Traffic Analysis an Intersection Redesign at South Amherst Highway and Old Town Connector

A Technical Report
presented to the faculty of the
School of Engineering and Applied Science
University of Virginia

by

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Introduction

Project Background

A senior living community has been proposed in Amherst, VA. It will include commercial space, multi-family apartments, attached single family villas, an independent senior living facility and an assisted senior living and memory care center. The developers of the community plan to use 45 acres of land located to the north and northwest of the intersection of Amherst Highway (VA-163) and Old Town Connector (VA-210). The primary entry point from the existing roadway system to the community will be the intersection of the two aforementioned roads (Figure 1).

The two major responsibilities of this project include providing a traffic analysis of the intersection based on the increased volume due to the senior living community and proposing a redesign of the intersection that optimizes the safety, level of service (LOS), delay time, and queue length while minimizing the cost with corresponding signal timing splits. This report will present said traffic study and intersection redesign.

The information from this analysis will assist the developers in determining the impact of the site onto the existing intersection. The goal is for there to be zero negative or minimal negative impact on the intersection. If there is any negative impact on the intersection, it will be described how this negative impact will be mitigated.

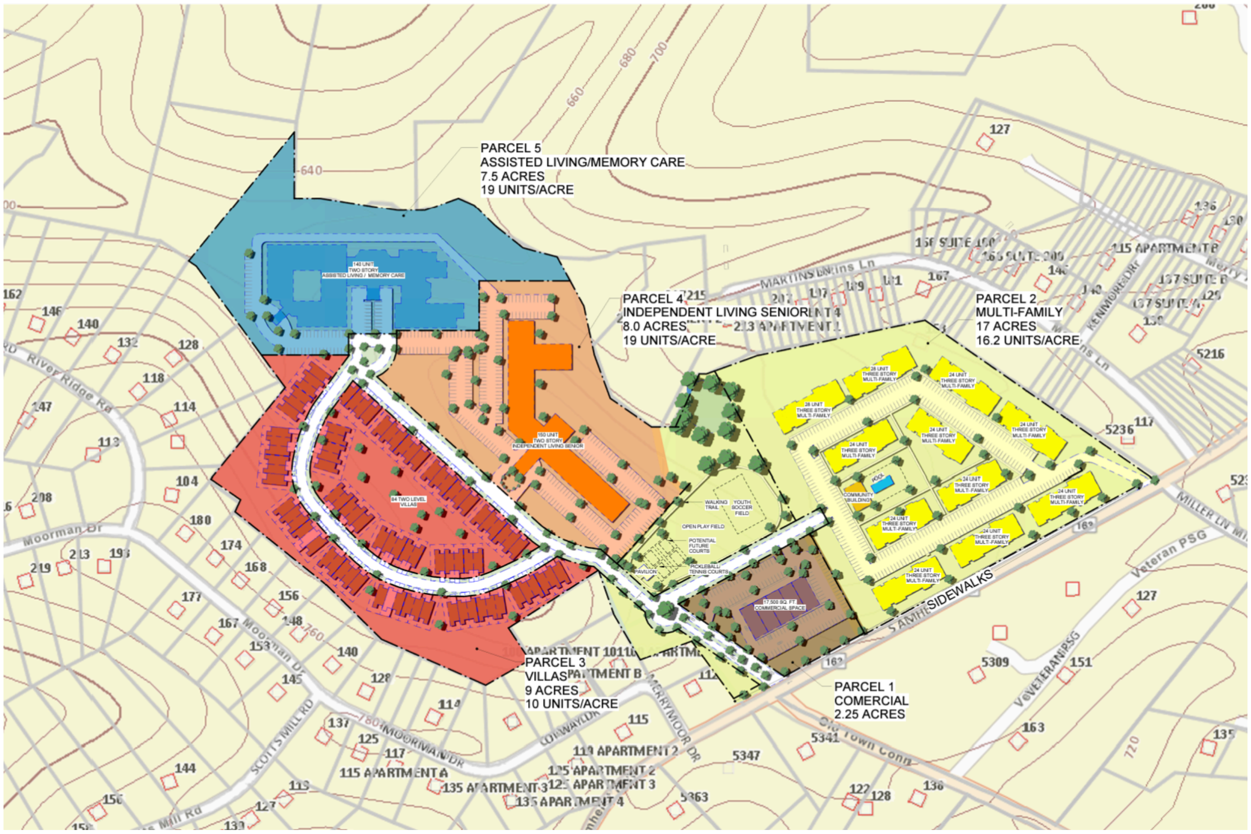

Figure 1: Amherst Senior Living Concept

Project Scope and Measures of Success

The first portion of the project will consist of a traffic analysis of the intersection. The analysis will look at three sets of conditions: the existing conditions and the intersection in 2030 with and without the site. The conditions analyzing the intersection in 2030 will use the word "future" to describe them. Each condition will have at least an AM and PM case. The goal of the analysis is to obtain the LOS, delay time and queue length of the intersection and each of its approaches for each case of each condition to present to the developers. There will also be unique signal timings for each of the 3 conditions. If optimal, the future conditions will have unique AM and PM signal timing splits. Furthermore, it is assumed that the new community will result only in the addition of a 4th leg to the existing intersection, with the exception of new turn lanes into the community, instead of redesigning the entire intersection from scratch to optimize the results of the analysis.

Another measure of success will be documentation of the analysis and a discussion on chosen design. The results of the analysis will be documented in this report. It will include at least a table of the LOS, delay time and queue length of each condition of each approach. Finally, if the inclusion of the site causes any negative impact on the intersection, its mitigation must be detailed.

Before discussing the design itself, however, the report will also compare if the intersection should remain a signalized intersection or be remade into a roundabout. The Virginia Department of Transportation (VDOT) requires consideration of a roundabout when designing new signalized intersections. Both in accordance with this and from the encouragement of mentors, a roundabout will be considered as an alternative design for the intersection. The comparison will look at the results of the analysis for both cases alongside comparing the costs of new traffic signals and an estimated cost of construction of the 4th leg versus the cost of construction of a roundabout. Once that decision is made, the chosen intersection will be designed within PTV Vistro and in accordance with VDOT and AASHTO standards. If the chosen design is a roundabout, then it will follow the process and guidelines detailed in *NCHRP Report 672 — Roundabouts: An Information Guide, 2nd Edition*.

Analysis

Goals and Background

The entire analysis will be done using the software PTV Vistro and calculations in Google Sheets. The goal of the analysis is to obtain the delay, LOS and queue length of each case of each condition. The LOS is a grade from ‘A’ to ‘F’ with ‘A’ as the highest grade and ‘F’ as the lowest grade assigned to either a singular approach to the intersection, or the intersection in its entirety. A higher grade indicates better operating conditions, with a grade of "D" being the minimum for the approach or intersection to be considered operational. There are qualitative and quantitative ways to assign meanings to the LOS grades (Tables 1 & 2).

Table 1: Qualitative Interpretation of LOS Grades and Quantitative Definitions of them in terms of V/C Ratio for Signalized Intersections from San Mateo County in California (C/CAG, 2005)

Intersection Level of Service Definitions

Level of Service	Interpretation	V/C Ratio
A	Uncongested operations; all queues clear in a single signal cycle.	Less Than 0.60
B	Very light congestion; an occasional approach phase is fully utilized.	0.60 to 0.69
C	Light congestion; occasional backups on critical approaches.	0.70 to 0.79
D	Significant congestion on critical approaches, but intersection functional. Cars required to wait through more than one cycle during short peaks. No long-standing queues formed.	0.80 to 0.89
E	Severe congestion with some long-standing queues on critical approaches. Blockage of intersection may occur if traffic signal does not provide for protected turning movements. Traffic queue may block nearby intersections(s) upstream of critical approach(es).	0.90 to 0.99
F	Total breakdown, stop-and-go operation.	1.00 and Greater

The delay, measured in seconds, is how long the average vehicle is stopped at the approach of the intersection. There are 3 components to delay: uniform delay, incremental delay and initial queue delay. The summation of these results in the control delay, which is used to quantitatively measure the LOS of an approach or intersection (Table 2). Once again, something needs to be at a grade 'D' or better to be considered operational.

Table 2: LOS Grade Definitions for Signalized Intersections from the Highway Capacity Manual in terms of Control Delay per Vehicle (TRB, 2000)

EXHIBIT 16-2. LOS CRITERIA FOR SIGNALIZED INTERSECTIONS

LOS	Control Delay per Vehicle (s/veh)
A	≤ 10
B	> 10–20
C	> 20–35
D	> 35–55
E	> 55–80
F	> 80

The queue length, measured in feet, is the length from the front of the first car in the queue to the end of the last one while the light is red within one cycle. This will be determined and measured in PTV Vistro.

Methodology

The intersection was created in PTV Vistro and duplicated for each condition. The 4th leg was added for the future condition with the site. The traffic counts, peak hour factor, heavy vehicle percentages, and signal timing planning sheet were all provided before the start of the analysis. All of this information can be found in Appendix D. If any of the provided values were below the minimum design values in the Traffic Operations and Safety Analysis Manual (TOSAM), then they were increased to them. The two values changed were the peak hour factor and the heavy vehicle percentages. The minimum values for each are 0.88 and 2% respectively.

The provided information was enough to complete the existing conditions, but for the future conditions, traffic growth needed to be considered. To compute them, traffic growth was observed from 2011 to 2022 at the intersection, and the growth rate over time was computed. The years 2020 and 2021 were excluded due to the COVID-19 pandemic. Furthermore, the TOSAM minimum design growth rate is 0.5%. Similar to the provided values, if the calculated growth rate was below 0.5%, it was increased to it.

For the condition with the new site, the trips generated from the new site needed to be added to the traffic values used for the future condition without the site. This was done using the ITE Trip Generation manual. The manual takes what the land is going to be used for (single family housing, office space, assisted living, etc.) and assigns it to a land use code. Then, it looks at previous traffic studies conducted with the numbers of trips generated versus the number of dwellings and generates an average rate, a line of best fit equation, and an r-squared value for each land use code. If the r-squared value is above 0.75, then the line of best fit equation is used to determine the trips. If it is lower than 0.75, the average rate is used instead. The assigned land use codes, equations and rates for each, can be found in Appendix D.

Table 3: Trip Generation Calculations

Trip Generation						
	Land Use Code	Descriptor	Dwellings	Daily Traffic	# of Trips In	# of Trips Out
Parcel 1	710	General Office Building	17500	255	127	127
Parcel 2	220	Multifamily Housing (Low-Rise) Not Close to Rail Transit	272	1819	909	909
Parcel 3	215	Single-Family Attached Housing	84	691	345	345
Parcel 4	252	Senior Adult Housing - Multifamily	150	458	229	229
Parcel 5	254	Assisted Living	140	364	182	182
	Peak AM Traffic	# of Trips In	# of Trips Out	Peak PM Traffic	# of Trips In	# of Trips Out
Parcel 1	37	33	4	39	7	32
Parcel 2	107	26	81	138	87	51
Parcel 3	38	9	28	46	27	19
Parcel 4	29	10	19	38	21	17
Parcel 5	25	15	10	34	13	20
	Total AM Trips:	93	144	Total PM Trips:	155	139

The trips were then distributed amongst the approaches entering and the approach exiting the site. For the trips entering the site, the future volumes of the other approaches were used. The total volume of each approach was added together and proportions of each approach's volume to the total volume of the intersection before the site were calculated from them. These proportions were multiplied by the number of trips entering the site to determine the traffic of each approach into the site. These trips became the volumes of the 3 approaches entering the site. For the trips exiting the site, a similar method was used. Instead of the individual approaches being used, the number of vehicles going in a specific direction were used instead. For example, instead of the westbound approach being used, the number of vehicles going west was used instead. The same method of adding up volumes and creating proportions from them was used. These trips became the volumes of the new southbound approach. Visual representations of the Trip Distributions can be found in Appendix D.

Table 4: AM Trip Distribution

Entering			
Approach	Total Volume of Approach	Proportion of Approach's Volume to Total Volume	Volume Entering Site from Approach
Eastbound	340	0.35602	33
Northbound	212	0.22199	21
Westbound	403	0.42199	39
Total	955	1.00	
Exiting			
Direction Headed	Total Volume of Direction	Proportion of Direction's Volume to Total Volume	Volume Exiting Site in Direction
West	577	0.60419	87
East	223	0.23351	34
South	155	0.16230	23
Total	955	1.00	

Table 5: PM Trip Distribution

Entering			
Approach	Total Volume of Approach	Proportion of Approach's Volume to Total Volume	Volume Entering Site from Approach
Eastbound	729	0.59803	93
Northbound	160	0.13126	20
Westbound	330	0.27071	42
Total	1219	1.00	
Exiting			
Direction Headed	Total Volume of Direction	Proportion of Direction's Volume to Total Volume	Volume Exiting Site in Direction
West	431	0.35357	49
East	513	0.42084	58
South	275	0.22559	31
Total	1219	1.00	

From there, the specific cases for each condition were developed and considered. From the signal timing sheet, the cycle length for the existing condition was determined to be 90 seconds. In addition, it was known that the intersection did not have unique signal timings for the AM and PM peak hours. Additional cases were made for the AM and PM cases to determine if the AM or PM signal timing split was ruling one. For both future conditions, the limitations of both the initial 90 second cycle length and the same splits being used all day no longer had to be under consideration. That means cases for separate AM and PM timing splits and different cycle lengths were used instead. Webster's optimal cycle length equation was used to determine the optimal cycle length for each future condition and time of day case. That was compared to the base 90 second cycle length from the initial signal timing sheet. All of this resulted in 12 cases total, 4 for each of the conditions. The future condition with the site was split into two additional cases as well, an AM and PM case for a roundabout. They will be represented separately in the results just like every other condition.

While developing the splits within PTV Vistro, there were discrepancies between the saturation flow rates calculated in the software and what was calculated within Google Sheets using equations from the Highway Capacity Manual. There were also discrepancies with balancing the splits within PTV Vistro and some other factors. This resulted in shifting the focus of the project from doing everything within PTV Vistro to calculating the saturation flow rates and green time splits in sheets and inputting them into PTV Vistro. The results from PTV Vistro were compared to what was calculated on sheets to ensure both of their accuracies. All of the calculations using sheets can be found in Appendix D. The associated equations and PTV Vistro's output values may also be found there. The tabulated results will contain the values outputted by PTV Vistro.

Results

Table 4: Existing Condition Using the AM Timing Splits

Existing Conditions AM Timing Plan				
	Time of Day	**Delay (s)**	**LOS**	**Queue Length (ft)**
Intersection	AM	18.43	B	--
Intersection	PM	40.24	D	--
Approach				
Northbound	AM	31.53	C	200.1
Eastbound	AM	19.81	B	237.0
Westbound	AM	10.29	B	174.3
Northbound	PM	28.97	C	146.6
Eastbound	PM	57.43	E	726.4
Westbound	PM	9.39	A	125.8

Table 5: Existing Condition Using the PM Timing Splits

Existing Conditions PM Timing Plan				
	Time of Day	Delay	LOS	Queue Length (ft)
Intersection	AM	26.56	C	--
Intersection	PM	20.68	C	--
Approach				
Northbound	AM	93.48	F	323.5
Eastbound	AM	11.10	B	169.5
Westbound	AM	4.74	A	93.6
Northbound	PM	57.47	E	211.5
Eastbound	PM	19.97	B	427.0
Westbound	PM	4.28	A	68.4

Table 6: Future Condition Without the Site Using Webster's Optimal Cycle Length Formula

Future Conditions w/out Site (With Websters Optimal Cycle Length Formula)				
	Time of Day	Delay (s)	LOS	Queue Length (ft)
Intersection (50 s)	AM	19.04	B	--
Intersection (80 s)	PM	24.18	C	--
Approach				
Northbound	AM	32.78	C	157.2
Eastbound	AM	24.60	C	201.7
Westbound	AM	6.93	A	83.4
Northbound	PM	63.45	E	221.1
Eastbound	PM	24.64	C	460.5
Westbound	PM	4.16	A	64.9

Table 7: Future Condition Without the Site Using a Cycle Length of 90 Seconds

Future Conditions w/out Site (With 90 sec. Cycle Length from Signal Timing Sheet)				
	Time of Day	Delay (s)	LOS	Queue Length (ft)
Intersection	AM	18.72	B	--
Intersection	PM	22.51	C	--
Approach				
Northbound	AM	32.00	C	203.6
Eastbound	AM	20.17	C	246.4
Westbound	AM	10.45	B	183.2
Northbound	PM	61.37	E	228.1
Eastbound	PM	22.19	C	471.5
Westbound	PM	4.35	A	71.8

Table 8: Future Condition With the Site Using Webster's Optimal Cycle Length Formula

Future Conditions w/out Site (With Websters Optimal Cycle Length Formula)				
	Time of Day	Delay (s)	LOS	Queue Length (ft)
Intersection (65 s)	AM	28.32	C	--
Intersection (75 s)	PM	27.12	C	--
Approach				
Northbound	AM	31.18	C	161.7
Southbound	AM	38.62	D	119.3
Eastbound	AM	23.86	C	141.1
Westbound	AM	27.22	C	247.8
Northbound	PM	43.92	D	166.8
Southbound	PM	49.24	D	144.8
Eastbound	PM	22.76	C	279.1
Westbound	PM	19.95	B	178.4

Table 9: Future Condition With the Site Using a Cycle Length of 90 Seconds

Future Conditions w/out Site (With 90 sec. Cycle Length from Signal Timing Sheet)				
	Time of Day	Delay (s)	LOS	Queue Length (ft)
Intersection	AM	31.69	C	--
Intersection	PM	29.12	C	--
Approach				
Northbound	AM	33.87	C	201.4
Southbound	AM	39.27	D	139.9
Eastbound	AM	28.68	C	183.4
Westbound	AM	30.59	C	305.5
Northbound	PM	46.85	D	184.6
Southbound	PM	52.77	D	163.3
Eastbound	PM	24.52	C	311.2
Westbound	PM	21.45	C	199.1

Table 10: Future Condition as a Roundabout

Roundabout				
	Time of Day	Delay (s)	LOS	Queue Length (ft)
Intersection	AM	7.59	A	--
Intersection	PM	13.16	B	--
Approach				
Northbound	AM	6.50	A	26.5
Southbound	AM	8.23	A	22.7
Eastbound	AM	6.30	A	37.4
Westbound	AM	9.13	A	64.8
Northbound	PM	9.42	A	32.6
Southbound	PM	6.91	A	21.3
Eastbound	PM	17.35	C	212.1
Westbound	PM	8.43	A	53.5

Discussion

The northbound approach is for Old Town Connector. The eastbound and westbound approaches are for South Amherst Highway. The eastbound approach is going toward Lynchburg and the westbound approach is going away from it. The southbound approach is for the new site and is only present in the conditions where the site is being considered.

While the northbound approach has a shorter delay, better LOS and shorter queue length in the AM, the westbound and eastbound approaches have shorter delays, better LOSs and shorter queue lengths in the PM. While comparing the delays for the both timing splits in their respective entireties, the PM timing plan's total is 23.3 seconds compared to the AM's 30.5 seconds. Therefore, in the remainder of the analysis, when referring to the existing conditions, the PM timing splits will be used.

Even in its existing conditions, the intersection has some congestion and delay problems. While the intersection is still functional according to the definitions in Tables 1 & 2, there is a lot of room for improvement. Even when balancing, the Northbound approach has long delay times and an inoperational LOS grade. With the large volume of the westbound approach, a lot of time is allotted for it to clear its queue, giving less time for the Northbound approach to do the same. This is the primary contributor to the lengthy delay time and queue length.

Despite Webster's optimal cycle length formula being used to determine the optimal cycle lengths for the AM and PM existing future conditions, the delays and LOSs for the cycle length of 90 seconds are slightly better than the optimized ones. This is because, when optimizing, the green split for the left turn South Amherst Highway to Old Town Connector is lower than the minimum green time on the original timing plan sheet. Since time from the other splits had to be reallocated to this once so it could reach the minimum, the other splits had to be recalculated, thus making the intersection no longer the optimal timing. Even though this only resulted in minor changes in the outputs, it was still enough to be slightly less operational than at 90 seconds.

Since the cycle length of 90 seconds has the prevailing splits, this will be used for the remainder of the discussion about the future condition without the site. The futures cases have the luxury of not needing to have one timing plan for the entire day. This improves the overall delay time and LOS of the intersections compared to the existing conditions. That being said, since the amount of growth from now until 2030 is low (all approaches have a growth rate <1%), the results from the future conditions without the site are very similar to the existing conditions. The delay and queue lengths are only slightly greater. This makes sense as this condition follows the same cycle length with a slightly higher volume, resulting in the intersection to have a slightly tougher ability to clear its queues.

For the analysis of the future conditions with the site, an additional turn lane was added to each existing approach to both help optimize the signal timing and help decrease delays and queue lengths. The new approach was given two lanes to follow the same principle. Unlike the previous condition, the splits produced with Webster's optimal cycle length formula are the ones with better outputs. That will be used for the remainder of the discussion about the future condition with the site.

The inclusion of the 4th leg of the intersection slightly decreased the signalized intersection's overall functionality. Both of South Amherst Highway's approaches saw longer delays. The westbound approach, in particular, saw longer queue lengths and a worse LOS grade as well. This is a result of the westbound through lane no longer being green during both the westbound left turn and the eastbound through lane phases. The northbound approach does grade out as better, especially in the afternoon where the eastbound approach is not eating up so much of the green time anymore, but it is still worse operationally than the east and westbound approaches. Furthermore, the new southbound approach has a LOS of ‘D’, meaning that its inclusion as a new leg is barely operational.

If kept as a signalized intersection, the inclusion of the new senior living facility does not drastically change the functionality of the intersection, but it does slightly decrease its functionality (Table 12). While all within the same LOS grade of C, there are longer delay times between the existing conditions and the future condition with the site.

Table 12: Intersection Conditions Over Time

Intersection Conditions Over Time		
AM Existing	26.56	C
AM Future No Site	18.72	B
AM Future Site	28.32	C
PM Existing	20.68	C
PM Future No Site	22.51	C
PM Future Site	27.12	C

An opportunity for mitigation comes in the form of the roundabout alternative. It has an ‘A’ grade for the morning and a ‘B’ grade for the afternoon. In addition, each individual approach has a grade of ‘A’ except for one, the eastbound approach in the PM, whose average delay in this case is lower than in any of the other PM cases. All of the other delay times and queue lengths are lowest in this condition versus the other ones as well. The analysis shows that a roundabout would be functionally more operational than a signalized intersection, eliminating the negative impact of the site onto the new intersection.

Design

Design Justification

As mentioned before, the two design alternatives under consideration for the intersection with the inclusion of the new site are a signalized intersection and a roundabout. The comparison will look at aforementioned results, the upfront cost of construction, other long term costs and safety.

Once again, according to the results of the analysis, turning this intersection into a roundabout instead of keeping it a signalized intersection is the better option. The roundabout has much lower delay times and queue lengths, resulting in a better LOS for every approach of the intersection. The lower delay times and queue lengths will keep drivers happy from not having to wait as long and will decrease the likelihood of drivers making poor decisions from impatience or other negative attitudes as a result of waiting.

"Delays can occur due to funding issues or if the state has a backlog of projects. The cost to purchase and install a traffic signal can range from $200,000-$500,000. Annual maintenance expenses are approximately $8,000" (CAT, 2024). "A single-lane roundabout costs roughly $1.2 to $1.8 million to construct while multilane roundabouts can cost more than $2 million each, according to estimates from the Wisconsin Department of Transportation" (Reid, 2021). That being said, costs can vary based on jurisdiction. A similar project in Virginia for the construction of a 3-leg single-lane roundabout has a total upfront cost of $5,400,000. This includes the design, right-of-way and construction phases of the project (LACA, 2023). While roundabouts have a very large upfront cost, "cities or counties no longer have to pay for the annual maintenance, electricity, and supplies for traffic lights at intersections that use roundabouts. And if a storm knocks out power, the roundabout keeps functioning; the city or county no longer needs to deploy police officers to direct traffic through intersections with dead traffic lights" (Reid, 2021). The only long term cost is the cost of maintaining the island in the center.

In order to keep this intersection a signalized one, there are multiple large upfront costs that would have to be made. First, the 3 new right turn lanes would need to be added. Second, since the width of all the existing approaches is increasing, new traffic signal heads, poles and mast arms would need to go on the existing legs in addition to the new one for the new approach. There are also some smaller ones including street name signs, nuts and bolts and other pieces to assemble and fasten the poles and mast arms. The materials aren't the only cost, however, as there is the cost of designing and constructing the intersection. Adding all of these costs together, the price of constructing an entire signalized intersection can soar to the millions depending on how much the cost of construction is and how much is able to be reused from the previous intersection. Once the project is complete, there are annual and maintenance costs for signalized intersections. These include power, routine traffic analysis, replacement of bulbs or other parts, new signing and more. Finally, there are opportunity costs unique to signalized intersections, including being unable to function from something like a lack of power or an otherwise malfunctioning traffic signal head.

One of the greatest measures of an intersection's safety is the number of points in which two cars can collide with each other, otherwise known as conflict points. In a 4-leg intersection where each approach can go through, turn left or turn right, there are 32 conflict points. In a roundabout, there are only 8. Furthermore, there are different kinds of conflict points: ones where vehicles are diverging from their lane of traffic to another, ones where vehicles are merging into another lane of traffic, and ones where the paths of two vehicles are crossing. The points that can cause the most harm to vehicles and the humans driving them are the crossing conflict points. In a 4-leg intersection, 16 of the 32 conflict points are these crossing points. In a roundabout, however, there are no instances of these points. Only merging and diverging points exist within a roundabout (KYTC, n.d.). This is reflected in accident data as well, as roundabouts have a lower rate of accidents than signalized intersections. "Roundabouts have been shown to reduce fatal and injury crashes by 75%" (Chesterfield, n.d.).

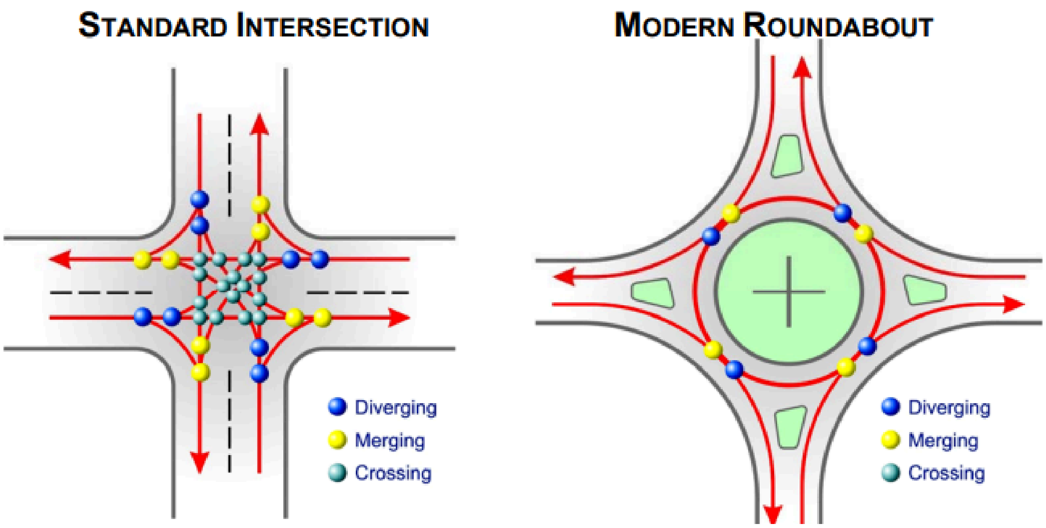

Figure 3: Visualization of Intersection's Conflict Points (KYTC, n.d.)

As detailed, there are a lot of advantages to roundabouts over signalized intersections. They have smaller long term costs, in this instance the roundabout would operate better, and they are safer. Despite that, the immediate cost is very high. It would ultimately be up to the developer if they would like to invest that much in order to completely mitigate the negative impact of the site on the existing intersection. With that being said, the proposed redesign will be a roundabout.

Design Overview and Considerations

Once again, the purpose of this roundabout will be to service the intersection of South Amherst Highway, Old Town Connector, and the new senior living facility. In total, there will be 4 legs to this intersection, one for each approach. This design will be done with the help of Version 1.1 of VDOT's Roundabout Design Guidance and *NCHRP Report 672 — Roundabouts: An Information Guide, 2nd Edition*. In both, it is recommended to follow the process detailed in Figure 4. The process detailed there will be loosely followed such that the needed inputs in PTV

Vistro can be fulfilled. The end of the design so screenshots of the design within PTV Vistro, alongside any other design considerations or decisions worth mentioning. Once again, there are no considerations of pedestrians or bicycles on this intersection.

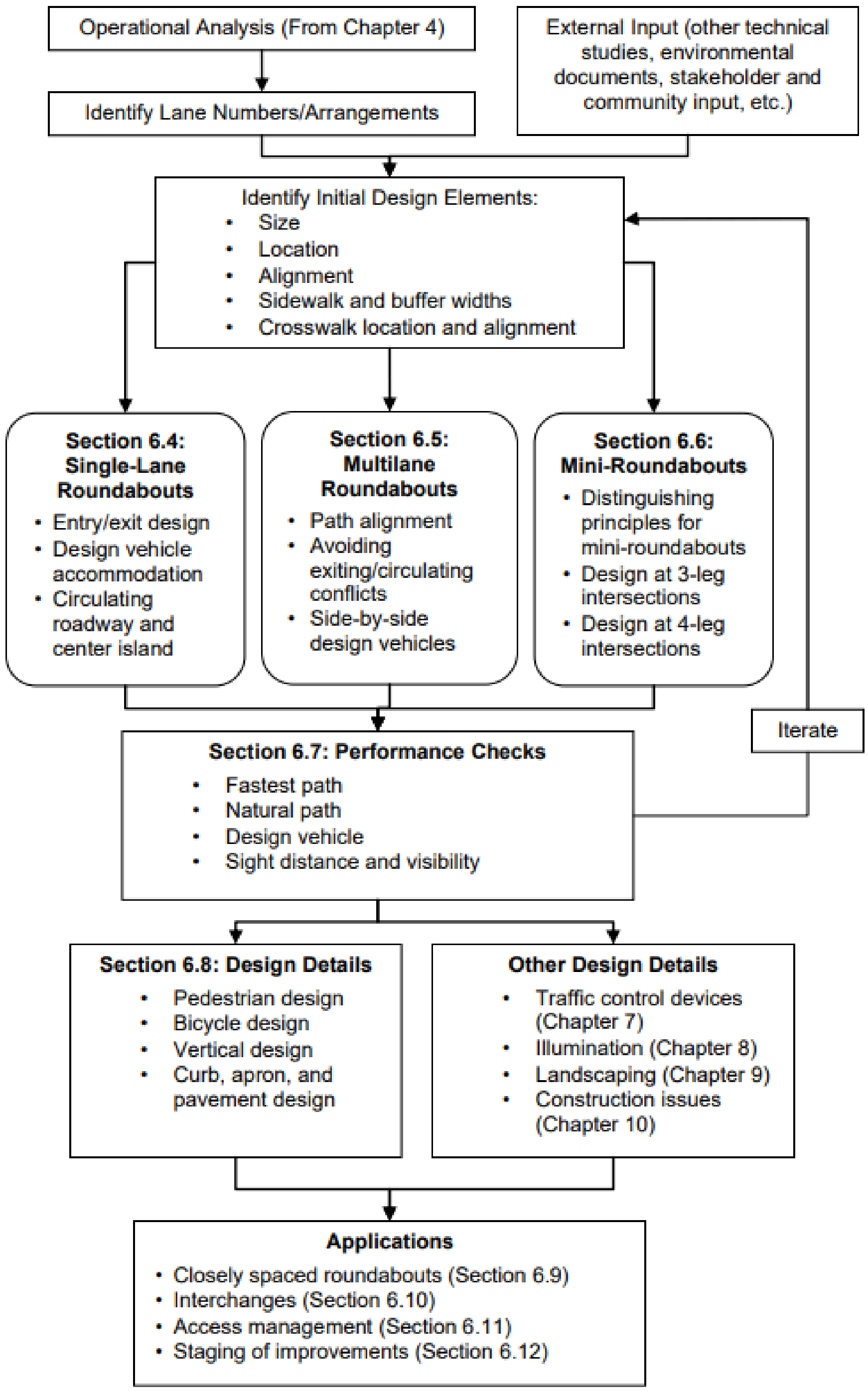

Figure 4: General Roundabout Design Process (NCHRP, 2010)

The operational analysis was completed in the previous section. All of the information from that analysis will be used to assist the design when possible. The next consideration is the number of lanes. At an approach of a roundabout, there are 3 different volumes, there is the volume entering the approach, the volume exiting the approach, and the volume going to the next approach. They are referred to as the entering, exiting and conflict volumes respectively. When evaluating how many lanes are going to be in a roundabout, the sum entering and conflicting volumes at a given approach can be used to estimate how many lanes will be needed (Figure 5).

Volume Range (sum of entering and conflicting volumes)	Number of Lanes Required
0 to 1,000 veh/h	▪ Single-lane entry likely to be sufficient
1,000 to 1,300 veh/h	▪ Two-lane entry may be needed ▪ Single-lane may be sufficient based upon more detailed analysis.
1,300 to 1,800 veh/h	▪ Two-lane entry likely to be sufficient
Above 1,800 veh/h	▪ More than two entering lanes may be required ▪ A more detailed capacity evaluation should be conducted to verify lane numbers and arrangements.

Source: New York State Department of Transportation

Figure 5: Planning-Level Lane Requirements for Roundabouts (NCHRP, 2010)

For this intersection, the sum of the entering volume and conflicting volume at each approach is less than 1,000. Therefore, this roundabout will suffice with a single lane. There will also be no need for any bypass lanes as well. This was assessed when the intersection was at its maximum hourly volume, which occurred during the PM peak hour.

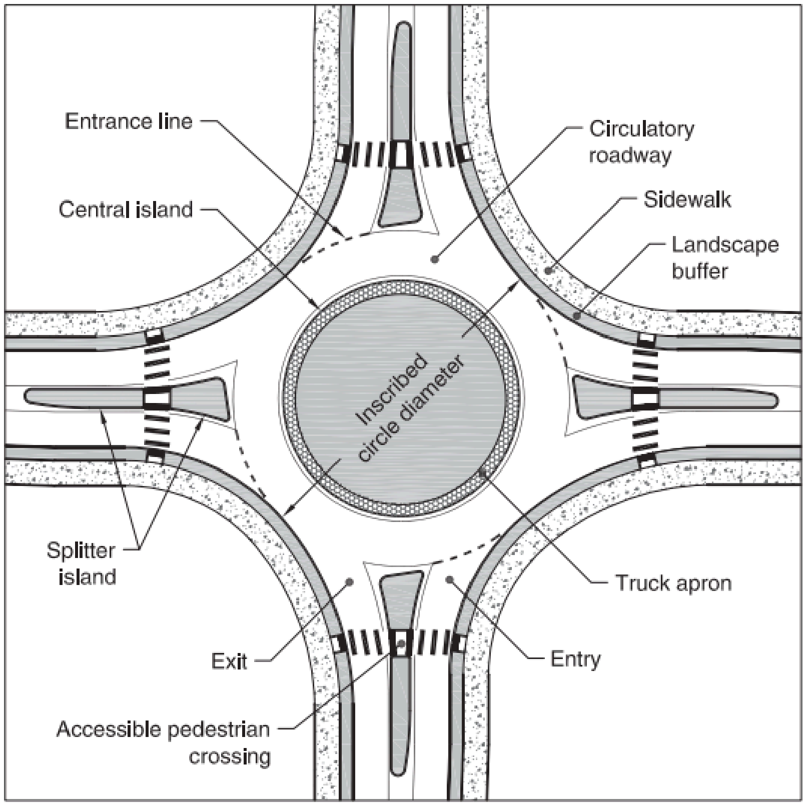

Figure 6: Basic Geometric Design Elements of a Roundabout (NCHRP, 2010)

The next important design criteria is the design vehicle. “Commonly, WB-50 (WB-15) vehicles are the largest vehicles along urban collectors and arterials” (NCHRP, 2010). The next closest design vehicle size in *A Policy on Geometric Design of Highways and Street, 7th Edition* is the WB-40 (WB-12). Below are its minimum turning radii specifications (Table 13).

Table 13: WB-40 (WB-12) Minimum Turning Design Specifications (AASHTO, 2018)

Descriptor	Radius (ft)
Minimum Design Turning Radius	39.9
Centerline Turning Radius	36
Minimum Inside Radius	19.3

There are a few other design aspects that can be considered at this point. First is the inscribed circle diameter, which is the diameter over the paved circle of the roundabout. For this project’s design vehicle, a minimum radius of 105 ft will be used. It’s also an output of a lot of other design elements that determine how large the radius needs to be. Once everything else is known, the radius can be chosen. Next, entries to the roundabout can be offset to better reflect the existing road geometry or help control the ability of the intersection to control the vehicle's speed. Approaches can be aligned either through or to the left or right of the approach centerline, each with their own advantages and disadvantages. When designing, a common starting point is to align the approach about the center of the roundabout. From there, the approach can be offset based on other parameters. Third is the angle between approaches. For 4 legs intersections, it is best to keep them as close to 90 degrees apart as possible, as angles higher than 90 degrees run the risk of drivers traversing those segments of the roundabout at a faster speed.

Referring to Figure 4, the next topics being talked about will be specifically in reference to single-lane roundabouts. The next design components are the splitter island and the driver's visibility. It is important to ensure that drivers have the necessary stopping sight distance and intersection sight distance such that their safety can be maximized. Based on the existing conditions, there will be sufficient stopping sight distance at all points around the roundabout. Because of the weird geometry of the original intersection and the uphill grade into the intersection from every approach, two changes will need to be made to maximize the intersection sight distance. First, the splitter islands will be longer than the design minimum to ensure that the drivers driving uphill and at a speed of 35 mph can successfully and safely recognize the roundabout. Second, since the northbound approach is skewed, removing the trees or close to the approach or changing the geometry of the approach can help drivers better recognize vehicles coming from that approach as they enter the intersection.

The splitter islands will also serve another purpose. "Maximum entering design speed based on a theoretical fastest path of 20 to 25 mph are recommended at single-lane roundabouts" (NCHRP, 2010). The speed of the east, west and northbound approaches is currently 35 mph, and that will not be changed as a result of the project. That being said, there needs to be something to help reduce the speed such that the speed at the entry of the roundabout reflects the maximum entering design speed. That's where the second purpose of the splitter islands come in, as they can help reduce the speed of entry into the roundabout. They would need to be significantly longer than the minimum length of 50 feet to fulfill this purpose. Finally, it is important that splitter islands extend past the end of the exit curve to prevent exiting traffic from accidentally crossing into the path of approaching traffic. The lengths of the splitter islands in this design will sufficiently meet this expectation.

The next consideration is the entry width of each approach. They usually range from 14-18 ft. A good starting value is 15 ft, and then iterating from there to find the optimal value. That process will be followed in this design. Entry widths should not exceed 18 ft such that drivers do not mistake the roundabout to have two lanes. The entry width leads right into the circulatory roadway width. This typically ranges from 100-120% the entry width or 16-20 ft. It usually remains constant throughout the entire roundabout. The circulatory roadway width should be wide enough to accommodate the WB-40 (WB-12) design vehicle. A truck apron can help out in that. "Usually, the left-turn movement is the critical path for determining circulatory roadway width. In accordance with AASHTO policy, a minimum clearance of 1 ft (preferably 2 ft) should be provided between the outside edge of the vehicle's tire track and the curb line" (NCHRP, 2010).

Roundabouts usually have a non-traversable area at its center called a central island. The island usually has some landscaping or other signifiers that help the driver recognize that there is a roundabout. "Raised central islands for single-lane roundabouts are preferred over depressed central islands, as depressed central islands are difficult for approaching drivers to recognize and drainage can be an issue” (NCHRP, 2010). It is best for central islands to be circular, but some can be ovular or irregularly shaped. That being said, this project will do its best to not deviate from a circular central island. These islands can also include a truck apron, which is a raised, paved area for trucks to have extra room for turning. The truck aprons are designed specifically for the design vehicles of the intersection. This roundabout will follow that expectation. These aprons are of a different material than the circular roadway and are raised by at least 2-3” to discourage passenger vehicles from using them. Without sufficient turning space for trucks, they can ride along the central island and harm either the landscaping or other things that are on them or even risk tipping over as a result of driving on something they are not designed to.

The final design considerations are the entry and exit ways. First is the entry design. For the entry curb radius, a single radius for all approaches is usually sufficient for a single-lane roundabout. They are to be between 50-100 ft. When iterating for it, start between 60-90 ft and go from there until the right value is found. Entry curb radii at 65 ft or below can negatively impact the roundabout’s capacity. Conversely, larger radii may result in unwanted higher speeds. Second is the exit design. “Exit curb radii are usually larger than the entry curb radii in order to minimize the likelihood of congestion and crashes at the exits” (NCHRP, 2010). The same minimum of 50 ft applies here, but when iterating, the usual range is now larger at 100-200 ft. Offset approach alignments can result in much larger radii, ranging from 300-800 ft.

Discussion and Conclusions

The following parameters are needed in PTV Vistro’s roundabout design interface: entry lane width (E), entry curb radius (R), entry angle (Φ), approach half width (V), flare length (L’), grade separation (SEP), exit land width and curb radius, inscribed circle diameter (D), circulatory roadway width and speed, and splitter island length and width. PTV Vistro uses the Kimber model for determining the geometry of the roundabout. While others can be considered, these parameters will be the only ones necessary for sufficient completion of the design component.

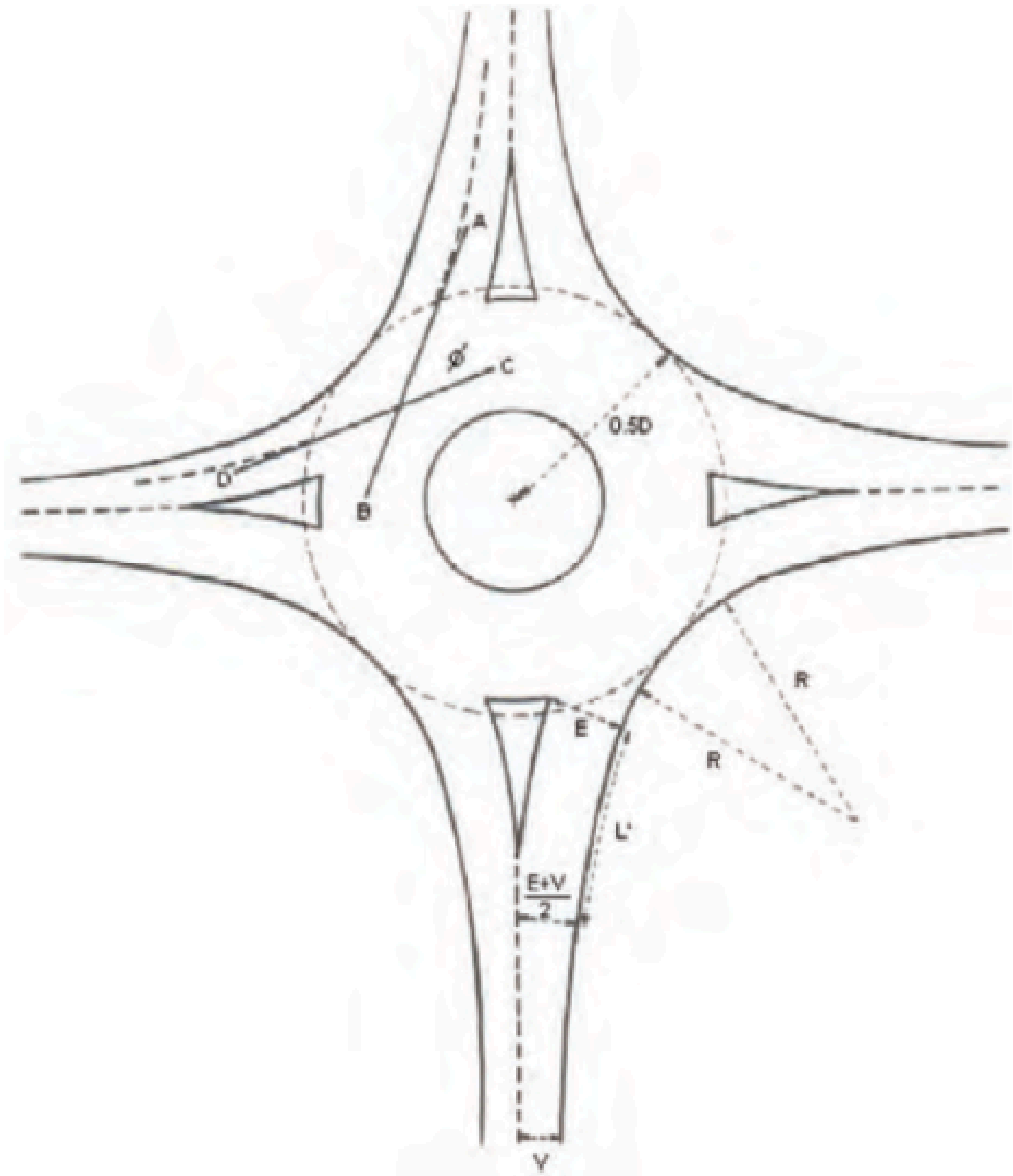

Figure 7: Description of the Node Geometry for the Kimber Model (PTV Vistro, n.d.)

In the NCHRP Report, there is a very in depth iteration method using various radii amongst each approach of the roundabout to determine the optimal values for each of the considered parameters. This is made significantly easier with the use of CAD software. Given more time for the project, this method could have been used to optimize the intersection's design such that it could function as efficiently and safely as possible.

With that being said, the iteration process will be outside of the expectation for this design, as a sufficient design will be enough within the time constraints. Due to those limitations and restrictions, a fair amount of assumptions will be made in order to meet all of the needed design criteria. The first and most important one is that the roundabout will be, for the most part, completely symmetrical. This assumption may not be realistic, but it allows for the same values to be used across each approach. Next, the lanes will remain 10 ft wide, just like in the existing conditions. Even though the northbound approach is skewed, the entry angles, using Figure 7 as a reference, will be at 45 degrees. In order to help alleviate this, the pathway before the entry of the roundabout on the northbound approach will be curved to help have a uniform entry angle among all approaches and help drivers slow down before reaching the approach. This design is simply one possibility of what the roundabout could look like. Undergoing the iterative process could reveal that this intersection is suboptimal, which within the scope of the project is to be expected.

Table 14: Design Parameters for PTV Vistro

	Northbound	Southbound	Eastbound	Westbound
Entry Lane Width (ft)	16.0	16.0	16.0	16.0
Entry Radius (ft)	80.0	80.0	80.0	80.0
Entry Angle (degrees)	45.0	45.0	45.0	45.0
Approach Half Width (ft)	10.0	10.0	10.0	10.0
Flare Length (ft)	60.0	60.0	60.0	60.0
Grade Separation (ft)	0.0	0.0	0.0	0.0
Exit Lane Width (ft)	20.0	20.0	20.0	20.0
Exit Radius (ft)	100.0	100.0	100.0	100.0
Inscribed Circle Diameter (ft)	110.0	110.0	110.0	110.0
Circulating Roadway Width (ft)	19.0			
Circulating Roadway Speed (mph)	18.0			
Splitter Island Length (ft)	100.0	100.0	100.0	100.0
Splitter Island Width (ft)	20.0	20.0	20.0	20.0

The entry lane width, entry radius, exit lane width, exit radius and circulating roadway width were all values within the acceptable ranges of values from the NCHRP report. Since the lane widths are still 10 ft, the approach half-width is 10ft. For simplicity's sake, flare length and grade separation are left at default values. In order to satisfy the requirements of the design vehicle, the inscribed circle diameter will be 110 ft. The circulating roadway speed was estimated from Figure 8. The central island will include a truck apron as well to help with them turning. It will have a radius of 10 feet and be raised 3" off the ground. Like previously mentioned, splitter islands will be implemented to help reduce the speed before entry into the roundabout and increase sight distance.

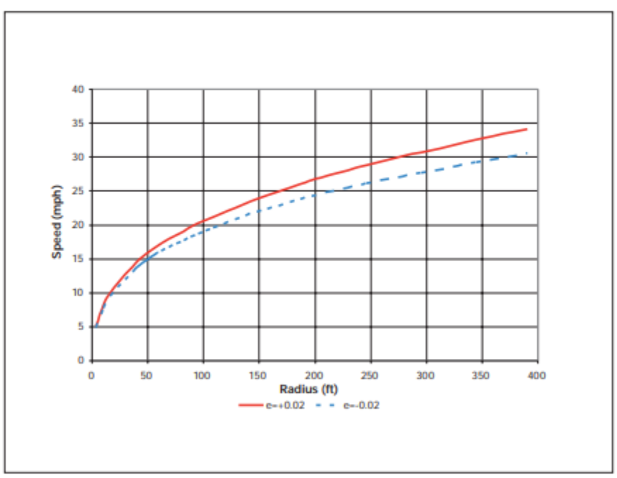

Figure 8: Speed-Radius Relationship (U.S. Customary Units) (citation)

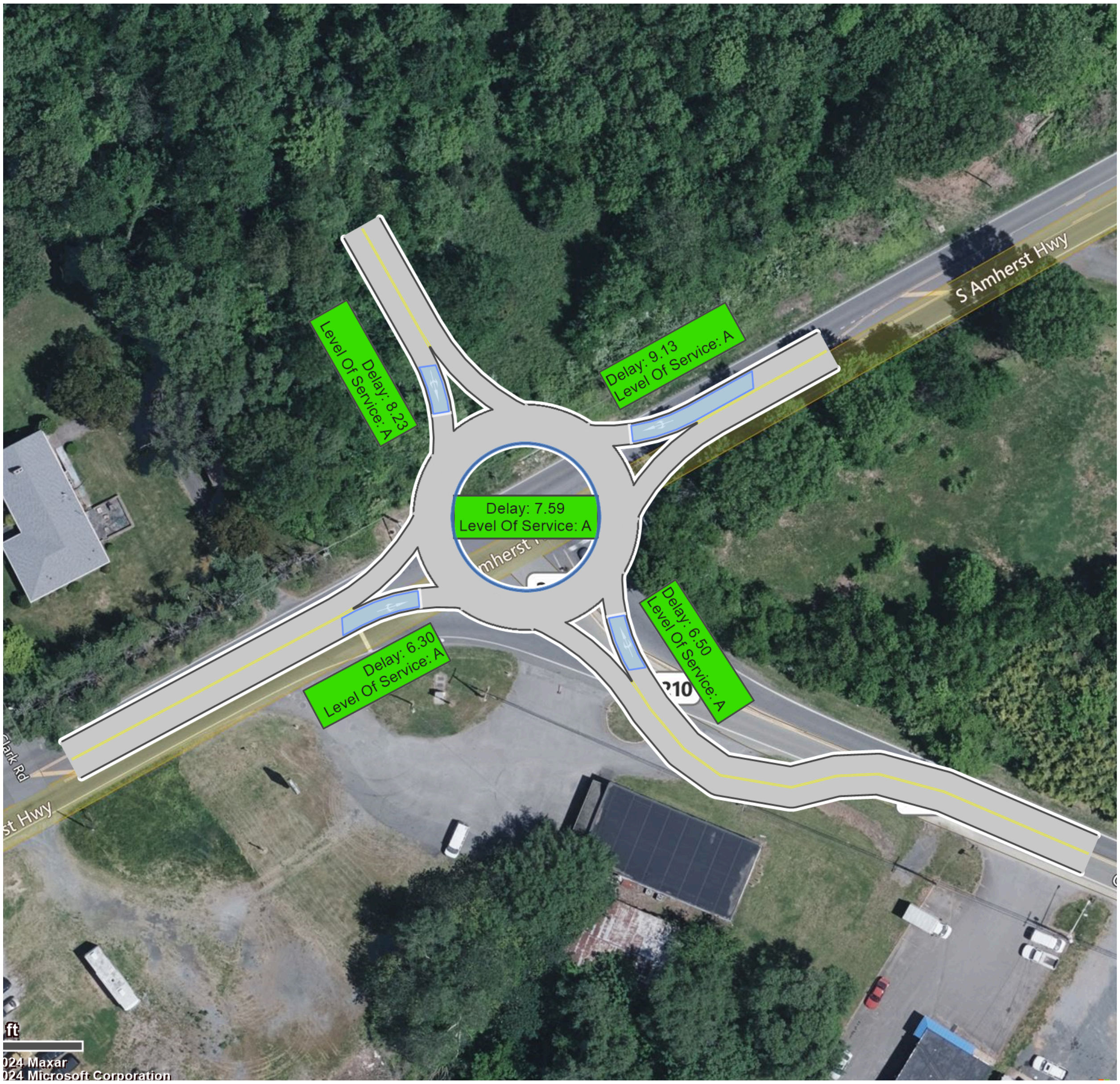

Figure 9: Design of Roundabout in PTV Vistro

Once again, this is simply one possible iteration of the design that could help mitigate traffic at the intersection. This design is not without fault, as redirecting the northbound approach would require lots of right-of-way cost in order to get access to restructure the road. Furthermore, the cost of construction would increase a lot with that decision as well. With that being said, this design still serves its purpose of providing one alternative that would mitigate or eliminate the negative impact the site would have on the intersection.

Conclusion

The roundabout design completely mitigates the negative impact of the site on the intersection at the expense of a lot of money. Ultimately, it is up to the developer to determine what the design will look like. What was detailed above is nothing more than a recommendation based on chosen elements such as how long and short term costs are compared and safety. This project functioned as a traffic impact study and then some in that regard.

It is realistic for the developer to keep it a signalized intersection due to the large immediate cost of a roundabout, lower construction time and impact, and low amount of negative impact of the site on the intersection. Once again, the intersection maintains its LOS grade of 'C' while gaining only small amounts of delay. There are ways for the negative impact to be mitigated while keeping it as a signalized intersection, such as further optimizing the timing splits, increasing the amount of protected/permissive left turns, or implementing some actuation into the cycle to help during non-peak hours. That is not to discredit the work done above, only to further contextualize it.

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Appendix A - Detailed Schedule

Task	1/19/24	1/26/24	2/2/24	2/9/24	2/16/24	2/23/24	3/1/24	3/8/24	3/15/24	3/22/24	3/29/24	4/5/24	4/12/24	4/19/24	4/26/24
Proposal															
Planning															
Data Collection															
Analysis of Existing Conditions															
Future Analysis Without Site															
Future Analysis With Site															
Documentation															
Learn MicroStation															
Design Intersection With 4th Leg															
Design of Neighborhood Connector															
Final Poster															
Final Report															

Figure A1: Initial Schedule as of 2/9/24

As time progressed, the analysis portion kept taking longer and longer and a greater understanding of both how to carry out traffic analyses and how to use PTV Vistro were obtained. This results in the exclusion of the design of the connector to the nearby neighborhood and pushing the documentation and design phase further back.

NEW SCHEDULE															
Task	1/19/24	1/26/24	2/2/24	2/9/24	2/16/24	2/23/24	3/1/24	3/8/24	3/15/24	3/22/24	3/29/24	4/5/24	4/12/24	4/19/24	4/26/24
Proposal															
Planning															
Data Collection															
Analysis of Existing Conditions															
Future Analysis Without Site															
Future Analysis With Site															
Documentation															
Learn MicroStation															
Design Intersection With 4th Leg															
Final Poster															
Final Report															

Figure A2: Revised Schedule as of 3/27/24

As time progressed even further, it was decided that PTV Vistro would be sufficient for completing the intersection design component of the project. In addition, the analysis needed a lot of reworking, sparking the change of a lot of the background calculations to be done on Google Sheets instead of PTV Vistro. This resulted in the final poster no longer being an expectation for completion of the project.

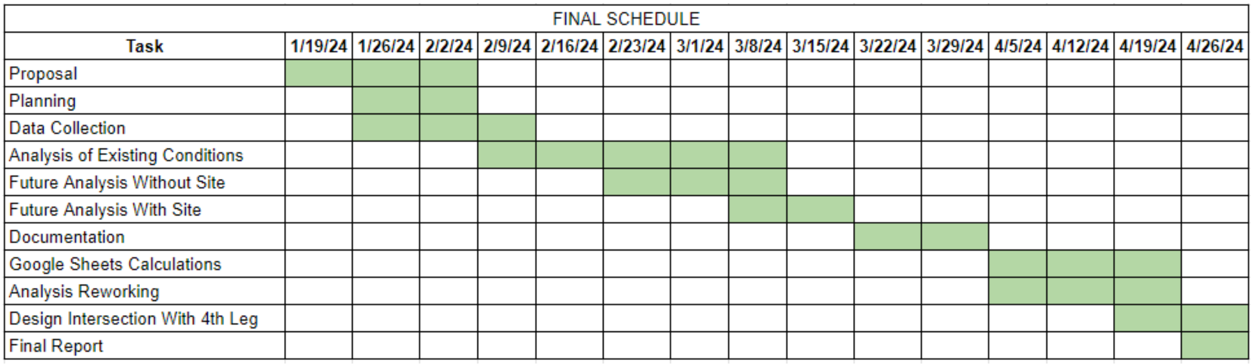


Figure A3: Final Schedule to Reflect Actual Progression of Project

<u>Appendix B - Design Evolution</u>

There are 3 design evolutions in this project. First, the scope of the design was reduced to eliminate the design of the neighborhood connector. Second, the design consideration was expanded to include a roundabout alongside a signalized intersection. Finally, it was determined that the design component would be able to sufficiently be completed within just PTV Vistro. This eliminated the need to learn and use CAD software, which was planned to be MicroStation. Any other changes and assumptions made to the design are mentioned in the Design section.

<u>Appendix C - Engineering Standards</u>

The analysis was done using the HCM 7th Edition standards within PTV Vistro. The analysis and design followed the VDOT Traffic Operations and Safety Analysis (TOSAM) Manuel, VDOT Road Design Manual and of the guidelines and constraints from the American Association of State Highway and Transportation Officials (AASHTO), including the “Green Book”. Any more standards were guidelines provided in the NCHRP Report 672. The use of many standards and guidelines were mentioned in the body of the report.

<u>Appendix D - Associated Technical Deliverables</u>

This will include the initial info, calculated growth rates, future volumes, trip generations, the spreadsheet math I did, and the output intersection information and signal timings.

	Amherst Hwy From North				Amherst Hwy From South			Merrymoor Dr From West			
Start Time	Right	Thru	Left	App. Total	Thru	Left	App. Total	Right	Left	App. Total	Int. Total
Peak Hour Analysis From 07:00 AM to 11:45 AM - Peak 1 of 1											
Peak Hour for Entire Intersection Begins at 07:45 AM											
07:45 AM	0	**155**	0	**155**	74	**1**	75	**3**	**5**	**8**	**238**
08:00 AM	**3**	123	0	126	86	0	86	1	3	4	216
08:15 AM	2	132	**1**	135	68	1	69	1	5	6	210
08:30 AM	0	143	0	143	**90**	0	**90**	1	4	5	238
Total Volume	5	553	1	559	318	2	320	6	17	23	902
% App. Total	0.9	98.9	0.2		99.4	0.6		26.1	73.9		
PHF	.417	.892	.250	.902	.883	.500	.889	.500	.850	.719	.947
Passenger Veh	4	544	1	549	310	2	312	6	15	21	882
% Passenger Veh	80.0	98.4	100	98.2	97.5	100	97.5	100	88.2	91.3	97.8
Trucks	1	9	0	10	8	0	8	0	2	2	20
% Trucks	20.0	1.6	0	1.8	2.5	0	2.5	0	11.8	8.7	2.2

Figure D1: AM Initial Information

	Amherst Hwy From North				Amherst Hwy From South			Merrymoor Dr From West			
Start Time	Right	Thru	Left	App. Total	Thru	Left	App. Total	Right	Left	App. Total	Int. Total
Peak Hour Analysis From 12:00 PM to 05:45 PM - Peak 1 of 1											
Peak Hour for Entire Intersection Begins at 04:30 PM											
04:30 PM	4	93	0	97	167	**2**	169	1	**7**	**8**	274
04:45 PM	**6**	84	0	90	150	1	151	**2**	2	4	245
05:00 PM	5	103	0	108	173	2	175	1	1	2	285
05:15 PM	3	**111**	0	**114**	**189**	2	**191**	2	2	4	**309**
Total Volume	18	391	0	409	679	7	686	6	12	18	1113
% App. Total	4.4	95.6	0		99	1		33.3	66.7		
PHF	.750	.881	.000	.897	.898	.875	.898	.750	.429	.563	.900
Passenger Veh	18	381	0	399	666	7	673	6	12	18	1090
% Passenger Veh	100	97.4	0	97.6	98.1	100	98.1	100	100	100	97.9
Trucks	0	10	0	10	13	0	13	0	0	0	23
% Trucks	0	2.6	0	2.4	1.9	0	1.9	0	0	0	2.1

Figure D2: PM Initial Information

210 @ 163

Phase Timing

10/4/2023 3:17:26 PM

Phase	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
Min Green	0	12	0	7	7	12	0	0	5	5	5	5	5	5	5	5
Veh Ext	0.0	7.0	0.0	3.0	3.0	7.0	0.0	0.0	3.0	4.0	3.0	4.0	3.0	4.0	3.0	4.0
Max Green 1	0	50	0	40	25	50	0	0	30	40	30	40	30	40	30	40
Max Green 2	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Max Green 3	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Max Ext	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Yellow	0.0	3.9	0.0	3.0	3.9	3.9	0.0	0.0	4.0	5.0	4.0	5.0	4.0	5.0	4.0	5.0
Red Clr	0.0	1.8	0.0	2.7	1.8	1.8	0.0	0.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0
Adv Flash	0.0	5.0	0.0	0.0	0.0	5.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Bike MG	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Walk	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Ped Clr	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Walk2	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Sol DW	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Early Wlk	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Delay Wlk	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Added	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Max Initial	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Min Gap	0.0	2.5	0.0	2.5	0.0	2.5	0.0	2.5	0.0	2.5	0.0	2.5	0.0	2.5	0.0	2.5
Reduce After	0	8	0	8	0	8	0	8	0	8	0	8	0	8	0	8
TTReduce	0	10	0	10	0	10	0	10	0	10	0	10	0	10	0	10
CS Min Green	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
CS Max Green	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Red Revert	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0
Neg Ped	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
AP Disc	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Pmt Green	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Pmt Walk	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Pmt Ped Clr	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Return Green	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

Figure D3: Provided Signal Timing Sheet

	AADT		Growth		
Year	VA163	VA210	163	210	
2022	8457	4334	0.0103	-0.0197	
2021					excluded for COVID
2020					excluded for COVID
2019	8200	4600	-0.1277	0.0455	
2018	9400	4400	-0.0408	-0.0222	
2017	9800	4500	0.0103	0.0227	
2016	9700	4400	0.0899	0.1282	
2015	8900	3900	0.0349	0.0541	
2014	8600	3700	0.0118	0.0000	
2013	8500	3700	-0.1500	-0.1395	
2012	10000	4300	0	0	
2011	10000	4300			
			-1.79%	0.77%	average growth rate
			0.50%		

Figure D4: Growth Rate Calculations

	AM					
	Old Town Connector		S Amherst Highway (Eastbound)		N Amherst Highway (Westbound)	
2023	193	12	204	125	25	364
2030	200	12	211	129	26	377

	PM					
	Old Town Connector		S Amherst Highway (Eastbound)		N Amherst Highway (Westbound)	
2023	136	16	471	221	40	273
2030	143	17	496	233	42	288

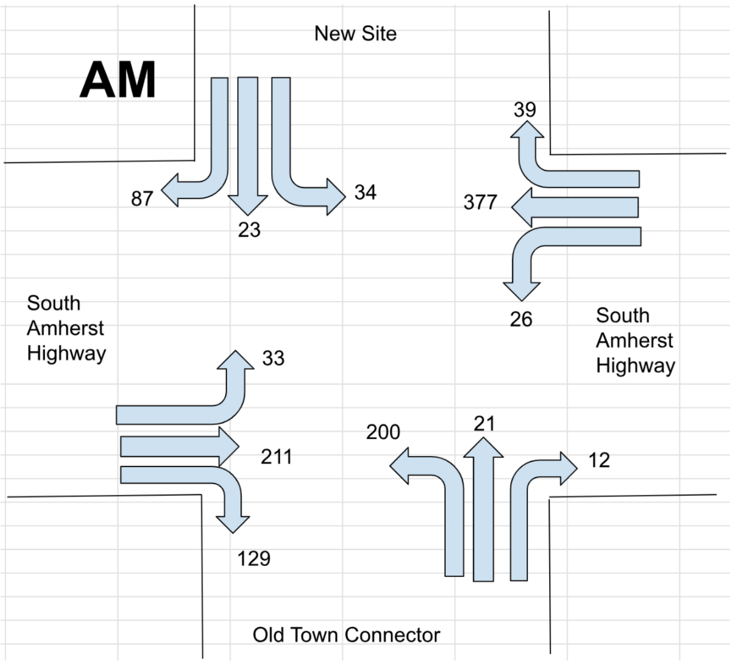
Figure D5: Future Volumes for Existing Legs

Information Used from Trip Generation Manual					
Weekday					
	Average Rate	Equation	% Entering	% Exiting	R^2
Parcel 1	10.84	Ln(T) = 0.87Ln(x) + 3.05	50%	50%	0.78
Parcel 2	6.74	T = 6.41(x) + 75.31	50%	50%	0.86
Parcel 3	7.20	T = 7.62(x) - 50.48	50%	50%	0.94
Parcel 4	3.24	T = 2.89(x) + 24.82	50%	50%	0.99
Parcel 5	2.60	Not Given	50%	50%	---
Weekday AM Peak					
Parcel 1	1.52	Ln(T) = 0.86Ln(x) + 1.16	88%	12%	0.78
Parcel 2	0.40	T = 0.31(x) + 22.85	24%	76%	0.79
Parcel 3	0.48	T = 0.52(x) - 5.70	31%	69%	0.92
Parcel 4	0.20	T = 0.19(x) = 0.90	34%	66%	0.85
Parcel 5	0.18	Not Given	60%	40%	---
Weekday PM Peak					
Parcel 1	1.44	Ln(T) = 0.83Ln(x) + 1.29	17%	83%	0.77
Parcel 2	0.51	T = 0.43(x) + 20.55	63%	37%	0.84
Parcel 3	0.57	T = 0.60(x) - 3.93	57%	43%	0.91
Parcel 4	0.25	T = 0.25(x) + 0.07	56%	44%	0.84
Parcel 5	0.24	Not Given	39%	61%	---

Figure D6: Information Used From ITE Trip Generation Manual

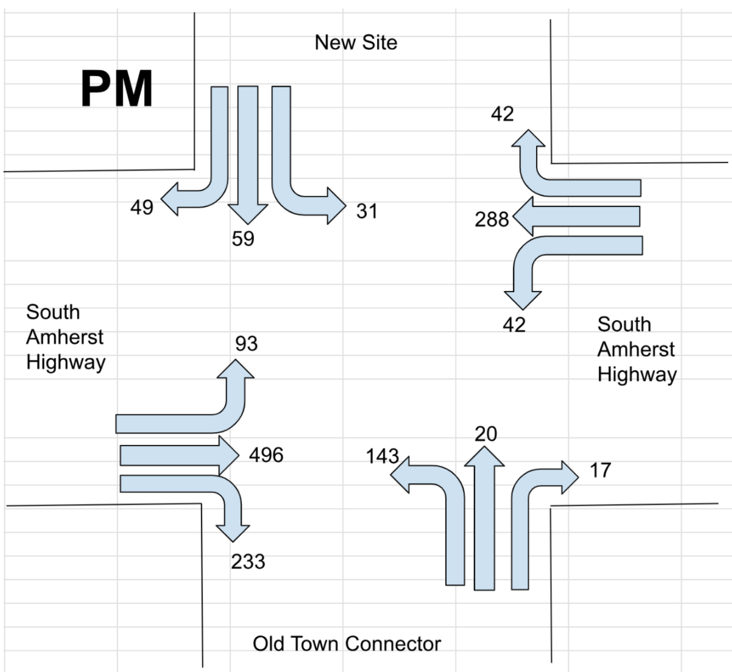
Trip Generation						
	Land Use Code	Descriptor	Dwellings	Daily Traffic	# of Trips In	# of Trips Out
Parcel 1	710	General Office Building	17500	255	127	127
Parcel 2	220	Multifamily Housing (Low-Rise) Not Close to Rail Transit	272	1819	909	909
Parcel 3	215	Single-Family Attached Housing	84	691	345	345
Parcel 4	252	Senior Adult Housing - Multifamily	150	458	229	229
Parcel 5	254	Assisted Living	140	364	182	182
	Peak AM Traffic	# of Trips In	# of Trips Out	Peak PM Traffic	# of Trips In	# of Trips Out
Parcel 1	37	33	4	39	7	32
Parcel 2	107	26	81	138	87	51
Parcel 3	38	9	28	46	27	19
Parcel 4	29	10	19	38	21	17
Parcel 5	25	15	10	34	13	20
	Total AM Trips:	93	144	Total PM Trips:	155	139

Figure D7: Trip Generation Math Outputs



Entering			
Approach	Total Volume of Approach	Proportion of Approach's Volume to Total Volume	Volume Entering Site from Approach
Eastbound	340	0.35602	33
Northbound	212	0.22199	21
Westbound	403	0.42199	39
Total	955	1.00	
Exiting			
Direction Headed	Total Volume of Direction	Proportion of Direction's Volume to Total Volume	Volume Exiting Site in Direction
West	577	0.60419	87
East	223	0.23351	34
South	155	0.16230	23
Total	955	1.00	

Figure D8: Future Site AM Trip Distribution



Entering			
Approach	Total Volume of Approach	Proportion of Approach's Volume to Total Volume	Volume Entering Site from Approach
Eastbound	729	0.59803	93
Northbound	160	0.13126	20
Westbound	330	0.27071	42
Total	1219	1.00	
Exiting			
Direction Headed	Total Volume of Direction	Proportion of Direction's Volume to Total Volume	Volume Exiting Site in Direction
West	431	0.35357	49
East	513	0.42084	58
South	275	0.22559	31
Total	1219	1.00	

Figure D9: Future Site PM Distribution

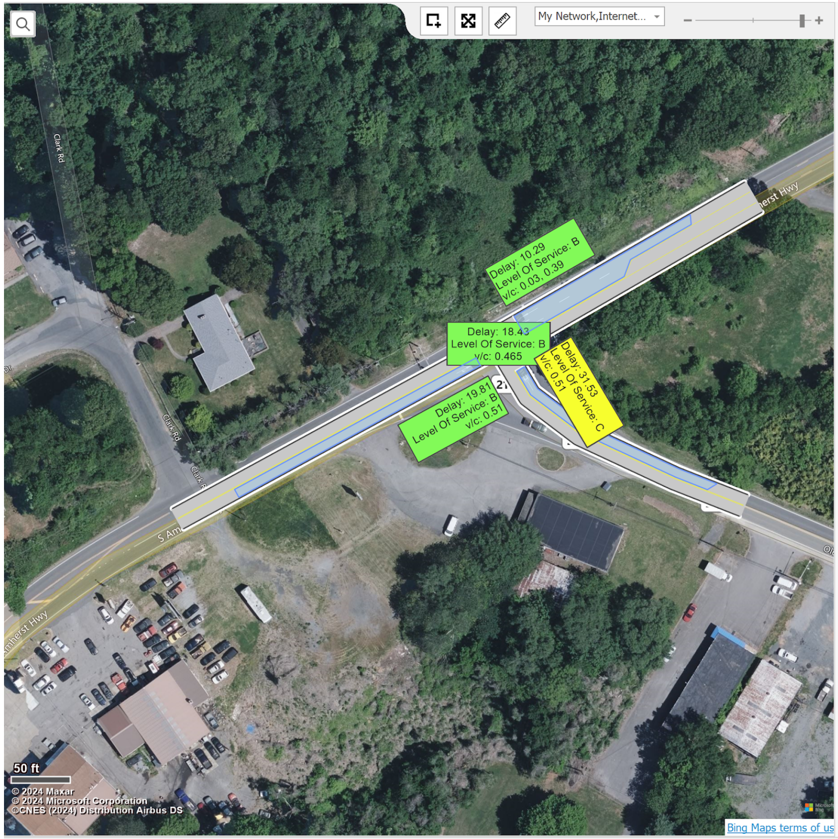


Figure D10: Existing Conditions AM Time w/ AM Plan

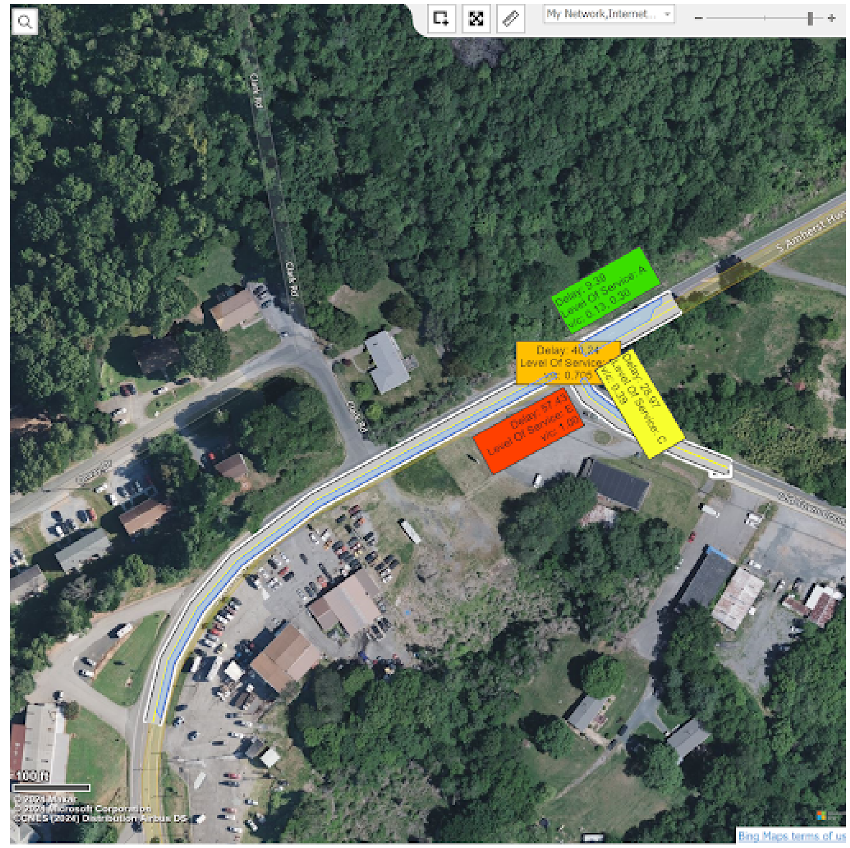


Figure D11: Existing Conditions PM Time w/ AM Plan

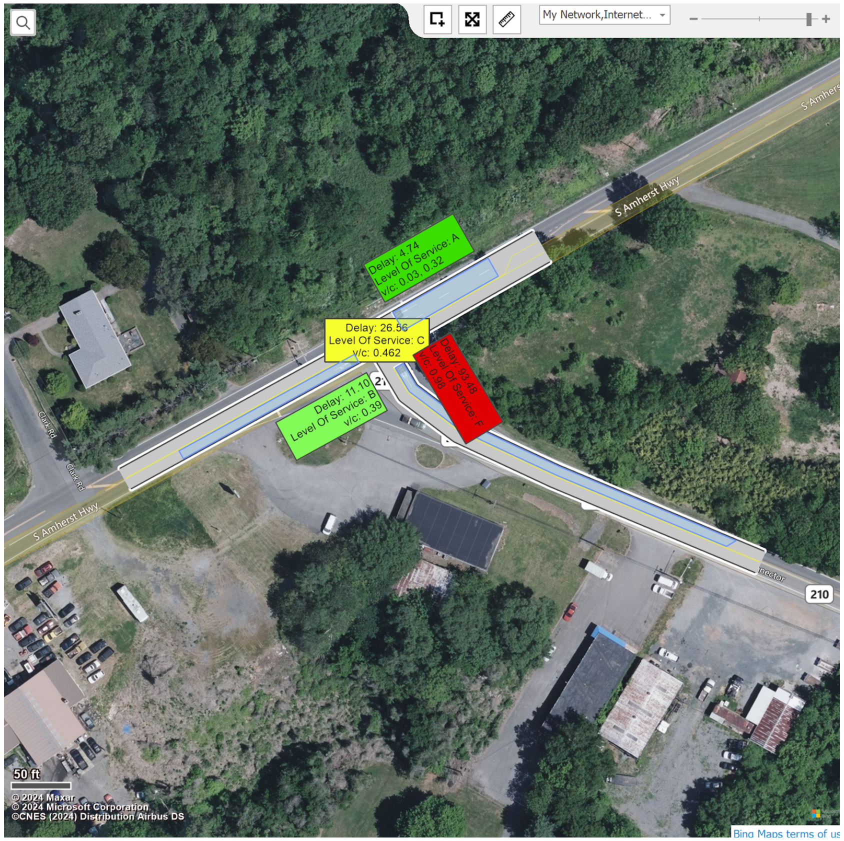


Figure D12: Existing Conditions AM Time w/ PM Plan

Figure D13: Existing Conditions PM Time w/ PM Plan

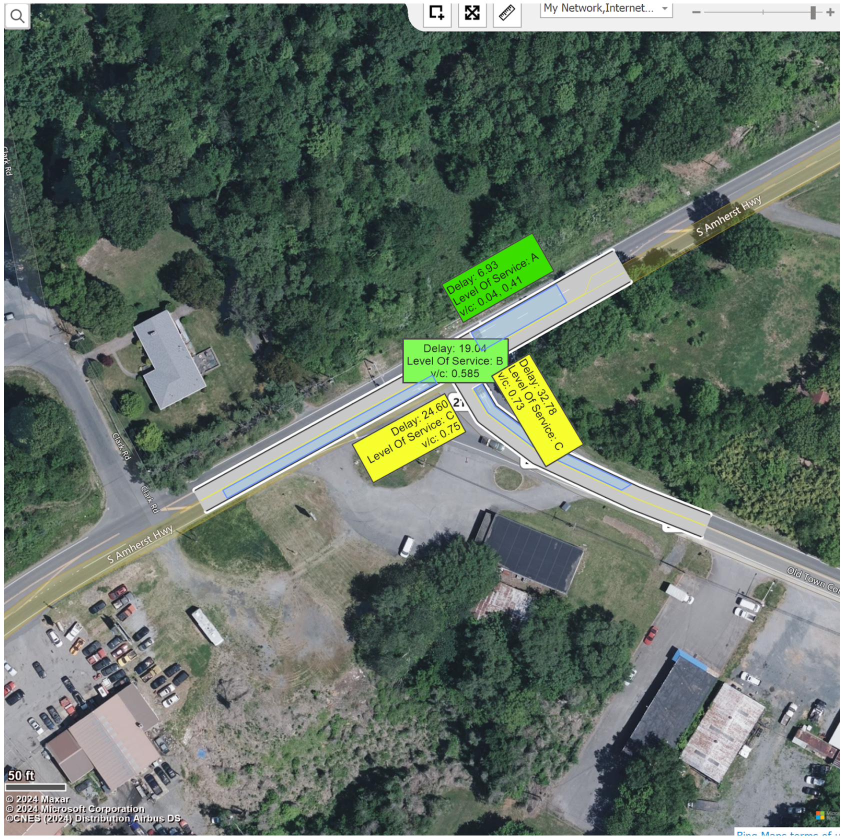


Figure D14: Future w/out Site AM, Cycle = 50 s

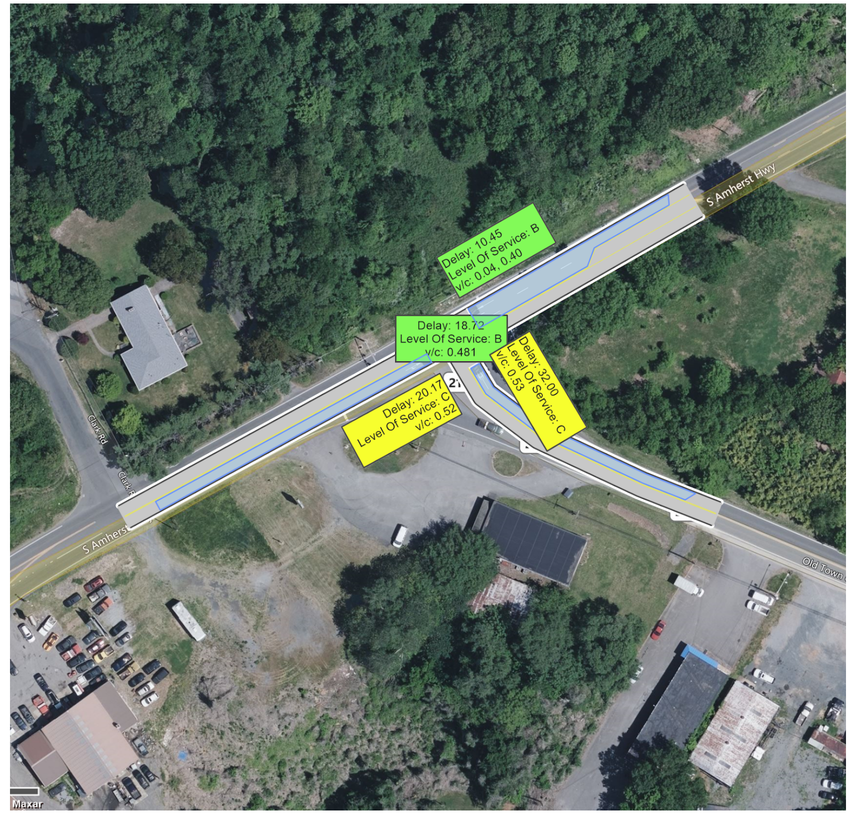


Figure D15: Future w/out Site PM, Cycle = 90 s

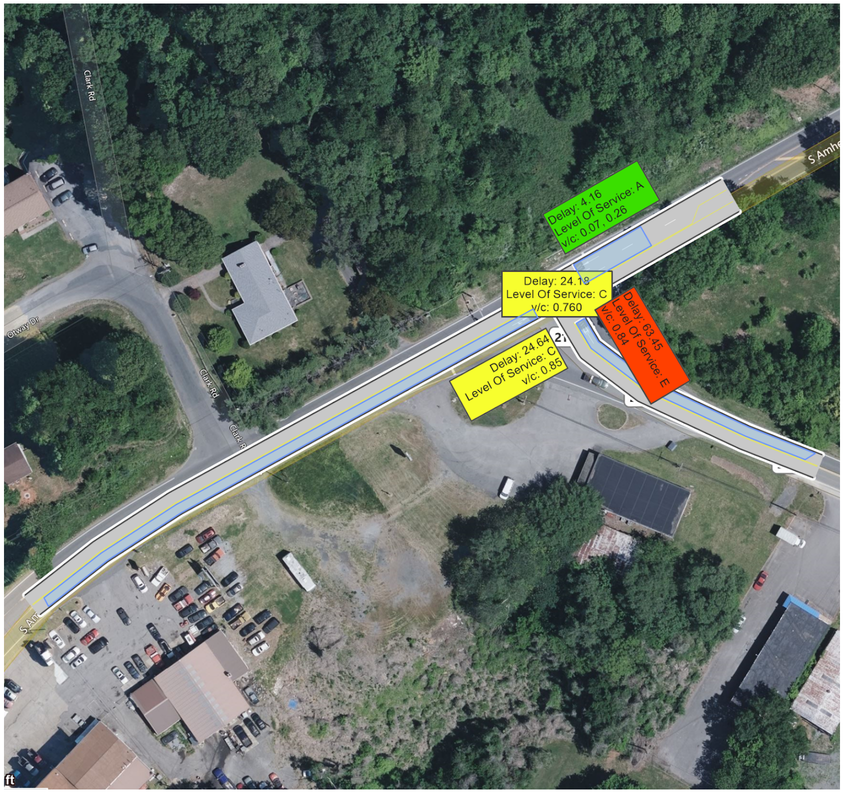


Figure D16: Future w/out Site PM, Cycle = 80 s

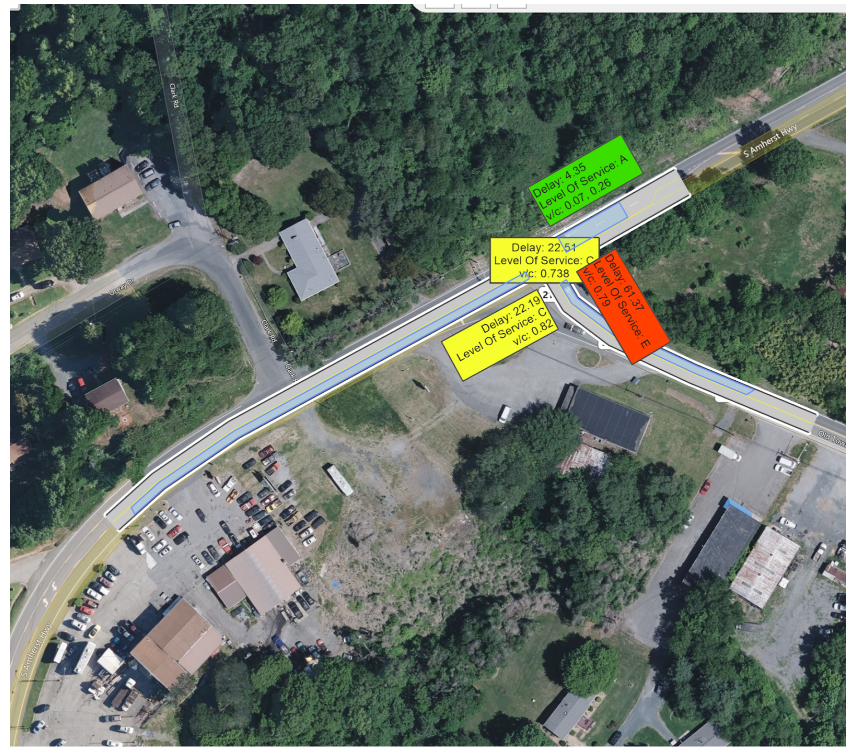


Figure D17: Future w/out Site PM, Cycle = 90 s

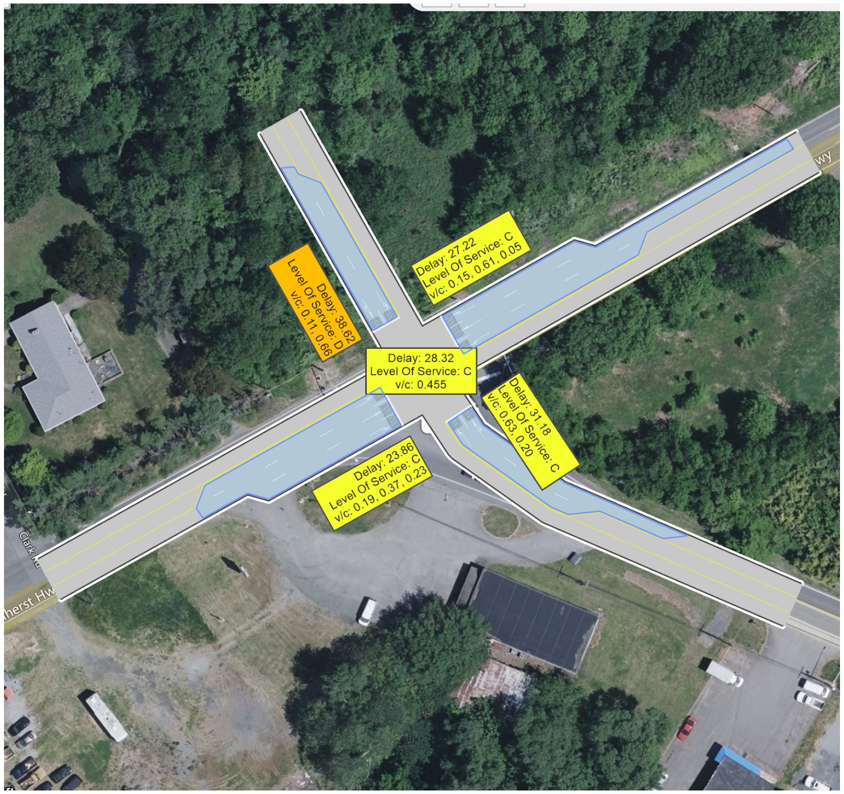


Figure D18: Future w/ Site AM, Cycle = 65 s

Figure D19: Future w/ Site PM, Cycle = 90 s

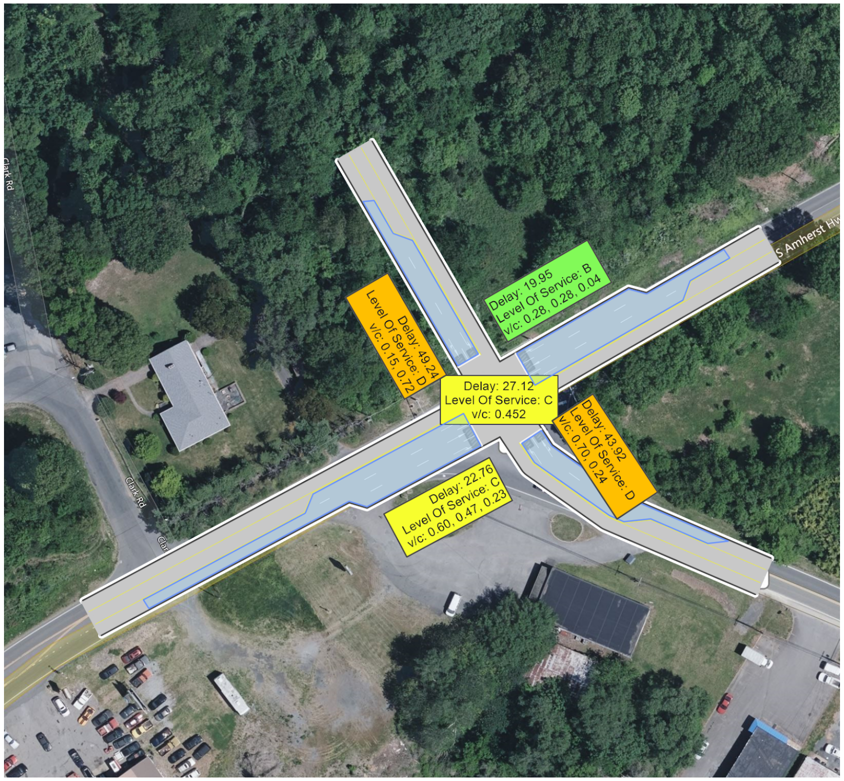


Figure D20: Future w/ Site PM, Cycle = 75 s

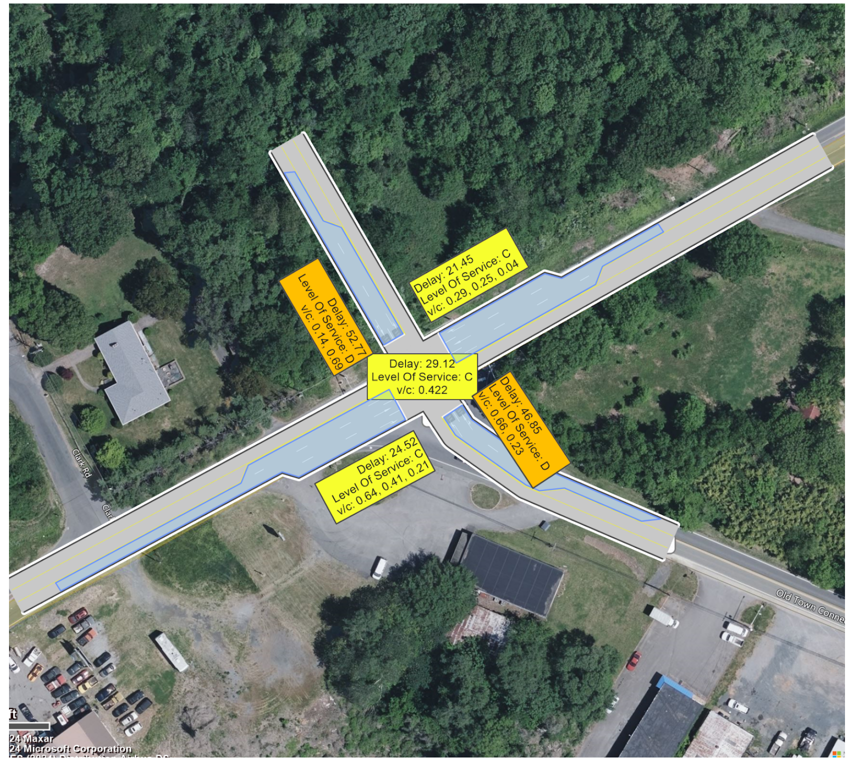


Figure D21: Future w/ Site PM, Cycle = 90 s

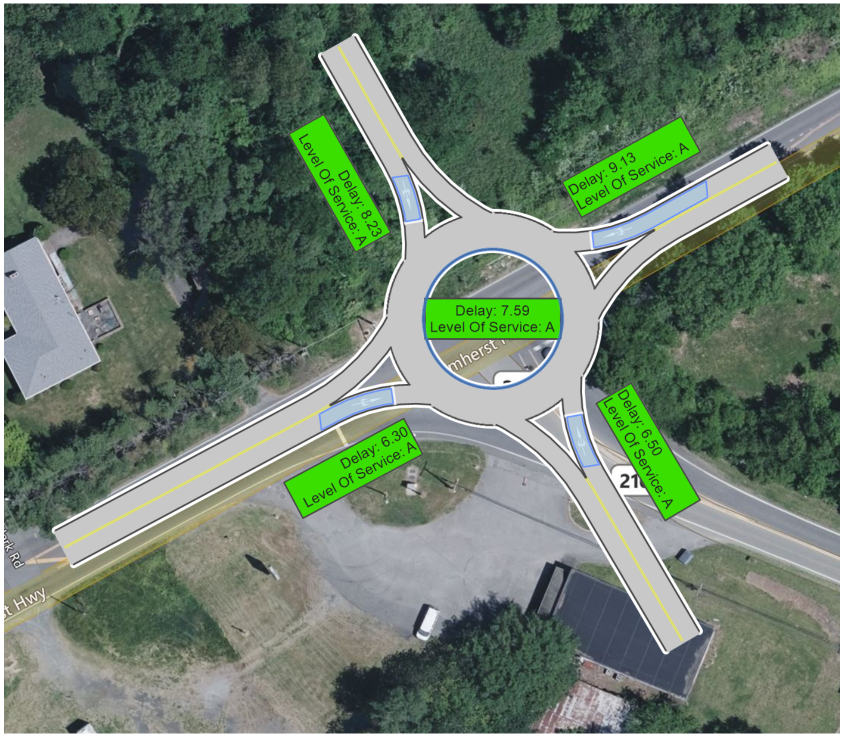


Figure D22: Roundabout AM

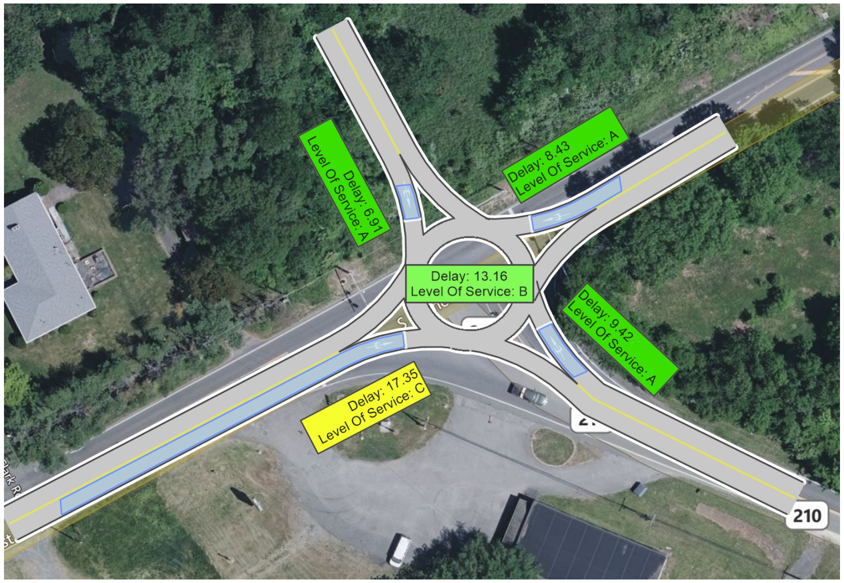


Figure D23: Roundabout PM

Figure D24: Existing Conditions AM Timing Splits

Figure D25: Existing Conditions PM Timing Splits

Figure D26: Future Conditions w/out Site AM, Cycle = 50 seconds

Figure D27: Future Conditions w/out Site AM, Cycle = 90 seconds

Figure D28: Future Conditions w/out Site PM, Cycle = 80 seconds

Figure D29: Future Conditions w/out Site PM, Cycle = 90 seconds

Figure D30: Future Conditions w/ Site AM, Cycle = 65 seconds

Figure D31: Future Conditions w/ Site AM, Cycle = 90 seconds

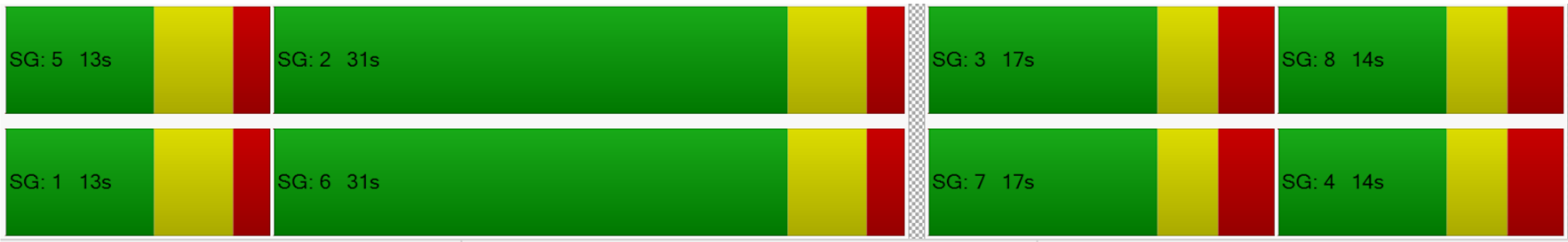


Figure D32: Future Conditions w/out Site PM, Cycle = 75 seconds

Figure D33: Future Conditions w/out Site PM, Cycle = 90 seconds

AM Existing Conditions (90 second cycle length)

																3.048	C =	90
	Volume	s_o	N	f_w	f_HV	f_g	f_p	f_bb	f_a	f_LU	f_LT	f_RT	f_Lpb	f_Rpb	s	v/s	Critical?	
WB LT	28	1900	1	0.9387	0.9615	0.9850	1.0	1.0	1.0	1.0	0.9500	1.0	1.0	1.0	1605	0.0174	Y	
WB TH	400	1900	1	0.9387	0.9804	0.9850	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1722	0.2323	N	
EB TH + RT	374	1900	1	0.9387	0.9709	0.9925	1.0	1.0	1.0	1.0	1.0	0.9430	1.0	1.0	1621	0.2308	Y	
NB LT + RT	226	1900	1	0.9387	0.9625	0.9800	1.0	1.0	1.0	1.0	0.9552	0.9907	1.0	1.0	1592	0.1420	Y	
															Y_c =	0.3902	C_opt =	47.556
															X_c =	0.4746	C_min =	90

	L_3 =	16.0	L_4 =	22.0						T =	0.25	k_max =	0.5				
		g_i		new g_i	G	Y	AR	Split		Capacity	v/c	d_1	d_2	Total Delay	LOS		
WB LT	Phase 1	3.3	bumped up to 7	7.7	7	3.8	1.9	12.7	13	958.32	0.0292	7.43	0.057	7.49	A		
WB TH	Phase 2	52.1	time to readjust	53.7	53	3.8	1.9	58.7	59	1028.53	0.3889	9.51	1.109	10.62	B	10.41	B
EB TH + RT		43.8		41.0	40.3	3.8	1.9	46.0	46	739.16	0.5060	17.31	2.467	19.77	B		
NB LT + RT	Phase 3	26.9	time to readjust	25.3	25.6	3.0	2.7	31.3	31	446.66	0.5060	27.14	4.054	31.20	C		
	Y_c2&3 =	0.3727		TRUE				90.0	90					Intersection Delay	18.39		
	X_c2&3 =	0.4346												Intersection LOS	B		

PM Existing Conditions (90 second cycle length)

	Volume	s_o	N	f_w	f_HV	f_g	f_p	f_bb	f_a	f_LU	f_LT	f_RT	f_Lpb	f_Rpb	s	v/s	Critical?	
WB LT	45	1900	1	0.9387	0.9756	0.9850	1.0	1.0	1.0	1.0	0.9500	1.0	1.0	1.0	1628	0.0276	Y	
WB TH	310	1900	1	0.9387	0.9785	0.9850	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1719	0.1803	N	
EB TH + RT	751	1900	1	0.9387	0.9804	0.9925	1.0	1.0	1.0	1.0	1.0	0.9499	1.0	1.0	1648	0.4556	Y	
NB LT + RT	173	1900	1	0.9387	0.9625	0.9800	1.0	1.0	1.0	1.0	0.9571	0.9844	1.0	1.0	1585	0.1092	Y	
															Y_c =	0.5924	C_opt =	71.146
															X_c =	0.7205	C_min =	90

		g_i		new g_i	G	Y	AR	Split		Capacity	v/c	d_1	d_2	Total Delay	LOS		
WB LT	Phase 1	3.5	bumped up to 7	7.7	7	3.8	1.9	12.7	13	1197.36	0.0376	3.24	0.059	3.30	A		
WB TH	Phase 2	65.4		66.2	65.5	3.8	1.9	71.2	71	1264.07	0.2452	3.84	0.462	4.31	A	4.18	A
EB TH + RT		56.9	time to readjust	53.5	52.8	3.8	1.9	58.5	58	979.61	0.7666	13.61	5.724	19.33	B		
NB LT + RT	Phase 3	13.6	time to readjust	12.8	13.1	3.0	2.7	18.8	19	225.66	0.7666	37.15	21.714	58.87	E		
	Y_c2&3 =	0.5648		TRUE				90.0	90					Intersection Delay	20.47		
	X_c2&3 =	0.6584												Intersection LOS	C		

AM Future w/out Site Conditions (50 second cycle length)

	Volume	s_o	N	f_w	f_HV	f_g	f_p	f_bb	f_a	f_LU	f_LT	f_RT	f_Lpb	f_Rpb	s	v/s	Critical?	
WB LT	29	1900	1	0.9387	0.9615	0.9850	1.0	1.0	1.0	1.0	0.9500	1.0	1.0	1.0	1605	0.0181	Y	
WB TH	414	1900	1	0.9387	0.9804	0.9850	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1722	0.2404	N	
EB TH + RT	387	1900	1	0.9387	0.9709	0.9925	1.0	1.0	1.0	1.0	1.0	0.9430	1.0	1.0	1621	0.2388	Y	
NB LT + RT	234	1900	1	0.9387	0.9625	0.9800	1.0	1.0	1.0	1.0	0.9551	0.9910	1.0	1.0	1592	0.1470	Y	
															Y_c =	0.4038	C_opt =	48.644
															X_c =	0.5939	C_min =	50

		g_i		new g_i	G	Y	AR	Split		Capacity	v/c	d_1	d_2	Total Delay	LOS		
WB LT	Phase 1	1.5	bumped up to 7	7.7	7	3.8	1.9	12.7	13	516.72	0.0561	4.57	0.207	4.77	A		
WB TH	Phase 2	26.6	time to readjust	29.0	28.3	3.8	1.9	34.0	34	998.25	0.4147	5.82	1.272	7.09	A	6.94	A
EB TH + RT		20.1		16.3	15.6	3.8	1.9	21.3	21	527.69	0.7334	14.94	8.745	23.68	C		
NB LT + RT	Phase 3	12.4	time to readjust	10.0	10.3	3.0	2.7	16.0	16	319.07	0.7334	18.74	13.906	32.64	C		
	Y_c2&3 =	0.3858		TRUE				50.0	50					Intersection Delay	18.68		
	X_c2&3 =	0.5471												Intersection LOS	B		

AM Future w/out Site Conditions (90 second cycle length)

	Volume	s_o	N	f_w	f_HV	f_g	f_p	f_bb	f_a	f_LU	f_LT	f_RT	f_Lpb	f_Rpb	s	v/s	Critical?	
WB LT	29	1900	1	0.9387	0.9615	0.9850	1.0	1.0	1.0	1.0	0.9500	1.0	1.0	1.0	1605	0.0181	Y	
WB TH	414	1900	1	0.9387	0.9804	0.9850	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1722	0.2404	Y	
EB TH + RT	387	1900	1	0.9387	0.9709	0.9925	1.0	1.0	1.0	1.0	1.0	0.9430	1.0	1.0	1621	0.2388	N	
NB LT + RT	234	1900	1	0.9387	0.9625	0.9800	1.0	1.0	1.0	1.0	0.9551	0.9910	1.0	1.0	1592	0.1470	Y	
															Y_c =	0.4038	C_opt =	48.644
															X_c =	0.4911	C_min =	90

		g_i		new g_i	G	Y	AR	Split		Capacity	v/c	d_1	d_2	Total Delay	LOS		
WB LT	Phase 1	3.3	bumped up to 7	7.7	7	3.8	1.9	12.7	13	958.21	0.0303	7.44	0.059	7.50	A		
WB TH	Phase 2	52.1	time to readjust	53.7	53	3.8	1.9	58.7	59	1028.42	0.4026	9.61	1.174	10.79	B	10.57	B
EB TH + RT		43.8		41.0	40.3	3.8	1.9	46.0	46	739.03	0.5237	17.49	2.645	20.14	B		
NB LT + RT	Phase 3	26.9	time to readjust	25.3	25.6	3.0	2.7	31.3	31	446.86	0.5237	27.30	4.340	31.64	C		
	Y_c2&3 =	0.3858		TRUE				90.0	90					Intersection Delay	18.69		
	X_c2&3 =	0.4498												Intersection LOS	B		

Figure D34: Spreadsheet Math Part 1

	PM Future w/out Site Conditions (80 second cycle length)																																		
	Volume	s_o	N	f_w	f_HV	f_g	f_p	f_bb	f_a	f_LU	f_LT	f_RT	f_Lpb	f_Rpb	s	v/s	Critical?			g_i		new g_i	G	Y	AR	Split		Capacity	v/c	d_1	d_2	Total Delay	LOS		
WB LT	48	1900	1	0.9387	0.9756	0.9850	1.0	1.0	1.0	1.0	0.9500	1.0	1.0	1.0	1628	0.0295	Y		Phase 1	3.0	bumped up to 7	7.7	7	3.8	1.9	12.7	13	139.30	0.3446	33.79	6.644	40.44	D		
WB TH	327	1900	1	0.9387	0.9785	0.9850	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1719	0.1902	N		Phase 2	57.2		58.1	57.4	3.8	1.9	63.1	63	1249.21	0.2618	3.69	0.510	4.20	A	8.84	A
EB TH + RT	792	1900	1	0.9387	0.9804	0.9925	1.0	1.0	1.0	1.0	1.0	0.9498	1.0	1.0	1648	0.4805	Y			49.2	time to readjust	45.4	44.7	3.8	1.9	50.4	50	936.23	0.8459	14.37	9.308	23.68	C		
NB LT + RT	182	1900	1	0.9387	0.9625	0.9800	1.0	1.0	1.0	1.0	0.9571	0.9843	1.0	1.0	1585	0.1148	Y		Phase 3	11.8	time to readjust	10.9	11.2	3.0	2.7	16.9	17	215.14	0.8459	33.75	31.568	65.32	E		
															Y_c =	0.6248	C_opt =	77.295	Y_c2&3 =	0.5953		TRUE				80.0	80					Intersection Delay	25.17		
															X_c =	0.7810	C_min =	80	X_c2&3 =	0.7117												Intersection LOS	C		

	PM Future w/out Site Conditions (90 second cycle length)																																		
	Volume	s_o	N	f_w	f_HV	f_g	f_p	f_bb	f_a	f_LU	f_LT	f_RT	f_Lpb	f_Rpb	s	v/s	Critical?			g_i		new g_i	G	Y	AR	Split		Capacity	v/c	d_1	d_2	Total Delay	LOS		
WB LT	48	1900	1	0.9387	0.9756	0.9850	1.0	1.0	1.0	1.0	0.9500	1.0	1.0	1.0	1628	0.0295	Y		Phase 1	3.5	bumped up to 7	7.7	7	3.8	1.9	12.7	13	139.30	0.3446	38.77	6.644	45.42	D		
WB TH	327	1900	1	0.9387	0.9785	0.9850	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1719	0.1902	N		Phase 2	65.4		66.2	65.5	3.8	1.9	71.2	71	1264.56	0.2586	3.88	0.496	4.38	A	9.63	A
EB TH + RT	792	1900	1	0.9387	0.9804	0.9925	1.0	1.0	1.0	1.0	1.0	0.9498	1.0	1.0	1648	0.4805	Y			56.9	time to readjust	53.5	52.8	3.8	1.9	58.5	58	980.02	0.8081	14.24	7.145	21.38	C		
NB LT + RT	182	1900	1	0.9387	0.9625	0.9800	1.0	1.0	1.0	1.0	0.9571	0.9843	1.0	1.0	1585	0.1148	Y		Phase 3	13.6	time to readjust	12.8	13.1	3.0	2.7	18.8	19	225.21	0.8081	37.42	25.899	63.32	E		
															Y_c =	0.6248	C_opt =	77.295	Y_c2&3 =	0.5953		TRUE				90.0	90					Intersection Delay	23.77		
															X_c =	0.7599	C_min =	90	X_c2&3 =	0.6941												Intersection LOS	C		

	AM Future w/ Site Conditions (65 second cycle length)																																		
	Volume	s_o	N	f_w	f_HV	f_g	f_p	f_bb	f_a	f_LU	f_LT	f_RT	f_Lpb	f_Rpb	s	v/s	Critical?			g_i		new g_i	G	Y	AR	Split		Capacity	v/c	d_1	d_2	Total Delay	LOS		
NB LT	220	1900	1	0.9387	0.9699	0.9800	1.0	1.0	1.0	1.0	0.9500	1.0	1.0	1.0	1610	0.1366	Y		Phase 3	15.8		13.8	14.1	3.0	2.7	19.8	20	341.70	0.6438	23.36	9.015	32.38	D		
NB TH + RT	38	1900	1	0.9387	0.9320	0.9800	1.0	1.0	1.0	1.0	1.0	0.9526	1.0	1.0	1552	0.0245	N		Phase 4	9.4		8.2	8.5	3.0	2.7	14.2	14	196.56	0.1933	25.41	2.182	27.59	C	31.67	D
SB LT	39	1900	1	0.9387	0.9804	0.9788	1.0	1.0	1.0	1.0	0.9500	1.0	1.0	1.0	1626	0.0240	N		Phase 3	15.8		13.8	14.1	3.0	2.7	19.8	20	344.95	0.1131	20.67	0.664	21.33	C		
SB TH + RT	125	1900	1	0.9387	0.9804	0.9788	1.0	1.0	1.0	1.0	1.0	0.8956	1.0	1.0	1533	0.0816	Y		Phase 4	9.4		8.2	8.5	3.0	2.7	14.2	14	194.15	0.6438	26.99	15.299	42.29	E	37.30	D
EB LT	38	1900	1	0.9387	0.9804	0.9925	1.0	1.0	1.0	1.0	0.9500	1.0	1.0	1.0	1649	0.0230	Y		Phase 1	2.7	bumped to 7	7.7	7	3.8	1.9	12.7	13	195.30	0.1946	25.85	2.213	28.07	C		
EB TH + RT	387	1900	2	0.9387	0.9709	0.9925	1.0	1.0	1.0	1.0	1.0	0.9500	1.0	1.0	3265	0.1185	N		Phase 2	15.2		13.3	12.6	3.8	1.9	18.3	18	666.86	0.5803	23.35	3.662	27.01	C	27.10	C
WB LT	29	1900	1	0.9387	0.9615	0.9850	1.0	1.0	1.0	1.0	0.9500	1.0	1.0	1.0	1605	0.0181	N		Phase 1	2.7		7.7	7	3.8	1.9	12.7	13	190.09	0.1526	25.72	1.697	27.42	C		
WB TH + RT	447	1900	2	0.9387	0.9804	0.9850	1.0	1.0	1.0	1.0	1.0	0.9869	1.0	1.0	3399	0.1315	Y		Phase 2	15.2		13.3	12.6	3.8	1.9	18.3	18	694.27	0.6438	23.70	4.557	28.25	C	28.20	C
															Y_c =	0.3727	C_opt =	60.577	Y_c2,3&4 =	0.3497		TRUE				65.0	65					Intersection Delay	29.65		
															X_c =	0.5634	C_min =	65	X_c2,3&4 =	0.5180												Intersection LOS	C		

Figure D35: Spreadsheet Math Part 2

AM Future w/ Site Conditions (65 second cycle length)

	Volume	s_o	N	f_w	f_HV	f_g	f_p	f_bb	f_a	f_LU	f_LT	f_RT	f_Lpb	f_Rpb	s	v/s	Critical?	
NB LT	220	1900	1	0.9387	0.9699	0.9800	1.0	1.0	1.0	1.0	0.9500	1.0	1.0	1.0	1610	0.1366	Y	
NB TH + RT	38	1900	1	0.9387	0.9320	0.9800	1.0	1.0	1.0	1.0	1.0	0.9526	1.0	1.0	1552	0.0245	N	
SB LT	39	1900	1	0.9387	0.9804	0.9788	1.0	1.0	1.0	1.0	0.9500	1.0	1.0	1.0	1626	0.0240	N	
SB TH + RT	125	1900	1	0.9387	0.9804	0.9788	1.0	1.0	1.0	1.0	1.0	0.8956	1.0	1.0	1533	0.0816	Y	
EB LT	38	1900	1	0.9387	0.9804	0.9925	1.0	1.0	1.0	1.0	0.9500	1.0	1.0	1.0	1649	0.0230	Y	
EB TH + RT	387	1900	2	0.9387	0.9709	0.9925	1.0	1.0	1.0	1.0	1.0	0.9500	1.0	1.0	3265	0.1185	N	
WB LT	29	1900	1	0.9387	0.9615	0.9850	1.0	1.0	1.0	1.0	0.9500	1.0	1.0	1.0	1605	0.0181	N	
WB TH + RT	447	1900	2	0.9387	0.9804	0.9850	1.0	1.0	1.0	1.0	1.0	0.9869	1.0	1.0	3399	0.1315	Y	
															Y_c =	0.3727	C_opt =	60.577
															X_c =	0.5634	C_min =	65

		g_i		new g_i	G	Y	AR	Split		Capacity	v/c	d_1	d_2	Total Delay	LOS		
NB LT	Phase 3	15.8		13.8	14.1	3.0	2.7	19.8	20	341.70	0.6438	23.36	9.015	32.38	D		
NB TH + RT	Phase 4	9.4		8.2	8.5	3.0	2.7	14.2	14	196.56	0.1933	25.41	2.182	27.59	C	31.67	D
SB LT	Phase 3	15.8		13.8	14.1	3.0	2.7	19.8	20	344.95	0.1131	20.67	0.664	21.33	C		
SB TH + RT	Phase 4	9.4		8.2	8.5	3.0	2.7	14.2	14	194.15	0.6438	26.99	15.299	42.29	E	37.30	D
EB LT	Phase 1	2.7	bumped to 7	7.7	7	3.8	1.9	12.7	13	195.30	0.1946	25.85	2.213	28.07	C		
EB TH + RT	Phase 2	15.2		13.3	12.6	3.8	1.9	18.3	18	666.86	0.5803	23.35	3.662	27.01	C	27.10	C
WB LT	Phase 1	2.7		7.7	7	3.8	1.9	12.7	13	190.09	0.1526	25.72	1.697	27.42	C		
WB TH + RT	Phase 2	15.2		13.3	12.6	3.8	1.9	18.3	18	694.27	0.6438	23.70	4.557	28.25	C	28.20	C
	Y_c2,3&4 =	0.3497		TRUE				65.0	65					Intersection Delay	29.65		
	X_c2,3&4 =	0.5180												Intersection LOS	C		

AM Future w/ Site Conditions (90 second cycle length)

	Volume	s_o	N	f_w	f_HV	f_g	f_p	f_bb	f_a	f_LU	f_LT	f_RT	f_Lpb	f_Rpb	s	v/s	Critical?	
NB LT	220	1900	1	0.9387	0.9699	0.9800	1.0	1.0	1.0	1.0	0.9500	1.0	1.0	1.0	1610	0.1366	Y	
NB TH + RT	38	1900	1	0.9387	0.9320	0.9800	1.0	1.0	1.0	1.0	1.0	0.9526	1.0	1.0	1552	0.0245	N	
SB LT	39	1900	1	0.9387	0.9804	0.9788	1.0	1.0	1.0	1.0	0.9500	1.0	1.0	1.0	1626	0.0240	N	
SB TH + RT	125	1900	1	0.9387	0.9804	0.9788	1.0	1.0	1.0	1.0	1.0	0.8956	1.0	1.0	1533	0.0816	Y	
EB LT	38	1900	1	0.9387	0.9804	0.9925	1.0	1.0	1.0	1.0	0.9500	1.0	1.0	1.0	1649	0.0230	Y	
EB TH + RT	387	1900	2	0.9387	0.9709	0.9925	1.0	1.0	1.0	1.0	1.0	0.9500	1.0	1.0	3265	0.1185	N	
WB LT	29	1900	1	0.9387	0.9615	0.9850	1.0	1.0	1.0	1.0	0.9500	1.0	1.0	1.0	1605	0.0181	N	
WB TH + RT	447	1900	2	0.9387	0.9804	0.9850	1.0	1.0	1.0	1.0	1.0	0.9869	1.0	1.0	3399	0.1315	Y	
															Y_c =	0.3727	C_opt =	60.577
															X_c =	0.4933	C_min =	90

		g_i		new g_i	G	Y	AR	Split		Capacity	v/c	d_1	d_2	Total Delay	LOS		
NB LT	Phase 3	24.9	time to readjust	23.6	23.8	3.0	2.7	29.5	29	421.56	0.5219	28.41	4.564	32.97	C		
NB TH + RT	Phase 4	14.9		14.1	14.4	3.0	2.7	20.1	20	242.50	0.1567	32.84	1.374	34.21	C	33.15	C
SB LT	Phase 3	24.9		23.6	23.8	3.0	2.7	29.5	29	425.56	0.0916	25.13	0.426	25.55	C		
SB TH + RT	Phase 4	14.9	time to readjust	14.1	14.4	3.0	2.7	20.1	20	239.52	0.5219	34.88	7.912	42.79	D	38.69	D
EB LT	Phase 1	4.2	bumped up to 7	7.7	7	3.8	1.9	12.7	13	141.05	0.2694	38.52	4.640	43.16	D		
EB TH + RT	Phase 2	24.0		22.7	22	3.8	1.9	27.7	28	822.71	0.4704	28.57	1.928	30.49	C	31.63	C
WB LT	Phase 1	4.2		7.7	7	3.8	1.9	12.7	13	137.29	0.2112	38.32	3.477	41.80	D		
WB TH + RT	Phase 2	24.0	time to readjust	22.7	22	3.8	1.9	27.7	28	856.53	0.5219	28.99	2.270	31.26	C	31.90	C
	Y_c2,3&4 =	0.3497		TRUE				90.0	90					Intersection Delay	32.90		
	X_c2,3&4 =	0.4482												Intersection LOS	C		

PM Future w/ Site Conditions (75 second cycle length)

	Volume	s_o	N	f_w	f_HV	f_g	f_p	f_bb	f_a	f_LU	f_LT	f_RT	f_Lpb	f_Rpb	s	v/s	Critical?	
NB LT	163	1900	1	0.9387	0.9579	0.9800	1.0	1.0	1.0	1.0	0.9500	1.0	1.0	1.0	1590	0.1025	Y	
NB TH + RT	42	1900	1	0.9387	0.9804	0.9800	1.0	1.0	1.0	1.0	1.0	0.9393	1.0	1.0	1609	0.0261	N	
SB LT	35	1900	1	0.9387	0.9804	0.9788	1.0	1.0	1.0	1.0	0.9500	1.0	1.0	1.0	1626	0.0215	N	
SB TH + RT	123	1900	1	0.9387	0.9804	0.9788	1.0	1.0	1.0	1.0	1.0	0.9402	1.0	1.0	1609	0.0764	Y	
EB LT	106	1900	1	0.9387	0.9804	0.9925	1.0	1.0	1.0	1.0	0.9500	1.0	1.0	1.0	1649	0.0643	Y	
EB TH + RT	792	1900	2	0.9387	0.9728	0.9925	1.0	1.0	1.0	1.0	1.0	0.9498	1.0	1.0	3271	0.2421	Y	
WB LT	48	1900	1	0.9387	0.9756	0.9850	1.0	1.0	1.0	1.0	0.9500	1.0	1.0	1.0	1628	0.0295	N	
WB TH + RT	375	1900	2	0.9387	0.9785	0.9850	1.0	1.0	1.0	1.0	1.0	0.9832	1.0	1.0	3380	0.1109	N	
															Y_c =	0.4854	C_opt =	73.838
															X_c =	0.6868	C_min =	75

		g_i		new g_i	G	Y	AR	Split		Capacity	v/c	d_1	d_2	Total Delay	LOS		
NB LT	Phase 3	11.2	time to readjust	11.0	11.3	3.0	2.7	17.0	17	233.82	0.6971	30.40	15.871	46.27	D		
NB TH + RT	Phase 4	8.3		8.2	8.6	3.0	2.7	14.3	14	176.48	0.2380	30.52	3.156	33.68	C	43.69	D
SB LT	Phase 3	11.2		11.0	11.3	3.0	2.7	17.0	17	239.01	0.1464	27.88	1.288	29.17	C		
SB TH + RT	Phase 4	8.3	time to readjust	8.2	8.6	3.0	2.7	14.3	14	176.44	0.6971	32.19	20.422	52.61	D	47.42	D
EB LT	Phase 1	7.0	bumped up to 7	7.7	7	3.8	1.9	12.7	13	169.26	0.6263	32.27	16.250	48.52	D		
EB TH + RT	Phase 2	26.4	time to readjust	26.0	25.3	3.8	1.9	31.0	31	1136.09	0.6971	21.08	3.554	24.63	C	27.45	C
WB LT	Phase 1	7.0		7.7	7	3.8	1.9	12.7	13	167.16	0.2872	31.11	4.281	35.39	D		
WB TH + RT	Phase 2	26.4		26.0	25.3	3.8	1.9	31.0	31	1174.00	0.3194	17.97	0.718	18.69	B	20.58	C
	Y_c2,3&4 =	0.4211		TRUE				75.0	75					Intersection Delay	29.58		
	X_c2,3&4 =	0.5791												Intersection LOS	C		

Figure D36: Spreadsheet Math Part 3