BERRY SURVEYING & ENGINEERING
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NHDOT
District VI – Attention James Hewitt P.E.
PO Box 740
Durham, NH 03824

Rev.: July 20th, 2018

RE: Traffic Impact Analysis & Distribution

1962 Real Estate LLC
40 Wakefield Street
Rochester, NH 03866

John & Linda Svenson
P.O. Box 10
Barrington, NH 03825

Mr. Hewitt,

On behalf of the applicants, 1962 Real Estate LLC and John & Linda Svenson, Berry Surveying & Engineering (BS&E) is submitting for your review a Traffic Impact Analysis, for the reestablishment of Christmas Lane. This will serve to better assist the existing homes, including the Christmas Dove and the proposed 2,500 Sq. Ft. bank (Holy Rosary Credit Union) with a drive through.

The following conclusions were reached as a result Traffic Impact Analysis:

- A total of 33 vehicle trips (21 enter/12 exit) is predicted to occur at the AM peak hour and 95 vehicle trips (46 enter/49 exit) at the PM peak hour.
- This is an increase in trip generation of 30 AM trips, 61 PM trips, and 370 weekday trips.
- The 2018 and 2028 build traffic volumes DO NOT satisfy the NCHRP 457 guidelines for the implementation of a right-turn lane.
- The 2018 and 2028 build traffic volumes DO NOT satisfy the NCHRP 457 guidelines for the implementation of a left-turn lane.
- It is recommended that the existing and surrounding infrastructure will be sufficient to handle the projected increase in vehicle trips and peak hour and all other hours.

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Proposed Development & Introduction

The proposal is to re-develop Christmas Lane for better access to NH 9 for the existing houses, the Christmas Dove and the proposed Holy Rosary Credit Union. The purpose of this analysis is to determine the maximum number of trips coming to and leaving Christmas Lane during certain peak periods of the day. This information is then used in determining the impact on safety as it relates to the existing roadway infrastructure. The following components of the analysis are typical for a project of this size pursuant to the Institute of Traffic Engineers (ITE) manual.

Existing Conditions

Existing Site Description

The existing site consists of two lots, Tax Map 239, Lot 2 and Tax Map 235, Lot 1. These parcels contain 114,280 Sq. Ft. (2.62 Ac.) and 706,280 Sq. Ft. (16.21 Ac.) of land, respectively. The site is combination of wooded and open land, and currently on site are three single family detached homes and the "Christmas Dove", a specialty retail store. The site is located in the town center zone, and is surrounded by other commercial and residential lots. There is a commercial driveway approximately 250 feet to the east of the existing driveway cut, the "Village Barn", and a residential driveway across from the site.

NH Route 9 Road Description

NH Route 9 is a two lane major collector road, according to the NHDOT MS2 Transportation Management System (NHDOT). This road provides access to NH Route 125 and the Barrington town center to the east and more rural parts of Barrington to the west. It has an Average Annual Daily Traffic (AADT) of approximately 6,355 (2017) divided between east and west, also as shown by the NHDOT.

NH Route 9 in the area of the project is composed of a twenty-seven foot wide paved surface with a variable shoulder widths on the north and south side of the road. There is a centerline delineation and fog / edge lines provided. The posted speed limit of the roadway is 30 miles per hour (MPH). The geometry of NH Route 9 in the project area is situated on a curve, and is super elevated to the south. The proposed driveway is on the apex of the curve to maximize sight distance. There are no existing sidewalks, crosswalks, or other pedestrian amenities in the area of the project.

Existing Traffic Volumes

According to traffic counts recorded by the NHDOT for July 1st 2016, the NH Route 9 AM and PM two-way peaks were 568 trips and 821 trips, respectively. It was found that NH Route 9 has an AADT of 6,355 vehicles.

The highest peak hour traffic volume on this section of NH Route 9 east bound occurred from 7-8 AM with 398 vehicles and from 4-5 PM with 235 vehicles. West bound highest peak hour traffic volume occurred from 7-8 AM with 208 vehicles and from 4-5 PM with 586 vehicles. Table #1 shows the traffic direction breakdown of NH Route 9 and Figures #1-3 are graphical representations of the traffic variations occurring throughout the day.

Traffic Distribution NH Route 9				
Date	East Bound		West Bound	
7/1/2016	AM Peak	398	AM Peak	208
	PM Peak	235	PM Peak	586
% Distribution	AM Peak	65.7	AM Peak	34.3
	PM Peak	28.6	PM Peak	71.4

Table 1: Directional breakdown of trips occurring on NH Route 9

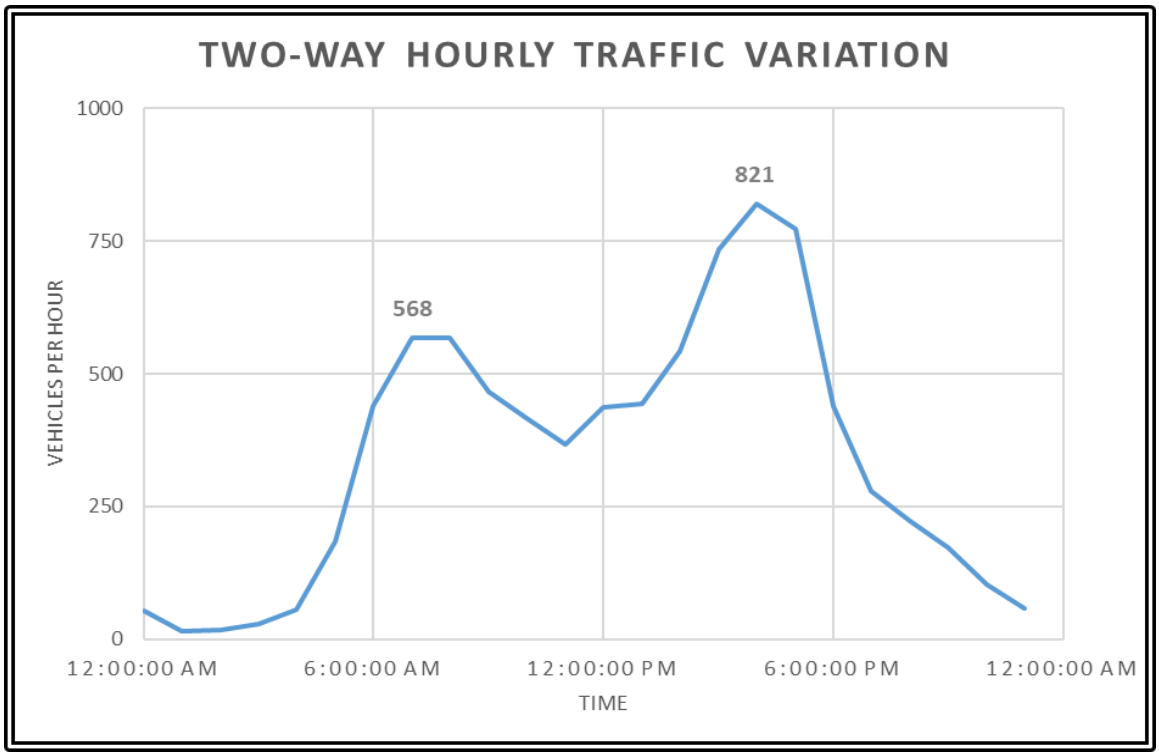

Figure 1: Graph of NH Route 9 two-way hourly traffic variation with peak hour values

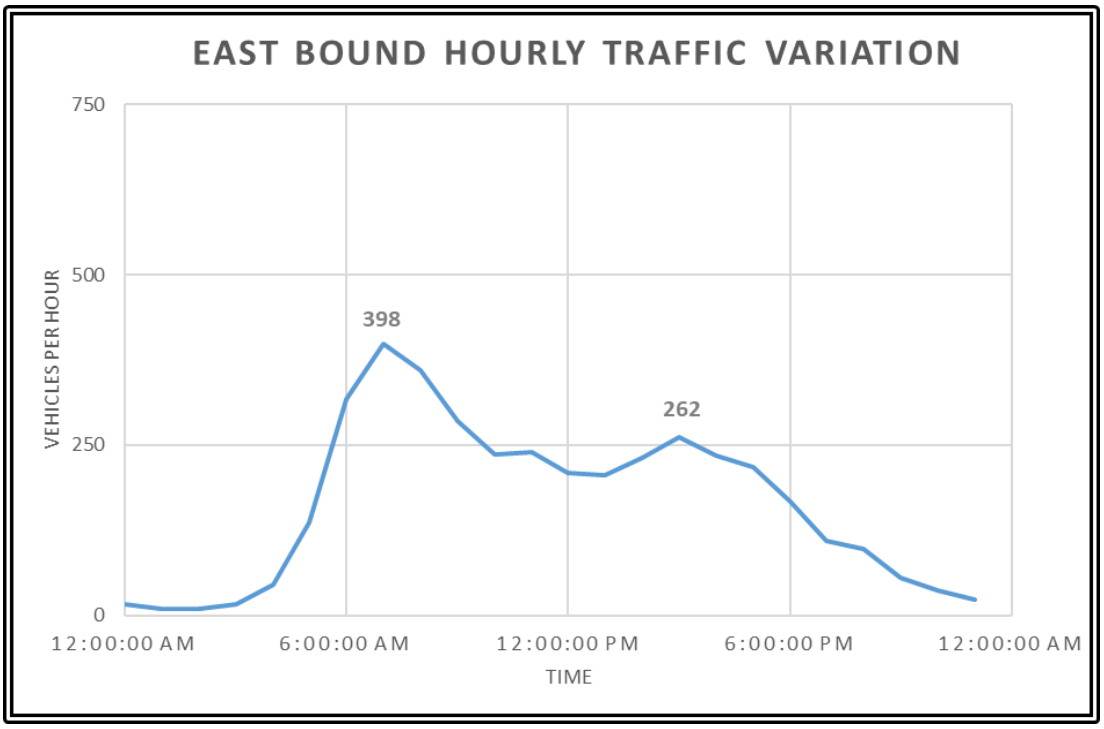

Figure 2: Graph of NH Route 9 east bound hourly traffic variation with peak hour values

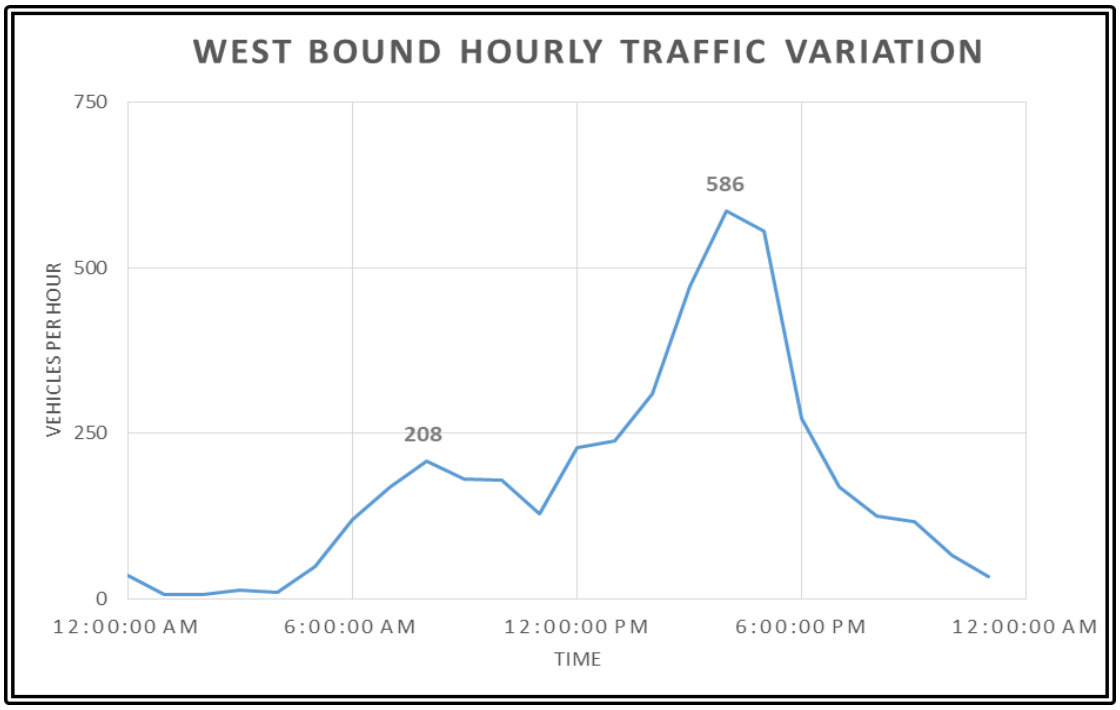

Figure 3: Graph of NH Route 9 west bound hourly traffic variation with peak hour values

Existing Vehicle Speeds

As previously mentioned, the posted speed limit of NH Route 9 is 30 MPH. For the purposes of the safety analysis below, the 85th percentile of speed is required. This particular section of NH Route 9 was observed by Berry Surveying & Engineering to analyze the pass by traffic, reviewing speed. Excessive speeds were rare, and most operators obeyed the posted speed limits within a deviation of 5 MPH. This is consistent with speeds found on urban roads. The 85th percentile derived by observation and consistency with general practice is 35 MPH.

Intersection of NH Route 9 and NH Route 125

Approximately 0.1 miles to the east, NH Route 9 connects to NH Route 125 at an angle of 90 degrees, where a signalized intersection is used to control traffic movements. NH Route 125 has a posted speed limit of 35 MPH and is considered a principal arterial road according to the NHDOT. NH Route 125 consists of three north bound lanes (left, through, through + right) and three south bound lanes (left, through, through + right) in the area of the intersection of NH Route 9 and NH Route 125, with an Average Annual Daily Traffic (AADT) of 16,561 (2017) divided between north and south. The directional volume split is nearly 50/50, with a north bound AADT of 8,157 (2017) and south bound AADT of 8,404 (2017) and shows an increase in the PM peak hour traffic volumes proportional to what NH Route 9 experiences.

The directional breakdown of trips entering and exiting the site has taken into account the potential draw from NH Route 125. As NH Route 125 is a principal arterial road, this intersection will influence the trips to and from the site. Turn movements must be evaluated differently than the typical directional breakdown that would be derived strictly from the pass by traffic of NH Route 9. The ratio of AADT's from NH Route 9 and NH Route 125 was used to account for this. As trips enter and exit the project site to and from NH Route 125, entrance trips will influence the NH Route 9 west bound volume and exit trips will influence the NH Route 9 east bound volume. Table 2 shows a summary of AADT values for NH Route 9 and NH Route 125.

NH Route 125 AADT	16,561 (2017)
NH Route 9 AADT	6,355 (2017)
Combined AADT	22,916 (2017)

Table 2: AADT values for NH Route 9 and NH Route 125

These AADT's where then applied to determine what percentage of vehicles would potentially enter or exit to the east or west of the site. It was determined that 72.3% of vehicles would enter from NH Route 9 west bound, coming from the intersection of NH Route 9 and NH Route 125, and 27.7% would enter from NH Route 9 east bound. It was then determined that 72.3% of vehicles would exit to NH Route 9 east bound, going to the intersection of NH Route 9 and NH Route 125, and 27.7% of vehicles would exit to NH Route 9 west bound. This directional break

down is used later in the document in the determination of turning movements and turn bay warrant analyses. In addition, Figure 4 shows the configuration of the intersection of NH Route 9 and NH Route 125 with surrounding roadways, including AADT values (NHDOT).

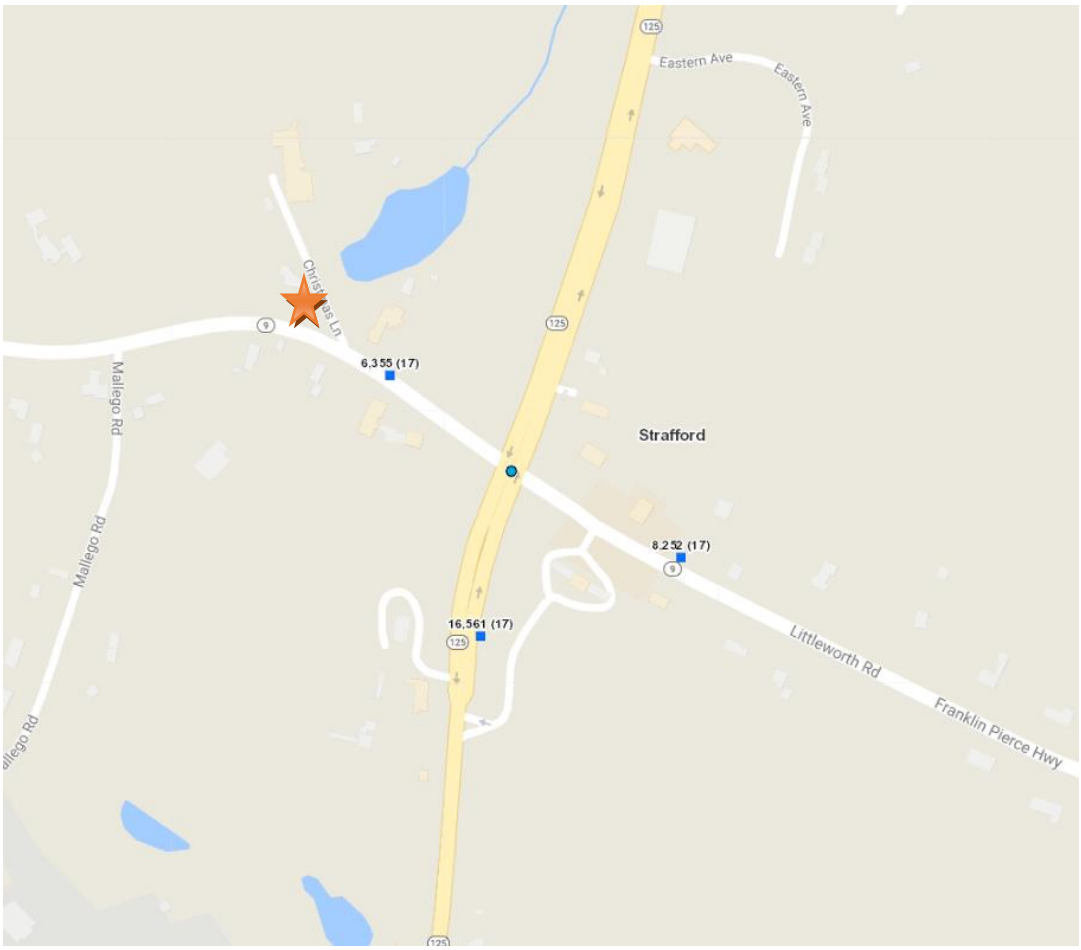

Figure 4: Figure of the intersection of NH Route 9 and NH Route 125 with AADT values (NHDOT)

Existing Trip Generation

The 9th Edition ITE Trip Generation Manual was used to determine the existing volume of trips, as well as the percentage of entrance-to-exit traffic experienced during the AM & PM peak hours between 7 and 9 AM and 4 and 6 PM. Land Use Codes Single Family Detached Housing (210) and Specialty Retail Center (826) were used in deriving the trip generation for the existing site. Tables 2-4 provide average trip rate, total trips generated, enter to exit ratio, and the enter to exit distribution. Given the extremely seasonal draw the Christmas Dove has, it is anticipated that the ITE generation rates given for this use, on this site, will generate a conservatively high volume of traffic for most times of the year. BS&E has witnessed far less traffic than stated below in the PM peak hour.

Single Family Detached Housing Existing Trip Generation:

Time Method	AM Peak Adj. Street (Page 297) Dwelling Units			Time Method	PM Peak Adj. Street (Page 298) Dwelling Units		
# Units	3			# Units	3		
Avg. Rate	0.75			Avg. Rate	1		
Total Trips	2.3			Total Trips	3.0		
% Enter	25.0	Total Enter	0.6	% Enter	63.0	Total Enter	1.9
% Exit	75.0	Total Exit	1.7	% Exit	37.0	Total Exit	1.1

Table 3: (Single Family Detached Housing) Peak hour of adjacent street traffic AM & PM

Specialty Retail Center Existing Trip Generation:

Time Method	PM Peak Adj. Street (Page 1580) 1000 Sq. Ft Gross Leasable Area		
GLA (Ft. Sq.)	11.45		
Avg. Rate	2.71		
Total Trips	31.0		
% Enter	44.0	Total Enter	13.7
% Exit	56.0	Total Exit	17.4

Table 4: (Specialty Retail Center) Peak hour of adjacent street traffic PM

Total Existing Trip Generation:

Time	AM Peak Adj. Street Traffic			Time	PM Peak Adj. Street Traffic		
Total Trips	2.3			Total Trips	34.0		
% Enter	25.0	Total Enter	0.6	% Enter	49.8	Total Enter	17.0
% Exit	75.0	Total Exit	1.7	% Exit	50.2	Total Exit	17.1

Table 5: Total existing trip generation peak hour of adjacent street traffic AM & PM

Proposed Trip Generation Increase

The 9th Edition ITE Trip Generation Manual was used to determine the proposed volume of trips, as well as the percentage of entrance-to-exit traffic experienced at the AM & PM peak hours between 7 and 9 AM and 4 and 6 PM. Land Use Code Drive-in Bank (912) was used in deriving the trip generation for the proposed bank. Tables 5 and 6 provide average trip rate, total trips generated, enter to exit ratio, and the enter to exit distribution. Table 7 shows the changes in trip generation during the AM and PM peak hours, as well as the weekday total.

Drive-in Bank Proposed Trip Generation:

Time Method	AM Peak Adj. Street (Page 1843) 1000 Sq. Ft Gross Floor Area			Time Method	PM Peak Adj. Street (Page 1844) 1000 Sq. Ft Gross Floor Area		
GFA (Ft. Sq.)	2.5			GFA (Ft. Sq.)	2.5		
Avg. Rate	12.08			Avg. Rate	24.3		
Total Trips	30.2			Total Trips	60.8		
% Enter	62.0	Total Enter	18.7	% Enter	48.0	Total Enter	29.2
% Exit	38.0	Total Exit	11.5	% Exit	52.0	Total Exit	31.6

Table 6: (Drive-in Bank) Peak hour of adjacent street traffic weekdays AM & PM

Build Existing & Proposed Trip Generation:

Time	AM Peak Adj. Street Traffic			Time	PM Peak Adj. Street Traffic			Time
Total Trips	32.5			Total Trips	94.8			Total Trips
% Enter	63.9	Total Enter	20.7	% Enter	48.7	Total Enter	46.1	% Enter
% Exit	36.1	Total Exit	11.7	% Exit	51.3	Total Exit	48.7	% Exit

Table 7: Total Peak hour of adjacent street traffic weekdays AM & PM generation

Changes in Trip Generation	
Time	# Trips Increased
AM Peak	30.2
PM Peak	60.8
Weekday Total	370.4

Table 8: Changes in trip generation

Build Traffic Projections and Turning Analysis

Traffic data obtained from the NH DOT's Transportation Data Management System has been projected to 2018 and ten years further to 2028. This has been done using a peak seasonal adjustment factor of 1.08 (AM & PM) and using an annual growth rate of 1%, compounded annually. The derivation of the peak seasonal adjustment factor comes from an average series of values from other urban highways from across New Hampshire, which can be found as Table 15 in Appendix D. Figures 5 and 6 show the build turning movements to and from the site. These figures also show the volume of traffic east bound and west bound on NH Route 9 in 2018 and the projected 2028 volumes.

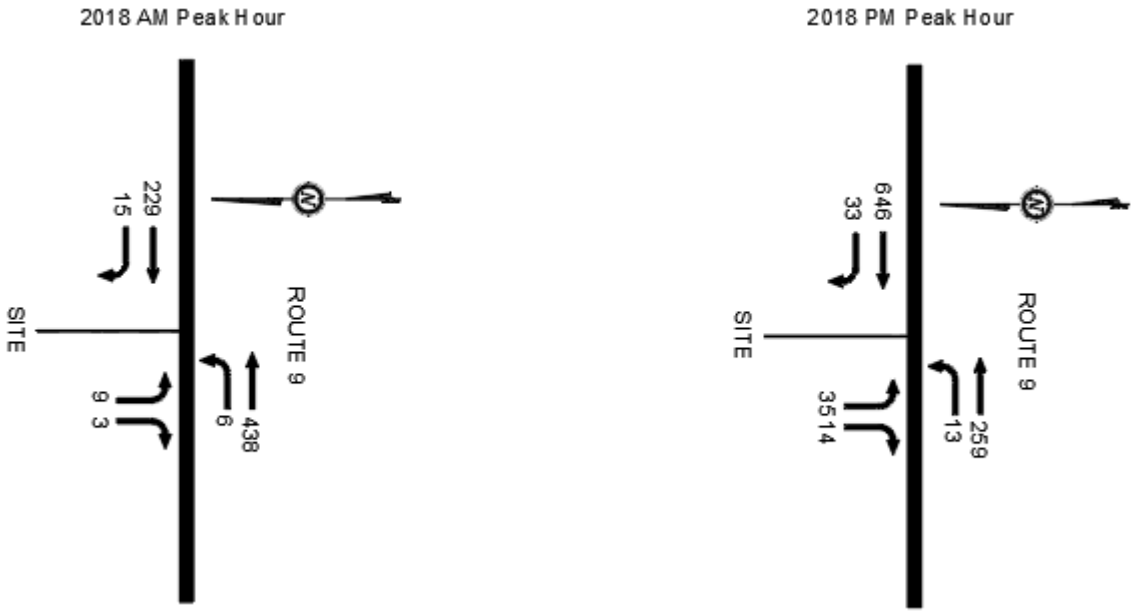

Figure 5: 2018 build traffic volumes and turning movements

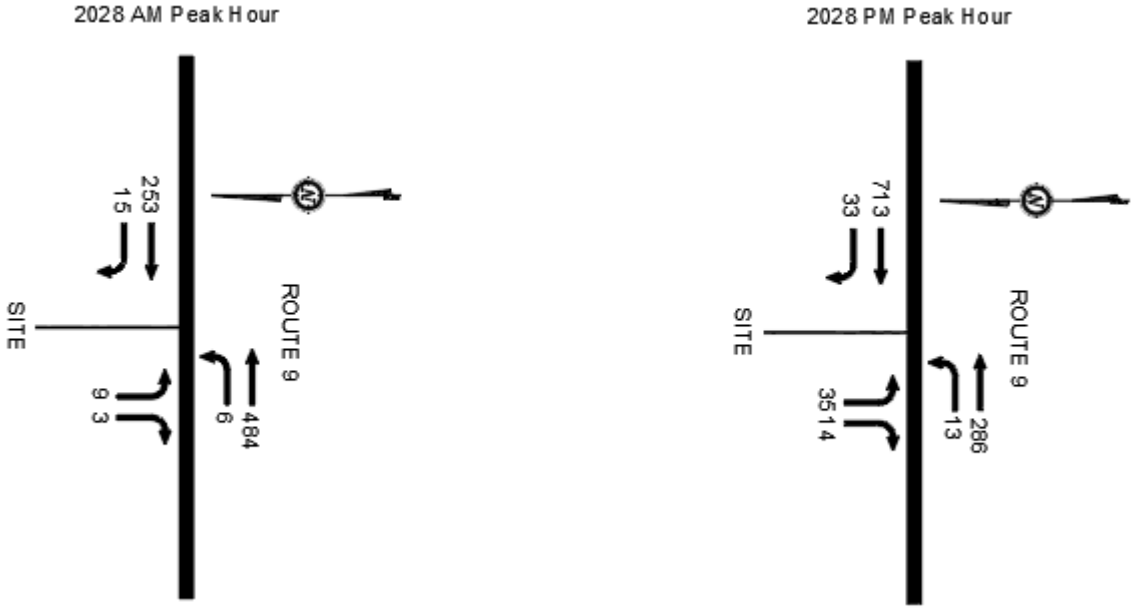

Figure 6: 2028 projected traffic volumes and turning movements

In Tables 9 and 10, the total trips that are calculated to occur from NH Route 9 are shown at AM and PM weekday peak hours. These are further broken down to entrance and exit into the site as well as percentage of left and right turns.

Time	AM Peak Adj. Street Traffic	# Trips	Turn Type	% Distribution
Total Trips	32.5			
Trips Enter from Route 9 East Bound		5.7	Left	17.7
Trips Enter from Route 9 West Bound		15.0	Right	46.2
Trips Exit to Route 9 East Bound		8.5	Left	26.1
Trips Exit to Route 9 West Bound		3.3	Right	10.0

Table 9: Summary of AM turning movements to and from the site

Time	PM Peak Adj. Street Traffic	# Trips	Turn Type	% Distribution
Total Trips	94.8			
Trips Enter from Route 9 East Bound		12.8	Left	13.5
Trips Enter from Route 9 West Bound		33.3	Right	35.2
Trips Exit to Route 9 East Bound		35.2	Left	37.1
Trips Exit to Route 9 West Bound		13.5	Right	14.2

Table 10: Summary of PM turning movements to and from the site

Left-Turn Warrants Analysis

Depending on vehicle speed, advancing vehicular volumes, opposing vehicular volumes, and the percent of left turns that vehicles are predicted to make, certain roadways may require special treatment for vehicles making left turning maneuvers. The determination of this special treatment is determined by the NCHRP 457 left turn bay guidelines. Calibration constants of 3.0 seconds are used for average left turn time, 5.0 seconds for critical headway, and 1.9 seconds for vehicles to clear the advancing lane. If warranted, the left turn bay would allow for deceleration of vehicles and storage in the queue to wait safely for advancing traffic to pass.

The AM and PM peak traffic volumes from 2016 projected to 2018 and 2028 were used to determine if a left-turn bay is warranted to safely enter the site. It has been calculated that approximately 6 trips are to occur turning left into the site during the AM peak hour and 13 during the PM peak hour. For 2018, the projected AM total advancing and total opposing volumes are 459 and 244, respectively. It was projected for 2028 that total advancing and total opposing volumes would be 505 and 268, respectively. For 2018, the projected PM advancing and opposing volumes are 305 and 679, respectively. It was projected for 2028 that advancing and opposing volumes would be 332 and 746, respectively. Using the 85th percentile speed of 35 MPH, it was determined that a left turn lane will not be warranted to safely enter the site. The projection of the traffic volumes for AM and PM peak hours is included as Figures 11 and 12. The full analysis can be found in Appendix B as Figures 13-16. Tables 11 and 12 are summaries of the left-turn bay analyses.

Left-Turn Lane Warrants Analysis Route 9		
Factors	2018 AM Build Volume	2028 AM Build Volume
Left-Turn Volume (EB)	6	6
Advancing Volume (EB) (L+TR+R)	459	505
Opposing Volume (WB) (TR+R)	244	268
Percent Lefts	1%	1%
85th Percentile Speed (MPH)	35	35
Limiting Adv. Volume (veh/hr)	1,247	1,272
Left Turn Bay Warranted	NO	NO

Table 11: Summary of AM NCHRP 457 left-turn bay analysis

Left-Turn Lane Warrants Analysis Route 9		
Factors	2018 PM Build Volume	2028 PM Build Volume
Left-Turn Volume (EB)	13	13
Advancing Volume (EB) (L+TR+R)	305	332
Opposing Volume (WB) (TR+R)	679	746
Percent Lefts	4%	4%
85th Percentile Speed (MPH)	35	35
Limiting Adv. Volume (veh/hr)	439	427
Left Turn Bay Warranted	NO	NO

Table 12: Summary of PM NCHRP 457 left-turn bay analysis

Right-Turn Warrants Analysis

Depending on vehicle speed, advancing vehicular volumes, and the percent of right turns that vehicles are predicted to make, certain roadways may require special treatment for vehicles making right turning maneuvers. The determination of this special treatment is determined by the NCHRP 457 right turn bay guidelines. If warranted, the right turn bay would allow for deceleration of vehicles and storage in the queue to wait safely for right turning traffic to clear.

The AM and PM peak traffic volumes from 2016 projected to 2018 and 2028 were used to determine if a right-turn bay is warranted to safely enter the site. It has been calculated that approximately 14 trips are to occur turning right into the site during the AM peak hour and 34 for the PM peak hour. For 2018, the projected AM total advancing volume is 250 and PM total advancing volume is 692. It was projected for 2028 that the AM total advancing volume is 274 and the PM total advancing volume is 759. Using the 85th percentile speed of 35 MPH, it was determined that a right turn lane will not be warranted to safely enter the site. The projection of the traffic volumes for AM and PM peak hours is included as Figures 17 and 18. The full analysis can be found in Appendix B as Figures 19-22. Tables 13 and 14 are summaries of the right-turn bay analyses.

Right-Turn Lane Warrants Analysis Route 9		
Factors	2018 AM Build Volume	2028 AM Build Volume
Right-Turn Volume (WB)	15	15
Advancing Volume (WB) (L+TR+R)	250	274
85th Percentile Speed (MPH)	35	35
Limiting Adv. Volume (veh/hr)	2560	1835
Right-Turn Bay Warranted	NO	NO

Table 13: Summary of AM NCHRP 457 right-turn bay analysis

Right-Turn Lane Warrants Analysis Route 9		
Factors	2018 PM Build Volume	2028 PM Build Volume
Right-Turn Volume (WB)	35	35
Advancing Volume (WB) (L+TR+R)	692	759
85th Percentile Speed (MPH)	35	35
Limiting Adv. Volume (veh/hr)	63	45
Right-Turn Bay Warranted	NO	NO

Table 14: Summary of PM NCHRP 457 right-turn bay analysis

Sight Distance and Safety Analysis

The proposed driveway is located at the apex of the road curve to maximize sight distance. Sight distance to the east and west, as well as driveway alignment are the two determining factors of safety. Sight distance to the east is un-obstructed for well over 400 feet (measured) while sight distance to the west is un-obstructed for well over 400 feet (measured.) Using Exhibit 3-1 (Stopping Sight Distance) (Figure 28) in the Geometric Design Manual, a 35 mph 85th percentile speed requires the stopping sight distance be 250 feet from the both directions. The standard sight distance required by NHDOT is 400 feet in cases where the Geometric Design Manual would not require more. In this instance both the easterly and westerly sight distances meet the design required warrant as well as the standard practice of NHDOT of 400 feet. There are no improvements required to maintain this site distance.

With respect to general safety of NH Route 9 in relation to the peak hour trip generation and AADT, it is our assessment that the cross section of pavement and shoulder widths are appropriate.

*****AASHTO Geometric Design of Highways and Streets (2011)

Conclusions and Recommendations

1.) A total of 33 vehicle trips (21 enter/12 exit) is predicted to occur at the AM peak hour and 95 vehicle trips (46 enter/49 exit) at the PM peak hour.

2.) This is an increase in trip generation of 30 AM trips, 61 PM trips, and 370 weekday trips.

3.) The 2018 and 2028 build traffic volumes DO NOT satisfy the NCHRP 457 guidelines for the implementation of a right-turn lane.

4.) The 2018 and 2028 build traffic volumes DO NOT satisfy the NCHRP 457 guidelines for the implementation of a left-turn lane.

5.) It is recommended that the existing and surrounding infrastructure will be sufficient to handle the projected increase in vehicle trips and peak hour and all other hours.

Respectfully Submitted,

BERRY SURVEYING & ENGINEERING

Christopher R. Berry SIT
Principal, President

Kenneth A. Berry, PE, LLS,
CPSWQ, CPESC, CESSWI
Principal, VP-Technical Operations

KRP/krp

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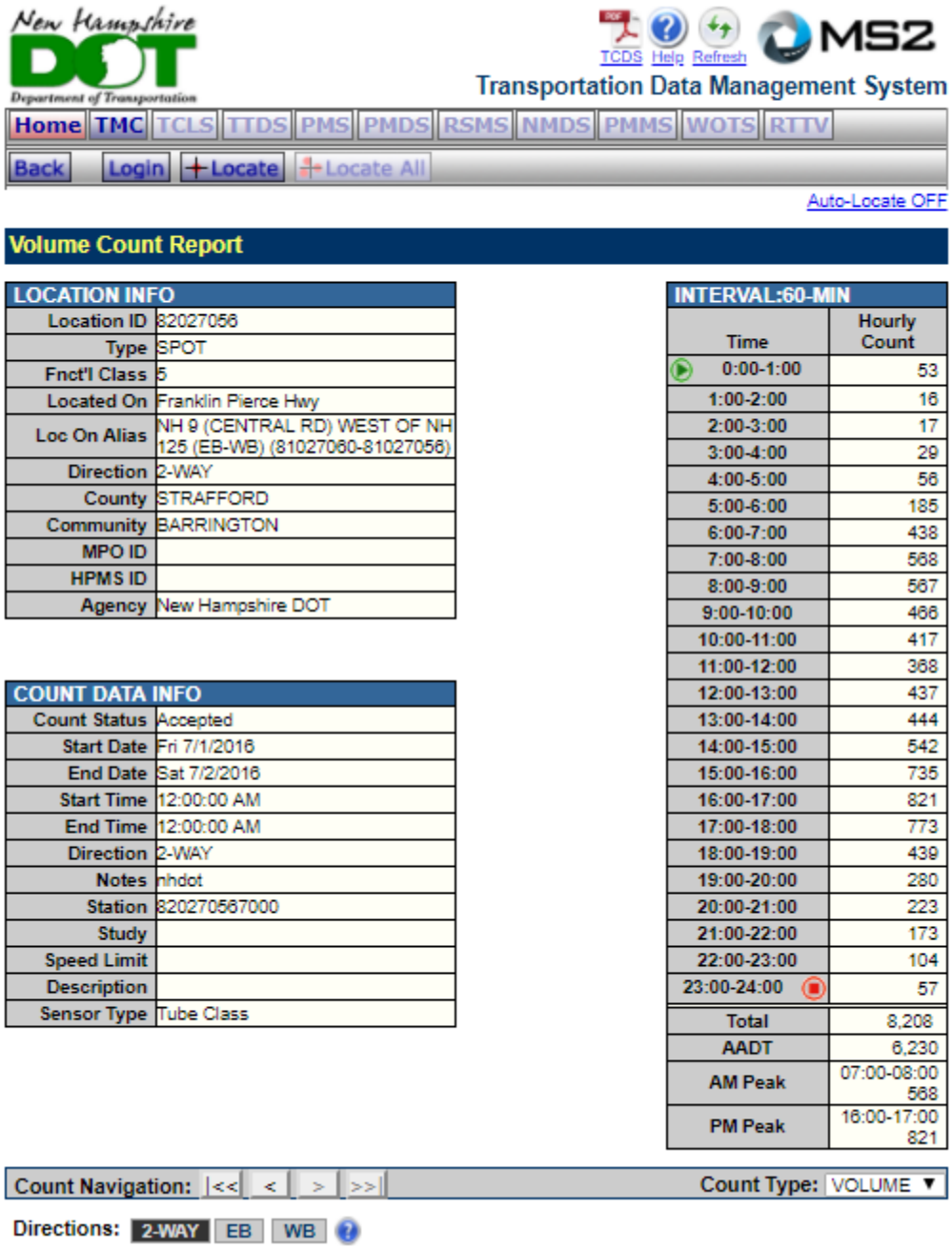
Appendix A

NH Route 9 and NH Route 125 Traffic Counts



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Volume Count Report

LOCATION INFO	
Location ID	82027056
Type	SPOT
Fnct'l Class	5
Located On	Franklin Pierce Hwy
Loc On Alias	NH 9 (CENTRAL RD) WEST OF NH 125 (EB-WB) (81027060-81027056)
Direction	2-WAY
County	STRAFFORD
Community	BARRINGTON
MPO ID	
HPMS ID	
Agency	New Hampshire DOT

COUNT DATA INFO	
Count Status	Accepted
Start Date	Fri 7/1/2016
End Date	Sat 7/2/2016
Start Time	12:00:00 AM
End Time	12:00:00 AM
Direction	2-WAY
Notes	nhdot
Station	820270567000
Study	
Speed Limit	
Description	
Sensor Type	Tube Class

INTERVAL:60-MIN	
Time	Hourly Count
0:00-1:00	53
1:00-2:00	16
2:00-3:00	17
3:00-4:00	29
4:00-5:00	56
5:00-6:00	185
6:00-7:00	438
7:00-8:00	568
8:00-9:00	567
9:00-10:00	466
10:00-11:00	417
11:00-12:00	368
12:00-13:00	437
13:00-14:00	444
14:00-15:00	542
15:00-16:00	735
16:00-17:00	821
17:00-18:00	773
18:00-19:00	439
19:00-20:00	280
20:00-21:00	223
21:00-22:00	173
22:00-23:00	104
23:00-24:00	57
Total	8,208
AADT	6,230
AM Peak	07:00-08:00 568
PM Peak	16:00-17:00 821

Count Navigation: |<< < > >> Count Type: VOLUME

Directions: 2-WAY EB WB

Figure 7: Friday July 1st, 2016 NH Route 9 two-way hourly traffic count

New Hampshire DOT Department of Transportation

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Volume Count Report

LOCATION INFO	
Location ID	82027056_EB
Type	SPOT
Fnct'l Class	5
Located On	NH 9 (CENTRAL RD) EB WEST OF NH 125 (82027056)
Loc On Alias	NH 9 (CENTRAL RD) WEST OF NH 125 (EB-WB) (81027060-81027056)
Direction	EB
County	STRAFFORD
Community	BARRINGTON
MPO ID	
HPMS ID	
Agency	New Hampshire DOT

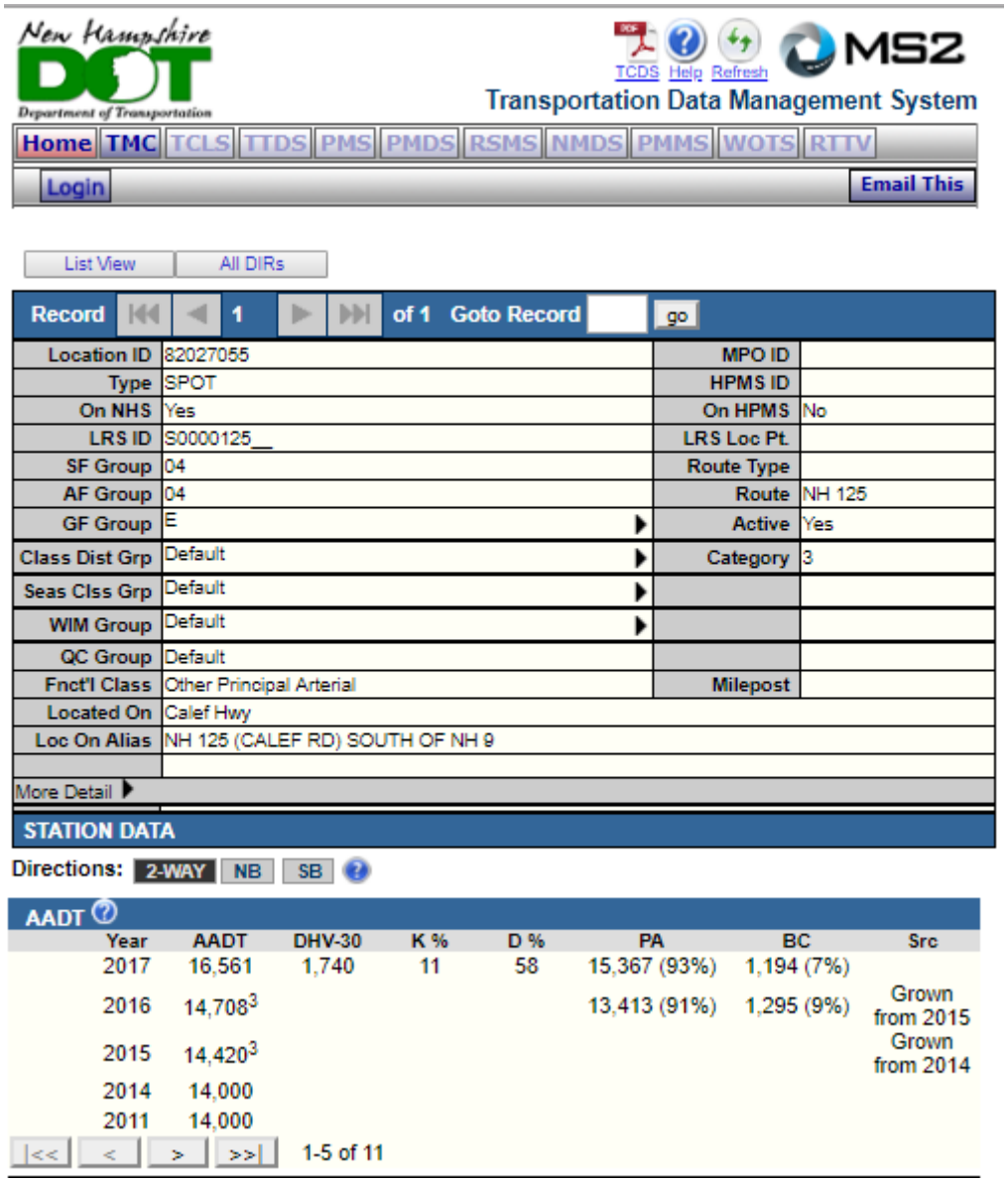
COUNT DATA INFO	
Count Status	Accepted
Start Date	Fri 7/1/2016
End Date	Sat 7/2/2016
Start Time	12:00:00 AM
End Time	12:00:00 AM
Direction	
Notes	nhdot
Station	820270563000
Study	
Speed Limit	
Description	
Sensor Type	Tube Class

INTERVAL:60-MIN	
Time	Hourly Count
0:00-1:00	17
1:00-2:00	9
2:00-3:00	10
3:00-4:00	16
4:00-5:00	45
5:00-6:00	136
6:00-7:00	318
7:00-8:00	398
8:00-9:00	359
9:00-10:00	285
10:00-11:00	237
11:00-12:00	239
12:00-13:00	209
13:00-14:00	206
14:00-15:00	232
15:00-16:00	262
16:00-17:00	235
17:00-18:00	217
18:00-19:00	167
19:00-20:00	110
20:00-21:00	97
21:00-22:00	56
22:00-23:00	37
23:00-24:00	23
Total	3,920
AADT	2,975
AM Peak	07:00-08:00 398
PM Peak	15:00-16:00 262

Count Navigation: |<< < > >>| Count Type: VOLUME

Directions: 2-WAY | EB | WB

Figure 8: Friday July 1st, 2016 NH Route 9 east bound hourly traffic count



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Record 1 of 1 Goto Record go

Location ID	82027055	MPO ID	
Type	SPOT	HPMS ID	
On NHS	Yes	On HPMS	No
LRS ID	S0000125__	LRS Loc Pt.	
SF Group	04	Route Type	
AF Group	04	Route	NH 125
GF Group	E	Active	Yes
Class Dist Grp	Default	Category	3
Seas Clss Grp	Default		
WIM Group	Default		
QC Group	Default		
Fnct'l Class	Other Principal Arterial	Milepost	
Located On	Calef Hwy		
Loc On Alias	NH 125 (CALEF RD) SOUTH OF NH 9		

More Detail

STATION DATA

Directions: 2-WAY | NB | SB

AADT

Year	AADT	DHV-30	K %	D %	PA	BC	Src
2017	16,561	1,740	11	58	15,367 (93%)	1,194 (7%)	
2016	14,708[3]				13,413 (91%)	1,295 (9%)	Grown from 2015
2015	14,420[3]						Grown from 2014
2014	14,000						
2011	14,000						

|<< < > >>| 1-5 of 11

Figure 9: History of AADT values and classification for NH Route 125

STATE OF NEW HAMPSHIRE
DEPARTMENT OF TRANSPORTATION
BUREAU OF TRAFFIC

Bureau of Planning, Traffic Section, Traffic Reports *18-Feb-16*

STAT.	TYPE	LOCATION	FC	2008	2009	2010	2011	2012	2013	2014	2015
Town: BARRINGTON											
027017	82	US 202 (WASHINGTON ST) AT ISINGLASS RIVER	07	4700	*	*	4600	*	*	6200	*
027052	62	NH 202A AT ROCHESTER TL (EB-WB) (61027010-61027011)	07	*	*	2200	*	*	2000	*	*
027053	62	NH 9 (CENTRAL RD) AT MADBURY TL (EB-WB) (61027012-61027013)	07	*	*	7400	*	*	7400	*	*
027054	82	NH 9 (CENTRAL RD) EAST OF NH 125 (EB-WB) (81027054-027061)	07	*	*	7200	*	*	7100	*	*
027055	82	NH 125 (CALEF RD) SOUTH OF NH 9	02	14000	*	*	14000	*	*	14000	*
027056	82	NH 9 (CENTRAL RD) WEST OF NH 125 (EB-WB) (81027060-81027056)	07	*	*	6700	*	*	6400	*	*
027057	82	SECOND CROWN POINT RD WEST OF POND HILL RD	09	1100	*	*	1100	*	*	1100	*
027058	82	WOODS RD AT NOTTINGHAM TL	09	300	*	*	190	*	*	380	*
027059	82	PROVINCE RD WEST OF NH 126	09	1400	*	*	1400	*	*	1100	*
027062	82	NH 126 (LOCKS HILL RD) WEST OF WATERHOUSE RD	08	*	*	2800	*	*	2700	*	*
027066	82	GREEN HILL RD OVER ISINGLASS RIVER	19	1700	*	*	1800	*	*	1600	*
027067	82	MALEGO RD OVER MALEGO BROOK	09	1000	*	*	1100	*	*	1000	*

Figure 10: Historical traffic volumes ((027056) NH 9 (Central Road) West of NH 125)

Appendix B

Data Used in Left-Turn Bay Warrants Analysis

Year	Advancing Volume	Advancing Volume Peaked	Opposing Volume	Opposing Volume Peaked	Left Turns	Right Turns	Total Advancing Volume (L+TR+R)	Total Opposing Volume (TR+R)
2016	398	430	208	225	6	15	451	240
2017	402	434	210	227	6	15	455	242
2018	406	438	212	229	6	15	459	244
2019	410	443	214	231	6	15	464	246
2020	414	447	216	234	6	15	468	249
2021	418	452	219	236	6	15	472	251
2022	422	456	221	238	6	15	477	253
2023	427	461	223	241	6	15	482	256
2024	431	465	225	243	6	15	486	258
2025	435	470	227	246	6	15	491	261
2026	440	475	230	248	6	15	496	263
2027	444	480	232	251	6	15	500	266
2028	448	484	234	253	6	15	505	268
Seasonal Peaking Factor (July)		1.08						

Figure 11: Data used for AM Peak hour left-turn warrant analysis

Year	Advancing Volume	Advancing Volume Peaked	Opposing Volume	Opposing Volume Peaked	Left Turns	Right Turns	Total Advancing Volume (L+TR+R)	Total Opposing Volume (TR+R)
2016	235	254	586	633	13	33	300	666
2017	237	256	592	639	13	33	302	673
2018	240	259	598	646	13	33	305	679
2019	242	261	604	652	13	33	308	685
2020	245	264	610	659	13	33	310	692
2021	247	267	616	665	13	33	313	698
2022	249	269	622	672	13	33	316	705
2023	252	272	628	679	13	33	318	712
2024	254	275	635	685	13	33	321	719
2025	257	278	641	692	13	33	324	725
2026	260	280	647	699	13	33	326	732
2027	262	283	654	706	13	33	329	739
2028	265	286	660	713	13	33	332	746
Seasonal Peaking Factor (July)		1.08						

Figure 12: Data used for PM Peak hour left-turn warrant analysis

Left-Turn Bay Warrants Analysis

Figure 2 - 5. Guideline for determining the need for a major-road left-turn bay at a two-way stop-controlled intersection.

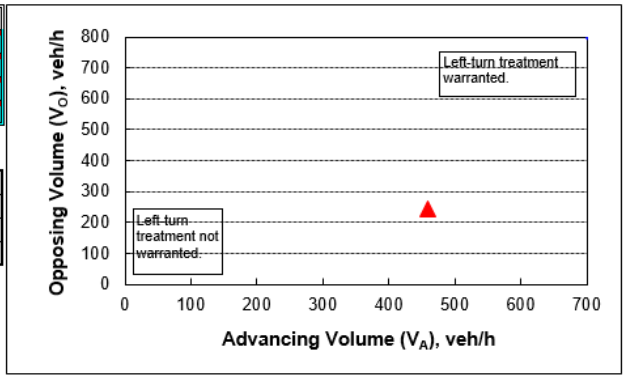
2-lane roadway (English)

INPUT

Variable	Value
85th percentile speed, mph:	35
Percent of left-turns in advancing volume (V_A), %:	1%
Advancing volume (V_A), veh/h:	459
Opposing volume (V_O), veh/h:	244

OUTPUT

Variable	Value
Limiting advancing volume (V_A), veh/h:	1247
Guidance for determining the need for a major-road left-turn bay:	
Left-turn treatment NOT warranted.	



CALIBRATION CONSTANTS

Variable	Value
Average time for making left-turn, s:	3.0
Critical headway, s:	5.0
Average time for left-turn vehicle to clear the advancing lane, s:	1.9

Figure 13: 2018 AM NCHRP 457 left-turn bay analysis

Figure 2 - 5. Guideline for determining the need for a major-road left-turn bay at a two-way stop-controlled intersection.

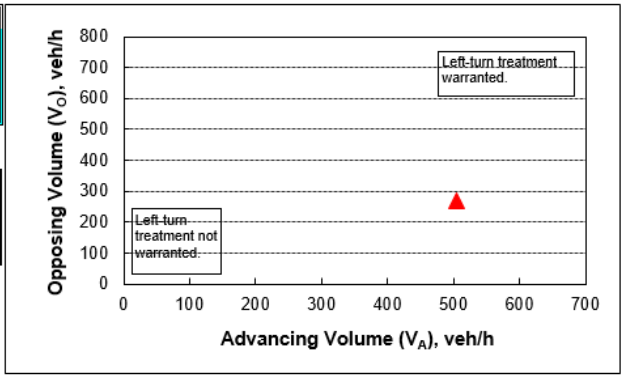
2-lane roadway (English)

INPUT

Variable	Value
85th percentile speed, mph:	35
Percent of left-turns in advancing volume (V_A), %:	1%
Advancing volume (V_A), veh/h:	505
Opposing volume (V_O), veh/h:	268

OUTPUT

Variable	Value
Limiting advancing volume (V_A), veh/h:	1272
Guidance for determining the need for a major-road left-turn bay:	
Left-turn treatment NOT warranted.	



CALIBRATION CONSTANTS

Variable	Value
Average time for making left-turn, s:	3.0
Critical headway, s:	5.0
Average time for left-turn vehicle to clear the advancing lane, s:	1.9

Figure 14: 2028 AM NCHRP 457 left-turn bay analysis

Figure 2 - 5. Guideline for determining the need for a major-road left-turn bay at a two-way stop-controlled intersection.

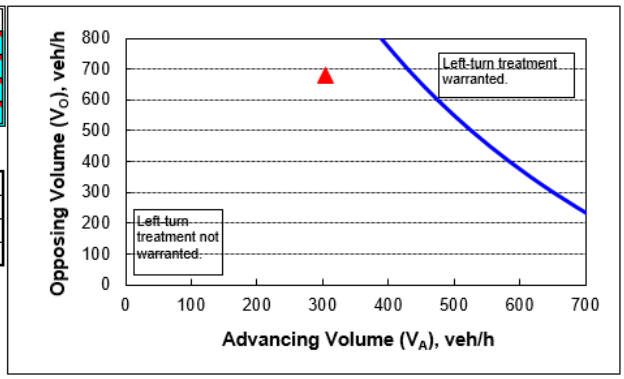
2-lane roadway (English)

INPUT

Variable	Value
85th percentile speed, mph:	35
Percent of left-turns in advancing volume (V_A), %:	4%
Advancing volume (V_A), veh/h:	305
Opposing volume (V_O), veh/h:	679

OUTPUT

Variable	Value
Limiting advancing volume (V_A), veh/h:	439
Guidance for determining the need for a major-road left-turn bay:	
Left-turn treatment NOT warranted.	



CALIBRATION CONSTANTS

Variable	Value
Average time for making left-turn, s:	3.0
Critical headway, s:	5.0
Average time for left-turn vehicle to clear the advancing lane, s:	1.9

Figure 15: 2018 PM NCHRP 457 left-turn bay analysis

Figure 2 - 5. Guideline for determining the need for a major-road left-turn bay at a two-way stop-controlled intersection.

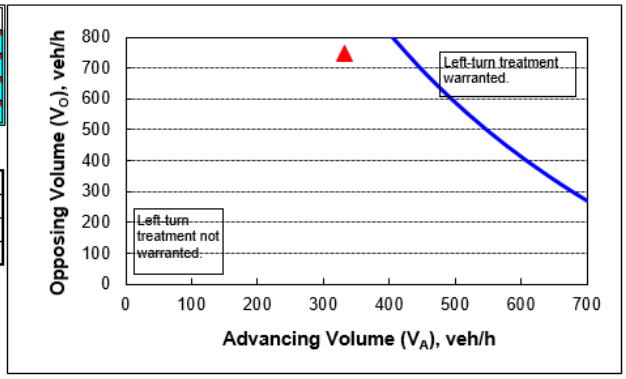
2-lane roadway (English)

INPUT

Variable	Value
85th percentile speed, mph:	35
Percent of left-turns in advancing volume (V_A), %:	4%
Advancing volume (V_A), veh/h:	332
Opposing volume (V_O), veh/h:	746

OUTPUT

Variable	Value
Limiting advancing volume (V_A), veh/h:	427
Guidance for determining the need for a major-road left-turn bay:	
Left-turn treatment NOT warranted.	



CALIBRATION CONSTANTS

Variable	Value
Average time for making left-turn, s:	3.0
Critical headway, s:	5.0
Average time for left-turn vehicle to clear the advancing lane, s:	1.9

Figure 16: 2028 PM NCHRP 457 left-turn bay analysis

Data Used in Right-Turn Bay Warrants Analysis

Year	Advancing Volume	Advancing Volume Peaked	Left Turns	Right Turns	Total Advancing Volume (L+TR+R)
2016	208	225	6	15	246
2017	210	227	6	15	248
2018	212	229	6	15	250
2019	214	231	6	15	252
2020	216	234	6	15	254
2021	219	236	6	15	257
2022	221	238	6	15	259
2023	223	241	6	15	262
2024	225	243	6	15	264
2025	227	246	6	15	266
2026	230	248	6	15	269
2027	232	251	6	15	271
2028	234	253	6	15	274
Seasonal Peaking Factor (July)		1.08			

Figure 17: Data used for AM Peak hour right-turn warrant analysis

Year	Advancing Volume	Advancing Volume Peaked	Left Turns	Right Turns	Total Advancing Volume (L+TR+R)
2016	586	633	13	33	679
2017	592	639	13	33	685
2018	598	646	13	33	692
2019	604	652	13	33	698
2020	610	659	13	33	705
2021	616	665	13	33	711
2022	622	672	13	33	718
2023	628	679	13	33	725
2024	635	685	13	33	731
2025	641	692	13	33	738
2026	647	699	13	33	745
2027	654	706	13	33	752
2028	660	713	13	33	759
Seasonal Peaking Factor (July)		1.08			

Figure 18: Data used for PM Peak hour right-turn warrant analysis

Right-Turn Bay Warrants Analysis

Figure 2 - 6. Guideline for determining the need for a major-road right-turn bay at a two-way stop-controlled intersection.

INPUT

Roadway geometry:	2-lane roadway
Variable	Value
Major-road speed, mph:	35
Major-road volume (one direction), veh/h:	250
Right-turn volume, veh/h:	15

OUTPUT

Variable	Value
Limiting right-turn volume, veh/h:	2560
Guidance for determining the need for a major-road right-turn bay for a 2-lane roadway:	
Do NOT add right-turn bay.	

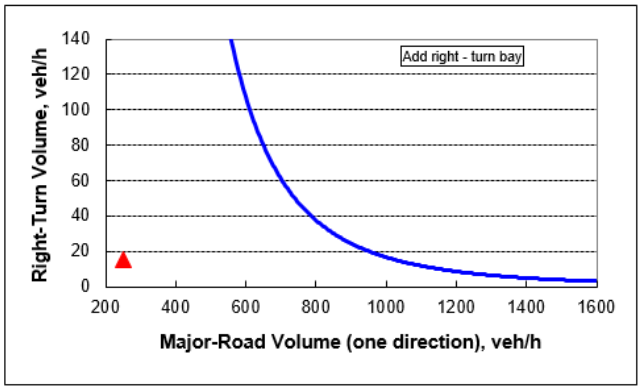


Figure 19: 2018 AM NCHRP 457 right-turn bay analysis

Figure 2 - 6. Guideline for determining the need for a major-road right-turn bay at a two-way stop-controlled intersection.

INPUT

Roadway geometry:	2-lane roadway
Variable	Value
Major-road speed, mph:	35
Major-road volume (one direction), veh/h:	274
Right-turn volume, veh/h:	15

OUTPUT

Variable	Value
Limiting right-turn volume, veh/h:	1835
Guidance for determining the need for a major-road right-turn bay for a 2-lane roadway:	
Do NOT add right-turn bay.	

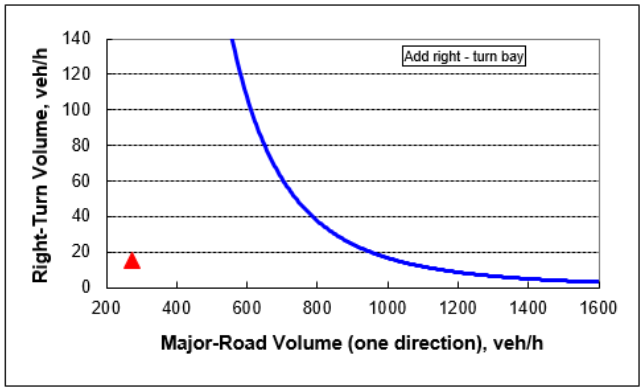


Figure 20: 2028 AM NCHRP 457 right-turn bay analysis

Figure 2 - 6. Guideline for determining the need for a major-road right-turn bay at a two-way stop-controlled intersection.

INPUT

Roadway geometry:	2-lane roadway
Variable	Value
Major-road speed, mph:	35
Major-road volume (one direction), veh/h:	692
Right-turn volume, veh/h:	33

OUTPUT

Variable	Value
Limiting right-turn volume, veh/h:	63
Guidance for determining the need for a major-road right-turn bay for a 2-lane roadway:	
Do NOT add right-turn bay.	

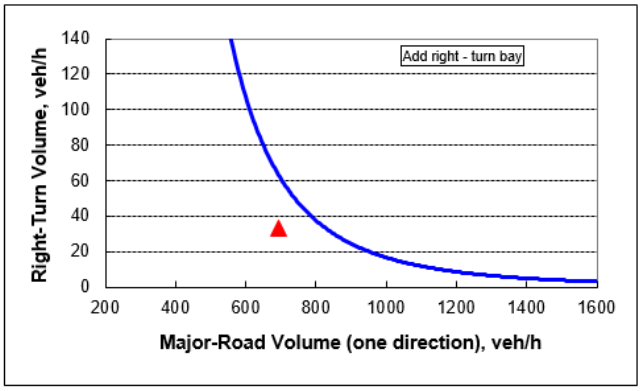


Figure 21: 2018 PM NCHRP 457 right-turn bay analysis

Figure 2 - 6. Guideline for determining the need for a major-road right-turn bay at a two-way stop-controlled intersection.

INPUT

Roadway geometry:	2-lane roadway
Variable	Value
Major-road speed, mph:	35
Major-road volume (one direction), veh/h:	759
Right-turn volume, veh/h:	33

OUTPUT

Variable	Value
Limiting right-turn volume, veh/h:	45
Guidance for determining the need for a major-road right-turn bay for a 2-lane roadway:	
Do NOT add right-turn bay.	

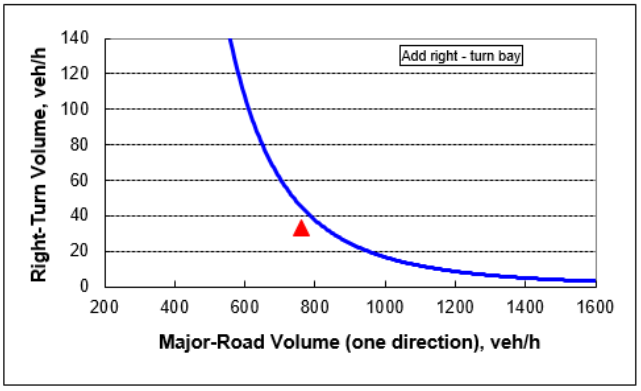
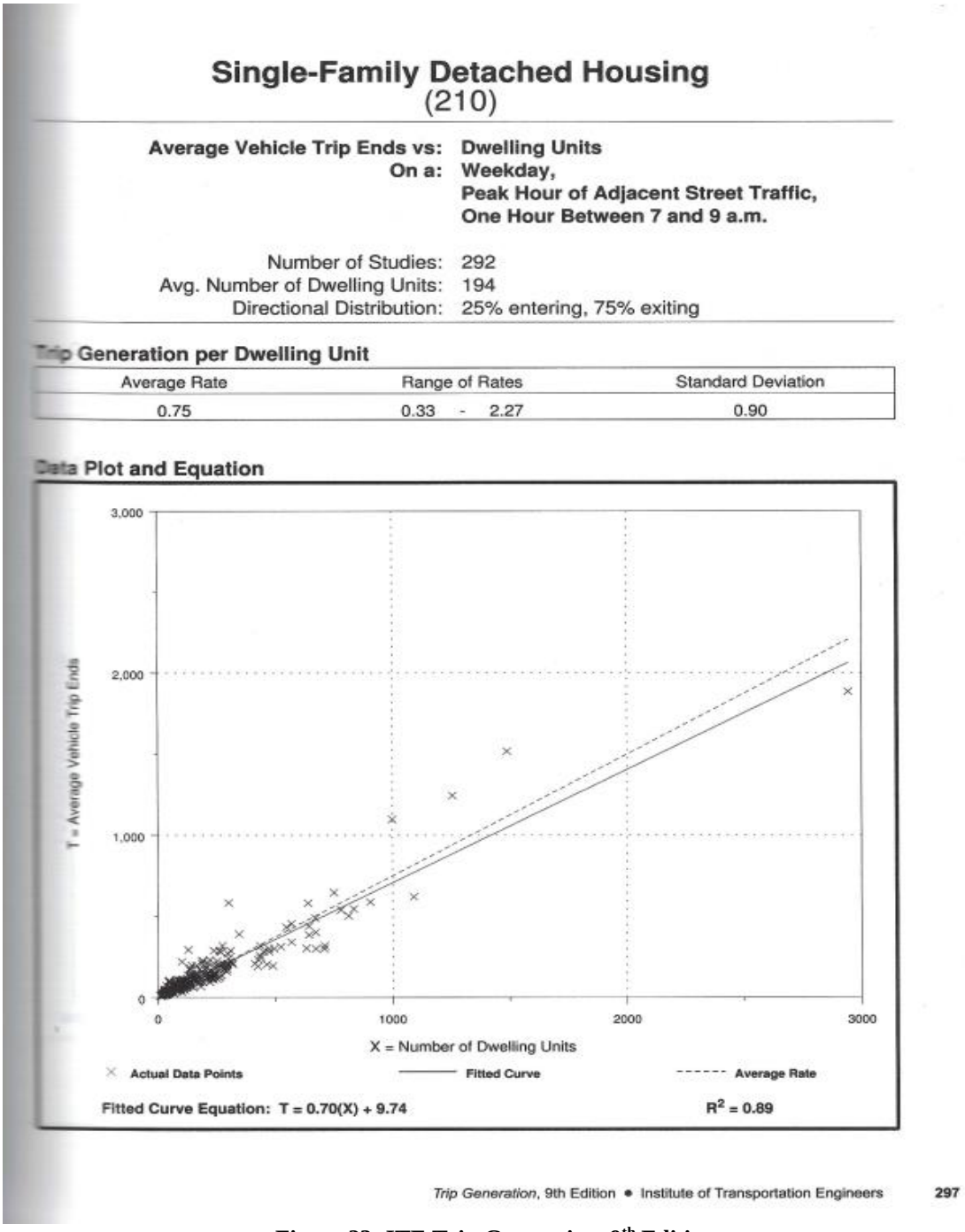


Figure 22: 2028 PM NCHRP 457 right-turn bay analysis

Appendix C

Trip Generation Derivation



Single-Family Detached Housing
(210)

Average Vehicle Trip Ends vs: **Dwelling Units**
On a: **Weekday,**
Peak Hour of Adjacent Street Traffic,
One Hour Between 7 and 9 a.m.

Number of Studies: 292
Avg. Number of Dwelling Units: 194
Directional Distribution: 25% entering, 75% exiting

Trip Generation per Dwelling Unit

Average Rate	Range of Rates	Standard Deviation
0.75	0.33 - 2.27	0.90

Data Plot and Equation

Fitted Curve Equation: T = 0.70(X) + 9.74

$R^2 = 0.89$

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Figure 23: ITE Trip Generation, 9th Edition

Single-Family Detached Housing
(210)

Average Vehicle Trip Ends vs: **Dwelling Units**
On a: **Weekday,**
Peak Hour of Adjacent Street Traffic,
One Hour Between 4 and 6 p.m.

Number of Studies: 321
Avg. Number of Dwelling Units: 207
Directional Distribution: 63% entering, 37% exiting

Trip Generation per Dwelling Unit

Average Rate	Range of Rates	Standard Deviation
1.00	0.42 - 2.98	1.05

Data Plot and Equation

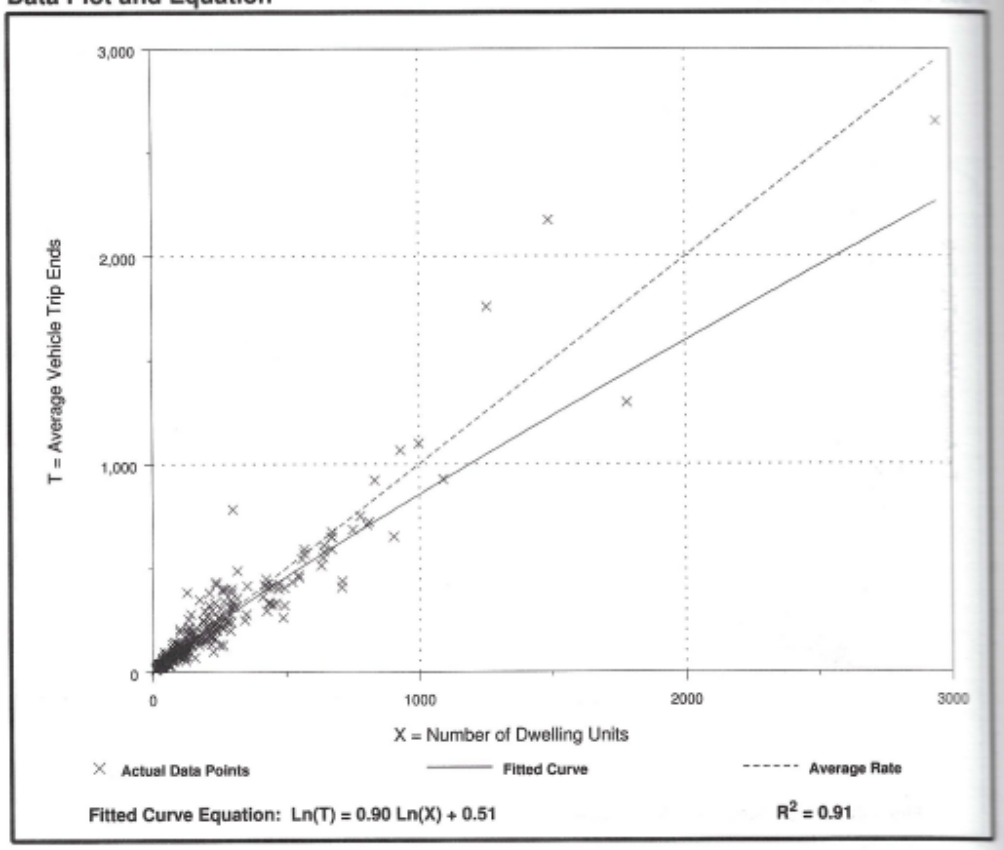

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Figure 24: ITE Trip Generation, 9th Edition

Specialty Retail Center
(826)

Average Vehicle Trip Ends vs: 1000 Sq. Feet Gross Leasable Area
On a: Weekday,
P.M. Peak Hour of Generator

Number of Studies: 3
Average 1000 Sq. Feet GLA: 75
Directional Distribution: 56% entering, 44% exiting

Trip Generation per 1000 Sq. Feet Gross Leasable Area

Average Rate	Range of Rates	Standard Deviation
5.02	4.59 - 6.18	2.31

Data Plot and Equation *Caution - Use Carefully - Small Sample Size*

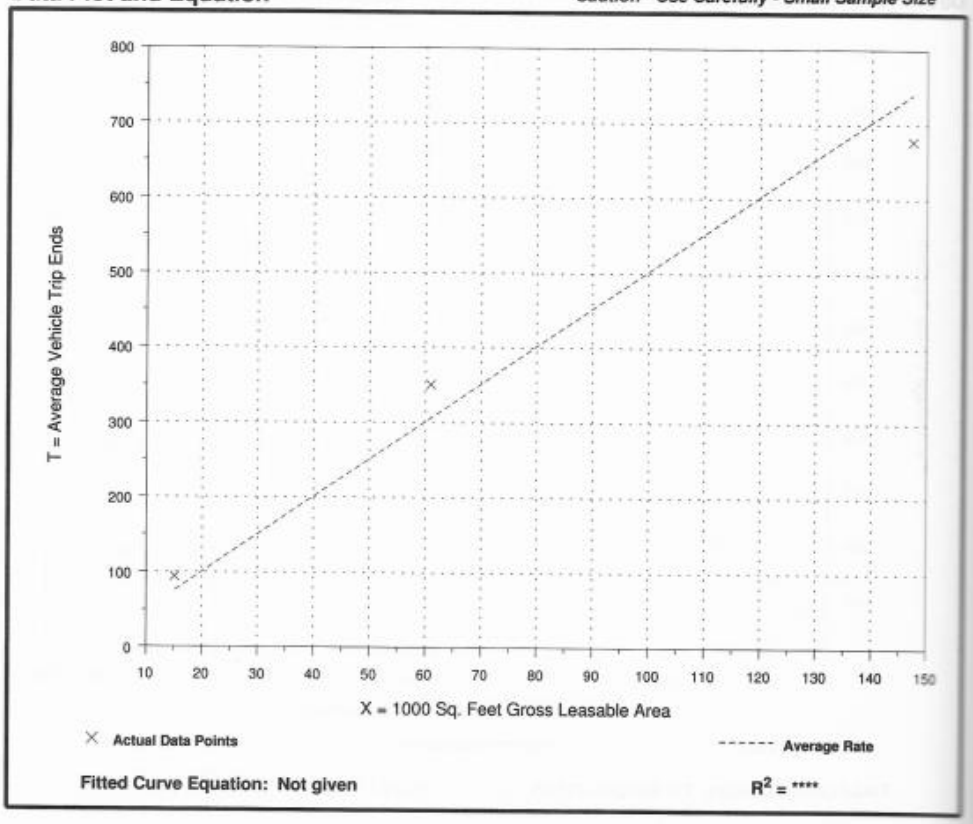

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Figure 25: ITE Trip Generation, 9th Edition

Drive-in Bank
(912)

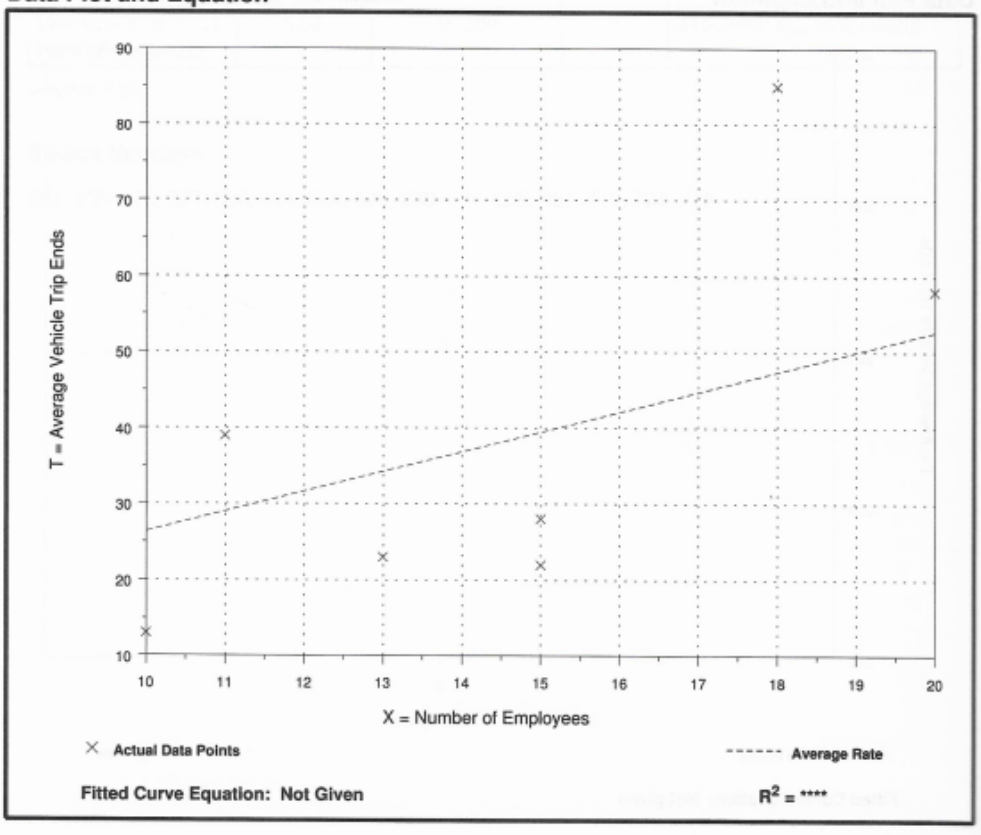
Average Vehicle Trip Ends vs: **Employees**
On a: **Weekday,**
Peak Hour of Adjacent Street Traffic,
One Hour Between 7 and 9 a.m.

Number of Studies: 7
Avg. Number of Employees: 15
Directional Distribution: 62% entering, 38% exiting

Trip Generation per Employee

Average Rate	Range of Rates	Standard Deviation
2.63	1.30 - 4.72	1.98

Data Plot and Equation



Fitted Curve Equation: Not Given R^2 = ****

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Figure 26: ITE Trip Generation, 9th Edition

Drive-in Bank
(912)

Average Vehicle Trip Ends vs: Employees
On a: Weekday,
Peak Hour of Adjacent Street Traffic,
One Hour Between 4 and 6 p.m.

Number of Studies: 11
Avg. Number of Employees: 13
Directional Distribution: 48% entering, 52% exiting

Trip Generation per Employee

Average Rate	Range of Rates	Standard Deviation
5.42	1.70 - 8.90	2.93

Data Plot and Equation

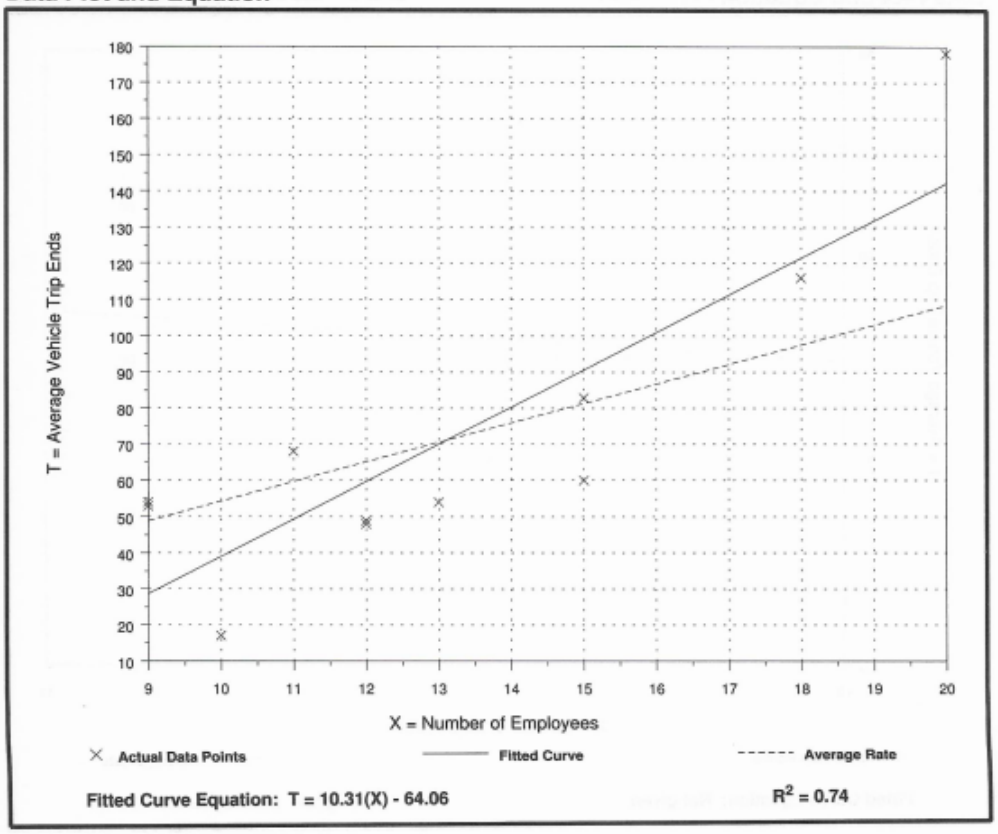

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Figure 27: ITE Trip Generation, 9th Edition

Appendix D

Miscellaneous

Year 2016 Monthly Data			
Group 4 Averages:		Urban Highways	
		Adjustment to	
Month	ADT	Average	Peak
January	13,573	1.16	1.25
February	14,038	1.12	1.21
March	15,731	1.00	1.08
April	16,139	0.97	1.05
May	15,705	1.00	1.08
June	16,766	0.94	1.01
July	15,752	1.00	1.08
August	16,529	0.95	1.03
September	17,007	0.92	1.00
October	16,598	0.94	1.02
November	15,649	1.00	1.09
December	14,638	1.07	1.16
Average ADT:	15,677		
Peak ADT:	17,007		

Table 15: Derivation of the seasonal peaking factor

Metric					US Customary				
			Stopping sight distance					Stopping sight distance	
Design speed (km/h)	Brake reaction distance (m)	Braking distance on level (m)	Calculated (m)	Design (m)	Design speed (mph)	Brake reaction distance (ft)	Braking distance on level (ft)	Calculated (ft)	Design (ft)
20	13.9	4.6	18.5	20	15	55.1	21.6	76.7	80
30	20.9	10.3	31.2	35	20	73.5	38.4	111.9	115
40	27.8	18.4	46.2	50	25	91.9	60.0	151.9	155
50	34.8	28.7	63.5	65	30	110.3	86.4	196.7	200
60	41.7	41.3	83.0	85	35	128.6	117.6	246.2	250
70	48.7	56.2	104.9	105	40	147.0	153.6	300.6	305
80	55.6	73.4	129.0	130	45	165.4	194.4	359.8	360
90	62.6	92.9	155.5	160	50	183.8	240.0	423.8	425
100	69.5	114.7	184.2	185	55	202.1	290.3	492.4	495
110	76.5	138.8	215.3	220	60	220.5	345.5	566.0	570
120	83.4	165.2	248.6	250	65	238.9	405.5	644.4	645
130	90.4	193.8	284.2	285	70	257.3	470.3	727.6	730
					75	275.6	539.9	815.5	820
					80	294.0	614.3	908.3	910

Note: Brake reaction distance predicated on a time of 2.5 s; deceleration rate of 3.4 m/s^2 [11.2 ft/s^2] used to determine calculated sight distance.

Exhibit 3-1. Stopping Sight Distance

Figure 28: Derivation of stopping sight distance requirements

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