University of Central Florida

Traffic Analysis Report

N Alafaya Trail

From Waterford Lakes Town Center
To State Road 408 (West)

Orange County

Prepared For:

TTE 6256 - Traffic Operations

October 2020

Revised and Final
Traffic Analysis Report

N Alafaya Trail
From Waterford Lakes Town Center to State Road 408 (West)
Orange County

Prepared For:

University of Central Florida
TTE 6256 - Traffic Operations

Prepared By:

Team 4

Jorge Ugan, Md Rakibul Alam, Johnathan Nagoda

University of Central Florida
4000 Central Florida Blvd
Orlando, FL 32816

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

Executive Summary *(Johnathan, Rakibul, Jorge)*

The purpose of the signalized intersection study was to identify bottlenecks and issues with traffic congestions by experiencing a field traffic event. The data was collected in between two intersections: Alafaya Trail (SR434) and Waterford Lakes Town Center (intersection 1), and Alafaya Trail (SR434) and On-Ramp to SR408 Westbound (intersection 2). The location was a traffic facility outside of UCF due to the COVID-19 pandemic not only enforcing restrictions to UCF's main campus but also providing uncommon traffic patterns. The analysis was performed utilizing HCS (Highway Capacity Software) 7.9 for the results from our data collection. Everyone that had participated in the field observation were prepared for safety and gathered the data without any interruptions. Throughout the event, the bottlenecks occurred on the turning lanes moving inbound and outbound from the Waterford Lake Town Center. The westbound left turn at intersection 2 had the most critical condition with LOS F. In order to resolve the bottleneck issue, we suggest three alternatives: adjusting the left signal timings, adding another left lane, and the combination of additional left lane and increasing left turn phase split time. Among these, the first alternative is both economic and efficient.

1. Introduction *(Johnathan, Rakibul, Jorge)*

1.1 Background

The study of traffic volume, signal timing and control delay for roads and intersections are crucial part to understand the efficiency of the roads and intersections. Additionally, left turn at an intersection is risky maneuver. Therefore, transportation engineers and planners pay attention for the safety and mobility of such areas. As an important part of TTE 6256 (Traffic Operations Course), we conducted this project which focuses on the existing 870 ft segment of N Alafaya Trail from Waterford Lakes Town Center to State Road 408 (West). This project analyzes the signal timing, queue delay, and vehicle counts using highway capacity software (HCS) 7.9 with the data collected at the two intersections on N Alafaya Trail - N Alafaya Trail & Waterford Lakes Town Center (Intersection 1) and N Alafaya Trail & State Road 408 West (Intersection 2) on October 6, 2020 during the afternoon peak hours (4:00 pm - 7:00 pm). At intersection 1 all the four directions (N, E, S, W) were examined, and only 3 directions (N, E, S) were selected at intersection 2, since the westbound segment led to State Road 408 West, as shown in Figure 1.3.1.

1.2 Objectives

There are three objectives for which this project is undertaken. The objectives of this project are as follows:

- To identify traffic flows from the bottlenecks of the two intersections.
- To analyze the traffic condition of these intersections using the latest version of HCS.
- To propose alternatives for improving bottleneck condition.

1.3 Existing condition

Both intersections are signalized four-way intersections in Orange County; however, excluding the west ramp converted Intersection 2 into a three-way intersection. N Alafaya Trail runs in a north-south direction with Waterford Lakes Town Center running in the east-west direction. The intersections have dotted line extensions through the intersections, painted crosswalks and pedestrian signals, and painted directional arrows on all lanes for the eastbound and westbound intersection approaches; however, the north and south bound intersection approaches only have painted directional arrows for left and right turning lanes. All the intersections' approach directions have painted stop lines, as well as solid white lines for each lane.

At Intersection 1, the eastbound intersection approach contains one protected left turn lane and one shared through and right turn lane; the tire marks wear off the painted directional arrows near the stop line. The west bound intersection approach has three lanes: a protected left turn lane, a thru lane, and a right turn lane. The northbound intersection approach had two protected left turn lanes, three thru lanes, and one right turn lane. The southbound intersection approach has 6 lanes: two protected left turn lanes, three thru lanes, and one right turn lane.

Figure 1.3.1 The two red boxes show the signalized intersections of interest being N Alafaya Trail & Waterford Lakes Town Center (Intersection 1) and N Alafaya Trail & State Road 408 West (Intersection 2). Courtesy of Google Maps, accessed October 20, 2020.

At Intersection 2, the west bound intersection approach contains one protected left turn lane, one shared through and left turn lane, and one right turn lane. The northbound intersection approach has two protected left turn lanes which were excluded from the data collection, three thru lanes, and one right turn lane. In the southbound intersection approach, there was a single protected left turn lane and three thru lanes.

2. Data Collection Methodology *(Johnathan, Jorge, Rakibul)*

2.1 Signal timing

The signal timing durations were recorded from 4:00 PM to 5:00 PM and 6:00 PM to 7:00 PM for each time interval in 30 minutes. Every observer had a traffic signal for every intersection approach to watch the intersection signals cycling from green, red, and yellow time.

2.2 Back of queue

The control delay was measured through the queue-count technique which required two observers: one was to keep track of vehicles in the count interval, and the other counted the approach volumes that stopped and didn't stop during the observation for every cycle. The control device at the subject intersection was the traffic signal. All the vehicles in the queue at the left movements passed through the green light without issues, and there was no overflow queue at these intersections during the PM peak hour.

2.3 Vehicle counts

The traffic vehicle volume was counted on Tuesday, October 6, 2020 to represent both typical traffic intersection studies on a weekday, and the traffic flow patterns on Intersection 1 during the PM peak hour are lesser than the traffic flow patterns on Intersection 2 due to vehicles exiting out of the Waterford Town Center during the evening.

3. Data Analysis & Results *(Rakibul, Jorge, Johnathan)*

The collected data was properly formatted and input to HCS 7.9. After the reports were generated from HCS 7.9, an analysis was done to compare the report and the observations collected. The sections below describe in detail the calculations done to conduct a comprehensive analysis.

3.1 Input data analysis

Based on the hourly traffic volume as shown in Table 3.1.1, the peak hour was selected. For intersection 1, 5:30-6:30 was the peak hour which contained the highest total volume 4415. For intersection 2, the peak hour is 4:30-5:30 with the highest total volume 4463. From the peak hour, one of the 15-minute periods which has peak volume was measured. For intersection 1, the peak 15 minute is 6:00-6:15 with traffic volume 1223, and for intersection 2, the peak 15 minute is 4:45-5:00 with traffic volume 1234. Using Equation 3.1 peak hour factor (PHF) for intersection 1 and 2 are 0.90 each.

$$PHF = \frac{V_{hr}}{V_{15\,min} \times 4} \qquad \text{[Eqn 3.1]}$$

Table 3.1.1: Estimation of PHF

Intersection 1				Intersection 2			
Time	15 minutes traffic volume	Time	Hourly traffic volume	Time	15 minutes traffic volume	Time	Hourly traffic volume
4:30-4:45	754	4:30-5:30	3752	4:30-4:45	997	4:30-5:30	4463
4:45-5:00	1016	4:45-5:45	4109	4:45-5:00	1234	4:45-5:45	4460
5:00-5:15	984	5:00-6:00	4124	5:00-5:15	1162	5:00-6:00	4265
5:15-5:30	998	5:15-6:15	4363	5:15-5:30	1070	5:15-6:15	4095
5:30-5:45	1111	5:30-6:30	4415	5:30-5:45	994	5:30-6:30	4194
5:45-6:00	1031			5:45-6:00	1039		
6:00-6:15	1223			6:00-6:15	992		
6:15-6:30	1050			6:15-6:30	1169		

After identifying the geometric configuration in the software, we input average traffic volume per hour. As this project deals with coordination of two intersection signal timing, uncoordinated intersection box blank was unchecked. Additionally, a field-measured phase-times box was checked. Based on the collected traffic signal field data, we input phase split and yellow change values. See the appendix for the details of these inputs.

Table 3.1.2: Lane width and storage length information

	Lane direction	Lane width (ft)	Storage length (ft)
Intersection 1	Eastbound left	12	212
	Westbound left	12	117.56
	Northbound left	12	256
	Southbound left	12	343.72
Intersection 2	Eastbound left	N/A	N/A
	Westbound left	12	74.92
	Northbound left	12	225
	Southbound left	12	360.53

Lane width and storage length were collected from the Google map web tool as shown in Table 3.1.2. In our field survey, the number of vehicle arrivals at red time was not observed. It is assumed that a random number of vehicles arrive during red time. Therefore, arrival Type 3 was used.

3.2 Results

In this section, the level of service (LOS) of the roadway lanes and lane groups are discussed based on HCS 7.9 results summary. Additionally, control delay, which was estimated from HCS, was compared with field study. Moreover, the bottleneck condition in the turning lane for both inbound and outbound from the town center is also discussed.

HCS 7.9 has provided the LOS based on signalized intersection input data as shown in Figure 7.1 (see appendix). The software provided the Levels of Service (LOS) for each intersection based on the control delay per vehicle. This intersection has an average vehicle delay of 33.5 s/veh which resulted in a LOS “E”. All the lanes of major roads (E-W) have a LOS “E” which controlled the overall intersection LOS.

The lane with southbound left and westbound left traffic has control delay 77.1s/veh and 61.7 s/veh respectively. These lanes are inbound and outbound from the town center respectively. They both have LOS “E”. Westbound left vehicles are coming out from the town center. From intersection control delay data sheet of field observation, the values of control delay of the lanes with southbound left and westbound left traffic are 68.5 and 60.7 s/veh, respectively, which are fairly close to the HCS 7.9 results. This intersection 2 has an average vehicle delay of 31.2 s/veh which resulted in a LOS “C”. Here, both lane groups of major roads (N-S) had a LOS “B”. However, the westbound lane group had a LOS “F” with approach delay 125.1 s/veh. In this lane group, the lane with westbound left traffic has queue storage ratio more than 1 and control delay 162.8 s/veh which is a fairly large. The lane with southbound left traffic has control delay 10.5 s/veh which is close to the value (12.46 s/veh) from intersection control delay data sheet of field observation.

From intersection control delay data sheet of field observation, the values of control delay of the lane westbound left traffic are 6.06 s/veh which shows a large disparity with the value (162.8 s/veh) from HCS 7.9. The reason for such disparity resulted from the field survey issues. We came to know from the surveyor, who measured the number of vehicles from the queue of that lane, that it was hard to count the number of queue lengths properly because the queue was very long beyond the storage length. That is why, it was confusing to mark the vehicle if that vehicle is going through or left. The Google map's aerial photo (Figure 3.2.1) also verifies their statement.

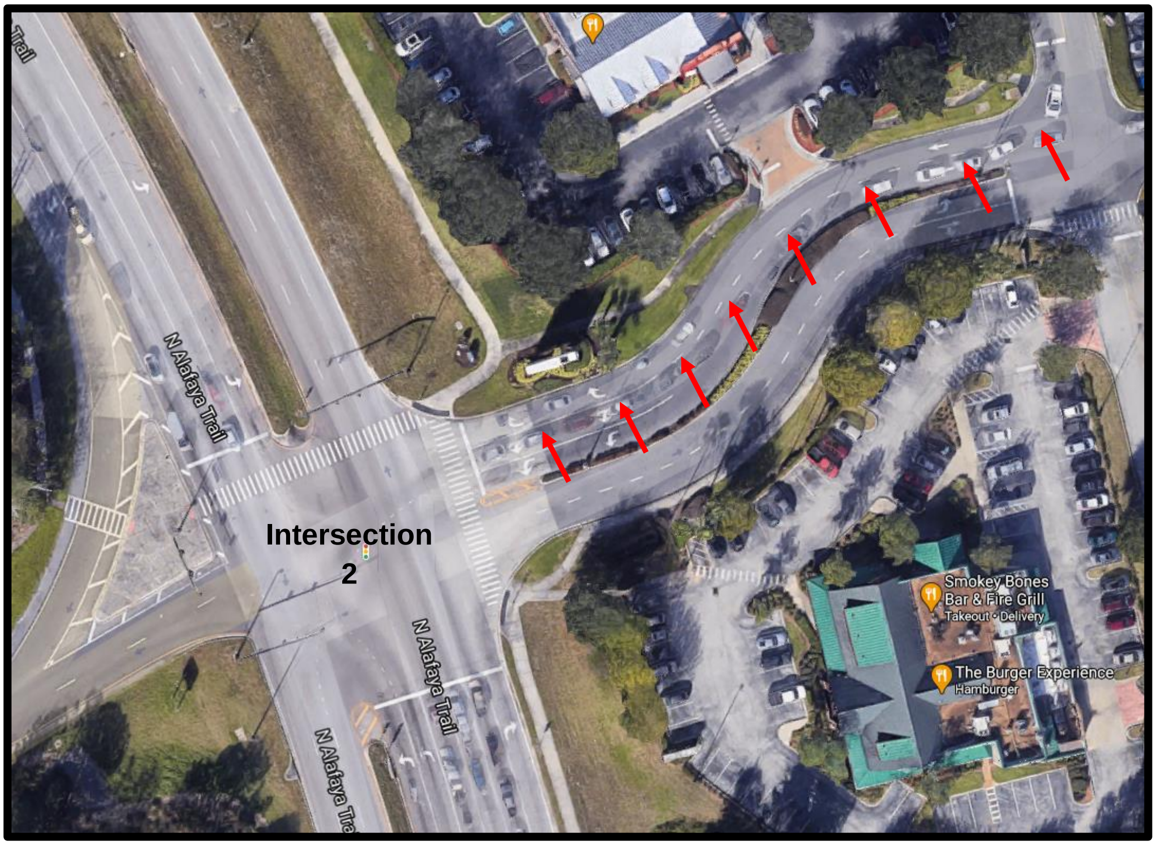


Figure 3.2.1: Bottleneck in the lane with westbound traffic passes beyond the storage length. Courtesy of Google Maps, accessed October 20, 2020.

3.3 Signal Coordination Analysis

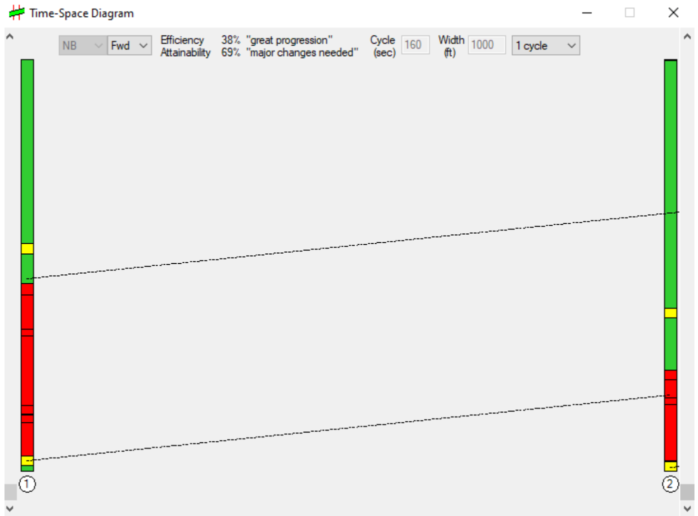


Figure 3.3.1 - Time-Space Diagram 1 – NB Forward Direction

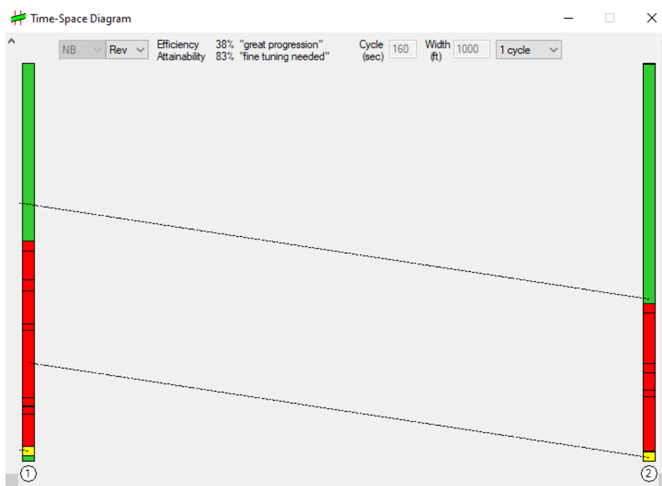


Figure 3.3.2 - Time-Space Diagram 2 – NB Backward Direction

From the Northbound (NB) forward and backward direction time-space diagram, it is found that the efficiency of intersection coordination is poor (38%). HCS suggests 69% major changes for forward direction efficiency and 83% fine tuning for backward direction.

4. Alternative Solutions *(Jorge, Rakibul, Johnathan)*

The alternative solutions were proposed to the intersections that did not meet regional level of service (LOS) standards. For two-hour p.m. peak, the regional level of service standards are: LOS E or better in urban centers and corridors, LOS D or better elsewhere in the remainder of the cities and unincorporated urban growth areas, and LOS C or better in rural areas (outside of the urban areas and Tribal Reservations) (Thurston Regional Planning Council, 2016) . Since both intersections were with an urban center, we should suggest solutions for any lanes with LOS F. Intersection 1 did not have any lanes with LOS F, so no solutions were needed. Intersection 2 had a LOS F for the westbound left, therefore the solutions proposed below are intended for Intersection 2 westbound left.

4.1 Alternative 1: Adjusted Signal Timings for Intersection 2

An alternative to improve the control delay was to change the signal timing, meaning the LOS would also improve for the given approach. Since the Westbound left approach for intersection 2 had a large control delay, the signal timing was adjusted for this approach. It is important to note that the Westbound left, through, and right signals all had the same signal timings. Therefore, it would be unreasonable to decrease the timings for the through and right signal timings, if in real-life they are equal. Since the signal timings did not propose a problem for the other approaches in intersection 2 (i.e. Northbound and Southbound), it was best not to adjust these timings. In order to visualize the effect that signal timing has on the control delay, a graph was developed showing the increase to the signal timing, and the corresponding control delay as shown in Figure 4.1.1. The figure shows a nonlinear relationship between the control delay and the signal timing for intersection 2. A trendline was also developed using westbound control delay in vertical axis and signal timing in horizontal axis for intersection 2 as shown in Equation 4.1.1. The minimum control delay can be achieved by simply adjusting the signal timing from Equation 4.1.1. It was found that the optimal signal timing increase was 53.25%, which corresponds to a control delay of 60.63 s/veh.

$$y = 0.0354x\text{^}2 - 3.77x + 161 \; [Eqn\ 4.1.1]$$
$$Here, x = westbound\ phase\ split\ signal\ timing\ increase\ (\%) and$$
$$y = control\ delay\ (s/veh)$$
$$y' = 0.0708x - 3.77$$
$$For\ maximum\ y, \qquad y' = 0$$
$$x = 53.24\%, y = 60.62\ s/veh$$

Table 4.1.1: LOS and Approach Delay for Intersection 2

Intersection 2 LOS and control delay (s/veh)			
	Northbound	Southbound	Westbound
Existing	LOS B (15.4)	LOS B (17.5)	LOS F (125.1)
Alternative 1	LOS B (19.4)	LOS C (26.5)	LOS D (51.1)

Intersection 2		
Westbound phase split Signal Timing Increase (%)	Westbound phase split Signal Timing (s)	Control Delay (s/veh)
0	24.5	162.8
15	28.2	108.2
30	31.9	79.2
45	35.5	66.8
60	39.2	59.6

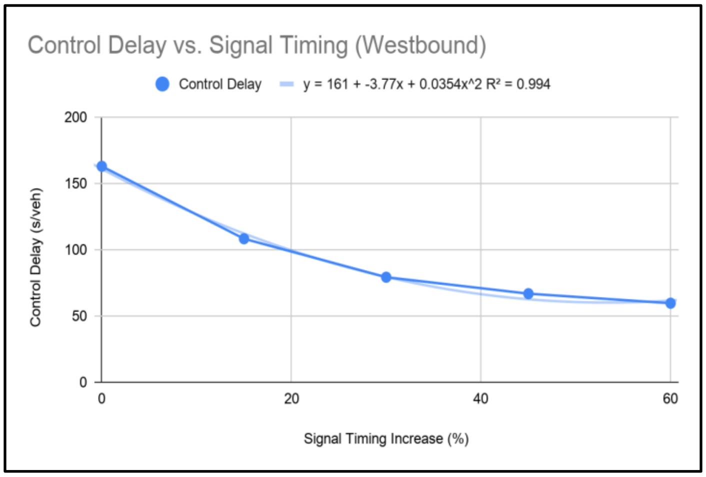


Figure 4.1.1: The relationship between the signal timing and control delay for Alternative 1

Table 4.1.2: Percentage Reduction in Control Delay for Intersection 2

Percentage Reduction in Control Delay (Westbound left)		
Existing Control Delay (s/veh)	Predicted Optimal Control Delay (s/veh)	Percent Reduced (%)
162.80	60.2	63.02%

4.2 Alternative 2: Geometry Change for Intersection 2

An alternative to improve the control delay was to modify the existing geometry of the intersections. To understand the effect that the intersection geometry has on the control delay, HCS was utilized. Since, the control delay was the highest for the westbound left lane in intersection 2, it was reasonable to add an additional left turn lane. This also worked well, since no adjustments would have to be made for the other approaches. Table 4.2.1 shows the change in the control delay with just an additional left turn lane. The percentage change due to the additional left lane was 59.83% decrease in the control delay.

Table 4.2.1: Performance of alternative 2

Intersection 2			
Existing		Adjusted	
Control Delay (s/veh)	LOS	Control Delay (s/veh)	LOS
162.8	F	65.4	E

4.3 Alternative 3: Combinations of Adjustments for Intersection 2

An alternative to improve the control delay was to modify both the signal timings and the existing geometry of the intersections. In other words, this alternative is a combination of Alternative 1 and Alternative 2. After applying this alternative, there is a linear relationship between the control delay and signal timing, as shown in Figure 4.3.1.

Intersection 2		
Westbound Phase Split Signal Timing Increase (%)	Signal Timing	Control Delay (s/veh)
0	(s)	65.4
15	24.5	61.4
30	28.2	58.0
45	31.9	54.8
60	35.5	51.6

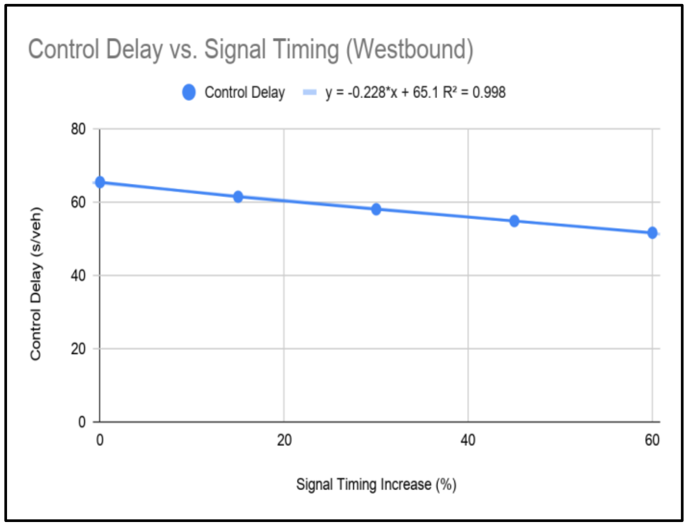


Figure 4.3.1: The relationship between the signal timing and control delay for Alternative 3

4.4 Evaluations of Alternatives

Each of the proposed alternatives (i.e. Alternative 1, Alternative 2, and Alternative 3) has a different cost associated with the adjustments that would be made in terms for signal timing and/or intersection geometry. For Alternative 1, there would be no cost associated with the adjustments, since the only change would be increasing the signal timing. Alternative 2, would have to bear the cost of an additional lane, which was found to be around $1-2 million per mile (Feigenbaum, 2011). Alternative 3 would have the same cost as Alternative 2, since it is the combination of Alternative 1 and Alternative 2. Considering the cost for each of the alternatives, it was decided that Alternative 1 be the alternative that should be implemented currently. If future volumes increase to the point that Alternative 1 does not reduce the control delay, one might consider Alternative 2 or Alternative 3.

5. Conclusions *(Jorge, Rakibul, Johnathan)*

In conclusion, both methods of obtaining the control delay proved to be accurate. The only disparity between the calculating the control delay and the calculated control delay through HCS 7.9 was the westbound left on intersection 2. The disparity was due to the difficulty of known which vehicles would turn left by the short storage length and the shared through and left lane. Besides this instance the percent error among the other calculated control delay was below 23%. To improve the control delay, 3 alternatives were proposed. Alternative 1 suggests to increase 53.25% westbound phase split timing which results in minimum control delay 60.63 s/veh. Alternative 2 focused on the addition of a left turn lane, and Alternative 3 focused on the combination of adjusting the signal timing and adding a left turn lane. As expected, each of these alternatives reduced the control delay, but the degree of reduction differs among each alternative. Alternative 1 is the most economical alternative with the satisfactory control delay. Alternative 3 is the most expensive one although the control delay is the lowest.

6. Limitations and Recommendations *(Rakibul, Jorge, Johnathan)*

- Generally, there are limitations due to some reasons in each study. In our study, because of time constraint only afternoon peak hour time was chosen. If the data for morning peak hour time could be collected that will make our project report more informative. Besides working with both the weekend and weekdays data would make the project more comprehensive.

- The data was collected in the pandemic of COVID-19 because of which the traffic volume is not the same as regular time, i.e., before the pandemic. In future, we hope the data will be available for a pandemic free regular traffic condition.
- The data collection was done by humans (students) in the field. The more accurate data can be collected through video surveillance for better accuracy.

7. References *(Rakibul, Jorge, Johnathan)*

1. McTrans Center, 2020.Urban streets HCS 7.9 User's manual.
2. Thurston Regional Planning Council, Washington, 2016. Appendix O: Level of service standard and measurements. Retrieved from https://www.trpc.org/DocumentCenter/View/2798/Appendix-O--Level-of-Service-Standard-and-Measurement
3. Dr. Sandt's lecture and supporting handouts.
4. Photograph of Intersection 2. Google Maps, 20 Oct. 2020. Retrieved from https://www.google.com/maps/place/Waterford+Lakes+Town+Center/@28.5513469,-81.2041614,159m/data=!3m1!1e3!4m5!3m4!1s0x88e767764fd29dfb:0x403c54495dbd2618!8m2!3d28.5545475!4d-81.2003099
5. Feigenbaum, B., 2011. Reducing Congestion on a Budget: Turn Lanes. Retrieved from https://reason.org/commentary/reducing-congestion-on-a-budget-tur/#:~:text=Constructing%20a%20continuous%20turn%20lane,cost%20of%20widening%20the%20highway.

8. Appendix *(Rakibul, Johnathan, Jorge)*

Table 7.1: Summary of the control delay for intersection 1

	Control Delay Summary			
Southbound Left	ΣV_{iq}	93	N_c	75
	d_{vp}	12.31	FVS	0.03
	$\frac{V_{stop}}{N_c \times N}$	0.04	d_{ad}	0.15
	CF	5	d	12.46

	Control Delay Summary			
Westbound Left	ΣV_{iq}	152	N_c	43
	d_{vp}	5.95	FVS	0.02
	$\frac{V_{stop}}{N_c \times N}$	0.19	d_{ad}	0.12
	CF	5	d	6.06

ΣV_{iq}	Vehicle in queue
N_c	Number of cycles
V_{stop}	Stopped approach volume
V_{tot}	Total approach volume
d_{vp}	Time in queue per vehicle
$\frac{V_{stop}}{N_c \times N}$	Number of vehicles stopping per lane per cycle
CF	Acceleration/deceleration factor
FVS	Fraction of vehicle stopping
d_{ad}	Acceleration/deceleration correction delay
d	Control delay per vehicle

Table 7.2: Summary of the control delay for intersection 2

	Control Delay Summary			
Southbound Left	ΣV_{iq}	1819	N_c	47
	d_{vp}	64.12	FVS	0.87
	$\frac{V_{stop}}{N_c \times N}$	3.54	d_{ad}	4.35
	CF	5	d	68.46

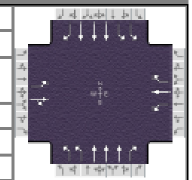
	Control Delay Summary			
Westbound Left	ΣV_{iq}	1819	N_c	47
	d_{vp}	56.69	FVS	0.80
	$\frac{V_{stop}}{N_c \times N}$	7.29	d_{ad}	4.02
	CF	5	d	60.70

ΣV_{iq}	Vehicle in queue
N_c	Number of cycles
V_{stop}	Stopped approach volume
V_{tot}	Total approach volume
d_{vp}	Time in queue per vehicle
$\frac{V_{stop}}{N_c \times N}$	Number of vehicles stopping per lane per cycle
CF	Acceleration/deceleration factor
FVS	Fraction of vehicle stopping
d_{ad}	Acceleration/deceleration correction delay
d	Control delay per vehicle

Figure 7.1: Summary of HCS input data sheet for intersection 1

HCS7 Signalized Intersection Input Data

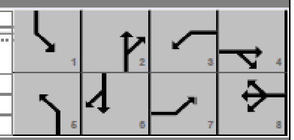
General Information				Intersection Information	
Agency	Group 4			Duration, h	0.250
Analyst	Jorge Ugan and Md Rakibul Alam	Analysis Date	10/18/2020	Area Type	Other
Jurisdiction	HCM	Time Period	4pm to 7pm	PHF	0.90
Urban Street	North Alafaya Trail	Analysis Year	2020	Analysis Period	1> 7:00
Intersection	North Alafaya Trail & Wa…	File Name	Project1PlanB.xus		
Project Description	TTE6256 Project				



Demand Information	EB			WB			NB			SB		
Approach Movement	L	T	R	L	T	R	L	T	R	L	T	R
Demand (*v*), veh/h	124	32	125	194	55	138	267	1025	149	183	1144	651

Signal Information			
Cycle, s	160.0	Reference Phase	2
Offset, s	0	Reference Point	End
Uncoordinated	No	Simult. Gap E/W	On
Force Mode	Fixed	Simult. Gap N/S	On

Green	13.3	11.4	73.6	13.6	0.4	26.9
Yellow	4.3	4.0	3.6	3.1	3.3	2.5
Red	0.0	0.0	0.0	0.0	0.0	0.0



Traffic Information	EB			WB			NB			SB		
Approach Movement	L	T	R	L	T	R	L	T	R	L	T	R
Demand (v), veh/h	124	32	125	194	55	138	267	1025	149	183	1144	651
Initial Queue (Q_b), veh/h	0	0	0	0	0	0	0	0	0	0	0	0
Base Saturation Flow Rate (s_o), veh/h	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900
Parking (N_m), man/h		None			None			None			None	
Heavy Vehicles (P_{HV}), %	0	0		0	0	0	0	0	0	0	0	0
Ped / Bike / RTOR, /h	0	0	0	0	0	0	0	0	0	0	0	0
Buses (N_b), buses/h	0	0	0	0	0	0	0	0	0	0	0	0
Arrival Type (*AT*)	3	3	3	3	3	3	3	3	3	3	3	3
Upstream Filtering (*I*)	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.82	0.82	0.82
Lane Width (*W*), ft	12.0	12.0		12.0	12.0	12.0	12.0	12.0	12.0	12.0	12.0	12.0
Turn Bay Length, ft	212	0		118	0	0	256	0	0	344	0	0
Grade (P_g), %		0			0			0			0	
Speed Limit, mi/h	35	35	35	35	35	35	35	35	35	35	35	35

Phase Information	EBL	EBT	WBL	WBT	NBL	NBT	SBL	SBT
Maximum Green (G_{max}) or Phase Split, s	20.4	33.1	16.7	29.4	33.0	92.6	17.6	77.2
Yellow Change Interval (*Y*), s	3.3	3.2	3.1	2.5	4.0	4.0	4.3	3.6
Red Clearance Interval (R_c), s	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Minimum Green (G_{min}), s	6	6	6	6	6	6	6	6
Start-Up Lost Time (l_t), s	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0
Extension of Effective Green (*e*), s	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0
Passage (*PT*), s	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0
Recall Mode	Off	Off	Off	Off	Off	Min	Off	Min
Dual Entry	No	Yes	No	Yes	No	Yes	No	Yes
Walk (*Walk*), s		0.0		0.0		0.0		0.0
Pedestrian Clearance Time (*PC*), s		0.0		0.0		0.0		0.0

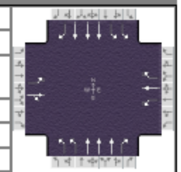
Multimodal Information	EB			WB			NB			SB		
85th % Speed / Rest in Walk / Corner Radius	0	No	25	0	No	25	0	No	25	0	No	25
Walkway / Crosswalk Width / Length, ft	9.0	12	0	9.0	12	0	9.0	12	0	9.0	12	0
Street Width / Island / Curb	0	0	No	0	0	No	0	0	No	0	0	No
Width Outside / Bike Lane / Shoulder, ft	12	5.0	2.0	12	5.0	2.0	12	5.0	2.0	12	5.0	2.0
Pedestrian Signal / Occupied Parking	No	0.50		No	0.50		No	0.50		No	0.50	

 HCS™ Streets Version 7.9 Generated: 10/18/2020 9:27:30 PM

Figure 7.2: Results summary from HCS for intersection 1

HCS7 Signalized Intersection Results Summary

General Information				Intersection Information	
Agency	Group 4			Duration, h	0.250
Analyst	Jorge Ugan and Md Rakibul Alam	Analysis Date	10/18/2020	Area Type	Other
Jurisdiction	HCM	Time Period	4pm to 7pm	PHF	0.90
Urban Street	North Alafaya Trail	Analysis Year	2020	Analysis Period	1> 7:00
Intersection	North Alafaya Trail & Wa…	File Name	Project1PlanB.xus		
Project Description	TTE6256 Project				



Demand Information	EB			WB			NB			SB		
Approach Movement	L	T	R	L	T	R	L	T	R	L	T	R
Demand (*v*), veh/h	124	32	125	194	55	138	267	1025	149	183	1144	651

Signal Information			
Cycle, s	160.0	Reference Phase	2
Offset, s	0	Reference Point	End
Uncoordinated	No	Simult. Gap E/W	On
Force Mode	Fixed	Simult. Gap N/S	On

Green	13.3	11.4	73.6	13.6	0.4	26.9
Yellow	4.3	4.0	3.6	3.1	3.3	2.5
Red	0.0	0.0	0.0	0.0	0.0	0.0

Timer Results	EBL	EBT	WBL	WBT	NBL	NBT	SBL	SBT
Assigned Phase	7	4	3	8	5	2	1	6
Case Number	2.0	4.0	1.1	3.0	2.0	3.0	2.0	3.0
Phase Duration, s	20.4	33.1	16.7	29.4	33.0	92.6	17.6	77.2
Change Period, ($Y+R_c$), s	3.3	3.2	3.1	3.2	4.0	4.0	4.3	4.0
Max Allow Headway (*MAH*), s	3.1	3.3	3.1	3.3	3.1	0.0	3.1	0.0
Queue Clearance Time (g_s), s	13.8	17.3	15.6	16.1	14.1		8.7	
Green Extension Time (g_e), s	0.1	0.7	0.0	0.6	0.6	0.0	0.1	0.0
Phase Call Probability	1.00	1.00	1.00	1.00	1.00		1.00	
Max Out Probability	0.79	0.00	1.00	0.01	0.00		0.14	

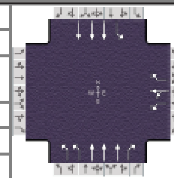
Movement Group Results	EB			WB			NB			SB		
Approach Movement	L	T	R	L	T	R	L	T	R	L	T	R
Assigned Movement	7	4	14	3	8	18	5	2	12	1	6	16
Adjusted Flow Rate (*v*), veh/h	138	174		216	61	153	297	1139	166	147	919	523
Adjusted Saturation Flow Rate (*s*), veh/h/ln	1810	1662		1810	1900	1610	1757	1725	1610	1757	1725	1610
Queue Service Time (g_s), s	11.8	15.3		13.6	4.4	14.1	12.1	20.1	8.2	6.7	13.6	29.8
Cycle Queue Clearance Time (g_c), s	11.8	15.3		13.6	4.4	14.1	12.1	20.1	8.2	6.7	13.6	29.8
Green Ratio (*g/C*)	0.11	0.19		0.25	0.16	0.16	0.18	0.55	0.55	0.08	0.46	0.46
Capacity (*c*), veh/h	193	311		296	311	264	637	2866	892	292	2368	737
Volume-to-Capacity Ratio (*X*)	0.712	0.562		0.728	0.196	0.582	0.466	0.397	0.186	0.503	0.388	0.710
Back of Queue (*Q*), ft/ln (50 th percentile)	150	164		55.6	54	148.2	135.2	206	79.9	79.5	123.4	166
Back of Queue (*Q*), veh/ln (50 th percentile)	6.0	6.6		2.2	2.2	5.9	5.4	8.2	3.2	3.2	4.9	6.6
Queue Storage Ratio (*RQ*) (50 th percentile)	0.71	0.00		0.47	0.00	0.00	0.53	0.00	0.00	0.23	0.00	0.00
Uniform Delay (d_1), s/veh	69.1	59.1		54.1	57.8	61.8	58.6	20.4	17.8	76.7	18.1	16.4
Incremental Delay (d_2), s/veh	10.1	1.4		7.7	0.1	2.2	0.2	0.4	0.5	0.4	0.4	4.7
Initial Queue Delay (d_3), s/veh	0.0	0.0		0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Control Delay (*d*), s/veh	79.1	60.5		61.7	57.9	64.0	58.8	20.8	18.2	77.1	18.5	21.1
Level of Service (LOS)	E	E		E	E	E	E	C	B	E	B	C
Approach Delay, s/veh / LOS	68.7	E		62.0	E		27.6	C		24.8	C	
Intersection Delay, s/veh / LOS	33.5						C					

Multimodal Results	EB		WB		NB		SB	
Pedestrian LOS Score / LOS	2.87	C	2.87	C	2.27	B	2.11	B
Bicycle LOS Score / LOS	1.00	A	1.20	A	1.37	A	1.70	B

Figure 7.3: Summary of HCS input data sheet for intersection 2

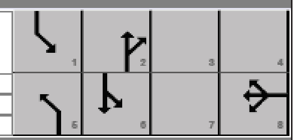
HCS7 Signalized Intersection Input Data

General Information				Intersection Information	
Agency	Group 4			Duration, h	0.250
Analyst	Jorge Ugan and Md Rakibul Alam	Analysis Date	10/18/2020	Area Type	Other
Jurisdiction	HCM	Time Period	4pm to 7pm	PHF	0.90
Urban Street	North Alafaya Trail	Analysis Year	2020	Analysis Period	1> 7:00
Intersection	North Alafaya Trail & 40…	File Name	Project1PlanB.xus		
Project Description	TTE6256 Project				



Demand Information	EB			WB			NB			SB		
Approach Movement	L	T	R	L	T	R	L	T	R	L	T	R
Demand (*v*), veh/h				250	67	85	471	1628	570	53	1207	

Signal Information										
Cycle, s	160.0	Reference Phase	2							
Offset, s	0	Reference Point	End	Green	7.2	20.3	96.3	21.9	0.0	0.0
Uncoordinated	No	Simult. Gap E/W	On	Yellow	3.8	3.8	4.1	2.6	0.0	0.0
Force Mode	Fixed	Simult. Gap N/S	On	Red	0.0	0.0	0.0	0.0	0.0	0.0



Traffic Information	EB			WB			NB			SB		
Approach Movement	L	T	R	L	T	R	L	T	R	L	T	R
Demand (*v*), veh/h				250	67	85	471	1628	570	53	1207	
Initial Queue (Q_b), veh/h				0	0	0	0	0	0	0	0	
Base Saturation Flow Rate (s_o), veh/h				1900	1900	1900	1900	1900	1900	1900	1900	
Parking (N_m), man/h					None			None			None	
Heavy Vehicles (P_{HV}), %				0	0	0	0	0	0	0	0	
Ped / Bike / RTOR, /h	0	0		0	0	0	0	0	0	0	0	
Buses (N_b), buses/h				0	0	0	0	0	0	0	0	0
Arrival Type (*AT*)				3	3	3	3	3	3	3	3	
Upstream Filtering (*I*)				1.00	1.00	1.00	0.89	0.89	0.89	1.00	1.00	
Lane Width (*W*), ft				12.0	12.0	12.0	12.0	12.0	12.0	12.0	12.0	
Turn Bay Length, ft				75	0	0	225	0	0	361	0	
Grade (*Pg*), %		0			0			0			0	
Speed Limit, mi/h				35	35	35	35	35	35	35	35	

Phase Information	EBL	EBT	WBL	WBT	NBL	NBT	SBL	SBT
Maximum Green (G_{max}) or Phase Split, s				24.5	35.1	124.5	11.0	100.4
Yellow Change Interval (*Y*), s				2.6	3.8	4.5	3.8	4.1
Red Clearance Interval (R_c), s				0.0	0.0	0.0	0.0	0.0
Minimum Green (G_{min}), s				6	6	6	6	6
Start-Up Lost Time (*lt*), s			2.0	2.0	2.0	2.0	2.0	2.0
Extension of Effective Green (*e*), s			2.0	2.0	2.0	2.0	2.0	2.0
Passage (*PT*), s				2.0	2.0	2.0	2.0	2.0
Recall Mode				Off	Off	Min	Off	Min
Dual Entry				Yes	No	Yes	No	Yes
Walk (*Walk*), s		0.0		0.0		0.0		
Pedestrian Clearance Time (*PC*), s		0.0		0.0		0.0		

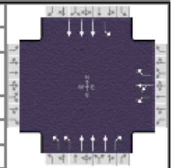
Multimodal Information	EB			WB			NB			SB		
85th % Speed / Rest in Walk / Corner Radius	0	No	25	0	No	25	0	No	25			
Walkway / Crosswalk Width / Length, ft	9.0	12	0	9.0	12	0	9.0	12	0			
Street Width / Island / Curb		0		0	0	No	0	0	No	0		No
Width Outside / Bike Lane / Shoulder, ft				12	5.0	2.0	12	5.0	2.0	12	5.0	2.0
Pedestrian Signal / Occupied Parking	No			No	0.50		No	0.50			0.50	

 HCS™ Streets Version 7.9 Generated: 10/18/2020 9:27:30 PM

Figure 7.4: Results summary from HCS for intersection 2

HCS7 Signalized Intersection Results Summary

General Information				Intersection Information	
Agency	Group 4			Duration, h	0.250
Analyst	Jorge Ugan and Md Rakibul Alam	Analysis Date	10/18/2020	Area Type	Other
Jurisdiction	HCM	Time Period	4pm to 7pm	PHF	0.90
Urban Street	North Alafaya Trail	Analysis Year	2020	Analysis Period	1> 7:00
Intersection	North Alafaya Trail & 40…	File Name	Project1PlanB.xus		
Project Description	TTE6256 Project				



Demand Information	EB			WB			NB			SB		
Approach Movement	L	T	R	L	T	R	L	T	R	L	T	R
Demand (*v*), veh/h				250	67	85	471	1628	570	53	1207	

Signal Information										
Cycle, s	160.0	Reference Phase	2							
Offset, s	0	Reference Point	End							
Uncoordinated	No	Simult. Gap E/W	On	Green	7.2	20.3	96.3	21.9	0.0	0.0
Force Mode	Fixed	Simult. Gap N/S	On	Yellow	3.8	3.8	4.1	2.6	0.0	0.0
				Red	0.0	0.0	0.0	0.0	0.0	0.0

Timer Results	EBL	EBT	WBL	WBT	NBL	NBT	SBL	SBT
Assigned Phase				8	5	2	1	6
Case Number				9.0	2.0	3.0	1.1	4.0
Phase Duration, s				24.5	35.1	124.5	11.0	100.4
Change Period, (*Y+R* c), s				2.6	3.8	4.5	3.8	4.5
Max Allow Headway (*MAH*), s				3.2	3.1	0.0	3.1	0.0
Queue Clearance Time (*g* s), s				23.9	12.9		3.9	
Green Extension Time (*g* e), s				0.0	0.5	0.0	0.0	0.0
Phase Call Probability				1.00	1.00		1.00	
Max Out Probability				1.00	0.00		0.77	

Movement Group Results	EB			WB			NB			SB		
Approach Movement	L	T	R	L	T	R	L	T	R	L	T	R
Assigned Movement				3	8	18	5	2	12	1	6	
Adjusted Flow Rate (*v*), veh/h				278	74	94	252	872	305	59	1341	
Adjusted Saturation Flow Rate (*s*), veh/h/ln				1810	1900	1610	1757	1725	1610	1810	1725	
Queue Service Time (*g* s), s				21.9	5.6	8.6	10.9	6.9	6.7	1.9	22.4	
Cycle Queue Clearance Time (*g* c), s				21.9	5.6	8.6	10.9	6.9	6.7	1.9	22.4	
Green Ratio (*g/C*)				0.14	0.14	0.14	0.20	0.75	0.75	0.64	0.60	
Capacity (*c*), veh/h				248	260	220	687	3882	1208	513	3102	
Volume-to-Capacity Ratio (*X*)				1.122	0.286	0.429	0.367	0.225	0.253	0.115	0.432	
Back of Queue (*Q*), ft/ln (50 th percentile)				414.2	68.6	89.2	128.5	58.2	53.1	19.4	225.1	
Back of Queue (*Q*), veh/ln (50 th percentile)				16.6	2.7	3.6	5.1	2.3	2.1	0.8	9.0	
Queue Storage Ratio (*RQ*) (50 th percentile)				5.52	0.00	0.00	0.57	0.00	0.00	0.05	0.00	
Uniform Delay (*d* 1), s/veh				69.1	62.0	63.3	64.1	5.0	4.1	10.5	17.3	
Incremental Delay (*d* 2), s/veh				93.8	0.2	0.5	0.1	0.1	0.5	0.0	0.4	
Initial Queue Delay (*d* 3), s/veh				0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	
Control Delay (*d*), s/veh				162.8	62.3	63.8	64.2	5.1	4.5	10.5	17.8	
Level of Service (LOS)				F	E	E	E	A	A	B	B	
Approach Delay, s/veh / LOS	0.0			125.1		F	15.4		B	17.5		B
Intersection Delay, s/veh / LOS	31.2						C					

Multimodal Results	EB		WB		NB		SB	
Pedestrian LOS Score / LOS	2.88	C	2.63	C	2.05	B	1.67	B
Bicycle LOS Score / LOS			1.22	A	2.12	B	1.26	A

 HCS™ Streets Version 7.9 Generated: 10/18/2020 9:27:30 PM