US 278 (William Hilton Parkway) Traffic Signal Retiming

Final Report

November 2018

Prepared for:

South Carolina Department of Transportation

Prepared by:

Stantec Consulting Services, Inc
4969 Centre Pointe Drive, Suite 200
North Charleston, SC 29418

Table of Contents

LIST OF TABLES

LIST OF FIGURES

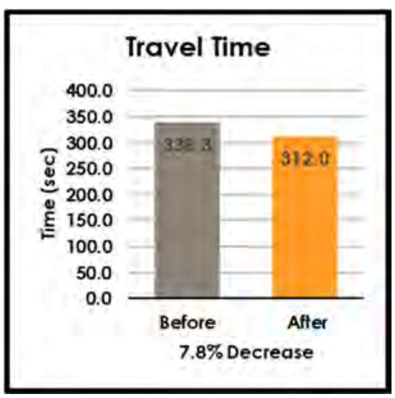
LIST OF APPENDICES

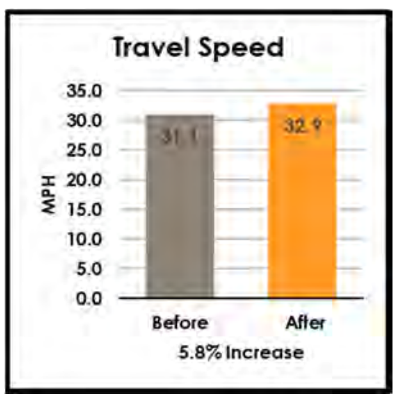
Executive Summary

Stantec Consulting Services, Inc. (Stantec), under contract with the South Carolina Department of Transportation (SCDOT), has developed and implemented new coordinated traffic signal timing plans for seven signals along US 278 (William Hilton Parkway) in the Town of Hilton Head Island, Beaufort County, South Carolina. The timing plans developed for this project include the weekday AM peak period, weekday Midday period, weekday PM peak period, Saturday outflow peak period, Saturday inflow peak period, Sunday outflow peak period, Sunday inflow peak period, and an Off-Peak plan. Two sets of timing plans were developed for each of these peak periods—an Off-Season set which runs from approximately mid-September through mid-March, and a Peak-Season set which runs from mid-March through mid-September to better serve the increased traffic demands during the summer months.

To determine the effectiveness of the implemented new signal timing plans, travel time studies were performed using a GPS receiver. This data was processed with Tru-Traffic software to evaluate and document the results of the timing plan development process. This report presents the results of the “before” and “after” studies that were conducted for the seven signals along the US 278 (William Hilton Parkway) corridor included in this project. Due to the abbreviated schedule of this project, before and after studies were conducted for the Off-Season only, with travel time runs recorded on Spring and Fall weeks selected for their comparable daily volume as based upon historical trends.

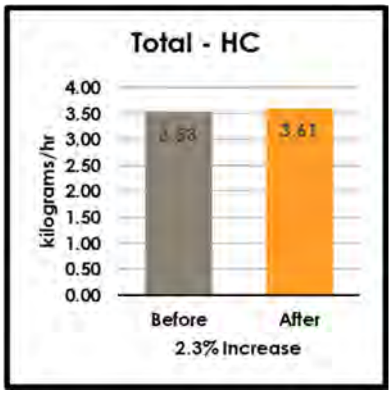
The travel time studies were conducted on typical weekdays during three time periods of the day: AM peak (06:00-09:30), Midday (09:30-14:20), and PM peak (14:20-19:00). The before runs were recorded on Wednesday, March 28, 2018. The after runs were recorded on Thursday, October 4, 2018, a week selected based upon comparable daily volume when considering seasonal adjustments. The following charts show the average improvements experienced along US 278 (William Hilton Parkway) for both directions of travel during all three time periods. Charts summarizing the detailed results by each timing plan and direction of travel are presented later in this report. All results shown below were calculated using the Tru-Traffic software (version 10.0).

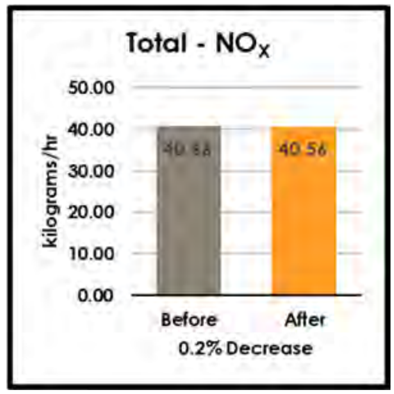


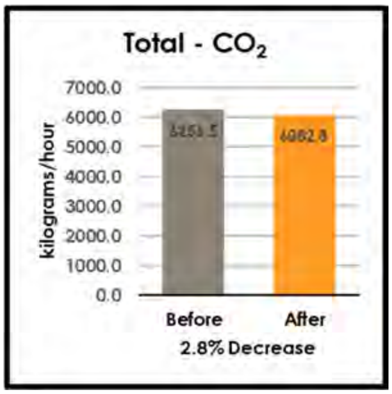


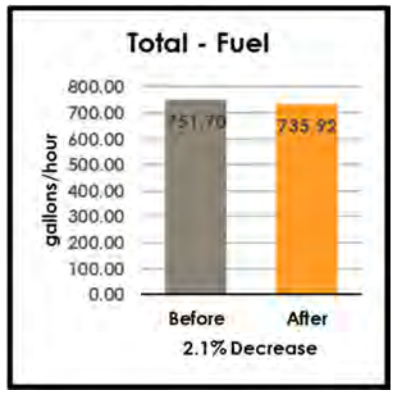
As evident in the graphs above, improvements were shown in travel time, delay and speed for the US 278 (William Hilton Parkway) corridor.

Hydrocarbons (HC) and oxides of nitrogen (NO_x), which are vehicle emissions regulated by federal law, along with carbon dioxide (CO_2) emissions and fuel consumption were estimated by processing the travel time runs using the Tru-Traffic software. The following charts show the cumulative average improvements experienced along US 278 (William Hilton Parkway) for both directions of travel during all three time periods. Charts summarizing the detailed results by each weekday timing plan are presented in subsequent sections of this report.









As evident in the graphs above, improvements were shown in fuel consumption and the emissions of carbon dioxide and oxides of nitrogen. Estimated emissions of hydrocarbons along US 278 (William Hilton Parkway) show a minor increase during the three time periods measured.

Delay incurs direct costs upon motorists in the form of increased fuel consumption and the value of their time wasted while waiting in traffic. Motorists using US 278 (William Hilton Parkway) during the weekday AM, Midday, and PM peak periods are expected to save 17,632 hours each year because of the improved traffic flow due to the new timing plans.

Conservatively assuming a vehicle occupancy of 1.2 persons/vehicle, $12.00 per hour for the value of motorists' time, and $2.59 per gallon for gasoline, annual savings to motorists along US 278 (William Hilton Parkway) are expected to be $253,907 in the form of reduced delay and $10,221 decrease in cost due to decreased fuel consumption, for a total annual savings of $264,128. The estimates for hours and fuel saved per year assume that similar improvements in travel time and fuel consumption as seen in the Off-Season would be experienced during the Peak-Season weekday peak periods, as well.

Other benefits not considered in this analysis include lower driver frustration levels and a potential reduction of collisions. All of the results mentioned in the report are for three hours a day for each weekday during the AM, Midday, and PM peak periods. New signal timing plans were also implemented during the weekend peak hours. However, because benefit/cost "before" and "after" studies were not conducted during these time periods, additional savings could not be quantified during these periods.

Based on equivalent annual cost of designing, implementing, and documenting signal timing plan improvements, the benefit to cost ratios for interest rates ranging from 4% to 8% were calculated to be between 12.3:1 and 13.0:1 for this project.

1.0 INTRODUCTION

This document describes the development of preliminary timing plans by Stantec for seven (7) intersections along US 278 (William Hilton Parkway) in the Town of Hilton Head Island, Beaufort County, South Carolina. The intersections are listed in **Table 1** and shown on the following page in **Figure 1**.

The purpose of this project is to improve traffic flow along the US 278 (William Hilton Parkway) corridor by developing and implementing coordinated traffic signal timing plans for the following intersections:

Table 1 – Project Intersections

Signal ID	Intersection
6100	US 278 (William Hilton Parkway) & S-141 (Squire Pope Road) & Chamberlin Drive
6101	US 278 (William Hilton Parkway) & Wild Horse Road & S-79 (Spanish Wells Road)
6102	US 278 Business (William Hilton Parkway) & S-482 (Gum Tree Road) & Ramps to/from Cross Island Parkway
6103	US 278 Business (William Hilton Parkway) & S-626 (Wilborn Road) & Jarvis Park Road
6104	US 278 Business (William Hilton Parkway) & Museum Street & Pembroke Drive
6105	US 278 Business (William Hilton Parkway) & Whooping Crane Way & Indigo Run Drive
6106	US 278 Business (William Hilton Parkway) & Beach City Road & Gardner Drive

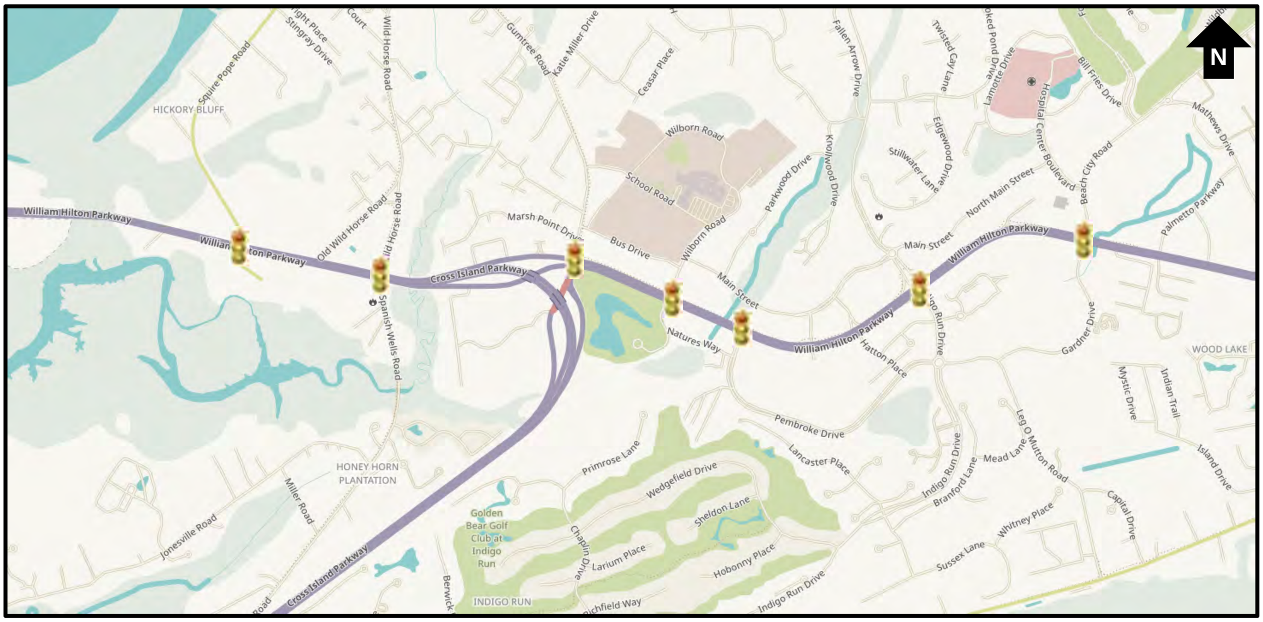


Figure 1 – Project Location

US 278 and US 278 Business together form a loop around Hilton Head Island, providing not only access between destinations on-island but also the primary means of conveyance for motorists travelling to and from the mainland to the west. This section of US 278, located along the northern end of the island, is a principal arterial divided highway approximately 2.7 miles in length with a posted speed limit of 45 mph that serves as the sole roadway connecting Hilton Head Island with the mainland to the west. The Cross Island Parkway, a toll expressway which provides a controlled access connection to the southern end of Hilton Head Island, merges and diverges interior to this segment of William Hilton Parkway and contributes a modest change in traffic volumes toward the bridges. West of Squire Pope Road, US 278 has two travel lanes in each direction, separated by a flush median. Between Squire Pope Road and Wild Horse Road/Spanish Wells Road, US 278 has three travel lanes in each direction separated by a flush median which transitions to a raised landscaped median toward the east. Continuing to the east of Wild Horse Road/Spanish Wells Road, US 278 has four travel lanes in each direction until the free-flowing ramps to/from the Cross Island Parkway merge and diverge from the median of US 278, leaving three travel lanes in each direction east of the Cross Island Parkway. East of Wilborn Road/Jarvis Park Road, US 278 is divided by a wide earthen median and has two travel lanes in each direction with auxiliary lanes dedicated to turning traffic at most intersections.

This report is divided into the following sections:

I. Introduction
II. Inventory & Data Collection
III. Local Timing Parameters
IV. Coordination Parameters
V. Operational Analysis
VI. Results Summary
VII. Effectiveness Evaluation
VIII. Conclusions

2.0 INVENTORY & DATA COLLECTION

2.1 INVENTORY

Stantec staff completed an inventory of each of the project intersections. Information obtained consists of the intersection configuration, signing and marking configurations, signal phasing, and pedestrian crossing dimensions. The inventory limits were approximately 500-feet from the intersection along the mainline. The measured clearance distances for each vehicular and pedestrian movement were utilized to calculate new yellow, all-red, and flashing don't walk clearance intervals. The completed form for each intersection is provided in **Appendix A**

2.2 DATA COLLECTION

Stantec utilized various traffic data sources to develop traffic volumes for the off-peak season and peak season conditions. These sources include historical data from a continuous count stations (CCS) for US 278 just west of this signal system, historical turning movement counts and directional tube counts within this system conducted by the Town of Hilton Head Island in June 2017, and peak hour turning movement counts and 24-hour directional tube counts collected by Stantec in February 2018. **Table 2** lists the 24-hour count locations while the turning movement counts were conducted during weekday AM, weekday midday, weekday PM, and weekend peak hours at the study corridor intersections shown previously in **Table 1**.

Table 2 – 24-Hour Directional Tube Count Locations

#	Location	Direction of Travel
A	US 278 (William Hilton Parkway) east of bridge over Skull Creek	Eastbound and Westbound
B	US 278 (William Hilton Parkway) east of Spanish Wells Road	Eastbound and Westbound
C	US 278 (William Hilton Parkway) east of Whooping Crane Way	Eastbound and Westbound

The 24-hour directional tube counts were graphed, as shown on the following page in **Figure 2,** to show the traffic volumes throughout a typical weekday. The existing and proposed TOD schedules are shown for reference on the Figures. The turning movement count (TMC) data was collected based upon the historical tube counts and after determining the peak periods in two-hour increments. The TMC's were collected and the peak hour was determined; this count data is included in **Appendix B**.

Using the available hourly historical count data taken from the CCS west of this system, off-season (school) and peak-season (summer) weekend hourly volumes were graphed, as shown on the following pages in **Figures 3 and 4**, to show the Saturday and Sunday traffic volumes and how they vary between the off-season and the peak-season. These traffic volumes, along with the existing and implemented time-of-day coordination plan periods, are shown below in **Figures 3** for Saturdays and **Figure 4** for Sundays.

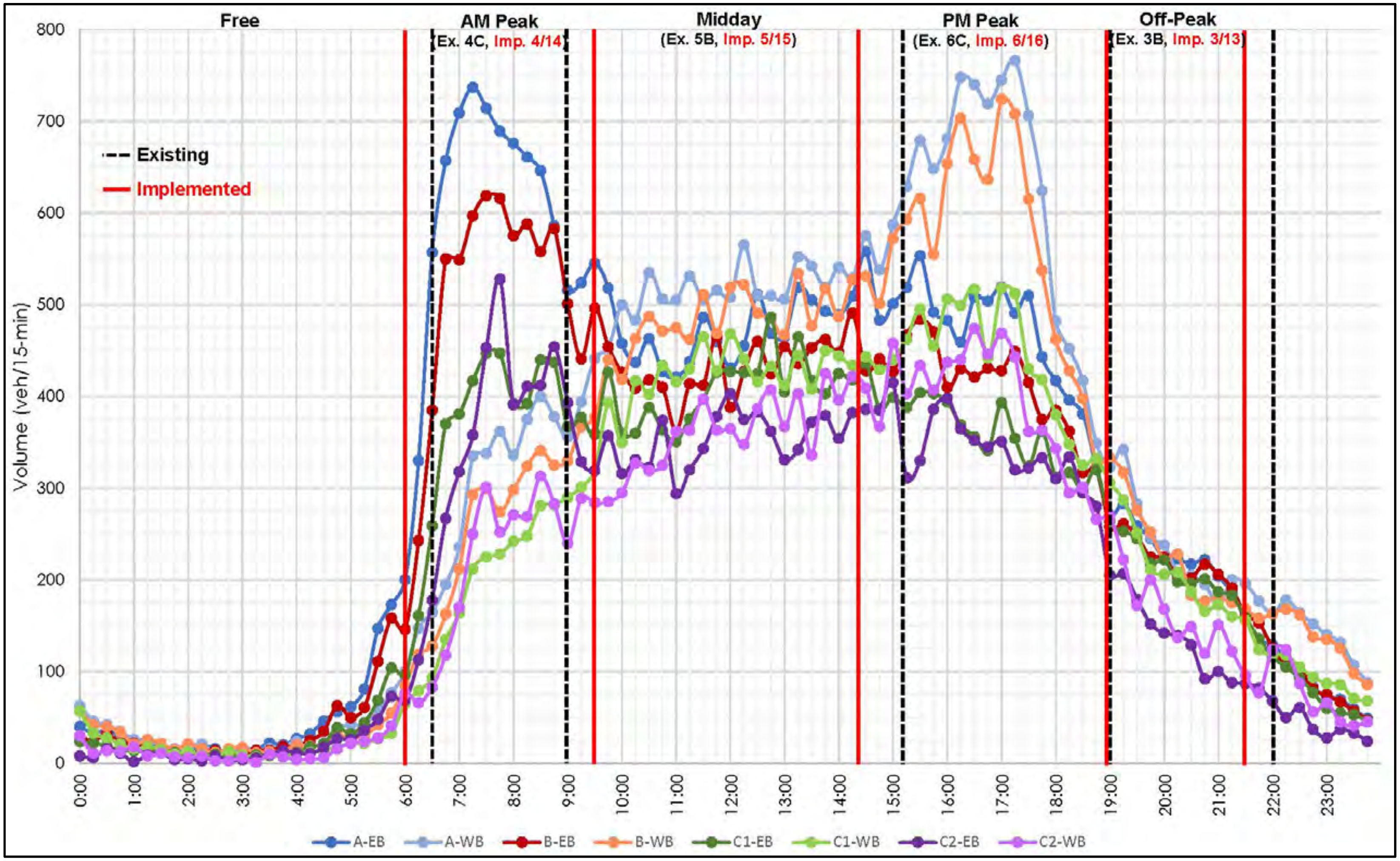


Figure 2 – Weekday Directional Traffic Volumes (June 2017 [A, B, C1] and February 2018 [C2])

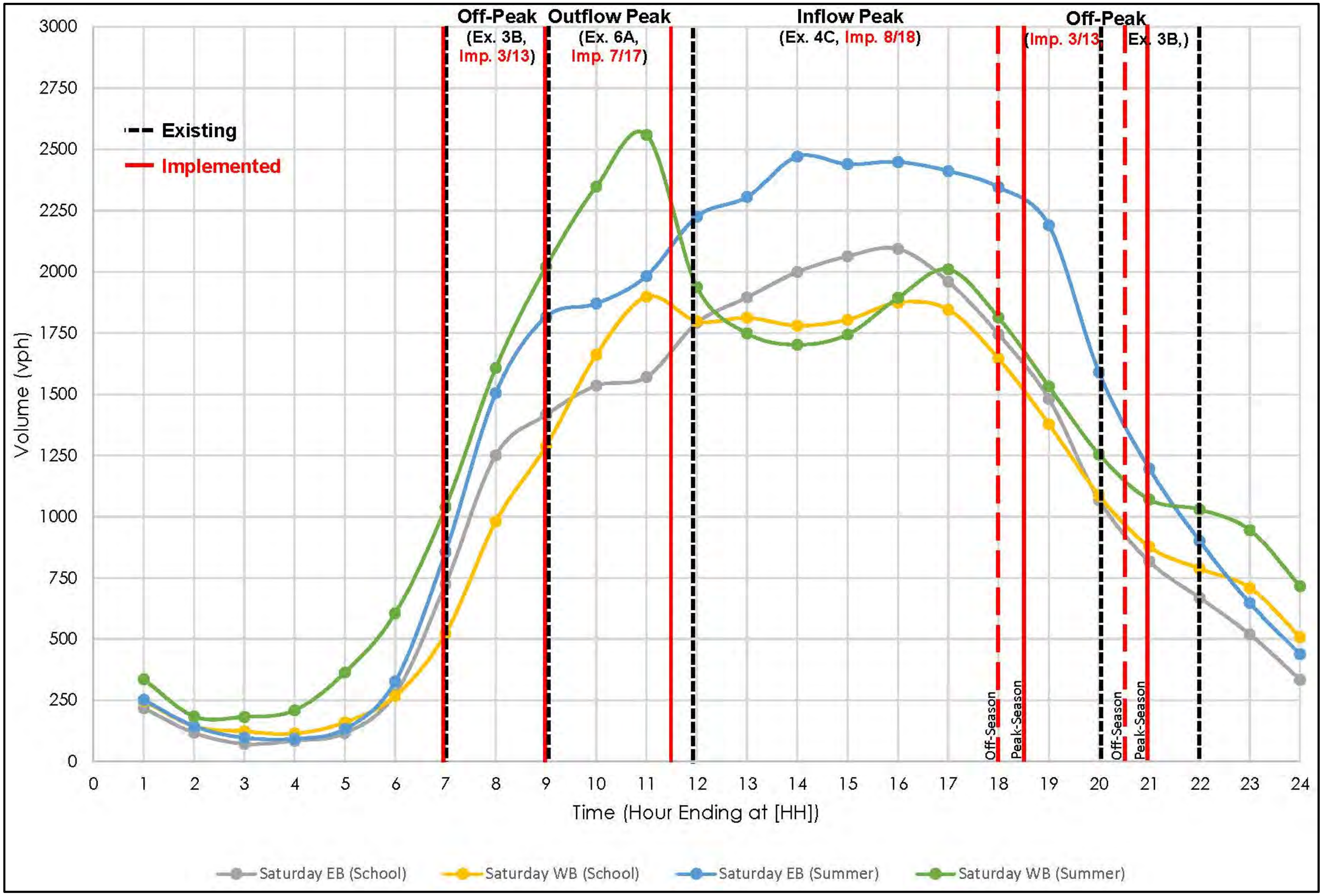


Figure 3 – Saturday Directional Traffic Volumes (Summer [May-August] and School [September-April])

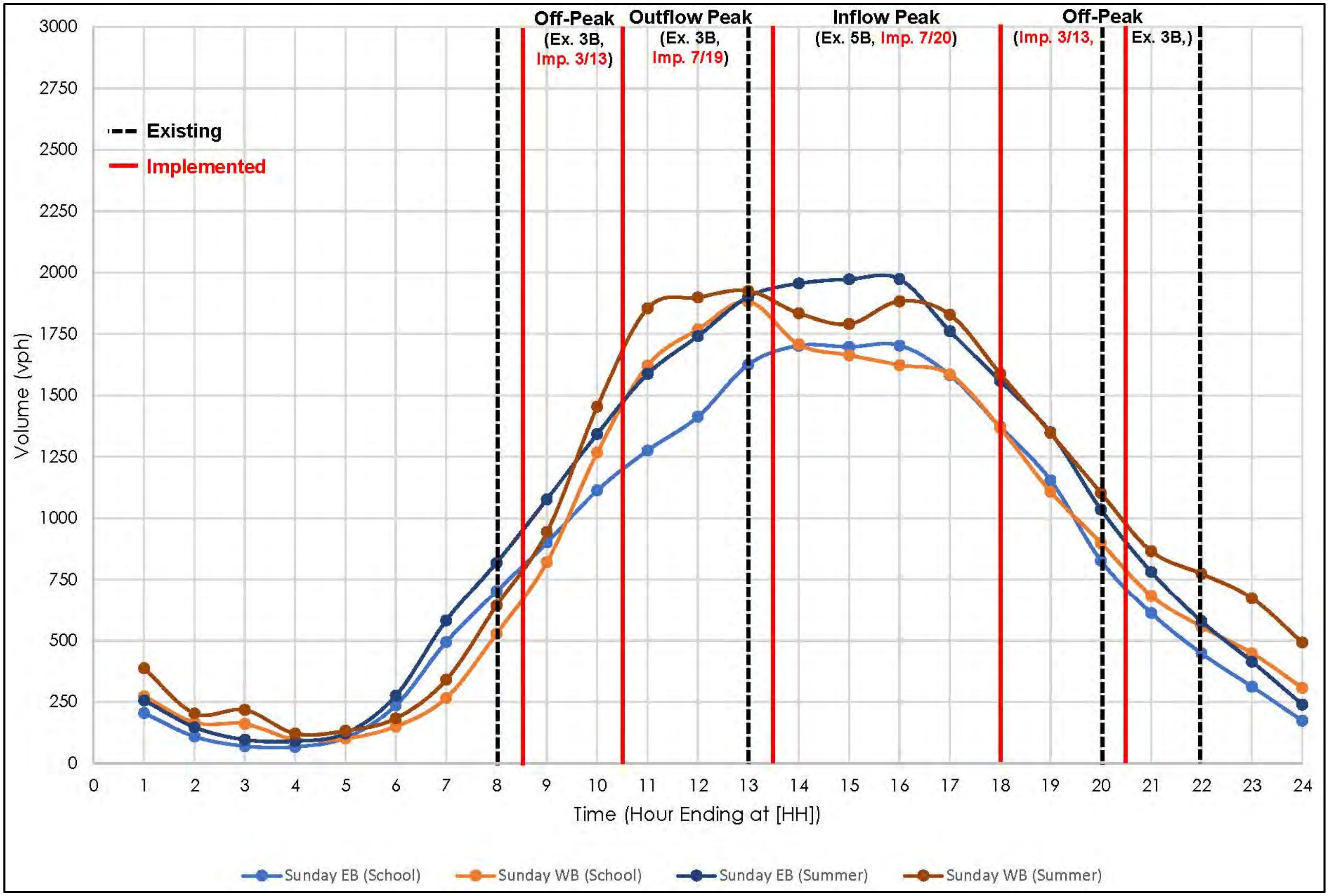


Figure 4 – Sunday Directional Traffic Volumes (Summer [May-August] and School [September-April])

2.3 TRAFFIC VOLUME DEVELOPMENT

To meet the project schedule of developing and implementing new coordinated signal timing plans before the 2018 peak season, Stantec analyzed the available traffic count data and applied factors based upon historical trends to develop projected peak season 2018 traffic volumes. Similarly, the data collected in June 2017 was adjusted to develop projected off-season 2018 traffic volumes.

Hourly traffic volume data recorded by SCDOT's continuous count station (CCS) just west of Hilton Head Island was reviewed for the 2014, 2015, 2016, and 2017 calendar years. The traffic volume data set for January 1, 2017 through December 31, 2017, as recorded by the CCS, was analyzed to develop directional peak hour and daily volume adjustment factors for the off-season, peak season, weekday, and weekend traffic projections.

Based upon the seasonal variation in daily volume observed at the CCS, data collected in February was increased by 14% to estimate June conditions. Variations in the intensity of directional peaks between the off-season and peak season were also measured from the 2017 CCS data and range between +2% and +35% for the weekend in-flow and out-flow peak hours. Based upon the historical growth in average annual daily traffic at the CCS entering and exiting Hilton Head Island from 2014 to 2017 (the most recent full year of data available), an annual growth factor of 1.9% was determined.

3.0 LOCAL TIMING PARAMETERS

Local controller timings were developed for each of the seven intersections in this project. **Table 3** details the methods used to develop the controller values that were used for each intersection. Clearance calculations for each intersection are shown in **Appendix A**.

Table 3 – Local Timing Parameters

<table>
<tr><th>Parameter</th><th colspan="2">Value</th></tr>
<tr><td colspan="3">PEDESTRIAN INTERVAL</td></tr>
<tr><td>Pedestrian Change Interval</td><td colspan="2">((Curb to Curb Distance) / (Walking Speed))</td></tr>
<tr><td>Walking Speed</td><td colspan="2">3.5 Feet per Second</td></tr>
<tr><td>Walk</td><td colspan="2">7 Seconds – Also calculated (Push button to far curb distance) / (walking speed of 3.0fps). If this number was greater than the calculated Pedestrian Change Interval then the difference was added to the Walk time.</td></tr>
<tr><td>Buffer Interval</td><td colspan="2">Following the pedestrian change interval, a buffer interval consisting of a steady UPRAISED HAND (symbolizing DON’T WALK) signal indication shall be displayed for at least 3 seconds prior to the release of any conflicting vehicular movement</td></tr>
<tr><td colspan="3">VEHICLE INTERVAL</td></tr>
<tr><td>Yellow Interval</td><td>t + (V/(2A + 64.4g))
Minimum of 3 seconds. Rounded up to the nearest tenth second.
Left turn clearance calculations based on 20-MPH</td><td rowspan="13">t = perception reaction time (1 second)

V = posted speed in feet/second (20 mph for left turn clearances)

A = deceleration rate (10 feet/second/second)

W = intersection width measured from stop bar to the far edge of the last conflict lane (or crosswalk when the crosswalk is greater than 20’ from the intersection)

L = length of vehicle (assume 20 feet)

g = The approximate approach grade

n = detection distance / 20

N = number of lanes</td></tr>
<tr><td>All Red Interval</td><td>(W + L) / V
Minimum of 2.0 seconds. Rounded up to the nearest tenth second</td></tr>
<tr><td>Minimum Green</td><td>Maintain existing</td></tr>
<tr><td>Volume Density</td><td>No Change</td></tr>
<tr><td>Minimum Cycle Length</td><td>90 seconds</td></tr>
<tr><td>Maximum Cycle Length</td><td>240 Seconds</td></tr>
<tr><td>Offset Reference</td><td>End of Green</td></tr>
<tr><td>Offset Seeking</td><td>Short/Long Way</td></tr>
<tr><td>Free Operation</td><td>Late night</td></tr>
<tr><td>Lead/Lag by TOD?</td><td>Yes</td></tr>
<tr><td>Traffic Responsive Operation</td><td>No</td></tr>
<tr><td>Special Events</td><td>No</td></tr>
<tr><td colspan="2"></td></tr>
<tr><td colspan="3">CONTACT INFORMATION</td></tr>
<tr><td>Traffic & Transportation Engineer</td><td colspan="2">Darrin Shoemaker, P.E., Town of Hilton Head Island</td></tr>
<tr><td>Law Enforcement</td><td colspan="2">Beaufort County Sheriff’s Office</td></tr>
</table>

4.0 COORDINATION PARAMETERS

The objective of the proposed signal timing is to minimize delay for all vehicles within the system and provide improved progression at the posted speed limit through the signal system for the mainline while minimizing side-street delay.

The turning movement count inventory data was entered into Synchro 10 using the following guidelines:

- All movements were coded as they appear in the field.
- Signing and marking restrictions were coded as they appear in the field.
- A saturated flow rate of 1,900 vehicles per hour was used.
- Posted speed limits were used for progression speeds.

Multiple runs of Synchro 10 were completed to determine the most appropriate combination of cycle length, splits, and offsets for each signal in the system. The existing coordinated signal timing plans and daily schedules are summarized below in **Table 4**. The implemented plans developed for both the off-season and peak-season conditions are summarized in **Table 5** on the following page. Detailed Synchro timing reports are included in **Appendix C** and time-space diagrams for the corridor are included in **Appendix D**.

Table 4 – Time-of-Day/Day-of-Week Schedule (Existing)

Start Time (HH:MM)	Monday-Friday		Saturday		Sunday	
	Day Plan 1		Day Plan 2		Day Plan 3	
	#	CL	#	CL	#	CL
00:00	99	Free	99	Free	99	Free
06:30	4C	180				
07:00			3B	130		
08:00					3B	130
09:00	5B	140	6A	170		
12:00			4C	180		
13:00					5B	140
15:15	6A	170				
19:00	3B	130				
20:00			3B	130	3B	130
22:00	99	Free	99	Free	99	Free

Table 5 – Time-of-Day/Day-of-Week Schedule (Implemented)

Start Time (HH:MM)	Off-Season (September 9th – March 14th)						Peak-Season (March 15th – September 8th)					
	Monday-Friday		Saturday		Sunday		Monday-Friday		Saturday		Sunday	
	Day Plan 1		Day Plan 2		Day Plan 3		Day Plan 4		Day Plan 5		Day Plan 6	
	#	CL	#	CL	#	CL	#	CL	#	CL	#	CL
00:00	**99**	**Free**	**99**	**Free**	**99**	**Free**	**99**	**Free**	**99**	**Free**	**99**	**Free**
06:00	**4**	**140**					**14**	**140**				
07:00			**3**	**100**					**13**	**100**		
08:30					**3**	**100**					**13**	**100**
09:00			**7**	**160**					**17**	**180**		
09:30	**5**	**120**					**15**	**120**				
10:30					**7**	**160**					**19**	**160**
11:30			**8**	**170**					**18**	**190**		
13:30					**7**	**160**					**20**	**190**
14:20	**6**	**130**					**16**	**150**				
18:00			**3**	**100**	**3**	**100**					**13**	**100**
18:30									**13**	**100**		
19:00	**3**	**100**					**13**	**100**				
20:30			**99**	**Free**	**99**	**Free**					**99**	**Free**
21:00									**99**	**Free**		
21:30	**99**	**Free**					**99**	**Free**				

Note: The implemented plans 3-8 are programmed for year-round operation on plan days 1, 2, and 3. The implemented plans 13-20 are programmed to supersede the off-season plans and operate during the peak season using plan days 4, 5, and 6.

4.1 PROPOSED TIMING PLANS

The existing corridor utilizes four coordinated timing plans with cycle lengths that range from 130-180 seconds for everyday operation with no change for the summer peak season. As with the existing plans, several of the implemented timing plans have some directional bias for either eastbound or westbound due to the flow of traffic to and from the mainland west of this system. The implemented plans operate with cycle lengths that range from 100 seconds for the off-peak plan to 190 seconds for the weekend inflow peak plan. The weekday AM implemented timing plans operate with a 140 second cycle length during both the off-season and peak-season. The weekday Midday implemented timing plans operate with a 120 second cycle length in both the off-season and peak-season. The weekday PM implemented timing plans operate with a 130 second cycle length in the off-season and 150 second cycle length in the peak-season. The Saturday outflow implemented plans operate with a 160 second cycle length during the off-season and with a 180 second cycle length during the peak-season. The Saturday inflow implemented plans operate with a 170 second cycle length in the off-season and with a 190 second cycle length during the peak-season. During the off-season, the Saturday inflow plan is recycled for the Sunday peak period. During the peak-season, the Sunday outflow implemented plan operates with a 160 second cycle length and the Sunday inflow implemented plan operates with a 190 second cycle length. A detailed comparison of the changes to level of service and delay by each intersection is provided for each of the existing and implemented plans is shown in **Tables 14, 15, 16, 17, 18, 19, 20, 21, 22,** and **23** in section 5.2 of this report. Level of service comparisons between existing and implemented conditions were not performed for the off-peak and Sunday peak plans as the traffic data used to develop and evaluate timing plans was collected during other periods and adjusted to represent typical Sunday and off-peak volumes.

The existing weekday AM peak plan has a 180 second cycle length and runs from 06:30 to 09:00. This plan was replaced with a 140 second cycle length plan for both the off-season and peak-season that runs from 06:00 to 09:30. As shown in **Tables 14 and 15**, the implemented weekday AM plan reduced the delay at five of the seven intersections during the off-season and at six of the seven intersections during the peak-season. The implemented weekday AM plan improved or maintained the level of service at five of the intersections during the off-season and six of the intersections during the peak-season.

The existing weekday Midday plan runs a 140 second cycle length and runs from 09:00 to 15:15. The implemented weekday Midday plan has a 120 second cycle length in both the off-season and peak-season and runs from 09:30 to 14:20. Initially, an off-season plan with a 100 second cycle length was selected but was replaced during fine-tuning with a 120 second cycle length plan as necessary to adequately serve all movements throughout the corridor. As shown in **Tables 16 and 17**, the implemented Midday plan reduced the delay at four of the seven intersections during the off-season and at five of the seven intersections during the peak-season. The implemented weekday Midday plan improved or maintained the level of service at all seven intersections during both the off-season and the peak-season.

The existing weekday PM peak plan runs from 15:15 to 19:00 and has a cycle length of 170 seconds. This plan has been replaced with a 130 second cycle length for the off-season and a 150 second cycle length for the peak-season that runs from 14:20 to 19:00. Initially, an off-season plan with a 110 second cycle length was selected but during fine-tuning was replaced with a 130 second cycle length plan as necessary to adequately serve all movements throughout the corridor. As shown in **Tables 18 and 19**, the implemented PM plan reduced the delay at five of the seven intersections during the off-season and at five of the seven intersections during the peak-season. The implemented weekday PM plan improved or maintained the level of service at all seven intersections during both the off-season and the peak-season.

The existing Saturday outflow peak plan recycles the weekday PM peak plan with a 170 second cycle length and runs from 09:00 to 12:00. The implemented plan runs from 09:00 to 11:30 with a cycle length of 160 seconds for the off-season and 180 seconds for the peak-season. As shown in **Table 20**, the implemented Saturday (off-season) outflow peak plan maintained or improved the level of service at all seven intersections and reduced or had minor change to delay at five of the seven intersections. As shown in **Table 21**, the implemented Saturday (peak-season) outflow peak plan maintained or improved the level of service at four of the seven intersections and reduced or had minor change to delay at three of the seven intersections.

The existing Saturday inflow peak plan has a 180 second cycle length and runs from 12:00 to 20:00. The implemented plan runs from 11:30 to 18:00 with a cycle length of 170 seconds for the off-season and from 11:30 to 18:30 with a cycle length of 190 seconds for the peak-season. As shown in **Table 22**, the implemented Saturday (off-season) inflow peak plan improved or maintained the level of service at six of the seven intersections and reduced delay at three intersections. As shown in **Table 23**, the implemented Saturday (peak-season) inflow peak plan improved or maintained the level of service at all seven intersections and reduced or nominally changed delay at four of the seven intersections.

The existing Sunday peak plan has a 140 second cycle length and runs from 13:00 to 20:00 during both the off-season and peak-season. The implemented Saturday peak outflow plan, with a 160 second cycle length, is recycled for Sundays in the off-season and runs from 10:30 to 18:00. For the peak-season, the implemented Sunday outflow peak plan runs from 10:30 to 13:30 with a cycle length of 160 seconds and the implemented Sunday inflow peak plan runs from 13:30 to 18:00 with a cycle length of 190 seconds. Level of service comparisons between existing and implemented conditions were not performed for the Sunday peak plans as the traffic data used to develop and evaluate timing plans was collected during other periods and adjusted to represent typical Sunday volumes. While level of service comparisons and before and after studies were not performed for the Sunday peak periods, the new timing plans were optimized and fine-tuned to the observed conditions to adequately serve all movements along the corridor.

The existing off-peak plan has a 130 second cycle length and runs from 19:00 to 22:00 on weekdays, from 07:00 to 09:00 and again from 20:00 to 22:00 on Saturdays, from 08:00 to 13:00 and again from 20:00 to 22:00 on Sundays. During the off-season, the implemented off-peak plan has a 100 second cycle length and runs from 19:00 to 21:30 on weekdays, from 07:00 to 09:00 and again from 18:00 to 20:30 on Saturdays, and from 08:30 to 10:30 and again from 18:00 to 20:30 on Sundays. During the peak-season, the implemented off-peak plan has a 100 second cycle length and runs from 19:00 to 21:30 on weekdays, from 07:00 to 09:00 and again from 18:30 to 21:00 on Saturdays, and from 08:30 to 10:30 and again from 18:00 to 20:30 on Sundays.

5.0 OPERATIONAL ANALYSIS

5.1 METHODOLOGY FOR BEFORE AND AFTER STUDIES

The travel time, average speed, and delay studies were conducted in accordance with the procedures given in the *Manual of Transportation Engineering Studies*, published by the Institute of Transportation Engineers. Travel time, average speed, and delay studies were conducted in both the eastbound and westbound directions on US 278 (William Hilton Parkway) during the off-season weekday AM peak, weekday midday peak, and weekday PM peak periods. A minimum of six runs was made in each direction. The "floating car" technique was used, whereby the driver passes as many cars as pass the driver. During times of congestion and unbalanced lane utilization, the average car method was used whereby the driver matched the "average" vehicle in the traffic flow to account for unbalanced lane utilization. The following route were determined along the US 278 (William Hilton Parkway) system:

- Route 1: US 278 (William Hilton Parkway), eastbound/westbound between Squire Pope Road/Chamberlin Drive and Beach City Road/Gardner Drive.

The study vehicle was unmarked and operated as inconspicuously as possible. The operator recorded the stops and travel time experienced during each run. The "before" runs were collected for US 278 (William Hilton Parkway) on Wednesday, March 28, 2018. The "after" runs were collected for US 278 (William Hilton Parkway) on Thursday, October 4, 2018. Travel run data was collected using a GPS receiver and was processed with Tru-Traffic version 10 software. **Tables 6, 7, 8 and 9** below and on the following pages, summarize the recorded "before" and "after" travel time, average speed, delay, and number of stops. Hydrocarbons (HC), and oxides of nitrogen (NO_X), which are vehicle emissions regulated by federal law, along with carbon dioxide (CO_2) emissions and fuel consumption were estimated by processing the travel time runs using the Tru-Traffic software. These values are summarized for the "before" and "after" runs on the following pages in **Tables 10, 11, 12, and 13**. The data below is shown as the average of multiple runs for each time period and direction of travel. The raw travel time reports are included in **Appendix E**.

Table 6 – Average Travel Time (sec)

US 278 (William Hilton Parkway)									
	AM			MD			PM		
Direction of Travel	**EB**	**WB**	**All Runs**	**EB**	**WB**	**All Runs**	**EB**	**WB**	**All Runs**
Before	293	272	282	281	292	286	375	512	447
After	293	265	280	279	268	273	312	448	383
% Difference	**0.0%**	**-2.6%**	**-0.7%**	**-0.7%**	**-8.2%**	**-4.5%**	**-16.8%**	**-12.5%**	**-14.3%**

Table 7 – Average Delay (sec)

US 278 (William Hilton Parkway)									
	AM			MD			PM		
Direction of Travel	**EB**	**WB**	**All Runs**	**EB**	**WB**	**All Runs**	**EB**	**WB**	**All Runs**
Before	81	59	70	68	79	74	163	299	235
After	81	53	68	66	56	61	99	235	171
% Difference	**0.0%**	**-10.2%**	**-2.9%**	**-2.9%**	**-29.1%**	**-17.6%**	**-39.3%**	**-21.4%**	**-27.2%**

Table 8 – Average Speed (mph)

US 278 (William Hilton Parkway)									
	AM			MD			PM		
Direction of Travel	**EB**	**WB**	**All Runs**	**EB**	**WB**	**All Runs**	**EB**	**WB**	**All Runs**
Before	33.8	35.7	34.8	34.6	33.2	33.9	25.9	23.3	24.5
After	34.0	36.7	35.3	34.8	36.4	35.6	31.0	24.6	27.7
% Difference	**+0.6%**	**+2.8%**	**+1.4%**	**+0.6%**	**+9.6%**	**+5.0%**	**+19.7%**	**+5.6%**	**+13.1%**

Table 9 – Average Number of Stops

US 278 (William Hilton Parkway)									
	AM			MD			PM		
Direction of Travel	**EB**	**WB**	**All Runs**	**EB**	**WB**	**All Runs**	**EB**	**WB**	**All Runs**
Before	1.4	1.7	1.6	1.9	1.7	1.8	2.8	4.7	3.8
After	1.6	1.5	1.5	1.5	1.2	1.3	2.7	3.7	3.2
% Difference	**+14.3%**	**-11.8%**	**-6.3%**	**-21.1%**	**-29.4%**	**-27.8%**	**-3.6%**	**-21.3%**	**-15.8%**

Table 10 – Average Oxides of Nitrogen Emissions (kg/hr)

US 278 (William Hilton Parkway)									
	AM			MD			PM		
Direction of Travel	**EB**	**WB**	**All Runs**	**EB**	**WB**	**All Runs**	**EB**	**WB**	**All Runs**
Before	8.07	4.17	12.25	6.78	6.87	13.65	6.11	8.66	14.77
After	7.97	3.93	11.90	6.66	6.58	13.23	6.29	9.14	15.43
% Difference	**-1.3%**	**-5.8%**	**-2.8%**	**-1.9%**	**-4.2%**	**-3.0%**	**+2.9%**	**+5.6%**	**+4.5%**

Table 11 – Average Hydrocarbon Emissions (kg/hr)

US 278 (William Hilton Parkway)									
	AM			MD			PM		
Direction of Travel	**EB**	**WB**	**All Runs**	**EB**	**WB**	**All Runs**	**EB**	**WB**	**All Runs**
Before	0.72	0.35	1.07	0.58	0.59	1.17	0.53	0.75	1.29
After	0.72	0.32	1.04	0.59	0.56	1.15	0.56	0.85	1.42
% Difference	**0.0%**	**-8.1%**	**-2.6%**	**+1.9%**	**-5.1%**	**-1.6%**	**+5.4%**	**+13.4%**	**+10.1%**

Table 12 – Average Carbon Dioxide Emissions (kg/hr)

US 278 (William Hilton Parkway)									
	AM			MD			PM		
Direction of Travel	**EB**	**WB**	**All Runs**	**EB**	**WB**	**All Runs**	**EB**	**WB**	**All Runs**
Before	1180	613	1793	997	1009	2007	928	1527	2455
After	1169	588	1757	988	968	1957	925	1442	2367
% Difference	**-1.0%**	**-4.1%**	**-2.0%**	**-0.9%**	**-4.0%**	**-2.5%**	**-0.3%**	**-5.6%**	**-3.6%**

Table 13 – Average Fuel Consumption (gal/hr)

US 278 (William Hilton Parkway)									
	AM			MD			PM		
Direction of Travel	**EB**	**WB**	**All Runs**	**EB**	**WB**	**All Runs**	**EB**	**WB**	**All Runs**
Before	142.12	73.53	215.66	120.13	121.48	241.61	110.94	183.49	294.43
After	140.81	70.53	211.35	119.19	116.64	235.83	111.74	177.00	288.74
% Difference	**-0.9%**	**-4.1%**	**-2.0%**	**-0.8%**	**-4.0%**	**-2.4%**	**+0.7%**	**-3.5%**	**-1.9%**

5.2 LOS AND DELAY ANALYSIS

Synchro 10 was also used to prepare an evaluation of intersection operations to determine the Level of Service (LOS) and average delay of the existing condition (existing geometry, existing signal timings, and existing traffic volumes) and the proposed condition (existing geometry, proposed signal timings, and existing traffic volumes). This capacity analysis methodology is based on the *2010 Highway Capacity Manual* (HCM), a standard guidance for capacity analysis, which defines LOS at signalized intersections in terms of average control delay per vehicle, which is composed of initial deceleration delay, queue move-up time, stopped delay, and final acceleration delay. LOS ranges from A to F, with LOS A indicating operations with very low control delay and LOS F describing operations with extremely high average control delay. In the comparison between existing, proposed, and final timings, the LOS and delay should improve for the overall corridor but may increase or decrease at individual intersections depending on what was running before.

Currently, the corridor has very directional peak hour traffic during the weekday AM and PM peak periods, particularly along the segment west of the Cross Island Parkway. The weekday midday traffic volumes are more evenly balanced. The primary goal for timing the US 278 (William Hilton Parkway) corridor was to minimize delay for all vehicles within the system and provide improved progression at the posted speed limit through the signal system for the mainline while minimizing side-street queuing and delay. The volume-to-capacity (V/C) ratios were also included in the analysis to measure capacity demand of each intersection since delay on side streets and protected-only left-turns can sometimes skew an intersection delay even if the respective queues are under capacity. Overall, corridor offsets looked to reduce queuing and delay.

The results of the existing and implemented conditions are shown on the following pages in **Tables 14, 15, 16, 17, 18, 19, 20, 21, 22, and 23**. Reports detailing the Synchro LOS and Delay outputs are included in **Appendix C**. Although some of LOS and delays results under the implemented plans yielded worse values than the existing models, the signal timings have been optimized and fine-tuned to accommodate improved progression along US 278 (William Hilton Parkway) and capacity efficiency throughout the system. Meanwhile, all LOS results remain at 'D' or better with one exception and with many intersections performing with even better results.

The LOS, delay, and V/C ratios varied in the field upon implementation and fine-tuning of the proposed timing plans. Overall, the US 278 (William Hilton Parkway) system has numerous volume and lane additions and subtractions at major intersections and at the ramps to/from the Cross Island Parkway that result in unbalanced lane utilization and varying speeds in advance of intersections when vehicles are preparing to turn or merge. Consequently, varying speeds, lane utilization, geometric constraints, volume and lane additions and subtractions between the study intersections contributed various effects on the overall progression and flow of the corridor. Adjustments to the splits and offsets were incorporated during fine tuning and also in the weeks following implementation upon observation of actual driver behavior.

Level of Service (LOS) and Intersection Delay Summary Legend

Level of Service (LOS)		Change in Intersection Delay
Improved by two letter grades		Delay decreased by more than 2.0 seconds.
Improved by one letter grade		Delay decreased by 1.0 second or more but less than 2.0 seconds.
No change to LOS		Delay changed by less than 1.0 second (+/-)
Degraded by one letter grade		Delay increased by 1.0 second or more but less than 2.0 seconds.
Degraded by two letter grades		Delay increased by 2.0 seconds or more.

Table 14 – Existing and Implemented Intersection Level of Service and Delay (#4 Weekday AM Peak Period – Off Season)

#	ID #	Intersection	Existing		Implemented	
			LOS	Control Delay (sec/veh)	LOS	Control Delay (sec/veh)
1	6100	US 278 & S-141 (Squire Pope Road) & Chamberlin Drive	A	7.5	A	5.9
2	6101	US 278 & Wild Horse Road & S-79 (Spanish Wells Road)	C	28.3	C	21.6
3	6102	US 278 Business & S-482 (Gum Tree Road) & Ramps to/from Cross Island Parkway	D	46.8	D	40.1
4	6103	US 278 Business & S-626 (Wilborn Road) & Jarvis Park Road	C	28.4	C	24.5
5	6104	US 278 Business & Museum Street & Pembroke Drive	A	9.5	B	17.1
6	6105	US 278 Business & Whooping Crane Way & Indigo Run Drive	B	17.7	C	22.6
7	6106	US 278 Business & Beach City Road & Gardner Drive	B	18.6	B	15.9

Table 15 – Existing and Implemented Intersection Level of Service and Delay (#14 Weekday AM Peak Period – Peak Season)

#	ID #	Intersection	Existing		Implemented	
			LOS	Control Delay (sec/veh)	LOS	Control Delay (sec/veh)
1	6100	US 278 & S-141 (Squire Pope Road) & Chamberlin Drive	A	9.4	A	6.9
2	6101	US 278 & Wild Horse Road & S-79 (Spanish Wells Road)	C	21.7	B	14.0
3	6102	US 278 Business & S-482 (Gum Tree Road) & Ramps to/from Cross Island Parkway	D	35.9	C	27.8
4	6103	US 278 Business & S-626 (Wilborn Road) & Jarvis Park Road	B	10.1	A	6.0
5	6104	US 278 Business & Museum Street & Pembroke Drive	A	9.6	B	11.9
6	6105	US 278 Business & Whooping Crane Way & Indigo Run Drive	B	18.5	B	17.6
7	6106	US 278 Business & Beach City Road & Gardner Drive	B	19.0	B	15.1

Table 16 – Existing and Implemented Intersection Level of Service and Delay (#5 Weekday Midday Peak Period – Off Season)

#	ID #	Intersection	Existing		Implemented	
			LOS	Control Delay (sec/veh)	LOS	Control Delay (sec/veh)
1	6100	US 278 & S-141 (Squire Pope Road) & Chamberlin Drive	A	8.8	A	8.7
2	6101	US 278 & Wild Horse Road & S-79 (Spanish Wells Road)	B	14.8	B	17.6
3	6102	US 278 Business & S-482 (Gum Tree Road) & Ramps to/from Cross Island Parkway	C	31.0	C	22.6
4	6103	US 278 Business & S-626 (Wilborn Road) & Jarvis Park Road	B	10.4	B	11.4
5	6104	US 278 Business & Museum Street & Pembroke Drive	B	17.0	B	12.5
6	6105	US 278 Business & Whooping Crane Way & Indigo Run Drive	B	19.6	B	15.8
7	6106	US 278 Business & Beach City Road & Gardner Drive	B	16.0	B	18.4

Table 17 – Existing and Implemented Intersection Level of Service and Delay (#15 Weekday Midday Peak Period – Peak Season)

#	ID #	Intersection	Existing		Implemented	
			LOS	Control Delay (sec/veh)	LOS	Control Delay (sec/veh)
1	6100	US 278 & S-141 (Squire Pope Road) & Chamberlin Drive	A	9.4	A	9.8
2	6101	US 278 & Wild Horse Road & S-79 (Spanish Wells Road)	B	16.6	B	18.9
3	6102	US 278 Business & S-482 (Gum Tree Road) & Ramps to/from Cross Island Parkway	D	36.5	C	34.1
4	6103	US 278 Business & S-626 (Wilborn Road) & Jarvis Park Road	B	16.5	B	11.2
5	6104	US 278 Business & Museum Street & Pembroke Drive	B	18.6	B	17.4
6	6105	US 278 Business & Whooping Crane Way & Indigo Run Drive	C	26.0	C	20.1
7	6106	US 278 Business & Beach City Road & Gardner Drive	B	19.8	B	14.5

Table 18 – Existing and Implemented Intersection Level of Service and Delay (#6 Weekday PM Peak Period – Off Season)

#	ID #	Intersection	Existing		Implemented	
			LOS	Control Delay (sec/veh)	LOS	Control Delay (sec/veh)
1	6100	US 278 & S-141 (Squire Pope Road) & Chamberlin Drive	C	23.3	C	29.3
2	6101	US 278 & Wild Horse Road & S-79 (Spanish Wells Road)	B	18.3	B	16.4
3	6102	US 278 Business & S-482 (Gum Tree Road) & Ramps to/from Cross Island Parkway	D	39.5	C	31.0
4	6103	US 278 Business & S-626 (Wilborn Road) & Jarvis Park Road	C	26.2	C	28.0
5	6104	US 278 Business & Museum Street & Pembroke Drive	B	16.6	B	14.7
6	6105	US 278 Business & Whooping Crane Way & Indigo Run Drive	C	26.5	C	23.7
7	6106	US 278 Business & Beach City Road & Gardner Drive	C	23.5	C	21.7

Table 19 – Existing and Implemented Intersection Level of Service and Delay (#16 Weekday PM Peak Period – Peak Season)

#	ID #	Intersection	Existing		Implemented	
			LOS	Control Delay (sec/veh)	LOS	Control Delay (sec/veh)
1	6100	US 278 & S-141 (Squire Pope Road) & Chamberlin Drive	E	66.7	E	63.3
2	6101	US 278 & Wild Horse Road & S-79 (Spanish Wells Road)	C	22.3	C	25.0
3	6102	US 278 Business & S-482 (Gum Tree Road) & Ramps to/from Cross Island Parkway	C	34.8	C	24.2
4	6103	US 278 Business & S-626 (Wilborn Road) & Jarvis Park Road	B	13.1	B	12.0
5	6104	US 278 Business & Museum Street & Pembroke Drive	B	18.1	B	15.6
6	6105	US 278 Business & Whooping Crane Way & Indigo Run Drive	C	29.3	C	30.7
7	6106	US 278 Business & Beach City Road & Gardner Drive	C	27.6	C	25.4

Table 20 – Existing and Implemented Intersection Level of Service and Delay (#7 Saturday Outflow Peak Period – Off Season)

#	ID #	Intersection	Existing		Implemented	
			LOS	Control Delay (sec/veh)	LOS	Control Delay (sec/veh)
1	6100	US 278 & S-141 (Squire Pope Road) & Chamberlin Drive	A	8.4	A	9.4
2	6101	US 278 & Wild Horse Road & S-79 (Spanish Wells Road)	B	16.0	B	15.7
3	6102	US 278 Business & S-482 (Gum Tree Road) & Ramps to/from Cross Island Parkway	C	29.7	C	22.2
4	6103	US 278 Business & S-626 (Wilborn Road) & Jarvis Park Road	A	4.0	A	4.0
5	6104	US 278 Business & Museum Street & Pembroke Drive	B	15.7	B	14.4
6	6105	US 278 Business & Whooping Crane Way & Indigo Run Drive	B	10.3	B	11.0
7	6106	US 278 Business & Beach City Road & Gardner Drive	B	12.7	B	13.8

Table 21 – Existing and Implemented Intersection Level of Service and Delay (#17 Saturday Outflow Peak Period – Peak Season)

#	ID #	Intersection	Existing		Implemented	
			LOS	Control Delay (sec/veh)	LOS	Control Delay (sec/veh)
1	6100	US 278 & S-141 (Squire Pope Road) & Chamberlin Drive	C	27.2	D	45.6
2	6101	US 278 & Wild Horse Road & S-79 (Spanish Wells Road)	B	15.5	B	16.2
3	6102	US 278 Business & S-482 (Gum Tree Road) & Ramps to/from Cross Island Parkway	D	35.8	C	31.7
4	6103	US 278 Business & S-626 (Wilborn Road) & Jarvis Park Road	A	9.1	A	6.9
5	6104	US 278 Business & Museum Street & Pembroke Drive	B	18.1	C	26.6
6	6105	US 278 Business & Whooping Crane Way & Indigo Run Drive	B	19.6	C	21.2
7	6106	US 278 Business & Beach City Road & Gardner Drive	C	23.8	C	27.9

Table 22 – Existing and Implemented Intersection Level of Service and Delay (#8 Saturday Inflow Peak Period – Off Season)

#	ID #	Intersection	Existing		Implemented	
			LOS	Control Delay (sec/veh)	LOS	Control Delay (sec/veh)
1	6100	US 278 & S-141 (Squire Pope Road) & Chamberlin Drive	B	13.1	B	10.4
2	6101	US 278 & Wild Horse Road & S-79 (Spanish Wells Road)	C	21.8	C	20.1
3	6102	US 278 Business & S-482 (Gum Tree Road) & Ramps to/from Cross Island Parkway	C	26.4	C	32.0
4	6103	US 278 Business & S-626 (Wilborn Road) & Jarvis Park Road	A	7.7	A	4.8
5	6104	US 278 Business & Museum Street & Pembroke Drive	B	13.1	B	16.7
6	6105	US 278 Business & Whooping Crane Way & Indigo Run Drive	A	8.4	A	9.6
7	6106	US 278 Business & Beach City Road & Gardner Drive	A	9.9	B	14.0

Table 23 – Existing and Implemented Intersection Level of Service and Delay (#18 Saturday Inflow Peak Period – Peak Season)

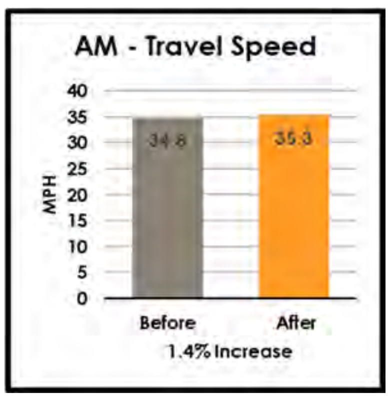
#	ID #	Intersection	Existing		Implemented	
			LOS	Control Delay (sec/veh)	LOS	Control Delay (sec/veh)
1	6100	US 278 & S-141 (Squire Pope Road) & Chamberlin Drive	C	20.2	B	14.3
2	6101	US 278 & Wild Horse Road & S-79 (Spanish Wells Road)	C	24.4	C	25.3
3	6102	US 278 Business & S-482 (Gum Tree Road) & Ramps to/from Cross Island Parkway	D	36.9	C	34.2
4	6103	US 278 Business & S-626 (Wilborn Road) & Jarvis Park Road	B	10.1	A	8.6
5	6104	US 278 Business & Museum Street & Pembroke Drive	C	20.4	C	23.7
6	6105	US 278 Business & Whooping Crane Way & Indigo Run Drive	B	12.6	B	16.2
7	6106	US 278 Business & Beach City Road & Gardner Drive	B	13.6	B	18.6

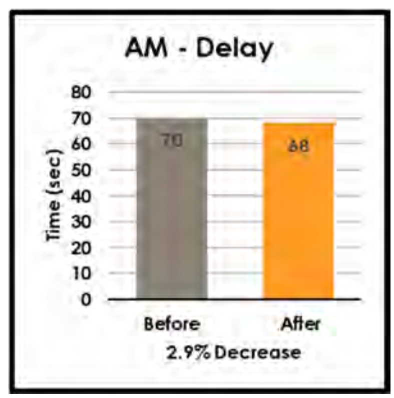
6.0 RESULTS SUMMARY

Due to the abbreviated project schedule for retiming US 278 (William Hilton Parkway) in advance of the 2018 peak-season, before and after studies were completed for the Off-Season weekday peak timing plans only. The before and after studies were conducted on weeks that historically have experienced similar daily traffic volumes in order to minimize the effect of any seasonal variation along this system. The results of the before and after study are shown in detail in the following sections and summarize the average changes to travel time, speed, and delay for each peak period.

6.1 WEEKDAY AM PEAK PLAN

The existing weekday AM peak plan has a 180 second cycle length and runs from 06:30 to 09:00. This plan was replaced with a 140 second cycle length plan for both the off-season and peak-season that runs from 06:00 to 09:30. As seen in **Figure 2**, the eastbound direction of travel is the predominate flow of traffic throughout the corridor during the weekday AM peak period, with approximately two-thirds of vehicles arriving on Hilton Head Island. As shown in the charts below, the implemented AM plan improved the combined eastbound and westbound averages of travel time, travel speed, and delay along the corridor. The changes by direction of travel are detailed in section 5.1 of this report.

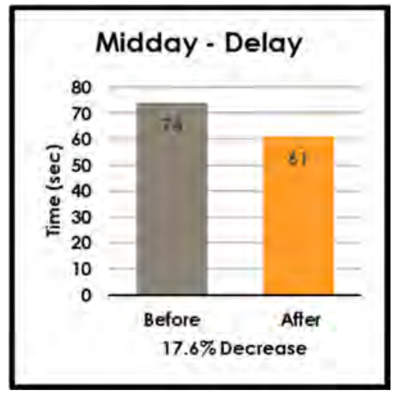




Travel time was reduced by almost one percent, speed was increased by just over one percent, and delay was reduced by nearly three percent. While the improvements to travel time, speed, and delay were modest, the new timing plans resulted in decreased overall intersection delay at five of the seven intersections during the off-season and at six of the seven intersections during the peak-season, as seen previously in **Table 14** and **Table 15**.

6.2 WEEKDAY MIDDAY PEAK PLAN

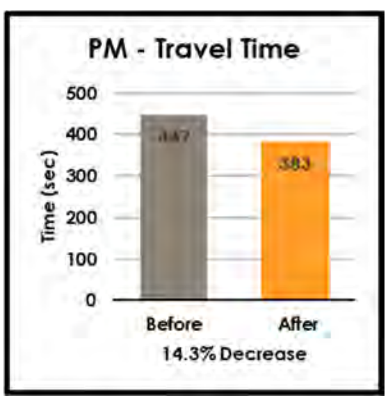
The existing weekday Midday plan runs a 140 second cycle length and runs from 09:00 to 15:15. The implemented weekday Midday plan has a 120 second cycle length for both the off-season and the peak-season and runs from 09:30 to 14:20. As seen in **Figure 2**, the midday traffic volumes are fairly well balanced between the eastbound and westbound directions of travel during this period. As shown in the charts below, the implemented weekday Midday peak plan improved the combined eastbound and westbound averages of travel time, travel speed, and delay along the corridor.

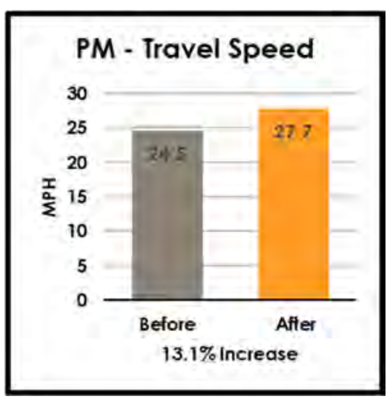


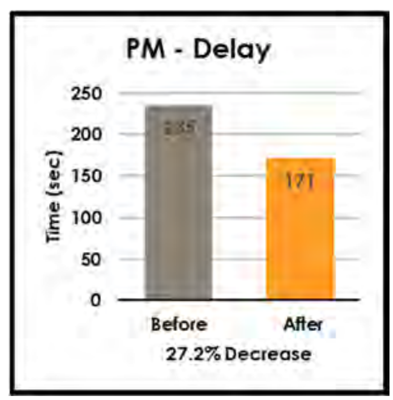
Travel time was reduced by four and a half percent, speed was increased by five percent, and delay was reduced by nearly eighteen percent. In addition to the improvements to travel time, speed, and delay, the new timing plans resulted in decreased overall intersection delay at four of the seven intersections during the off-season and at five of the seven intersections during the peak-season, as shown previously in **Table 16** and **Table 17**. The implemented timing plans maintained or improved the level of service at all intersections during the weekday Midday peak period.

6.3 WEEKDAY PM PEAK PLAN

The existing weekday PM peak plan runs from 15:15 to 19:00 and has a cycle length of 170 seconds. This plan has been replaced with a 130 second cycle length during the off-season and a 150 second cycle length during the peak-season that runs from 14:20 to 19:00. As seen in **Figure 2**, the predominate flow of traffic throughout the corridor during the weekday PM peak period is westbound, with nearly 60 percent of traffic traveling toward the mainland. As shown in the charts below, the implemented PM plan improved the combined eastbound and westbound averages of travel time, travel speed, and delay along the corridor.







Travel time was reduced by fourteen percent, speed was increased by thirteen percent, and delay was reduced by more than twenty seven percent. In addition to the improvements to travel time, speed, and delay, the new timing plans resulted in decreased overall intersection delay at five of the seven intersections during the off-season and at five of the seven intersections during the peak-season, as shown previously in **Table 18** and **Table 19**. The implemented timing plans maintained or improved the level of service at all intersections during the weekday PM peak period.

6.4 SATURDAY OUTFLOW PEAK PLAN

The existing Saturday outflow peak plan reuses the weekday PM peak plan with a 170 second cycle length and runs from 09:00 to 12:00. The implemented plan runs from 09:00 to 11:30 with a cycle length of 160 seconds during the off-season and 180 seconds during the peak-season. As shown in **Table 20**, the implemented Saturday (off-season) outflow peak plan maintained or improved the level of service at all seven intersections and reduced or had minor change to delay at five of the seven intersections. As shown in **Table 21**, the implemented Saturday (peak-season) outflow peak plan maintained or improved the level of service at four of the seven intersections and reduced or had minor change to delay at three of the seven intersections. Saturday outflow peak operations were observed and the splits and offsets were fine-tuned during implementation to ensure that queueing and delay were within acceptable ranges. Before and after studies were not performed for this period, therefore, the changes to travel time, speed, delay, number of stops, and emissions were not quantified.

6.5 SATURDAY INFLOW PEAK PLAN

The existing Saturday inflow peak plan reuses the weekday AM peak plan with a 180 second cycle length and runs from 12:00 to 20:00. The implemented plan runs from 11:30 to 18:30 with a cycle length of 170 seconds during the off-season and 190 seconds during the peak-season. As shown in **Table 22**, the implemented Saturday (off-season) inflow peak plan improved or maintained the level of service at six of the seven intersections and reduced delay at three intersections. As shown in **Table 23**, the implemented Saturday (peak-season) inflow peak plan improved or maintained the level of service at all seven intersections and reduced or nominally changed delay at four of the seven intersections. Saturday inflow peak operations were observed and the splits and offsets were fine-tuned during implementation to ensure that queueing and delay were within acceptable ranges. Before and after studies were not performed for this period, therefore, the changes to travel time, speed, delay, number of stops, and emissions were not quantified.

6.6 OFF-PEAK PLAN AND SUNDAY PEAK PLANS

Off-Peak periods throughout the week and Sunday peak operations were observed to ensure that queueing and delay were within acceptable ranges. During implementation, the splits and offsets for these plans were optimized and fine-tuned to the observed conditions to adequately serve all movements along the corridor while maintaining steady progression. Before and after studies were not performed for these periods, therefore, the changes to travel time, speed, delay, number of stops, and emissions were not quantified.

7.0 EFFECTIVENESS EVALUATION

Improvements in traffic signal timing can also be measured using a cost versus benefit ratio. If the financial benefits to the drivers outweigh the financial cost of the project over its lifespan, then the project is worth the investment. The financial benefit to the drivers is seen through decreased driving time and fuel consumption due to improved traffic flow from the signal timing plans.

The signal timing plans will last until changes in volume or roadway characteristics decrease the efficiency of the signal system to move traffic. Development in the area can increase the volume and cause the need for roadway expansion. In order to determine the cost/benefit ratio for this report, the life span of the new signal timing plans was assumed to be 2 years.

7.1 ANNUAL COSTS

The cost of designing, implementing, and recording the timing plans and the interest associated with the capital invested are all factors involved in calculating the equivalent annual cost.

The formulas used to determine the project's costs are:

$E = R \times C$

Where:

E = Equivalent Cost

R = Capital Recovery Cost

C = Initial Cost

$R = i(1+i)^n / ((1+i)^n - 1)$

Where:

R = Capital Recovery Cost

i = Annual Interest Rate

n = Useful Life of Timing Plans

The equivalent annual costs, as calculated, using the above formulas, for US 278 (William Hilton Parkway) are shown in **Table 24**. The table shows interest rates ranging from 4% to 8%, which are assumed to be reasonable rates for the current market. As stated previously, the useful life of the timing plans was assumed to be 2 years. Based on contracted fees for traffic data collection, development of timing plans, implementing and field tuning of timing plans, the total cost was $38,378.00.

Table 24 - Equivalent Annual Cost of Timing Plans

Annual Interest Rate	Capital Recovery Factor	Equivalent Annual Cost
4%	0.5302	$20,348
5%	0.5378	$20,640
6%	0.5454	$20,933
7%	0.5531	$21,227
8%	0.5608	$21,521

* $38,378.00 Initial Cost and 2-year Service Life

7.2 BENEFITS

Many benefits can be derived from the improved signal timing, including vehicular emissions, reduced vehicular crashes, time savings, and fuel savings. Unfortunately, it is hard to put a dollar value on the public health benefits received by decreased vehicular emissions. Also, this study did not include a crash analysis; therefore, a dollar value for potential decreased vehicular crashes due to improved traffic flow was not included. However, it is possible to assign a dollar value to the time motorists save due to decreased travel time and the decreased fuel usage. The time saved can be measured by a dollar value using the following formula.

S = R x V x D x O x C

Where:

S = Dollars Saved

R = Travel Time Reduction

V = Volume

D = Days Timing in Effect

O = Average Vehicle Occupancy

C = Cost of Delay per Person Hour

The days the timings are in effect is assumed to be 250 days. The average vehicle occupancy is assumed to be 1.2, and the cost of delay per person is assumed to be $12.00 per person-hour.

The values for fuel consumption were obtained from travel run data collected using a GPS receiver and the Tru-Traffic software for the existing timing plans and the final timing plans. The cost of fuel is assumed to be $2.59 per gallon. **Table 25** shows the annual dollar value of the US 278 (William Hilton Parkway) signal timing improvements for the three analyzed peak periods.

Other benefits not considered in this analysis include lower driver frustration levels and a potential reduction of accidents. All of the improvements mentioned in the report are for three hours a day for each weekday during the AM, MD, and PM peak hours along US 278 (William Hilton Parkway).

Table 25 - Annual Travel Time and Fuel Consumption Cost Savings

Time Period	Volume (veh/hr)	Annual Improvement				
		Travel Time (Veh-Hrs)	Value	Fuel Consumption (gallons)	Value	Total
US 278 (William Hilton Parkway)						
AM - EB	1,806	0	$0	327	$848	$848
AM - WB	936	455	$6,552	751	$1,945	$8,497
MIDDAY - EB	1,504	209	$3,008	236	$612	$3,620
MIDDAY - WB	1,509	2,515	$36,216	1,209	$3,132	$39,348
PM - EB	1,341	5,867	$84,483	(200)	($518)	$83,965
PM - WB	1,932	8,587	$123,648	1,622	$4,202	$127,850
Total		**17,632**	**$253,907**	**3,946**	**$10,221**	**$264,128**

Notes: Values shown in red and in parentheses represent negative savings.
Traffic volumes shown are the average of the through movement from each intersection in the signal system during the off-season.

7.3 COST/BENEFIT ANALYSIS

The benefit to cost ratio is a measure of effectiveness for the new signal timing plans. It validates the time and money spent to improve the timing along the corridor. The ratio for the US 278 (William Hilton Parkway) corridor was obtained by dividing the value of the annual benefits (reduced travel time and fuel consumption) by the equivalent annual cost. A benefit to cost ratio greater than one indicates the project's benefits outweigh the costs.

The total value of the benefits received by the motorists on US 278 (William Hilton Parkway) is $264,128. The equivalent annual cost of designing, implementing, and documenting the improved signal timing plans ranges from $20,348 at 4% interest to $21,521 at 8% interest. **Table 26** shows the benefit to cost ratios for the interest rates ranging from 4% to 8%.

Table 26 - Cost/Benefit Analysis

Costs		Benefits			Benefit/ Cost Ratio
Interest Rate	Equivalent Annual Cost	Reduced Delay	Reduced Fuel Consumption	Total	
4%	$20,348	$253,907	$10,221	$264,128	13.0
5%	$20,640	$253,907	$10,221	$264,128	12.8
6%	$20,933	$253,907	$10,221	$264,128	12.6
7%	$21,227	$253,907	$10,221	$264,128	12.4
8%	$21,521	$253,907	$10,221	$264,128	12.3

As evident in **Table 26**, the benefit to cost ratio ranges from 12.3:1 to 13.0:1. The benefits calculated are only for the weekday AM, weekday Midday, and weekday PM peak hours.

8.0 CONCLUSIONS

New coordinated traffic signal timings were developed and implemented for seven signals along US 278 (William Hilton Parkway) in the Town of Hilton Head Island, Beaufort County, South Carolina. The timing plans developed for this project include the weekday AM peak period, weekday Midday period, weekday PM peak period, Saturday outflow peak period, Saturday inflow peak period, Sunday outflow peak period, Sunday inflow peak period, and an Off-Peak plan. Two sets of timing plans were developed for each of these peak periods—an Off-Season set which runs from approximately mid-September through mid-March, and a Peak-Season set which runs from mid-March through mid-September to better serve the increased traffic demands during the summer months

To determine the effectiveness of the implemented new signal timing plans, travel time studies were performed using a GPS receiver. This data and processed with Tru-Traffic software to evaluate and document the results of the timing plan development process. This report presents the results of the “before” and “after” studies that were conducted for the seven signals along the US 278 (William Hilton Parkway) corridor included in this project. Due to the abbreviated schedule of this project, before and after studies were conducted for the Off-Season only, with travel time runs recorded on Spring and Fall weeks selected for their comparable daily volume as based upon historical trends.

The new signal timing plans implemented for the weekday AM peak, weekday Midday peak, and weekday PM peak show improvements along US 278 (William Hilton Parkway). The new timing plans have decreased travel time and delay and increased the speeds through the corridor. With respect to these time periods, the improvements in traffic flow are expected to result in reduced fuel consumption and decreased emissions of carbon monoxide and hydrocarbons.

Delay incurs direct costs upon motorists in the form of increased fuel consumption and the value of their time wasted while waiting in traffic. Motorists using US 278 (William Hilton Parkway) during the weekday AM, Midday, and PM peak periods are expected to save 17,632 hours each year because of the improved traffic flow due to the new timing plans.

Conservatively assuming a vehicle occupancy of 1.2 persons/vehicle, $12.00 per hour for the value of motorists’ time, and $2.59 per gallon for gasoline, annual savings to motorists along US 278 (William Hilton Parkway are expected to be $253,907 in the form of reduced delay and $10,221 decrease in cost due to decreased fuel consumption, for a total annual savings of $264,128. The estimates for hours and fuel saved per year assume that similar improvements in travel time and fuel consumption as seen in the Off-Season would be experienced during the Peak-Season weekday peak periods, as well.

Other benefits not considered in this analysis include lower driver frustration levels and a potential reduction of collisions. All of the results mentioned in the report are for three hours a day for each weekday during the AM, Midday, and PM peak periods. New signal timing plans were also implemented during the weekend peak hours. However, because benefit/cost “before” and “after” studies were not conducted during these time periods, additional savings could not be quantified for these periods.

The Benefit to Cost ratio is between 12.3:1 and 13.0:1 for the US 278 (William Hilton Parkway) corridor with consideration of three weekday peak hours.