Warrants for Exclusive Left Turn Lanes at Unsignalized Intersections and Driveways

Dr. John N. Ivan, PI
Dr. Adel W. Sadek, Co PI
Hongmei Zhou
Surang Ranade

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16. Abstract

Accommodating left turns at unsignalized intersections is one of the most challenging problems in traffic engineering. Over the last forty years, a small number of studies developed guidelines for traffic engineers to help in deciding when a left turn lane is warranted for a given situation. Building on these previous attempts, this report describes the development of a refined decision support system (DSS) for assessing the likely benefits of left-turn lane installations as an aid to deciding whether a left-turn lane is warranted. The developed DSS is designed to predict these likely benefits based on several measures including delay savings, reductions in percent stops, increases in fuel efficiency, and reductions in emissions. The first step in developing the DSS was to use microscopic simulation to model several real-world unsignalized intersections with different geometric configurations and located in different area types. After carefully calibrating these models, several scenarios covering a wide range of operational conditions were simulated. The output from these simulation runs was then used to train a set of Multi-layer Perceptron Neural Networks (NNs), and to generalize the results from the models' runs. The NNs were then incorporated into a DSS that can help an analyst quantify the impacts of a proposed new development as well as estimate the benefits of left-turn lane installations.

The report also presents an investigation of the safety effects of exclusive left turn lane installation at unsignalized intersections. Crash prediction models were estimated using crash and volume data from intersections without left turn lanes by intersection category and crash category. Intersections throughout Connecticut were selected, representing six different types of intersections based on rural versus urban location, number of approach legs and number of through lanes on the main road. Negative binomial modeling was used with generalized estimation equations to account for the correlation among the crash counts for an intersection through the years. The expected number of crashes was predicted using the prediction models for intersections with left turn lanes assuming no left turn lanes were present. If the observed crash counts were significantly lower than the predicted, then there was evidence that the left turn lane created a safer condition. The results of comparing the observed and predicted crashes showed that the intersections were safer for same direction crashes when left turn lanes were installed, except for those on urban two lane roads, at which no safety effect is discernable.

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SI* (MODERN METRIC) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
in	inches	25.4	millimetres	mm
ft	feet	0.305	metres	m
yd	yards	0.914	metres	m
mi	miles	1.61	kilometres	km
		AREA		
in^2	square inches	645.2	millimetres squared	mm^2
ft^2	square feet	0.093	metres squared	m^2
yd^2	square yards	0.836	metres squared	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	kilometres squared	km^2
		VOLUME		
fl oz	fluid ounces	29.57	mililitres	mL
gal	gallons	3.785	Litres	L
ft^3	cubic feet	0.028	metres cubed	m^3
yd^3	cubic yards	0.765	metres cubed	m^3

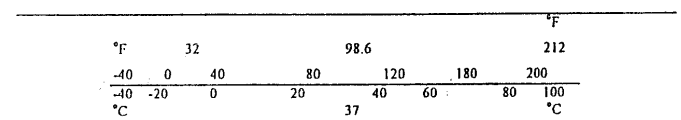
NOTE: Volumes greater than 1000 L shall be shown in m^3

Symbol	When You Know	Multiply By	To Find	Symbol
		MASS		
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams	Mg
		TEMPERATURE (exact)		
^oF	Fahrenheit temperature	5(F-32)/9	Celcius temperature	^oC

* SI is the symbol for the International System of Measurement

APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
mm	millimetres	0.039	inches	in
m	metres	3.28	feet	ft
m	metres	1.09	yards	yd
km	kilometres	0.621	miles	mi
		AREA		
mm^2	millimetres squared	0.0016	square inches	in^2
m^2	metres squared	10.764	square feet	ft^2
ha	hectares	2.47	acres	ac
km^2	kilometres squared	0.386	square miles	mi^2
		VOLUME		
mL	millilitres	0.034	fluid ounces	fl oz
L	litres	0.264	gallons	gal
m^3	metres cubed	35.315	cubic feet	ft^3
m^3	metres cubed	1.308	cubic yards	yd^3
		MASS		
g	grams	0.035	ounces	oz
kg	kilograms	2.205	pounds	lb
Mg	megagrams	1.102	short tons (2000 lb)	T
		TEMPERATURE (exact)		
^oC	Celcius temperature	1.8C+32	Fahrenheit temperature	^oF



Executive Summary

The objectives of this project were to develop warrants for installing exclusive left turn lanes (LTL) at unsignalized intersections and driveways considering operations and safety. The operational warrants were developed by determining the volume at which the average delay or the number of stops becomes unacceptable without a LTL. Safety was considered by investigating the conditions under which a LTL improves the safety of the intersection. A second objective was to learn how to design exclusive LTL at unsignalized intersections and driveways to make them safer. The project was motivated by the following concerns: left turning vehicles waiting in the through lane impose potentially unacceptable delay on through vehicles that can be avoided by installing a LTL, and left turning vehicles waiting in the through lane are exposed to the potential for being struck by through vehicles. The question to be answered is "when do conditions warrant a LTL based on each factor?"

For the operational analysis, microscopic simulation models were developed for a representative set of real-world unsignalized intersections with different geometric configurations (i.e. two and four lanes) and different locations (i.e. urban, suburban, and rural). The selected intersections were videotaped during the peak hour and the following data were extracted:

- Advancing, opposing and left turning volumes
- Basic geometric information
- Discharge headway from a queue;
- Average & maximum queue length
- Stopped delay at the subject link; and
- Gap acceptance behavior

The following parameters were adjusted to calibrate the models: queue discharge headway, gap acceptance behavior, and vehicle entry distribution. Calibration aimed to match the following measures of effectiveness (MOE): stopped delay at the subject link, the average queue on the subject link and the maximum queue on the subject link.

The calibrated models were run 150 times for different volume and speed combinations, with the output used to train two separate neural networks for two cases: one with a left turn lane and the second one without. The NN training results were used to generate warrants based on two criteria: control delay and the number of stops in the advancing traffic stream. Then a decision support system (DSS) was developed using artificial neural networks for predicting the likely benefits of left-turn installations. From this analysis, new warrants were developed and compared against the existing ones endorsed by AASHTO. These warrants are presented in the report as a series of graphs indicating thresholds above which a LTL is warranted. Also, a series of graphs is provided to illustrate the expected benefits of a LTL in different scenarios.

The objectives of the safety analysis were to examine the safety effects of LTL at unsignalized intersections and driveways, and in particular to answer the following questions:

- Where does a LTL significantly reduce accidents?
- Where does a LTL have no appreciable effect on accidents?
- Is there any conflict between these findings and those of the operational analysis?

Intersections were selected to represent all combinations of the following criteria: (1) with and without LTL (Y or N), rural and urban/suburban areas (R or U), two-lane and four-lane roads (2 or 4), T- and four-way intersections (T or X). The crash counts observed at intersections with LTL were compared to counts predicted using crash prediction models estimated for intersections without LTL. Differences between what was observed with a LTL and what was predicted without are then used to identify the potential safety effects of installing LTL.

A sample of intersections was selected from roads in Connecticut using the Connecticut Department of Transportation (ConnDOT) photolog system. For each combination, a target of five intersections with LTL for study and fifty intersections without LTL for estimating the prediction functions were selected. Data were collected from ConnDOT archives describing crashes within 250 ft of the intersection and the Annual Average Daily Traffic (AADT) on the main road for ten years from 1995 to 2004. Intersections on four-lane rural roads were not studied because there were not enough intersections available from which to select. Statistical models were estimated for predicting same direction crashes and injury and fatal crashes. We focused on these crash subsets because same direction crashes are the category expected to be reduced by having a LTL, and because injury and fatal crashes have more severe consequences and are therefore of greater interest for crash reduction.

The models were estimated using negative binomial regression, which permits estimation of a dispersion parameter (*k*) that represents how much the variance of expected crashes differs from the mean. We also used a repeated measures framework, in which each year of data was considered a separate observation. This required that we also use the Generalized Estimating Equations procedure to account for potential correlation among years at the same site. The results were the following:

- For four-leg intersections on rural two-lane roads and three-leg intersections on urban four-lane roads, there is strong evidence that installing a LTL will reduce rear-end crashes.
- For three-leg intersections on rural two-lane roads and four-leg intersections on urban four-lane roads, there is some evidence that installing a LTL will reduce rear-end crashes.
- For three-leg and four-leg intersections on urban two-lane roads, there is no evidence that installing a LTL will reduce rear-end crashes.

In no situations was it found that installing a LTL is likely to increase the number of rear-end crashes. The results for injury and fatal crashes are similar.

Physical intersection characteristics were also examined to identify how their characteristics affect safety with and without left turn lanes. It was found that LTL safety benefits are limited when taper and storage length are short, multiple driveways use the same LTL and other geometric features increase the crash potential (e.g., horizontal curves). Consequently, it is recommended to follow AASHTO guidelines for taper and storage length and avoid designing the LTL to serve multiple driveways. These situations can likely lead to confusion for drivers behind the left turning vehicle as to where the turn will be made. In any case, the benefits of a left turn lane will be difficult to achieve under these conditions.

In conclusion, there is no contradiction between the operational and safety findings. That is, there are no cases where the volume warrants suggest a LTL but safety analysis recommends none. It is, however, possible that a LTL may not be warranted by volume, but it could improve safety substantially. The recommendation therefore is that a LTL should be considered at all four-leg intersections on rural two-lane roads and three-leg intersections on urban four-lane roads on the basis of safety irrespective of whether or not the LTL is warranted on the basis of traffic volume. At other locations, the warrants in the report should be followed.

Table of Contents

List of Figures

List of Tables

List of Symbols and Acronyms

λ - Arrival rate
μ - Service rate
L - Percentage of left turning volume in the advancing volume
V_A - Advancing Volume (veh/hour)
t_w - Average time a left turning vehicle have to wait to find a suitable gap in the opposing stream
t_e - Time required for a left-turning vehicle to clear itself from the advancing queue
t_A - The median headway of the advancing stream
V_O - Opposing Volume (veh/hour)
t_l - Time taken to complete a left-turn maneuver (sec)
G_C - Critical gap for a particular site (sec)
t_l - Time taken to complete a left-turn maneuver (sec)
μ_{ij} - Expected mean of the crashes occurring at intersection i in year j
V_{ij} - AADT of the major road of the intersection i in year j

AADT – Annual average daily traffic
ACTHD - Average delay to through vehicles caught behind the left turning vehicles
ALTD - Average delay to left turning vehicles
ATD - Average delay to caught and not caught vehicles
CCMPO - Chittenden County Metropolitan Planning Organization
ConnDOT - Connecticut Department of Transportation
DS - Total delay saving per hour as a result of the left turn lane
DSS - Decision support system
GLIM - Generalized linear modeling
MOE's - Measures of effectiveness
NN's - Neural Networks
NTVC - Number of through vehicles caught behind the left turning vehicles
R2T - Rural, two-lane, T-intersection
R2X - Rural, two-lane, four-leg intersection
SPF – Safety performance function
TD - Total hourly delay
TWLTL - Two-way-left-turn lanes
U2T - Urban, two-lane, T- intersection
U2X - Urban, two-lane, four-leg intersection
U4T - Urban, four-lane, T- intersection
U4X - Urban, four-lane, four-leg intersection

1 INTRODUCTION

1.1 Background and Motivation

Accommodating left turns from main roads is one of the most challenging problems in traffic engineering. At unsignalized intersections and driveways, when there is no protected left turn signal phasing, vehicles turning left from a main road into a minor street or a driveway must yield to the oncoming traffic stream and wait for a suitable gap to complete the left turn maneuver safely. When an exclusive left turn lane is provided, only the vehicle waiting to make the left turn incurs the resulting delay. However, when such an exclusive left turn lane is not provided, left turning vehicles must wait in the same lane as the through traffic in the same direction, thus causing the through traffic to incur some of the delay (when there are at least two-lane in that direction), or all of the delay (when there is only one lane). Left turning vehicles, waiting for a suitable gap in the oncoming traffic are also exposed to the possibility of being struck by the through vehicles approaching from behind. In addition to that, the desire to avoid this risk may also induce left turning vehicles to choose insufficiently long gaps in the oncoming traffic which might further potentially increase the risk of a crash. Providing an exclusive left turn lane reduces this risk to some extent.

Many agencies which have jurisdiction over arterial roads are faced with the need to install exclusive left turn lanes at unsignalized intersections and driveways, to reduce unnecessary delays to through vehicles and also to reduce the crash risk. This need is most often generated as a consequence of increasing land development along major roads, which increases the number of driveways, and hence, the number of left turns and the resulting delays and crash risk.

At first, it might seem appropriate to install exclusive left turn lanes in all such situations. However, it is also important to remember that such a policy is not financially feasible, as each left turn lane installation requires either widening of the pavement or re-striping of lanes without widening. Widening the pavement obviously involves substantial planning, design, land acquisition and construction costs. Re-striping without widening also involves the costs of losing either a through lane or a portion or all of the shoulder or parking lane. This in turn results in narrower lanes, which again might adversely affect the operational efficiency and safety of the subject link. Moreover, it also can require relocation of storm water collecting devices, which also increases the cost. Thus, the decision about whether to install an exclusive left turn lane or not cannot be taken without detailed analysis of the situation. In traffic engineering, “warrants” are used to help with decisions like this, for example for deciding where to install traffic signals. In this context, the warrant justifies a given action when the specified conditions are met. That action could be the installation of an exclusive left or right turn lane or signalization.

1.2 Objectives and Scope

There are two objectives to this research project. The first is to review the left turn lane warrants currently used by various road authorities; build, calibrate and validate traffic simulation models; develop a decision support system to evaluate likely benefits of installation of left turn lane; and develop new left turn lane warrants based on traffic volume.

The second objective is to examine the safety effects of left turn lane installation. Negative binomial modeling is used to compare the crash experience at intersections with and without left turn lanes, especially noting the contributions of other conditions (e.g., volume level, area type, and roadway geometry). Together with the volume-based warrants, the results consider not only traffic volumes, but also observed safety experience and other pertinent characteristics of the intersection or driveway that affect the overall safety effect of installing the left turn lane.

1.3 Outline of Report

In order to clearly explain the fundamental concepts as well as the developed methodologies and analysis of results in this research project, the report has been assembled as follows:

Chapter 1 describes the motivation and objectives and presents the problem statement and objectives.

Chapter 2 describes a review of the relevant literature, including not only critically review of previous attempts at the development of left-turn lane warrants at unsignalized intersections, but also warrants that are currently used by different transportation agencies to evaluate the theoretical background, strengths and limitations. Also included is a review of previous studies related to left turn safety at unsignalized intersections, especially the safety effects of left turn lane installation.

Chapter 3 describes the methodology for development of volume warrants for left turn lanes, starting with the general study approach followed by description of the different steps followed such as site selection, data collection and reduction, simulation model development and calibration, development of artificial neural networks, and finally, the development of the decision support system.

Chapter 4 describes the development of new left turn lane warrants, based on the models described in Chapter 3, for different road categories such as urban two-lane roads, rural two-lane roads and urban four-lane roads, based on different warrant criteria such as total delay and total number of stops per hour.

Chapter 5 describes the study design of the safety analysis. General study approaches are given. How the study sites are selected is explained as well as the steps of collecting and compilation of data. This chapter also provides the methodology, first with a review of Negative Binomial modeling that was used in modeling the crashes in intersections without left turn lanes. The prediction models are then applied to those intersections with left turn lanes. Safety performance functions are defined to compute the expected number of crashes in intersections with left turn lanes if no left turn lanes were there.

Chapter 6 describes the results of the safety analysis. The expected number of crashes in each intersection is compared with the actual number of crashes occurring there, so the “dangerous” intersections are detected. Pertinent characteristics of intersections with unusual safety experience are considered to identify explanations for the findings.

Chapter 7 summarizes the results and the conclusions of both parts of the study, including suggestions for application and future research.

2 LITERATURE REVIEW

There are several studies carried out in the past which contribute towards the development of left turn lane warrants. The first major study was carried out by Harmelink (1967), which was followed by another major study by Kikuchi and Chakroborty (1991) which is a major improvement over Harmelink's study. Another remarkable study was carried out by Fitzpatrick, Brewer and Parham (2003). The most recent study was carried out by Lakkundi *et al.* (2004). This chapter focuses on these four seminal studies, but also addresses a few more relevant studies about left turn lane operation and safety.

2.1 Harmelink's Study

As mentioned above, the oldest left-turn lane warrants were published by Harmelink (1967). These warrants are the basis for AASHTO (2001) guidelines for justifying a left-turn lane at an unsignalized intersection. The warrants developed by Harmelink are in the form of sets of different volume combinations, specifically, the advancing volume (V_A), the percentage of left-turns in the advancing volume (P_L), and the opposing volume (V_O). The warrants were developed for the approach speeds of 40, 50 and 60 mph.

The warrants developed by Harmelink try to minimize the conflict between the left turning vehicles and through vehicles approaching from behind. To be specific, these warrants are based on the probability that one or more through vehicles are present in the queue formed by the left-turning vehicles that is waiting for a suitable gap. Harmelink determined values for the maximum allowable probabilities based upon the judgment of a panel of traffic engineers. He then computed the combination of the three volumes (*i.e.* advancing, left-turn and opposing) for each value of the probabilities suggested by the panel of traffic engineers. This was done analytically on the basis of queuing theory. Specifically, Harmelink's queuing system assumes that the arriving units are the through vehicles arriving behind the left-turning vehicles, and that the service is the departure of the left-turning vehicles. Given this, Harmelink formulated the arrival rate (λ) and the service rate (μ) of the queuing system as follows:

For the arrival rate, λ:

$$\lambda = L(1-L)(V_a)\left(\frac{t_w + t_e}{\left(\frac{2}{3}\right)t_A}\right) \tag{2.1}$$

Where,
L = Percentage of left turning volume in the advancing volume
V_A = Advancing Volume (veh/hour)
t_w = Average time a left turning vehicle have to wait to find a suitable gap in the opposing stream
t_e = Time required for a left-turning vehicle to clear itself from the advancing queue
t_A = The median headway of the advancing stream

t_w, the average time a left turning vehicle must wait to find a suitable gap in the opposing stream which is used in equation 2.1 can be calculated using equation 2.2.

$$t_w = \frac{3600}{V_O}\left(e^{\frac{V_O}{3600}G_C} - \frac{V_O}{3600}G_C - 1\right) \tag{2.2}$$

Where,
V_O = Opposing Volume (veh/hour)
t_l = Time taken to complete a left-turn maneuver (sec).
G_C = Critical gap for a particular site (sec)

For the service rate, μ:

$$\mu = \frac{Total\ unblocked\ time}{t_l} \tag{2.3}$$

Where,
t_l = Time taken to complete a left-turn maneuver (sec)

With the arrival and service rates determined from equations 2.1 and 2.2 above, the probability that one or more units are in the system can be calculated.

Given λ and μ, the probability of k units in the system would be:

$$P(K) = \left(\frac{\lambda}{\mu}\right)^k \left(1 - \frac{\lambda}{\mu}\right) \tag{2.4}$$

So, the probability of no vehicles behind the left turning vehicles would be,

$$P(0) = \left(\frac{\lambda}{\mu}\right)^0 \left(1 - \frac{\lambda}{\mu}\right) \tag{2.5}$$

$1 - P(0)$ represents the probability of one or more through vehicles behind a left turning vehicle in the system,

$$1 - P(0) = \frac{\lambda}{\mu} \leq \alpha = 0.02 \tag{2.6}$$

So, according to Harmelink if, the probability of one or more through vehicles present behind the left turning vehicle is greater than 0.02 for 40 mph. operating speed, an exclusive left turn lane is justified. Probability values that Harmelink uses to base the warrants are different for different operating speed as shown in Table 2.1. Assumptions for the values of different parameters which are used in the Harmelink's model as follows:

- Average time required for making a left turn (t_l) is 3.0 sec for two-lane highways and 4.0 sec for four-lane highways as determined from field studies.
- Critical headway in the opposing traffic stream (*Gc*) for a left-turn maneuver is 5.0 sec on two-lane highways and 6.0 sec on four-lane highway as determined from field studies.
- Average time required for a left-turning vehicle to clear or "exit" from the advancing lane (t_e) is 1.9 sec as determined from field studies.

Table 2.1: Probability Values for Different Operating Speeds as Suggested by Harmelink (1967)

Approach Speed (mph)		Probability of through vehicles behind left turn vehicle
Design	Operating	
50	40	0.02
60	50	0.015
70	60	0.01

These parameters can be established from field measurements at unsignalized intersections. The other parameters that are used in the queuing system are a direct function of the three volumes (advancing, left and opposing). It should be noted, however, that while the Harmelink warrants are still in use, they actually have several limitations as pointed out by Kikuchi and Chakroborty (1991), and described in the following section. Table 2.2 shows the warrants developed by Harmelink for different operating speeds as well as different left turning percentages.

2.2 Kikuchi and Chakraborty Study

In 1991, Kikuchi and Chakraborty of the University of Delaware not only critically reviewed the Harmelink warrants and pointed out a number of serious theoretical flaws and limitations with the Harmelink model, but they also developed three new sets of warrants using (1) probability criteria as suggested by Harmelink, (2) delay to through vehicles, and (3) degradation in Level of Service as warrants criteria. This section describes their work.

2.2.1 Probability criteria as suggested by Harmelink

Kikuchi and Chakraborty (1991) pointed out the three main flaws in the Harmelink model. The first flaw concerns the inconsistent definitions of the arrival and service rates. In queuing theory both the arrival and departure rate should have the same units. This, however, is not the case in the Harmelink model. As mentioned above, in Harmelink's model the arrival rate refers to the *through* vehicles behind the left turning vehicles, whereas the service rate refers to the *left-turning* vehicles. This inconsistency leads to erroneous results when more than one through vehicle is queued behind the left-turning vehicle.

Table 2.2: AASHTO Guidelines for Left Turn Lanes on Two-lane Highways

Opposing Volumes	Advancing Volumes			
	5 % Left Turns	10 % Left Turns	20 % Left Turns	30 % Left Turns
	40 - mph operating speed			
800	330	240	180	160
600	410	305	225	200
400	510	380	275	245
200	640	470	350	305
100	720	575	390	340
	50 - mph operating speed			
800	280	210	165	135
600	350	260	195	170
400	430	320	240	210
200	550	400	300	270
100	615	445	335	295
	60 - mph operating speed			
800	230	170	125	115
600	290	210	160	140
400	365	270	200	175
200	450	330	250	215
100	505	370	275	240

Source: AASHTO (2001)

The second flaw concerns the issue of residual gaps. In the Harmelink model, the service rate is calculated by considering the sum of gaps that are greater than the critical gap and dividing that sum by the time required for completing a left-turn maneuver. The problem here, however, is that the residual gaps (*i.e.* the remainder of individual gap after subtracting the value of the critical gap) are added up and that sum is considered to be part of the time available for making left-turns. As pointed out by Kikuchi and Chakroborty, this tends to exaggerate the number of opportunities available for making left-turns. For example, suppose there are a total of four gaps of seven seconds each available in the opposing traffic and the time required for completing the left turn maneuver is four seconds. Then, according to Harmelink's equation seven left turning vehicles would be served in that period of time but practically, only four vehicles should depart in that much time.

Third, the basis of the warrants (*i.e.*, the probability that one or more through vehicles queue behind the left-turning vehicle) is somewhat questionable. First, the probabilities that Harmelink used are quite subjective. Second, as pointed out by Kikuchi and Chakroborty, if the probability-based warrants are used, the total delay savings vary more than 20 times for the same threshold probability. Third, judging by the speeds that Harmelink assumed in his model, the warrants appear to have been mainly intended for high-speed rural highways.

Finally, the different values of the parameters used by Harmelink (such as Critical Gap headway (G_c), average time a left turn vehicle has to wait before finding a suitable gap in the opposing traffic (t_w) and time required to clear advancing lane (t_l)) correspond to conditions of the roads and state of vehicles present four decades ago (*i.e.* in 1967) and which may not be applicable to the current state of roads as well as vehicles. In addition, the warrants were developed primarily for rural areas, and their application to the urban setting, therefore, may be inappropriate.

To address the above-mentioned problems, Kikuchi and Chakroborty first suggested a more refined analytical formulation that avoids the two theoretical flaws of the Harmelink model. The newly developed equations by Kikuchi and Chakraborty use arrival and departure rates which have consistent units and also make sure that the residual gaps are not added up leading to erroneous results.

The modified equations for the arrival, λ, and the service rate, μ, which Kikuchi and Chakraborty (1991) developed are given below in equations 2.7 and 2.8 respectively:

$$\lambda = L \cdot V_A \left[1 - e^{\frac{-(1-L)V_A}{3600}(t_w + t_e)} \right] \tag{2.7}$$

$$\mu = \left[1 - e^{-3\left(\frac{Vo}{3600}\right)}\right] \cdot Vo \cdot \sum_{n=1}^{N} n \left\{ e^{-\frac{Vo}{3600}[Gc+3(n-1)]} \right\} \tag{2.8}$$

Using this newer formulation, they then revised the volume warrants based on the probability values suggested by Harmelink as shown in Table 2.3. The value of N is the maximum number of left turning opportunities per a single headway. The value of N is calculated by solving equation 2.9.

$$probability\ \{headway \geq G_C + N.G_S\} \approx 0 \tag{2.9}$$

where Gs = Follow up gap size (sec)

Table 2.3: Modified Volume Warrants by Kikuchi and Chakraborty (1991) Based on the Probability Criteria as Suggested by Harmelink (1967)

Opposing Volumes (vph)	Advancing Volumes (vph)			
	5 % Left Turns	10 % Left Turns	20 % Left Turns	30 % Left Turns
	40 - mph operating speed			
800	434	300	219	189
600	542	375	272	134
400	682	472	343	293
200	863	600	435	375
100	946	679	493	424
50 - mph operating speed				
800	366	257	185	162
600	460	320	234	202
400	577	403	294	255
200	735	513	373	324
100	830	576	424	365
60 - mph operating speed				
800	294	207	154	146
600	365	259	187	165
400	461	324	238	206
200	586	414	303	263
100	663	468	344	297

So, according the newer formulation the arrival rate (λ) is the number of arriving units per unit time. One arriving unit is a left turning vehicle followed by one or more through vehicles. The departure rate (μ) is the departure of the arriving units per unit time.

2.2.2 Delay to Through Vehicles as a Warrant Criteria

In addition to correcting the analytical formulation of Harmelink, Kikuchi and Chakroborty developed another set of volume warrants based on delay to through vehicles. To do this, Kikuchi and Chakroborty first developed their own simulation model, because, at that time the commercially-available models had several limitations with respect to modeling unsignalized intersections as well as computing the different delay values. The warrants were then developed from the model output. The model output as suggested by Kikuchi and Chakraborty was the following.

- Total hourly delay (TD)
- Average delay to left turning vehicles (ALTD)
- Average delay to through vehicles caught behind the left turning vehicles (ACTHD)
- Average delay to caught and not caught vehicles (ATD)
- Total delay saving per hour as a result of the left turn lane (DS)
- Number of through vehicles caught behind the left turning vehicles (NTVC)

The measures of effectiveness used for the calibration of the model were average delay to left turning vehicles (ALTD) and number of vehicles caught behind the left turning vehicles (NTVC). For calibrating the simulation model which they developed, they compared NTVC from the simulation model with the λ from the equation 2.6. Similarly, the ALTD from the simulation model was compared with the t_w from the equation 2.2. Then, the simulation model was run many times and regression analysis was used to develop general relationships of the different parameters such as TD, ACTHD, ATD, DS with the Opposing, Advancing, left turning volumes and operating speed. Then these regression equations were used to developed warrants based on different criteria such as average delay to through vehicles, total delay and delay savings.

Figure 2.1 shows the warrants developed by Kikuchi and Chakraborty based on the average delay to through vehicles as warrant criteria and the thresholds they used to warrant the left turning vehicle for two-lane facilities. The justification behind the values of thresholds they used for developing warrants could not be found in the paper. So, as shown in figure, for a particular volume combination if the plotted point is above the curve, an exclusive left turn lane is warranted.

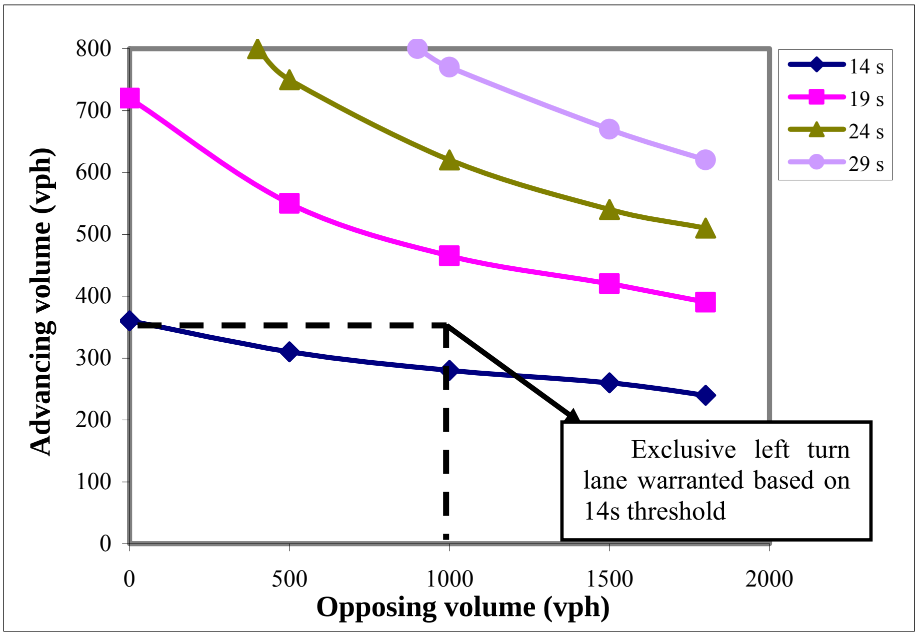


Figure 2.1: Warrants Developed by Kikuchi and Chakraborty Based on the Average Delay to Through Vehicles as Warrant Criteria

2.2.3 Degradation of Level of Service from A to B as a Warrant Criteria

Kikuchi and Chakraborty then developed warrants based on the degradation of Level of Service from A to B based on different volume combinations of V_A, V_O, and L. Figure 2.2 shows the volume warrants for installation of a left turn lane for different left turn percentages and operating speed of 40 mph based on degradation of level of service from A to B as warrant criteria, applied to the peak hour. This criterion may not seem to be reasonable for traffic engineers as in the field a level of service of C is considered acceptable.

The simulation model used by Kikuchi and Chakraborty was rather simplistic as compared to current microscopic simulation models. The model was also deterministic and suffered from the limitation that the critical gap was assumed to be fixed. Apart from the limitations of the simulation model developed by Kikuchi and Chakraborty, the new warrants represent a major improvement over the Harmelink warrants which incorporate other warrants criteria that are much easier for the general public to appreciate compared to probability values, in deciding whether or not a left-turn is warranted at unsignalized intersections.

2.3 Fitzpatrick, Brewer and Parham

More recently, Fitzpatrick, *et al.* (2003) reviewed eight methods currently used to determine warrants for left turn lanes. The researchers also reviewed several state guidelines on installing the left turn lane. In this project researchers found that most of the methods currently used to warrant left turn lanes, are based on the Harmelink's model. They also found out that the assumptions made by Harmelink for the values of different parameters such as G_c, t_e and t_l are on the lower side or overly cautious and should be modified. They also carried out a study and proposed modified values for the above mentioned parameters and using modified values, they developed new set of warrants using Harmelink's model. The modified values as suggested by the Fitzpatrick, Brewer and Parham for the different parameters were as following.

- Critical headway (G_c) for a left-turn maneuver is 5.5 sec.
- Time to complete the left turn and clear the opposing lane (t_l) is 4.3 sec.
- Time to clear the advancing lane (t_e) is 3.2 sec.

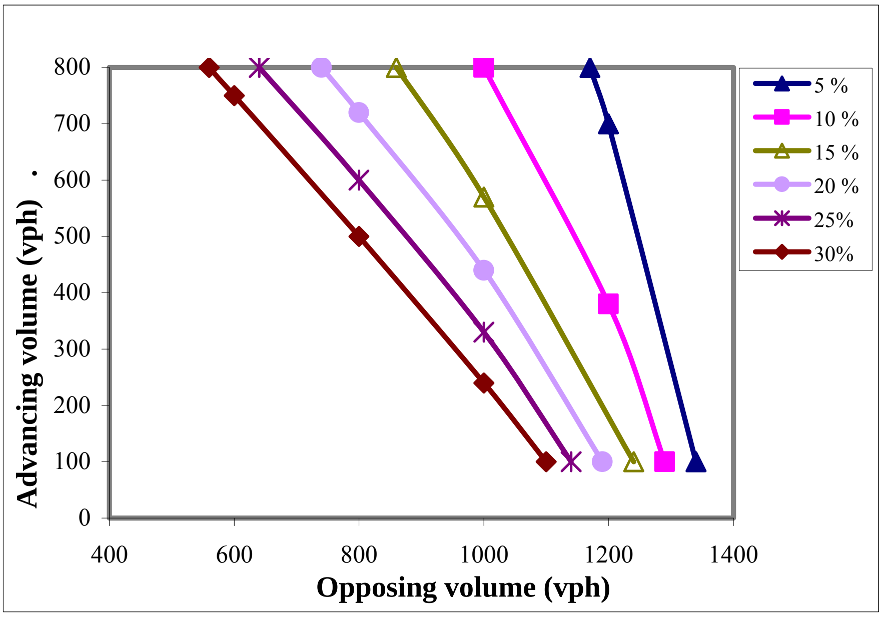


Figure 2.2: Warrants Developed By Kikuchi and Chakraborty (1991) Based on the Degradation of Level of Service from A to B as Warrant Criteria

Figure 2.3 shows the comparison of the Harmelink's warrants and the modified Harmelink's warrants using the different values of parameters such as G_c, t_l and t_e as suggested by Fitzpatrick, Brewer and Parham. As compared with the Harmelink's warrants these new warrants developed seem to be more conservative.

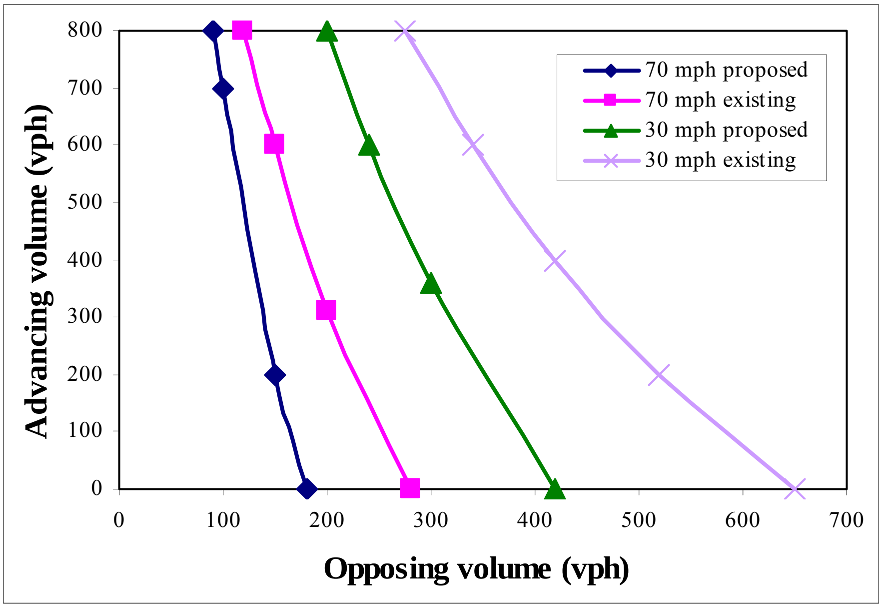
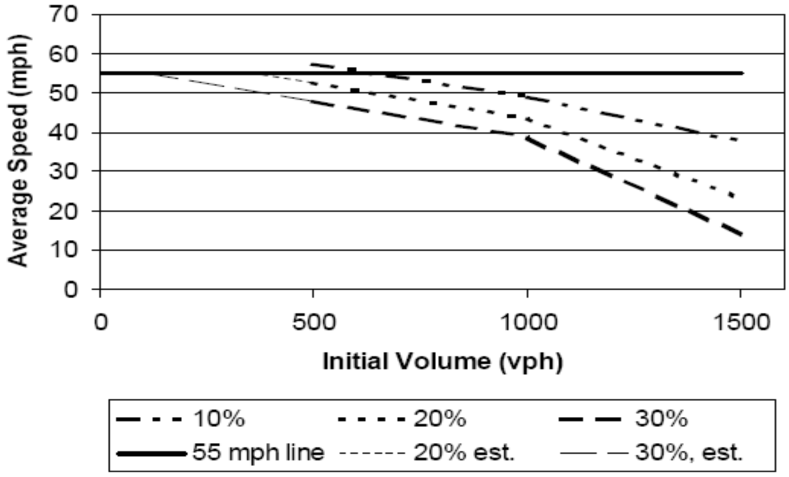


Figure 2.3: Comparison of the Harmelink's Warrants and Modified Harmelink's Warrants

However, Fitzpatrick *et al.* failed to address the several shortcomings in the Harmelink's model as pointed out Kikuchi and Chkraborty (1991). They also carried out simulation modeling of rural two-lane unsignalized intersections using VISSIM simulation model. One noticeable aspect of the simulation modeling was that they also considered scenarios when the impeded vehicles used the shoulders to overtake the left turning vehicle waiting for a suitable gap in the opposing traffic. So, for each operating speed they had three scenarios 1) when no vehicle use shoulders to overtake left turning vehicle 2) when 25% of the vehicles use shoulders to pass the left turning vehicle, and 3) when 90% of the vehicles use shoulders to pass the left turning vehicle. They also presented a new set of warrants for the two-lane facilities. The new warrants are based on the change in the average speed of the subject link. So, if the average speed on the subject link is lower than the specified threshold value, a left turn lane is warranted. Figure 2.4 shows the warrants based on the average speed on the subject link warrant criteria developed by Fitzpatrick, Brewer and Parham. The x-axis of Figure 2.4 represents the initial volumes which is a combination of opposing and advancing volumes (40% opposing and 60% advancing).



(1 mph = 1.61 km/h)

Figure 2.4: Warrants Developed by Fitzpatrick, Brewer and Parham

2.4 Virginia Transportation Research Council study

Recently, a study was undertaken by the Virginia Transportation Research Council to develop new left-turn guidelines for both unsignalized and signalized intersections (Lakkundi *et al.*, 2004). These warrants were developed on the basis of well-validated, event-based simulation programs "LTGAP" which the authors developed themselves, and calibrated based upon field data collected at a number of intersections from the Commonwealth of Virginia. One advantage of the VTRC study over Harmelink's (1967) and Kikuchi and Chakraborty (1991) was that they used more accurate modeling techniques which incorporated a stochastic gap acceptance module. The models were calibrated based on the number of left turning vehicles stopped on the subject link.

For the unsignalized intersections they developed left turn lane volume warrants based on the probability criteria as suggested by Harmelink. The authors of the VTRC study also developed warrants for signalized intersections. If it is decided to provide an exclusive left turn lane at a particular intersection, the length of the lane also needs to be determined. So, the authors of this study also recommended the length of the proposed left turn lane. Since, the purpose of installing a left turn lane is to prevent left-turn overflows; the probability of left-turn lane overflows for varying left-turn lane lengths was investigated, which, was later used to recommend the left-turn lane length for the candidate intersections. In addition to the general guidelines, the authors of the Virginia Transportation Research Council also developed a prioritization tool that can be used to prioritize candidate intersections which accounts for both operational and safety aspects.

Despite extensive improvements over the previous attempts at developing left turn lane warrants, the VTRC warrants are still based on the probability criterion suggested by Harmelink. As pointed out by Kikuchi and Chakroborty, this practice is quite subjective and somewhat questionable. Figure 2.5 shows the left turn lane warrants developed by the authors of VTRC for the different operating speeds and left turning percentage of 10 percent. If compared with the other warrants such as Harmelink's and modified Harmelink's warrants, these new warrants seem to be less conservative for lower opposing volumes but for higher opposing volumes there is not much difference between these new warrants and earlier warrants.

2.5 NCHRP Report 279

In 1985, the Transportation Research Board published NCHRP Report 279, Intersection Channelization Design Guide (Neuman 1985). In this report, to decide the necessity of the left turn lane, Harmelink's model was used. The guide provides the following advice for unsignalized intersections:

- Left-turn lanes should be considered at all median cross-overs on divided, high-speed highways.
- Left-turn lanes should be provided at all unstopped (*i.e.*, through) approaches of primary, high-speed rural highway intersections with other arterials or collectors.

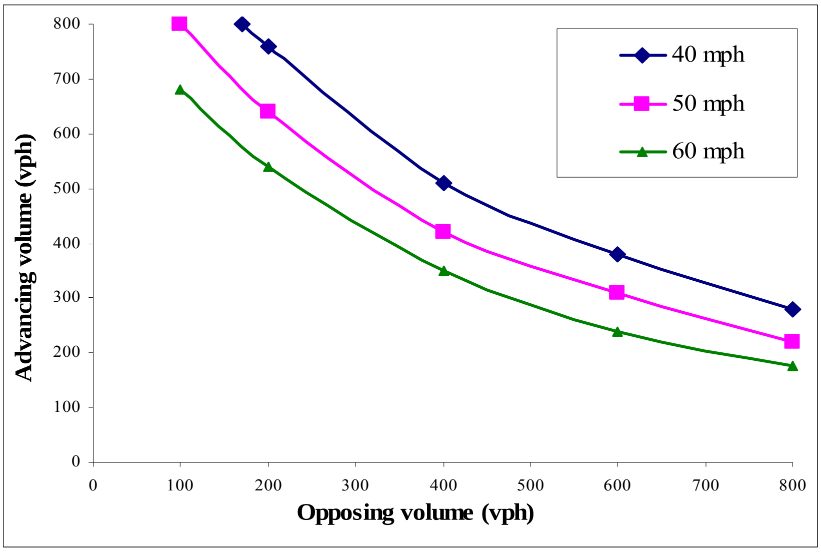


Figure 2.5: Left Turn Lane Warrants by VTRC

- Left-turn lanes are recommended at approaches to intersections for which the combination of through left, and opposing volumes exceeds the warrants shown in Figure 2.6.
- Left-turn lanes on stopped or secondary approaches should be provided based on analysis of the capacity and operations of the unsignalized intersection. Considerations include minimizing delays to right turning or through vehicles and total approach capacity.

The last point suggests that for developing left turn lane warrants, performance measures such as delays and number of stops on the subject link could also be used instead of probability as warrant criteria.

The warrants developed in NCHRP Report 279, however, were once again developed using Harmelink's method, which, firstly has two major shortcomings as pointed out Kikuchi and Chakroborty (1991) and secondly is based on the probability of through vehicles present behind the left turning vehicles which is quite subjective and somewhat questionable.

2.6 Other Studies

Many others have examined the operational effects of various means of serving left turns on main roads. For example, Oppenlander and Bianchi (1990) expanded Harmelink's warrant for additional operating speed and left turn lane percentages. As shown in the Figure 2.7, they added warrants for operating speeds of 30 mph and 70 mph and left turn percentage ranging from 0.5 % to 50% to the previous Harmelnk's warrants.

Basha (1992), Chakraborty *et al.* (1995) and Lertworawanich and Elefteriadou (2003) developed methods for estimating storage lengths needed for left turn bays at unsignalized intersections. Simpson and Matthias (2000) validated estimates of left turn delay computed using the 1997 Highway Capacity Manual (HCM) methodology against values observed in field studies, and found that the HCM method over-estimates delay when total approach volumes are high. Many studies have investigated alternative median treatments for multi-lane highways, including two-way-left-turn lanes (TWLTL), flush medians and raised medians, including McCoy *et al.* (1982), Ballard and McCoy (1983) and Venigalla *et al.* (1992). In all cases TWLTL reduced delay to through vehicles compared to undivided sections, but there was not always a difference in delay between sections with TWLTL and raised medians, except when traffic volumes or the number of driveways is very high.

2.7 Safety Implications

The installation of a left turn lane has operational as well as the safety impacts on the traffic flow. Studies discussed in the earlier section take into account only the operational impact of installing a left turn lane. Following are some of the studies which take in to account the safety impacts of installation of left turn lane. There are many studies which try to compare the safety benefits of each of the three median types (raised median, flush median and TWLTL), including McCoy and Malone (1989), Squires and Parsonson (1989), Fitzpatrick and Balke (1995), Margiotta and Chatterjee (1995), and Bonneson and McCoy (1997). These studies generally have found road sections with TWLTL and flush medians to have lower crash frequencies than undivided sections, and sections with raised medians to have fewer crashes than those with TWLTL or flush medians, as would be expected.

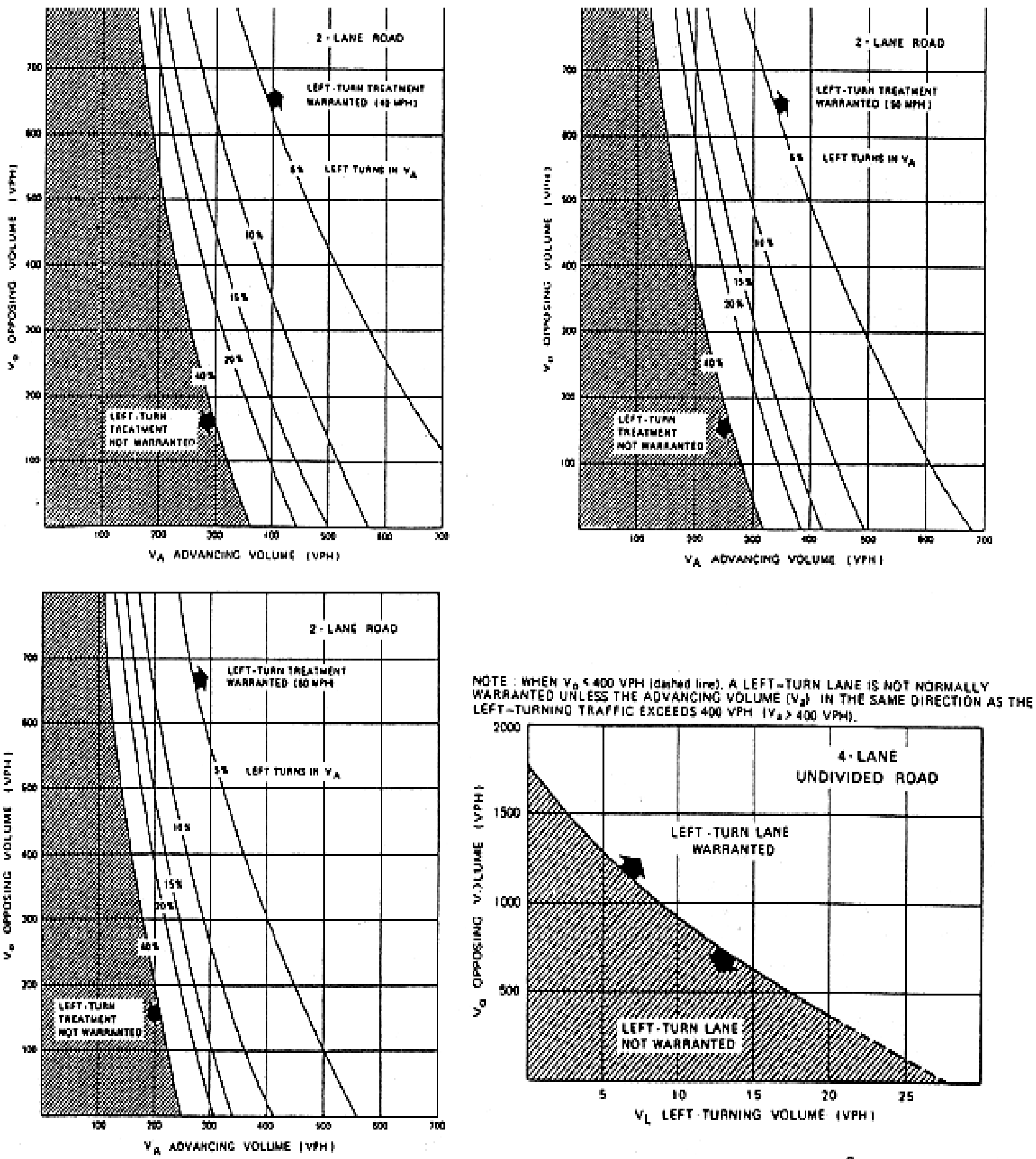


Figure 2.6: Warrants Developed in NCHRP Report 279

However, McCoy and Malone (1989) analyzed crash experience at signalized and unsignalized intersections on urban four-lane roadways and found that at intersections with left turn lanes, rear-end, sideswipe and left-turn crashes were reduced compared to intersections without left turn lanes, but right-angle crashes increased. Because right-angle crashes are generally more severe than rear-end or sideswipe crashes (thought not left-turn crashes), this complicates the notion that installing left turn lanes always improves safety.

More recent research by Rimiller *et al.* (2003) conducted a before-after study of the safety effect of adding left turn lanes at sixteen intersections in Connecticut. Adding left turn lanes was found to reduce crashes in some, but not all situations. For example, in areas with higher traffic volumes the rate of crashes between vehicles traveling in the same direction actually increased, and opposite direction turning crashes decreased significantly only at intersections with lower traffic volumes. In general, installing left turn lanes was less effective at reducing crashes under conditions associated with higher traffic intensity, such as at traffic signals, on four-lane roads and four-legged (rather than three-legged) intersections. When considering crash severity, installing left turn lanes reduced the average crash severity, suggesting that even if the number of crashes is not reduced or even increases, if the resulting crashes are less severe, the improvement may still be beneficial in terms of safety.

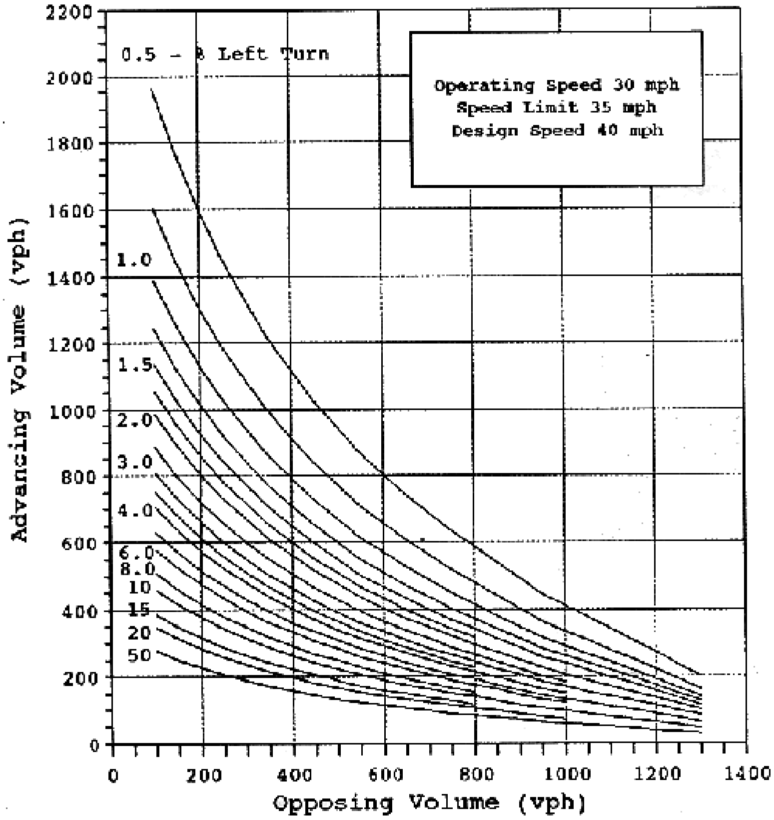


Figure 2.7: Warrants Developed by Openlander and Bianchi (1990) for 30 mph Operating Speed

The above research findings demonstrate that it is not as simple as it seems to choose whether or not to install an exclusive left turn lane at an unsignalized intersection or driveway. Installing a left turn lane might increase some types of crashes, but decrease more serious types of crashes, but only under certain road and area type conditions. Consequently, it would be extremely helpful for highway agencies to have a rational, defendable set of warrants for determining where it is – and is not –safe to install exclusive left turn lanes at unsignalized intersections and driveways.

2.8 Simulation Modeling

One of the most important analytical tools of traffic engineering is computer simulation, as it can be effectively used to evaluate or predict the impacts of various design alternatives on traffic conditions. Traffic simulation is often preferred over field experiments because; it's not as expensive as field experiment. The results can be obtained quickly as compared to the field experiment. In one simulation run of a traffic simulation model many measures of effectiveness can be obtained. The most important benefit of traffic simulation over field experimentation is that it avoids unnecessary disruption of live traffic. Lastly, sometimes field experiments require major physical or operational changes to the facility, which might not be favorable for live traffic conditions.

There are three basic types of simulation models which are currently used for the analysis of the traffic flow.

- **Microscopic simulation modeling.** Each individual vehicle is tracked. A vehicle's movement in the system is determined by the driver behavior, characteristics of the vehicles, and its interactions with network geometry, control devices and surrounding vehicles. Microscopic models are capable of modeling complex traffic networks with a great level of detail but on the other hand they require extensive data input as well as higher execution period. The core of a microscopic model is its car-following and lane-changing logic.
- **Macroscopic simulation modeling.** Macroscopic models use basic relationships in traffic engineering such as flow-density relationships to govern vehicle movement in the network. Individual vehicles are not tracked. Macroscopic models are not as detailed as microscopic models. Instead, they are capable of modeling larger networks and require less execution period.
- **Mesoscopic simulation modeling.** Mesoscopic models fall somewhere between the microscopic models and macroscopic models. Unlike, microscopic models they usually simulate the vehicles as

group or platoon. Unlike macroscopic models they can simulate lane-changing, merging and diverging behavior.

2.8.1 CORSIM

In selecting the microscopic simulation model to use, researchers considered the use of either the Corridor Simulation (CORSIM) model, developed by the Federal Highway Administration (FHWA) or the PARAMICS model, developed by Quadstone, Limited. After some initial investigation, the research team elected to use CORSIM. The primary reason in selecting CORSIM over PARAMICS was that CORSIM allowed for greater flexibility and accuracy in modeling and calibrating gap acceptance behavior and discharge headways at each intersection. CORSIM allows the user to specify the decile gap acceptance distribution. This is not possible with PARAMICS, where calibration primarily depends upon adjusting the value of the “target headway”, one of the parameters of the car following model that PARAMICS uses. Obtaining values for the “target headway” from field observations, however, is much more challenging than specifying the gap acceptance distribution.

CORSIM is a combination of two different models: an urban arterial simulation model which is called NETSIM; and a freeway simulation model called FRESIM. CORSIM is a part of the several traffic simulation models developed by FHWA. Combining the NETSIM and FRESIM models allows users to carry out system-level analysis of networks including freeways as well as urban arterials. CORSIM is a microscopic simulation model which applies a time step simulation process. CORSIM therefore tracks the position and movement of each vehicle in the network once each second. Likewise, control devices are updated once each second. CORSIM is a stochastic model, which means that random numbers are assigned to each of the vehicles depending upon the characteristics of the vehicle and driver behavior. The movement of vehicles is based on car following theory, vehicle performance, driver behavior and its interaction with the control devices and surrounding vehicles. With respect to freeways, CORSIM is capable of modeling up to five mainline lanes, up to three auxiliary lanes, and one to three lane ramps. The model can also measure the impacts of restricted use lanes, HOV, incidents, and ramp metering and can replicate the presence of surveillance detectors. The latest version of CORSIM available in the market right now is CORSIM 5.1 which was used for modeling purpose in this research project. CORSIM 5.1 was released in 2003.

2.8.2 SCRIPT TOOL

The TSIS (Traffic Software Integrated System’s) Script Tool comes with the standard TSIS package (which includes NETSIM and FRESIM). TSIS script tool uses Microsoft's Script Control to provide an application that enables researchers to generate and execute scripts within the TSIS environment. The Script Tool also provides access to several interfaces that allow the scripts to interact with the TSIS user interface, TShell, and other tools within the TSIS environment.

Scripting enables researchers to automate frequently performed tasks by writing standard Visual Basic scripts. Although scripting is a very powerful tool, it does require some computer programming experience and knowledge of the VBScript language. For example in this research project the researchers developed customized scripts in Visual Basics environment which enabled them to automate recurrent simulation modeling followed by the extraction of the required output for the subject link only from the huge output file. Thus, researchers found the script tool quite useful, which used effectively, is capable of saving a lot of time.

3 VOLUME WARRANTS: METHODOLOGY

3.1 General Approach

The primary objective of this part of the project is to build on previous attempts to develop left-turn lane warrants in an effort to provide a more refined decision support system (DSS), as well as to develop new left turn lane warrants which would assist in decisions regarding installing left-turn lanes at unsignalized intersections. Figure 3.1 summarizes the different steps that were followed in the research study's methodology. The basic idea behind the development of the DSS and new left turn lane warrants in the current study was to first use microscopic simulation to model, in great detail, several real-world unsignalized intersections with different geometric configurations (*i.e.* two-lane vs. multi-lane) and located in different area types (*i.e.* urban vs. suburban vs. rural areas).

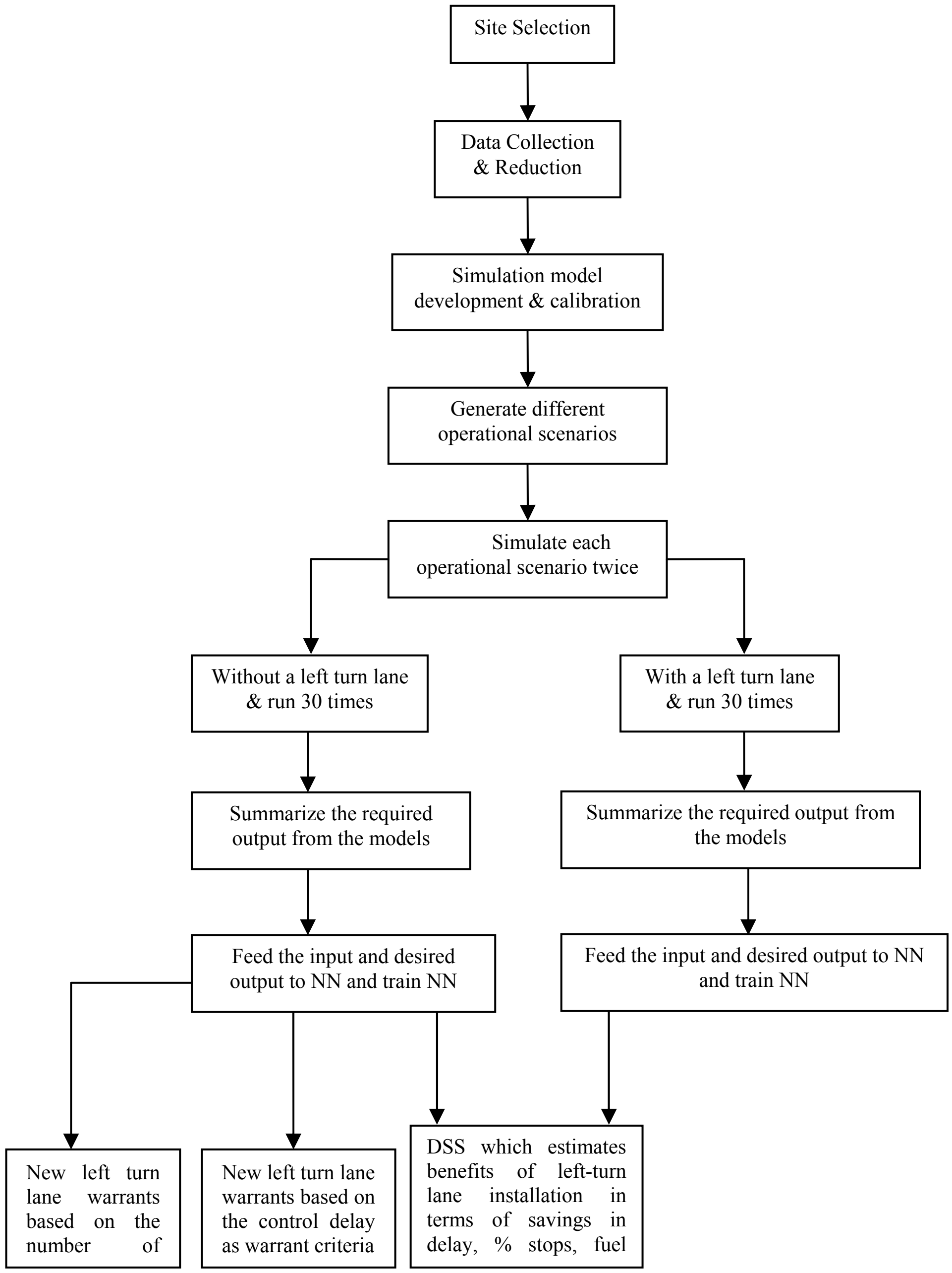


Figure 3.1: Steps in the Study Methodology

After building the models in the CORSIM, the next step was to carefully calibrate those models by comparing several of their output parameters (e.g. stopped delay and queue lengths) to detailed field observations to make sure the models are replicating real-world conditions. The measures of effectiveness (MOE's) used in the calibration were stopped delay, average queue length and maximum queue length on the subject link. The parameters adjusted to minimize the difference between the MOE's obtained from model and those observed in the field, were gap acceptance distribution, vehicle entry distribution and queue discharge headway.

Following this, a large number of different operational scenarios were simulated. These scenarios covered a wide range of possible values for the advancing, opposing, and left-turn volumes at unsignalized intersections, as well as a range of operating speeds. For each scenario, two cases were modeled, one time without an exclusive left-turn lane and the second time with a left-turn installed, thereby providing an estimate of the benefits of installing a left turn lane which helped in the development of DSS. The output from the different simulation model runs was then used to train a Neural Network (NN) (while it was also possible to use different tools such as regression analysis to generalize the results, it was found that the NN did a good job in that regard). Once trained, the NN can serve as a DSS for predicting the likely benefits of installing a left-turn lane for a given intersection. In estimating these benefits, the study considered a wide range of performance measures including savings in delay, reduction in the number of stops, improvement in the fuel consumption efficiency, as well as reduction in emissions to provide a comprehensive picture for the likely benefits of installing a left-turn lane. Furthermore, the output from the scenarios without an exclusive left turn lane was used to develop the new left turn lane warrants. Likewise, in the process of the development of new left turn lane warrants, the total delay (veh-sec/hour) and total number of stops on the subject link were used to warrant the left turn lane.

3.2 Site Selection

As Figure 3.1 shows, the first step in the methodology was to select a set of unsignalized intersections for the purpose of developing and calibrating the simulation models. In selecting those sites, the research team made sure that the selected intersections:

- covered a wide range of traffic volumes (left turning, opposing and advancing);
- included both multi-lane as well as two-lane roads;
- were located in urban, suburban and rural locations; and
- had a significant percentage of left-turn vehicles.

To do this, the research team first compiled a list of candidate intersections in Chittenden County, Vermont. Average annual daily traffic (AADT) and turning movement counts for those intersections were obtained from the Chittenden County Metropolitan Planning Organization (CCMPO) website and used to screen potential sites. The categorization of intersections into urban, suburban and rural was based on a general assessment of the area in which the intersections were located. Urban intersections included in the study were typically located in built-up areas, near the urban core of major cities in Chittenden County having a population of greater than 5,000. Rural intersections, on the other hand, were generally surrounded by farm land. Categorizing intersections as suburban, however, was more subjective. In general, intersections categorized as suburban were located in areas which were not as built up as their urban counterparts; at the same time, the surrounding development could not justify classifying the area as rural.

The research group then visited all of the potential sites to collect practical information regarding possible nearby parking and good locations to mount the video cameras which were used for data collection. Sites with a small shoulder width were preferred over sites with larger shoulder widths because, as was observed in the field, where a wide shoulder was available, several through vehicles tend to use that shoulder to overtake the left turning vehicles waiting for a suitable gap in opposing traffic. This obviously would cause problems in collecting queuing and stopped delay information from the field observations. Eventually, the study ended up with a final list of 8 intersections where data were collected. Table 3.1 gives some basic information about the selected intersections' locations, geometric configuration, the timing of the peak hour and the volume combinations. Figure 3.2 shows the location of those intersections.

3.3 Data Collection and Reduction

The eight intersections were videotaped using two SONY DV cameras with an accuracy of 1/10th of a second. This allowed for measuring gaps and gap acceptance behavior to an accuracy of 1/10th of a second. All the video taping at different intersections was conducted for a period of one hour during the peak period. Once, the video taping was completed, the tapes were watched, and the following data were extracted:

Table 3.1: Data Describing Selected Intersections

No	Intersection Description	Category	Speed Limit (mph)	Peak* hour	Peak hourly volume*		
					Advancing (vph)	Opposing (vph)	Left turning (vph)
1	Colchester Av and Votey Parking Driveway	4-lane Urban	30	8 to 9 AM	933	828	21
2	Cheese Factory and Hinesburg	2-lane Rural	45	8 to 9 AM	384	292	113
3	Spear street and Barstow Road	2-lane Suburban	35	4:30 to 5:30 PM	627	486	146
4	Williston Road and Commerce Street	2-lane Urban	40	8 to 9 AM	623	449	73
5	Williston Road and Talcott (eastern intersection)	2-lane Rural	40	4:30 to 5:30 PM	606	990	234
6	Williston Road and Old Stage	2-lane Urban	40	4:30 to 5:30 PM	240	247	21
7	Vt. 2A and Creamery Road	2-lane Suburban	40	4:30 to 5:30 PM	664	253	263
8	Williston Road and McDonald's Driveway	4-lane Urban	35	4 to 5 PM	1942	1308	22

*Peak hourly volumes and peak hour were obtained from the Chittenden County Metropolitan Planning Organization (CCMPO) website

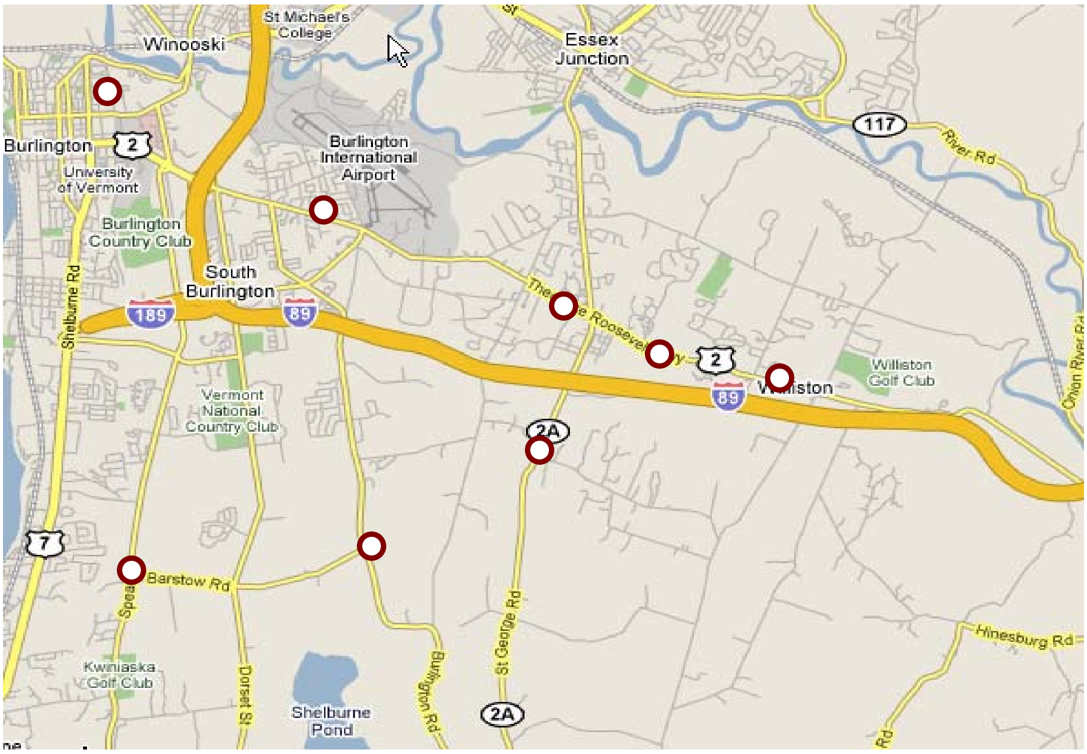


Figure 3.2: Location of Selected Intersections

- **Advancing, opposing and left turning volumes.** The videos of the intersections were observed several times to get the advancing, opposing and left turning vehicles were counted manually to get the respective volumes.
- **Basic geometric information (number of lanes, lane channelization and operating speed).** Basic geometric information was obtained during the site visits of the research team to the different intersections and also the recorded video tapes helped to collect these data.
- **The discharge headway from a queue.** The default value of the queue discharge headway used in CORSIM is 1.8 sec/vehicle (which is 2000 veh/hour/ln).
- **Average and maximum queue length during the one hour of observation.** Average and maximum queue lengths were measured from the videotapes using the procedure explained in the Highway Capacity Manual (HCM 2000). Every 30 seconds, the videotape was stopped, and the queue length on the subject link (*i.e.* the number of vehicles in the queue) was recorded (V_i). If no vehicles were in the queue at a given time snap shot, a zero was recorded for the queue length. From this, the average queue and maximum queue were computed as follows:

Average queue length = $\frac{\sum_{i=1}^{n} V_i}{n}$ where, n is the total number of intervals.

Maximum queue length = Maximum (V_i)

- **Stopped delay at the subject link.** Average stopped delay was measured from the videotapes again using the same procedure explained in the Highway Capacity Manual (HCM 2000). At each 30 seconds interval, the videotape was stopped, and the queue length on the subject link (*i.e.* the number of vehicles in the queue) was recorded (V_i). If no vehicles were in the queue at a given time snap shot, a zero was recorded for the queue length. From this, the average stopped delay was calculated using the following formula.

Stopped Delay = $\frac{\sum_{i=1}^{n} V_i \times l}{V_T}$ (sec/vehicle),

Where,
l is the time interval (30 sec in our case) and
V_T is the Total advancing volume

- **Posted Speed Limit.** Posted speed limit was also collected during the site visit of the research team to the different intersections.

3.3.1 Gap acceptance distribution data

As shown in Figure 3.3, suppose a left turning vehicle A arrives at the stop line at time T_1 and vehicle B arrives at T_2 then the available gap for vehicle to complete the left turn maneuver is given by the following equation $\Delta t = T_2 - T_1$ sec. After this, the time difference between vehicle B and vehicle C measured in seconds is called the time headway (sec).

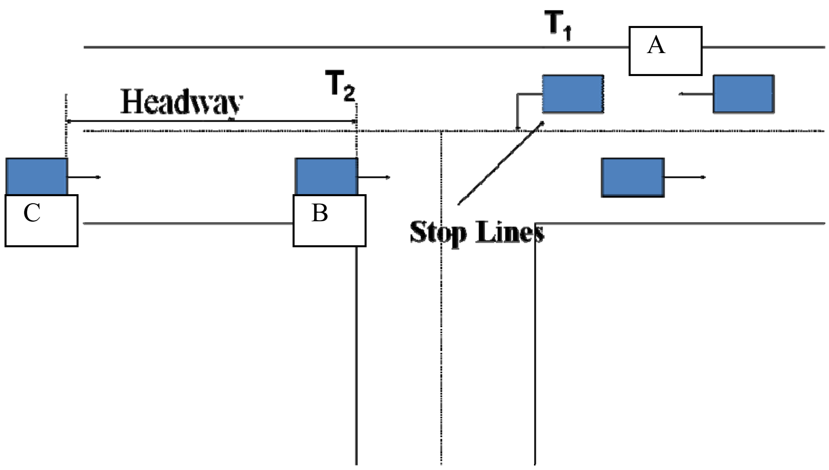


Figure 3.3: Typical Layout of T-intersections

CORSIM allows the user to specify a decile distribution for the accepted gaps in the oncoming traffic stream facing permissive left-turning vehicles (the decile distribution would specify the values of the minimum gap that will be accepted by 10% of the drivers' population, 20%, 30%... and so on). This allows for greater accuracy in modeling local gap acceptance behavior at a given intersection or region. To calibrate CORSIM, therefore, it was necessary to obtain the gap acceptance distribution for each of the 8 selected intersections from the recorded videotapes. The following procedure was used:

- The lengths of all the time gaps were measured from the video, and were labeled as either an "accepted" or a "rejected" gap according to observed drivers' behavior.
- Acceptable and rejected gaps were then arranged in descending order.
- The largest or longest rejected gap was noted, and accepted gaps that were larger than the longest rejected gap were excluded from further analysis. The reason behind this was the assumption that all the time gaps longer than the largest rejected gap will be accepted by all of the advancing vehicles.
- With the accepted gaps that are equal to or less than the largest rejected gap arranged in descending order, the values corresponding to the different 10ths percentiles were noted (*i.e.* values that were accepted by 10% of the population, by 20% and so on). Table 3.2 below shows an example of the gap acceptance distribution derived for intersection number 4 (Williston Road and Commerce Street). According to the observed distribution, only 10% of the drivers would be willing to accept gaps as short as 3.6 seconds, 20% would accept gaps as short as 3.9 seconds, while 100% of the drivers would be willing to accept gaps that are at least 6.8 seconds long.

Table 3.2: Gap Acceptance Distribution at Williston Road and Commerce Street

Gap (Seconds)	6.8	6.5	6	5.8	5.5	5.2	4.8	4.3	3.9	3.6
Percentile	100	90	80	70	60	50	40	30	20	10

Figure 3.4 compares the gap acceptance cumulative distributions for the 8 intersections observed in this study. It is interesting to note here that each area type (*i.e.* urban, suburban or rural) appears to have a slightly different gap acceptance distribution. It shows that the gap acceptance distributions for the two rural locations are almost identical, and seem to have higher minimum gap acceptance values compared to suburban and urban distributions. Gap acceptance distributions for the suburban locations also appear to be very close to one another, with values lower than rural locations and slightly higher than urban locations. Finally, the four urban locations also appear to have distributions that are quite similar to one another. This basically shows that the gap acceptance distribution differs based on the area type and the earlier warrants are just for two-lane categories and they do not differentiate between urban and rural road categories.

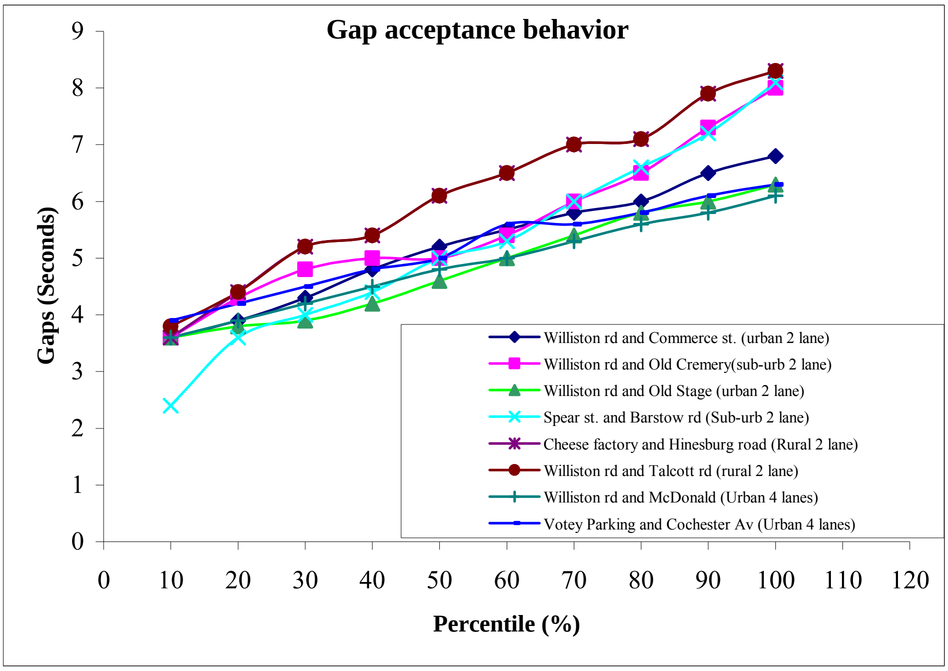


Figure 3.4: Gap Acceptance Distributions for the Different Sites

3.4 Simulation Model Development and Calibration

With the simulation model selected, the study then proceeded to develop and calibrate the simulation models. Aerial photographs of the intersections were obtained and imported into CORSIM for each intersection. The different links and nodes were overlaid on the aerial photographs. Thirty different runs with different random seed numbers were performed using the CORSIM Script tool, and the results, only for the subject link, from these runs were averaged to get a more accurate picture of the CORSIM's reported performance measures. Figure 3.5 shows a model of the intersection of Williston Road and Commerce Street built in the CORSIM.

Figure 3.5: A Simulation Model of the Intersection of Williston Road and Commerce Street

For calibration, the gap acceptance distribution, the discharge headway and the vehicle entry distribution for each intersection were adjusted based on the values obtained from the videotapes. For each intersection, the model's output (*i.e.* the average of 30 runs) was compared against several field measurements. Specifically, the study compared (1) average stopped delay; (2) average queue lengths; and (3) maximum queue as measured from the videotapes to the values estimated by CORSIM. Excellent calibration results were obtained as evidenced by Figure 3.6, 3.7 and 3.8 compare the average stopped delay and maximum queue length and average queue lengths on the subject link (as determined from the 30 CORSIM runs) to the values measured from the field for each of the eight intersections respectively. Also shown is the range of values obtained from the 30 CORSIM runs. As can be seen, the model and field results are almost identical for all eight sites.

For Figure 3.8, which shows the calibration of the different simulation models based on the average queue length, it should be noted that the field observed values are a little bit different from the modeled average queue values. This difference could be attributed to the default property of CORSIM 5.1 to round up or down the queue length to either 1.0 or 0 vehicles.

3.5 Development of Decision Support System

3.5.1 Generation of the Different Operational Scenarios

Once all 8 models were calibrated, the next step was to choose a representative intersection for each intersection category: 2-lane urban; 2-lane rural; and 4-lane urban. To keep the representative models as general as possible, intersections with grade, high queue discharge values, and sharp turns were ruled out. The representative intersections for each of the category are as shown in Table 3.3.

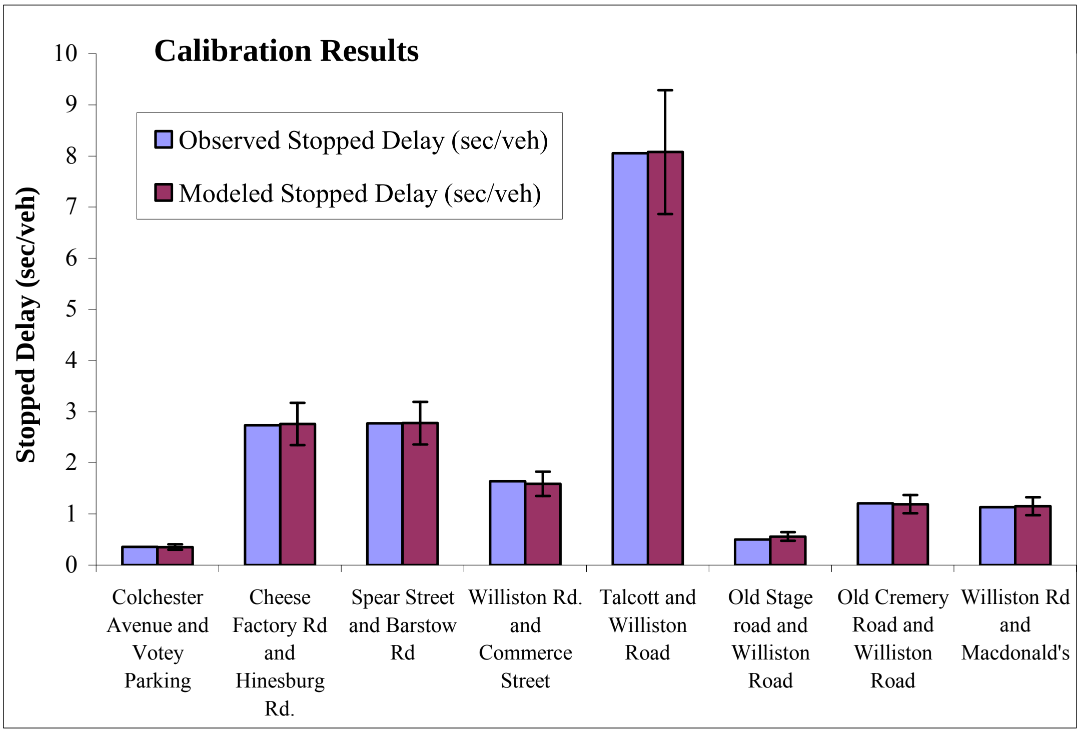


Figure 3.6: Results of the Calibration Procedure (Stopped Delay)

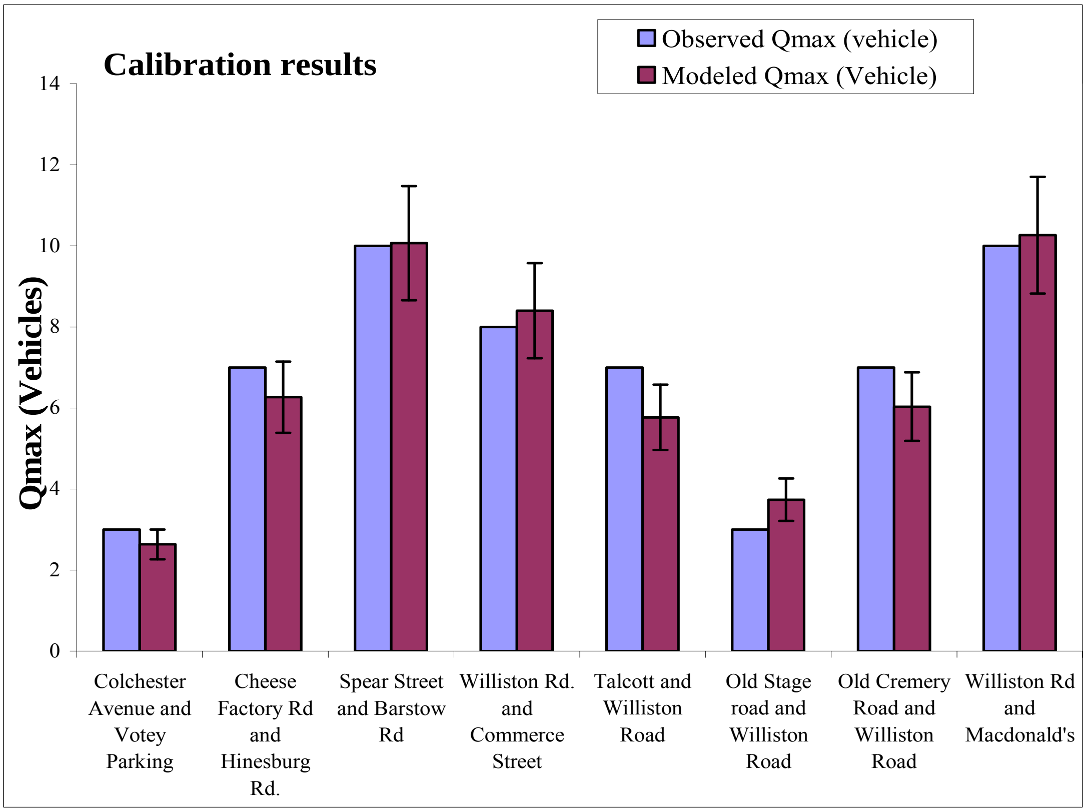


Figure 3.7: Results of the Calibration Procedure (Qmax)

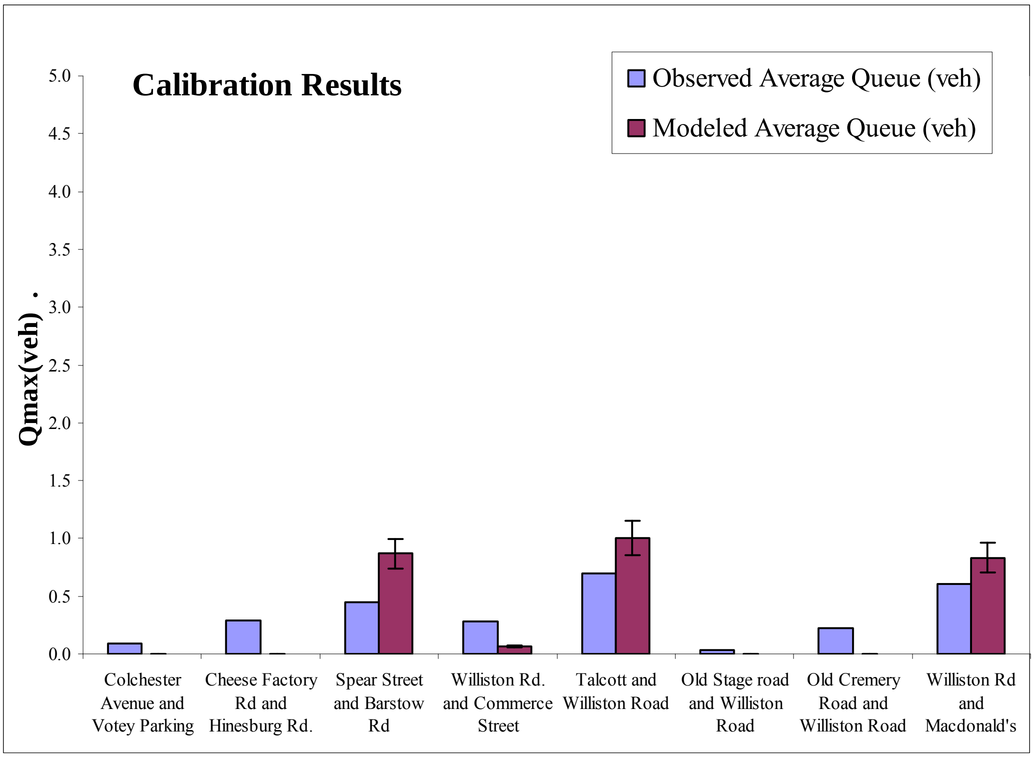


Figure 3.8: Results of the Calibration Procedure (Qavg)

Table 3.3: Representative intersections for each of the category

1	Cheese Factory and Hinesburg Road	2-lane Rural
2	Williston Rd. and Commerce Street	2-lane Urban
3	Williston Rd and McDonald's	4-lane Urban

To develop the Decision Support System as well as the new left turn lane warrants, it was necessary to look at combinations of different opposing, advancing and left turning volumes as well as different operating speeds. The opposing and advancing volumes were varied between 100 and 800 vehicles per hour per lane in the increments of 100 per hour per lane. For left turning percentages, values of 5%, 10%, 20% and 30% and for operating speeds values of 30 mph, 40 mph and 50 mph were used. Therefore a total of 768 operation scenarios for each of the urban two-lane and rural two-lane category were used.

Similarly, for the case of urban four-lane roads, the opposing and advancing volumes were varied between 200 and 1600 vehicles per hour per lane in increments of 200 per hour per lane. For left turning percentages, values of 5%, 10%, 20%, and 30% and for operating speeds values of 30 mph, 40 mph and 50 mph were used.

As a result, there were in total 768 operational scenarios for each road category and 2304 operational scenarios for all of the three road categories. So, to quantify the benefits of installation of a left turn lane it was necessary to simulate 2304 operational scenarios twice, once without a left turn lane and then with a left turn lane. By considering the scope of the project, this task was computationally highly time consuming as each operational scenario needed to be run 30 times and the required output had to be summarized from the 30 runs and averaged to account for the stochasticity of the simulation. So, looking at the scope of the project, the research team started looking for a different option which could reduce computational work. After careful consideration, the research team decided to investigate the use of Neural Networks.

For each intersection category, to reduce the computational work load, it was decided to develop a set of 150 random operational scenarios for each category, each scenario representing a certain combination of advancing volume, opposing volume, left-turn percentage and speed. In generating these scenarios, advancing and opposing volumes were randomly varied between 100 and 800 vehicles/hr/lane, the left-turn percentage was varied between 3% and 30%, and speed was varied between 40 and 60 mph. For each scenario, two cases were simulated; once without a left-turn lane and another with a left-turn lane. As before, each case was run 30 times, each time with a different random seed number and the results were averaged over the 30 runs. The output from all these runs was a dataset for each operational scenario, which gave the estimated values for the following performance measures: control delay (sec/veh), percent stops, fuel consumption (mpg), Carbon monoxide (CO), Nitrogen oxide (NO) and Hydrocarbon (HC) emissions (gram/mile) for two cases, without a left-turn lane and with a left-turn lane. The developed dataset was then used to train the NN, which constitutes the heart of the DSS, as explained below.

3.5.2 Neural Network Development, Training and Testing

Neural Networks (NN's) are biologically-inspired systems consisting of massively connected networks of computational "neurons", organized in layers. By adjusting the link weights in the network, NN's can be "trained" to approximate virtually any nonlinear function to a required degree of accuracy. NNs typically learn by providing the network with a set of input and output exemplars (Principe, J.C., N.W. Euliano, and W.C. Lefebvre 2000). A learning algorithm (such as back propagation) is then used to adjust the weights of the network so that the network would give the desired output, in a type of learning commonly called *supervised* learning. The interested reader is referred to Transportation Research Circular E-C113, Artificial Intelligence in Transportation: Information for Application for more details. The Circular is available online at: http://onlinepubs.trb.org/onlinepubs/circulars/ec113.pdf.

In this study, the research team decided to use a NN to generalize the results obtained from the simulation models and to serve as a DSS for predicting the likely benefits of installing left-turn lanes at unsignalized intersections. Over the years, several NN types and architectures have been developed. The most important of these for transportation is the Multi-layer Perceptron (MLP) Neural Network. The MLP typically consists of three layers: the input layer, the hidden layer(s), and the output layer. The type of connections in the MLP is of the feed forward type, where all possible connections between the different neurons are made. The MLP was NN architecture used in this study. Figure 16 shows the structure of NN's used in this research study.

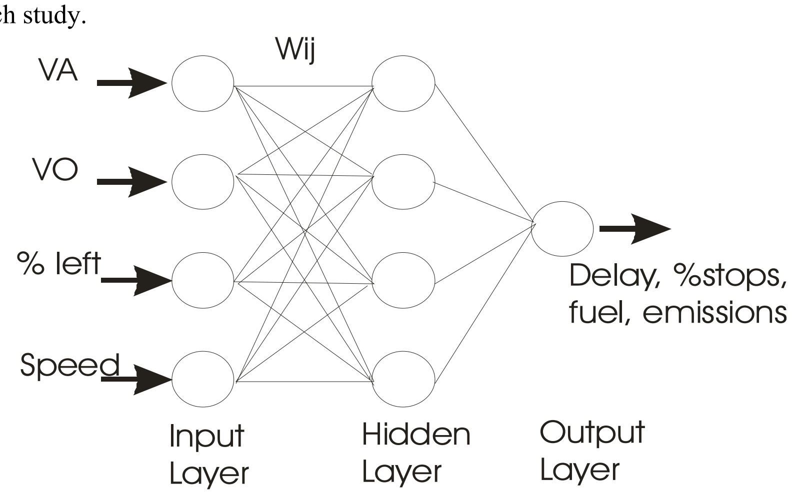


Figure 3.9: The Neural Network Used in the Study

As shown in Figure 3.9, the NN used has one hidden layer consisting of four neurons. As input, the NN will receive four input variables: (1) advancing volume (V_A); opposing volume (V_O); (3) percent left turns; and (4) speed. The output will be: (1) the stopped delay (sec/veh); (2) percent stops (%); (3) fuel consumption (miles/gallon); and (4) emissions for CO, NO and HC (gram/mile). A separate NN was developed for each intersection category (e.g. rural, suburban, urban multi- and two- lanes). In addition, one NN was developed for the "without left-turn lane" case, and one for the "with left-turn lane" case. Differences between the outputs of these two NN's can therefore be used to evaluate the likely benefits of installing left turn lanes based on delay savings, reductions in percent stops, increases in fuel efficiency, and reductions in emissions levels.

Using the dataset of 150 operational scenarios, the NN's were trained using the back propagation algorithm. Ten percent of the data were kept aside for testing purposes (*i.e.* were not used in training the networks). The training continued for 2000 epochs (training cycles). The research team came up with this number after several trials and errors. Following the training, the testing set was presented to the NN's, and the networks' output was compared to the values in the test set. The results are shown in Figures 3.10 and 3.11, indicating that the NN was able to predict the different performance measures produced by the simulation models with great accuracy, as indicated by the high R^2 values of the fitted lines.

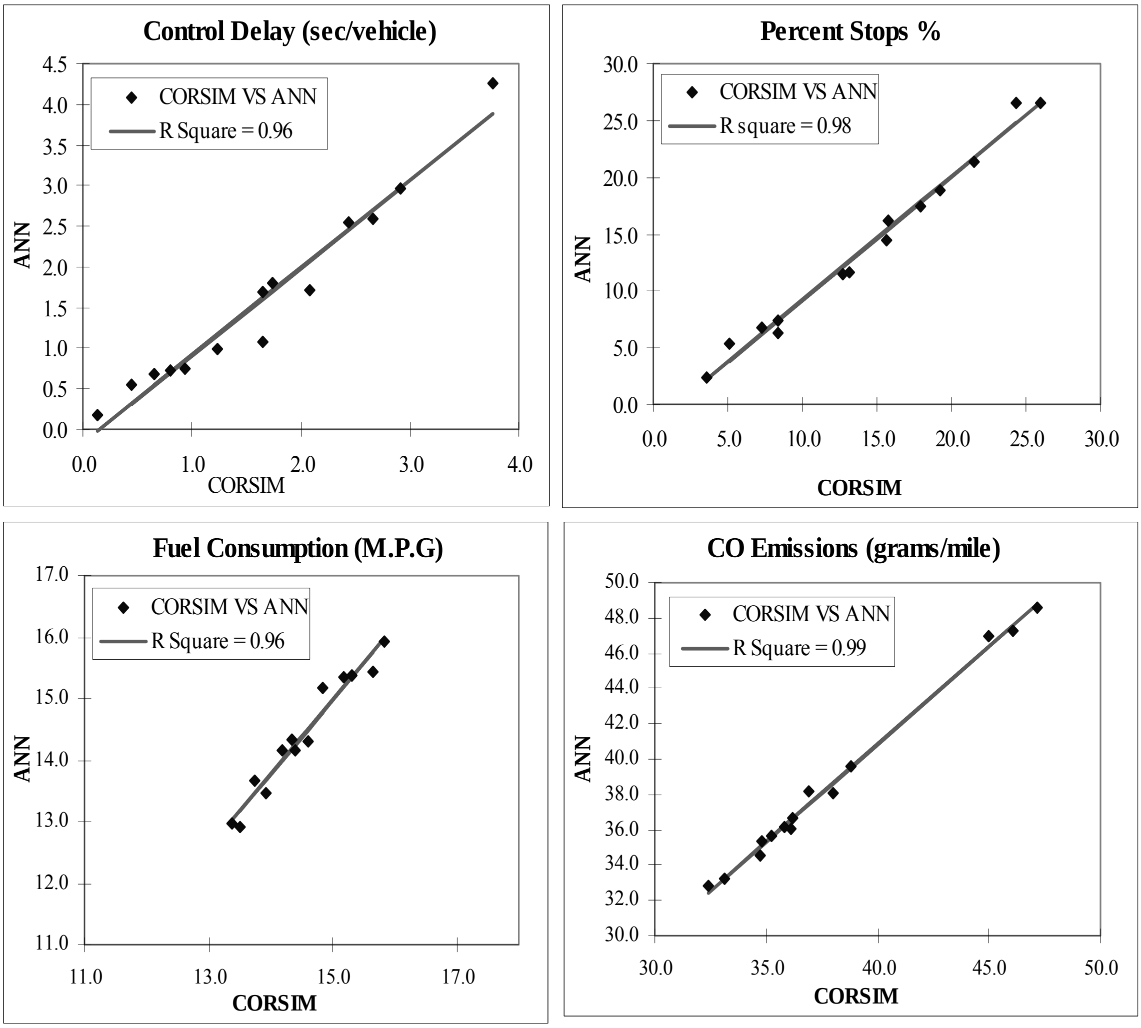


Figure 3.10: Result of Neural Network Testing for Urban Two-lane Without a Left Turn Lane Category

3.6 Applications and Discussion

The NNs are designed to serve as a DSS to aid traffic engineers in assessing the likely benefits of installing a left-turn lane at an unsignalized intersection and in deciding when such installation is warranted. So, for a given road category, in order to use this DSS traffic engineers would need to input the different opposing, advancing left turning volumes and operating speed, once into the NNs trained for without a left turn lane scenario and once for with a left turn scenario. Then by comparing the ouput from the two different NNs, they can get the benefits of installing a left turn lane in terms of reduction in delay, reduction in the total number of stops and increase in the fuel efficiency. To illustrate their use, the NNs were used, for a case study, to determine the likely benefits of installing a left-turn lane for a combination of advancing and opposing volumes ranging from 100 to 800 veh/hr. For this case study, the left turn percentage was fixed at 20% and the speed at 40 mph.

The results are shown in Figure 3.12 which gives the benefits in terms of savings in: (1) average control delay per vehicle (sec/veh); (2) total delay savings in veh.sec/hr; (3) reductions in percent stops; and (4) reductions in fuel consumption (miles/gallon). As shown in Figure 19, the benefits of installation of a left turn lane increase as the opposing and advancing volumes increase. Using the NNs, an analyst therefore can easily quantify the impacts of a proposed new development as well as estimate the benefits of installing a left-turn lane for that situation. The NNs can also be used to establish warrants for left-turn lane installations similar to those proposed by Harmelink (1967) by establishing thresholds on control delay, percent stops or savings in fuel consumption. Plots similar to those shown in Figure 19 can be a great aid in this regard.

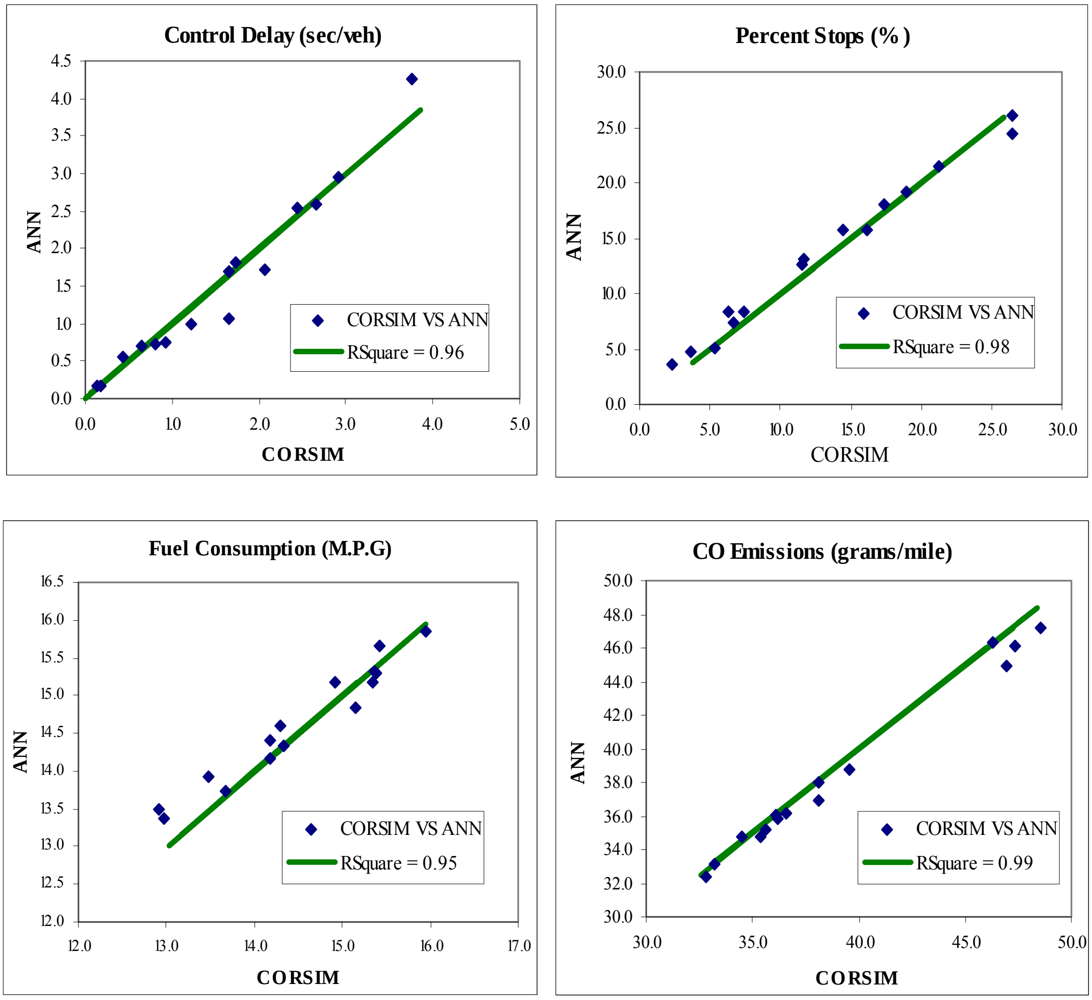


Figure 3.11: Result of Neural Network Testing for Urban two-lane with a left turn lane category.

It should be noted that for emissions, the research team found out that there was not a significant reduction in emissions levels resulting from the installation of left turn lanes. Gives that these results are based on the CORSIM simulation model, two explanations are possible. The first explanation is that the emission model in CORSIM is not that sensitive to changes in operational conditions (*i.e.* speed, stops, etc.). The alternative explanation is that it is not possible to justify the installation of left-turn lanes on the basis of reductions in emissions levels. This point deserves further investigation.

The benefits of installation of left turn lane at different road categories, for different operating speeds and for different left turning percentage are presented in the Appendix B (Figure B.1 to B.27).

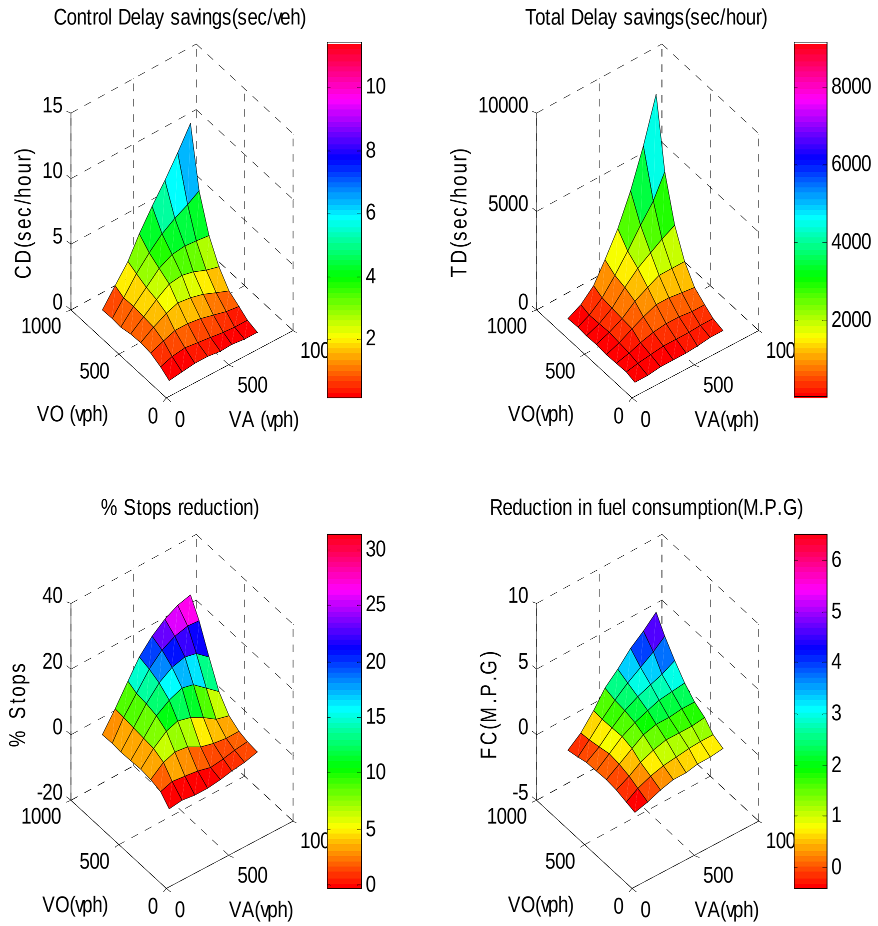


Figure 3.12: Left Turn Installation Benefits

4 VOLUME WARRANTS: DEVELOPMENT

For the development of left turn lane warrants, the output from the NN's trained for the without left turn lane scenarios were used. As discussed in the last chapter, the researchers found that the emissions model in CORSIM is not that sensitive to changes in operational conditions (*i.e.* speed, stops, etc). Given this, it was decided that both fuel consumption and emissions warrants could not be developed at this time. Instead, the warrant criteria, used for the justification of a left turn lane, were the total delay (sec/hour) and the number of stops per hour on the subject link. The set of warrants developed in this study differ based on the road category. For each road category, two sets of warrants were developed: 1) one developed using the control delay as warrant criteria, and 2) the second one using the number of stops per hour as warrant criteria.

In developing the warrants for the left turn lane based on control delay and the number of stops per hour, the first step was to set up the thresholds for both of the warrant criteria. While setting up the thresholds for the control delay and number of stops per hour, the following points were considered. First, it was necessary to look at the rate of change in the delay and number of stops with respect to the opposing, advancing, left turning volumes and operating speed. For that, total delay and the total number of stops on the subject link were plotted against the various combinations of advancing, opposing, left turning volumes and different operating speed.

The thresholds selected were kept constant regardless of the volumes and category (e.g. urban two-lane and rural two-lane categories had same thresholds), but varied with the operating speeds. For example, volume combinations for 30 mph speed are higher than the volume combinations for 40 mph speed as the thresholds are higher for the former. This same logic was followed by Harmelink (1967), Kikuchi and Chakraborty (1991) and Lakkundi *et al.*, (2004). The thresholds were selected such that the warrants developed would be somewhat comparable with the other warrants presented by Harmelink (1967), Kikuchi and Chakraborty (1991) and Lakkundi *et al.*, (2004).

As shown in Figures 4.1 and 4.2, the thresholds were set just below the point at which the curves for delay and the number of stops rise sharply for the relatively high opposing volumes. These figures are for the case of an operating speed of 50 mph and a left turning percentage of 30%. As shown, if the delay is above the specified threshold values for a certain combination of opposing, advancing and left turning volume, a left turn lane is warranted.

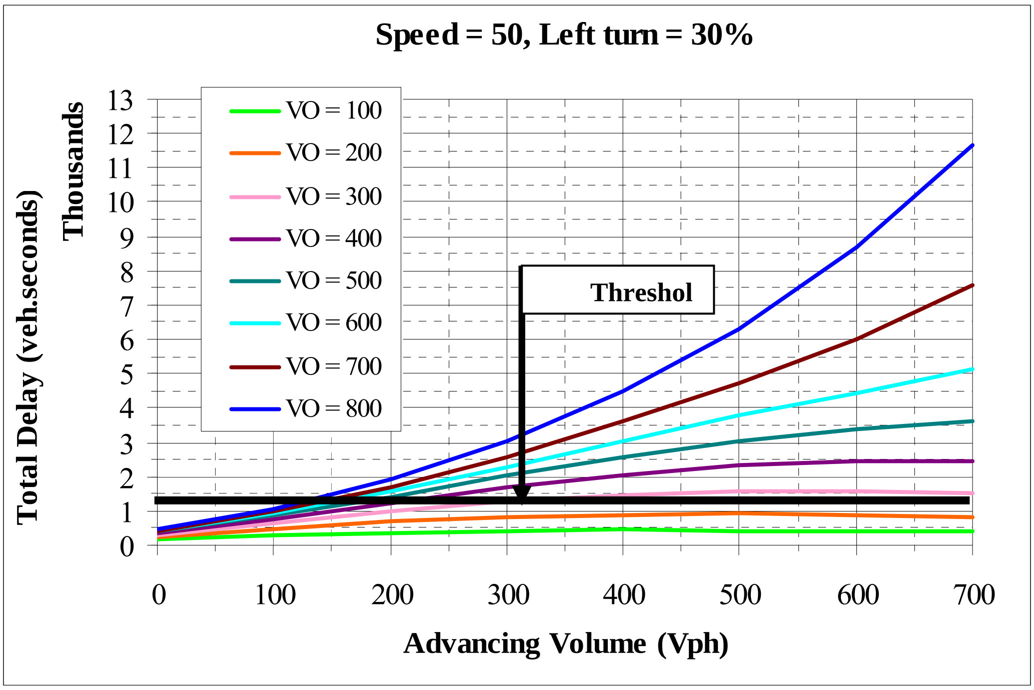


Figure 4.1: Total Delay Plotted for Urban Two-lane Category

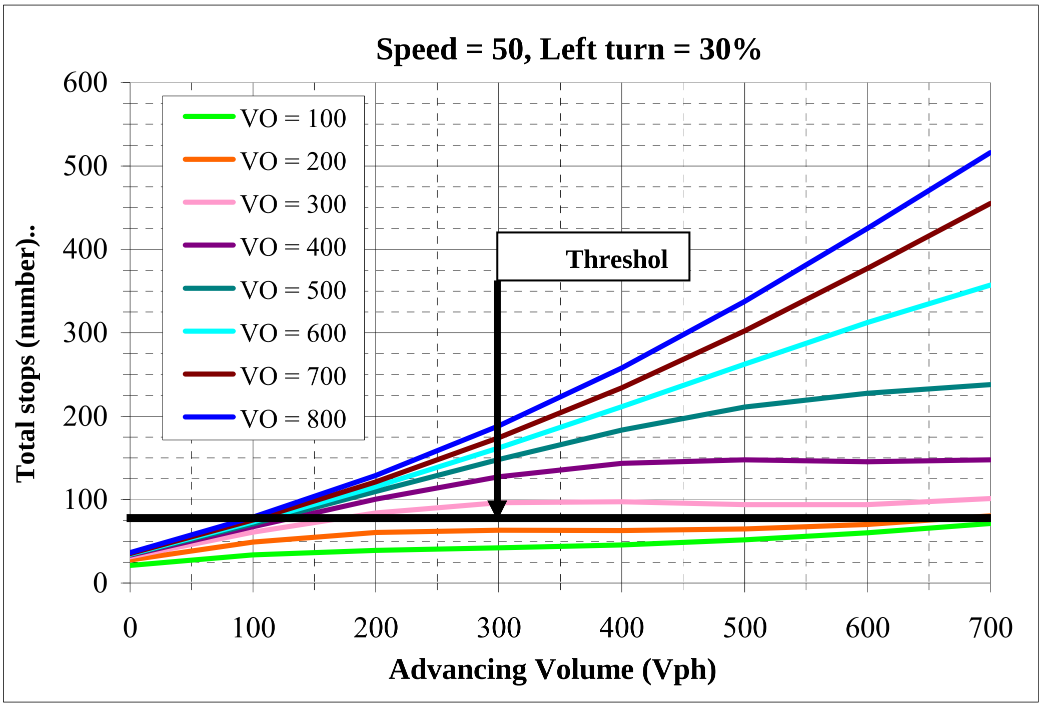


Figure 4.2: Total Number of Stops Plotted for Urban Two-lane Category

4.1 Urban Two-Lane Roads

4.1.1 Total Delay

Thresholds selected for the urban two-lane category for developing the warrants based on the total delay (sec/hour) criterion, were as shown in the Table 4.1.

Table 4.1: Thresholds Selected for the Urban Two-lane Category

Operating Speed	Thresholds
(mph)	(vehicle. seconds/hour)
50	1000
40	2000
30	4000

Using the thresholds values as shown in Table 4.1, warrants were developed for urban two-lane category based on total delay (sec/hour) s shown in Figures 4.3, 4.4 and 4.5 for operating speeds of 30 mph, 40 mph and 50 mph respectively.

Figures 4.6 and 4.7 compare the warrants developed for the urban two-lane category using control delay as warrant criteria to: (1) the Harmelink warrants; (2) the modified Harmelink's warrants developed by Kikuchi and Chakraborty; and (3) VTRC's warrants, which were all developed using the probability of one or more through vehicles behind the left turning vehicle as a warrant criteria. As shown in Figure 4.6 and 4.7, the warrants developed in this study seem to match well with the other warrants when the opposing volume is higher than 400 vph. For lower volumes, the current's study warrants seem to allow for higher volumes compared to the other warrants.

It should be noted that this is consistent with the Kikuchi and Chakraborty (1991) study, in which the researchers report that the warrants based on delay as a warrant criterion tend to yield higher volume threholds compared to those based on the probability of vehicles stopping behind the left-turning vehicle, especially for low volumes. Basically, the warrants developed in this study are based on the operational performance measures and the warrants developed earlier are based on conflict avoidance. Fitzpatrick, Brewer and Parham (2003) also found that the methods based on delay or other operational performance measures typically do not recommend a left-turn lane at lower left or through volumes when compared to methods based on conflict avoidance or safety.

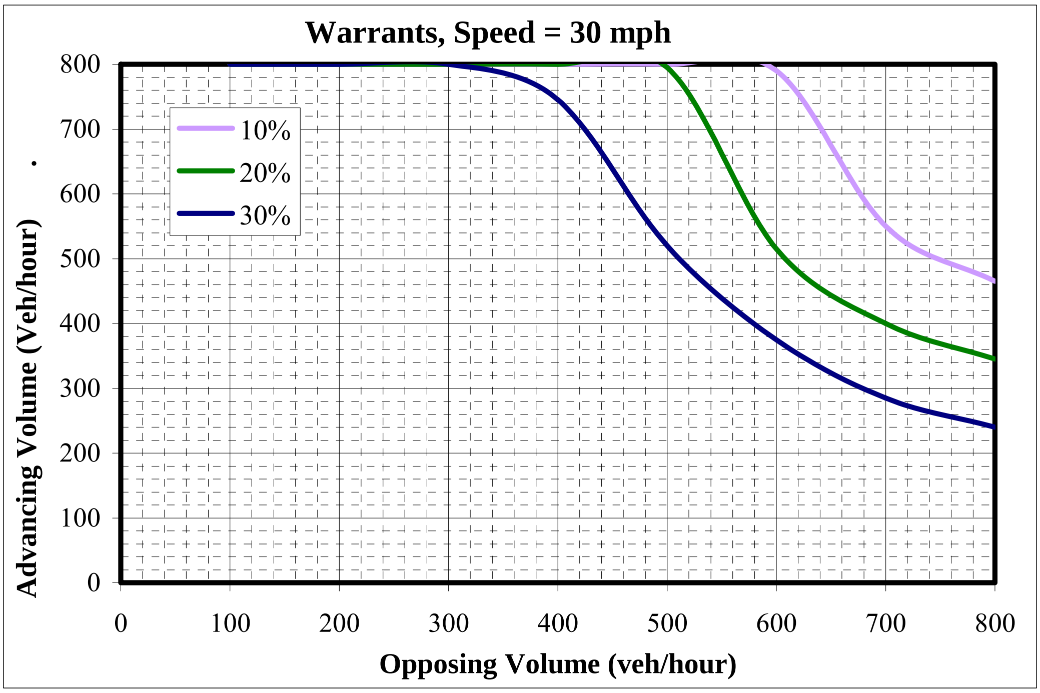


Figure 4.3: Warrants for urban two-lane category for operating speed 30 mph

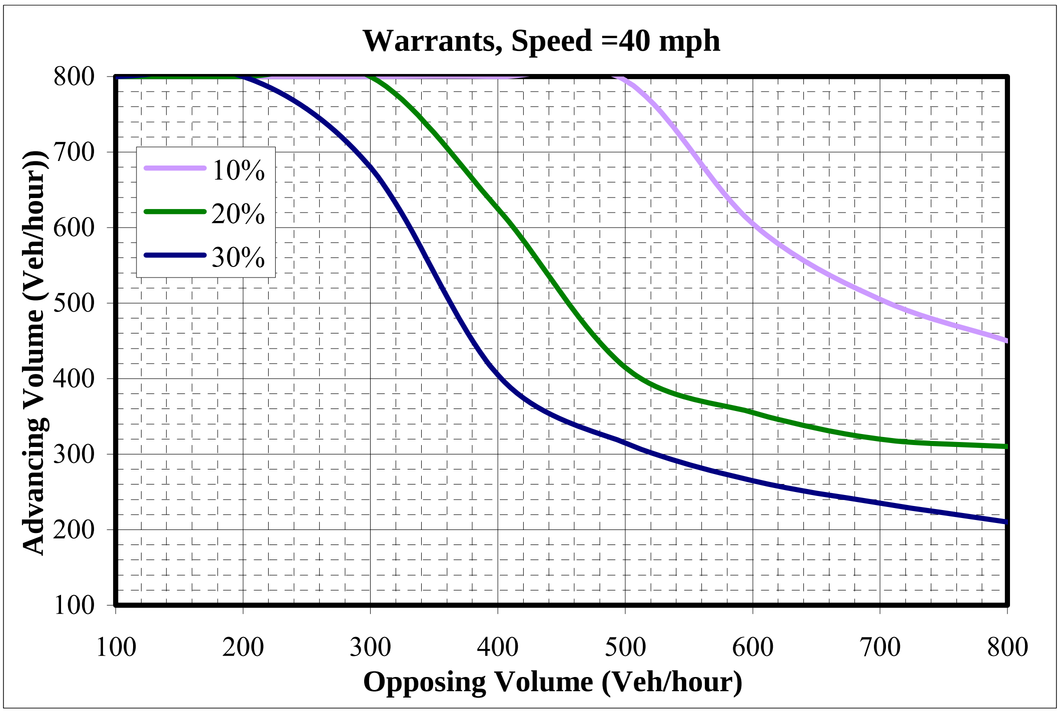


Figure 4.4: Warrants for urban two-lane category for operating speed 40 mph

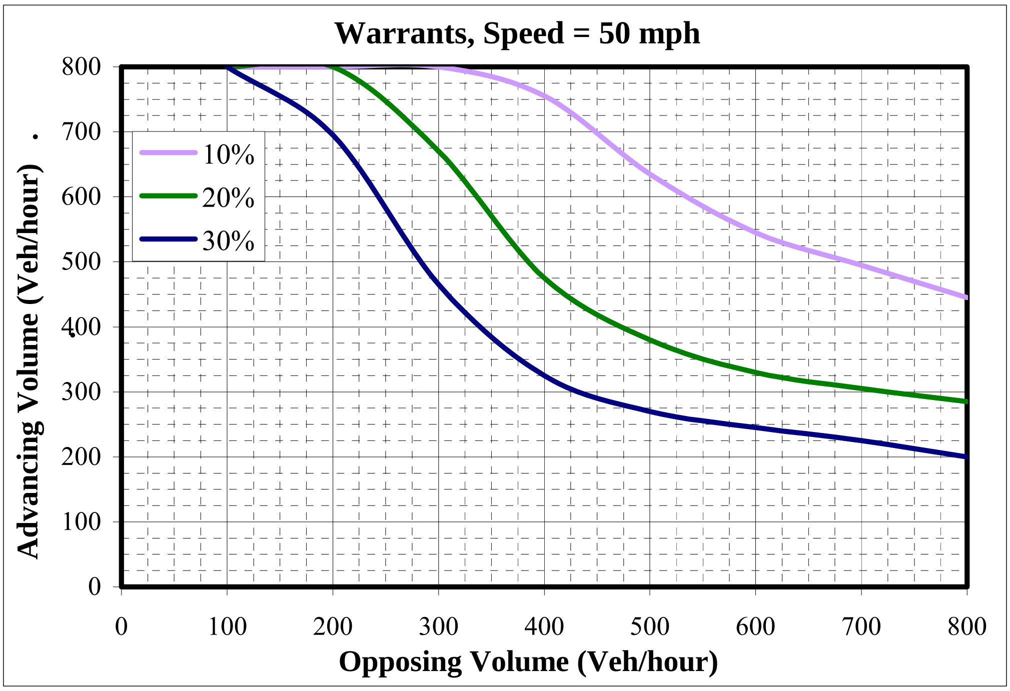


Figure 4.5: Warrants for urban two-lane category for operating speed 50 mph

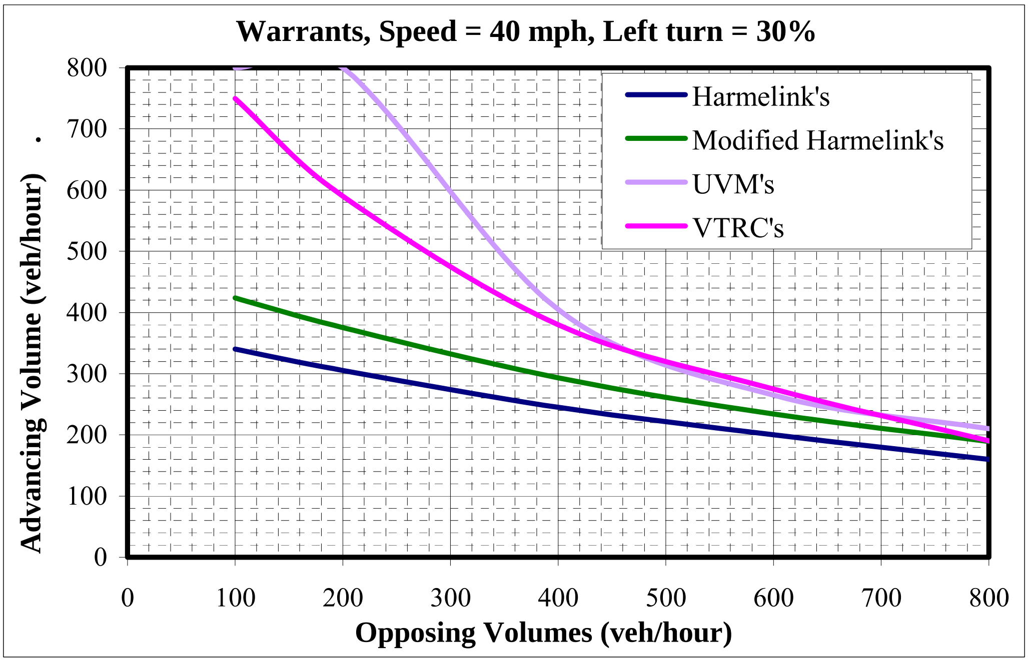


Figure 4.6: Comparison of the new left turn lane warrants with different warrants

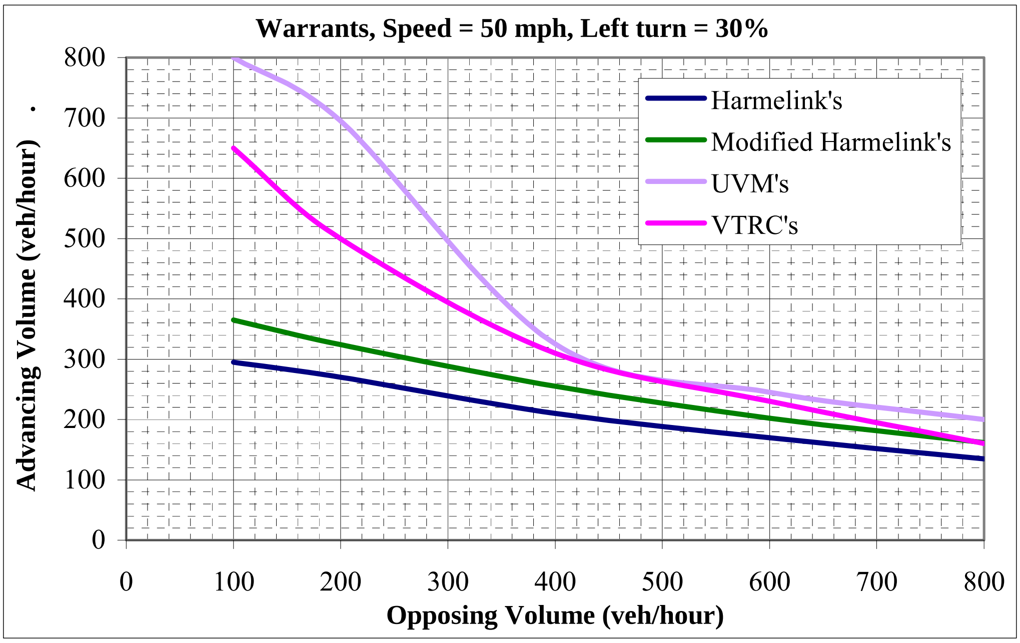


Figure 4.7: Comparison of the new left turn lane warrants with different warrants

4.1.2 Number of Stops

Using the same guiding principles as discussed above, thresholds were selected for the development of left turn lane warrants for urban two-lane category based on total number of stops (number) as warrant criteria. These are as shown in the Table 4.2.

Table 4.2: Thresholds selected for the urban two-lane category

Operating Speed	Thresholds
(mph)	(no. of stops/hour)
50	70
40	90
30	130

Using the thresholds values shown in Table 4.2, warrants were developed for the urban two-lane category based on total number of stops (number) as warrant criteria. Figures 4.8, 4.9 and 4.10 show the warrants for left turn lanes for operating speeds of 30 mph, 40 mph and 50 mph respectively.

Comparisons of the warrants developed by the researchers in this study for the urban two-lane category using percent stops as warrant criteria, with Harmelink, modified Harmelink and VTRC's warrants are shown in Figures 4.11 and 4.12. As before, the developed warrants seem to match well with the other warrants when the opposing volume is above 400 vph, but tend to yield higher volumes at lower opposing volume values.

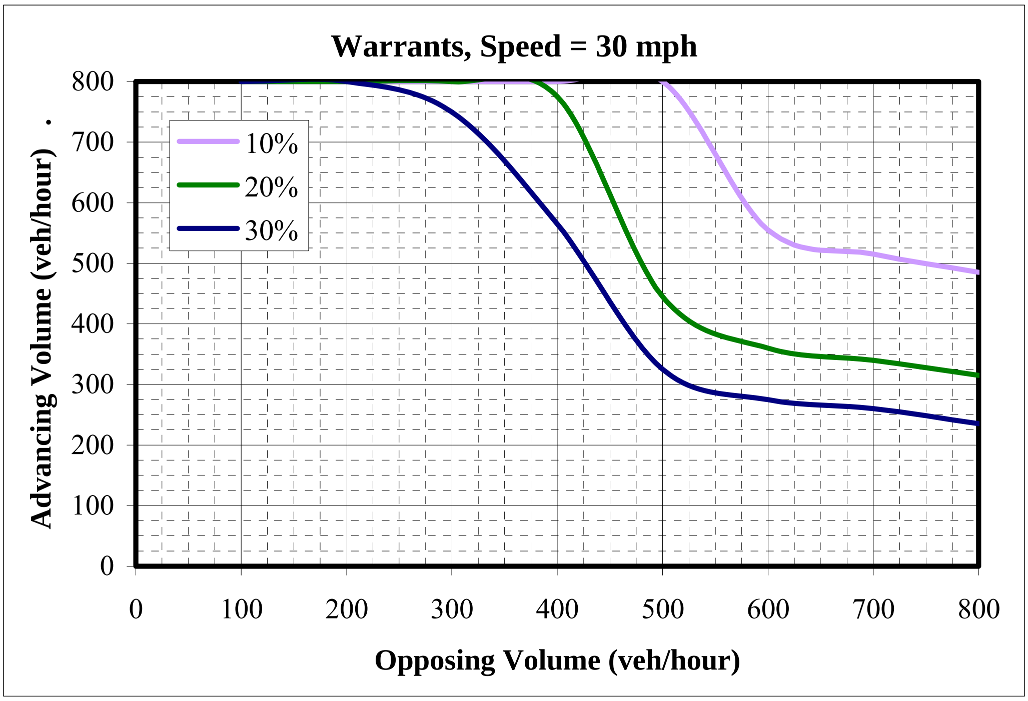


Figure 4.8: Warrants for urban two-lane category for operating speed 30 mph

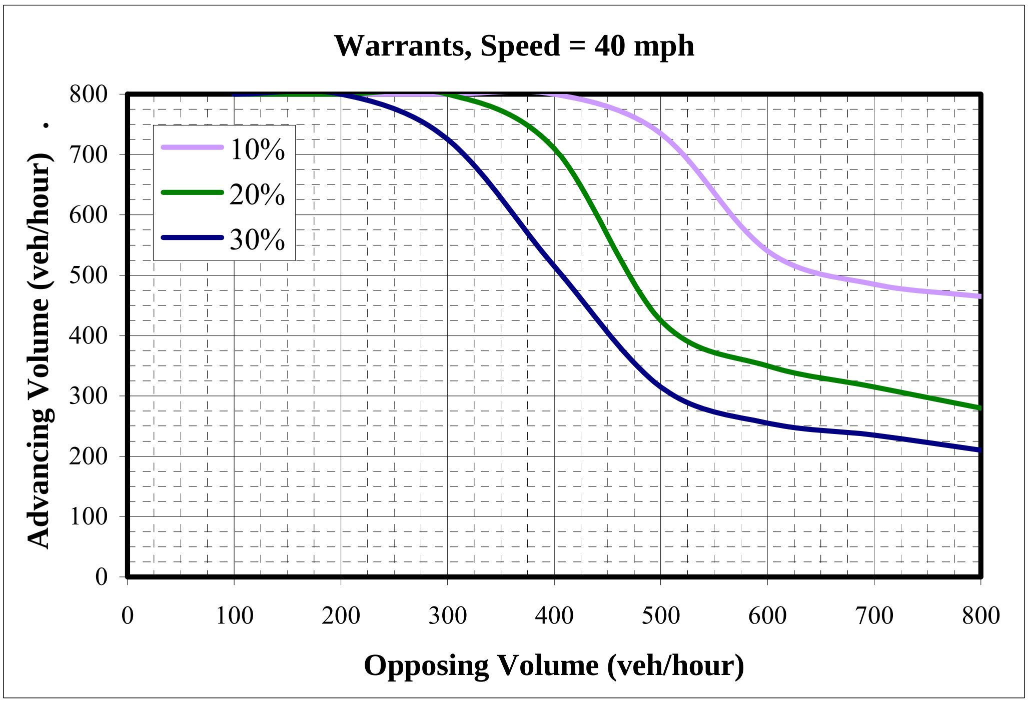


Figure 4.9: Warrants for urban two-lane category for operating speed 40 mph

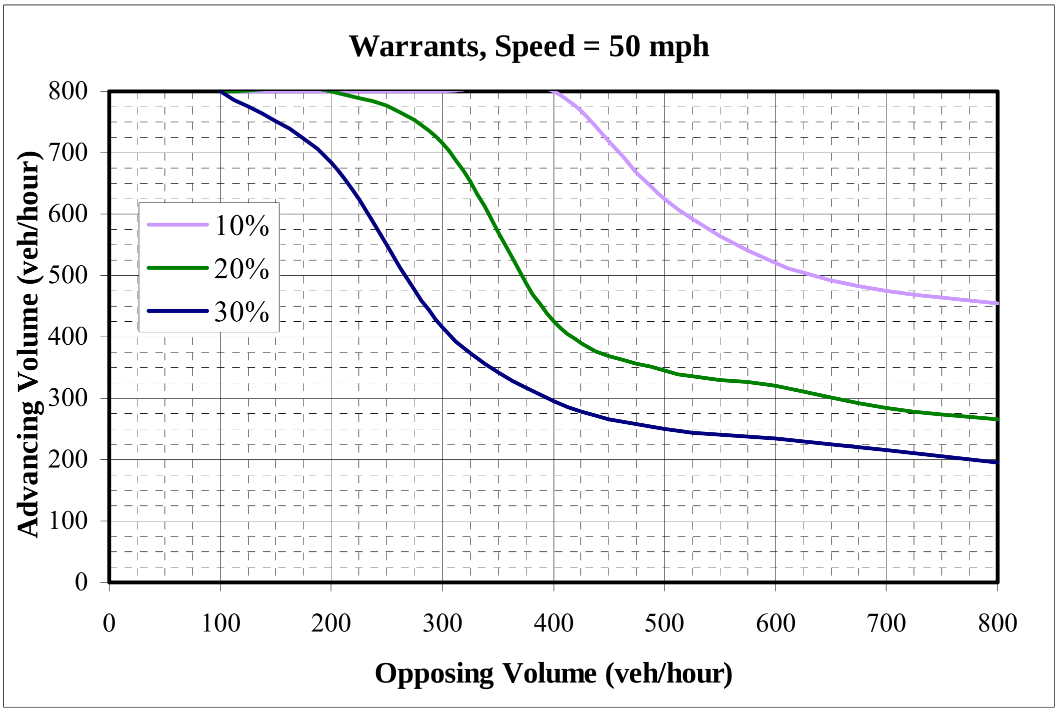


Figure 4.10: Warrants for urban two-lane category for operating speed 40 mph

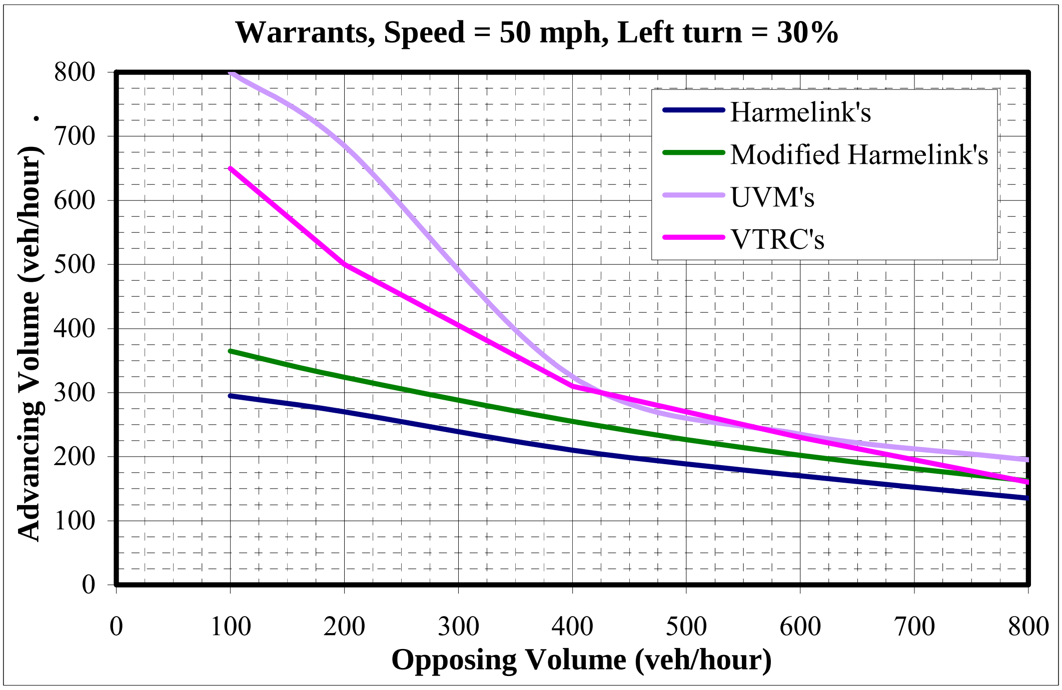


Figure 4.11: Comparison of the new left turn lane warrants with different warrants

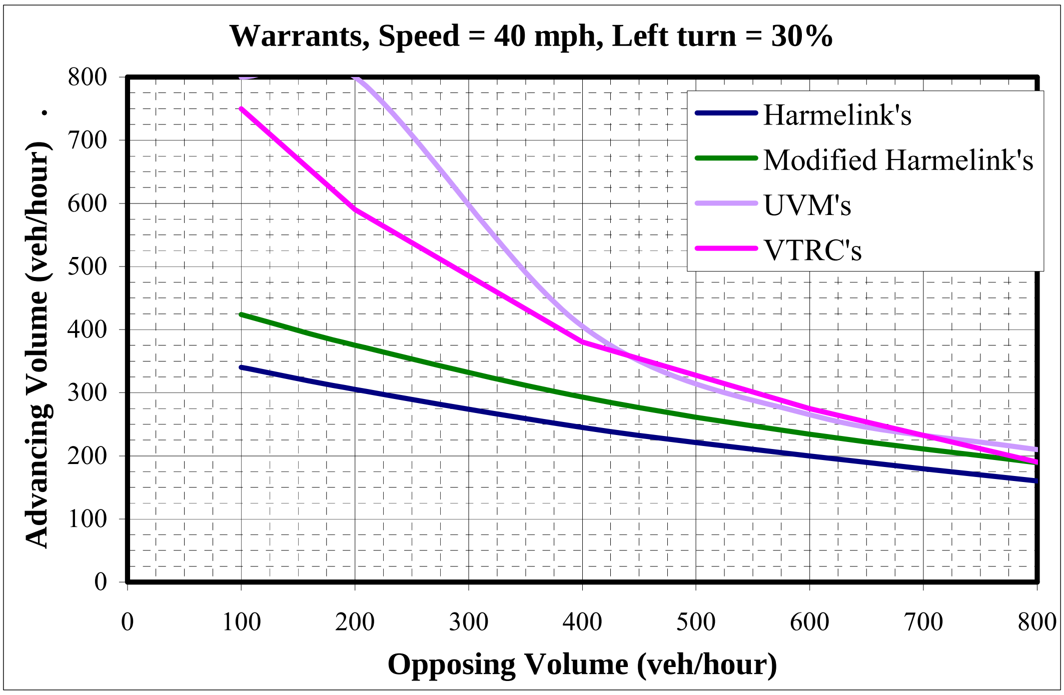


Figure 4.12: Comparison of the new left turn lane warrants with different warrants

4.1.3 Comparison between Criteria

The warrants for the urban two-lane category developed using total delay as warrant criteria were compared to the warrants developed using total number of stops. As can be observed from Figures 4.13, 4.14 and 4.15, the warrants developed using the different warrant criteria tend to produce results which are very close to each other, which is quite favorable because it increases the confidence level in the warrants developed in this study. Since, both sets of warrants developed in this study are based on performance operational measures; it is not surprising if they are very much close to each other.

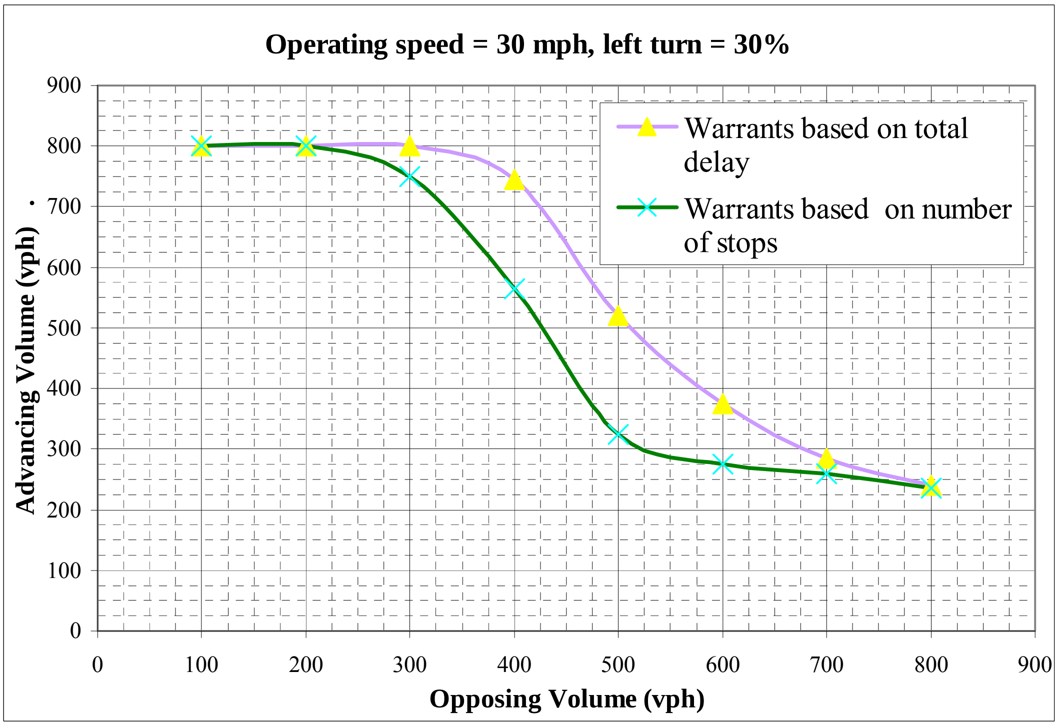


Figure 4.13: Comparison between Warrants Developed Using Different Criteria

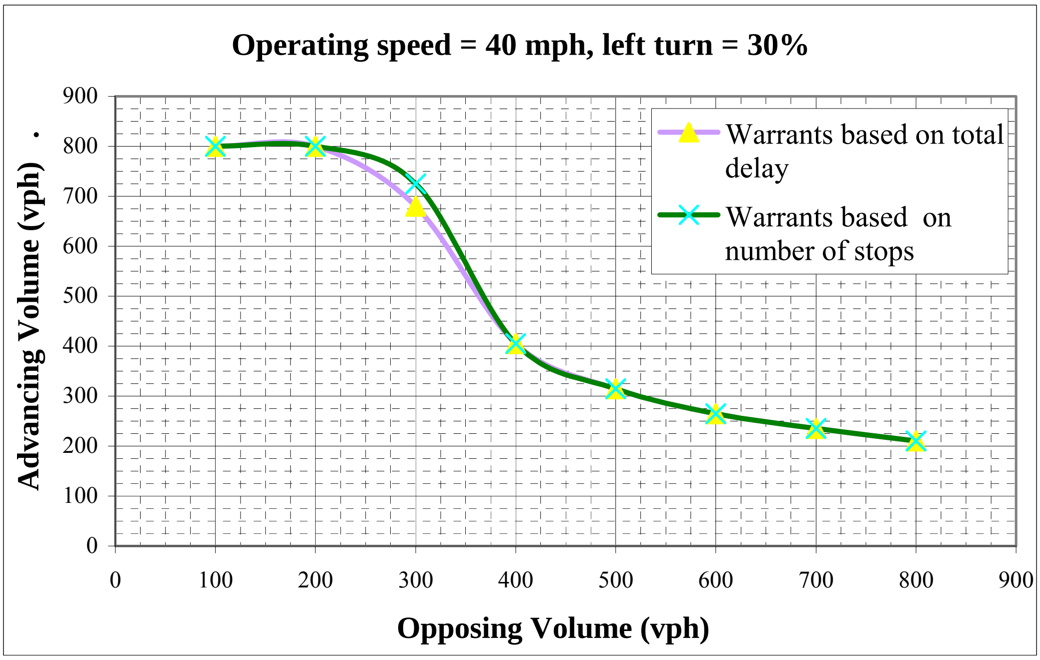


Figure 4.14: Comparison of the Warrants Developed Using Different Warrants Criteria

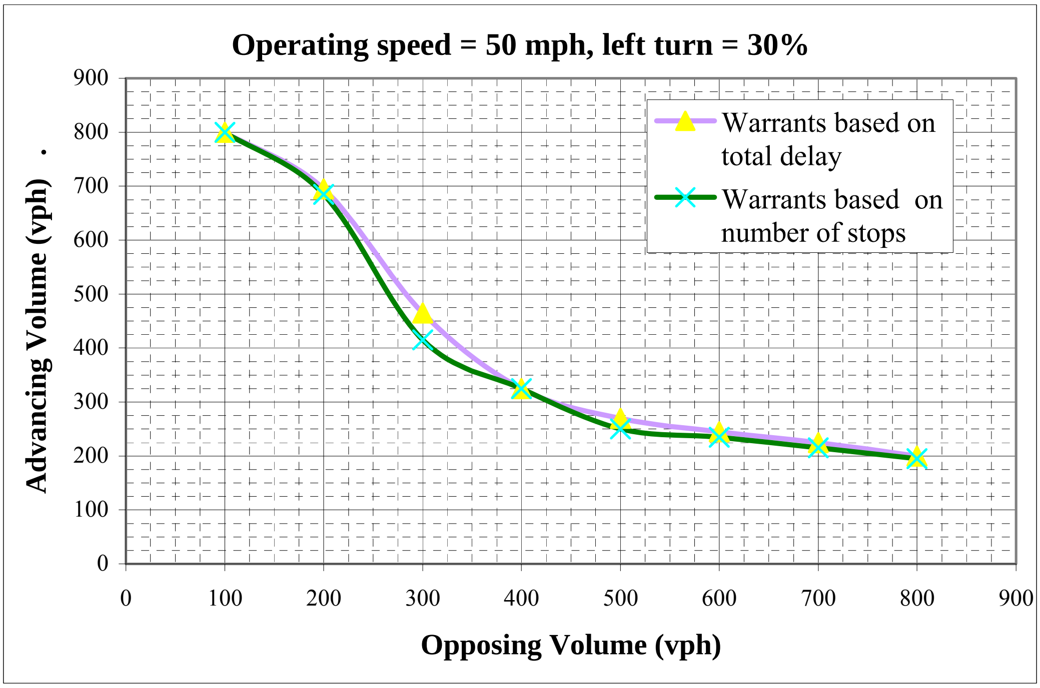


Figure 4.15: Comparison of the Warrants Developed Using Different Warrants Criteria

4.2 Rural Two-Lane Roads

4.2.1 Total Delay

Thresholds selected for the rural two-lane category for developing the warrants based on total delay (sec/hour) criteria, were similar to those for the urban two-lane category as shown in the Table 4.3. Figure 4.16 shows an example of the warrants developed for rural two-lane category based on the total delay as warrant criteria for operating speed of 40 mph and left turning percentages of 10%, 20% and 30%. The warrants for the other operating speeds such as 30 mph and 50 mph and the different left turning percentage are presented in Appendix A (Figure A.1 and Figure A.2).

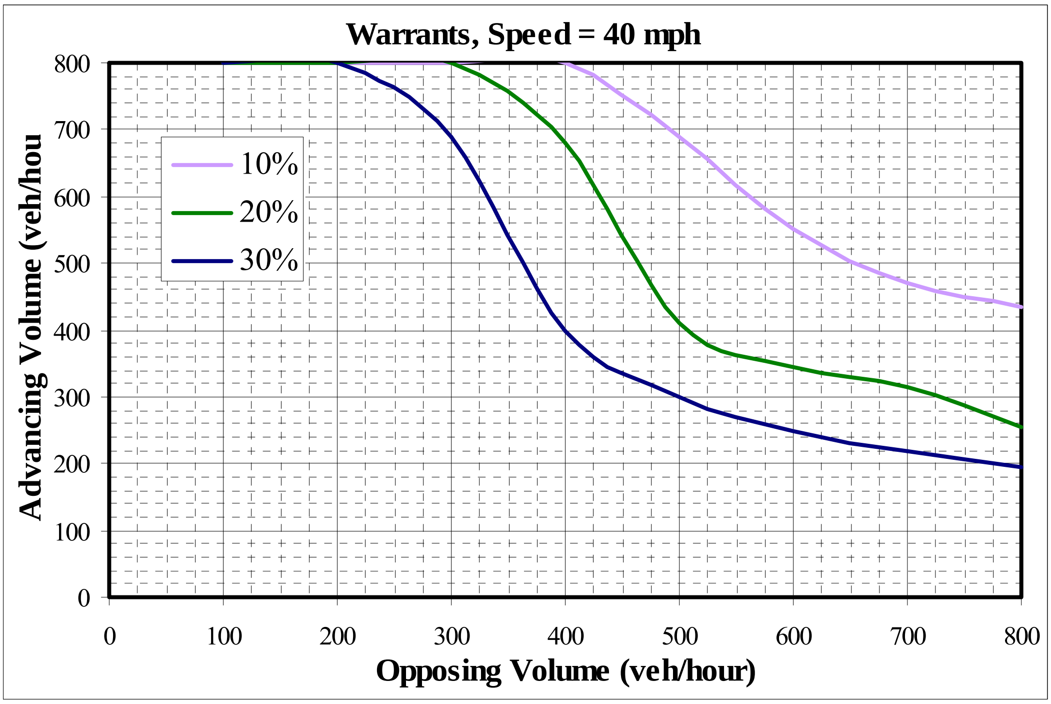


Figure 4.16: Warrants for Rural Two-Lane Category for Operating Speed of 40 Mph

4.2.2 Number of Stops

The total number of stops thresholds selected for the rural two-lane category was similar to those for the urban two-lane category (to be shown in Table 4.4). Figure 4.17 shows a sample of the warrants developed for rural two-lane category based on the total number of stops for operating speed of 40 mph and left turning percentages of 10%, 20% and 30%. The warrants for the other operating speeds such as 30 mph and 50 mph and the different left turning percentage are presented in Appendix A (Figure A.3 and Figure A.4).

4.2.3 Comparison between Criteria

Like the urban two-lane warrants, the warrants for rural two-lane category developed using different warrant criteria such as total delay (veh.sec/hour) and total number of stops (number) on the subject link were compared to each other. As observed in the Figure 4.18, the warrants based on the different criteria again are very close to each other which is quite favorable for the reason explained earlier.

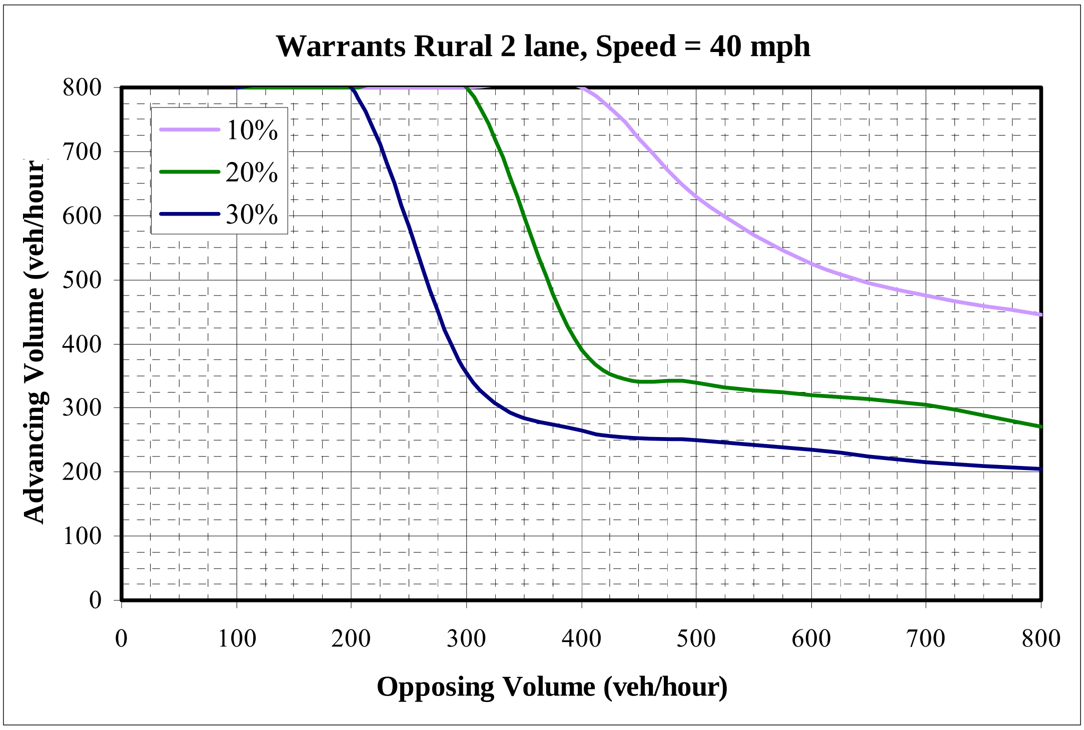


Figure 4.17: Warrants for Rural Two-Lane Category for Operating Speed of 40 Mph

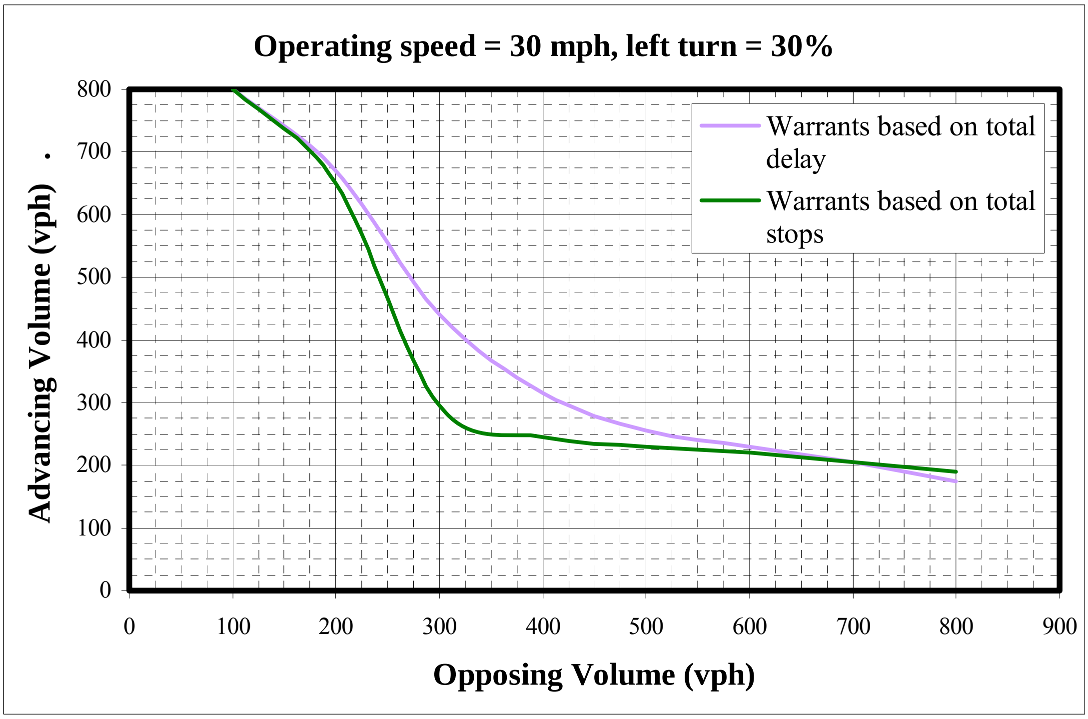


Figure 4.18: Comparison of the Warrants Developed Using Different Warrants Criteria

4.3 Urban Four-Lane Roads

4.3.1 Total Delay

Warrants were also developed for the urban four-lane category based on the two different warrant criteria. The thresholds selected for the development of left turn lane warrants based on total delay (veh.sec/hour) are shown in Table 4.3.

Table 4.3: Thresholds Selected for the Urban Four-lane Category

Operating Speed	Thresholds
(mph)	(vehicle. seconds/hour)
50	4000
40	10000
30	30000

Figure 4.19 shows the warrants developed using the thresholds shown in Table 4.3 for an operating speed of 50 mph and left turning percentages of 10%, 20% and 30%. Warrants for other operating speeds such as 30 mph and 40 mph are presented in the Appendix A (Figure A.5 and Figure A.6).

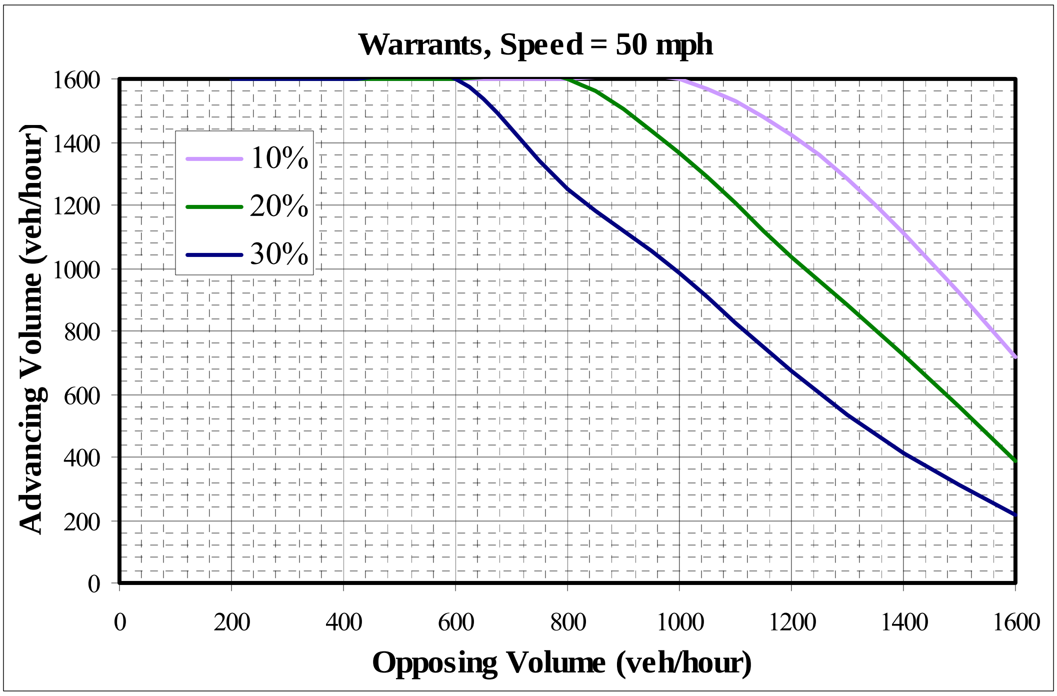


Figure 4.19: Warrants for the Left Turn Lane for the Urban Four-lane Category

4.3.2 Number of Stops

The thresholds selected for the development of left turn lane warrants based on the total number of stops on the subject link (number) for the urban four-lane category, are shown in the Table 4.4. Figure 4.20 shows the developed warrants for an operating speed of 50 mph and left turning percentages of 10%, 20% and 30%. Warrants for other operating speeds such as 30 mph and 40 mph are presented in the Appendix A (Figure A.7 and Figure A.8).

Table 4.4: Thresholds Selected for the Urban Four-lane Category

Operating Speed	Thresholds
(mph)	(vehicle. seconds/hour)
50	80
40	100
30	130

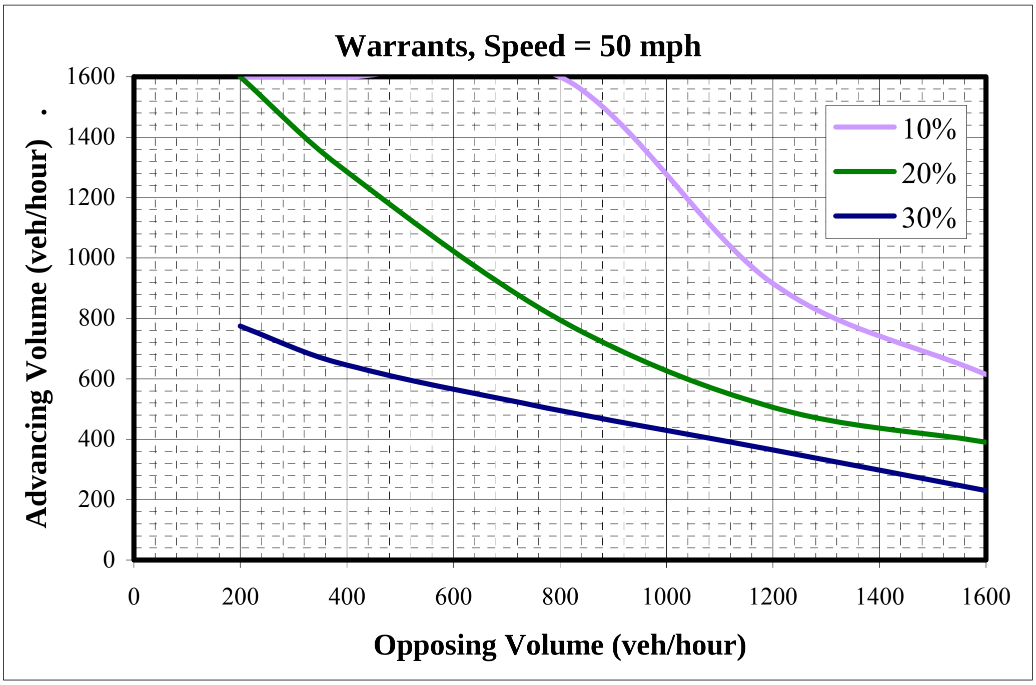


Figure 4.20: Warrants for the Left Turn Lane for the Urban Four-lane Category

4.4 Comparison among Road Categories

Though, the thresholds used for developing the warrants were the same for both of the categories, the warrants developed for rural two-lane category were different than the warrants developed for urban two-lane category. The main reason behind this was the fact that the gap acceptance distribution for rural two-lane category was found to be higher than that for the urban two-lane category as discussed earlier. Figure 4.21 shows the comparison between warrants for the different road categories for an operating speed of 50 mph and left turning percentage of 30%. As observed, while the warrants for the urban two-lane and rural two-lane appear to be quite similar to one another, the warrants for the four-lane urban category are quite different. This indicates the need to distinguish between two-lane and four-lane roads when assessing the need for a left-turn lane at an unsignalized intersection.

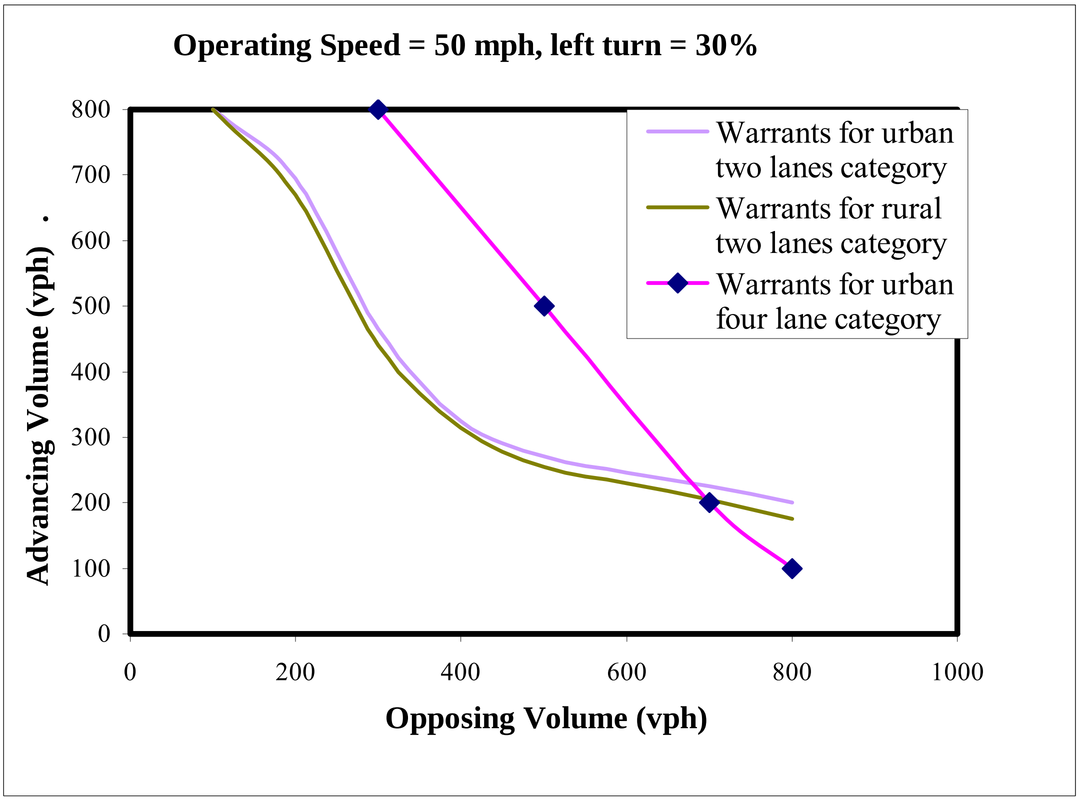


Figure 4.21: Comparison of warrants for urban two-lane and rural two-lane based on total delay

5 SAFETY ANALYSIS: METHODOLOGY

5.1 General Study Approach

This part of the study is focused on learning whether or not the presence of a left turn lane at an unsignalized intersection or driveway has a significant effect on the occurrence of crashes. This requires assembling a data set of crashes observed at unsignalized intersections both with and without left turn lanes, controlling for other features expected to influence the occurrence of crashes. We originally considered conducting this as a before-after analysis of the safety effect of installing left turn lanes, however, we found that the number of intersections at which left turn lanes were added was very small, and the number at which all of the needed information to conduct a proper analysis was available was even smaller so as to make this impractical.

Instead we chose a different approach. We collected crash and traffic volume data at a large number of unsignalized intersections without left turn lanes and estimated base line crash prediction models that could be used to predict the expected number of crashes at similar intersections with left turn lanes. Using generalized linear modeling, we estimated models for different types of intersections categorized by the area type (rural or urban), the number of lanes on the major road (2 or 4), and the number of approach legs (3 or 4). These characteristics are considered to be important because they affect the nature and character of the vehicle interactions in the intersection, and thus, potentially the number of crashes and their distribution with respect to crash type as well as its severity. In rural areas, the number of unsignalized intersections on four-lane roads is very limited, so such intersections are not included in this study. This leaves six categories of intersections, coded as R2T, R2X, U2T, U2X, U4T and U4X, with the symbols denoted as follows:

R = rural; U = urban
2 = two-lane; 4 = four-lane
T = t-intersection, or three-leg; X = crossing intersection, or four-leg

For the crash prediction models, a negative binomial distribution is assumed, in which the occurrence of crashes at a given location is a Poisson process, with the mean crash rate at locations with identical characteristics following a Gamma distribution. Rather than using covariates to represent the intersection characteristics, a separate model was estimated for each of the six intersection categories. The only covariate used was the variable Annual Average Daily Traffic (AADT), which is important for representing the effect of exposure on the total crash risk. The estimation models take the form:

$$\mu_{ij} = e^{\beta_0} V_{ij}^{\beta_1} \tag{5.1}$$

Where

μ_{ij} = the expected mean of the crashes occurring at intersection i in year j;

V_{ij} = the AADT of the major road of the intersection i in year j.

The models were estimated using generalized linear modeling (GLIM) in the statistical software SAS®.

We also identified intersections with left turn lanes in each of the six categories and collected crash and volume data at them as well. For each intersection with a left turn lane, the estimated models were then used to predict the number of crashes expected at that intersection *if it did not have a left turn lane.* The idea is that if the observed crash count at an intersection with a left turn lane is significantly lower than what would be expected if it did not have a left turn lane, then there is evidence that the left turn lane creates a safer condition. This information can then be used to derive safety-based warrants to work in conjunction with the volume-based warrants. Roadway geometry was also considered in explaining differences in crash experience among different locations.

5.2 Selection of Study Sites

The study sites were selected using the Connecticut Department of Transportation (ConnDOT) photolog program, in which the entire state-maintained highway system is photographed annually using roadway image and data collection technology. Fifty intersections without left turn lanes were chosen for each intersection category except for the U4X category, in which only forty intersections could be found. Due to the small number of unsignalized intersections found with left turn lanes in one or both directions of the major road, the number of the study intersections in each intersections category varies from a minimum of 3 to a maximum of 21. Intersections are defined by the route number of the major road and the route mileages at which they are located. All of the selected intersections are located on undivided roads, without signalization and with no stop signs on the major road approaches. For all intersections selected, it was also required that important geometric characteristics (e.g., lane width, shoulders, curvature), the surrounding environment, and traffic control remained unchanged during the 10-year period from 1995 to 2004 for which crash counts were collected. For some of the study intersections (those with left turn lanes), the presence of a left turn lane could not be verified before 1999. In those cases, crash records were only considered for the years from 1999 on. The study sites selected are summarized in Table 5.1.

Table 5.1: Data Describing Crash Study Intersections

Intersection Category	Left Turn Lane	Number of Intersections	Average AADT	Min AADT	Max AADT	Number of Crashes
R2T	YES	13	13801	5400	30200	157
	NO	50	10918	2200	30200	623
R2X	YES	3	13988	7700	18700	44
	NO	50	7425	1400	16300	620
U2T	YES	21	13126	6700	25800	407
	NO	50	12016	3500	28900	1103
U2X	YES	4	11695	8300	15400	52
	NO	50	9599	1600	26000	943
U4T	YES	8	18714	8400	26700	326
	NO	50	18479	8500	30000	1763
U4X	YES	8	17082	12600	34800	355
	NO	40	16472	8400	29400	1398

Crash records were collected from the ConnDOT Office of Policy and Planning over a 10-year period from 1995 to 2004 at each selected study location, both with and without left turn lanes. These crash records are categorized by crash type and severity to more appropriately identify their consequences, as the objective is not just to reduce crashes, but to minimize their consequences as well. Since intersection-related crashes do not necessarily occur within the boundary of an intersection, crashes were gathered from within a buffering distance of 250 ft (76.2 m) along both major road approaches in each intersection. For cases in which there was another intersection within this 250 ft (76.2 m) distance, only crashes in the space between the intersections were used. The information collected from the crash records includes the year, date, time, severity, collision type, weather, and road surface condition.

The AADT's on the major roads for each intersection were obtained from the ConnDOT Traffic Log for the 10-year period. The Traffic Log gives AADT at certain locations on the road, generally for every segment delineated by significant road intersections. The AADT nearest to the intersection and not separated from the study intersection by any other major intersection was used. In this case, a major intersection would be an intersection with comparable or greater importance in the road hierarchy.

The major roads are all state roads for which AADT data are available. Unfortunately, most of the intersecting roads are local roads and therefore have no such traffic data. It is desirable to have traffic data for both the major roads and the intersecting roads because some types of collisions, especially those involving turning or crossing vehicles, are related to interactions between vehicles entering from both from the major and the intersecting roads. The percentage of turning vehicles at the intersection might also be important for representing exposure to collisions related to vehicles turning.

5.3 Compilation of Data

The presence of a left turn lane is very likely to affect, whether increase or decrease, some types of crashes but not all. Rather than considering the total crash counts, which would potentially neutralize or cancel out the effect of the left turn lanes, we analyze crashes by collision type to determine what effect a left turn lane may have on a specific crash type.

The ConnDOT crash database categorizes each crash into one of 17 collision types, as listed in Table 5.2. Records coded as collision type 6, "Miscellaneous Non-Collision", are not considered in this study. For the remaining 16 collision types, however, it is impractical to study each of them separately. It is more applicable to aggregate together several collision types, for which the safety is expected to be affected similarly by adding left turn lanes. Based on previous findings (Zhang *et al.* 2007) these collision types are combined into three categories for analysis as follows:

- **Category 1: same direction crashes,** including Turning-Same Direction, Sideswipe-Same Direction, Rear-end.
- **Category 2: intersecting direction crashes,** including Turning-Opposite Direction, Turning-Intersecting Paths and Angle.

- **Category 3: all other types of crashes**, including Sideswipe-Opposite direction, Overturn, Head-on, Backing, etc.

Table 5.2: Crash Type in ConnDOT's Crash Database

Crash Type	Description	Crash Type	Description
01	Turning-Same direction	10	Head-on
02	Turning-Opposite Direction	11	Backing
03	Turning-Intersecting Paths	12	Parking
04	Sideswipe-Same Direction	13	Pedestrian
05	Sideswipe-Opposite Direction	14	Jackknife
06	Miscellaneous Non-Collision	15	Fixed Object
07	Overturn	16	Moving Object
08	Angle	17	Unknown
09	Rear-end		

Source: ConnDOT (2006)

Another important aspect of the crash is its severity level. The ConnDOT crash database defines three levels of severity: fatal, injury (no fatality) and property damage only. We are more concerned on whether or not the left turn lanes can reduce those serious crashes or mitigate their consequences. Therefore, fatal and injury crashes were summed up for each of the intersections.

Then, for each category of intersections with and without left turn lanes, two data sets were prepared containing crash counts by crash category and severity. Tables 5.3 and 5.4 show samples of the structure of each data set for R2T intersections as an example. There is one of each table for each of the six intersection categories. One crash record in each year for each intersection from 1995 to 2004 gives a total number of 500 crash records of the intersections without left turn lanes in one table, except for the U4X category, which has 400 records for 40 intersections that meet the requirement of site selection. Compared with 50 or 40 records if we sum the total crashes for all years for each intersection, this gives a much larger sample size which is preferable for modeling. A separate generalized linear model was fitted to the data for each intersection category. The number of intersections with left turn lanes in each table varies and their traffic volume data would be applied to the prediction models to predict crashes assuming no left turn lanes were added and the expected crashes will be compared with the actual crashes to evaluate the safety effect of left turn lanes.

Table 5.3: Number of Crashes by Category for R2T Intersections

No.	ID	Year	AADT	Crash_1	Crash_2	Crash_3
1	R2TN01	1995	21600	0	0	1
2	R2TN01	1996	22600	0	0	0
.						
10	R2TN01	2004	19500	0	1	0
. . .						
500	R2TN50	2004	8200	0	0	1
1	R2TY01	1995	23700	0	1	1
2	R2TY01	1996	23900	1	0	1
.						
10	R2TY01	2004	25300	0	0	0
. . .						
126	R2TY13	2004	6800	0	0	0

Table 5.4: Number of Fatal and Injury Crashes for R2T Intersections

No.	ID	Year	AADT	Crash
1	R2TN01	1995	21600	0
2	R2TN01	1996	22600	0
.				
10	R2TN01	2004	19500	2
. . .				
500	R2TN50	2004	8200	1
1	R2TY01	1995	23700	0
2	R2TY01	1996	23900	0
.				
10	R2TY01	2004	25300	0
. . .				
126	R2TY13	2004	6800	0

5.4 Negative Binomial Modeling

A traditional linear model restricts the response variable to a normal distribution. However, data are not always normally distributed but they can be counts or proportions, etc. and their variances are not necessarily constant for all observations. The traditional linear model is not appropriate for modeling such data. In this case, the generalized linear model, as extensions of the traditional linear model, is applicable to more general distributions than the normal for the response. It allows the response probability distribution to be any member of an exponential family. A link function g is used to relate the linear component to the mean of the response variable, *i.e.*

$$g(\mu_i) = x_i^{'}\beta \tag{5.2}$$

Where x_i is a column vector of covariates or explanatory variables, for observation i; β is the coefficients to be estimated; and μ_i is the expected value of the response value for the ith observation. The link function g can be logit, log, or identical, etc. The variance of the response variable y_i depends on the mean μ_i by

$$Var(y_i) = \frac{\phi V(\mu_i)}{\omega_i} \tag{5.3}$$

Where ϕ is a dispersion parameter either known or estimated and ω_i is a known weight for each observation. A generalized linear model is fitted to the data by maximum likelihood estimation of the parameter vector β. In SAS, GENMOD procedure provides modeling for the generalized linear model.

Negative Binomial distribution, as a member of exponential distributions, is appropriately modeled by a generalized linear model. In the traffic engineering area, Negative Binomial modeling is an accepted practice in crash modeling among the leading researchers today (Miaou 1994). It assumes that the number of crashes at a given location follows a Poisson distribution, while the mean of the crashes follows a Gamma distribution. The variance of the Negative Binomial distribution is

$$Var(y) = \mu + \kappa\mu^2 \tag{5.4}$$

The distribution of y,

$$P(y) = \frac{\Gamma(\kappa^{-1} + y)}{\Gamma(\kappa^{-1})y!}\left(\frac{\kappa\mu}{1+\kappa\mu}\right)^{y}\left(\frac{1}{1+\kappa\mu}\right)^{1/\kappa} \tag{5.5}$$

approaches Poisson (μ) as κ becomes close to zero. The Negative Binomial distribution can accommodate overdispersion. When a Poisson regression model does not fit the data and it appears that the variance of y is increasing faster than the Poisson model allows, a Negative Binomial model that is more dispersed can be an alternate choice. In SAS, the Negative Binomial modeling is requested by using dist = NB statement in the GENMOD procedure.

When the responses are discrete and correlated, Generalized Estimating Equations (GEEs) provides a practical method to analyze such data. The GEEs was introduced by Liang and Zeger in 1986 to deal with discrete correlated data. These data can be modeled as a generalized linear model except for the correlation among the responses. They are modeled using the same link function, but the covariance structure of the correlated measurement must also be modeled. Let y_{ij}, $j = 1,...,n_i$, $i = 1,...,K$, represent the j th measure on the i th subject, and let $R_i(\alpha)$ be a $n_i \times n_i$ working correlation matrix. The covariance matrix of Y_i can then be modeled as

$$V_i = \phi A_i^{\frac{1}{2}} R(\alpha) A_i^{\frac{1}{2}} \tag{5.6}$$

where A_i is a $n_i \times n_i$ diagonal matrix with $Var(\mu_{ij})$ as the j th diagonal element and ϕ is the dispersion parameter. In SAS, the structure of the working correlation can be set as fixed, independent, exchangeable, unstructured and autoregressive. When the structure is autoregressive AR (1), the correlation between y_{ij} and $y_{i,j+t}$ would be

$$Corr(y_{ij}, y_{i,j+t}) = \alpha^t \quad \text{for } t = 0,1,2,...n_i - j \tag{5.7}$$

The fitting algorithm for GEEs is not a likelihood-based method of estimation, so inference measures based on likelihood are not applicable to GEE methods. First, it computes an initial estimate of β with an ordinary generalized linear model and then the working correlations R is computed based on the standardized residuals; covariance is estimated by $V_i = \phi A_i^{\frac{1}{2}} R(\alpha) A_i^{\frac{1}{2}}$; β is then updated by

$$\beta_{r+1} = \beta_r + \left[\sum_{i=1}^{K} \frac{\partial \mu_i'}{\partial \beta} V_i^{-1} \frac{\partial \mu_i}{\partial \beta} \right]^{-1} \left[\sum_{i=1}^{K} \frac{\partial \mu_i'}{\partial \beta} V_i^{-1} (Y_i - \mu_i) \right] \tag{5.8}$$

and then iterated until convergence. The model-based covariance of $\hat{\beta}$ is given by

$$Cov(\hat{\beta}) = I_0^{-1} \tag{5.9}$$

Where

$$I_0 = \sum_{i=1}^{K} \frac{\partial \mu_i'}{\partial \beta} V_i^{-1} \frac{\partial \mu_i}{\partial \beta} \tag{5.10}$$

It's the GEE equivalent of the inversed of the Fisher information matrix that is often used in generalized linear model as an estimator of the covariance of the maximum likelihood estimator of β. The estimator

$$M = I_0^{-1} I_1 I_0^{-1} \tag{5.11}$$

Where

$$I_1 = \sum_{i=1}^{K} \frac{\partial \mu_i'}{\partial \beta} V_i^{-1} Cov(Y_i) V_i^{-1} \frac{\partial \mu_i}{\partial \beta} \tag{5.12}$$

is called the empirical, or robust estimator of the covariance matrix of $\hat{\beta}$. It is a constant estimator even if the working correlation matrix is mis-specified. Statistics that are helpful in assessing the goodness of fit of an ordinary generalized linear model, such as the scale deviance, is not applicable when GEE method has been used. Lin, Wei and Ying (2002) present graphical and numerical methods for model assessment based on cumulative sums of residuals over certain coordinates, which can be used in model-checking for GEE modeling. The distribution of the residuals under the assumed model can be approximated by certain zero-mean Gaussian process. The observed residuals can then be compared with a number of simulations from the null distribution, which enable objective assessment whether the observed residual pattern reflects anything beyond random fluctuation. In SAS, the GEE solution is requested by using the REPEATED statement in the GENMOD procedure.

5.5 Safety Performance Functions

Once the crash and traffic volume data were ready, generalized linear models assuming Negative Binomial distribution were fitted to the data for the intersections without left turn lanes. The GEE method was used in all of the model fitting process to account for the correlation among the crashes in 10 years at each intersection arising from this repeated measures study design. The Safety Performance Functions (SPF) are obtained, relating the traffic volume to the occurrence of crashes, in the form of $\mu_{ij} = e^{\beta_0} V_{ij}^{\beta_1}$, where μ_{ij} is the expected mean of the crashes occurring at intersection i in year j; V_{ij} is the AADT on the major road for the intersection i in year j. β_0 and β_1 are the parameters estimated from the Negative Binomial modeling. Separate models were fitted to each of the 6 intersection types by crash category.

5.5.1 Model Specification

In addition to the "AADT" variable, a "Year" dummy variable was also considered, which is to account for time trend or factors other than traffic volume that change from year to year. The models are in the form of

$$\mu_{kij} = e^{\beta_t} e^{\beta_{k0}} V_{kij}^{\beta_{k1}} \tag{5.13}$$

where β_t is the parameter for year t, k is the intersection type. Models with same estimates of both β_0 and β_1, with same estimate of β_0 but different β_1, and with different estimates of both β_0 and β_1, for the 6 intersection types were fitted and tested respectively. The results show that models with different β_0 and β_1 yield better model fit than models in which the different intersection types share the same β_0 and/or β_1. This suggests that the six intersection types have significantly different SPF's, and thus, different crash occurrence mechanisms.

The parameter estimates for the "Year" variable are found to be insignificant in most cases. Models with and without the year variables are compared. The results are shown in Appendix C (Table C.1 to C.3). Including the year variables in the model doesn't seem to improve the model fit very much. Therefore, the simpler model without the year variables was adopted, that is the original SPF, $\mu_{ij} = e^{\beta_0} V_{ij}^{\beta_1}$ for each type of intersections is used.

5.5.2 By Crash Category

The parameter estimates for the simple SPF's are shown in Appendix C (Table C.1 to C.3) for each of the 6 intersection types by crash category. The parameter estimates in Table C.2 and C.3 for intersecting direction crashes (crash category 2) and other crashes (crash category 3), however, are not always significant at 0.05 level. For example, β_0 and β_1 are only significant for R2T intersections for crash category 2 and the fit P-values for those models are small, which is not very preferable. For crash category 3, β_0 and β_1 are both significant for only two intersection categories, that is R2X and U4X intersections, and the fit P-value of the model for U4X category is merely 0.05, indicating that only the intercept and the variable of traffic volume can not adequately explain the variation of the crash occurrence. More variables are preferred to be added in.

Based on the results obtained, therefore, we are only confident in the models for crash category 1, same direction crashes, including turning-same direction, sideswipe-same direction and rear-end crashes, for each of the intersection types. The models with parameters estimated are listed in Table C.1 and shown in Figure 5.1 as below.

$$\begin{aligned}
&\text{R2T intersection: } \mu_{ij} = e^{-10.4} V_{ij}^{1.0347}\\
&\text{R2X intersection: } \mu_{ij} = e^{-13.49} V_{ij}^{1.3891}\\
&\text{U2T intersection: } \mu_{ij} = e^{-10.81} V_{ij}^{1.1648}\\
&\text{U2X intersection: } \mu_{ij} = e^{-11.25} V_{ij}^{1.1963}\\
&\text{U4T intersection: } \mu_{ij} = e^{-7.255} V_{ij}^{0.8051}\\
&\text{U4X intersection: } \mu_{ij} = e^{-12.3} V_{ij}^{1.3247}
\end{aligned} \tag{5.14}$$

We can see from Figure 5.1 that generally, there are more crashes at the same level of AADT in non-rural areas than in rural areas regardless of other intersection characteristics. As AADT level increases, crashes in R2X intersections increase faster than in R2T intersections, and faster in U2T than in U2X intersections. Generally, the U4X intersection type has a much faster increasing rate of crashes than any other categories. U4T intersections have more crashes as the AADT level is lower than 15,000 but the rate increases at a relatively mild pace thereafter. Generally, it has smaller number of crashes than U2T and U4X intersections at higher level of AADT.

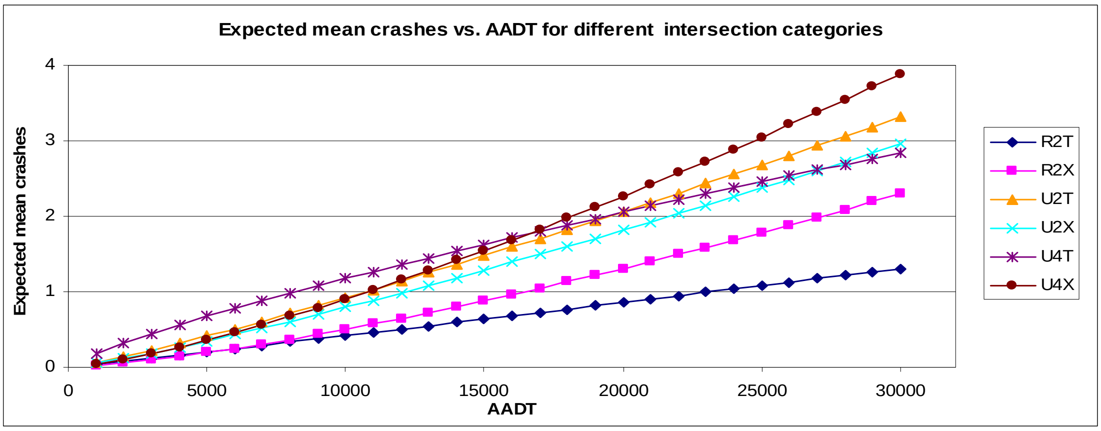


Figure 5.1: Models for the Six Intersection Categories

Since the observed crashes in each year is a random variable with variation around the mean expected value, in some years, the number of crashes might be higher than the mean and in other years, it might be lower than the mean. If we aggregate them all in several years, it would yield more consistent results. Therefore, we define a Safety Performance Function (SPF) for each intersection category, which is in the form of

$$SPF = \mu_i = \sum_{j=1}^{J} \mu_{ij} = \sum_{j=1}^{J} e^{\beta_0} V_{ij}^{\beta_1} \tag{5.15}$$

where N_{ij}, V_{ij}, β_0 and β_1 are the same as stated before and N_i is the sum of the expected mean crashes for intersections in J years, which is 10 years from 1995 to 2004 in our case. This results in a dataset with a much reduced number of cases, but can be estimated using GLM rather than GEE methods. Thus we have the following SPF for crash category 1 for the 6 categories of intersections:

$$\begin{aligned}
&\text{R2T intersection: } SPF = \sum_{j=1}^{J} e^{-10.4} V_{ij}^{1.0347} \\
&\text{R2X intersection: } SPF = \sum_{j=1}^{J} e^{-13.49} V_{ij}^{1.3891} \\
&\text{U2T intersection: } SPF = \sum_{j=1}^{J} e^{-10.81} V_{ij}^{1.1648} \\
&\text{U2X intersection: } SPF = \sum_{j=1}^{J} e^{-11.25} V_{ij}^{1.1963} \\
&\text{U4T intersection: } SPF = \sum_{j=1}^{J} e^{-7.255} V_{ij}^{0.8051} \\
&\text{U4X intersection: } SPF = \sum_{j=1}^{J} e^{-12.3} V_{ij}^{1.3247}
\end{aligned} \tag{5.16}$$

The value computed from the SPF for an intersection can then be compared with the actual observed crashes in one year for that intersection to evaluate the performance of the intersection.

5.5.3 By Crash Severity

The SPF's as obtained above consider same direction crashes only, which seems not enough to completely assess the safety performance of an intersection. Therefore, models were estimated with total fatal and injury crashes as the dependent variable using the same methods as described above for estimating models of collision type. The parameter estimates are shown in Appendix C (Table C.4). For U2T intersections, neither of the estimates of β_0 and β_1 are significant. The fit P-values for U2X and U4T models are not high enough, indicating that a model with only intercept and traffic volume is not sufficiently complete to explain the variation in crashes. Given that the intercept and traffic volume do have significant effect on crash occurrence, we will use these models to make complementary analysis to the study we have done previously on crash categories.

The model estimation results are listed in Appendix C (Table C.4) and the models are graphed in Figure 5.2 and listed in Equation 5.17. Compared with the SPF's for same direction crashes, the estimates of β_1 are smaller for all intersection types with fatal and injury crashes, all of which are less than 1 except for U4T category.

$$\begin{aligned}
&\text{R2T intersection: } \mu_{ij} = e^{-4.7492} V_{ij}^{0.4282} \\
&\text{R2X intersection: } \mu_{ij} = e^{-5.4145} V_{ij}^{0.5331} \\
&\text{U2X intersection: } \mu_{ij} = e^{-5.9579} V_{ij}^{0.6057} \\
&\text{U4T intersection: } \mu_{ij} = e^{-11.951} V_{ij}^{1.22531} \\
&\text{U4X intersection: } \mu_{ij} = e^{-8.7589} V_{ij}^{0.94697}
\end{aligned} \tag{5.17}$$

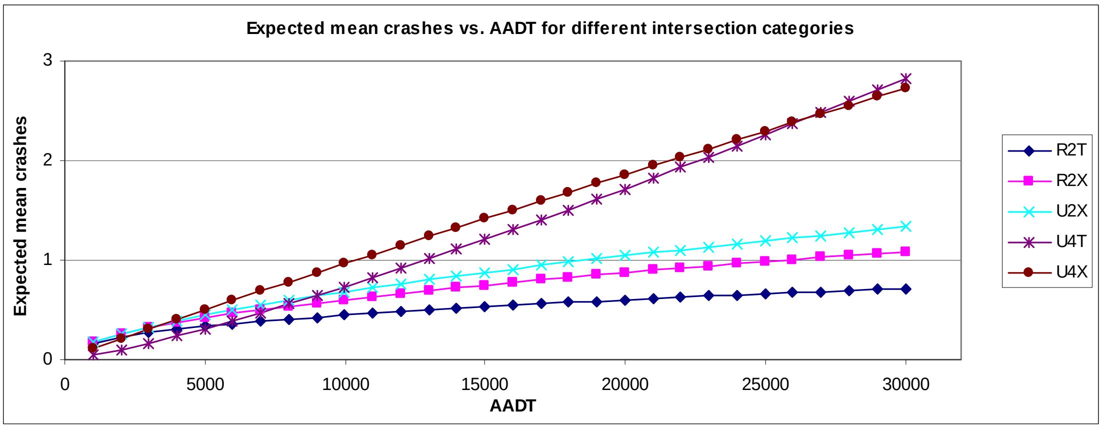


Figure 5.2: Models for Fatalities and Injuries

Figure 5.2 shows that at the same level of AADT, there are more crashes in non-rural areas than in rural areas. Intersections with 4 lanes in the major road have much higher crash rates than any other intersections. Intersections with 2 lane roads have much slower increase of crashes along the increase of AADT. In rural area, R2X intersections have more expected crashes than R2T intersections. In all, we may come to the conclusion that U4T and U4X categories of intersections are the most dangerous places not only because a large number of crashes are expected to occur there but also because these crashes tend to be more severe than those in other intersections.

Similarly, we established an overall SPF for fatal and injury crashes over all 10 years for each of the intersection category except for the U2T category. They are shown below.

$$\begin{aligned}
&\text{R2T intersection: } SPF = \sum_{j=1}^{J} e^{-4.7492} V_{ij}^{0.4282} \\
&\text{R2X intersection: } SPF = \sum_{j=1}^{J} e^{-5.4145} V_{ij}^{0.5331} \\
&\text{U2X intersection: } SPF = \sum_{j=1}^{J} e^{-5.9579} V_{ij}^{0.6057} \\
&\text{U4T intersection: } SPF = \sum_{j=1}^{J} e^{-11.951} V_{ij}^{1.22531} \\
&\text{U4X intersection: } SPF = \sum_{j=1}^{J} e^{-8.7589} V_{ij}^{0.94697}
\end{aligned} \tag{5.18}$$

The value computed from the SPF can then be compared to the actual observed crashes with fatalities or injuries to evaluate the safety performance of the intersection.

5.6 Prediction and Comparison

As stated above, the expected number of crashes was predicted using the SPF for the intersections with left turn lanes for each year. The prediction will then be compared with the actual observed crashes to see whether having the left turn lane results in safer condition. However, we are not confident to say that the intersection has positive safety effect so long as the observed crashes are fewer than the expected or vice versa. It is reasonable that variation around the expected value exists since the number of crashes is a random variable. Therefore, it is necessary to work out an interval corresponding to each expected value so that the observed crashes can be compared to the interval rather than the expected mean value. Observed crashes that fall within the interval could be considered to have the same expected mean value obtained from the SPF. Conversely, those beyond the interval are to be considered to have significantly different mean values than values estimated from the SPF. Thus we can make a conclusion whether the safety at an intersection is significantly different from its peers based on its crash records. In order to predict a single crash count for each intersection it is necessary to aggregate the predictions for each of the 10 years of observed AADT, thus masking the true distribution of the predicted counts. However, the crash counts aggregated over 10 years are less likely to be overdispersed, so a Poisson distribution should be a reasonable assumption. We use the Probability Distribution Function of the Poisson distribution,

$$P(y) = \frac{\lambda^y e^{-\lambda}}{y!} \tag{5.19}$$

to compute $P(y \geq N)$ and $P(y \leq N)$, where N is the total number of observed crashes within ten years at an intersection. Then, for any left turn lane intersection for which $P(y \geq N)$ or $P(y \leq N)$ is less than a set value (for example, 0.05), we can say that its expected mean of crashes is different from that obtained from the SPF, that is, from the sample of similar intersections without left turn lanes. What this means is that an intersection can be considered significantly safer if $P(y \leq N)$ is less than the set value and significantly more dangerous if the $P(y \geq N)$ is less than that set value.

5.7 Consideration of Physical Characteristics

Another concern of this study is to develop safety design guidelines for exclusive left turn lanes. After we have found those that are significantly safer and more dangerous, we can further investigate their physical characteristics as well as the design feature of their left turn lanes to find some clues for the safety performance of the intersection. More reasonable judgment can then be made on whether it is the left turn lanes or the other features that make the intersections safer or more dangerous.

In those intersections where left turn lanes have a positive safety effect, the design of the left turn lanes will be good examples for us to set up our design guidelines, whereas in those intersections where left turn lanes have negative safety effects, careful investigation will be conducted to avoid failure in future design.

6 SAFETY ANALYSIS: RESULTS

6.1 Safety Performance Functions

The SPF estimate for each intersection is computed for same direction crashes and crashes involving fatality and injury respectively.

6.1.1 By Crash Category

The results are listed in Appendix D (Table D.1 to D.6) and also shown in Figures 6.1 to 6.12. The estimates from the SPF and the corresponding observed crashes in 10 years for each intersection in different categories as well as $P(y \geq N)$ and $P(y \leq N)$ are listed in the Tables (Table D.1 to D.6). The maximum, minimum and average AADT for that intersection within the 10-year period are also given.

The SPF estimated and observed crashes for each intersection are also shown in the following figures (Figure 6.1 to 6.12) by intersection category, with two figures showing intersections with and without left turn lanes, respectively, for each category. They can then be compared and some brief findings are described as below.

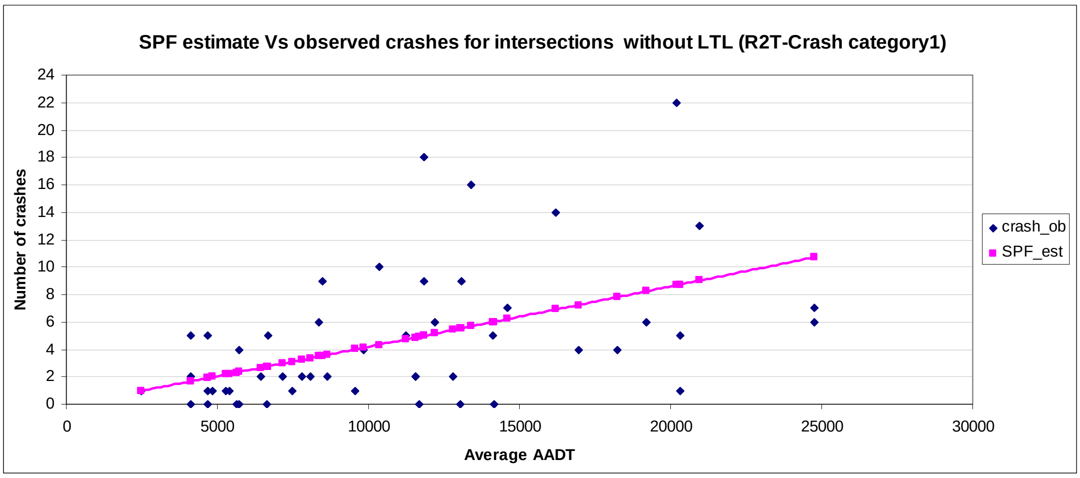


Figure 6.1: SPF Estimate and Observed Crashes for R2T Intersections without Left Turn Lanes

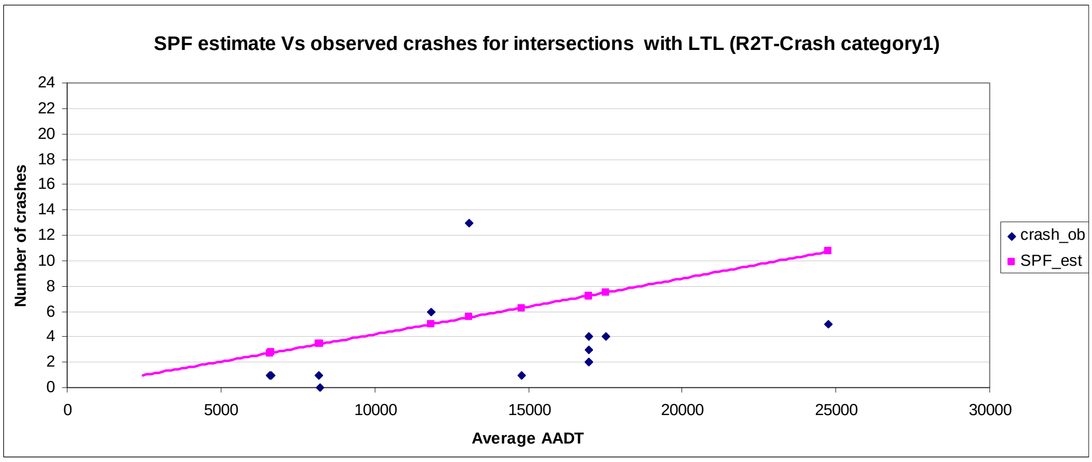


Figure 6.2: SPF Estimate and Observed Crashes for R2T Intersections with Left Turn Lanes

R2T Intersections:

More crashes occur within the AADT range of 10,000 to 15,000 for both intersections with and without left turn lanes. Due to the small sample size, there is a gap for the AADT range between 19,000 and 24,000 for intersections with left-turn lanes. Preliminary check has been made to those intersections which appear to be more dangerous than others. Most of them are on curves (e.g. R2TN18, 45; R2TY12).

Generally, adding left-turn lane(s) seems to be safer for R2T intersections within the AADT range of 5000 to 25,000. More specifically, when the AADT is between 10,000 and 15,000, the safety effect of left-turn lanes is not very obvious. One intersection with left-turn lanes and with AADT in this range has many more crashes than expected. However, there is a horizontal curve at this intersection along the major road, which might also affect the occurrence of crashes. We are also uncertain about the safety effect of the left-turn lanes when the AADT is between 19,000 and 24,000 since we lack intersections with left-turn lanes and within this AADT range.

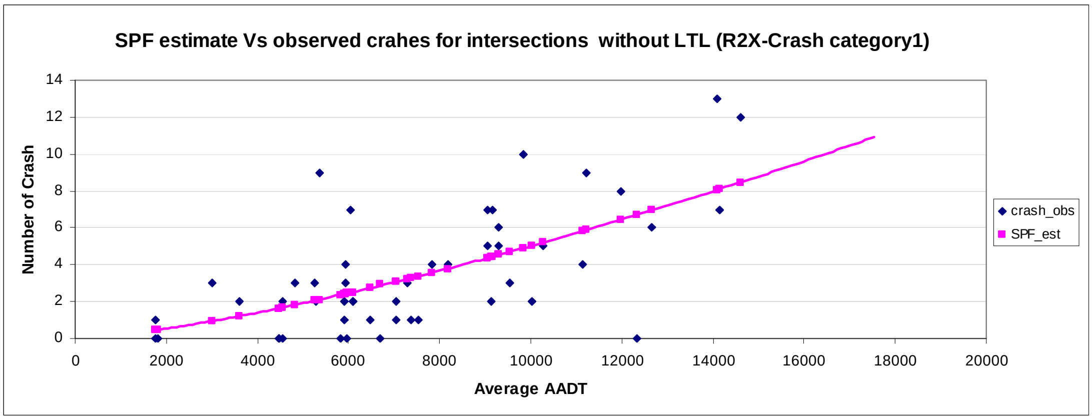


Figure 6.3: SPF Estimate and Observed Crashes for R2X Intersections without Left Turn Lanes

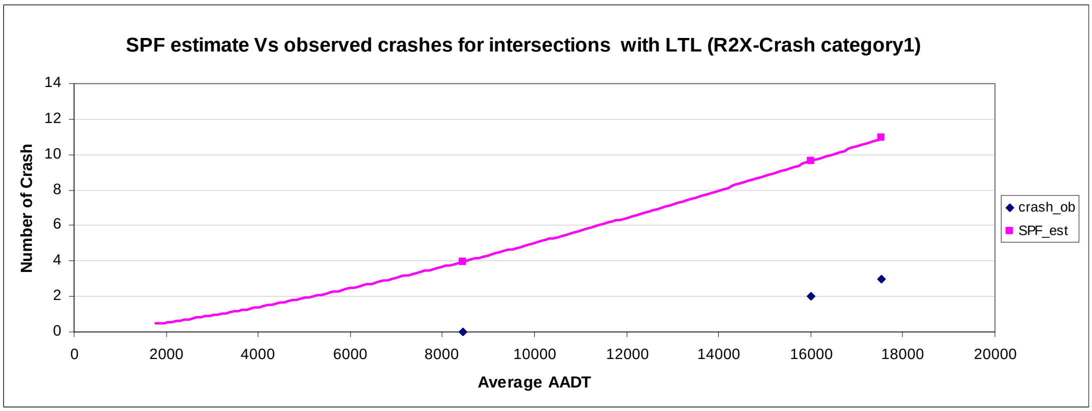


Figure 6.4: SPF Estimate and Observed Crashes for R2X Intersections with Left Turn Lanes

R2X Intersections:

Only 3 intersections are with left-turn lane(s) in this category. Due to the small sample size, there is a gap for the AADT range between 10,000 and 15,000 for intersections with left-turn lanes. Meantime, there is no intersection without left-turn lanes and with AADT larger than 16,000. Similarly to the R2T intersections, those more dangerous intersections have curves along the major road.

From the data we have, we may conclude that left-turn lanes do have positive safety effect for this intersection category, though we are not quite sure about the effect when the AADT is between 10,000 and 15,000.

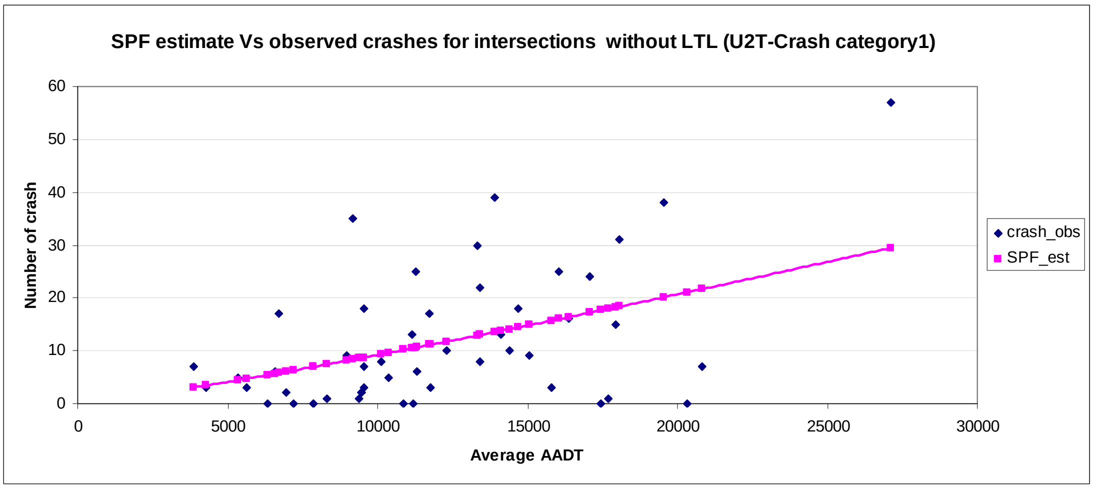


Figure 6.5: SPF Estimate and Observed Crashes for U2T Intersections without Left Turn Lanes

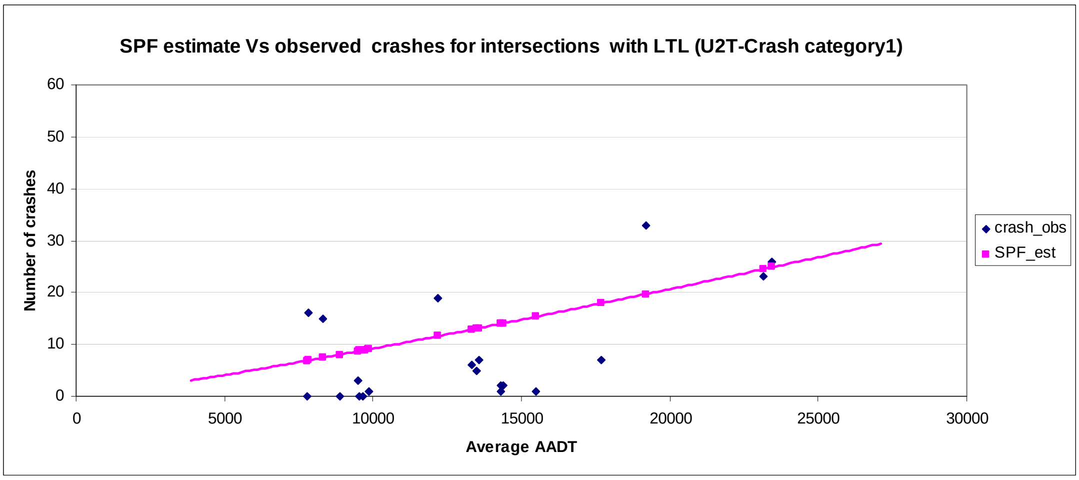


Figure 6.6: SPF Estimate and Observed Crashes for U2T Intersections with Left Turn Lanes

U2T Intersections:

4 out of 21 intersections with left-turn lanes tend to be more dangerous with $P(y \geq crash\_obs) < 0.05$. The AADT of these intersections are widely distributed from 8000 to 20,000. There are fewer crashes than expected for intersections with left-turn lanes when the AADT is around 10,000 as well as from 13,000 to 18,000.

To further look at the 4 intersections with $P(y \geq crash\_obs) < 0.05$ in the Photolog, one intersection (U2TY02) is very close to another four-way intersection with a left-turn lane, which might be one reason that more crashes occur. Another intersection (U2TY17) has a curve along the major road as well as on the minor road, which also makes it more dangerous than other intersections. When the number of driveways increases near the target intersections, there seems to be more crashes (e.g. U2TN09, 10, 21, 29, 40; U2TY12 and 18).

Conservatively, we could not conclude that left-turn lane addition improves safety for U2T intersections. Within certain AADT ranges (1000, 13000-18000), they do have fewer crashes than expected, however, we are unsure about the above effect throughout the entire AADT range since the AADT for those more dangerous intersections is spread widely. Thus, the safety effect of left-turn lanes is inconclusive for this intersection category.

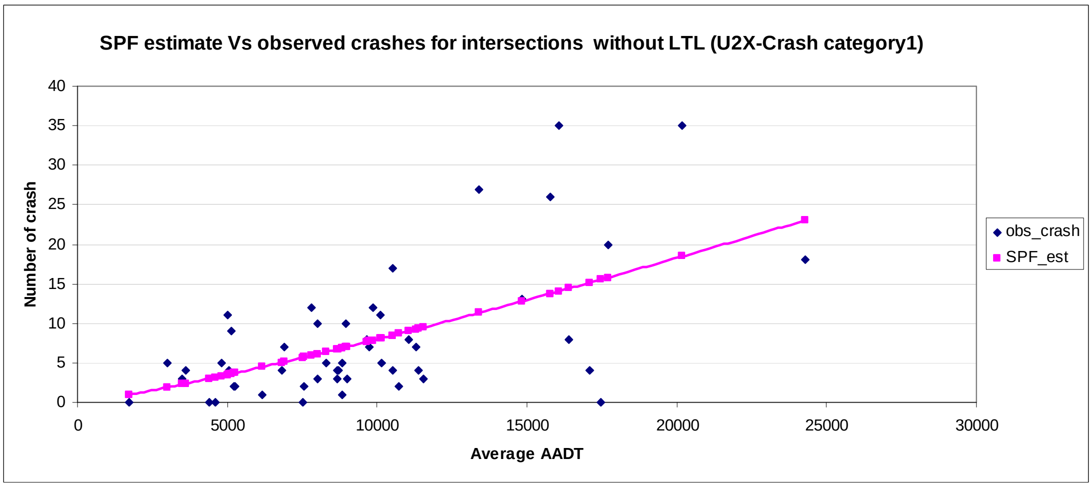


Figure 6.7: SPF Estimate and Observed Crashes for U2X Intersections without Left Turn Lanes

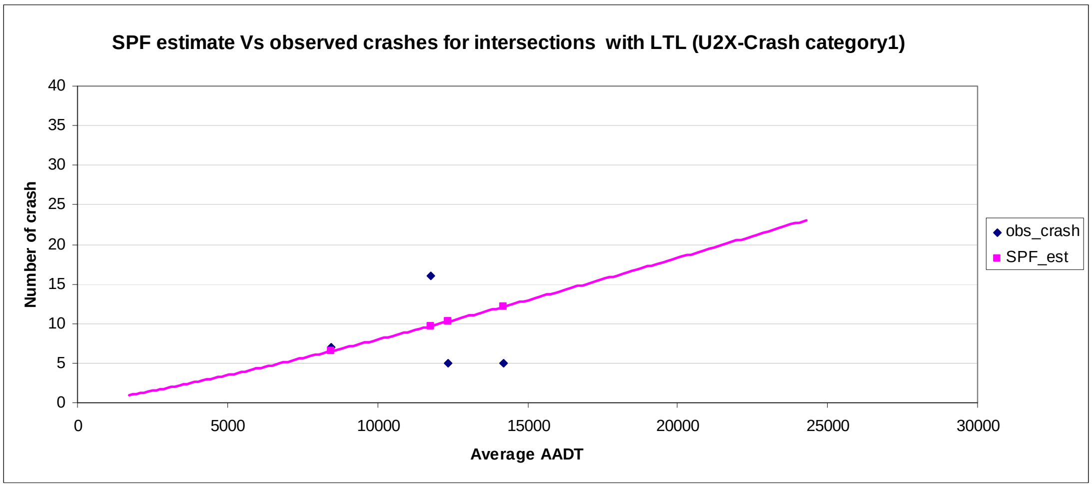


Figure 6.8: SPF Estimate and Observed Crashes for U2X Intersections with Left Turn Lanes

U2X intersections:

Only 4 intersections with left-turn lanes are in this category. One intersection (U2XY04) has $P(y \geq crash\_obs) < 0.05$ and another one (U2XY01) has the number of the crashes a little bit larger than the SPF estimate. The AADT range for intersections with left-turn lanes is between 5000 and 14,000. One reason for few intersections with large AADT might be that they are mostly signalized, and signalized intersections were excluded from this study. When the AADT is around 13,000 and 14,000, there seems to be fewer crashes than the SPF estimate. Driveways near the target intersections influence one another and there seems to be more crashes in that case. (e.g. U2XN04, 09, 11, 12, 13, 14; U2XY04)

Taking into account that there are only 4 intersections with left-turn lanes in this category and 2 of them are not significantly safer, we can not come to the conclusion that adding left-turn lanes has positive safety effect for this intersection category.

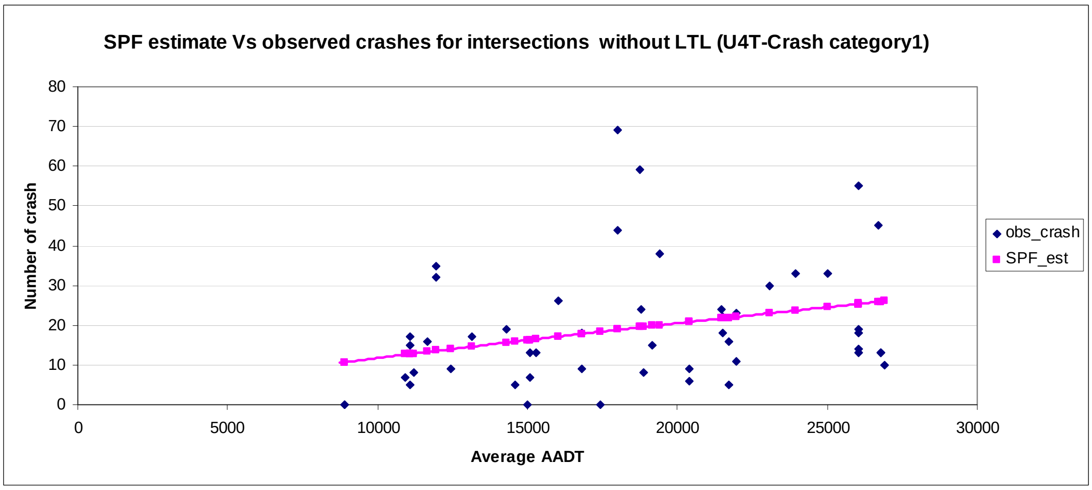


Figure 6.9: SPF Estimate and Observed Crashes for U4T Intersections without Left Turn Lanes

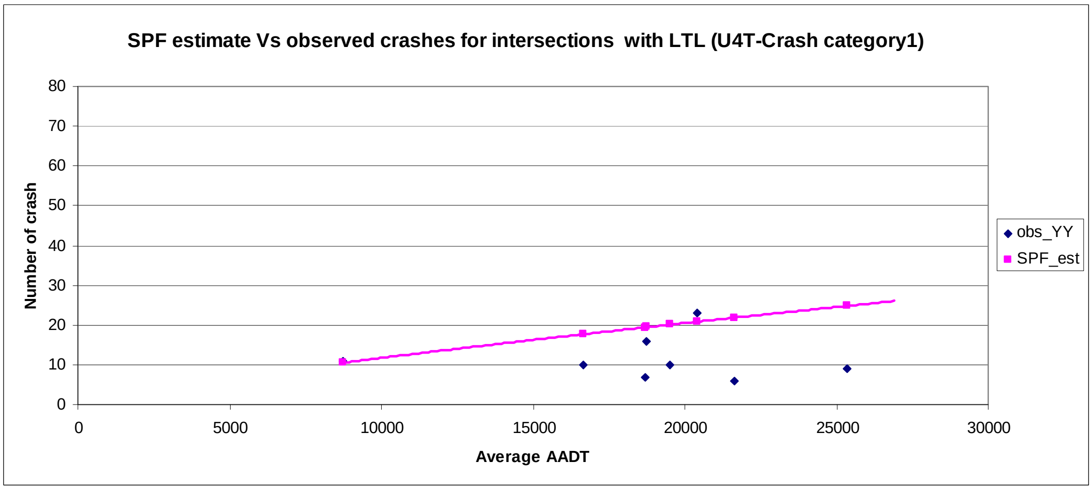


Figure 6.10: SPF Estimate and Observed Crashes for U4T Intersections with Left Turn Lanes

U4T intersections:

Crashes are prone to occur around AADT of 18,000 for intersections without left-turn lanes. None of the intersections with left-turn lane(s) has $P(y \geq crash\_obs) < 0.05$, though there is one intersection (U4TY02) that has a larger number of crashes than the SPF estimate. Due to the small sample size, there is a gap for the AADT range between 10,000 and 15,000 for intersections with left-turn lanes.

From the data we have, we may conclude that adding left-turn lanes has a positive safety effect for U4T intersections with AADT range from 15,000 to 26,000.

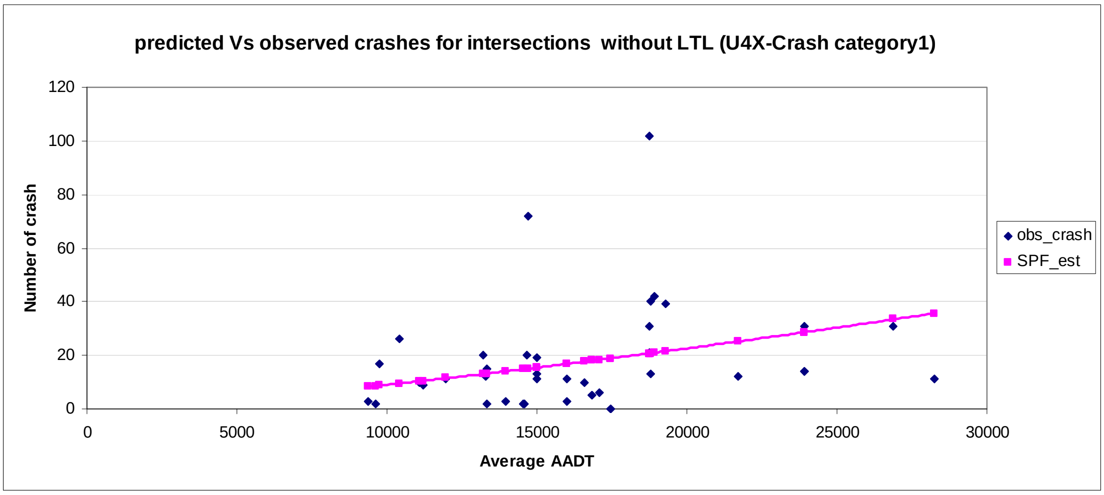


Figure 6.11: SPF Estimate and Observed Crashes for U4X Intersections without Left Turn Lanes

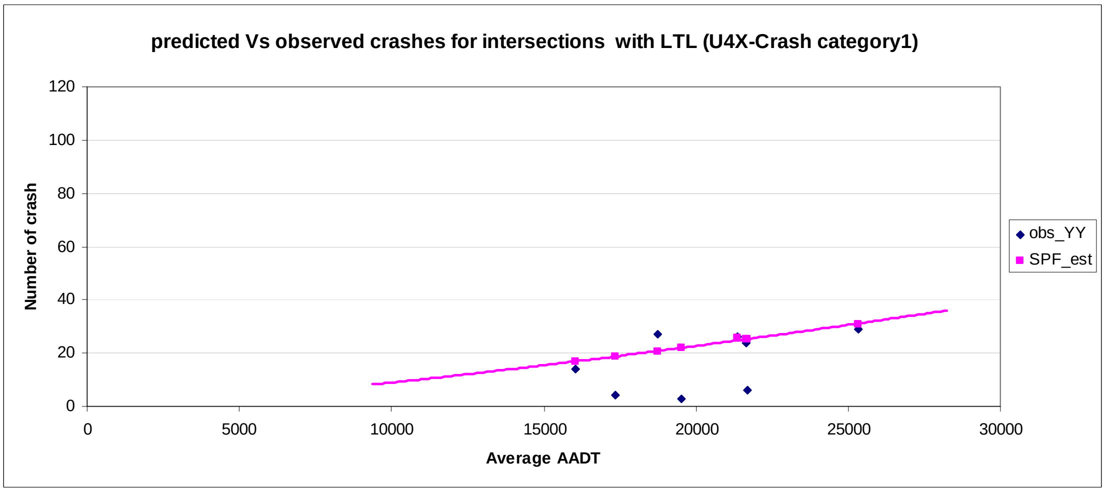


Figure 6.12: SPF Estimate and Observed Crashes for U4X Intersections with Left Turn Lanes

U4X intersections:

More crashes are observed around an AADT of 18,000 for intersections without left-turn lanes. Due to the small sample size, there are no intersections with left-turn lanes and with AADT less than 15,000. One intersection with left-turn lanes and with average AADT of 18,000 has more crashes than the SPF estimate.

Generally, adding left turn lanes has a positive safety effect for U4X intersections. Although we are not quite sure about the effect when the AADT is less than 15,000, it is likely there will be few U4T intersections with AADT in that range in reality.

Summary:

The results are summarized in Table 6.1. The percentage of the more dangerous intersections decreases by 5%-23% for intersection categories of R2T, R2X, U2T, U4T and U4X when left turn lanes are added, while it increases for the U2X category from 18% to 25%. There are 4 such intersections in the U2T category, which makes the safety effect of left turn lanes for this category inconclusive. The percentage of much safer intersections increases for all the intersections categories by 7%-98% after left turn lanes are added, except for the U4X category, with the percentage decreasing from 45% to 38%.

Based on the analysis above, left turn lanes has positive effect on same direction crashes in R2T, R2X, U4T and U4X intersections. Due to the small sample size of the U2T and U2X intersections with left turn lanes and the existence of a few dangerous intersections, the safety effect of those two categories is inconclusive.

Table 6.1: Summary for Crash Category 1

	w/o LTL					w/ LTL				
	# of int.	P(y≥crash_ob)<0.05	%	P(y≤crash_ob)<0.05	%	# of int.	P(y≥crash_ob)<0.05	%	P(y≤crash_ob)<0.05	%
R2T	50	8	16%	4	8%	13	1	8%	5	38%
R2X	50	3	6%	1	2%	3	0	0%	3	100%
U2T	50	12	24%	17	34%	21	4	19%	14	67%
U2X	50	9	18%	9	18%	4	1	25%	1	25%
U4T	50	10	20%	18	36%	8	0	0%	5	63%
U4X	40	9	23%	18	45%	8	0	0%	3	38%

6.1.2 By Crash Severity

The results are listed in Appendix D (Tables D.1 to D.6) and also shown in Figures 6.13 to 6.22. The SPF estimated and observed crashes for each intersection category with and without left turn lanes are also shown in the following figures.

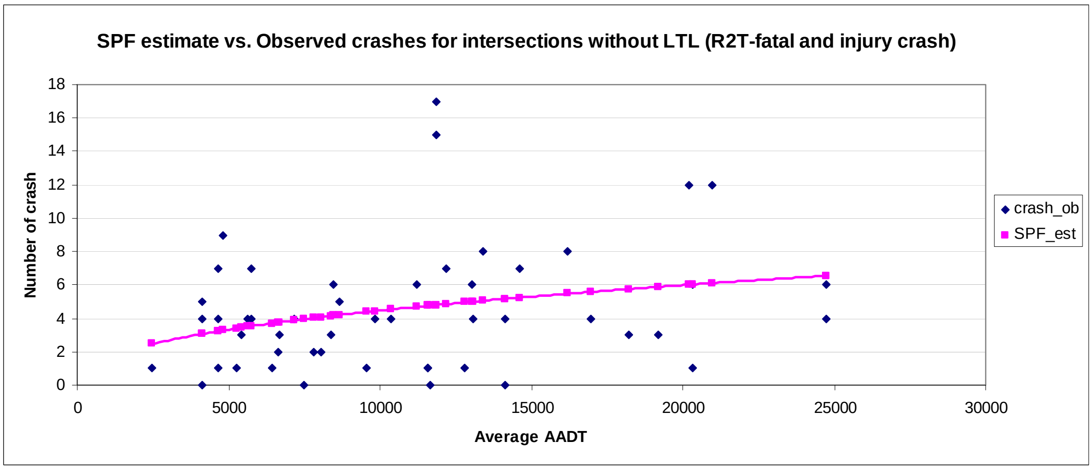


Figure 6.13: SPF and Observed Crashes for R2T Intersections without LTL (Fatal and Injury Crash)

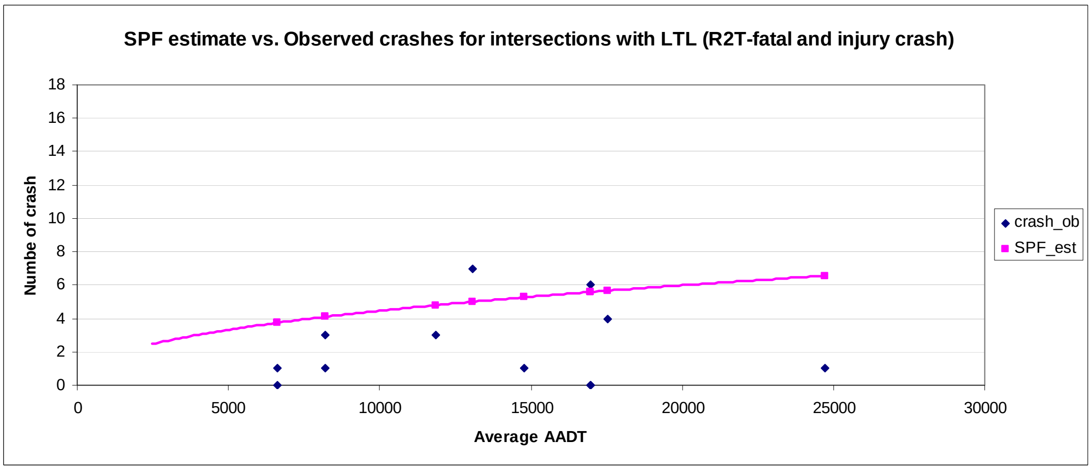


Figure 6.14: SPF and Observed Crashes for R2T Intersections with LTL (Fatal and Injury Crash)

R2T intersections:

6 intersections without left turn lanes have $P(y \geq crash\_obs) < 0.05$, while there are none for intersections with left turn lanes. 2 intersections with left turn lanes have crash rate a little bit higher than the SPF estimate. We can conclude that after left turn lanes are added, the number of fatal and injury crashes decreases significantly.

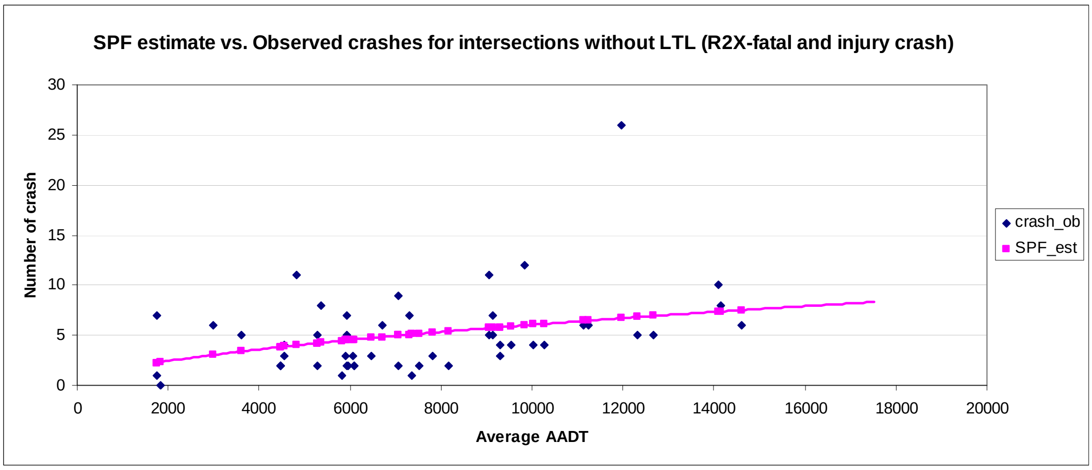


Figure 6.15: SPF and Observed Crashes for R2X Intersections without LTL (Fatal and Injury Crash)

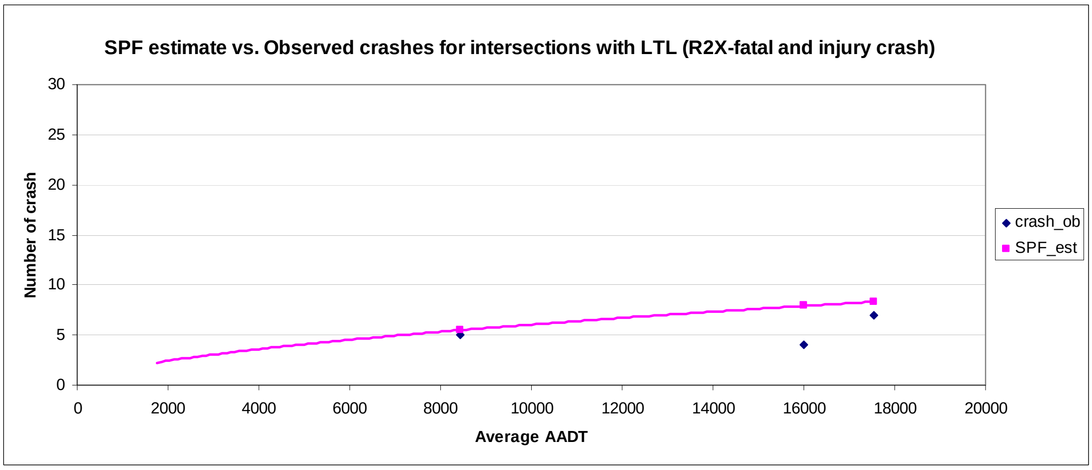


Figure 6.16: SPF and Observed Crashes for R2X Intersections with LTL (Fatal and Injury Crash)

R2X intersections:

There is a gap for the AADT range for the intersections with left turn lanes due to the small sample size. The crash rates for all of 3 intersections with left turn lanes are below the SPF estimates, whereas 5 intersections without left turn lanes have $P(y \geq crash\_obs) < 0.05$. We may conclude that adding a left turn lane at least doesn't have a negative effect for R2X intersections.

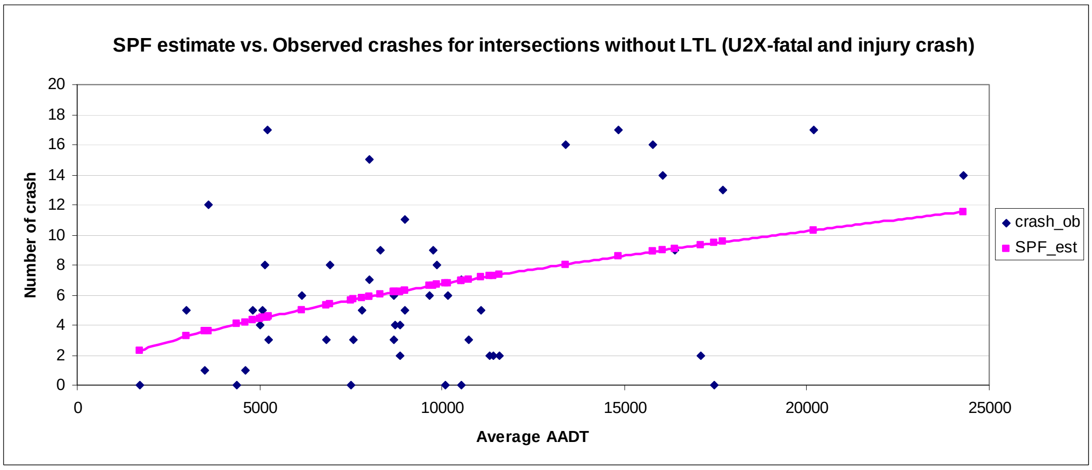


Figure 6.17: SPF and Observed Crashes for U2X Intersections without LTL (Fatal and Injury Crash)

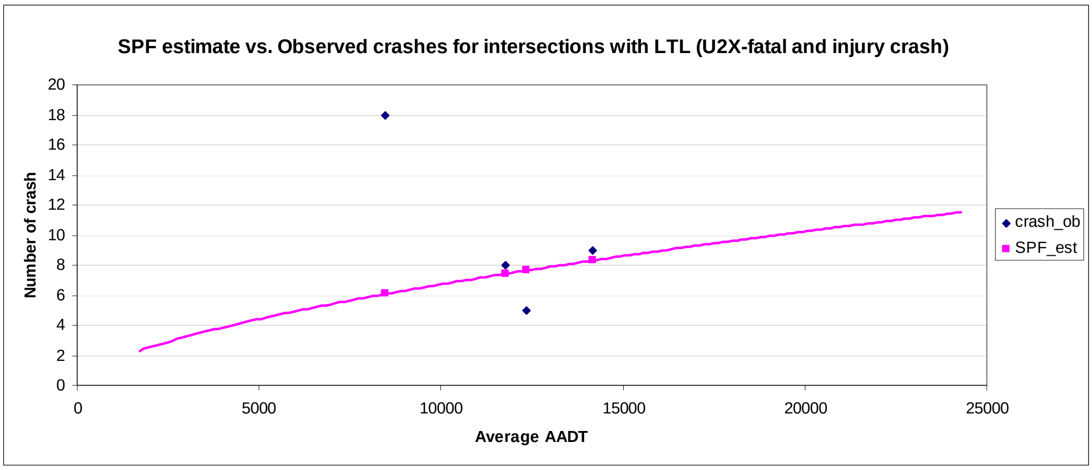


Figure 6.18: SPF and Observed Crashes for U2X Intersections with LTL (Fatal and Injury Crash)

U2X intersections:

There are 7 intersections without left turn lanes with $P(y \geq crash\_obs) < 0.05$. Only 4 intersections are with left turn lanes, one of them having $P(y \geq crash\_obs) < 0.05$ and another 2 having the number of crashes a bit larger than the SPF estimate. Adding a left turn lane doesn't seem to make the intersection safer.

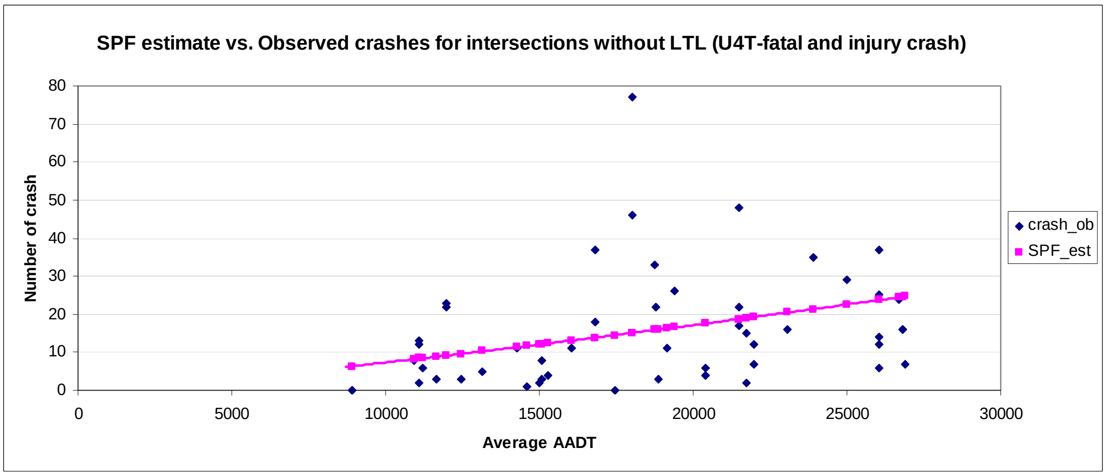


Figure 6.19: SPF and Observed Crashes for U4T Intersections without LTL (Fatal and Injury Crash)

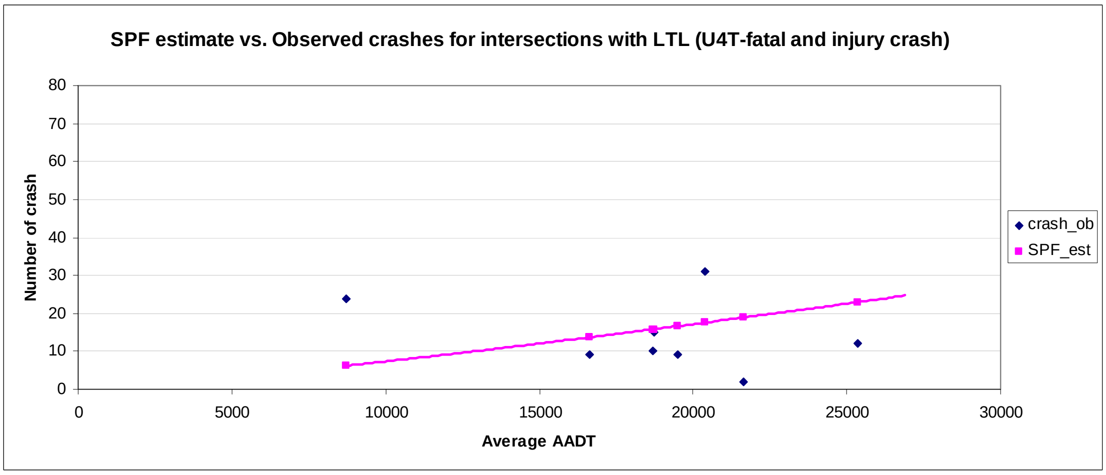


Figure 6.20: SPF and Observed Crashes for U4T Intersections with LTL (Fatal and Injury Crash)

U4T intersections:

There are more crashes occurring in intersections without left turn lanes with AADT larger than 15000. 2 intersections with left turn lanes have $P(y \geq crash\_obs) < 0.05$. Meanwhile there are no intersections with left turn lanes with AADT range between 10,000 and 15,000 due to the small sample size. Generally, adding left turn lanes has a positive safety effect for intersections with AADT higher than 15,000.

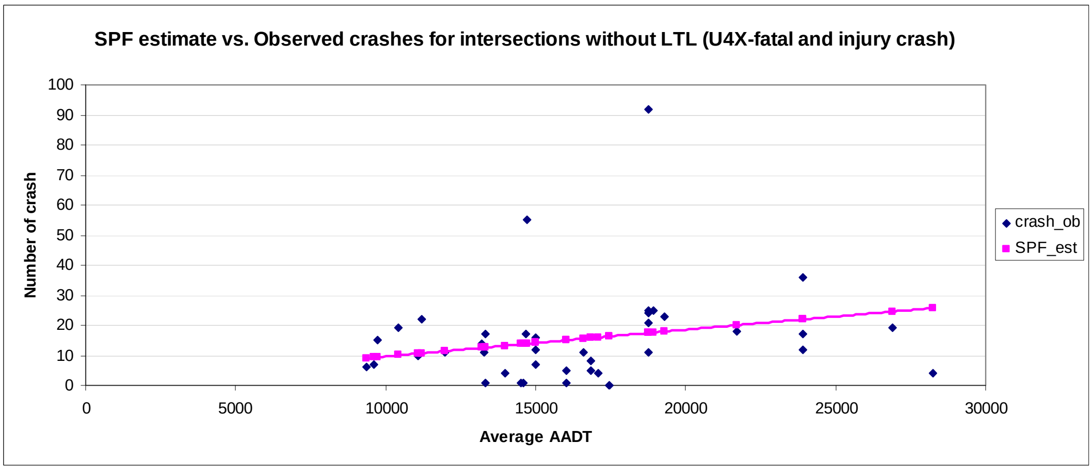


Figure 6.21: SPF and Observed Crashes for U4X Intersections without LTL (Fatal and Injury Crash)

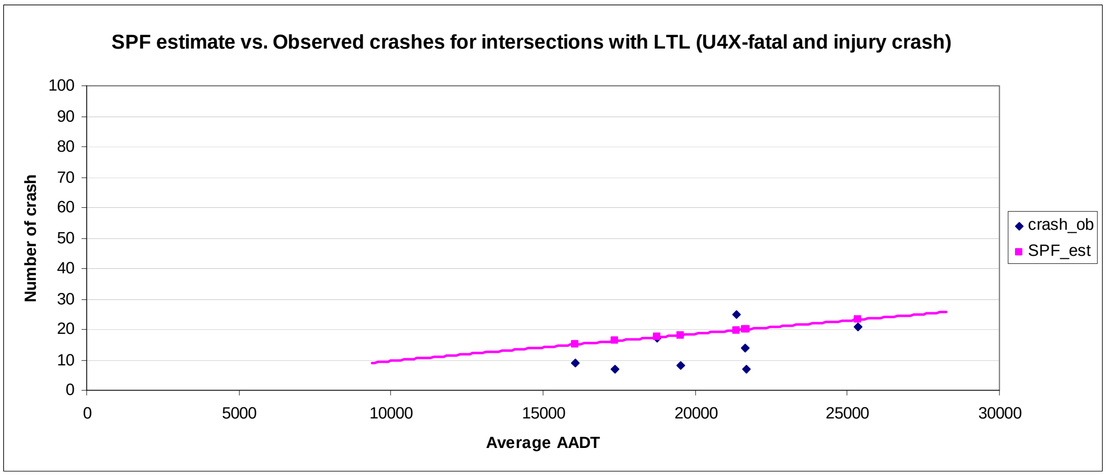


Figure 6.22: SPF and Observed Crashes for U4X Intersections with LTL (Fatal and Injury Crash)

U4X intersections:

5 intersections without left turn lanes have $P(y \geq crash\_obs) < 0.05$. It's obvious that with left turn lanes added, there is a significant decrease of crashes and we may conclude that the safety effect of left turn lanes in U4X intersections is generally positive.

Summary:

Comparing the SPF estimates in Appendix D (Table D.1 to D.6), intersections detected as more dangerous for the occurrence of same direction crashes are not necessarily more dangerous as far as fatal and injury crashes are concerned, especially for the R2T and R2X categories.

The results are summarized in Table 6.2. The percentages of dangerous intersections decrease for the R2T, R2X and U4X categories by at least 10% when left turn lanes are added, while it increases for the U2X and U4T categories by 11% and 5% respectively. The percentages of much safer intersections increase for R2T and U4X intersections with left turn lanes. Combined with the conclusion obtained from section 6.1.1, adding left turn lanes can reduce both same direction crashes and fatal and injury intersections in R2T, R2X, U4T and U4X intersections. The effects are more significant for same direction crashes than for fatal and injury crashes.

Table 6.2: Summary for fatal and injury crashes

	w/o LTL				w/ LTL				
	# of int.	P(y≥crash_ob)<0.05	%	P(y≤crash_ob)<0.05	%	# of int.	P(y≥crash_ob)<0.05	%	P(y≤crash_ob)<0.05	%
R2T	50	6	12%	7	14%	13	0	0%	7	54%
R2X	50	5	10%	1	2%	3	0	0%	0	0%
U2X	50	7	14%	8	16%	4	1	25%	0	0%
U4T	50	10	20%	21	42%	8	2	25%	3	38%
U4X	40	5	13%	14	35%	8	0	0%	3	38%

6.2 Assessment of Physical Characteristics

Intersections with left turn lanes yet have $P(y \geq crash\_obs) < 0.05$ for either same direction crashes or fatal and injury crashes were further examined for their physical characteristics. The photolog images of these intersections, showing the geometric features as well as the surrounding environment, were displayed in Appendix E (Figures E.1 to E.9). We are aiming to find out whether it is the left turn lane installation, or the inherent geometric features of the intersection, or the combination of the two that causes the intersections to be more dangerous than others.

Six intersections have $P(y \geq crash\_obs) < 0.05$ for same direction crashes. R2TY12, the only intersection detected to be more dangerous in the rural area, is on a curve with limited sight distance. U2TY02, just close to anther 4-leg intersections with a left turn lane, has a very short taper length as well as the storage length for the left turn lane, which may cause problem when the left turn vehicles do not have enough deceleration distance. U2TY12 is also close to another intersection with a left turn lane. A relatively high percentage of left turn vehicles and vehicles frequently coming from the minor approach make this intersection more complex. U2TY17 is on a curve, with no taper yet very short length of the left turn lane. Besides, vehicles on the minor approach have limited sight distance for the vehicles heading north on major road. U2TY18 is close to a number of driveways within the study area of the intersections. Vehicles to and from the driveways interfere with the traffic on the major roads, increasing the probability of crash occurrence. Similar for U2XY04, along the major road within the study area of the intersection exist a number of driveways with considerable traffic volume. As explained previously, it is possible that the driveways rather than the left turn lanes make this area more crash-prone.

Three intersections have $P(y \geq crash\_obs) < 0.05$ for fatal and injury crashes. U2XY02 provides very short length of taper and storage for the left turn lane. U4TY02 is on a curve and also is very close to other intersections, resulting in more vehicle interactions on the study area. U4TY08 is an unusual site in this study for it has two way left turn lanes installed close to the study intersection. It is likely this special feature also has effects on the crash rate at the study area of this intersection.

Based on the above examination, of these more dangerous intersections, some have curves at the intersections causing insufficient sight distance, some are close to driveways with considerable traffic, especially in urban areas, and some have inappropriate design of the left turn lanes, such as insufficient length of the taper and storage lane, which often appears when two intersections are close to each other leaving short distance for the left turn lane.

In the former two cases, the analysis of the effect of left turn lanes on crashes becomes more complex since these features and factors may also contribute to the high crash rate. The effects are mixed and hard to separate, which leave these cases to be inconclusive as far as the safety effect of left turn lanes are concerned.

However, the above examination also suggests that if the left turn lanes were designed inappropriately, which is the third case, they definitely have negative impact on the safety of the intersection.

Due to the small number of intersections detected, nine in total, and lack of other geometric data in detail, we are unable to conduct a systematic analysis of these physical characteristics on safety of an intersection. However, the review on each intersection individually also provides some insight to complement our previous study.

7 CONCLUSIONS AND FUTURE RESEARCH DIRECTIONS

7.1 Development of Volume Warrants

In the first part of the this study, a DSS for predicting the likely benefits of left-turn installations at unsignalized intersections was developed for each of the road categories such as urban two-lane, rural two-lane and urban four-lane. The DSS's are based on the use of NN's to generalize the results from microscopic simulation models that were carefully calibrated to reflect real-world conditions. Given the different volume combinations such as opposing, advancing and left turning volumes, and the operating speed, the DSS will predict the benefits of installation of left turn lane in terms of the following:

- Total savings in the delay;
- Reduction in the total number of stops on the subject link; and
- Increased fuel consumption efficiency.

In the second part of the study, two new sets of left turn lane warrants were developed for the different categories of roads such as urban two-lane, rural two-lane and urban four-lane. The first set of warrants uses the control delay (vehicle/sec/hour) as the warrant criteria and the second set uses the total number of stops per hour. The study also collected a lot of field data and paid special attention to carefully calibrating the simulation models used for developing the DSS and the new warrants.

Besides developing the DSS and developing new left turn lane warrants, several interesting conclusions are derived from the study, including:

- Gap acceptance behavior at unsignalized intersections differs based on the area type (*i.e.* urban, suburban or rural) in which the intersection is located.
- The CORSIM model, after calibration, does an excellent job in accurately modeling operations at unsignalized intersections. For calibration, the user should focus on adjusting the gap acceptance distribution, the discharge headway, and the entry distribution type.
- NN's are quite capable of accurately generalizing the results from microscopic simulation models when trained on a dataset of a reasonable size.
- Warrants based on performance measures such as delay and numbers of stops are higher than the traditional warrants based on probability as.
- The emissions model in CORSIM does not seem to be sensitive to the changes in the operational conditions such as change in the speed or acceleration. Further investigation into the environmental benefits (*i.e.* reductions in emissions levels) resulting from left-turn installations is warranted.

7.2 Safety Analysis

Reduction of delay and increase of capacity is often the major concern when deciding whether or not a left turn lane should be installed. However, safety issues should by no means be ignored when such decisions are made. This research analyzes the safety effect of installing left turn lanes at different locations with various features. Along with the volume based warrants, it aims to more comprehensively and better accounts for the concerns related to the installing of left turn lanes.

Some results of this research are summarized as follows:

- For intersections without left turn lanes, more crashes (same direction) are expected to occur in urban areas than in rural areas, in R2X than in R2T intersections, and in U2T than in U2X intersections, given the same level of AADT. U4T intersections have more expected crashes than U4X intersections at lower level of AADT and fewer at higher level.
- Results are similar for fatal and injury crashes, more expected crashes for intersections in urban areas than in rural areas and in R2X than in R2T intersections. Besides, the number of crashes expected for U4T and U4X intersections is much greater than that for any other categories of intersections. Compared with conclusion (1), more crashes are expected for U4T than U4X intersections at high level of AADT. In other words, there are fewer same direction crashes but more fatal and injury crashes for U4T than for U4X intersections when AADT is high.
- It is safer for intersections to install left turn lanes if only the same direction crashes, including rear-end, turning-same direction and sideswipe-same direction, are concerned. Installing left turn lanes can effectively reduce the number of such types of crashes at unsignalized intersections. The exception is U2X intersections, for which adding left turn lanes does not seem to improve safety. However, whether or not other features other than the existence of left turn lanes in these intersections contribute to the high crash rate is unclear.

- Installing left turn lanes dramatically reduces fatal and injury crashes at intersections in rural areas. However, reduction of such crashes in urban areas is less significant except for U4X intersections, for which a positive effect of left turn lanes is very strong.
- Improper design of left turn lanes results in more dangerous intersections. In such cases, crashes are expected to be reduced when the design is improved. At intersections with such features as located on curves, close to a number of driveways, etc., the effect of left turn lanes cannot be easily separated from the impact of these features. Further study is required.

As far as future research is concerned, the following is suggested:

- Before and after study can be conducted when data is available, by which all of the other features other than the existence of left turn lanes can be controlled the same for each intersection in the before and after period. Thus the effect of left turn lanes can be separated from that of the other features, which may change from one another when comparison sites are used.
- When data permits, more variables, such as traffic volume of the intersecting minor roads, percentage of left turn traffic, average speed of vehicles traveling, number of driveways near the intersection, may be included in the model. This is particularly necessary for analyzing the crash categories 2 and 3 (intersecting direction and segment related crashes) as well as fatal and injury crashes.
- If more intersections with left turn lanes can be found and thus more may be detected to be dangerous, research can be then conducted on these dangerous intersections to decide which factors, left turn lane related as well as others, cause the intersection to be more prone to crash occurrence. Also, comparison can be made between these dangerous intersections and the safe intersections identified by the SPF functions.
- All of the geometric characteristics of the intersections, along with the precise application of traffic control devices used, including pavement markings and signage, as well as lane and pavement width can be carefully examined to provide guidelines for how to physically design and control exclusive left turn lanes to maximize safety for all road users.

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Appendix A: Left Turn Lane Warrants

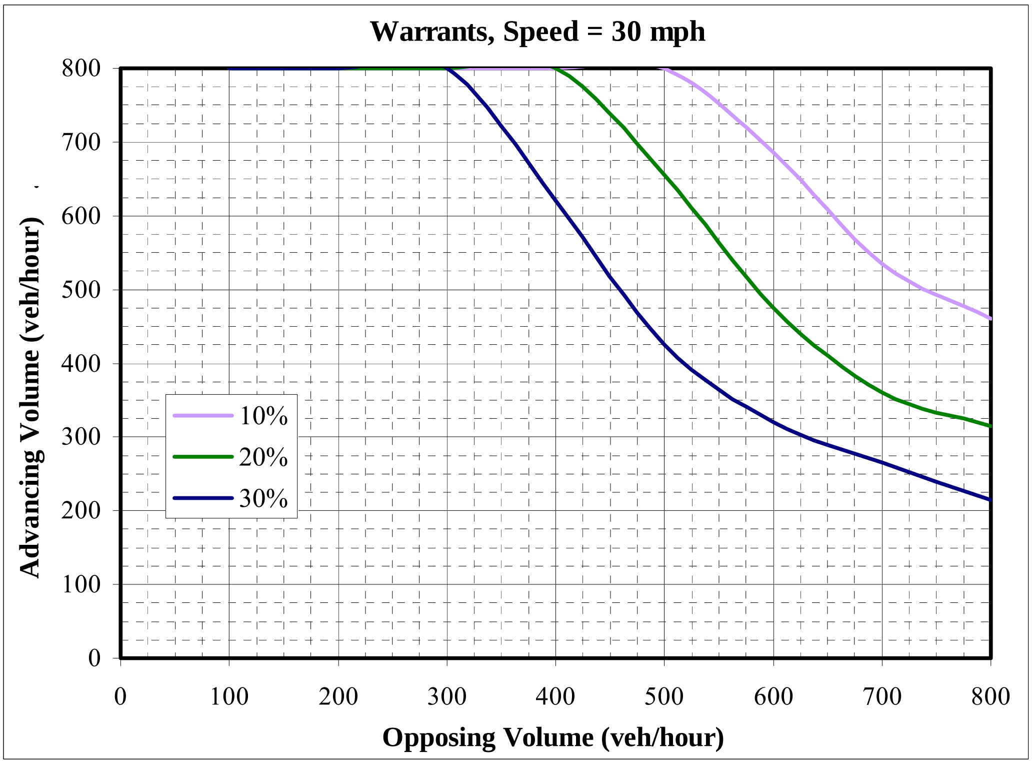

Figure A.1: Rural two-lane, Operating speed = 30, Warrants based on Total Delay (sec.veh/hour) on the subject link

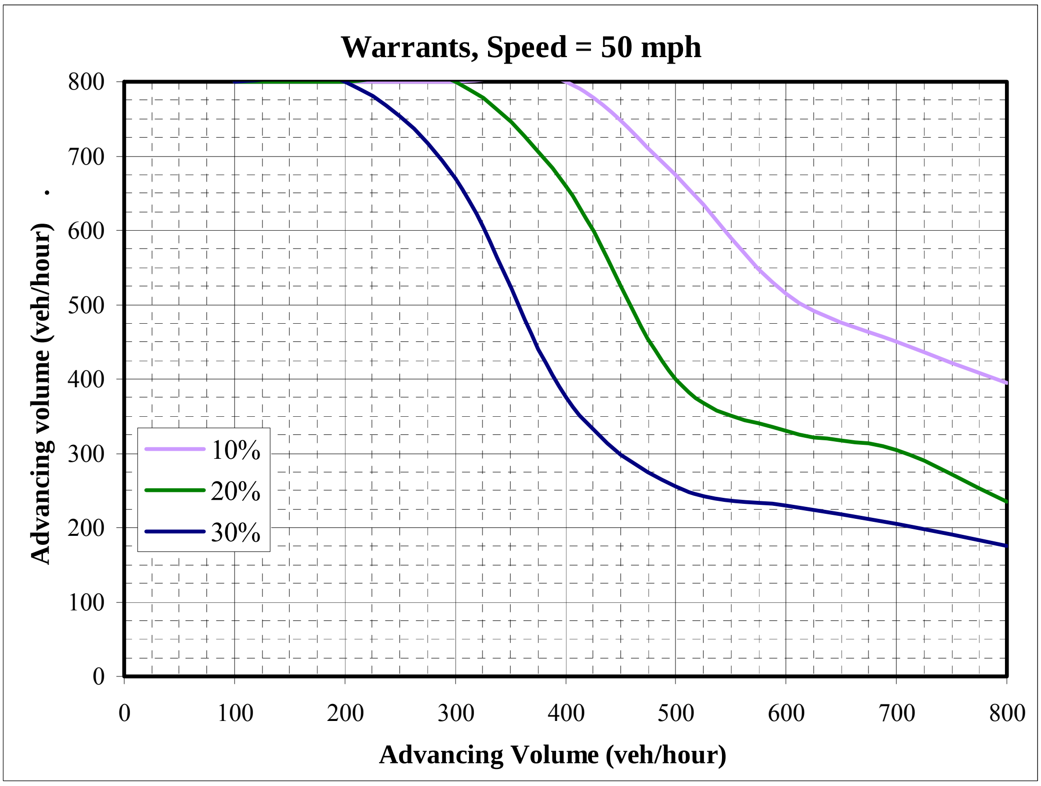

Figure A.2: Rural two-lane, Operating speed = 50, Warrants based on Total Delay (sec.veh/hour) on the subject link

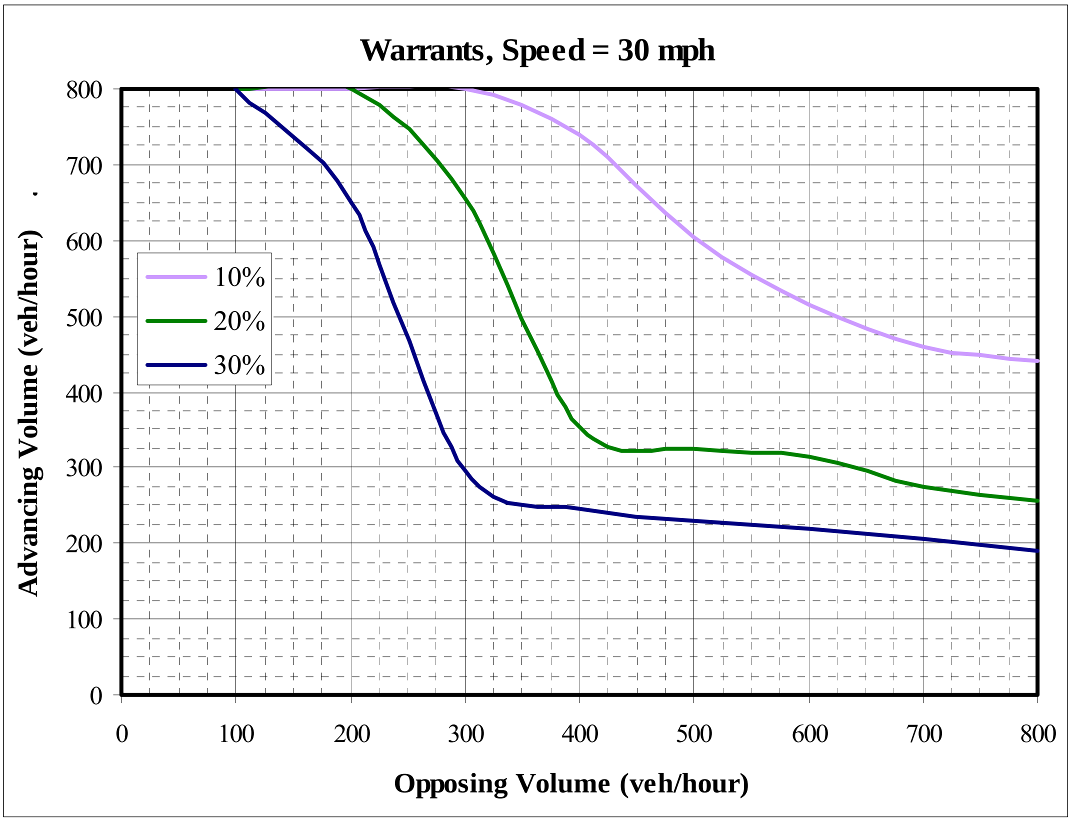

Figure A.3: Rural two-lane, Operating speed = 30, Warrants based on Total number of stops (number) on the subject link

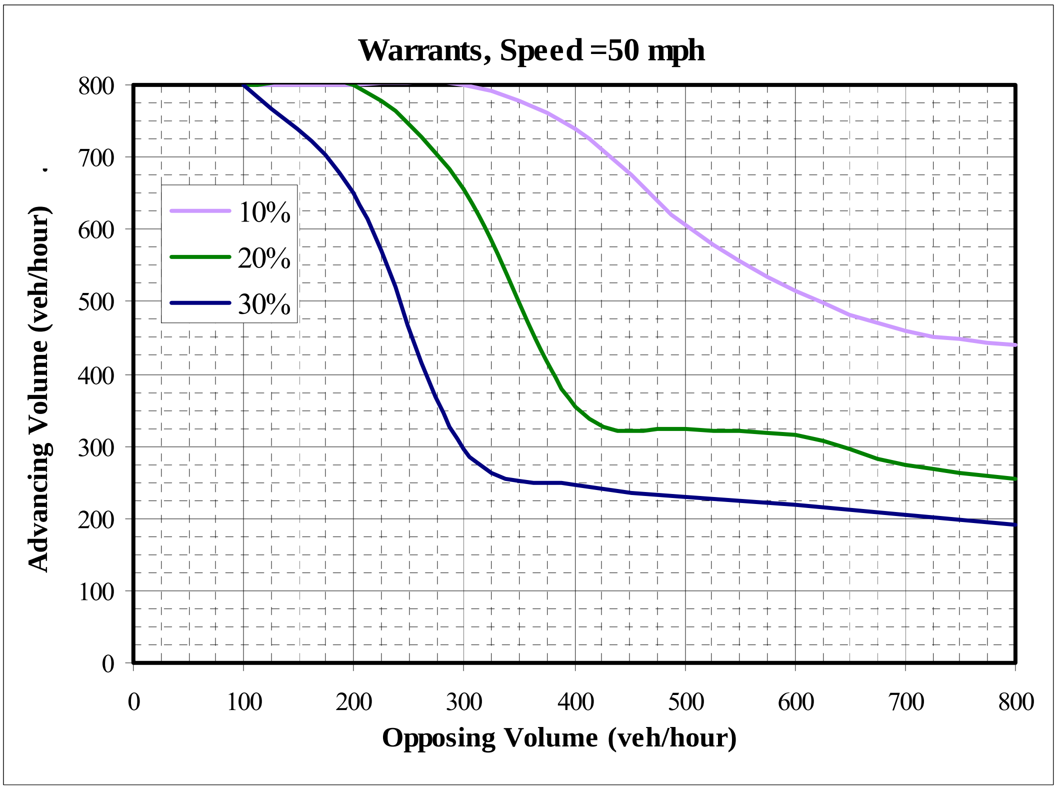

Figure A.4: Rural two-lane, Operating speed = 50, Warrants based on Total number of stops (number) on the subject link

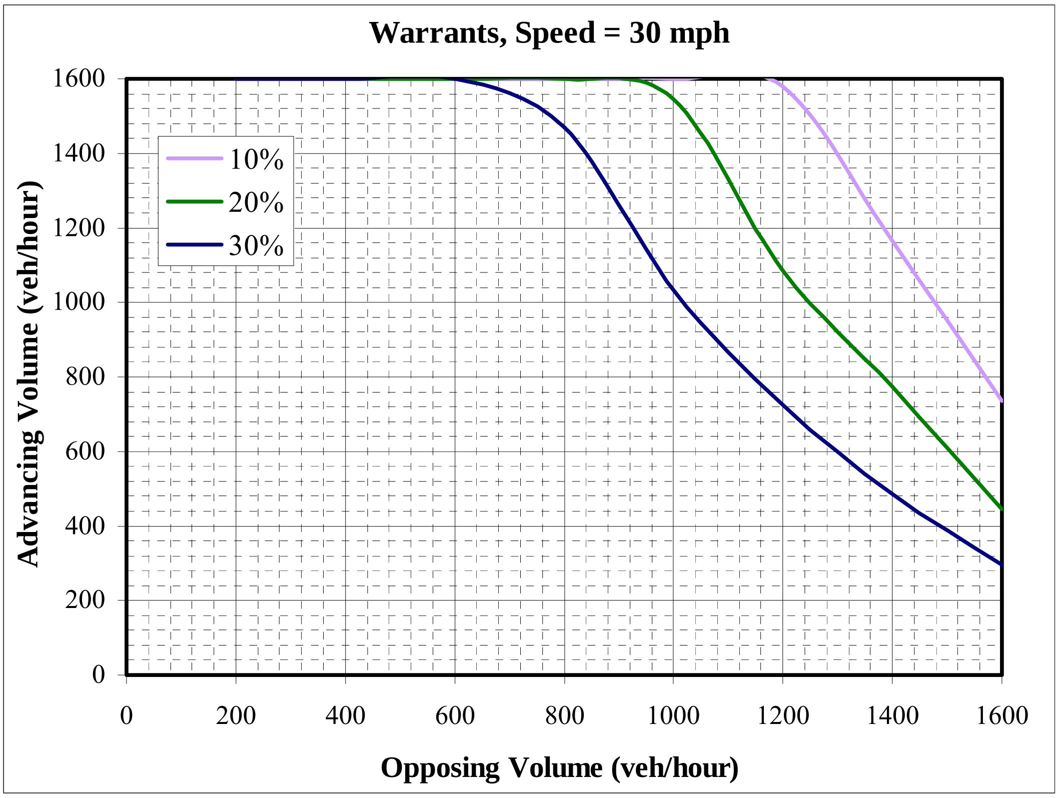

Figure A.5: Urban four-lane, Operating speed = 30, Warrants based on Total Delay (veh.sec/hour) on the subject link

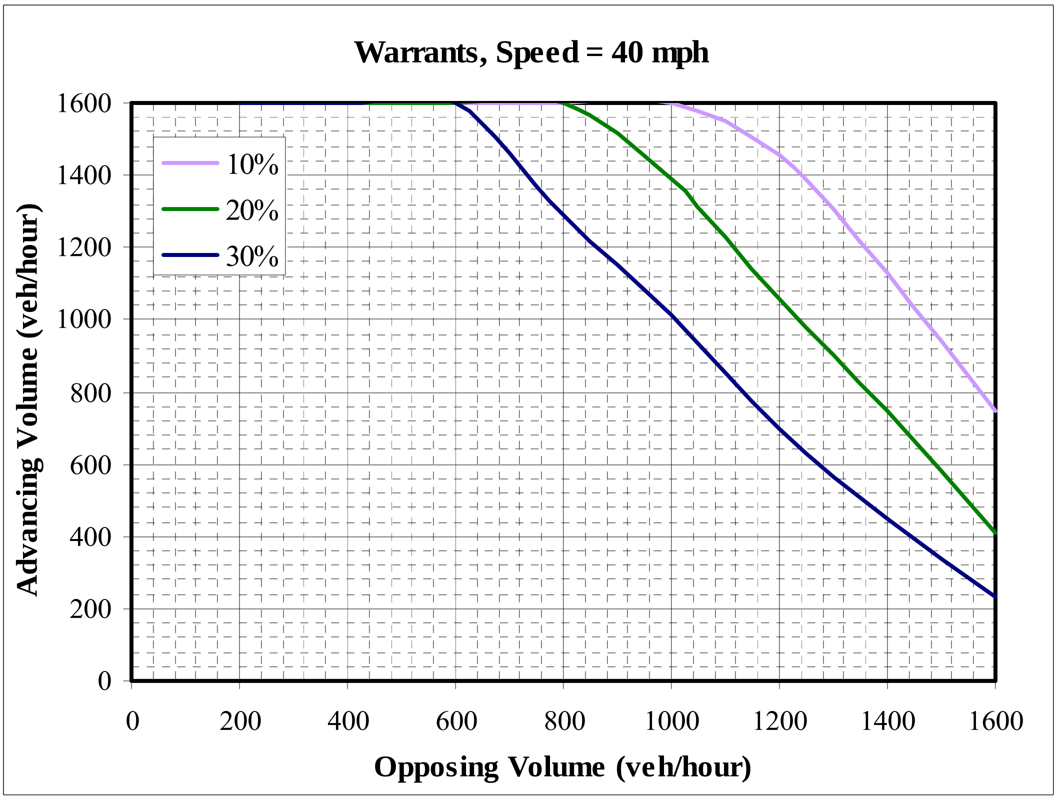

Figure A.6: Urban four-lane, Operating speed = 40, Warrants based on Total Delay (veh.sec/hour) on the subject link

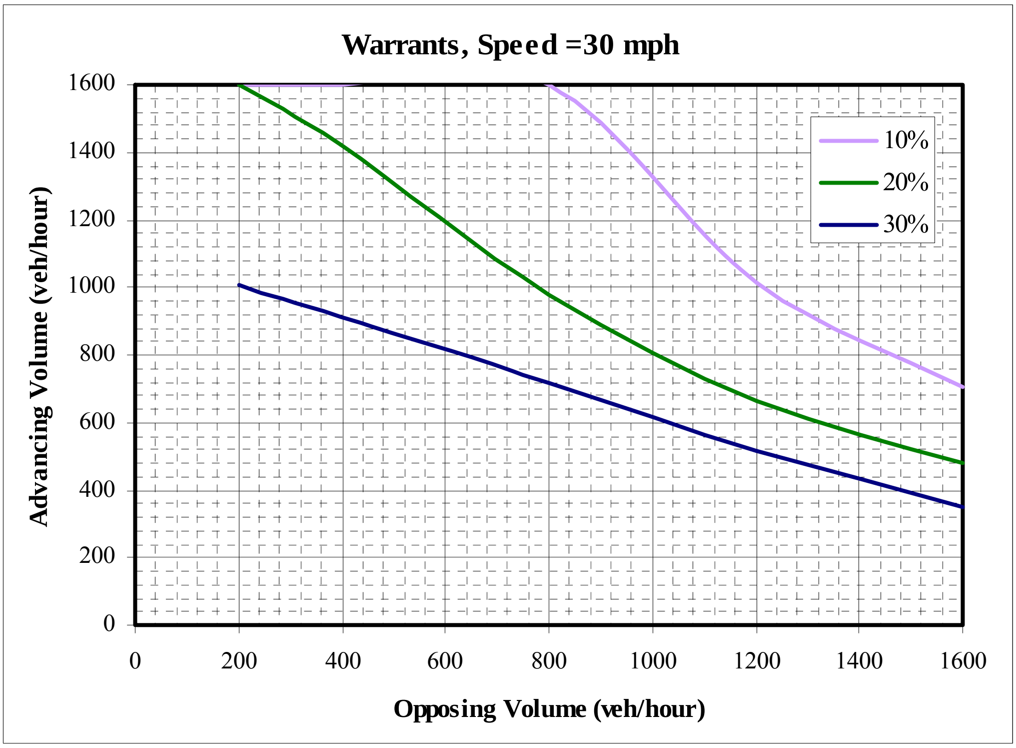

Figure A.7: Urban four-lane, Operating speed = 30, Warrants based on Total number of stops (number) on the subject link

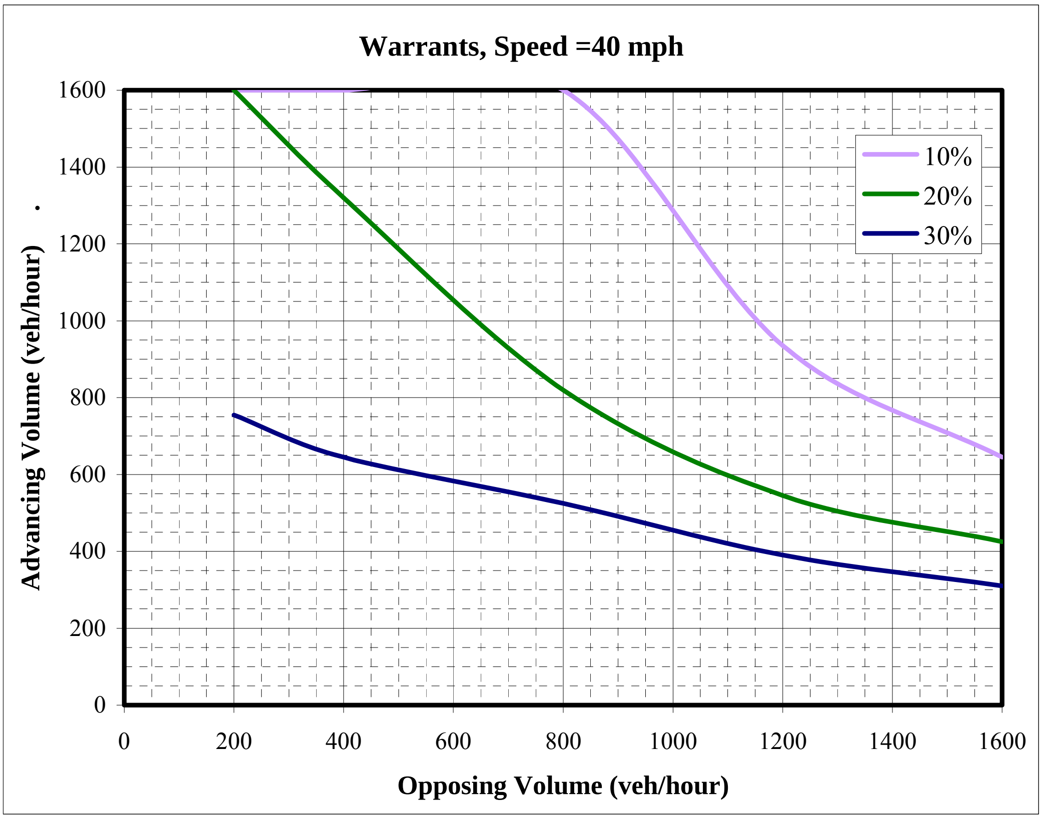

Figure A.8: Urban four-lane, Operating speed = 40, Warrants based on Total number of stops (number) on the subject link

Appendix B: Benefits of Left Turn Lane Installation

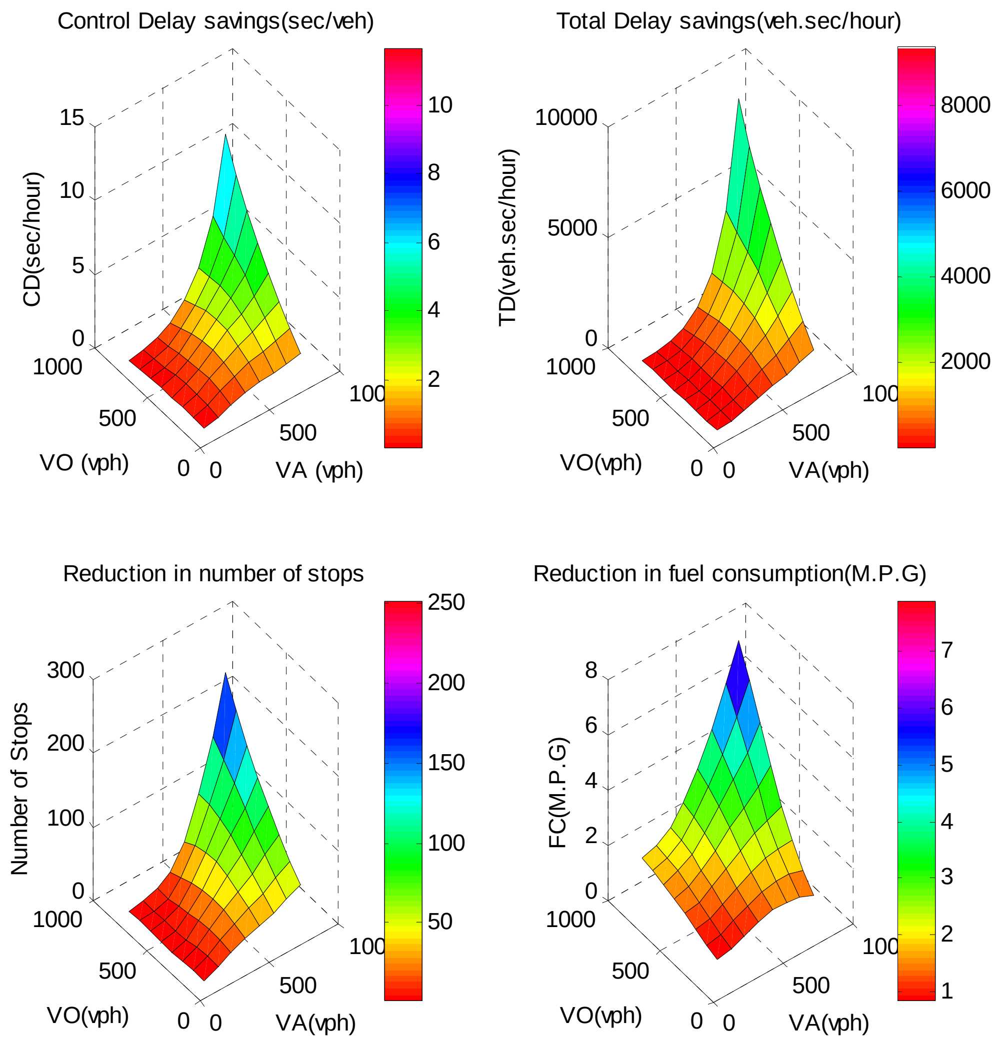

Figure B.1: Urban two-lane category, operating speed = 30mph, Left turn Percent = 10%

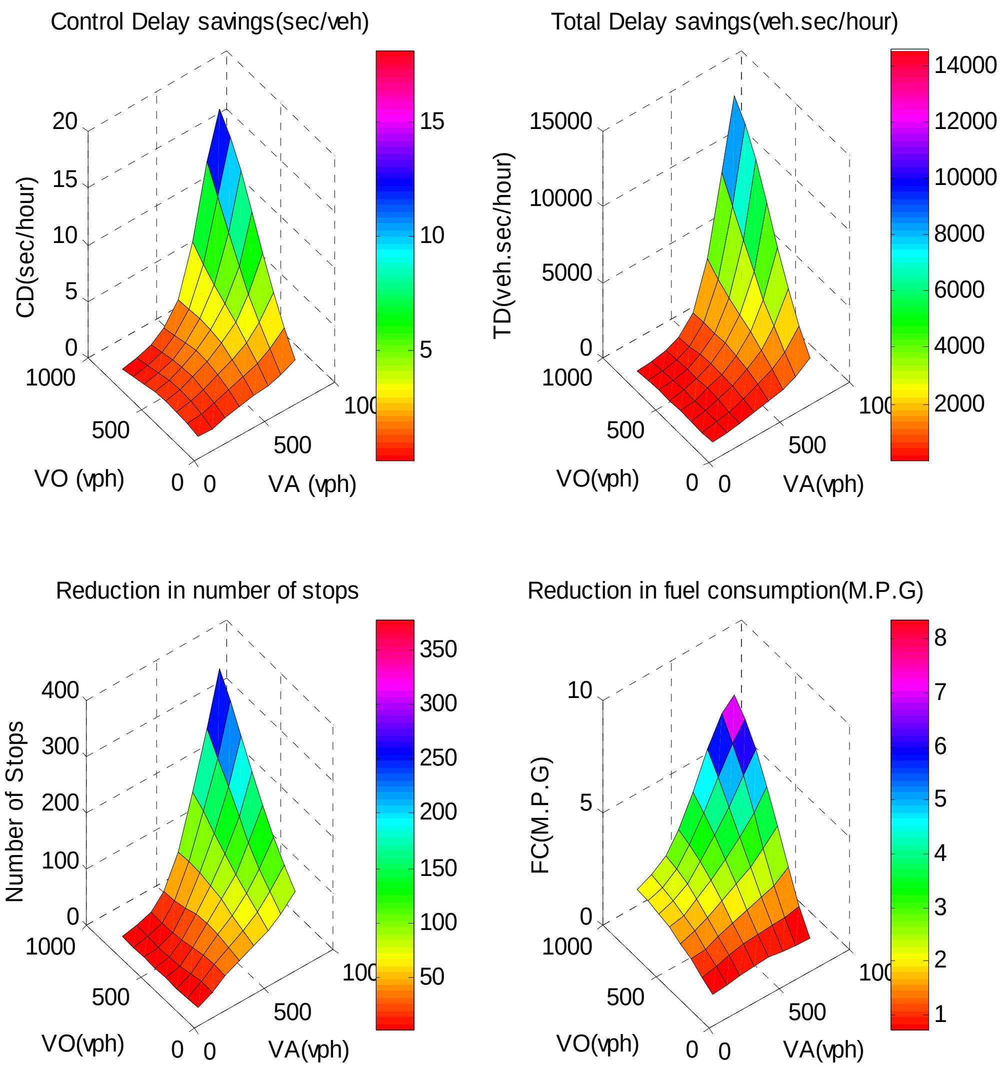

Figure B.2: Urban two-lane category, operating speed = 30mph, Left turn Percent = 20%

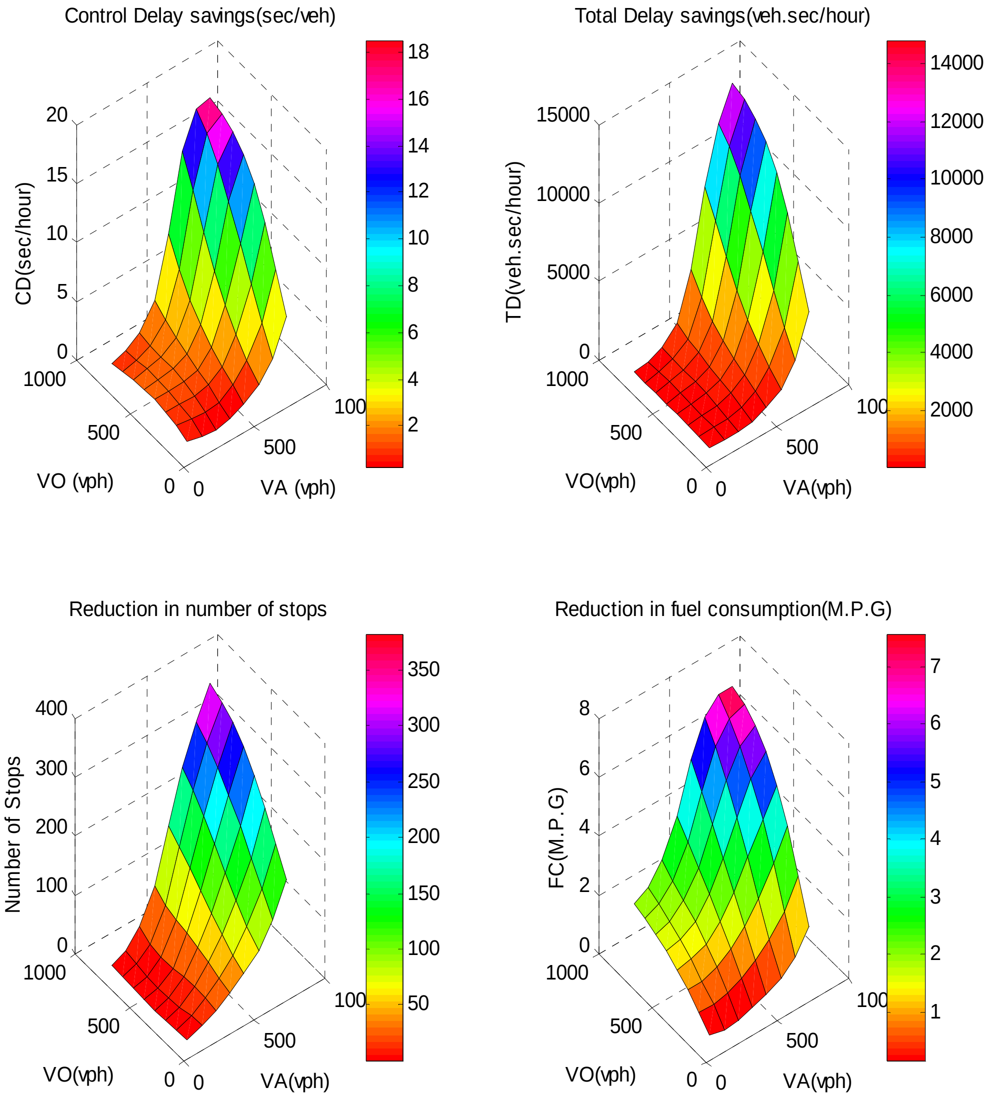

Figure B.3: Urban two-lane category, operating speed = 30mph, Left turn Percent = 30%

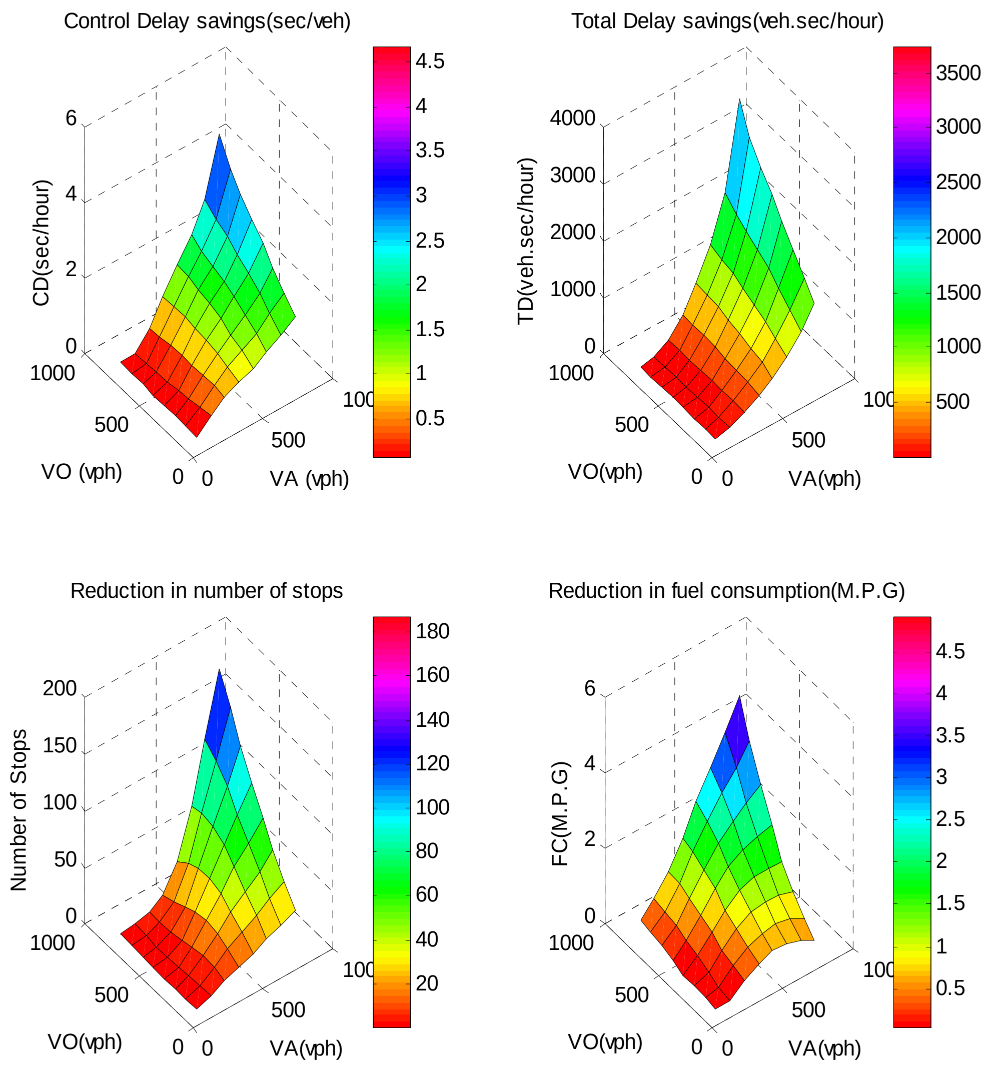

Figure B.4: Urban two-lane category, operating speed = 40mph, Left turn Percent = 10%

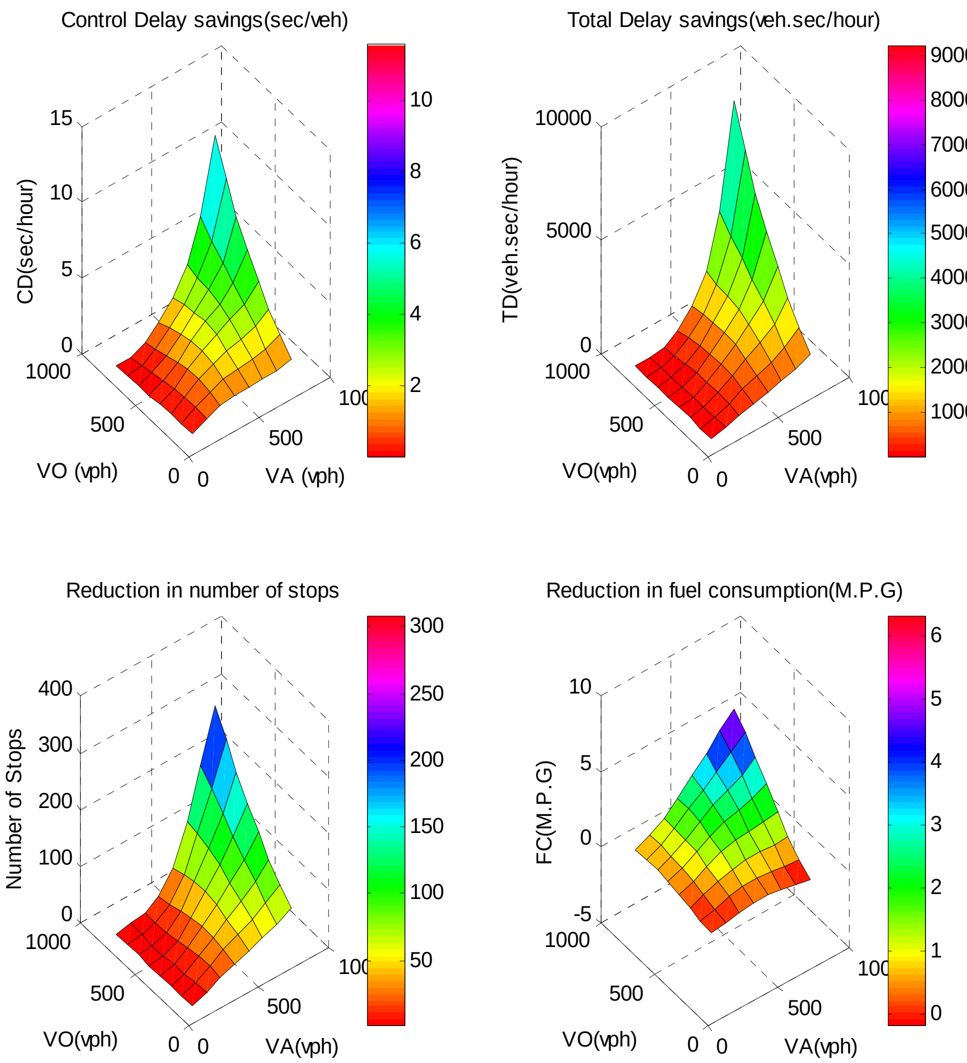

Figure B.5: Urban two-lane category, operating speed = 40mph, Left turn Percent = 20%

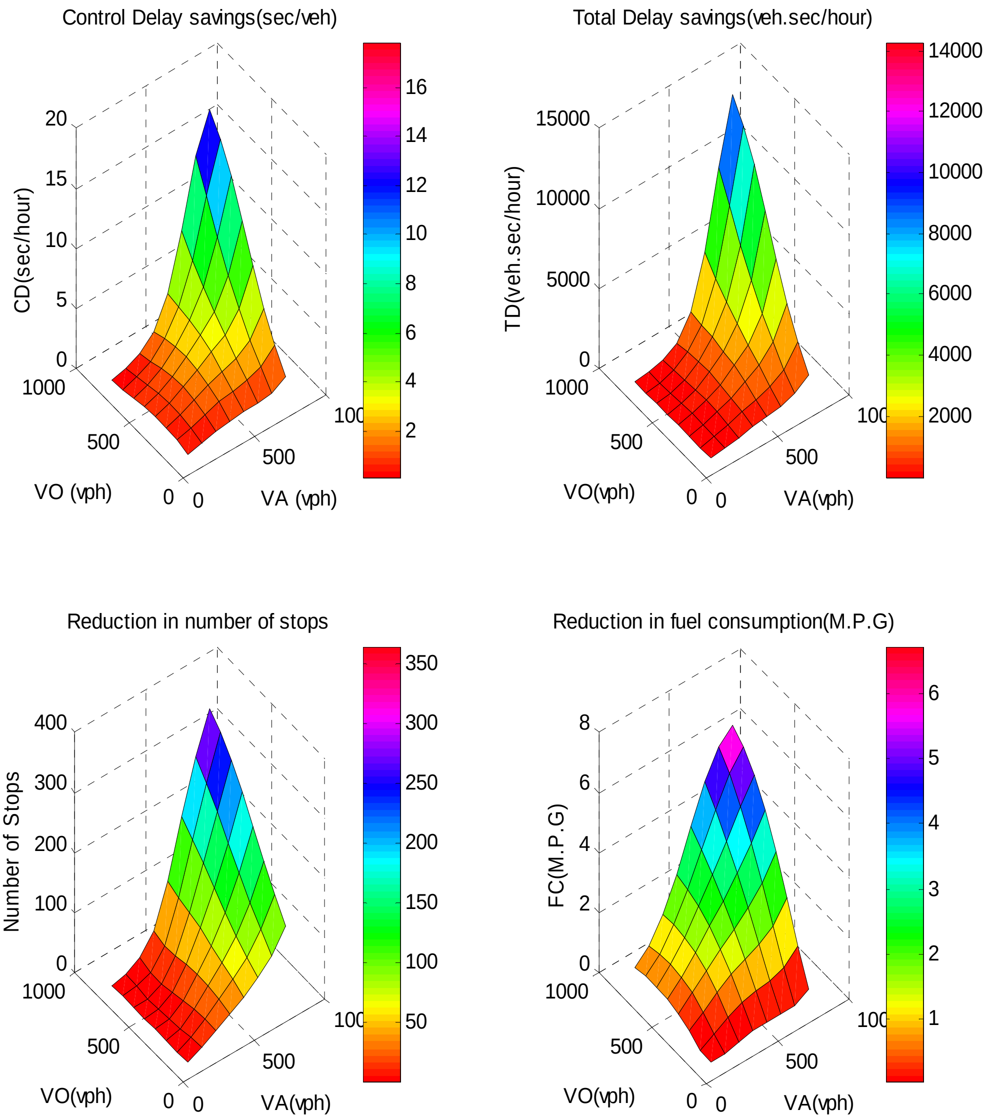

Figure B.6: Urban two-lane category, operating speed = 40mph, Left turn Percent = 30%

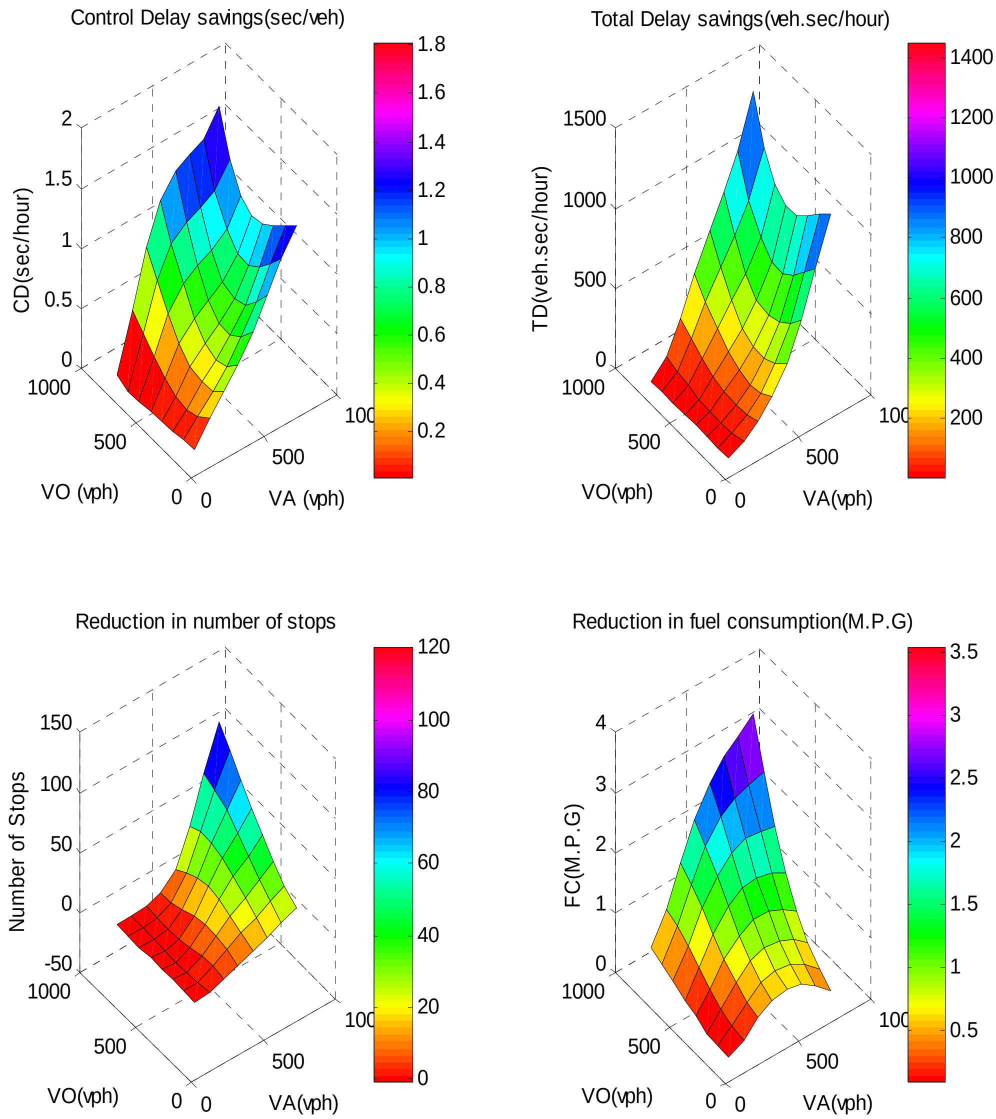

Figure B.7: Urban two-lane category, operating speed = 50mph, Left turn Percent = 10%

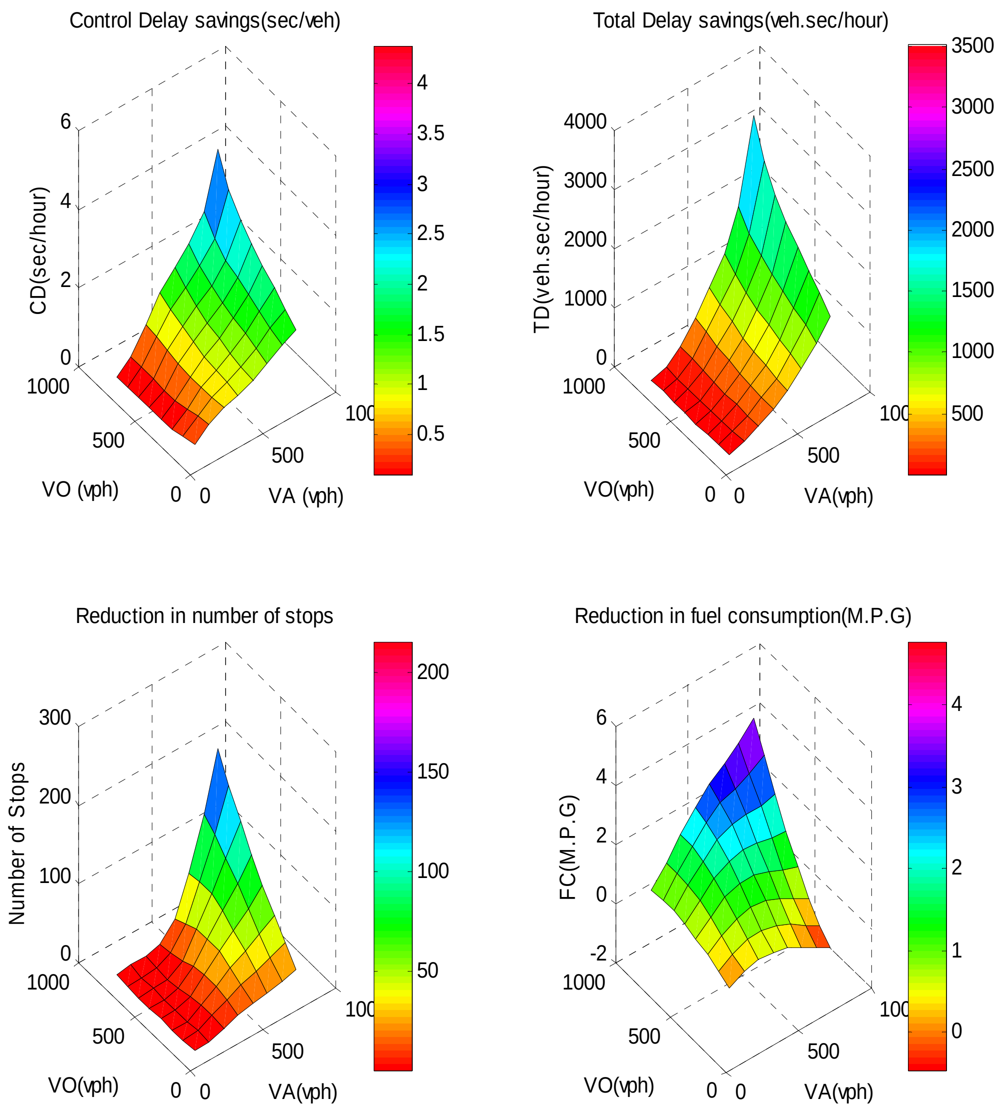

Figure B.8: Urban two-lane category, operating speed = 50mph, Left turn Percent = 20%

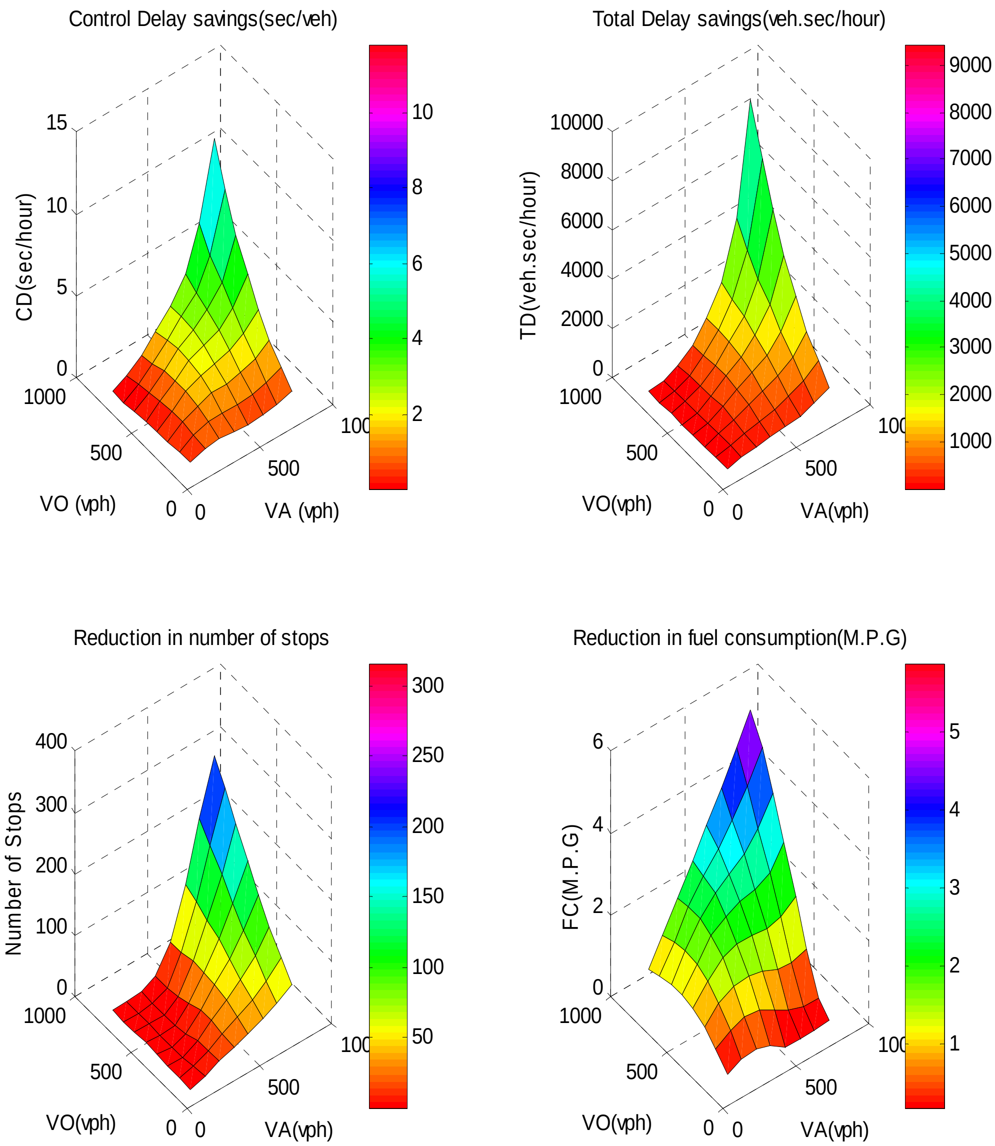

Figure B.9: Urban two-lane category, operating speed = 50mph, Left turn Percent = 30%

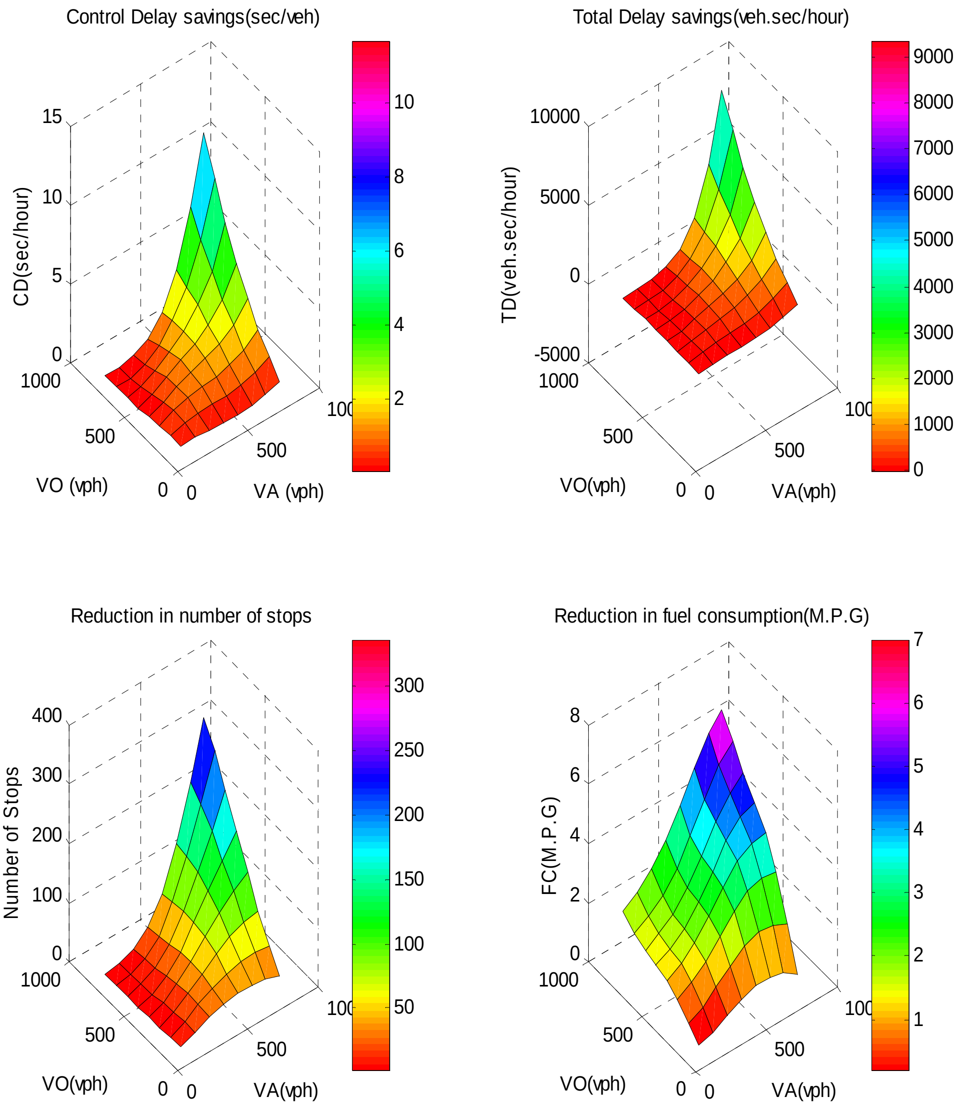

Figure B.10: Rural two-lane category, operating speed = 30mph, Left turn Percent = 10%

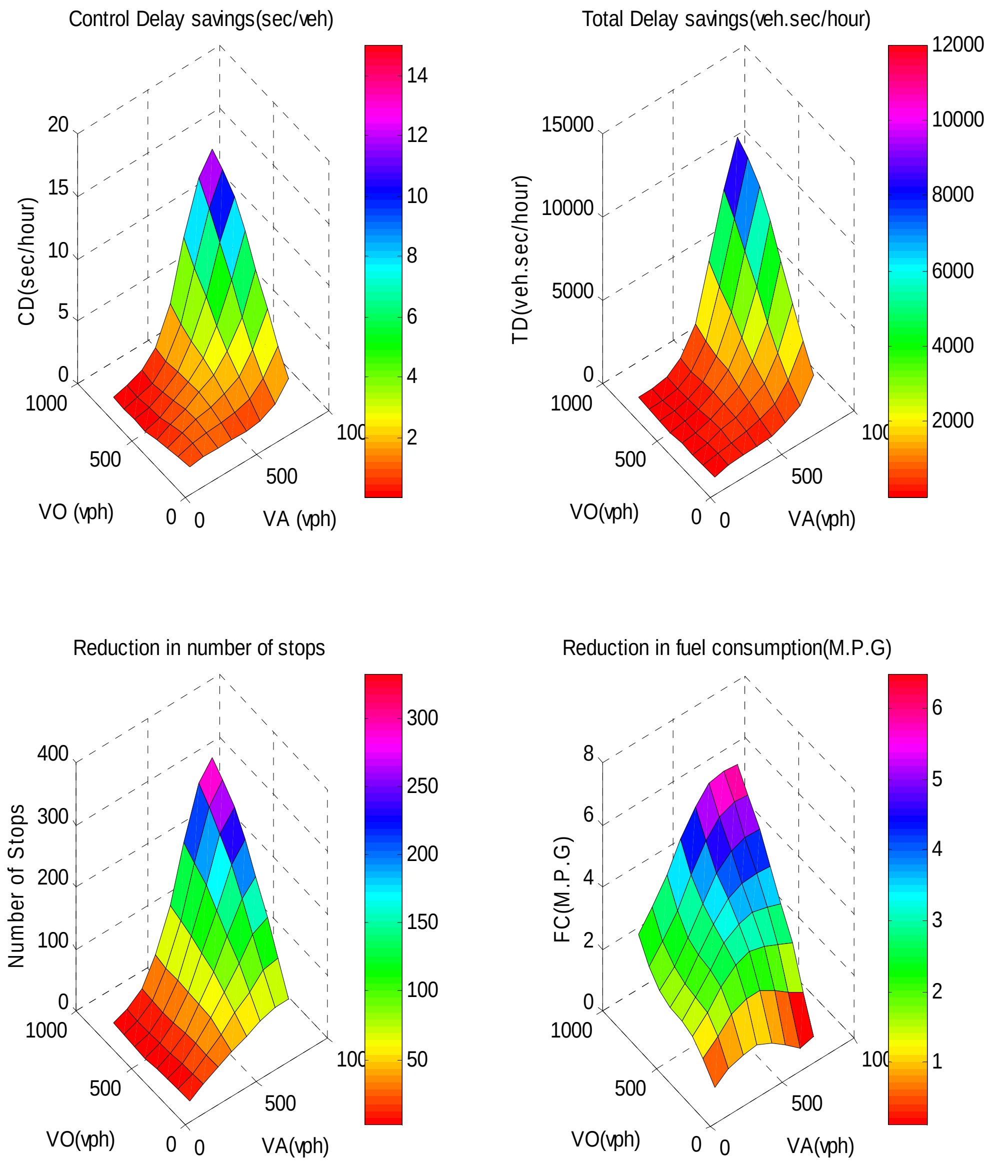

Figure B. 11: Rural two-lane category, operating speed = 30mph, Left turn Percent = 20%

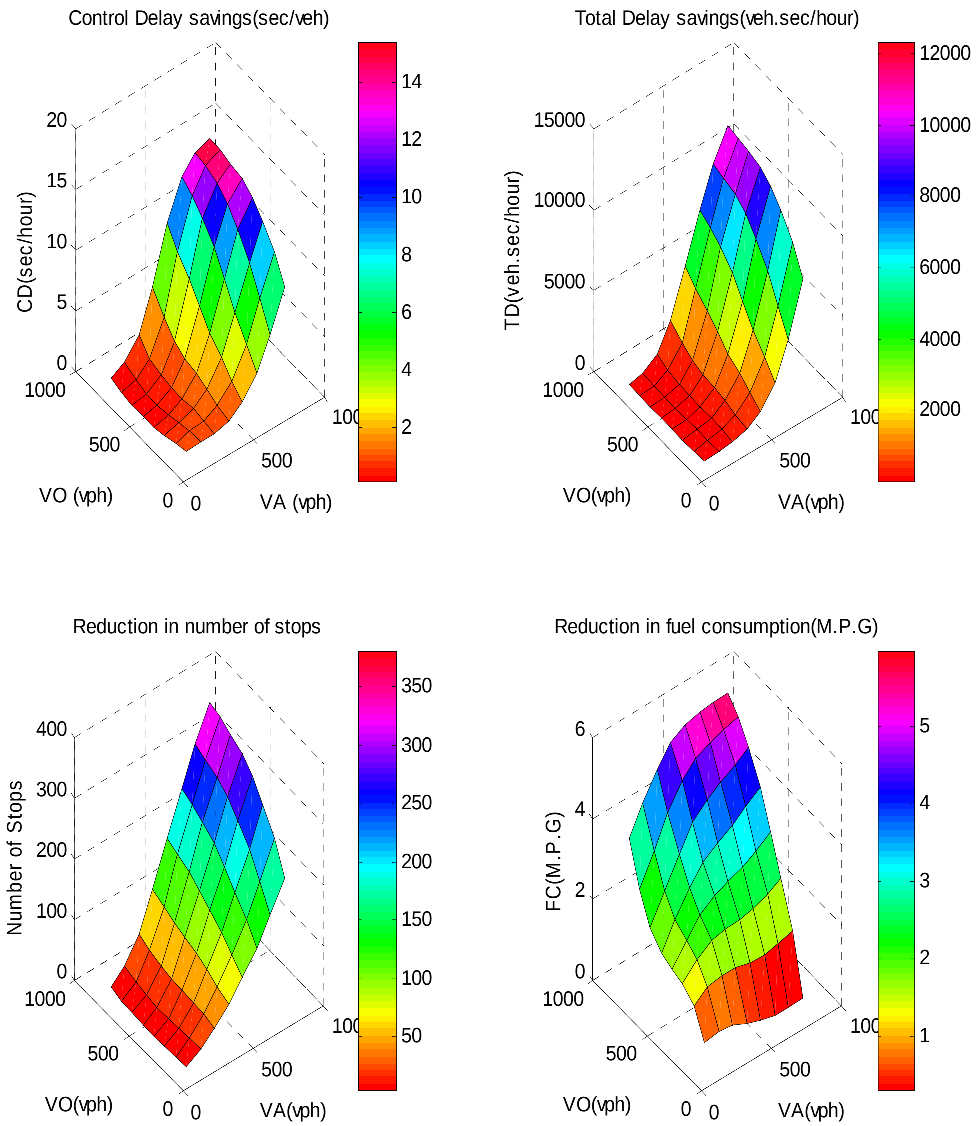

Figure B.12: Rural two-lane category, operating speed = 30mph, Left turn Percent = 30%

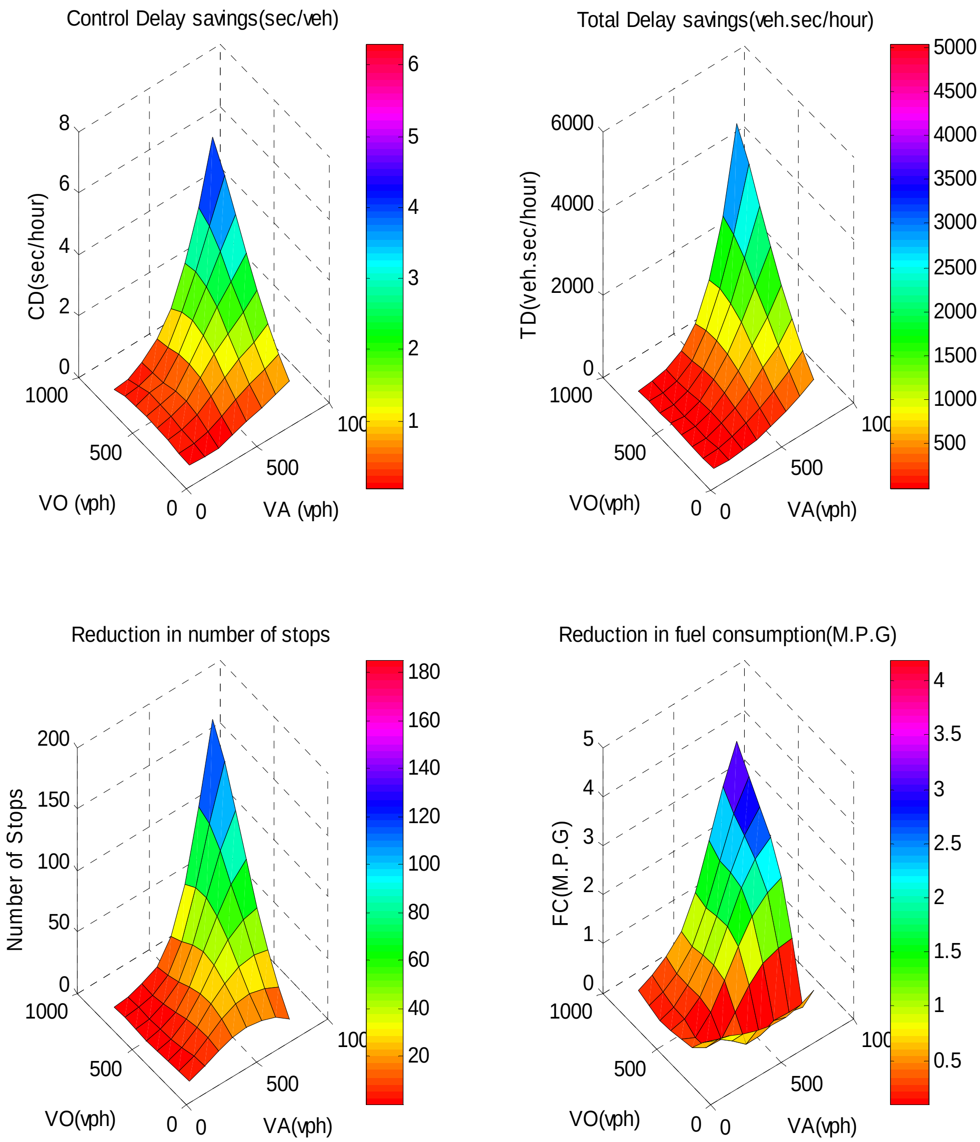

Figure B.13: Rural two-lane category, operating speed = 40mph, Left turn Percent = 10%

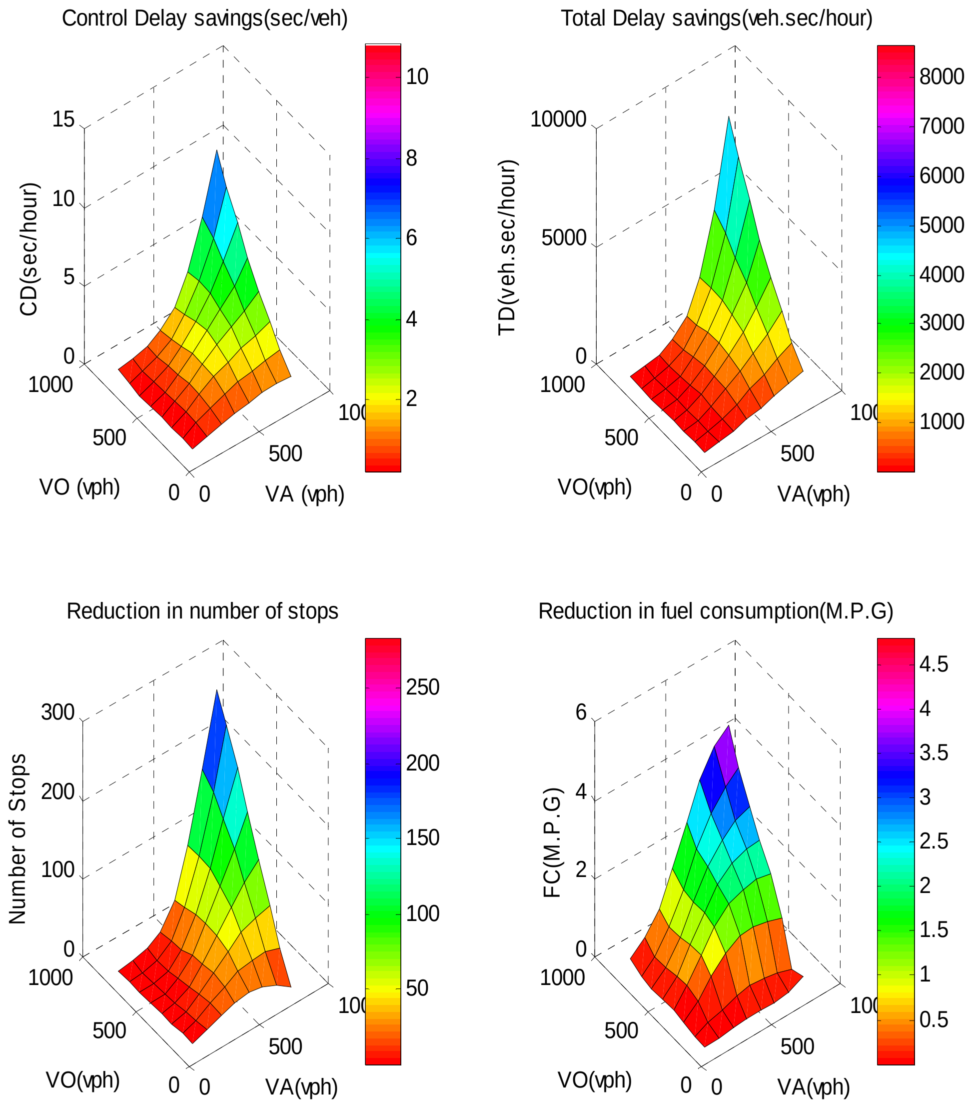

Figure B.14: Rural two-lane category, operating speed = 40mph, Left turn Percent = 20%

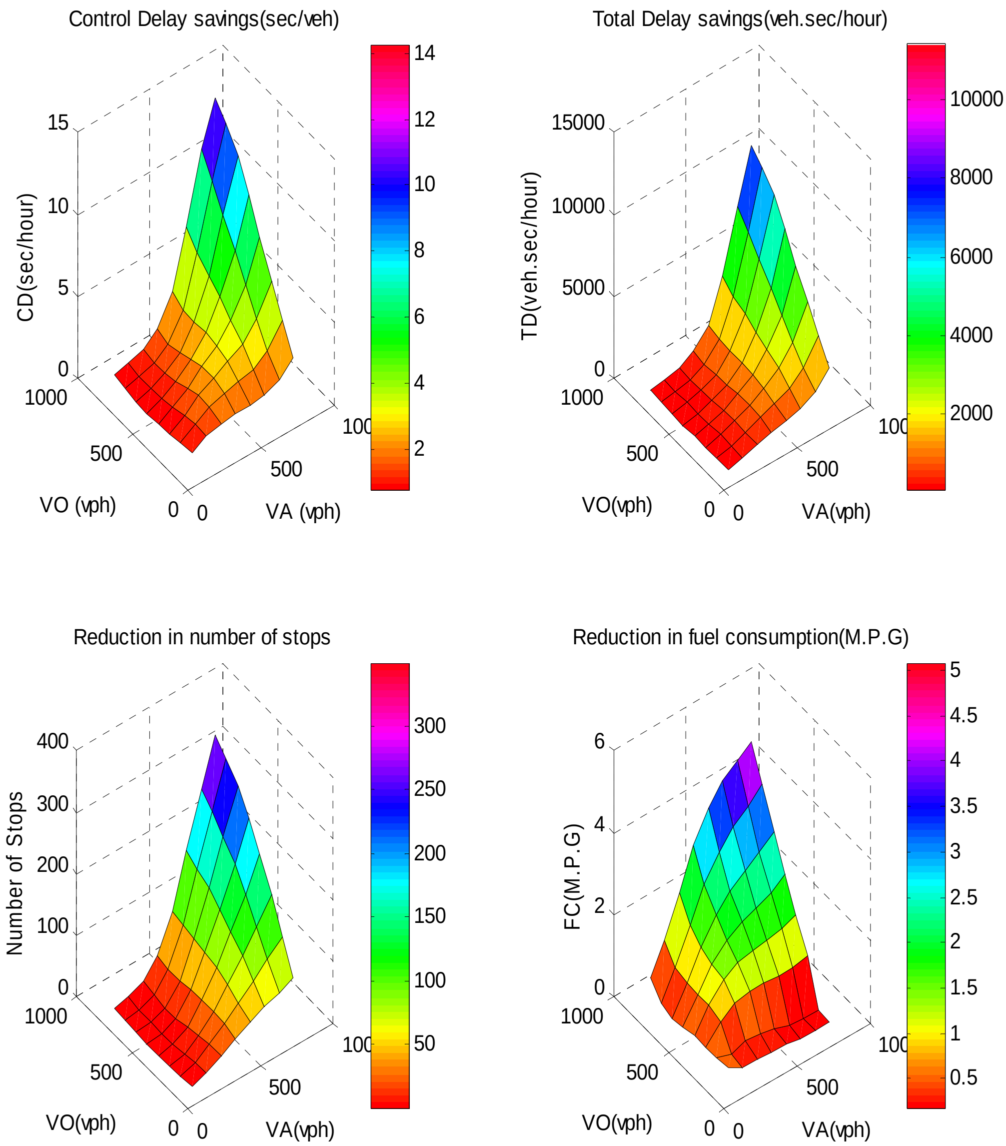

Figure B.15: Rural two-lane category, operating speed = 40mph, Left turn Percent = 30%

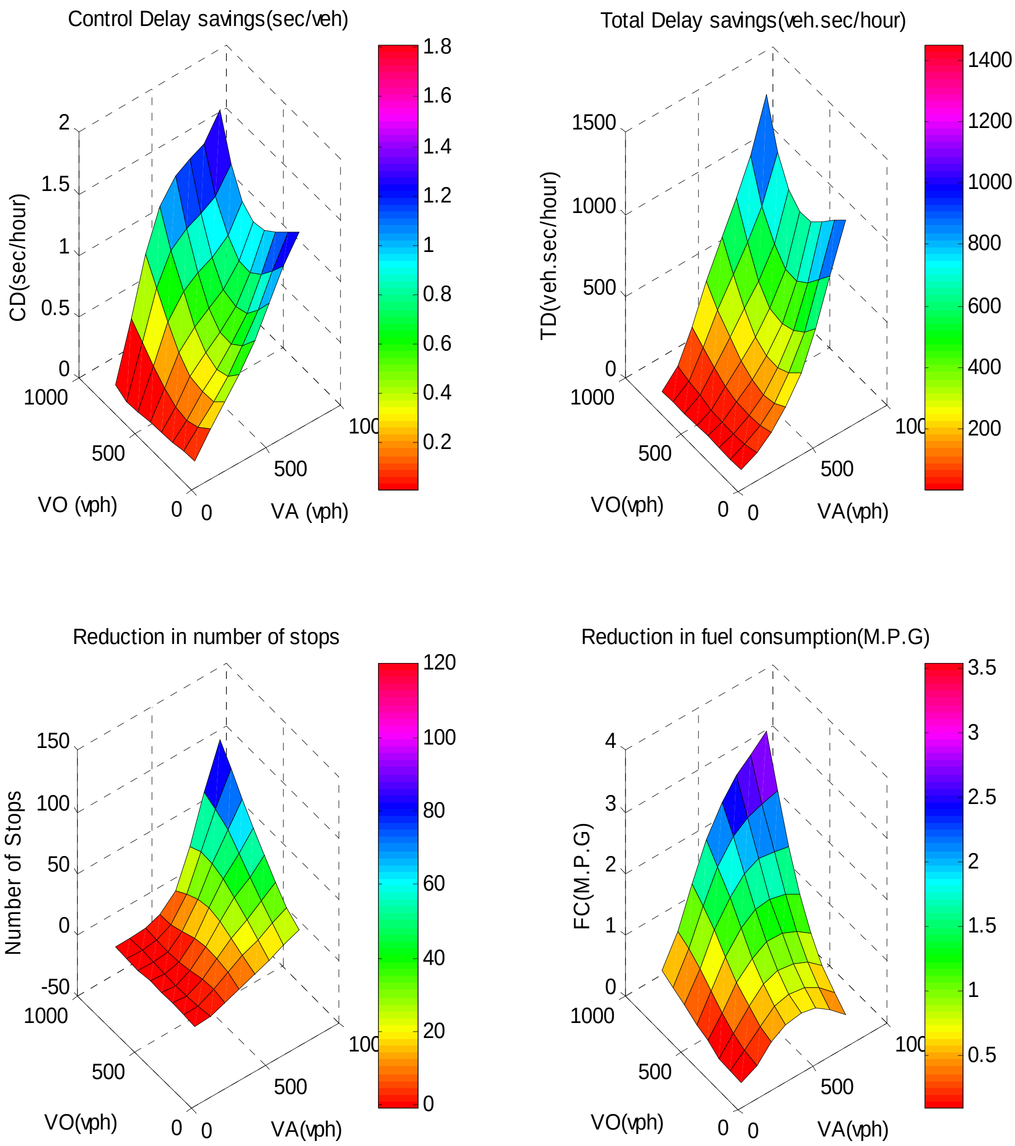

Figure B.16: Rural two-lane category, operating speed = 50mph, Left turn Percent = 10%

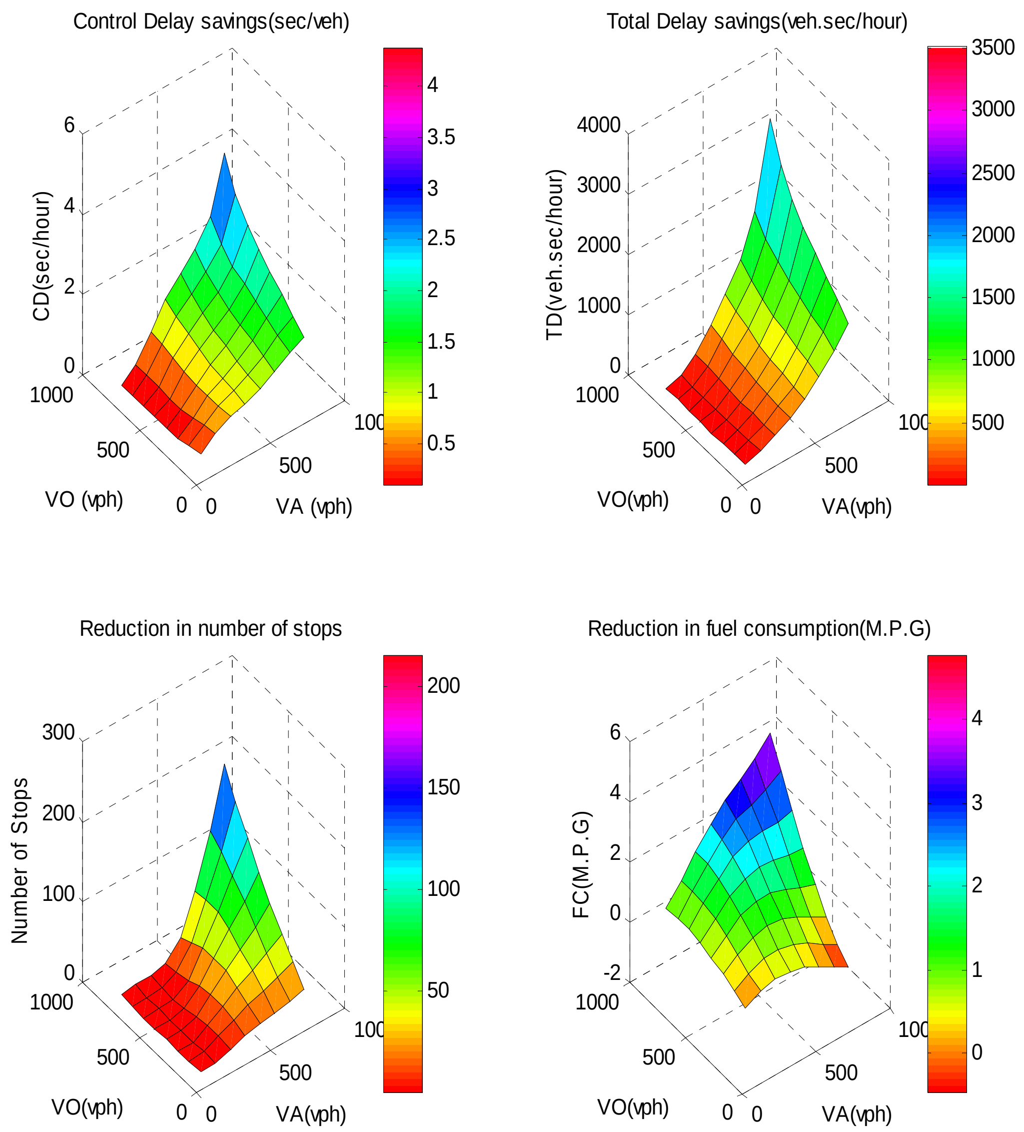

Figure B.17: Rural two-lane category, operating speed = 50mph, Left turn Percent = 20%

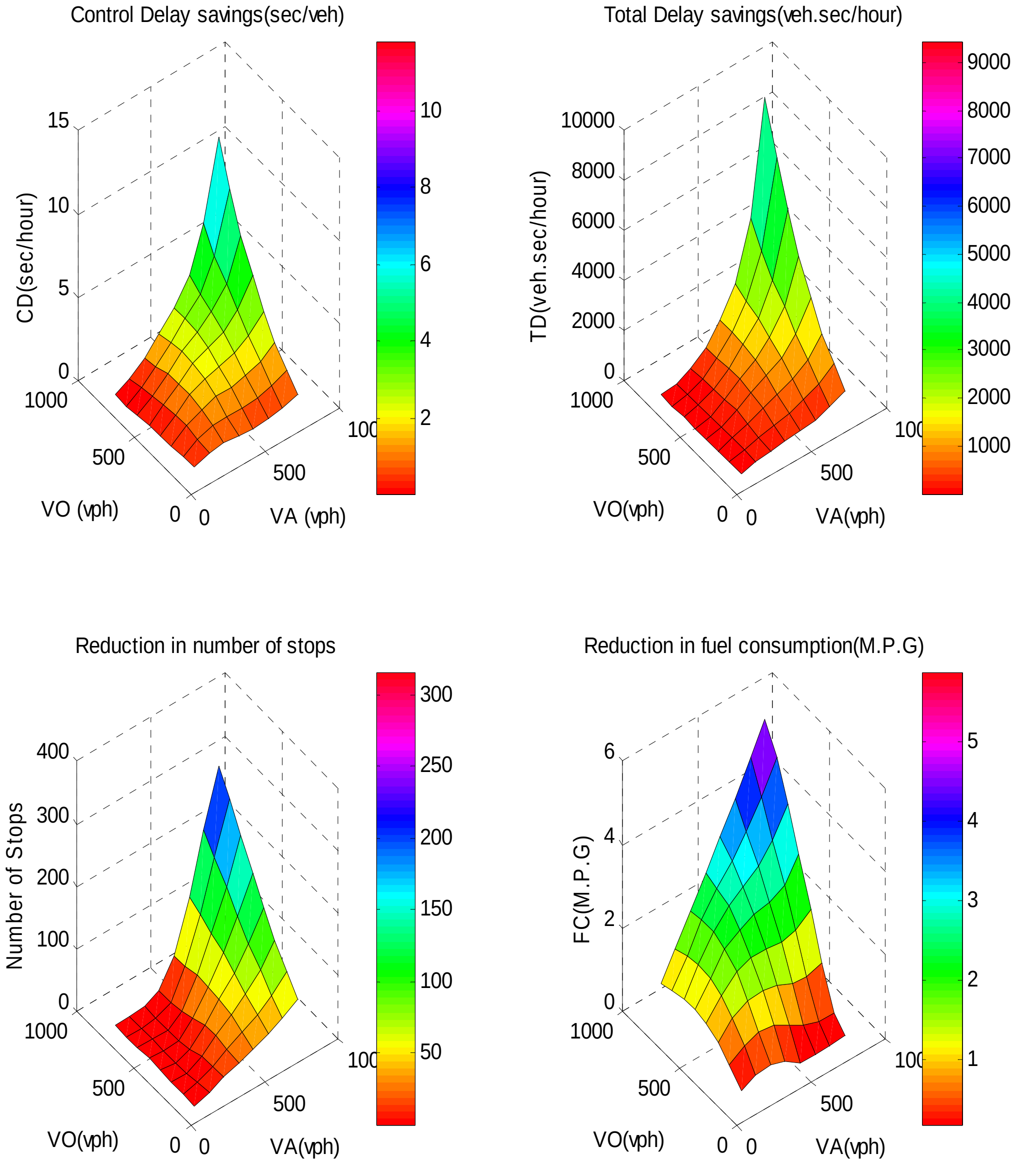

Figure B.18: Rural two-lane category, operating speed = 50mph, Left turn Percent = 30%

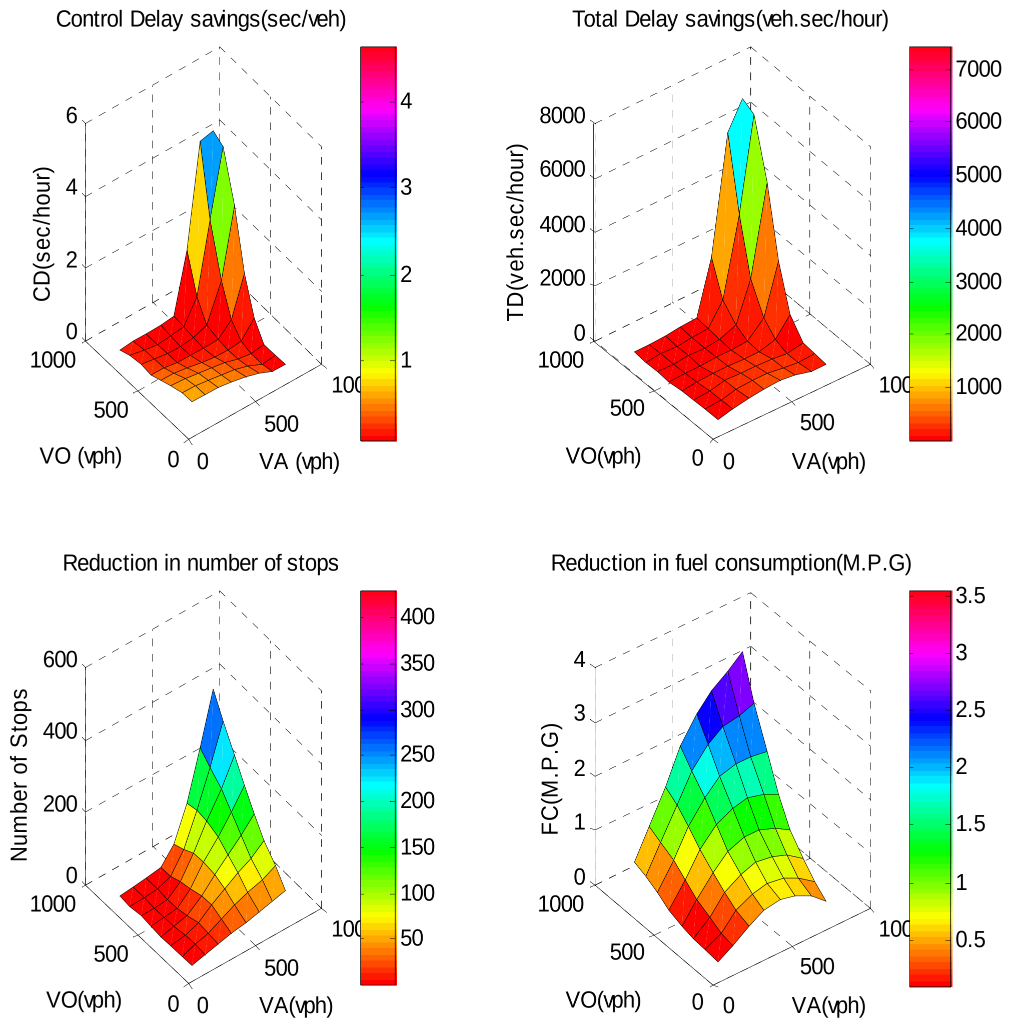

Figure B.19: Urban four-lane category, operating speed = 50mph, Left turn Percent = 10%

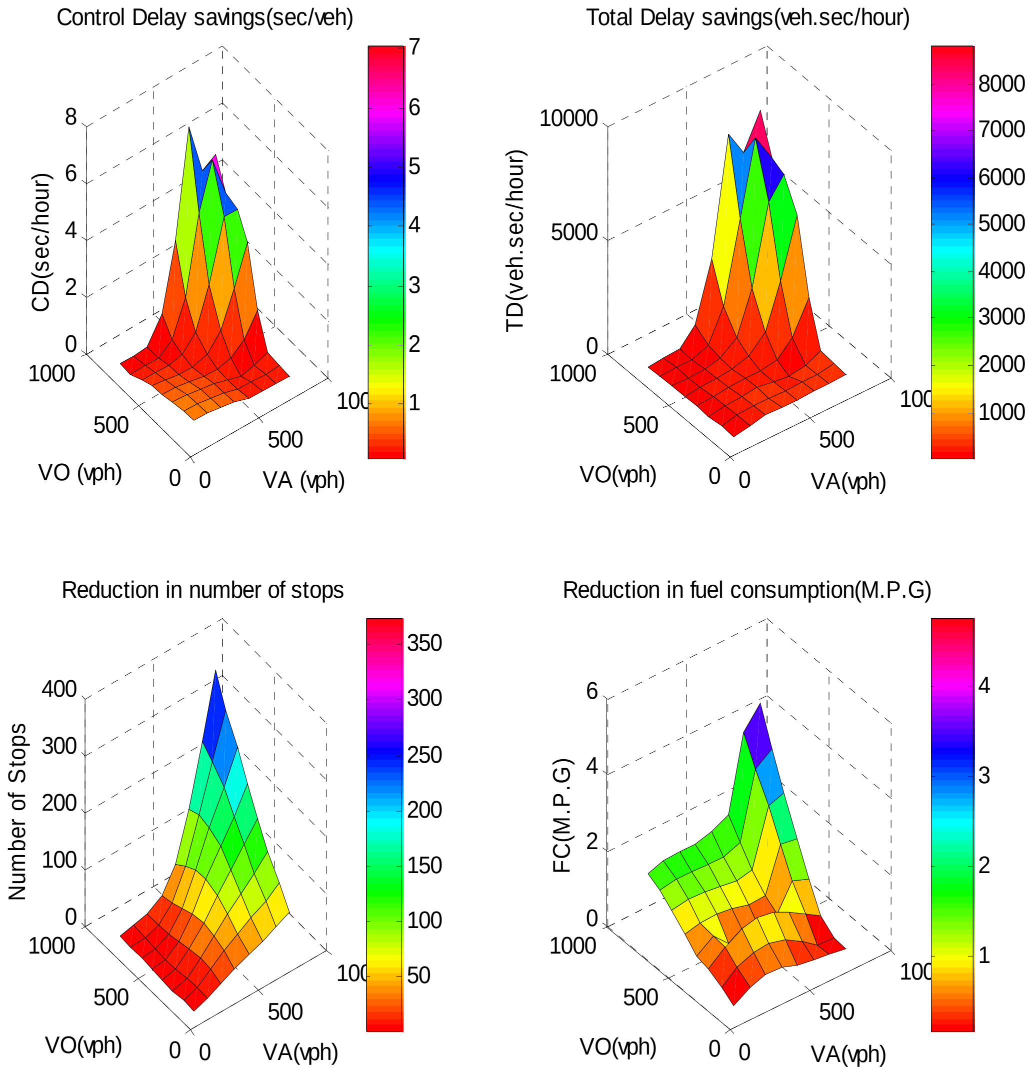

Figure B.20: Urban four-lane category, operating speed = 50mph, Left turn Percent = 20%

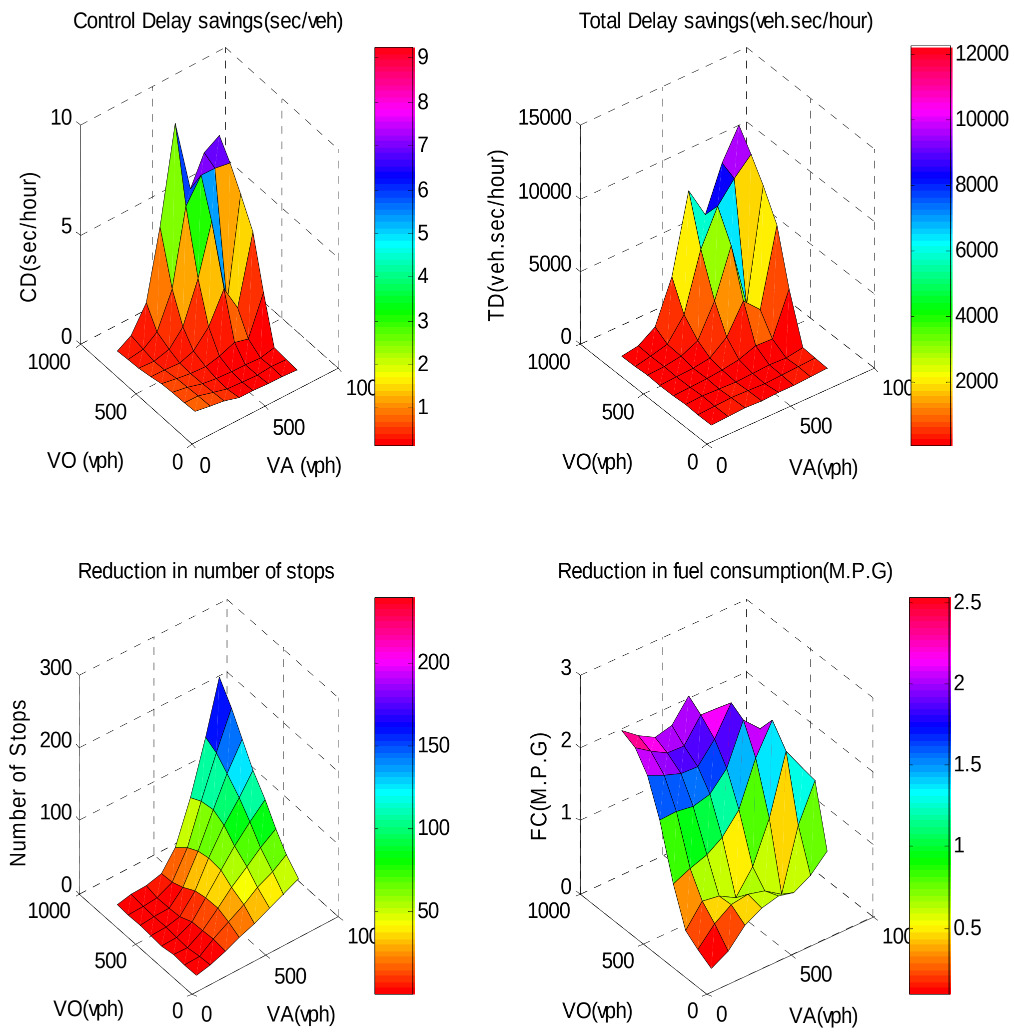

Figure B.21: Urban four-lane category, operating speed = 50mph, Left turn Percent = 30%

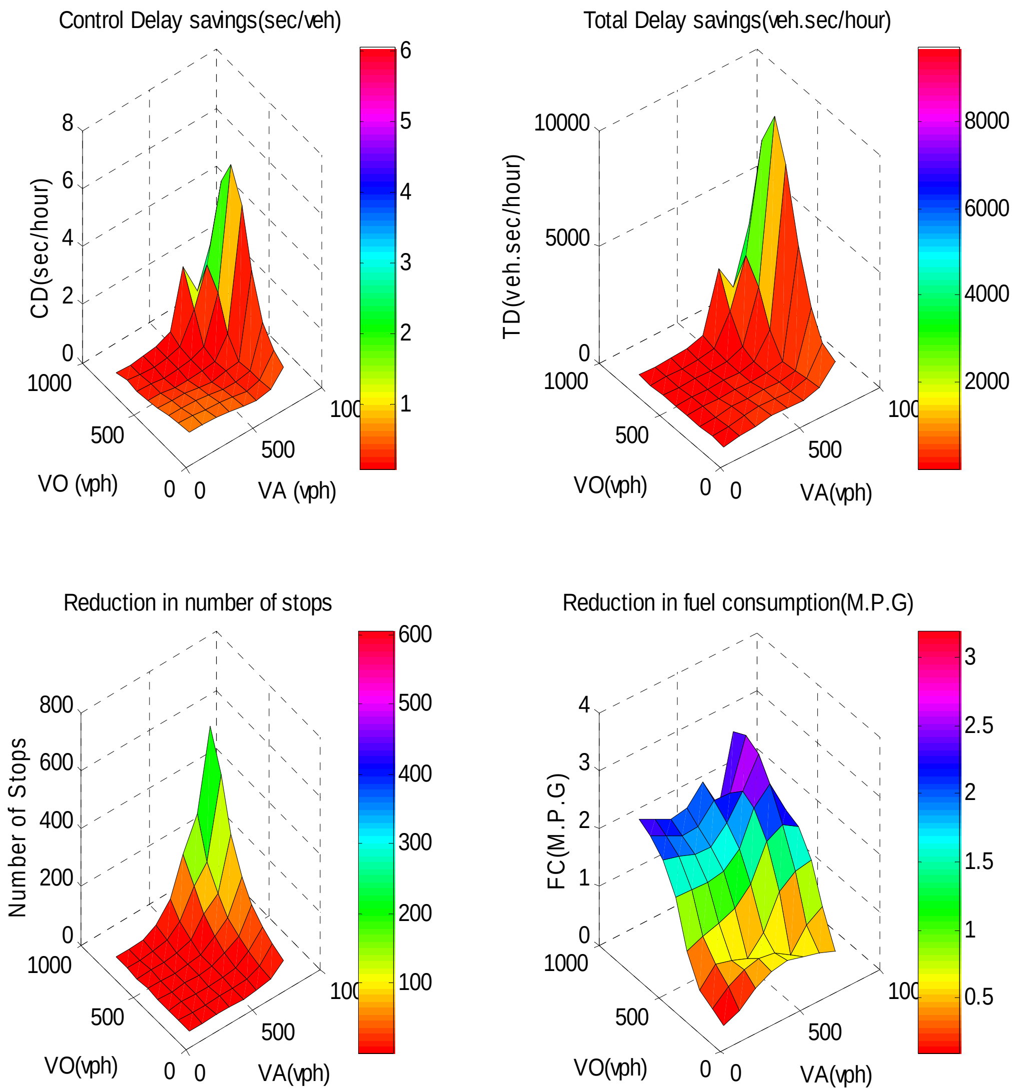

Figure B.22: Urban four-lane category, operating speed = 40mph, Left turn Percent = 10%

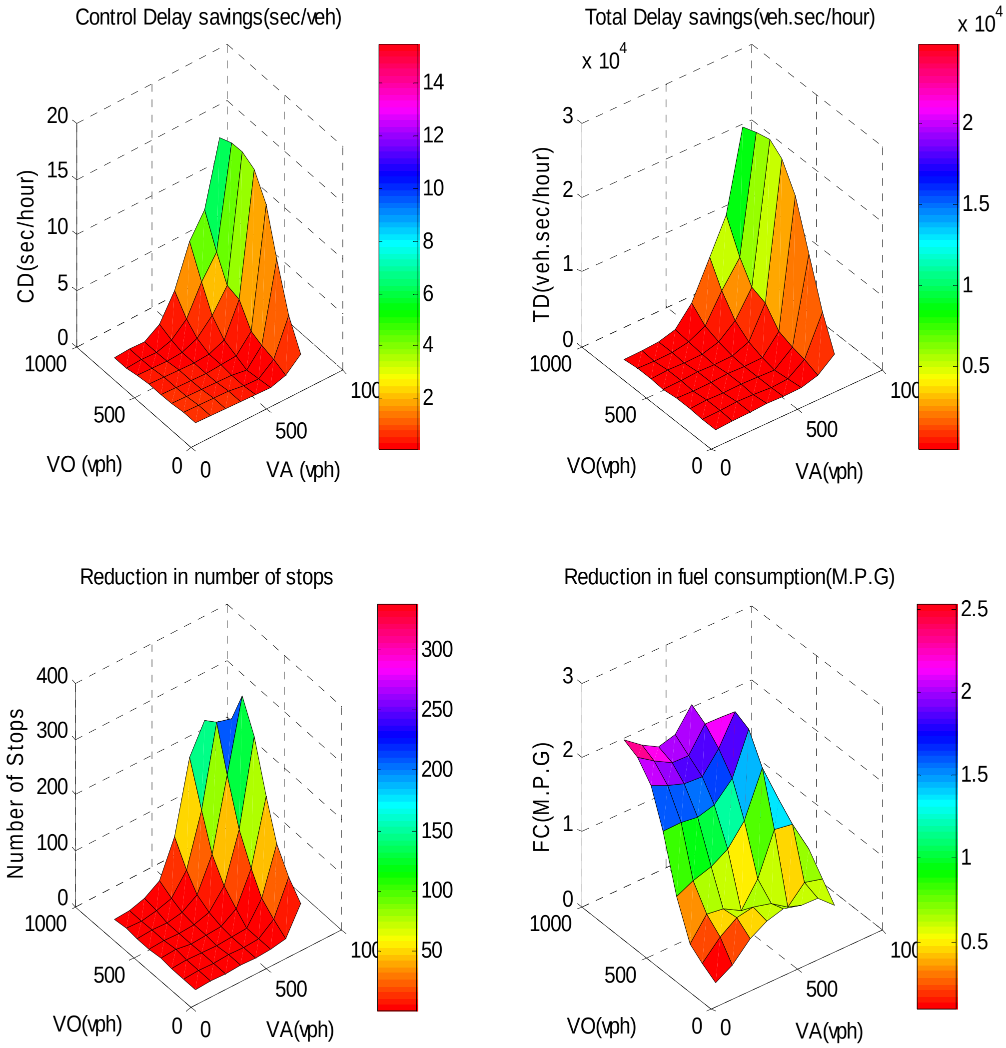

Figure B.23: Urban four-lane category, operating speed = 40mph, Left turn Percent = 20%

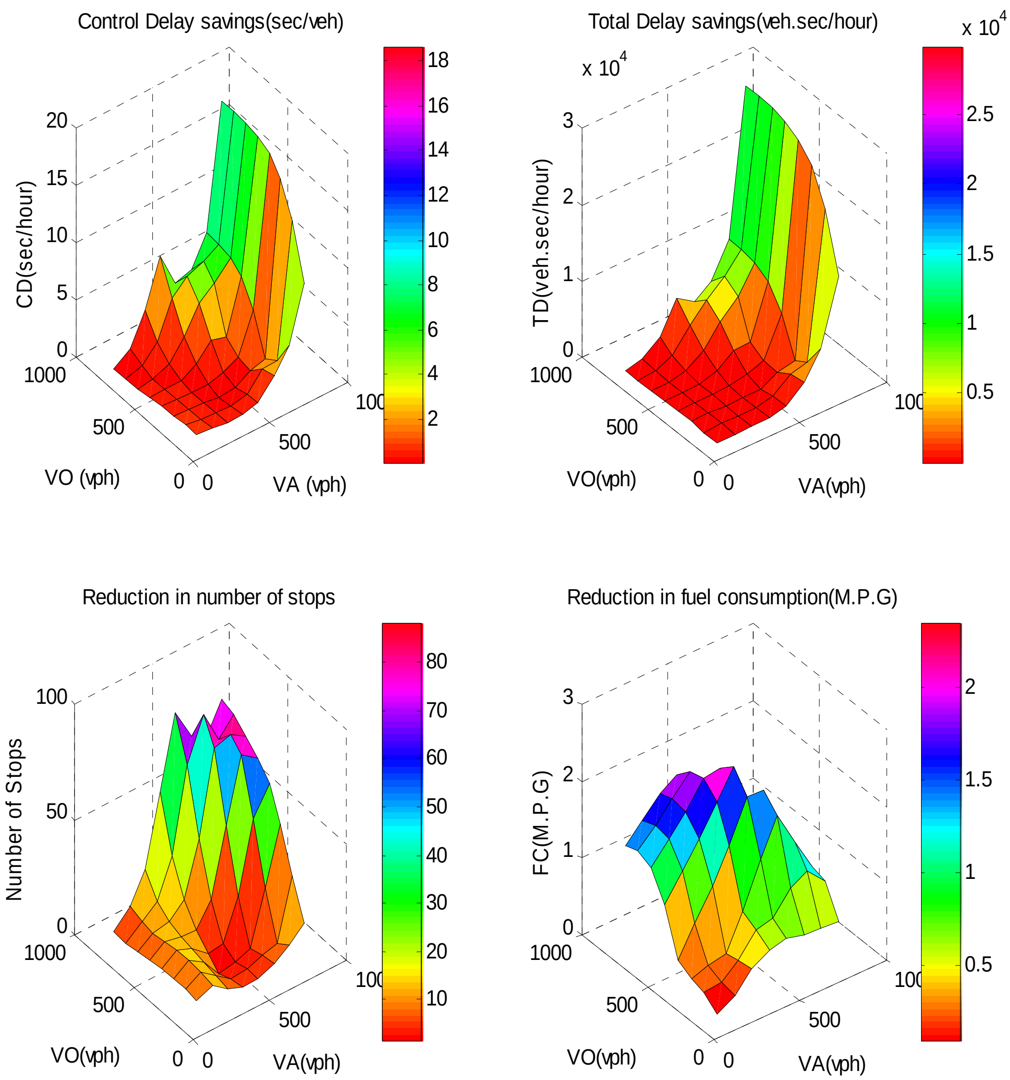

Figure B.24: Urban four-lane category, operating speed = 40mph, Left turn Percent = 30%

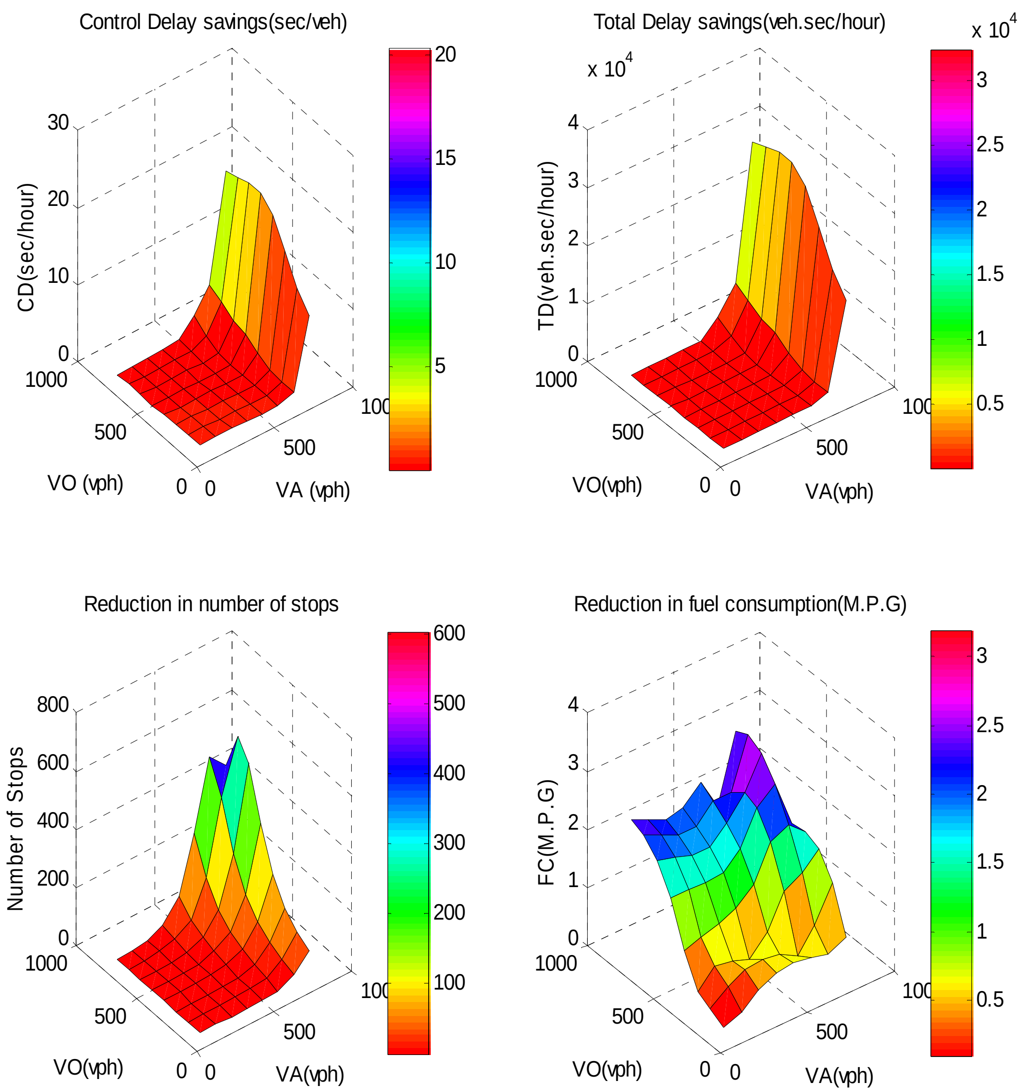

Figure B.25: Urban four-lane category, operating speed = 30mph, Left turn Percent = 10%

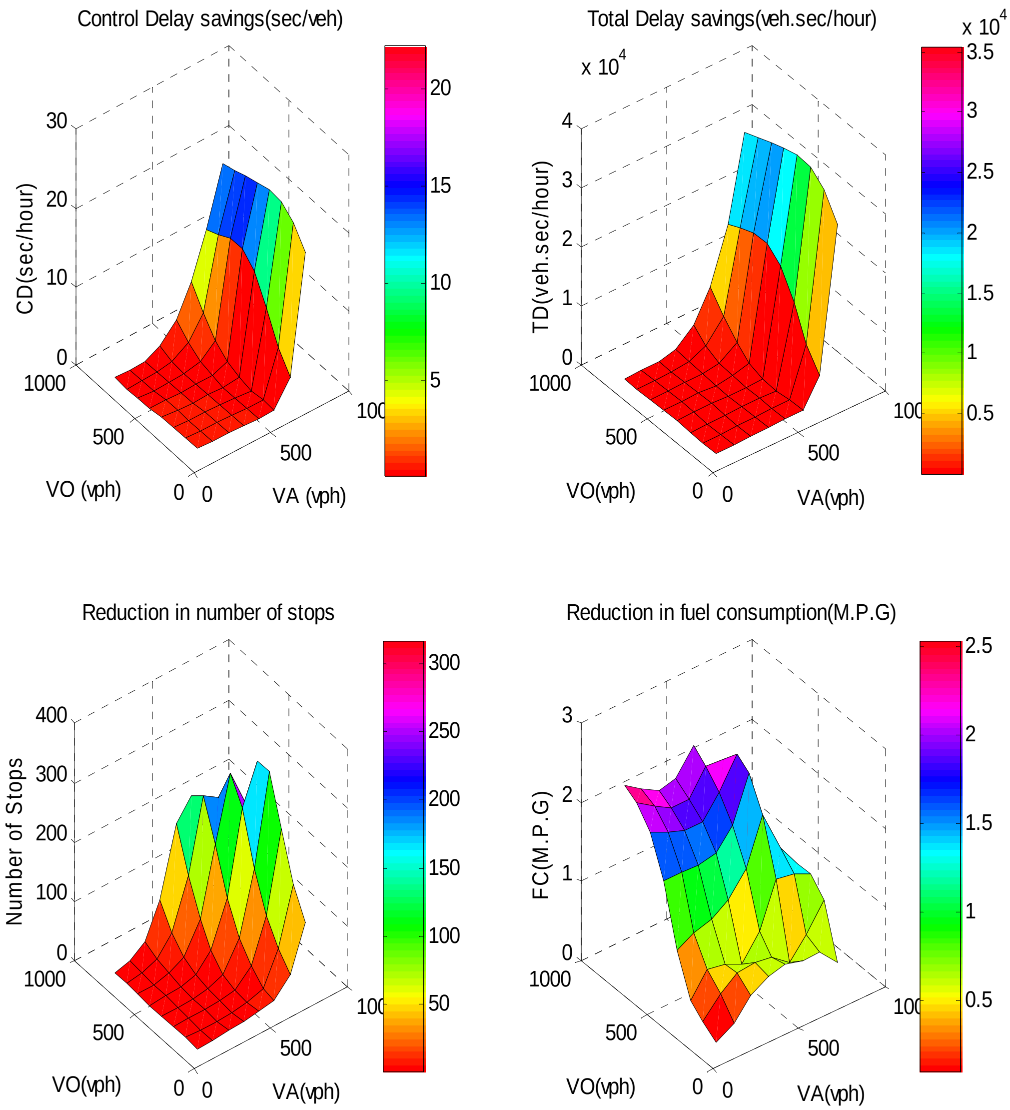

Figure B.26: Urban four-lane category, operating speed = 40mph, Left turn Percent = 20%

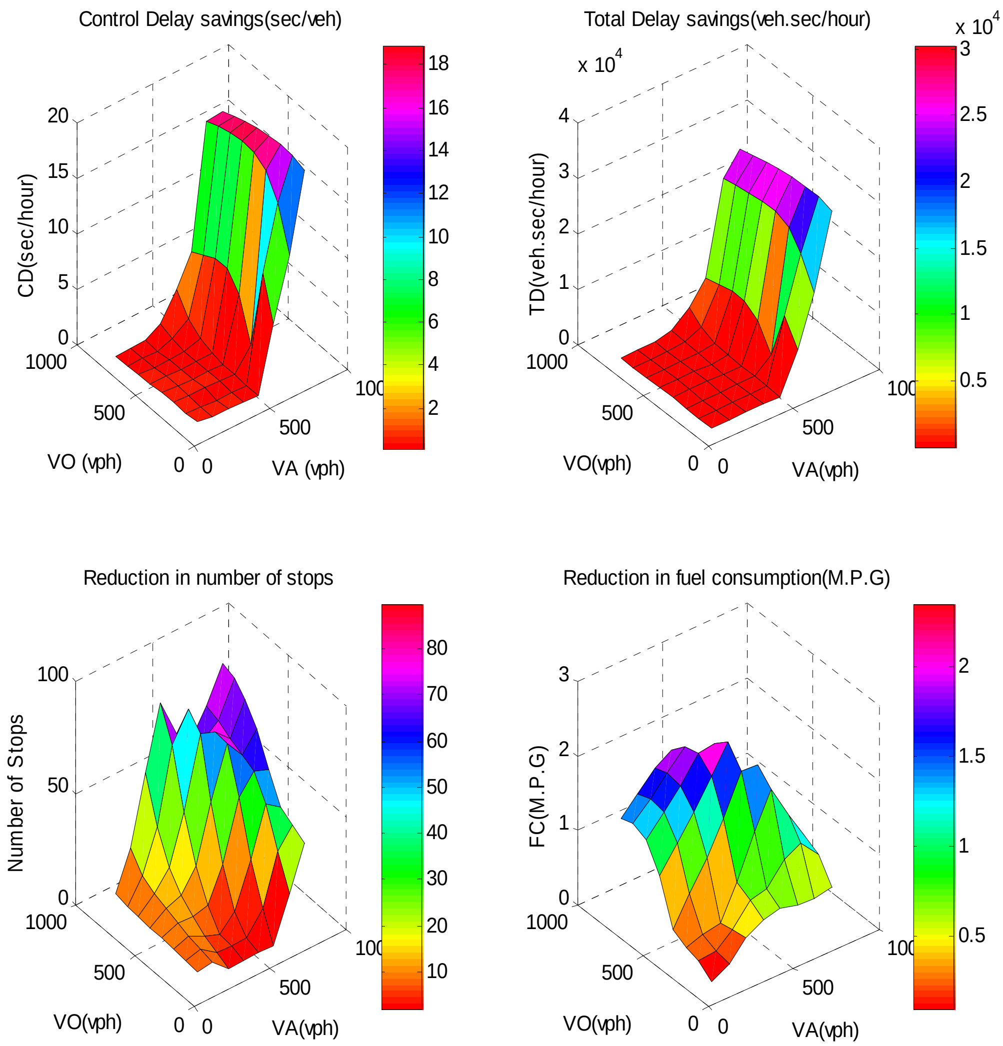

Figure B.27: Urban four-lane category, operating speed = 40mph, Left turn Percent = 30%

Appendix C: Crash Prediction Model Fit Statistics

Table C.1: Model fit for Crash Category 1 with and without the "Year" variable

	Intersection type											
	R2T		R2X		U2T		U2X		U4T		U4X	
	estimate	s.e.	estimate	s.e.	estimate	s.e.	estimate	s.e.	estimate	s.e.	estimate	s.e.
With the "Year" Variable												
Intercept	**-10.01**	**1.9132**	**-14.2**	**2.536**	**-10.58**	**3.1102**	**-11.05**	**2.252**	**-7.108**	**2.7846**	**-12.28**	**3.2947**
logtraffic	**1.0169**	**0.2077**	**1.483**	**0.2809**	**1.162**	**0.3283**	**1.1913**	**0.2477**	**0.7906**	**0.2793**	**1.3276**	**0.3377**
1995	**-0.572**	**0.2897**	**-0.997**	**0.441**	-0.133	0.1784	-0.417	0.2818	-0.064	0.1827	-0.293	0.1913
1996	-0.258	0.276	0.0945	0.2972	-0.242	0.1973	-0.117	0.2497	0.1167	0.1476	-0.039	0.2211
1997	-0.381	0.2631	0.172	0.2533	**-0.381**	**0.193**	-0.113	0.2395	-0.201	0.2072	-0.186	0.1886
1998	***-0.591***	***0.3136***	**-0.917**	**0.3686**	-0.23	0.212	-0.138	0.2405	-0.033	0.144	0.0485	0.1412
1999	-0.096	0.2759	0.241	0.2642	***-0.236***	***0.1415***	**-0.538**	**0.2502**	0.0031	0.163	0.2621	0.1728
2000	-0.031	0.1984	0.0005	0.2873	-0.269	0.1667	-0.009	0.2695	-0.078	0.1731	0.0641	0.1409
2001	0.0072	0.2224	-0.061	0.2932	-0.15	0.1955	-0.006	0.2236	0.04	0.1823	-0.047	0.1284
2002	-0.409	0.2841	-0.318	0.2885	-0.256	0.1878	-0.2	0.2376	0.1821	0.1605	-0.057	0.1574
2003	-0.152	0.3027	0.1481	0.253	**-0.489**	**0.1917**	-0.134	0.2041	-0.027	0.1373	-0.145	0.2074
2004	0	0	0	0	0	0	0	0	0	0	0	0
Fit P-value	Max Abs Value	Pr>M.A.V.	Max Abs Value	Pr>M.A.V.	Max Abs Value	Pr>M.A.V.	Max Abs Value	Pr>M.A.V.	Max Abs Value	Pr>M.A.V.	Max Abs Value	Pr>M.A.V.
	1.4496	0.85	1.3931	0.482	3.4286	0.832	3.827	0.303	7.5973	0.335	7.4132	0.51
Without the "Year" Variable												
Intercept	**-10.4**	**1.9367**	**-13.49**	**2.4629**	**-10.81**	**3.1044**	**-11.25**	**2.2515**	**-7.255**	**2.7332**	**-12.3**	**3.059**
logtraffic	**1.0347**	**0.209**	**1.3891**	**0.27**	**1.1648**	**0.3286**	**1.1963**	**0.2468**	**0.8051**	**0.2771**	**1.3247**	**0.3146**
Fit P-value	Max Abs Value	Pr>M.A.V.	Max Abs Value	Pr>M.A.V.	Max Abs Value	Pr>M.A.V.	Max Abs Value	Pr>M.A.V.	Max Abs Value	Pr>M.A.V.	Max Abs Value	Pr>M.A.V.
	1.5354	0.811	1.4973	0.416	4.6823	0.534	3.7227	0.328	7.9358	0.305	9.6986	0.306

Note: parameters in bold are significant at 0.05 level; parameters in bold and italic are marginally significant at 0.1 level.

Table C.2: Model fit for Crash Category 2 with and without the "Year" variable

	Intersection type											
	R2T		R2X		U2T		U2X		U4T		U4X	
	estimate	s.e.	estimate	s.e.	estimate	s.e.	estimate	s.e.	estimate	s.e.	estimate	s.e.
With the "Year" Variable												
Intercept	**-7.529**	**2.98**	***-3.103***	***1.8802***	-0.684	2.6618	-3.678	2.9237	***-6.304***	***3.4183***	-6.33	3.993
logtraffic	**0.6846**	**0.319**	0.2754	0.2164	0.0111	0.2757	0.3596	0.3085	***0.6307***	***0.3559***	***0.6964***	***0.4186***
1995	**-0.601**	**0.3165**	-0.461	0.3298	-0.335	0.2907	-0.015	0.2476	0.0057	0.1879	0.1562	0.2318
1996	-0.059	0.3676	-0.39	0.3544	-0.196	0.3338	0.0312	0.2574	0.2638	0.1827	0.1329	0.2107
1997	-0.329	0.4458	-0.057	0.3263	-0.287	0.2992	-0.229	0.2366	0.0536	0.218	0.267	0.2003
1998	**-1.331**	**0.4578**	-0.109	0.2923	-0.197	0.2981	0.0996	0.2392	-0.028	0.2193	-0.118	0.2157
1999	-0.314	0.4468	-0.104	0.3175	0.1024	0.2428	-0.089	0.2651	0.0985	0.2153	0.1655	0.2082
2000	-0.511	0.431	-0.025	0.3442	0.1643	0.2677	0.1684	0.2112	0.0875	0.2486	-0.015	0.2127
2001	-0.277	0.3869	0.2436	0.2597	0.0357	0.267	-0.003	0.1931	0.2524	0.1834	0.0967	0.223
2002	**-1.123**	**0.5685**	0.1671	0.2792	-0.336	0.327	0.0394	0.2145	0.363	0.2238	-0.035	0.2052
2003	-0.225	0.4964	-0.218	0.2877	0.0351	0.281	0.0863	0.1807	0.0169	0.2177	0.0145	0.2344
2004	0	0	0	0	0	0	0	0	0	0	0	0
Fit P-value	Max Abs Value	Pr>M.A.V.	Max Abs Value	Pr>M.A.V.	Max Abs Value	Pr>M.A.V.	Max Abs Value	Pr>M.A.V.	Max Abs Value	Pr>M.A.V.	Max Abs Value	Pr>M.A.V.
	1.1315	0.612	3.3919	0.072	3.0415	0.453	2.1589	0.826	7.5882	0.118	8.2989	0.277
Without the "Year" Variable												
Intercept	**-7.926**	**2.9191**	***-3.393***	***1.8856***	-1.005	2.5871	-3.722	2.829	***-6.256***	***3.4236***	-5.542	3.8658
logtraffic	**0.6845**	**0.3132**	0.2991	0.2154	0.0353	0.2743	0.3657	0.3049	***0.6364***	***0.3531***	0.6226	0.4014
Fit P-value	Max Abs Value	Pr>M.A.V.	Max Abs Value	Pr>M.A.V.	Max Abs Value	Pr>M.A.V.	Max Abs Value	Pr>M.A.V.	Max Abs Value	Pr>M.A.V.	Max Abs Value	Pr>M.A.V.
	1.0847	0.665	3.4482	0.064	2.9808	0.512	2.0666	0.875	8.5257	0.074	8.9383	0.254

Note: parameters in bold are significant at 0.05 level; parameters in bold and italic are marginally significant at 0.1 level.

Table C.3: Model fit for Crash Category 3 with and without the "Year" variable

	Intersection type											
	R2T		R2X		U2T		U2X		U4T		U4X	
	estimate	s.e.	estimate	s.e.	estimate	s.e.	estimate	s.e.	estimate	s.e.	estimate	s.e.
With the "Year" Variable												
Intercept	2.1545	2.1962	**-4.749**	**1.4511**	1.392	2.7586	-3.034	2.0631	**-6.374**	**3.1327**	**-9.331**	**3.8491**
logtraffic	-0.285	0.2274	**0.4559**	**0.1605**	-0.191	0.2926	0.2428	0.2202	***0.5886***	***0.3174***	**0.9196**	**0.3925**
1995	-0.263	0.3668	-0.003	0.2976	**-0.546**	**0.2515**	-0.073	0.2744	0.08	0.2684	0.0722	0.2347
1996	0.0317	0.3641	-0.412	0.3762	-0.245	0.2632	0.2229	0.2703	-0.234	0.2676	-0.053	0.2838
1997	-0.14	0.3403	-0.299	0.2827	-0.103	0.2762	-0.127	0.3214	0.0316	0.3052	-0.272	0.3983
1998	-0.241	0.3394	**-0.897**	**0.346**	**-0.488**	**0.2208**	-0.366	0.3349	0.391	0.2618	0.2347	0.2873
1999	-0.264	0.3303	0.0099	0.3101	0.0198	0.2416	-0.884	0.4073	-0.028	0.2977	0.2449	0.2129
2000	-0.294	0.1971	0	0.2761	-0.169	0.195	-0.039	0.3109	0.1931	0.2785	0.0764	0.2768
2001	0.0493	0.3408	-0.447	0.302	-0.236	0.3184	-0.042	0.3309	0.4195	0.3026	-0.118	0.3233
2002	0.0924	0.2344	-0.342	0.3403	**-0.537**	**0.2646**	0.0078	0.2712	0.0978	0.274	0.016	0.25
2003	-0.037	0.283	-0.52	0.3207	-0.229	0.2141	-0.044	0.2949	-0.058	0.2417	***-0.557***	***0.2895***
2004	0	0	0	0	0	0	0	0	0	0	0	0
Fit P-value	Max Abs Value	Pr>M.A.V.	Max Abs Value	Pr>M.A.V.	Max Abs Value	Pr>M.A.V.	Max Abs Value	Pr>M.A.V.	Max Abs Value	Pr>M.A.V.	Max Abs Value	Pr>M.A.V.
	2.405	0.502	1.1594	0.528	2.9058	0.14	1.9239	0.344	3.8102	0.245	4.6615	0.053
Without the "Year" Variable												
Intercept	1.992	2.1095	**-5.027**	**1.4294**	1.0232	2.7244	-3.15	2.033	**-6.372**	**3.0906**	**-9.567**	**3.6535**
logtraffic	-0.278	0.2273	**0.459**	**0.1601**	-0.176	0.2911	0.2448	0.2201	***0.5987***	***0.3124***	**0.9429**	**0.3797**
Fit P-value	Max Abs Value	Pr>M.A.V.	Max Abs Value	Pr>M.A.V.	Max Abs Value	Pr>M.A.V.	Max Abs Value	Pr>M.A.V.	Max Abs Value	Pr>M.A.V.	Max Abs Value	Pr>M.A.V.
	2.3523	0.539	1.1529	0.567	2.8677	0.154	1.9752	0.316	3.9852	0.236	4.6736	0.051

Note: parameters in bold are significant at 0.05 level; parameters in bold and italic are marginally significant at 0.1 level.

Table C.4: Model fit for fatal and injuries for the 6 intersection type

	Intersection type											
	R2T		R2X		U2T		U2X		U4T		U4X	
	estimate	s.e.	estimate	s.e.	estimate	s.e.	estimate	s.e.	estimate	s.e.	estimate	s.e.
Intercept	**-4.7492**	**1.6045**	**-5.4145**	**2.4112**	-3.0696	2.8209	**-5.9579**	**2.109**	**-11.5951**	**2.93**	**-8.7589**	**3.5918**
logtraffic	**0.4282**	**0.1744**	**0.5331**	**0.2721**	0.2958	0.295	**0.6057**	**0.2283**	**1.2253**	**0.2954**	**0.9469**	**0.3718**
Fit P-value	Max Abs Value	Pr>M.A.V.	Max Abs Value	Pr>M.A.V.	Max Abs Value	Pr>M.A.V.	Max Abs Value	Pr>M.A.V.	Max Abs Value	Pr>M.A.V.	Max Abs Value	Pr>M.A.V.
	1.6606	0.58	2.6956	0.229	2.4965	0.745	3.2533	0.196	10.8954	0.152	8.1717	0.264

Note: parameters in bold are significant at 0.05 level.

Appendix D: SPF Estimates

Table D.1: SPF estimate for R2T intersections

Int_ ID	Rt #	Cum_ Mileage	Ave_ AADT	Min_ AADT	Max_ AADT	Crash Category 1				Fatal and injury crashes			
						Crash_ ob	SPF_ estimate	P(y>= crash_ob)	P(y<= crash_ob)	Crash_ ob	SPF_ estimate	P(y>= crash_ob)	P(y<= crash_ob)
R2TN01	2	43.97	20310	16900	23600	1	8.735591	0.99983924	**0.001565**	1	6.024368	0.997581	**0.016993**
R2TN02	2	44.27	20310	16900	23600	5	8.735591	0.93543331	0.132716	6	6.024368	0.558226	0.602389
R2TN03	2	44.66	24740	23300	30200	6	10.71296	0.95551912	0.091206	6	6.554368	0.638813	0.51798
R2TN04	2	45.88	24740	23300	30200	7	10.71296	0.90879448	0.162714	4	6.554368	0.891837	0.217656
R2TN05	4	25.981	8475	8100	8900	9	3.535825	**0.01049189**	0.996443	6	4.164399	0.241063	0.871503
R2TN06	4	34.971	9555	9100	10100	1	4.003036	0.98173988	0.091356	1	4.381414	0.987492	0.067309
R2TN07	4	35.483	13035	12400	14000	0	5.520066	1	**0.004006**	6	4.997864	0.383664	0.762496
R2TN08	4	38.087	20965	19800	22000	13	9.025745	0.12610791	0.947281	**12**	6.112865	**0.022756**	0.989828
R2TN09	5	48.341	6625	5400	8400	0	2.741986	1	0.064442	2	3.738	0.887228	0.279058
R2TN10	5	51.352	13405	12500	14300	16	5.682291	**0.00028528**	0.999715	8	5.057337	0.139429	0.928099
R2TN11	5	51.733	14125	13300	15000	5	5.998428	0.71473306	0.445932	4	5.170339	0.758076	0.41113
R2TN12	6	24.83	12805	12200	13200	2	5.419247	0.97155964	0.093498	1	4.960755	0.992992	**0.041771**
R2TN13	6	29.63	4110	3800	4700	0	1.672387	1	0.187798	0	3.062603	1	0.046766
R2TN14	6	30.22	4110	3800	4700	2	1.672387	0.49813026	0.764494	4	3.062603	0.366791	0.804636
R2TN15	6	32.14	4835	4500	5100	1	1.978353	0.8617032	0.411897	**9**	3.28258	**0.00669**	0.997886
R2TN16	6	37.04	7175	6600	7600	2	2.976293	0.79728265	0.428522	4	3.880057	0.542753	0.652256
R2TN17	6	80.135	16955	16100	18000	4	7.245811	0.9301823	0.151725	4	5.587174	0.807985	0.344097
R2TN18	25	10.21	20205	18900	22600	22	8.687508	**0.01657038**	0.98343	12	6.017586	**0.020491**	0.990973
R2TN19	25	12.32	19195	17100	22900	6	8.239435	0.82972613	0.285013	3	5.884101	0.932656	0.16185
R2TN20	25	21.55	14145	12800	15300	0	6.007414	1	**0.00246**	0	5.172269	1	**0.005672**
R2TN21	25	21.95	11665	10600	12600	0	4.921188	1	**0.00729**	0	4.766323	1	**0.008512**
R2TN22	25	24.78	8060	6800	9100	2	3.357511	0.84826347	0.348008	2	4.071061	0.913491	0.227875
R2TN23	25	24.96	7490	6800	9100	1	3.112421	0.95550689	0.182974	0	3.944329	1	**0.019364**
R2TN24	30	14.627	6670	6500	7100	5	2.75978	0.14611467	0.938342	3	3.762266	0.724954	0.481235
R2TN25	44	4.357	7800	7400	8500	2	3.244979	0.83457581	0.370596	2	4.019481	0.909839	0.235263
R2TN26	44	5.177	4665	4400	4900	5	1.906429	**0.0446054**	0.986581	1	3.233137	0.960566	0.166928
R2TN27	44	7.016	4665	4400	4900	0	1.906429	1	0.14861	**7**	3.233137	**0.046663**	0.982231

Int_ ID	Rt #	Cum_ Mileage	Ave_ AADT	Min_ AADT	Max_ AADT	Crash Category 1				Fatal and injury crashes			
						Crash_ ob	SPF_ estimate	P(y>= crash_ob)	P(y<= crash_ob)	Crash_ ob	SPF_ estimate	P(y>= crash_ob)	P(y<= crash_ob)
R2TN28	44	8.542	4665	4400	4900	1	1.906429	0.85138987	0.431925	4	3.233137	0.404849	0.774687
R2TN29	44	10.332	5645	4900	6500	0	2.322594	1	0.098019	4	3.501449	0.46368	0.725171
R2TN30	44	30.814	14600	12800	15500	7	6.207413	0.42697348	0.714965	7	5.242409	0.274051	0.840115
R2TN31	44	66.931	18230	16700	20400	4	7.810834	0.95187335	0.110987	3	5.758818	0.926363	0.174058
R2TN32	44	73.551	6440	5900	7500	2	2.661526	0.74427336	0.503096	1	3.705324	0.975408	0.115715
R2TN33	58	4.09	5735	5200	6200	0	2.36065	1	0.094359	4	3.527696	0.469332	0.720205
R2TN34	58	5.24	5735	5200	6200	4	2.36065	0.21309377	0.909001	7	3.527696	0.067445	0.972179
R2TN35	58	6.01	4110	3600	4300	5	1.672378	**0.02789148**	0.992582	5	3.062656	0.195373	0.909634
R2TN36	63	48.75	2475	2200	2800	1	0.989567	0.62826231	0.739597	1	2.469838	0.915401	0.293543
R2TN37	66	15.125	10345	9600	11000	10	4.345964	**0.01379146**	0.994792	4	4.531365	0.662968	0.526161
R2TN38	66	18.417	11230	10800	11700	5	4.731167	0.51114133	0.663015	6	4.691982	0.330168	0.805696
R2TN39	66	26.891	9845	9500	10000	4	4.128731	0.59126681	0.603704	4	4.437842	0.647109	0.543941
R2TN40	66	33.828	8655	8000	9200	2	3.61358	0.87564012	0.30035	5	4.201445	0.410454	0.752907
R2TN41	80	11.11	5405	4900	6000	1	2.220284	0.89142176	0.349653	3	3.439843	0.667883	0.549668
R2TN42	80	15.25	5270	4400	7100	1	2.164403	0.88518148	0.363332	1	3.387846	0.966219	0.148227
R2TN43	110	5.35	12185	11000	13300	6	5.148396	0.41005923	0.740179	7	4.854889	0.216921	0.881338
R2TN44	122	2.5	11855	11500	12200	9	5.003799	0.06834189	0.968034	15	4.801352	**0.000148**	0.999957
R2TN45	122	2.67	11855	11500	12200	18	5.003799	**6.9608E-05**	0.99993	17	4.801352	**1.2E-05**	0.999988
R2TN46	123	1.89	16210	14200	17600	14	6.917319	**0.01167923**	0.994846	8	5.478127	0.187823	0.896204
R2TN47	123	2.84	13065	11100	14100	9	5.533724	0.10852746	0.944455	4	4.999521	0.734907	0.440577
R2TN48	123	5.33	11560	10100	12600	2	4.875317	0.9551556	0.135554	1	4.74839	0.991334	**0.049813**
R2TN49	123	5.82	11560	10100	12600	2	4.875317	0.9551556	0.135554	1	4.74839	0.991334	**0.049813**
R2TN50	123	7.01	8365	7100	9300	6	3.488683	0.14089438	0.935581	3	4.138702	0.781524	0.406852
R2TY01	2	46.333	24740	23300	30200	5	10.71296	0.98168817	**0.044481**	1	6.554368	0.998576	**0.010757**
R2TY02	5	48.234	6625	5400	8400	1	2.741986	0.93555778	0.241142	1	3.738	0.976198	0.112772
R2TY03	6	78.549	16955	16100	18000	2	7.245811	0.99411946	**0.024602**	0	5.587174	1	**0.003746**
R2TY04	6	79.019	16955	16100	18000	4	7.245811	0.9301823	0.151725	0	5.587174	1	**0.003746**
R2TY05	6	79.594	16955	16100	18000	2	7.245811	0.99411946	**0.024602**	6	5.587174	0.485961	0.672289

Int_ ID	Rt #	Cum_ Mileage	Ave_ AADT	Min_ AADT	Max_ AADT	Crash Category 1				Fatal and injury crashes			
						Crash_ ob	SPF_ estimate	P(y>= crash_ob)	P(y<= crash_ob)	Crash_ ob	SPF_ estimate	P(y>= crash_ob)	P(y<= crash_ob)
R2TY06	6	80.435	16955	16100	18000	3	7.245811	0.97539849	0.069818	0	5.587174	1	**0.003746**
R2TY07	6	82.949	14765	14100	15300	1	6.279683	0.99812601	**0.013642**	1	5.269338	0.994853	**0.032268**
R2TY08	25	24.08	8190	6800	9600	1	3.41373	0.96708182	0.145292	3	4.097324	0.775813	0.414691
R2TY09	44	69.705	17533	16300	18700	4	7.5019	0.94092839	0.059072	4	5.66557	0.816367	0.332308
R2TY10	58	3.82	8215	7900	8600	0	3.423661	1	**0.032593**	1	4.109689	0.983587	**0.083865**
R2TY11	122	2.16	11855	11500	12200	6	5.003799	0.38470589	0.761628	3	4.801352	0.857589	0.294025
R2TY12	123	2.97	13065	11100	14100	13	5.533724	**0.00467578**	0.998219	7	4.999521	0.237746	0.866678
R2TY13	140	10.058	6605	6300	6900	1	2.73194	0.93490711	0.242923	0	3.74685	1	**0.023592**

Table D.2: SPF estimate for R2X intersections

						Crash Category 1				Fatal and injury crashes			
Int_ ID	Rt #	Cum_ Mileage	Ave_ AADT	Min_ AADT	Max_ AADT	Crash_ ob	SPF_ estimate	P(y>= crash_ob)	P(y<= crash_ob)	Crash_ ob	SPF_ estimate	P(y>= crash_ob)	P(y<= crash_ob)
R2XN01	4	20.75	4465	3700	5800	0	1.638753	1	0.194222	2	3.82977	0.895123	0.264122
R2XN02	6	31.49	4835	4500	5100	3	1.82246	0.2754	0.887656	11	4.014313	**0.002916**	0.999057
R2XN03	6	99.15	9135	8500	10200	2	4.414819	0.934498	0.183388	5	5.75503	0.680817	0.485784
R2XN04	6	101.97	7360	6800	8400	1	3.272315	0.962081	0.162	1	5.090179	0.993843	**0.037497**
R2XN05	31	7.869	9150	8900	9600	7	4.419977	0.158836	0.919858	7	5.763251	0.35573	0.775881
R2XN06	31	8.878	8180	7600	8800	4	3.784645	0.523369	0.670827	2	5.407356	0.971273	0.094275
R2XN07	44	19.503	4560	4200	4800	0	1.680712	1	0.186241	3	3.882581	0.744182	0.45674
R2XN08	44	29.93	14600	12800	15500	12	8.465497	0.14841	0.911153	6	7.508314	0.759472	0.377019
R2XN09	44	77.096	5365	5100	5500	9	2.105527	**0.000344**	0.999929	8	4.258329	0.068022	0.969911
R2XN10	44	80.749	10020	9500	10700	2	5.015227	0.960082	0.123375	4	6.067278	0.854699	0.276152
R2XN11	63	38.12	3005	2900	3300	3	0.941643	0.069886	0.984383	6	3.065217	0.090635	0.963096
R2XN12	66	13.687	10275	9200	11000	5	5.19489	0.593013	0.581806	4	6.153495	0.861977	0.265036
R2XN13	66	18.901	11230	10800	11700	9	5.875374	0.140165	0.924388	6	6.472532	0.626954	0.530854
R2XN14	66	21.348	9300	8800	10500	5	4.523586	0.472367	0.698896	3	5.815105	0.929255	0.168481
R2XN15	66	23.375	9300	8800	10500	6	4.523586	0.301104	0.828017	4	5.815105	0.831519	0.310567
R2XN16	66	25.585	14140	12800	16300	7	8.110406	0.699924	0.437659	8	7.368034	0.455952	0.680003
R2XN17	66	28.228	9845	9500	10000	10	4.89244	**0.028093**	0.988152	12	6.00783	**0.020269**	0.991084
R2XN18	66	32.068	11145	10200	11900	4	5.815583	0.831565	0.310499	6	6.443759	0.622731	0.535399
R2XN19	67	24.04	14090	12600	16200	13	8.0632	0.066887	0.96391	10	7.356204	0.20738	0.874282
R2XN20	67	30.51	5920	5400	6200	2	2.415343	0.694885	0.565705	7	4.5016	0.169154	0.913282
R2XN21	68	18.74	12320	11400	13100	0	6.685415	1	**0.001249**	5	6.819988	0.809944	0.324287
R2XN22	68	20.84	9055	8700	9600	7	4.356717	0.151042	0.924744	5	5.729044	0.677041	0.490121
R2XN23	68	21.92	9055	8700	9600	5	4.356717	0.440501	0.727193	11	5.729044	**0.032416**	0.985353
R2XN24	80	14.85	5270	4400	7100	3	2.077619	0.344322	0.842853	5	4.194143	0.409034	0.754099
R2XN25	80	20.31	3600	3200	3900	2	1.211301	0.341453	0.877027	5	3.394341	0.254767	0.871257
R2XN26	81	5.78	7820	6900	9000	4	3.560179	0.476295	0.714036	3	5.267807	0.896166	0.229425
R2XN27	81	10.29	5970	5300	7000	0	2.446867	1	0.086564	2	4.520489	0.939917	0.171287

						Crash Category 1				Fatal and injury crashes			
Int_ ID	Rt #	Cum_ Mileage	Ave_ AADT	Min_ AADT	Max_ AADT	Crash_ ob	SPF_ estimate	P(y>= crash_ob)	P(y<= crash_ob)	Crash_ ob	SPF_ estimate	P(y>= crash_ob)	P(y<= crash_ob)
R2XN28	83	3.129	5275	4700	5900	2	2.060138	0.610027	0.660404	2	4.214419	0.922926	0.208338
R2XN29	83	5.022	6465	6200	6900	1	2.728449	0.934679	0.243544	3	4.732996	0.850983	0.304522
R2XN30	85	20.311	4465	3700	5800	0	1.638753	1	0.194222	2	3.82977	0.895123	0.264122
R2XN31	85	21.94	4465	3700	5800	0	1.638753	1	0.194222	2	3.82977	0.895123	0.264122
R2XN32	85	36.887	5940	5400	6300	4	2.426127	0.226755	0.900826	2	4.510692	0.939433	0.172379
R2XN33	85	37.038	5940	5400	6300	3	2.426127	0.437102	0.773245	5	4.510692	0.469925	0.701103
R2XN34	101	7.23	6090	5900	6600	2	2.511502	0.715055	0.540866	2	4.575052	0.942545	0.165311
R2XN35	101	7.23	6090	5900	6600	2	2.511502	0.715055	0.540866	2	4.575052	0.942545	0.165311
R2XN36	104	5.39	7050	5500	7900	2	3.09191	0.814165	0.402917	9	4.96022	0.065528	0.969592
R2XN37	104	6.37	7050	5500	7900	1	3.09191	0.954585	0.185835	2	4.96022	0.958211	0.128043
R2XN38	106	8.35	6055	5400	6800	7	2.494236	**0.014028**	0.99581	3	4.557839	0.832824	0.332632
R2XN39	106	10.27	6705	5200	9700	0	2.920511	1	0.053906	6	4.794868	0.348097	0.791522
R2XN40	107	0.17	11965	10800	12700	8	6.41794	0.315282	0.801229	26	6.708544	**1.25E-08**	1
R2XN41	109	3.57	1820	1600	2100	0	0.470107	1	0.624936	0	2.304834	1	0.099775
R2XN42	109	8.68	1765	1400	2100	1	0.452291	0.363831	0.923902	7	2.260826	**0.00858**	0.997666
R2XN43	109	9.45	1765	1400	2100	0	0.452291	1	0.636169	1	2.260826	0.895736	0.339988
R2XN44	110	7.04	12655	11100	13500	6	6.94122	0.691722	0.458506	5	6.923583	0.819922	0.310574
R2XN45	118	2.78	7300	6300	8100	3	3.235236	0.627401	0.594685	7	5.066564	0.247613	0.859584
R2XN46	118	5.4	7525	6900	8300	1	3.370981	0.965644	0.150169	2	5.157061	0.964544	0.112032
R2XN47	159	15.63	5915	5200	6300	1	2.412453	0.910405	0.30574	3	4.499483	0.826364	0.342383
R2XN48	202	32.87	5815	5300	6400	0	2.356975	1	0.094706	1	4.455386	0.988384	0.063369
R2XN49	202	52.31	9550	8700	10800	3	4.700116	0.847711	0.309665	4	5.89933	0.839584	0.298757
R2XN50	214	1.15	4560	3800	5100	2	1.685014	0.502086	0.761174	4	3.878018	0.542343	0.652653
R2XY01	44	68.125	16000	15800	16300	2	9.604565	0.999285	**0.003825**	4	7.911812	0.955025	0.104794
R2XY02	44	68.637	17533.33	16300	18700	3	10.91666	0.998702	**0.005234**	7	8.329557	0.725201	0.40798
R2XY03	44	70.804	8433.333	7700	9100	0	3.951793	1	**0.01922**	5	5.49952	0.642407	0.529001

Table D.3: SPF estimate for U2T intersections

Int_ID	Rt #	Cum_Mileage	Ave_AADT	Min_AADT	Max_AADT	Crash_ob	SPF_estimate	P(y>=crash_ob)	P(y<=crash_ob)
U2TN01	2	42.29	20310	16900	23600	0	21.05831	1	**7.15E-10**
U2TN02	3	5.417	11180	10100	12000	0	10.4993	1	**2.76E-05**
U2TN03	3	7.653	9530	9000	10100	3	8.714913	0.992173158	**0.025932**
U2TN04	3	8.582	8315	7500	8600	1	7.434856	0.999409686	**0.004979**
U2TN05	3	9.871	11295	10900	11800	25	10.62116	**0.000118087**	0.999953
U2TN06	4	23.126	14125	11900	18300	13	13.81478	0.623075577	0.484202
U2TN07	4	25.103	10390	9800	11000	5	9.637395	0.963091901	0.082109
U2TN08	4	33.787	9555	9100	10100	7	8.741314	0.768630661	0.35505
U2TN09	4	39.737	18080	16900	19200	31	18.37414	**0.004438852**	0.997536
U2TN10	5	11.664	16065	14800	17200	25	16.01384	**0.022515059**	0.986754
U2TN11	5	12.022	17925	14800	22700	15	18.24262	0.807382363	0.267977
U2TN12	5	17.397	14700	13700	17000	18	14.44071	0.205653008	0.856682
U2TN13	7	6.46	27105	24900	28900	57	29.44769	**0.002844887**	0.997155
U2TN14	10	3.6	13415	11900	15200	8	12.98328	0.945499514	0.100525
U2TN15	10	3.8	13415	11900	15200	22	12.98328	**0.013899577**	0.992485
U2TN16	10	15.04	20820	19300	22000	7	21.65731	0.999923712	**0.000251**
U2TN17	22	1.45	6945	6800	7000	2	6.027504	0.983053141	0.060753
U2TN18	22	2.88	12300	12000	12600	10	11.72949	0.733176793	0.376213
U2TN19	30	9.23	14395	12600	15700	10	14.09349	0.894953329	0.169567
U2TN20	32	9.29	13910	10000	15600	39	13.56835	**1.39192E-08**	1
U2TN21	32	10.12	9540	7500	11000	18	8.739255	**0.003975952**	0.998236
U2TN22	44	11.778	5345	5000	5800	5	4.444301	0.457292581	0.712415
U2TN23	44	18.427	5630	5200	6100	3	4.721349	0.849831087	0.306337
U2TN24	44	34.47	11745	11300	12300	17	11.11625	0.060301685	0.964962
U2TN25	44	60.133	17450	16400	19200	0	17.63097	1	**2.2E-08**
U2TN26	63	13.22	17065	15800	18100	24	17.18046	0.069251598	0.955113
U2TN27	63	16.19	10875	10400	11200	0	10.16267	1	**3.86E-05**
U2TN28	66	26.188	11325	9500	15000	6	10.69481	0.955041934	0.092057
U2TN29	66	35.407	19530	18400	20800	38	20.1022	**0.000239049**	0.99988

Int_ID	Rt #	Cum_Mileage	Ave_AADT	Min_AADT	Max_AADT	Crash_ob	SPF_estimate	P(y>=crash_ob)	P(y<=crash_ob)
U2TN30	67	8.7	3865	3500	4100	7	3.046291	**0.03589633**	0.987064
U2TN31	67	15.84	6320	5900	6500	0	5.401179	1	**0.004511**
U2TN32	67	19.04	10125	9200	10900	8	9.353807	0.715883518	0.410032
U2TN33	68	6.26	7220	6600	8000	0	6.30832	1	**0.001821**
U2TN34	68	7.91	7870	6600	9400	0	6.980752	1	**0.00093**
U2TN35	68	8.59	7870	6600	9400	0	6.980752	1	**0.00093**
U2TN36	70	3.24	17715	16200	18800	1	17.94593	0.999999984	**3.05E-07**
U2TN37	70	3.93	13315	12500	14200	30	12.86663	**3.156E-05**	0.999987
U2TN38	70	4.64	11155	10600	12000	13	10.47035	0.25498115	0.827813
U2TN39	71	3.933	8970	8600	9400	9	8.1208	0.424309611	0.701527
U2TN40	71	12.558	9175	7700	11900	35	8.356882	**5.51659E-12**	1
U2TN41	80	1.87	15810	14300	19500	3	15.73279	0.999979347	**0.000116**
U2TN42	80	7.86	9470	9100	10400	2	8.650965	0.998311487	**0.008235**
U2TN43	80	8.28	9470	9100	10400	2	8.650965	0.998311487	**0.008235**
U2TN44	80	8.77	9390	9100	9600	1	8.565089	0.999809353	**0.001824**
U2TN45	97	0.86	6690	6300	7400	17	5.772964	**0.000111924**	0.999965
U2TN46	101	4.67	15045	14300	15900	9	14.83322	0.959181708	0.075442
U2TN47	101	5.18	11765	10400	12300	3	11.13938	0.998922223	**0.004425**
U2TN48	202	20.54	16385	15800	16900	16	16.38131	0.570588006	0.528187
U2TN49	202	43.2	6595	5200	7100	6	5.681592	0.501923384	0.657299
U2TN50	214	3.89	4290	3800	4900	3	3.440809	0.668065963	0.549457
U2TY01	1	82.82	7775	7100	8700	0	6.878181	1	**0.00103**
U2TY02	3	8.998	8315	7500	8600	15	7.434856	**0.009544555**	0.995748
U2TY03	6	47.78	23440	21500	25800	26	24.86758	0.436549544	0.639483
U2TY04	7	35.4	23150	19800	25400	23	24.51584	0.647503676	0.431507
U2TY05	10	25.52	17700	16000	18500	7	17.92599	0.998901971	**0.003034**
U2TY06	12	35.86	9540	6700	11100	0	8.758371	1	**0.000157**
U2TY07	22	5.11	9515	8900	10600	3	8.699817	0.992078521	**0.026207**
U2TY08	30	9.133	14395	12600	15700	2	14.09349	0.999988569	**8.66E-05**
U2TY09	30	9.976	9750	8700	10700	9	8.953103	0.538152563	0.593587

Int_ID	Rt #	Cum_Mileage	Ave_AADT	Min_AADT	Max_AADT	Crash_ob	SPF_estimate	P(y>=crash_ob)	P(y<=crash_ob)
U2TY10	63	20.17	15475	15200	15700	1	15.32615	0.999999779	**3.6E-06**
U2TY11	66	26.475	9845	9500	10000	1	9.050225	0.999882635	**0.00118**
U2TY12	66	35.012	19190	17200	20800	33	19.69692	**0.003796203**	0.997872
U2TY13	68	9.19	9645	9200	10100	0	8.836974	1	**0.000145**
U2TY14	70	4.19	13315	12500	14200	6	12.86663	0.988294136	**0.027982**
U2TY15	83	17.103	14305	13600	15100	2	13.98631	0.999987367	**9.51E-05**
U2TY16	83	17.171	14305	13600	15100	1	13.98631	0.999999157	**1.26E-05**
U2TY17	97	0.69	7845	7300	8400	16	6.948115	**0.002239825**	0.999115
U2TY18	110	4.57	12185	11000	13300	19	11.60712	**0.019799**	0.989479
U2TY19	167	9.966	13475	12800	14700	5	13.04823	0.996387408	**0.010402**
U2TY20	190	7.606	13585	13000	14400	7	13.17008	0.976575369	**0.049426**
U2TY21	202	68.182	8900	8500	9600	0	8.047446	1	**0.00032**

Table D.4: SPF estimate for U2X intersections

Int_ ID	Rt #	Cum_ Mileage	Ave_ AADT	Min_ AADT	Max_ AADT	Crash Category 1				Fatal and injury crashes			
						Crash_ ob	SPF_ estimate	P(y>= crash_ob)	P(y<= crash_ob)	Crash_ ob	SPF_ estimate	P(y>= crash_ob)	P(y<= crash_ob)
U2XN01	3	8.404	9650	9000	10700	8	7.636833	0.495306478	0.643098	6	6.583496	0.642979	0.513419
U2XN02	3	8.677	8315	7500	8600	5	6.389306	0.763685341	0.385335	9	6.014911	0.154306	0.915046
U2XN03	3	9.688	11295	10900	11800	7	9.215813	0.812176593	0.299234	2	7.247499	0.994128	**0.02457**
U2XN04	4	23.054	10525	9400	12100	17	8.475929	**0.006440735**	0.997084	7	6.937454	0.540928	0.608032
U2XN05	4	24.819	10530	9800	11100	4	8.475712	0.969381115	0.075446	0	6.94359	1	**0.000965**
U2XN06	4	25.899	8675	8100	9700	4	6.723573	0.902637128	0.199734	3	6.170203	0.945208	0.13665
U2XN07	4	29.159	11055	10500	11600	8	8.982724	0.674075881	0.457931	5	7.1529	0.840505	0.281608
U2XN08	4	40.552	24295	21700	26000	18	23.04736	0.879144574	0.172319	14	11.54293	0.271269	0.811714
U2XN09	5	11.928	16065	14800	17200	35	14.05086	**1.83497E-06**	0.999999	14	8.976125	0.072654	0.959302
U2XN10	6	13.21	17085	15900	17900	4	15.1208	0.999808434	**0.000782**	2	9.320955	0.999076	**0.004813**
U2XN11	6	48.7	20190	19100	21100	35	18.46388	**0.00038861**	0.999806	17	10.3175	**0.03461**	0.981198
U2XN12	6	72.438	15785	14800	17500	26	13.75789	**0.002076372**	0.998975	16	8.881036	**0.019814**	0.990135
U2XN13	32	49.23	13380	12200	14100	27	11.28836	**5.00739E-05**	0.99998	16	8.032354	**0.008527**	0.996134
U2XN14	44	4.613	7800	7400	8500	12	5.919763	**0.018343992**	0.992041	5	5.784609	0.685079	0.480866
U2XN15	44	4.717	4825	4400	5400	5	3.333137	0.243469729	0.878857	5	4.318351	0.433107	0.733608
U2XN16	44	18.786	5145	4500	5800	9	3.600017	**0.011671741**	0.995976	8	4.489354	0.085712	0.960234
U2XN17	44	60.768	17450	16400	19200	0	15.50971	1	**1.84E-07**	0	9.440552	1	**7.94E-05**
U2XN18	44	99.816	10105	9100	10900	11	8.068879	0.191009576	0.88304	0	6.771125	1	**0.001146**
U2XN19	63	24.46	6915	6400	7500	7	5.125391	0.256371846	0.853208	8	5.376163	0.175491	0.90457
U2XN20	63	26.37	5070	4700	5400	4	3.535444	0.470995977	0.718736	5	4.451872	0.458738	0.711129
U2XN21	63	27.36	3510	3000	4100	3	2.279977	0.398644497	0.803405	1	3.555285	0.971427	0.130159
U2XN22	67	19.98	10720	10400	11300	2	8.65798	0.998322071	**0.00819**	3	7.020355	0.970815	0.08071
U2XN23	68	3.62	10150	8500	12100	5	8.119956	0.907032467	0.180493	6	6.782685	0.670681	0.482567
U2XN24	68	5.34	7995	7000	9000	10	6.101036	0.091054842	0.953064	15	5.868367	**0.001132**	0.999597
U2XN25	68	5.55	7995	6600	10200	3	6.111708	0.942833721	0.141511	7	5.858258	0.370943	0.763253
U2XN26	70	3.01	17715	16200	18800	20	15.79479	0.173687463	0.879348	13	9.525673	0.165745	0.896544
U2XN27	70	6.55	5220	4400	6000	2	3.665829	0.880634292	0.291261	17	4.525303	5.66E-06	0.999999

Int_ ID	Rt #	Cum_ Mileage	Ave_ AADT	Min_ AADT	Max_ AADT	Crash Category 1				Fatal and injury crashes			
						Crash_ ob	SPF_ estimate	P(y>= crash_ob)	P(y<= crash_ob)	Crash_ ob	SPF_ estimate	P(y>= crash_ob)	P(y<= crash_ob)
U2XN28	70	9.94	8855	7800	10100	5	6.89421	0.817136974	0.314423	4	6.244594	0.869324	0.253649
U2XN29	71	4.092	8970	8600	9400	10	6.995368	0.169034677	0.901808	11	6.299344	0.056247	0.972285
U2XN30	71	11.548	9760	7300	12800	7	7.760893	0.656558808	0.486805	9	6.611576	0.221851	0.867605
U2XN31	71	15.111	8715	7600	9500	4	6.762968	0.90501034	0.195733	4	6.185268	0.864579	0.261023
U2XN32	71	16.161	11390	10900	11900	4	9.309701	0.982966907	**0.045372**	2	7.283649	0.994312	0.023903
U2XN33	74	1.37	5265	4600	6500	2	3.704438	0.884204698	0.284683	3	4.548178	0.831769	0.334233
U2XN34	74	5.151	7510	6400	7900	0	5.65783	1	**0.00349**	0	5.652378	1	**0.003509**
U2XN35	74	6.081	9860	9200	10300	12	7.833895	0.10030782	0.943867	8	6.672106	0.352557	0.770732
U2XN36	79	1.56	8835	8400	9300	1	6.869836	0.998961353	**0.008174**	2	6.241302	0.985899	0.052029
U2XN37	79	9.87	7575	6900	8300	2	5.716388	0.977892483	0.075887	3	5.682212	0.922254	0.181893
U2XN38	80	7.14	14835	13600	16000	13	12.77489	0.511945852	0.597777	17	8.550903	**0.006989**	0.996809
U2XN39	80	19.09	3600	3200	3900	4	2.34828	0.210538977	0.910505	12	3.613239	**0.000382**	0.999896
U2XN40	83	24.435	2990	2500	3200	5	1.880906	**0.042547026**	0.987361	5	3.226828	0.224181	0.891511
U2XN41	85	11.998	4380	3400	4900	0	2.970505	1	0.051277	0	4.069054	1	**0.017094**
U2XN42	94	1.629	16385	13700	24700	8	14.46582	0.975598809	**0.049221**	9	9.036232	0.549111	0.582635
U2XN43	94	4.389	6155	5700	6800	1	4.459208	0.988428473	0.063171	6	5.008433	0.385519	0.760949
U2XN44	99	2.01	8690	7500	10100	3	6.747276	0.964178838	0.095929	6	6.167535	0.580841	0.579413
U2XN45	113	2.83	1710	1600	1900	0	0.963481	1	0.381562	0	2.298345	1	0.100425
U2XN46	113	2.94	4610	4200	5100	0	3.156505	1	**0.042574**	1	4.199932	0.985003	0.077981
U2XN47	123	4.5	11560	10100	12600	3	9.478715	0.995763864	**0.015089**	2	7.347545	0.994623	**0.022766**
U2XN48	202	27.35	5015	4500	5400	11	3.490027	**0.000996711**	0.999718	4	4.421896	0.644355	0.54699
U2XN49	202	40.16	8990	8700	9700	3	7.014212	0.970679733	0.081028	5	6.307722	0.754026	0.3976
U2XN50	202	42.3	6835	6400	7100	4	5.053605	0.742418332	0.431138	3	5.339243	0.901164	0.22059
U2XY01	2	41.044	12365	10100	14800	5	10.29149	0.975803309	0.056831	5	7.641175	0.877898	0.226358
U2XY02	3	8.952	8467	8300	8600	7	6.527918	0.47786824	0.668671	18	6.08241	**6.74E-05**	0.999947
U2XY03	66	36.537	14183	12900	15400	5	12.10604	0.992943077	**0.019031**	9	8.320787	0.452136	0.67611
U2XY04	101	5.11	11765	10400	12300	16	9.678209	**0.038447**	0.979291	8	7.428024	0.464794	0.671827

Table D.5: SPF estimate for U4T intersections

Int_ ID	Rt #	Cum_ Mileage	Ave_ AADT	Min_ AADT	Max_ AADT	Crash Category 1				Fatal and injury crashes			
						Crash_ ob	SPF_ estimate	P(y>= crash_ob)	P(y<= crash_ob)	Crash_ ob	SPF_ estimate	P(y>= crash_ob)	P(y<= crash_ob)
U4TN01	1	0.811	21490	18900	23000	24	21.72636	0.340547693	0.731786	17	18.74588	0.687949	0.400583
U4TN02	1	2.53	21735	18800	24400	16	21.91606	0.920624588	0.120448	15	19.02251	0.851403	0.213333
U4TN03	1	5.16	16040	11900	19300	26	17.13349	**0.027365999**	0.983433	11	13.14457	0.760525	0.338659
U4TN04	4	44.705	18890	17400	20200	8	19.58679	0.999657127	**0.002017**	3	16.00171	0.999984	**9.3E-05**
U4TN05	4	46.578	15015	14100	15800	0	16.28386	1	**8.47E-08**	2	12.07454	0.999925	**0.00049**
U4TN06	5	36.286	18775	17700	21300	24	19.49077	0.17979788	0.870143	22	15.88245	0.08431	0.945322
U4TN07	5	36.445	21980	19300	24900	11	22.11929	0.996689191	**0.007157**	7	19.2788	0.999573	**0.001259**
U4TN08	5	36.531	21980	19300	24900	23	22.11929	0.453731808	0.627731	12	19.2788	0.969627	0.053705
U4TN09	5	52.198	14585	14100	15500	5	15.90746	0.999570115	**0.001478**	1	11.65207	0.999991	**0.00011**
U4TN10	5	53.697	13165	10600	14900	17	14.62578	0.300442545	0.779736	5	10.30499	0.976016	0.056392
U4TN11	10	0.36	18010	17400	18700	44	18.85276	**9.07625E-08**	1	46	15.08753	**1.23E-10**	1
U4TN12	10	0.62	18010	17400	18700	69	18.85276	**9.07625E-08**	1	77	15.08753	**1.23E-10**	1
U4TN13	10	5.296	25010	23000	30000	33	24.54035	0.059137353	0.959468	29	22.58888	0.109633	0.922443
U4TN14	30	7.366	14285	13600	15000	19	15.64362	0.228761394	0.836308	11	11.35891	0.582193	0.532753
U4TN15	44	40.116	21725	21000	22200	5	21.92616	0.999996501	**1.62E-05**	2	18.98417	1	**1.14E-06**
U4TN16	44	41.673	26710	25300	28100	45	25.89253	**0.000227192**	0.999959	24	24.45421	0.563614	0.517194
U4TN17	44	41.916	26890	25900	28100	10	26.03357	0.999888679	**0.000306**	7	24.65523	0.999992	**2.95E-05**
U4TN18	44	48.517	20395	18700	23000	9	20.82912	0.998765913	**0.003063**	6	17.58453	0.99956	**0.001388**
U4TN19	44	52.161	18745	17600	22100	59	19.46383	**2.18223E-07**	1	33	15.85411	**0.000111**	0.999949
U4TN20	44	52.87	15270	14900	16000	13	16.50695	0.838305191	0.235233	4	12.32524	0.998219	**0.006049**
U4TN21	44	54.85	11665	11200	12000	16	13.28955	0.262557595	0.813996	3	8.860909	0.993034	**0.023412**
U4TN22	44	55.608	11090	9800	12900	15	12.752	0.299812693	0.785116	13	8.337397	0.081421	0.954739
U4TN23	44	55.85	11090	9800	12900	5	12.752	0.995532232	**0.012607**	2	8.337397	0.997765	**0.010556**
U4TN24	44	55.944	11090	9800	12900	17	12.752	0.147196644	0.903577	12	8.337397	0.137803	0.918579
U4TN25	44	56.493	11970	10300	14200	32	13.55585	**1.39562E-05**	0.999994	23	9.160832	**8.64E-05**	0.999968
U4TN26	44	56.714	11970	10300	14200	35	13.55585	**8.39881E-07**	1	22	9.160832	**0.000222**	0.999914
U4TN27	44	62.753	17450	16400	19200	0	18.37738	1	**1.04E-08**	0	14.51746	1	**4.96E-07**

Int_ ID	Rt #	Cum_ Mileage	Ave_ AADT	Min_ AADT	Max_ AADT	Crash Category 1				Fatal and injury crashes			
						Crash_ ob	SPF_ estimate	P(y>= crash_ob)	P(y<= crash_ob)	Crash_ ob	SPF_ estimate	P(y>= crash_ob)	P(y<= crash_ob)
U4TN28	66	5.668	26050	24400	27800	18	25.37441	0.947539347	0.080828	25	23.71875	0.423187	0.653678
U4TN29	66	5.912	26800	25700	28100	13	25.96289	0.998161595	**0.003912**	16	24.55499	0.972893	**0.038896**
U4TN30	66	5.912	26800	25700	28100	13	25.96289	0.998161595	**0.003912**	16	24.55499	0.972893	**0.0452**
U4TN31	66	6.61	16795	15700	19900	18	17.81644	0.514088775	0.57942	37	13.8575	**1.9E-07**	1
U4TN32	66	6.783	16795	15700	19900	9	17.81644	0.992139708	**0.016981**	18	13.8575	0.162798	0.890382
U4TN33	66	7.802	23060	21400	25800	30	22.99733	0.091425536	0.936056	16	20.43481	0.864742	0.194221
U4TN34	71	18.8	19150	17700	19900	15	19.80706	0.887442875	0.16672	11	16.26695	0.931262	0.114297
U4TN35	71	18.871	15095	12700	18100	7	16.33301	0.996781995	**0.00818**	3	12.18049	0.999552	**0.001993**
U4TN36	71	19.123	15095	12700	18100	13	16.33301	0.827990704	0.24826	8	12.18049	0.918088	0.143554
U4TN37	80	0.9	23930	22900	26500	33	23.69615	**0.040619935**	0.973013	35	21.37901	**0.004186**	0.997594
U4TN38	82	27.73	19405	17700	20600	38	20.01809	**0.000220831**	0.999889	26	16.53427	**0.018822**	0.98897
U4TN39	104	0.33	21500	18500	23300	22	21.73599	0.505864532	0.578734	48	18.75431	**7.85E-08**	1
U4TN40	104	0.39	21500	18500	23300	18	21.73599	0.816873181	0.249665	22	18.75431	0.255518	0.80943
U4TN41	104	0.74	26035	23100	28600	19	25.35214	0.918538328	0.119558	14	23.71916	0.987692	**0.022538**
U4TN42	104	0.797	26035	23100	28600	13	25.35214	0.997411759	**0.005391**	6	23.71916	0.999996	**1.63E-05**
U4TN43	104	0.92	26035	23100	28600	14	25.35214	0.994608815	**0.010467**	12	23.71916	0.997039	**0.00627**
U4TN44	104	1.63	26035	23100	28600	55	25.35214	**0.000144445**	1	37	23.71916	**0.006918**	0.995829
U4TN45	104	2.04	20395	17700	22600	6	20.82236	0.999961786	**0.000141**	4	17.59433	0.999975	**0.000116**
U4TN46	113	4.64	10935	9300	12500	7	12.60712	0.967489691	0.066137	8	8.196229	0.574078	0.565178
U4TN47	113	6.9	11205	9700	11700	8	12.86387	0.942018431	0.106148	6	8.437055	0.845605	0.262949
U4TN48	159	2.24	12465	11400	13700	9	14.01365	0.938358972	0.108755	3	9.617132	0.996214	**0.013656**
U4TN49	202	57.06	8900	8500	9600	0	10.68762	1	**2.28E-05**	0	6.361585	1	**0.001727**
U4TN50	202	57.35	8900	8500	9600	0	10.68762	1	**2.28E-05**	0	6.361585	1	**0.001727**
U4TY01	1	17.57	18735	18100	19400	16	19.46188	0.813793745	0.257681	15	15.83442	0.616914	0.483254
U4TY02	3	3.243	20400	19200	21200	23	20.84162	0.346486675	0.727906	31	17.5773	**0.002371**	0.998738
U4TY03	3	6.533	18700	15600	22600	7	19.41002	0.999611178	**0.001155**	10	15.83068	0.952957	0.083355
U4TY04	5	8.97	16633	14600	18200	10	17.67193	0.981652502	**0.035656**	9	13.70259	0.928297	0.124244
U4TY05	99	7.8	19520	16000	22300	10	20.10042	0.995288704	**0.010242**	9	16.6735	0.984853	**0.030897**

						Crash Category 1				Fatal and injury crashes			
Int_ ID	Rt #	Cum_ Mileage	Ave_ AADT	Min_ AADT	Max_ AADT	Crash_ ob	SPF_ estimate	P(y>= crash_ob)	P(y<= crash_ob)	Crash_ ob	SPF_ estimate	P(y>= crash_ob)	P(y<= crash_ob)
U4TY06	99	8.29	25345	23500	26700	9	24.81891	0.999914389	**0.00025**	12	22.93646	0.995415	**0.009425**
U4TY07	99	8.74	21650	19900	23500	6	21.85724	0.99998292	**6.58E-05**	2	18.916	1	**1.21E-06**
U4TY08	113	0.88	8725	8400	9000	11	10.51887	0.48159263	0.636498	24	6.20785	**4.61E-08**	1

Table D.6: SPF estimate for U4X intersections

Int_ ID	Rt #	Cum_ Mileage	Ave_ AADT	Min_ AADT	Max_ AADT	Crash Category 1				Fatal and injury crashes			
						Crash_ ob	SPF_ estimate	P(y>= crash_ob)	P(y<= crash_ob)	Crash_ ob	SPF_ estimate	P(y>= crash_ob)	P(y<= crash_ob)
U4XN01	4	46.665	15015	14100	15800	11	15.44394	0.901567352	0.157042	12	14.14773	0.752224	0.344075
U4XN02	5	17.285	14700	13700	17000	72	15.02357	**2.44308E-08**	1	55	13.86575	**8.27E-12**	1
U4XN03	5	36.114	18775	17700	21300	40	20.77412	**0.000116013**	1	25	17.48084	0.05265	0.966474
U4XN04	5	36.378	18775	17700	21300	13	20.77412	0.972689925	**0.047795**	11	17.48084	0.960896	0.068971
U4XN05	5	52.237	14525	13900	15500	2	14.78024	0.999993986	**4.76E-05**	1	13.71013	0.999999	**1.63E-05**
U4XN06	5	52.275	14585	14100	15500	2	14.8605	0.999994422	**4.44E-05**	1	13.76383	0.999999	**1.55E-05**
U4XN07	5	53.527	13285	10600	14900	12	13.18163	0.665005907	0.443283	11	12.59368	0.711801	0.395637
U4XN08	6	73.33	9740	9400	10300	17	8.704154	**0.008225401**	0.996181	15	9.390967	0.055544	0.969328
U4XN09	6	73.77	9360	8400	9800	3	8.260495	0.988785345	**0.035502**	6	9.043253	0.886911	0.202869
U4XN10	6	74.6	9610	8900	11100	2	8.561284	0.998170224	**0.008843**	7	9.270897	0.816793	0.293138
U4XN11	44	40.565	28250	26800	29400	11	35.6746	0.999999595	**1.36E-06**	4	25.73943	1	**1.42E-07**
U4XN12	44	41.852	26890	25900	28100	31	33.41382	0.685204961	0.380146	19	24.56491	0.893367	0.152604
U4XN13	44	49.757	17075	15300	18700	6	18.33583	0.999747502	**0.000827**	4	15.97673	0.999905	**0.000408**
U4XN14	44	49.831	16835	15300	18600	5	17.98848	0.999915053	**0.000327**	5	15.76471	0.999521	**0.001635**
U4XN15	44	49.977	16835	15300	18600	5	17.98848	0.999915053	**0.000327**	8	15.76471	0.988493	**0.024979**
U4XN16	44	50.034	16595	15300	18600	10	17.64878	0.981424472	**0.036059**	11	15.55192	0.90599	0.150811
U4XN17	44	51.77	18745	17600	22100	21	20.73579	0.505982851	0.580584	21	17.45385	0.22712	0.834718
U4XN18	44	51.82	18745	17600	22100	31	20.73579	**0.020841678**	0.987063	24	17.45385	0.078991	0.948084
U4XN19	44	52.238	18745	17600	22100	102	20.73579	**5.53073E-05**	1	92	17.45385	**1.01E-08**	1
U4XN20	44	55.994	11090	9800	12900	10	10.35291	0.585373513	0.538978	10	10.61722	0.616549	0.506292
U4XN21	44	56.031	11970	10300	14200	11	11.46607	0.594430498	0.523807	11	11.41203	0.588279	0.530204
U4XN22	44	58.166	13195	12900	14000	20	13.01232	**0.043004249**	0.97477	14	12.51872	0.374203	0.723211
U4XN23	44	63.071	17450	16400	19200	0	18.84911	1	**6.52E-09**	0	16.31102	1	**8.25E-08**
U4XN24	44	63.905	17450	16400	19200	0	18.84911	1	**6.52E-09**	0	16.31102	1	**8.25E-08**
U4XN25	66	6.289	21690	20200	22900	12	25.14324	0.998698198	**0.002906**	18	20.04136	0.706103	0.377939
U4XN26	80	0.857	23930	22900	26500	14	28.64495	0.999106949	**0.001935**	12	21.99546	0.992352	**0.01515**
U4XN27	80	0.91	23930	22900	26500	31	28.64495	0.354242654	0.710789	36	21.99546	**0.003743**	0.997845

Int_ ID	Rt #	Cum_ Mileage	Ave_ AADT	Min_ AADT	Max_ AADT	Crash Category 1				Fatal and injury crashes			
						Crash_ ob	SPF_ estimate	P(y>= crash_ob)	P(y<= crash_ob)	Crash_ ob	SPF_ estimate	P(y>= crash_ob)	P(y<= crash_ob)
U4XN28	80	1	23930	22900	26500	14	28.64495	0.999106949	**0.001935**	17	21.99546	0.882778	0.169232
U4XN29	100	0.15	11190	10400	12000	9	10.46689	0.717233248	0.401039	22	10.70903	**0.001635**	0.999262
U4XN30	104	0.58	14660	14300	14900	20	14.9584	0.122475047	0.918756	17	13.8312	0.229652	0.839013
U4XN31	187	1.777	13325	12100	16000	15	13.20867	0.34632993	0.744857	17	12.63257	0.139236	0.909521
U4XN32	187	1.816	13325	12100	16000	2	13.20867	0.999973932	**0.000186**	1	12.63257	0.999997	**4.45E-05**
U4XN33	187	1.872	18925	18400	19400	42	20.97962	**7.07433E-05**	1	25	17.61448	0.056397	0.963844
U4XN34	187	2.572	10435	9200	13200	26	9.558543	**8.29604E-06**	0.999997	19	10.02159	**0.007341**	0.996464
U4XN35	349	1.871	13960	12900	15200	3	14.02994	0.999908446	**0.000463**	4	13.20392	0.999106	**0.003229**
U4XN36	401	1.8	16005	14800	17000	11	16.81244	0.946391975	0.09156	5	15.02891	0.999162	**0.002737**
U4XN37	401	1.862	16005	14800	17000	3	16.81244	0.999992052	**4.75E-05**	1	15.02891	1	**4.76E-06**
U4XN38	502	4.59	15005	14300	17000	19	15.43445	0.2124975	0.84961	16	14.13837	0.344393	0.743825
U4XN39	502	4.869	15005	14300	17000	13	15.43445	0.766812051	0.322932	7	14.13837	0.986929	**0.029295**
U4XN40	529	1.166	19310	17700	20800	39	21.55867	**0.000460024**	0.999758	23	17.95244	0.197636	0.845585
U4XY01	1	8.37	21350	12600	34800	26	25.54106	0.4900227	0.587647	25	19.65847	0.138395	0.902393
U4XY02	5	6.953	18735	17300	20300	27	20.71313	0.105004316	0.927062	17	17.4458	0.574551	0.521182
U4XY03	5	7.562	17367	16300	18200	4	18.7293	0.999990525	**4.71E-05**	7	16.23732	0.996572	**0.008669**
U4XY04	6	93.603	16050	15400	16600	14	16.86767	0.79027679	0.291537	9	15.0697	0.963887	0.067626
U4XY05	99	7.702	21680	20800	22200	6	25.11956	0.999998732	**5.57E-06**	7	20.03339	0.999751	**0.000761**
U4XY06	99	7.754	19520	16000	22300	3	21.90455	0.999999919	**6.18E-07**	8	18.13388	0.997344	**0.006519**
U4XY07	99	8.38	25345	23500	26700	29	30.90639	0.658419446	0.411145	21	23.22537	0.705971	0.371693
U4XY08	99	9.1	21650	19900	23500	24	25.09782	0.61356253	0.465635	14	20.00487	0.934004	0.104676

Appendix E: Physical Characteristics of Intersections

Figure E.1: Physical characteristics of intersections – R2TY12

Figure E.2: Physical characteristics of intersections – U2TY02

Figure E.3: Physical characteristics of intersections – U2TY12

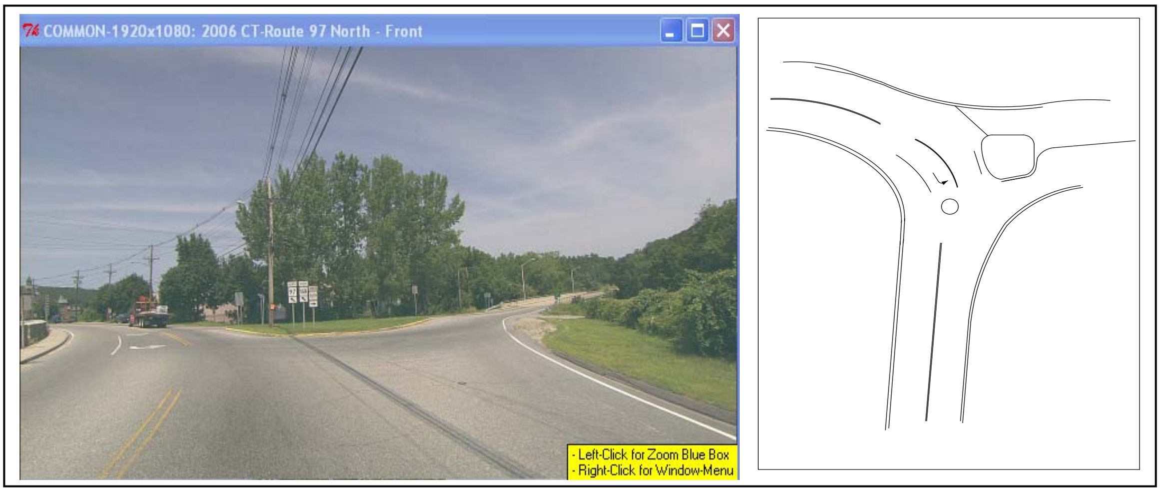

Figure E.4: Physical characteristics of intersections – U2TY17

Figure E.5 Physical characteristics of intersections – U2TY18

Figure E.6: Physical characteristics of intersections – U2XY04

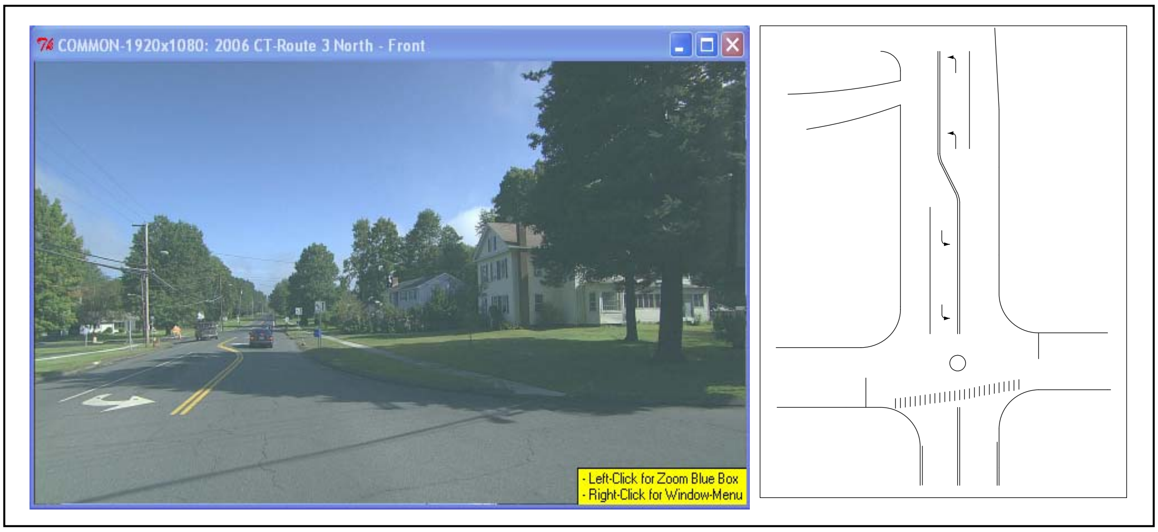

Figure E.7: Physical characteristics of intersections – U2XY02

Figure E.8: Physical characteristics of intersections – U4TY02

Figure E.9: Physical characteristics of intersections – U4TY08