

WELLS + ASSOCIATES
MEMORANDUM



To: Ravindra Raut
City of Alexandria, VA

Cc: Bob Garbacz
Katie North
City of Alexandria, VA

From: Larry Sefcik
Grady Vaughan, EIT
Wells + Associates, Inc.

Re: Alfred Street Baptist Church (301 Alfred Street)

Subject: Supplemental Traffic Analysis
Based on Comments Received at Meeting on November 16, 2016

Date: January 6, 2017

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INTRODUCTION

This memorandum provides responses to comments received from the City of Alexandria staff during a meeting on Tuesday, November 16, 2016 regarding the Traffic Impact Analysis (TIA) revised through September 23, 2016 in support of the renovation and expansion of the Alfred Street Baptist Church.

The meeting was requested by City staff to gain further clarification on certain sections of the traffic study. It was agreed that the previous submitted TIA would not need to be amended. However, City staff requested supplemental analyses to be performed under separate cover to address concerns of City staff.

The parameters of the supplemental analysis are listed below:

- The following signalized intersections from the TIA were included in the supplemental analysis (numbering based on the TIA):
 8. Duke Street/Henry Street
 9. Duke Street/Patrick Street
 10. Duke Street/Alfred Street
 15. Patrick Street/Gibbon Street
- The peak hour factors would be set to “1.0” and the peak 15-minute vehicular traffic counts would be multiplied by four (4) to forecast a hypothetical “surge” Sunday peak hour period.
- The 10 percent non-auto adjustment originally assumed in the TIA would be removed from the trip generation.

A further explanation of the parameters and the subsequent results of the analysis are discussed below.

BACKGROUND

The Alfred Street Baptist Church project is located in the City of Alexandria, Virginia and is bounded by Duke Street to the north, Wolfe Street to the south, South Patrick Street to the west and South Alfred Street to the east. The subject site is currently occupied by a 43,784 SF church with an observed typical attendance of approximately 1,208 parishioners per service. Additionally, 22 affordable townhouses are located on the southern portion of the site that are currently built and occupied.

The applicant proposes to raze the townhomes and redevelop and expand the existing church with approximately 232,368 gross square feet of church space (from an observed 1,208 to proposed 2,163 seats) and a structured parking garage. The proposal includes a total of 129,130 SF of net floor area (38,031 SF existing + 91,099 SF proposed). The existing 1,208 seats is a recorded attendance of a typical Sunday service, with 920 seats provided within the existing sanctuary and chapel. The number of seats included in the proposed church space consists of the proposed main sanctuary and restored main chapel with their corresponding balconies. Of the 2,163 total seats, 1,906 seats would be located in the proposed main sanctuary while 257 seats would be located in the restored chapel. Parking would be provided via a proposed two level below-grade parking garage on site (216 spaces), a two level below-grade parking garage across South Patrick Street from the site (194 spaces), and a surface lot across of South Patrick Street (48 spots) totaling 458 spaces. A total of 34 bicycle parking spaces will be provided at grade and/or within the below grade parking garage.

The church currently has services at 7:30 AM, 9:30 AM, and 11:30 AM. It is a main goal of the renovation and expansion of the church to reduce the number of services from three (3) to two (2) on Sundays. The two (2) proposed services would be spread farther apart in the morning, likely allowing the earlier service to exit the study area prior to the later services arrival, reducing the effects of church traffic on the adjacent roadway network.

In an effort to provide a conservative analysis, both inbound and outbound site traffic was added to the network for total future conditions with the development within the same peak study hour. Additionally, proposed conditions include both a full sanctuary and separated chapel (2,163 total seats), with the assumption that every seat would be occupied. It is noted that the existing observed attendance would not occupy all 2,163 during any one (1) service.

SITE TRIP GENERATION METHODOLOGY

The TIA assumed a 10 percent non-auto adjustment that included the synergy that would occur between the proposed development and adjacent mix of uses in Old Town, the shuttle service provided to Metro, local walkable residences, and bus routes adjacent to the church. The shuttle service records, on average, approximately 370 trips per month, and runs on a

continuous loop. It is noted that the majority of trips are requested during the peak service on Sundays. A covered bicycle parking area is proposed within the new parking structure to further encourage non-auto mode share. The trip generation without 10% non-auto mode split is included as Table 1.

An internal survey was conducted by Alfred Street Church on June 4, 2016. The survey included the following question: “What primary means of transportation do you usually use to arrive at Alfred Street Baptist Church?”

Approximately 91 percent of the responses were “drive”. The remainder of the responses were divided among the remaining non-auto options:

- Ride with Someone,
- Bike,
- Public Transportation,
- Taxi, Uber, Lyft (similar service)
- Walk,
- Other

Although both the initial traffic study scoping meeting with the City and the church survey show that a significant non-auto component exists, this supplemental analysis assumes no mode split. Therefore, the results of this study should be considered conservative.

A copy of the survey question, the number of responses, and response ratio is shown as Attachment I.

SITE TRAFFIC DISTRIBUTIONS AND TOTAL FUTURE TRAFFIC FORECASTS

The trip assignment percentages used in the most recent TIA were derived based on input from City staff, private parishioner data, and knowledge of the local roadway network. With the 10 percent non-auto factor being eliminated from the analysis, the “new” trips were conservatively forecasted to arrive at the new on-site, underground parking garage. Further, resident information for Church members consistent with assumptions made in the TIA was requested by City staff and is included in Attachment II.

In order to conservatively analyze the peak “surge” period of church operations on a Sunday, City staff requested that the analysis be completed by extrapolating the peak 15-minute observed vehicular traffic counts over a forecasted study hour. The “surge” peak hour observed traffic counts were used as the existing traffic volumes in traffic forecasts calculated consistent with future forecasts described in detail in the previously submitted TIA. The forecasts include volumes associated with regional growth, pipeline developments, new site trips, and peak hour rerouting. Additionally, site generated traffic was doubled based on the assumption that all trips entering and exiting the site arrive and leave within a 30-minute period. The doubled site trips provide capacity analyses that assume a “surge” period rather than results averaged over a typical peak hour. Peak hour traffic forecasts from the previously

submitted TIA and the updated analysis provided within this addendum are included in Attachment III. Traffic forecasts used in the updated analysis are shown on Figure 1.

PEAK HOUR FACTORS (PHF)

In order to analyze the peak 15-minute period at study intersections, peak hour factors between 0.85 and 0.92 were used in the previously submitted TIA based on existing peak hour traffic counts. Future conditions included peak hour factors of 0.92 for all movements as included in the signed scoping document. It is noted that a peak hour factor of 0.5 was used for the site driveways to model the spike in traffic before and after services as requested previously by City staff. The “surge” traffic forecasts included in the updated analysis represent the peak 15-minute period at each study location. Therefore, the peak hour factors were also set to 1.0 for the four (4) study intersections.

CAPACITY ANALYSIS

Future peak hour levels of service and 50th and 95th percentile queues with the proposed development are summarized in Tables 2 and 3. The results for the key study intersections and analysis parameters previously described indicate the following:

- The “surge” forecasting would increase average overall delays by approximately five (5) seconds or less at any of the four (4) key study intersections when compared to the previous TIA results.
- All signalized study intersections would continue to operate at acceptable overall LOS “D” or better during Sunday peak hours.
- The northbound left movement at the intersection of Duke Street/S. Patrick Street degrades from LOS “D” to LOS “F” with the “surge” forecasting, adjusted peak hour factors, and removal of the 10 percent non-auto reduction. Additionally, the northbound queue would increase by approximately 200 feet during the peak hour “surge” period.
- Peak hour operations along S. Patrick Street degrade during the “surge” period of the services beginning or ending. The average overall levels of service at the intersections would continue to be acceptable during the average Sunday peak hour.

CONCLUSIONS

Based on the results of the *conservative* supplemental analysis outlined within this addendum including update site trip generation methodology, adjustment of peak hour factors, and updated total future traffic forecasts, the following conclusions are provided:

- Peak 15-minute “surge” operations of the four (4) key study intersections minimally increase delays and queues across the roadway network when compared to the results of the previously submitted TIA.
- All signalized intersections would continue to operate at acceptable overall levels of service during the total future with development conditions.

If you have any questions or require additional information, do not hesitate to contact Larry Sefcik at 703-676-3604 or at lesefcik@wellsandassociates.com or Grady Vaughan at (703) 676-3627 or at gpvaughan@wellsandassociates.com.

O:\PROJECTS\6001-6500\6383 ALFRED BAPTIST CHURCH\DOCUMENTS\REPORTS\ALFRED STREET BAPTIST CHURCH TIA ADDENDUM (W+A SUBMISSION 1.6.17).DOCX

Table 1
Alfred Street Baptist Church
Site Trip Generation Analysis

Land Use	ITE Land Use Code	Size	Units	AM Peak Hour			PM Peak Hour			Weekday ADT ⁽²⁾	Sunday Peak Hour			Sunday ADT
				In	Out	Total	In	Out	Total		In	Out	Total	
Existing ⁽²⁾														
Church	560	43,784	SF	14	11	25	10	10	20	399	369	369	737	2,235
Townhomes	230	1,208	seats	3	12	15	11	6	17	172	27	28	55	106
		22	DU											
Existing Subtotal				17	23	40	21	16	37	571	396	397	792	2,341
Proposed Development														
Church	560	232,368	SF	72	58	130	40	44	84	2,117	660	660	1,320	4,002
		2,163	seats ⁽³⁾											
Total Proposed Site Trips				72	58	130	40	44	84	2,117	660	660	1,320	4,002
NET NEW TRIPS (Proposed vs. Existing)				55	35	90	19	28	47	1,546	265	264	528	1,661

Notes: (1) Traffic estimates based on Institute of Transportation Engineers (ITE) Trip Generation, Ninth Edition.

(2) Existing seat count was quantified using attendance recorded during a typical Sunday service.

(3) Proposed seat number includes both seats in the main sanctuary (1906 seats) and overflow seating in the chapel (257 seats).

Table 2
 Alfred Street Baptist Church
 Total Future with Development Intersection Level of Service Summary ⁽¹⁾

Intersection	Intersection Control	Approach/Movement	Alfred Street Baptist Church TIA Submitted September 23, 2016		Revised Addendum Including Removal of Non-Auto Mode Split and Peak Hour Factor Adjustments	
			Sunday Peak Hour		Sunday Peak Hour	
			LOS	Delay (sec.)	LOS	Delay (sec.)
8. Henry Street/Duke Street	Signalized	EBT	A	6.4	A	5.9
		EBR	A	5.2	A	5.4
		WBL	A	1.4	A	1.5
		WBT	A	0.2	A	0.2
		Overall	A	2.8	A	2.6
9. Patrick Street/Duke Street	Signalized	EBL	B	17.5	B	17.0
		WBR	B	15.7	B	15.1
		NBL	D	38.9	F	49.0
		NBT	C	30.4	D	35.4
		NBR	C	30.4	D	36.6
		Overall	C	27.7	C	32.9
10. Alfred Street/Duke Street	Signalized	EBLTR	B	12.9	B	13.4
		WBLTR	A	1.9	A	3.2
		NBLTR	B	18.4	B	17.5
		SBLTR	C	21.7	C	21.9
		Overall	B	10.1	B	10.2
15. Patrick Street/Gibbon Street *HCM 2010 analysis is unavailable for the existing intersection geometry at this intersection.	Signalized	WBL	C	29.7	C	31.2
		WBLTR	C	20.3	C	20.5
		NBLT	B	16.6	B	19.8
		SBTR	B	12.2	B	13.7
		Overall	B	15.8	B	17.8

Notes:
 (1) Capacity analysis based on Highway Capacity Manual 2010 methodology, using Synchro 9.1 unless otherwise noted.

Table 3
Alfred Street Baptist Church

Total Future with Development Intersection Queue Summary ⁽¹⁾		Intersection Control	Approach/ Movement	Storage Length (ft)	Alfred Street Baptist Church TIA Submitted September 23, 2016		Revised Addendum Including Removal of Non-Auto Mode Split and Peak Hour Factor Adjustments	
Intersection	Signalized				Sunday Peak Hour	Sunday Peak Hour	50th	95th
8. Henry Street/Duke Street	Signalized	EBT EBR WBL WBT SBLTR*	- 125 - - -	175 88 63 98 25	#316 131 m67 m104 32	140 100 74 114 18	#229 147 m78 m119 24	
9. Patrick Street/Duke Street	Signalized	EBT WBTR NBLTR	- - -	89 132 318	m137 #555 383	66 131 ~363	151 #549 #564	
10. Alfred Street/Duke Street	Signalized	EBLTR WBLTR NBLTR SBLTR	- - - -	80 88 28 35	m65 179 61 77	m109 422 31 87	245 227 398 348	
15. Patrick Street/Gibbon Street	Signalized	WBL WBT NBLT SBTR	- - - -	143 79 272 399	301 114 437 #559	151 84 305 452	316 120 #586 #561	

Notes:

- (1) Queue length is based on the 50th and 95th percentile queues in feet as reported by Synchro, Version 9.
- (2) "e" - 50th percentile volume exceeds capacity, queue may be longer than shown.
- (3) "#" - 95th percentile volume exceeds capacity, queue may be longer than shown.
- (4) "m" - Volume for 95th percentile queue is metered by upstream signal.

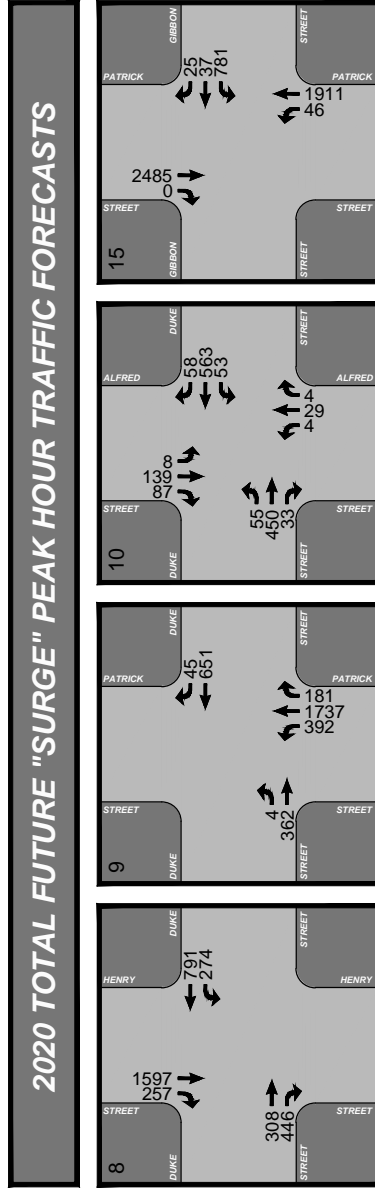
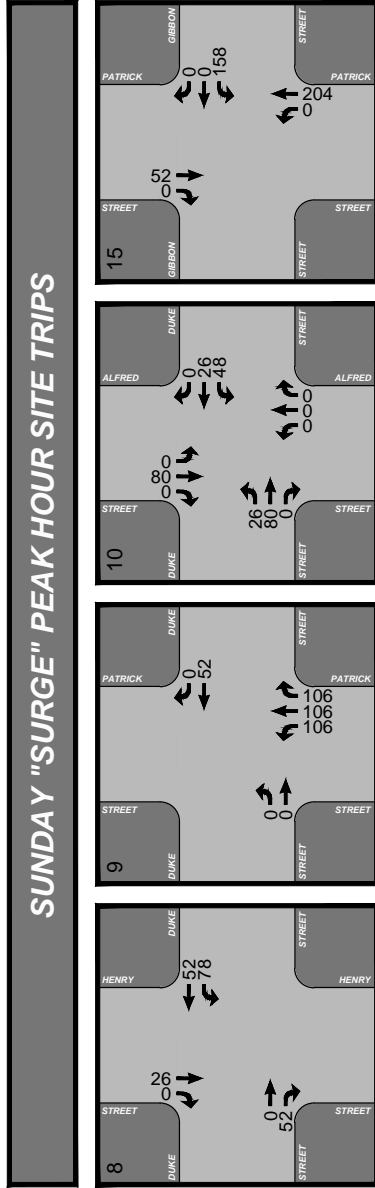
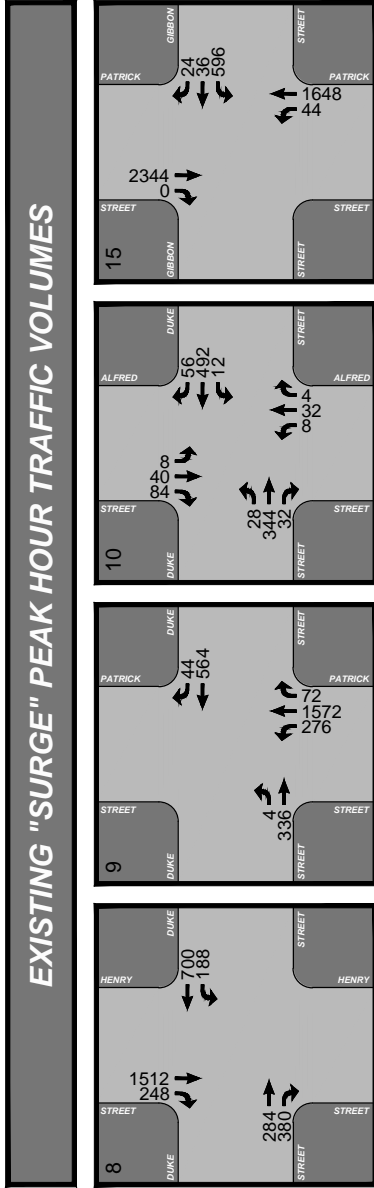


Figure 1
"Surge" Peak Hour Traffic Forecasts

Allred Street Baptist Church
City of Alexandria, Virginia

ATTACHMENT I

Constant Contact Survey Results

Survey Name: NEW Transportation Survey June 4 2016 Survey

Response Status: Partial & Completed

Filter: None

6/8/2016 6:53 AM EDT

*What primary means of transportation do you usually use to arrive at Alfred Street Baptist Church?

Answer	0%	100%	Number of Response(s)	Response Ratio
Drive			486	91.3 %
Ride with someone			13	2.4 %
Bike (Bikeshare)			1	<1 %
Use Public Transportation			14	2.6 %
Taxi, Uber, Lyft (similar service)			3	<1 %
Walk			3	<1 %
Other			6	1.1 %
No Response(s)			6	1.1 %
Totals			532	100%

ATTACHMENT II

ASBC Demographics

6750 members total

780 don't live in District Maryland and Virginia (“DMV”)

The average attendance is @3400 over the entire weekend spread over four services and @6000 live in the DMV area and

61% of the members that live in the DMV live in Virginia.

- 50% of the members that live in Virginia live in Alexandria.

25% of the members that live in the DMV live in Maryland

12% of the members that live in the DMV live in the District

1300 watch services on line very week - not related to members or non-members

ATTACHMENT III

From Alfred Street Baptist Church TIA dated September 23, 2016

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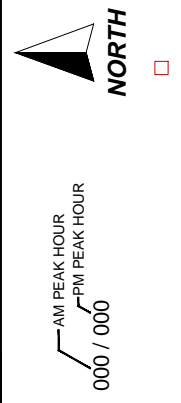
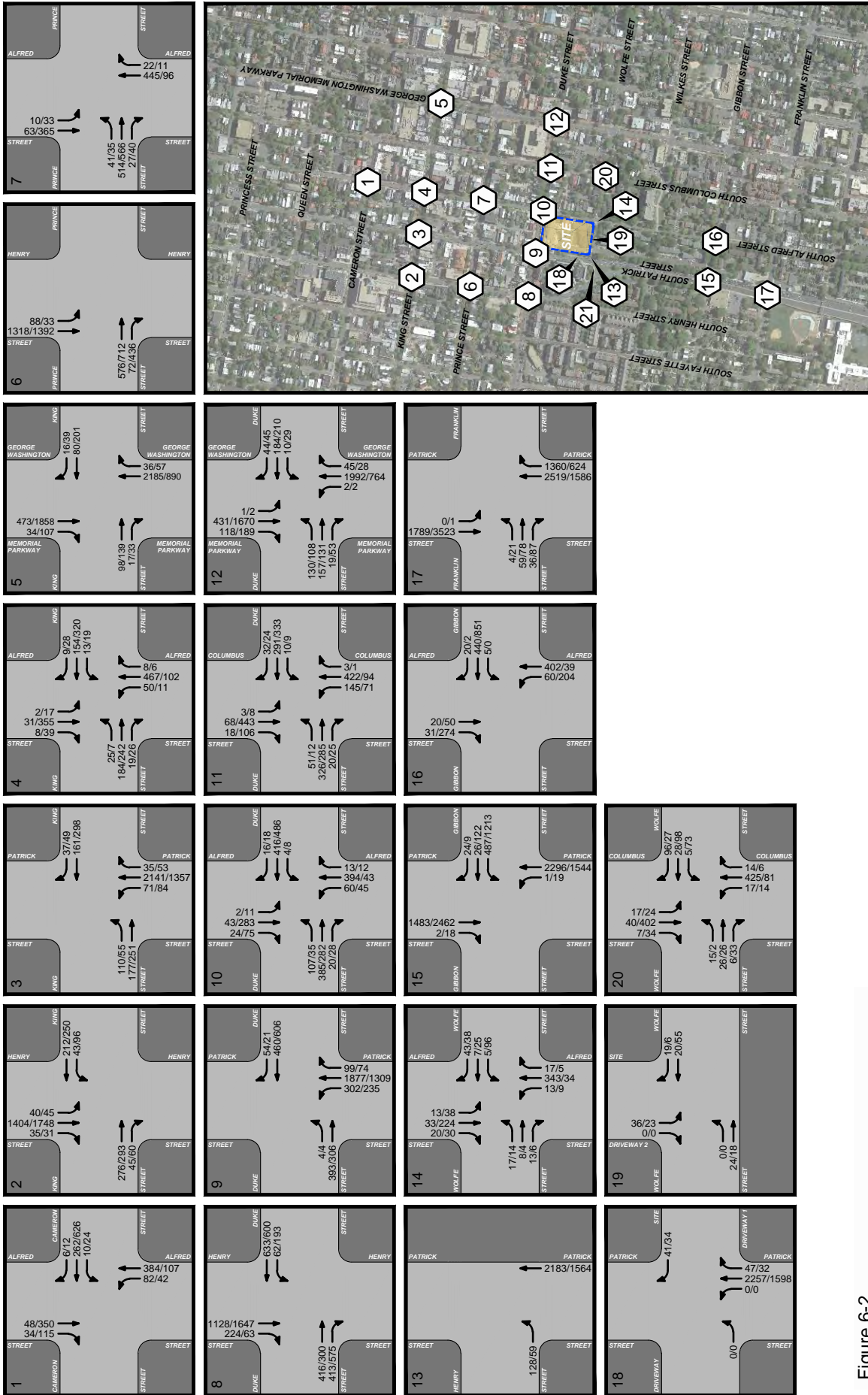


Figure 6-2
Future Peak Hour Traffic Forecasts
With Development (2022) - Weekday
Alfred Street Baptist Church
City of Alexandria, Virginia

ATTACHMENT III

From Alfred Street Baptist Church TIA dated September 23, 2016

Transportation Consultants ■ INNOVATION + SOLUTIONS

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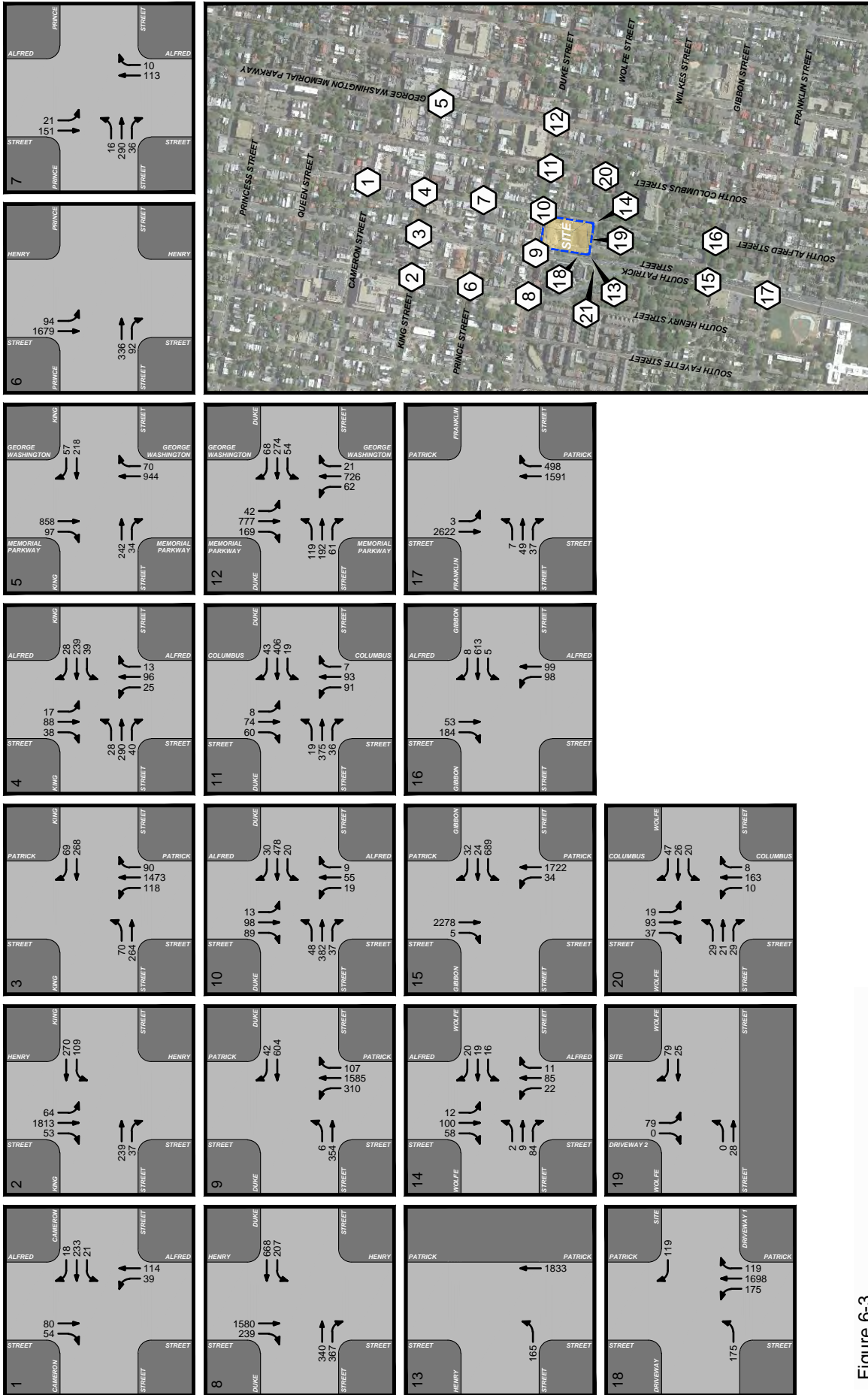


Figure 6-3
Future Peak Hour Traffic Forecasts
With Development (2022) - Sunday
 Alfred Street Baptist Church
 City of Alexandria, Virginia

ATTACHMENT III

Queues

8: Henry St & Duke St

Total Future Sunday w/ Development 2022



Lane Group	EBT	EBR	WBL	WBT	SBT
Lane Group Flow (vph)	308	446	274	791	1854
v/c Ratio	0.69	0.64	0.82	0.58	0.82
Control Delay	36.5	32.1	21.4	14.6	3.8
Queue Delay	0.0	0.0	0.0	1.4	0.1
Total Delay	36.5	32.1	21.4	16.1	4.0
Queue Length 50th (ft)	140	100	74	114	18
Queue Length 95th (ft)	#229	147	m78	m119	24
Internal Link Dist (ft)	72			232	350
Turn Bay Length (ft)		125			
Base Capacity (vph)	449	697	333	1357	2251
Starvation Cap Reductn	0	0	0	355	37
Spillback Cap Reductn	0	0	0	0	0
Storage Cap Reductn	0	0	0	0	0
Reduced v/c Ratio	0.69	0.64	0.82	0.79	0.84

Intersection Summary

95th percentile volume exceeds capacity, queue may be longer.













Queue shown is maximum after two cycles.

m Volume for 95th percentile queue is metered by upstream signal.

ATTACHMENT III

HCM 2010 Signalized Intersection Summary 8: Henry St & Duke St

Total Future Sunday w/ Development 2022

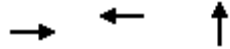
												
Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations		↑	↑↑	↑	↑↑						↑↑↑	
Traffic Volume (veh/h)	0	308	446	274	791	0	0	0	0	0	1597	257
Future Volume (veh/h)	0	308	446	274	791	0	0	0	0	0	1597	257
Number	3	8	18	7	4	14				5	2	12
Initial Q (Qb), veh	0	0	0	0	0	0				0	0	0
Ped-Bike Adj(A_pbT)	1.00		0.97	1.00		1.00				1.00		1.00
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00				1.00	1.00	1.00
Adj Sat Flow, veh/h/ln	0	1863	1863	1863	1863	0				0	1845	1900
Adj Flow Rate, veh/h	0	308	446	274	791	0				0	1597	257
Adj No. of Lanes	0	1	2	1	2	0				0	3	0
Peak Hour Factor	1.00	1.00	1.00	1.00	1.00	1.00				1.00	1.00	1.00
Percent Heavy Veh, %	0	2	2	2	2	0				0	3	0
Cap, veh/h	0	972	1607	961	3157	0				0	0	0
Arrive On Green	0.00	0.52	0.52	0.52	1.00	0.00				0.00	0.01	0.00
Sat Flow, veh/h	0	1863	3080	1774	3632	0					0	
Grp Volume(v), veh/h	0	308	446	274	791	0					0.0	
Grp Sat Flow(s),veh/h/ln	0	1863	1540	1774	1770	0						
Q Serve(g_s), s	0.0	3.5	3.0	0.9	0.0	0.0						
Cycle Q Clear(g_c), s	0.0	3.5	3.0	0.9	0.0	0.0						
Prop In Lane	0.00		1.00	1.00		0.00						
Lane Grp Cap(c), veh/h	0	972	1607	961	3157	0						
V/C Ratio(X)	0.00	0.32	0.28	0.29	0.25	0.00						
Avail Cap(c_a), veh/h	0	972	1607	961	3157	0						
HCM Platoon Ratio	1.00	1.00	1.00	2.00	2.00	1.00						
Upstream Filter(I)	0.00	1.00	1.00	1.00	1.00	0.00						
Uniform Delay (d), s/veh	0.0	5.1	5.0	0.7	0.0	0.0						
Incr Delay (d2), s/veh	0.0	0.9	0.4	0.7	0.2	0.0						
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0						
%ile BackOfQ(50%),veh/ln	0.0	2.0	1.3	0.5	0.1	0.0						
LnGrp Delay(d),s/veh	0.0	5.9	5.4	1.5	0.2	0.0						
LnGrp LOS		A	A	A	A							
Approach Vol, veh/h		754			1065							
Approach Delay, s/veh		5.6			0.5							
Approach LOS		A			A							
Timer	1	2	3	4	5	6	7	8				
Assigned Phs				4			7	8				
Phs Duration (G+Y+Rc), s				37.0			13.7	23.3				
Change Period (Y+Rc), s				* 5.3			* 5.3	* 5.3				
Max Green Setting (Gmax), s				* 32			* 8.4	* 18				
Max Q Clear Time (g_c+I1), s				2.0			2.9	5.5				
Green Ext Time (p_c), s				9.6			0.5	6.4				
Intersection Summary												
HCM 2010 Ctrl Delay			2.6									
HCM 2010 LOS			A									
Notes												

ATTACHMENT III

Queues

9: Patrick St & Duke St

Total Future Sunday w/ Development 2022



Lane Group	EBT	WBT	NBT
Lane Group Flow (vph)	366	696	2310
v/c Ratio	0.40	0.97	1.01
Control Delay	15.9	36.5	37.8
Queue Delay	0.7	10.8	0.0
Total Delay	16.6	47.3	37.8
Queue Length 50th (ft)	66	131	~363
Queue Length 95th (ft)	151	#549	#564
Internal Link Dist (ft)	232	245	252
Turn Bay Length (ft)			
Base Capacity (vph)	908	716	2286
Starvation Cap Reductn	268	44	0
Spillback Cap Reductn	58	0	0
Storage Cap Reductn	0	0	0
Reduced v/c Ratio	0.57	1.04	1.01

Intersection Summary
















- ~ Volume exceeds capacity, queue is theoretically infinite.
Queue shown is maximum after two cycles.
- # 95th percentile volume exceeds capacity, queue may be longer.
Queue shown is maximum after two cycles.

ATTACHMENT III

HCM 2010 Signalized Intersection Summary

9: Patrick St & Duke St

Total Future Sunday w/ Development 2022

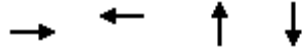
												
Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (veh/h)	4	362	0	0	651	45	392	1737	181	0	0	0
Future Volume (veh/h)	4	362	0	0	651	45	392	1737	181	0	0	0
Number	7	4	14	3	8	18	5	2	12			
Initial Q (Qb), veh	0	0	0	0	0	0	0	0	0			
Ped-Bike Adj(A_pbT)	1.00		1.00	1.00		0.98	1.00		0.99			
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00			
Adj Sat Flow, veh/h/ln	1900	1937	0	0	1863	1900	1900	1845	1900			
Adj Flow Rate, veh/h	4	362	0	0	651	45	392	1737	181			
Adj No. of Lanes	0	1	0	0	1	0	0	3	0			
Peak Hour Factor	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00			
Percent Heavy Veh, %	2	2	0	0	2	2	0	3	0			
Cap, veh/h	48	842	0	0	752	52	387	1843	195			
Arrive On Green	0.44	0.44	0.00	0.00	0.88	0.85	0.46	0.46	0.45			
Sat Flow, veh/h	5	1924	0	0	1720	119	837	3985	422			
Grp Volume(v), veh/h	366	0	0	0	0	696	844	710	757			
Grp Sat Flow(s),veh/h/ln	1929	0	0	0	0	1839	1803	1679	1763			
Q Serve(g_s), s	0.0	0.0	0.0	0.0	0.0	15.7	37.0	31.5	32.3			
Cycle Q Clear(g_c), s	10.5	0.0	0.0	0.0	0.0	15.7	37.0	31.5	32.3			
Prop In Lane	0.01		0.00	0.00		0.06	0.46		0.24			
Lane Grp Cap(c), veh/h	890	0	0	0	0	804	834	776	815			
V/C Ratio(X)	0.41	0.00	0.00	0.00	0.00	0.87	1.01	0.91	0.93			
Avail Cap(c_a), veh/h	890	0	0	0	0	804	834	776	815			
HCM Platoon Ratio	1.00	1.00	1.00	1.00	2.00	2.00	1.00	1.00	1.00			
Upstream Filter(I)	1.00	0.00	0.00	0.00	0.00	1.00	1.00	1.00	1.00			
Uniform Delay (d), s/veh	15.6	0.0	0.0	0.0	0.0	3.9	21.5	20.0	20.4			
Incr Delay (d2), s/veh	1.4	0.0	0.0	0.0	0.0	11.2	27.5	15.4	16.2			
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0			
%ile BackOfQ(50%),veh/ln	5.9	0.0	0.0	0.0	0.0	9.1	24.7	17.7	19.2			
LnGrp Delay(d),s/veh	17.0	0.0	0.0	0.0	0.0	15.1	49.0	35.4	36.6			
LnGrp LOS	B					B	F	D	D			
Approach Vol, veh/h		366			696			2310				
Approach Delay, s/veh		17.0			15.1			40.8				
Approach LOS		B			B			D				
Timer	1	2	3	4	5	6	7	8				
Assigned Phs		2		4				8				
Phs Duration (G+Y+Rc), s		41.0		39.0				39.0				
Change Period (Y+Rc), s		5.0		* 5.2				* 5.2				
Max Green Setting (Gmax), s		36.0		* 34				* 34				
Max Q Clear Time (g_c+I1), s		39.0		12.5				17.7				
Green Ext Time (p_c), s		0.0		5.3				4.8				
Intersection Summary												
HCM 2010 Ctrl Delay			32.9									
HCM 2010 LOS			C									
Notes												

ATTACHMENT III

Queues

10: Alfred St & Duke St

Total Future Sunday w/ Development 2022



Lane Group	EBT	WBT	NBT	SBT
Lane Group Flow (vph)	538	674	37	234
v/c Ratio	0.64	0.76	0.07	0.47
Control Delay	13.1	17.1	16.5	16.4
Queue Delay	0.0	1.7	0.0	0.3
Total Delay	13.1	18.8	16.5	16.7
Queue Length 50th (ft)	121	251	11	52
Queue Length 95th (ft)	m109	422	31	87
Internal Link Dist (ft)	245	227	398	348
Turn Bay Length (ft)				
Base Capacity (vph)	844	882	525	495
Starvation Cap Reductn	2	52	0	0
Spillback Cap Reductn	0	91	0	45
Storage Cap Reductn	0	0	0	0
Reduced v/c Ratio	0.64	0.85	0.07	0.52

















Intersection Summary

m Volume for 95th percentile queue is metered by upstream signal.

ATTACHMENT III

HCM 2010 Signalized Intersection Summary 10: Alfred St & Duke St

Total Future Sunday w/ Development 2022

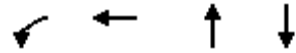
												
Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (veh/h)	55	450	33	53	563	58	4	29	4	8	139	87
Future Volume (veh/h)	55	450	33	53	563	58	4	29	4	8	139	87
Number	5	2	12	1	6	16	3	8	18	7	4	14
Initial Q (Qb), veh	0	0	0	0	0	0	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	0.95		0.86	0.99		0.86	0.92		0.79	0.83		0.81
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Adj Sat Flow, veh/h/ln	1900	1937	1900	1900	1937	1900	1900	1863	1900	1900	1863	1900
Adj Flow Rate, veh/h	55	450	33	53	563	58	4	29	4	8	139	87
Adj No. of Lanes	0	1	0	0	1	0	0	1	0	0	1	0
Peak Hour Factor	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Percent Heavy Veh, %	2	2	2	2	2	2	2	2	2	2	2	2
Cap, veh/h	113	831	58	99	845	84	83	498	64	53	341	206
Arrive On Green	0.55	0.55	0.54	1.00	1.00	1.00	0.35	0.35	0.34	0.35	0.35	0.34
Sat Flow, veh/h	114	1511	106	91	1537	153	94	1424	184	19	975	588
Grp Volume(v), veh/h	538	0	0	674	0	0	37	0	0	234	0	0
Grp Sat Flow(s),veh/h/ln	1731	0	0	1781	0	0	1701	0	0	1582	0	0
Q Serve(g_s), s	0.1	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Cycle Q Clear(g_c), s	14.3	0.0	0.0	0.0	0.0	0.0	1.1	0.0	0.0	9.0	0.0	0.0
Prop In Lane	0.10		0.06	0.08		0.09	0.11		0.11	0.03		0.37
Lane Grp Cap(c), veh/h	1002	0	0	1028	0	0	645	0	0	600	0	0
V/C Ratio(X)	0.54	0.00	0.00	0.66	0.00	0.00	0.06	0.00	0.00	0.39	0.00	0.00
Avail Cap(c_a), veh/h	1002	0	0	1028	0	0	645	0	0	600	0	0
HCM Platoon Ratio	1.00	1.00	1.00	2.00	2.00	2.00	1.00	1.00	1.00	1.00	1.00	1.00
Upstream Filter(I)	1.00	0.00	0.00	1.00	0.00	0.00	1.00	0.00	0.00	1.00	0.00	0.00
Uniform Delay (d), s/veh	11.3	0.0	0.0	0.0	0.0	0.0	17.3	0.0	0.0	20.0	0.0	0.0
Incr Delay (d2), s/veh	2.1	0.0	0.0	3.2	0.0	0.0	0.2	0.0	0.0	1.9	0.0	0.0
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	7.9	0.0	0.0	0.9	0.0	0.0	0.6	0.0	0.0	4.2	0.0	0.0
LnGrp Delay(d),s/veh	13.4	0.0	0.0	3.2	0.0	0.0	17.5	0.0	0.0	21.9	0.0	0.0
LnGrp LOS	B			A			B			C		
Approach Vol, veh/h		538			674			37				234
Approach Delay, s/veh		13.4			3.2			17.5				21.9
Approach LOS		B			A			B				C
Timer	1	2	3	4	5	6	7	8				
Assigned Phs		2		4		6		8				
Phs Duration (G+Y+Rc), s		48.0		32.0		48.0		32.0				
Change Period (Y+Rc), s		5.0		5.0		5.0		5.0				
Max Green Setting (Gmax), s		43.0		27.0		43.0		27.0				
Max Q Clear Time (g_c+I1), s		16.3		11.0		2.0		3.1				
Green Ext Time (p_c), s		7.2		1.1		7.8		1.2				
Intersection Summary												
HCM 2010 Ctrl Delay				10.2								
HCM 2010 LOS				B								

ATTACHMENT III

Queues

15: Patrick St & Gibbon St

Total Future Sunday w/ Development 2022



Lane Group	WBL	WBT	NBT	SBT
Lane Group Flow (vph)	390	453	1957	2485
v/c Ratio	0.80	0.86dl	0.89	0.84
Control Delay	34.6	21.1	21.5	15.2
Queue Delay	0.8	0.0	0.0	15.8
Total Delay	35.4	21.1	21.5	31.0
Queue Length 50th (ft)	151	84	305	452
Queue Length 95th (ft)	316	120	#586	#561
Internal Link Dist (ft)		273	346	346
Turn Bay Length (ft)				
Base Capacity (vph)	558	1059	2190	2943
Starvation Cap Reductn	38	0	0	558
Spillback Cap Reductn	0	0	0	159
Storage Cap Reductn	0	0	0	0
Reduced v/c Ratio	0.75	0.43	0.89	1.04

Intersection Summary

95th percentile volume exceeds capacity, queue may be longer.

Queue shown is maximum after two cycles.

dl Defacto Left Lane. Recode with 1 though lane as a left lane.

ATTACHMENT III

HCM Signalized Intersection Capacity Analysis

15: Patrick St & Gibbon St

Total Future Sunday w/ Development 2022



Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR	
Lane Configurations				↖	↔			↗			↗	↔	
Traffic Volume (vph)	0	0	0	781	37	25	46	1911	0	0	2485	0	
Future Volume (vph)	0	0	0	781	37	25	46	1911	0	0	2485	0	
Ideal Flow (vphpl)	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	
Lane Width	12	12	12	11	11	11	12	12	12	12	12	12	
Grade (%)		0%			2%			0%			0%		
Total Lost time (s)				4.0	4.0			4.0			4.0		
Lane Util. Factor				0.91	0.91			0.91			0.91		
Frpb, ped/bikes				1.00	1.00			1.00			1.00		
Flpb, ped/bikes				1.00	1.00			1.00			1.00		
Frt				1.00	0.99			1.00			1.00		
Flt Protected				0.95	0.96			1.00			1.00		
Satd. Flow (prot)				1541	2908			5030			5036		
Flt Permitted				0.95	0.96			0.74			1.00		
Satd. Flow (perm)				1541	2908			3747			5036		
Peak-hour factor, PHF	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	
Adj. Flow (vph)	0	0	0	781	37	25	46	1911	0	0	2485	0	
RTOR Reduction (vph)	0	0	0	0	5	0	0	0	0	0	0	0	
Lane Group Flow (vph)	0	0	0	390	448	0	0	1957	0	0	2485	0	
Confl. Peds. (#/hr)	3		2	2			3						
Confl. Bikes (#/hr)							2						
Heavy Vehicles (%)	2%	2%	2%	2%	2%	2%	3%	3%	3%	3%	3%	3%	
Bus Blockages (#/hr)	0	0	0	0	2	0	0	0	0	0	0	0	
Parking (#/hr)					1								
Turn Type				Split	NA		Perm	NA			NA		
Protected Phases				8	8			2			6		
Permitted Phases							2						
Actuated Green, G (s)				23.2	23.2			45.3			45.3		
Effective Green, g (s)				25.2	25.2			46.8			46.8		
Actuated g/C Ratio				0.31	0.31			0.58			0.58		
Clearance Time (s)				6.0	6.0			5.5			5.5		
Vehicle Extension (s)				2.0	2.0			2.0			2.0		
Lane Grp Cap (vph)				485	916			2191			2946		
v/s Ratio Prot				c0.25	0.15						0.49		
v/s Ratio Perm								c0.52					
v/c Ratio				0.80	0.86dl			0.89			0.84		
Uniform Delay, d1				25.1	22.2			14.4			13.6		
Progression Factor				0.93	0.92			0.99			0.84		
Incremental Delay, d2				7.7	0.1			5.5			2.3		
Delay (s)				31.2	20.5			19.8			13.7		
Level of Service				C	C			B			B		
Approach Delay (s)		0.0			25.4			19.8			13.7		
Approach LOS		A			C			B			B		
Intersection Summary													
HCM 2000 Control Delay			17.8		HCM 2000 Level of Service						B		
HCM 2000 Volume to Capacity ratio			0.86										
Actuated Cycle Length (s)			80.0		Sum of lost time (s)						8.0		
Intersection Capacity Utilization			104.8%		ICU Level of Service						G		