Bowman
CONSULTING

Traffic Impact Analysis
Rappahannock Landing Apartments
Stafford County, Virginia

Prepared for:
The Breeden Company

March 30, 2018

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Table of Contents

List of Figures

List of Tables

Appendixes

Executive Summary

- The purpose of this study is to determine the impact to the existing roadway network in the vicinity of the proposed Rappahannock Landing Apartments. This proposed development will consists of 324 apartment units.

- The overall Rappahannock Landing Development currently consists of the following sections:

    - Section 1 (Condos) - This project was approved in 2010 and the construction has been completed.
    - Section 2 (Townhomes) - This project was approved in 2015 and it is currently under construction.
    - Section 3 (Townhomes) - This project was approved in 2015 and it is currently under construction.
    - Section 4 (Townhomes) - This project was approved in 2017 and it is currently under construction.
    - Section 5 (Apartments) - This project is currently under development.

- The following intersections were evaluated in this Traffic Impact Assessment:

    1. Warrenton Road and Short Street (Signalized Intersection)
    2. Warrenton Road and Olde Forge Drive (Unsignalized Intersection)
    3. Warrenton Road and Solomon Drive/Lendall Lane

- The proposed development is expected to generate 108 trips (28 in and 80 out) during the morning peak hour and 137 trips (84 in and 53 out) during the evening peak hour.

- For purposes of this analysis, it is anticipated that the proposed Rappahannock Landing Apartments will be constructed by the year 2022. The following scenarios will be evaluated as part of this study:

    - Existing conditions (2018)
    - The build out year without the proposed development (background) (2022 No Build)
    - The build out year with the proposed development (2022 Build)
    - The build out year without the proposed development (background) (2028 No Build)
    - The build out year with the proposed development (2028 Build)

- VDOT has currently completed a corridor improvement study for US Route 17 using the STARS program. The project limits are from I-95 to Route 1001 (Washington Street). The study provides recommendations to alleviate traffic congestion and improve safety along the corridor. General information from this study will be included and evaluated as part of this analysis.

- The VDOT US Route 17 corridor improvement study mentioned above includes improvements to the Warrenton Road/Short Street and Warrenton Road/Olde Forge Drive intersections. For the purpose of this study it was assumed that the existing traffic signal at the intersection of Warrenton Road/Short Street will be relocated to the intersection of Warrenton Road/Olde Forge Drive by the build out year (2022). It was also assumed that a raised median will be installed along Warrenton Road within the intersection of Warrenton Road/Short Street by the build out year (2022). Both of these improvements are discussed in the VDOT US Route 17 corridor improvement study.

- The proposed Rappahannock Landing Apartments Development was found to present minor increases of delay and/or changes in Levels of Service on the existing roadway network during the 2022 & 2028 no Build and Build conditions.

- The minor increases and/or changes can be mitigated by adjusting/optimizing the traffic signal timings to accommodate future traffic demand. Traffic signal timing evaluations are recommended once the proposed development is constructed by the year 2022.

- It is also recommended to coordinate with VDOT regarding the progress and timetable for the proposed improvements along Warrenton Road.

- The proposed Rappahannock Landing Apartments Development is not expected to adversely impact the existing roadway network with or without the proposed VDOT improvements.

Introduction

This report summarizes the findings of the traffic impact analysis performed by Bowman Consulting Group (BCG) for the proposed development (Rappahannock Landing Apartments) located in Stafford County, Virginia on Tax Map 53, Parcel 1E. The site consists of 25.53 acres and is currently zoned R-1. The developer is seeking rezoning to UD-3 in order to develop 324 apartment units.

The purpose of this analysis is to determine the potential impact (if any) to the existing traffic operations within the surrounding roadway network caused by the proposed development. This analysis has been coordinated with VDOT and Stafford County. The Pre-Scope of Work Meeting Form which includes the agreement of the major components of this study can be found in **Appendix A**.

Background Information

The proposed site is located on the south east quadrant of the I-95/US 17 interchange. The proposed Rappahannock Landing Apartments is one of the sections for the overall Rappahannock Landing Development in this quadrant. **Figure 1**, depicts the site location.

Figure 1. Site location.

The overall Rappahannock Landing Development currently consists of the following sections:

- Section 1 (Condos) - This project was approved in 2010 and the construction has been completed.
- Section 2 (Townhomes) - This project was approved in 2015 and it is currently under construction.
- Section 3 (Townhomes) - This project was approved in 2015 and it is currently under construction.
- Section 4 (Townhomes) - This project was approved in 2017 and it is currently under construction.
- Section 5 (Apartments) - This project is currently under development.

Access to the site is currently provided via two existing local roadways (Short Street & Olde Forge Drive) that intersect Warrenton Road (US 17). **Figure 2** depicts the existing access points for the proposed site.

Figure 2. Site access points.

For purposes of this analysis, the following three (3) intersections will be analyzed in this report:

1. Warrenton Road (Route 17) and Short Street/Hotel Entrance (Signalized Intersection)
2. Warrenton Road (Route 17) and Olde Forge Drive (Unsignalized Intersection)
3. Warrenton Road (Route 17) and Solomon Drive/Lendall Lane (Signalized Intersection)

To assess the traffic operations at these intersections, the following tasks were undertaken:

- Field inspections were conducted to obtain an inventory of existing roadway geometry, traffic control, and location of adjacent intersections.
- Turning movement counts were obtained from VDOT for the morning and evening peak periods. These counts will be used to identify peak hours, determine traffic patterns, and evaluate intersection Levels of Service.
- Capacity analyses were prepared to determine existing and projected Levels of Service (LOS) and maximum queue lengths.

Existing Roadway Network

Warrenton Road (US 17) within the identified study area is a four-lane undivided roadway with a mix of auxiliary lanes and a center two-way left turn lane. Route 17 is identified as a principle arterial roadway on VDOT's 2014 Functional Classification Map. It has an east-west alignment within the study area with a posted speed limit of 45 miles per hour.

Short Street (Route 1034) within the identified study area is a two-lane undivided roadway identified as a major collector roadway on VDOT's 2014 Functional Classification Map. Short Street has a north-south alignment with the intersection of Route 17 and an east-west alignment with the intersection of Musselman Road. Short Street has a posted speed limit of 25 miles per hour.

Olde Forge Drive (Route 1580) within the identified study area is a two-lane undivided roadway identified as a local street on VDOT's 2014 Functional Classification Map. Olde Foge Drive has a north-south alignment with a posted speed limit of 25 miles per hour. It is median divided only at the intersection with Warrenton Road (Route 17).

Solomon Drive (Route 1001) within the identified study area is a two-lane undivided roadway identified as a local street on VDOT's 2014 Functional Classification Map. Solomon Drive has a north-south alignment and does not have a posted speed limit. Solomon Drive is a newly constructed roadway serving parcels which are largely undeveloped.

Lendall Lane (Route 1015) within the identified study area is a two-lane undivided roadway identified as a local street on VDOT's 2014 Functional Classification Map. Lendall Lane has a north-south alignment with a posted speed limit of 30 miles per hour.

Existing Intersection Characteristics

Intersection of Warrenton Road (US 17) and Short Street/Hotel Entrance

This intersection is currently a four legged signalized intersection where Short Street has a north-south alignment and Warrenton Road (Route 17) has an east-west alignment as shown in **Figure 3**. The fourth leg of this intersection is a private parking lot entrance which serves a hotel.

Figure 3. Aerial of Warrenton Road & Short Street/Hotel Entrance

The eastbound approach consists of one shared left-through lane, one through lane and one exclusive right turn lane with 150 feet of storage. The northbound approach consists of one shared left-through lane and one exclusive right turn lane with 150 feet of storage. The westbound approach consists of one exclusive left turn lane with 175 feet of storage, one through lane and one shared through-right turn lane. The southbound approach consists of one shared left-through-right lane.

Intersection of Warrenton Road (US 17) and Olde Forge Drive

This intersection is currently a three legged unsignalized intersection where Olde Forge Drive has a north-south alignment and Warrenton Road (Route 17) has an east-west alignment as shown in **Figure 4**. Olde Forge Drive operates as stop controlled while Warrenton Road (Route 17) is free flowing.

Figure 4. Aerial of Warrenton Road & Olde Forge Drive

The eastbound approach consists of a center dual left turn lane, a through lane and a shared through-right turn lane. The northbound approach consists of a divided median with one right turn lane and one left turn lane. The westbound approach consists of a center dual left turn lane and two through lanes. The outer most through lane also serves as a way for motorists to access private driveways to the north of Warrenton Road as a right turn lane is not provided. Note that there is a private driveway which provides a fourth leg to this intersection. This private driveway is not included in this analysis as the existing traffic volumes for this driveway are minimal.

<u>Intersection of Warrenton Road (US 17) and Solomon Drive/Lendall Lane</u>

This intersection is currently a four legged signalized intersection where Solomon Drive and Lendall Lane have a north-south alignment and Warrenton Road (Route 17) has an east-west alignment as shown in **Figure 5**.

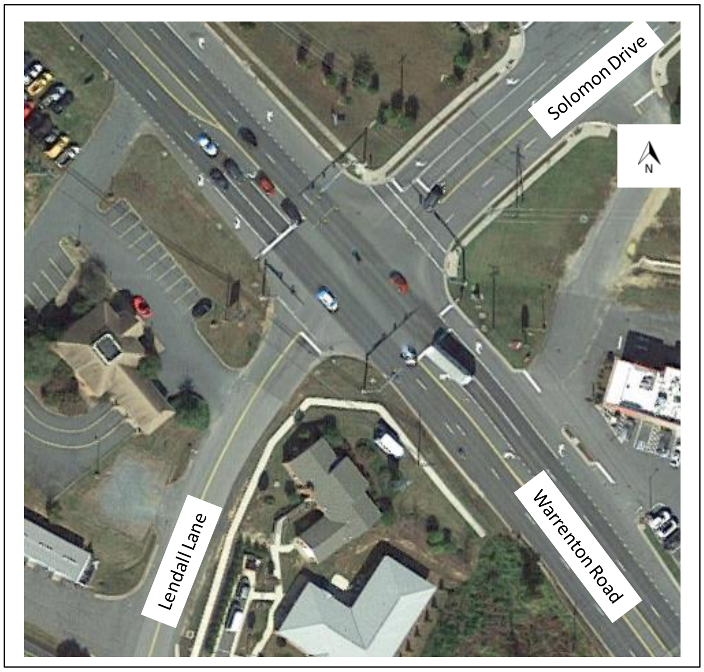


Figure 5. Aerial of Warrenton Road & Solomon Drive/Lendall Lane

The eastbound approach consists of an exclusive left turn lane with 50 feet of storage and a 75-foot taper, two through lanes and an exclusive right turn lane with 50 feet of storage and a 120 foot taper. The northbound approach consists one shared left-through-right lane. The westbound approach consists of one exclusive left turn lane with 100 feet of storage and a 100-foot taper, two through lanes and one exclusive right turn lane with 325 feet of storage. The southbound approach consists of one exclusive left turn lane with 140 feet of storage and 100-foot taper, one through lane and one exclusive right turn lane with 140 feet of storage.

Data Collection

Field inspections were conducted to obtain an inventory of existing roadway geometry, traffic control, and location of adjacent intersections.

Turning movement counts for the morning and evening peak periods were provided by VDOT for the US Route 17 corridor. These counts were used to identify peak hours, determine traffic patterns, and evaluate intersection Levels of Service. The 2017 traffic counts are included in **Appendix B**.

Information from VDOT

VDOT has completed a corridor study along US 17 from I-95 to Route 1 through the STARS program. The program objective is to develop solutions to improve traffic congestion and safety issues that can be programmed in the VDOT Six-Year Improvement Plan.

The results of the study were presented to the public at a Citizen Information Meeting on May 10, 2017. The improvements presented still need to be prioritized and the study needs to be finalized.

The study is proposing the following improvements along Warrenton Road (Route 17):

Intersection of Warrenton Road and Short Street

- Install a raised median to prohibit left turns from the side streets.
- Remove existing traffic signal

Intersection of Warrenton Road and Olde Forge Drive

- Install a new traffic signal
- Provide dedicated left turn lanes for traffic along Warrenton Road.
- Install a right turn lane to access Olde Forge Drive.
- Realign RV Parkway with Olde Forge Drive.

The study at this point indicates cost estimates and a timeframe for construction (4 years). It is our understanding that the majority of these recommendations will be completed by others (except the realignment of the RV Parkway with Olde Forge Drive).

For purposes of this analysis, it was assumed that the improvements mentioned above for the intersection of Warrenton Road/Short Street will be completed by the build out year (2022). All trip distributions and capacity analyses for years 2022 and 2028 within this report were conducted with these improvements.

The corridor study information can be found in **Appendix C**.

Traffic Forecast and Background Traffic

For purposes of this analysis, it is anticipated that the proposed Rappahannock Landing Apartments 5 will be constructed by the year 2022. The following scenarios will be evaluated as part of this analysis:

- Existing conditions (2018)
- The build out year without the proposed development (background) (2022 No Build)
- The build out year with the proposed development (2022 Build)
- The build out year without the proposed development (background) (2028 No Build)
- The build out year with the proposed development (2028 Build)

Three steps were considered on the traffic forecast (background traffic) evaluation.

The first step was to determine a background growth rate applicable for the area. A growth rate of 2.0% was used for traffic along US Route 17. This growth rate was only applied to through movements along Route 17 as there is no commercial or residential development within this area which this growth rate would apply.

The traffic counts provided by VDOT completed in 2017. These counts were projected to 2018 using a 2% growth rate for through movements along Route 17. The 2018 traffic volumes (Existing) are summarized and depicted in **Exhibit 1** of **Appendix D**. Please note that these volumes have been redistributed to account for the roadway improvements mentioned previously. These newly distributed existing traffic volumes are summarized and depicted in **Exhibit 2** of **Appendix D**.

The 2018 traffic volumes were then projected using the 2% growth rate to the year 2022 and 2028 to develop the No-Build conditions.

The second step was to consider the status of the Rappahannock Landing development at the time the traffic counts were conducted (2017). At this time, the following sections of Rappahannock Landing had been constructed:

- Section 1 – 100% Constructed/Occupied
- Section 2 – 40% Constructed/Occupied
- Section 3 – 40% Constructed/Occupied
- Section 4 – 0% Constructed/Occupied

Since sections 2 through 4 are not 100% constructed/occupied, a trip generation was conducted for these sections to identify trips that were not captured during the traffic counts. The trips generated by background developments as shown in **Table 1** are summarized and depicted in **Exhibit 5** of **Appendix D**.

Table 1 below summarizes the trip generation which has been completed for Rappahannock Landing sections 2-4. With 40% of sections 2 and 3 of Rappahannock Landing having been established prior to when the existing traffic counts were conducted, only 60% of the trips generated by these sections are being added to the existing traffic counts.

Table 1. Trip Generation for Background Developments

Land Use	Size	Units	Land Use Code	Weekday AM Peak Hour In	AM Out	AM Total	PM Peak Hour In	PM Out	PM Total	Average Daily Trips
Rappahannock Landing - Section 2										
Townhomes	131	D.U.	220	14	48	62	47	28	75	949
60% Generated Trips				8	29	37	28	17	45	569
Rappahannock Landing - Section 3										
Townhomes	154	D.U.	220	17	55	72	55	32	87	1,123
60% Generated Trips				10	33	43	33	19	52	674
Rappahannock Landing - Section 4										
Townhomes	276	D.U.	220	29	96	125	92	54	146	2,046
Total Trips From Other Planned Developments (2)				**47**	**158**	**205**	**153**	**90**	**243**	**3,289**

Notes: (1) Based on the Institute of Transportation Engineers Trip Generation, 10th Edition.
(2) Total includes 60% of Rappahannock Landing Section 2 & 3 trip generation.

The arrival/departure trip distribution was prepared based on existing roadway traffic patterns and in coordination with VDOT and the County (See Scope of work). Generally, it is expected that 75% of the traffic will be generated to/from the west of the Rappahannock Landing Site while 25% of the traffic will be generated to/from the east. The arrival/departure rates are depicted on **Exhibit 4 in Appendix D**.

The third step was to identify nearby developments within VDOT's LandTrack System that are projected to be completed within the near future. The following projects and respective traffic impact assessments were found in the system.

- Westlake Traffic Impact Study prepared by Wells + Associates (4/13/17)
- Traffic Analysis For McDonald's at Retail Buildings At Stafford Lakes prepared by DRW Consultants, LLC (11/16/14)
- Proposed Grocery Store on Warrenton Road Traffic Impact Study by Kimley-Horn (9/17/15).

Although these studies are located along the US 17 corridor, they are located west of the I-95/US 17 interchange and would not be applicable for this analysis. Therefore, background traffic from these developments will not be considered in this analysis.

The only background projects applicable for this analysis are Rappahannock Landing Sections 2-4. The traffic forecast for these sections of Rappahannock Landing are accounted for in step two.

The No Build traffic volumes plus the background traffic are depicted on **Exhibits 7 and 9** in **Appendix D**. These volumes create the total no build conditions for years 2022 and 2028.

Proposed Development (Rappahannock Landing Apartments)

The Applicant, Breeden Company, is proposing to develop section 5 of the Rappahannock Landing development with a residential land use. The proposed development will consist of the following land use:

- 324 Apartment Units

Site Trip Generation

A trip generation for the proposed development is shown in **Table 2** and depicted on **Exhibit 6** in **Appendix D**. The Institute of Transportation Engineers (ITE) *Trip Generation Manual, 10th edition* was used to determine the number of trips generated by the land use (Multifamily Housing (Mid-Rise), land use code 221). The average weekday morning and evening peak hour trips were determined along with the average daily trips.

Table 2: Rappahannock Landing Apartments Trip Generation

				Weekday						
				AM Peak Hour			PM Peak Hour			Average Daily Trips
Land Use	Size	Units	Land Use Code	In	Out	Total	In	Out	Total	
Rappahannock Landing - Section 5										
Apartments	324	D.U.	221	28	80	108	84	53	137	1,764
Total Trips				28	80	108	84	53	137	1,764

Notes: (1) Based on the Institute of Transportation Engineers Trip Generation, 10th Edition.

These traffic volumes were then distributed to the roadway system in accordance with a site traffic distribution pattern developed based on a review of traffic patterns in the area. For purposes of this study, it was determined that 75% of the site generated trips will travel to/from the west along Warrenton Road with 25% of the site generated trips traveling to/from the east.

With Olde Forge Drive serving as the main entrance for the Rappahannock Landing Apartments development, it was determined that 95% of the traffic leaving the development will use Olde Forge Drive (future signalized intersection) to access Warrenton Road. These trips will access Olde Forge Drive by navigating through previously developed sections of Rappahannock Landing. The remaining 5% of the traffic leaving the development will use Short Street (future unsignalized intersection) to access Warrenton Road.

It was also determined that 75% of the trips entering the site will use Short Street while 25% of the trips entering the site will use Olde Forge Drive. This is due to 75% of the trips coming from the west as these trips will arrive at the unsignalized Short Street before approaching the signalized Olde Forge Drive. This assumption will allow drivers to avoid the Olde Forge traffic signal and take a free-flowing right turn onto Short Street to access the development.

With the large number of incoming trips using Short Street and Olde Forge Drive, Rappahannock Landing Apartments is prepared to install guide signs in the area to facilitate and provide guidance to the development. The site generated trip distribution is depicted on **Exhibit 3** and **Exhibit 4** in **Appendix D.**

The projected 2022 and 2028 Build conditions (including the morning and evening trip volumes for the Rappahannock Landing Apartments) are depicted on **Exhibit 8** and **Exhibit 10** in **Appendix D.**

The 2022 and 2028 build scenarios were created using the 2022 and 2028 No Build traffic volumes plus the traffic volumes from the Rappahannock Landing Apartments.

Capacity Analysis

The three previously mentioned study intersections were analyzed for each scenario using the 2016 Highway Capacity Manual (HCM) methodologies using the computer software package Synchro 10 with Sim Traffic. The analysis uses capacity, Level of Service, and control delay as the criteria for the performance of the intersections.

Capacity, as defined by the HCM, is a measure of the maximum number of vehicles in an hour that can travel through an intersection or section of roadway under typical conditions. Level of Service (LOS) is a marker of the driving conditions and perception of drivers while traveling during the given time period. LOS ranges from LOS "A" which represents free flow conditions, to LOS "F" which represents breakdown conditions. **Table 3** shows the LOS for intersections as defined by the HCM.

Table 3: HCM Level of Service Criteria

Unsignalized Intersections		Signalized Intersections	
Level of Service	Average Control Delay (sec/veh)	Level of Service	Average Control Delay (sec/veh)
A	≤10	A	≤10
B	>10-15	B	>10-20
C	>15-25	C	>20-35
D	>25-35	D	>35-55
E	>35-50	E	>55-80
F	≥50	F	≥80

Typically, LOS "A" through "D" is considered acceptable, while LOS "E" and "F" are considered failing or unacceptable. Control delay is a measure of the total amount of delay experienced by an individual vehicle and includes delay related to deceleration, queue delay, stopped delay, and acceleration. **Table 3** displays the amount of control delay (in seconds per vehicle) that corresponds to the LOS for signalized and unsignalized intersections.

Capacity analyses were completed for the above referenced study intersections during the No Build conditions and the Build conditions.

Capacity Analysis – Existing Conditions (2018)

Intersection of Warrenton Road and Short Street

Based on the results of the capacity analysis, the intersection of Warrenton Road and Short Street is projected to experience acceptable overall levels of service during the morning and evening peak hours. All turning movements and approaches currently operate at acceptable levels of service "D" or better. The results are summarized in **Table 4**. The capacity analysis results are included in **Appendix E**.

Table 4: Existing Conditions (2018) Capacity Analysis – Warrenton Road and Short Street

INTERSECTION			AM Peak			PM Peak		
			Conditions			Conditions		
	Approach	Movement	DELAY (S)	LOS	Maximum Queue (ft)*	DELAY (S)	LOS	Maximum Queue (ft)*
Intersection #1: **Warrenton Road (Route 17) & Short Street/Hotel Entrance (2018 Existing Conditions)**	NB	LT	41.5	D	114.0	41.0	D	134.0
		R	36.7	D	26.0	37.0	D	27.0
		Approach	40.8	D		40.4	D	
	SB	LTR	47.9	D	29.0	46.1	D	35.0
	EB	LT	16.9	B	233.0	25.5	C	233.0
		T	16.9	B	256.0	25.5	C	388.0
		R	8.0	A	136.0	9.2	A	74.0
		Approach	16.5	B		24.1	C	
	WB	L	3.0	A	21.0	4.0	A	42.0
		T	3.4	A	163.0	2.4	A	84.0
		TR	3.4	A	125.0	2.4	A	205.0
		Approach	3.4	A		2.4	A	
	OVERALL		**11.7**	**B**		**16.6**	**B**	

*Extracted from SimTraffic simulation software

Intersection of Warrenton Road and Olde Forge Drive

Based on the results of the capacity analysis, the unsignalized intersection of Warrenton Road and Olde Forge Drive is projected to experience acceptable overall levels of service during the morning and evening peak hours. All turning movements and approaches currently operate at acceptable levels of service "B" or better.

The results are summarized in **Table 5**. The capacity analysis results are included in **Appendix E**.

Table 5: Existing Conditions (2018) Capacity Analysis – Warrenton Road and Olde Forge Drive

INTERSECTION			AM Peak Conditions			PM Peak Conditions		
	Approach	Movement	DELAY (S)	LOS	Maximum Queue (ft)*	DELAY (S)	LOS	Maximum Queue (ft)*
Intersection #2: Warrenton Road (Route 17) & Olde Forge Drive (2018 Existing Conditions)	NB	L	10.9	B		13.9	B	72.0
		R	-	A	30.0	-	A	53.0
		Approach	10.9	B		13.9	B	
	EB	T	-	A	54.0	-	A	
		TR	-	A		-	A	
		Approach	-	A		-	A	
	WB	L	10.7	B	53.0	13.4	B	51.0
		T	-	A		-	A	
		Approach	0.2	A		0.6	A	

*Extracted from SimTraffic simulation software

Intersection of Warrenton Road and Solomon Drive/Lendall Lane

Based on the results of the capacity analysis, the intersection of Warrenton Road and Solomon Drive/Lendall Lane is expected to experience acceptable overall levels of service during the morning and evening peak hours. All turning movements and approaches currently operate at acceptable levels of service "D" or better.

The results are summarized in **Table 6**. The capacity analysis results are included in **Appendix E**.

Table 6: Existing Conditions (2018) Capacity Analysis – Warrenton Road and Solomon Drive/Lendall Lane

INTERSECTION			AM Peak Conditions			PM Peak Conditions		
	Approach	Movement	DELAY (S)	LOS	Maximum Queue (ft)*	DELAY (S)	LOS	Maximum Queue (ft)*
Intersection #3: Warrenton Road & Solomon Drive/Lendall Lane (2018 Existing Conditions)	NB	LTR	42.3	D	72.0	43.9	D	72.0
	SB	L	42.3	D	72.0	43.1	D	49.0
		T	40.0	D	52.0	41.2	D	
		R	40.2	D	46.0	41.2	D	52.0
		Approach	41.1	D		42.2	D	
	EB	L	22.2	C	88.0	1.3	A	68.0
		T	12.0	B	45.0	9.3	A	105.0
		R	10.0	B		8.6	A	15.0
		Approach	12.8	B		9.1	A	
	WB	L	11.8	B	6.0	14.7	B	6.0
		T	25.7	C	178.0	16.5	B	211.0
		R	12.6	B		9.7	A	
		Approach	25.1	C		16.4	B	
	OVERALL		**20.5**	**C**		**13.9**	**B**	

*Extracted from SimTraffic simulation software

Capacity Analysis Comparison – No Build vs Build Out Conditions (Year 2022)

Capacity Analyses were conducted for the No Build and Build conditions (year 2022). The primary purpose for this approach was to compare the results in order to identify areas impacted by the proposed development.

<u>Intersection of Warrenton Road and Short Street</u>

Based on the results of the capacity analysis during the morning peak hour, the intersection of Warrenton Road and Short Street is projected to experience overall acceptable levels of service during the no build and build out conditions.

All turning movements and approaches are expected to operate at acceptable levels of service "C" or better during the morning peak hour for the no build and build out conditions. The results are summarized in **Table 7**. The capacity analysis results are included in **Appendix F**.

Table 7: 2022 AM Peak Hour Capacity Analysis – Warrenton Road and Short Street

INTERSECTION			AM Peak (No Build) Conditions			AM Peak (Build Out) Conditions		
	Approach	Movement	DELAY (S)	LOS	Maximum Queue (ft)*	DELAY (S)	LOS	Maximum Queue (ft)*
Intersection #1: Warrenton Road (Route 17) & Short Street/Hotel Entrance (2022 No Build vs. Build Out AM Conditions)	NB	R	17.9	C	22.0	18.1	C	80.0
	SB	R	9.8	A	25.0	10.0	A	28.0
	EB	T	-	A	121.0	-	A	245.0
		R	-	A		-	A	
		Approach	-	A		-	A	
	WB	T	-	A		-	A	
		TR	-	A		-	A	
		Approach	-	A		-	A	

*Extracted from SimTraffic simulation software

Based on the results of the capacity analysis during the evening peak hour, the intersection of Warrenton Road and Short Street is projected to experience overall acceptable levels of service during the no build and build out conditions.

All turning movements and approaches are expected to operate at acceptable levels of service "C" or better during the evening peak hour for the no build and build out conditions. The results are summarized in **Table 8**. The capacity analysis results are included in **Appendix F**.

Table 8: 2022 PM Peak Hour Capacity Analysis – Warrenton Road and Short Street

INTERSECTION			PM Peak (No Build) Conditions			PM Peak (Build Out) Conditions		
	Approach	Movement	DELAY (S)	LOS	Maximum Queue (ft)*	DELAY (S)	LOS	Maximum Queue (ft)*
Intersection #1: Warrenton Road (Route 17) & Short Street/Hotel Entrance (2022 No Build vs. Build Out PM Conditions)	NB	R	24.4	C	63.0	24.7	C	41.0
	SB	R	11.4	B	28.0	11.6	B	48.0
	EB	T	-	A	305.0	-	A	369.0
		R	-	A	307.0	-	A	333.0
		Approach	-	A		-	A	
	WB	T	-	A		-	A	
		TR	-	A		-	A	
		Approach	-	A		-	A	

*Extracted from SimTraffic simulation software

Based on the capacity analyses for the 2022 No Build and Build conditions, the intersection of Warrenton Road and Short Street is not expected to experience a significant increase in delay from the proposed development. The overall level of service did not change for the morning or evening peak hour when comparing the no build and build out condition.

Intersection of Warrenton Road and Olde Forge Drive

Based on the results of the capacity analysis during the morning peak hour, the intersection of Warrenton Road and Olde Forge Drive is projected to experience overall acceptable levels of service during the no build and build out conditions.

All turning movements and approaches are expected to operate at acceptable levels of service "D" or better during the morning peak hour for the no build and build out conditions. The results are summarized in **Table 9**. The capacity analysis results are included in **Appendix F**.

Table 9: 2022 AM Peak Hour Capacity Analysis – Warrenton Road and Olde Forge Drive

INTERSECTION			AM Peak (No Build) Conditions			AM Peak (Build Out) Conditions		
	Approach	Movement	DELAY (S)	LOS	Maximum Queue (ft)*	DELAY (S)	LOS	Maximum Queue (ft)*
Intersection #2: Warrenton Road (Route 17) & Olde Forge Drive (2022 No Build vs. Build Out AM Conditions)	NB	L	29.2	C	137.0	32.4	C	234.0
		R	23.2	C	51.0	23.6	C	73.0
		Approach	27.9	C		30.5	C	
	EB	L	13.9	B		14.7	B	
		T	31.3	C	352.0	35.9	D	364.0
		TR	31.3	C	352.0	35.9	D	357.0
		Approach	31.3	C		35.9	D	
	WB	L	12.0	B	90.0	13.8	B	219.0
		T	5.1	A	228.0	5.0	A	398.0
		Approach	5.4	A		5.3	A	
	OVERALL		**19.1**	**B**		**21.5**	**C**	

*Extracted from SimTraffic simulation software

Based on the results of the capacity analysis during the evening peak hour, the intersection of Warrenton Road and Olde Forge Drive is projected to experience overall acceptable levels of service during the no build and build out conditions.

All turning movements and approaches are expected to operate at acceptable levels of service "D" or better during the evening peak hour for the no build and build out conditions. The results are summarized in **Table 10**. The capacity analysis results are included in **Appendix F**.

Table 10: 2022 PM Peak Hour Capacity Analysis – Warrenton Road and Olde Forge Drive

INTERSECTION			PM Peak (No Build)			PM Peak (Build Out)		
			Conditions			Conditions		
	Approach	Movement	DELAY (S)	LOS	Maximum Queue (ft)*	DELAY (S)	LOS	Maximum Queue (ft)*
Intersection #2: Warrenton Road (Route 17) & Olde Forge Drive (2022 No Build vs. Build Out PM Conditions)	NB	L	44.5	D	189.0	48.3	D	246.0
		R	35.2	D	30.0	35.3	D	175.0
		Approach	42.4	D		45.5	D	
	EB	L	11.6	B	31.0	11.6	B	
		T	50.7	D	380.0	51.2	D	385.0
		TR	50.7	D	366.0	51.2	D	368.0
		Approach	50.6	D		51.0	D	
	WB	L	25.8	C	116.0	30.7	C	112.0
		T	8.0	A	72.0	8.0	A	250.0
		Approach	9.3	A		9.9	A	
	OVERALL		**33.8**	**C**		**34.5**	**C**	

*Extracted from SimTraffic simulation software

Based on the capacity analyses for the 2022 No Build and Build conditions, the intersection of Warrenton Road and Short Street is not expected to experience a significant increase in delay from the proposed development. During the morning peak hour the intersection will experience an increase in overall delay of 2.4 seconds between the no build and build conditions. During the evening peak hour the intersection will experience an increase in overall delay of 0.7 seconds between the no build and build conditions. While the overall level of service did not change for the evening peak hour, the morning peak hour experienced a change in the overall level of service from "B" to "C". While this change should be noted, with a change in delay of less than one second drivers will not experience a change in service while using the intersection. This level of service change can be mitigated by optimizing/adjusting the traffic signal timings in the future.

<u>Intersection of Warrenton Road and Solomon Drive/Lendall Lane</u>

Based on the results of the capacity analysis during the morning peak hour, the intersection of Warrenton Road and Solomon Drive/Lendall Lane is projected to experience overall acceptable levels of service during the no build and build out conditions.

All turning movements and approaches are expected to operate at acceptable levels of service "D" or better during the morning peak hour for the no build and build out conditions. The results are summarized in **Table 11**. The capacity analysis results are included in **Appendix F**.

Table 11: 2022 AM Peak Hour Capacity Analysis – Warrenton Road and Solomon Drive/Lendall Lane

INTERSECTION			AM Peak (No Build) Conditions			AM Peak (Build Out) Conditions		
	Approach	Movement	DELAY (S)	LOS	Maximum Queue (ft)*	DELAY (S)	LOS	Maximum Queue (ft)*
Intersection #3: **Warrenton Road & Solomon Drive/Lendall Lane (2022 No Build vs. Build Out AM Conditions)**	NB	LTR	44.8	D	54.0	44.8	D	91.0
	SB	L	43.1	D	149.0	43.1	D	58.0
		T	40	D	262.0	40.0	D	30.0
		R	40.2	D	164.0	40.2	D	72.0
		Approach	41.4	D		41.4	D	
	EB	L	30.4	C	195.0	32.0	C	81.0
		T	3.2	A	206.0	3.0	A	248.0
		R	7.6	A		7.6	A	74.0
		Approach	5.1	A		5.1	A	
	WB	L	9.8	A	18.0	10.0	B	129.0
		T	19.6	B	273.0	19.7	B	242.0
		R	9.1	A		9.1	A	24.0
		Approach	19.2	B		19.3	B	
	OVERALL		**14.0**	**B**		**14.0**	**B**	

*Extracted from SimTraffic simulation software

Based on the results of the capacity analysis during the evening peak hour, the intersection of Warrenton Road and Solomon Drive/Lendall Lane is projected to experience overall acceptable levels of service during the no build and build out conditions.

All turning movements and approaches are expected to operate at acceptable levels of service "D" or better during the evening peak hour for the no build and build out conditions except for the northbound approach. This approach operates at a level of service "E" for both the no build and build conditions. This failing level of service can be mitigated by optimizing/adjusting the traffic signal timings in the future along with adding auxiliary lanes for the approach. The results are summarized in **Table 12**. The capacity analysis results are included in **Appendix F**.

Table 12: 2022 PM Peak Hour Capacity Analysis – Warrenton Road and Solomon Drive/Lendall Lane

INTERSECTION			PM Peak (No Build) Conditions			PM Peak (Build Out) Conditions		
	Approach	Movement	DELAY (S)	LOS	Maximum Queue (ft)*	DELAY (S)	LOS	Maximum Queue (ft)*
Intersection #3: Warrenton Road & Solomon Drive/Lendall Lane (2022 No Build vs. Build Out PM Conditions)	NB	LTR	63	E	96.0	65.6	E	195.0
	SB	L	51.5	D	49.0	51.5	D	93.0
		T	48.0	D	31.0	48.0	D	30.0
		R	47.8	D	66.0	47.8	D	53.0
		Approach	49.8	D		49.8	D	
	EB	L	8.6	A	44.0	8.7	A	44.0
		T	23.3	C	141.0	23.6	C	216.0
		R	6.5	A	74.0	6.4	A	
		Approach	22.8	C		23.1	C	
	WB	L	18.4	B	18.0	18.7	B	18.0
		T	14.1	B	223.0	14.2	B	288.0
		R	7.2	A	1.0	7.1	A	285.0
		Approach	14.0	B		14.1	B	
	OVERALL		**21.2**	**C**		**21.4**	**C**	

*Extracted from SimTraffic simulation software

Based on the capacity analyses for the 2022 No Build and Build conditions, the intersection of Warrenton Road and Solomon Drive/Lendall Lane is not expected to experience a significant increase in delay from the proposed development. During the morning hour the intersection will experience no change in overall delay between the no build and build conditions. During the evening peak hour, the intersection will experience an increase in overall delay of 0.2 seconds. The overall level of service did not change for the morning or evening peak hour when comparing the no build and build out condition.

Capacity Analysis Comparison – No Build vs Build Out Conditions (Year 2028)

Capacity Analyses were conducted for the No Build and Build conditions (year 2028). The primary purpose for this approach was to compare the results in order to identify areas impacted by the proposed development.

Intersection of Warrenton Road and Short Street

Based on the results of the capacity analysis during the morning peak hour, the intersection of Warrenton Road and Short Street is projected to experience overall acceptable levels of service during the no build and build out conditions.

All turning movements and approaches are expected to operate at acceptable levels of service "C" or better during the morning peak hour for the no build and build out conditions. The results are summarized in **Table 13**. The capacity analysis results are included in **Appendix G**.

Table 13: 2028 AM Peak Hour Capacity Analysis – Warrenton Road and Short Street

INTERSECTION			AM Peak (No Build) Conditions			AM Peak (Build Out) Conditions		
	Approach	Movement	DELAY (S)	LOS	Maximum Queue (ft)*	DELAY (S)	LOS	Maximum Queue (ft)*
Intersection #1: Warrenton Road (Route 17) & Short Street/Hotel Entrance (2028 No Build vs. Build Out AM Conditions)	NB	R	20.0	C	110.0	20.2	C	61.0
	SB	R	10.0	B	28.0	10.4	B	
	EB	T	-	A	282.0	-	A	370.0
		R	-	A	235.0	-	A	267.0
		Approach	-	A		-	A	
	WB	T	-	A		-	A	
		TR	-	A		-	A	
		Approach	-	A		-	A	

*Extracted from SimTraffic simulation software

Based on the results of the capacity analysis during the evening peak hour, the intersection of Warrenton Road and Short Street is projected to experience overall acceptable levels of service during the no build and build out conditions.

All turning movements and approaches are expected to operate at acceptable levels of service “D” or better during the evening peak hour for the no build and build out conditions. The results are summarized in **Table 14**. The capacity analysis results are included in **Appendix G**.

Table 14: 2028 PM Peak Hour Capacity Analysis – Warrenton Road and Short Street

INTERSECTION			PM Peak (No Build) Conditions			PM Peak (Build Out) Conditions		
	Approach	Movement	DELAY (S)	LOS	Maximum Queue (ft)*	DELAY (S)	LOS	Maximum Queue (ft)*
Intersection #1: Warrenton Road (Route 17) & Short Street/Hotel Entrance (2028 No Build vs. Build Out PM Conditions)	NB	R	29.4	D	44.0	29.9	D	147.0
	SB	R	11.9	B	28.0	12.2	B	28.0
	EB	T	-	A	370.0	-	A	342.0
		R	-	A	353.0	-	A	341.0
		Approach	-	A		-	A	
	WB	T	-	A		-	A	
		TR	-	A		-	A	
		Approach	-	A		-	A	

*Extracted from SimTraffic simulation software

Based on the capacity analyses for the 2028 No Build and Build conditions, the intersection of Warrenton Road and Short Street is not expected to experience an increase in delay from the proposed development. The overall level of service did not change for the morning or evening peak hour when comparing the no build and build out conditions.

Intersection of Warrenton Road and Olde Forge Drive

Based on the results of the capacity analysis during the morning peak hour, the intersection of Warrenton Road and Olde Forge Drive is projected to experience overall acceptable levels of service during the no build and build out conditions.

All turning movements and approaches are expected to operate at acceptable levels of service "D" or better during the morning peak hour for the no build and build out conditions. The results are summarized in **Table 15**. The capacity analysis results are included in **Appendix G**.

Table 15: 2028 AM Peak Hour Capacity Analysis – Warrenton Road and Olde Forge Drive

INTERSECTION			AM Peak (No Build) Conditions			AM Peak (Build Out) Conditions		
	Approach	Movement	DELAY (S)	LOS	Maximum Queue (ft)*	DELAY (S)	LOS	Maximum Queue (ft)*
Intersection #2: Warrenton Road (Route 17) & Olde Forge Drive (2028 No Build vs. Build Out AM Conditions)	NB	L	37.0	D	225.0	41.7	D	200.0
		R	28.3	C	175.0	28.4	C	74.0
		Approach	35.2	D		38.9	D	
	EB	L	12.5	B		12.5	B	
		T	31.6	C	360.0	31.9	C	383.0
		TR	31.6	C	358.0	31.9	C	357.0
		Approach	31.6	C		31.9	C	
	WB	L	11.8	B	72.0	18.4	B	52.0
		T	4.2	A	404.0	4.2	A	328.0
		Approach	4.4	A		4.7	A	
	OVERALL		**19.3**	**B**		**20.2**	**C**	

*Extracted from SimTraffic simulation software

Based on the results of the capacity analysis during the evening peak hour, the intersection of Warrenton Road and Olde Forge Drive is projected to experience overall acceptable levels of service "D" during the no build and build out conditions.

Multiple turning movements during the build and build out scenarios are expected to operate at an unacceptable level of service "E" during the evening peak hour. These turning movements all experience an unacceptable level of service and long delays during the no build and build out conditions. The main cause of these delays is due to the planned growth in the area projected to 2028. These unacceptable levels of service can be mitigated by optimizing/adjusting the traffic signal timings and/or intersection geometry in the future. The results are summarized in **Table 16**. The capacity analysis results are included in **Appendix G**.

Table 16: 2028 PM Peak Hour Capacity Analysis – Warrenton Road and Olde Forge Drive

INTERSECTION			PM Peak (No Build) Conditions			PM Peak (Build Out) Conditions		
	Approach	Movement	DELAY (S)	LOS	Maximum Queue (ft)*	DELAY (S)	LOS	Maximum Queue (ft)*
Intersection #2: Warrenton Road (Route 17) & Olde Forge Drive (2028 No Build vs. Build Out PM Conditions)	NB	L	64.5	E	168.0	68.6	E	307.0
		R	53.7	D	30.0	54.3	D	175.0
		Approach	62.1	E		65.5	E	
	EB	L	13.3	B	31.0	13.7	B	30
		T	58.4	E	359.0	62.5	E	360.0
		TR	58.4	E	360.0	62.5	E	362.0
		Approach	58.4	E		62.3	E	
	WB	L	62.4	E	184.0	70.6	E	224.0
		T	11.4	B	565.0	11.4	B	681.0
		Approach	14.7	B		16.0	B	
	OVERALL		**41.2**	**D**		**44.2**	**D**	

*Extracted from SimTraffic simulation software

Based on the capacity analyses for the 2028 No Build and Build conditions, the intersection of Warrenton Road and Short Street is not expected to experience a significant increase in overall delay from the proposed development. The overall level of service did not change for the morning or evening peak hour when comparing the no build and build out conditions.

Intersection of Warrenton Road and Solomon Drive/Lendall Lane

Based on the results of the capacity analysis during the morning peak hour, the intersection of Warrenton Road and Solomon Drive/Lendall Lane is projected to experience overall acceptable levels of service during the no build and build out conditions.

All turning movements and approaches are expected to operate at acceptable levels of service "D" or better during the morning peak hour for the no build and build out conditions. The results are summarized in **Table 17**. The capacity analysis results are included in **Appendix G**.

Table 17: 2028 AM Peak Hour Capacity Analysis – Warrenton Road and Solomon Drive/Lendall Lane

INTERSECTION			AM Peak (No Build)			AM Peak (Build Out)		
			Conditions			Conditions		
	Approach	Movement	DELAY (S)	LOS	Maximum Queue (ft)*	DELAY (S)	LOS	Maximum Queue (ft)*
Intersection #3: Warrenton Road & Solomon Drive/Lendall Lane (2028 No Build vs. Build Out AM Conditions)	NB	LTR	51.7	D	94.0	51.7	D	92.0
	SB	L	46.6	D	71.0	46.6	D	71.0
		T	42.9	D	28.0	42.9	D	
		R	43.0	D	72.0	43.0	D	95.0
		Approach	44.5	D		44.5	D	
	EB	L	36.1	D	129.0	36.2	D	67.0
		T	3	A	153.0	3.3	A	345.0
		R	6.9	A		6.9	A	
		Approach	5.1	A		5.3	A	
	WB	L	10.9	B	18.0	11.3	B	16.0
		T	20.7	C	363.0	20.9	C	354.0
		R	8.2	A	3.0	8.2	A	2.0
		Approach	20.3	C		20.5	C	
	OVERALL		**14.6**	**B**		**14.8**	**B**	

*Extracted from SimTraffic simulation software

Based on the results of the capacity analysis during the evening peak hour, the intersection of Warrenton Road and Solomon Drive/Lendall Lane is projected to experience overall acceptable levels of service during the no build and build out conditions.

Eastbound and westbound turning movements and approaches are expected to operate at acceptable levels of service “D” or better during the evening peak hour for the no build and build out conditions. While these approaches are expected to operate at acceptable levels of service, the northbound and southbound approaches are expected to operate at unacceptable levels of service during the no build and build out conditions. These failing levels of service can be mitigated by optimizing/adjusting the traffic signal timings in the future along with adding auxiliary lanes for the approaches. The results are summarized in **Table 18**. The capacity analysis results are included in **Appendix G**.

Table 18: 2028 PM Peak Hour Capacity Analysis – Warrenton Road and Solomon Drive/Lendall Lane

INTERSECTION			PM Peak (No Build) Conditions			PM Peak (Build Out) Conditions		
	Approach	Movement	DELAY (S)	LOS	Maximum Queue (ft)*	DELAY (S)	LOS	Maximum Queue (ft)*
Intersection #3: Warrenton Road & Solomon Drive/Lendall Lane (2028 No Build vs. Build Out PM Conditions)	NB	LTR	98.5	F	158.0	95.8	F	140.0
	SB	L	89.6	F	66.0	89.6	F	72.0
		T	77.4	E		77.4	E	50.0
		R	77.0	E	71.0	77.0	E	45.0
		Approach	83.7	F		83.7	E	
	EB	L	10.0	B	21.0	10.4	B	44.0
		T	22.8	C	54.0	23.4	C	60.0
		R	5.5	A		5.5	A	6.0
		Approach	22.4	C		23.0	C	
	WB	L	28.1	C	14.0	29.1	C	18.0
		T	13.5	B	240.0	13.7	B	216.0
		R	6.4	A	2.0	6.4	A	1.0
		Approach	13.4	B		13.7	B	
	OVERALL		**22.2**	**C**		**22.5**	**C**	

*Extracted from SimTraffic simulation software

Based on the capacity analyses for the 2028 No Build and Build conditions, the intersection of Warrenton Road and Solomon Drive/Lendall Lane is not expected to experience a significant increase in overall delay from the proposed development. During the morning hour the intersection will experience an increase in overall delay of 0.2 second between the no build and build conditions. During the evening peak hour, the intersection will experience an increase in overall delay of 0.3 seconds. The overall level of service did not change for the morning or evening peak hour when comparing the no build and build out conditions.

<u>Crash Data Evaluation</u>

Crash data was extracted from the Tableau VDOT database for the two (2) intersections of the which serve as the main entrances for the proposed development (Warrenton Road/Short Street and Warrenton Road/Olde Forge Drive). Crash data was extracted for the past three years (2015-2017).

<u>Intersection of Warrenton Road and Short Street</u>

The crash data extracted from the Tableau VDOT database for the intersection of Warrenton Road and Short Street is summarized in **Table 19**. Although this intersection does not meet the threshold of 5 crashes per year; it is recommended that VDOT and/or county monitor or further evaluate the roadway conditions since the majority of crashes are of angle type. This crash data should be accounted for when considering the proposed improvements as outlined in the US Route 17 study performed by VDOT.

Table 19: Crash Data – Warrenton Road and Short Street

Crash Data (2015-2017) Warrenton Road (Route 17) & Short Street			
Collision Type	Year		
	2015	2016	2017
Rear End			
Angle	3		3
Side Swipe	1		
Fixed Object		1	
Other			
Total	4	1	3

Intersection of Warrenton Road and Olde Forge Drive

The crash data extracted from the Tableau VDOT database for the intersection of Warrenton Road and Olde Forge Drive is summarized in **Table 20**. This intersection does not meet the threshold of 5 crashes per year and it does not appear that any crash patterns exist for this intersection.

Table 20: Crash Data – Warrenton Road and Olde Forge Drive

Crash Data (2015-2017) Warrenton Road (Route 17) & Olde Forge Drive			
Collision Type	Year		
	2015	2016	2017
Rear End			
Angle			
Side Swipe	1		1
Fixed Object			
Other		1	
Total	1	1	1

Other Modes of Transportation

This study also reviews the potential for walking, bicycling, and transit trips to and from the area.

Walking Facilities

Currently there are no sidewalks along Short Street and Olde Forge Drive in the vicinity of the site. There are sidewalks along the frontage of certain commercial parcels along Warrenton Road, but there is not a continuous pedestrian facility which is adjacent to Warrenton Road. Walking facilities are provided within other sections of Rappahannock Landing, but these facilities halt as they leave the development as there are no other continuous facilities provided in the area. The proposed Rappahannock Landing Apartments is expected to be consistent with walking facilities within previously approved Rappahannock Landing sections. Given the residential nature of the development and the lack of sidewalks on existing streets in the area to provide pedestrian connections, it is unlikely that a significant number of trips would be made via walking. Therefore, no reductions in site generated trips were taken in this analysis for walking.

Bicycle Facilities

Currently, there are no bicycle facilities on any of the roadways in the vicinity of the site. The Stafford County Comprehensive Plan makes no mention of adding bicycling facilities to any of those roadways. While bicycling trips are possible, without bicycle facilities it is unlikely that a significant portion of the site trips would be made via bicycle. Therefore, no reductions in site generated trips were taken in this analysis for bicycling.

Transit Facilities

Fredericksburg Regional Transit (FRED) provides bus service along Warrenton Road (Route 17), with a bus stop at Olde Forge Drive. Without sidewalks to connect from the proposed development to the transit stop at the intersection of Warrenton Road and Olde Forge Drive, it is unlikely that a significant portion of the site trips would be made via transit. It is recommended that the County coordinate with FRED to provide a bus route to serve existing and proposed residential units in the Rappahannock Landing development. Therefore, no reductions in site generated trips were taken in this analysis for transit. It is recommended that the County coordinate with FRED to have a formal transit stop constructed at the intersection of Olde Forge Drive and Warrenton Road to promote the use of this transit stop.

Conclusions

- The proposed development is expected to generate 108 trips (28 in and 80 out) during the morning peak hour and 137 trips (84 in and 53 out) during the evening peak hour.
- The proposed Rappahannock Landing Apartments Development was found to present minor increases of delay and/or changes in Levels of Service on the existing roadway network during the 2022 & 2028 build out conditions.
- The minor increases and/or changes can be mitigated by adjusting/optimizing the traffic signal timings to accommodate future traffic demand. Traffic signal timing evaluations and adjustments are recommended once the proposed development is constructed by the year 2022.
- The proposed Rappahannock Landing Apartments development is not expected to adversely impact the existing roadway network with the proposed VDOT improvements.

Bowman
CONSULTING

Appendix A: VDOT Approved Pre-Scope of Work Meeting Form

PRE-SCOPE OF WORK MEETING FORM

Information on the Project
Traffic Impact Analysis Base Assumptions

The applicant is responsible for entering the relevant information and submitting the form to VDOT and the locality no less than three (3) business days prior to the meeting. If a form is not received by this deadline, the scope of work meeting may be postponed.

Contact Information	
Consultant Name: Tele: E-mail:	Bowman Consulting Group - Carlos G. Garcia, PE 804-616-3240 cgarcia@bowmanconsulting.com
Developer/Owner Name: Tele: E-mail:	Mr. Brian Revere / The Breeden Company 4501 Marshall Run Circle,Glen Allen, Virginia 3059 brianr@breedenconstruction.com

Project Information				
Project Name:	Rappahannock Landing Apartments (Section 5)-(See Figure 1 and 2)			
Project Location: (Attach regional and site specific location map)	Rappahannock Landing Apartments			
Project Description: Including type of application (rezoning, subdivision, site plan), acreage, business square ft, number of dwelling units, access location, etc. Attach additional sheet if necessary)	The project consists of a rezoning the site the is currently zoned as R-1 to UD-3. The site contains approximately 25.53 acres. The proposed development is expecting to accommodate 324 dwelling units of apartments.			
Locality/County:	Stafford County			
Proposed Use: (Check all that apply; attach additional pages as necessary)	**Residential** ☒	**Commercial** ☐	**Mixed Use** ☐	**Other** ☐
	Residential # of Units: 324 **Commercial** Use Sq Ft: _____ ITE LU Code(s): 221 _____ _____ _____		**Mixed Use**: # Res. Units: _____ ITE LU Code(s): _____ Commercial Use Sq Ft:_____ ITE LU Code(s): _____ **Other**: _____ ITE LU Code(s): _____ Sq Ft: _____	

It is important for the applicant to provide sufficient information to county and VDOT staff so that questions regarding geographic scope, alternate methodology, or other issues can be answered at the scoping meeting.

Traffic Impact Analysis Assumptions

Study Period	Existing Year: 2018	Build-out Year: 2022	Design Year: 2028
Study Area Boundaries (Attach map)	North: Route 17 (Warrenton Rd)	South: Rappahannock River	
	East: Rappahannock Landing	West: I-95	
External Factors That Could Affect Project (Planned road improvements, other nearby developments)	Route 17 Corridor Timing adjustments (By VDOT) Roadway Improvements along Route 17 (Median Installation along Rte 17 in front of Short Street) Rappahannock Development (Sections 1-4) Traffic Signal Relocation on Rte 17 from Short Drive to Olde Forge Road		
Consistency With Comprehensive Plan	The property is within the County's targeted growth designation area		
Available Traffic Data (Historical, forecasts)	See Table 1 and Table 2		

Trip Distribution (Attach sketch)	Road Name: Route 17 (Warrenton Rd)	75% To/From the West of the Site 25% To/from the East of the Site			
See attachments for additional information.	Road Name: Short Street	N 5% %	S 75%%	E ____%	W ____%
	Road Name: Olde Forge Road	N 95%%	S 25% %	E ____%	W ____%
	Road Name:	N ____%	S ____%	E ____%	W ____%

Annual Vehicle Trip Growth Rate:	2% (Rte 17 Only)	Peak Period for Study (check all that apply)	☒ AM ☒ PM ☐ SAT

Study Intersections and/or Road Segments (Attach additional sheets as necessary)	1. Short St and Warrenton Rd	6.
	2. Old Forge Dr and Warrenton Rd	7.
	3. Warrenton Rd. and Solomon Dr.	8.
	4.	9.
	5.	10.

Trip Adjustment Factors	Internal allowance: ☐ Yes ☒ No Reduction:______ % trips	Pass-by allowance: ☐ Yes ☒ No Reduction:______% trips
Software Methodology	☒ Synchro ☒ HCS (v.2000/+) ☐ aaSIDRA ☐ CORSIM ☐ Other _____	
Traffic Signal Proposed or Affected (Analysis software to be used, progression speed, cycle length)	The study will include the relocation of the traffic signal along Rte 17. from Short Street to Olde Forge Road. The intersection of Rte 17 and Solomon Drive will be included as this intersection will be coordinated with the new signal at Olde FOrge Road.	

It is important for the applicant to provide sufficient information to county and VDOT staff so that questions regarding geographic scope, alternate methodology, or other issues can be answered at the scoping meeting.

Improvement(s) Assumed or to be Considered	Traffic signal timing adjustments at Short Street would be considered if needed.
Background Traffic Studies Considered	Other than additional sections of the Rappahannock Landing Development; the immediate area boundaries do not have additional existing traffic studies that can be considered as part of this study.
Plan Submission	☐ Master Development Plan (MDP) ■ Generalized Development Plan (GDP) ☐ Preliminary/Sketch Plan ☐ Other Plan type (Final Site, Subd. Plan)
Additional Issues to be addressed	■ Queuing analysis ■ Actuation/Coordination ☐ Weaving analysis ☐ Merge analysis ■ Bike/Ped Accommodations ■ Intersection(s) ■ TDM Measures ☐ Other ________________

NOTES on ASSUMPTIONS: *See attached list of notes and assumptions*

SIGNED: ______________________________ DATE: 03/26/2018
Applicant or Consultant

PRINT NAME: Carlos G. Garcia, P.E.
Applicant or Consultant

It is important for the applicant to provide sufficient information to county and VDOT staff so that questions regarding geographic scope, alternate methodology, or other issues can be answered at the scoping meeting.

SCOPE OF WORK MEETING CONCLUSIONS

ADDITIONS TO THE VDOT REQUIRED ELEMENTS, CHANGES TO THE METHODOLOGY OR STANDARD ASSUMPTIONS, AND SIGNATURE PAGE

Any additions to the VDOT Required Elements or changes to the Methodology or Standard Assumptions due to special circumstances that are approved by VDOT:

Based on the proposed site trip generation and the internal arrival/departure distribution, the report will discuss the need for improvements on Musselman Road from Krieger Lane to Streamview Drive. The discussion will be based on the 527 requirements.

It is also understood that this section of Musselman Road has limitations such as available ROW, Utilities and ditches.

The applicant will contact VDOT and the locality prior to the preparation of the traffic impact analysis study in the event there are any substantial changes in the existing conditions that will affect the scope of the study.

AGREED: ______________________ DATE: 03/26/2018
Applicant or Consultant

PRINT NAME: Carlos G. Garcia, P.E.
Applicant or Consultant

SIGNED: ______________________ DATE: ____________
VDOT Representative

PRINT NAME: ______________________
VDOT Representative

SIGNED: ______________________ DATE: ____________
Local Government Representative

PRINT NAME: ______________________
Local Government Representative

Intersections to be evaluated

Intersections:

1. Short Street and Warrenton Road (signalized)
2. Old Forge Drive and Warrenton Road (un-signalized)
3. Warrenton Drive and Solomon Drive (Signalized)

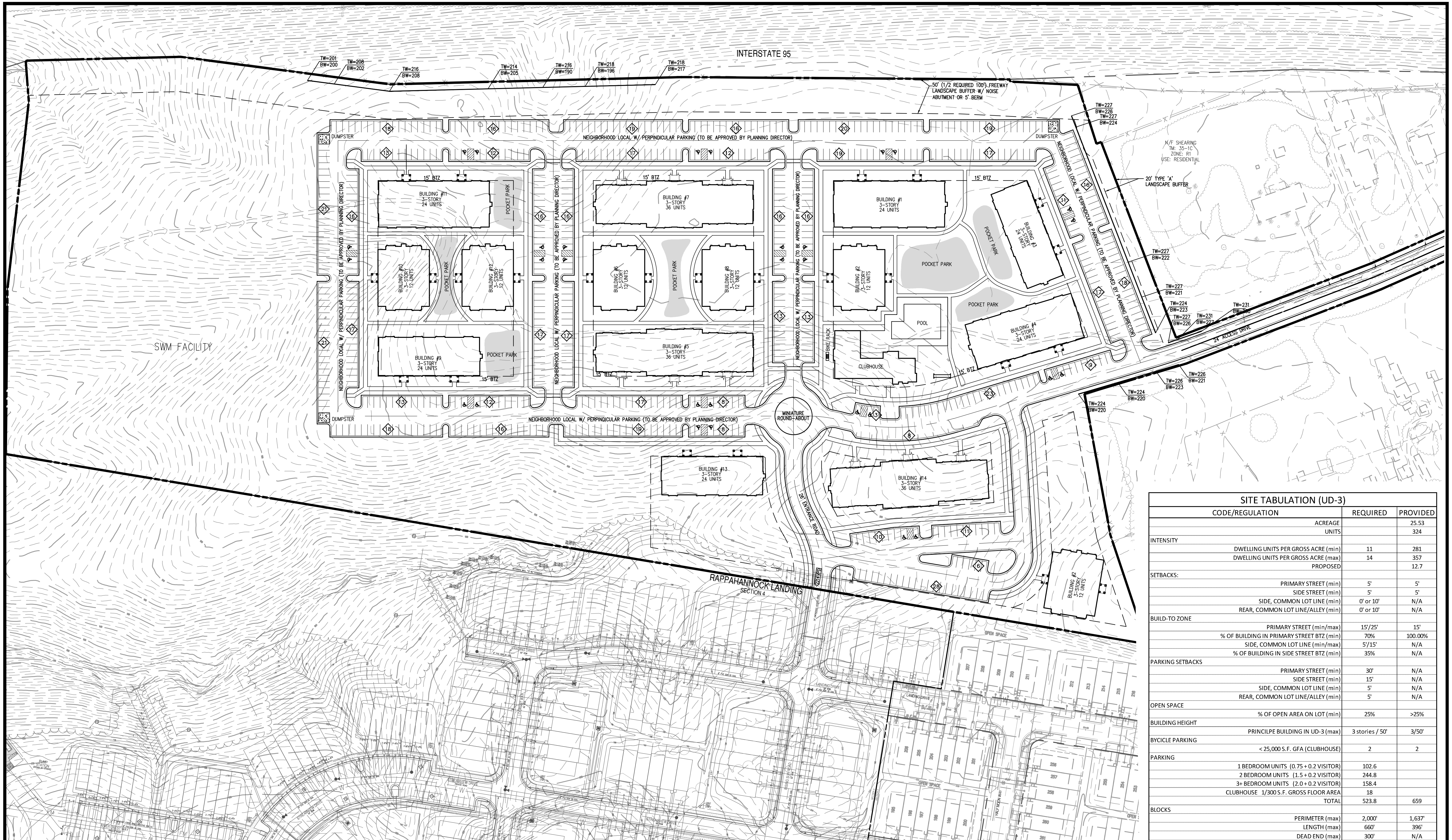
Trip Generation

Land use	LU Code	Size	Units	AM Peak Hour			PM Peak Hour			Daily trips
				In	Out	Total	In	Out	Total	
Multifamily Housing (Mid-Rise)	221	324	D.U.	27	81	108	84	53	137	1764

Average Daily Traffic Volumes (VA Roads)

Route	Average daily traffic
Route 17 (Warrenton Road)	35,000
Old Forge Drive	850
Short Street	2,000
Musselman Road (Between Bellows Avenue and Anvil Road)	1,600
Musselman Road (Between Anvil Road and Krieger Lane)	460
Bellows Avenue	560
Anvil Road	660

Arrival / Departure Distribution
N
Falls Run
75%
75%
5%
25%
75%
20%
25%
Riverside Center for the Performing Arts
I-95
95
657
17
698
1004
1001
Lennar at Rappahannock Landing
Gari Melchers Complex
Route 17
BUS 17
Legend
Overall % Arrival/Departure
% Vehicles Entering the site
% Vehicles Exiting the site



SITE TABULATION (UD-3)		
CODE/REGULATION	REQUIRED	PROVIDED
ACREAGE		25.53
UNITS		324
INTENSITY		
DWELLING UNITS PER GROSS ACRE (min)	11	281
DWELLING UNITS PER GROSS ACRE (max)	14	357
PROPOSED		12.7
SETBACKS:		
PRIMARY STREET (min)	5'	5'
SIDE STREET (min)	5'	5'
SIDE, COMMON LOT LINE (min)	0' or 10'	N/A
REAR, COMMON LOT LINE/ALLEY (min)	0' or 10'	N/A
BUILD-TO ZONE		
PRIMARY STREET (min/max)	15'/25'	15'
% OF BUILDING IN PRIMARY STREET BTZ (min)	70%	100.00%
SIDE, COMMON LOT LINE (min/max)	5'/15'	N/A
% OF BUILDING IN SIDE STREET BTZ (min)	35%	N/A
PARKING SETBACKS		
PRIMARY STREET (min)	30'	N/A
SIDE STREET (min)	15'	N/A
SIDE, COMMON LOT LINE (min)	5'	N/A
REAR, COMMON LOT LINE/ALLEY (min)	5'	N/A
OPEN SPACE		
% OF OPEN AREA ON LOT (min)	25%	>25%
BUILDING HEIGHT		
PRINCILPE BUILDING IN UD-3 (max)	3 stories / 50'	3/50'
BYCICLE PARKING		
< 25,000 S.F. GFA (CLUBHOUSE)	2	2
PARKING		
1 BEDROOM UNITS (0.75 + 0.2 VISITOR)	102.6	
2 BEDROOM UNITS (1.5 + 0.2 VISITOR)	244.8	
3+ BEDROOM UNITS (2.0 + 0.2 VISITOR)	158.4	
CLUBHOUSE 1/300 S.F. GROSS FLOOR AREA	18	
TOTAL	523.8	659
BLOCKS		
PERIMETER (max)	2,000'	1,637'
LENGTH (max)	660'	396'
DEAD END (max)	300'	N/A

GRAPHIC SCALE

60 0 30 60 120 240

(IN FEET)
1 inch = 60 ft.

NORTH

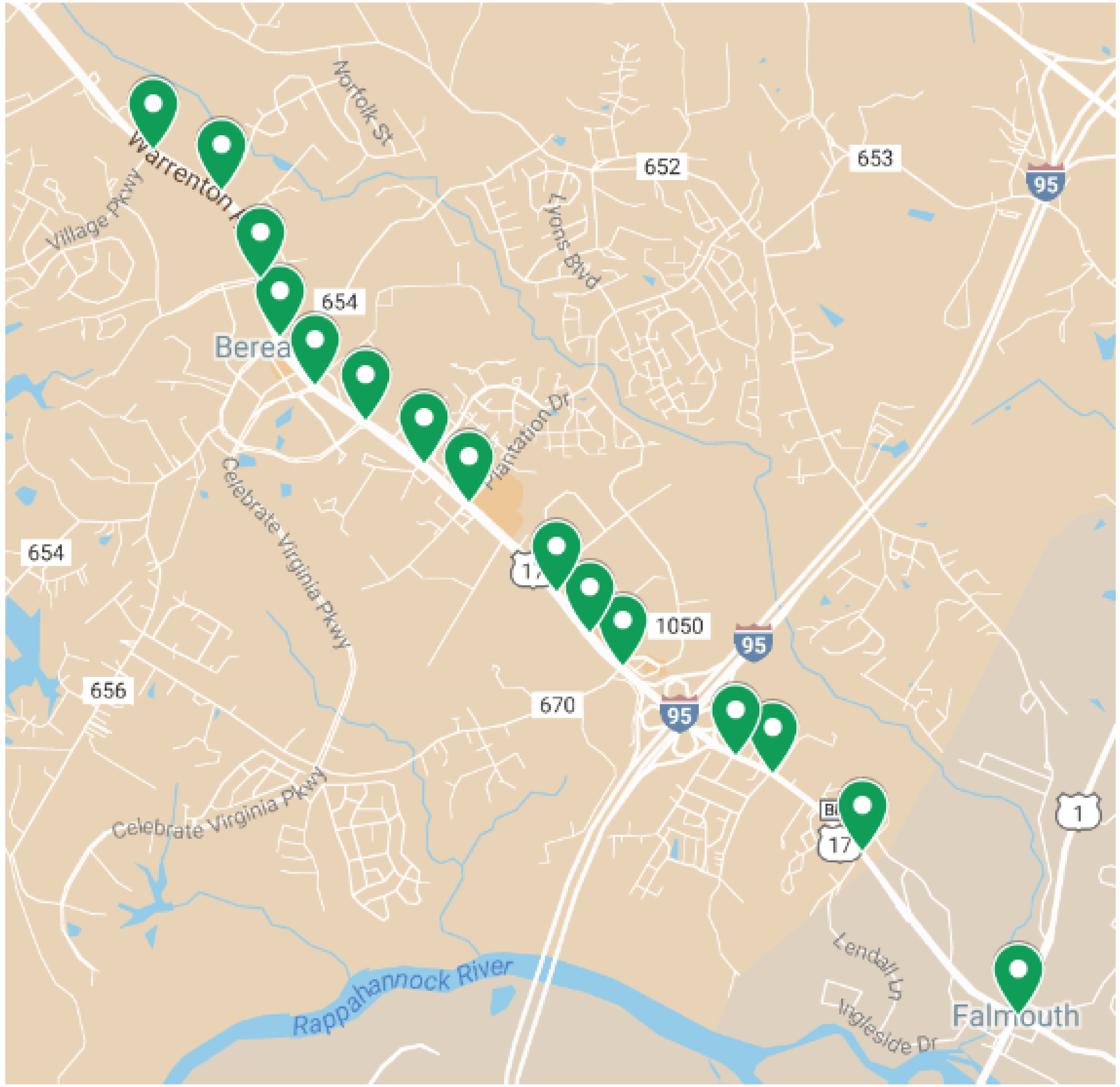
CONCEPTUAL PLAN
RAPPAHANNOCK LANDING – SECTION 5
STAFFORD, VIRGINIA

Bowman
CONSULTING

JUNE 30, 2017

1 OF 1

Appendix B: Raw Traffic Data



NOT TO SCALE

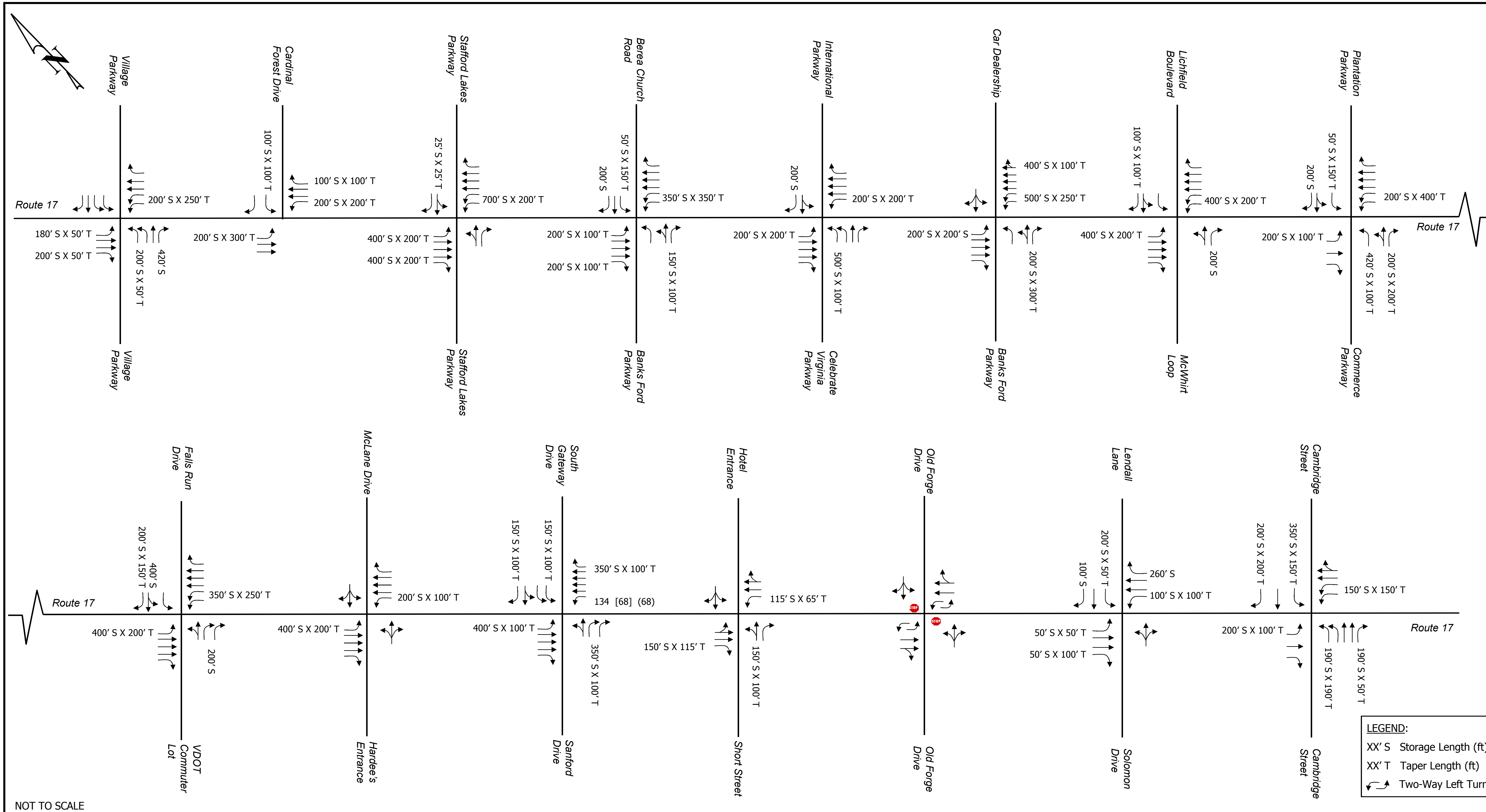
LEGEND:

Study Intersections

TIMMONS GROUP
YOUR VISION ACHIEVED THROUGH OURS.

Study Location Map and Intersections
U.S. Route 17 Corridor

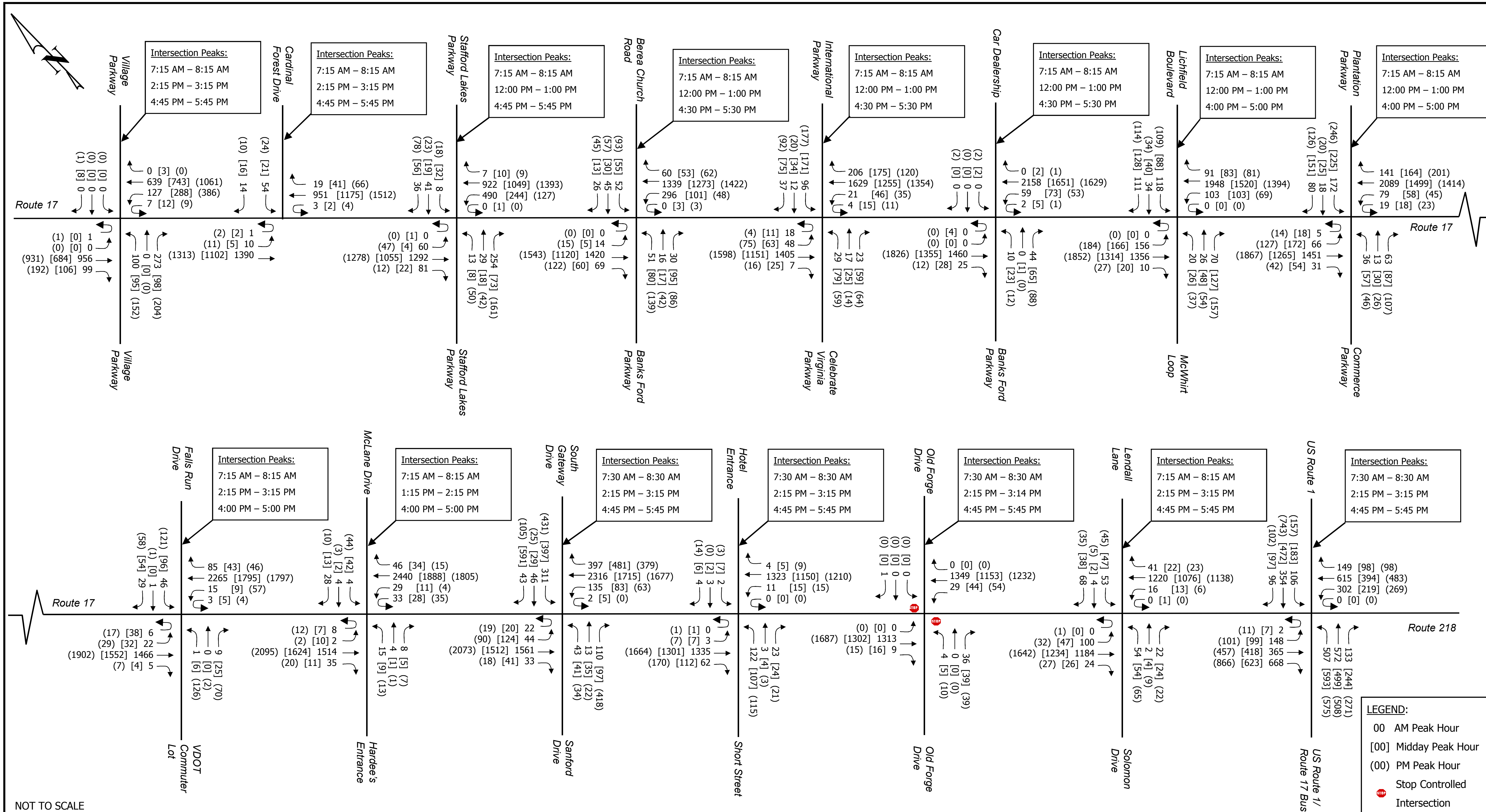
Figure
1-1



TIMMONS GROUP
YOUR VISION ACHIEVED THROUGH OURS.

U.S. Route 17 Corridor
Existing Lane Geometry
June 2017

Figure 1-2



TIMMONS GROUP
YOUR VISION ACHIEVED THROUGH OURS.

U.S. Route 17 Corridor
Weekday Peak Hour Volumes – Individual Intersection Peak Hour
June 2017

Figure 2-1

Appendix C: U.S. 17 Corridor Study Performed by VDOT

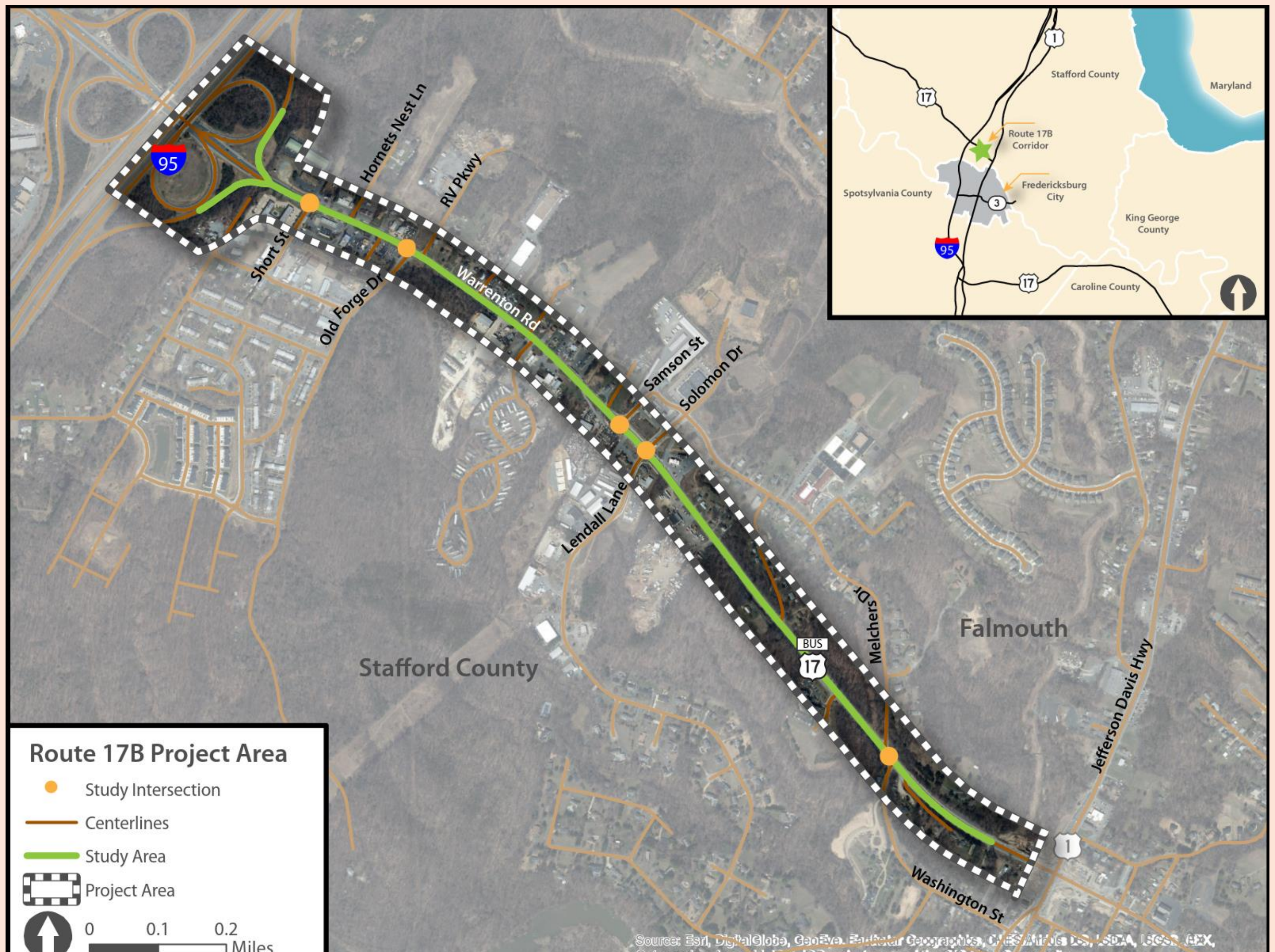
US 17 BUSINESS CORRIDOR STUDY

FROM INTERSTATE-95 TO ROUTE 1001 (WASHINGTON STREET)

Project Description

Route 17 Business between I-95 and Route 1 at Falmouth is a critical east west route in southern Stafford County providing access between the City of Fredericksburg to retail centers and residences east and west of I-95. It experiences heavy congestion during peak hours and has a higher crashes when compared to the statewide average. VDOT has recently completed improvement projects surrounding this corridor, highlighting the need to identify and evaluate additional improvements along this corridor to address congestion and safety issues.

US 17 Business Project Area and Location Map



Planning Level Cost Estimate

Phase	Six Year Improvement Program
Preliminary Engineering	$650,000
ROW and Utility Relocation	$219,000
Construction	$4,243,000
Total Cost =	**$5,112,000**

Note: Cost estimates reported in 2017 dollars

Traffic Operations Improvements

- Addition of lane capacity
- Turn lane storage length extensions
- Traffic signal timing/phasing improvements
- Lane re-configurations
- Traffic signage modifications and improvements

Targeted Safety Improvements

- Access management measures
- Geometric improvements
- Pavement marking improvements
- Pedestrian/bike facilities improvements
- Sight Distance improvements

Crash Reduction

2030 – No Build	2030 – Build
53 Expected Crashes	46 to 50 Expected Crashes

6-12% REDUCTION

Project Benefits

Traffic Operations Measures	
2030 No Build Delay*	423,054 seconds
2030 Build Delay*	384,092 seconds
Δ Delay (% Change)	-38,962 seconds (-9%)
20-Year Operations Savings	$2,189,798.00

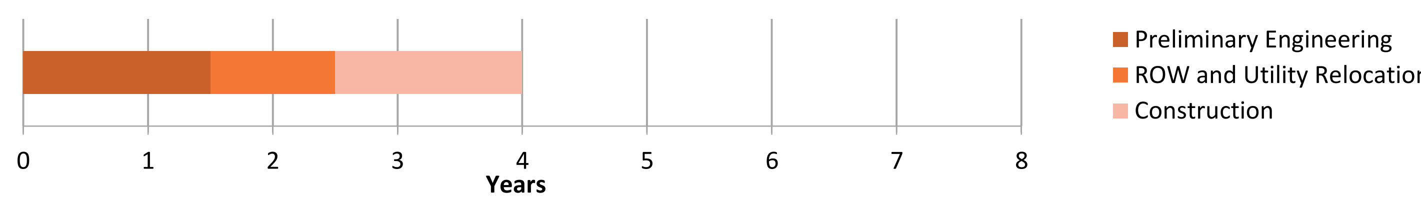
*Compounded AM and PM weekday travel delay in the influence area of all the proposed improvements within the corridor

- Reduced travel time and delay through the corridor
- Improved travel speeds through the corridor
- Improved signal timing and phasing
- Improved pavement markings and signing
- Improved sight distances
- Improved safety for road users

Benefit/Cost Ratio: 0.4

Benefit/Cost calculated using the midpoint of the cost estimate range

Project Schedule



STARS VDOT

US 17 BUSINESS CORRIDOR STUDY

PREFERRED IMPROVEMENT, SHORT STREET & OLDE FORGE DRIVE (ALTERNATIVE A)

Existing Conditions

Short Street

- Major Collector
- 4-legged signalized intersection

Olde Forge Drive

- Collector facility
- Unsignalized T-intersection

- Minor street movements experience heavier delay and a LOS D or worse
- Eastbound and westbound directions experience lengthy queues

Planning Level Cost Estimate

Phase	Six Year Improvement Program
Preliminary Engineering	$470,000
ROW and Utility Relocation	$188,000
Construction	$3,066,000
Total Cost =	**$3,724,000**

Note: Cost estimates reported in 2017 dollars

Eastbound Approach at Short Street

Project Benefits

Traffic Operations Measures	
2030 No Build Delay*	225,673 seconds
2030 Build Delay*	196,166 seconds
Δ Delay (% Change)	-29,507 seconds (-13%)
20-Year Operations Savings	$1,658,366.00

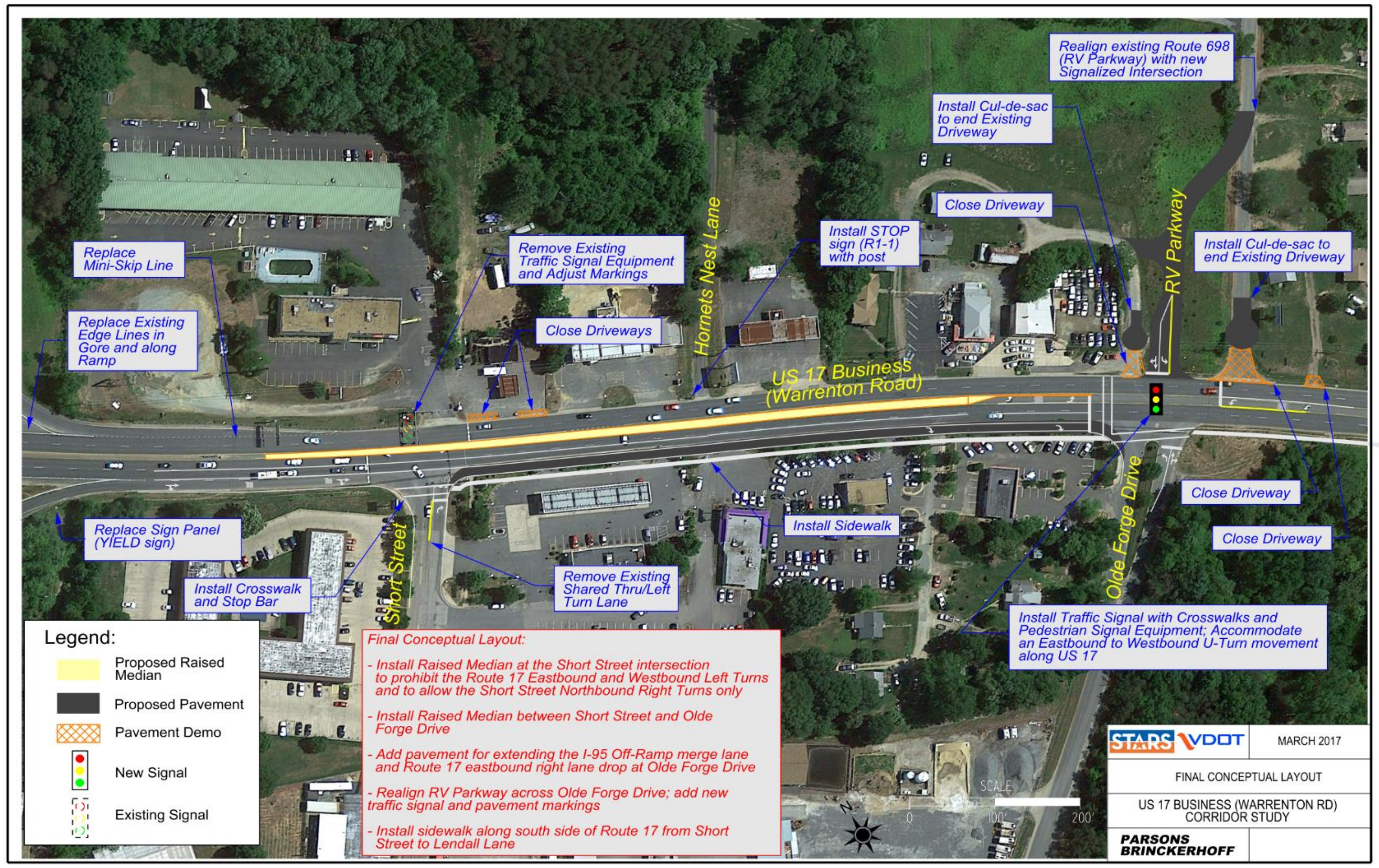
*Total of AM and PM weekday travel times in the influence area of the proposed improvement

- Provides longer merging distance for I-95 off-ramp traffic
- Adds capacity at the Older Forge Drive intersection
- Provides improved access for RV Parkway
- Improved safety for road users

Benefit/Cost Ratio: 0.4

Benefit/Cost calculated using the midpoint of the cost estimate range

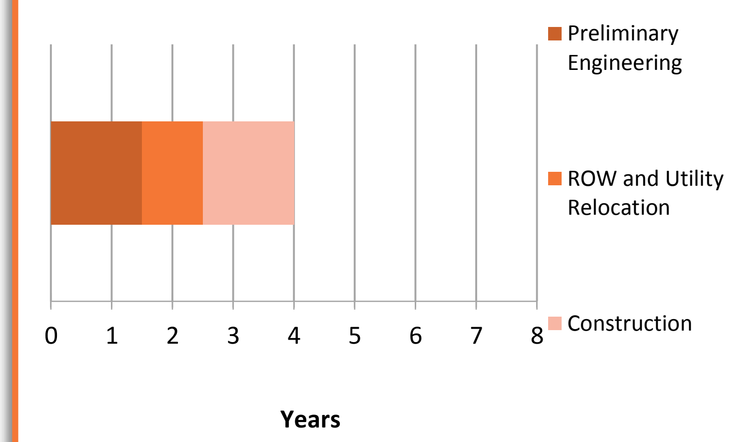
Final Alternative Layout, Alternative A



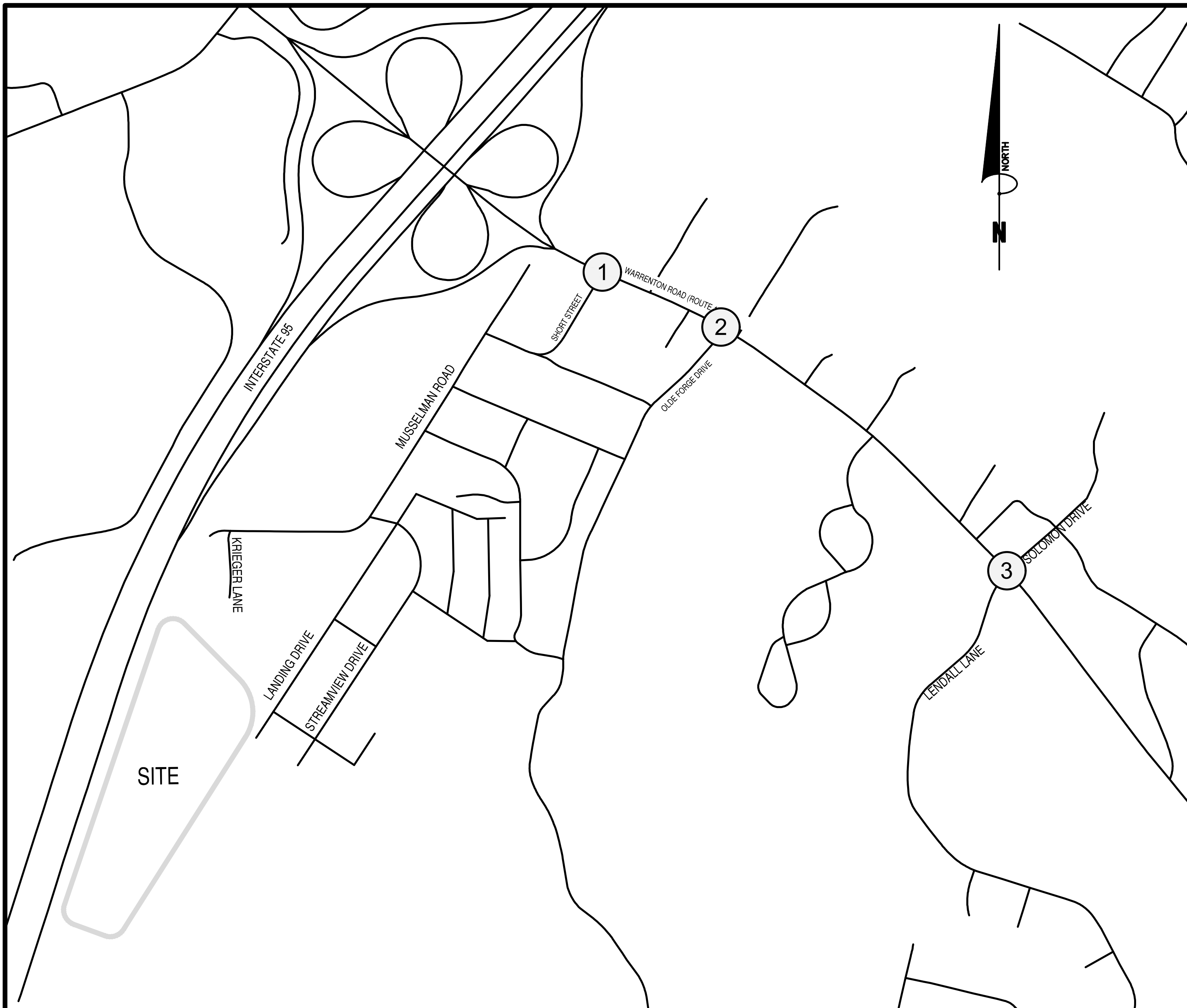
Operations Benefits

Short Street Intersection Queues (feet)				
Movement	2030 No-Build (Signalized)		2030 Build (Unsignalized)	
	AM	PM	AM	PM
Eastbound LT	267	281	--	--
Eastbound TH	247	287	0	0
Eastbound R	13	47	0	0
Westbound L	44	50	4	9
Westbound TH	136	132	0	0
Westbound TR	177	156	0	0
Northbound LT	180	210	--	--
Northbound R	49	73	11	14
Southbound LT	23	52	--	--
Southbound R	50	38	2	3

Project Schedule



Appendix D: Traffic Volume Figures



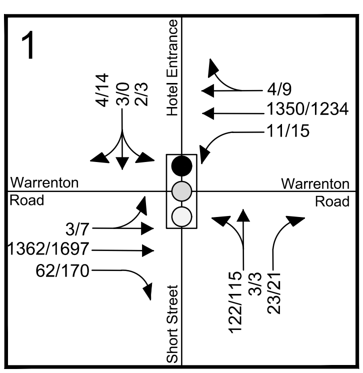
LEGEND

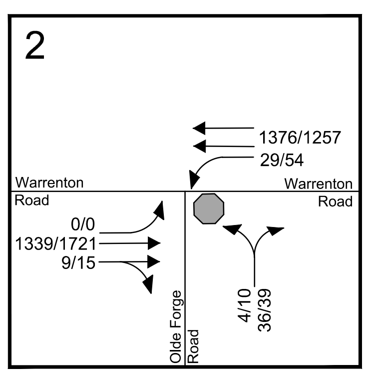
Traffic Signal

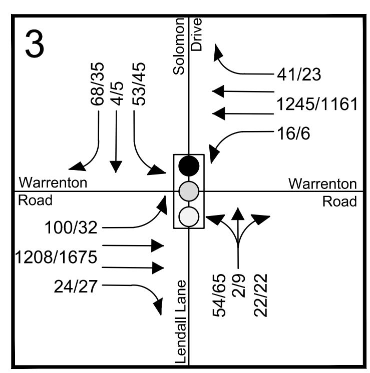
Stop Sign

Represents One Travel Lane

xx/yy AM/PM Peak Hour Volume





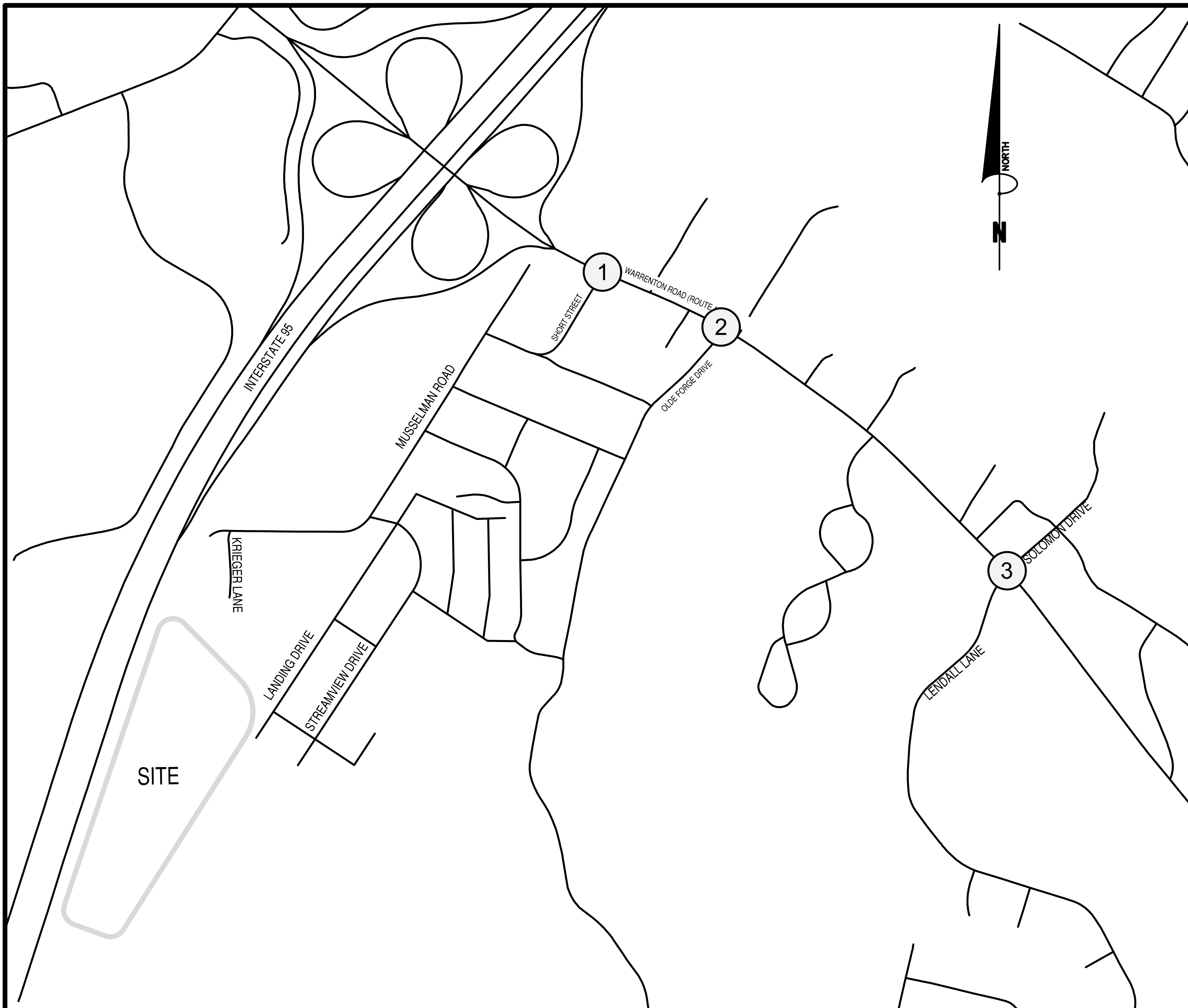


Bowman CONSULTING

Existing Conditions (2018)
Rappahannock Landing
Stafford County, Virginia

Exhibit 1

NOT TO SCALE



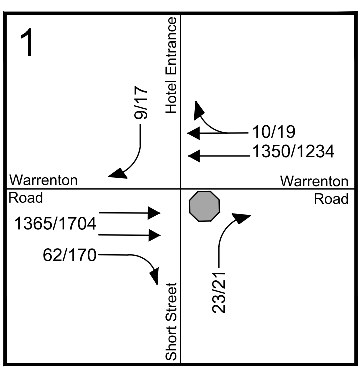
LEGEND

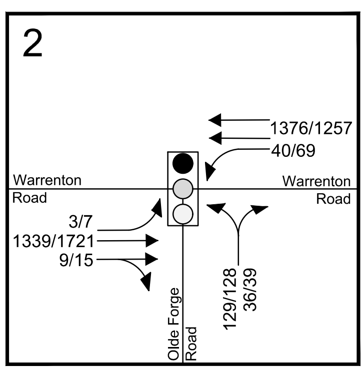
Traffic Signal

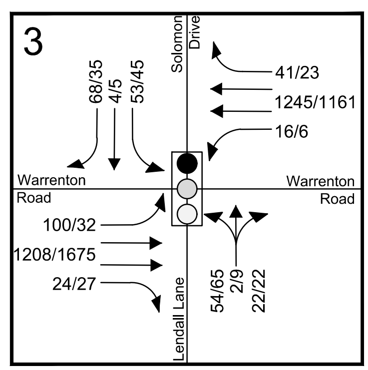
Stop Sign

Represents One Travel Lane

xx/yy AM/PM Peak Hour Volume





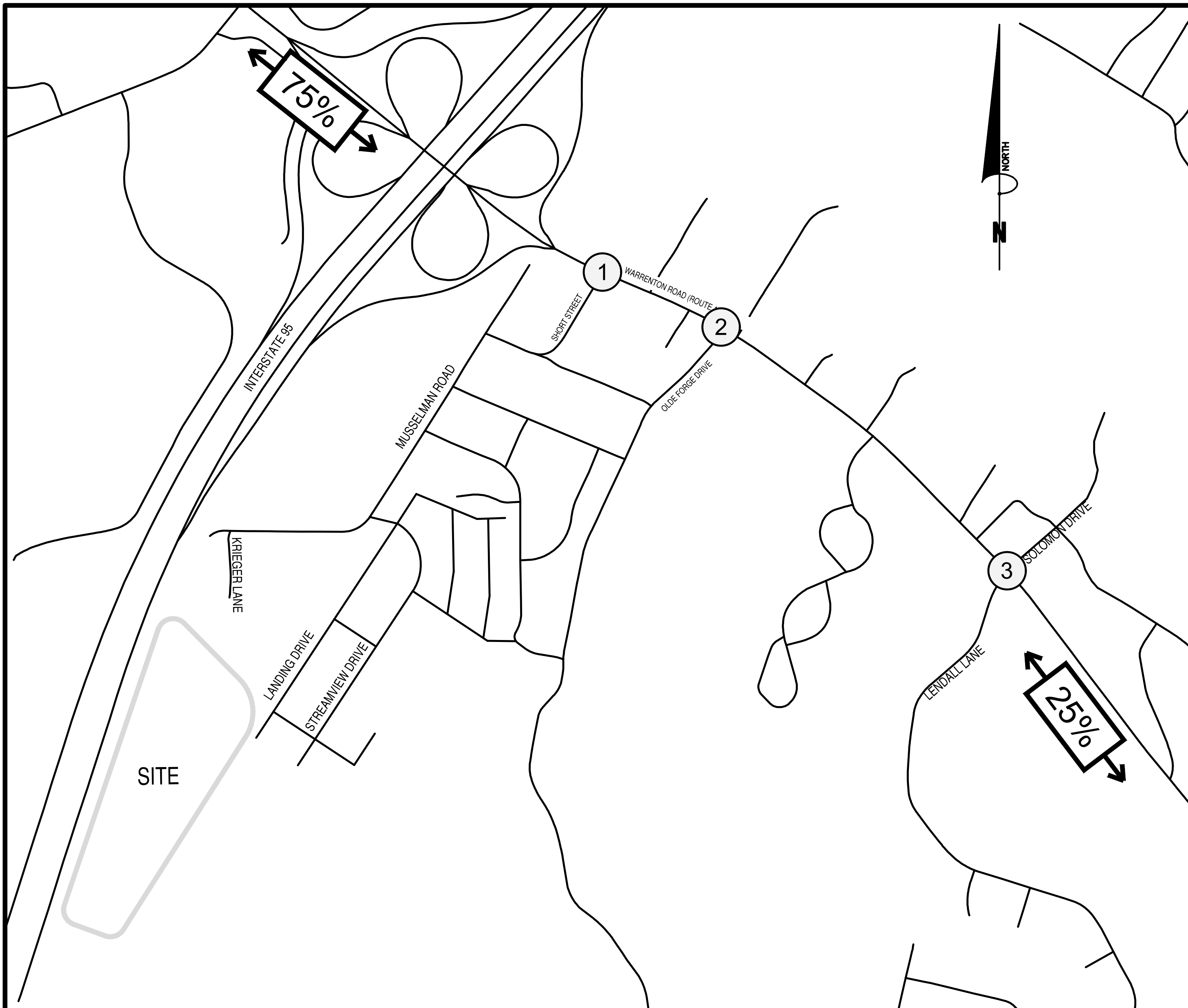


Bowman CONSULTING

Existing Conditions with Relocated Signal (2018)
Rappahannock Landing
Stafford County, Virginia

Exhibit 2

NOT TO SCALE



LEGEND

XX% Overall Traffic Distribution (to and from)

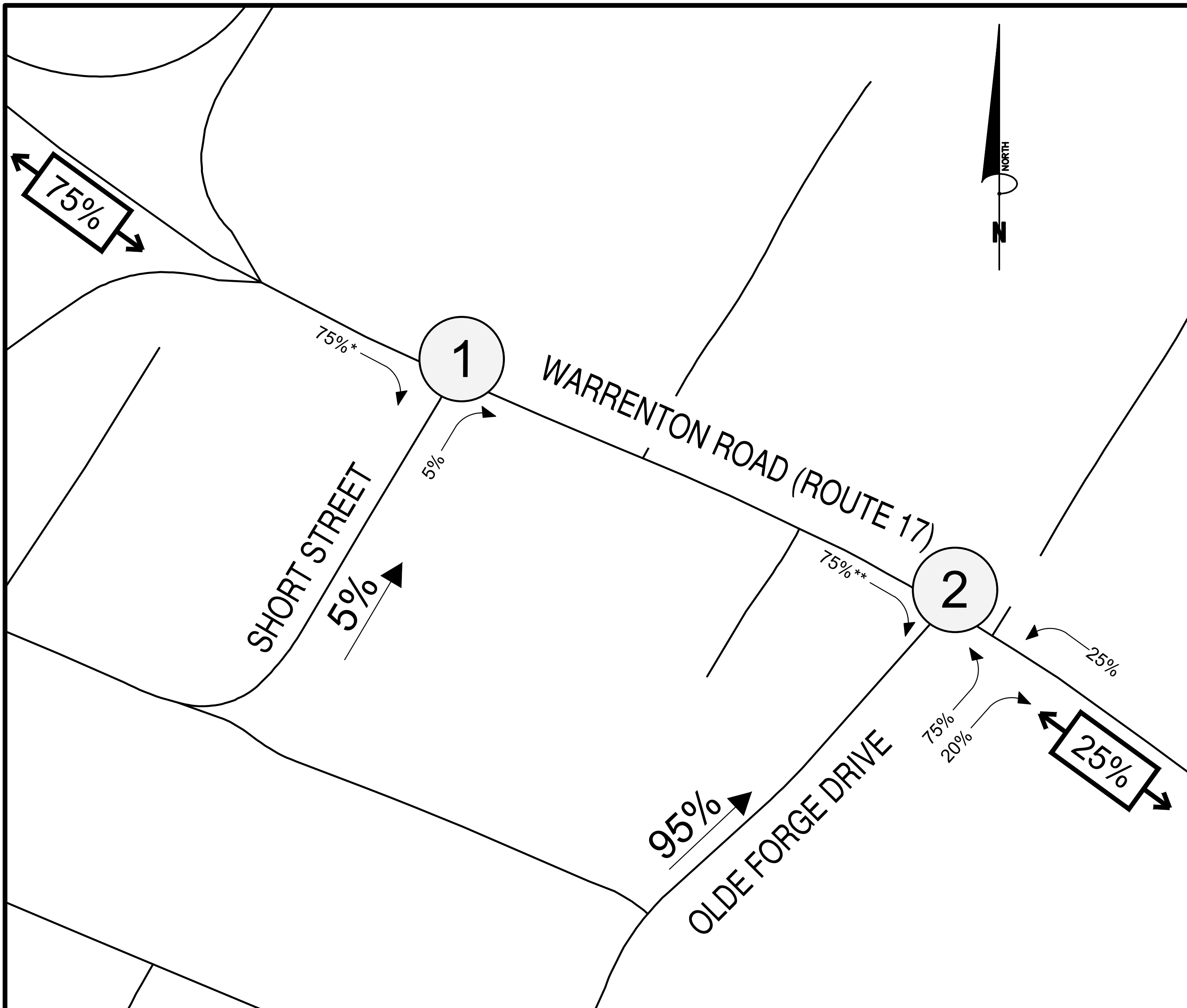
Bowman CONSULTING

Site Generated Trip Distribution - Route 17

Rappahannock Landing
Stafford County, Virginia

Exhibit 3

NOT TO SCALE



LEGEND

XX% Overall Traffic Distribution (to and from)

* Incoming Trip Distribution for Rappahannock Landing Section 5

** Incoming Trip Distribution for Rappahannock Landing Sections 2-4

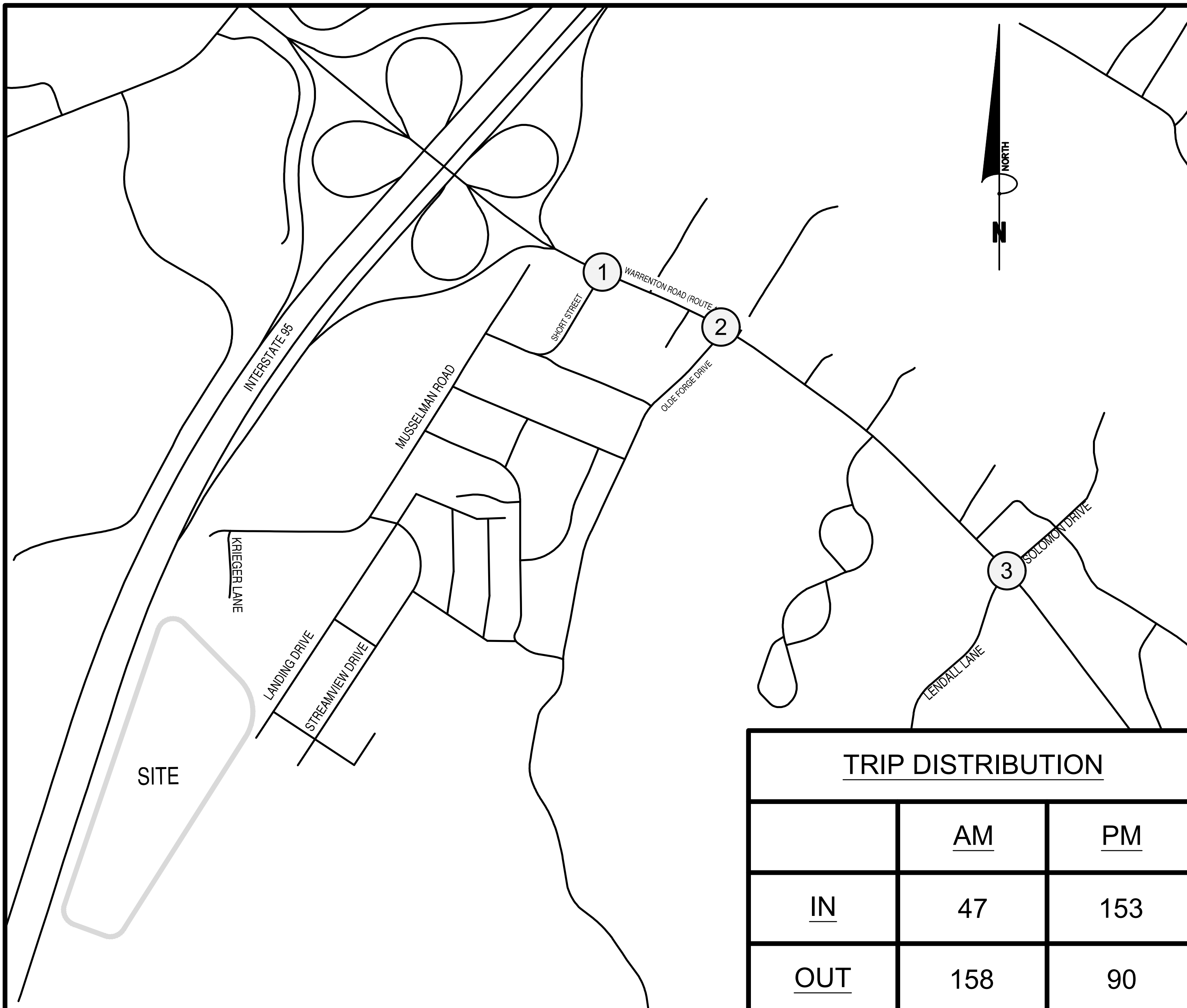
Bowman CONSULTING

Site Generated Trip Distribution - Intersections 1 & 2

Rappahannock Landing
Stafford County, Virginia

Exhibit 4

NOT TO SCALE



TRIP DISTRIBUTION

	AM	PM
IN	47	153
OUT	158	90

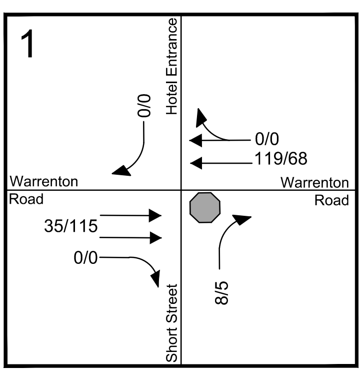
LEGEND

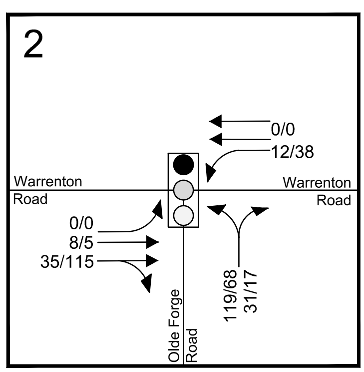
Traffic Signal

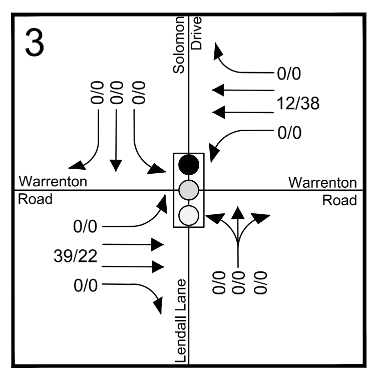
Stop Sign

Represents One Travel Lane

xx/yy AM/PM Peak Hour Volume







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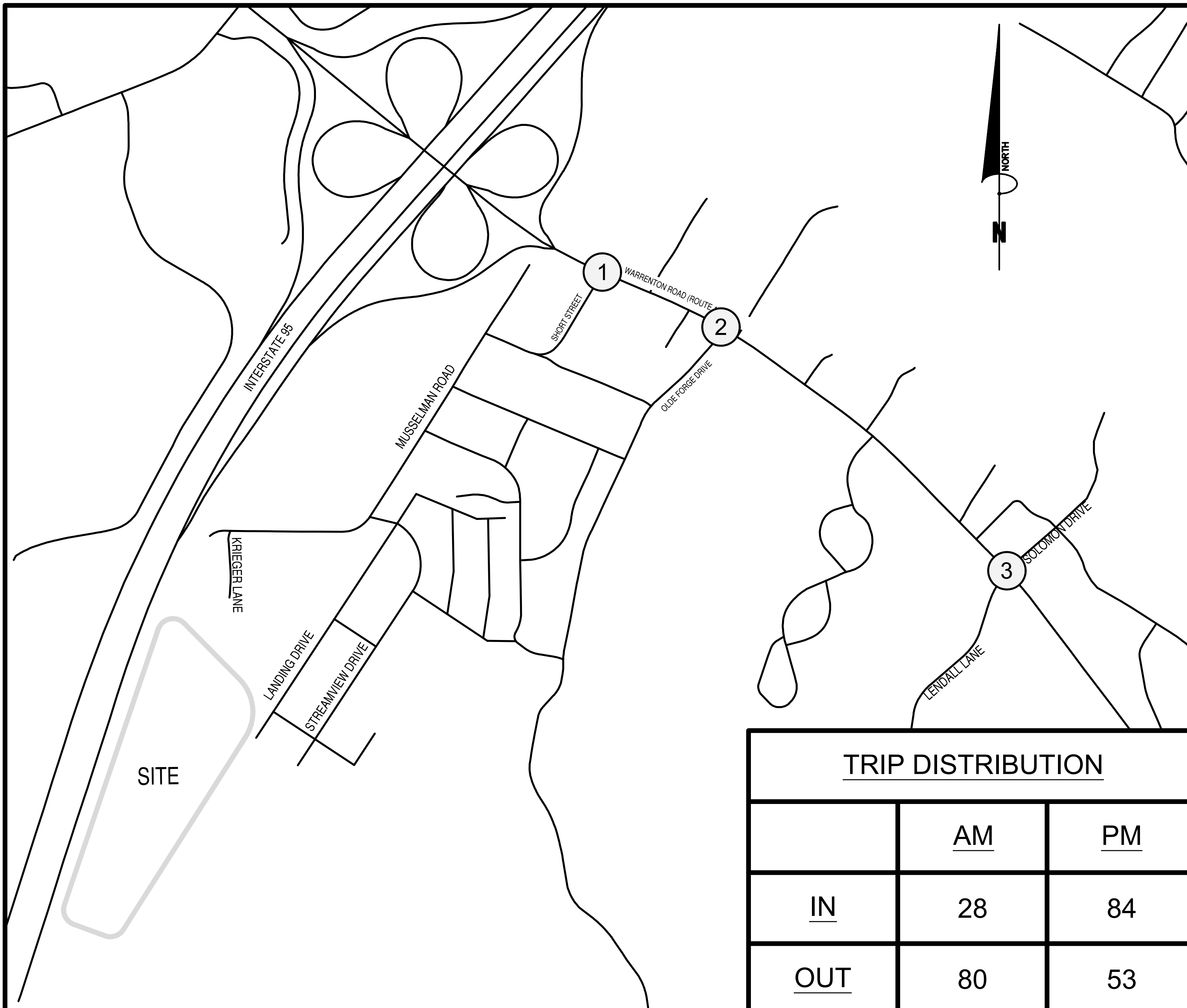
Background Development Trip Generation

Rappahannock Landing
Stafford County, Virginia

Exhibit 5

NOT TO SCALE



TRIP DISTRIBUTION

	AM	PM
IN	28	84
OUT	80	53

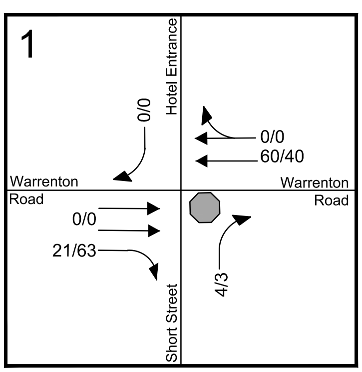
LEGEND

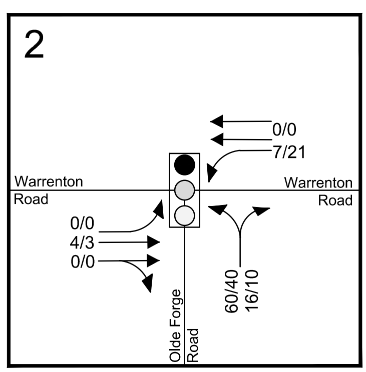
Traffic Signal

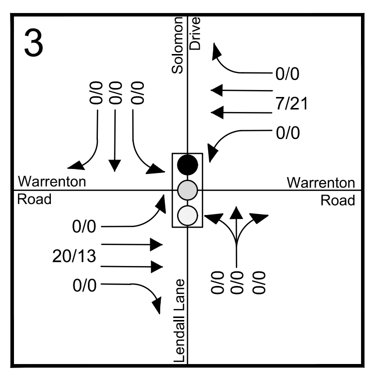
Stop Sign

Represents One Travel Lane

xx/yy AM/PM Peak Hour Volume





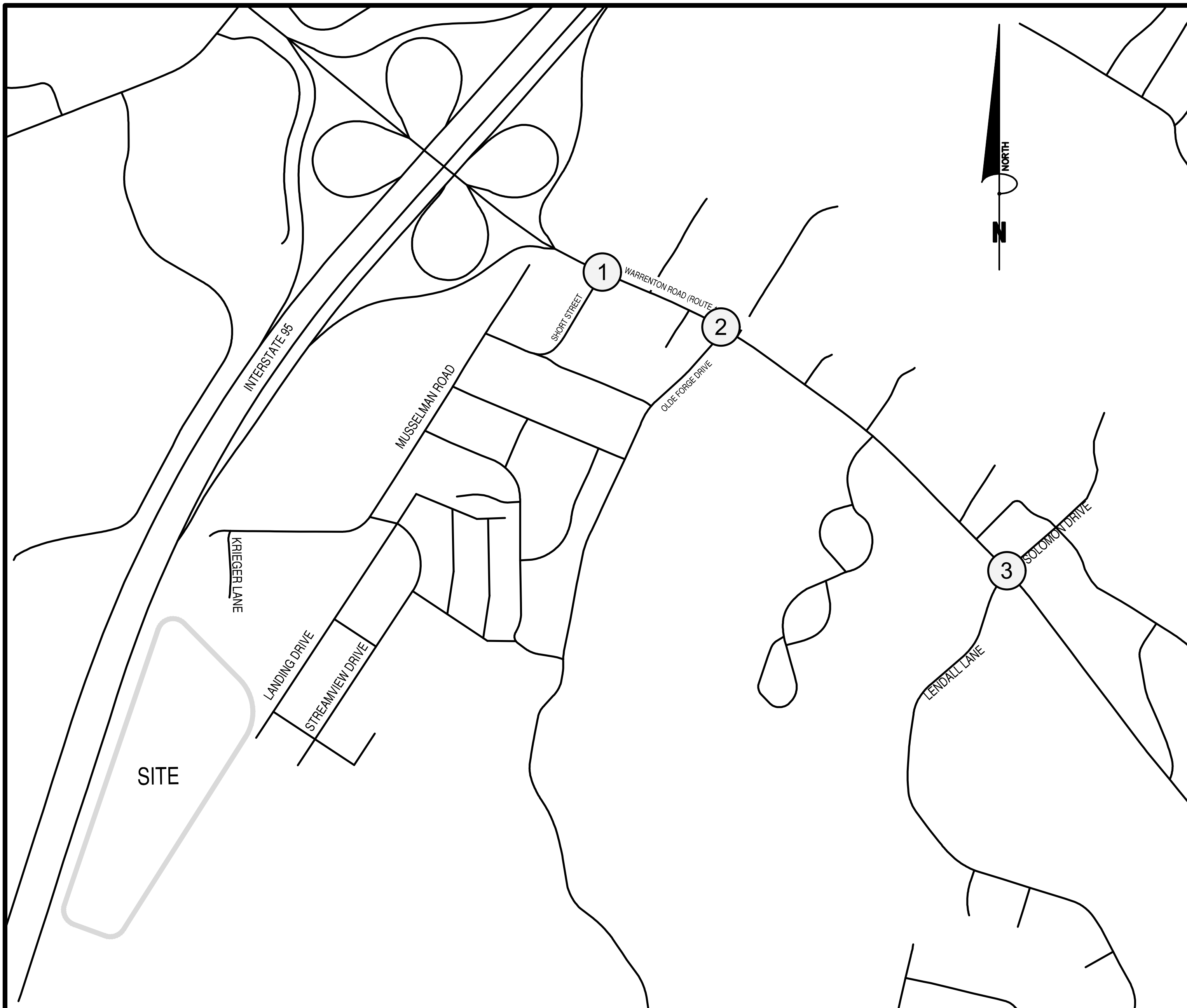


Bowman CONSULTING

Site Generated Trips
Rappahannock Landing
Stafford County, Virginia

Exhibit 6

NOT TO SCALE



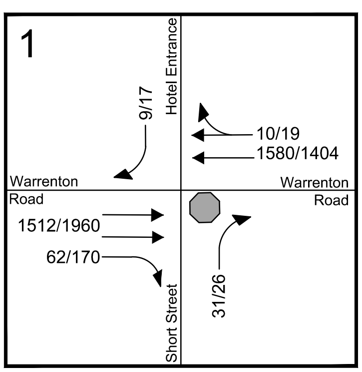
LEGEND

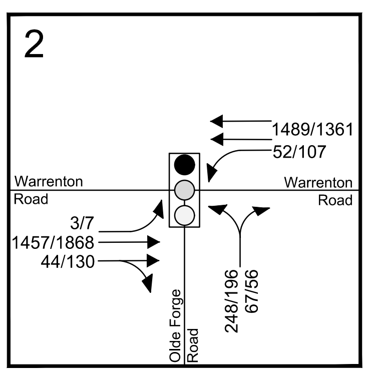
Traffic Signal

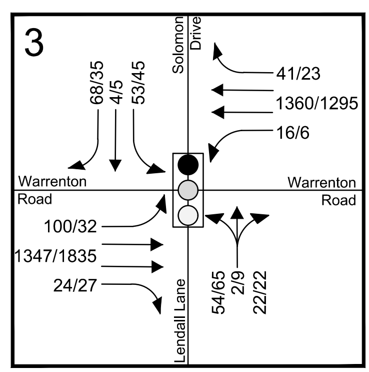
Stop Sign

Represents One Travel Lane

xx/yy AM/PM Peak Hour Volume





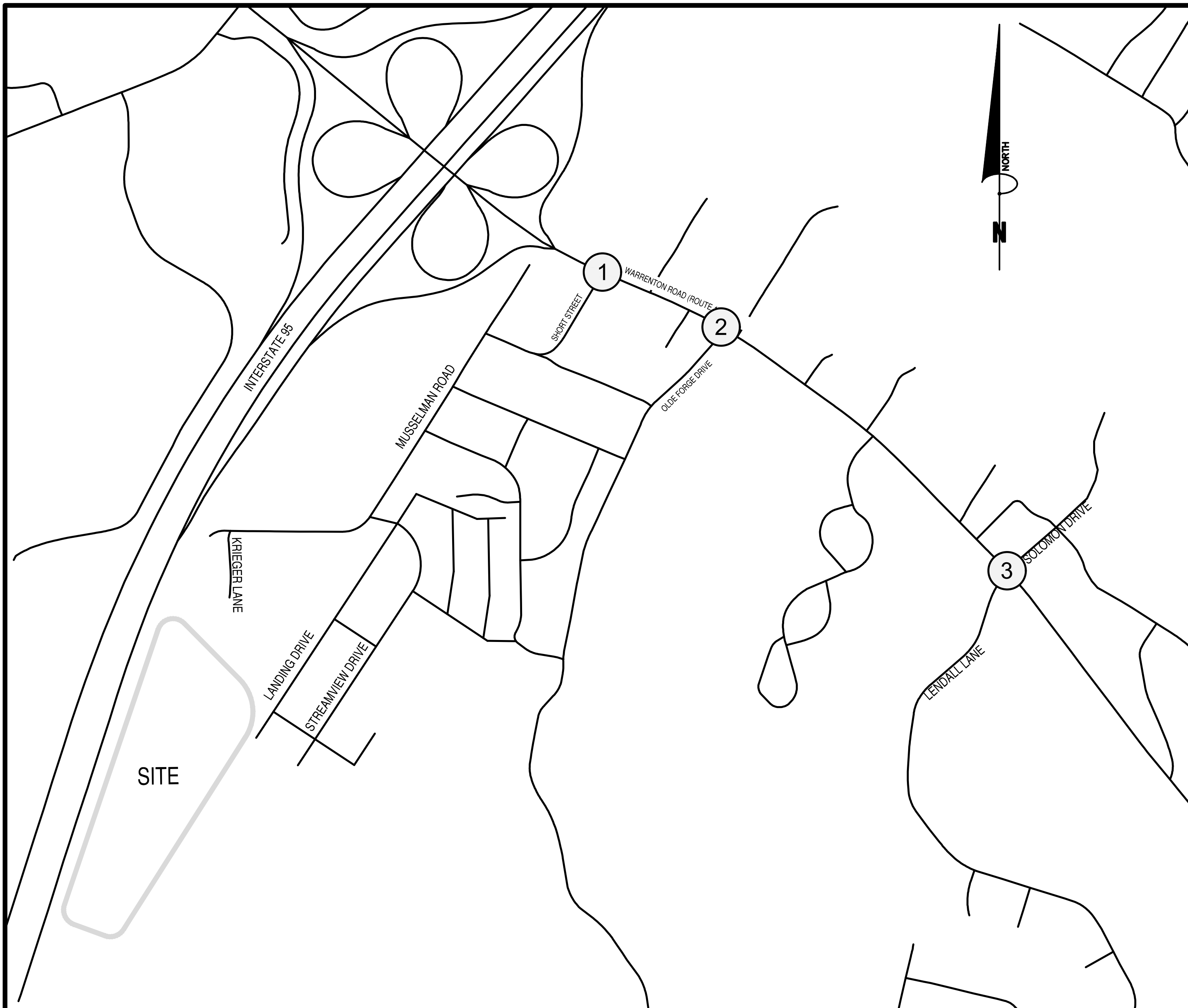


Bowman
CONSULTING

Total No Build Conditions (2022)
Rappahannock Landing
Stafford County, Virginia

Exhibit 7

NOT TO SCALE



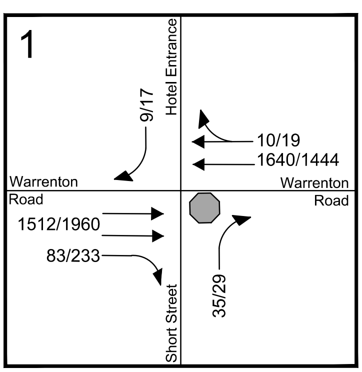
LEGEND

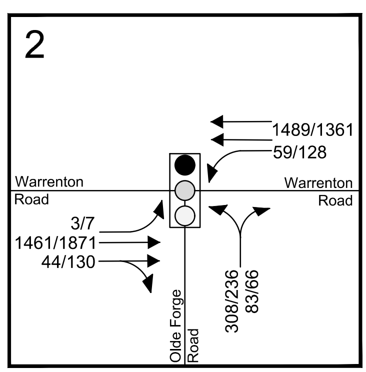
Traffic Signal

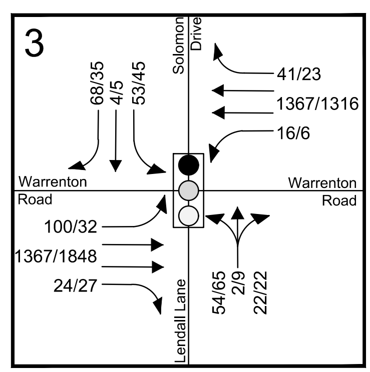
Stop Sign

Represents One Travel Lane

xx/yy AM/PM Peak Hour Volume





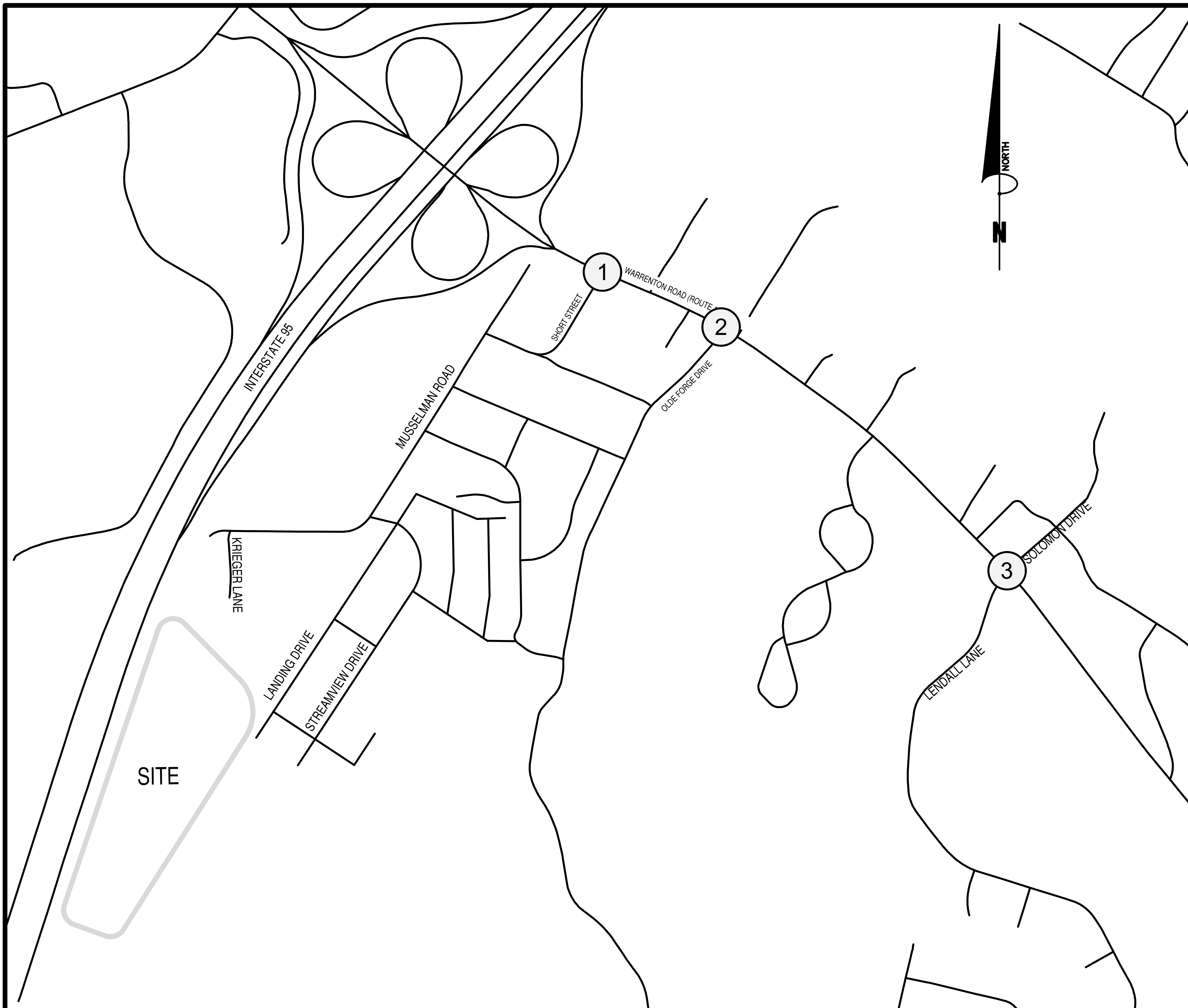


Bowman CONSULTING

Total Build Out Conditions (2022)
Rappahannock Landing
Stafford County, Virginia

Exhibit 8

NOT TO SCALE



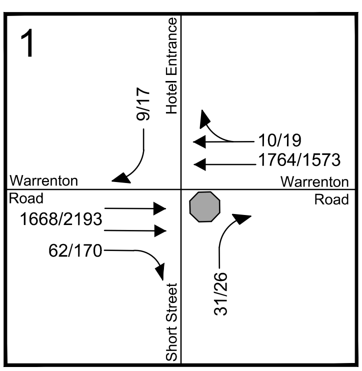
LEGEND

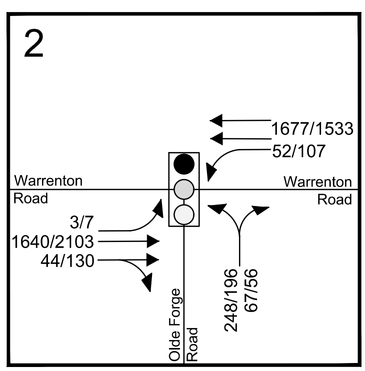
Traffic Signal

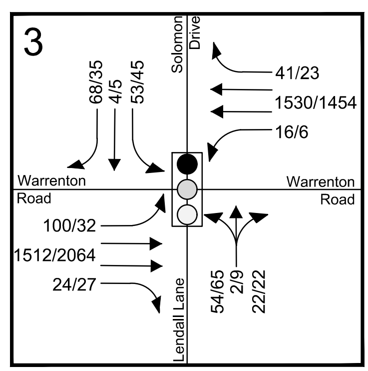
Stop Sign

Represents One Travel Lane

xx/yy AM/PM Peak Hour Volume





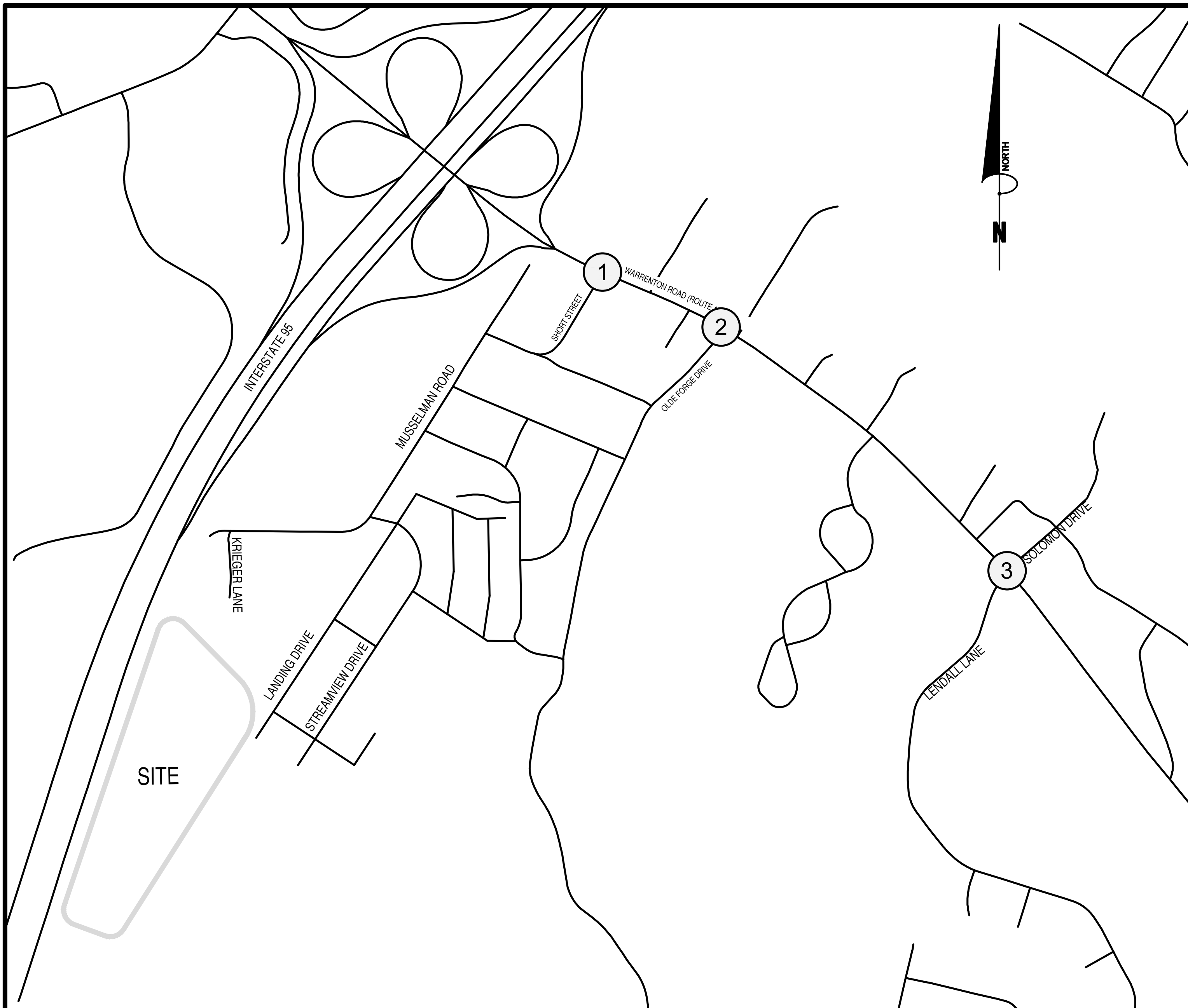


Bowman CONSULTING

Total No Build Conditions (2028)
Rappahannock Landing
Stafford County, Virginia

Exhibit 9

NOT TO SCALE



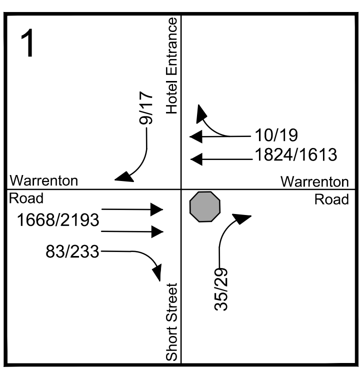
LEGEND

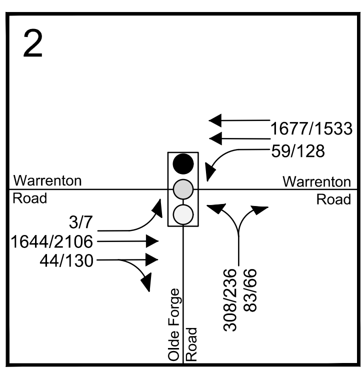
Traffic Signal

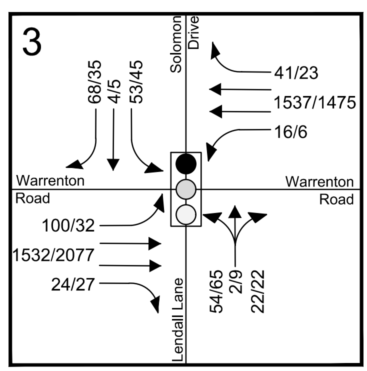
Stop Sign

Represents One Travel Lane

xx/yy AM/PM Peak Hour Volume







Bowman CONSULTING

Total Build Out Conditions (2028)
Rappahannock Landing
Stafford County, Virginia

Exhibit 10

NOT TO SCALE

Appendix E: Existing Conditions (2018) Capacity Analysis

HCM Signalized Intersection Capacity Analysis
14: Short St/Short St (1034) & Warrenton Rd (Rte.17)

03/27/2018

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (vph)	3	1362	62	11	1350	4	122	3	23	2	3	4
Future Volume (vph)	3	1362	62	11	1350	4	122	3	23	2	3	4
Ideal Flow (vphpl)	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900
Grade (%)		0%			1%			-2%			3%	
Total Lost time (s)		4.0	4.0	4.0	4.0			4.0	4.0		4.0	
Lane Util. Factor		0.95	1.00	1.00	0.95			1.00	1.00		1.00	
Frt		1.00	0.85	1.00	1.00			1.00	0.85		0.94	
Flt Protected		1.00	1.00	0.95	1.00			0.95	1.00		0.99	
Satd. Flow (prot)		3438	1583	1761	3420			1794	1599		1706	
Flt Permitted		0.95	1.00	0.09	1.00			0.95	1.00		0.99	
Satd. Flow (perm)		3273	1583	169	3420			1794	1599		1706	
Peak-hour factor, PHF	0.91	0.91	0.91	0.91	0.91	0.91	0.91	0.91	0.91	0.91	0.91	0.91
Adj. Flow (vph)	3	1497	68	12	1484	4	134	3	25	2	3	4
RTOR Reduction (vph)	0	0	27	0	0	0	0	0	21	0	4	0
Lane Group Flow (vph)	0	1500	41	12	1488	0	0	137	4	0	5	0
Heavy Vehicles (%)	2%	5%	2%	2%	5%	2%	2%	2%	2%	2%	2%	2%
Turn Type	Perm	NA	Perm	pm+pt	NA		Split	NA	Perm	Split	NA	
Protected Phases		6		5	2		4	4		3	3	
Permitted Phases	6		6	2					4			
Actuated Green, G (s)		56.5	56.5	66.1	66.1			12.1	12.1		1.2	
Effective Green, g (s)		60.8	60.8	70.4	70.4			14.5	14.5		3.1	
Actuated g/C Ratio		0.61	0.61	0.70	0.70			0.14	0.14		0.03	
Clearance Time (s)		8.3	8.3	8.3	8.3			6.4	6.4		5.9	
Vehicle Extension (s)		3.0	3.0	3.0	3.0			3.0	3.0		3.0	
Lane Grp Cap (vph)		1989	962	208	2407			260	231		52	
v/s Ratio Prot				0.00	c0.44			c0.08			c0.00	
v/s Ratio Perm		c0.46	0.03	0.04					0.00			
v/c Ratio		0.75	0.04	0.06	0.62			0.53	0.02		0.10	
Uniform Delay, d1		14.2	7.9	9.3	7.8			39.6	36.6		47.1	
Progression Factor		1.00	1.00	0.31	0.32			1.00	1.00		1.00	
Incremental Delay, d2		2.7	0.1	0.1	0.9			1.9	0.0		0.8	
Delay (s)		16.9	8.0	3.0	3.4			41.5	36.7		47.9	
Level of Service		B	A	A	A			D	D		D	
Approach Delay (s)		16.5			3.4			40.8			47.9	
Approach LOS		B			A			D			D	

Intersection Summary			
HCM 2000 Control Delay	11.7	HCM 2000 Level of Service	B
HCM 2000 Volume to Capacity ratio	0.69		
Actuated Cycle Length (s)	100.0	Sum of lost time (s)	16.0
Intersection Capacity Utilization	61.6%	ICU Level of Service	B
Analysis Period (min)	15		

c Critical Lane Group

Intersection: 14: Short St/Short St (1034) & Warrenton Rd (Rte.17)

Movement	EB	EB	EB	WB	WB	WB	NB	NB	SB
Directions Served	LT	T	R	L	T	TR	LT	R	LTR
Maximum Queue (ft)	233	256	136	21	163	125	114	26	29
Average Queue (ft)	161	147	38	4	64	64	88	10	6
95th Queue (ft)	256	252	122	18	165	142	114	32	25
Link Distance (ft)	233	233			334	334		423	403
Upstream Blk Time (%)	0	2							
Queuing Penalty (veh)	3	15							
Storage Bay Dist (ft)			112	135			245		
Storage Blk Time (%)		7			2				
Queuing Penalty (veh)		5			0				

Movement	SET	SER	NWL	NWT	NEL	NER
Lane Configurations						
Traffic Volume (veh/h)	1339	9	29	1376	4	36
Future Volume (Veh/h)	1339	9	29	1376	4	36
Sign Control	Free			Free	Stop	
Grade	-1%			1%	0%	
Peak Hour Factor	1.00	1.00	1.00	1.00	1.00	1.00
Hourly flow rate (vph)	1339	9	29	1376	4	36
Pedestrians						
Lane Width (ft)						
Walking Speed (ft/s)						
Percent Blockage						
Right turn flare (veh)						6
Median type	TWLTL			TWLTL		
Median storage veh)	2			2		
Upstream signal (ft)	415					
pX, platoon unblocked			0.70		0.70	0.70
vC, conflicting volume			1348		2090	674
vC1, stage 1 conf vol					1344	
vC2, stage 2 conf vol					746	
vCu, unblocked vol			639		1699	0
tC, single (s)			4.1		6.8	6.9
tC, 2 stage (s)					5.8	
tF (s)			2.2		3.5	3.3
p0 queue free %			96		98	95
cM capacity (veh/h)			658		261	759

Direction, Lane #	SE 1	SE 2	NW 1	NW 2	NW 3	NE 1
Volume Total	893	455	29	688	688	40
Volume Left	0	0	29	0	0	4
Volume Right	0	9	0	0	0	36
cSH	1700	1700	658	1700	1700	843
Volume to Capacity	0.53	0.27	0.04	0.40	0.40	0.05
Queue Length 95th (ft)	0	0	3	0	0	4
Control Delay (s)	0.0	0.0	10.7	0.0	0.0	10.9
Lane LOS			B			B
Approach Delay (s)	0.0		0.2			10.9
Approach LOS						B

Intersection Summary			
Average Delay	0.3		
Intersection Capacity Utilization	48.0%	ICU Level of Service	A
Analysis Period (min)	15		

Intersection: 1: Olde Forge Road & Warrenton Rd (Rte.17)

Movement	SE	NW	NE
Directions Served	T	L	R
Maximum Queue (ft)	54	53	30
Average Queue (ft)	11	25	24
95th Queue (ft)	46	53	43
Link Distance (ft)	334		
Upstream Blk Time (%)			
Queuing Penalty (veh)			
Storage Bay Dist (ft)		150	150
Storage Blk Time (%)			
Queuing Penalty (veh)			

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (vph)	100	1208	24	16	1245	41	54	2	22	53	4	68
Future Volume (vph)	100	1208	24	16	1245	41	54	2	22	53	4	68
Ideal Flow (vphpl)	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900
Grade (%)		-1%			1%			1%			2%	
Total Lost time (s)	4.0	4.0	4.0	4.0	4.0	4.0		4.0		4.0	4.0	4.0
Lane Util. Factor	1.00	0.95	1.00	1.00	0.95	1.00		1.00		1.00	1.00	1.00
Frt	1.00	1.00	0.85	1.00	1.00	0.85		0.96		1.00	1.00	0.85
Flt Protected	0.95	1.00	1.00	0.95	1.00	1.00		0.97		0.95	1.00	1.00
Satd. Flow (prot)	1778	3455	1591	1761	3421	1575		1723		1752	1844	1567
Flt Permitted	0.07	1.00	1.00	0.12	1.00	1.00		0.97		0.95	1.00	1.00
Satd. Flow (perm)	138	3455	1591	215	3421	1575		1723		1752	1844	1567
Peak-hour factor, PHF	0.88	0.88	0.88	0.88	0.88	0.88	0.88	0.88	0.88	0.88	0.88	0.88
Adj. Flow (vph)	114	1373	27	18	1415	47	61	2	25	60	5	77
RTOR Reduction (vph)	0	0	12	0	0	23	0	15	0	0	0	69
Lane Group Flow (vph)	114	1373	15	18	1415	24	0	73	0	60	5	8
Heavy Vehicles (%)	2%	5%	2%	2%	5%	2%	2%	2%	2%	2%	2%	2%
Turn Type	pm+pt	NA	Perm	pm+pt	NA	Perm	Split	NA		Split	NA	Perm
Protected Phases	1	6		5	2		3	3		4	4	
Permitted Phases	6		6	2		2						4
Actuated Green, G (s)	59.4	51.6	51.6	49.0	46.4	46.4		8.1		7.1	7.1	7.1
Effective Green, g (s)	66.0	55.5	55.5	56.8	50.3	50.3		11.3		10.7	10.7	10.7
Actuated g/C Ratio	0.66	0.56	0.56	0.57	0.50	0.50		0.11		0.11	0.11	0.11
Clearance Time (s)	7.9	7.9	7.9	7.9	7.9	7.9		7.2		7.6	7.6	7.6
Vehicle Extension (s)	3.0	3.0	3.0	3.0	3.0	3.0		3.0		3.0	3.0	3.0
Lane Grp Cap (vph)	282	1917	883	222	1720	792		194		187	197	167
v/s Ratio Prot	c0.05	c0.40		0.01	c0.41			c0.04		c0.03	0.00	
v/s Ratio Perm	0.22		0.01	0.04		0.02						0.01
v/c Ratio	0.40	0.72	0.02	0.08	0.82	0.03		0.38		0.32	0.03	0.05
Uniform Delay, d1	14.9	16.4	10.0	11.7	21.1	12.5		41.1		41.3	40.0	40.1
Progression Factor	1.44	0.61	1.00	1.00	1.00	1.00		1.00		1.00	1.00	1.00
Incremental Delay, d2	0.8	2.0	0.0	0.2	4.6	0.1		1.2		1.0	0.1	0.1
Delay (s)	22.2	12.0	10.0	11.8	25.7	12.6		42.3		42.3	40.0	40.2
Level of Service	C	B	B	B	C	B		D		D	D	D
Approach Delay (s)		12.8			25.1			42.3			41.1	
Approach LOS		B			C			D			D	

Intersection Summary			
HCM 2000 Control Delay	20.5	HCM 2000 Level of Service	C
HCM 2000 Volume to Capacity ratio	0.64		
Actuated Cycle Length (s)	100.0	Sum of lost time (s)	16.0
Intersection Capacity Utilization	61.1%	ICU Level of Service	B
Analysis Period (min)	15		

c Critical Lane Group

Intersection: 15: Lendall Lane/Solomon Drive & Warrenton Rd (Rte.17)

Movement	EB	EB	EB	WB	WB	WB	NB	SB	SB	SB
Directions Served	L	T	T	L	T	T	LTR	L	T	R
Maximum Queue (ft)	88	45	40	6	178	165	72	72	52	46
Average Queue (ft)	49	25	16	1	85	85	46	42	18	27
95th Queue (ft)	92	58	47	5	181	161	93	70	55	52
Link Distance (ft)		2728	2728		4517	4517	517		674	
Upstream Blk Time (%)										
Queuing Penalty (veh)										
Storage Bay Dist (ft)	170			105				125		140
Storage Blk Time (%)			0		4					
Queuing Penalty (veh)			0		1					

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (vph)	7	1697	170	15	1234	9	115	3	21	3	0	14
Future Volume (vph)	7	1697	170	15	1234	9	115	3	21	3	0	14
Ideal Flow (vphpl)	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900
Grade (%)		0%			1%			-2%			3%	
Total Lost time (s)		4.0	4.0	4.0	4.0			4.0	4.0		4.0	
Lane Util. Factor		0.95	1.00	1.00	0.95			1.00	1.00		1.00	
Frt		1.00	0.85	1.00	1.00			1.00	0.85		0.89	
Flt Protected		1.00	1.00	0.95	1.00			0.95	1.00		0.99	
Satd. Flow (prot)		3438	1583	1761	3418			1794	1599		1617	
Flt Permitted		0.95	1.00	0.06	1.00			0.95	1.00		0.99	
Satd. Flow (perm)		3261	1583	118	3418			1794	1599		1617	
Peak-hour factor, PHF	0.98	0.98	0.98	0.98	0.98	0.98	0.98	0.98	0.98	0.98	0.98	0.98
Adj. Flow (vph)	7	1732	173	15	1259	9	117	3	21	3	0	14
RTOR Reduction (vph)	0	0	71	0	0	0	0	0	18	0	16	0
Lane Group Flow (vph)	0	1739	102	15	1268	0	0	120	3	0	1	0
Heavy Vehicles (%)	2%	5%	2%	2%	5%	2%	2%	2%	2%	2%	2%	2%
Turn Type	Perm	NA	Perm	pm+pt	NA		Split	NA	Perm	Split	NA	
Protected Phases		6		5	2		4	4		3	3	
Permitted Phases	6		6	2					4			
Actuated Green, G (s)		54.7	54.7	65.5	65.5			11.7	11.7		2.2	
Effective Green, g (s)		59.0	59.0	69.8	69.8			14.1	14.1		4.1	
Actuated g/C Ratio		0.59	0.59	0.70	0.70			0.14	0.14		0.04	
Clearance Time (s)		8.3	8.3	8.3	8.3			6.4	6.4		5.9	
Vehicle Extension (s)		3.0	3.0	3.0	3.0			3.0	3.0		3.0	
Lane Grp Cap (vph)		1923	933	194	2385			252	225		66	
v/s Ratio Prot				0.01	c0.37			c0.07			c0.00	
v/s Ratio Perm		c0.53	0.06	0.05					0.00			
v/c Ratio		0.90	0.11	0.08	0.53			0.48	0.01		0.01	
Uniform Delay, d1		18.0	9.0	14.0	7.3			39.5	37.0		46.0	
Progression Factor		1.00	1.00	0.27	0.23			1.00	1.00		1.00	
Incremental Delay, d2		7.5	0.2	0.1	0.7			1.4	0.0		0.1	
Delay (s)		25.5	9.2	4.0	2.4			41.0	37.0		46.1	
Level of Service		C	A	A	A			D	D		D	
Approach Delay (s)		24.1			2.4			40.4			46.1	
Approach LOS		C			A			D			D	

Intersection Summary			
HCM 2000 Control Delay	16.6	HCM 2000 Level of Service	B
HCM 2000 Volume to Capacity ratio	0.77		
Actuated Cycle Length (s)	100.0	Sum of lost time (s)	16.0
Intersection Capacity Utilization	71.6%	ICU Level of Service	C
Analysis Period (min)	15		

c Critical Lane Group

Intersection: 14: Short St/Short St (1034) & Warrenton Rd (Rte.17)

Movement	EB	EB	EB	WB	WB	WB	NB	NB	SB
Directions Served	LT	T	R	L	T	TR	LT	R	LTR
Maximum Queue (ft)	233	388	74	42	84	205	134	27	35
Average Queue (ft)	172	225	37	24	48	139	90	11	19
95th Queue (ft)	269	384	78	46	95	240	136	32	44
Link Distance (ft)	233	233	233		336	336		423	505
Upstream Blk Time (%)	1	3							
Queuing Penalty (veh)	4	16							
Storage Bay Dist (ft)				135			245		
Storage Blk Time (%)									
Queuing Penalty (veh)									

Movement	SET	SER	NWL	NWT	NEL	NER
Lane Configurations						
Traffic Volume (veh/h)	1721	15	54	1257	10	39
Future Volume (Veh/h)	1721	15	54	1257	10	39
Sign Control	Free			Free	Stop	
Grade	-1%			1%	0%	
Peak Hour Factor	1.00	1.00	1.00	1.00	1.00	1.00
Hourly flow rate (vph)	1721	15	54	1257	10	39
Pedestrians						
Lane Width (ft)						
Walking Speed (ft/s)						
Percent Blockage						
Right turn flare (veh)						6
Median type	TWLTL			TWLTL		
Median storage veh)	2			2		
Upstream signal (ft)	415					
pX, platoon unblocked			0.56		0.56	0.56
vC, conflicting volume			1736		2465	868
vC1, stage 1 conf vol					1728	
vC2, stage 2 conf vol					736	
vCu, unblocked vol			752		2048	0
tC, single (s)			4.1		6.8	6.9
tC, 2 stage (s)					5.8	
tF (s)			2.2		3.5	3.3
p0 queue free %			89		95	94
cM capacity (veh/h)			480		198	610

Direction, Lane #	SE 1	SE 2	NW 1	NW 2	NW 3	NE 1
Volume Total	1147	589	54	628	628	49
Volume Left	0	0	54	0	0	10
Volume Right	0	15	0	0	0	39
cSH	1700	1700	480	1700	1700	766
Volume to Capacity	0.67	0.35	0.11	0.37	0.37	0.06
Queue Length 95th (ft)	0	0	9	0	0	5
Control Delay (s)	0.0	0.0	13.4	0.0	0.0	13.9
Lane LOS			B			B
Approach Delay (s)	0.0		0.6			13.9
Approach LOS						B

Intersection Summary			
Average Delay	0.5		
Intersection Capacity Utilization	58.0%	ICU Level of Service	B
Analysis Period (min)	15		

Intersection: 1: Old Forge Road & Warrenton Rd (Rte.17)

Movement	NW	NE	NE
Directions Served	L	L	R
Maximum Queue (ft)	51	72	53
Average Queue (ft)	32	40	43
95th Queue (ft)	48	84	60
Link Distance (ft)		712	
Upstream Blk Time (%)			
Queuing Penalty (veh)			
Storage Bay Dist (ft)	150		150
Storage Blk Time (%)			
Queuing Penalty (veh)			

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (vph)	32	1675	27	6	1161	23	65	9	22	45	5	35
Future Volume (vph)	32	1675	27	6	1161	23	65	9	22	45	5	35
Ideal Flow (vphpl)	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900
Grade (%)		-1%			1%			1%			2%	
Total Lost time (s)	4.0	4.0	4.0	4.0	4.0	4.0		4.0		4.0	4.0	4.0
Lane Util. Factor	1.00	0.95	1.00	1.00	0.95	1.00		1.00		1.00	1.00	1.00
Frt	1.00	1.00	0.85	1.00	1.00	0.85		0.97		1.00	1.00	0.85
Flt Protected	0.95	1.00	1.00	0.95	1.00	1.00		0.97		0.95	1.00	1.00
Satd. Flow (prot)	1778	3455	1591	1761	3421	1575		1737		1752	1844	1567
Flt Permitted	0.14	1.00	1.00	0.07	1.00	1.00		0.97		0.95	1.00	1.00
Satd. Flow (perm)	262	3455	1591	132	3421	1575		1737		1752	1844	1567
Peak-hour factor, PHF	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95
Adj. Flow (vph)	34	1763	28	6	1222	24	68	9	23	47	5	37
RTOR Reduction (vph)	0	0	12	0	0	11	0	11	0	0	0	34
Lane Group Flow (vph)	34	1763	16	6	1222	13	0	89	0	47	5	3
Heavy Vehicles (%)	2%	5%	2%	2%	5%	2%	2%	2%	2%	2%	2%	2%
Turn Type	pm+pt	NA	Perm	pm+pt	NA	Perm	Split	NA		Split	NA	Perm
Protected Phases	1	6		5	2		3	3		4	4	
Permitted Phases	6		6	2		2						4
Actuated Green, G (s)	58.5	54.8	54.8	53.5	52.3	52.3		7.6		5.8	5.8	5.8
Effective Green, g (s)	66.3	58.7	58.7	61.3	56.2	56.2		10.8		9.4	9.4	9.4
Actuated g/C Ratio	0.66	0.59	0.59	0.61	0.56	0.56		0.11		0.09	0.09	0.09
Clearance Time (s)	7.9	7.9	7.9	7.9	7.9	7.9		7.2		7.6	7.6	7.6
Vehicle Extension (s)	3.0	3.0	3.0	3.0	3.0	3.0		3.0		3.0	3.0	3.0
Lane Grp Cap (vph)	288	2028	933	163	1922	885		187		164	173	147
v/s Ratio Prot	c0.01	c0.51		0.00	0.36			c0.05		c0.03	0.00	
v/s Ratio Perm	0.07		0.01	0.02		0.01						0.00
v/c Ratio	0.12	0.87	0.02	0.04	0.64	0.02		0.48		0.29	0.03	0.02
Uniform Delay, d1	8.6	17.4	8.6	14.6	14.9	9.7		41.9		42.2	41.2	41.1
Progression Factor	0.14	0.32	1.00	1.00	1.00	1.00		1.00		1.00	1.00	1.00
Incremental Delay, d2	0.1	3.7	0.0	0.1	1.6	0.0		1.9		1.0	0.1	0.1
Delay (s)	1.3	9.3	8.6	14.7	16.5	9.7		43.9		43.1	41.2	41.2
Level of Service	A	A	A	B	B	A		D		D	D	D
Approach Delay (s)		9.1			16.4			43.9			42.2	
Approach LOS		A			B			D			D	

Intersection Summary			
HCM 2000 Control Delay	13.9	HCM 2000 Level of Service	B
HCM 2000 Volume to Capacity ratio	0.71		
Actuated Cycle Length (s)	100.0	Sum of lost time (s)	16.0
Intersection Capacity Utilization	65.1%	ICU Level of Service	C
Analysis Period (min)	15		

c Critical Lane Group

Intersection: 15: Lendall Lane/Solomon Drive & Warrenton Rd (Rte.17)

Movement	EB	EB	EB	EB	WB	WB	WB	NB	SB	SB
Directions Served	L	T	T	R	L	T	T	LTR	L	R
Maximum Queue (ft)	68	105	103	15	6	204	211	72	49	52
Average Queue (ft)	22	54	69	3	2	100	134	51	27	20
95th Queue (ft)	63	102	105	13	6	197	216	83	65	52
Link Distance (ft)		2727	2727			4517	4517	517		
Upstream Blk Time (%)										
Queuing Penalty (veh)										
Storage Bay Dist (ft)	170			50	105				125	140
Storage Blk Time (%)			11			3				
Queuing Penalty (veh)			3			0				

Appendix F: No Build and Build (2022) Capacity Analysis

HCM Unsignalized Intersection Capacity Analysis
14: Short St/Short St (1034) & Warrenton Rd (Rte.17)

03/29/2018

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (veh/h)	0	1512	62	0	1580	10	0	0	31	0	0	9
Future Volume (Veh/h)	0	1512	62	0	1580	10	0	0	31	0	0	9
Sign Control		Free			Free			Stop			Stop	
Grade		0%			1%			-2%			3%	
Peak Hour Factor	0.91	0.91	0.91	0.91	0.91	0.91	0.91	0.91	0.91	0.91	0.91	0.91
Hourly flow rate (vph)	0	1662	68	0	1736	11	0	0	34	0	0	10
Pedestrians												
Lane Width (ft)												
Walking Speed (ft/s)												
Percent Blockage												
Right turn flare (veh)												
Median type		None			None							
Median storage veh)												
Upstream signal (ft)					418							
pX, platoon unblocked	0.70						0.70	0.70		0.70	0.70	0.70
vC, conflicting volume	1747			1730			2540	3409	831	2606	3472	874
vC1, stage 1 conf vol												
vC2, stage 2 conf vol												
vCu, unblocked vol	1211			1730			2343	3584	831	2438	3673	0
tC, single (s)	4.1			4.1			7.5	6.5	6.9	7.5	6.5	6.9
tC, 2 stage (s)												
tF (s)	2.2			2.2			3.5	4.0	3.3	3.5	4.0	3.3
p0 queue free %	100			100			100	100	89	100	100	99
cM capacity (veh/h)	400			361			13	4	313	10	3	760

Direction, Lane #	EB 1	EB 2	EB 3	WB 1	WB 2	NB 1	SB 1
Volume Total	831	831	68	1157	590	34	10
Volume Left	0	0	0	0	0	0	0
Volume Right	0	0	68	0	11	34	10
cSH	1700	1700	1700	1700	1700	313	760
Volume to Capacity	0.49	0.49	0.04	0.68	0.35	0.11	0.01
Queue Length 95th (ft)	0	0	0	0	0	9	1
Control Delay (s)	0.0	0.0	0.0	0.0	0.0	17.9	9.8
Lane LOS						C	A
Approach Delay (s)	0.0			0.0		17.9	9.8
Approach LOS						C	A

Intersection Summary			
Average Delay	0.2		
Intersection Capacity Utilization	54.0%	ICU Level of Service	A
Analysis Period (min)	15		

Intersection: 14: Short St/Short St (1034) & Warrenton Rd (Rte.17)

Movement	EB	EB	NB	SB
Directions Served	T	T	R	R
Maximum Queue (ft)	72	121	22	25
Average Queue (ft)	14	38	17	5
95th Queue (ft)	62	120	31	22
Link Distance (ft)	233	233	416	396
Upstream Blk Time (%)				
Queuing Penalty (veh)				
Storage Bay Dist (ft)				
Storage Blk Time (%)				
Queuing Penalty (veh)				

Movement	SEU	SET	SER	NWL	NWT	NEL	NER
Lane Configurations							
Traffic Volume (vph)	3	1457	44	52	1489	248	67
Future Volume (vph)	3	1457	44	52	1489	248	67
Ideal Flow (vphpl)	1900	1900	1900	1900	1900	1900	1900
Grade (%)		-1%			1%	0%	
Total Lost time (s)	4.0	4.0		7.0	4.0	4.0	4.0
Lane Util. Factor	1.00	0.95		1.00	0.95	1.00	1.00
Frt	1.00	1.00		1.00	1.00	1.00	0.85
Flt Protected	0.95	1.00		0.95	1.00	0.95	1.00
Satd. Flow (prot)	1778	3541		1761	3522	1770	1583
Flt Permitted	0.13	1.00		0.08	1.00	0.95	1.00
Satd. Flow (perm)	242	3541		156	3522	1770	1583
Peak-hour factor, PHF	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Adj. Flow (vph)	3	1457	44	52	1489	248	67
RTOR Reduction (vph)	0	2	0	0	0	0	46
Lane Group Flow (vph)	3	1499	0	52	1489	248	21
Turn Type	Perm	NA		pm+pt	NA	Prot	Perm
Protected Phases		6		5	2	4	
Permitted Phases	6			2			4
Actuated Green, G (s)	40.4	40.4		53.4	53.4	26.9	26.9
Effective Green, g (s)	44.7	44.7		53.4	57.7	29.3	29.3
Actuated g/C Ratio	0.47	0.47		0.56	0.61	0.31	0.31
Clearance Time (s)	8.3	8.3		7.0	8.3	6.4	6.4
Vehicle Extension (s)	3.0	3.0		3.0	3.0	3.0	3.0
Lane Grp Cap (vph)	113	1666		189	2139	545	488
v/s Ratio Prot		c0.42		0.02	c0.42	c0.14	
v/s Ratio Perm	0.01			0.14			0.01
v/c Ratio	0.03	0.90		0.28	0.70	0.46	0.04
Uniform Delay, d1	13.5	23.1		19.2	12.7	26.4	23.0
Progression Factor	1.00	1.00		0.60	0.30	1.00	1.00
Incremental Delay, d2	0.4	8.2		0.6	1.3	2.7	0.2
Delay (s)	13.9	31.3		12.0	5.1	29.2	23.2
Level of Service	B	C		B	A	C	C
Approach Delay (s)		31.3			5.4	27.9	
Approach LOS		C			A	C	

Intersection Summary			
HCM 2000 Control Delay	19.1	HCM 2000 Level of Service	B
HCM 2000 Volume to Capacity ratio	0.76		
Actuated Cycle Length (s)	95.0	Sum of lost time (s)	15.0
Intersection Capacity Utilization	63.6%	ICU Level of Service	B
Analysis Period (min)	15		
c Critical Lane Group			

Intersection: 2: Olde Forge Road & Warrenton Rd (Rte.17)

Movement	SE	SE	NW	NW	NW	NE	NE
Directions Served	T	TR	L	T	T	L	R
Maximum Queue (ft)	352	352	90	140	228	137	51
Average Queue (ft)	315	306	52	95	154	99	22
95th Queue (ft)	360	398	87	145	248	154	54
Link Distance (ft)	351	351		2725	2725	873	
Upstream Blk Time (%)	2	2					
Queuing Penalty (veh)	14	15					
Storage Bay Dist (ft)			200				150
Storage Blk Time (%)	39					0	
Queuing Penalty (veh)	1					0	

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (vph)	100	1347	24	16	1360	41	54	2	22	53	4	68
Future Volume (vph)	100	1347	24	16	1360	41	54	2	22	53	4	68
Ideal Flow (vphpl)	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900
Grade (%)		-1%			1%			1%			2%	
Total Lost time (s)	4.0	4.0	4.0	4.0	4.0	4.0		4.0		4.0	4.0	4.0
Lane Util. Factor	1.00	0.95	1.00	1.00	0.95	1.00		1.00		1.00	1.00	1.00
Frt	1.00	1.00	0.85	1.00	1.00	0.85		0.96		1.00	1.00	0.85
Flt Protected	0.95	1.00	1.00	0.95	1.00	1.00		0.97		0.95	1.00	1.00
Satd. Flow (prot)	1778	3455	1591	1761	3421	1575		1723		1752	1844	1567
Flt Permitted	0.07	1.00	1.00	0.10	1.00	1.00		0.97		0.95	1.00	1.00
Satd. Flow (perm)	131	3455	1591	176	3421	1575		1723		1752	1844	1567
Peak-hour factor, PHF	0.88	0.88	0.88	0.88	0.88	0.88	0.88	0.88	0.88	0.88	0.88	0.88
Adj. Flow (vph)	114	1531	27	18	1545	47	61	2	25	60	5	77
RTOR Reduction (vph)	0	0	11	0	0	20	0	15	0	0	0	71
Lane Group Flow (vph)	114	1531	16	18	1545	27	0	73	0	60	5	6
Heavy Vehicles (%)	2%	5%	2%	2%	5%	2%	2%	2%	2%	2%	2%	2%
Turn Type	pm+pt	NA	Perm	pm+pt	NA	Perm	Split	NA		Split	NA	Perm
Protected Phases	1	6		5	2		3	3		4	4	
Permitted Phases	6		6	2		2						4
Actuated Green, G (s)	58.5	53.3	53.3	52.1	50.1	50.1		4.7		4.4	4.4	4.4
Effective Green, g (s)	66.3	57.2	57.2	59.9	54.0	54.0		7.9		8.0	8.0	8.0
Actuated g/C Ratio	0.70	0.60	0.60	0.63	0.57	0.57		0.08		0.08	0.08	0.08
Clearance Time (s)	7.9	7.9	7.9	7.9	7.9	7.9		7.2		7.6	7.6	7.6
Vehicle Extension (s)	3.0	3.0	3.0	3.0	3.0	3.0		3.0		3.0	3.0	3.0
Lane Grp Cap (vph)	249	2080	957	209	1944	895		143		147	155	131
v/s Ratio Prot	c0.04	0.44		0.01	c0.45			c0.04		c0.03	0.00	
v/s Ratio Perm	0.27		0.01	0.05		0.02						0.00
v/c Ratio	0.46	0.74	0.02	0.09	0.79	0.03		0.51		0.41	0.03	0.05
Uniform Delay, d1	13.7	13.5	7.6	9.6	16.1	9.0		41.7		41.3	39.9	40.0
Progression Factor	2.16	0.12	1.00	1.00	1.00	1.00		1.00		1.00	1.00	1.00
Incremental Delay, d2	0.9	1.5	0.0	0.2	3.5	0.1		3.1		1.8	0.1	0.2
Delay (s)	30.4	3.2	7.6	9.8	19.6	9.1		44.8		43.1	40.0	40.2
Level of Service	C	A	A	A	B	A		D		D	D	D
Approach Delay (s)		5.1			19.2			44.8			41.4	
Approach LOS		A			B			D			D	

Intersection Summary			
HCM 2000 Control Delay	14.0	HCM 2000 Level of Service	B
HCM 2000 Volume to Capacity ratio	0.69		
Actuated Cycle Length (s)	95.0	Sum of lost time (s)	16.0
Intersection Capacity Utilization	64.2%	ICU Level of Service	C
Analysis Period (min)	15		

c Critical Lane Group

Intersection: 15: Lendall Ln (1015)/Solomon Drive & Warrenton Rd (Rte.17)

Movement	EB	EB	EB	WB	WB	WB	NB	SB	SB	SB
Directions Served	L	T	T	L	T	T	LTR	L	T	R
Maximum Queue (ft)	195	206	126	18	204	273	54	149	262	164
Average Queue (ft)	88	104	85	17	132	184	38	76	58	71
95th Queue (ft)	180	193	148	20	205	274	73	152	227	150
Link Distance (ft)		2725	2725		2215	2215	526		674	
Upstream Blk Time (%)										
Queuing Penalty (veh)										
Storage Bay Dist (ft)	170			105				125		140
Storage Blk Time (%)		1	14		9	1		19		
Queuing Penalty (veh)		1	3		1	1		14		

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (veh/h)	0	1512	83	0	1640	10	0	0	35	0	0	9
Future Volume (Veh/h)	0	1512	83	0	1640	10	0	0	35	0	0	9
Sign Control		Free			Free			Stop			Stop	
Grade		0%			1%			-2%			3%	
Peak Hour Factor	0.91	0.91	0.91	0.91	0.91	0.91	0.91	0.91	0.91	0.91	0.91	0.91
Hourly flow rate (vph)	0	1662	91	0	1802	11	0	0	38	0	0	10
Pedestrians												
Lane Width (ft)												
Walking Speed (ft/s)												
Percent Blockage												
Right turn flare (veh)												
Median type		None			None							
Median storage veh)												
Upstream signal (ft)					418							
pX, platoon unblocked	0.70						0.70	0.70		0.70	0.70	0.70
vC, conflicting volume	1813			1753			2573	3475	831	2676	3560	906
vC1, stage 1 conf vol												
vC2, stage 2 conf vol												
vCu, unblocked vol	1315			1753			2394	3674	831	2541	3796	28
tC, single (s)	4.1			4.1			7.5	6.5	6.9	7.5	6.5	6.9
tC, 2 stage (s)												
tF (s)	2.2			2.2			3.5	4.0	3.3	3.5	4.0	3.3
p0 queue free %	100			100			100	100	88	100	100	99
cM capacity (veh/h)	368			353			12	3	313	8	3	733

Direction, Lane #	EB 1	EB 2	EB 3	WB 1	WB 2	NB 1	SB 1
Volume Total	831	831	91	1201	612	38	10
Volume Left	0	0	0	0	0	0	0
Volume Right	0	0	91	0	11	38	10
cSH	1700	1700	1700	1700	1700	313	733
Volume to Capacity	0.49	0.49	0.05	0.71	0.36	0.12	0.01
Queue Length 95th (ft)	0	0	0	0	0	10	1
Control Delay (s)	0.0	0.0	0.0	0.0	0.0	18.1	10.0
Lane LOS						C	A
Approach Delay (s)	0.0			0.0		18.1	10.0
Approach LOS						C	A

Intersection Summary			
Average Delay	0.2		
Intersection Capacity Utilization	55.7%	ICU Level of Service	B
Analysis Period (min)	15		

Intersection: 14: Short St/Short St (1034) & Warrenton Rd (Rte.17)

Movement	EB	EB	NB	SB
Directions Served	T	T	R	R
Maximum Queue (ft)	245	245	80	28
Average Queue (ft)	90	104	25	6
95th Queue (ft)	272	294	73	24
Link Distance (ft)	233	233	416	396
Upstream Blk Time (%)	2	3		
Queuing Penalty (veh)	11	18		
Storage Bay Dist (ft)				
Storage Blk Time (%)				
Queuing Penalty (veh)				

Movement	SEU	SET	SER	NWL	NWT	NEL	NER
Lane Configurations							
Traffic Volume (vph)	3	1461	44	59	1489	308	83
Future Volume (vph)	3	1461	44	59	1489	308	83
Ideal Flow (vphpl)	1900	1900	1900	1900	1900	1900	1900
Grade (%)		-1%			1%	0%	
Total Lost time (s)	4.0	4.0		7.0	4.0	4.0	4.0
Lane Util. Factor	1.00	0.95		1.00	0.95	1.00	1.00
Frt	1.00	1.00		1.00	1.00	1.00	0.85
Flt Protected	0.95	1.00		0.95	1.00	0.95	1.00
Satd. Flow (prot)	1778	3541		1761	3522	1770	1583
Flt Permitted	0.14	1.00		0.09	1.00	0.95	1.00
Satd. Flow (perm)	257	3541		162	3522	1770	1583
Peak-hour factor, PHF	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Adj. Flow (vph)	3	1461	44	59	1489	308	83
RTOR Reduction (vph)	0	2	0	0	0	0	58
Lane Group Flow (vph)	3	1503	0	59	1489	308	25
Turn Type	Perm	NA		pm+pt	NA	Prot	Perm
Protected Phases		6		5	2	4	
Permitted Phases	6			2			4
Actuated Green, G (s)	38.9	38.9		53.9	53.9	26.4	26.4
Effective Green, g (s)	43.2	43.2		53.9	58.2	28.8	28.8
Actuated g/C Ratio	0.45	0.45		0.57	0.61	0.30	0.30
Clearance Time (s)	8.3	8.3		7.0	8.3	6.4	6.4
Vehicle Extension (s)	3.0	3.0		3.0	3.0	3.0	3.0
Lane Grp Cap (vph)	116	1610		226	2157	536	479
v/s Ratio Prot		c0.42		0.02	c0.42	c0.17	
v/s Ratio Perm	0.01			0.13			0.02
v/c Ratio	0.03	0.93		0.26	0.69	0.57	0.05
Uniform Delay, d1	14.3	24.5		18.7	12.4	27.9	23.4
Progression Factor	1.00	1.00		0.71	0.30	1.00	1.00
Incremental Delay, d2	0.4	11.4		0.4	1.3	4.4	0.2
Delay (s)	14.7	35.9		13.8	5.0	32.4	23.6
Level of Service	B	D		B	A	C	C
Approach Delay (s)		35.9			5.3	30.5	
Approach LOS		D			A	C	

Intersection Summary			
HCM 2000 Control Delay	21.5	HCM 2000 Level of Service	C
HCM 2000 Volume to Capacity ratio	0.82		
Actuated Cycle Length (s)	95.0	Sum of lost time (s)	15.0
Intersection Capacity Utilization	72.8%	ICU Level of Service	C
Analysis Period (min)	15		
c Critical Lane Group			

Intersection: 2: Olde Forge Road & Warrenton Rd (Rte.17)

Movement	SE	SE	NW	NW	NW	NE	NE
Directions Served	T	TR	L	T	T	L	R
Maximum Queue (ft)	364	357	219	341	398	234	73
Average Queue (ft)	311	317	66	190	227	156	51
95th Queue (ft)	398	399	195	385	435	239	83
Link Distance (ft)	351	351		2725	2725	873	
Upstream Blk Time (%)	11	12					
Queuing Penalty (veh)	88	91					
Storage Bay Dist (ft)			200				150
Storage Blk Time (%)	38			10		11	
Queuing Penalty (veh)	1			6		9	

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (vph)	100	1367	24	16	1367	41	54	2	22	53	4	68
Future Volume (vph)	100	1367	24	16	1367	41	54	2	22	53	4	68
Ideal Flow (vphpl)	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900
Grade (%)		-1%			1%			1%			2%	
Total Lost time (s)	4.0	4.0	4.0	4.0	4.0	4.0		4.0		4.0	4.0	4.0
Lane Util. Factor	1.00	0.95	1.00	1.00	0.95	1.00		1.00		1.00	1.00	1.00
Frt	1.00	1.00	0.85	1.00	1.00	0.85		0.96		1.00	1.00	0.85
Flt Protected	0.95	1.00	1.00	0.95	1.00	1.00		0.97		0.95	1.00	1.00
Satd. Flow (prot)	1778	3455	1591	1761	3421	1575		1723		1752	1844	1567
Flt Permitted	0.07	1.00	1.00	0.09	1.00	1.00		0.97		0.95	1.00	1.00
Satd. Flow (perm)	131	3455	1591	168	3421	1575		1723		1752	1844	1567
Peak-hour factor, PHF	0.88	0.88	0.88	0.88	0.88	0.88	0.88	0.88	0.88	0.88	0.88	0.88
Adj. Flow (vph)	114	1553	27	18	1553	47	61	2	25	60	5	77
RTOR Reduction (vph)	0	0	11	0	0	20	0	15	0	0	0	71
Lane Group Flow (vph)	114	1553	16	18	1553	27	0	73	0	60	5	6
Heavy Vehicles (%)	2%	5%	2%	2%	5%	2%	2%	2%	2%	2%	2%	2%
Turn Type	pm+pt	NA	Perm	pm+pt	NA	Perm	Split	NA		Split	NA	Perm
Protected Phases	1	6		5	2		3	3		4	4	
Permitted Phases	6		6	2		2						4
Actuated Green, G (s)	58.5	53.3	53.3	52.1	50.1	50.1		4.7		4.4	4.4	4.4
Effective Green, g (s)	66.3	57.2	57.2	59.9	54.0	54.0		7.9		8.0	8.0	8.0
Actuated g/C Ratio	0.70	0.60	0.60	0.63	0.57	0.57		0.08		0.08	0.08	0.08
Clearance Time (s)	7.9	7.9	7.9	7.9	7.9	7.9		7.2		7.6	7.6	7.6
Vehicle Extension (s)	3.0	3.0	3.0	3.0	3.0	3.0		3.0		3.0	3.0	3.0
Lane Grp Cap (vph)	249	2080	957	204	1944	895		143		147	155	131
v/s Ratio Prot	c0.04	0.45		0.01	c0.45			c0.04		c0.03	0.00	
v/s Ratio Perm	0.28		0.01	0.05		0.02						0.00
v/c Ratio	0.46	0.75	0.02	0.09	0.80	0.03		0.51		0.41	0.03	0.05
Uniform Delay, d1	13.8	13.7	7.6	9.8	16.2	9.0		41.7		41.3	39.9	40.0
Progression Factor	2.26	0.11	1.00	1.00	1.00	1.00		1.00		1.00	1.00	1.00
Incremental Delay, d2	0.8	1.5	0.0	0.2	3.5	0.1		3.1		1.8	0.1	0.2
Delay (s)	32.0	3.0	7.6	10.0	19.7	9.1		44.8		43.1	40.0	40.2
Level of Service	C	A	A	B	B	A		D		D	D	D
Approach Delay (s)		5.1			19.3			44.8			41.4	
Approach LOS		A			B			D			D	

Intersection Summary			
HCM 2000 Control Delay	14.0	HCM 2000 Level of Service	B
HCM 2000 Volume to Capacity ratio	0.69		
Actuated Cycle Length (s)	95.0	Sum of lost time (s)	16.0
Intersection Capacity Utilization	64.4%	ICU Level of Service	C
Analysis Period (min)	15		

c Critical Lane Group

Intersection: 15: Lendall Ln (1015)/Solomon Drive & Warrenton Rd (Rte.17)

Movement	EB	EB	EB	EB	WB	WB	WB	WB	NB	SB	SB	SB
Directions Served	L	T	T	R	L	T	T	R	LTR	L	T	R
Maximum Queue (ft)	81	207	248	74	129	242	237	24	91	58	30	72
Average Queue (ft)	54	97	105	15	33	209	200	5	44	40	6	57
95th Queue (ft)	87	211	228	64	113	268	260	21	83	66	26	79
Link Distance (ft)		2725	2725			2215	2215		526		674	
Upstream Blk Time (%)												
Queuing Penalty (veh)												
Storage Bay Dist (ft)	170			50	105			260		125		140
Storage Blk Time (%)		2	15			23						
Queuing Penalty (veh)		2	4			4						

HCM Unsignalized Intersection Capacity Analysis
14: Short St/Short St (1034) & Warrenton Rd (Rte.17) 03/29/2018

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (veh/h)	0	1960	170	0	1404	19	0	0	26	0	0	17
Future Volume (Veh/h)	0	1960	170	0	1404	19	0	0	26	0	0	17
Sign Control		Free			Free			Stop			Stop	
Grade		0%			1%			-2%			3%	
Peak Hour Factor	0.91	0.91	0.91	0.91	0.91	0.91	0.91	0.91	0.91	0.91	0.91	0.91
Hourly flow rate (vph)	0	2154	187	0	1543	21	0	0	29	0	0	19
Pedestrians												
Lane Width (ft)												
Walking Speed (ft/s)												
Percent Blockage												
Right turn flare (veh)												
Median type		None			None							
Median storage veh)												
Upstream signal (ft)					418							
pX, platoon unblocked	0.82						0.82	0.82		0.82	0.82	0.82
vC, conflicting volume	1564			2341			2944	3718	1077	2660	3894	782
vC1, stage 1 conf vol												
vC2, stage 2 conf vol												
vCu, unblocked vol	1240			2341			2932	3880	1077	2583	4097	281
tC, single (s)	4.1			4.1			7.5	6.5	6.9	7.5	6.5	6.9
tC, 2 stage (s)												
tF (s)	2.2			2.2			3.5	4.0	3.3	3.5	4.0	3.3
p0 queue free %	100			100			100	100	86	100	100	97
cM capacity (veh/h)	455			208			5	3	215	9	2	584

Direction, Lane #	EB 1	EB 2	EB 3	WB 1	WB 2	NB 1	SB 1
Volume Total	1077	1077	187	1029	535	29	19
Volume Left	0	0	0	0	0	0	0
Volume Right	0	0	187	0	21	29	19
cSH	1700	1700	1700	1700	1700	215	584
Volume to Capacity	0.63	0.63	0.11	0.61	0.31	0.14	0.03
Queue Length 95th (ft)	0	0	0	0	0	11	3
Control Delay (s)	0.0	0.0	0.0	0.0	0.0	24.4	11.4
Lane LOS						C	B
Approach Delay (s)	0.0			0.0		24.4	11.4
Approach LOS						C	B

Intersection Summary			
Average Delay	0.2		
Intersection Capacity Utilization	64.2%	ICU Level of Service	C
Analysis Period (min)	15		

Intersection: 14: Short St/Short St (1034) & Warrenton Rd (Rte.17)

Movement	EB	EB	EB	NB	SB
Directions Served	T	T	R	R	R
Maximum Queue (ft)	225	305	307	63	28
Average Queue (ft)	184	243	107	37	6
95th Queue (ft)	228	348	327	62	24
Link Distance (ft)	233	233	233	416	396
Upstream Blk Time (%)	0	10	5		
Queuing Penalty (veh)	1	63	29		
Storage Bay Dist (ft)					
Storage Blk Time (%)					
Queuing Penalty (veh)					

Movement	SEU	SET	SER	NWL	NWT	NEL	NER
Lane Configurations							
Traffic Volume (vph)	7	1868	130	107	1361	196	56
Future Volume (vph)	7	1868	130	107	1361	196	56
Ideal Flow (vphpl)	1900	1900	1900	1900	1900	1900	1900
Grade (%)		-1%			1%	0%	
Total Lost time (s)	4.0	4.0		8.3	4.0	4.0	4.0
Lane Util. Factor	1.00	0.95		1.00	0.95	1.00	1.00
Frt	1.00	0.99		1.00	1.00	1.00	0.85
Flt Protected	0.95	1.00		0.95	1.00	0.95	1.00
Satd. Flow (prot)	1778	3522		1761	3522	1770	1583
Flt Permitted	0.20	1.00		0.06	1.00	0.95	1.00
Satd. Flow (perm)	374	3522		113	3522	1770	1583
Peak-hour factor, PHF	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Adj. Flow (vph)	7	1868	130	107	1361	196	56
RTOR Reduction (vph)	0	4	0	0	0	0	44
Lane Group Flow (vph)	7	1994	0	107	1361	196	12
Turn Type	Perm	NA		pm+pt	NA	Prot	Perm
Protected Phases		6		5	2	4	
Permitted Phases	6			2			4
Actuated Green, G (s)	57.4	57.4		75.7	75.7	20.9	20.9
Effective Green, g (s)	61.7	61.7		75.7	80.0	23.3	23.3
Actuated g/C Ratio	0.55	0.55		0.68	0.72	0.21	0.21
Clearance Time (s)	8.3	8.3		8.3	8.3	6.4	6.4
Vehicle Extension (s)	3.0	3.0		3.0	3.0	3.0	3.0
Lane Grp Cap (vph)	207	1952		224	2531	370	331
v/s Ratio Prot		c0.57		0.04	c0.39	c0.11	
v/s Ratio Perm	0.02			0.28			0.01
v/c Ratio	0.03	1.02		0.48	0.54	0.53	0.04
Uniform Delay, d1	11.3	24.8		24.2	7.2	39.1	35.0
Progression Factor	1.00	1.00		1.00	1.00	1.00	1.00
Incremental Delay, d2	0.3	25.9		1.6	0.8	5.3	0.2
Delay (s)	11.6	50.7		25.8	8.0	44.5	35.2
Level of Service	B	D		C	A	D	D
Approach Delay (s)		50.6			9.3	42.4	
Approach LOS		D			A	D	

Intersection Summary			
HCM 2000 Control Delay	33.8	HCM 2000 Level of Service	C
HCM 2000 Volume to Capacity ratio	0.88		
Actuated Cycle Length (s)	111.3	Sum of lost time (s)	16.3
Intersection Capacity Utilization	88.5%	ICU Level of Service	E
Analysis Period (min)	15		
c Critical Lane Group			

Intersection: 2: Olde Forge Road & Warrenton Rd (Rte.17)

Movement	SE	SE	SE	NW	NW	NW	NE	NE
Directions Served	U	T	TR	L	T	T	L	R
Maximum Queue (ft)	31	380	366	116	72	71	189	30
Average Queue (ft)	10	352	356	63	51	48	159	16
95th Queue (ft)	31	395	365	134	84	91	186	39
Link Distance (ft)		351	351		2725	2725	873	
Upstream Blk Time (%)		13	14					
Queuing Penalty (veh)		128	136					
Storage Bay Dist (ft)	100			200				150
Storage Blk Time (%)		34					9	
Queuing Penalty (veh)		2					5	

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (vph)	32	1835	27	6	1295	23	65	9	22	45	5	35
Future Volume (vph)	32	1835	27	6	1295	23	65	9	22	45	5	35
Ideal Flow (vphpl)	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900
Grade (%)		-1%			1%			1%			2%	
Total Lost time (s)	4.0	4.0	4.0	4.0	4.0	4.0		4.0		4.0	4.0	4.0
Lane Util. Factor	1.00	0.95	1.00	1.00	0.95	1.00		1.00		1.00	1.00	1.00
Frt	1.00	1.00	0.85	1.00	1.00	0.85		0.97		1.00	1.00	0.85
Flt Protected	0.95	1.00	1.00	0.95	1.00	1.00		0.97		0.95	1.00	1.00
Satd. Flow (prot)	1778	3455	1591	1761	3421	1575		1737		1752	1844	1567
Flt Permitted	0.11	1.00	1.00	0.06	1.00	1.00		0.97		0.95	1.00	1.00
Satd. Flow (perm)	206	3455	1591	105	3421	1575		1737		1752	1844	1567
Peak-hour factor, PHF	0.88	0.88	0.88	0.88	0.88	0.88	0.88	0.88	0.88	0.88	0.88	0.88
Adj. Flow (vph)	36	2085	31	7	1472	26	74	10	25	51	6	40
RTOR Reduction (vph)	0	0	11	0	0	9	0	10	0	0	0	37
Lane Group Flow (vph)	36	2085	20	7	1472	17	0	99	0	51	6	3
Heavy Vehicles (%)	2%	5%	2%	2%	5%	2%	2%	2%	2%	2%	2%	2%
Turn Type	pm+pt	NA	Perm	pm+pt	NA	Perm	Split	NA		Split	NA	Perm
Protected Phases	1	6		5	2		3	3		4	4	
Permitted Phases	6		6	2		2						4
Actuated Green, G (s)	71.6	68.6	68.6	67.6	66.6	66.6		5.8		4.0	4.0	4.0
Effective Green, g (s)	79.4	72.5	72.5	75.4	70.5	70.5		9.0		7.6	7.6	7.6
Actuated g/C Ratio	0.72	0.66	0.66	0.69	0.64	0.64		0.08		0.07	0.07	0.07
Clearance Time (s)	7.9	7.9	7.9	7.9	7.9	7.9		7.2		7.6	7.6	7.6
Vehicle Extension (s)	3.0	3.0	3.0	3.0	3.0	3.0		3.0		3.0	3.0	3.0
Lane Grp Cap (vph)	247	2277	1048	145	2192	1009		142		121	127	108
v/s Ratio Prot	c0.01	c0.60		0.00	0.43			c0.06		c0.03	0.00	
v/s Ratio Perm	0.10		0.01	0.03		0.01						0.00
v/c Ratio	0.15	0.92	0.02	0.05	0.67	0.02		0.70		0.42	0.05	0.03
Uniform Delay, d1	8.3	16.1	6.5	18.2	12.5	7.2		49.2		49.1	47.8	47.7
Progression Factor	1.00	1.00	1.00	1.00	1.00	1.00		1.00		1.00	1.00	1.00
Incremental Delay, d2	0.3	7.2	0.0	0.1	1.7	0.0		13.8		2.4	0.2	0.1
Delay (s)	8.6	23.3	6.5	18.4	14.1	7.2		63.0		51.5	48.0	47.8
Level of Service	A	C	A	B	B	A		E		D	D	D
Approach Delay (s)		22.8			14.0			63.0			49.8	
Approach LOS		C			B			E			D	

Intersection Summary			
HCM 2000 Control Delay	21.2	HCM 2000 Level of Service	C
HCM 2000 Volume to Capacity ratio	0.82		
Actuated Cycle Length (s)	110.0	Sum of lost time (s)	16.0
Intersection Capacity Utilization	69.5%	ICU Level of Service	C
Analysis Period (min)	15		

c Critical Lane Group

Intersection: 15: Lendall Ln (1015)/Solomon Drive & Warrenton Rd (Rte.17)

Movement	EB	EB	EB	EB	WB	WB	WB	WB	NB	SB	SB	SB
Directions Served	L	T	T	R	L	T	T	R	LTR	L	T	R
Maximum Queue (ft)	44	87	141	74	18	223	203	1	96	49	31	66
Average Queue (ft)	26	33	48	20	7	146	131	0	76	34	13	36
95th Queue (ft)	52	86	133	66	21	238	214	1	115	47	32	61
Link Distance (ft)		2725	2725			2215	2215		526		674	
Upstream Blk Time (%)												
Queuing Penalty (veh)												
Storage Bay Dist (ft)	170			50	105			260		125		140
Storage Blk Time (%)			4	0		17						
Queuing Penalty (veh)			1	0		1						

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (veh/h)	0	1960	233	0	1444	19	0	0	29	0	0	17
Future Volume (Veh/h)	0	1960	233	0	1444	19	0	0	29	0	0	17
Sign Control		Free			Free			Stop			Stop	
Grade		0%			1%			-2%			3%	
Peak Hour Factor	0.91	0.91	0.91	0.91	0.91	0.91	0.91	0.91	0.91	0.91	0.91	0.91
Hourly flow rate (vph)	0	2154	256	0	1587	21	0	0	32	0	0	19
Pedestrians												
Lane Width (ft)												
Walking Speed (ft/s)												
Percent Blockage												
Right turn flare (veh)												
Median type		None			None							
Median storage veh)												
Upstream signal (ft)					418							
pX, platoon unblocked	0.82						0.82	0.82		0.82	0.82	0.82
vC, conflicting volume	1608			2410			2966	3762	1077	2706	4008	804
vC1, stage 1 conf vol												
vC2, stage 2 conf vol												
vCu, unblocked vol	1294			2410			2959	3934	1077	2640	4235	308
tC, single (s)	4.1			4.1			7.5	6.5	6.9	7.5	6.5	6.9
tC, 2 stage (s)												
tF (s)	2.2			2.2			3.5	4.0	3.3	3.5	4.0	3.3
p0 queue free %	100			100			100	100	85	100	100	97
cM capacity (veh/h)	434			195			5	3	215	8	2	561

Direction, Lane #	EB 1	EB 2	EB 3	WB 1	WB 2	NB 1	SB 1
Volume Total	1077	1077	256	1058	550	32	19
Volume Left	0	0	0	0	0	0	0
Volume Right	0	0	256	0	21	32	19
cSH	1700	1700	1700	1700	1700	215	561
Volume to Capacity	0.63	0.63	0.15	0.62	0.32	0.15	0.03
Queue Length 95th (ft)	0	0	0	0	0	13	3
Control Delay (s)	0.0	0.0	0.0	0.0	0.0	24.7	11.6
Lane LOS						C	B
Approach Delay (s)	0.0			0.0		24.7	11.6
Approach LOS						C	B

Intersection Summary			
Average Delay	0.2		
Intersection Capacity Utilization	64.2%	ICU Level of Service	C
Analysis Period (min)	15		

Intersection: 14: Short St/Short St (1034) & Warrenton Rd (Rte.17)

Movement	EB	EB	EB	NB	SB
Directions Served	T	T	R	R	R
Maximum Queue (ft)	369	358	333	41	48
Average Queue (ft)	291	311	230	21	20
95th Queue (ft)	415	411	424	42	51
Link Distance (ft)	233	233	233	416	396
Upstream Blk Time (%)	17	40	11		
Queuing Penalty (veh)	106	241	69		
Storage Bay Dist (ft)					
Storage Blk Time (%)					
Queuing Penalty (veh)					

Movement	SEU	SET	SER	NWL	NWT	NEL	NER
Lane Configurations							
Traffic Volume (vph)	7	1871	130	128	1361	236	66
Future Volume (vph)	7	1871	130	128	1361	236	66
Ideal Flow (vphpl)	1900	1900	1900	1900	1900	1900	1900
Grade (%)		-1%			1%	0%	
Total Lost time (s)	4.0	4.0		8.3	4.0	4.0	4.0
Lane Util. Factor	1.00	0.95		1.00	0.95	1.00	1.00
Frt	1.00	0.99		1.00	1.00	1.00	0.85
Flt Protected	0.95	1.00		0.95	1.00	0.95	1.00
Satd. Flow (prot)	1778	3522		1761	3522	1770	1583
Flt Permitted	0.20	1.00		0.06	1.00	0.95	1.00
Satd. Flow (perm)	374	3522		113	3522	1770	1583
Peak-hour factor, PHF	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Adj. Flow (vph)	7	1871	130	128	1361	236	66
RTOR Reduction (vph)	0	4	0	0	0	0	52
Lane Group Flow (vph)	7	1997	0	128	1361	236	14
Turn Type	Perm	NA		pm+pt	NA	Prot	Perm
Protected Phases		6		5	2	4	
Permitted Phases	6			2			4
Actuated Green, G (s)	57.4	57.4		75.7	75.7	20.9	20.9
Effective Green, g (s)	61.7	61.7		75.7	80.0	23.3	23.3
Actuated g/C Ratio	0.55	0.55		0.68	0.72	0.21	0.21
Clearance Time (s)	8.3	8.3		8.3	8.3	6.4	6.4
Vehicle Extension (s)	3.0	3.0		3.0	3.0	3.0	3.0
Lane Grp Cap (vph)	207	1952		224	2531	370	331
v/s Ratio Prot		c0.57		0.05	c0.39	c0.13	
v/s Ratio Perm	0.02			0.34			0.01
v/c Ratio	0.03	1.02		0.57	0.54	0.64	0.04
Uniform Delay, d1	11.3	24.8		27.2	7.2	40.1	35.1
Progression Factor	1.00	1.00		1.00	1.00	1.00	1.00
Incremental Delay, d2	0.3	26.4		3.5	0.8	8.2	0.2
Delay (s)	11.6	51.2		30.7	8.0	48.3	35.3
Level of Service	B	D		C	A	D	D
Approach Delay (s)		51.0			9.9	45.5	
Approach LOS		D			A	D	

Intersection Summary			
HCM 2000 Control Delay	34.5	HCM 2000 Level of Service	C
HCM 2000 Volume to Capacity ratio	0.91		
Actuated Cycle Length (s)	111.3	Sum of lost time (s)	16.3
Intersection Capacity Utilization	90.8%	ICU Level of Service	E
Analysis Period (min)	15		
c Critical Lane Group			

Intersection: 2: Olde Forge Road & Warrenton Rd (Rte.17)

Movement	SE	SE	NW	NW	NW	NE	NE
Directions Served	T	TR	L	T	T	L	R
Maximum Queue (ft)	385	368	112	198	250	246	175
Average Queue (ft)	349	358	77	89	121	178	92
95th Queue (ft)	402	368	123	183	241	252	204
Link Distance (ft)	351	351		2725	2725	873	
Upstream Blk Time (%)	31	34					
Queuing Penalty (veh)	304	337					
Storage Bay Dist (ft)			200				150
Storage Blk Time (%)	50			0		24	0
Queuing Penalty (veh)	4			1		16	0

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (vph)	32	1848	27	6	1316	23	65	9	22	45	5	35
Future Volume (vph)	32	1848	27	6	1316	23	65	9	22	45	5	35
Ideal Flow (vphpl)	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900
Grade (%)		-1%			1%			1%			2%	
Total Lost time (s)	4.0	4.0	4.0	4.0	4.0	4.0		4.0		4.0	4.0	4.0
Lane Util. Factor	1.00	0.95	1.00	1.00	0.95	1.00		1.00		1.00	1.00	1.00
Frt	1.00	1.00	0.85	1.00	1.00	0.85		0.97		1.00	1.00	0.85
Flt Protected	0.95	1.00	1.00	0.95	1.00	1.00		0.97		0.95	1.00	1.00
Satd. Flow (prot)	1778	3455	1591	1761	3421	1575		1737		1752	1844	1567
Flt Permitted	0.11	1.00	1.00	0.06	1.00	1.00		0.97		0.95	1.00	1.00
Satd. Flow (perm)	198	3455	1591	105	3421	1575		1737		1752	1844	1567
Peak-hour factor, PHF	0.88	0.88	0.88	0.88	0.88	0.88	0.88	0.88	0.88	0.88	0.88	0.88
Adj. Flow (vph)	36	2100	31	7	1495	26	74	10	25	51	6	40
RTOR Reduction (vph)	0	0	11	0	0	9	0	10	0	0	0	37
Lane Group Flow (vph)	36	2100	20	7	1495	17	0	99	0	51	6	3
Heavy Vehicles (%)	2%	5%	2%	2%	5%	2%	2%	2%	2%	2%	2%	2%
Turn Type	pm+pt	NA	Perm	pm+pt	NA	Perm	Split	NA		Split	NA	Perm
Protected Phases	1	6		5	2		3	3		4	4	
Permitted Phases	6		6	2		2						4
Actuated Green, G (s)	71.8	68.8	68.8	67.8	66.8	66.8		5.6		4.0	4.0	4.0
Effective Green, g (s)	79.6	72.7	72.7	75.6	70.7	70.7		8.8		7.6	7.6	7.6
Actuated g/C Ratio	0.72	0.66	0.66	0.69	0.64	0.64		0.08		0.07	0.07	0.07
Clearance Time (s)	7.9	7.9	7.9	7.9	7.9	7.9		7.2		7.6	7.6	7.6
Vehicle Extension (s)	3.0	3.0	3.0	3.0	3.0	3.0		3.0		3.0	3.0	3.0
Lane Grp Cap (vph)	242	2283	1051	145	2198	1012		138		121	127	108
v/s Ratio Prot	c0.01	c0.61		0.00	0.44			c0.06		c0.03	0.00	
v/s Ratio Perm	0.10		0.01	0.03		0.01						0.00
v/c Ratio	0.15	0.92	0.02	0.05	0.68	0.02		0.72		0.42	0.05	0.03
Uniform Delay, d1	8.4	16.1	6.4	18.5	12.5	7.1		49.4		49.1	47.8	47.7
Progression Factor	1.00	1.00	1.00	1.00	1.00	1.00		1.00		1.00	1.00	1.00
Incremental Delay, d2	0.3	7.5	0.0	0.1	1.7	0.0		16.2		2.4	0.2	0.1
Delay (s)	8.7	23.6	6.4	18.7	14.2	7.1		65.6		51.5	48.0	47.8
Level of Service	A	C	A	B	B	A		E		D	D	D
Approach Delay (s)		23.1			14.1			65.6			49.8	
Approach LOS		C			B			E			D	

Intersection Summary			
HCM 2000 Control Delay	21.4	HCM 2000 Level of Service	C
HCM 2000 Volume to Capacity ratio	0.82		
Actuated Cycle Length (s)	110.0	Sum of lost time (s)	16.0
Intersection Capacity Utilization	69.8%	ICU Level of Service	C
Analysis Period (min)	15		

c Critical Lane Group

Intersection: 15: Lendall Ln (1015)/Solomon Drive & Warrenton Rd (Rte.17)

Movement	EB	EB	EB	WB	WB	WB	WB	NB	SB	SB	SB
Directions Served	L	T	T	L	T	T	R	LTR	L	T	R
Maximum Queue (ft)	44	216	215	18	227	288	285	195	93	30	53
Average Queue (ft)	18	54	58	4	139	161	57	113	52	6	38
95th Queue (ft)	45	190	192	16	256	286	245	199	98	26	57
Link Distance (ft)		2725	2725		2215	2215		526		674	
Upstream Blk Time (%)											
Queuing Penalty (veh)											
Storage Bay Dist (ft)	170			105			260		125		140
Storage Blk Time (%)		2	10		13	1	0				
Queuing Penalty (veh)		1	3		1	0	0				

Appendix G: No Build and Build (2028) Capacity Analysis

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (veh/h)	0	1668	62	0	1764	10	0	0	31	0	0	9
Future Volume (Veh/h)	0	1668	62	0	1764	10	0	0	31	0	0	9
Sign Control		Free			Free			Stop			Stop	
Grade		0%			1%			-2%			3%	
Peak Hour Factor	0.91	0.91	0.91	0.91	0.91	0.91	0.91	0.91	0.91	0.91	0.91	0.91
Hourly flow rate (vph)	0	1833	68	0	1938	11	0	0	34	0	0	10
Pedestrians												
Lane Width (ft)												
Walking Speed (ft/s)												
Percent Blockage												
Right turn flare (veh)												
Median type		None			None							
Median storage veh)												
Upstream signal (ft)					418							
pX, platoon unblocked	0.68						0.68	0.68		0.68	0.68	0.68
vC, conflicting volume	1949			1901			2812	3782	916	2894	3844	974
vC1, stage 1 conf vol												
vC2, stage 2 conf vol												
vCu, unblocked vol	1447			1901			2722	4156	916	2843	4248	6
tC, single (s)	4.1			4.1			7.5	6.5	6.9	7.5	6.5	6.9
tC, 2 stage (s)												
tF (s)	2.2			2.2			3.5	4.0	3.3	3.5	4.0	3.3
p0 queue free %	100			100			100	100	88	100	100	99
cM capacity (veh/h)	314			309			7	2	275	5	1	727

Direction, Lane #	EB 1	EB 2	EB 3	WB 1	WB 2	NB 1	SB 1
Volume Total	916	916	68	1292	657	34	10
Volume Left	0	0	0	0	0	0	0
Volume Right	0	0	68	0	11	34	10
cSH	1700	1700	1700	1700	1700	275	727
Volume to Capacity	0.54	0.54	0.04	0.76	0.39	0.12	0.01
Queue Length 95th (ft)	0	0	0	0	0	10	1
Control Delay (s)	0.0	0.0	0.0	0.0	0.0	20.0	10.0
Lane LOS						C	B
Approach Delay (s)	0.0			0.0		20.0	10.0
Approach LOS						C	B

Intersection Summary			
Average Delay	0.2		
Intersection Capacity Utilization	59.1%	ICU Level of Service	B
Analysis Period (min)	15		

Intersection: 14: Short St/Short St (1034) & Warrenton Rd (Rte.17)

Movement	EB	EB	EB	NB	SB
Directions Served	T	T	R	R	R
Maximum Queue (ft)	227	282	235	110	28
Average Queue (ft)	65	92	47	50	11
95th Queue (ft)	205	253	202	119	34
Link Distance (ft)	233	233	233	416	396
Upstream Blk Time (%)	1	3	1		
Queuing Penalty (veh)	3	19	7		
Storage Bay Dist (ft)					
Storage Blk Time (%)					
Queuing Penalty (veh)					

Movement	SEU	SET	SER	NWL	NWT	NEL	NER
Lane Configurations							
Traffic Volume (vph)	3	1640	44	52	1677	248	67
Future Volume (vph)	3	1640	44	52	1677	248	67
Ideal Flow (vphpl)	1900	1900	1900	1900	1900	1900	1900
Grade (%)		-1%			1%	0%	
Total Lost time (s)	4.0	4.0		7.0	4.0	4.0	4.0
Lane Util. Factor	1.00	0.95		1.00	0.95	1.00	1.00
Frt	1.00	1.00		1.00	1.00	1.00	0.85
Flt Protected	0.95	1.00		0.95	1.00	0.95	1.00
Satd. Flow (prot)	1778	3543		1761	3522	1770	1583
Flt Permitted	0.11	1.00		0.07	1.00	0.95	1.00
Satd. Flow (perm)	201	3543		137	3522	1770	1583
Peak-hour factor, PHF	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Adj. Flow (vph)	3	1640	44	52	1677	248	67
RTOR Reduction (vph)	0	2	0	0	0	0	50
Lane Group Flow (vph)	3	1682	0	52	1677	248	17
Turn Type	Perm	NA		pm+pt	NA	Prot	Perm
Protected Phases		6		5	2	4	
Permitted Phases	6			2			4
Actuated Green, G (s)	47.2	47.2		62.2	62.2	23.1	23.1
Effective Green, g (s)	51.5	51.5		62.2	66.5	25.5	25.5
Actuated g/C Ratio	0.52	0.52		0.62	0.66	0.26	0.26
Clearance Time (s)	8.3	8.3		7.0	8.3	6.4	6.4
Vehicle Extension (s)	3.0	3.0		3.0	3.0	3.0	3.0
Lane Grp Cap (vph)	103	1824		215	2342	451	403
v/s Ratio Prot		c0.47		0.02	c0.48	c0.14	
v/s Ratio Perm	0.01			0.13			0.01
v/c Ratio	0.03	0.92		0.24	0.72	0.55	0.04
Uniform Delay, d1	11.9	22.4		20.3	10.7	32.3	28.1
Progression Factor	1.00	1.00		0.56	0.28	1.00	1.00
Incremental Delay, d2	0.5	9.3		0.4	1.2	4.8	0.2
Delay (s)	12.5	31.6		11.8	4.2	37.0	28.3
Level of Service	B	C		B	A	D	C
Approach Delay (s)		31.6			4.4	35.2	
Approach LOS		C			A	D	

Intersection Summary			
HCM 2000 Control Delay	19.3	HCM 2000 Level of Service	B
HCM 2000 Volume to Capacity ratio	0.83		
Actuated Cycle Length (s)	100.0	Sum of lost time (s)	15.0
Intersection Capacity Utilization	67.1%	ICU Level of Service	C
Analysis Period (min)	15		
c Critical Lane Group			

Intersection: 2: Olde Forge Road & Warrenton Rd (Rte.17)

Movement	SE	SE	NW	NW	NW	NE	NE
Directions Served	T	TR	L	T	T	L	R
Maximum Queue (ft)	360	358	72	393	404	225	175
Average Queue (ft)	336	332	37	255	282	152	67
95th Queue (ft)	396	403	76	473	499	232	169
Link Distance (ft)	351	351		2725	2725	873	
Upstream Blk Time (%)	8	8					
Queuing Penalty (veh)	65	70					
Storage Bay Dist (ft)			200				150
Storage Blk Time (%)	46			17		9	
Queuing Penalty (veh)	1			9		6	

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (vph)	100	1512	24	16	1530	41	54	2	22	53	4	68
Future Volume (vph)	100	1512	24	16	1530	41	54	2	22	53	4	68
Ideal Flow (vphpl)	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900
Grade (%)		-1%			1%			1%			2%	
Total Lost time (s)	4.0	4.0	4.0	4.0	4.0	4.0		4.0		4.0	4.0	4.0
Lane Util. Factor	1.00	0.95	1.00	1.00	0.95	1.00		1.00		1.00	1.00	1.00
Frt	1.00	1.00	0.85	1.00	1.00	0.85		0.96		1.00	1.00	0.85
Flt Protected	0.95	1.00	1.00	0.95	1.00	1.00		0.97		0.95	1.00	1.00
Satd. Flow (prot)	1778	3455	1591	1761	3421	1575		1723		1752	1844	1567
Flt Permitted	0.06	1.00	1.00	0.07	1.00	1.00		0.97		0.95	1.00	1.00
Satd. Flow (perm)	118	3455	1591	129	3421	1575		1723		1752	1844	1567
Peak-hour factor, PHF	0.88	0.88	0.88	0.88	0.88	0.88	0.88	0.88	0.88	0.88	0.88	0.88
Adj. Flow (vph)	114	1718	27	18	1739	47	61	2	25	60	5	77
RTOR Reduction (vph)	0	0	10	0	0	19	0	15	0	0	0	71
Lane Group Flow (vph)	114	1718	17	18	1739	28	0	73	0	60	5	6
Heavy Vehicles (%)	2%	5%	2%	2%	5%	2%	2%	2%	2%	2%	2%	2%
Turn Type	pm+pt	NA	Perm	pm+pt	NA	Perm	Split	NA		Split	NA	Perm
Protected Phases	1	6		5	2		3	3		4	4	
Permitted Phases	6		6	2		2						4
Actuated Green, G (s)	64.4	59.3	59.3	58.2	56.2	56.2		4.1		4.0	4.0	4.0
Effective Green, g (s)	72.2	63.2	63.2	66.0	60.1	60.1		7.3		7.6	7.6	7.6
Actuated g/C Ratio	0.72	0.63	0.63	0.66	0.60	0.60		0.07		0.08	0.08	0.08
Clearance Time (s)	7.9	7.9	7.9	7.9	7.9	7.9		7.2		7.6	7.6	7.6
Vehicle Extension (s)	3.0	3.0	3.0	3.0	3.0	3.0		3.0		3.0	3.0	3.0
Lane Grp Cap (vph)	234	2183	1005	181	2056	946		125		133	140	119
v/s Ratio Prot	c0.04	0.50		0.01	c0.51			c0.04		c0.03	0.00	
v/s Ratio Perm	0.31		0.01	0.06		0.02						0.00
v/c Ratio	0.49	0.79	0.02	0.10	0.85	0.03		0.59		0.45	0.04	0.05
Uniform Delay, d1	16.6	13.5	6.8	10.7	16.2	8.1		44.9		44.2	42.8	42.8
Progression Factor	2.12	0.10	1.00	1.00	1.00	1.00		1.00		1.00	1.00	1.00
Incremental Delay, d2	0.9	1.7	0.0	0.2	4.5	0.1		6.8		2.4	0.1	0.2
Delay (s)	36.1	3.0	6.9	10.9	20.7	8.2		51.7		46.6	42.9	43.0
Level of Service	D	A	A	B	C	A		D		D	D	D
Approach Delay (s)		5.1			20.3			51.7			44.5	
Approach LOS		A			C			D			D	

Intersection Summary			
HCM 2000 Control Delay	14.6	HCM 2000 Level of Service	B
HCM 2000 Volume to Capacity ratio	0.75		
Actuated Cycle Length (s)	100.0	Sum of lost time (s)	16.0
Intersection Capacity Utilization	68.9%	ICU Level of Service	C
Analysis Period (min)	15		

c Critical Lane Group

Intersection: 15: Lendall Ln (1015)/Solomon Drive & Warrenton Rd (Rte.17)

Movement	EB	EB	EB	WB	WB	WB	WB	NB	SB	SB	SB
Directions Served	L	T	T	L	T	T	R	LTR	L	T	R
Maximum Queue (ft)	129	153	128	18	346	363	3	94	71	28	72
Average Queue (ft)	77	112	102	7	251	250	1	64	45	6	46
95th Queue (ft)	121	168	172	21	367	380	3	100	71	25	73
Link Distance (ft)		2725	2725		2215	2215		526		674	
Upstream Blk Time (%)											
Queuing Penalty (veh)											
Storage Bay Dist (ft)	170			105			260		125		140
Storage Blk Time (%)		0	18		28	10					
Queuing Penalty (veh)		0	4		4	4					

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (veh/h)	0	1668	83	0	1824	10	0	0	35	0	0	9
Future Volume (Veh/h)	0	1668	83	0	1824	10	0	0	35	0	0	9
Sign Control		Free			Free			Stop			Stop	
Grade		0%			1%			-2%			3%	
Peak Hour Factor	0.91	0.91	0.91	0.91	0.91	0.91	0.91	0.91	0.91	0.91	0.91	0.91
Hourly flow rate (vph)	0	1833	91	0	2004	11	0	0	38	0	0	10
Pedestrians												
Lane Width (ft)												
Walking Speed (ft/s)												
Percent Blockage												
Right turn flare (veh)												
Median type		None			None							
Median storage veh)												
Upstream signal (ft)					418							
pX, platoon unblocked	0.68						0.68	0.68		0.68	0.68	0.68
vC, conflicting volume	2015			1924			2845	3848	916	2964	3934	1008
vC1, stage 1 conf vol												
vC2, stage 2 conf vol												
vCu, unblocked vol	1544			1924			2771	4253	916	2947	4380	55
tC, single (s)	4.1			4.1			7.5	6.5	6.9	7.5	6.5	6.9
tC, 2 stage (s)												
tF (s)	2.2			2.2			3.5	4.0	3.3	3.5	4.0	3.3
p0 queue free %	100			100			100	100	86	100	100	99
cM capacity (veh/h)	288			303			6	1	275	4	1	677

Direction, Lane #	EB 1	EB 2	EB 3	WB 1	WB 2	NB 1	SB 1
Volume Total	916	916	91	1336	679	38	10
Volume Left	0	0	0	0	0	0	0
Volume Right	0	0	91	0	11	38	10
cSH	1700	1700	1700	1700	1700	275	677
Volume to Capacity	0.54	0.54	0.05	0.79	0.40	0.14	0.01
Queue Length 95th (ft)	0	0	0	0	0	12	1
Control Delay (s)	0.0	0.0	0.0	0.0	0.0	20.2	10.4
Lane LOS						C	B
Approach Delay (s)	0.0			0.0		20.2	10.4
Approach LOS						C	B

Intersection Summary			
Average Delay	0.2		
Intersection Capacity Utilization	60.7%	ICU Level of Service	B
Analysis Period (min)	15		

Intersection: 14: Short St/Short St (1034) & Warrenton Rd (Rte.17)

Movement	EB	EB	EB	NB
Directions Served	T	T	R	R
Maximum Queue (ft)	370	358	267	61
Average Queue (ft)	180	172	53	41
95th Queue (ft)	383	399	229	61
Link Distance (ft)	233	233	233	416
Upstream Blk Time (%)	13	12	3	
Queuing Penalty (veh)	78	70	20	
Storage Bay Dist (ft)				
Storage Blk Time (%)				
Queuing Penalty (veh)				

Movement	SEU	SET	SER	NWL	NWT	NEL	NER
Lane Configurations							
Traffic Volume (vph)	3	1644	44	59	1677	308	83
Future Volume (vph)	3	1644	44	59	1677	308	83
Ideal Flow (vphpl)	1900	1900	1900	1900	1900	1900	1900
Grade (%)		-1%			1%	0%	
Total Lost time (s)	4.0	4.0		7.0	4.0	4.0	4.0
Lane Util. Factor	1.00	0.95		1.00	0.95	1.00	1.00
Frt	1.00	1.00		1.00	1.00	1.00	0.85
Flt Protected	0.95	1.00		0.95	1.00	0.95	1.00
Satd. Flow (prot)	1778	3543		1761	3522	1770	1583
Flt Permitted	0.11	1.00		0.07	1.00	0.95	1.00
Satd. Flow (perm)	201	3543		137	3522	1770	1583
Peak-hour factor, PHF	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Adj. Flow (vph)	3	1644	44	59	1677	308	83
RTOR Reduction (vph)	0	2	0	0	0	0	62
Lane Group Flow (vph)	3	1686	0	59	1677	308	21
Turn Type	Perm	NA		pm+pt	NA	Prot	Perm
Protected Phases		6		5	2	4	
Permitted Phases	6			2			4
Actuated Green, G (s)	47.2	47.2		62.2	62.2	23.1	23.1
Effective Green, g (s)	51.5	51.5		62.2	66.5	25.5	25.5
Actuated g/C Ratio	0.52	0.52		0.62	0.66	0.26	0.26
Clearance Time (s)	8.3	8.3		7.0	8.3	6.4	6.4
Vehicle Extension (s)	3.0	3.0		3.0	3.0	3.0	3.0
Lane Grp Cap (vph)	103	1824		215	2342	451	403
v/s Ratio Prot		c0.48		0.02	c0.48	c0.17	
v/s Ratio Perm	0.01			0.15			0.01
v/c Ratio	0.03	0.92		0.27	0.72	0.68	0.05
Uniform Delay, d1	11.9	22.4		20.4	10.7	33.6	28.1
Progression Factor	1.00	1.00		0.88	0.28	1.00	1.00
Incremental Delay, d2	0.5	9.4		0.4	1.2	8.1	0.2
Delay (s)	12.5	31.9		18.4	4.2	41.7	28.4
Level of Service	B	C		B	A	D	C
Approach Delay (s)		31.9			4.7	38.9	
Approach LOS		C			A	D	

Intersection Summary			
HCM 2000 Control Delay	20.2	HCM 2000 Level of Service	C
HCM 2000 Volume to Capacity ratio	0.87		
Actuated Cycle Length (s)	100.0	Sum of lost time (s)	15.0
Intersection Capacity Utilization	72.8%	ICU Level of Service	C
Analysis Period (min)	15		
c Critical Lane Group			

Intersection: 2: Olde Forge Road & Warrenton Rd (Rte.17)

Movement	SE	SE	NW	NW	NW	NE	NE
Directions Served	T	TR	L	T	T	L	R
Maximum Queue (ft)	383	357	52	264	328	200	74
Average Queue (ft)	364	341	35	161	232	149	39
95th Queue (ft)	382	380	49	292	351	207	68
Link Distance (ft)	351	351		2725	2725	873	
Upstream Blk Time (%)	17	15					
Queuing Penalty (veh)	141	129					
Storage Bay Dist (ft)			200				150
Storage Blk Time (%)	40			5		7	
Queuing Penalty (veh)	1			3		6	

HCM Signalized Intersection Capacity Analysis
15: Lendall Ln (1015)/Solomon Drive & Warrenton Rd (Rte.17)

03/29/2018

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (vph)	100	1532	24	16	1537	41	54	2	22	53	4	68
Future Volume (vph)	100	1532	24	16	1537	41	54	2	22	53	4	68
Ideal Flow (vphpl)	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900
Grade (%)		-1%			1%			1%			2%	
Total Lost time (s)	4.0	4.0	4.0	4.0	4.0	4.0		4.0		4.0	4.0	4.0
Lane Util. Factor	1.00	0.95	1.00	1.00	0.95	1.00		1.00		1.00	1.00	1.00
Frt	1.00	1.00	0.85	1.00	1.00	0.85		0.96		1.00	1.00	0.85
Flt Protected	0.95	1.00	1.00	0.95	1.00	1.00		0.97		0.95	1.00	1.00
Satd. Flow (prot)	1778	3455	1591	1761	3421	1575		1723		1752	1844	1567
Flt Permitted	0.06	1.00	1.00	0.07	1.00	1.00		0.97		0.95	1.00	1.00
Satd. Flow (perm)	118	3455	1591	123	3421	1575		1723		1752	1844	1567
Peak-hour factor, PHF	0.88	0.88	0.88	0.88	0.88	0.88	0.88	0.88	0.88	0.88	0.88	0.88
Adj. Flow (vph)	114	1741	27	18	1747	47	61	2	25	60	5	77
RTOR Reduction (vph)	0	0	10	0	0	19	0	15	0	0	0	71
Lane Group Flow (vph)	114	1741	17	18	1747	28	0	73	0	60	5	6
Heavy Vehicles (%)	2%	5%	2%	2%	5%	2%	2%	2%	2%	2%	2%	2%
Turn Type	pm+pt	NA	Perm	pm+pt	NA	Perm	Split	NA		Split	NA	Perm
Protected Phases	1	6		5	2		3	3		4	4	
Permitted Phases	6		6	2		2						4
Actuated Green, G (s)	64.4	59.3	59.3	58.2	56.2	56.2		4.1		4.0	4.0	4.0
Effective Green, g (s)	72.2	63.2	63.2	66.0	60.1	60.1		7.3		7.6	7.6	7.6
Actuated g/C Ratio	0.72	0.63	0.63	0.66	0.60	0.60		0.07		0.08	0.08	0.08
Clearance Time (s)	7.9	7.9	7.9	7.9	7.9	7.9		7.2		7.6	7.6	7.6
Vehicle Extension (s)	3.0	3.0	3.0	3.0	3.0	3.0		3.0		3.0	3.0	3.0
Lane Grp Cap (vph)	234	2183	1005	177	2056	946		125		133	140	119
v/s Ratio Prot	c0.04	0.50		0.01	c0.51			c0.04		c0.03	0.00	
v/s Ratio Perm	0.31		0.01	0.06		0.02						0.00
v/c Ratio	0.49	0.80	0.02	0.10	0.85	0.03		0.59		0.45	0.04	0.05
Uniform Delay, d1	16.7	13.7	6.8	11.0	16.3	8.1		44.9		44.2	42.8	42.8
Progression Factor	2.10	0.11	1.00	1.00	1.00	1.00		1.00		1.00	1.00	1.00
Incremental Delay, d2	0.9	1.9	0.0	0.3	4.6	0.1		6.8		2.4	0.1	0.2
Delay (s)	36.2	3.3	6.9	11.3	20.9	8.2		51.7		46.6	42.9	43.0
Level of Service	D	A	A	B	C	A		D		D	D	D
Approach Delay (s)		5.3			20.5			51.7			44.5	
Approach LOS		A			C			D			D	

Intersection Summary			
HCM 2000 Control Delay	14.8	HCM 2000 Level of Service	B
HCM 2000 Volume to Capacity ratio	0.75		
Actuated Cycle Length (s)	100.0	Sum of lost time (s)	16.0
Intersection Capacity Utilization	69.1%	ICU Level of Service	C
Analysis Period (min)	15		

c Critical Lane Group

Intersection: 15: Lendall Ln (1015)/Solomon Drive & Warrenton Rd (Rte.17)

Movement	EB	EB	EB	WB	WB	WB	WB	NB	SB	SB
Directions Served	L	T	T	L	T	T	R	LTR	L	R
Maximum Queue (ft)	67	345	322	16	352	354	2	92	71	95
Average Queue (ft)	48	144	138	3	229	221	0	61	40	68
95th Queue (ft)	64	333	302	13	381	390	2	92	79	95
Link Distance (ft)		2725	2725		2215	2215		526		
Upstream Blk Time (%)										
Queuing Penalty (veh)										
Storage Bay Dist (ft)	170			105			260		125	140
Storage Blk Time (%)		5	19		16	6				
Queuing Penalty (veh)		5	5		3	2				

HCM Unsignalized Intersection Capacity Analysis
14: Short St/Short St (1034) & Warrenton Rd (Rte.17) 03/29/2018

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (veh/h)	0	2193	170	0	1573	19	0	0	26	0	0	17
Future Volume (Veh/h)	0	2193	170	0	1573	19	0	0	26	0	0	17
Sign Control		Free			Free			Stop			Stop	
Grade		0%			1%			-2%			3%	
Peak Hour Factor	0.91	0.91	0.91	0.91	0.91	0.91	0.91	0.91	0.91	0.91	0.91	0.91
Hourly flow rate (vph)	0	2410	187	0	1729	21	0	0	29	0	0	19
Pedestrians												
Lane Width (ft)												
Walking Speed (ft/s)												
Percent Blockage												
Right turn flare (veh)												
Median type		None			None							
Median storage veh)												
Upstream signal (ft)					418							
pX, platoon unblocked	0.79						0.79	0.79		0.79	0.79	0.79
vC, conflicting volume	1750			2597			3294	4160	1205	2974	4336	875
vC1, stage 1 conf vol												
vC2, stage 2 conf vol												
vCu, unblocked vol	1420			2597			3371	4467	1205	2966	4690	313
tC, single (s)	4.1			4.1			7.5	6.5	6.9	7.5	6.5	6.9
tC, 2 stage (s)												
tF (s)	2.2			2.2			3.5	4.0	3.3	3.5	4.0	3.3
p0 queue free %	100			100			100	100	84	100	100	96
cM capacity (veh/h)	376			164			2	1	176	4	1	540

Direction, Lane #	EB 1	EB 2	EB 3	WB 1	WB 2	NB 1	SB 1
Volume Total	1205	1205	187	1153	597	29	19
Volume Left	0	0	0	0	0	0	0
Volume Right	0	0	187	0	21	29	19
cSH	1700	1700	1700	1700	1700	176	540
Volume to Capacity	0.71	0.71	0.11	0.68	0.35	0.16	0.04
Queue Length 95th (ft)	0	0	0	0	0	14	3
Control Delay (s)	0.0	0.0	0.0	0.0	0.0	29.4	11.9
Lane LOS						D	B
Approach Delay (s)	0.0			0.0		29.4	11.9
Approach LOS						D	B

Intersection Summary			
Average Delay	0.2		
Intersection Capacity Utilization	70.6%	ICU Level of Service	C
Analysis Period (min)	15		

Intersection: 14: Short St/Short St (1034) & Warrenton Rd (Rte.17)

Movement	EB	EB	EB	NB	SB
Directions Served	T	T	R	R	R
Maximum Queue (ft)	370	349	353	44	28
Average Queue (ft)	337	308	321	28	19
95th Queue (ft)	387	347	386	50	38
Link Distance (ft)	233	233	233	416	396
Upstream Blk Time (%)	30	46	36		
Queuing Penalty (veh)	185	279	220		
Storage Bay Dist (ft)					
Storage Blk Time (%)					
Queuing Penalty (veh)					

Movement	SEU	SET	SER	NWL	NWT	NEL	NER
Lane Configurations							
Traffic Volume (vph)	7	2103	130	107	1533	196	56
Future Volume (vph)	7	2103	130	107	1533	196	56
Ideal Flow (vphpl)	1900	1900	1900	1900	1900	1900	1900
Grade (%)		-1%			1%	0%	
Total Lost time (s)	4.0	4.0		8.3	4.0	4.0	4.0
Lane Util. Factor	1.00	0.95		1.00	0.95	1.00	1.00
Frt	1.00	0.99		1.00	1.00	1.00	0.85
Flt Protected	0.95	1.00		0.95	1.00	0.95	1.00
Satd. Flow (prot)	1778	3526		1761	3522	1770	1583
Flt Permitted	0.15	1.00		0.04	1.00	0.95	1.00
Satd. Flow (perm)	279	3526		67	3522	1770	1583
Peak-hour factor, PHF	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Adj. Flow (vph)	7	2103	130	107	1533	196	56
RTOR Reduction (vph)	0	2	0	0	0	0	42
Lane Group Flow (vph)	7	2231	0	107	1533	196	14
Turn Type	Perm	NA		pm+pt	NA	Prot	Perm
Protected Phases		6		5	2	4	
Permitted Phases	6			2			4
Actuated Green, G (s)	102.0	102.0		122.7	122.7	34.6	34.6
Effective Green, g (s)	106.3	106.3		122.7	127.0	37.0	37.0
Actuated g/C Ratio	0.62	0.62		0.71	0.74	0.22	0.22
Clearance Time (s)	8.3	8.3		8.3	8.3	6.4	6.4
Vehicle Extension (s)	3.0	3.0		3.0	3.0	3.0	3.0
Lane Grp Cap (vph)	172	2179		169	2600	380	340
v/s Ratio Prot		c0.63		0.05	c0.44	c0.11	
v/s Ratio Perm	0.03			0.40			0.01
v/c Ratio	0.04	1.02		0.63	0.59	0.52	0.04
Uniform Delay, d1	12.9	32.9		54.9	10.4	59.6	53.5
Progression Factor	1.00	1.00		1.00	1.00	1.00	1.00
Incremental Delay, d2	0.4	25.6		7.5	1.0	4.9	0.2
Delay (s)	13.3	58.4		62.4	11.4	64.5	53.7
Level of Service	B	E		E	B	E	D
Approach Delay (s)		58.3			14.7	62.1	
Approach LOS		E			B	E	

Intersection Summary			
HCM 2000 Control Delay	41.2	HCM 2000 Level of Service	D
HCM 2000 Volume to Capacity ratio	0.89		
Actuated Cycle Length (s)	172.0	Sum of lost time (s)	16.3
Intersection Capacity Utilization	95.0%	ICU Level of Service	F
Analysis Period (min)	15		
c Critical Lane Group			

Intersection: 2: Olde Forge Road & Warrenton Rd (Rte.17)

Movement	SE	SE	SE	NW	NW	NW	NE	NE
Directions Served	U	T	TR	L	T	T	L	R
Maximum Queue (ft)	31	359	360	184	504	565	168	30
Average Queue (ft)	6	355	355	123	349	388	132	24
95th Queue (ft)	26	360	361	175	510	570	218	43
Link Distance (ft)		351	351		2725	2725	873	
Upstream Blk Time (%)		35	38					
Queuing Penalty (veh)		385	417					
Storage Bay Dist (ft)	100			200				150
Storage Blk Time (%)		43		0	19		5	
Queuing Penalty (veh)		3		1	20		3	

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (vph)	32	2064	27	6	1454	23	65	9	22	45	5	35
Future Volume (vph)	32	2064	27	6	1454	23	65	9	22	45	5	35
Ideal Flow (vphpl)	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900
Grade (%)		-1%			1%			1%			2%	
Total Lost time (s)	4.0	4.0	4.0	4.0	4.0	4.0		4.0		4.0	4.0	4.0
Lane Util. Factor	1.00	0.95	1.00	1.00	0.95	1.00		1.00		1.00	1.00	1.00
Frt	1.00	1.00	0.85	1.00	1.00	0.85		0.97		1.00	1.00	0.85
Flt Protected	0.95	1.00	1.00	0.95	1.00	1.00		0.97		0.95	1.00	1.00
Satd. Flow (prot)	1778	3455	1591	1761	3421	1575		1737		1752	1844	1567
Flt Permitted	0.10	1.00	1.00	0.03	1.00	1.00		0.97		0.95	1.00	1.00
Satd. Flow (perm)	186	3455	1591	60	3421	1575		1737		1752	1844	1567
Peak-hour factor, PHF	0.88	0.88	0.88	0.88	0.88	0.88	0.88	0.88	0.88	0.88	0.88	0.88
Adj. Flow (vph)	36	2345	31	7	1652	26	74	10	25	51	6	40
RTOR Reduction (vph)	0	0	8	0	0	7	0	6	0	0	0	38
Lane Group Flow (vph)	36	2345	23	7	1652	19	0	103	0	51	6	2
Heavy Vehicles (%)	2%	5%	2%	2%	5%	2%	2%	2%	2%	2%	2%	2%
Turn Type	pm+pt	NA	Perm	pm+pt	NA	Perm	Split	NA		Split	NA	Perm
Protected Phases	1	6		5	2		3	3		4	4	
Permitted Phases	6		6	2		2						4
Actuated Green, G (s)	127.4	123.2	123.2	121.0	120.0	120.0		10.4		4.8	4.8	4.8
Effective Green, g (s)	135.2	127.1	127.1	128.8	123.9	123.9		13.6		8.4	8.4	8.4
Actuated g/C Ratio	0.80	0.75	0.75	0.76	0.73	0.73		0.08		0.05	0.05	0.05
Clearance Time (s)	7.9	7.9	7.9	7.9	7.9	7.9		7.2		7.6	7.6	7.6
Vehicle Extension (s)	3.0	3.0	3.0	3.0	3.0	3.0		3.0		3.0	3.0	3.0
Lane Grp Cap (vph)	223	2583	1189	94	2493	1147		138		86	91	77
v/s Ratio Prot	c0.01	c0.68		0.00	0.48			c0.06		c0.03	0.00	
v/s Ratio Perm	0.12		0.01	0.05		0.01						0.00
v/c Ratio	0.16	0.91	0.02	0.07	0.66	0.02		0.74		0.59	0.07	0.03
Uniform Delay, d1	9.7	16.9	5.5	27.8	12.1	6.3		76.5		79.1	77.1	76.9
Progression Factor	1.00	1.00	1.00	1.00	1.00	1.00		1.00		1.00	1.00	1.00
Incremental Delay, d2	0.3	6.0	0.0	0.3	1.4	0.0		19.3		10.5	0.3	0.1
Delay (s)	10.0	22.8	5.5	28.1	13.5	6.4		95.8		89.6	77.4	77.0
Level of Service	B	C	A	C	B	A		F		F	E	E
Approach Delay (s)		22.4			13.4			95.8			83.7	
Approach LOS		C			B			F			F	

Intersection Summary			
HCM 2000 Control Delay	22.2	HCM 2000 Level of Service	C
HCM 2000 Volume to Capacity ratio	0.85		
Actuated Cycle Length (s)	170.0	Sum of lost time (s)	16.0
Intersection Capacity Utilization	75.8%	ICU Level of Service	D
Analysis Period (min)	15		

c Critical Lane Group

Intersection: 15: Lendall Ln (1015)/Solomon Drive & Warrenton Rd (Rte.17)

Movement	EB	EB	EB	WB	WB	WB	WB	NB	SB	SB
Directions Served	L	T	T	L	T	T	R	LTR	L	R
Maximum Queue (ft)	21	43	54	14	224	240	2	158	66	71
Average Queue (ft)	8	30	11	3	161	167	0	108	39	36
95th Queue (ft)	24	56	46	12	259	277	2	197	69	65
Link Distance (ft)		2725	2725		2215	2215		526		
Upstream Blk Time (%)										
Queuing Penalty (veh)										
Storage Bay Dist (ft)	170			105			260		125	140
Storage Blk Time (%)			3		13	0				
Queuing Penalty (veh)			1		1	0				

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (veh/h)	0	2193	233	0	1613	19	0	0	29	0	0	17
Future Volume (Veh/h)	0	2193	233	0	1613	19	0	0	29	0	0	17
Sign Control		Free			Free			Stop			Stop	
Grade		0%			1%			-2%			3%	
Peak Hour Factor	0.91	0.91	0.91	0.91	0.91	0.91	0.91	0.91	0.91	0.91	0.91	0.91
Hourly flow rate (vph)	0	2410	256	0	1773	21	0	0	32	0	0	19
Pedestrians												
Lane Width (ft)												
Walking Speed (ft/s)												
Percent Blockage												
Right turn flare (veh)												
Median type		None			None							
Median storage veh)												
Upstream signal (ft)					418							
pX, platoon unblocked	0.79						0.79	0.79		0.79	0.79	0.79
vC, conflicting volume	1794			2666			3316	4204	1205	3020	4450	897
vC1, stage 1 conf vol												
vC2, stage 2 conf vol												
vCu, unblocked vol	1475			2666			3399	4522	1205	3026	4833	341
tC, single (s)	4.1			4.1			7.5	6.5	6.9	7.5	6.5	6.9
tC, 2 stage (s)												
tF (s)	2.2			2.2			3.5	4.0	3.3	3.5	4.0	3.3
p0 queue free %	100			100			100	100	82	100	100	96
cM capacity (veh/h)	358			154			2	1	176	4	1	518

Direction, Lane #	EB 1	EB 2	EB 3	WB 1	WB 2	NB 1	SB 1
Volume Total	1205	1205	256	1182	612	32	19
Volume Left	0	0	0	0	0	0	0
Volume Right	0	0	256	0	21	32	19
cSH	1700	1700	1700	1700	1700	176	518
Volume to Capacity	0.71	0.71	0.15	0.70	0.36	0.18	0.04
Queue Length 95th (ft)	0	0	0	0	0	16	3
Control Delay (s)	0.0	0.0	0.0	0.0	0.0	29.9	12.2
Lane LOS						D	B
Approach Delay (s)	0.0			0.0		29.9	12.2
Approach LOS						D	B

Intersection Summary			
Average Delay	0.3		
Intersection Capacity Utilization	70.6%	ICU Level of Service	C
Analysis Period (min)	15		

Intersection: 14: Short St/Short St (1034) & Warrenton Rd (Rte.17)

Movement	EB	EB	EB	NB	SB
Directions Served	T	T	R	R	R
Maximum Queue (ft)	369	342	341	147	28
Average Queue (ft)	308	283	252	109	20
95th Queue (ft)	434	375	466	149	38
Link Distance (ft)	233	233	233	416	396
Upstream Blk Time (%)	27	30	19		
Queuing Penalty (veh)	163	180	115		
Storage Bay Dist (ft)					
Storage Blk Time (%)					
Queuing Penalty (veh)					

Movement	SEU	SET	SER	NWL	NWT	NEL	NER
Lane Configurations							
Traffic Volume (vph)	7	2106	130	128	1533	236	66
Future Volume (vph)	7	2106	130	128	1533	236	66
Ideal Flow (vphpl)	1900	1900	1900	1900	1900	1900	1900
Grade (%)		-1%			1%	0%	
Total Lost time (s)	4.0	4.0		8.3	4.0	4.0	4.0
Lane Util. Factor	1.00	0.95		1.00	0.95	1.00	1.00
Frt	1.00	0.99		1.00	1.00	1.00	0.85
Flt Protected	0.95	1.00		0.95	1.00	0.95	1.00
Satd. Flow (prot)	1778	3526		1761	3522	1770	1583
Flt Permitted	0.15	1.00		0.04	1.00	0.95	1.00
Satd. Flow (perm)	282	3526		68	3522	1770	1583
Peak-hour factor, PHF	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Adj. Flow (vph)	7	2106	130	128	1533	236	66
RTOR Reduction (vph)	0	2	0	0	0	0	41
Lane Group Flow (vph)	7	2234	0	128	1533	236	25
Turn Type	Perm	NA		pm+pt	NA	Prot	Perm
Protected Phases		6		5	2	4	
Permitted Phases	6			2			4
Actuated Green, G (s)	101.0	101.0		122.7	122.7	34.6	34.6
Effective Green, g (s)	105.3	105.3		122.7	127.0	37.0	37.0
Actuated g/C Ratio	0.61	0.61		0.71	0.74	0.22	0.22
Clearance Time (s)	8.3	8.3		8.3	8.3	6.4	6.4
Vehicle Extension (s)	3.0	3.0		3.0	3.0	3.0	3.0
Lane Grp Cap (vph)	172	2158		180	2600	380	340
v/s Ratio Prot		c0.63		0.06	c0.44	c0.13	
v/s Ratio Perm	0.02			0.45			0.02
v/c Ratio	0.04	1.04		0.71	0.59	0.62	0.07
Uniform Delay, d1	13.3	33.4		58.2	10.4	61.1	53.8
Progression Factor	1.00	1.00		1.00	1.00	1.00	1.00
Incremental Delay, d2	0.4	29.1		12.4	1.0	7.4	0.4
Delay (s)	13.7	62.5		70.6	11.4	68.6	54.3
Level of Service	B	E		E	B	E	D
Approach Delay (s)		62.3			16.0	65.5	
Approach LOS		E			B	E	

Intersection Summary			
HCM 2000 Control Delay	44.2	HCM 2000 Level of Service	D
HCM 2000 Volume to Capacity ratio	0.92		
Actuated Cycle Length (s)	172.0	Sum of lost time (s)	16.3
Intersection Capacity Utilization	97.3%	ICU Level of Service	F
Analysis Period (min)	15		
c Critical Lane Group			

Intersection: 2: Olde Forge Road & Warrenton Rd (Rte.17)

Movement	SE	SE	SE	NW	NW	NW	NE	NE
Directions Served	U	T	TR	L	T	T	L	R
Maximum Queue (ft)	30	360	362	224	578	681	307	175
Average Queue (ft)	12	354	356	128	341	455	149	66
95th Queue (ft)	37	361	364	238	590	649	330	159
Link Distance (ft)		351	351		2725	2725	873	
Upstream Blk Time (%)		34	36					
Queuing Penalty (veh)		375	399					
Storage Bay Dist (ft)	100			200				150
Storage Blk Time (%)		42			17		10	0
Queuing Penalty (veh)		3			22		7	1

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (vph)	32	2077	27	6	1475	23	65	9	22	45	5	35
Future Volume (vph)	32	2077	27	6	1475	23	65	9	22	45	5	35
Ideal Flow (vphpl)	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900
Grade (%)		-1%			1%			1%			2%	
Total Lost time (s)	4.0	4.0	4.0	4.0	4.0	4.0		4.0		4.0	4.0	4.0
Lane Util. Factor	1.00	0.95	1.00	1.00	0.95	1.00		1.00		1.00	1.00	1.00
Frt	1.00	1.00	0.85	1.00	1.00	0.85		0.97		1.00	1.00	0.85
Flt Protected	0.95	1.00	1.00	0.95	1.00	1.00		0.97		0.95	1.00	1.00
Satd. Flow (prot)	1778	3455	1591	1761	3421	1575		1737		1752	1844	1567
Flt Permitted	0.10	1.00	1.00	0.03	1.00	1.00		0.97		0.95	1.00	1.00
Satd. Flow (perm)	178	3455	1591	60	3421	1575		1737		1752	1844	1567
Peak-hour factor, PHF	0.88	0.88	0.88	0.88	0.88	0.88	0.88	0.88	0.88	0.88	0.88	0.88
Adj. Flow (vph)	36	2360	31	7	1676	26	74	10	25	51	6	40
RTOR Reduction (vph)	0	0	8	0	0	7	0	6	0	0	0	38
Lane Group Flow (vph)	36	2360	23	7	1676	19	0	103	0	51	6	2
Heavy Vehicles (%)	2%	5%	2%	2%	5%	2%	2%	2%	2%	2%	2%	2%
Turn Type	pm+pt	NA	Perm	pm+pt	NA	Perm	Split	NA		Split	NA	Perm
Protected Phases	1	6		5	2		3	3		4	4	
Permitted Phases	6		6	2		2						4
Actuated Green, G (s)	127.4	123.2	123.2	121.0	120.0	120.0		10.4		4.8	4.8	4.8
Effective Green, g (s)	135.2	127.1	127.1	128.8	123.9	123.9		13.6		8.4	8.4	8.4
Actuated g/C Ratio	0.80	0.75	0.75	0.76	0.73	0.73		0.08		0.05	0.05	0.05
Clearance Time (s)	7.9	7.9	7.9	7.9	7.9	7.9		7.2		7.6	7.6	7.6
Vehicle Extension (s)	3.0	3.0	3.0	3.0	3.0	3.0		3.0		3.0	3.0	3.0
Lane Grp Cap (vph)	217	2583	1189	94	2493	1147		138		86	91	77
v/s Ratio Prot	c0.01	c0.68		0.00	0.49			c0.06		c0.03	0.00	
v/s Ratio Perm	0.12		0.01	0.05		0.01						0.00
v/c Ratio	0.17	0.91	0.02	0.07	0.67	0.02		0.74		0.59	0.07	0.03
Uniform Delay, d1	10.1	17.1	5.5	28.7	12.3	6.3		76.5		79.1	77.1	76.9
Progression Factor	1.00	1.00	1.00	1.00	1.00	1.00		1.00		1.00	1.00	1.00
Incremental Delay, d2	0.4	6.3	0.0	0.3	1.5	0.0		19.3		10.5	0.3	0.1
Delay (s)	10.4	23.4	5.5	29.1	13.7	6.4		95.8		89.6	77.4	77.0
Level of Service	B	C	A	C	B	A		F		F	E	E
Approach Delay (s)		23.0			13.7			95.8			83.7	
Approach LOS		C			B			F			F	

Intersection Summary			
HCM 2000 Control Delay	22.5	HCM 2000 Level of Service	C
HCM 2000 Volume to Capacity ratio	0.86		
Actuated Cycle Length (s)	170.0	Sum of lost time (s)	16.0
Intersection Capacity Utilization	76.2%	ICU Level of Service	D
Analysis Period (min)	15		

c Critical Lane Group

Intersection: 15: Lendall Ln (1015)/Solomon Drive & Warrenton Rd (Rte.17)

Movement	EB	EB	EB	EB	WB	WB	WB	WB	NB	SB	SB	SB
Directions Served	L	T	T	R	L	T	T	R	LTR	L	T	R
Maximum Queue (ft)	44	37	60	6	18	206	216	1	140	72	50	45
Average Queue (ft)	29	15	44	1	7	121	136	0	86	45	25	27
95th Queue (ft)	48	38	74	5	20	243	266	1	153	76	52	52
Link Distance (ft)		2725	2725			2215	2215		526		674	
Upstream Blk Time (%)												
Queuing Penalty (veh)												
Storage Bay Dist (ft)	170			50	105			260		125		140
Storage Blk Time (%)			7			15						
Queuing Penalty (veh)			2			1						