Virginia Department of Transportation

VDOT Signal Justification Report (SJR)
Northern District

Note: Text in small gray font is sample input or guidance only & should be removed from the final document before conversion to PDF. The full SJR, including appendices (A, B, and C as noted at the back of this template), should be submitted electronically as a PDF file to avoid unnecessary printing and allow for efficient review by VDOT.

Refer to the latest edition of IIM-TE-387 for additional information about the application of the SJR process in various scenarios.

COMMONWEALTH OF VIRGINIA
KAYLA M. ORD
Lic. No. 060923
08/24/2020
PROFESSIONAL ENGINEER

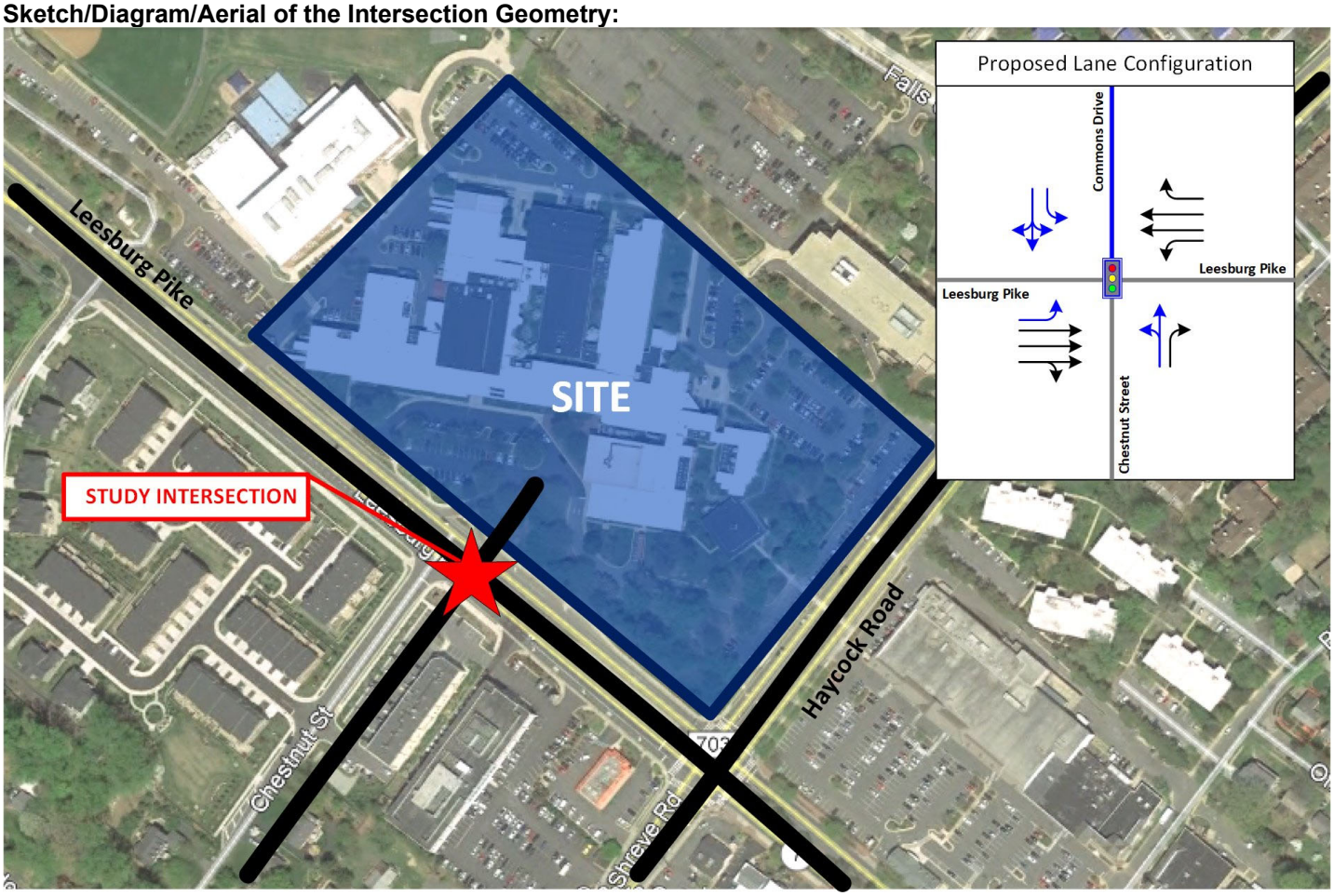
Kayla Ord
08/24/2020

Gorove Slade
Chantilly, VA
Traffic Engineer

Date: August 24, 2020

I. Study Intersection

Major Street Route # and Name: Leesburg Pike (Route 7) **Direction:** East/West
Minor Street Route # and Name: Chestnut Street (Route 1750) & Commons Drive (NIS) **Direction:** Choose an item.
County or Locality: City of Falls Church
Is the Intersection on the Arterial Preservation Network (APN)?: Yes
Sketch/Diagram/Aerial of the Intersection Geometry:



Describe the origin and nature of the request. If this SJR is based on a recommendation from another study (e.g. Traffic Impact Analysis or Safety Study), then note the name/date of the study and attach the study to this SJR.
Submit the relevant study (e.g. traffic impact analysis, safety study, etc.) as Appendix C to the SJR if the study documents are available. If the relevant document is not available, please note the name and date of the study. VDOT reserves the right to request additional information related to the studies if the information included is incomplete.
The Traffic Impact Study dated August 24, 2020 for the West Falls Church Economic Development Project recommends a traffic signal at this intersection in the future with development conditions. This TIA has been submitted concurrently with this SJR. Further, this intersection has undergone extensive coordination with VDOT, the City of Falls Church, and Fairfax County to determine that a signal at this location is the best regional solution.

This study is being completed to determine the need for a signal at the subject intersection for the portion of development that is anticipated to occur in the next two years.

If the origin of this SJR comes from another study, what were the key conclusions from that study that are related to this intersection? Provide a brief narrative summarizing the applicable recommendations at this intersection.
The Traffic Impact Study recommended a traffic signal at this location for multiple reasons.

1. The addition of a traffic signal allows for left turns into and out of the site from Leesburg Pike. This reduces the turning volumes at Leesburg Pike and Haycock Road and improves operations.
2. The signal provides a pedestrian connection across Leesburg Pike which is anticipated to be used by the schools, the mixed-use development, and the region.
3. This intersection currently experiences a high volume of crashes. A signal is anticipated to improve safety.

II. Signal Warrant Analysis Summary

Fill in the checkboxes and tables. Submit the full and complete signal warrant analysis as Appendix A to the SJR. An up-to-date signal warrant analysis is required to be submitted along with this SJR.

Intersection Approach Information: The intersection approach information described in this section, including approach directions, should be consistent with the information described in Section 1.

Approach Direction	Road Name/Route Number	Approach Speed	Approach Speed Type	Approach Speed Notes[1]	Number of Through Lanes	Annual Average Daily Traffic (AADT)
Northbound	Chestnut Street	25 MPH	Posted Speed Limit	N/A	1	1,110
Southbound	Commons Drive	25 MPH	Assumed Speed Limit	N/A	1	7,220
Eastbound	Leesburg Pike	25 MPH	Posted Speed Limit	N/A	3	32,600
Westbound	Leesburg Pike	25 MPH	Posted Speed Limit	N/A	2	33,580

[1] If approach speed type is not the posted/statutory speed limit, explain the reason why the posted/statutory speed limit was not used.

Summary of Traffic Count Source:
Turning movement traffic data was collected at the study intersection on Thursday, February 14, 2019 between 7:00 AM and 7:00 PM.

Summary of MUTCD Signal Warrant Analysis:

MUTCD Signal Warrants	Warrant Satisfied?			Notes / Summary
Warrant 1: Eight-Hour Vehicular Volume	☒ Yes	☐ No	☐ N/A	
Warrant 1: VDOT ADT Option[1]	☐ Yes	☐ No	☒ N/A	
Warrant 2: Four-Hour Vehicular Volume	☒ Yes	☐ No	☐ N/A	
Warrant 3: Peak Hour[2]	☐ Yes	☐ No	☒ N/A	
Warrant 4: Pedestrian Volume	☐ Yes	☐ No	☒ N/A	Not Pursued
Warrant 5: School Crossing	☐ Yes	☐ No	☒ N/A	Not Pursued
Warrant 6: Coordinated Signal System	☐ Yes	☐ No	☒ N/A	
Warrant 7: Crash Experience[3]	☒ Yes	☐ No	☐ N/A	
Warrant 8: Roadway Network	☐ Yes	☒ No	☐ N/A	
Warrant 9: Intersection Near a Grade Crossing	☐ Yes	☒ No	☐ N/A	

[1] The VDOT ADT Estimate Warrant in the VA Supplement to the MUTCD may be used instead of MUTCD Warrants 1 and 2 if the DTE concurs that it is infeasible to project estimated opening-day volumes over 8 or more hours of the day. Refer to Chapter 4C of the Virginia Supplement to the MUTCD for additional information on the use of this option.
[2] As per MUTCD Section 4C.04, Warrant 3 shall only be applied in unusual cases, such as facilities that attract or discharge large numbers of vehicles over a short period of time.
[3] The Alternative Signal Warrant 7 – Crash Experience documented in FHWA Interim Approval #19 (IA-19) shall be used as per the Virginia Supplement to the MUTCD and the latest edition of IIM-TE-387. The most recent available three years of available crash data shall be used.

Are the signal warrant analyses based on current volumes or anticipated future volumes?
☐ Current volumes ☒ Anticipated future volumes/conditions

If the signal warrant is only met under future conditions, provide a summary of trip generation assumptions and anticipated development thresholds that will trigger the signal being justified:
Traffic volumes in the future (2022) conditions in the SJR were calculated based on the following assumptions:

- An inherent growth rate of 0.5% applied to the through volumes on Leesburg Pike
- Rerouted volumes based on the VDOT Route 7 connector ramp
- Rerouted volumes due to the proposed configuration of Chestnut Street
- Phase 1 of the West Falls Church Economic Development Program which includes:
    - 1,500-student high school
    - 600-student middle-school
    - 130,000 square feet of office
    - 134,000 square feet of retail
    - 530 multi family dwelling units
    - 225 senior living dwelling units
    - 10,000 SF daycare
    - 150-room hotel

The signal warrants are met in the future conditions with the above assumptions

Was the 70% volume reduction factor applied to various signal warrant thresholds?
☐ Yes ☒ No

If yes, please describe the additional justification based on engineering judgment for use of the 70% volume reduction factor:

Was a right-turn volume adjustment used?
☒ Yes ☐ No

Please describe the rationale for this determination (whether yes or no above) and how the right turn vehicle volumes were considered in the analysis:
The MUTCD/NCHRP 457 suggests reducing some or all of the right-turning volumes for minor street approaches having an exclusive right turn lane. The basis for such a justification is the degree of conflict between the minor street right-turning traffic and the traffic on the major street. Chestnut Street is planned to have an exclusive northbound right turn lane in the future conditions. Commons Drive is planned to have a shared left/thru/right lane in addition to an exclusive left lane. As per guidance received from VDOT reviewers on recent studies, since the minor street approach of Chestnut Street has an exclusive right turn lane, the right turn on red reduction was checked in Synchro assuming a traffic signal at the intersection. The results revealed a reduction ranging from 93% to 100%. Hence, a reduction of 100% was applied to the right turning traffic from the Chestnut Street approach.

III. VJuST Innovative Intersection Consideration

Summary of Potentially Feasible Innovative Intersections according to VJuST results:
The VDOT Junction Screening Tool (VJuST) was utilized to consider and screen innovative intersection configurations that address mobility and safety issues. A conventional signal, roundabout, and a two-way stop-controlled intersection were analyzed in the study.

No other configurations were analyzed due to right-of-way complications.

Innovative Intersection Type	Feasibility Decision and Reason
Roundabout	Is this Innovative Intersection type feasible? ☐ Yes ☒ No Explanation: The multilane roundabout design would contain right-of-way complications. Further, the overall intersection would operate at an unacceptable LOS and the VJuST screening showed a large v/c ratio.

IV. Intersection Configuration and Control Recommendations and Signal Justification

Intersection configuration and control recommendations:
A traffic signal is recommended at this intersection. The lane configuration is presented on the Page 1 figure.

Signal Justification:
The traffic signal is recommended at the intersection because the conventional signal operates at the lowest v/c ratio in the VJuST screening and the signal is warranted based on multiple warrants (Warrant 1, Warrant 2, and Warrant 7)

Is an Access Management Exception or Access Management Waiver necessitated by the recommended intersection control method?:
☒ Yes ☐ No

If yes, please describe the specific spacing requirement that is not met (e.g., which spacing requirement from the Road Design Manual Appendix F) and brief rationale for recommending this condition:
The spacing standard is met to the west. However, the signal is located approximately 206 feet from a right-out only commercial entrance (Alley 1) to the east. The spacing standard requires 250 feet. Due to the low volume and limited movement at Alley 1 and the regional improvement the signal would provide, the signal is recommended. This AME would be required with or without a signal.

VDOT Signal Justification Report (SJR) Signature Page

Major Street Route # and Name: Leesburg Pike (Route 7)
Minor Street Route # and Name: Chestnut Street (Route 1750) and Commons Drive (NIS)
County or Locality: City of Falls Church
Is the Intersection on Arterial Preservation Network (APN)?: Yes

I. Approvals

If the proposed traffic signal is on an Arterial Preservation Network (APN), the SJR requires approval from the District Traffic Engineer (DTE), District Engineer/Administrator (DE/DA), and State Traffic Engineer (STE). For a proposed traffic signal not on the APN, SJR only requires DTE approval. After the PE signs and seals the SJR, attach this Signature Page onto the signed and sealed SJR so that this Signature Page appears on Page 4 of the SJR.

Note that new crossovers on the APN must also be approved by the DE/DA and the State Location & Design Engineer as per IIM-LU-501.1.

Signal Justification Report Approvals:

District Traffic Engineer (DTE): Required for all SJRs, including those on locality-maintained roads to be installed using SMART SCALE or HSIP funds.

☐ Concur ☐ Do Not Concur

Name	Signature	Date

Comments:

_____

_____

_____

District Engineer/Administrator (DE/DA): Only required if SJR recommends a proposed signal on the APN.

☐ Concur ☐ Do Not Concur

Name	Signature	Date

Comments:

_____

_____

_____

State Traffic Engineer (STE): Only required if SJR recommends a proposed signal on the APN.

☐ Approved ☐ Denied

Name	Signature	Date

Comments:

_____

_____

_____

If additional comments are necessary, please attach the comments on another sheet.

Appendix A:
Signal Warrant Analysis

Please see Appendix C: Signal Justification Report

Appendix B:
VJuST Input and Results Worksheets

Please see Appendix C: Signal Justification Report

Appendix C (optional):
TIA, Safety Study, and/or other Studies (if available)

Signal Justification Report

Future with Phase 1 Development (2022) Conditions at Leesburg Pike and Chestnut Street/Commons Drive

Falls Church, Virginia

August 24, 2020

Prepared for:

City of Falls Church
Department of Public Works
300 Park Avenue
Suite 100 West Wing
Falls Church, VA 22046

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Executive Summary

The following report presents the findings of a Signal Justification Report (SJR) for the Future with Development (2022) conditions at the intersection of Leesburg Pike and Chestnut Street/Commons Drive in the City of Falls Church, Virginia. Traffic conditions and physical characteristics of this location were considered in order to determine the appropriateness of and need for the installation of a traffic control signal or alternative traffic control measures under the future conditions.

As stated in the VDOT Requirements for Signal Justification Reports for New and Reconstructed Signals, dated June 5, 2017:

> "The SJR shall justify why a signal is not merely warranted but also necessary, as per the standard statements in Section 4C.01 of the VA Supplement, "The satisfaction of a signal warrant or warrants shall not in itself require the installation of a traffic control signal … In order for a traffic signal to be justified, evidence of the need for right of way assignment beyond that which could be provided by a stop sign or other unsignalized intersection configuration shall be demonstrated. Examples of such a need include excessive delay, congestion, unfavorable approach conditions, or surrounding conditions that cause driver confusion"

This study is being conducted in conjunction with the EYA West Falls Church mixed-use development within the City of Falls Church. It is located north of Leesburg Pike, east of I-66, west of Haycock Road, and south of Falls Church Drive.

The subject site is composed of three parcels (City of Falls Church Parcel ID # 51-221-001, 51-221-002 and 51-221-003) totaling approximately 35 acres. Currently the site is occupied by an 800-student high school and a 600-student middle school. The proposed development is planned to include a 1,500-student high school, a 600-student middle-school, 330,000 square feet of office, 134,000 square feet of retail, 680 residential dwelling units, 225 senior living dwelling units, a 10,000 SF daycare, and a 150-room hotel at build-out, which is anticipated to be by 2025. It is important to note that, of the build-out development, the entirety of the school re-development, 130,000 square feet of office, 134,000 square feet of retail, 530 multi family dwelling units, 225 senior living dwelling units, a 10,000 SF daycare, and a 150-room hotel are anticipated to be built by 2022. For the purposes of this study, the development program, anticipated to be completed by 2022 is utilized in the signal warrant analysis.

The site is currently accessed by one (1) full access point on Leesburg Pike, one (1) right-in/right-out point on Leesburg Pike, and two (2) full access points on Haycock Road. With the proposed development, an additional full access and a right-out only driveway is planned along Leesburg Pike. The internal circulation will create connectivity between all the proposed buildings on site.

A previous TIA dated November 26, 2018 was previously approved for this development. However, changes to the development and site access has required additional analysis. This SJR is being submitted consecutively with the updated TIA per the City of Falls Church's request. The updated TIA assumes a signalized access point on Leesburg Pike at the subject intersection and relies on the approval of this SJR. Therefore, the updated TIA and the SJR are being submitted together.

Scope of Study

The following intersection was identified for inclusion in this study:

- Leesburg Pike and Chestnut Street/Commons Drive

An aerial map showing the study intersection location and the location of the development is provided in Figure 1.

Figure 1: Site Location

This Signal Justification Report evaluates standard signal warrants along with geometric and traffic flow characteristics to determine the appropriateness and need for a traffic signal or other alternative traffic control measures at the study intersection for the portion of the planned development that is anticipated to occur in the next two years.

Certain corridors of statewide significance in Virginia are designated as part of Arterial Preservation Network. This Network consists of two components:

- "Mobility Preservation Segments" (MPS's) – arterials outside of Urban Areas (population of 50,000 or more) that serve a critical function for interurban mobility where no parallel Interstate/freeway route exists.
- "Mobility Enhancement Segments" (MES's) – arterials within Urban Areas that serve a critical function for commerce, commuting, and multimodal mobility.

The Commonwealth Transportation Board (CTB) has expressed concern regarding the proliferation of new signals on the Arterial Preservation Network, and VDOT is currently promoting multiple strategies to enhance and/or preserve mobility on these arterial corridors. Therefore, additional policies and procedures were established in the IIM-TE-387.0 (*Requirements for Signal Justification Reports (SJRs) for New and Reconstructed Signals, dated July 5, 2017*) for all new traffic signals proposed on the

Arterial Preservation Network. Figure 2 shows that Leesburg Pike is part of Arterial Preservation Network and is recognized as Mobility Enhancement Segment (Non-CoSS).

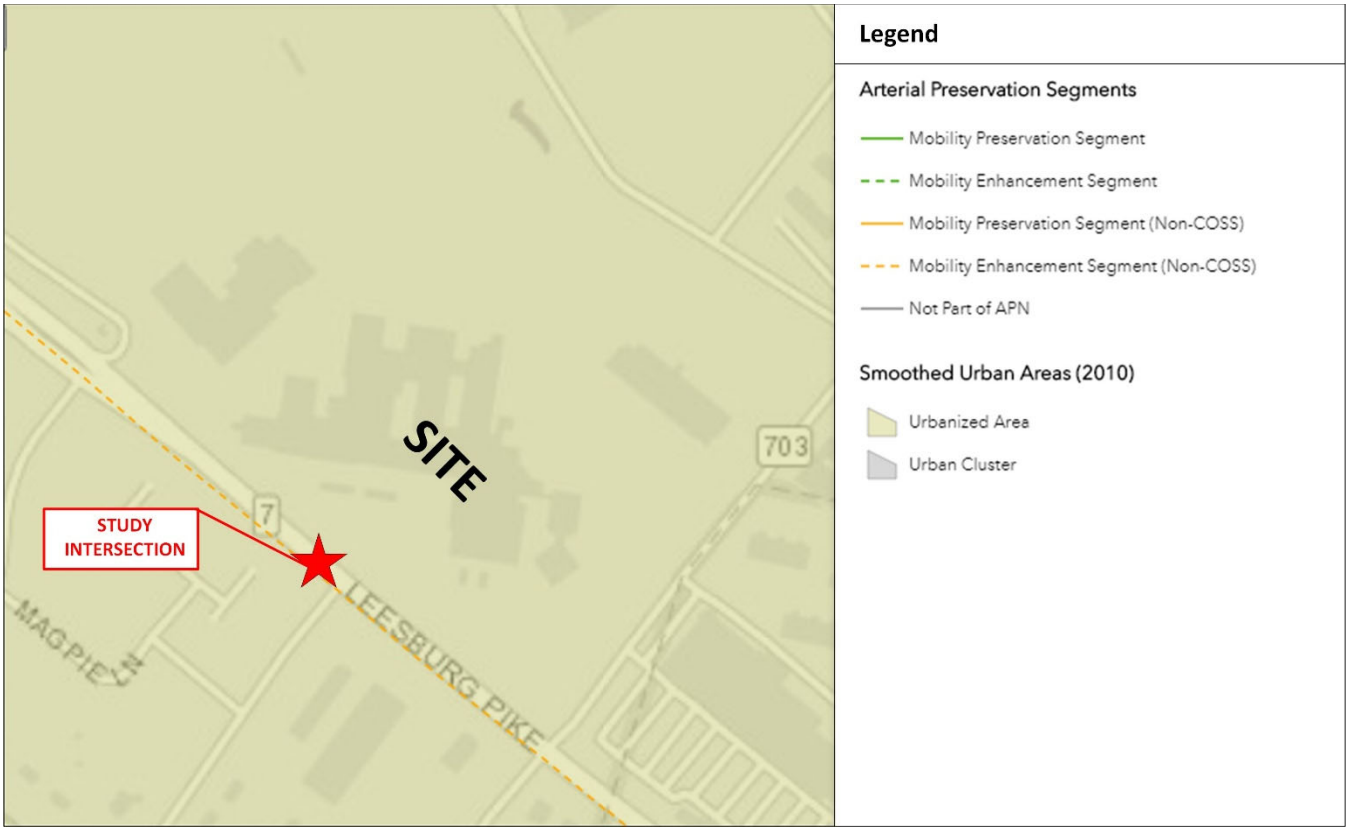


Figure 2: Arterial Preservation Network

Methodology

The following section presents the detailed evaluation of the traffic signal control warrants for the study intersection under the Future with Development (2022) conditions. The signal warrant analyses were performed following the procedures outlined in the 2011 update of the Virginia Supplement to the 2009 Manual on Uniform Traffic Control Devices (MUTCD).

Traffic signal warrant studies recommend 12-hour traffic volume counts during an average day. It is desirable that the hours selected contain the greatest percentage of the 24-hour traffic volume. In addition, the study may include vehicular volumes for each traffic movement from each approach collected in 15-minute increments during the morning and afternoon peak periods where the total traffic entering the intersection is greatest. Data that may be helpful pertains to vehicular type, pedestrian volume, information regarding uses of nearby facilities, the posted or statutory speed limit or 85th percentile speed on the uncontrolled approaches, the physical layout of the intersection, and collision diagrams.

The criterion of each warrant was evaluated using the information obtained for the projected future (2022) conditions.

Existing Conditions (2019)

Currently, the intersection of Leesburg Pike and Chestnut Street/Site Entrance operates under stop-control along the minor street. The site entrance operates as a right-in/right-out access for the school, while Chestnut Street allows for all turning movements into and out of the street.

With the proposed development, the site entrance is planned to operate as a full access point. With the re-development, the existing site entrance is planned to be designated as Commons Drive.

Existing Roadway Network

A description of the existing roadways within the study vicinity is presented below.

Leesburg Pike (Route 7) is a four-lane divided highway classified as a principal arterial by VDOT. Published historical traffic count data for the year 2019 from VDOT showed that Route 55 carried approximately 30,000 vpd between I-66 and WCL Falls Church. The posted speed limit in the vicinity of the study intersection is 25 mph. As mentioned previously, the roadway is identified as a "Mobility Enhancement Segment" under VDOT's Arterial Preservation Network. Figure 3 below illustrates the existing conditions at the study intersection.

Existing Traffic Volumes

In order to determine the weekday peak hour volumes, turning movement count data was collected for twelve hours at the study intersection on Thursday, February 14, 2019 between 7:00 AM and 7:00 PM. Analysis of the traffic data determined the following peak hours:

- AM Peak Hour: 7:45 AM to 8:45 AM
- PM Peak Hour: 4:30 PM to 5:30 PM

The existing volumes for the peak hours are illustrated on Figure 3. The twelve hour hourly existing volumes at the study intersection are illustrated in Table 1 and the peak hours are shown in Table 2. The raw traffic data in 15-minute interval is included in Appendix A.

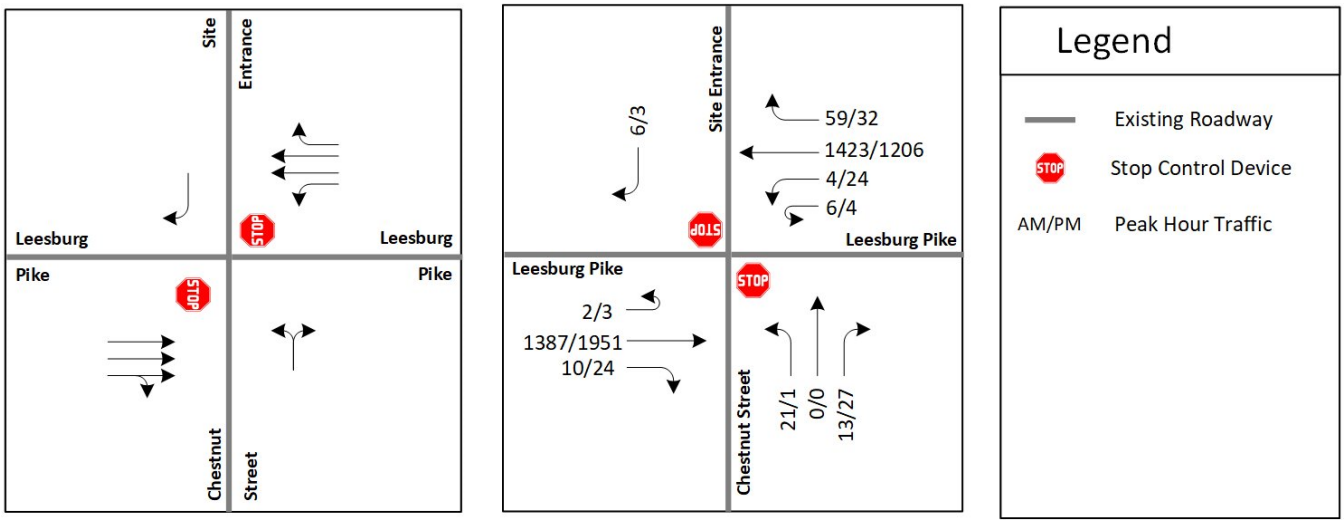


Figure 3: Lane Configuration and Peak Hour Volumes for Existing (2019) Conditions

Table 1: Existing Conditions (2019) – Hourly Traffic Volumes

			Major EB Leesburg Pike				Major WB Leesburg Pike				Minor NB Chestnut Street				Minor SB Commons Drive			
Hour			Left	Thru	Right	Total EB	Left	Thru	Right	Total WB	Left	Thru	Right	Total NB	Left	Thru	Right	Total SB
7:00 AM	to	8:00 AM	2	1,306	5	**1,313**	7	1,069	60	**1,136**	20	0	8	**28**	0	0	4	**4**
8:00 AM	to	9:00 AM	2	1,325	13	**1,340**	9	1,382	24	**1,415**	17	0	11	**28**	0	0	4	**4**
9:00 AM	to	10:00 AM	7	1,187	13	**1,207**	13	1,017	11	**1,041**	8	0	8	**16**	0	0	2	**2**
10:00 AM	to	11:00 AM	7	1,189	12	**1,208**	17	914	6	**937**	10	0	15	**25**	0	0	5	**5**
11:00 AM	to	12:00 PM	7	1,117	22	**1,146**	19	956	14	**989**	25	0	20	**45**	0	0	5	**5**
12:00 PM	to	1:00 PM	6	1,060	22	**1,088**	33	1,044	3	**1,080**	27	0	21	**48**	0	0	8	**8**
1:00 PM	to	2:00 PM	11	1,045	28	**1,084**	23	996	3	**1,022**	24	0	21	**45**	0	0	6	**6**
2:00 PM	to	3:00 PM	5	1,121	21	**1,147**	24	1,057	20	**1,101**	26	0	18	**44**	0	0	8	**8**
3:00 PM	to	4:00 PM	4	1,431	28	**1,463**	25	1,135	17	**1,177**	10	0	28	**38**	0	0	18	**18**
4:00 PM	to	5:00 PM	5	1,813	19	**1,837**	27	1,254	9	**1,290**	1	0	26	**27**	0	0	3	**3**
5:00 PM	to	6:00 PM	2	1,940	29	**1,971**	27	1,168	59	**1,254**	3	0	24	**27**	0	0	1	**1**
6:00 PM	to	7:00 PM	4	1,781	33	**1,818**	30	1,096	48	**1,174**	5	0	19	**24**	0	0	3	**3**

Table 2: Existing Conditions (2019) – Peak Hour Traffic Volumes

			Major EB Leesburg Pike				Major WB Leesburg Pike				Minor NB Chestnut Street				Minor SB Commons Drive			
Hour			Left	Thru	Right	Total EB	Left	Thru	Right	Total WB	Left	Thru	Right	Total NB	Left	Thru	Right	Total SB
7:45 AM	to	8:45 AM	2	1,387	10	**1,399**	10	1,423	59	**1,492**	21	0	13	**34**	0	0	6	**6**
4:30 PM	to	5:30 PM	3	1,951	24	**1,978**	28	1,206	32	**1,266**	1	0	27	**28**	0	0	3	**3**

Existing Safety Assessment

Crash Analysis

Crash Data at the intersection of Leesburg Pike and Chestnut Street/Commons Drive for the most recent five years was obtained from VDOT. Table 3 provides a summary of the reported crashes by severity and type.

Table 3: Intersection Crash Data

Intersection	Crash Data for the Intersection of Leesburg Pike and Chestnut Street (2015 - 2019)						
	2015	2016	2017	2018	2019	Total	Relative Frequency
Crash Severity							
Fatal Collision							0.00%
Injury Collision	1	6	3	2	1	13	38.24%
Type A							
Type B		*2*	*3*	*2*	*1*	*8*	
Type C	*1*	*4*				*5*	
Property Damage Only	5	5	5	2	4	21	61.76%
TOTAL*	**6**	**11**	**8**	**4**	**5**	**34**	**100.00%**
Crash Type							
Fixed Object/ Single-Vehicle Crash							0.00%
Head-On							0.00%
Sideswipe / Same Direction							0.00%
Sideswipe / Opposite Direction							0.00%
Rear-End Collision	2	1				3	8.82%
Angle Collision	4	10	8	4	3	29	85.29%
Backed Into							0.00%
Pedestrian Collision							0.00%
Deer/Animal							0.00%
Other					2	2	5.88%
TOTAL*	**6**	**11**	**8**	**4**	**5**	**34**	**100.00%**
Other Factors							
Distracted Driver		2				2	5.88%
Alcohol Related**							0.00%
Work-Zone Related							0.00%
Inclement Weather (Non-Dry)		2		1		3	8.82%
Speeding	1	1	1			3	8.82%
Disregard of Traffic Control Device							0.00%
Pedestrian Injury***							N/A
Time of Day							
AM Peak Period (6 - 10 AM)	3	2	2	1		8	23.53%
Off Peak - Daytime (10 AM - 3 PM)					2	2	5.88%
PM Peak Period (3 - 7 PM)	2	9	6	3	3	23	67.65%
Off Peak - Nighttime (7 PM - 6 AM)	1					1	2.94%
CALCULATED CRASH RATE						***0.62***	***Crashes per MEV***

* It should be noted that an intersection radius of 100 feet was used in this analysis. Crashes also thought to be caused by the intersection may have been added based on the description of the crash and engineering judgement.

** Instances where the event was classified as "Unknown", "Not Known Whether Impaired", "Ability Not Impaired" were classified as alcohol related to provide a more conservative analysis.

*** Pedestrian injuries are based on the number of pedestrians injured and may not be directly be related to the number of crash incidences (i.e., if one crash occurred injuring two pedestrians, the table would show a "2" instead of a "1").

As can be seen a total of 34 crashes occurred at the study intersection between 2015 and 2019. A majority of the crashes had a severity of “Property Damage Only (PDO)” with the remaining crash reported as “Injury Collison (IC)”. Of the 34 crashes, 29 were classified as angled collisions.

With the proposed development, Chestnut Street is planned to be converted to a signalized full access configuration. In the interim, VDOT has however installed traffic signs indicating No Left Turn and No U-Turn for Rt. 7 making left onto Chestnut as well as "No Left Turn" between 4:00 PM – 7:00 PM for Chestnut Street making a left out to Rt. 7. This decision was partially based on severity and quantity of accidents for motorists making left from Rt. 7 onto Chestnut. These signs have been recently installed. This is anticipated to reduce the number of crashes due to vehicles making the left turns onto Chestnut Street. Converting Chestnut Street to a signalized intersection is further expected to reduce these crashes.

The crash data for the subject intersection is included in Appendix B.

Future with Development (2022)

The proposed intersection will allow full access to both Chestnut Street and Commons Drive. The lane configuration for the intersection with the proposed development in place is shown in Figure 4.

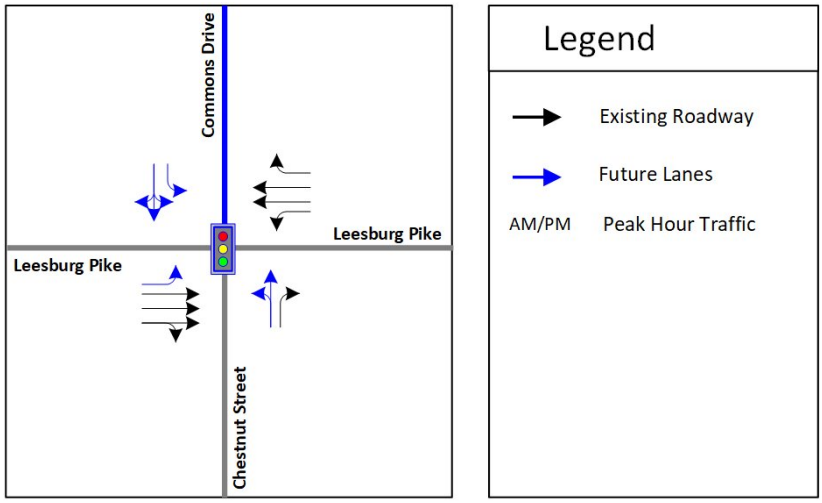


Figure 4: Future Lane Configuration (2022)

Future Traffic Volumes (2022)

The following steps were taken to estimate the traffic volumes at the study intersection in the future (2022) conditions:

1. **Inherent Growth**: Consistent with the Traffic Impact Study, an annual inherent growth of 0.5% per year was applied to all the through volumes on Leesburg Pike to account for the regional growth in traffic volumes. The traffic volumes due to the inherent growth are shown in Table 4.

Table 4: Inherent Growth Volumes (2022)

	Major EB Leesburg Pike			Major WB Leesburg Pike			Minor NB Chestnut Street			Minor SB Commons Drive		
Hour	Left	Thru	Right	Left	Thru	Right	Left	Thru	Right	Left	Thru	Right
7:00 AM to 8:00 AM	0	20	0	0	16	0	0	0	0	0	0	0
8:00 AM to 9:00 AM	0	20	0	0	21	0	0	0	0	0	0	0
9:00 AM to 10:00 AM	0	18	0	0	15	0	0	0	0	0	0	0
10:00 AM to 11:00 AM	0	18	0	0	14	0	0	0	0	0	0	0
11:00 AM to 12:00 PM	0	17	0	0	14	0	0	0	0	0	0	0
12:00 PM to 1:00 PM	0	16	0	0	16	0	0	0	0	0	0	0
1:00 PM to 2:00 PM	0	16	0	0	15	0	0	0	0	0	0	0
2:00 PM to 3:00 PM	0	17	0	0	16	0	0	0	0	0	0	0
3:00 PM to 4:00 PM	0	22	0	0	17	0	0	0	0	0	0	0
4:00 PM to 5:00 PM	0	27	0	0	19	0	0	0	0	0	0	0
5:00 PM to 6:00 PM	0	29	0	0	18	0	0	0	0	0	0	0
6:00 PM to 7:00 PM	0	27	0	0	17	0	0	0	0	0	0	0

2. **Route 7 Connector Ramp Reroute Traffic Volumes:** The Route 7 Connector Ramp is assumed to be in place by the future conditions (2022). The purpose of the VDOT Route 7 Connector Ramp is to provide vehicles on eastbound I-66 direct access to the West Falls Church Metrorail station parking. It is anticipated to redirect vehicles that would normally utilize eastbound Route 7 and make a left onto Haycock Road to access the Metrorail parking off Falls Church Drive. The reroute is consistent with the TIA. Hourly factors were applied to the peak hour trips to determine the off-peak hour

trips; the factors were based on the existing traffic observed on Leesburg Pike and are shown in Appendix C. The hourly traffic volumes for the Route 7 Connector Ramp Reroute are shown in Table 5.

Table 5: Route 7 Connector Ramp Reroute Volumes (2022)

Time Period	Major EB Leesburg Pike			Major WB Leesburg Pike			Minor NB Chestnut Street			Minor SB Commons Drive		
	Left	Thru	Right	Left	Thru	Right	Left	Thru	Right	Left	Thru	Right
7:00 AM 8:00 AM	0	-140	0	0	0	0	0	0	0	0	0	0
8:00 AM 9:00 AM	0	-157	0	0	0	0	0	0	0	0	0	0
9:00 AM 10:00 AM	0	-129	0	0	0	0	0	0	0	0	0	0
10:00 AM 11:00 AM	0	-123	0	0	0	0	0	0	0	0	0	0
11:00 AM 12:00 PM	0	-122	0	0	0	0	0	0	0	0	0	0
12:00 PM 1:00 PM	0	-202	0	0	0	0	0	0	0	0	0	0
1:00 PM 2:00 PM	0	-197	0	0	0	0	0	0	0	0	0	0
2:00 PM 3:00 PM	0	-210	0	0	0	0	0	0	0	0	0	0
3:00 PM 4:00 PM	0	-246	0	0	0	0	0	0	0	0	0	0
4:00 PM 5:00 PM	0	-292	0	0	0	0	0	0	0	0	0	0
5:00 PM 6:00 PM	0	-301	0	0	0	0	0	0	0	0	0	0
6:00 PM 7:00 PM	0	-279	0	0	0	0	0	0	0	0	0	0

3. **Site Generated Traffic Volumes**: As mentioned earlier, The proposed 2022 development is planned to include a 1,500-student high school, a 600-student middle-school, 130,000 square feet of office, 134,000 square feet of retail, 530 multi family dwelling units, 225 senior living dwelling units, a 10,000 SF daycare, and a 150-room hotel. The trip distribution assumptions for the development were kept consistent with the TIA. The trip generation/distribution assumptions used in this study are included in Appendix D. Hourly factors were applied to the peak hour trips to determine the off-peak hour trips; the factors were based on ITE's Trip Generation Manual 10th Edition and are shown in Appendix D. The hourly traffic volumes, including pass-by trips, for the West Falls Church Economic Development are shown in Table 6.

Table 6: Site Generated Traffic Volumes (2022)

Hour	Major EB Leesburg Pike			Major WB Leesburg Pike			Minor NB Chestnut Street			Minor SB Commons Drive		
	Left	Thru	Right	Left	Thru	Right	Left	Thru	Right	Left	Thru	Right
7:00 AM to 8:00 AM	126	-20	0	0	74	518	0	0	0	339	0	70
8:00 AM to 9:00 AM	156	-22	0	0	73	221	0	0	0	153	0	68
9:00 AM to 10:00 AM	203	-35	0	0	54	130	0	0	0	92	0	81
10:00 AM to 11:00 AM	293	-53	0	0	59	154	0	0	0	105	0	113
11:00 AM to 12:00 PM	425	-77	0	0	83	218	0	0	0	146	0	159
12:00 PM to 1:00 PM	167	-37	0	0	29	83	0	0	0	108	0	99
1:00 PM to 2:00 PM	154	-35	0	0	25	81	0	0	0	105	0	93
2:00 PM to 3:00 PM	166	-39	0	0	23	142	0	0	0	169	0	104
3:00 PM to 4:00 PM	165	-38	0	0	28	164	0	0	0	193	0	102
4:00 PM to 5:00 PM	188	-41	0	0	33	159	0	0	0	190	0	111
5:00 PM to 6:00 PM	202	-42	0	0	37	185	0	0	0	218	0	118
6:00 PM to 7:00 PM	152	-30	0	0	29	141	0	0	0	162	0	89

4. **Existing Removed Traffic Volumes:** The existing development is being removed with the proposed development. Therefore, the existing site traffic based on existing counts have been removed from the intersection as shown in Table 7.

Table 7: Existing Removed Traffic Volumes

Hour	Major EB Leesburg Pike			Major WB Leesburg Pike			Minor NB Chestnut Street			Minor SB Commons Drive		
	Left	Thru	Right	Left	Thru	Right	Left	Thru	Right	Left	Thru	Right
7:00 AM to 8:00 AM	-2	0	0	0	0	-60	0	0	0	0	0	-4
8:00 AM to 9:00 AM	-1	0	0	0	0	-24	0	0	0	0	0	-4
9:00 AM to 10:00 AM	-5	0	0	0	0	-11	0	0	0	0	0	-2
10:00 AM to 11:00 AM	-2	0	0	0	0	-6	0	0	0	0	0	-5
11:00 AM to 12:00 PM	-4	0	0	0	0	-14	0	0	0	0	0	-5
12:00 PM to 1:00 PM	-4	0	0	0	0	-3	0	0	0	0	0	-8
1:00 PM to 2:00 PM	-5	0	0	0	0	-3	0	0	0	0	0	-6
2:00 PM to 3:00 PM	-2	0	0	0	0	-20	0	0	0	0	0	-8
3:00 PM to 4:00 PM	0	0	0	0	0	-17	0	0	0	0	0	-18
4:00 PM to 5:00 PM	-1	0	0	0	0	-9	0	0	0	0	0	-3
5:00 PM to 6:00 PM	0	0	0	0	0	-59	0	0	0	0	0	-1
6:00 PM to 7:00 PM	-1	0	0	0	0	-48	0	0	0	0	0	-3

5. **Chestnut Street Rerouted Volumes**: As part of the proposed application, Chestnut Street is being aligned with Commons Drive. This improvement removes one of the existing entrances to the commercial center and provides an improved intersection to the users. Existing peak hour counts at Chestnut Street and the commercial center RIRO were combined to reflect the existing commercial building and office building trips that would utilize this intersection. In order to be conservative, additional trips were added to the northbound left movement as the proposed signal could attract additional trips from the office building. These additional trips were assumed based on the office building trip generation, parking locations, and the proposed distributions and resulted in an additional 3 trips in the AM peak hour and 15 trips in the PM peak hour. Hourly factors were applied to the peak hour trips to determine the off-peak hour trips; the factors were based on ITE's Trip Generation Manual 10th Edition for office use. The hourly volumes for the Chestnut Street Rerouted Volumes are shown in Table 8.

Table 8: Chestnut Street Rerouted Traffic Volumes

Time Period	Major EB Leesburg Pike			Major WB Leesburg Pike			Minor NB Chestnut Street			Minor SB Commons Drive		
	Left	Thru	Right	Left	Thru	Right	Left	Thru	Right	Left	Thru	Right
7:00 AM 8:00 AM	0	-15	15	0	0	0	3	0	0	0	0	0
8:00 AM 9:00 AM	0	-19	19	0	0	0	4	0	0	0	0	0
9:00 AM 10:00 AM	0	-12	12	0	0	0	2	0	0	0	0	0
10:00 AM 11:00 AM	0	-13	13	0	0	0	3	0	0	0	0	0
11:00 AM 12:00 PM	0	-18	18	0	0	0	4	0	0	0	0	0
12:00 PM 1:00 PM	0	-7	7	0	0	0	15	0	13	0	0	0
1:00 PM 2:00 PM	0	-6	6	0	0	0	12	0	10	0	0	0
2:00 PM 3:00 PM	0	-5	5	0	0	0	11	0	9	0	0	0
3:00 PM 4:00 PM	0	-5	5	0	0	0	11	0	9	0	0	0
4:00 PM 5:00 PM	0	-7	7	0	0	0	15	0	13	0	0	0
5:00 PM 6:00 PM	0	-7	7	0	0	0	15	0	13	0	0	0
6:00 PM 7:00 PM	0	-2	2	0	0	0	3	0	3	0	0	0

6. **Total Traffic Volumes:** The traffic generated by the inherent growth, the Route 7 Connector reroute, the site generated traffic, the existing removed traffic, and the Chestnut Street Reroute was added to the existing volumes to determine the future hourly volumes (2022) as shown in Table 9.

Table 9: Future Conditions (2022) Traffic Volumes

			Major EB Leesburg Pike				Major WB Leesburg Pike				Minor NB Chestnut Street				Minor SB Commons Drive			
Hour			Left	Thru	Right	Total EB	Left	Thru	Right	Total WB	Left	Thru	Right	Total NB	Left	Thru	Right	Total SB
7:00 AM	to	8:00 AM	126	1,151	20	**1,297**	7	1,159	518	**1,684**	23	0	8	**31**	339	0	70	**409**
8:00 AM	to	9:00 AM	157	1,147	32	**1,336**	9	1,476	221	**1,706**	21	0	11	**32**	153	0	68	**221**
9:00 AM	to	10:00 AM	205	1,029	25	**1,259**	13	1,086	130	**1,229**	10	0	8	**18**	92	0	81	**173**
10:00 AM	to	11:00 AM	298	1,018	25	**1,341**	17	987	154	**1,158**	13	0	15	**28**	105	0	113	**218**
11:00 AM	to	12:00 PM	428	917	40	**1,385**	19	1,053	218	**1,290**	29	0	20	**49**	146	0	159	**305**
12:00 PM	to	1:00 PM	169	830	29	**1,028**	33	1,089	83	**1,205**	42	0	34	**76**	108	0	99	**207**
1:00 PM	to	2:00 PM	160	823	34	**1,017**	23	1,036	81	**1,140**	36	0	31	**67**	105	0	93	**198**
2:00 PM	to	3:00 PM	169	884	26	**1,079**	24	1,096	142	**1,262**	37	0	27	**64**	169	0	104	**273**
3:00 PM	to	4:00 PM	169	1,164	33	**1,366**	25	1,180	164	**1,369**	21	0	37	**58**	193	0	102	**295**
4:00 PM	to	5:00 PM	192	1,500	26	**1,718**	27	1,306	159	**1,492**	16	0	39	**55**	190	0	111	**301**
5:00 PM	to	6:00 PM	204	1,619	36	**1,859**	27	1,223	185	**1,435**	18	0	37	**55**	218	0	118	**336**
6:00 PM	to	7:00 PM	155	1,497	35	**1,687**	30	1,142	141	**1,313**	8	0	22	**30**	162	0	89	**251**

Adjustments for Right Turns

The MUTCD/NCHRP 457 suggests reducing some or all of the right-turning volumes for minor street approaches having an exclusive right turn lane. The basis for such a justification is the degree of conflict between the minor street right-turning traffic and the traffic on the major street. Chestnut Street is planned to have an exclusive northbound right turn lane in the future conditions. Commons Drive is planned to have a shared left/thru/right lane in addition to an exclusive left lane. As per guidance received from VDOT reviewers on recent studies, since the minor street approach of Chestnut Street has an exclusive right turn lane, the right turn on red reduction was checked in Synchro assuming a traffic signal at the intersection. The results revealed a reduction ranging from 93% to 100%. Hence, **a reduction of 100% was applied to the right turning traffic from the Chestnut Street approach.**

Future Volumes (2022) for Warrant Evaluation

The traffic volumes on Leesburg Pike (major street), Chestnut Street (NB minor street), and Commons Drive (SB minor street) used for the evaluation of the warrants are shown in Table 10, Table 11, and Table 12 respectively. Please note that the volumes shown in Table 11 reflect a reduction of 100% applied to the right turning traffic on the northbound approach.

Table 10: Future Conditions (2022) Traffic Volumes – Leesburg Pike (Major Street)

Hour	Existing Volume EB/WB (A)	Growth Volume EB/WB (B)	Rerouted Volumes & Existing Removed (C)	Site Volume EB/WB (D)	Total Volume EB/WB E = A + B + C + D
7:00 AM to 8:00 AM	2,449	36	-202	698	2,981
8:00 AM to 9:00 AM	2,755	41	-182	428	3,042
9:00 AM to 10:00 AM	2,248	33	-145	352	2,488
10:00 AM to 11:00 AM	2,145	32	-131	453	2,499
11:00 AM to 12:00 PM	2,135	31	-140	649	2,675
12:00 PM to 1:00 PM	2,168	32	-209	242	2,233
1:00 PM to 2:00 PM	2,106	31	-205	225	2,157
2:00 PM to 3:00 PM	2,248	33	-232	292	2,341
3:00 PM to 4:00 PM	2,640	39	-263	319	2,735
4:00 PM to 5:00 PM	3,127	46	-302	339	3,210
5:00 PM to 6:00 PM	3,225	47	-360	382	3,294
6:00 PM to 7:00 PM	2,992	44	-328	292	3,000

Table 11: Future Conditions (2022) Traffic Volumes – Chestnut Street (NB Minor Street)

Hour	Existing Volume NB (A)	Growth Volume NB (B)	Rerouted Volumes & Existing Removed (C)	Site Volume NB (D)	Total Volume NB E = A + B + C + D
7:00 AM to 8:00 AM	20	0	3	0	23
8:00 AM to 9:00 AM	17	0	4	0	21
9:00 AM to 10:00 AM	8	0	2	0	10
10:00 AM to 11:00 AM	10	0	3	0	13
11:00 AM to 12:00 PM	25	0	4	0	29
12:00 PM to 1:00 PM	27	0	15	0	42
1:00 PM to 2:00 PM	24	0	12	0	36
2:00 PM to 3:00 PM	26	0	11	0	37
3:00 PM to 4:00 PM	10	0	11	0	21
4:00 PM to 5:00 PM	1	0	15	0	16
5:00 PM to 6:00 PM	3	0	15	0	18
6:00 PM to 7:00 PM	5	0	3	0	8

*** 100% of the right turning traffic was removed from the NB approach**

Table 12: Future Conditions (2022) Traffic Volumes – Commons Drive (SB Minor Street)

Hour	Existing Volume SB (A)	Growth Volume SB (B)	Rerouted Volumes & Existing Removed (C)	Site Volume SB (D)	Total Volume SB E = A + B + C + D
7:00 AM to 8:00 AM	4	0	-4	409	409
8:00 AM to 9:00 AM	4	0	-4	221	221
9:00 AM to 10:00 AM	2	0	-2	173	173
10:00 AM to 11:00 AM	5	0	-5	218	218
11:00 AM to 12:00 PM	5	0	-5	305	305
12:00 PM to 1:00 PM	8	0	-8	207	207
1:00 PM to 2:00 PM	6	0	-6	198	198
2:00 PM to 3:00 PM	8	0	-8	273	273
3:00 PM to 4:00 PM	18	0	-18	295	295
4:00 PM to 5:00 PM	3	0	-3	301	301
5:00 PM to 6:00 PM	1	0	-1	336	336
6:00 PM to 7:00 PM	3	0	-3	251	251

*** No Right Turn on Reduction was applied**

Warrant Analysis Results – Future (2022) Conditions

Warrant 1: Eight-Hour Vehicular Volume

Warrant 1 is satisfied when for each of any 8 hours of an average day, the traffic volumes given in the tables shown below exist on the major-street and on the higher-volume minor-street approaches to the intersection. If the vehicles per hour given in both of the 100% columns in the MUTCD Table 4C-1 exist on the major-street and the higher-volume minor-street approaches, respectively, to the intersection and satisfy either Condition A or Condition B for any eight hours of an average weekday, then Warrant 1 is satisfied. The condition for the major-street and minor-street shall be for the same 8 hours. On the minor-street, the higher volume shall not be required to be on the same approach during each of these 8 hours.

If the posted or statutory speed limit or the 85th-percentile speed on the major street exceeds 40mph, or if the intersection lies within an isolated community having a population of less than 10,000, the traffic volumes in the 70% columns in Table 4C-1 may be used in place of the 100% columns.

The 80% columns may be used in place of the 100% columns when street volumes for both the major-street and minor-street approaches meet or exceed the 80% values set forth in the MUTCD and satisfy both Conditions A and B for each of any 8 hours of an average day. The condition for the major-street and minor-street shall be for the same 8 hours but do not need to be the same for Condition A and Condition B. On the minor-street, the higher volume shall not be required to be on the same approach during each of these 8 hours.

If the posted or statutory speed limit or the 85th-percentile speed on the major street exceeds 40 mph, or if the intersection lies within an isolated community having a population of less than 10,000, the traffic volumes in the 56% columns in Table 4C-1 may be used in place of the 80% columns.

The specific volumes used in this study for Conditions A and B were taken from the MUTCD Table 4C-1 considering two lanes for moving traffic on the major approach and two lanes on the minor approach operating at the speed limit under the future conditions. The MUTCD Table 4C-1 with the 100% and 80% thresholds highlighted is shown in Figure 5.

Traffic data for eight-hours of vehicular volumes at the study intersection were used for the evaluation of Warrant 1. Table 13 summarizes the eight-hour vehicular volumes and thresholds to meet Warrant 1 under the future conditions (2022).

Table 4C-1. Warrant 1, Eight-Hour Vehicle Volume

Condition A—Minimum Vehicular Volume

Number of lanes for moving traffic on each approach		Vehicles per hour on major street (total of both approaches)				Vehicles per hour on higher-volume minor-street approach (one direction only)			
Major Street	Minor Street	100%[a]	80%[b]	70%[c]	56%[d]	100%[a]	80%[b]	70%[c]	56%[d]
1	1	500	400	350	280	150	120	105	84
2 or more	1	600	480	420	336	150	120	105	84
2 or more	2 or more	600	480	420	336	200	160	140	112
1	2 or more	500	400	350	280	200	160	140	112

Condition B—Interruption of Continuous Traffic

Number of lanes for moving traffic on each approach		Vehicles per hour on major street (total of both approaches)				Vehicles per hour on higher-volume minor-street approach (one direction only)			
Major Street	Minor Street	100%[a]	80%[b]	70%[c]	56%[d]	100%[a]	80%[b]	70%[c]	56%[d]
1	1	750	600	525	420	75	60	53	42
2 or more	1	900	720	630	504	75	60	53	42
2 or more	2 or more	900	720	630	504	100	80	70	56
1	2 or more	750	600	525	420	100	80	70	56

[a] Basic minimum hourly volume
[b] Used for combination of Conditions A and B after adequate trial of other remedial measures
[c] May be used when the major-street speed exceeds 40 mph or in an isolated community with a population of less than 10,000
[d] May be used for combination of Conditions A and B after adequate trial of other remedial measures when the major-street speed exceeds 40 mph or in an isolated community with a population of less than 10,000

Figure 5: MUTCD Table 4C-1

Table 13: Future Conditions (2022) – Volumes and Thresholds

		Either Condition A or B Must be Satisfied		Both Conditions A and B Must be Satisfied	
Hour	Volumes (Major Road/ Minor Road)	100% Threshold Conditions A	100% Threshold Conditions B	80% Threshold Conditions A	80% Threshold Conditions B
	VPH	600 / 200	900 / 100	480 / 160	720 / 80
7:00 AM to 8:00 AM	2,981 / 409	Y / Y	Y / Y	Y / Y	Y / Y
8:00 AM to 9:00 AM	3,042 / 221	Y / Y	Y / Y	Y / Y	Y / Y
9:00 AM to 10:00 AM	2,488 / 173	Y / N	Y / Y	Y / Y	Y / Y
10:00 AM to 11:00 AM	2,499 / 218	Y / Y	Y / Y	Y / Y	Y / Y
11:00 AM to 12:00 PM	2,675 / 305	Y / Y	Y / Y	Y / Y	Y / Y
12:00 PM to 1:00 PM	2,233 / 207	Y / Y	Y / Y	Y / Y	Y / Y
1:00 PM to 2:00 PM	2,157 / 198	Y / N	Y / Y	Y / Y	Y / Y
2:00 PM to 3:00 PM	2,341 / 273	Y / Y	Y / Y	Y / Y	Y / Y
3:00 PM to 4:00 PM	2,735 / 295	Y / Y	Y / Y	Y / Y	Y / Y
4:00 PM to 5:00 PM	3,210 / 301	Y / Y	Y / Y	Y / Y	Y / Y
5:00 PM to 6:00 PM	3,294 / 336	Y / Y	Y / Y	Y / Y	Y / Y
6:00 PM to 7:00 PM	3,000 / 251	Y / Y	Y / Y	Y / Y	Y / Y

Y- Threshold is Satisfied

N- Threshold is Not Satisfied

Based on the data summarized, the future hourly volumes on the major-street and higher-volume minor street meet the minimum requirements for the 100% Thresholds under Condition B. Therefore, Warrant 1 is satisfied in the future conditions (2022).

Warrant 1 is satisfied in the Future Conditions (2022).

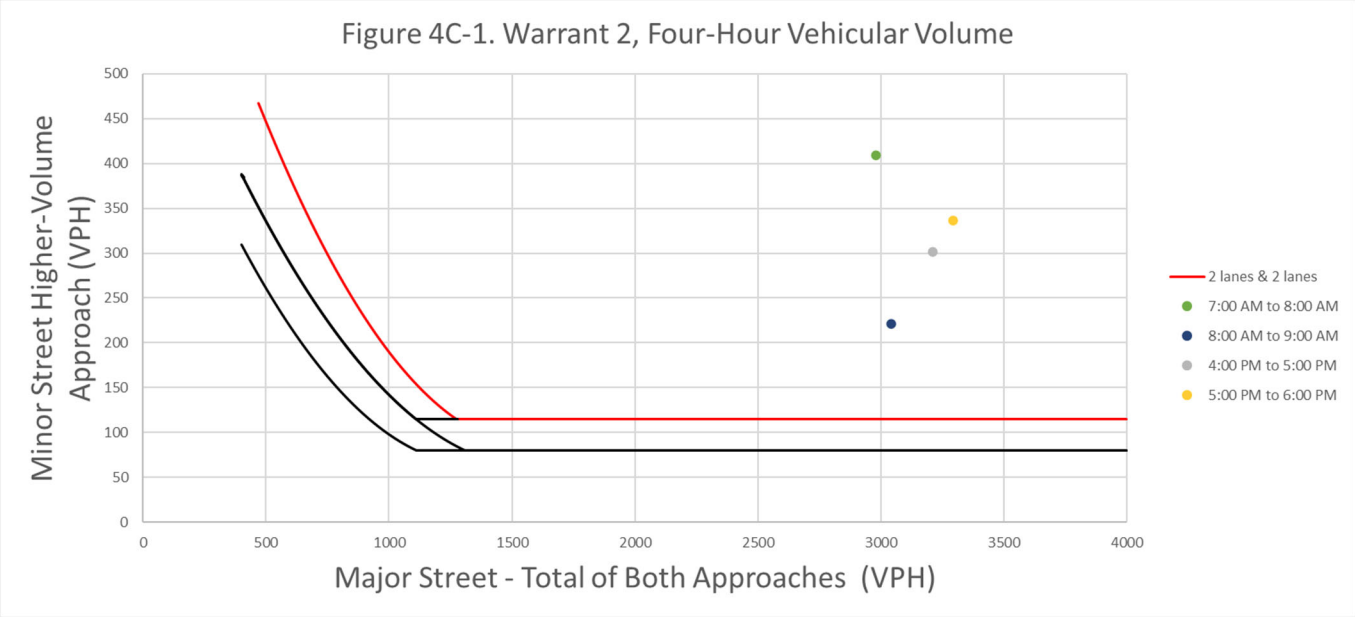
Warrant 2: Four-Hour Vehicular Volume

Warrant 2 is satisfied when the plotted points representing the vehicles per hour on the major street (total of both approaches) and the corresponding vehicles per hour on the higher-volume minor-street approach (one direction only) for each of any 4 hours of an average day all fall above Figure 4C-1 shown in the MUTCD for the existing combination of approach lanes. In the case where the 85th percentile speed on the major street is greater than 40 mph, the 70% factor applies, and Figure 4C-2 can be used. This analysis assumed two lanes for moving traffic on the major approach and two lanes on the minor approach operating at the speed limit using the 100% threshold. The future traffic volumes for the four hours of an average day that were used to evaluate Warrant 2 are shown in Table 14.

Table 14: Future Conditions (2022) – Four-Hourly Traffic Volumes at Study Intersection

Hour	Future Volume EB/WB	Future Volume NB	Future Volume SB	Warrant Satisfied?
7:00 AM to 8:00 AM	2,981	23	409	Y
8:00 AM to 9:00 AM	3,042	21	221	Y
4:00 PM to 5:00 PM	3,210	16	301	Y
5:00 PM to 6:00 PM	3,294	18	336	Y

Typically, four hourly traffic volumes of an average day are plotted on the MUTCD Figure 4C-1 or Figure 4C-2 and, if all points are above the appropriate curve, the warrant criterion is met. As noted in the MUTCD Figure 4C-1, 115 vehicles per hour apply as the lower threshold volume for a minor-street approach with two lanes.



As shown in Figure 4C-1, the four hours fall above the warrant threshold. Hence, Warrant 2 is satisfied in the future conditions.

Warrant 2 is satisfied in Future Conditions (2022).

Warrant 3: Peak Hour

Warrant 3 "shall be applied only in unusual cases, such as office complexes, manufacturing plants, industrial complexes, or high-occupancy vehicle facilities that attract or discharge large numbers of vehicles over a short time." The need for a traffic control signal shall be considered if the criteria in either of the following two categories are met:

A. Warrant 3 is satisfied when, for the same 1 hour of an average day, the total delay on one minor-street approach (one direction only) controlled by a STOP sign equals or exceeds: 4 vehicle-hours for a one-lane approach or 5 vehicle-hours for a two-lane approach; and the volume on the same minor-street approach equals or exceeds 100 vehicles per hour for one moving lane of traffic or 150 vehicles per hour for two moving lanes; and the total entering volume serviced during the hour equals or exceeds 650 vehicles per hour for intersections with three approaches or 800 vehicles per hour for intersections with four or more approaches; or

B. Warrant 3 is satisfied when the plotted points representing the vehicles per hour on the major street (total of both approaches) and the corresponding vehicles per hour on the higher-volume minor-street approach (one direction only) for 1 hour of an average day falls above the applicable figure shown in the MUTCD for the existing and future combination of approach lanes.

The highest peak hour volumes on the minor and major approaches at the intersection occurred from 4:30 PM to 5:30 PM. However, the minor approaches at the intersection cannot be considered an unusual case which discharge or attract large numbers of vehicles in a short time. Hence, Warrant 3 is not applicable in this case in the future conditions (2022).

Warrant 3 is not applicable in the Future Conditions (2022).

Warrant 4: Pedestrian Volume

Warrant 4 is satisfied when the pedestrian volume crossing the major street at the study intersection meets one of the following criteria:

A. For each of any 4 hours of an average day, the plotted points representing the vehicles per hour on the major street (total of both approaches) and the corresponding pedestrians per hour crossing the major street (total of all crossings) all fall above the curve in Figure 4C-5; or

B. For 1 hour (any four consecutive 15-minute periods) of an average day, the plotted point representing the vehicles per hour on the major street (total of both approaches) and the corresponding pedestrians per hour crossing the major street (total of all crossings) fall above the curve in Figure 4C-7.

If the posted or statutory speed limit or the 85th-percentile speed on the major street exceeds 35mph, or if the intersection lies within the built-up area of an isolated community having a population of less than 10,000, Figure 4C-6 may be used in place of Figure 4C-5 to evaluate Criterion A and Figure 4C-8 may be used in place of Figure 4C-7 to evaluate Criterion B.

There are currently no crosswalks at the study intersection. The proposed signal will include a safe pedestrian crossing which is anticipated to serve the mixed-use development, provide a safe path for school children, and further connect the area to the metro station. However, due to the unknown number of future pedestrians, Warrant 4 is not pursued at this time.

Warrant 4 is not pursued in the Future Conditions (2022).

Warrant 5: School Crossing

Warrant 5 is applicable where school children crossing the major street are the principal reason for a traffic control signal installation. A minimum of 20 school children are required to cross the road during the highest crossing hour to satisfy the warrant. A traffic signal will include pedestrian crossing phases which will provide a safe option for school children to cross the road.

A crosswalk is proposed at the subject intersection to provide a safe pedestrian crossing to school children at Mary Ellen Henderson Middle School and George Mason High School which are located adjacent to the study intersection. The crosswalk is anticipated to serve a number of school children; however, due to the unknow number of future pedestrians Warrant 5 is not pursued at this time.

Warrant 5 is not pursued in the Future Conditions (2022).

Warrant 6: Coordinated Signal System

Warrant 6 is satisfied when, “on a two-way street, adjacent traffic control signals do not provide the necessary degree of platooning and the proposed and adjacent traffic control signals will collectively provide a progressive operation.” In addition, this warrant should not be applied where the resultant spacing of traffic control signals would be less than 1,000 feet.

The closest signal on Leesburg Pike is approximately 485 feet to the east at Haycock Road. Therefore, Warrant 6 is not applicable**.**

Warrant 6 is not applicable in the Future Conditions (2022).

Warrant 7: Crash Experience

Warrant 7 is applicable where the severity and frequency of crashes are the principal reasons to consider the installation of a traffic control signal. This warrant is valid when **all** of the following criteria are met:

A. “Adequate trial of alternatives with satisfactory observance and enforcement has failed to reduce the crash frequency; and

B. Five or more reported crashes, of types susceptible to correction by a traffic control signal, have occurred within a 12-month period. Each crash should involve personal injury or property damage apparently exceeding the applicable requirements for a reportable crash; and

C. For each of any 8 hours of an average day, the vehicles per hour given in both of the 80% columns of Condition A in the MUTCD Table 4C-1, or the vehicles per hour in both of the 80% columns of Condition B in the MUTCD Table 4C-1 exists on the major-street and the higher-volume minor-street approach, respectively, to the intersection, or the volume of the pedestrian traffic is not less than 80% of the requirements specified in Warrant 4. These major-street and minor-street volumes shall be for the same 8 hours. On the minor street, the higher volume shall not be required to be on the same approach during each of the 8 hours."

As previously explained, the intersection experienced a total of 34 crashes between 2015 and 2019. Of the 34 crashes approximately 38% were reported as "Injury Collision (IC)" while 62% were "Property Damage Only (PDO)". No fatal crashes were recorded at the intersection during the five-year period. A majority of the crashes (85%) were classified as angled-collisions resulting primarily due to vehicles attempting to turn left onto Chestnut Street from Leesburg Pike.

Additionally, VDOT installed traffic signs indicating No Left Turn and No U-Turn for Rte. 7 making left onto Chestnut as well as "No Left Turn" between 4:00 PM – 7:00 PM for Chestnut Street making a left out to Rte. 7. This decision was partially based on severity and quantity of accidents for motorists making left from Rte. 7 onto Chestnut. These signs have been recently installed.

Considering VDOT's concern with the number of crashes at the intersection, if approved, the provision of a signal in addition to the proposed configuration of the side streets, is anticipated to reduce the nature and severity of the crashes at the location. Therefore, this warrant is satisfied in the future conditions.

Warrant 7 is satisfied in the Future Conditions (2022).

Warrant 8: Roadway Network

Warrant 8 is applied when a traffic control signal is considered for the intersection of two or more major routes and if the intersection meets one or both of the following criteria:

A. The intersection has a total existing, or immediately projected, entering volume of at least 1,000 vehicles per hour during the peak hour of a typical weekday and has 5-year projected traffic volumes, based on an engineering study, that meet one or more of Warrants 1, 2, and 3 during an average weekday; or

B. The intersection has a total existing or immediately projected entering volume of at least 1,000 vehicles per hour for each of any 5 hours of a non-normal business day (Saturday or Sunday).

Leesburg Pike (major street) is classified as a major collector and Chestnut Street and Commons Drive (minor streets) are local roads/commercial entrances. Hence, the intersection does not qualify as an intersection of two major routes. Therefore, this warrant is not satisfied in the future conditions.

Warrant 8 is not satisfied in the Future Conditions (2022).

Warrant 9: Intersection Near a Grade Crossing

Warrant 9 is applied after adequate consideration has been given to other alternative or after a trial of an alternative has failed to alleviate the safety concerns associated with the grade crossing (railroad crossing).

A Grade Crossing is not in close proximity to the study intersection; therefore, this criterion is not satisfied.

Warrant 9 is not satisfied in the Future Conditions (2022).

Signal Justification Analysis

As stated in the VDOT Requirements for Signal Justification Reports (SJRs) For New and Reconstructed Signals, dated June 5, 2017:

> "The SJR shall justify why a signal is not merely warranted but also necessary, as per the standard statements in Section 4C.01 of the VA Supplement, "The satisfaction of a signal warrant or warrants shall not in itself require the installation of a traffic control signal … In order for a traffic signal to be justified, evidence of the need for right of way assignment beyond that which could be provided by a stop sign or other unsignalized intersection configuration shall be demonstrated. Examples of such a need include: excessive delay, congestion, unfavorable approach conditions, or surrounding conditions that cause driver confusion"

As shown in the earlier section, a traffic signal is warranted at the location. However, the following alternative configurations were considered and analyzed, consistent with the guidelines for Signal Justification Reports:

- Conventional Signal
- Roundabout
- Two-Way Stop Control

The VDOT Junction Screening Tool (VJuST) was utilized to consider and screen innovative intersection configurations that address mobility and safety issues. Table 15 shows the possible configurations from VJuST.

Table 15: VJuST – Possible Configurations

VDOT Junction Screening Tool				
Possible Configurations				
Indicate with a "Y" or "N" if each intersection or interchange configuration should or should not be considered. Use the information links for guidance. Then, click the "Show/Hide Configurations button" to hide the worksheets for the configurations that will not be considered.				
#	**Intersections**	**Information**	**Consider?**	**Justification**
	Signalized Intersections			
1	Conventional	-	Y	
2	Bowtie	Link	N	Right-of-way restrictions identified
3	Center Turn Overpass	Link	N	Right-of-way restrictions identified
4	Continuous Green-T	Link	N	Unable to accommodate traffic patterns
5	Echelon	Link	N	Right-of-way restrictions identified
6	Full Displaced Left Turn	Link	N	Right-of-way restrictions identified
7	Median U-Turn	Link	N	Right-of-way restrictions identified
8	Partial Displaced Left Turn	Link	N	Right-of-way restrictions identified
9	Partial Median U-Turn	Link	N	Right-of-way restrictions identified
10	Quadrant Roadway N-E	Link	N	Right-of-way restrictions identified
11	Quadrant Roadway N-W	Link	N	Right-of-way restrictions identified
12	Quadrant Roadway S-E	Link	N	Right-of-way restrictions identified
13	Quadrant Roadway S-W	Link	N	Right-of-way restrictions identified
14	Restricted Crossing U-Turn	Link	N	Right-of-way restrictions identified
15	Single Loop	Link	N	Right-of-way restrictions identified
16	Split Intersection	Link	N	Right-of-way restrictions identified
	Unsignalized Intersections			
17	50 Mini Roundabout	Link	N	Unable to accommodate magnitude of traffic volumes
18	75 Mini Roundabout	Link	N	Unable to accommodate magnitude of traffic volumes
19	Roundabout	Link	Y	
20	Two-Way Stop Control	-	Y	
#	**Interchanges**	**Information**	**Consider?**	**Justification**
21	Traditional Diamond	Link	N	Right-of-way restrictions identified
22	Contraflow Left	Link	N	Right-of-way restrictions identified
23	Displaced Left Turn	Link	N	Right-of-way restrictions identified
24	Diverging Diamond	Link	N	Right-of-way restrictions identified
25	Double Roundabout	Link	N	Right-of-way restrictions identified
26	Michigan Urban Diamond	Link	N	Right-of-way restrictions identified
27	Partial Cloverleaf	Link	N	Right-of-way restrictions identified
28	Single Point	Link	N	Right-of-way restrictions identified
29	Single Roundabout	Link	N	Right-of-way restrictions identified

Conventional Signalized Intersection

An Intersection Capacity Analysis was performed using *Synchro 10* based on the Highway Capacity Manual (HCM 2000) data and methodology. The analysis was performed for the AM and PM peak hours under the future (2022) conditions. Figure 6 shows the peak hour volumes and lane configuration at the intersection. A peak hour factor of 0.92 was used for the study intersection unless the existing peak hour factor was higher. The results of the intersection capacity analysis are shown in Table 16. The detailed analysis worksheets are included in Appendix E.

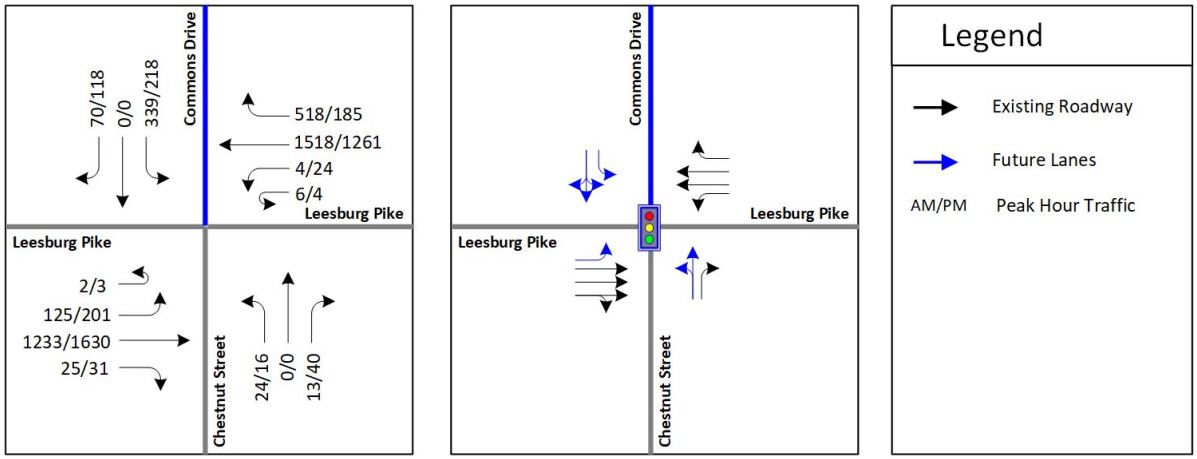


Figure 6: Future Traffic Volumes and Lane Configuration (2022)

Table 16: Signalized Conditions Intersection Capacity Analysis – Future Conditions (2022)

Intersection (Movement)	Storage Length	Total Future 2022							
		AM Peak				PM Peak			
		LOS	Delay (sec/veh)	Queue 50th (ft)	Queue 95th (ft)	LOS	Delay (sec/veh)	Queue 50th (ft)	Queue 95th (ft)
3 Chestnut St/Commons Drive (N/S) & Leesburg Pike (E/W)									
Overall Intersection (Signalized)		**D**	**45.7**			**C**	**31.1**		
Eastbound Left	415	E	69.4	~133	#254	E	71.8	145	#293
Eastbound Thru/Right	415	B	17.3	173	300	C	20.6	249	435
Westbound Left	75	E	66.3	10	m12	E	55.2	28	m41
Westbound Thru	350	E	72.8	~668	m#1155	C	35.0	272	#870
Westbound Right	350	C	27.9	173	m193	C	22.5	0	m52
Northbound Thru/Left	50	D	53.7	17	#47	E	58.9	11	35
Nortbound Right	50	D	48.8	0	0	D	45.5	0	0
Southbound Left	115	D	47.9	153	221	D	46.7	128	185
Southbound Left/Thru/Right	115	D	35.7	0	47	D	37.1	0	18

95th percentile volume exceeds capacity, queue may be longer
m Volume for 95th percentile queue is metered by upstream signal
~ Volume exceeds capacity, queue is theoretically infinite
Err = Volume greatly exceeds capacity

As shown, the conventional signal would result in acceptable overall levels of service during both peak hours. Some movements perform at unacceptable LOS and experience extended queues; however, the signal provides a regional benefit by providing an access point on Leesburg Pike that allows for left turns. This reduces the number of vehicles turning at the Leesburg Pike and Haycock Drive intersection which helps improve regional operations.

Roundabout

A preliminary roundabout assessment was conducted for the study intersection based on the VDOT Roundabout Screening Guidelines. The results of the roundabout evaluation per the screening criteria are:

- The ADT at the study intersection is approximately 37,310 vpd and the left turn percentage is approximately 12%. As shown in Figure 7, the intersection would likely operate as a double-lane roundabout.

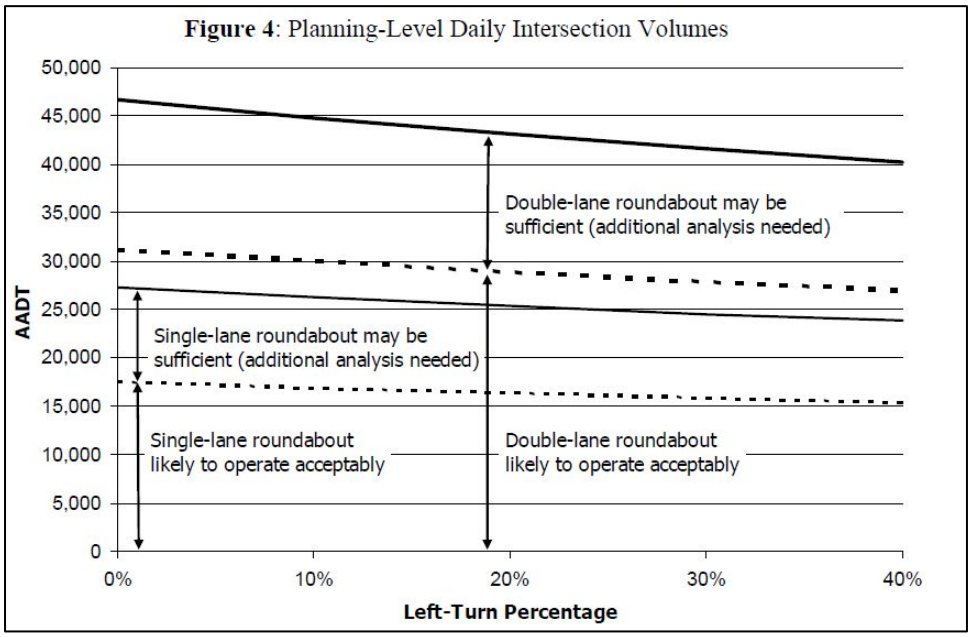


Figure 7: NCHRP Report 62 Exhibit 3-12 (VDOT Roundabout Screening Guidelines)

- The range of the Inscribed Circle Diameter for a two-lane roundabout is 150 feet to 300 feet as shown in Figure 8.

Figure 2: Roundabout Category Comparison

Design Element	Mini-Roundabout	Single-Lane Roundabout	Multilane Roundabout
Desirable maximum entry design speed	15 to 20 mph (25 to 30 km/h)	20 to 25 mph (30 to 40 km/h)	25 to 30 mph (40 to 50 km/h)
Maximum number of entering lanes per approach	1	1	2-3
Typical inscribed circle diameter	45 to 90 ft (13 to 27 m)	90 to 180 ft (27 to 55 m)	150 to 300 ft (46 to 91 m)
Central island treatment	Fully traversable	Raised (may have traversable apron)	Raised (may have traversable apron)
Typical daily service volumes on 4-leg roundabout below which may be expected to operate without requiring a detailed capacity analysis (veh/day)*	Up to approximately 15,000	Up to approximately 25,000	Up to approximately 45,000 for two-lane roundabout

*Operational analysis needed to verify upper limit for specific applications or for roundabouts with more than two lanes or four legs.

Figure 8: Minimum Inscribed Diameters of roundabouts (*Virginia DOT, Roundabout Design Guidance)*

- A preliminary roundabout analysis was performed using SIDRA Intersection 7.0 Plus software, utilizing the SIDRA Standard Model and an environmental factor of 1.05 for future (2022) conditions. The results of the analysis are summarized in Table 17. The detailed analysis worksheets are included in Appendix F.

Table 17: Roundabout Analysis – Future Conditions (2022)

Intersection (Movement)	Total Future 2022							
	AM Peak				PM Peak			
	LOS	Delay (sec/veh)	95th Queue	v/c Ratio	LOS	Delay (sec/veh)	95th Queue	v/c Ratio
3 Chestnut St/Commons Drive (N/S) & Leesburg Pike (E/W)								
Overall Intersection (Roundabout)	**E**	**35.5**			**E**	**40.2**		
Eastbound Approach	D	31.7	399	0.888	F	66.3	1175	1.059
Westbound Approach	D	31.7	618	0.935	B	14.4	206	0.716
Northbound Approach	B	11.3	9	0.065	B	14.8	19	0.133
Southbound Approach	F	69.4	375	0.993	B	12.9	60	0.409

- A roundabout is not recommended at this intersection due to the following reasons:
    - The overall intersection operates at an unacceptable LOS.
    - A roundabout can be a good alternative when the volumes on the major and minor streets are approximately even. Leesburg Pike has an ADT of approximately 30,390 vpd and the minor streets have a combined ADT of approximately 3,920 vpd. Hence, there is a significant difference between the traffic volumes on the major and minor streets.
    - A roundabout is a good alternative when there is a high percentage of turning traffic. As shown in the 'Roundabout Screening Analysis', the percentage of left turns at the intersection is only approximately 12%.
    - Right-of-way constraints would make a roundabout at this location unsuitable for a large roundabout.
    - Due to all the above reasons, a roundabout is not a desirable alternative at the study intersection.

Two-Way Stop Controlled Intersection

The results of the intersection capacity analyses with a stop-controlled intersection are summarized in Table 18. The detailed analysis worksheets are included in Appendix G.

Table 18: Unsignalized Conditions Intersection Capacity Analysis – Future Conditions (2022)

Intersection (Movement)	Storage Length	Total Future 2022							
		AM Peak				PM Peak			
		LOS	Delay (sec/veh)	Queue 50th (ft)	Queue 95th (ft)	LOS	Delay (sec/veh)	Queue 50th (ft)	Queue 95th (ft)
3 Chestnut St/Commons Drive (N/S) & Leesburg Pike (E/W)									
Overall Intersection (Unsignalized)									
Eastbound Left	175	E	35.4	-	71	C	18.3	-	56
Westbound Left	75	B	11.9	-	2	C	15.9	-	7
Northbound Thru/Left	50	F	1342.8	-	105	F	Err	-	Err
Nortbound Right	50	B	11.5	-	2	B	13.8	-	8
Southbound Left	115	F	Err	-	Err	F	Err	-	Err
Southbound Left/Thru/Right	115	F	Err	-	Err	F	Err	-	Err

Err = Volume greatly exceeds capacity

As shown in the table, the intersection operates at unacceptable levels of service during the AM peak hour and the side streets operate at Err during both peak hours.

Summary – Justification

A summary of the evaluation of the justification criteria from the VJuST for the future (2022) conditions is presented in Table 19 and Table 20. The VJuST Input and Results worksheets are included in Appendix H.

Table 19: VJuST Intersection Results (AM Peak Hour)

Intersection Results					
		Congestion	Pedestrian	Safety	Notes
Type	Dir	Maximum V/C	Accommodation Compared to Conventional	Weighted Total Conflict Points	
Conventional	-	0.69		48	
Roundabout	-	1.10		8	
Two-Way Stop Control	-	N/A*		48	

Table 20: VJuST Intersection Results (PM Peak Hour)

Intersection Results					
		Congestion	Pedestrian	Safety	Notes
Type	Dir	Maximum V/C	Accommodation Compared to Conventional	Weighted Total Conflict Points	
Conventional	-	0.62		48	
Roundabout	-	0.97		8	
Two-Way Stop Control	-	28.73		48	

As shown in the table, the conventional traffic signal operates with the lowest v/c ratio, and this configuration is recommended at the intersection.

Intersection Discussion

The intersection of Leesburg Pike and Chestnut Street/Commons Drive has been considered with multiple design options. Extensive coordination between Fairfax County, VDOT, and the City of Falls Church has been conducted to determine the current design.

A signal is desired at this location for multiple reasons, but the main benefits are listed below:

- The signalized intersection safety allows full movement access on Commons Drive. Allowing a left in and a left out relieves pressure at Leesburg Pike and Haycock Road by allowing for a second location for left turns for vehicles coming to and from the north, including to and from the metro station.
- The intersection currently experiences a large number of crashes. A signal allows the vehicles entering and exiting the sides streets a protected phase which will enhance safety.
- The realignment of Chestnut Street advances Fairfax County's goals of reducing cut through traffic in the neighborhoods to the south.
- A pedestrian crossing is provided across Leesburg Pike which is in line with the City of Falls Church's multimodal goals and provides a safe route to school.
- The proposed intersection removes a RIRO on Leesburg Pike which reduces the number of curb cuts on Leesburg Pike.

As described, there are multiple benefits from the proposed intersection improvement.

Conclusions

The purpose of this analysis was to determine if the installation of a traffic control signal would be justified at the intersection of Leesburg Pike and Commons Drive/Chestnut Street under the future (2022) conditions. A summary of the evaluation of the warrant criteria from the Manual on Uniform Traffic Control Devices (MUTCD), 2009 Edition is presented in Table 21.

Table 21: Summary of Warrant Analysis

Warrant No.	Warrant Description	Future Conditions (2022)
1	Eight-Hour Vehicular Volume	Satisfied
2	Four-Hour Vehicular Volume	Satisfied
3	Peak Hour	Not Applicable
4	Pedestrian Volume	Not Pursued
5	School Crossing	Not Pursued
6	Coordinated Signal System	Not Applicable
7	Crash Experience	Satisfied
8	Roadway Network	Not Satisfied
9	Intersection Near a Grade Crossing	Not Satisfied

According to the MUTCD, at least one warrant needs to be satisfied to allow for the installation of a traffic control signal. Based on the results presented in Table 21, the installation of a traffic control signal is **warranted under the future (2022) conditions**.

The VDOT Junction Screening Tool (VJuST) was utilized to consider and screen alternative intersection configurations that address mobility and safety issues. The possible configurations for the study intersection were a conventional signal, a roundabout, and a two-way stop control. The results of the analysis showed that the **conventional signal control operates with the lowest v/c ratio, and this configuration is recommended at the intersection.**

APPENDIX

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APPENDIX A

Turning Movement Counts

2/14/2019

ALL VEHICLES		Direction:	Major					Major					Minor					Minor				
		Roadway:	EB					WB					NB					SB				
			Leesburg Pike					Leesburg Pike					Chestnut Street					Commons Drive				
		Movement:	Left	Thru	Right	U	Ped	Left	Thru	Right	U	Ped	Left	Thru	Right	U	Ped	Left	Thru	Right	U	Ped
6:00 AM	to	6:15 AM																				
6:15 AM	to	6:30 AM																				
6:30 AM	to	6:45 AM																				
6:45 AM	to	7:00 AM																				
7:00 AM	to	7:15 AM	0	255	1	0	0	2	211	1	0	1	4	0	0	0	0	0	0	0	0	0
7:15 AM	to	7:30 AM	0	337	2	0	0	1	243	4	1	0	2	0	2	0	0	0	0	0	0	0
7:30 AM	to	7:45 AM	2	369	1	0	0	0	256	19	1	0	5	0	3	0	0	0	0	2	0	0
7:45 AM	to	8:00 AM	0	345	1	0	0	2	359	36	0	0	9	0	3	0	0	0	0	2	0	4
8:00 AM	to	8:15 AM	1	335	1	1	0	1	364	20	0	0	6	0	0	0	0	0	0	4	0	1
8:15 AM	to	8:30 AM	0	331	5	0	0	0	371	1	4	0	4	0	5	0	0	0	0	0	0	0
8:30 AM	to	8:45 AM	0	376	3	0	0	1	329	2	2	0	2	0	5	0	0	0	0	0	0	0
8:45 AM	to	9:00 AM	0	283	4	0	0	1	318	1	0	0	5	0	1	0	0	0	0	0	0	0
9:00 AM	to	9:15 AM	1	271	2	0	1	2	279	3	3	0	0	0	2	0	0	0	0	0	0	1
9:15 AM	to	9:30 AM	0	321	2	0	0	3	259	3	0	0	1	0	4	0	0	0	0	0	0	0
9:30 AM	to	9:45 AM	4	280	4	1	0	2	259	3	2	0	5	0	1	0	2	0	0	2	0	1
9:45 AM	to	10:00 AM	0	315	5	1	0	1	220	2	0	0	2	0	1	0	1	0	0	0	0	0
10:00 AM	to	10:15 AM	1	338	1	3	0	2	187	3	4	0	1	0	4	0	0	0	0	2	0	0
10:15 AM	to	10:30 AM	1	314	5	0	0	0	229	0	1	0	1	0	4	0	0	0	0	1	0	0
10:30 AM	to	10:45 AM	0	277	3	1	0	5	268	2	0	1	6	0	3	0	1	0	0	1	0	1
10:45 AM	to	11:00 AM	0	260	3	1	0	4	230	1	1	0	2	0	4	0	0	0	0	1	0	0
11:00 AM	to	11:15 AM	2	269	4	1	0	2	227	3	1	0	4	0	2	0	0	0	0	1	0	0
11:15 AM	to	11:30 AM	0	273	5	0	0	2	247	3	0	0	6	0	6	0	1	0	0	0	0	0
11:30 AM	to	11:45 AM	2	289	8	1	0	2	241	3	2	0	9	0	3	0	0	0	0	4	0	0
11:45 AM	to	12:00 PM	0	286	5	1	0	8	241	5	2	0	6	0	9	0	0	0	0	0	0	0
12:00 PM	to	12:15 PM	1	264	4	1	0	4	242	0	2	0	8	0	5	0	0	0	0	1	0	0
12:15 PM	to	12:30 PM	1	292	5	1	0	8	261	0	2	0	4	0	4	0	0	0	0	2	0	3
12:30 PM	to	12:45 PM	1	257	4	0	0	3	263	2	5	0	3	0	8	0	0	0	0	2	0	0
12:45 PM	to	1:00 PM	1	247	9	0	0	8	278	1	1	0	12	0	4	0	0	0	0	3	0	0
1:00 PM	to	1:15 PM	2	261	6	2	0	4	244	0	2	0	10	0	4	0	0	0	0	2	0	0
1:15 PM	to	1:30 PM	1	261	9	2	0	6	267	0	1	0	5	0	7	0	0	0	0	1	0	0
1:30 PM	to	1:45 PM	1	259	8	2	0	3	238	1	3	0	4	0	6	0	1	0	0	3	0	1
1:45 PM	to	2:00 PM	1	264	5	0	0	3	247	2	1	0	5	0	4	0	2	0	0	0	0	1
2:00 PM	to	2:15 PM	1	264	7	0	0	8	260	2	1	0	5	0	2	0	0	0	0	3	0	0
2:15 PM	to	2:30 PM	1	272	3	2	0	4	279	1	4	0	9	0	7	0	0	0	0	4	0	0
2:30 PM	to	2:45 PM	0	284	6	1	0	3	245	5	1	0	6	0	1	0	1	0	0	1	0	0
2:45 PM	to	3:00 PM	0	301	5	0	0	3	273	12	0	0	6	0	8	0	0	0	0	0	0	0
3:00 PM	to	3:15 PM	0	337	6	0	0	6	283	9	1	0	0	0	10	0	0	0	0	8	0	6
3:15 PM	to	3:30 PM	0	341	9	0	0	3	272	3	0	0	3	0	6	1	0	0	0	6	0	4
3:30 PM	to	3:45 PM	0	355	8	1	1	7	282	2	2	0	3	0	10	0	1	0	0	2	0	0
3:45 PM	to	4:00 PM	0	398	5	3	0	5	298	3	1	0	3	0	2	0	0	0	0	2	0	1
4:00 PM	to	4:15 PM	0	399	2	1	0	4	305	2	0	0	0	0	7	0	0	0	0	0	0	0
4:15 PM	to	4:30 PM	0	427	4	2	1	6	304	1	1	0	1	0	7	0	1	0	0	1	0	1
4:30 PM	to	4:45 PM	1	473	7	0	1	8	322	1	2	0	0	0	6	0	0	0	0	1	0	0
4:45 PM	to	5:00 PM	0	514	6	1	0	6	323	5	0	0	0	0	6	0	2	0	0	1	0	1
5:00 PM	to	5:15 PM	0	481	4	1	0	3	290	8	2	0	0	0	9	0	1	0	0	1	0	1
5:15 PM	to	5:30 PM	0	483	7	0	0	7	271	18	0	0	1	0	6	0	1	0	0	0	0	3
5:30 PM	to	5:45 PM	0	489	7	1	0	8	278	24	0	0	1	0	5	0	0	0	0	0	0	0
5:45 PM	to	6:00 PM	0	487	11	0	3	5	329	9	2	0	1	0	4	0	0	0	0	0	0	0
6:00 PM	to	6:15 PM	0	406	6	0	0	11	278	8	1	0	1	0	5	0	0	0	0	0	0	2
6:15 PM	to	6:30 PM	1	445	8	1	0	7	292	15	2	0	0	0	3	0	2	0	0	0	0	3
6:30 PM	to	6:45 PM	0	497	9	1	2	3	285	11	1	0	1	0	7	0	0	0	0	0	0	1
6:45 PM	to	7:00 PM	0	433	10	1	0	4	241	14	1	0	3	0	4	0	0	0	0	3	0	0
7:00 PM	to	7:15 PM																				
7:15 PM	to	7:30 PM																				
7:30 PM	to	7:45 PM																				
7:45 PM	to	8:00 PM																				

8:00 PM	to	8:15 PM				
8:15 PM	to	8:30 PM				
8:30 PM	to	8:45 PM				
8:45 PM	to	9:00 PM				
9:00 PM	to	9:15 PM				
9:15 PM	to	9:30 PM				
9:30 PM	to	9:45 PM				
9:45 PM	to	10:00 PM				
10:00 PM	to	10:15 PM				

APPENDIX B

Crash Data

Crash Data for the Intersection of Leesburg Pike and Chestnut Street (2015 - 2019)

Document Number	Date	Crash Severity	Collsion Type	Pedestrain Injury	Persons Injured	Fatalities	Direction of Travel	Work Zone Related	Adverse Weather Conditions	Distracted Driver
151320024	4/27/2015	C.Nonvisible Injury	2. Angle	0	1	0	West;East	no	no	no
151420081	5/5/2015	PDO.Property Damage Only	2. Angle	0	0	0	South;East	no	no	no
151420043	5/13/2015	PDO.Property Damage Only	1. Rear End	0	0	0	East;East	no	no	no
151420138	5/15/2015	PDO.Property Damage Only	2. Angle	0	0	0	South;East	no	no	no
152310117	8/8/2015	PDO.Property Damage Only	2. Angle	0	0	0	North;East	no	no	no
152610152	9/3/2015	PDO.Property Damage Only	1. Rear End	0	0	0	East;East;East	no	no	no
160780136	3/5/2016	PDO.Property Damage Only	1. Rear End	0	0	0	East;East	no	no	yes
160960016	3/8/2016	PDO.Property Damage Only	2. Angle	0	0	0	South;East	no	no	no
161040044	4/4/2016	PDO.Property Damage Only	2. Angle	0	0	0	South;South	no	yes	no
161470278	5/6/2016	PDO.Property Damage Only	2. Angle	0	0	0	South;East	no	yes	no
162030023	6/23/2016	C.Nonvisible Injury	2. Angle	0	2	0	South;East;North	no	no	no
162040036	6/29/2016	B.Visible Injury	2. Angle	0	1	0	West;East	no	no	yes
162350024	7/29/2016	PDO.Property Damage Only	2. Angle	0	0	0	South;East	no	no	no
162580063	8/22/2016	C.Nonvisible Injury	2. Angle	0	1	0	South;East	no	no	no
162910003	8/23/2016	B.Visible Injury	2. Angle	0	4	0	South;East	no	no	no
162695187	9/23/2016	C.Nonvisible Injury	2. Angle	0	2	0	East;South	no	no	no
162990027	10/4/2016	C.Nonvisible Injury	2. Angle	0	1	0	South;West	no	no	no
170575110	2/20/2017	PDO.Property Damage Only	2. Angle	0	0	0	East;East	no	no	no
170845067	3/24/2017	PDO.Property Damage Only	2. Angle	0	0	0	South;East	no	no	no
170985239	4/7/2017	B.Visible Injury	2. Angle	0	3	0	South;East	no	no	no
172085332	7/17/2017	PDO.Property Damage Only	2. Angle	0	0	0	South;East	no	no	no
172085331	7/17/2017	B.Visible Injury	2. Angle	0	1	0	East;South	no	no	no
172835433	10/5/2017	PDO.Property Damage Only	2. Angle	0	0	0	East;West	no	no	no
172835423	10/10/2017	B.Visible Injury	2. Angle	0	1	0	South;East;East	no	no	no
172965538	10/19/2017	PDO.Property Damage Only	2. Angle	0	0	0	South;East	no	no	no
181535157	5/23/2018	B.Visible Injury	2. Angle	0	1	0	East;East	no	no	no
182635412	7/31/2018	PDO.Property Damage Only	2. Angle	0	0	0	South;East	no	no	no
182675239	9/24/2018	PDO.Property Damage Only	2. Angle	0	0	0	North;East	no	yes	no
182825427	10/9/2018	B.Visible Injury	2. Angle	0	2	0	South;East	no	no	no
190175535	1/17/2019	PDO.Property Damage Only	2. Angle	0	0	0	South;East	no	no	no
190485251	2/7/2019	PDO.Property Damage Only	2. Angle	0	0	0	South;East	no	no	no
191165129	4/22/2019	PDO.Property Damage Only	16. Other	0	0	0	North;East	no	no	no
192195420	8/7/2019	PDO.Property Damage Only	2. Angle	0	0	0	South;East	no	no	no
192835373	10/10/2019	B.Visible Injury	16. Other	0	1	0	South;East	no	no	no

APPENDIX C

Hourly Factors for Existing Traffic on Leesburg Pike

Hourly Factors for Existing Traffic on Leesburg Pike

Hourly Factor Calculation

Time Period	Leesburg Pike Volumes	Hourly Factor
7:00 AM 8:00 AM	2449	1.00
8:00 AM 9:00 AM	2755	1.12
9:00 AM 10:00 AM	2248	0.92
10:00 AM 11:00 AM	2145	0.88
11:00 AM 12:00 PM	2135	0.87
12:00 PM 1:00 PM	2168	0.67
1:00 PM 2:00 PM	2106	0.65
2:00 PM 3:00 PM	2248	0.70
3:00 PM 4:00 PM	2640	0.82
4:00 PM 5:00 PM	3127	0.97
5:00 PM 6:00 PM	3225	1.00
6:00 PM 7:00 PM	2992	0.93

APPENDIX D

Trip Generation and Distribution – West Falls Church Economic Development

Hourly Factors from ITE Trip Generation Manual – 10th Edition

2022 Trip Generation

ITE Land Use Code		Trip Generation, 10th Ed.	Quantity	Weekday AM Peak Hour In	Weekday AM Peak Hour Out	Weekday AM Peak Hour Total	Weekday PM Peak Hour In	Weekday PM Peak Hour Out	Weekday PM Peak Hour Total	Weekday Daily Total	Saturday Peak Hour In	Saturday Peak Hour Out	Saturday Peak Hour Total	Saturday Daily Total
Existing Development*														
High School	530	High School	800 students	222	125	346	91	82	173	1,725	66	53	119	1,185
Middle School [A]	522	Middle/Jr High School	600 students	221	124	346	91	81	172	1,725	65	53	118	1,185
		Existing Trips		**443**	**249**	**692**	**182**	**163**	**345**	**3,450**	**131**	**106**	**237**	**2,370**
Proposed Development														
High School	530	High School	1,500 students	523	257	780	101	109	210	3,035	95	55	150	870
Middle School [A]	522	Middle/Jr High School	600 students	188	160	348	50	52	102	1,427	38	22	60	348
Office	710	General Office Building	130,000 sf	128	21	149	23	123	146	1,369	37	32	69	287
		Mode Split/TDM Reduction	*35%*	*-45*	*-7*	*-52*	*-8*	*-43*	*-51*	*-479*	*-13*	*-11*	*-24*	*-100*
		Internal Reduction	*(1) (3)*	*-6*	*-2*	*-8*	*-2*	*-7*	*-9*	*-88*	*-2*	*-2*	*-4*	*-18*
Retail[B]	820	Shopping Center	134,000 sf	136	83	219	324	351	675	7,336	406	374	780	10,686
		Pass-By Reduction	*25%/34%/26%*	*-34*	*-21*	*-55*	*-110*	*-119*	*-230*	*-1,834*	*-106*	*-97*	*-203*	*-2,672*
		Internal Reduction	*(2) (3)*	*-6*	*-5*	*-11*	*-12*	*-10*	*-22*	*-431*	*-19*	*-18*	*-37*	*-682*
Residential	220	Multifamily	530 du	54	179	233	164	97	261	3,966	270	269	539	6,904
		Mode Split/TDM Reduction	*35%*	*-19*	*-63*	*-82*	*-57*	*-34*	*-91*	*-1,388*	*-95*	*-94*	*-189*	*-2,416*
		Internal Reduction	*(1) (2)*	*-4*	*-5*	*-9*	*-12*	*-9*	*-21*	*-431*	*-19*	*-18*	*-37*	*-682*
Assisted Living	252	Senior Living	225 du	16	29	45	31	25	56	879	48	29	77	833
		Mode Split/TDM Reduction	*35%*	*-6*	*-10*	*-16*	*-11*	*-9*	*-20*	*-308*	*-17*	*-10*	*-27*	*-292*
Hotel	310	Hotel	150 rooms	41	29	70	44	42	86	1,267	60	48	108	1,148
		Mode Split/TDM Reduction	35%	*-14*	*-10*	*-25*	*-15*	*-15*	*-30*	*-443*	*-21*	*-17*	*-38*	*-402*
		Internal Reduction	*(4)*	*-3*	*-1*	*-4*	*-1*	*-3*	*-4*	*-82*	*-2*	*-2*	*-4*	*-19*
Day Care	565	Day Care Center	10,000 sf	58	52	110	52	59	111	476	11	6	17	62
		Pass-By/Diverted Reduction	*55%*	*-32*	*-29*	*-61*	*-29*	*-32*	*-61*	*-262*	*-6*	*-3*	*-9*	*-34*
		Proposed Development Site Trips		**975**	**657**	**1,633**	**532**	**577**	**1,108**	**14,009**	**666**	**562**	**1,228**	**13,821**
		New Site Trips (Proposed - Existing)		**532**	**408**	**941**	**350**	**414**	**763**	**10,559**	**535**	**456**	**991**	**11,451**

**Based on Existing Counts*

A) ITE does not have data for Saturday for Middle School, the Saturday rates for High School use were utilized

B) The pass by reduction for the shopping center is based on the ITE Trip Generation methodology, as provided in the 10th Edition Handbook. The average rate for shopping centers is 34% for the PM Peak and 26% for the Saturday Peak. For all other time periods, the default pass by rate is 25%.

C) The pass-by/diverted trip reduction for the day care is based on the ITE Trip Generation methodology, as provided in the 10th Edition Handbook

(1) residential / office - smaller of 5% of residential trips or 5% of office trips

(2) residential / retail - smaller of X% of residential trips or X% of retail trips; AM: X = 5%, PM: X = 10%, Sat: X= 10%, Daily: X = 15%

(3) office/ retail - smaller of 5% of office trips or 5% of retail trips

(4) hotel/office - use 15% of hotel/motel trips, unless the overall volume of the office traffic is more than the overall volume of hotel/motel traffic use in which case use the smaller of 10% of the hotel/motel traffic or the office traffic

Trip Distribution

The trip distribution is based on the West Falls Church Economic Development TIA.

Hourly Traffic Volumes – Office

Time Period	Office ITE's 24 Hour Percentage	Hourly Factor	Major EB Leesburg Pike			Major WB Leesburg Pike			Minor NB Chestnut Street			Minor SB Commons Drive		
			Left	Thru	Right	Left	Thru	Right	Left	Thru	Right	Left	Thru	Right
7:00 AM 8:00 AM	7.0%	1.00	31	0	0	0	30	0	0	0	0	2	0	0
8:00 AM 9:00 AM	8.8%	1.26	39	0	0	0	38	0	0	0	0	3	0	0
9:00 AM 10:00 AM	5.4%	0.77	24	0	0	0	23	0	0	0	0	2	0	0
10:00 AM 11:00 AM	5.9%	0.84	26	0	0	0	25	0	0	0	0	2	0	0
11:00 AM 12:00 PM	8.4%	1.20	37	0	0	0	36	0	0	0	0	2	0	0
12:00 PM 1:00 PM	10.4%	1.00	13	0	0	0	13	0	0	0	0	15	0	0
1:00 PM 2:00 PM	8.2%	0.79	10	0	0	0	10	0	0	0	0	12	0	0
2:00 PM 3:00 PM	7.5%	0.72	9	0	0	0	9	0	0	0	0	11	0	0
3:00 PM 4:00 PM	7.4%	0.71	9	0	0	0	9	0	0	0	0	11	0	0
4:00 PM 5:00 PM	10.1%	0.97	13	0	0	0	13	0	0	0	0	15	0	0
5:00 PM 6:00 PM	10.4%	1.00	13	0	0	0	13	0	0	0	0	15	0	0
6:00 PM 7:00 PM	2.4%	0.23	3	0	0	0	3	0	0	0	0	3	0	0

Hourly factors for Office (ITE Code 710) are provided in the Trip Generation Manual, 10th Edition and were used.

Hourly Traffic Volumes – Residential

Time Period	Residential ITE's 24 Hour Percentage	Hourly Factor	Major EB Leesburg Pike			Major WB Leesburg Pike			Minor NB Chestnut Street			Minor SB Commons Drive		
			Left	Thru	Right	Left	Thru	Right	Left	Thru	Right	Left	Thru	Right
7:00 AM 8:00 AM	7.5%	1.00	16	0	0	0	19	5	0	0	0	14	0	34
8:00 AM 9:00 AM	6.2%	0.83	13	0	0	0	16	4	0	0	0	12	0	28
9:00 AM 10:00 AM	4.3%	0.57	9	0	0	0	11	3	0	0	0	8	0	19
10:00 AM 11:00 AM	3.7%	0.49	8	0	0	0	9	2	0	0	0	7	0	17
11:00 AM 12:00 PM	4.5%	0.60	10	0	0	0	11	3	0	0	0	8	0	20
12:00 PM 1:00 PM	4.7%	0.47	21	0	0	0	5	7	0	0	0	3	0	8
1:00 PM 2:00 PM	4.4%	0.44	20	0	0	0	4	6	0	0	0	3	0	8
2:00 PM 3:00 PM	5.4%	0.53	25	0	0	0	5	7	0	0	0	4	0	10
3:00 PM 4:00 PM	5.8%	0.57	26	0	0	0	6	8	0	0	0	4	0	10
4:00 PM 5:00 PM	8.3%	0.82	38	0	0	0	8	12	0	0	0	6	0	15
5:00 PM 6:00 PM	10.1%	1.00	46	0	0	0	10	14	0	0	0	7	0	18
6:00 PM 7:00 PM	7.9%	0.78	36	0	0	0	8	11	0	0	0	5	0	14

Hourly factors for Residential (ITE Code 221) are provided in the Trip Generation Manual, 10th Edition and were used.

Hourly Traffic Volumes – Retail

Time Period	Retail ITE's 24 Hour Percentage	Hourly Factor	Major EB Leesburg Pike			Major WB Leesburg Pike			Minor NB Chestnut Street			Minor SB Commons Drive		
			Left	Thru	Right	Left	Thru	Right	Left	Thru	Right	Left	Thru	Right
7:00 AM 8:00 AM	1.1%	1.00	49	-10	0	0	4	21	0	0	0	13	0	18
8:00 AM 9:00 AM	2.0%	1.82	89	-18	0	0	7	38	0	0	0	24	0	33
9:00 AM 10:00 AM	3.6%	3.27	160	-33	0	0	13	69	0	0	0	43	0	59
10:00 AM 11:00 AM	5.6%	5.09	249	-51	0	0	20	107	0	0	0	66	0	92
11:00 AM 12:00 PM	8.3%	7.55	370	-75	0	0	30	158	0	0	0	98	0	136
12:00 PM 1:00 PM	10.0%	1.08	123	-35	0	0	6	60	0	0	0	70	0	87
1:00 PM 2:00 PM	9.3%	1.00	114	-33	0	0	6	56	0	0	0	65	0	81
2:00 PM 3:00 PM	9.0%	0.97	110	-32	0	0	6	54	0	0	0	63	0	78
3:00 PM 4:00 PM	8.8%	0.95	108	-31	0	0	6	53	0	0	0	62	0	77
4:00 PM 5:00 PM	9.2%	0.99	113	-33	0	0	6	55	0	0	0	64	0	80
5:00 PM 6:00 PM	9.3%	1.00	114	-33	0	0	6	56	0	0	0	65	0	81
6:00 PM 7:00 PM	8.0%	0.86	98	-28	0	0	5	48	0	0	0	56	0	70

Hourly factors for Retail (ITE Code 820) are provided in the Trip Generation Manual, 10th Edition and were used.

Hourly Traffic Volumes – Hotel

Time Period	Hotel ITE's 24 Hour Percentage	Hourly Factor	Major EB Leesburg Pike			Major WB Leesburg Pike			Minor NB Chestnut Street			Minor SB Commons Drive		
			Left	Thru	Right	Left	Thru	Right	Left	Thru	Right	Left	Thru	Right
7:00 AM 8:00 AM	7.1%	1.00	10	0	0	0	9	0	0	0	0	4	0	0
8:00 AM 9:00 AM	5.5%	0.77	8	0	0	0	7	0	0	0	0	3	0	0
9:00 AM 10:00 AM	4.7%	0.66	7	0	0	0	6	0	0	0	0	3	0	0
10:00 AM 11:00 AM	4.1%	0.58	6	0	0	0	5	0	0	0	0	2	0	0
11:00 AM 12:00 PM	3.7%	0.52	5	0	0	0	5	0	0	0	0	2	0	0
12:00 PM 1:00 PM	3.9%	0.57	6	0	0	0	6	0	0	0	0	3	0	0
1:00 PM 2:00 PM	3.5%	0.51	6	0	0	0	6	0	0	0	0	3	0	0
2:00 PM 3:00 PM	4.5%	0.65	7	0	0	0	7	0	0	0	0	3	0	0
3:00 PM 4:00 PM	5.2%	0.75	8	0	0	0	8	0	0	0	0	4	0	0
4:00 PM 5:00 PM	5.7%	0.83	9	0	0	0	9	0	0	0	0	4	0	0
5:00 PM 6:00 PM	6.9%	1.00	11	0	0	0	11	0	0	0	0	5	0	0
6:00 PM 7:00 PM	7.0%	1.01	11	0	0	0	11	0	0	0	0	5	0	0

Hourly factors for Hotel (ITE Code 312) are provided in the Trip Generation Manual, 10th Edition and were used.

Hourly Traffic Volumes – School

Time Period	School ITE's 24 Hour Percentage	Hourly Factor	Major EB Leesburg Pike			Major WB Leesburg Pike			Minor NB Chestnut Street			Minor SB Commons Drive		
			Left	Thru	Right	Left	Thru	Right	Left	Thru	Right	Left	Thru	Right
7:00 AM 8:00 AM	18.2%	1.00	0	0	0	0	21	476	0	0	0	296	0	0
8:00 AM 9:00 AM	6.6%	0.36	0	0	0	0	8	173	0	0	0	107	0	0
9:00 AM 10:00 AM	2.1%	0.12	0	0	0	0	2	55	0	0	0	34	0	0
10:00 AM 11:00 AM	1.6%	0.09	0	0	0	0	2	42	0	0	0	26	0	0
11:00 AM 12:00 PM	2.1%	0.12	0	0	0	0	2	55	0	0	0	34	0	0
12:00 PM 1:00 PM	1.5%	0.13	0	0	0	0	1	13	0	0	0	15	0	0
1:00 PM 2:00 PM	1.9%	0.16	0	0	0	0	1	16	0	0	0	19	0	0
2:00 PM 3:00 PM	8.0%	0.68	0	0	0	0	3	69	0	0	0	78	0	0
3:00 PM 4:00 PM	10.6%	0.91	0	0	0	0	5	92	0	0	0	103	0	0
4:00 PM 5:00 PM	9.3%	0.79	0	0	0	0	4	80	0	0	0	91	0	0
5:00 PM 6:00 PM	11.7%	1.00	0	0	0	0	5	101	0	0	0	114	0	0
6:00 PM 7:00 PM	9.2%	0.79	0	0	0	0	4	79	0	0	0	90	0	0

Hourly factors for School (ITE Code 522) are provided in the Trip Generation Manual, 10th Edition and were used.

Hourly Traffic Volumes – Day Care

Time Period	Day Care ITE's 24 Hour Percentage	Hourly Factor	Major EB Leesburg Pike			Major WB Leesburg Pike			Minor NB Chestnut Street			Minor SB Commons Drive		
			Left	Thru	Right	Left	Thru	Right	Left	Thru	Right	Left	Thru	Right
7:00 AM 8:00 AM	25.2%	1.00	20	-10	0	0	-9	16	0	0	0	10	0	18
8:00 AM 9:00 AM	9.3%	0.37	7	-4	0	0	-3	6	0	0	0	4	0	7
9:00 AM 10:00 AM	4.1%	0.16	3	-2	0	0	-1	3	0	0	0	2	0	3
10:00 AM 11:00 AM	5.5%	0.22	4	-2	0	0	-2	3	0	0	0	2	0	4
11:00 AM 12:00 PM	3.8%	0.15	3	-2	0	0	-1	2	0	0	0	2	0	3
12:00 PM 1:00 PM	2.3%	0.20	4	-2	0	0	-2	3	0	0	0	2	0	4
1:00 PM 2:00 PM	2.5%	0.21	4	-2	0	0	-2	3	0	0	0	3	0	4
2:00 PM 3:00 PM	9.7%	0.83	15	-7	0	0	-7	12	0	0	0	10	0	16
3:00 PM 4:00 PM	9.1%	0.78	14	-7	0	0	-6	11	0	0	0	9	0	15
4:00 PM 5:00 PM	9.8%	0.84	15	-8	0	0	-7	12	0	0	0	10	0	16
5:00 PM 6:00 PM	11.7%	1.00	18	-9	0	0	-8	14	0	0	0	12	0	19
6:00 PM 7:00 PM	2.9%	0.25	4	-2	0	0	-2	3	0	0	0	3	0	5

Hourly factors for Day Care (ITE Code 565) are provided in the Trip Generation Manual, 10th Edition and were used.

APPENDIX E

Intersection Capacity Analysis Worksheets – Future Conditions Signalized (2022)

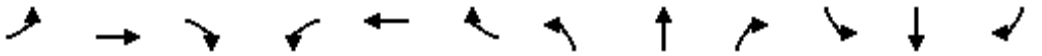
Lane Group	EBL	EBT	WBL	WBT	WBR	NBT	NBR	SBL	SBT
Lane Group Flow (vph)	130	1284	11	1650	563	26	14	228	216
v/c Ratio	1.88	0.42	0.11	0.92	0.58	0.25	0.04	0.72	0.43
Control Delay	476.6	15.0	48.5	29.8	9.3	54.0	0.3	52.5	6.0
Queue Delay	0.0	0.0	0.0	2.4	0.1	0.0	0.0	4.9	0.6
Total Delay	476.6	15.0	48.5	32.2	9.4	54.0	0.3	57.4	6.6
Queue Length 50th (ft)	~133	173	10	~668	173	17	0	153	0
Queue Length 95th (ft)	#254	300	m12	m#1155	m193	#47	0	221	47
Internal Link Dist (ft)		143		196		222			190
Turn Bay Length (ft)			180		215				
Base Capacity (vph)	69	3042	104	1799	979	102	312	432	593
Starvation Cap Reductn	0	0	0	76	38	0	0	0	0
Spillback Cap Reductn	0	63	0	0	0	0	0	141	150
Storage Cap Reductn	0	0	0	0	0	0	0	0	0
Reduced v/c Ratio	1.88	0.43	0.11	0.96	0.60	0.25	0.04	0.78	0.49

Intersection Summary

~ Volume exceeds capacity, queue is theoretically infinite.
Queue shown is maximum after two cycles.

95th percentile volume exceeds capacity, queue may be longer.
Queue shown is maximum after two cycles.

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



Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (vph)	127	1233	25	10	1518	518	24	0	13	339	0	70
Future Volume (vph)	127	1233	25	10	1518	518	24	0	13	339	0	70
Ideal Flow (vphpl)	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900
Grade (%)		-7%			-1%			0%			0%	
Total Lost time (s)	6.8	5.3		6.8	5.0	5.0		7.2	7.2	7.2	7.2	
Lane Util. Factor	1.00	0.91		1.00	0.95	1.00		1.00	1.00	0.95	0.95	
Frpb, ped/bikes	1.00	1.00		1.00	1.00	0.98		1.00	0.98	1.00	1.00	
Flpb, ped/bikes	1.00	1.00		1.00	1.00	1.00		1.00	1.00	1.00	1.00	
Frt	1.00	1.00		1.00	1.00	0.85		1.00	0.85	1.00	0.95	
Flt Protected	0.95	1.00		0.95	1.00	1.00		0.95	1.00	0.95	0.97	
Satd. Flow (prot)	1832	5146		1778	3489	1555		1770	1551	1681	1624	
Flt Permitted	0.95	1.00		0.95	1.00	1.00		0.95	1.00	0.95	0.97	
Satd. Flow (perm)	1832	5146		1778	3489	1555		1770	1551	1681	1624	
Peak-hour factor, PHF	0.98	0.98	0.98	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92
Adj. Flow (vph)	130	1258	26	11	1650	563	26	0	14	368	0	76
RTOR Reduction (vph)	0	1	0	0	0	206	0	0	13	0	175	0
Lane Group Flow (vph)	130	1283	0	11	1650	357	0	26	1	228	41	0
Confl. Peds. (#/hr)	3		1	3		1			1			
Confl. Bikes (#/hr)			3									
Heavy Vehicles (%)	2%	4%	2%	2%	4%	2%	2%	2%	2%	2%	2%	2%
Turn Type	Prot	NA		Prot	NA	Perm	Split	NA	Perm	Split	NA	
Protected Phases	1	6		5	2		8	8		4	4	
Permitted Phases						2			8			
Actuated Green, G (s)	9.4	53.7		1.2	45.8	45.8		3.8	3.8	19.8	19.8	
Effective Green, g (s)	9.4	53.7		1.2	45.8	45.8		3.8	3.8	19.8	19.8	
Actuated g/C Ratio	0.09	0.51		0.01	0.44	0.44		0.04	0.04	0.19	0.19	
Clearance Time (s)	6.8	5.3		6.8	5.0	5.0		7.2	7.2	7.2	7.2	
Vehicle Extension (s)	3.0	3.0		3.0	3.0	3.0		3.0	3.0	3.0	3.0	
Lane Grp Cap (vph)	164	2631		20	1521	678		64	56	316	306	
v/s Ratio Prot	c0.07	0.25		0.01	c0.47			c0.01		c0.14	0.03	
v/s Ratio Perm						0.23			0.00			
v/c Ratio	0.79	0.49		0.55	1.08	0.53		0.41	0.01	0.72	0.13	
Uniform Delay, d1	46.8	16.7		51.6	29.6	21.7		49.5	48.8	40.0	35.5	
Progression Factor	1.00	1.00		1.01	0.97	1.22		1.00	1.00	1.00	1.00	
Incremental Delay, d2	22.5	0.6		14.2	44.2	1.3		4.2	0.1	7.9	0.2	
Delay (s)	69.4	17.3		66.3	72.8	27.9		53.7	48.8	47.9	35.7	
Level of Service	E	B		E	E	C		D	D	D	D	
Approach Delay (s)		22.1			61.4			52.0			41.9	
Approach LOS		C			E			D			D	

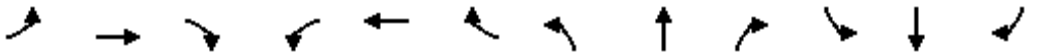
Intersection Summary			
HCM 2000 Control Delay	45.7	HCM 2000 Level of Service	D
HCM 2000 Volume to Capacity ratio	0.93		
Actuated Cycle Length (s)	105.0	Sum of lost time (s)	26.5
Intersection Capacity Utilization	83.0%	ICU Level of Service	E
Analysis Period (min)	15		

c Critical Lane Group

Lane Group	EBL	EBT	WBL	WBT	WBR	NBT	NBR	SBL	SBT
Lane Group Flow (vph)	213	1730	30	1371	201	17	43	190	175
v/c Ratio	0.97	0.60	0.28	0.82	0.24	0.18	0.13	0.67	0.38
Control Delay	101.6	19.5	57.9	27.7	4.4	51.9	0.9	51.4	3.7
Queue Delay	0.0	0.0	0.0	0.0	0.0	0.0	0.2	2.4	0.5
Total Delay	101.6	19.5	57.9	27.7	4.4	51.9	1.1	53.8	4.1
Queue Length 50th (ft)	145	249	28	272	0	11	0	128	0
Queue Length 95th (ft)	#293	435	m41	#870	m52	35	0	185	18
Internal Link Dist (ft)		143		196		222			190
Turn Bay Length (ft)			180		215				
Base Capacity (vph)	219	2886	106	1674	844	97	320	432	572
Starvation Cap Reductn	0	0	0	0	0	0	0	0	0
Spillback Cap Reductn	0	89	0	0	0	0	70	140	145
Storage Cap Reductn	0	0	0	0	0	0	0	0	0
Reduced v/c Ratio	0.97	0.62	0.28	0.82	0.24	0.18	0.17	0.65	0.41

Intersection Summary

95th percentile volume exceeds capacity, queue may be longer.
Queue shown is maximum after two cycles.
m Volume for 95th percentile queue is metered by upstream signal.



Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (vph)	204	1630	31	28	1261	185	16	0	40	218	0	118
Future Volume (vph)	204	1630	31	28	1261	185	16	0	40	218	0	118
Ideal Flow (vphpl)	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900
Grade (%)		-7%			-1%			0%			0%	
Total Lost time (s)	6.8	5.3		6.8	5.0	5.0		7.2	6.8	7.2	7.2	
Lane Util. Factor	1.00	0.91		1.00	0.95	1.00		1.00	1.00	0.95	0.95	
Frpb, ped/bikes	1.00	1.00		1.00	1.00	0.98		1.00	1.00	1.00	0.99	
Flpb, ped/bikes	1.00	1.00		1.00	1.00	1.00		1.00	1.00	1.00	1.00	
Frt	1.00	1.00		1.00	1.00	0.85		1.00	0.85	1.00	0.89	
Flt Protected	0.95	1.00		0.95	1.00	1.00		0.95	1.00	0.95	0.99	
Satd. Flow (prot)	1832	5147		1778	3489	1557		1770	1583	1681	1540	
Flt Permitted	0.95	1.00		0.95	1.00	1.00		0.95	1.00	0.95	0.99	
Satd. Flow (perm)	1832	5147		1778	3489	1557		1770	1583	1681	1540	
Peak-hour factor, PHF	0.96	0.96	0.96	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92
Adj. Flow (vph)	212	1698	32	30	1371	201	17	0	43	237	0	128
RTOR Reduction (vph)	0	1	0	0	0	109	0	0	40	0	145	0
Lane Group Flow (vph)	213	1729	0	30	1371	92	0	17	3	190	30	0
Confl. Peds. (#/hr)			3	3			1					1
Confl. Bikes (#/hr)						2						
Heavy Vehicles (%)	2%	4%	2%	2%	4%	2%	2%	2%	2%	2%	2%	2%
Turn Type	Prot	NA		Prot	NA	Perm	Split	NA	pm+ov	Split	NA	
Protected Phases	1	6		5	2		8	8	5	4	4	
Permitted Phases						2			8			
Actuated Green, G (s)	14.0	53.2		5.2	44.7	44.7		2.3	7.5	17.8	17.8	
Effective Green, g (s)	14.0	53.2		5.2	44.7	44.7		2.3	7.5	17.8	17.8	
Actuated g/C Ratio	0.13	0.51		0.05	0.43	0.43		0.02	0.07	0.17	0.17	
Clearance Time (s)	6.8	5.3		6.8	5.0	5.0		7.2	6.8	7.2	7.2	
Vehicle Extension (s)	3.0	3.0		3.0	3.0	3.0		3.0	3.0	3.0	3.0	
Lane Grp Cap (vph)	244	2607		88	1485	662		38	113	284	261	
v/s Ratio Prot	c0.12	0.34		0.02	c0.39			c0.01	0.00	c0.11	0.02	
v/s Ratio Perm						0.06			0.00			
v/c Ratio	0.87	0.66		0.34	0.92	0.14		0.45	0.03	0.67	0.11	
Uniform Delay, d1	44.6	19.2		48.2	28.5	18.4		50.7	45.4	40.8	36.9	
Progression Factor	1.00	1.00		1.11	0.91	1.20		1.00	1.00	1.00	1.00	
Incremental Delay, d2	27.2	1.3		1.8	9.0	0.3		8.2	0.1	5.9	0.2	
Delay (s)	71.8	20.6		55.2	35.0	22.5		58.9	45.5	46.7	37.1	
Level of Service	E	C		E	C	C		E	D	D	D	
Approach Delay (s)		26.2			33.8			49.3			42.1	
Approach LOS		C			C			D			D	

Intersection Summary			
HCM 2000 Control Delay	31.1	HCM 2000 Level of Service	C
HCM 2000 Volume to Capacity ratio	0.85		
Actuated Cycle Length (s)	105.0	Sum of lost time (s)	26.5
Intersection Capacity Utilization	78.5%	ICU Level of Service	D
Analysis Period (min)	15		

c Critical Lane Group

APPENDIX F

Roundabout Analysis Worksheets – Future Conditions (2022)

MOVEMENT SUMMARY

Site: 101 [AM Peak]

WFC
Roundabout

Movement Performance - Vehicles											
Mov ID	OD Mov	Demand Flows Total veh/h	HV %	Deg. Satn v/c	Average Delay sec	Level of Service	95% Back of Queue Vehicles veh	Distance ft	Prop. Queued	Effective Stop Rate per veh	Average Speed mph
South: Chestnut Street											
3	L2	26	2.0	0.065	10.0	LOS A	0.4	9.1	0.88	0.82	22.8
8	T1	1	2.0	0.054	13.7	LOS B	0.3	6.6	0.84	0.83	22.3
18	R2	14	2.0	0.054	13.7	LOS B	0.3	6.6	0.84	0.83	21.8
Approach		41	2.0	0.065	11.3	LOS B	0.4	9.1	0.86	0.82	22.4
East: Leesburg Pike											
1	L2	11	2.0	0.935	32.2	LOS D	24.3	617.7	1.00	0.89	19.4
6	T1	1650	2.0	0.935	31.9	LOS D	24.3	617.7	1.00	0.88	19.1
16	R2	563	2.0	0.935	31.3	LOS D	24.3	617.6	1.00	0.86	18.7
Approach		2224	2.0	0.935	31.7	LOS D	24.3	617.7	1.00	0.87	19.0
North: Commons Drive											
7	L2	368	2.0	0.993	78.8	LOS F	14.8	375.0	1.00	1.82	13.7
4	T1	1	2.0	0.327	24.2	LOS C	1.7	42.5	0.88	0.91	20.2
14	R2	76	2.0	0.327	24.2	LOS C	1.7	42.5	0.88	0.91	19.7
Approach		446	2.0	0.993	69.4	LOS F	14.8	375.0	0.98	1.66	14.4
West: Leesburg Pike											
5	L2	138	2.0	0.888	31.7	LOS D	15.7	398.9	1.00	1.26	19.4
2	T1	1340	2.0	0.888	31.7	LOS D	15.7	398.9	1.00	1.26	19.1
12	R2	27	2.0	0.888	31.7	LOS D	15.7	398.9	1.00	1.26	18.8
Approach		1505	2.0	0.888	31.7	LOS D	15.7	398.9	1.00	1.26	19.1
All Vehicles		4216	2.0	0.993	35.5	LOS E	24.3	617.7	1.00	1.09	18.4

Site Level of Service (LOS) Method: Delay & v/c (HCM 6). Site LOS Method is specified in the Parameter Settings dialog (Site tab).
Roundabout LOS Method: Same as Sign Control.
Vehicle movement LOS values are based on average delay and v/c ratio (degree of saturation) per movement.
LOS F will result if v/c > 1 irrespective of movement delay value (does not apply for approaches and intersection).
Intersection and Approach LOS values are based on average delay for all movements (v/c not used as specified in HCM 6).
Roundabout Capacity Model: SIDRA Standard.
HCM Delay Formula option is used. Control Delay does not include Geometric Delay since Exclude Geometric Delay option applies.
Gap-Acceptance Capacity: SIDRA Standard (Akçelik M3D).
HV (%) values are calculated for All Movement Classes of All Heavy Vehicle Model Designation.

MOVEMENT SUMMARY

Site: 101 [PM Peak]

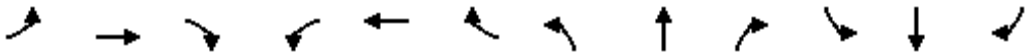
WFC
Roundabout

Movement Performance - Vehicles											
Mov ID	OD Mov	Demand Flows Total veh/h	HV %	Deg. Satn v/c	Average Delay sec	Level of Service	95% Back of Queue Vehicles veh	Distance ft	Prop. Queued	Effective Stop Rate per veh	Average Speed mph
South: Chestnut Street											
3	L2	17	2.0	0.083	18.0	LOS C	0.4	10.2	0.87	0.87	21.2
8	T1	1	2.0	0.083	18.0	LOS C	0.4	10.2	0.87	0.87	20.8
18	R2	43	2.0	0.133	13.4	LOS B	0.8	19.2	0.92	0.92	21.8
Approach		62	2.0	0.133	14.8	LOS B	0.8	19.2	0.91	0.91	21.6
East: Leesburg Pike											
1	L2	30	2.0	0.716	14.8	LOS B	8.1	205.9	0.76	0.61	22.7
6	T1	1371	2.0	0.716	14.5	LOS B	8.1	206.2	0.76	0.60	22.4
16	R2	201	2.0	0.716	14.2	LOS B	8.1	206.2	0.75	0.59	21.9
Approach		1602	2.0	0.716	14.4	LOS B	8.1	206.2	0.76	0.60	22.3
North: Commons Drive											
7	L2	237	2.0	0.409	12.5	LOS B	2.4	59.9	0.84	0.89	22.2
4	T1	1	2.0	0.305	13.7	LOS B	1.5	37.1	0.80	0.81	22.3
14	R2	128	2.0	0.305	13.7	LOS B	1.5	37.1	0.80	0.81	21.7
Approach		366	2.0	0.409	12.9	LOS B	2.4	59.9	0.83	0.86	22.1
West: Leesburg Pike											
5	L2	222	2.0	1.059	66.3	LOS F	46.2	1174.5	1.00	2.06	15.0
2	T1	1772	2.0	1.059	66.3	LOS F	46.2	1174.5	1.00	2.06	14.8
12	R2	34	2.0	1.059	66.3	LOS F	46.2	1174.5	1.00	2.06	14.6
Approach		2027	2.0	1.059	66.3	LOS F	46.2	1174.5	1.00	2.06	14.8
All Vehicles		4058	2.0	1.059	40.2	LOS E	46.2	1174.5	0.89	1.36	17.8

Site Level of Service (LOS) Method: Delay & v/c (HCM 6). Site LOS Method is specified in the Parameter Settings dialog (Site tab).
Roundabout LOS Method: Same as Sign Control.
Vehicle movement LOS values are based on average delay and v/c ratio (degree of saturation) per movement.
LOS F will result if v/c > 1 irrespective of movement delay value (does not apply for approaches and intersection).
Intersection and Approach LOS values are based on average delay for all movements (v/c not used as specified in HCM 6).
Roundabout Capacity Model: SIDRA Standard.
HCM Delay Formula option is used. Control Delay does not include Geometric Delay since Exclude Geometric Delay option applies.
Gap-Acceptance Capacity: SIDRA Standard (Akçelik M3D).
HV (%) values are calculated for All Movement Classes of All Heavy Vehicle Model Designation.

APPENDIX G

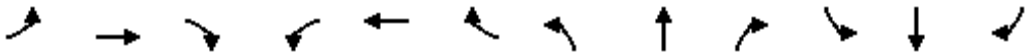
Intersection Capacity Analysis Worksheets – Future Conditions Unsignalized (2022)



Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (veh/h)	127	1233	25	10	1518	518	24	0	13	339	0	70
Future Volume (Veh/h)	127	1233	25	10	1518	518	24	0	13	339	0	70
Sign Control		Free			Free			Stop			Stop	
Grade		-7%			-1%			0%			0%	
Peak Hour Factor	0.98	0.98	0.98	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92
Hourly flow rate (vph)	130	1258	26	11	1650	563	26	0	14	368	0	76
Pedestrians					1			3			3	
Lane Width (ft)					12.0			12.0			12.0	
Walking Speed (ft/s)					4.0			4.0			4.0	
Percent Blockage					0			0			0	
Right turn flare (veh)												
Median type		None			None							
Median storage veh)												
Upstream signal (ft)					588							
pX, platoon unblocked	0.46						0.46	0.46		0.46	0.46	0.46
vC, conflicting volume	2216			1287			2457	3772	436	2369	3222	828
vC1, stage 1 conf vol												
vC2, stage 2 conf vol												
vCu, unblocked vol	1290			1287			1816	4684	436	1624	3484	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 %	47			98			0	100	98	0	100	85
cM capacity (veh/h)	244			533			11	0	566	17	1	496

Direction, Lane #	EB 1	EB 2	EB 3	EB 4	WB 1	WB 2	WB 3	WB 4	NB 1	NB 2	SB 1	SB 2
Volume Total	130	503	503	278	11	825	825	563	26	14	245	199
Volume Left	130	0	0	0	11	0	0	0	26	0	245	123
Volume Right	0	0	0	26	0	0	0	563	0	14	0	76
cSH	244	1700	1700	1700	533	1700	1700	1700	11	566	17	27
Volume to Capacity	0.53	0.30	0.30	0.16	0.02	0.49	0.49	0.33	2.42	0.02	14.23	7.27
Queue Length 95th (ft)	71	0	0	0	2	0	0	0	105	2	Err	Err
Control Delay (s)	35.4	0.0	0.0	0.0	11.9	0.0	0.0	0.0	1342.8	11.5	Err	Err
Lane LOS	E				B				F	B	F	F
Approach Delay (s)	3.3				0.1				876.9		Err	
Approach LOS									F		F	

Intersection Summary				
Average Delay	1086.7			
Intersection Capacity Utilization	77.2%	ICU Level of Service	D	
Analysis Period (min)	15			



Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (veh/h)	204	1630	31	28	1261	185	16	0	40	218	0	118
Future Volume (Veh/h)	204	1630	31	28	1261	185	16	0	40	218	0	118
Sign Control		Free			Free			Stop			Stop	
Grade		-7%			-1%			0%			0%	
Peak Hour Factor	0.96	0.96	0.96	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92
Hourly flow rate (vph)	213	1698	32	30	1371	201	17	0	43	237	0	128
Pedestrians		1						3				
Lane Width (ft)		12.0						12.0				
Walking Speed (ft/s)		4.0						4.0				
Percent Blockage		0						0				
Right turn flare (veh)												
Median type		None			None							
Median storage veh)												
Upstream signal (ft)					588							
pX, platoon unblocked	0.74						0.74	0.74		0.74	0.74	0.74
vC, conflicting volume	1572			1733			3018	3775	585	2466	3590	686
vC1, stage 1 conf vol												
vC2, stage 2 conf vol												
vCu, unblocked vol	1062			1733			3024	4052	585	2275	3801	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 %	56			92			0	100	91	0	100	84
cM capacity (veh/h)	480			359			2	1	453	9	1	798

Direction, Lane #	EB 1	EB 2	EB 3	EB 4	WB 1	WB 2	WB 3	WB 4	NB 1	NB 2	SB 1	SB 2
Volume Total	213	679	679	372	30	686	686	201	17	43	158	207
Volume Left	213	0	0	0	30	0	0	0	17	0	158	79
Volume Right	0	0	0	32	0	0	0	201	0	43	0	128
cSH	480	1700	1700	1700	359	1700	1700	1700	2	453	9	23
Volume to Capacity	0.44	0.40	0.40	0.22	0.08	0.40	0.40	0.12	7.93	0.09	17.76	9.04
Queue Length 95th (ft)	56	0	0	0	7	0	0	0	Err	8	Err	Err
Control Delay (s)	18.3	0.0	0.0	0.0	15.9	0.0	0.0	0.0	Err	13.8	Err	Err
Lane LOS	C				C				F	B	F	F
Approach Delay (s)	2.0				0.3				2842.9		Err	
Approach LOS									F		F	

Intersection Summary				
Average Delay	963.4			
Intersection Capacity Utilization	72.5%	ICU Level of Service	C	
Analysis Period (min)	15			

APPENDIX H

VJuST Input and Results Worksheets

AM PEAK

VDOT Junction Screening Tool

Input Worksheet

Project Title:	*Leesburg Pike and Chestnut Street/Commons Drive*
E-W Facility:	*Leesburg Pike*
N-S Facility:	*Chestnut Street/Commons Drive*
Date:	*August 4, 2020*

Traffic Volume Demand

Direction	Volume (veh/hr)			Truck Percent (%)
	U-Turn / Left	Through	Right	
Eastbound	127	1233	25	2.00%
Westbound	10	1518	518	2.00%
Northbound	24	0	13	2.00%
Southbound	339	0	70	2.00%
Adjustment Factor	0.80 / 0.95		0.85	
Suggested	**U - 0.8 / L - 0.95**		**0.85**	
Truck to PCE Factor		**Suggested = 2.00**		2.00
Critical Lane Volume		1600		

Equivalent Passenger Car Volume

	Volume (pc/hr)			
	U-Turn / Left	Through	Right	Approach
Eastbound	130	1258	26	1414
Westbound	10	1548	528	2086
Northbound	24	0	13	37
Southbound	346	0	71	417

Notes:

Left-turn Adjustment Factor	*Conversion of left-turning vehicles to equivalent through vehicles*
Right-turn Adjustment Factor	*Conversion of right-turning vehicles to equivalent through vehicles*
U-turn Adjustment Factor	*Conversion of U-turning vehicles to equivalent through vehicles*
Truck to PCE Factor	*1 truck = X Passenger Car Equivalents*
Critical Lane Volume Sum Limit	*Saturation value for critical lane volume sum at an intersection*

VDOT Junction Screening Tool

Possible Configurations

Indicate with a "Y" or "N" if each intersection or interchange configuration should or should not be considered. Use the information links for guidance. Then, click the "Show/Hide Configurations button" to hide the worksheets for the configurations that will not be considered.

#	Intersections	Information	Consider?	Justification
	Signalized Intersections			
1	Conventional	-	Y	
2	Bowtie	Link	N	Right-of-way restrictions identified
3	Center Turn Overpass	Link	N	Right-of-way restrictions identified
4	Continuous Green-T	Link	N	Unable to accommodate traffic patterns
5	Echelon	Link	N	Right-of-way restrictions identified
6	Full Displaced Left Turn	Link	N	Right-of-way restrictions identified
7	Median U-Turn	Link	N	Right-of-way restrictions identified
8	Partial Displaced Left Turn	Link	N	Right-of-way restrictions identified
9	Partial Median U-Turn	Link	N	Right-of-way restrictions identified
10	Quadrant Roadway N-E	Link	N	Right-of-way restrictions identified
11	Quadrant Roadway N-W	Link	N	Right-of-way restrictions identified
12	Quadrant Roadway S-E	Link	N	Right-of-way restrictions identified
13	Quadrant Roadway S-W	Link	N	Right-of-way restrictions identified
14	Restricted Crossing U-Turn	Link	N	Right-of-way restrictions identified
15	Single Loop	Link	N	Right-of-way restrictions identified
16	Split Intersection	Link	N	Right-of-way restrictions identified
	Unsignalized Intersections			
17	50 Mini Roundabout	Link	N	Unable to accommodate magnitude of traffic volumes
18	75 Mini Roundabout	Link	N	Unable to accommodate magnitude of traffic volumes
19	Roundabout	Link	Y	
20	Two-Way Stop Control	-	Y	
#	Interchanges	Information	Consider?	Justification
21	Traditional Diamond	Link	N	Right-of-way restrictions identified
22	Contraflow Left	Link	N	Right-of-way restrictions identified
23	Displaced Left Turn	Link	N	Right-of-way restrictions identified
24	Diverging Diamond	Link	N	Right-of-way restrictions identified
25	Double Roundabout	Link	N	Right-of-way restrictions identified
26	Michigan Urban Diamond	Link	N	Right-of-way restrictions identified
27	Partial Cloverleaf	Link	N	Right-of-way restrictions identified
28	Single Point	Link	N	Right-of-way restrictions identified
29	Single Roundabout	Link	N	Right-of-way restrictions identified

VDOT Junction Screening Tool

Directional Questions and Base Lane Configurations

Before entering a base number of through lanes for each direction, answer all applicable directional question for each intersection or interchange configuration selected for consideration. Navigate to the lane configuration worksheet for example diagrams, if provided.

Intersections	Question	Direction
Bowtie	N/A	N/A
Continuous Green-T	N/A	N/A
Echelon	N/A	N/A
Median U-Turn	N/A	N/A
Partial Displaced Left Turn	N/A	N/A
Partial Median U-Turn	N/A	N/A
Restricted Crossing U-Turn	N/A	N/A
Single Loop	N/A	N/A
Split Intersection	N/A	N/A
Interchanges	**Question**	**Direction**
All	N/A	N/A

Base Number of Through Lanes

Enter a base number of through lanes for each direction. The number of through lanes entered will populate on each non-roundabout lane configuration worksheet. This tool also allows the user to enter the number of through lanes on the lane configuration worksheets directly. This base number may be overwritten on individual lane configuration worksheets. Turn lanes, shared lanes, and channelized lanes must still be entered in each lane configuration worksheet.

Eastbound	3
Westbound	2
Northbound	1
Southbound	1

VDOT Junction Screening Tool

Results Worksheet

General Information	
Project Title:	Leesburg Pike and Chestnut Street/Commons Drive
EW Facility:	Leesburg Pike
NS Facility:	Chestnut Street/Commons Drive
Date:	August 4, 2020

Volumes (veh/hr)	U-Turn / Left	Through	Right
Eastbound	127	1233	25
Westbound	10	1518	518
Northbound	24	0	13
Southbound	339	0	70

General Instructions: All intersection and interchange configurations have a default assumption of one exclusive lane per movement. No results shall be interpreted until the user has verified the lane configurations on each worksheet.

Intersection Results

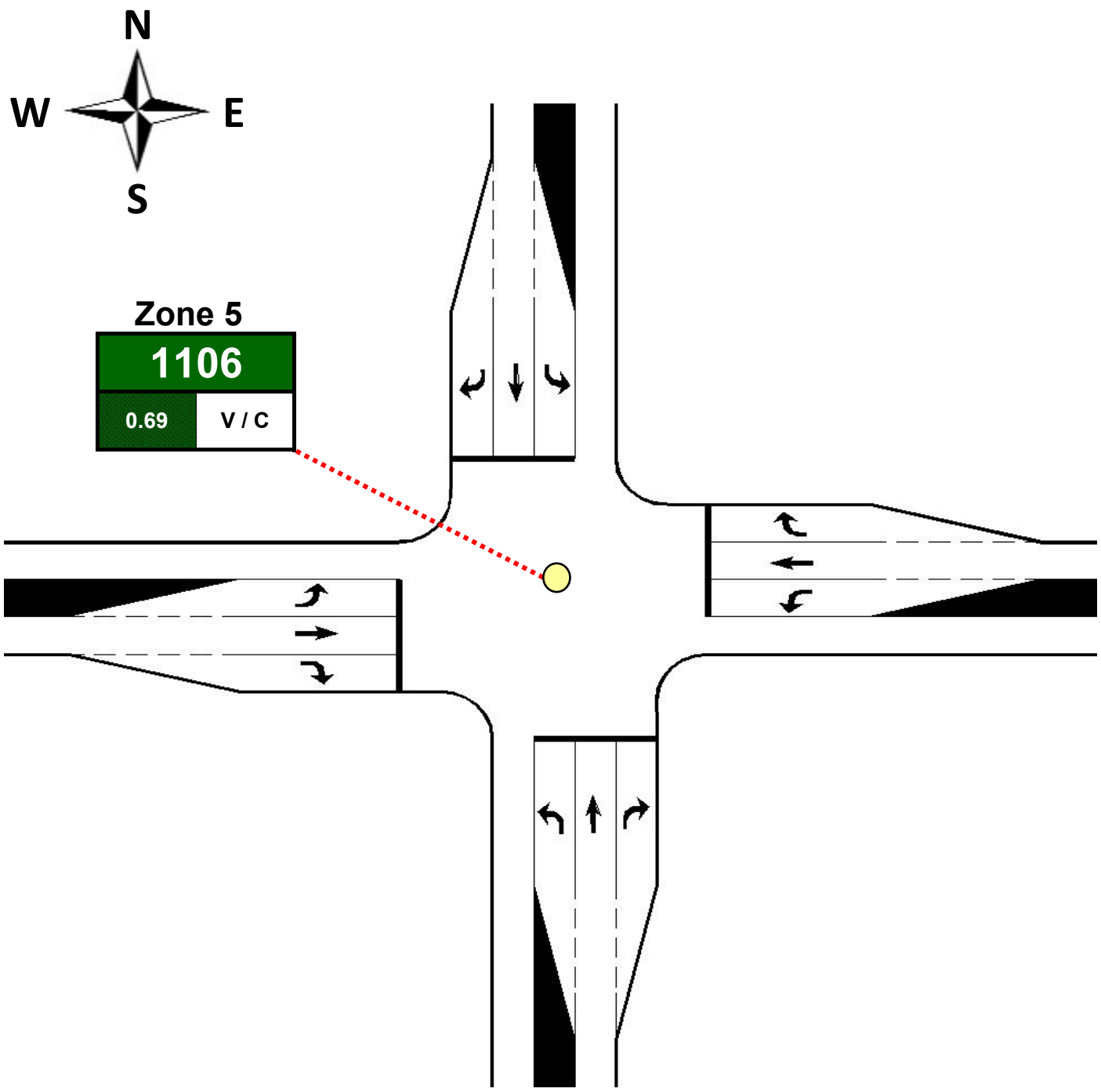
		Congestion	Pedestrian	Safety	Notes
Type	**Dir**	**Maximum V/C**	**Accommodation Compared to Conventional**	**Weighted Total Conflict Points**	
Conventional	-	0.69		48	
Roundabout	-	1.10		8	
Two-Way Stop Control	-	N/A*		48	

Information	
Congestion	The maximum v/c ratio represents the worst v/c of all zones that make up an intersection.
Pedestrian	Compares the potential of each design to accommodate pedestrians based on safety, wayfinding, and delay. Potential is qualitatively defined as better (+), similar (blank cell), or worse (-) than a conventional intersection or traditional diamond interchange.
Safety	Weighted Total = (2 x Crossing Conflicts) + Merging Conflicts + Diverging Conflicts

Conventional

DESIGN AND RESULTS

		Critical Lane Volume Sum			
Project Name:	*Leesburg Pike and Chestnut Street/Commons Drive*	< 1200	1200 - 1399	1400 - 1599	≥ 1600
EW Facility:	*Leesburg Pike*				
NS Facility:	*Chestnut Street/Commons Drive*	VOLUME / CAPACITY RATIO:		0.69	
Date:	*August 4, 2020*				

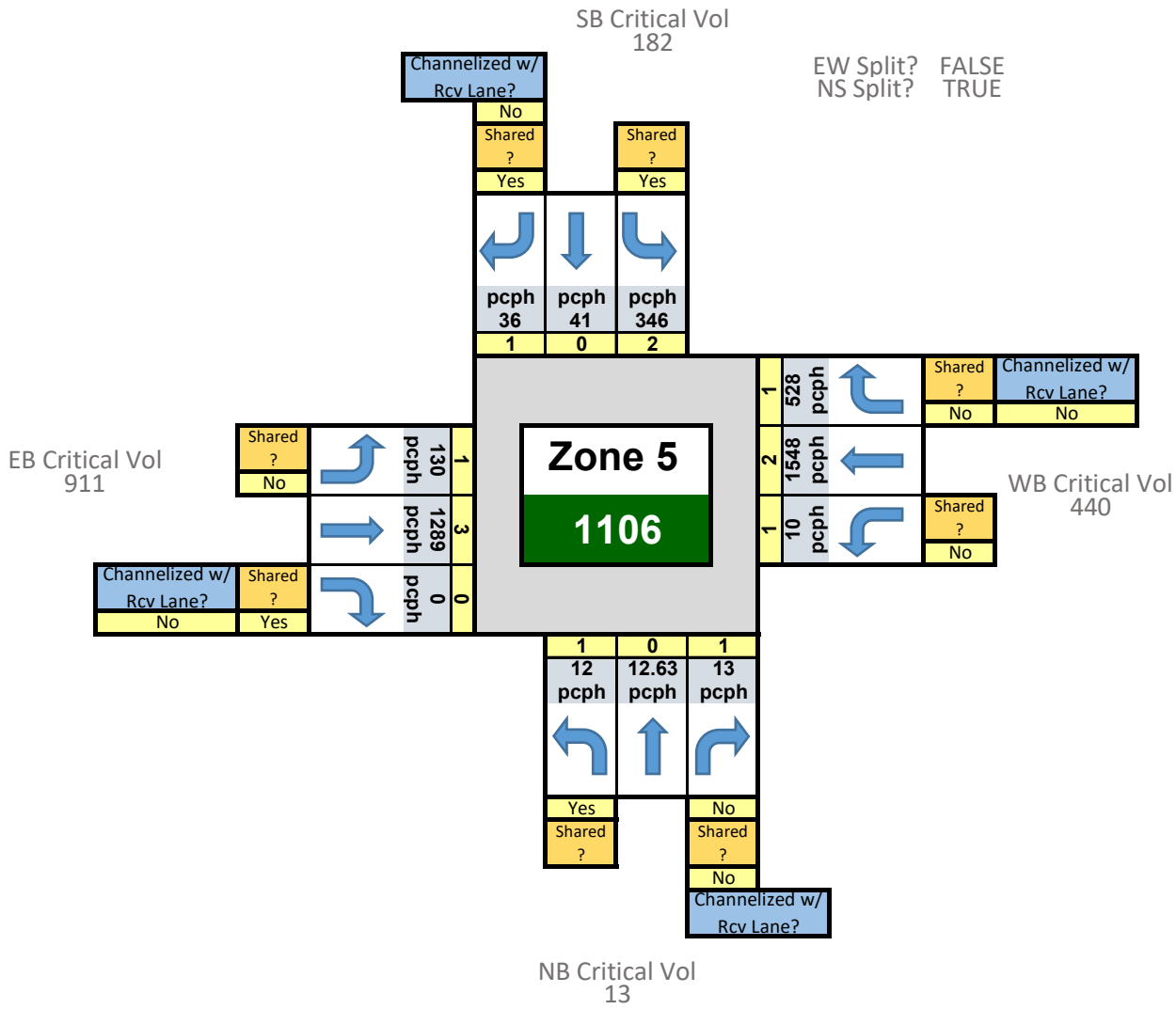


Note: This diagram does not reflect the actual lane configuration of the intersection

Conventional

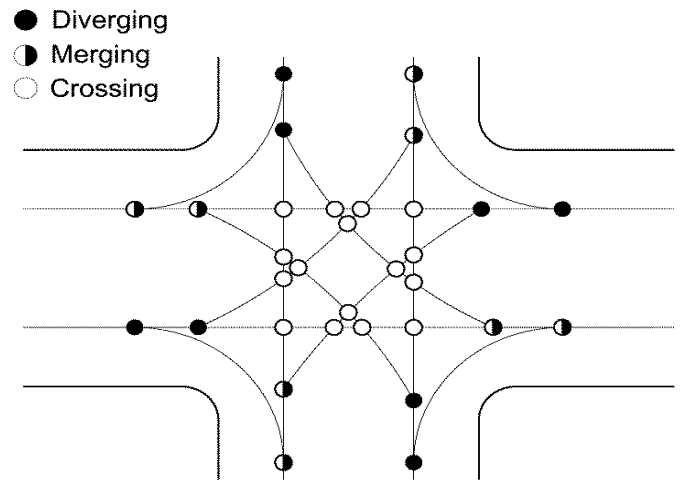
DATA INPUT AND CONFIGURATION

Enter the lane configurations in the yellow cells.



Back to Results

Safety - Conflict Point Diagram



Conflict Type	Count
Crossing	16
Merging	8
Diverging	8
Total	**32**

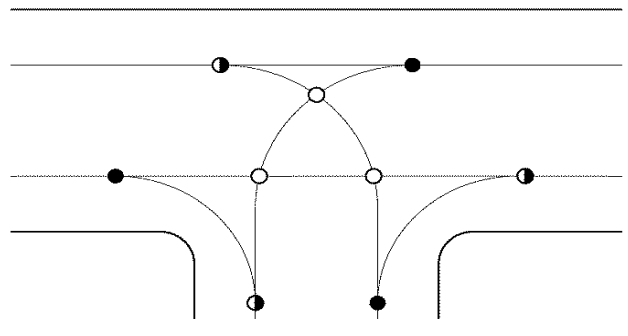
Conflict Type	Weight
Crossing	2
Merging	1
Diverging	1

Weighted Total Conflict Points

48

Safety - Conflict Point Diagram (Three Legs)

● Diverging
◑ Merging
○ Crossing



Conflict Type	Count
Crossing	3
Merging	3
Diverging	3
Total	**9**

Conflict Type	Weight
Crossing	2
Merging	1
Diverging	1

Weighted Total Conflict Points

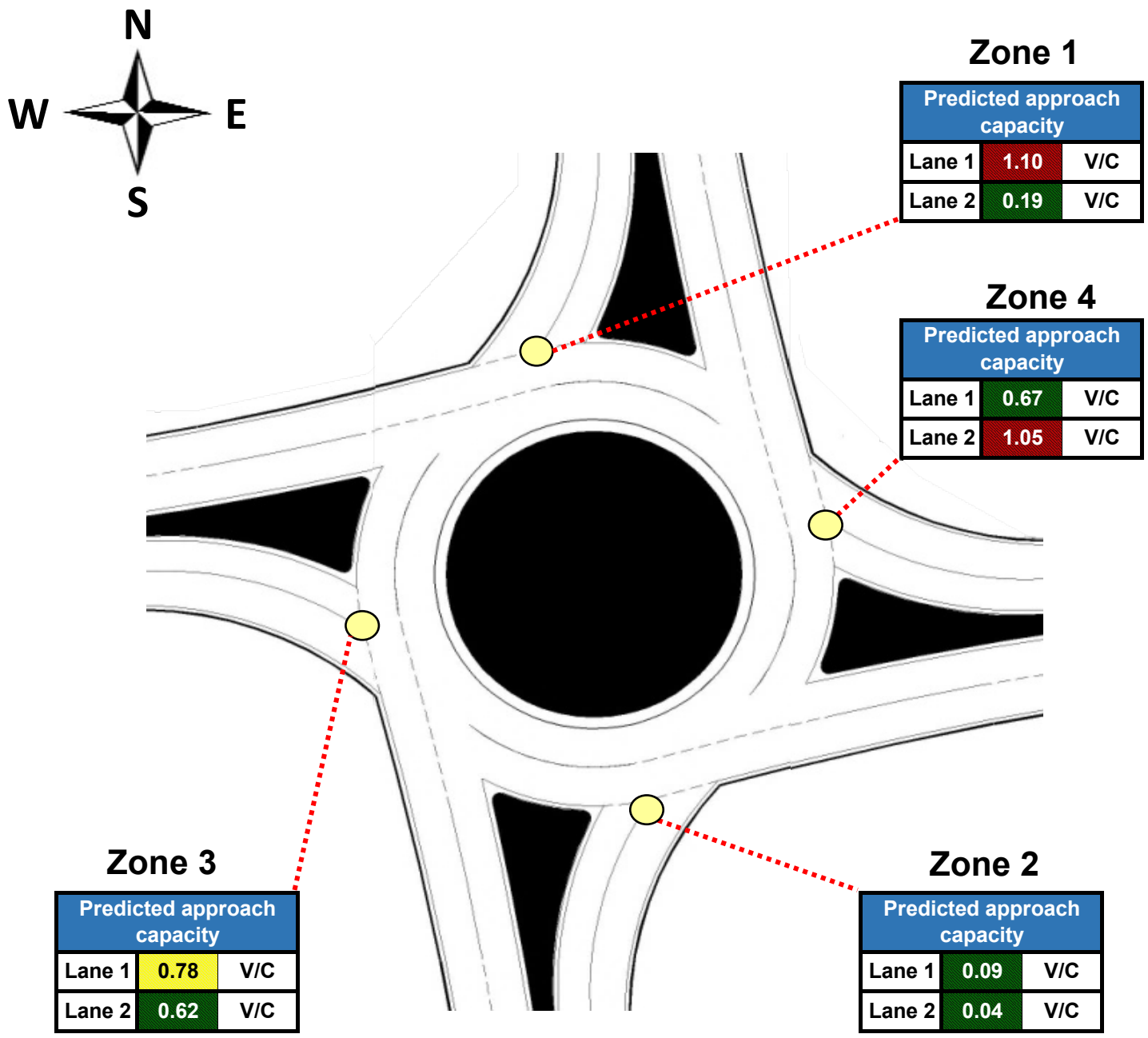
12

Roundabout

DESIGN AND RESULTS

Project Name:	*esburg Pike and Chestnut Street/Commons Dr*
EW Facility:	*Leesburg Pike*
NS Facility:	*Chestnut Street/Commons Drive*
Date:	*August 4, 2020*

Critical Lane Volume Sum			
< 1200	**1200 - 1399**	**1400 - 1599**	**≥ 1600**
VOLUME / CAPACITY RATIO:	**1.10**		



Zone 1

Predicted approach capacity		
Lane 1	1.10	V/C
Lane 2	0.19	V/C

Zone 4

Predicted approach capacity		
Lane 1	0.67	V/C
Lane 2	1.05	V/C

Zone 3

Predicted approach capacity		
Lane 1	0.78	V/C
Lane 2	0.62	V/C

Zone 2

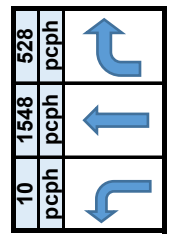
Predicted approach capacity		
Lane 1	0.09	V/C
Lane 2	0.04	V/C

Roundabout

DATA INPUT AND CONFIGURATION

Enter the lane configurations in the yellow cells.

pcph	pcph	pcph
71	0	346



SB

Slip Lane?	Number of Entry Lanes
No	2

Number of Entry Lanes	Slip Lane?
2	No

Number of Circulating Lanes
2

Number of Circulating Lanes
2

EB

WB

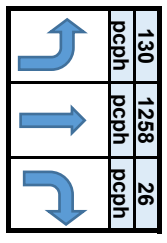
Number of Entry Lanes	Slip Lane?
2	No

Number of Circulating Lanes
2

Number of Circulating Lanes
2

Number of Entry Lanes	Slip Lane?
2	No

NB

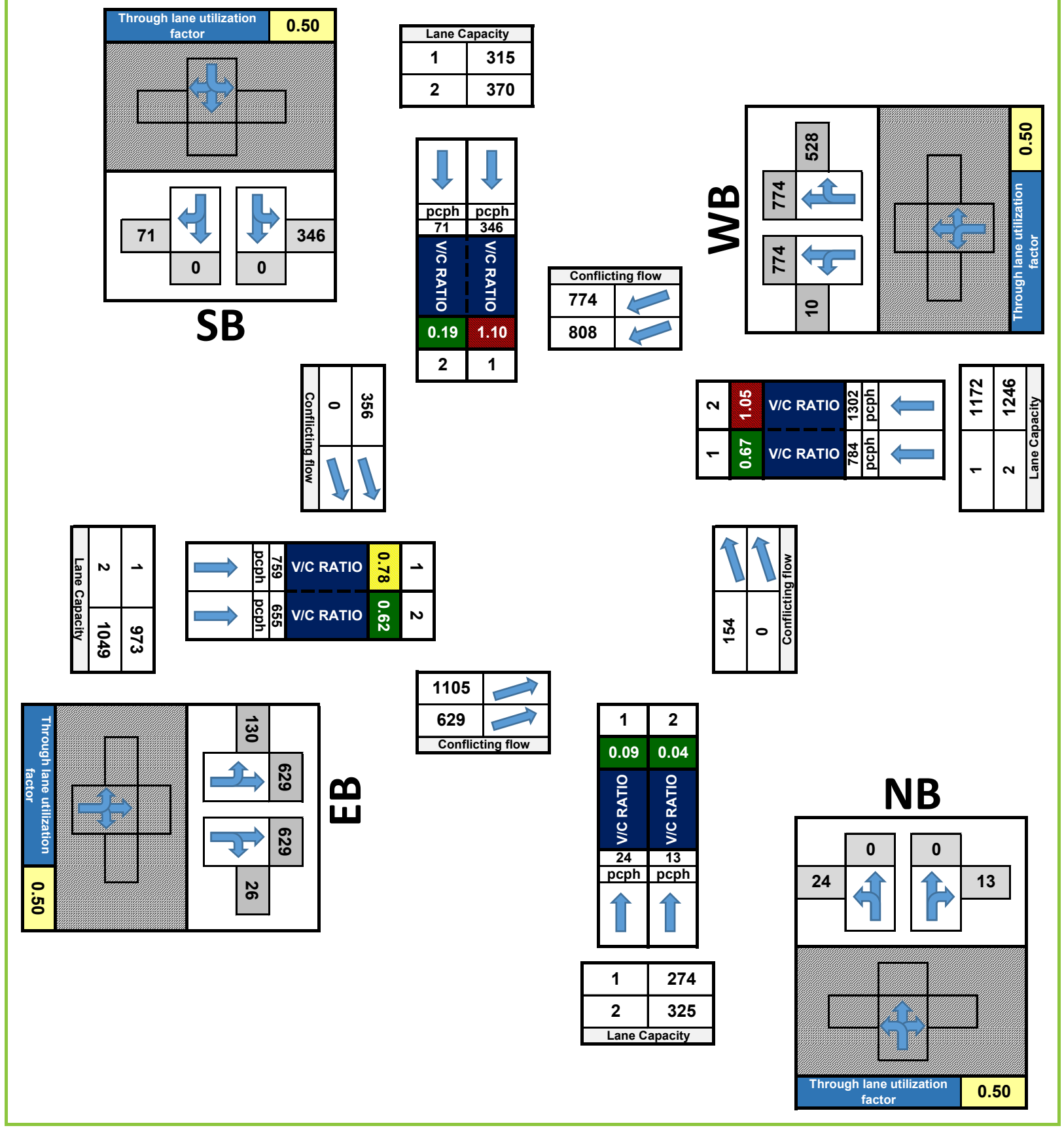


24	0	13
pcph	pcph	pcph

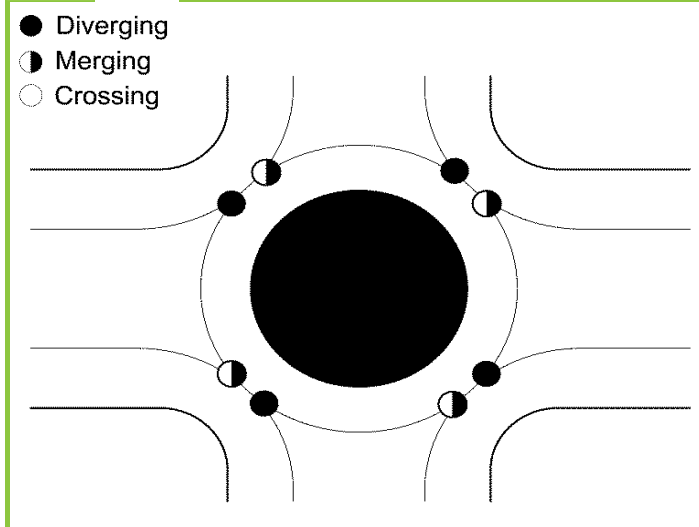
Back to Results

Roundabout

CAPACITY CALCULATIONS



Safety - Conflict Point Diagram



Conflict Type	Count
Crossing	0
Merging	4
Diverging	4
Total	**8**

Conflict Type	Weight
Crossing	2
Merging	1
Diverging	1

Weighted Total Conflict Points

8

Assumptions

● The number of circulating lanes in one quadrant is assumed to be equal to the number of exiting lanes in the next quadrant.

● The roundabout is limited to a maximum of two entry lanes and two circulating lanes.

● All left-turning vehicles are assumed to stay in the innermost lane until exiting the roundabout.

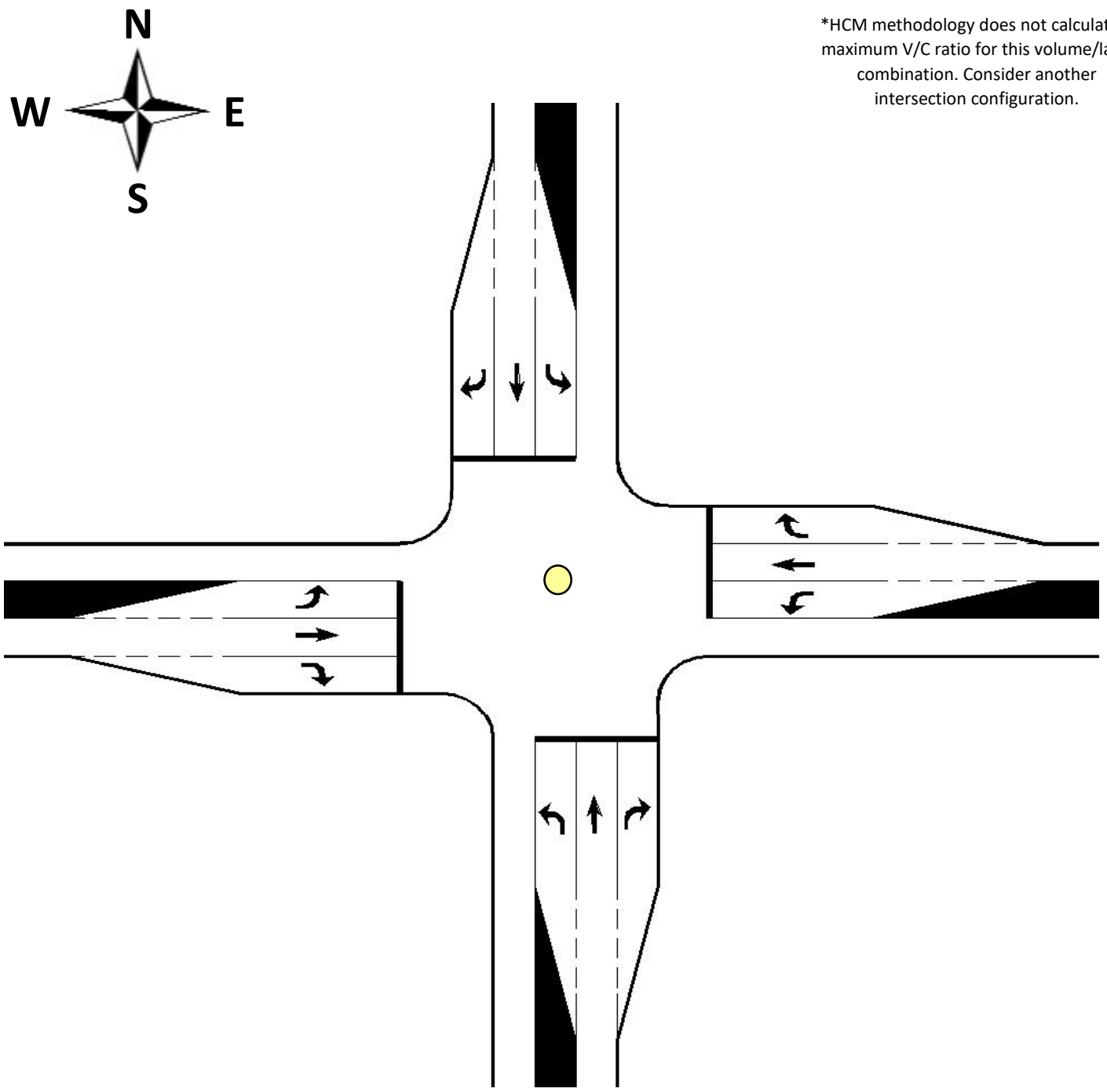
● This worksheet does not use the CLV methodology. The calculations are based on the *HCM 6th Edition* .

Two-Way Stop Control (TWSC)

DESIGN AND RESULTS

Project Name:	*Leesburg Pike and Chestnut Street/Commons Drive*	***Critical Lane Volume Sum***			
EW Facility:	*Leesburg Pike*	**< 1200**	**1200 - 1399**	**1400 - 1599**	**≥ 1600**
NS Facility:	*Chestnut Street/Commons Drive*	**VOLUME / CAPACITY RATIO:**		**N/A***	
Date:	*August 4, 2020*				

*HCM methodology does not calculate a maximum V/C ratio for this volume/lane combination. Consider another intersection configuration.



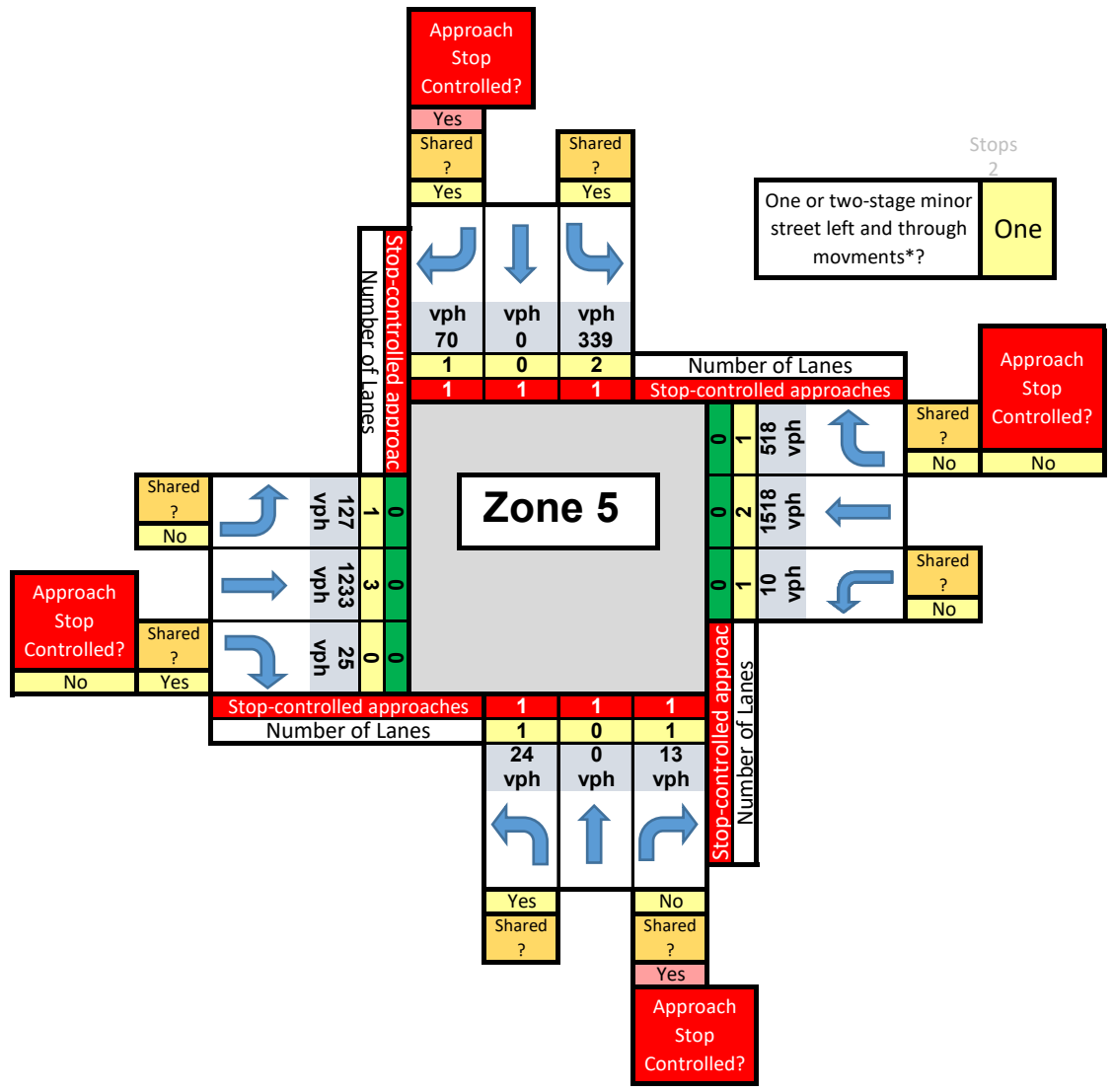
Note: This diagram does not reflect the actual lane configuration of the intersection

Two-Way Stop Control (TWSC)

DATA INPUT AND CONFIGURATION

Step 1: Identify which approaches are stop-controlled by selecting "Yes" from the drop-down box.

Step 2: Enter the lane configurations in the yellow cells.



Back to Results

Two-Way Stop Control (TWSC)

HCM 6 CALCULATIONS

Priority	MVMT	Rank
1	EBL	2
2	EBT	1
3	EBR	1
4	WBL	2
5	WBT	1
6	WBR	1
7	NBL	4
8	NBT	3
9	NBR	2
10	SBL	4
11	SBT	3
12	SBR	2

MAJOR	MINOR
EB	NB
WB	SB

Major street lanes	5
M1 Shared?	FALSE
M4 Shared?	FALSE

Priority	Rank	Flow Rates	Lanes	Shared?	Stop controlled?	Truck %
1	2	127	1	No		0.02
4	2	10	1	No		0.02
7	4	24	1	Yes	Yes	0.02
8	3	0	0		Yes	0.02
9	2	13	1	No	Yes	0.02
10	4	339	2	Yes	Yes	0.02
11	3	0	0		Yes	0.02
12	2	70	1	Yes	Yes	0.02
2	1	1233	3			0.02
3	1	25	0	Yes	No	0.02
5	1	1518	2			0.02
6	1	518	1	No	No	0.02

Conflicting Flows	
$v_{c,1}$	2036.00
$v_{c,4}$	1258.00
$v_{c,7}$	2278.50
$v_{c,8}$	3555.50
$v_{c,9}$	629.00
$v_{c,10}$	2285.20
$v_{c,11}$	3050.00
$v_{c,12}$	759.00
$v_{c,I,7}$	1499.50
$v_{c,II,7}$	779.00
$v_{c,I,8}$	1499.50
$v_{c,II,8}$	2056.00
$v_{c,I,10}$	1538.00
$v_{c,II,10}$	747.20
$v_{c,I,11}$	1538.00
$v_{c,II,11}$	1512.00

Critical Headways	
$t_{c,1}$	5.34
$t_{c,4}$	5.34
$t_{c,7}$	6.44
$t_{c,8}$	6.54
$t_{c,9}$	7.14
$t_{c,10}$	6.44
$t_{c,11}$	6.54
$t_{c,12}$	7.14
$t_{c,I,7}$	7.34
$t_{c,II,7}$	6.74
$t_{c,I,8}$	5.54
$t_{c,II,8}$	5.54
$t_{c,I,10}$	7.34
$t_{c,II,10}$	6.74
$t_{c,I,11}$	5.54
$t_{c,II,11}$	5.54

Follow-Up Headways	
$t_{f,1}$	3.12
$t_{f,4}$	3.12
$t_{f,7}$	3.82
$t_{f,8}$	4.02
$t_{f,9}$	3.92
$t_{f,10}$	3.82
$t_{f,11}$	4.02
$t_{f,12}$	3.92

Poten Capac
$c_{p,1}$
$c_{p,4}$
$c_{p,7}$
$c_{p,8}$
$c_{p,9}$
$c_{p,10}$
$c_{p,11}$
$c_{p,12}$

Two-Stage Capaci
$c_{p,I,7}$
$c_{p,II,7}$
$c_{p,I,8}$
$c_{p,II,8}$
$c_{p,I,10}$
$c_{p,II,10}$
$c_{p,I,11}$
$c_{p,II,11}$

y_7
y_8
y_{10}
$y11$

Mvmt 1,
Mvmt 4,

Mvmt 1, sh
Mvmt 4, sh

Mvmt 7
Mvmt 1(

a	0.91

***Assumption:**	One storage space in median (n_m = 1) for two-stage turns

Saturation Flow Rates	
Through	1800
Right	1500

ıtial ities
119.88
293.22
42.46
5.68
364.33
42.05
12.38
299.52

Movement Capacities	
$c_{m,1}$	119.88
$c_{m,4}$	293.22
$c_{m,7}$	0.00
$c_{m,8}$	0.00
$c_{m,9}$	364.33
$c_{m,10}$	0.00
$c_{m,11}$	0.00
$c_{m,12}$	299.52

Shared Movement Capacities	
1	0.00
1	
0	
1	0.00
1	
1	

Movement Capacities	
1	119.88
2	5400.00
3	1500.00
4	293.22
5	3600.00
6	1500.00
--	--
7-8	0.00
9	364.33
--	--
10-11-12	0.00
--	--

Movement V/C	
1	1.06
2	0.23
3	0.02
4	0.03
5	0.42
6	0.35
--	--
7-8	0.00
9	0.04
--	--
10-11-12	0.00
--	--

Intersection V/C
N/A*

V/C Not Reported for Any Movements?
Yes

*HCM methodology does not calculate a maximum V/C ratio for this volume/lane combination. Consider another intersection configuration.

Potential ities
88.53
322.14
183.62
96.61
83.10
336.93
175.79
181.04

Two-Stage Movement Capacities	
$c_{m,I,7}$	0.00
$c_{m,II,7}$	238.43
$c_{m,I,8}$	0.00
$c_{m,II,8}$	93.31
$c_{m,I,10}$	80.26
$c_{m,II,10}$	0.00
$c_{m,I,11}$	169.80
$c_{m,II,11}$	0.00

Single-Stage Movement Capacities	
$c_{m,7}$	0.00
$c_{m,8}$	0.00
$c_{m,10}$	0.00
$c_{m,11}$	0.00

0.00
0.00
-8.03
-16.98

$c_{T,7}$	0.00	
$c_{T,8}$	0.00	
$c_{T,10}$	0.00	
$c_{T,11}$	0.00	
$p_{0,1}$	0.00	excl left
$p_{0,4}$	0.97	excl left
$p^*_{0,1}$	0.00	hared left
$p^*_{0,4}$	0.78	hared left
p''_{7}	0.000	, 4-leg
p''_{10}	0.000	), 4-leg
$x_{1i,1+2}$	0.70	
$x_{4i,1+2}$	0.84	

$p_{0,8}$	0.00	$p_{0,9}$ 0.96
$p_{0,11}$	1.00	$p_{0,12}$ 0.77
p'_{7}	0.00	$f_{p,7}$ 0.00
p'_{10}	0.00	$f_{p,10}$ 0.00

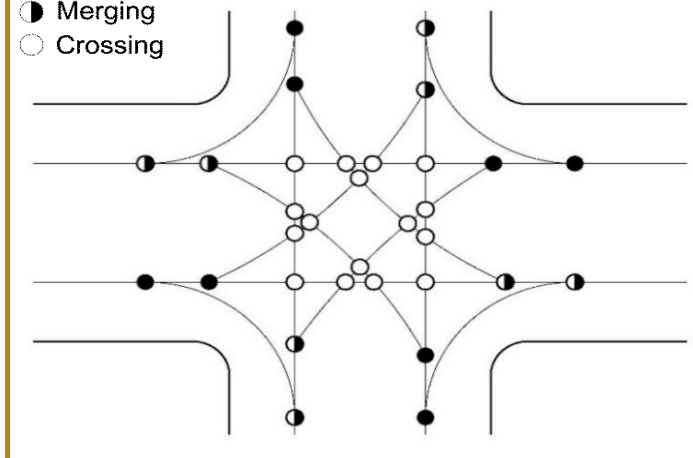
One Stage

f_{8}	0.00
f_{11}	0.00
f_{7}	0.00
f_{10}	0.00

Two Stage

$f_{I,8}$	0.00	$f_{II,8}$ 0.97	$p_{0,I,8}$ 1.00
$f_{I,11}$	0.97	$f_{II,11}$ 0.00	$p_{0,I,11}$ 1.00
$f_{I,7}$	0.00	$f_{II,7}$ 0.74	
$f_{I,10}$	0.97	$f_{II,10}$ 0.00	

Safety - Conflict Point Diagram



Conflict Type	Count
Crossing	16
Merging	8
Diverging	8
Total	**32**

Conflict Type	Weight
Crossing	2
Merging	1
Diverging	1

Weighted Total Conflict Points

48

Assumptions

● This worksheet does not use the CLV methodology. The calculations are based on the *HCM, 6th Edition* . The calculations are based on vehicles per hour.

PM PEAK

VDOT Junction Screening Tool

Input Worksheet

Project Title:	*Leesburg Pike and Chestnut Street/Commons Drive*
E-W Facility:	*Leesburg Pike*
N-S Facility:	*Chestnut Street/Commons Drive*
Date:	*August 4, 2020*

Traffic Volume Demand

Direction	Volume (veh/hr)			Truck Percent (%)
	U-Turn / Left	Through	Right	
Eastbound	204	1630	31	2.00%
Westbound	28	1261	185	2.00%
Northbound	16	0	40	2.00%
Southbound	218	0	118	2.00%
Adjustment Factor	0.80 / 0.95		0.85	
Suggested	**U - 0.8 / L - 0.95**		**0.85**	
Truck to PCE Factor		**Suggested = 2.00**		2.00
Critical Lane Volume		1600		

Equivalent Passenger Car Volume

	Volume (pc/hr)			
	U-Turn / Left	Through	Right	Approach
Eastbound	208	1663	32	1903
Westbound	29	1286	189	1504
Northbound	16	0	41	57
Southbound	222	0	120	342

Notes:

Left-turn Adjustment Factor	*Conversion of left-turning vehicles to equivalent through vehicles*
Right-turn Adjustment Factor	*Conversion of right-turning vehicles to equivalent through vehicles*
U-turn Adjustment Factor	*Conversion of U-turning vehicles to equivalent through vehicles*
Truck to PCE Factor	*1 truck = X Passenger Car Equivalents*
Critical Lane Volume Sum Limit	*Saturation value for critical lane volume sum at an intersection*

VDOT Junction Screening Tool

Possible Configurations

Indicate with a "Y" or "N" if each intersection or interchange configuration should or should not be considered. Use the information links for guidance. Then, click the "Show/Hide Configurations button" to hide the worksheets for the configurations that will not be considered.

#	Intersections	Information	Consider?	Justification
	Signalized Intersections			
1	Conventional	-	Y	
2	Bowtie	Link	N	Right-of-way restrictions identified
3	Center Turn Overpass	Link	N	Right-of-way restrictions identified
4	Continuous Green-T	Link	N	Unable to accommodate traffic patterns
5	Echelon	Link	N	Right-of-way restrictions identified
6	Full Displaced Left Turn	Link	N	Right-of-way restrictions identified
7	Median U-Turn	Link	N	Right-of-way restrictions identified
8	Partial Displaced Left Turn	Link	N	Right-of-way restrictions identified
9	Partial Median U-Turn	Link	N	Right-of-way restrictions identified
10	Quadrant Roadway N-E	Link	N	Right-of-way restrictions identified
11	Quadrant Roadway N-W	Link	N	Right-of-way restrictions identified
12	Quadrant Roadway S-E	Link	N	Right-of-way restrictions identified
13	Quadrant Roadway S-W	Link	N	Right-of-way restrictions identified
14	Restricted Crossing U-Turn	Link	N	Right-of-way restrictions identified
15	Single Loop	Link	N	Right-of-way restrictions identified
16	Split Intersection	Link	N	Right-of-way restrictions identified
	Unsignalized Intersections			
17	50 Mini Roundabout	Link	N	Unable to accommodate magnitude of traffic volumes
18	75 Mini Roundabout	Link	N	Unable to accommodate magnitude of traffic volumes
19	Roundabout	Link	Y	
20	Two-Way Stop Control	-	Y	
#	Interchanges	Information	Consider?	Justification
21	Traditional Diamond	Link	N	Right-of-way restrictions identified
22	Contraflow Left	Link	N	Right-of-way restrictions identified
23	Displaced Left Turn	Link	N	Right-of-way restrictions identified
24	Diverging Diamond	Link	N	Right-of-way restrictions identified
25	Double Roundabout	Link	N	Right-of-way restrictions identified
26	Michigan Urban Diamond	Link	N	Right-of-way restrictions identified
27	Partial Cloverleaf	Link	N	Right-of-way restrictions identified
28	Single Point	Link	N	Right-of-way restrictions identified
29	Single Roundabout	Link	N	Right-of-way restrictions identified

VDOT Junction Screening Tool

Directional Questions and Base Lane Configurations

Before entering a base number of through lanes for each direction, answer all applicable directional question for each intersection or interchange configuration selected for consideration. Navigate to the lane configuration worksheet for example diagrams, if provided.

Intersections	Question	Direction
Bowtie	N/A	N/A
Continuous Green-T	N/A	N/A
Echelon	N/A	N/A
Median U-Turn	N/A	N/A
Partial Displaced Left Turn	N/A	N/A
Partial Median U-Turn	N/A	N/A
Restricted Crossing U-Turn	N/A	N/A
Single Loop	N/A	N/A
Split Intersection	N/A	N/A
Interchanges	**Question**	**Direction**
All	N/A	N/A

Base Number of Through Lanes

Enter a base number of through lanes for each direction. The number of through lanes entered will populate on each non-roundabout lane configuration worksheet. This tool also allows the user to enter the number of through lanes on the lane configuration worksheets directly. This base number may be overwritten on individual lane configuration worksheets. Turn lanes, shared lanes, and channelized lanes must still be entered in each lane configuration worksheet.

Eastbound	3
Westbound	2
Northbound	1
Southbound	1

VDOT Junction Screening Tool

Results Worksheet

General Information	
Project Title:	Leesburg Pike and Chestnut Street/Commons Drive
EW Facility:	Leesburg Pike
NS Facility:	Chestnut Street/Commons Drive
Date:	August 4, 2020

Volumes (veh/hr)	U-Turn / Left	Through	Right
Eastbound	204	1630	31
Westbound	28	1261	185
Northbound	16	0	40
Southbound	218	0	118

General Instructions: All intersection and interchange configurations have a default assumption of one exclusive lane per movement. No results shall be interpreted until the user has verified the lane configurations on each worksheet.

Intersection Results

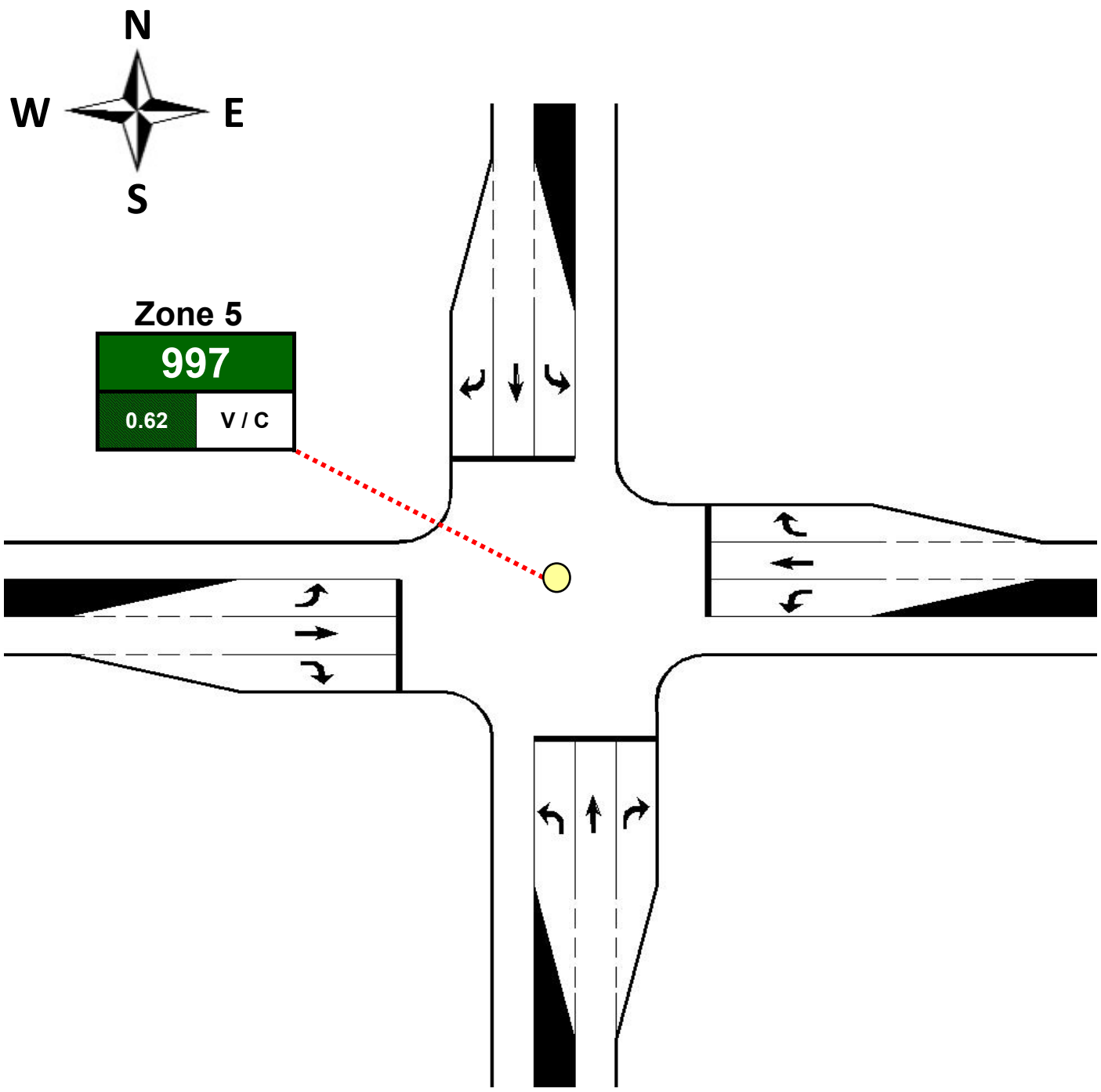
		Congestion	Pedestrian	Safety	Notes
Type	**Dir**	**Maximum V/C**	**Accommodation Compared to Conventional**	**Weighted Total Conflict Points**	
Conventional	-	0.62		48	
Roundabout	-	0.97		8	
Two-Way Stop Control	-	28.73		48	

Information	
Congestion	The maximum v/c ratio represents the worst v/c of all zones that make up an intersection.
Pedestrian	Compares the potential of each design to accommodate pedestrians based on safety, wayfinding, and delay. Potential is qualitatively defined as better (+), similar (blank cell), or worse (-) than a conventional intersection or traditional diamond interchange.
Safety	Weighted Total = (2 x Crossing Conflicts) + Merging Conflicts + Diverging Conflicts

Conventional

DESIGN AND RESULTS

		Critical Lane Volume Sum			
Project Name:	*Leesburg Pike and Chestnut Street/Commons Drive*	< 1200	1200 - 1399	1400 - 1599	≥ 1600
EW Facility:	*Leesburg Pike*				
NS Facility:	*Chestnut Street/Commons Drive*	VOLUME / CAPACITY RATIO:		0.62	
Date:	*August 4, 2020*				

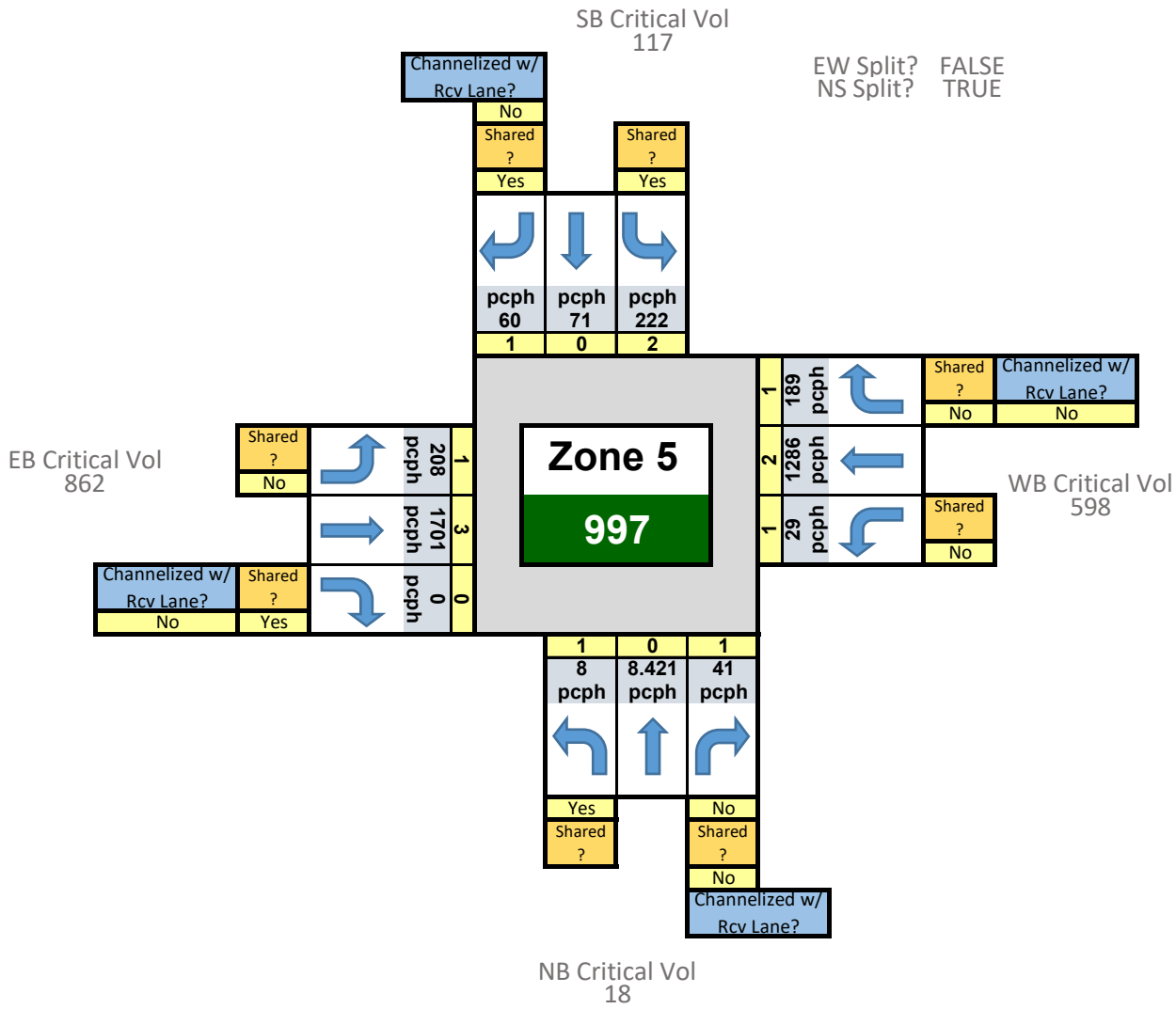


Note: This diagram does not reflect the actual lane configuration of the intersection

Conventional

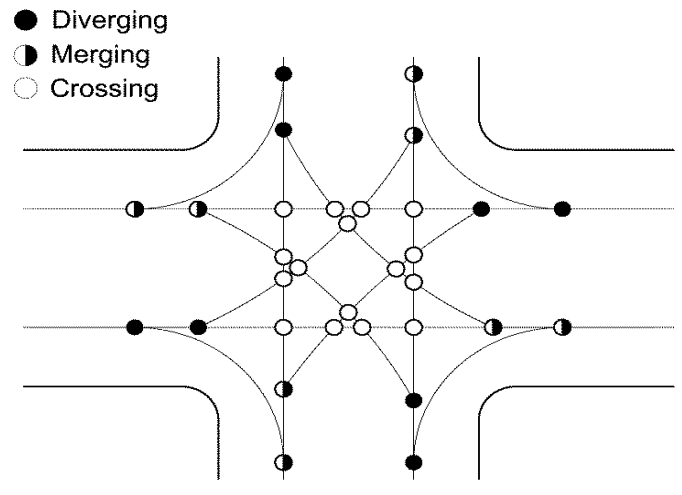
DATA INPUT AND CONFIGURATION

Enter the lane configurations in the yellow cells.



Back to Results

Safety - Conflict Point Diagram



Conflict Type	Count
Crossing	16
Merging	8
Diverging	8
Total	**32**

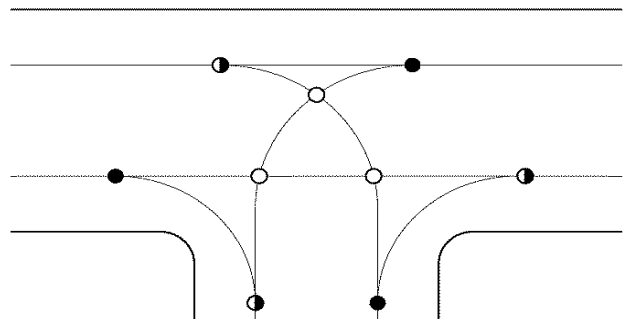
Conflict Type	Weight
Crossing	2
Merging	1
Diverging	1

Weighted Total Conflict Points

48

Safety - Conflict Point Diagram (Three Legs)

● Diverging
◑ Merging
○ Crossing



Conflict Type	Count
Crossing	3
Merging	3
Diverging	3
Total	**9**

Conflict Type	Weight
Crossing	2
Merging	1
Diverging	1

Weighted Total Conflict Points

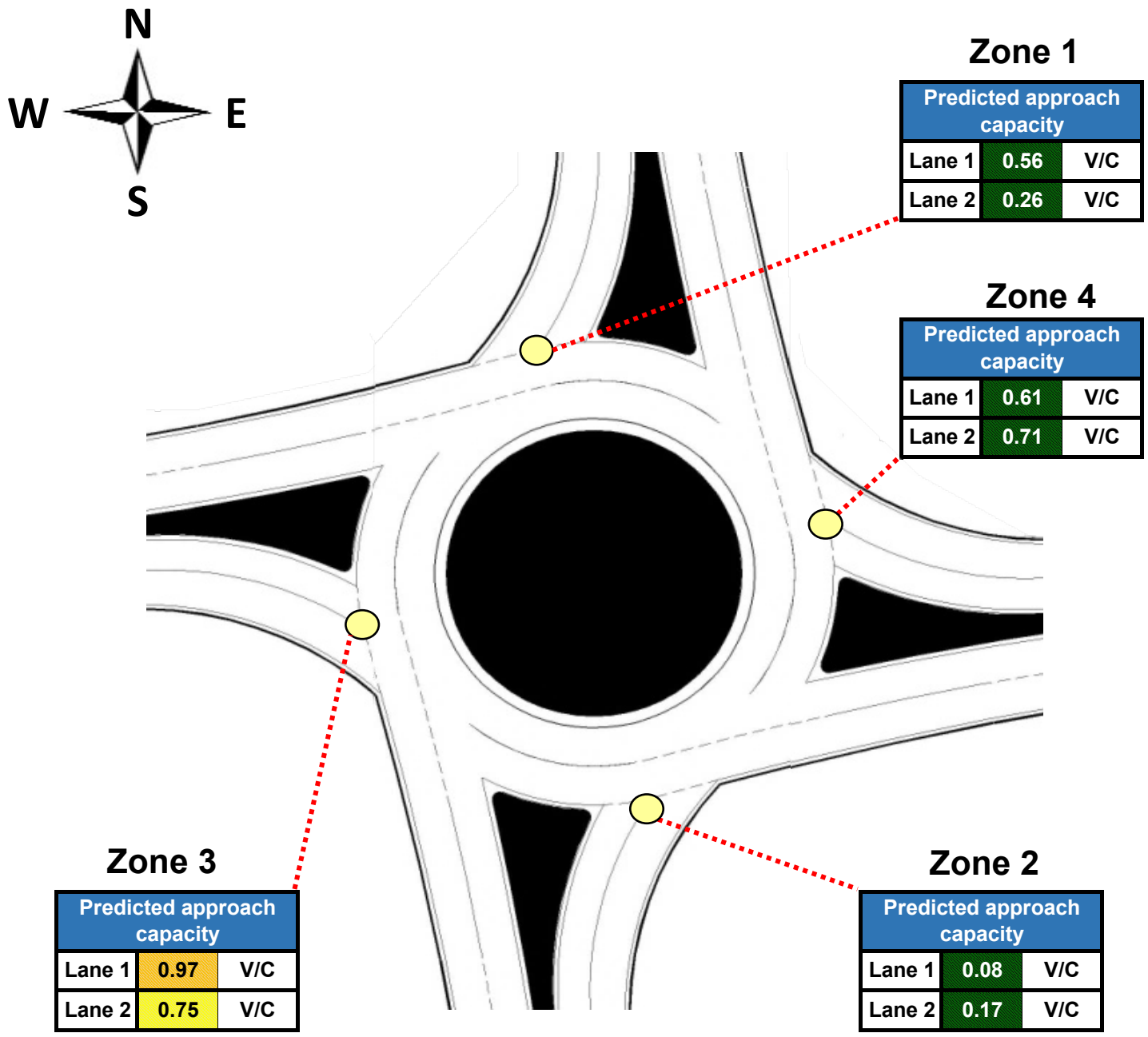
12

Roundabout

DESIGN AND RESULTS

Project Name:	*esburg Pike and Chestnut Street/Commons Dr*
EW Facility:	*Leesburg Pike*
NS Facility:	*Chestnut Street/Commons Drive*
Date:	*August 4, 2020*

Critical Lane Volume Sum			
< 1200	**1200 - 1399**	**1400 - 1599**	**≥ 1600**
VOLUME / CAPACITY RATIO:	**0.97**		



Zone 1

Predicted approach capacity		
Lane 1	0.56	V/C
Lane 2	0.26	V/C

Zone 4

Predicted approach capacity		
Lane 1	0.61	V/C
Lane 2	0.71	V/C

Zone 3

Predicted approach capacity		
Lane 1	0.97	V/C
Lane 2	0.75	V/C

Zone 2

Predicted approach capacity		
Lane 1	0.08	V/C
Lane 2	0.17	V/C

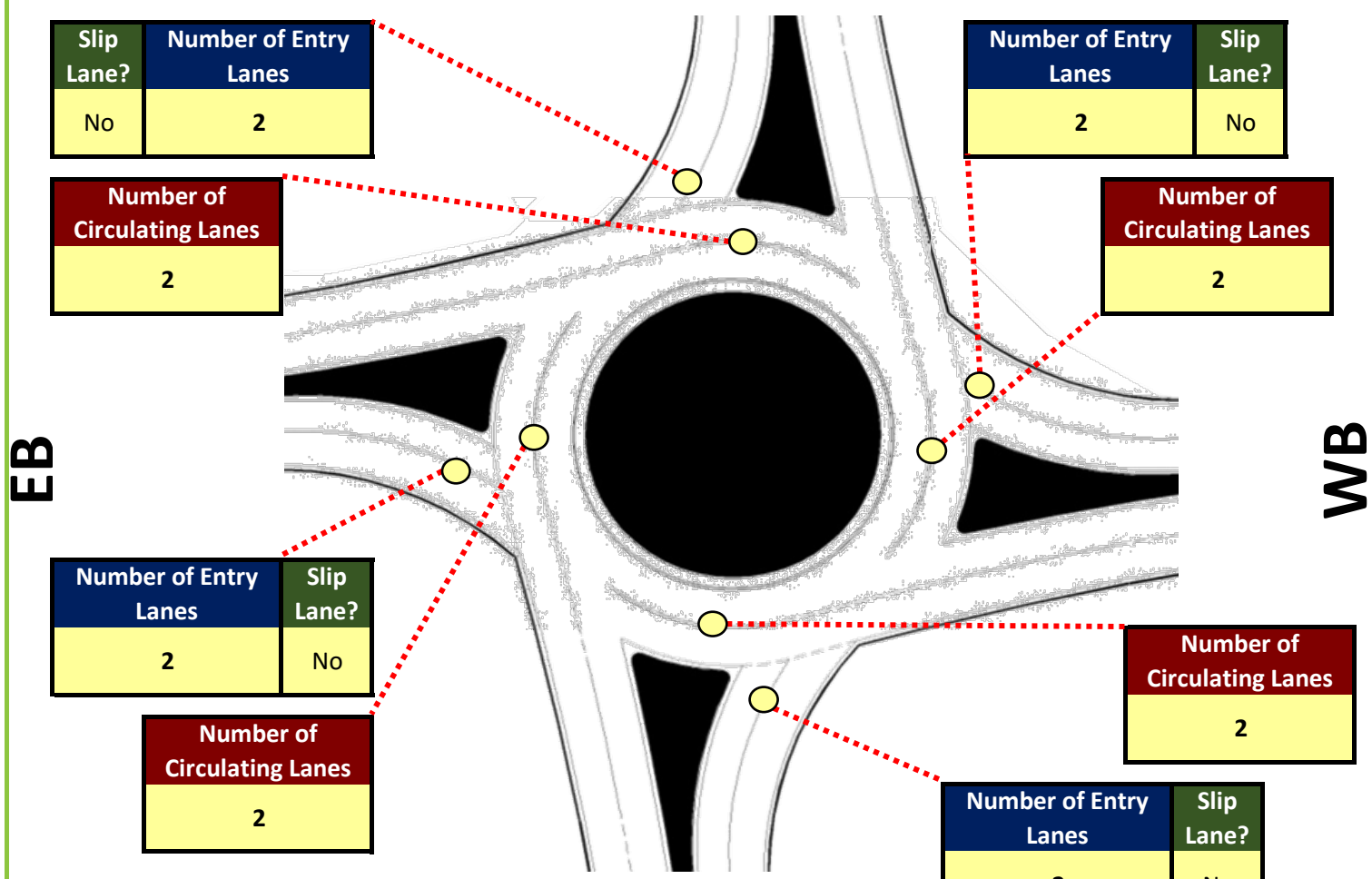
Roundabout

DATA INPUT AND CONFIGURATION

Enter the lane configurations in the yellow cells.

pcph	pcph	pcph
120	0	222

189	1286	29
pcph	pcph	pcph



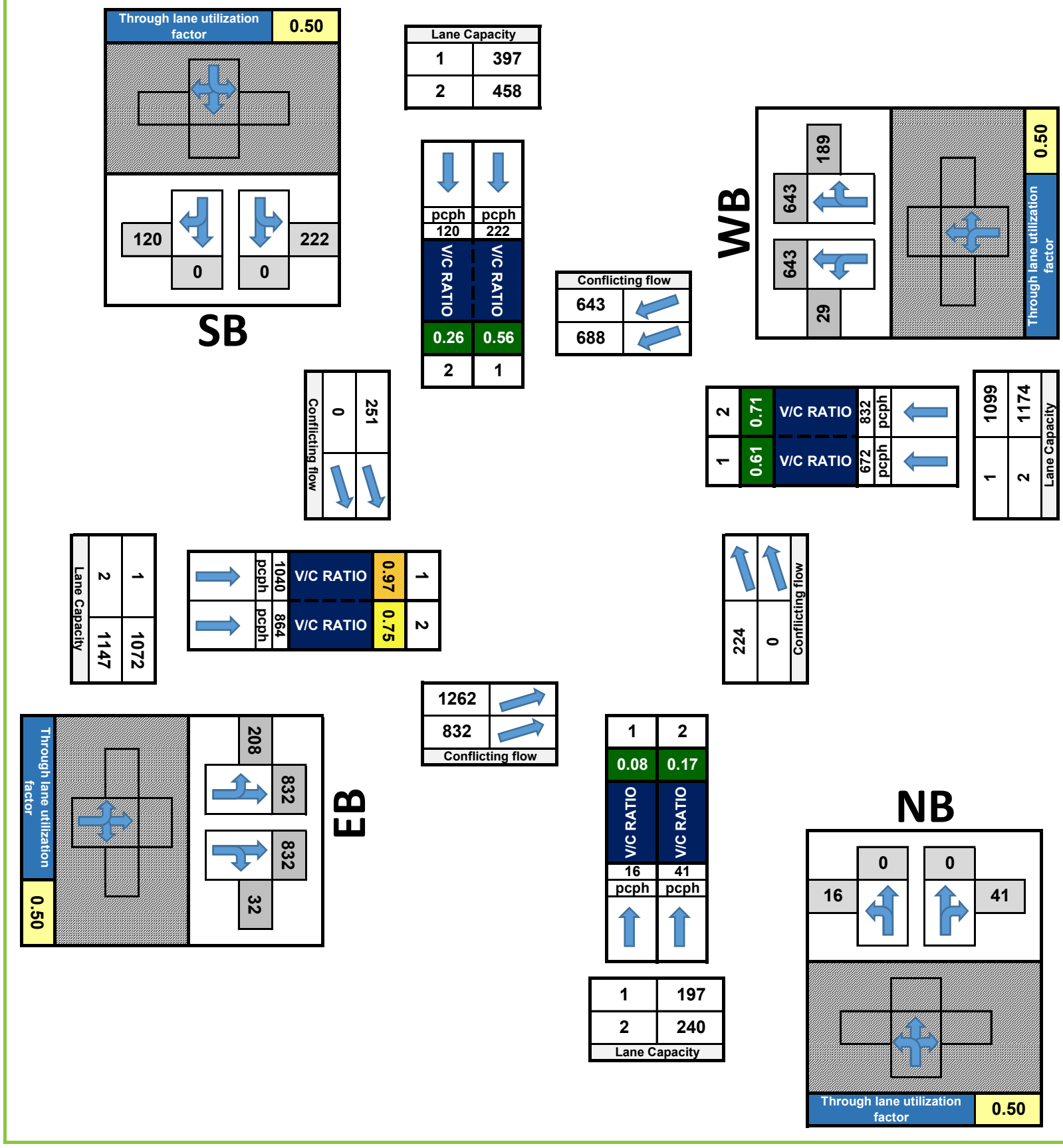
208	1663	32
pcph	pcph	pcph

16	0	41
pcph	pcph	pcph

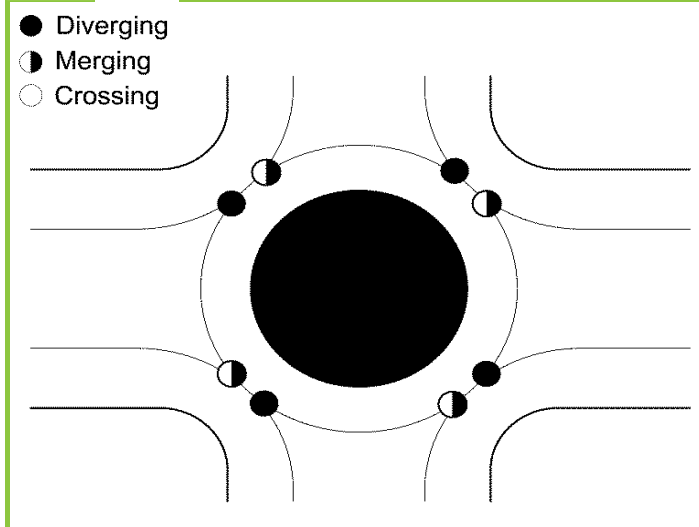
Back to Results

Roundabout

CAPACITY CALCULATIONS



Safety - Conflict Point Diagram



Conflict Type	Count
Crossing	0
Merging	4
Diverging	4
Total	**8**

Conflict Type	Weight
Crossing	2
Merging	1
Diverging	1

Weighted Total Conflict Points

8

Assumptions

● The number of circulating lanes in one quadrant is assumed to be equal to the number of exiting lanes in the next quadrant.

● The roundabout is limited to a maximum of two entry lanes and two circulating lanes.

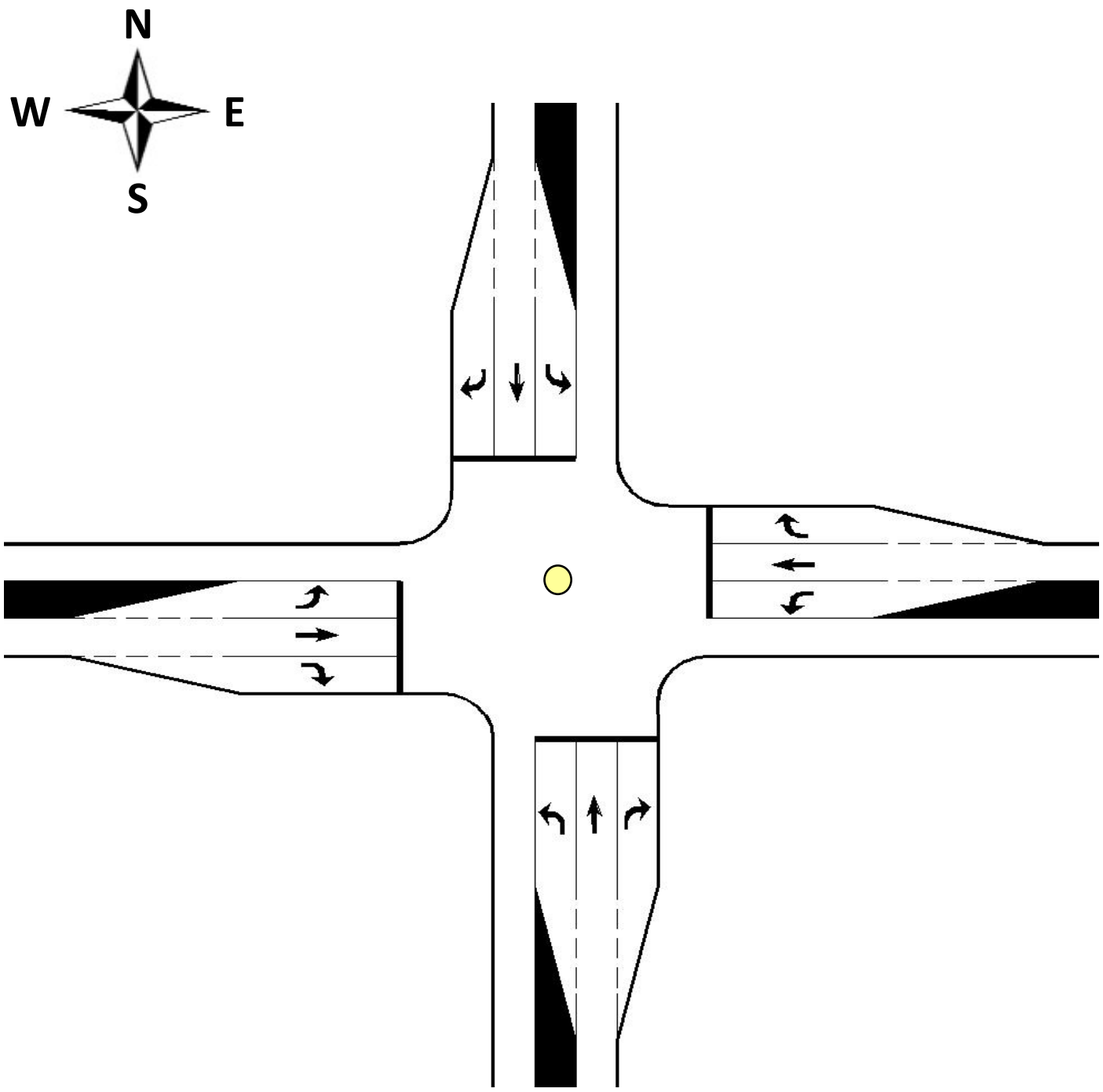
● All left-turning vehicles are assumed to stay in the innermost lane until exiting the roundabout.

● This worksheet does not use the CLV methodology. The calculations are based on the *HCM 6th Edition* .

Two-Way Stop Control (TWSC)

DESIGN AND RESULTS

Project Name:	*Leesburg Pike and Chestnut Street/Commons Drive*	***Critical Lane Volume Sum***			
EW Facility:	*Leesburg Pike*	**< 1200**	**1200 - 1399**	**1400 - 1599**	**≥ 1600**
NS Facility:	*Chestnut Street/Commons Drive*	**VOLUME / CAPACITY RATIO:**		**28.73**	
Date:	*August 4, 2020*				



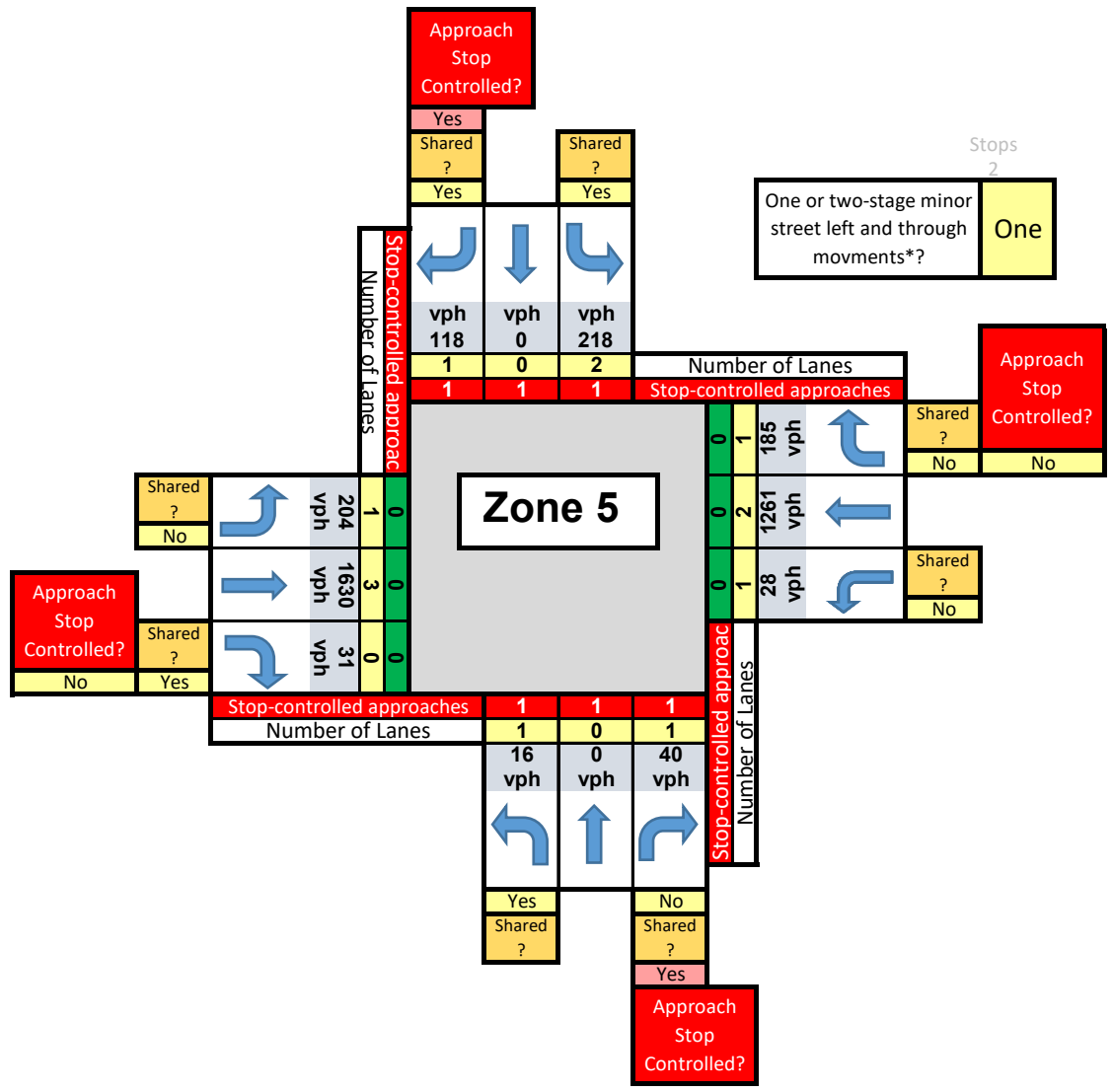
Note: This diagram does not reflect the actual lane configuration of the intersection

Two-Way Stop Control (TWSC)

DATA INPUT AND CONFIGURATION

Step 1: Identify which approaches are stop-controlled by selecting "Yes" from the drop-down box.

Step 2: Enter the lane configurations in the yellow cells.



Back to Results

Two-Way Stop Control (TWSC)

HCM 6 CALCULATIONS

Priority	MVMT	Rank
1	EBL	2
2	EBT	1
3	EBR	1
4	WBL	2
5	WBT	1
6	WBR	1
7	NBL	4
8	NBT	3
9	NBR	2
10	SBL	4
11	SBT	3
12	SBR	2

MAJOR	MINOR
EB	NB
WB	SB

Major street lanes	5
M1 Shared?	FALSE
M4 Shared?	FALSE

Priority	Rank	Flow Rates	Lanes	Shared?	Stop controlled?	Truck %
1	2	204	1	No		0.02
4	2	28	1	No		0.02
7	4	16	1	Yes	Yes	0.02
8	3	0	0		Yes	0.02
9	2	40	1	No	Yes	0.02
10	4	218	2	Yes	Yes	0.02
11	3	0	0		Yes	0.02
12	2	118	1	Yes	Yes	0.02
2	1	1630	3			0.02
3	1	31	0	Yes	No	0.02
5	1	1261	2			0.02
6	1	185	1	No	No	0.02

Conflicting Flows	
$v_{c,1}$	1446.00
$v_{c,4}$	1661.00
$v_{c,7}$	2740.00
$v_{c,8}$	3555.50
$v_{c,9}$	830.50
$v_{c,10}$	2377.00
$v_{c,11}$	3386.00
$v_{c,12}$	630.50
$v_{c,I,7}$	2053.50
$v_{c,II,7}$	686.50
$v_{c,I,8}$	2053.50
$v_{c,II,8}$	1502.00
$v_{c,I,10}$	1317.00
$v_{c,II,10}$	1060.00
$v_{c,I,11}$	1317.00
$v_{c,II,11}$	2069.00

Critical Headways	
$t_{c,1}$	5.34
$t_{c,4}$	5.34
$t_{c,7}$	6.44
$t_{c,8}$	6.54
$t_{c,9}$	7.14
$t_{c,10}$	6.44
$t_{c,11}$	6.54
$t_{c,12}$	7.14
$t_{c,I,7}$	7.34
$t_{c,II,7}$	6.74
$t_{c,I,8}$	5.54
$t_{c,II,8}$	5.54
$t_{c,I,10}$	7.34
$t_{c,II,10}$	6.74
$t_{c,I,11}$	5.54
$t_{c,II,11}$	5.54

Follow-Up Headways	
$t_{f,1}$	3.12
$t_{f,4}$	3.12
$t_{f,7}$	3.82
$t_{f,8}$	4.02
$t_{f,9}$	3.92
$t_{f,10}$	3.82
$t_{f,11}$	4.02
$t_{f,12}$	3.92

Poten Capac
$c_{p,1}$
$c_{p,4}$
$c_{p,7}$
$c_{p,8}$
$c_{p,9}$
$c_{p,10}$
$c_{p,11}$
$c_{p,12}$

Two-Stage Capaci
$c_{p,I,7}$
$c_{p,II,7}$
$c_{p,I,8}$
$c_{p,II,8}$
$c_{p,I,10}$
$c_{p,II,10}$
$c_{p,I,11}$
$c_{p,II,11}$

y_7
y_8
y_{10}
$y11$

Mvmt 1,
Mvmt 4,

Mvmt 1, sh
Mvmt 4, sh

Mvmt 7
Mvmt 1(

a	0.91

***Assumption:**	One storage space in median (n_m = 1) for two-stage turns

Saturation Flow Rates	
Through	1800
Right	1500

ıtial ities
236.97
185.29
21.55
5.68
268.74
36.78
7.38
363.51

Movement Capacities	
$c_{m,1}$	236.97
$c_{m,4}$	185.29
$c_{m,7}$	3.57
$c_{m,8}$	0.67
$c_{m,9}$	268.74
$c_{m,10}$	7.67
$c_{m,11}$	0.87
$c_{m,12}$	363.51

Shared Movement Capacities	
1	3.57
1	
0	
1	11.69
1	
1	

Movement Capacities	
1	236.97
2	5400.00
3	1500.00
4	185.29
5	3600.00
6	1500.00
--	--
7-8	3.57
9	268.74
--	--
10-11-12	11.69
--	--

Movement V/C	
1	0.86
2	0.30
3	0.02
4	0.15
5	0.35
6	0.12
--	--
7-8	4.49
9	0.15
--	--
10-11-12	28.73
--	--

Intersection V/C
28.73

V/C Not Reported for Any Movements?
No

Potential ities
35.18
367.00
96.89
183.10
119.33
215.75
225.31
95.14

Two-Stage Movement Capacities	
$c_{m,I,7}$	4.90
$c_{m,II,7}$	210.41
$c_{m,I,8}$	13.48
$c_{m,II,8}$	155.43
$c_{m,I,10}$	101.30
$c_{m,II,10}$	25.55
$c_{m,I,11}$	191.26
$c_{m,II,11}$	13.24

Single-Stage Movement Capacities	
$c_{m,7}$	3.57
$c_{m,8}$	0.67
$c_{m,10}$	7.67
$c_{m,11}$	0.87

0.47
-0.26
-9.25
-12.18

$c_{T,7}$	4.08
$c_{T,8}$	16.42
$c_{T,10}$	0.00
$c_{T,11}$	0.00

excl left	$p_{0,1}$	0.14
excl left	$p_{0,4}$	0.85

ıared left	$p^*_{0,1}$	0.00	$p_{0,8}$	1.00	$p_{0,9}$ 0.85
ıared left	$p^*_{0,4}$	0.50	$p_{0,11}$	1.00	$p_{0,12}$ 0.68

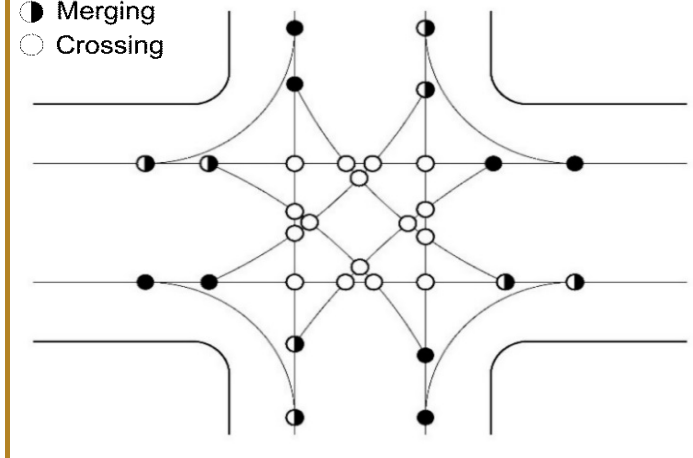
, 4-leg	p''_7	0.118	p'_7	0.25	$f_{p,7}$ 0.17
), 4-leg	p''_{10}	0.118	p'_{10}	0.25	$f_{p,10}$ 0.21

$x_{1i,1+2}$	0.93
$x_{4i,1+2}$	0.70

One Stage	f_8	0.12
	f_{11}	0.12
	f_7	0.00
	f_{10}	0.00

Two Stage	$f_{I,8}$	0.14	$f_{II,8}$	0.85	$p_{0,I,8}$ 1.00
	$f_{I,11}$	0.85	$f_{II,11}$	0.14	$p_{0,I,11}$ 1.00
	$f_{I,7}$	0.14	$f_{II,7}$	0.57	
	$f_{I,10}$	0.85	$f_{II,10}$	0.12	

Safety - Conflict Point Diagram



Conflict Type	Count
Crossing	16
Merging	8
Diverging	8
Total	**32**

Conflict Type	Weight
Crossing	2
Merging	1
Diverging	1

Weighted Total Conflict Points

48

Assumptions

● This worksheet does not use the CLV methodology. The calculations are based on the *HCM, 6th Edition* . The calculations are based on vehicles per hour.