	90%	95%	Total	90%	95%
Volusia	8609	19	28	4529	21	27
Alachua	4150	16	30	417	14	20
Bay	4343	13	23	3422	13	24

County	Long Form			Short Form		
	Total	90%	95%	Total	90%	95%
Sarasota	6893	18	29	4077	20	30
Polk	10967	21	29	4381	20	29
Pinellas	21822	24	34	5433	23	33
Manatee	6750	16	30	2042	18	35

APPENDIX J: Monthly Variation of Long and Short Form Crashes

To examine the issue of crash reporting practices not changed over the analysis period, we looked at the monthly variations in the proportion of long- and short- form crashes over the period January 2013 – November 2017 in the counties of interest (Figures J-1 through J-7).

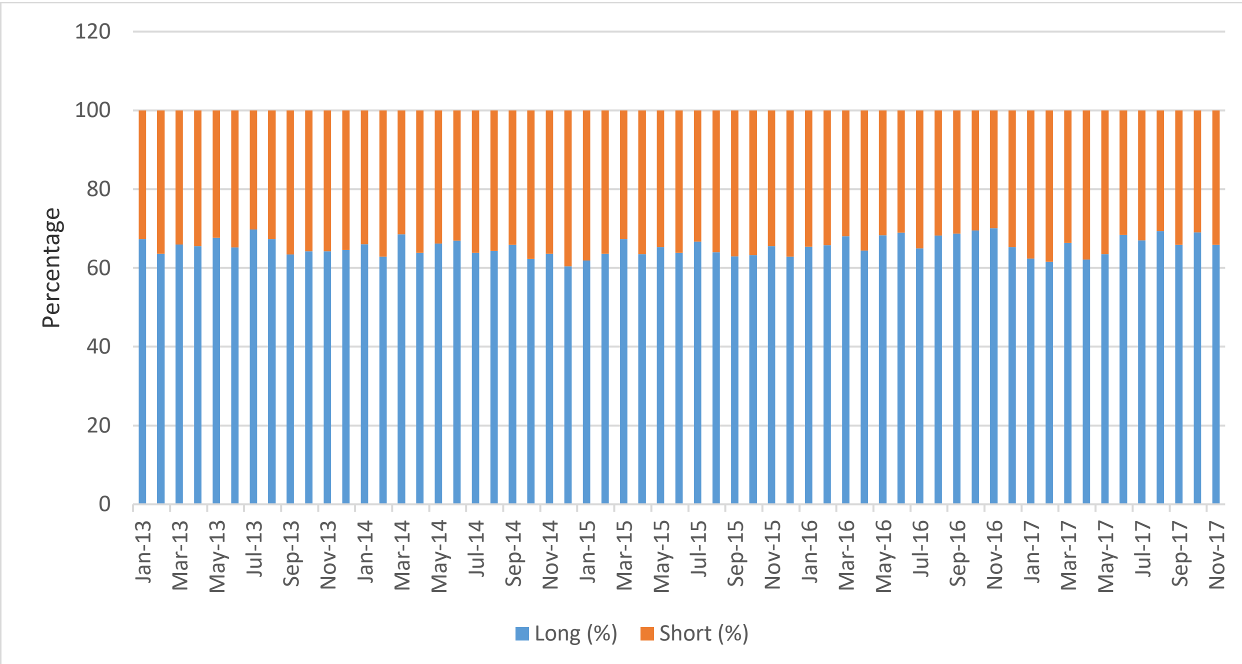


Figure J-1 Proportion of Short Form Crashes to Long Form Crashes in Volusia County

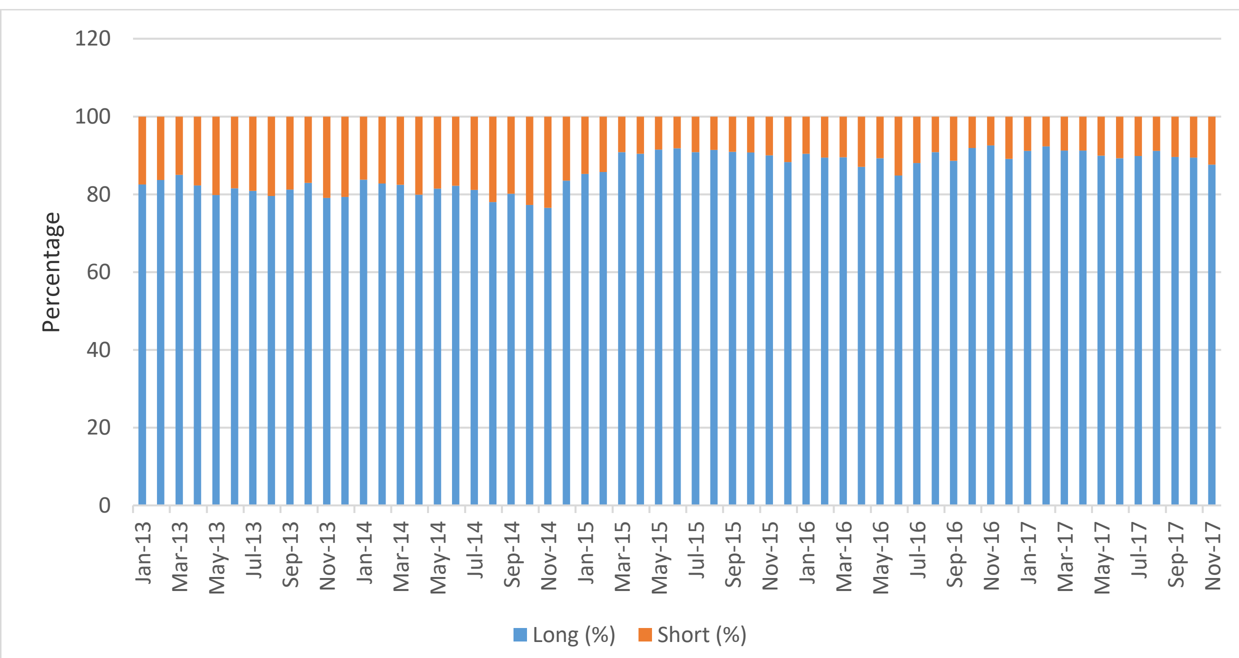


Figure J-2 Proportion of Short Form Crashes to Long Form Crashes in Alachua County

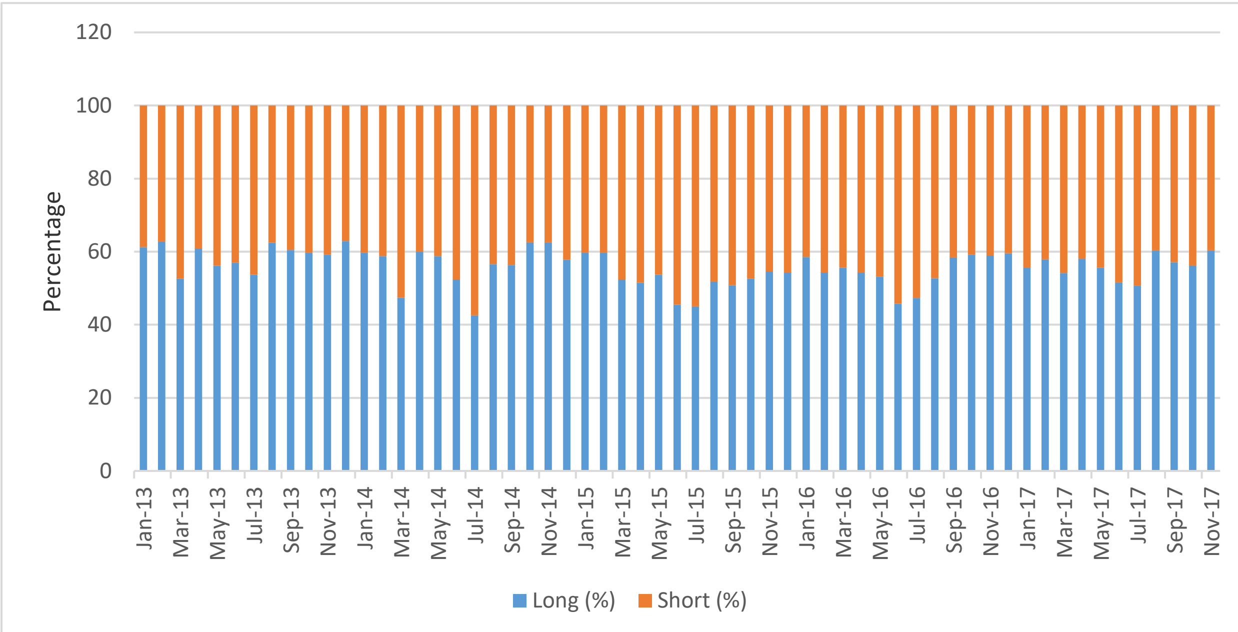


Figure J-3 Proportion of Short Form Crashes to Long Form Crashes in Bay County

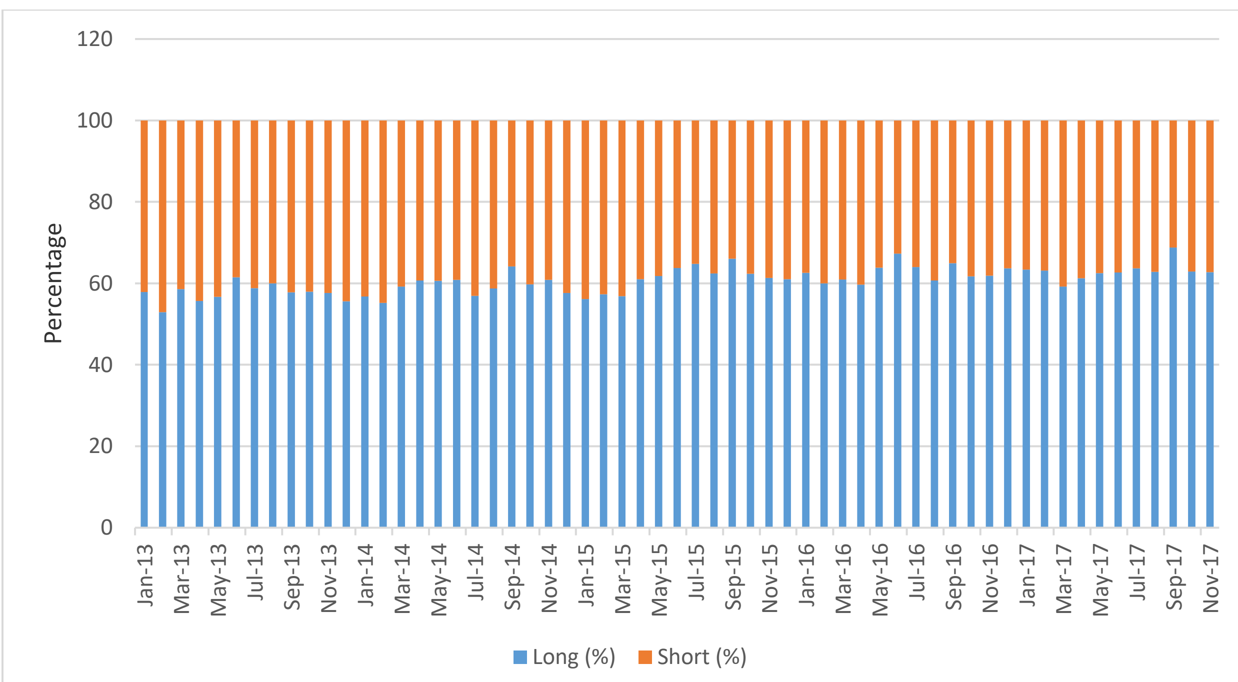


Figure J-4 Proportion of Short Form Crashes to Long Form Crashes in Sarasota County

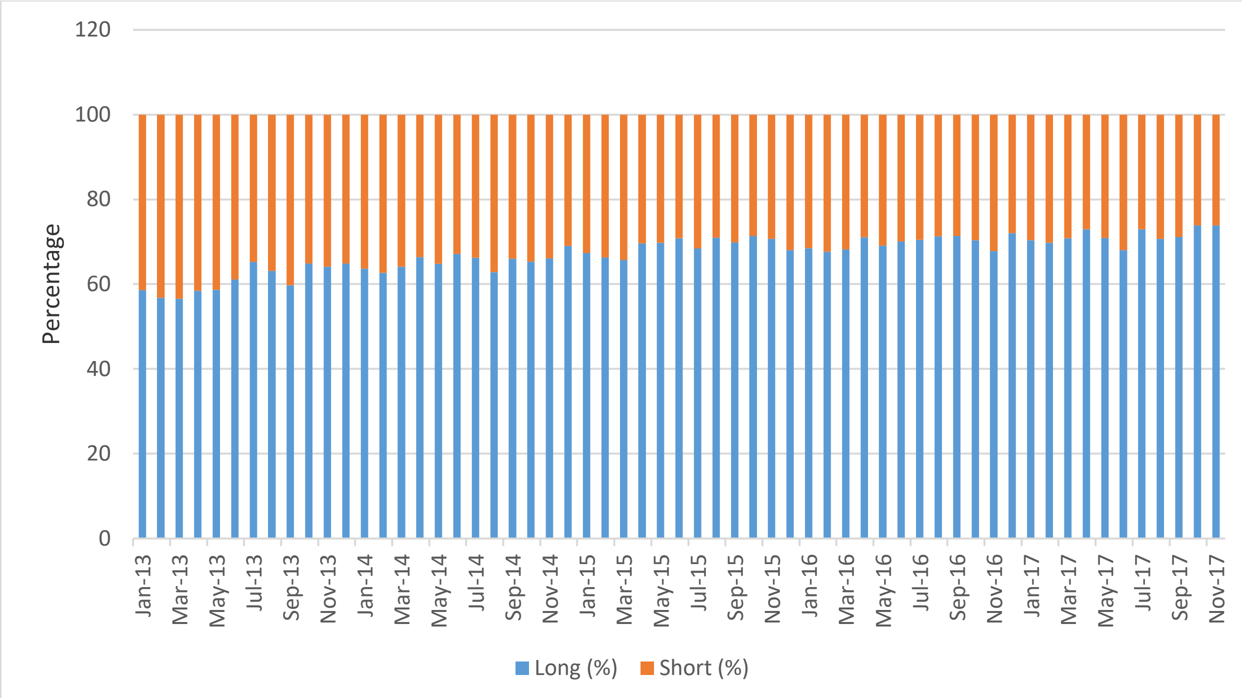


Figure J-5 Proportion of Short Form Crashes to Long Form Crashes in Polk County

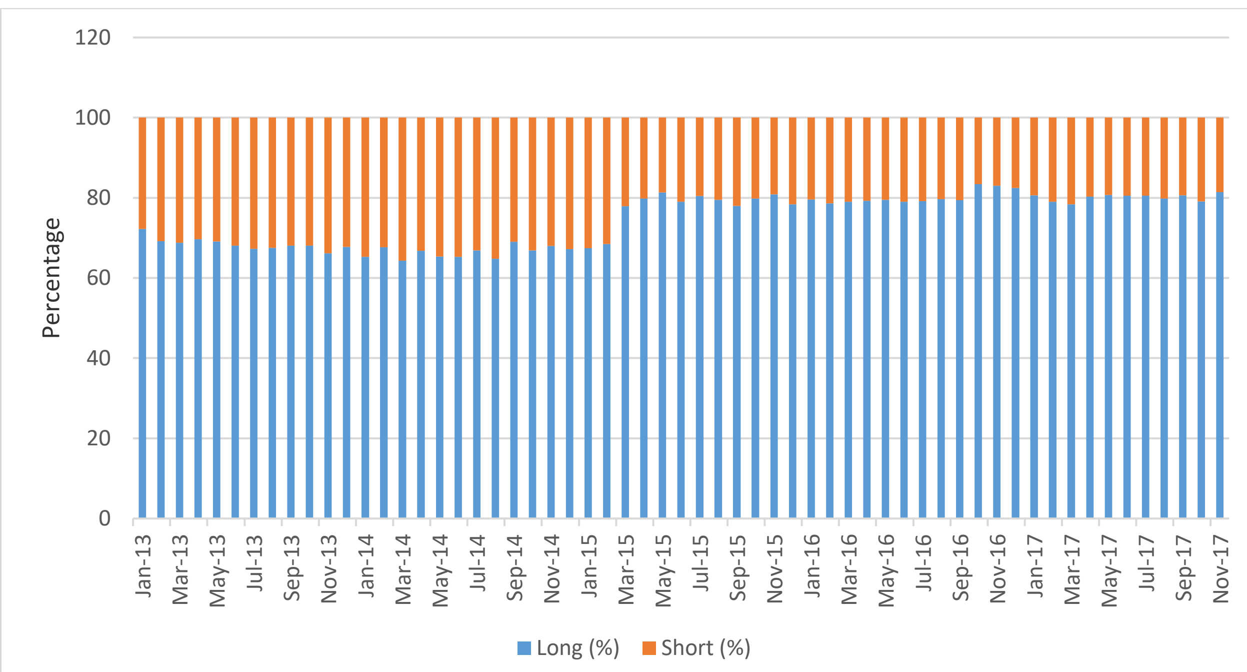


Figure J-6 Proportion of Short Form Crashes to Long Form Crashes in Pinellas County

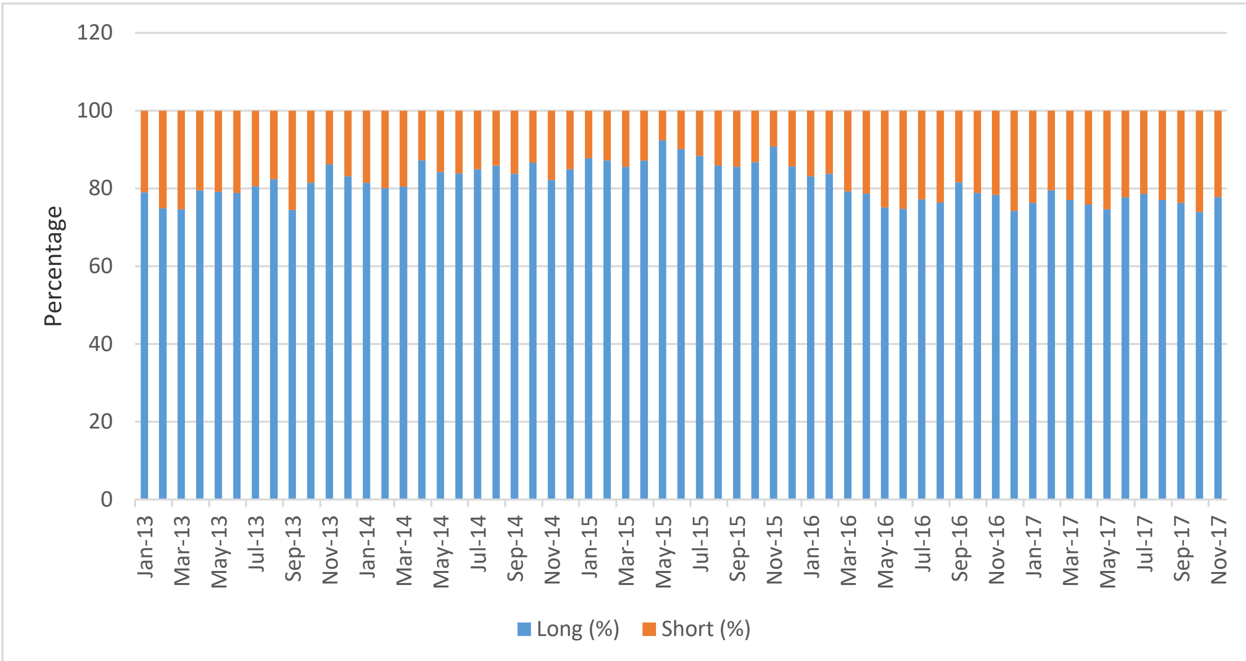


Figure J-7 Proportion of Short Form Crashes to Long Form Crashes in Manatee County

APPENDIX K: Historical AADT for Each Corridor

As stated earlier in the document, the AADT was obtained from the FDOT Florida Traffic online. The AADT for the corridors obtained were from years 2013-2017. Each corridor is made up of at least one segment. Therefore, the AADT for each segment of the selected corridors were obtained (see Figure K-1) and the total AADT for the corridor (see Table K-1) for each year was calculated based on the length of the segment. The formula used to calculate the AADT is shown below:

$$\frac{\sum(Segment\ Shape\ Length\ x\ AADT\ of\ \ segment)}{\sum Segment\ Shape\ Length}$$

CORRIDOR	COUNTY NAME	DESC_FRM	DESC_TO	AADT_2013	AADT_2014	AADT_2015	AADT_2016	AADT_2017	Shape_Length
BARTOW	POLK	SR 60/VAN FLEET DR	LYLE PKWY	40000	33000	38000	39000	35800	980.03
BARTOW	POLK	SR 60	SR 555/700,US17/98	18900	19300	19400	19900	22000	300.20
BARTOW	POLK	BROADWAY AVE	SR 60	34000	36500	40000	42000	40500	984.99
SR 693	PINELLAS	40TH AVE N	PARK BLVD N	38500	39500	41000	39000	40000	3429.54
SR 693	PINELLAS	CR-346/126 AVE N	CR-376/142 AVE N	27500	27500	31000	31500	32500	1605.81
SR 693	PINELLAS	N/A	CR-346/126 AVE N	37500	37000	38000	33000	34000	1602.69
SR 693	PINELLAS	CR-694/PARK BLVD	N/A	36000	36500	37000	36000	37000	3628.06
US 17/92	VOLUSIA	E TAYLOR RD	E BERESFORD AVE	27000	27500	23500	33500	30000	1648.40
US 17/92	VOLUSIA	N/A	E TAYLOR RD	45000	42000	44000	47000	47500	3199.91
23RD STREET	BAY	CR-2341/JENKS AVE	SR-77/MLK BLVD	0	26500	31000	34000	34000	1214.22
23RD STREET	BAY	STATE AVE	CR-2341/JENKS AVE	36000	35000	36000	36500	36500	399.82
23RD STREET	BAY	SR-391/AIRPORT RD	STATE AVE	31000	31500	31000	33000	34500	1173.95
23RD STREET	BAY	CR-385/FRANKFORD AVE	SR-391/AIRPORT RD	30000	30000	31000	31000	32000	1252.98
PCB PARKWAY	BAY	SR-30/FRONT BEACH RD	BULLNOSE AT 46160000	37500	37500	40500	40500	36200	474.02
PCB PARKWAY	BAY	PHYSICAL GORE	US-98/SR-30	22700	23800	25500	24500	21300	500.25
PCB PARKWAY	BAY	R JACKSON BLVD	N/A	37500	37500	40500	40500	39500	4259.78
PCB PARKWAY	BAY	HILL RD	R JACKSON BLVD	43000	46500	46500	49000	54000	5457.04
PCB PARKWAY	BAY	SR-79	HILL RD	39000	45500	46000	45000	46500	2915.98
PCB PARKWAY	BAY	BULLNOSE OF 46160104	CR-3031/THOMAS DR	51500	53000	53500	53000	54000	669.97
PCB PARKWAY	BAY	CR-3031/THOMAS DR	BULLNOSE OF BRIDGE	55000	58000	58500	58500	58500	2490.58
UNIVERSITY PARKWAY	SARASOTA	HONORE AVE	OFF 17075003	53000	54000	58000	60000	47500	1673.75
UNIVERSITY PARKWAY	SARASOTA	LOCKWOOD RIDGE RD	HONORE AVE	51000	52000	57000	59000	48500	4137.63
UNIVERSITY PARKWAY	SARASOTA	N/A	LOCKWOOD RIDGE RD	37500	38500	43500	45500	42500	2394.66
UNIVERSITY PARKWAY	SARASOTA	N/A	N/A	28000	28000	30000	31000	29000	1986.59
UNIVERSITY PARKWAY	SARASOTA	SR45/US41/TAMIAMI T	N/A	21000	21000	22000	23000	21500	973.92
UNIVERSITY PARKWAY	MANATEE	I 75 RAMPS	LAKEWOOD RANCH BLVD	34256	32597	34000	34000	33500	1407.37

Figure K-1 Segment Level AADT Data

Table K-1 Corridor AADT from 2013-2016

Corridor	2013	2014	2015	2016	2017
US 17/92	38880	37070	37030	42410	41550
Panama City Beach Parkway	42268	45076	46155	46745	48202
23rd Street	21870	29879	31495	33027	33772
University Parkway	40864	41331	45011	46552	40373
Bartow	34595	32706	36405	37773	36015
SR 693	35740	36172	37554	35830	36830

APPENDIX L: Short Term Changes in Crashes

Short-Term (7 Months) Safety Effects of Signal Improvement

Table L-1 US 17/92 (Deland)

			All crashes	Fatal and Injury	Intersection Related	Rear End	Weekday AM Peak	Weekday PM Peak	Weekday Off Peak	Weekend
US 17/92 (Mainline +Side Streets)	Before	Crash Freq.	106	22	18	26	11	15	56	24
	After	Crash Freq.	85	23	18	17	11	18	38	18
	Diff (%)		-19.81	4.55	0.00	-34.62	0.00	20.00	-32.14	-25.00
US 17/92 (Mainline Corridor)	Before	Crash Freq.	75	16	10	21	7	11	37	20
	After	Crash Freq.	48	11	8	9	5	10	23	10
	Diff (%)		-36.00	-31.25	-20.00	-57.14	-28.57	-9.09	-37.84	-50.00
Volusia County	Before	Crash Freq.	6983	1952	1473	2110	775	1204	3303	1701
	After	Crash Freq.	7274	2083	1623	2268	785	1312	3419	1758
	Diff (%)		4.17	6.71	10.18	7.49	1.29	8.97	3.51	3.35

Table L-2 Panama City Beach (PCB)

			All crashes	Fatal and Injury	Intersection Related	Rear End	Weekday AM Peak	Weekday PM Peak	Weekday Off Peak	Weekend
PCB Parkway (Mainline +Side Streets)	Before	Crash Freq.	293	50	21	71	28	59	131	75
	After	Crash Freq.	413	45	26	72	32	78	189	114
	Diff (%)		40.96	-10.00	23.81	1.41	14.29	32.20	44.27	52.00
PCB Parkway (Mainline Corridor)	Before	Crash Freq.	231	41	17	73	22	49	96	64
	After	Crash Freq.	332	39	17	75	24	70	156	82
	Diff (%)		43.72	-4.88	0.00	2.74	9.09	42.86	62.50	28.13
Bay County	Before	Crash Freq.	4108	762	667	1249	475	818	1890	925
	After	Crash Freq.	5029	878	597	1460	548	926	2367	1188
	Diff (%)		22.42	15.22	-10.49	16.89	15.37	13.20	25.24	28.43

Table L-3 23rd Street

			All crashes	Fatal and Injury	Intersection Related	Rear End	Weekday AM Peak	Weekday PM Peak	Weekday Off Peak	Weekend
23rd St. (Mainline +Side Streets)	Before	Crash Freq.	306	50	42	113	27	49	176	54
	After	Crash Freq.	272	36	24	94	23	48	147	54
	Diff (%)		-11.11	-28.00	-42.86	-16.81	-14.81	-2.04	-16.48	0.00
23rd St. (Mainline Corridor)	Before	Crash Freq.	157	34	28	93	14	30	92	21
	After	Crash Freq.	110	25	13	62	9	26	52	23
	Diff (%)		-29.94	-26.47	-53.57	-33.33	-35.71	-13.33	-43.48	9.52
Bay County	Before	Crash Freq.	4265	804	674	1291	543	801	2026	895
	After	Crash Freq.	4964	880	652	1536	592	961	2253	1158
	Diff (%)		16.39	9.45	-3.26	18.98	9.02	19.98	11.20	29.39

Table L-4 University Parkway

			All crashes	Fatal and Injury	Intersection Related	Rear End	Weekday AM Peak	Weekday PM Peak	Weekday Off Peak	Weekend
University Parkway (Mainline +Side Streets)	Before	Crash Freq.	111	39	51	69	12	23	44	32
	After	Crash Freq.	84	28	38	56	19	13	36	16
	Diff (%)		-24.32	-28.21	-25.49	-18.84	58.33	-43.48	-18.18	-50.00
University Parkway (Mainline Corridor)	Before	Crash Freq.	85	30	36	58	9	17	39	20
	After	Crash Freq.	81	22	32	52	23	11	34	13
	Diff (%)		-4.71	-26.67	-11.11	4.00	155.56	-35.29	-12.82	-35.00
Sarasota County	Before	Crash Freq.	6519	1547	1738	2505	811	1359	3010	1339
	After	Crash Freq.	6725	1588	1487	2550	896	1330	3186	1313
	Diff (%)		3.16	2.65	-14.44	1.80	10.48	-2.13	5.85	-1.94

Table L-5 Bartow

			All crashes	Fatal and Injury	Intersection Related	Rear End	Weekday AM Peak	Weekday PM Peak	Weekday Off Peak	Weekend
Bartow (Mainline +Side Streets)	Before	Crash Freq.	75	19	27	34	5	19	27	24
	After	Crash Freq.	54	13	14	31	9	10	20	15
	Diff (%)		-28.00	-31.58	-48.15	-8.82	80.00	-47.37	-25.93	-37.50
Bartow (Mainline Corridor)	Before	Crash Freq.	36	9	11	18	3	6	16	11
	After	Crash Freq.	31	9	4	19	6	4	11	10
	Diff (%)		-13.89	0.00	-63.64	4.00	100.00	-33.33	-31.25	-9.09
Polk County	Before	Crash Freq.	8353	2624	2094	2926	1045	1666	3762	1880
	After	Crash Freq.	8652	2597	1994	3048	1107	1639	4052	1854
	Diff (%)		3.58	-1.03	-4.78	4.17	5.93	-1.62	7.71	-1.38

Table L-6 SR 693

			All crashes	Fatal and Injury	Intersection Related	Rear End	Weekday AM Peak	Weekday PM Peak	Weekday Off Peak	Weekend
SR 693 (Mainline +Side Streets)	Before	Crash Freq.	235	78	108	127	25	41	130	39
	After	Crash Freq.	266	72	125	138	43	56	123	44
	Diff (%)		13.19	-7.69	15.74	8.66	72.00	36.59	-5.38	12.82
SR 693 (Mainline Corridor)	Before	Crash Freq.	111	38	46	63	14	16	66	15
	After	Crash Freq.	125	35	53	67	24	23	55	23
	Diff (%)		12.61	-7.89	15.22	4.00	71.43	43.75	-16.67	53.33
Pinellas County	Before	Crash Freq.	16536	4315	4999	5878	2279	3264	7357	3636
	After	Crash Freq.	15889	3957	5382	5799	2119	3350	7173	3247
	Diff (%)		-3.91	-8.30	7.66	-1.34	-7.02	2.63	-2.50	-10.70