RSG INC.
TRANSPORTATION

Staniford Farms, LLC Traffic Impact Study

Burlington, VT

May 2011

DATA ■ ANALYSIS ■ SOLUTIONS

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TABLE OF CONTENTS

1.0 INTRODUCTION 1

2.0 PROJECT DESCRIPTION 1

3.0 GEOGRAPHIC SCOPE OF STUDY 2

3.1 Local Highway Network, Traffic & Conditions 4

3.2 Other Development Volumes 4

3.3 Volume Adjustment Factors 4

3.4 Trip Generation 5

3.5 Trip Distribution 5

3.6 Scenario Volume Graphics 6

4.0 CONGESTION ANALYSIS 8

4.1 Level-of-Service Definition 8

4.2 Level-of-Service Results 9

5.0 SAFETY ANALYSIS 10

5.1 Crash Histories 10

5.2 Sight Distances 12

5.3 Turn Lane Warrant Analysis 13

5.4 Safety Conclusions 14

6.0 PEDESTRIAN AND BICYCLE FACILITIES 15

7.0 SUMMARY AND CONCLUSIONS 17

LIST OF FIGURES

Figure 1: Site Plan 2
Figure 2. Study Area and Study Intersection 3
Figure 3. Adjusted Development Traffic 6
Figure 4. 2012 Peak Hours No Build 6
Figure 5. 2012 Peak Hours Build 7
Figure 6. 2017 Peak Hours No Build 7
Figure 7. 2017 Peak Hours Build 7
Figure 8. Sight Distance Photographs at North Ave and Staniford Rd 13
Figure 9. View from Bike Path (looking west along Appletree Point Road) 16
Figure 10.Staniford Road Looking West Toward the Bike Path 16

LIST OF TABLES

Table 1. Trip Generation - ODV 4
Table 2. Trip Generation - Proposed Land Uses 5
Table 3. Trip Distribution 6
Table 4. Level-of-Service Criteria for Signalized and Unsignalized Intersections 8
Table 5. AM Peak Hour LOS Results 9
Table 6. PM Peak Hour LOS Results 10
Table 7. Reported Crashes (2005-2009) at North Avenue and Staniford Road (mile markers 2.76-2.78) 11
Table 8. High Crash Location Intersection Analysis 11
Table 9. Left Turn Lane Warrant Summary for the North Avenue – Staniford Road Intersection 14
Table 10. Percentage of a Queue Free State 14
Table 11. Project's Relative Share of Impact on Left Turns 15

1.0 INTRODUCTION

Resource Systems Group has been retained by the Staniford Farms, LLC to evaluate the traffic impacts resulting from the development of the Staniford Farms site, located off of Appletree Point Lane in Burlington, Vermont.

This traffic impact study includes the following items:

- The project description and study scope
- Traffic volumes in 2013 and 2018 with and without the project
- A safety analysis of the study intersection
- A summary with conclusions and recommendations

This study relies upon design standards and analysis procedures documented in the 2000 *Highway Capacity Manual*,[1] *Trip Generation*,[2] *A Policy on Geometric Design of Highways and Streets*,[3] *Manual on Uniform Traffic Control Devices* (MUTCD),[4] *Traffic Impact Evaluation: Study and Review Guide*,[5] and the *Vermont State Design Standards*.[6]

2.0 PROJECT DESCRIPTION

The existing site lies off of Appletree Point Lane and consists of several lots, including six single family homes. Redevelopment of theses parcels will result in twenty-three new single family homes, one accessory apartment and four new duplexes (8 dwelling units). One of existing single family homes will be removed and five others will continue to use the project access in addition to the project traffic (Figure 1).

The main access point to the proposed development would be off of North Avenue via Staniford Road.

Realignment of the site access point is proposed which will improve sight distance and reduce vehicle speed entering and exiting the project area. (see Figure 1).

[1] Transportation Research Board, National Research Council, *Highway Capacity Manual* (Washington, DC: National Academy of Sciences, 2000).

[2] Institute of Transportation Engineers, *Trip Generation* 8th Edition (Washington, D.C.: Institute of Transportation Engineers, 2008).

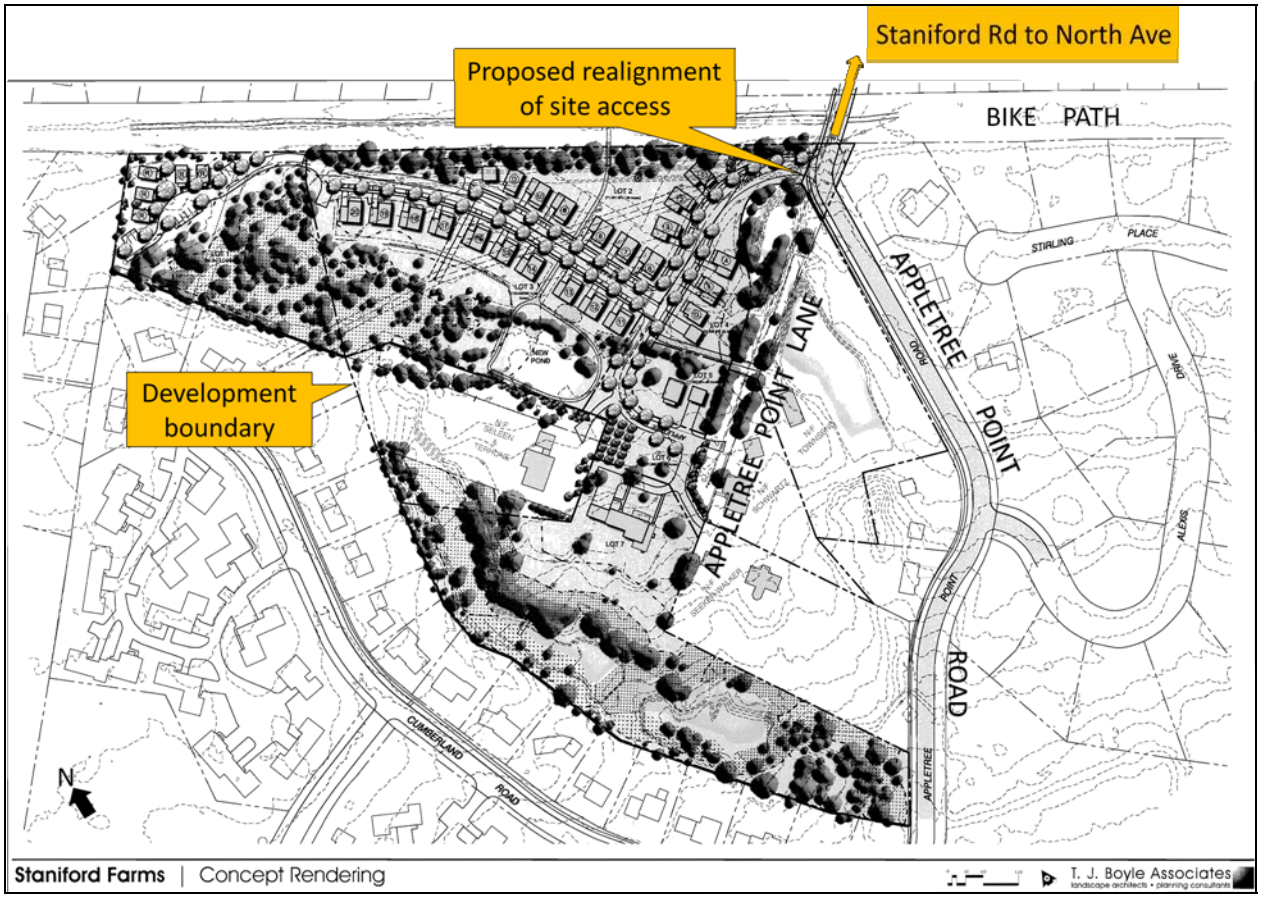
[3] American Association of State Highway and Transportation Officials (AASHTO), *A Policy on Geometric Design of Highways and Streets*, 4th Edition (Washington DC: AASHTO, 2004).

[4] American Traffic Safety Services Association (ATSSA), ITE, and AASHTO, *Manual on Uniform Traffic Control Devices*, 2003 Edition (Washington DC: FHWA, 2003).

[5] Vermont Agency of Transportation, Development Review Section, *Traffic Impact Evaluation Study and Review Guide* (January 2003).

[6] State of Vermont Agency of Transportation, *Vermont State Standards* (Montpelier: VTrans, 1 July 1997).

Figure 1: Site Plan



3.0 Geographic Scope of Study

VTrans guidelines specify that a traffic study should be considered if the proposed development will generate 75 or more peak hour trips. The geographic scope of the study should also include the immediate access points and those intersections or highway segments receiving 75 or more project-generated peak hour trips.[1]

Based on conversations with Carol Weston, Engineer for the City Department of Public Works, one study intersection was identified – North Ave/Staniford Rd as well as a review of the proposed project entrance and safety related to the Bike Path crossing. The location of the study intersection relative to the proposed development site is shown in Figure 2.

[1] Vermont Agency of Transportation, Development Review Section, *Traffic Impact Evaluation Study and Review Guide* (January 2003)2.

Figure 2. Study Area and Study Intersection

Staniford Road runs east-west between Appletree Point Lane on the west and North Avenue on the east. The Burlington Bike Path runs north-south through the study area with the project's eastern edge bordering the Bike Path. Staniford Road is an urban local street within a tight grid network west of North Avenue. This study assumes all project traffic uses the study intersection. However, the grid network allows vehicles to disperse and use the many local streets for access to and from North Avenue. Staniford Road is 30 feet wide with one travel lane in each direction and parking allowed on both sides of the road. The speed limit is 25 mph.

North Avenue within the study area is classified as an urban minor arterial. The speed limit is 30 mph. The road has one travel lane in each direction and is 40 feet wide, allowing parking on both sides of the road.

North Avenue and Staniford Road is a 3-legged unsignalized intersection, stop controlled along Staniford Road. The driveway directly opposite Staniford Road creates an informal 4th leg to this intersection. The width of North Avenue allows for an informal defacto left-turn lane because of the underutilization of on-street parking in the vicinity of Staniford Road. There are a few vehicle trips resulting from a small business of Bessery's Quality Market on the east of this intersection. To be conservative the intersection was modeled without the informal left turn lane on the northbound approach and with the Bessery's Quality Market as the westbound approach.

3.1 Local Highway Network, Traffic & Conditions

In the study area, VTrans considers North Avenue an urban minor arterial with an Annual Average Daily Traffic (AADT) volume of 10,800 vehicles per day, as measured by VTrans in 2009.[1]

Resource Systems Group analyzed the two highest peak hours of traffic at the study intersection: the weekday AM and PM peak hours. It is standard practice in Vermont to analyze traffic conditions in the base year (the year construction is estimated to be complete) and five years in the future. Our base year of analysis will be 2013 – the anticipated construction completion date. Considerations for phasing the development have not been proposed, however full occupancy in 2013 would be a conservative approach, in that event.

Thus, the following scenarios are evaluated:

- 2013 No Build
- 2013 Build, including the net change in traffic due to the proposed land uses
- 2018 No Build
- 2018 Build, including the net change in traffic due to the proposed land uses

3.2 Other Development Volumes

Other development volumes (ODVs) represent trips generated by anticipated developments in the study area. Trips generated by ODVs are typically included in every scenario because we assume they are already present on the road network in the analysis years.

We have spoken with Burlington Planner, Scott Gustin, to identify developments that should be included in the background traffic volumes. The only developments in this area are those associated with the redevelopment of the Dept. of Motor Vehicles site.

Table 1 shows the trips generated by the redevelopment of the Dept. of Motor Vehicles site.

Table 1. Trip Generation - ODV

				Weekday AM		Weekday PM	
ITE Code	ITE Land Use Name	Size		Enter	Exit	Enter	Exit
223	Mid-Rise Apartment	47	units	4	10	11	8
223	Mid-Rise Apartment	33	units	3	7	7	5
252	Senior Adult Housing-Attached	59	occupied units	3	5	6	4
710	General Office Building	8500	sq ft	12	2	2	11
731	State Motor Vehicles Department	7500	sq ft	40	40	64	64
			Sub-Totals	62	63	90	92
			Total Peak Hr	126		182	

3.3 Volume Adjustment Factors

Resource Systems Group conducted turning movement counts at the study intersection on PM Tuesday, March 15, 2011 and AM Wednesday, March 16, 2011. The peak hour traffic volumes from these counts were adjusted to represent the design hour volume (DHV)[2] in 2013 and 2018 using two adjustment factors:

[1] The AADT was measured on North Avenue 130 feet south of Plattsburg Avenue.

[2] The DHV is the 30th highest hour of traffic for the year and is used as the design standard in Vermont.

1. A design hour adjustment factor is based on VTrans Design Hourly Volume (DHV) Policy for urban streets. The DHV adjustment used the AADT along North Avenue at the short-term counter S6D148, located 130 feet south of Plattsburg Avenue, to obtain the seasonal adjustment.[1] The Urban Area "k" factor of 10.4% was applied to the AADT to determine the design hour volume along North Avenue. The DHV design hour adjustment factor increases volumes of the study intersection by 9%.
2. An annual adjustment factor, which represents general background traffic growth, is based on estimated growth in the area. The base year annual adjustment factor holds volumes between 2011 and 2012 the same (annual adjustment factor of 0%). The future year annual adjustment factor is based on the expected growth rate from the *VTrans Continuous Traffic Counter Grouping Study and Regression Analysis*[2] and increases volumes by 1% between 2012 and 2017.

The AM peak hour of traffic at this intersection is from 7:15 AM to 8:15 AM and the PM peak hour is from 4:15 PM to 5:15 PM.

3.4 Trip Generation

Trip generation refers to the number of new vehicle trips originating at or destined for a particular development. Trip generation rates were determined using the Institute of Transportation Engineer's *Trip Generation* report. Two different land uses are considered. The corresponding Land Use Code (LUC) from *Trip Generation* report is as follows:

1) Single Family Detached Housing (LUC 210)
2) Apartment (LUC 220)
3) Residential Condominium/Townhouse – used to determine the trips generated by the proposed duplex units (LUC 230)

Table 2 shows the overall trip generation for the proposed development land uses. Please note that the net change in the number of single family homes proposed is 22 units. Raw turning movement volumes, adjustments, and trip generation calculations are available in Appendix A.

Table 2. Trip Generation - Proposed Land Uses

				Weekday AM		Weekday PM	
ITE Code	ITE Land Use Name	Size		Enter	Exit	Enter	Exit
210	Single Family Detached Housing	22	units	4	12	14	8
220	Apartment	1	unit	0	1	1	0
230	Residential Condominium	8	units	1	3	3	2
			Sub-Totals	5	16	18	10
			Total Peak Hr	21		28	

The net change in traffic due to the proposed development is an increase of 21 trip ends in the AM Peak Hour and an increase of 28 trip ends in the PM Peak Hour.

3.5 Trip Distribution

The distribution of new vehicle trips onto the network followed existing traffic patterns. For example: if 60% of the traffic on North Avenue is northbound in the PM Peak Hour, 60% of the entering/exiting

[1] Typically, continuous traffic counters, which collect data year round, are used in calculating the design hour adjustment factors. It was determined that there were no appropriate continuous traffic counters, so VTrans short term counters were used instead, as prescribed in the VTrans Traffic Impact Study Guidelines.

[2] VTrans - http://www.aot.state.vt.us/Planning/Documents/TrafResearch/Publications/Redbook09.pdf

project traffic is distributed to/from the south. Table 3 summarizes the distribution of project traffic trips.

Table 3. Trip Distribution

Proposed Development	originating or destined ***South***	originating or destined ***North***
Staniford Farms - AM	57%	43%
Staniford Farms - PM	68%	32%

3.6 Scenario Volume Graphics

No Build volumes represent the raw volumes plus the trips expected from the redevelopment of the Dept. of Motor Vehicles site. Adjustments to the intersection traffic volumes were made, as discussed in Section 3.3. Build volumes represent No Build volumes plus the site-generated trips expected by the proposed development. Site-generated trips were distributed onto the study intersection as discussed in Section 3.5.

Figure 3 through Figure 7 show the scenario volumes during the peak hours.

Figure 3. Adjusted Development Traffic

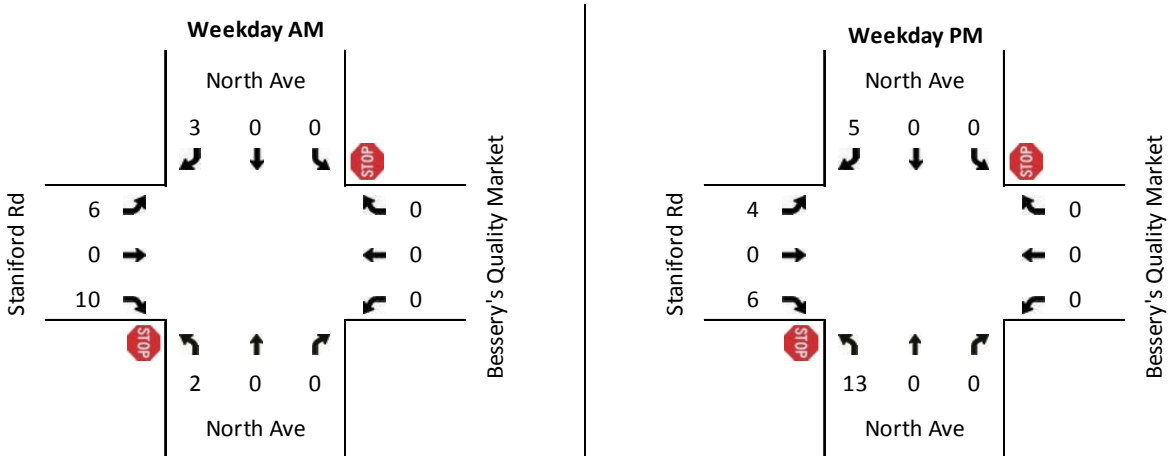


Figure 4. 2013 Peak Hours No Build

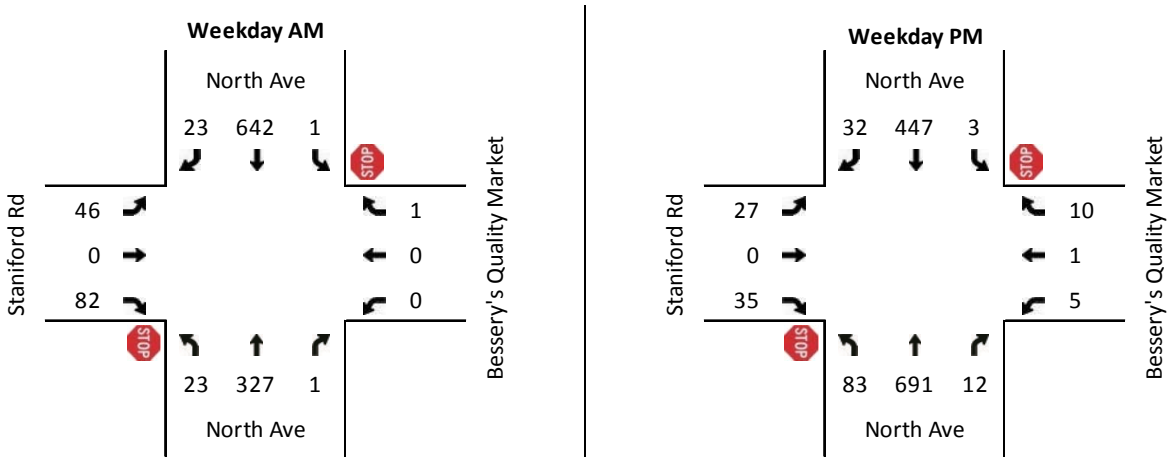


Figure 5. 2013 Peak Hours Build

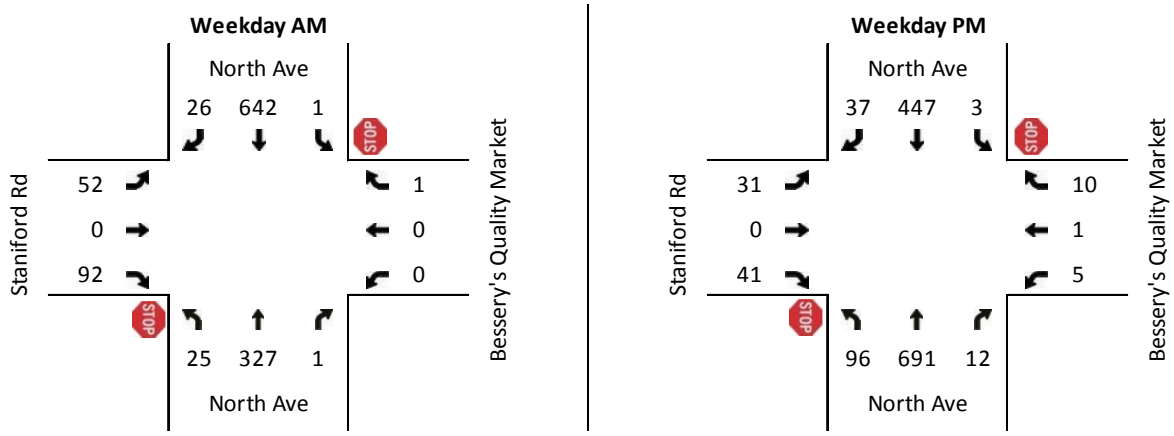


Figure 6. 2018 Peak Hours No Build

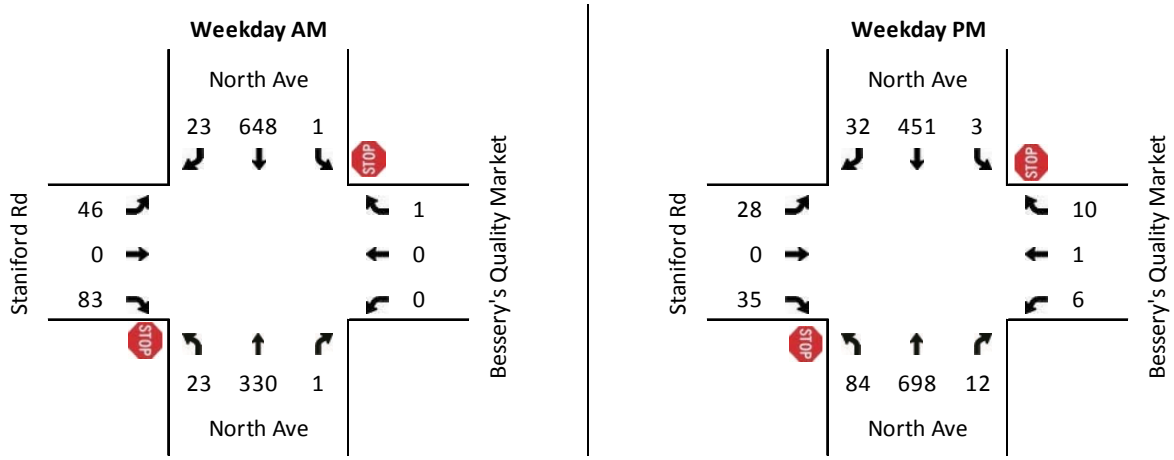
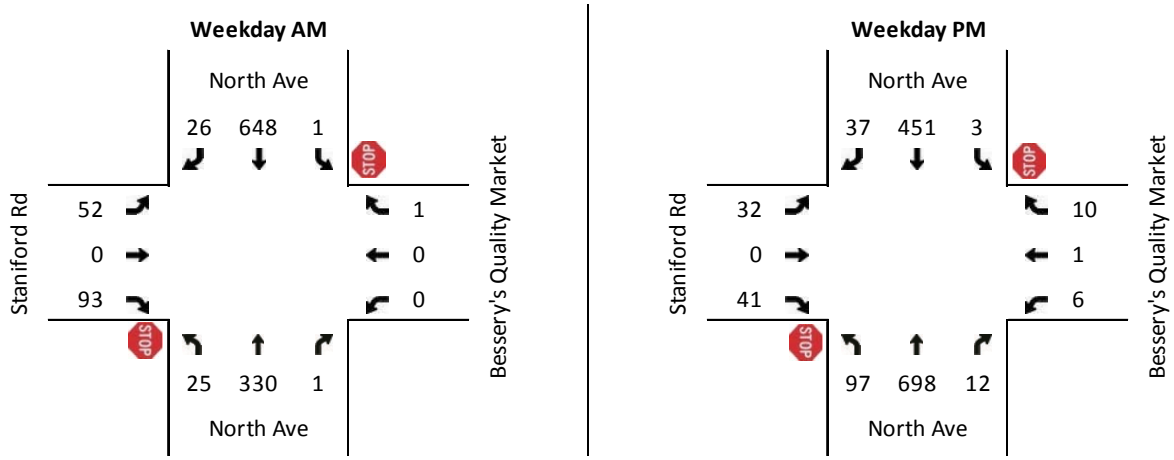


Figure 7. 2018 Peak Hours Build



4.0 CONGESTION ANALYSIS

4.1 Level-of-Service Definition

Level-of-service (LOS) is a qualitative measure describing the operating conditions as perceived by motorists driving in a traffic stream. LOS is estimated using the procedures outlined in the 2000 Highway Capacity Manual. In addition to traffic volumes, key inputs include the number of lanes and control types (signalized or unsignalized).

The 2000 Highway Capacity Manual defines six qualitative grades to describe the level of service at an intersection. Level-of-Service is based on the average control delay per vehicle. Table 4 shows the various LOS grades and descriptions for signalized and unsignalized intersections.

Table 4. Level-of-Service Criteria for Signalized and Unsignalized Intersections

LOS	Characteristics	Unsignalized Total Delay (sec)	Signalized Total Delay (sec)
A	Little or no delay	≤ 10.0	≤ 10.0
B	Short delays	10.1-15.0	10.1-20.0
C	Average delays	15.1-25.0	20.1-35.0
D	Long delays	25.1-35.0	35.1-55.0
E	Very long delays	35.1-50.0	55.1-80.0
F	Extreme delays	> 50.0	> 80.0

North Avenue and Staniford Road are owned and maintained by the City of Burlington. The City of Burlington does not have a level of service guideline; therefore RSG assumes that the City uses the VTrans LOS policy as a general guideline for development review.

The VTrans policy on LOS[1] states that:

> "It is the Agency's policy to design its highways and to require others accessing its facilities to effect improvements that will maintain a LOS "C" for the prescribed design period. In interpreting this policy, LOS refers to the overall LOS for the particular facility as defined in the latest HCM."

The level of service policy provides further guidance for minor stop controlled intersections:

> "For two-way stop controlled intersections, the HCM does not define a procedure for obtaining an overall LOS or a LOS for major street approaches. Therefore, VTrans LOS Policy for two-way stop controlled intersections is to maintain a LOS "D," or better, for side roads with volumes exceeding 100 vehicles/hour for a single lane approach, or 150 vehicles/hour for a two lane approach. No LOS criteria are in effect for volumes less than these.
>
> VTrans' main objective at unsignalized two-way stop controlled intersections is to minimize potential consequences when vehicle operators exit stop-controlled side streets by accepting unsafe gaps in the major street through traffic."

[1] Vermont Agency of Transportation (VTrans) Highway Design "Level of Service" Policy, 31 May 2007.

The traffic volumes exiting the study intersection minor approach do not exceed the 100 vehicle threshold during the PM peak hour, thereby no LOS criteria apply to the study intersection in the PM peak hour. The LOS criteria only apply to the AM peak hour.

4.2 Level-of-Service Results

The 2000 Highway Capacity Manual congestion reports within Synchro (v7), a traffic analysis software package from Trafficware, were used to assess congestion at the study intersection.

Unsignalized intersections experience delay only for vehicle movements that conflict with other movements. Side street stops and left turns are often the only movements that experience delay at unsignalized intersections.

Table 5 and Table 6 present the LOS results during the weekday AM and PM peak hours, respectively. In addition to LOS and delay, VTrans has recommended using Volume to Capacity (V/C) ratios when analyzing capacity at stop-controlled intersections. A V/C ratio less than 1.0 indicates that adequate capacity is available on a specific approach, even if the LOS is F. A ratio of 1.0 (or higher) indicates the facility is at (or over) capacity for the study period. 95th percentile queue lengths (given in feet) are also shown for each movement at the study intersection.

Detailed Synchro LOS worksheets are available in Appendix B.

Table 5. AM Peak Hour LOS Results

AM	2013 No Build				2013 Build			
North Ave/Staniford Rd	LOS	Delay (sec/veh)	Queue Length (ft)	v/c	LOS	Delay (sec/veh)	Queue Length (ft)	v/c
EB, Staniford Rd	D	29	62	0.48	D	32	77	0.55
WB, Bessery's Quality Market	B	10	0	0	B	10	0	0
NB, North Ave	A	1	2	0.03	A	1	2	0.03
SB, North Ave	A	0	0	0	A	0	0	0

AM	2018 No Build				2018 Build			
North Ave/Staniford Rd	LOS	Delay (sec/veh)	Queue Length (ft)	v/c	LOS	33	Queue Length (ft)	v/c
EB, Staniford Rd	D	29	64	0.49	D	32	79	0.56
WB, Bessery's Quality Market	B	10	0	0	B	10	0	0
NB, North Ave	A	1	2	0.03	A	1	2	0.03
SB, North Ave	A	0	0	0	A	0	0	0

Table 6. PM Peak Hour LOS Results

PM	2013 No Build				2013 Build			
North Ave/Staniford Rd	LOS	Delay (sec/veh)	Queue Length (ft)	v/c	LOS	Delay (sec/veh)	Queue Length (ft)	v/c
EB, Staniford Rd	E	36	40	0.37	E	42	52	0.45
WB, Bessery's Quality Market	D	27	8	0.1	D	29	8	0.1
NB, North Ave	A	2	7	0.09	A	3	8	0.1
SB, North Ave	A	0	0	0	A	0	0	0

PM	2018 No Build				2018 Build			
North Ave/Staniford Rd	LOS	Delay (sec/veh)	Queue Length (ft)	v/c	LOS	Delay (sec/veh)	Queue Length (ft)	v/c
EB, Staniford Rd	E	38	43	0.39	E	44	56	0.47
WB, Bessery's Quality Market	D	29	9	0.11	D	31	10	0.12
NB, North Ave	A	2	7	0.09	A	3	8	0.1
SB, North Ave	A	0	0	0	A	0	0	0

These results show all approaches at the study intersection except for the eastbound approach are operating under acceptable conditions in all AM and PM peak hour scenarios. Although the eastbound approach operates under LOS of E during the PM peak hour, this is not caused by the proposed development. Note that the traffic volume on the Staniford Road approach does not meet the threshold of 100 cars in this period, which would be necessary to apply the VTrans LOS criteria. In this case volume/capacity criteria applies, which shows ample capacity for this approach (0.47 v/c in the 2018 build condition).

5.0 Safety Analysis

5.1 Crash Histories

Crash histories were collected from VTrans (2005-2009) at the study intersection. VTrans maintains a statewide database of all reported crashes along all state highways and federal aid road segments.[1]

A reportable crash is a collision with at least one of the following results caused by the event:

- property damage exceeding $1,000
- personal injury
- fatality

In order to be classified as a High Crash Location (HCL), an intersection or road section (0.3 mile section) must meet the following two conditions: 1) it must have at least 5 accidents per year over a 5-year period; and 2) the Actual Crash rate must exceed the Critical Crash Rate.

Based on the latest release of the VTrans High Crash Location Report (2003-2007) the roadway segment of North Avenue, north and south of Staniford Road is classified as a High Crash Location (HCL). The data from 2003 to 2007 indicates that 54 crashes occurred between mile markers 2.6 (Gosse Court) and 2.9 (Green Acres Drive). The North Avenue and Staniford Road intersection is located approximately at mile marker 2.77. For urban areas, the length of influence for each approach on an intersection is 0.01 mile.

[1] This data is exempt from Discovery or Admission under 23 U.S.C. 409.

Table 7 explains the crashes that have occurred at the intersection of North Avenue and Staniford Road (mile markers 2.76-2.78). Five out of thirteen of these crashes occurred in the northbound direction, and were rear end and broadside collisions.

Table 7. Reported Crashes (2005-2009) at North Avenue and Staniford Road (mile markers 2.76-2.78)

Date	MM	Time	Weather	Contributing Circumstances	Type of Collision	# of Injuries	# of Fatalities	Direction
10/12/2005	2.77	18:24	Cloudy	No improper driving	No Turns, Thru moves only, Broadside --^<--	0	0	N
11/4/2005	2.77	15:26	Cloudy	No improper driving, Inattention, Distracted	Rear End	0	0	E
7/2/2006	2.77	18:59	Clear	Inattention, No improper driving	Rear End	0	0	N
10/30/2006	2.76	11:09	Clear	Followed too closely, Distracted	Rear End	0	0	N
11/12/2006	2.78	10:54	Unknown	-	Other	0	0	-
3/17/2007	2.77	14:00	Snow	Driving too fast for conditions, Under the influence of medication/drugs/alcohol, No improper driving	No Turns, Thru moves only, Broadside ^<	2	0	-
5/21/2007	2.77	13:00	Clear	Inattention, No improper driving	Rear End	0	0	N
6/28/2007	2.77	18:40	Clear	Unknown, Failed to yield right of way	No Turns, Thru moves only, Broadside ^<	3	0	S
7/9/2007	2.78	11:35	Rain	Under the influence of medication/drugs/alcohol, No improper driving	Rear End	0	0	-
7/29/2007	2.77	22:28	Clear	Under the influence of medication/drugs/alcohol	Same Direction Sideswipe	0	0	-
8/7/2007	2.77	-	Clear	Unknown	Rear End	0	0	N
3/13/2009	2.77	7:31	Clear	Inattention, No improper driving	No Turns, Thru moves only, Broadside ^<	0	0	E
4/10/2009	2.77	7:47	Clear	No improper driving, Distracted	Rear End	0	0	S

RSG conducted a high crash analysis using the latest available data and also concluded the intersection is not a High Crash Location, as shown in Table 8 below.

Table 8. High Crash Location Intersection Analysis

INTERSECTION NAME	North Ave & Staniford Rd
Number of Years	5
Total Crashes	13
ADT - EB Approach	1700
ADT - WB Approach	300
ADT - NB Approach	10800
ADT - SB Approach	10800
Statewide Average Crash Rate for Similar Intersections	0.495
Actual Observed Crash Rate (crashes per million vehicles)	0.604
Statewide Critical Rate for similar intersections	0.863
Actual/Critical Ratio	**0.700**
High Crash Location	**No**

Table 8 shows that the average rate of observed crashes at the study intersection is slightly above the statewide average crash rate for similar intersections of minor arterials and non-federal aid collectors. However, the analysis also indicates the actual to critical ratio for establishing High Crash Locations is less than 1 indicating the intersection does not qualify as a high crash location.

5.2 Sight Distances

Stopping sight distance is the distance required for a vehicle, traveling at the design speed, to stop before reaching a stationary object in its path, such as a stopped vehicle. Intersection (or corner) sight distance is the distance required for drivers to stop or adjust their speed, as appropriate, to avoid having to slow down a potentially conflicting vehicle leaving an intersection.

The provision of adequate stopping sight distance is critical for safe operations. The *2004 Policy on Geometric Design of Highways and Streets*[1] states that, "[i]f the available sight distance for an entering or crossing vehicle is at least equal to the appropriate stopping sight distance for the major road, then drivers have sufficient sight distance to anticipate and avoid collisions." The *2004 Policy on Geometric Design of Highways and Streets*[2] goes on to state that, "intersection sight distances that exceed stopping sight distances are desirable along the major road."

In the field, the available stopping sight distance is measured from a point 3.5 feet above the road surface of the major road approach lanes to a point 2.0 feet above the road surface at the stop bar of the minor street approach.[3] The available intersection sight distance is measured from a point 3.5 feet above the road surface at a point on the minor road approach 14.5 feet from the stop bar to a point 3.5 feet above the road surface of the major road approach lanes.[4] The minimum stopping sight distances are calculated based on factors such as design speed, response times, and grades as reported in the *2004 Policy on Geometric Design of Highways and Streets.*[5]

The required corner sight distance for vehicles making a left from North Ave onto Staniford Rd is 245 feet. The required corner sight distance for vehicles making a left (looking right) turn movement from Staniford Rd is 335 feet. The required corner sight distance for vehicles making a right (looking left) turn movement from Staniford Rd is 290 feet. Available sight distances are well in excess of the required values in each of these cases.

Figure 8 shows photographs taken in the field of the approximate sight distances for the study intersection.

See also sight distance discussion in Section 6.0 Pedestrian and Bicycle Facilities.

---

[1] American Association of State Highway and Transportation Officials, *A Policy on Geometric Design of Highways and Streets*, Fifth Edition (Washington D.C.: American Association of State Highway and Transportation Officials, 2004), pg. 651.

[2] American Association of State Highway and Transportation Officials, *A Policy on Geometric Design of Highways and Streets*, Fifth Edition (Washington D.C.: American Association of State Highway and Transportation Officials, 2004), pg. 651.

[3] American Association of State Highway and Transportation Officials, *A Policy on Geometric Design of Highways and Streets*, Fifth Edition (Washington D.C.: American Association of State Highway and Transportation Officials, 2004), pg. 127.

[4] American Association of State Highway and Transportation Officials, *A Policy on Geometric Design of Highways and Streets*, Fifth Edition (Washington D.C.: American Association of State Highway and Transportation Officials, 2004), pg. 653,657, 659.

[5] American Association of State Highway and Transportation Officials, *A Policy on Geometric Design of Highways and Streets*, Fifth Edition (Washington D.C.: American Association of State Highway and Transportation Officials, 2004), pg. 659.

Figure 8. Sight Distance Photographs at North Ave and Staniford Rd

View along North Ave Looking to Staniford Rd

View along North Ave Looking South

View along Staniford Rd Looking South

View along Staniford Rd Looking North

5.3 Turn Lane Warrant Analysis

Using the scenario volumes, a left-turn lane warrant was evaluated at the study intersection. The left turn warrant analysis was conducted using the accepted VTrans methodology, based on Harmelink's 1967 study. A second method developed by Kikuchi and Chakroborty (1991), which modified the Harmelink equation to correct errors in its application of queuing theory[1] was also evaluated. Both methods provided the same results during the AM and PM peak hours.

Table 9 shows that a left turn lane met applicable warrants during the PM peak hour under all analysis scenarios. The AM peak hour was not warranted.

[1] Larson, Larry & Fred L. Mannering, *Method for Prioritizing Intersection Improvements*, January 1997, Washington State Transportation Commission, Department of Transportation and U.S. Department of Transportation, Federal Highway Administration, 2 June 2003

Table 9. Left Turn Lane Warrant Summary for the North Avenue – Staniford Road Intersection

North Avenue - Staniford Road	NBL Warranted? Harmelink	Kikuchi & Chakraborty
AM Peak Hour		
2013 No Build	N	N
2013 Build	N	N
2018 No Build	N	N
2018 Build	N	N
PM Peak Hour		
2013 No Build	Y	Y
2013 Build	Y	Y
2018 No Build	Y	Y
2018 Build	Y	Y

Section 4.2 indicates that the North Avenue approach at the study intersection has minimal delay during all analysis scenarios. In addition to delay on an approach another helpful indicator of potential impacts is the Percentage of a Queue Free State. The measure provided by the Highway Capacity Software presents the probability that left turning vehicles will not block through vehicles.

Table 10. Percentage of a Queue Free State

AM Peak Hour	*Northbound Left - North Avenue*
2013 No Build	97
2013 Build	97
2018 No Build	97
2018 Build	97
PM Peak Hour	
2013 No Build	91
2013 Build	90
2018 No Build	91
2018 Build	90

As shown in Table 10, through traffic on North Avenue has a chance of encountering a vehicle waiting to turn left less than 10% (free 90% of the time) of the time during the Build condition during the PM peak hour.

5.4 Safety Conclusions

A left turn lane at the northbound of North Avenue is warranted under existing PM peak hour conditions without the proposed project. The crash record at this intersection suggests that a safety benefit may be obtained with the construction of a left turn lane at this location. However, the crash rate at this location is only slightly higher than the average crash rate for similar intersections in Vermont.

In the City's Transportation Plan[1] the City has expressed the objective to reconfigure North Avenue into a '*complete street*' that will accommodate all modes of travel safely and efficiently. Prior to the reconstruction of North Avenue, the City of Burlington has indicated that spot improvements, such as left

[1] Moving Forward Together: Transportation Plan for the City of Burlington. Draft dated August 31, 2007.

turn lane on North Avenue at Staniford Road, be put on hold until a comprehensive corridor study can be performed[1]. We recommend that the applicant make a financial contribution to the City toward future improvements at the North Avenue and Staniford Road intersection at a level commensurate with the development's peak hour traffic impact on that intersection.

Given that the intersection is not a high crash location and that the crash rate is only slightly above the Vermont state average for similar facilities, it is recommended that the developer participate in a cost sharing agreement with the City to fund the project's relative share of the left turn lane reconstruction. Those funds will be used to start a North Avenue Reconstruction Fund.

Table 11 presents the project's relative share of the northbound left turns that would utilize a left turn lane along North Avenue. Approximately 14% of the vehicles assumed to use the left turn lane would be generated by the proposed project.

Table 11. Project's Relative Share of Impact on Left Turns

	Left Turns PM Peak Hour			
Year	No Build Traffic Volume (veh/hr)	Project Vehicle Trips (veh/hr)	Build Traffic Volume (veh/hr)	Share of Project Traffic
2013	83	13	96	**14%**
2018	84	13	97	**13%**

The cost sharing provides for a fair share contribution towards ameliorating an existing deficiency.

The left-turn warrant analysis worksheets are available in Appendix C.

6.0 Pedestrian and Bicycle Facilities

The proposed project is located within a dense residential neighborhood with many opportunities for bicycling and walking. The Burlington Bike Path runs along the eastern edge of the proposed development, which allows for various recreation uses as well as a direct connection to downtown Burlington. RSG was asked to provide the developer with recommendations to improve pedestrian and bicycle facilities in the project vicinity.

Based on VTrans Crosswalk Guidelines (2004), the AADT on Staniford Road at the Bike Path crossing does not meet the criteria for a marked crosswalk installation. If we assume the bike path is a vehicular crossing, then the MUTCD warrants for a four-way stop are also not met, even if we assume the peak traffic occurs on both Staniford Road and the Bike Path at the same time[2]. Furthermore , the AASHTO Guide for the Development of Bicycle Facilities states "Four-way stops at path-roadway intersections are not recommended because of frequent confusion about or disregard for right of way rules. Yield signs may be acceptable at some locations, such as low-volume, low-speed neighborhood streets."[3]

The City of Burlington has recently enhanced the sidewalk along Staniford on the north side between the Bike Path and Western Avenue, connecting Appletree Point Road to North Avenue. There are speed bumps on approach from either side of the bike path (see Figure 9 & 10).

[1] Discussions with Carol Duncan, City Engineer.

[2] Peak traffic on the Bike Path typically occurs on Saturday or Sunday.

[3] AASHTO American Assoc. State Highway Officials, p. 50.

Vehicular sight distance from the west is limited due to a utility pole and control box, as well as a stockade fence (Figure 9). In addition vehicles speeds from this direction tend to be excessive due to the long straight section of Appletree Point Road prior to the sharp curve just west of the path crossing. The speed bumps here are quite subtle and appear to do little to slow traffic.

Figure 9. View from Bike Path (looking west along Appletree Point Road)

Figure 10.Staniford Road Looking West Toward the Bike Path

7.0 SUMMARY AND CONCLUSIONS

In summary the following items are highlighted from this analysis and report:

- The proposed development is estimated to generate additional 21 AM and 28 PM peak hour vehicle trips,
- Sight distances on North Avenue are more than adequate at Staniford Road,
- Sight distance at the Bike Path crossing on Staniford Road is limited to the west, and existing speed bumps on the Apple Tree Point Road approach to the path are ineffective,
- The proposed realignment of the project driveway will improve the access to the project area because of the improved sight distance and lowered travel speeds of entering vehicles,
- A northbound left turn lane is warranted on North Avenue under existing conditions during the PM peak hour. It remains warranted in all PM peak hours of the analysis. The movement would experience minimal delay and operate queue free for the majority of the time during the peak hours of analysis, and
- The proposed development will not cause unreasonable congestion or unsafe conditions on the local roadway network.

In conclusion, no traffic safety or capacity issues are expected due to the proposed development. Level of service, queuing, and approach capacity at the study intersection are not significantly impacted by the proposed project.

It is recommended that the project participates with the City to make improvements to the existing infrastructure to improve long term traffic conditions on North Avenue and bicycle and pedestrian safety around the Burlington Bike Path crossing of Staniford Road/Apple Tree Point Road. The project itself doesn't trigger the need for these recommendations as both of these issues predate the project proposal; rather, the project will improve traffic safety in the project vicinity if these recommendations are implemented.

These recommendations include:

- The developer contributes 14% of the cost of re-striping a left turn lane on North Avenue to a North Avenue Reconstruction Fund. The left turn lane could be implemented within the existing curb-to-curb travel area by re-striping North Avenue and restricting on-street parking for a section,
- The developer replace the speed bumps in the vicinity of the bike path to ensure their effectiveness, and
- The developer install "Yield to Pedestrian" signs on either side of the bike path, in accordance with the MUTCD.

These recommendations will provide long term improvements to North Avenue and provide immediate safety improvements to the Bike Path crossing.

END OF STUDY

APPENDIX A

DHV Adjustments, Volume Worksheets

AM

05/13/11 03:31 PM

Raw Count Data

North Ave/Staniford Rd
Burlington, VT
3/16/2011
3rd Wednesday
Resource Systems Group

0	EB	WB	NB	SB	
L	42	0	21	1	
T	0	0	283	554	
R	75	1	1	21	999
Enter	117	1	305	576	999
Exit	2	42	326	629	999
% Trucks	1.7%	0.0%	5.6%	2.8%	
Peds	6	24	0	1	PHF
Peak Hour	7:15 AM - 8:15 AM Peak				0.84

DHV & Annual Adjustments to

2013

DHV ATR/CTC	S6D148	Burlington: North Ave just Nof Cayuga
DHV Poll Group	Urban	
Annl Grwth ATR/CTC ID	S6D148	Burlington: North Ave just Nof Cayuga
Annl Grwth Poll Group	Urban	
TM Count Year	2011	
DHV Adjustment	1.09	
2011-2013 Growth	1.00	
Total Adjustment	1.09	

	DHV Calculations
ATR/CTC Year	2009
ATR/CTC AADT	10,800
2009-2011 Growth	1.01
Corr. AADT	10,908
DHV (Equation)	n/a
DHV (K-Factor)	1,134
Corr. Count	1,037

Adjusted Raw Counts

2013

	EB	WB	NB	SB	
L	46	0	23	1	
T	0	0	310	606	
R	82	1	1	23	1093
Enter	128	1	334	630	1093
Exit	2	46	357	688	1093

ODV Adjustments

Redevelopment of Depart. Of Motor Vehicles

	EB	WB	NB	SB	
L					
T			17	36	
R					53
Enter	0	0	17	36	53
Exit	0	0	17	36	53

No Build

2013

	EB	WB	NB	SB	
L	46	0	23	1	
T	0	0	327	642	
R	82	1	1	23	1146
Enter	128	1	351	666	1146
Exit	2	46	374	724	1146

AM

Trip Generation
22 SF Units (210)

	Enter	Exit	
PM	4	12	16

	EB	WB	NB	SB	
L	4		2		
T					
R	8			2	16
Enter	12	0	2	2	16
Exit	0	4	4	8	16

Trip Generation
4 Duplex Units (230)

	Enter	Exit	
PM	1	3	4

	EB	WB	NB	SB	
L	1		0		
T					
R	2			1	4
Enter	3	0	0	1	4
Exit	0	1	1	2	4

Build
2013

	EB	WB	NB	SB	
L	51	0	25	1	
T	0	0	327	642	
R	92	1	1	26	1166
Enter	143	1	353	669	1166
Exit	2	51	379	734	1166

Annual Adjustment
2018

1.01

Adjusted Raw Counts
2018

	EB	WB	NB	SB	
L	46	0	21	1	
T	0	0	286	560	
R	83	1	1	21	1020
Enter	129	1	308	582	1020
Exit	2	42	333	642	1020

No Build
2018

	EB	WB	NB	SB	
L	46	0	23	1	
T	0	0	330	648	
R	83	1	1	23	1157
Enter	129	1	354	673	1157
Exit	2	46	377	731	1157

Build
2018

	EB	WB	NB	SB	
L	51	0	25	1	
T	0	0	330	648	
R	93	1	1	26	1178
Enter	144	1	356	676	1178
Exit	2	51	382	741	1178

05/13/11 03:31 PM

Raw Count Data

North Ave/Staniford Rd
Burlington, VT
3/15/2011
3rd Tuesday
Resource Systems Group

0	EB	WB	NB	SB	
L	25	5	76	3	
T	0	1	588	383	
R	32	9	11	29	1162
Enter	57	15	675	415	1162
Exit	14	106	622	420	1162
% Trucks	0.0%	0.0%	0.9%	1.4%	
Peds	9	14	0	7	PHF
Peak Hour	4:15 PM - 5:15 PM Peak				0.96

DHV & Annual Adjustments to

2013

DHV ATR/CTC	S6D148	Burlington: North Ave just Nof Cayuga
DHV Poll Group	Urban	
Annl Grwth ATR/CTC ID	S6D148	Burlington: North Ave just Nof Cayuga
Annl Grwth Poll Group	Urban	
TM Count Year	2011	
DHV Adjustment	1.09	
2011-2013 Growth	1.00	
Total Adjustment	1.09	

	DHV Calculations
ATR/CTC Year	2009
ATR/CTC AADT	10,800
2009-2011 Growth	1.01
Corr. AADT	10,908
DHV (Equation)	n/a
DHV (K-Factor)	1,134
Corr. Count	1,037

Adjusted Raw Counts

2013

	EB	WB	NB	SB	
L	27	5	83	3	
T	0	1	643	419	
R	35	10	12	32	1271
Enter	62	16	738	454	1271
Exit	15	116	680	459	1271

ODV Adjustments

Redevelopment of Depart. Of Motor

	EB	WB	NB	SB	
L					
T			48	28	
R					76
Enter	0	0	48	28	76
Exit	0	0	48	28	76

No Build

2013

	EB	WB	NB	SB	
L	27	5	83	3	
T	0	1	691	447	
R	35	10	12	32	1347
Enter	62	16	786	482	1347
Exit	15	116	728	487	1347

PM

Trip Generation
22 SF Units (210)

	Enter	Exit	
PM	14	8	22

	EB	WB	NB	SB	
L	4		10		
T					
R	4			4	22
Enter	8	0	10	4	22
Exit	0	14	4	4	22

Trip Generation
4 Duplex Units (230)

	Enter	Exit	
PM	3	2	5

	EB	WB	NB	SB	
L	1		2		
T					
R	1			1	5
Enter	2	0	2	1	5
Exit	0	3	1	1	5

Build
2013

	EB	WB	NB	SB	
L	32	5	95	3	
T	0	1	691	447	
R	40	10	12	37	1374
Enter	72	16	798	487	1374
Exit	15	133	733	492	1374

Annual Adjustment
2018

1.01

Adjusted Raw Counts
2018

	EB	WB	NB	SB	
L	28	6	84	3	
T	0	1	650	423	
R	35	10	12	32	1284
Enter	63	17	746	459	1284
Exit	15	117	687	464	1284

No Build
2018

	EB	WB	NB	SB	
L	28	6	84	3	
T	0	1	698	451	
R	35	10	12	32	1361
Enter	63	17	794	487	1361
Exit	15	117	736	492	1361

Build
2018

	EB	WB	NB	SB	
L	33	6	96	3	
T	0	1	698	451	
R	40	10	12	37	1388
Enter	73	17	806	492	1388
Exit	15	134	741	497	1388

APPENDIX B

Congestion Analysis & Queue Reports (HCM / Synchro)

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Volume (veh/h)	46	0	82	0	0	1	23	327	1	1	642	23
Sign Control		Stop			Stop			Free			Free	
Grade		0%			0%			0%			0%	
Peak Hour Factor	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92
Hourly flow rate (vph)	50	0	89	0	0	1	25	355	1	1	698	25
Pedestrians												
Lane Width (ft)												
Walking Speed (ft/s)												
Percent Blockage												
Right turn flare (veh)												
Median type								None			None	
Median storage veh)												
Upstream signal (ft)												
pX, platoon unblocked												
vC, conflicting volume	1120	1119	710	1208	1131	356	723			357		
vC1, stage 1 conf vol												
vC2, stage 2 conf vol												
vCu, unblocked vol	1120	1119	710	1208	1131	356	723			357		
tC, single (s)	7.1	6.5	6.2	7.1	6.5	6.2	4.1			4.1		
tC, 2 stage (s)												
tF (s)	3.5	4.0	3.3	3.5	4.0	3.3	2.2			2.2		
p0 queue free %	72	100	79	100	100	100	97			100		
cM capacity (veh/h)	179	201	433	124	197	688	879			1202		

Direction, Lane #	EB 1	WB 1	NB 1	SB 1
Volume Total	139	1	382	724
Volume Left	50	0	25	1
Volume Right	89	1	1	25
cSH	287	688	879	1202
Volume to Capacity	0.48	0.00	0.03	0.00
Queue Length 95th (ft)	62	0	2	0
Control Delay (s)	28.8	10.2	0.9	0.0
Lane LOS	D	B	A	A
Approach Delay (s)	28.8	10.2	0.9	0.0
Approach LOS	D	B		

Intersection Summary			
Average Delay	3.5		
Intersection Capacity Utilization	56.4%	ICU Level of Service	B
Analysis Period (min)	15		

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Volume (veh/h)	51	0	92	0	0	1	25	327	1	1	642	26
Sign Control		Stop			Stop			Free			Free	
Grade		0%			0%			0%			0%	
Peak Hour Factor	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92
Hourly flow rate (vph)	55	0	100	0	0	1	27	355	1	1	698	28
Pedestrians												
Lane Width (ft)												
Walking Speed (ft/s)												
Percent Blockage												
Right turn flare (veh)												
Median type								None			None	
Median storage veh)												
Upstream signal (ft)												
pX, platoon unblocked												
vC, conflicting volume	1126	1125	712	1224	1139	356	726			357		
vC1, stage 1 conf vol												
vC2, stage 2 conf vol												
vCu, unblocked vol	1126	1125	712	1224	1139	356	726			357		
tC, single (s)	7.1	6.5	6.2	7.1	6.5	6.2	4.1			4.1		
tC, 2 stage (s)												
tF (s)	3.5	4.0	3.3	3.5	4.0	3.3	2.2			2.2		
p0 queue free %	69	100	77	100	100	100	97			100		
cM capacity (veh/h)	177	199	432	117	195	688	877			1202		

Direction, Lane #	EB 1	WB 1	NB 1	SB 1
Volume Total	155	1	384	727
Volume Left	55	0	27	1
Volume Right	100	1	1	28
cSH	286	688	877	1202
Volume to Capacity	0.54	0.00	0.03	0.00
Queue Length 95th (ft)	75	0	2	0
Control Delay (s)	31.6	10.2	1.0	0.0
Lane LOS	D	B	A	A
Approach Delay (s)	31.6	10.2	1.0	0.0
Approach LOS	D	B		

Intersection Summary			
Average Delay	4.2		
Intersection Capacity Utilization	58.9%	ICU Level of Service	B
Analysis Period (min)	15		

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Volume (veh/h)	46	0	83	0	0	1	23	330	1	1	648	23
Sign Control		Stop			Stop			Free			Free	
Grade		0%			0%			0%			0%	
Peak Hour Factor	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92
Hourly flow rate (vph)	50	0	90	0	0	1	25	359	1	1	704	25
Pedestrians												
Lane Width (ft)												
Walking Speed (ft/s)												
Percent Blockage												
Right turn flare (veh)												
Median type								None			None	
Median storage veh)												
Upstream signal (ft)												
pX, platoon unblocked												
vC, conflicting volume	1129	1129	717	1218	1141	359	729			360		
vC1, stage 1 conf vol												
vC2, stage 2 conf vol												
vCu, unblocked vol	1129	1129	717	1218	1141	359	729			360		
tC, single (s)	7.1	6.5	6.2	7.1	6.5	6.2	4.1			4.1		
tC, 2 stage (s)												
tF (s)	3.5	4.0	3.3	3.5	4.0	3.3	2.2			2.2		
p0 queue free %	72	100	79	100	100	100	97			100		
cM capacity (veh/h)	177	198	430	121	195	685	875			1199		

Direction, Lane #	EB 1	WB 1	NB 1	SB 1
Volume Total	140	1	385	730
Volume Left	50	0	25	1
Volume Right	90	1	1	25
cSH	284	685	875	1199
Volume to Capacity	0.49	0.00	0.03	0.00
Queue Length 95th (ft)	64	0	2	0
Control Delay (s)	29.4	10.3	0.9	0.0
Lane LOS	D	B	A	A
Approach Delay (s)	29.4	10.3	0.9	0.0
Approach LOS	D	B		

Intersection Summary			
Average Delay	3.6		
Intersection Capacity Utilization	56.8%	ICU Level of Service	B
Analysis Period (min)	15		

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Volume (veh/h)	51	0	93	0	0	1	25	330	1	1	648	26
Sign Control		Stop			Stop			Free			Free	
Grade		0%			0%			0%			0%	
Peak Hour Factor	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92
Hourly flow rate (vph)	55	0	101	0	0	1	27	359	1	1	704	28
Pedestrians												
Lane Width (ft)												
Walking Speed (ft/s)												
Percent Blockage												
Right turn flare (veh)												
Median type								None			None	
Median storage veh)												
Upstream signal (ft)												
pX, platoon unblocked												
vC, conflicting volume	1135	1135	718	1235	1148	359	733			360		
vC1, stage 1 conf vol												
vC2, stage 2 conf vol												
vCu, unblocked vol	1135	1135	718	1235	1148	359	733			360		
tC, single (s)	7.1	6.5	6.2	7.1	6.5	6.2	4.1			4.1		
tC, 2 stage (s)												
tF (s)	3.5	4.0	3.3	3.5	4.0	3.3	2.2			2.2		
p0 queue free %	68	100	76	100	100	100	97			100		
cM capacity (veh/h)	175	196	429	114	192	685	872			1199		

Direction, Lane #	EB 1	WB 1	NB 1	SB 1
Volume Total	157	1	387	734
Volume Left	55	0	27	1
Volume Right	101	1	1	28
cSH	283	685	872	1199
Volume to Capacity	0.55	0.00	0.03	0.00
Queue Length 95th (ft)	78	0	2	0
Control Delay (s)	32.4	10.3	1.0	0.0
Lane LOS	D	B	A	A
Approach Delay (s)	32.4	10.3	1.0	0.0
Approach LOS	D	B		

Intersection Summary			
Average Delay	4.3		
Intersection Capacity Utilization	59.1%	ICU Level of Service	B
Analysis Period (min)	15		

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Volume (veh/h)	27	0	35	5	1	10	83	691	12	3	447	32
Sign Control		Stop			Stop			Free			Free	
Grade		0%			0%			0%			0%	
Peak Hour Factor	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92
Hourly flow rate (vph)	29	0	38	5	1	11	90	751	13	3	486	35
Pedestrians												
Lane Width (ft)												
Walking Speed (ft/s)												
Percent Blockage												
Right turn flare (veh)												
Median type								None			None	
Median storage veh)												
Upstream signal (ft)												
pX, platoon unblocked												
vC, conflicting volume	1459	1454	503	1486	1465	758	521			764		
vC1, stage 1 conf vol												
vC2, stage 2 conf vol												
vCu, unblocked vol	1459	1454	503	1486	1465	758	521			764		
tC, single (s)	7.1	6.5	6.2	7.1	6.5	6.2	4.1			4.1		
tC, 2 stage (s)												
tF (s)	3.5	4.0	3.3	3.5	4.0	3.3	2.2			2.2		
p0 queue free %	70	100	93	94	99	97	91			100		
cM capacity (veh/h)	96	118	568	89	117	407	1046			849		

Direction, Lane #	EB 1	WB 1	NB 1	SB 1
Volume Total	67	17	854	524
Volume Left	29	5	90	3
Volume Right	38	11	13	35
cSH	182	179	1046	849
Volume to Capacity	0.37	0.10	0.09	0.00
Queue Length 95th (ft)	40	8	7	0
Control Delay (s)	36.1	27.2	2.2	0.1
Lane LOS	E	D	A	A
Approach Delay (s)	36.1	27.2	2.2	0.1
Approach LOS	E	D		

Intersection Summary			
Average Delay	3.3		
Intersection Capacity Utilization	82.9%	ICU Level of Service	E
Analysis Period (min)	15		

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Volume (veh/h)	32	0	40	5	1	10	95	691	12	3	447	37
Sign Control		Stop			Stop			Free			Free	
Grade		0%			0%			0%			0%	
Peak Hour Factor	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92
Hourly flow rate (vph)	35	0	43	5	1	11	103	751	13	3	486	40
Pedestrians												
Lane Width (ft)												
Walking Speed (ft/s)												
Percent Blockage												
Right turn flare (veh)												
Median type								None			None	
Median storage veh)												
Upstream signal (ft)												
pX, platoon unblocked												
vC, conflicting volume	1488	1483	506	1520	1497	758	526			764		
vC1, stage 1 conf vol												
vC2, stage 2 conf vol												
vCu, unblocked vol	1488	1483	506	1520	1497	758	526			764		
tC, single (s)	7.1	6.5	6.2	7.1	6.5	6.2	4.1			4.1		
tC, 2 stage (s)												
tF (s)	3.5	4.0	3.3	3.5	4.0	3.3	2.2			2.2		
p0 queue free %	62	100	92	93	99	97	90			100		
cM capacity (veh/h)	91	112	566	83	110	407	1041			849		

Direction, Lane #	EB 1	WB 1	NB 1	SB 1
Volume Total	78	17	867	529
Volume Left	35	5	103	3
Volume Right	43	11	13	40
cSH	171	170	1041	849
Volume to Capacity	0.46	0.10	0.10	0.00
Queue Length 95th (ft)	54	8	8	0
Control Delay (s)	42.8	28.6	2.5	0.1
Lane LOS	E	D	A	A
Approach Delay (s)	42.8	28.6	2.5	0.1
Approach LOS	E	D		

Intersection Summary			
Average Delay	4.0		
Intersection Capacity Utilization	84.8%	ICU Level of Service	E
Analysis Period (min)	15		

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Volume (veh/h)	28	0	35	6	1	10	84	698	12	3	451	32
Sign Control		Stop			Stop			Free			Free	
Grade		0%			0%			0%			0%	
Peak Hour Factor	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92
Hourly flow rate (vph)	30	0	38	7	1	11	91	759	13	3	490	35
Pedestrians												
Lane Width (ft)												
Walking Speed (ft/s)												
Percent Blockage												
Right turn flare (veh)												
Median type								None			None	
Median storage veh)												
Upstream signal (ft)												
pX, platoon unblocked												
vC, conflicting volume	1473	1468	508	1500	1479	765	525			772		
vC1, stage 1 conf vol												
vC2, stage 2 conf vol												
vCu, unblocked vol	1473	1468	508	1500	1479	765	525			772		
tC, single (s)	7.1	6.5	6.2	7.1	6.5	6.2	4.1			4.1		
tC, 2 stage (s)												
tF (s)	3.5	4.0	3.3	3.5	4.0	3.3	2.2			2.2		
p0 queue free %	68	100	93	93	99	97	91			100		
cM capacity (veh/h)	94	116	565	87	114	403	1042			843		

Direction, Lane #	EB 1	WB 1	NB 1	SB 1
Volume Total	68	18	863	528
Volume Left	30	7	91	3
Volume Right	38	11	13	35
cSH	175	166	1042	843
Volume to Capacity	0.39	0.11	0.09	0.00
Queue Length 95th (ft)	43	9	7	0
Control Delay (s)	38.1	29.4	2.2	0.1
Lane LOS	E	D	A	A
Approach Delay (s)	38.1	29.4	2.2	0.1
Approach LOS	E	D		

Intersection Summary			
Average Delay	3.5		
Intersection Capacity Utilization	83.3%	ICU Level of Service	E
Analysis Period (min)	15		

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Volume (veh/h)	33	0	40	6	1	10	96	698	12	3	451	37
Sign Control		Stop			Stop			Free			Free	
Grade		0%			0%			0%			0%	
Peak Hour Factor	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92
Hourly flow rate (vph)	36	0	43	7	1	11	104	759	13	3	490	40
Pedestrians												
Lane Width (ft)												
Walking Speed (ft/s)												
Percent Blockage												
Right turn flare (veh)												
Median type								None			None	
Median storage veh)												
Upstream signal (ft)												
pX, platoon unblocked												
vC, conflicting volume	1502	1497	510	1534	1511	765	530			772		
vC1, stage 1 conf vol												
vC2, stage 2 conf vol												
vCu, unblocked vol	1502	1497	510	1534	1511	765	530			772		
tC, single (s)	7.1	6.5	6.2	7.1	6.5	6.2	4.1			4.1		
tC, 2 stage (s)												
tF (s)	3.5	4.0	3.3	3.5	4.0	3.3	2.2			2.2		
p0 queue free %	60	100	92	92	99	97	90			100		
cM capacity (veh/h)	89	110	563	81	108	403	1037			843		

Direction, Lane #	EB 1	WB 1	NB 1	SB 1
Volume Total	79	18	876	534
Volume Left	36	7	104	3
Volume Right	43	11	13	40
cSH	165	157	1037	843
Volume to Capacity	0.48	0.12	0.10	0.00
Queue Length 95th (ft)	57	10	8	0
Control Delay (s)	45.5	31.0	2.5	0.1
Lane LOS	E	D	A	A
Approach Delay (s)	45.5	31.0	2.5	0.1
Approach LOS	E	D		

Intersection Summary			
Average Delay	4.3		
Intersection Capacity Utilization	85.1%	ICU Level of Service	E
Analysis Period (min)	15		

APPENDIX C

Turn Lane Warrant Analysis Sheets

Intersection:	North Avenue - Staniford Road
Location:	Burlington, Vemront
Analysis Period:	2013 No Build AM Peak Hour

HOURLY VOLUMES:

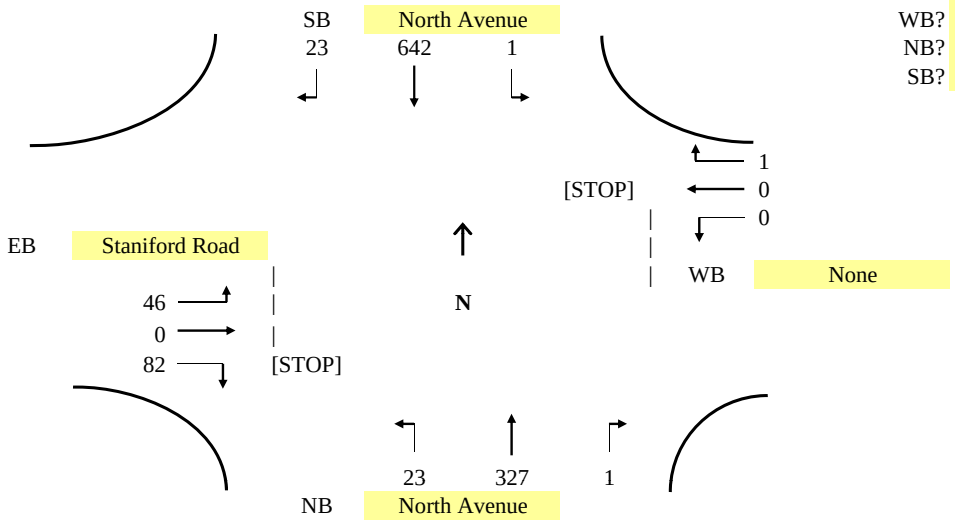
		EB	WB	NB	SB		
Base Volumes:	LT	46	0	23	1		
	TH	0	0	327	642		
	RT	82	1	1	23		
Devel Volumes:	LT						
	TH						
	RT						
Analysis Volumes:	LT	46	0	23	1		
	TH	0	0	327	642		
	RT	82	1	1	23		
	TOTAL	128	1	351	666	1146	vph

VOLUME ADJUSTMENTS:

Time of Day =	AM
EB/WB= Major or Minor?	minor
Base Year =	2013
Analysis Year =	2013
Growth/Year =	1.00
DHV Adjustment =	1.00
Total Vol Adjustment =	1.00
Speed Limit =	30 mph
Add Development?	n

Channelized right turn for:

EB?	N
WB?	N
NB?	N
SB?	N



CALCULATIONS:

LEFT TURN WARRANTS	EB	WB	NB	SB
Advancing Volume:	128	1	351	666
Opposing Volume:	1	128	666	351
% Left Turns:	36%	0%	7%	0%
Adv. Volume not to exceed:	NA	NA	379	3262
Harmelink Lane Warranted?	**NA**	**NA**	**N**	**N**
PLTF	0	1	0	1
PLTT	0	0	1	0
PLTW	0	0	0	0
Adv. Volume not to exceed:	576	1118	422	866
Kikuchi and Chakroborty Lane Warranted?	**NA**	**NA**	**N**	**N**

Intersection: North Avenue - Staniford Road
Location: Burlington, Vemront
Analysis Period: 2013 Build AM Peak Hour

HOURLY VOLUMES:

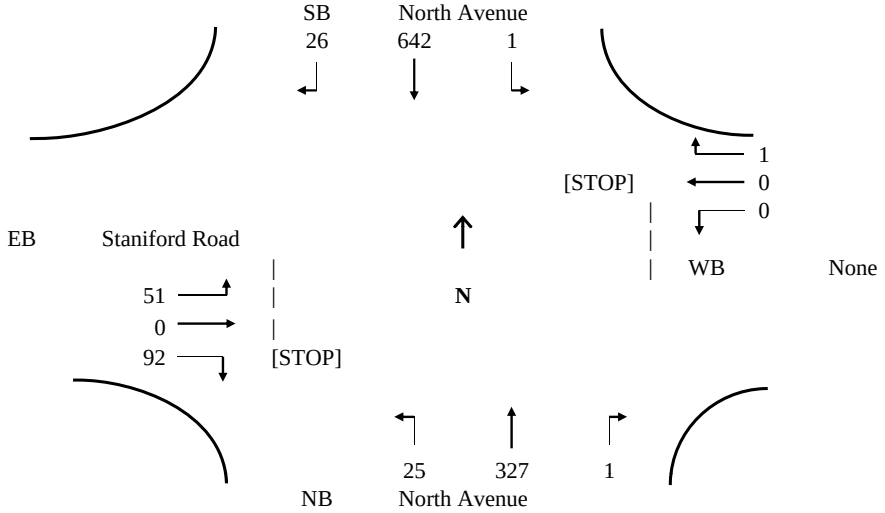
		EB	WB	NB	SB		
Base Volumes:	LT	51	0	25	1		
	TH	0	0	327	642		
	RT	92	1	1	26		
Devel Volumes:	LT						
	TH						
	RT						
Analysis Volumes:	LT	51	0	25	1		
	TH	0	0	327	642		
	RT	92	1	1	26		
	TOTAL	143	1	353	669	1166	vph

VOLUME ADJUSTMENTS:

Time of Day =	AM
EB/WB= Major or Minor?	minor
Base Year =	2013
Analysis Year =	2013
Growth/Year =	1.00
DHV Adjustment =	1.00
Total Vol Adjustment =	1.00
Speed Limit =	30 mph
Add Development?	y

Channelized right turn for:

EB?	N
WB?	N
NB?	N
SB?	N



CALCULATIONS:

LEFT TURN WARRANTS

	EB	WB	NB	SB
Advancing Volume:	143	1	353	669
Opposing Volume:	1	143	669	353
% Left Turns:	36%	0%	7%	0%
Adv. Volume not to exceed:	NA	NA	365	3263
Harmelink Lane Warranted?	**NA**	**NA**	**N**	**N**
PLTF	0	1	0	1
PLTT	0	0	1	0
PLTW	0	0	0	0
AV	576	1099	420	864
Kikuchi and Chakroborty Lane Warranted?	**NA**	**NA**	**N**	**N**

Intersection:	North Avenue - Staniford Road
Location:	Burlington, Vemront
Analysis Period:	2018 No Build AM Peak Hour

HOURLY VOLUMES:

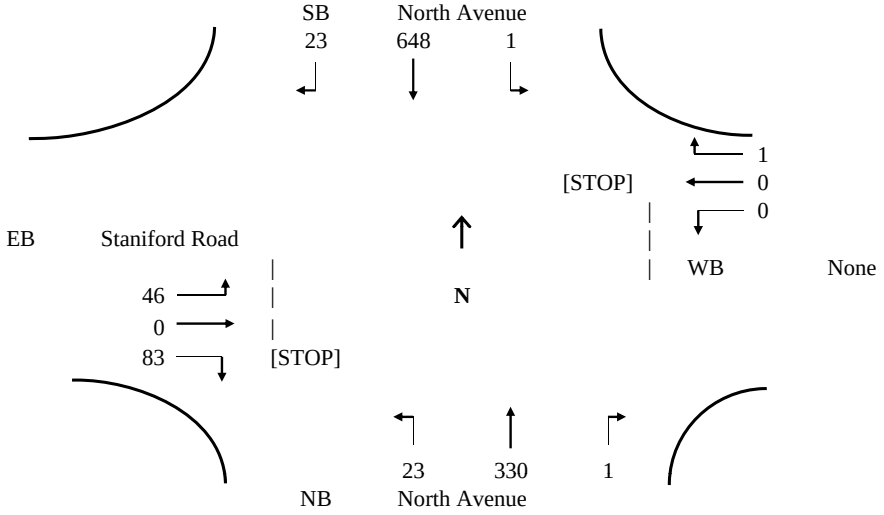
		EB	WB	NB	SB		
Base Volumes:	LT	46	0	23	1		
	TH	0	0	330	648		
	RT	83	1	1	23		
Devel Volumes:	LT						
	TH						
	RT						
Analysis Volumes:	LT	46	0	23	1		
	TH	0	0	330	648		
	RT	83	1	1	23		
	TOTAL	129	1	354	673	1157	vph

VOLUME ADJUSTMENTS:

Time of Day =	AM
EB/WB= Major or Minor?	minor
Base Year =	2018
Analysis Year =	2018
Growth/Year =	1.00
DHV Adjustment =	1.00
Total Vol Adjustment =	1.00
Speed Limit =	30 mph
Add Development?	n

Channelized right turn for:

EB?	N
WB?	N
NB?	N
SB?	N



CALCULATIONS:

LEFT TURN WARRANTS

	EB	WB	NB	SB
Advancing Volume:	129	1	354	673
Opposing Volume:	1	129	673	354
% Left Turns:	36%	0%	7%	0%
Adv. Volume not to exceed:	NA	NA	377	3250
Harmelink Lane Warranted?	**NA**	**NA**	**N**	**N**
PLTF	0	1	0	1
PLTT	0	0	1	0
PLTW	0	0	0	0
AV	576	1117	418	862
Kikuchi and Chakroborty Lane Warranted?	**NA**	**NA**	**N**	**N**

Intersection: North Avenue - Staniford Road
Location: Burlington, Vemront
Analysis Period: 2018 Build AM Peak Hour

HOURLY VOLUMES:

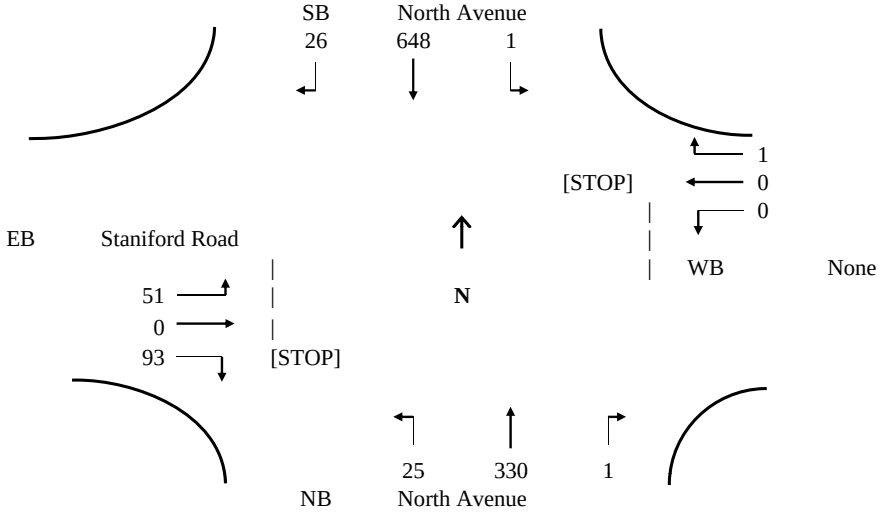
		EB	WB	NB	SB		
Base Volumes:	LT	51	0	25	1		
	TH	0	0	330	648		
	RT	93	1	1	26		
Devel Volumes:	LT						
	TH						
	RT						
Analysis Volumes:	LT	51	0	25	1		
	TH	0	0	330	648		
	RT	93	1	1	26		
	TOTAL	144	1	356	676	1178	vph

VOLUME ADJUSTMENTS:

Time of Day =	AM
EB/WB= Major or Minor?	minor
Base Year =	2018
Analysis Year =	2018
Growth/Year =	1.00
DHV Adjustment =	1.00
Total Vol Adjustment =	1.00
Speed Limit =	30 mph
Add Development?	y

Channelized right turn for:

EB?	N
WB?	N
NB?	N
SB?	N



CALCULATIONS:

LEFT TURN WARRANTS

	EB	WB	NB	SB
Advancing Volume:	144	1	356	676
Opposing Volume:	1	144	676	356
% Left Turns:	36%	0%	7%	0%
Adv. Volume not to exceed:	NA	NA	362	3250
Harmelink Lane Warranted?	**NA**	**NA**	**N**	**N**
PLTF	0	1	0	1
PLTT	0	0	1	0
PLTW	0	0	0	0
AV	576	1097	417	860
Kikuchi and Chakroborty Lane Warranted?	**NA**	**NA**	**N**	**N**

Intersection: North Avenue - Staniford Road
Location: Burlington, Vemront
Analysis Period: 2013 No Build PM Peak Hour

HOURLY VOLUMES:

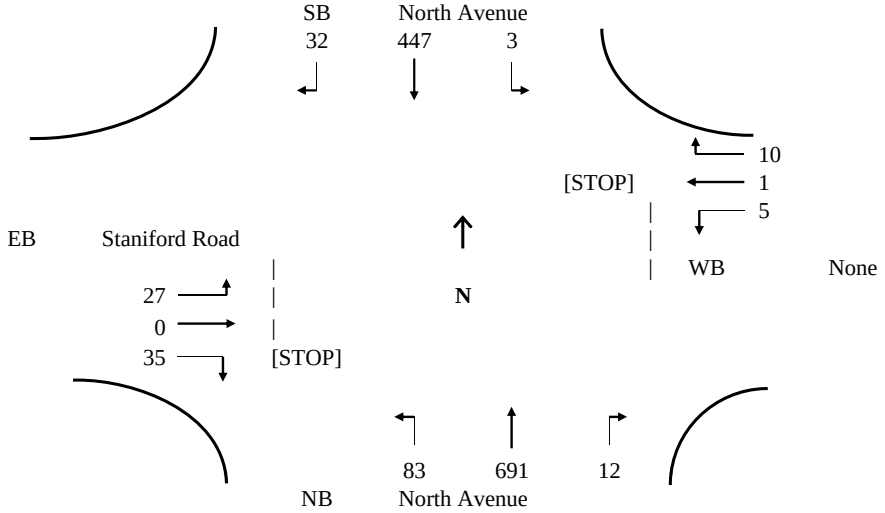
		EB	WB	NB	SB		
Base Volumes:	LT	27	5	83	3		
	TH	0	1	691	447		
	RT	35	10	12	32		
Devel Volumes:	LT						
	TH						
	RT						
Analysis Volumes:	LT	27	5	83	3		
	TH	0	1	691	447		
	RT	35	10	12	32		
	TOTAL	62	16	786	482	1347	vph

VOLUME ADJUSTMENTS:

Time of Day =	PM
EB/WB= Major or Minor?	minor
Base Year =	2013
Analysis Year =	2013
Growth/Year =	1.00
DHV Adjustment =	1.00
Total Vol Adjustment =	1.00
Speed Limit =	30 mph
Add Development?	n

Channelized right turn for:

EB?	N
WB?	N
NB?	N
SB?	N



CALCULATIONS:

LEFT TURN WARRANTS

	EB	WB	NB	SB
Advancing Volume:	62	16	786	482
Opposing Volume:	16	62	482	786
% Left Turns:	44%	33%	11%	1%
Adv. Volume not to exceed:	NA	NA	373	997
Harmelink Lane Warranted?	**NA**	**NA**	**Y**	**N**
PLTF	0	0	0	1
PLTT	0	0	0	0
PLTW	0	0	1	0
AV	566	537	380	524
Kikuchi and Chakroborty Lane Warranted?	**NA**	**NA**	**Y**	**N**

Intersection: North Avenue - Staniford Road
Location: Burlington, Vemront
Analysis Period: 2013 Build PM Peak Hour

HOURLY VOLUMES:

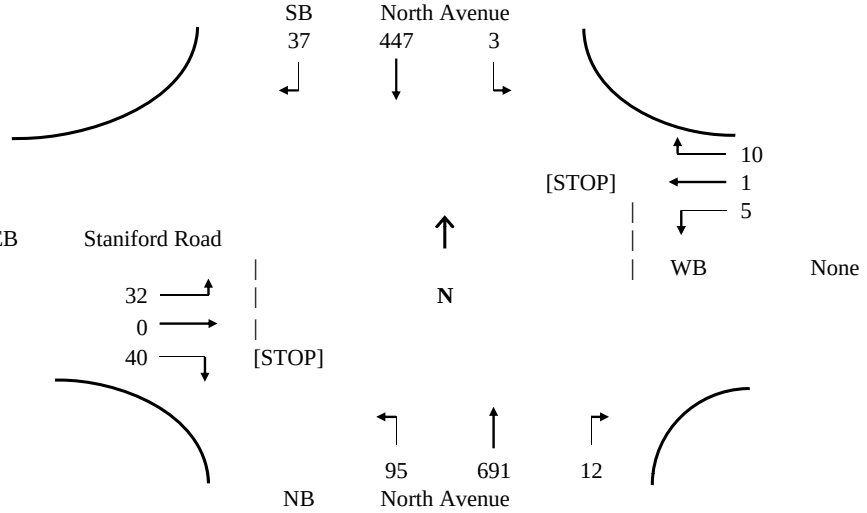
		EB	WB	NB	SB		
Base Volumes:	LT	32	5	95	3		
	TH	0	1	691	447		
	RT	40	10	12	37		
Devel Volumes:	LT						
	TH						
	RT						
Analysis Volumes:	LT	32	5	95	3		
	TH	0	1	691	447		
	RT	40	10	12	37		
	TOTAL	72	16	798	487	1374	vph

VOLUME ADJUSTMENTS:

Time of Day =	PM
EB/WB= Major or Minor?	minor
Base Year =	2013
Analysis Year =	2013
Growth/Year =	1.00
DHV Adjustment =	1.00
Total Vol Adjustment =	1.00
Speed Limit =	30 mph
Add Development?	y

Channelized right turn for:

EB?	N
WB?	N
NB?	N
SB?	N



CALCULATIONS:

LEFT TURN WARRANTS

	EB	WB	NB	SB
Advancing Volume:	72	16	798	487
Opposing Volume:	16	72	487	798
% Left Turns:	45%	33%	12%	1%
Adv. Volume not to exceed:	NA	NA	352	989
Harmelink Lane Warranted?	**NA**	**NA**	**Y**	**N**
PLTF	0	0	0	1
PLTT	0	0	0	0
PLTW	0	0	1	0
AV	566	530	378	517
Kikuchi and Chakroborty Lane Warranted?	**NA**	**NA**	**Y**	**N**

Intersection:	North Avenue - Staniford Road
Location:	Burlington, Vemront
Analysis Period:	2018 No Build PM Peak Hour

HOURLY VOLUMES:

		EB	WB	NB	SB
Base Volumes:	LT	28	6	84	3
	TH	0	1	698	451
	RT	35	10	12	32
Devel Volumes:	LT				
	TH				
	RT				
Analysis Volumes:	LT	28	6	84	3
	TH	0	1	698	451
	RT	35	10	12	32
	TOTAL	63	17	794	487

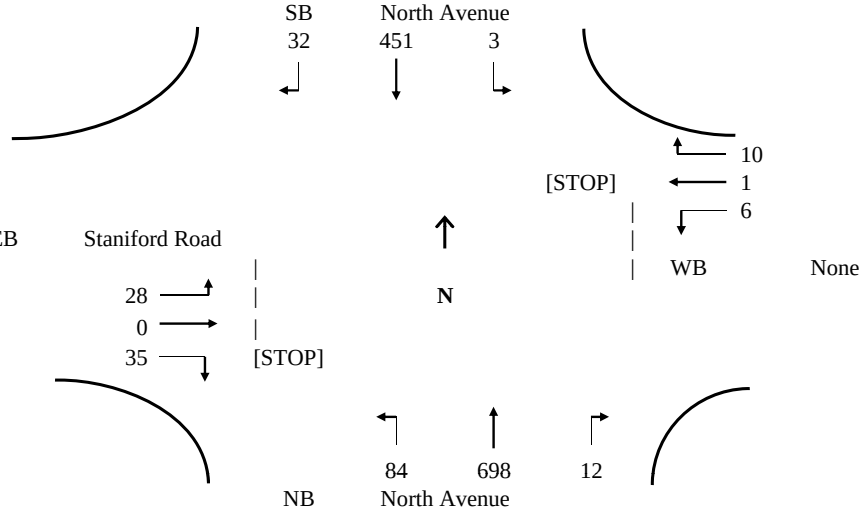
1361 vph

VOLUME ADJUSTMENTS:

Time of Day =	PM
EB/WB= Major or Minor?	minor
Base Year =	2018
Analysis Year =	2018
Growth/Year =	1.00
DHV Adjustment =	1.00
Total Vol Adjustment =	1.00
Speed Limit =	30 mph
Add Development?	n

Channelized right turn for:

EB?	N
WB?	N
NB?	N
SB?	N



CALCULATIONS:

LEFT TURN WARRANTS

	EB	WB	NB	SB
Advancing Volume:	63	17	794	487
Opposing Volume:	17	63	487	794
% Left Turns:	44%	33%	11%	1%
Adv. Volume not to exceed:	NA	NA	371	988
Harmelink Lane Warranted?	**NA**	**NA**	**Y**	**N**
PLTF	0	0	0	1
PLTT	0	0	0	0
PLTW	0	0	1	0
AV	566	536	378	519
Kikuchi and Chakroborty Lane Warranted?	**NA**	**NA**	**Y**	**N**

Intersection: North Avenue - Staniford Road
Location: Burlington, Vemront
Analysis Period: 2018 Build PM Peak Hour

HOURLY VOLUMES:

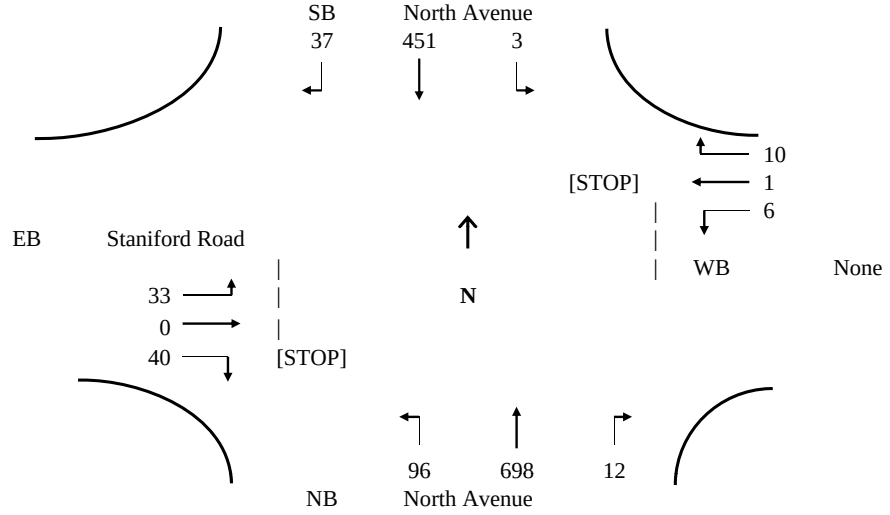
		EB	WB	NB	SB		
Base Volumes:	LT	33	6	96	3		
	TH	0	1	698	451		
	RT	40	10	12	37		
Devel Volumes:	LT						
	TH						
	RT						
Analysis Volumes:	LT	33	6	96	3		
	TH	0	1	698	451		
	RT	40	10	12	37		
	TOTAL	73	17	806	492	1388	vph

VOLUME ADJUSTMENTS:

Time of Day =	PM
EB/WB= Major or Minor?	minor
Base Year =	2018
Analysis Year =	2018
Growth/Year =	1.00
DHV Adjustment =	1.00
Total Vol Adjustment =	1.00
Speed Limit =	30 mph
Add Development?	y

Channelized right turn for:

EB?	N
WB?	N
NB?	N
SB?	N



CALCULATIONS:

LEFT TURN WARRANTS

	EB	WB	NB	SB
Advancing Volume:	73	17	806	492
Opposing Volume:	17	73	492	806
% Left Turns:	45%	33%	12%	1%
Adv. Volume not to exceed:	NA	NA	350	980
Harmelink Lane Warranted?	**NA**	**NA**	**Y**	**N**
PLTF	0	0	0	1
PLTT	0	0	0	0
PLTW	0	0	1	0
AV	566	530	376	512
Kikuchi and Chakroborty Lane Warranted?	**NA**	**NA**	**Y**	**N**