

12-2011

# Evaluation of NDOR's Actuated Advance Warning Systems

Justice Appiah

*University of Nebraska-Lincoln, jappiah2@unl.edu*

Laurence Rilett

*University of Nebraska - Lincoln, lrilett2@unl.edu*

Zifeng Wu

*University of Nebraska-Lincoln*

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TRANSPORTATION CENTER**



**Report TSPT1-03**

**Final Report**  
26-1118-0086-001

## **Evaluation of NDOR's Actuated Advance Warning Systems**

### **Justice Appiah, Ph.D.**

Postdoctoral Research Associate  
Department of Civil Engineering  
University of Nebraska-Lincoln

### **Laurence R. Rilett, Ph.D., P.E.**

Keith W. Klaasmeyer Chair in Engineering and Technology  
Director, Nebraska Transportation Center  
Distinguished Professor of Civil Engineering  
University of Nebraska-Lincoln

### **Zifeng Wu**

Graduate Research Assistant

---

### **2011**

Nebraska Transportation Center  
262 WHIT  
2200 Vine Street  
Lincoln, NE 68583-0851  
(402) 472-1975

"This report was funded in part through grant[s] from the Federal Highway Administration [and Federal Transit Administration], U.S. Department of Transportation. The views and opinions of the authors [or agency] expressed herein do not necessarily state or reflect those of the U. S. Department of Transportation."

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Justice Appiah, Ph.D.  
Postdoctoral Researcher  
Nebraska Transportation Center  
University of Nebraska–Lincoln

Laurence R. Rilett, Ph.D., P.E.  
Keith W. Klaasmeyer Chair in Engineering and Technology  
Director, Nebraska Transportation Center  
Distinguished Professor of Civil Engineering  
University of Nebraska–Lincoln

Zifeng Wu  
Graduate Research Assistant  
Department of Civil Engineering  
University of Nebraska–Lincoln

A Report on Research Sponsored by

Nebraska Department of Roads

Nebraska Transportation Center  
University of Nebraska–Lincoln

December 2011

## Technical Report Documentation Page

1. Report No. NTC Report TSPT1-03	2. Government Accession No.	3. Recipient's Catalog No.	
4. Title and Subtitle Evaluation of NDOR's Actuated Advance Warning Systems		5. Report Date December 2011	
		6. Performing Organization Code	
7. Author(s) Justice Appiah, Laurence R. Rilett, and Zifeng Wu		8. Performing Organization Report No. NTC Report TSPT1-03	
9. Performing Organization Name and Address Nebraska Transportation Center 2200 Vine St. 262 Whittier Research Center PO Box 830851 Lincoln, NE 68583-0851		10. Work Unit No. (TRAIS)	
		11. Contract or Grant No.	
12. Sponsoring Agency Name and Address Nebraska Department of Roads 1500 Nebraska Highway 2 Lincoln, Nebraska 68509-4567		13. Type of Report and Period Covered Draft Report	
		14. Sponsoring Agency Code	
15. Supplementary Notes			
16. Abstract Driver behavior within the dilemma zone can be a major safety concern at high-speed signalized intersections. The Nebraska Department of Roads (NDOR) has developed and implemented an actuated advance warning (AAW) dilemma zone protection system. Although these systems have received positive reviews from the public—and commercial vehicle operators in particular—there has been no comprehensive analysis of their effects on safety and traffic operations. The focus of this research was to conduct a quantitative study to ascertain the efficacy of the NDOR advance warning system. First, crash records from before and after the implementation of the system at 26 intersections were compared. In addition, 29 control intersections were used to compare crash rates over time, and a fully Bayesian technique was employed to ensure that no exogenous variables affected the study. Results of the safety analysis were promising (a 43.6% reduction in right-angle crashes) and suggested that the use of the system should be encouraged as an effective safety treatment for the dilemma zone problem at high-speed signalized intersections. Second, a non-intrusive data collection system was used to monitor traffic and to collect a continuous stream of data up to 1,000 ft upstream of the stop line at two high-speed signalized intersections equipped with the system. The results suggested that the system was effective at alerting drivers to the impending end of the green signal: approximately 78% of drivers observed in this study either maintained their speeds or slowed down when the signs began to flash. It was also found that the number of vehicles in their dilemma zones when the signal indication changed from green to amber was 77.2% smaller than the number that would have been expected if the NDOR AAW system had not been installed. Finally, a modeling framework was developed that could be used to perform consistent, detailed analyses of these systems. Results from two demonstration studies indicated that the proposed procedure had potential for studying these systems in a microsimulation environment.			
17. Key Words		18. Distribution Statement	
19. Security Classif. (of this report) Unclassified	20. Security Classif. (of this page) Unclassified	21. No. of Pages 115	22. Price

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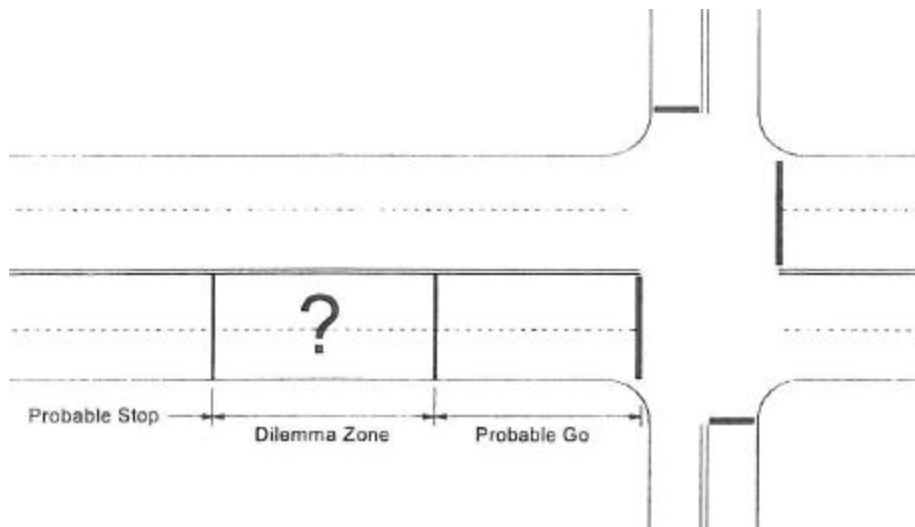
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## Chapter 1 Introduction

### 1.1 Background

As a traffic signal indication changes from green to amber, a driver approaching the intersection must decide whether to stop or proceed through the intersection. A situation in which neither decision is satisfactory occurs on a section of the roadway upstream of the intersection known as the dilemma zone, as shown in figure 1.1.



**Figure 1.1** Dilemma Zone (Source: McCoy and Pesti, 2002)

A driver upstream of the dilemma zone, who is traveling at the legal speed limit at the onset of the amber indication, can come to a stop without entering the intersection at a comfortable deceleration rate. A driver downstream of the dilemma zone, who is traveling at the legal speed limit at the onset of the amber indication, is able to clear the intersection before the end of the intergreen period (amber and all-red indications). However, a dilemma zone exists when a driver has neither sufficient distance to bring his/her vehicle to a complete stop nor

sufficient intergreen time to proceed safely through the intersection before the signal indication changes to red. Theoretically, it is possible to eliminate the dilemma zone with proper signal timing. However, the stochastic nature of driving means that some drivers will invariably find themselves in the dilemma zone. For example, they may misjudge the distances involved and elect to stop when they should proceed, they may have slower perception/reaction times than the design driver, or their vehicles may lack the necessary braking power required.

Drivers exhibit distinct differences in their desire or ability to stop when they are in the dilemma zone at the onset of the amber indication. Some drivers may stop abruptly, therefore increasing the risk of a rear-end collision. Other drivers might proceed through the intersection which increases the risk of red-light running and the possibility of a right-angle collisions with vehicles entering the intersection from the cross road. Due to the safety risks associated with drivers making incorrect decisions related to the decision to either stop or proceed through the intersection at the onset of amber, a number of dilemma zone protection strategies have been developed. These strategies 1) reduce the likelihood of a driver being in the dilemma zone at the onset of the amber indication, 2) increase the awareness of the driver that the green indication will be changing from red to amber in the near future thus allowing them a greater probability of choosing the appropriate action, or 3) both 1 and 2.

One common strategy used to provide dilemma zone protection at high-speed signalized intersections involves the use of active advance warning systems. These systems provide information, via flashing signal heads and warning signs, to drivers regarding whether or not they should be prepared to stop as they approach a signalized intersection. An example of a sign and signal head is provided in figure 1.2. The flashing signal head(s) is (are) activated at a predetermined time before the end of the green interval. The goal is to alert drivers to the

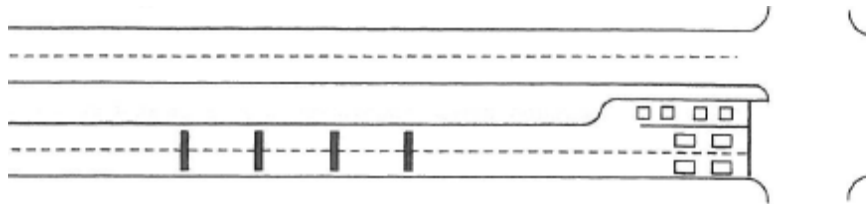
imminent loss of green and hopefully reduce indecision and variability in driver behavior at the onset of the amber interval.



**Figure 1.2** Active Advance Warning System

The documented effectiveness of AAW systems varies widely from study to study. Some studies (Obeng-Boampong 2004; Radalj 2003; Sunkari et al. 2005) have shown that AAW systems are very effective at reducing both red-light running and approach speed. Conversely, other studies (Pant and Xie 1995; Farraher et al. 1999) recommend caution in their use because they could inadvertently encourage some drivers to accelerate in order to make the green. Sayed et al. (1999) reported that after installing AAW systems in British Columbia, some intersections experienced crash reductions while others did not. In some cases, while the total number of collisions was reduced, the number of rear-end collisions increased and the number of lateral collision was reduced.

Advance detection (AD) systems have also been used to provide dilemma zone protection for high-speed approaches to isolated signalized intersections. In general, AD systems involve the installation of two to four detectors in each through lane of the high-speed approach.

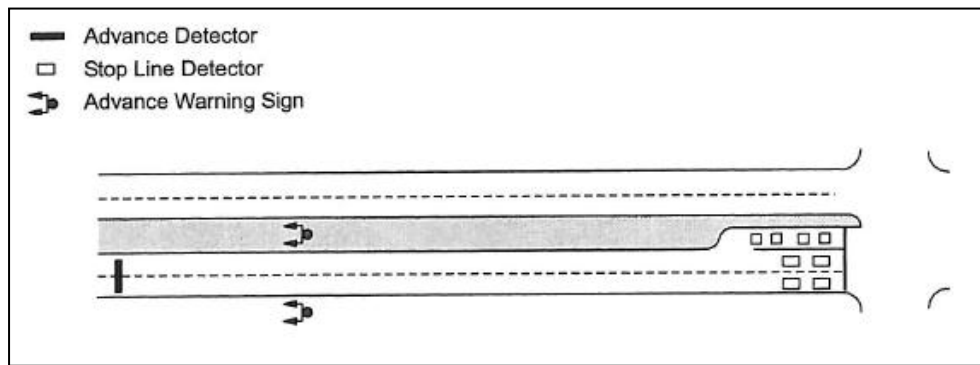


**Figure 1.3** Conventional Advance Detection System (Source: McCoy and Pesti, 2002)

The detector configuration (figure 1.3) enables the traffic signal controller to extend the green and prevent the onset of amber while approaching vehicles travel through the dilemma zone. The amount of green time that is extended is known as the “passage time”. This varies from system to system but most systems use a value of approximately 2 s per detector actuation (a total of 8 s for figure 1.3 above). Consequently, this reduces the number of drivers that have to make a decision whether to stop or proceed through the intersection. This green signal extension countermeasure has been found to reduce both the number of conflicts and the incidence of red-light running at signalized intersections (Pant et al. 2005; Zimmerman 2009). However, AD systems could be counterproductive as they tend to increase the likelihood that the green will be extended until it reaches the maximum green time allowable under the signal timing plan. At this point the signal transitions to amber and any dilemma zone protection is lost (Bonneson and McCoy 1994). This phenomenon is known as “max out” and is a key metric in analyzing the effectiveness of an AD system. Pant et al. (2005) indicated that while increasing

the green time by up to 3.0 seconds was beneficial, longer green time extensions actually resulted in an increase in conflicts.

In an effort to overcome this problem, the Nebraska Department of Roads (NDOR) developed an actuated advance warning (AAW) system that combines advance detection and advance warning. The detector layout for the NDOR AAW system is shown in figure 1.4.



**Figure 1.4** Schematic Diagram of NDOR AAW System (Source: McCoy and Pesti, 2002)

The system has one advance detector in each approach lane. Stop line detection is also provided in the through lanes and left-turn bays. The range of stop line detection is 30 to 40 ft in the through lanes and 40 to 50 ft in the left-turn bays. The advance detector operates in the pulse mode, which means that each vehicle crossing the detector transmits a single pulse to the controller, regardless of the time that the vehicle spends in the detection area. The stop line detectors operate in the presence mode (a continuous call is transmitted to the controller as long as a vehicle is within the detection area) but are not active during the extendible portion of the green interval (McCoy and Pesti 2002).

In addition to the advance detector, two flashing signal heads are mounted on top of advance warning signs with the legend, “PREPARE TO STOP WHEN FLASHING.” One

warning sign/flashing signal heads assembly is positioned on either side of the roadway approach downstream of the advance detector (see figure 1.5).



**Figure 1.5** Warning Sign/Flashing Signal Heads Assembly

The design algorithm continually monitors the upstream detector, as well as traffic at the intersection, in order to predict the onset of the amber signal indication. Depending on the approach speed, the signal heads are designed to flash from 5 s to 7 s before the onset of the amber indication. This is known as the lead flash or advance warning before end of green. The lead flash is dependent on several variables including approach speed, location of the advance detector and location of advance warning signs. Values recommended in the study by McCoy and Pesti (2002) are reproduced in table 1.1.

**Table 1.1** Design Parameters for the NDOR Actuated Advance Warning System\*

Design Speed (mph)	Distance Between Warning Sign and Stop Line (ft)	Distance Between Advance Detector and Stop Line (ft)	Lead Flash (Advance Warning Before End of Green) (s)
65	650	935	7
60	550	814	6
55	450	693	6
50	375	594	5
45	300	498	5

\*Passage time setting of 3.0 s

For each vehicle detected during the extensible portion of the green interval, the controller extends the green by an amount of time equal to the passage time setting on the controller. The passage time setting is typically set to 3 seconds. The number of extensions is a function of the number of vehicle detections at the advance detector location. The detector layout shown in figure 1.4 ensures that the maximum allowable headway, the largest headway at which no further green time extensions are allowed, equals the passage time setting on the controller (e.g. 3 s). It is important to note that this value is much shorter than the maximum allowable headway for conventional AD systems which are typically on the order of 10 s. As a result, the frequency of losing the dilemma zone protection through max-out (i.e. terminating the green interval at the preset maximum), as well as the average waiting time of vehicles on the cross road, is substantially reduced (McCoy and Pesti 2002). As of July, 2009, 35 intersections on the Nebraska state highway system had been equipped with NDOR AAW systems (see table 1.2).



**Table 1.2** Inventory of Intersections Equipped with AAW Systems in Nebraska

City/County	Highway	Cross Street	Date AAW Activated	Left-Turn From:
Beatrice	US-136	Orange Blvd	19991018	
Beatrice	US-77	Wal-Mart	20000119	N
Columbus	US-30	US-81 (S. JCT)	20040526	E,W
Columbus	US-30	29th Ave East	20040602	
Doniphan	US-34	S-40B	19990901	NB & SB
Douglas Co.	N-133	State St	20080619	W
Douglas Co.	N-36	N-133		
Douglas Co.	N-36	72nd St	20071031	W
Douglas Co.	US-6	Q St	20001016	
Grand Island	US-281	Airport Rd	19971211	EB
Grand Island	US-34	US-281	20001207	N, S, E, W
Grand Island	US-34/281	Wood River Rd	20000915	NB
Hastings	US-6	Showboat Rd	20040519	
Kearney	US-30	30th Ave	20010108	
Lancaster Co.	US-77	Saltillo Rd	19971105	N-S
Lincoln	US-34	N-79	20071218	
Lincoln	US-34	NW 48th St.	20071218	
Lincoln	US-6	Dorsey Lab Dr	20030623	
Lincoln	US-77	Old Cheney Rd	20011003	N,S
Lincoln	US-77	Pioneers	20001103	
McCook	US-6	Wal-Mart/Wedgwood	20040628	WB TO SB
McCook	US-83	J St	20040628	
Neb City Bypass	US-75	N-2 (S.JCT Bypass)	20040630	N,S,E,W,
Norfolk	US-275	N-24	20031001	
Norfolk	US-81	Ta-Ha-Zouka	20040602	
Papillion	N-370	108th St	20030818	E
Plattsmouth	US-75	N-66	2004????	
Plattsmouth	US-75	Ave B	20040405	NONE
Sarpy Co.	N-370	168th St	20020110	W
Sarpy Co.	N-370	132nd St	20040610	E, W
Sarpy Co.	US-75	Laplatte Rd	19980204	
Sarpy Co.	US-75	Platteview Rd	19980204	
Sidney	L17J	Old Post Rd	19970813	
Waverly Int	US-6	I-80 Ramp	20040623	
York	US-81	Lincoln Ave	20060608	

As presented in the study by McCoy and Pesti (2002), broad criteria for installing an NDOR AAW system vis-à-vis the conventional AD system include the average daily traffic on the main road and cross road, the traffic controller's maximum green time settings, approach speeds, and intersection location, such as rural or urban. Requests from citizens may also influence NDOR's decision to install an AAW system at an intersection (Kent Wohlers, personal communication, April 23, 2010).

While the benefits of these devices have been generally acknowledged by the traveling public and NDOR staff, there has not been a well-documented analysis of their effectiveness and under which conditions they are most effective. Furthermore, there are some general criteria on when these devices should be installed (McCoy and Pesti 2002), but there are currently no standards regarding when they should be removed because of changing demand.

## 1.2 Objectives

The goal of this research was to assess the effectiveness of the actuated advance warning devices in Nebraska in terms of safety and traffic operations efficiency. The specific objectives are listed below:

- Quantify the effectiveness of the NDOR AAW system in terms of safety and efficiency at isolated high-speed signalized intersections;
- Provide guidelines for NDOR engineers with respect to the installation of AAW systems; and
- Provide guidelines for NDOR engineers with respect to removing these devices if conditions change (e.g. traffic volumes increase substantially).

### 1.3 Organization

The report is organized into six chapters. Chapter 1 provides relevant background information and outlines the objectives of the study. Chapter 2 examines the NDOR AAW system with respect to safety. Crash records from across the state are used to compare the crash rates before and after the implementation of the system. In addition, control intersections are used to compare accident rates using a Bayesian technique to ensure that no exogenous variables affect the results. The operational effectiveness of the system, in terms of driver reactions and effects on conflicting traffic, is presented in chapter 3. Chapter 4 develops and calibrates a traffic micro-simulation modeling system that could be used by NDOR engineers to perform consistent, detailed analyses of AAW systems. The calibrated model is used in a sensitivity analysis in Chapter 5. The goal of the sensitivity analysis is to develop guidelines which NDOR traffic engineers may use for i) identifying candidate locations for installing AAW systems and ii) identifying which systems might be candidates for removal because of changing demand. Chapter 6 provides conclusions and recommendations.

## Chapter 2 Safety Analyses

The objective of this chapter was to examine the effectiveness of the actuated advance warning system in Nebraska with respect to safety.

### 2.1 Method

Crash records observed before and after the implementation of the system were compared using a fully Bayesian (FB) approach. The FB approach was adopted because recent studies indicate that uncertainty in the data, used in a before-after study, is better accounted for by a fully Bayesian (FB) approach than by the commonly used empirical Bayes's (EB) method (Persaud et al. 2009, Carriquiry and Pawlovich 2004, Lan et al. 2009). In addition, like the EB method, the FB technique also accounts for the regression-to-mean bias – a situation where the number of crashes at a site generally reverts to the expected mean value even if no treatment were applied (Persaud et al. 2009, Hauer 1997, Hauer 2002).

Rather than using a point estimate of the expected number of crashes and its variance, the FB approach generates a distribution of likely values which are then combined with site-specific crash data to obtain an estimate of the long-term expected crash frequency. Though a relatively complex alternative to the EB approach, the FB approach is desirable because it requires less data, it better accounts for uncertainty in the data, it allows for more detailed causal inferences, and it provides more flexibility in selecting crash count distributions (Persaud et al. 2009).

The Bayesian framework assumes that there is information about model parameters:  $\theta$  quantified by a probability distribution  $p(\theta)$  called the prior (before data are collected). The observed data,  $y$ , also contains information regarding the model parameters which is expressed in the likelihood,  $l(y|\theta)$ . In the context of a before-after safety analysis, the prior information could be the expected crash frequency from a group of similar, but untreated, intersections, whereas

current information is the observed crash frequency at the treated sites. The information in the likelihood and prior are combined to produce an updated probability distribution called the posterior distribution  $p(\theta|y)$ . That is,

$$p(\theta|y) = \frac{p(\theta) \times l(y|\theta)}{C} \quad (2.1)$$

where  $C$  is a normalizing constant.

As shown in equation 2.1, the posterior distribution is proportional to the product of the prior and the likelihood; therefore it is theoretically always available. However, in realistically complex models, the analytic computations required to obtain the normalizing constant and the posterior marginal distributions of individual parameters often are intractable (Cowles 2004). This partly explains why the relatively simpler EB approach, which allows for out-of-sample estimation of the prior, has been widely used in traffic safety analysis instead of the more rigorous FB approach. However, with the development of WinBUGS, a general-purpose software package that uses Markov chain Monte Carlo (MCMC) methods to fit arbitrarily complex Bayesian models the FB approach can be used for traffic safety analysis (Persuad *et al.* 2009, Carriquiry and Pawlovich 2004, Brooks 1998, Spiegelhalter 2003).

## 2.2 Data

The FB model was calibrated using data from 26 treated intersections and a reference population of 29 intersections across the state of Nebraska collected over a thirteen-year period (1996 to 2008). Each treated intersection was fully actuated controlled and had the NDOR actuated advance warning system installed on at least one high-speed approach. A summary of some relevant characteristics of the treated intersections is provided in table 2.1.

**Table 2.1** Summary Data for Treated Intersections

Intersection ID	Jurisdiction (City, County)	Year AD-AWS Activated	Approach with AD-AWS*	Main Road AADT		Cross Road AADT	
				Before	After	Before	After
1	Columbus, Platte	2004	E,W	9420	15831	5940	7076
2	Doniphan, Hall	1999	N,S,E,W	7728	9548	1802	2359
3	Grand Island, Hall	1997	N,S,E,W	5145	5483	2445	4477
4	Grand Island, Hall	2000	N,S	14098	16362	4751	6927
5	Grand Island, Hall	2000	N,S	14098	13946	4751	995
6	Gretna, Sarpy	1998	N,S	16073	20479	2708	2097
7	Gretna, Sarpy	1998	N,S	17971	21477	2391	2422
8	Gretna, Sarpy	2002	E,W	7025	12987	2072	4655
9	Gretna, Sarpy	2004	E,W	10544	14181	3102	5710
10	Hastings, Adams	2004	E	7230	9948	4138	4474
11	Kearney, Buffalo	2001	E,W	6700	9028	3864	4990
12	Lincoln, Lancaster	2003	E,W	15449	16995	5182	6980
13	Lincoln, Lancaster	2000	N,S	10475	11315	1763	4460
14	Lincoln, Lancaster	2001	N,S	8480	9333	5320	8948
15	Malcolm, Lancaster	2007	E,W	7456	10850	4574	4713
16	McCook, Red Willow	2004	W	5321	6821	4011	5103
17	Norfolk, Madison	2003	E,W	5856	6832	5094	5053
18	Norfolk, Madison	2004	S	7234	8464	3811	4071
19	Oak, Lancaster	2007	E,W	6927	7075	2096	1738
20	Omaha, Douglas	2005	N,S,E,W	9242	11113	6061	5919
21	Omaha, Douglas	2007	E,W	4882	4200	3791	3250
22	Plattsmouth, Cass	2004	N,S	10686	11791	5780	7191
23	Plattsmouth, Cass	2004	N,S	10279	10414	5579	7232
24	Roca, Lancaster	1997	N,S	11633	13432	2253	2909
25	Sidney, Cheyenne	1997	N,S	2420	4338	2193	3368
26	Waverly, Lancaster	2004	E,W	15619	17220	5106	7648

\*E = Eastbound, W = Westbound, N = Northbound, S = Southbound

Similar to the treated intersections, all intersections in the reference group were fully actuated. However, the reference group intersections did not have the NDOR AAW system installed at the time of the study. Instead, the reference group intersections were considered as potential candidates for AAW system installation by NDOR (Kent Wohlers, NDOR, personal communication, April 23, 2010).

For each treated intersection, crash data were obtained for periods before and after NDOR AAW system installation. The lengths of the before and after periods varied based on the availability of crash data with 12 months being the minimum. Crash data for the year in which

the NDOR AAW devices were activated were not used in the analysis. Summary crash data for the treated intersections are given in table 2.2.

**Table 2.2** Average Crashes per Year at Treated Intersections

Intersection ID	Rear-end Crashes		Angle Crashes		Injury Crashes		HV* Crashes		Total Crashes		Years	
	Before	After	Before	After	Before	After	Before	After	Before	After	Before	After
1	2.3	1.5	4.0	2.3	2.3	2.0	3.9	2.5	8.3	7.5	8	4
2	0.7	0.0	1.7	1.4	1.7	0.8	0.0	0.6	2.7	3.2	3	9
3	0.0	0.2	5.0	0.9	5.0	0.8	1.0	0.3	8.0	2.3	1	11
4	3.8	3.3	2.5	3.6	0.8	2.6	1.5	1.5	8.3	11.6	4	8
5	1.0	3.0	1.5	0.5	1.3	2.0	1.3	1.9	4.3	8.1	4	8
6	1.0	1.3	2.5	0.5	2.5	1.0	0.5	0.8	11.5	6.6	2	10
7	1.5	1.7	1.5	1.3	1.5	1.8	0.0	0.6	5.5	6.9	2	10
8	1.3	2.5	1.2	0.5	0.8	1.0	0.7	0.2	5.3	6.2	6	6
9	0.6	1.5	1.0	1.5	0.9	1.3	0.5	0.0	4.9	8.3	8	4
10	2.1	2.8	2.5	2.0	1.3	1.3	1.1	2.3	6.8	7.8	8	4
11	0.4	0.6	2.6	3.0	0.0	1.9	0.2	0.6	4.2	6.1	5	7
12	0.6	1.4	0.1	0.4	0.3	0.8	0.1	0.2	1.3	2.0	7	5
13	1.0	1.8	2.5	1.3	0.8	1.5	1.5	0.3	6.0	7.0	4	8
14	2.0	2.3	1.8	0.6	1.6	1.0	0.2	0.7	7.2	7.1	5	7
15	1.2	1.0	0.9	1.0	2.5	2.0	0.2	0.0	7.5	5.0	11	1
16	0.6	1.0	0.5	0.5	0.1	0.3	0.0	0.3	1.3	2.0	8	4
17	0.4	0.4	3.7	1.6	1.7	0.8	1.1	1.2	5.3	4.4	7	5
18	1.0	1.0	1.1	2.3	0.4	0.5	1.4	1.3	4.1	4.8	8	4
19	1.2	1.0	2.4	0.0	1.7	0.0	0.1	1.0	6.2	3.0	11	1
20	2.6	1.7	2.1	1.3	2.3	1.0	1.3	1.0	8.6	6.0	9	3
21	1.0	1.0	0.9	0.0	0.8	0.0	0.3	0.0	3.5	3.0	11	1
22	0.5	0.0	0.9	0.0	0.1	0.0	0.0	0.0	2.0	0.0	8	4
23	0.3	0.3	0.0	0.0	0.3	0.0	0.0	0.0	1.0	0.8	8	4
24	2.0	1.9	6.0	1.8	5.0	1.4	0.0	0.5	10.0	6.1	1	11
25	1.0	0.5	3.0	0.9	3.0	0.5	1.0	0.5	4.0	2.1	1	11
26	0.1	0.0	1.0	0.0	0.8	0.0	0.0	0.0	1.8	0.0	8	4
Average	1.2	1.3	2.0	1.1	1.5	1.0	0.7	0.7	5.4	4.9	6.1	5.9

\*HV denotes crashes involving heavy vehicles: buses, trucks, and farm / construction equipment.

It may be seen from table 2.2 that the total number of crashes was reduced from an average of 5.4 per year before NDOR AAW system installation to 4.9 per year after AAW system installation. However, this reduction in crashes must be viewed with caution as it neither accounts for the regression-to-mean bias nor the effects of exogenous variables such as changes in traffic volume.

Crash count data were retrieved from the Crash Outcome Data Evaluation System (CODES) at the Nebraska Department of Health and Human Services. In order to account for the expected impacts of changes in traffic volume on crash rates, average annual daily traffic (AADT) data were also collected for use in the FB model. The AADT data were obtained from the Nebraska Department of Roads. Similar data were collected for the reference population of 29 intersections (see table 2.3).

**Table 2.3** Summary Data for Reference Intersections

Intersection ID	Average AADT		Average Crashes per Year				
	Main Road	Cross Road	Rear-end	Angle	Injury	HV	Total
1	6384	2141	1.2	2.7	0.7	0.5	5.5
2	8665	5183	5.4	3.2	2.2	1.1	12.6
3	4094	5230	0.8	0.4	0.2	0.2	1.5
4	23088	11050	4.6	4.9	0.8	0.2	10.9
5	23957	15293	8.2	5.5	2.2	0.4	17.9
6	5589	5043	0.7	0.2	0.5	0.2	2.6
7	3871	6093	2.4	3.2	1.1	1.3	9.2
8	12904	4706	4.2	1.2	0.9	0.7	7.8
9	3764	5533	1.4	0.7	0.4	0.2	2.4
10	9950	4061	0.5	0.5	0.3	0.2	1.4
11	9950	4061	1.0	2.8	1.3	0.2	5.1
12	11414	4225	1.3	1.2	0.8	0.5	3.8
13	3585	3626	1.0	1.5	1.2	0.5	4.1
14	2656	2940	0.2	1.2	0.5	0.7	1.9
15	11468	4334	0.2	1.1	0.5	0.7	2.2
16	4364	4858	0.4	1.9	0.9	0.5	4.2
17	18758	8125	0.4	0.1	0.1	0.0	0.7
18	6520	6309	0.8	1.1	0.9	0.4	3.3
19	10011	7155	2.5	3.1	1.5	1.5	10.2
20	10308	8152	3.3	4.5	1.8	0.8	11.4
21	13218	9261	15.4	10.4	3.5	4.2	35.5
22	3085	5060	0.4	1.2	0.5	0.2	2.6
23	7686	5167	2.8	2.8	1.1	0.5	7.2
24	16374	4447	0.7	0.8	0.5	0.2	2.1
25	16953	12783	5.2	3.8	2.6	0.6	13.8
26	9157	3984	0.5	1.8	0.5	0.3	3.2
27	10217	4692	0.6	1.9	0.8	0.5	4.1
28	7945	3816	0.3	0.8	0.6	0.2	2.8
29	6744	5319	2.0	1.5	0.3	0.4	4.3
Average	9748	5953	2.4	2.3	1.0	0.6	6.7



### 2.3 Model

The FB approach to traffic safety analysis was used. Note that the sample size of 29 reference intersections is not large enough to estimate a reliable out-of-sample crash reduction model which is required for the EB approach, thus giving credence to the choice of an FB approach in this study. Based on the literature, four variations of the FB model were considered: the Poisson-Gamma with and without time trends and the Poisson-Lognormal with and without time trends (Persuad et al. 2009; Carriquiry and Pawlovich 2004; Lan et al. 2009). The Poisson-Gamma model with multiplicative non-time-dependent random effects was selected because it had the lowest value of the deviance information criterion statistic (Persuad et al. 2009, Lan et al. 2009). Using this model the expected crash reduction rate,  $R$ , was estimated as follows (Lan et al. 2009, Bonneson et al. 1993, Appiah et al. 2011):

$$R = 1 - \frac{\sum_{i \in I} \sum_{t > T_{0i}} y_{it}}{\sum_{i \in I} \sum_{t > T_{0i}} \delta_i \lambda_{it}} \quad (2.2)$$

where,

$T_{0i}$  = year in which AD-AW was implemented at intersection  $i$ ;

$I$  = set of treated intersections;

$y_{it}$  = observed number of crashes at intersection  $i$  in year  $t$ ;

$y_{it} \sim \text{Poisson}(\delta_i \lambda_{it})$ ;

$\lambda_{it}$  = expected number of crashes at intersection  $i$  in year  $t$ ;

$\ln \lambda_{it} = \beta_0 + \beta_1 \ln(AADTm_{it}/1000) + \beta_2 \ln(AADTc_{it}/1000)$ ;

$AADTm_{it}$  = Main road AADT at intersection  $i$  in year  $t$ ;

$AADTc_{it}$  = Cross road AADT at intersection  $i$  in year  $t$ ;

$\beta_0, \beta_1, \beta_2$  = regression coefficients;

$\delta_i$  = multiplicative random effect at intersection  $i$  such that,

$\delta_i \sim \text{Gamma}(\varphi, 1/\varphi)$  and

$\varphi \sim \text{Gamma}(1,1)$ .

Markov chain Monte Carlo methods were used to calibrate posterior distributions for the model parameters using the WinBUGS software (Spiegelhalter et al. 2003). Five sets of model parameters were calibrated – one for rear-end crashes, one for angle crashes, one for injury (fatal and non-fatal) crashes, another for crashes involving heavy vehicles, and a final one for crashes of all types and severities. In all instances, the calibration was done using the “before period” data for the treated intersections and the entire data from the reference intersections. Prior distributions for all parameters were chosen to reflect complete ignorance about their magnitudes. The calibrated parameters were used to obtain estimates of the total number of crashes expected at the treated sites in the “after periods” had the treatment not been implemented (denominator of fraction term in equation 2.2). The change in safety was then calculated as the percentage difference between the expected total number of crashes and the actual number of crashes observed in the “after period.”

## 2.4 Results

Three parallel Markov chains were run for 550,000 iterations to obtain posterior distributions of the model parameters and the crash reduction rates. Convergence was monitored using the Brooks-Gelman-Rubin diagnostic (Brooks and Gelman 1998), which showed that all model parameters converged after approximately 50,000 iterations. Consequently, the first 50,000 iterations of each chain were discarded as “burn-in” runs. In addition, the chains were

“thinned” using a factor of 50, which meant that the results were collected on model parameters at every 50th iteration. This was done so that the effects of any serial correlations would be minimized. Thus inferences were based on samples of size 30,000 for every model parameter. Model results are summarized in table 2.4.

**Table 2.4** Posterior Moments and Quantiles of Model Parameters

Node	Mean	Standard		95% Credible Interval	
		Error	Median	2.5%	97.5%
Rear-end					
$\beta_0$	-0.671	0.395	-0.639	-1.512	-0.048
$\beta_1$	0.144	0.085	0.136	0.011	0.329
$\beta_2$	0.208	0.101	0.203	0.027	0.417
$R$	0.012	0.115	0.019	-0.230	0.216
Angle					
$\beta_0$	-0.606	0.362	-0.570	-1.390	-0.043
$\beta_1$	0.030	0.028	0.022	0.001	0.105
$\beta_2$	0.101	0.066	0.092	0.005	0.250
$R$	0.436	0.056	0.439	0.321	0.539
Injury (fatal and non-fatal)					
$\beta_0$	-1.709	0.545	-1.702	-2.782	-0.655
$\beta_1$	0.117	0.086	0.101	0.005	0.317
$\beta_2$	0.077	0.061	0.063	0.003	0.225
$R$	0.207	0.087	0.210	0.026	0.369
Heavy vehicle					
$\beta_0$	-1.125	0.524	-1.111	-2.198	-0.172
$\beta_1$	0.140	0.103	0.121	0.006	0.385
$\beta_2$	0.156	0.110	0.137	0.007	0.411
$R$	0.005	0.133	0.014	-0.277	0.243
All					
$\beta_0$	-0.175	0.150	-0.136	-0.562	-0.006
$\beta_1$	0.036	0.029	0.029	0.001	0.108
$\beta_2$	0.031	0.026	0.024	0.001	0.095
$R$	0.082	0.058	0.084	-0.038	0.192
$P(R>0 \text{Data})$	0.920				

The results in table 2.4 show that there were reductions of 20.7% and 43.6% in injury and right-angle crashes, respectively, following the installation of the NDOR AAW devices at the study sites. The magnitude of these reductions may be considered relatively high. In contrast, the reductions in rear-end crashes (1.2%) and crashes involving heavy vehicles (0.5%) were relatively small. The posterior distribution of the overall crash reduction rate was approximately normal (squared error =  $1.41 \times 10^{-4}$ ) with mean 8.2% and standard deviation 5.8%.

Although the average overall crash reduction rate was fairly high at the study intersections, the results do not rule out potential increases in crashes following NDOR AAW system installation. As seen in table 2.4, the 95% credible interval for the overall crash reduction rate includes zero (no safety effects) and negative values (increased crashes). On the other hand, the results also suggest that total crash reduction rates as high as 19% are probable. The posterior probability that the installation of an NDOR AAW system improves overall intersection safety, as defined by the reduction in total crashes, was estimated to be 92%.

## 2.5 Summary

The Nebraska Department of Roads developed and implemented a dilemma zone protection design that combines advance detection and advance warning systems. The NDOR design uses a shorter maximum allowable headway than other conventional advance detection systems, thereby reducing the frequency of loss of dilemma zone protection due to max-out. The design algorithm continually monitors an upstream detector as well as traffic at the intersection in order to predict the onset of the amber signal indication. Flashing beacons are then used to get the driver's attention and also warn the driver of the impending end of the green indication.

This research examined results of the safety changes attained as a result of implementing the NDOR design at 26 signalized high-speed intersections in Nebraska. A fully Bayesian model

was used, which accounts for the regression-to-mean bias and can better account for model uncertainties with minimal data, relative to an empirical Bayes. WinBUGS was used to produce draws from the posterior distributions of model parameters. Given those draws, Monte Carlo methods were used to approximate quantities such as an intersection's expected long-term crash frequency and the expected crash reduction rate (and its distribution).

For the intersections studied in this research, the expected crash reduction rate, estimated based on 30,000 Monte Carlo samples, was 0.5% for crashes involving heavy vehicles, 1.2% for rear-end crashes, 43.6% for right-angle crashes, 20.7% for fatal and non-fatal injury crashes, and 8.2% for all crashes combined. The results also suggest that there is a greater than 90% probability that the installation of a new system that combines AD and AAW systems is effective at improving overall safety at high-speed signalized intersections.

These results confirm the hypothesis that the installation of the NDOR AAW systems in Nebraska improves overall intersection safety. While an economic analysis was not conducted, given the high cost of crashes (in terms of deaths, injuries, and property damage) and the relatively low cost of installation and maintenance of NDOR AAW devices, it is hypothesized that the design is worth considering for dilemma zone protection at other high-speed signalized intersections in Nebraska.

## Chapter 3 Operational Analyses

The objective of this chapter was to describe the performance of the NDOR AAW system with respect to traffic operations efficiency. The evaluation was based on data collected at two high-speed signalized intersections equipped with the NDOR AAW system. The following were the main characteristics studied:

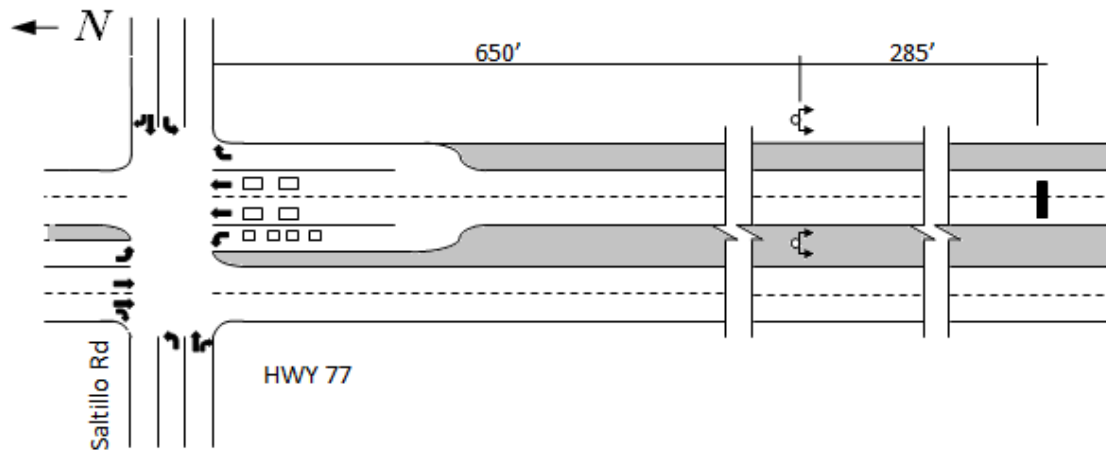
- Approach speed profiles;
- Acceleration (and deceleration) characteristics following the onset of amber signal;
- Acceleration (and deceleration) characteristics during the “lead flash”;
- Green time distributions and the rate at which green intervals ended by max-outs;
- The rate at which vehicles were “caught” in the dilemma zone;
- Waiting time on conflicting phases.

The goal was to study the operational effects of the intersection to identify if the NDOR AAW system was operating in the manner in which it was designed.

### 3.1 Study Sites

#### *3.1.1 Highway 77 and Saltillo Road*

The intersection of Highway 77 and Saltillo Road is an isolated intersection located about 5 miles south of Lincoln. As may be seen in figure 3.1, Saltillo Road is a two-lane undivided highway. The eastbound and westbound approaches both have an exclusive left-turn lane and a shared through/right-turn lane. The speed limit on both approaches is 55 mph.



**Figure 3.1** Northbound Approach on Highway 77 at Saltillo Road

Highway 77 is a four-lane divided expressway. The northbound approach on Highway 77 has two through lanes, an exclusive left-turn lane, and an exclusive right-turn lane. The southbound approach also has two through lanes and an exclusive left-turn lane. However, right-turners on the southbound approach share the outer lane with through traffic. The speed limit on both the southbound and the northbound approaches is 65 mph, which reduces to 55 mph at approximately 1,150 ft upstream of the intersection.

The traffic signal at Highway 77 and Saltillo Road operates in the fully-actuated mode and is not coordinated with any other signal. The timing for the Highway 77 phases is 7 s minimum green, a 3 s extension, 30 s maximum green, and 3 s of amber for the left-turn phases. The through phases have 15 s minimum green, a 3 s extension, 50 s maximum green, and a 4.5 s amber followed by 0.5 s all-red. The through phase extensions are from advance detectors located 935 ft from the stop line. The timing for the Saltillo Road phases is 10 s minimum green, a 3 s extension, 30 s maximum green, and 4 s of amber followed by 0.5 s all-red for all movements. The Saltillo Road phases have a 4 s delay on actuation. The through phases on

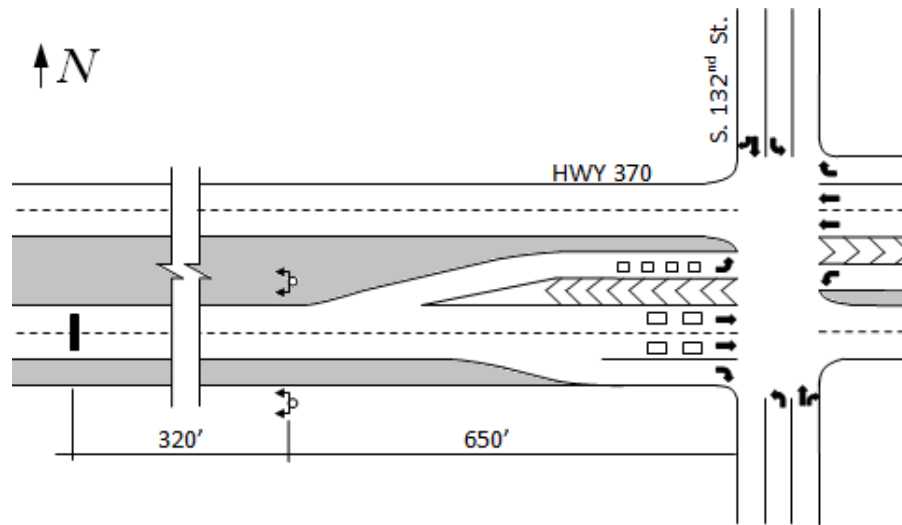
Highway 77 are on “recall” which means that the traffic signal “rests” in these phases when no traffic is present.

Whenever there is a vehicle call on a conflicting phase (to the Highway 77 through phases), the signal would “wait” until there is no active call on Highway 77 (i.e. gap-out) or until there is only 7 s (see table 1.1) before the green time reaches the preset maximum value (i.e. max-out). At this time, the controller “freezes” and the active warning signs, located 650 ft from the stop line, flash for 7 s to indicate the impending end of green. At the end of the 7 s “lead flash” interval, the controller resumes operation with the Highway 77 through phase amber indication. The active advance warning sign(s) continue to flash until the through phase(s) turn green again. The through phases on Highway 77 end together but may or may not start together depending upon which, if any, left turn phases are called. As the signal is fully actuated, any phase, except for the Highway 77 through phases (which are on recall), may be skipped if no demand is present.

### *3.1.2 Highway 370 and South 132nd Street*

The intersection of Highway 370 and South 132nd Street is an isolated intersection located about 10 miles south-west of Omaha. As may be seen in figure 3.2, South 132nd Street is a two-lane undivided highway.





**Figure 3.2** Eastbound Approach on Highway 370 at South 132nd Street

Both the northbound and southbound approaches on South 132nd Street have an exclusive left-turn lane and a shared through/right-turn lane. The speed limit is 35 mph on the southbound approach and 45 mph on the northbound approach. Highway 370 is a four-lane divided expressway. Both the northbound and southbound approaches have two through lanes, an exclusive left-turn lane, and an exclusive right-turn lane. The speed limit for both the southbound and the northbound approaches is 55 mph.

The signal at Highway 370 and South 132nd Street is an eight phase signal operating seven phases. The left turn phase from the north is not active. The signal operates in the free mode and is not coordinated with any other signal. The timing for the Highway 370 phases is 7 s minimum green, a 3 s extension, 25 s maximum green, and 3 s of amber for the left turn phases. The through phases have a 15 s minimum green, a 4 s extension, a 60 s maximum green, and a 4.5 s amber followed by a 0.5 s all-red. The extension is from the advance detector located 970 ft from the stop line. The timing for the South 132nd Street phases is 7 s minimum green, a 3 s extension, 25 s maximum green, and 3 s of amber for the northbound left turn phase. The

through phases have a 10 s minimum green, a 3 s extension, a 40 s maximum green, and a 4.5 s amber followed by a 0.5 s all-red. The extension is from the loop detectors located at the stop line. The through phases on Highway 370 are on “recall” and the signal would “rest” in these phases when no traffic is present.

Any time that a vehicle call is present on South 132nd Street, the signal “waits” until there is no active call on Highway 370. At this time the controller “freezes” and the active advance warning signs flash for 8 s (based on a 4 s travel time between the advance detector location and the warning sign location). At the end of the 8 s interval, the controller resumes operation with the Highway 370 amber indication. The advance warning signs continue to flash until the through phase signals on that leg turn green again. Since the signal is fully actuated, any phase, except for the Highway 370 through phases on “recall”, may be skipped if no demand is present.

The left turn phase from the south is protected only. It is green with the northbound through phase. After the left turn phase ends, the northbound and southbound phases are “ON” together. The southbound left is a permissive turn made during the through green. The through phases on Highway 370 end together, but may or may not start together depending upon which, if any, left turn phases are called.

## 3.2 Data

### *3.2.1 Equipment*

The data used in this study were collected using the Nebraska Transportation Center’s (NTC) mobile data collection system shown in Figure 3.3. The system consists of two Wavetronix SmartSensor Advance detectors and one Wavetronix SmartSensor HD detector mounted on a 30-ft mast attached to a trailer. The two SmartSensor Advance detectors are

mounted in the reverse-fire mode. One of the detectors tracks the speed and range of each vehicle it detects up to 500 ft upstream of the trailer's location; the other records similar information up to 500 ft downstream of the trailer's location. The SmartSensor HD is mounted in the side-fire mode and records the vehicle's length, travel lane, and speed as it passes the trailer (midstream).



**Figure 3.3** NTC Mobile Data Collection System



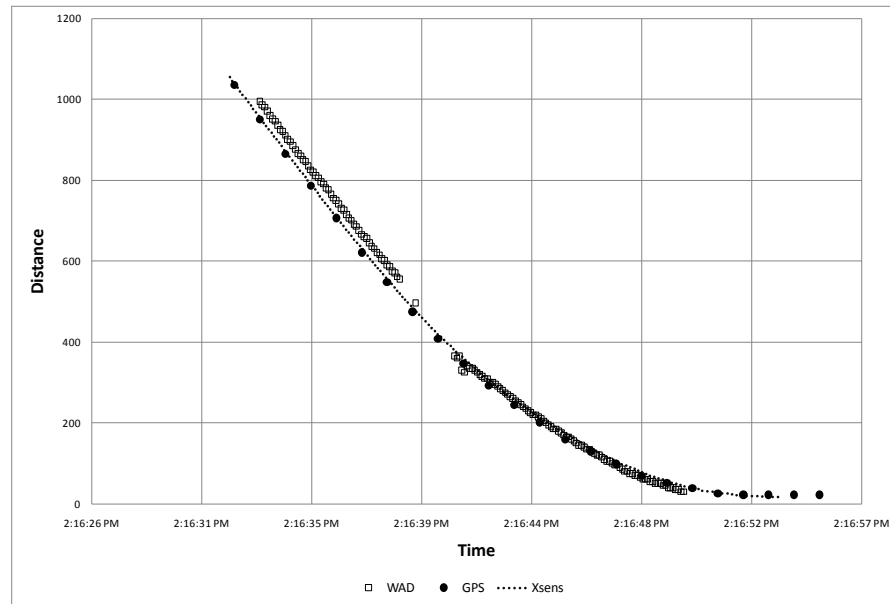
**Figure 3.4** Portable Click Cabinet System

Two portable Wavetronix Click Cabinet Systems (figure 3.4) facilitate communications between the detectors (installed on the mast) and the traffic control cabinet. One of the portable Click Cabinet Systems (CCS), installed in the traffic control cabinet, collects phase status information for onward transmission to the second CCS installed by the trailer. Data collected by the detectors are also routed to the second CCS. The portable CCS set-up also enables AC to DC power conversion as well as protects the detectors from possible damage caused by power surges (Graham 2011). A laptop computer, connected to the trailer's data-collecting portable cabinet, retrieves the detector and phase data. The data is saved (for later processing) in both the \*.DAT and \*.txt formats using MATLAB code provided by Wavetronix.

### *3.2.2 Sensor Validation*

The Wavetronix SmartSensors were validated against the Xsens MTi-G sensor. The MTi-G is an assembly of an inertial motion sensor, a DGPS receiver, and a processor capable of monitoring vehicle position, speed, and acceleration at a rate of 100 data points per second (Xsens 2011). Validation against the Xsens MTi-G was done only one time (Burnett 2011).

However, multiple runs using a handheld GPS device were done during every data collection session to ensure accurate vehicle tracking. An example of the tracking performance of the Xsens MTi-G (Xsens), the handheld GPS device (GPS), and the SmartSensors (WAD) is shown in figure 3.5 (Burnett 2011).



**Figure 3.5** Example Tracking Performance of Xsens, GPS, and SmartSensor (WAD)

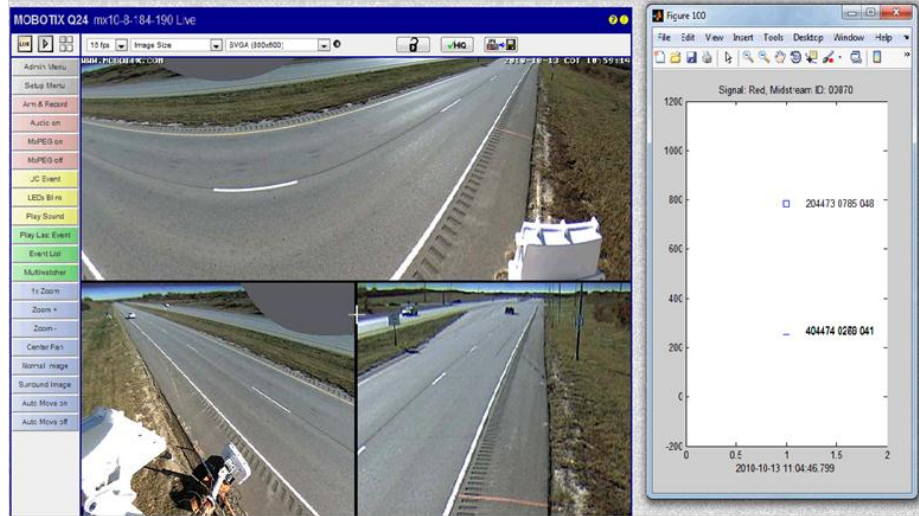
### 3.2.3 Data Collection

Data were collected at the northbound and the southbound approaches on Highway 77 at Saltillo Road and on the eastbound approach on Highway 370 at South 132nd Street using the NTC mobile data collection system. Even though the westbound approach at Highway 370 had an NDOR AAW system installed, data were not collected for this approach because of communication difficulties between the traffic control cabinet and the detectors. Data were collected under clear weather conditions when the pavement was dry and wind speeds were

below 10 mph (strong continuous winds or wind gusts sway the trailer's mast arm and produce erroneous data). Data were collected on the following dates and times:

- Northbound approach at Highway 77 and Saltillo Road: 8:00 am to 4:00 pm on September 29, 2010 and from 10:00 am to 6:00 pm on September 30, 2010;
- Southbound approach at Highway 77 and Saltillo Road: 2:00 pm to 6:00 pm on October 5 and October 6, 2010;
- Eastbound approach at Highway 370 and South 132nd Street: 12:00 pm to 4:00 pm on December 22, 2010.

During each data collection session, a Mobotix Q24M fisheye camera (Mobotix 2011) was used to record an overall view of traffic operations over the entire length of the study approach (upstream, midstream, and downstream of the trailer location). An Active Webcam (PY Software 2011) was used to capture high resolution images (up to 30 frames per second) from the three camera views (i.e. upstream, midstream, and downstream). These images were displayed on a computer screen side by side with a MATLAB window displaying data retrieved from the mobile data collection system as shown in figure 3.6. Displaying the data in this manner provided the ability to playback, when necessary, and crosscheck for possibly “suspicious” data. Screenshots of the display were then recorded and saved to standard \*.AVI movie files for later viewing using HyperCam 2 (Hyperionics 2011).



**Figure 3.6** Data Display Set-Up

In addition to the NTC mobile data collection system and the fisheye camera set-up, two camcorders were used to video-tape traffic and signal indications on the minor road approaches. These video recordings were used to obtain the data needed to determine the waiting time on conflicting phases. Two Jamar traffic data collectors (TDC-8) were also used to record turning movement counts at the study intersections.

### *3.2.4 Data Reduction and Diagnostics*

Data from the SmartSensors were retrieved from the field computer in the form of text files. A total of 168 text files ranging in size from 1 KB to 1.4 MB were retrieved at all sites during the data collection sessions. An example of the raw data format is shown in figure 3.7. The data were analyzed to obtain relevant traffic and signal status information on a cycle-by-cycle basis.

As a first step, extensive quality control and data screening were performed to identify and either discard or impute probably erroneous data. Data reduction was done using computer

programs written in the Perl programming language. Descriptions of the major modules implemented as part of the data screening program are provided below.

```
-----
#1 = timestamp (yyyy-mm-dd HH:MM:SS.FFF)
    Note: data columns may not be in timestamp order
#2 = data type (1 digit) & ID (5 digits)
    Note: data type is the first digit with following codes...
          2=Upstream,3=Midstream,4=Downstream,5=Phase)
    Note: ID is the lower 5 digits
#3 = range (ft)
    Note: for upstream data the range corresponds
          to the front of the vehicle
    Note: for Midstream & Downstream data the range corresponds
          to the back of the vehicle
    Note: for fusion you may wish to ignore upstream & Downstream
          data between 400 and 600 feet
#4 = speed (mph)
#5 = lane (1=closest to sensor, 0=Unknown)
#6 = length (ft, where 0=Unknown)
#7 = phase (0=Red, 1=yellow, 2=Green, 3=Unknown)
-----
000000000111111111122222222223333333333344444444445555555555666
12345678901234567890123456789012345678901234567890123456789012
#1      #2      #3      #4      #5      #6      #7
-----
2010-09-29 08:04:06.388 400001 0415 038 0 000 3
2010-09-29 08:04:05.449 300001 0500 052 0 020 3
2010-09-29 08:04:06.500 400001 0410 036 0 000 3
2010-09-29 08:04:06.588 400001 0405 041 0 000 3
2010-09-29 08:04:06.688 400001 0395 041 0 000 3
2010-09-29 08:04:06.789 400001 0400 034 0 000 3
2010-09-29 08:04:06.888 400001 0395 034 0 000 3
2010-09-29 08:04:06.989 400001 0385 040 0 000 3
2010-09-29 08:04:07.088 400001 0380 045 0 000 3
2010-09-29 08:04:07.188 400001 0375 045 0 000 3
2010-09-29 08:04:07.289 400001 0370 045 0 000 3
2010-09-29 08:04:07.388 400001 0360 044 0 000 3
2010-09-29 08:04:07.488 400001 0355 044 0 000 3
2010-09-29 08:04:07.588 400001 0350 042 0 000 3
2010-09-29 08:04:07.688 400001 0340 042 0 000 3
2010-09-29 08:04:07.789 400001 0335 043 0 000 3
2010-09-29 08:04:07.888 400001 0330 041 0 000 3
2010-09-29 08:04:07.997 400001 0325 041 0 000 3
2010-09-29 08:04:08.088 400001 0320 039 0 000 3
2010-09-29 08:04:08.188 400001 0315 039 0 000 3
2010-09-29 08:04:08.295 400001 0305 040 0 000 3
```

**Figure 3.7** Raw Data Format

#### Module 1 – Data Compilation:

This module was used to delete headers from the text files and to place all data collected during a given session end-to-end in chronological order to form one large text file. This facilitated processing the data on a cycle-by-cycle basis. That is, for all text files corresponding



to data collected on a given day, this module opened the file, scanned the data from the first line to the last, and printed the contents of each scanned line to a single output file (also in the \*.txt format) unless the line was part of a header. Additionally, if the value of the signal status variable (column #7 of figure 3.7) on a given line was 3, thus unknown, then it was generally not possible to classify data on the line that contained this signal status value as belonging to a particular cycle. Lines containing such data were therefore discarded.

#### Module 2 – Cycle-by-Cycle Data Retrieval:

Module 2 was used to split the output file from module 1 into several text files each of which contained data for only one cycle. Cycles of interest were those that had vehicles within the range of detection (approximately 1,100 ft) during the lead flash interval, or the amber signal indication, or both. Data partitioning was done using the variables signal status (see figure 3.7, column #7), time of day (column #1), and data type (column #2).

That is, a series of lines of data were identified as belonging to the same cycle if the pattern suggested that the entire series consisted of data collected during a continuous stretch of green signal indication followed by amber and then red. The data type variable was used to identify the boundaries of the signal indications. Thus a series of lines of data was identified as belonging to the same cycle if:

- (i) It started with data type equal to 500000 (i.e. phase information) and signal status equal to 2 (i.e. green);
- (ii) Is followed by a series of detector readings during green (signal status equal to 2 and data type not equal to 500000);
- (iii) Then another 500000 data type indicating the onset of amber (signal status equal to 1);

- (iv) Followed by a set of detector readings during amber (this ensured that all cycles that were used in the analyses had vehicles on the study approach during amber);
- (v) Then a 500000 data type for the beginning of red;
- (vi) Followed by a set of detector readings for which signal status is 1 and data type does not equal 500000; and finally
- (vii) A 500000 data type for the start of the next green phase (and the end of the current cycle).

The module also compared the differences in times between the starts and the ends of amber with the actual amber time implemented by the controller. Only those cycles that had amber times consistent with the controller setting were used. The cycle lengths were also checked in this module to ensure that they were consistent with the timing plans described in section 3.1. Lines of data that implied unreasonable change in signal status were excluded. An example of such data is shown in figure 3.8. As may be seen in the shaded area of figure 3.8, the values in the last column (“Signal Status”) change from “2” directly to “0” and suggest that the signal indication changed from green (“2”) to red (“0”) without displaying the amber (“1”). Because such a transition in signal indication is unreasonable, data from cycles that contained these types of transitions were not used.

2010-09-29	08:35:02.041	200246	1020	057	0	000	2
2010-09-29	08:35:02.141	200245	0890	051	0	000	2
2010-09-29	08:35:02.141	200246	1015	057	0	000	2
2010-09-29	08:35:02.241	200245	0905	051	0	000	2
2010-09-29	08:35:02.241	200247	1000	073	0	000	2
2010-09-29	08:35:02.341	200245	0900	051	0	000	2
2010-09-29	08:35:02.341	200247	0985	073	0	000	2
2010-09-29	08:35:02.441	200245	0895	048	0	000	2
2010-09-29	08:35:02.441	200247	0995	063	0	000	2
2010-09-29	08:35:02.486	500000	0000	000	0	000	0
2010-09-29	08:35:02.541	200245	0885	048	0	000	0
2010-09-29	08:35:02.541	200247	0980	063	0	000	0
2010-09-29	08:35:02.641	200245	0880	052	0	000	0
2010-09-29	08:35:02.641	200247	0970	063	0	000	0
2010-09-29	08:35:02.741	200245	0875	059	0	000	0
2010-09-29	08:35:02.741	200247	0965	063	0	000	0
2010-09-29	08:35:02.841	200245	0865	059	0	000	0
2010-09-29	08:35:02.841	200247	0955	063	0	000	0

**Figure 3.8** Data Showing Unreasonable Signal Status Change

Finally, this module was used to sort each cycle's data by vehicle ID and then by when it was recorded relative to the onset of amber. A sample output from Module 2 is shown in figure 3.9. Column 1 of figure 3.9 denotes the time the data was recorded relative to the onset of amber (negative values denote time before onset of amber and positive values are for time since onset of amber). Column 2, as well as the first digit of column 3, is the detector from which the data came (2 for upstream, 3 for midstream, and 4 for downstream). The last four digits in column 3 are the vehicle IDs. Columns 4, 5, 6, and 7 are the vehicle's distance from the stop line, speed, travel lane, and length, respectively. Column 8 is the signal status (2 for green, 1 for amber, and 0 for red).

25.80	2	20780	750	43	0	0	0
25.80	2	20780	1030	43	0	0	0
25.90	2	20780	745	43	0	0	0
25.90	2	20780	1025	43	0	0	0
26.00	2	20780	735	43	0	0	0
26.00	2	20780	1020	43	0	0	0
26.10	2	20780	725	43	0	0	0
26.10	2	20780	1010	43	0	0	0
29.20	2	20780	550	34	0	0	0
29.20	2	20780	815	42	0	0	0
-31.07	3	30154	500	47	1	18	2
-29.85	3	30154	500	39	0	17	2
-26.58	3	30154	500	57	1	17	2
-23.70	3	30154	500	41	0	38	2
-13.51	3	30154	500	41	0	19	2
-6.75	3	30154	500	61	1	15	2
-6.14	3	30154	500	62	0	17	2
1.27	3	30155	500	43	1	19	1
15.55	3	30155	500	51	0	20	0
22.02	3	30155	500	35	0	68	0
22.04	3	30155	500	43	1	79	0
-25.05	4	40770	380	38	0	0	2
-24.95	4	40770	370	38	0	0	2
-24.85	4	40770	360	60	0	0	2

**Figure 3.9** Example Output from Cycle-By-Cycle Data Retrieval Module

Module 2 resulted in 623 cycles of “usable” data – 423 for the northbound Highway 77 at Saltillo Road approach, 178 for the southbound Highway 77 at Saltillo Road approach, and 22 for the eastbound Highway 370 at South 132nd Street approach.

#### Module 3 – Data Cleaning and Imputation:

Module 3 was used to “clean” the output from Module 2 of “suspicious” detector readings and to identify “unreasonable” speed values. For example, it may be seen from figure 3.9 that there were instances where different vehicles were assigned the same ID. The first two lines of data in figure 3.9 seem to suggest that the same vehicle (with ID 20780) was observed at two different locations at the same time. For example, the same vehicle was observed at distances 750 ft and 1030 ft from the stop line 25.80 s following the onset of amber; and also at 745 ft and 1025 ft from the stop line 25.90 s following the onset of amber. This is not reasonable. A more plausible scenario is that the data were for two different vehicles – one tracked at distances 750 ft, 745 ft, etc. and the other at 1030 ft, 1025 ft, etc. from the stop line.

Module 3 was used to identify all such occurrences and to reassign unique ID's to the vehicles so identified. The module was also used to remove duplicate lines of data.

Speeds were checked for reasonableness by comparing the accelerations (and decelerations) calculated from pairs of sequential speed readings to a technically maximum feasible value (VISSIM Manual 2009, Mannering et al. 2009). In this study, the maximum value was set equal to 30 ft/s<sup>2</sup> (i.e., approximately 0.93g's). If the magnitude of the calculated acceleration (or deceleration) exceeded the predefined maximum value, then the speed value among the pair under consideration that was most consistent with the neighboring speed readings (usually the larger of the two) was assumed to hold true for both instances. This was deemed to be a reasonable assumption because vehicle speeds were tracked over approximately 5-ft intervals and the speed changes implied by accelerations greater than the preset maximum were considered impractical over such short distances. For example, the module would replace the speed reading of 5 mph for vehicle ID 40101 (figure 3.10; headers are same as in figure 3.9) observed when it was at a distance of 405 ft from the stop line with the value of 55 mph observed when it was at 390 ft from the stop line.

5.67	4	40100	275	38	0	0	0
5.77	4	40100	270	38	0	0	0
5.87	4	40100	265	37	0	0	0
5.97	4	40100	260	37	0	0	0
6.07	4	40100	255	37	0	0	0
6.17	4	40100	250	35	0	0	0
6.27	4	40100	245	35	0	0	0
6.37	4	40100	240	36	0	0	0
6.47	4	40100	235	35	0	0	0
6.47	4	40101	405	5	0	0	0
6.57	4	40101	390	55	0	0	0
10.87	4	40101	160	36	0	0	0
10.97	4	40101	155	36	0	0	0
11.07	4	40101	150	36	0	0	0
11.17	4	40101	140	36	0	0	0

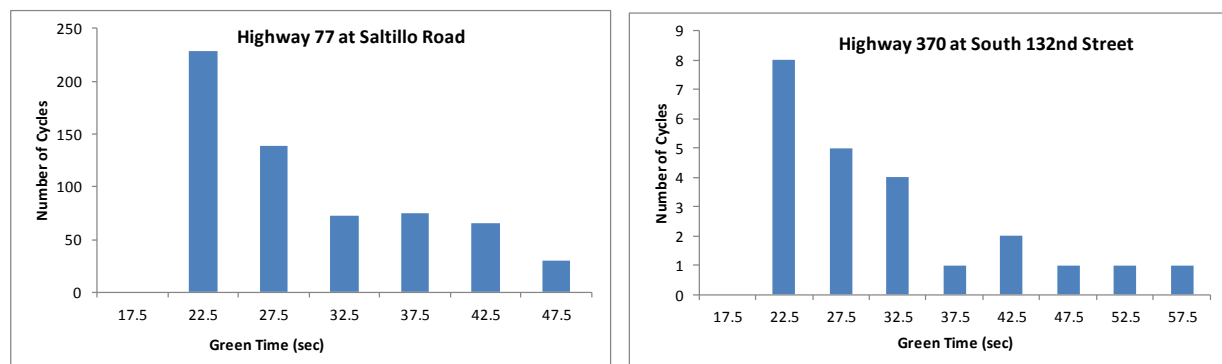
**Figure 3.10** Example “Suspicious” Speed Data

### 3.3 Analysis

The “cleaned” data from section 3.2.4 were used to calculate the accelerations, frequency of max-outs, dilemma-zone entrapment, and speed profiles needed to describe the performance of the NDOR AAW system.

#### 3.3.1 Max-Out Probabilities

Phase termination by maximum green or max-out refers to the immediate end of the green indication when it has been extended to the preset maximum green value. When max-out occurs, the green indication ends and the amber indication begins immediately, irrespective of vehicles being in their dilemma zones or not. Thus when a phase ends by max-out, dilemma zone protection is not provided. A low frequency of max-outs is therefore an indication that the NDOR AAW system is serving its primary objective of providing dilemma zone protection (McCoy and Pesti 2002).



**Figure 3.11** Distribution of Through Phase Green Time on High-Speed Approaches

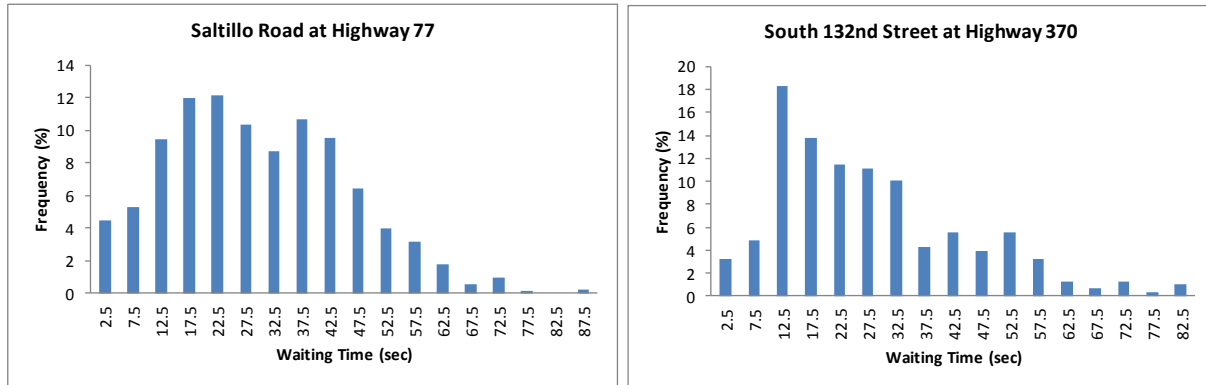
The distributions of green times at the study intersection approaches during the data collection periods are shown in figure 3.11. The expected probability that a cycle would end by max-out ( $p$  [Green time > 59.5 s];  $p$  [Green time > 49.5 s]) was estimated as 0.44% for the

eastbound Highway 370 high-speed approach and 0.49% for the two Highway 77 high-speed approaches. The low max-out probabilities (less than one half of a percentage point at both study sites) suggested the NDOR AAW system performed reasonably well under prevailing conditions for both study intersections.

### *3.3.2 Waiting Time on Conflicting Phases*

The average amount of time that the first vehicle arriving at a conflicting phase has to wait before it receives the green may serve as an indicator of traffic operations efficiency. One advantage of NDOR AAW system, relative to conventional advance detection systems, is that it uses a shorter maximum allowable headway (MAH) which tends to decrease the frequency at which the major road through phase ends by max-out. By decreasing the frequency at which max-outs occur, the average amount of time that the first vehicle arriving on the conflicting phase has to wait before receiving the green is reduced. Thus, the shorter MAH tends to decrease the delay experienced by vehicles waiting on the conflicting phases, thereby improving the efficiency of traffic operations (McCoy and Pesti 2002).

The distributions of the times that the first vehicle arriving on the minor road phases had to wait before receiving the green are shown in figure 3.12. The average waiting time on the minor road approaches at the intersection of Highway 77 and Saltillo Road was estimated as 29.8 s. The estimated average waiting time on the minor road approaches at the intersection of Highway 370 and South 132nd Street was 28.0 s. These estimates were consistent with the low frequencies of max-outs and the average green time durations of 30.0 s and 32.0 s observed for the major through phase movements on Highway 77 and Highway 370, respectively.



**Figure 3.12** Distribution of Waiting Times on Minor Road Phases

### 3.3.3 Driving Behavior during the Lead Flash Phase

Two driver behavior parameters were studied in relation to the lead flash phase:

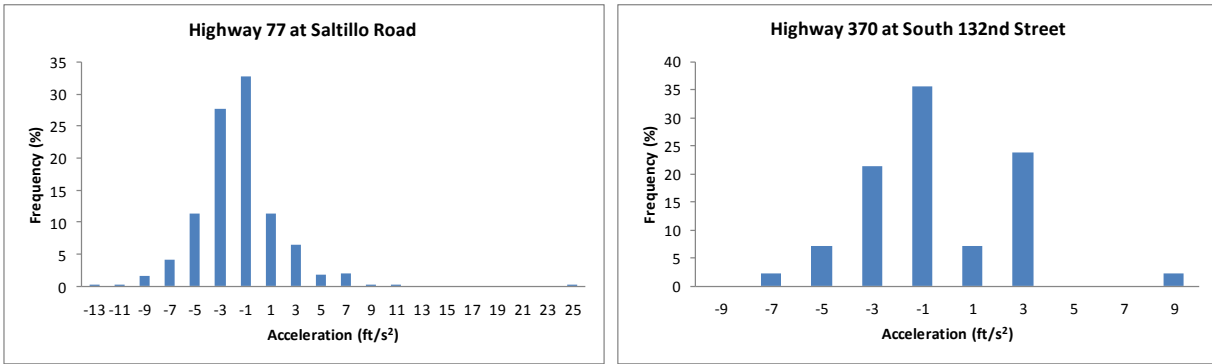
acceleration/deceleration rates and speed profile.

Acceleration/Deceleration Rates:

The lead flash is the time period before the onset of green during which the active advance warning devices flash. The MAH setting on the traffic controller enables an approaching vehicle to extend the green interval by a time equal to the MAH (usually 3.0 s) when it passes over the advance detector. If gap-out occurs after a green extension, the advance warning devices begin to flash alerting drivers to prepare to stop.

Driver reactions to the flashing advance warning device, as measured by the corresponding vehicle accelerations and decelerations, are summarized in figure 3.13. The data used in calculating the accelerations and decelerations are only for those vehicles that were observed upstream of the location of the advance warning device during the lead flash phase.





**Figure 3.13** Acceleration/Deceleration Rates on High-Speed Approaches during Lead Flash

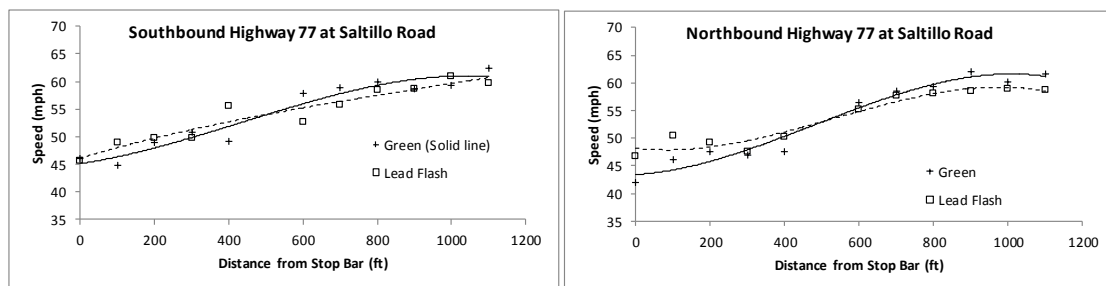
On average, drivers traveling on the Highway 77 approach decelerated at a rate of 1.55 ft/s<sup>2</sup> during the lead flash period. Overall, 77.8% of all drivers either maintained their speeds or decelerated during the lead flash. The average deceleration rate for the 75.0% of drivers who decelerated was 2.89 ft/s<sup>2</sup>. However, 22.2% of all drivers observed during the lead flash period increased their speeds, perhaps in an attempt to make the green. The average rate of acceleration for these drivers was 2.81 ft/s<sup>2</sup>. A review of the video files showed that 19.8% of vehicles observed upstream of the warning signs at the beginning of the lead flash proceeded through the intersection. Approximately 36.4% of the drivers who accelerated proceeded through the intersection while 18.8% of those who decelerated went through.

Approximately 33.3% of drivers observed on the Highway 370 approach accelerated during the lead flash period. The average rate of acceleration for these drivers was 3.49 ft/s<sup>2</sup>. Approximately, 4.8% of drivers maintained their speeds. Another 61.9% reduced their speeds at an average rate of 2.33 ft/s<sup>2</sup>. There was an overall average rate of deceleration of 0.277 ft/s<sup>2</sup> during the lead flash period. . A review of the video files for this intersection showed that 17.1% of vehicles observed upstream of the warning sign at the beginning of the lead flash

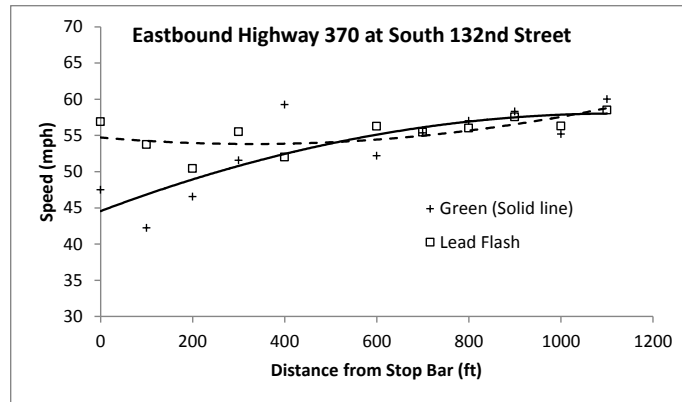
proceeded through the intersection. Approximately 27.8% of the drivers who accelerated and 14.1% of those who decelerated proceeded through the intersection.

#### Speed Profile:

Speed data collected along the high-speed approaches established a profile that could be used to estimate the speed of a vehicle at any point on the intersection approach over a range up to approximately 1,100 ft from the stop line. The average speeds of vehicles observed within 1,100 ft of the stop line on the high-speed approaches are shown in figures 3.14 and 3.15. These figures also show plots of the “best-fit” lines through the observed speed profiles. In both figures, speeds are compared over two equal time periods. The first period is the duration of the lead flash, which is the period before the onset of amber when the warning devices are flashing. The second is the portion of the green interval (of duration equal to the lead flash) that immediately precedes the start of flashing.



**Figure 3.14** Speed Profiles along Highway 77 at Saltillo Road during Lead Flash



**Figure 3.15** Speed Profile along Eastbound Highway 370 at S 132nd St. during Lead Flash

Figures 3.14 and 3.15 show that drivers generally tended to reduce their speeds as they approached the intersection (as suggested by the generally downward sloping speed profiles) in both time periods before and during the lead flash. However, consistent with the findings in section 3.3.3, there were moderate speed reductions (vehicle decelerations) along sections of the high-speed approaches upstream of the locations of the advance warning flashers (i.e. vehicles traveling 650 ft or more upstream of the stop line). The general trend of decreasing speeds upstream of the flashers continued as vehicles traveled the downstream portions. However, as may be seen from the plots, speeds downstream of the advance warning signs were generally higher during the portion of the green when the flashers were active (lead flash) than they were when the flashers were off (green). One probable explanation for this is that vehicles close to the advance warning sign when flashing began, assumed a realistic probability of making the green without unreasonably high accelerations. Consequently, the driver traveled at speeds higher than the typical profile (on green) in order to try and enter the intersection while the signal was still green or amber. Furthermore, it may also be determined from these plots that the speed increases were generally modest and that the average speeds were generally less than or equal to the posted speed limits (55 mph on all study approaches).

Accordingly, it appears that the advance warning signs were effective at alerting drivers to the impending end of the green. In response, it appears that those close enough to the flasher and who probably perceived realistic likelihoods of making the green tended to either maintain their speeds or accelerate to speeds generally higher than the typical profile speed. However, those that did increase their speed, on average, did not exceed the speed limit. Conversely, it appears that vehicles far from the advance warning sign when flashing began tended to travel at speeds lower than the typical profile. This was mostly likely because it was safe to do so or that there was not a realistic chance of making the green.

### *3.3.4 Driving Behavior following the Onset of Amber*

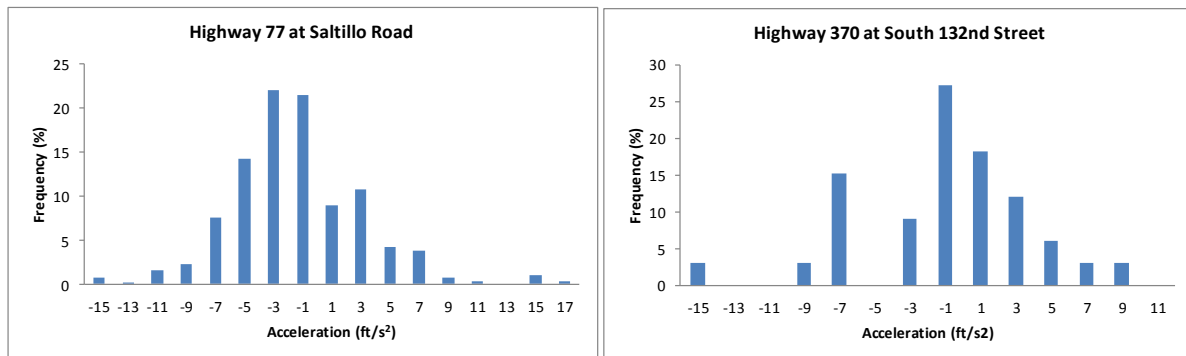
The two same driver behaviors were studied in relation to the onset of amber: acceleration/deceleration rates and speed profile.

#### *Acceleration/Deceleration Rates:*

High rates of acceleration at the onset of amber could be conducive to red-light running and right-angle collisions. Similarly, high deceleration rates may lead to abrupt stops and rear-end crashes. The distributions of vehicle accelerations at the high-speed approaches studied in this research are shown in figure 3.16. The data used were from vehicles observed upstream of the intersection during the first 2.0 s of the amber interval.

Approximately 30.2% of vehicles on the Highway 77 high-speed approaches accelerated following the onset of amber. The average rate of acceleration for these vehicles was  $4.08 \text{ ft/s}^2$ . For the 67.7% of vehicles that decelerated on amber, the average rate of deceleration was  $3.86 \text{ ft/s}^2$ . The overall rate of deceleration was  $1.38 \text{ ft/s}^2$ . Approximately 85.3% of all accelerations and decelerations could be considered “comfortable” (magnitude not greater than  $7 \text{ ft/s}^2$ ); 12.6% could be considered “moderate” (magnitude greater than  $7 \text{ ft/s}^2$  but less than  $13 \text{ ft/s}^2$ ) [Messer,

2002]. That is, 2.1% of vehicles engaged in “uncomfortable” accelerations and decelerations exceeding  $13 \text{ ft/s}^2$ . It is hypothesized that these vehicles are more likely to be involved in red-light running and abrupt stops. This hypothesis was not checked with the video files because of the quality.

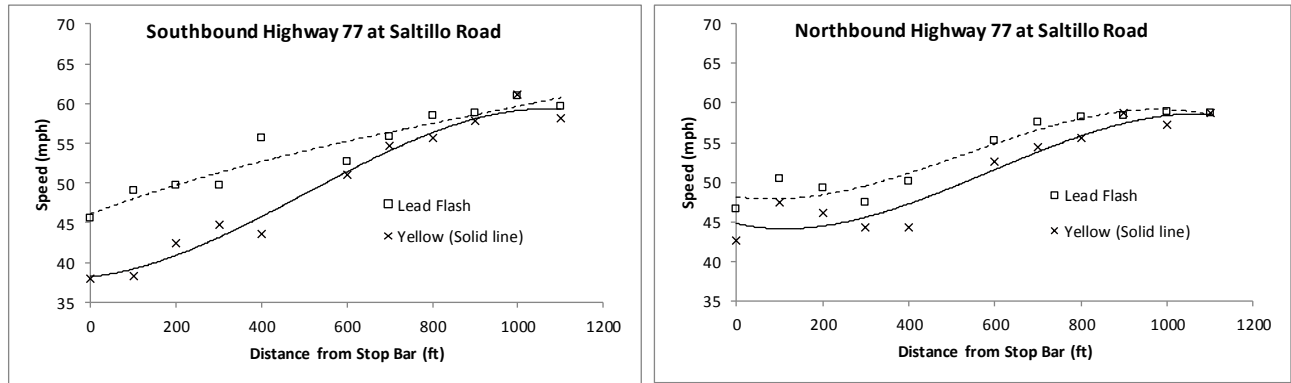


**Figure 3.16** Acceleration/Deceleration Behavior Following Onset of Amber

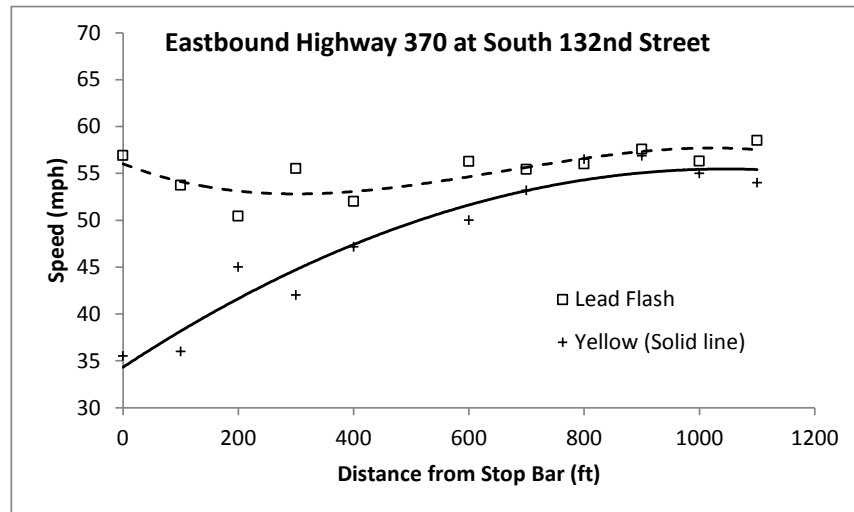
On the Highway 370 approach, 42.4% of drivers accelerated (at an average rate of  $3.10 \text{ ft/s}^2$ ) following the onset of amber. There was an overall average deceleration rate of  $0.90 \text{ ft/s}^2$  on amber. Approximately 82% (81.8%) and 3.0% of vehicles accelerated (or decelerated) at “comfortable” and “uncomfortable” rates, respectively.

#### Speed Profile:

The average speeds of vehicles observed within 1,100 ft of the intersections during the amber are shown in figures 3.17 and 3.18. The figures also show plots of the lines of “best-fit” through the observed speeds as well as present the speed profiles during the lead flash period. As expected, drivers on average reduced their speeds below the lead flash speeds and the amount of reduction appeared to increase as they approached the intersection.



**Figure 3.17** Speed Profile along Highway 77 at Saltillo Road on Amber



**Figure 3.18** Speed Profile along Eastbound Highway 370 at S 132nd St. on Amber

### 3.3.5 Vehicles in Dilemma Zone

The primary objective of the NDOR AAW system is to provide dilemma zone protection. If the controller gaps-out after a vehicle passes over the advance detector, the green is extended by an amount of time equal to the MAH. At the end of the green extension, the advance warning devices begin to flash for a period of time equal to the lead flash (see table 1.1). The amber indication starts immediately after the end of the lead flash. Vehicles traveling at or above the

design speed, up to a practically maximum speed value,  $V_m$  defined by equation 3.1, will reach the stop line before the onset of amber and are therefore provided dilemma zone protection. Equation 3.1 is based on stopping distance for a 2 s perception-reaction time and a 10 ft/s<sup>2</sup> deceleration rate (McCoy and Pesti 2002).

$$V_m = -20 + 2\sqrt{100 + 5D} \quad (3.1)$$

where,  $D$  = distance between advance detector and stop line.

Vehicles traveling at speeds lower than the design speed will not reach the stop line before the onset of amber and may be in the dilemma zone depending on their speed. A slower vehicle would be provided dilemma zone protection only if it had not reached the beginning of its dilemma zone by the time the amber indication started. In this study, the NDOR definition of dilemma zone was used (McCoy and Pesti, 2002). The NDOR definition assumes that the beginning of a vehicle's dilemma zone is one stopping distance upstream of the stop line; the stop line serves as the end of the dilemma zone.

Assuming a 2 s perception-reaction time and a 10 ft/s<sup>2</sup> deceleration rate, the maximum speed  $V_{m0}$ , at which a slower-moving vehicle (speed less than design speed) could travel and not reach the beginning of its dilemma zone may be calculated as a function of the controller's MAH setting, the duration of the lead flash ( $t$ ), and the distance of the advance detector from the stop line ( $D$ ) using equation 3.2 (McCoy and Pesti, 2002).

$$V_{m0} = -10(2 + MAH + t) + \sqrt{100(2 + MAH + t)^2 + 20D} \quad (3.2)$$

Therefore, a vehicle approaching an intersection equipped with NDOR AAW devices will be provided dilemma zone protection if it travels at a speed

$V : V \in \{0 < V \leq V_{m0} \cup V_{85} \leq V \leq V_m\}$  where  $V_{85}$  is the design speed. Vehicles traveling at speeds outside these ranges will not be provided dilemma zone protection; therefore, they will be traveling in their dilemma zones at the onset of amber.

If the distribution of speeds as vehicles pass the advance detector at the onset of amber is known, then the probability that a vehicle would be in its dilemma zone can be calculated. Summary statistics of speed data collected at the advance detector location at the onset of amber at the study intersections are provided in table 3.1. These were used to estimate the expected probability of being in a dilemma zone,  $p(V_{m0} < V < V_m)$  by noting that the speed data were approximately normally distributed. The results are summarized in table 3.1 along with vehicles observed in the dilemma zone. The vehicles included in computing the observed percentages were those observed during the first 2.0 s of the amber interval.

**Table 3.1** Expected and Observed Vehicles in Dilemma Zone

Approach	Speed Range with Dilemma Zone Protection			Vehicles at Advance Detector Location at Onset of Yellow		Cycles with Vehicle in Dilemma Zone at Onset of Yellow		
	Design Speed (mph)	Lower Range (mph)	Upper Range (mph)	Total	Mean Speed (mph)	Standard Deviation (mph)	Expected (%)	Observed (%)
SB Hwy 77 at Saltillo Rd	65	(0, 42]	[65, 81]	35	58.1	8.2	77.0	20.3
NB Hwy 77 at Saltillo Rd	65	(0, 42]	[65, 81]	56	56.6	11.4	66.6	12.2
EB Hwy 370 at S 132nd St	55	(0, 39]	[55, 121]	8	57.1	4.6	33.6	13.0

In general a lower percent of vehicles in their dilemma zones at the onset of amber indicates a higher degree of dilemma zone protection. It may be seen from table 3.1 that the



observed percentages were much lower than the statistically expected percentages. In particular, comparison of the last two columns suggested that the number of vehicles in their dilemma zones when the signal indication changed from green to amber was on average 77.2% smaller than that which would have been expected if the NDOR AAW system was not installed. The lower than expected number of vehicles in their dilemma zones at the onset of amber is an indication that the NDOR AAW devices increased the inclination of drivers to stop when they saw the warning devices flashing before the onset of amber. This appears consistent with the speed profiles which showed that drivers upstream of the flasher had a tendency to lower their speeds during the lead flash period and speeds were further reduced on amber.

### 3.4 Summary

This chapter examined the performance of the NDOR AAW system with respect to traffic operations efficiency. The study was based on data collected at two test intersections using the NTC mobile data collection equipment. The results suggested that there were lower than expected number of vehicles in their dilemma zones on the approaches equipped with NDOR AAW systems. The generally low max-out probabilities of less than one half of a percentage point at each study site also suggested that the NDOR AAW system performed reasonably well under prevailing conditions for both study intersections.

The average waiting times on the minor road approaches were 29.8 s and 28.0 s at the intersections of Highway 77 and Saltillo Road, and Highway 370 and South 132nd Street, respectively. These estimates appeared to be consistent with the low frequencies of max-outs and the average green time durations of 30.0 s and 32.0 s observed for the major through phase movements on Highway 77 and Highway 370, respectively.

The results also suggested that the advance warning signs were effective at alerting drivers to the impending end of the green. In response, it appears drivers close enough to the flasher when flashing began and who perceived realistic likelihoods of making the green tended to either maintain their speeds or accelerate to speeds generally higher than the typical profile speed (but lower than the speed limit, on average). On the contrary, it appears vehicles far from the advance warning sign when flashing began tended to travel at speeds lower than the typical profile speed probably because it was safe to do so or that there were no realistic chances of them making the green.

## Chapter 4 Traffic Microsimulation Model

The objective of this chapter was to develop a modeling system that could be used to perform consistent, detailed analyses of actuated advance warning systems on high-speed signalized intersection approaches in Nebraska. The proposed procedure is demonstrated using two test intersections – Highway 77 and Saltillo Road in Lincoln and the intersection of Highway 370 and South 132nd Street in Omaha. Traffic conditions at the study intersection approaches were emulated using the VISSIM microscopic traffic simulation model.

VISSIM was selected for this study because of the flexibility provided by its programmable vehicle actuated programming (VAP) traffic control logic. VISSIM is a discrete, stochastic, time step based microscopic traffic simulation model with driver-vehicle-units as single entities. The model produces measures of performance commonly used for intersection analysis such as speeds, average delay, queue lengths, and emissions (VISSIM Manual 2009). Because the ability to accurately and efficiently model traffic flow characteristics, drivers' behavior, and traffic control operations is critical for obtaining realistic microsimulation results, it is critical that the model is calibrated to local conditions. Calibration involves finding the appropriate combinations of model parameters that minimize errors between the observed and simulated performance measures.

The proposed procedure for developing, calibrating, and validating traffic microsimulation models of intersections equipped with NDOR AAW devices, as applied to the two test intersections, are described below.

#### 4.1 Test Intersection 1: Highway 77 and Saltillo Road, Lincoln, NE

A detailed description of the intersection of Highway 77 and Saltillo Road is provided in section 3.1.1.

##### *4.1.1 Measures of Performance*

The first step in the model calibration and validation process was to determine appropriate performance measures. The measures selected for this study were the average waiting time on conflicting phases and the speed profiles on the approach(es) equipped with NDOR AAW devices. Average waiting time was used to calibrate the model; speed profile was used to validate it. These performance measures were chosen because (i) they were considered reasonable indicators of the operational efficiency of an intersection equipped with NDOR AAW on its high-speed approach(es) and (ii) they were fairly easy to collect both in the field and from VISSIM.

##### *4.1.2 Input Parameters*

Input data required for the VISSIM model were existing geometry, traffic counts, signal timing plans and phase sequencing, and posted speed limits. Geometric characteristics and signal timing plans were provided by NDOR. Lane widths, approach grades, lengths of left-turn and right-turn lanes, and detector and advance warning flasher locations were retrieved from blueprints provided by NDOR.

Two JAMAR traffic data collectors (TDC-8) were used to record turning movement counts at the study intersections. Counts were collected from 2:00 to 4:00 pm on Wednesday September 29, 2010 and from 2:00 to 6:00 pm on Thursday September 29, Tuesday October 5, and Wednesday October 6, 2010. Average waiting times on conflicting phases and speed profiles required for model calibration and validation were collected with the NTC mobile data

collection trailer and by videotaping traffic operations and signal indications on the minor road approaches.

#### *4.1.3 Calibration Parameters*

VISSIM has over 50 tunable model parameters. Thirteen of these parameters were identified as the most relevant to this study and were thus selected for calibration. These parameters were: (i-ii) mean and variance of the desired speed distribution; (iii) number of observed preceding vehicles; (iv) average standstill distance; (v-vi) additive and multiplicative parts of desired safety distance; (vii) minimum headway; (viii) emergency stopping distance; (ix) waiting time before diffusion; (x) lane-changing distance; and (xi - xiii) the alpha, beta1, and beta2 coefficients of VISSIM's "reaction-to-amber" function. The thirteen parameters are explained in further detail below.

##### **Desired Speed Distributions:**

The desired speed distribution is an important factor that influences roadway capacity and the travel speeds that can be realized. The desired speed represents the speed at which a vehicle travels (with a small stochastic variation) when unimpeded. Of course, the presence of other vehicles on the roadway means that the speed that is actually realized by a vehicle may differ from its desired speed. Whenever possible, and if it is safe to do so, a vehicle that is traveling at a speed lower than its desired speed will overtake the vehicle ahead of it. A desired speed distribution is coded in VISSIM by specifying its shape as well as a minimum and a maximum speed value. Intermediate points such as the 15th, 50th, and 85th percentile speeds may also be specified.

The speed limit on Highway 77 is 65 mph and it is reduced to 55 mph at approximately 1,150 feet in advance of the intersection. A desired speed distribution of 50 to 70 mph was used

for the 65 mph speed limit. This corresponds approximately to the lower and upper bounds of the 95% confidence interval for speeds that are approximately normally distributed with mean 60 mph and standard deviation 5 mph. The approximate 15th and 85th percentile speeds were 55 mph and 65 mph, respectively. Similarly a desired speed distribution range of 40 to 60 mph was used to model the reduced speed limit of 55 mph. Thus vehicles were assigned a desired speed from the 50 to 70 mph distribution as they entered the network on Highway 77. However, as they crossed the speed limit sign at the beginning of the reduced speed section, they were reassigned “new” desired speeds from the 40 to 60 mph distribution. This was done in VISSIM by placing a “desired speed decision” point at the location of the speed limit sign. The approach speed on Saltillo Road was 55 mph and thus vehicles entering the network from this roadway were also assigned desired speeds from the 40 to 60 mph distribution.

In order to model expected changes in speed while the active advance warning signs flashed, another “desired speed decision” point (with a different desired speed distribution) was placed at the location of the advance warning flashers. Exact parameter values of this desired speed distribution were not specified, *a priori*, but were instead estimated as part of the model calibration process. This reflects the observation that drivers’ response to the flasher is not known, *a priori*. This decision is consistent with the findings of previous research that have indicated that, while there may be a general decrease in speeds following the installation of advance warning flashers, some drivers tend to accelerate in order to make the green.

The desired speed distribution used in this exercise was assumed to be normal with an unknown mean assumed to be in the range 40 to 50 mph and unknown standard deviation assumed to be between 4 and 8 mph. With the mean,  $\mu$  and standard deviation,  $\sigma$  calibrated, approximate values of the minimum, the maximum, and the 15th and 85th percentile speeds were

calculated as  $\mu - 2\sigma$ ,  $\mu + 2\sigma$ ,  $\mu - \sigma$ , and  $\mu + \sigma$ , respectively. For example, a calibrated mean speed of 48 mph and a standard deviation of 7 mph would suggest normally distributed speeds between 34 mph and 62 mph, and approximate 15th and 85th percentile speeds of 41 mph and 55 mph, respectively.

#### Number of Observed Vehicles:

The “number of observed vehicles” variable affects how well vehicles in the network can predict, and react to, other vehicles’ movements. VISSIM uses a default value of four for urban driving behavior and two for all others. A range of one to four vehicles was considered in this study.

#### Car-Following Parameters:

VISSIM includes two versions of the Wiedemann model – urban driver and freeway driver. The car-following mode of the urban driver model was used in this study. The model has three tunable parameters: average standstill distance, additive part of desired safety distance, and multiplicative part of desired safety distance. The safe distance between two vehicles is given by (VISSIM Manual 2009):

$$d_{safe} = d_1 + (a_1 + a_2 z) \sqrt{v}; \quad z \in [0,1] \text{ and } z \sim N(0.5, 0.15) \quad (4.1)$$

where,

$v$  = speed ( $\text{ms}^{-1}$ )

$d_1$  = average standstill distance. This is the average desired distance between stopped vehicles. The VISSIM default is 2.0 m. Values considered reasonable for this study were 1.0 m to 3.0 m.

$a_1, a_2$  = coefficients that affect computation of the desired safety distance. The default value for the additive part  $a_1$  is 2.0. Values considered in this study were 1.0 to 3.0. The multiplicative part  $a_2$  has a default value of 3.0. Values used in this study were 2.0 to 4.0.

#### Minimum Headway:

The minimum headway distance defines the minimum distance to the vehicle in front that must be available for a lane change in standstill condition. The default value is 0.5 m. The range of values used for this parameter was 0.5 m to 3.0 m. Larger or smaller values appeared to be unreasonable.

#### Emergency Stop Position:

For a vehicle following its route, the emergency stop position defines the last possible position from where a lane change can be made. If the lane change is not possible because of high traffic volumes, the vehicle will stop at this point and wait for an acceptable gap to do so. The default is 5.0 m. A range of 2.0 m to 7.0 m was considered reasonable for this study.

#### Waiting Time before Diffusion:

The “waiting time before diffusion” variable defines the maximum amount of time a vehicle can remain at the emergency stop position waiting for a gap to change lanes in order to stay on its route. When this time is reached the vehicle is taken out of the network (diffusion).



The default waiting time in VISSIM is 60 s. Values in the range 20 s to 60 s were used in this study.

#### Lane-Change Distance:

The lane-change distance parameter is used along with the emergency stop distance parameter to model drivers' lane-change behavior as they follow their routes. It is the distance, in anticipation of a lane change, at which a driver will begin maneuvering towards the desired lane. The default is 200.0 m. Values considered reasonable for this study were 150.0 m to 300.0 m.

#### Reaction-to-Amber:

VISSIM's probabilistic reaction-to-amber function is used to define vehicle behavior as it approaches a signal control showing amber. It is a binary logistic function that uses three parameters ( $\alpha$ ,  $\beta_1$ , and  $\beta_2$ ) to calculate the probability of a driver stopping when the signal indication is amber. A decision is kept until the vehicle passes the stop line. For a vehicle traveling at a speed  $v$  and at a distance  $dx$  from the stop line (at the start of the amber indication), the stop probability is calculated as:

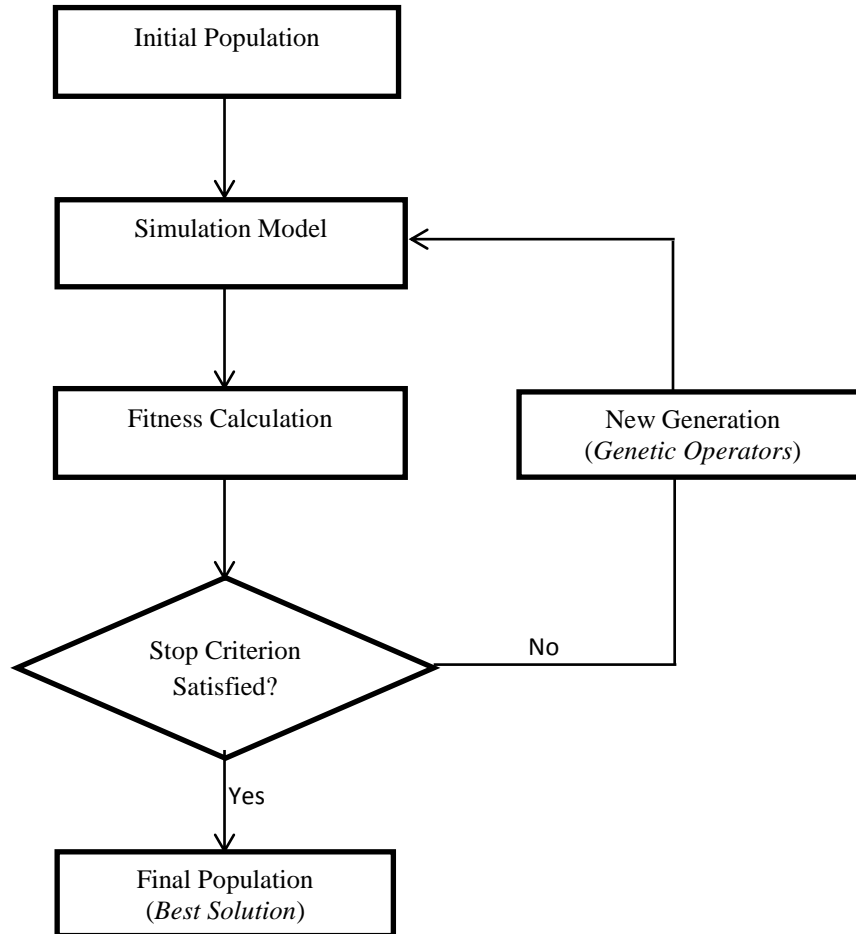
$$P_{stop} = \frac{1}{1 + e^{-\alpha - \beta_1 v - \beta_2 dx}} \quad (4.2)$$

The default parameter values used in VISSIM are  $\alpha = 1.59$ ,  $\beta_1 = -0.26$ , and  $\beta_2 = 0.27$ . Acceptable ranges used in this study were  $\alpha = [0.08, 3.10]$ ,  $\beta_1 = [-0.50, -0.01]$ , and  $\beta_2 = [0.01, 0.50]$ .

#### *4.1.4 Calibration Procedure*

The genetic algorithm (GA) was selected as the optimization tool for this study because of the following reasons: (i) it has been shown that it has advantages in dealing with non-convexity, locality, and the complex nature of transportation optimization; (ii) it searches over multiple locations and therefore has a very high likelihood of identifying a globally optimal solution; (iii) genetic algorithms only require the evaluation of an objective function with no need for gradient information; and (iv) they are rather robust when used in conjunction with simulation model calibration and can overcome the combinatorial explosion of model parameters (Kim and Rilett 2004, Yun and Park 2005, Mitchel 1998).

Genetic algorithms are stochastic algorithms whose search methods are based on the evolutionary ideas of natural selection or survival of the fittest. The GA calibration procedure starts with a randomly generated set or population of chromosomes each of which represents a potential solution to the problem under consideration; in this case a combination of simulation model parameters. The individual chromosomes undergo selection in the presence of variation-inducing operators such as mutation and crossover. A fitness function is used to evaluate each chromosome. Reproductive success varies with fitness. The processes of evaluation, selection, crossover, and mutation are repeated until a satisfactory solution is found. The main features of the GA calibration procedure are described in the following sections. A simplified flowchart of the main components is shown in figure 4.1. The GA was coded in Perl and integrated with VISSIM.



**Figure 4.1** Flowchart of Genetic Algorithm Calibration Process

Initial Population:

An initial population of 40 candidate solutions or chromosomes was used. Each chromosome is a string containing model parameter values (genes). In order to avoid any bias at the beginning of the evolutionary run, each of the 13 genes (parameters) defining a chromosome (candidate solution) was initialized with a random number within the predefined search space limits described in section 4.3.

#### Simulation Run:

A VISSIM simulation model was constructed with the input parameters described in section 4.1.2. The Perl control program was called to run the simulation for each of the 40 chromosomes. The time period for each simulation run was four hours. The main output collected at the end of every run was the average waiting time on conflicting phases (minor road approaches).

#### Fitness Calculation:

The quality of the solution provided by each chromosome was evaluated using a fitness function. The function used in this study was the mean absolute error ratio (MAER) which measures the average discrepancy between simulated and observed waiting times and is given by:

$$MAER_j = \frac{1}{m} \sum_{i=1}^m \left| \frac{TSIM_{ij} - TOBS_i}{TOBS_i} \right| \quad (4.3)$$

where,

$MAER_j$  = estimated MAER using chromosome  $j$ ;

$TSIM_{ij}$  = simulated average waiting time on minor approach  $i$  using chromosome  $j$ ;

$TOBS_i$  = observed average waiting time on minor road approach  $i$ ;

$m$  = number of minor road approaches considered ( $m = 2$ ).

#### Stop Criterion:

The stopping criterion used was a preset maximum number of generations (iterations) of the GA. This number was set equal to 100. Once this criterion was met, the chromosome with the smallest MAER was selected as the best solution.

#### New Generation:

An elitist selection strategy and the genetic operators of crossover and mutation were used to produce a new generation of chromosomes. If the stop criterion was not satisfied, then after evaluating the fitness function for the current generation of chromosomes, a subset of chromosomes was selected for use as parents in succeeding generations. The chromosomes were chosen according to their fitness value. In this study, an “elitist” selection strategy was used to ensure that the best chromosomes were preserved at each generation. This involved directly placing the best two chromosomes (as determined from their fitness values) into the next generation. A stochastic roulette wheel selection scheme was used for the process of choosing parents for subsequent recombination. That is, each chromosome was assigned a slice on a Monte Carlo-based roulette wheel proportional to its fitness. The “wheel” was spun in a simulated fashion 38 times and the parents were chosen based on where the “pointer” stopped (Gentle et al. 2004).

Once the pairs of parent chromosomes were selected, a crossover operator was used to create offspring. The offspring could be either a blend or a clone of the two parents depending on a pre-specified probability of crossover. The crossover probability used was 0.75. If no crossover took place, then the two offspring were clones of the two parents. On the other hand if crossover occurred, then the two offspring were formed by an interchange of genetic material between the two parents. This was accomplished by swapping parts based on a randomly chosen splice point on the pair of parent chromosomes.

While keeping the two elite chromosomes, the remaining 38 chromosomes were replaced by the offspring produced from crossover. Because the initial population might not contain enough variability to find the solution via crossover alone, a mutation operator was used to introduce some variability in the new set of chromosomes by randomly changing genes with probability  $P_m$ . The mutation rate ( $P_m$ ) was allowed to vary dynamically (between 0.0005 and 0.25) in the course of the evolutionary run. That is, the algorithm monitored the degree of convergence and adjusted the mutation rate accordingly. This was done to increase the chance that the algorithm did not converge prematurely to a local optimum.

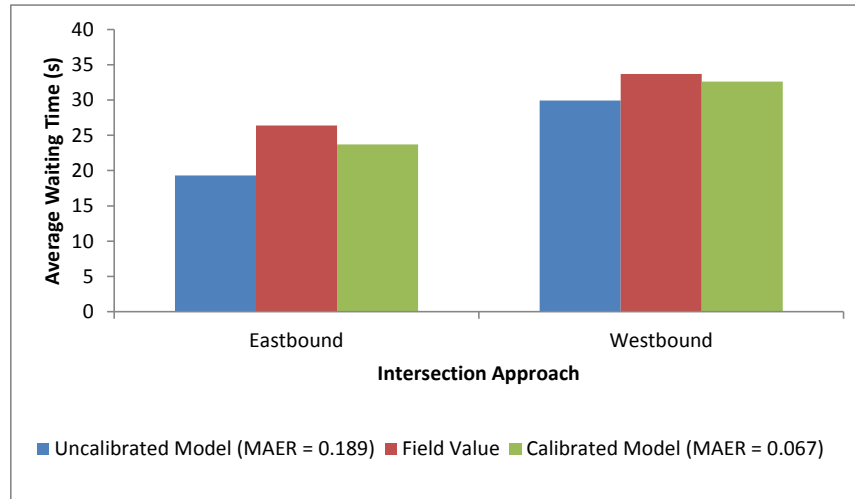
The resulting population was the new generation of chromosomes. The simulation was re-run with each member of this new generation and the processes of fitness evaluation, selection, crossover, and mutation were repeated until the stop criterion was satisfied.

#### *4.1.5 Calibration Results*

The lowest value of the mean absolute error ratio (MAER) after 100 iterations of the GA for a population of size 40 was 0.067 (figure 4.2). The VISSIM parameter values that corresponded to this MAER value were:

- Number of observed vehicles: 2
- Average standstill distance: 2.8 m
- Additive part of desired safety distance: 2.9
- Multiplicative part of desired safety distance: 2.8
- Minimum headway: 1.8 m
- Amber coefficient,  $\alpha$ : 2.417
- Amber coefficient,  $\beta_1$ : -0.033
- Amber coefficient,  $\beta_2$ : 0.167
- Desired speed at Flasher location (0-, 15-, 50-, 85-, 100-percentile): (30, 36, 42, 48, 54) mph

- Waiting time before diffusion: 36 s
- Emergency stop position: 3.1 m
- Lane change distance: 172 m



**Figure 4.2** Observed and Simulated Average Waiting Times on Minor Road Approaches

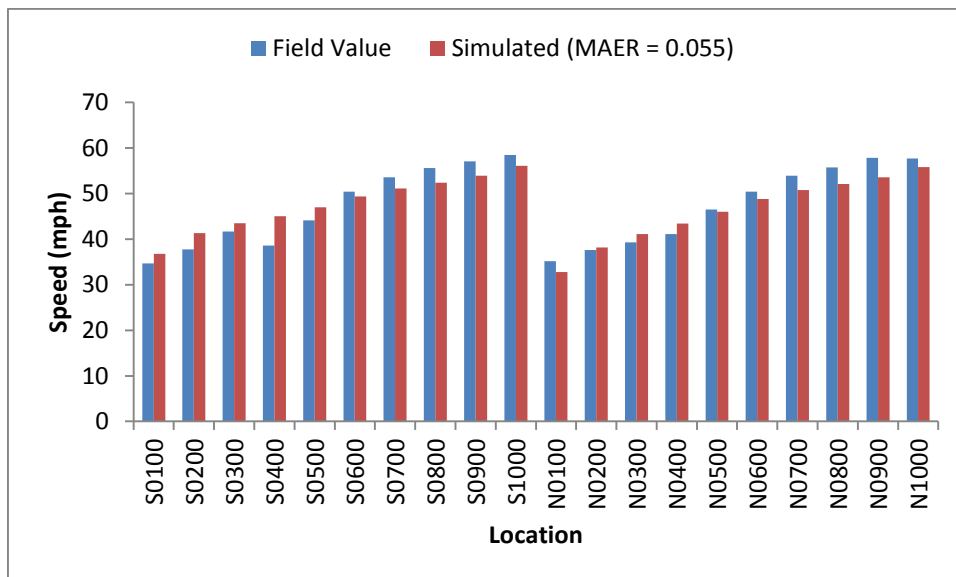
A comparison of the average waiting times obtained from the field data, the uncalibrated VISSIM model (default parameters), and the calibrated model are provided in figure 4.2. As may be seen in figure 4.2, the calibrated model compared much better with the field values (MAER = 0.067) than did the uncalibrated model (MAER = 0.189). The uncalibrated model indicated much shorter average waiting times than were observed in the field. Figure 4.2 highlights the importance of the calibration of microscopic traffic simulation models.

It should be noted that a calibrated simulation model that results in behavior not exhibited in the field cannot be credible. Consequently, an animation of the calibrated model was also viewed to ensure that the final parameters do not just produce performance measures close to

those observed in the field, but that the resulting model was visually reasonable and consistent with observed behavior.

#### 4.1.6 Model Validation

Finally, the calibrated model was validated with speed profile data from the northbound and the southbound high-speed approaches. A comparison of the simulated speed profiles and the observed profiles is provided in figure 4.3. Note that N0400 in this figure indicate average speed at a distance of 400 ft from the stop line on the northbound approach etc.



**Figure 4.3** Observed and Simulated Speed Profiles on High-Speed Approaches

As seen in figure 4.3, the plots suggest a good match ( $MAER = 0.055$ ) between the observed and simulated speed profiles. This indicated that the calibrated parameter values were appropriate for the study intersection.



## 4.2 Test Intersection 2: Highway 370 and South 132nd Street, Omaha, NE

The second intersection used for testing the model calibration and validation procedure was the intersection of Highway 370 and South 132nd Street in Omaha. A detailed description of this intersection is provided in section 3.1.2. The measures of performance and suggested ranges of calibration parameters used here were the same as those for the Highway 77 and Saltillo Road model calibration. The input parameters used in developing the baseline model were also the same; however, the parameter values were changed to reflect the description provided in section 3.1.2 and the turning movement counts observed at this site during the data collection period on December 22, 2010.

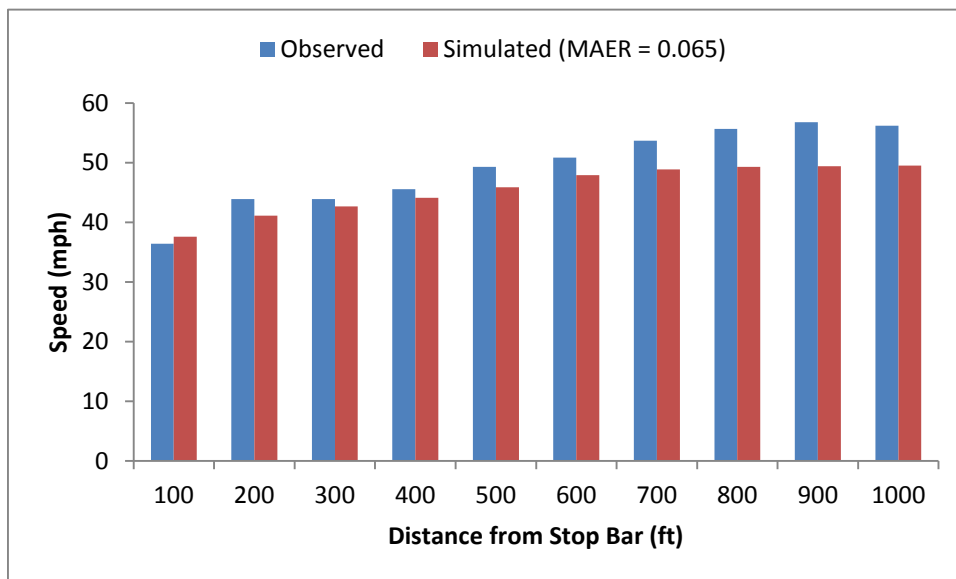
### *4.2.1 Model Calibration and Validation Results*

The model was calibrated using the average waiting time on minor road approaches and validated using the speed profile from the eastbound approach. The lowest MAER value after 100 iterations of the GA algorithm for a population size of 40 chromosomes was 0.0049; an MAER of 0.0716 was obtained with the default parameters. The following were the calibrated VISSIM parameter values that corresponded to the lowest MAER value:

- Number of observed vehicles: 3
- Average standstill distance: 1.6 m
- Additive part of desired safety distance: 1.8
- Multiplicative part of desired safety distance: 2.3
- Minimum headway: 3.0 m
- Amber coefficient,  $\alpha$ : 1.471
- Amber coefficient,  $\beta_1$ : -0.355
- Amber coefficient,  $\beta_2$ : 0.036
- Desired speed at Flasher location (0-, 15-, 50-, 85-, 100-percentile): (24, 32, 40, 48, 56) mph

- Waiting time before diffusion: 39 s
- Emergency stop position: 4.5 m
- Lane change distance: 166 m

The animation video appeared consistent with observations. The observed and simulated speed profiles for the eastbound approach are shown in figure 4.4. The plots suggest a good match (MAER = 0.065) between the observed and simulated speed profiles. These indicated that the calibrated parameter values were appropriate for the test intersection.



**Figure 4.4** Observed and Simulated Speed Profiles on Eastbound Approach

### 4.3 Summary

This chapter proposed a procedure for microscopic traffic simulation model calibration and validation and demonstrated the procedure through two case studies. The proposed procedure appeared to be effective in the calibration and validation, for VISSIM, of high-speed signalized intersections equipped with actuated advance warning systems. The procedure was

applicable to both the Lincoln and the Omaha test beds. It therefore appears that the procedure may be used to perform consistent, detailed analyses of actuated advance warning systems on similar high-speed signalized intersection approaches in Nebraska. An example of such an application is provided through a sensitivity analysis in the next chapter.

## Chapter 5 Sensitivity Analyses

This chapter examines the effects of different geometric, traffic, and signal timing parameters on the efficiency of signals where actuated advance warning systems are used. The goal was to establish general guidelines on when these devices might be installed, and perhaps even more importantly, when they should be removed because of changing demand. The effects of the different combinations of factors were evaluated by controlled experiments using VISSIM.

### 5.1 Experimental Design

#### *5.1.1 Geometric Conditions*

All simulation evaluations assumed a non-skewed, four-legged, isolated signalized intersection with high-speed major road approaches and lower-speed minor road approaches. The main road approaches each had two through/right-turn lanes and an exclusive left-turn lane. The cross road approaches each had one through/right-turn lane and one exclusive left-turn lane. All lanes were assumed to be 12 ft wide and on a zero grade.

#### *5.1.2 Traffic Conditions*

The simulations were run for two-way main road volumes of 500, 100, 1500, and 2,000 veh/h and minor road to main road two-way volume ratios of between 0.25 and 1.00. Left-turn and right-turn proportions were assumed equal on all approaches. The levels of turn percentages used were 5, 10, and 20%. Major road approach speeds were assumed equal to 45, 55, and 65 mph. The corresponding set of minor road approach speeds were 35, 45, or 55 mph with the speed on the major road assumed to be higher than that on the minor road. Input desired speeds were assumed to be approximately normally distributed with means equal to the approach speeds and a standard deviation equal to 5 mph. Pedestrian volumes were assumed equal to zero in all cases. A heavy vehicle percentage of 4% (similar to that at the Highway 370 and South 132nd

Street intersection) was used in all models. The various factors and levels used in the simulation are summarized in table 5.1.

**Table 5.1** Parameter Values Used in Sensitivity Analysis

Category	Parameter	Main Road	Cross Road
Geometry	Through / Right-Turn lanes	2	1
	Exclusive Left-Turn lanes	1	1
	Lane Width (ft)	12	12
	Grade	0	0
Traffic	Two-way volume (veh/h)	500 - 2000	500 - 2000
	Turn volumes (%)	5 - 20	5 - 20
	Approach speed (mph)	45 - 65	35 - 55
	Approach speed standard deviation (mph)	5	5
	Heavy vehicles (%)	4	4
	Pedestrians	None	None
Advance Warning System	Advance detector location (ft)	498 - 935	None
	Flashing beacon location (ft)	300 - 650	None
	Advance warning before end of green (s)	5 - 7	None
Signal Timing	Through/Right-turn phases		
	Minimum green (s)	15	10
	Maximum green (s)	50 - 60	30 - 40
	Yellow + All-red (s)	5	5
	Passage time (s)	3	3
	Left-turn phases		
	Minimum green (s)	7	10
	Maximum green (s)	30	30 - 40
	Yellow + All-red (s)	3	5
	Passage time (s)	3	3

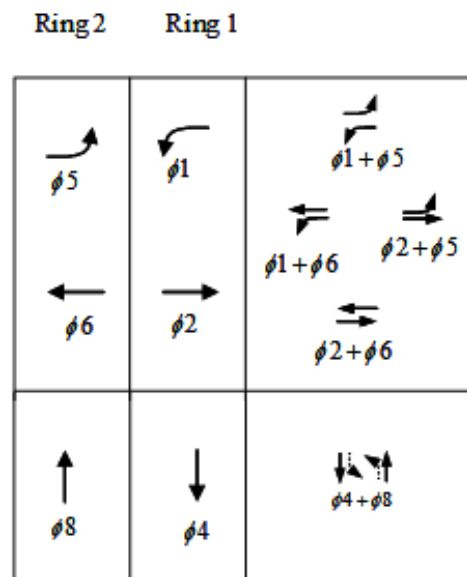
### 5.1.3 NDOR AAW System Parameters

The main parameters of the NDOR AAW SYSTEM are the locations of the advance detector, locations of the advance warning flasher, and the duration of advance warning before end of green (i.e. lead flash). The values of these parameters for the 65 mph approach speed were 935 ft from the stop line, 650 ft from the stop line, and 7 s respectively. The corresponding

values used for the 55 mph approach speed were 693 ft, 450 ft, and 6 s. Parameter values equal to 498 ft, 300 ft, and 5 s were used for the 45 mph speed. For further information regarding these parameter values, refer to the report by McCoy and Pesti (2002).

#### 5.1.4 Signal Timing

All signals were assumed to be fully actuated operating with six phases: two phases on the minor road and four phases on the major road. Each minor road phase serves all movements from the corresponding approach (figure 5.1).



**Figure 5.1** Signal Phasing Plan Used in Simulations

Any time that a vehicle call was present on the minor road, the controller would wait until there was no active call on the major road. At this time the controller would “freeze” and the advance beacons would flash for a duration equal to the pre-specified lead flash. At the end of the lead flash interval, the controller would resume operation with the major road amber indication. The beacons continued to flash until the through phase signals on that leg turned

green. Since the signals were fully actuated, any phase, except the major road through phases on recall, could be skipped if no demand was present.

Two sets of signal timings were used. One set had a maximum green time of 50 s on the major road through phases and a maximum green of 30 s on the minor road phases; the other was 60 s on the major road through and 40 s on the minor road. The minimum green time on the major road through phases was set equal to 15 s, and for the minor road phases it was 10 s. Amber times on all minor road phases and on all major road through phases were 4.5 s. These were followed by 0.5 s of all red. The major road left-turn phases were protected and had 30 s of maximum green, 7 s minimum green, and 3 s of amber. All gap extensions were for 3 s. The major road through phase extensions were from the advance detector locations. All other extensions were from detectors located at the stop line.

## 5.2 Measures of Performance

The effects of the different factors were assessed in terms of both traffic operations and safety. Operational effectiveness was assessed by determining the delay associated with waiting times on minor road phases. Safety was indirectly assessed by evaluating the total number of conflicts resulting from rear-end, lane-change, or path-crossing movements.

Conflicts were analyzed using the Federal Highway Administration's (FHWA) Surrogate Safety Assessment Model (SSAM). SSAM analyzes vehicle trajectories produced from VISSIM, and other traffic microsimulation models such as AIMSUN, Paramics, and Texas, to identify and classify conflict events. For each event SSAM also calculates several surrogate safety measures including: (i) post-encroachment time (PET), that is, the time between when the first vehicle last occupied a position and the second vehicle subsequently arrived to the same position, a value of zero indicating an actual collision; and (ii) time-to-collision (TTC), namely,

the time for two vehicles to collide if they continued at their present speed and stayed on the same path (SSAM Manual, Burnett 2011). These proximal safety measures are considered “valid and credible precursors of actual crashes” (Archer and Young 2009). This study used threshold values of 1.5 s and 4.0 s for TTC and PET, respectively (Nate 2011, Archer and Young 2009).

### 5.3 VISSIM Model Parameters

In general, the parameter values used in the simulations were the averages of the calibrated parameter values for the two intersections studied in chapter 4. In each simulation, the desired speed at the flasher location was assumed to follow an approximately normal distribution with a mean 10 mph less than the approach speed used in the simulation ( $S$ ) and a standard deviation equal to 7 mph. It is worth mentioning that these parameter values also produced results that were consistent with field data when used to model the two test intersections; MAER for observed and simulated speed profiles less than 7% in both cases. These parameter values were thus considered reasonable for modeling similar intersections in Nebraska. The following were the values used:

- Number of observed vehicles: 3
- Average standstill distance: 2.2 m
- Additive part of desired safety distance: 2.4
- Multiplicative part of desired safety distance: 2.5
- Minimum headway: 2.4 m
- Amber coefficient,  $\alpha$ : 1.944
- Amber coefficient,  $\beta_1$ : -0.194
- Amber coefficient,  $\beta_2$ : 0.102
- Desired speed at flasher location (0-, 85-, 100-percentile): ( $S-24$ ,  $S-3$ ,  $S+4$ ) mph



- Waiting time before diffusion: 38 s
- Emergency stop position: 3.8 m
- Lane change distance: 169 m

## 5.4 Simulation Runs

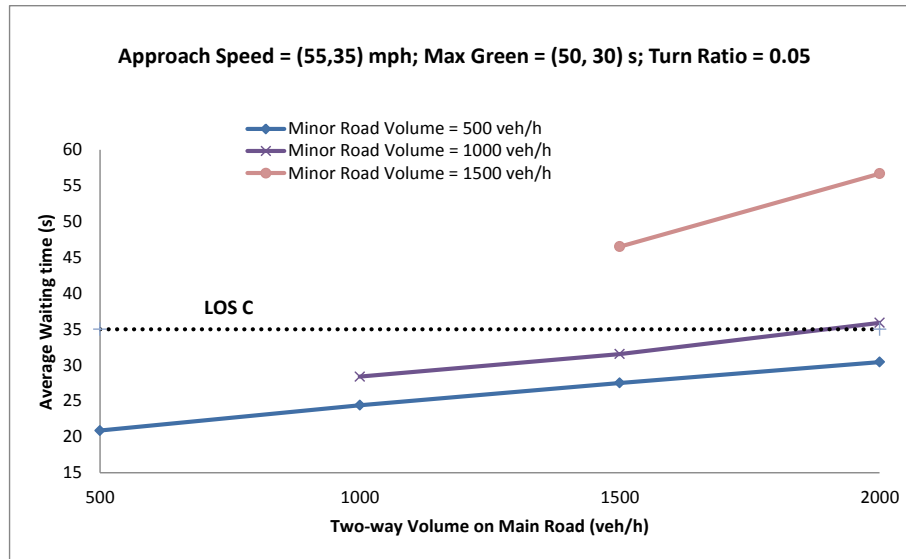
The simulation experiments consisted of eight levels of major road/minor road approach speed combinations, ten levels of major road/minor road two-way volume combinations, three levels of left-turn and right-turn proportions, and two levels of signal timing parameters for a total of 480 factor combinations. All experiments simulated one hour of operation at the intersection and were replicated ten times. Vehicle trajectory files from the simulation runs were exported to FHWA's SSAM software for conflict analysis. Results of the ten replications were then averaged to obtain the desired measures of performance.

## 5.5 Results

The results of the simulation runs are tabulated in appendix A. Some of the most important trends observed are described below.

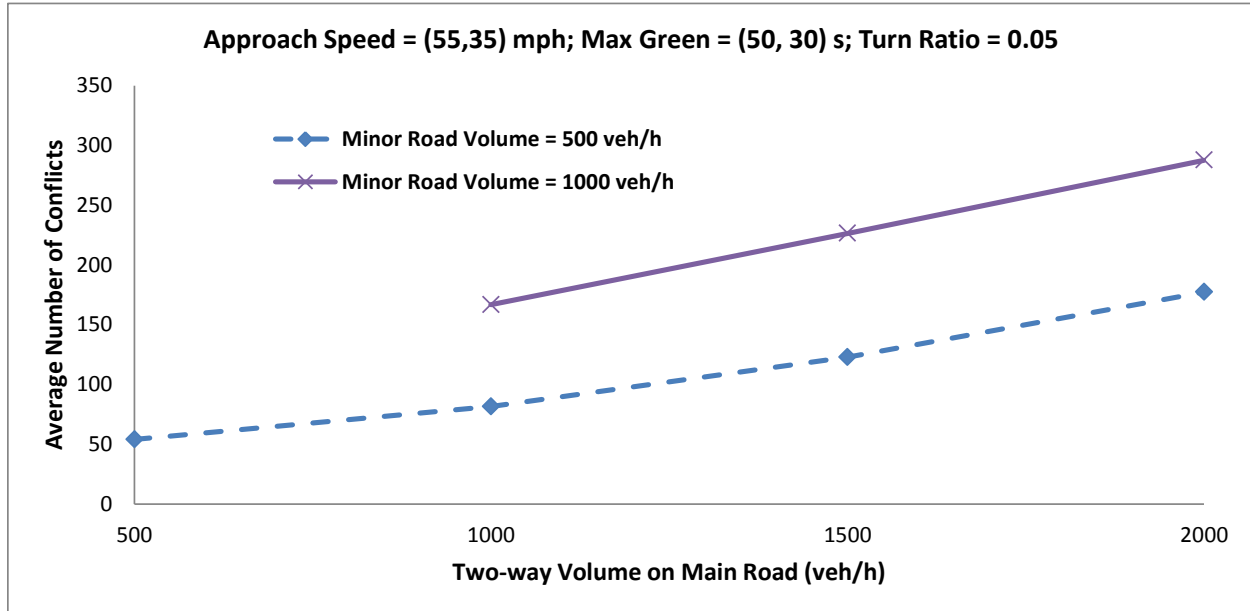
### *5.5.1 Effect of Approach Volume*

The effect of approach volume is demonstrated in figures 5.2 and 5.3. The general trends are illustrated with a signalized intersection having an approach speed of 55 mph on the approach that is equipped with an NDOR AAW system and 35 mph on the cross road. The maximum green time settings were assumed to be 50 s and 30 s on the major road through phases and the minor road through phases, respectively. The left-turn and right-turn percentage was assumed to be 5%. It should be noted that the other combinations of speed, volume, turn percentage, and signal timing exhibited similar patterns (see appendix B) and therefore are not discussed here.



**Figure 5.2** Effects of Approach Volumes on Average Waiting Times

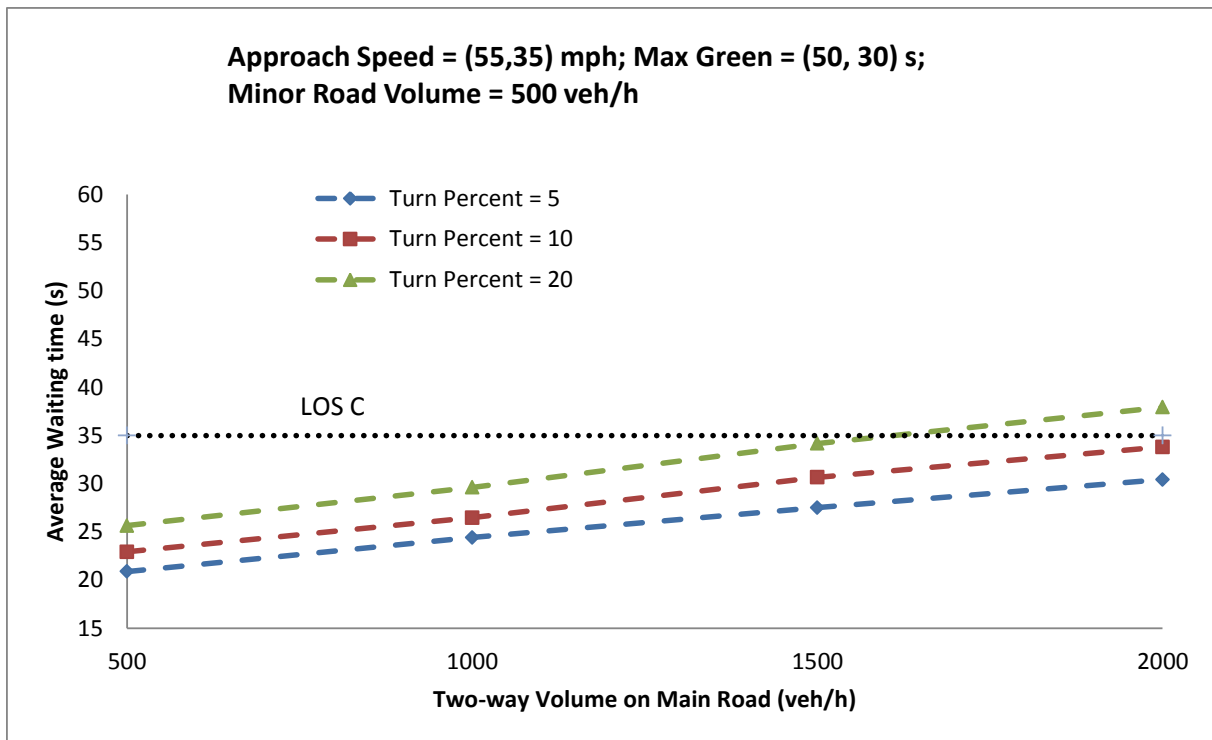
For a given volume of traffic on the minor road, as illustrated in figure 5.2, the average waiting time increased as the traffic volume on the major road was increased. This is consistent with more frequent green time extensions on the major road through phases. As expected, the number of conflicts also increased with increasing traffic volumes (figure 5.3). Additionally, at any given major road volume, both the number of conflicts and the average waiting time on the minor approaches increased as the minor road volume increased. It may also be seen that for a minor road volume of 500 veh/h, the average delay, or waiting time, on the minor road approaches remained “tolerable” for all major road volumes less than or equal to 2000 veh/h. Tolerable is defined in this report as not greater than 35 s/veh i.e., Level of Service (LOS) C. At a minor road volume of 1000 veh/h, the waiting times were tolerable only for major road volumes not exceeding 1800 veh/h. The corresponding total number of traffic conflicts was 250. The average waiting times were greater than 35 veh/h when the minor road volume increased to 1500 veh/h, irrespective of the corresponding volume on the major road (up to 2000 veh/h).



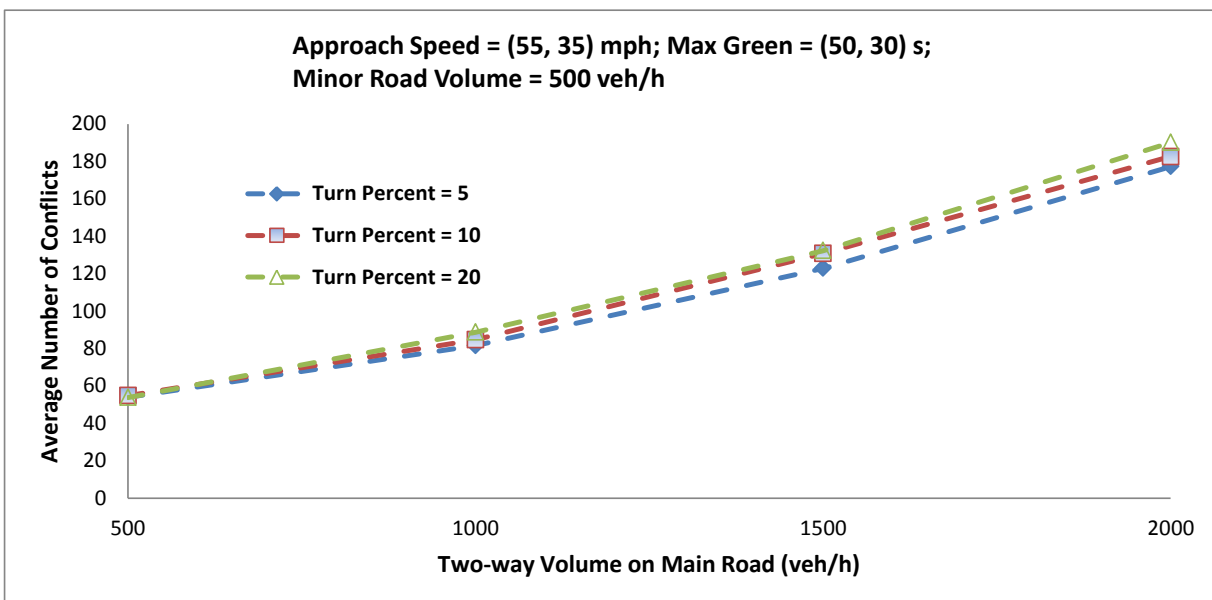
**Figure 5.3** Effects of Approach Volumes on Average Number of Traffic Conflicts

### 5.5.2 Effect of Turn Percentage

Figures 5.4 and 5.5 represent the effect of turn percentage on minor approach delay, or average waiting time, and total number of traffic conflicts. The trend lines indicate that average waiting time on the minor road phases increased with increasing turn percentage consistent with more frequent calls for, and increased duration of, left-turn phases. In general, figure 5.5 indicates that the number of conflicts also increased with increasing turn percentage. However, the relative sizes of the increases were generally only minimal, especially for approach volumes less than 100 veh/h. This was not surprising as all left-turns from the major road were protected whereas those from the minor road were permissive, made through gaps in the opposing through traffic. Increasing turn-percentages meant lower through traffic volumes and possibly more frequent, and longer, gaps.



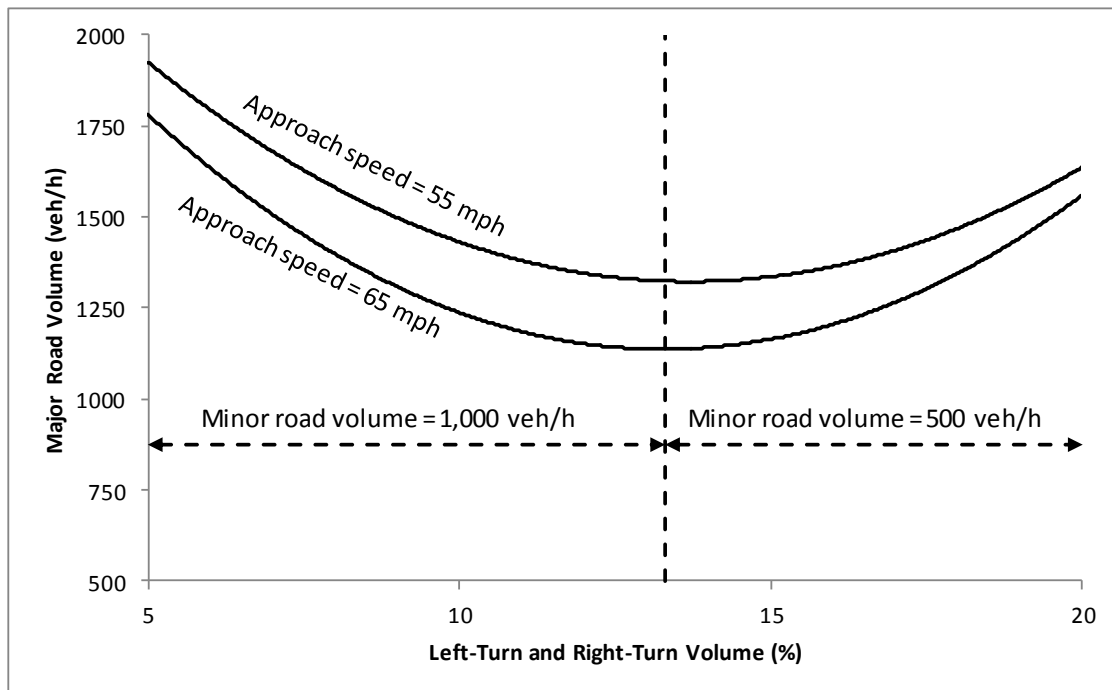
**Figure 5.4** Effects of Turn Percentages on Average Waiting Times



**Figure 5.5** Effects of Turn Percentages on Average Number of Traffic Conflicts

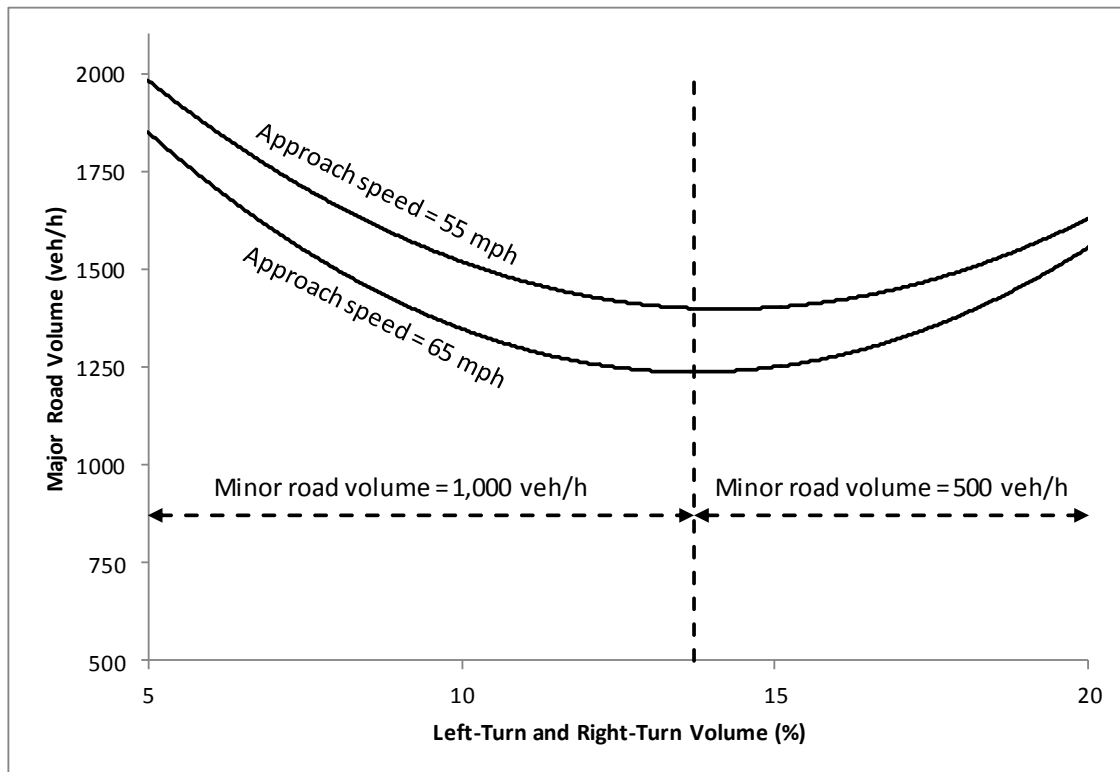
### 5.5.3 Delay Curves

The results of this study suggested that the average waiting time for vehicles on the minor road approaches was greater than 35 s/veh for all instances where the two-way approach volume on the cross street was 1,500 veh/h or more. On the other hand, when the minor road volume was 500 veh/h (or less) the average waiting time for all cases (with the exception of a 20% turn percentage and approach speeds greater than 45 mph) was less than 35 s/veh. Figures 5.6 and 5.7 summarize the interrelationships between approach speed, turn percentage, minor road volume, and major road volume. The curves represent the upper bounds of major road volumes beyond which the average delay to minor approach traffic exceeded 35 s/veh. Similar plots may be extracted from the plots in appendix B to establish other “delay boundaries” where necessary.



Note:  $G = (50, 30)$

**Figure 5.6** “Tolerable” Delay Boundaries for Minor Road Vehicles



Note:  $G = (50, 30)$

**Figure 5.7** “Tolerable” Delay Boundaries for Minor Road Vehicles

### 5.6 Suggested Guidelines

The study by McCoy and Pesti (2002) provided general guidelines regarding conditions for which installing the NDOR AAW system would be more beneficial than the traditional advance warning systems. Consequently, this study defers to the report by McCoy and Pesti (2002) for guidelines regarding installation.

For guidelines regarding removal, this study proposes the following approach based on the delay boundaries established in section 5.5. Note that the discussion is based on a “maximum tolerable delay” of 35 s/veh. Where local conditions or practice require a different limit on delay, the appropriate “delay boundary” plots may be extracted from appendices A and

B. The discussion also assumes that estimates of the expected maximum hourly two-way volume on the minor road, the expected turn percentages, and the approach speed on the subject major road are available.

1. Minor approach volumes of 1,500 veh/h or more, with equal or higher volumes on major road, result in an average delay to minor road vehicles of greater than 35 s/veh.

Therefore, these are candidates for removal.

2. For minor road volumes of 500 or 1,000 veh/h and approach speeds of 55 mph or 65 mph,
  - a. Figure 5.10 may be used if the maximum delay settings on the major road and the minor road are approximately 50 and 30 s, respectively;
  - b. Figure 5.11 may be used if the maximum delay settings on the major road and the minor road are approximately 60 and 40 s, respectively.

For example, if the speed on the high-speed approach is 65 mph, the minor road traffic volume is 1,000 veh/h, the turn percentage is 10%, and the signals have maximum green time settings of 50 s and 30 s on the major through and the minor road phases respectively, then according to figure 5.6 the volume on the major road beyond which delays to minor road traffic become intolerable or greater than 35 s/veh is 1,430 veh/h. Thus, removal of the NDOR AAW devices based on demand should be considered at approximately this volume.

## 5.7 Summary

The study by McCoy and Pesti (2002) provided general guidelines regarding conditions for which installing the NDOR AAW system could be more beneficial than the traditional advance warning systems. Yet, there are currently no standards as to when these devices should be removed because of changing demand. Using the traffic microsimulation software VISSIM,

this chapter examined the effects of different geometric, traffic, and signal timing parameters on the efficiency of signals where NDOR AAW systems are used. This chapter established general guidelines regarding when these devices might be removed because of changing demand. However, as this study did not consider the conventional advance detection alternative, installation guidelines were deferred to those provided by McCoy and Pesti. Nonetheless, it should be noted that the volumes obtained from this study may also be interpreted as the range of traffic volumes within which acceptable performance, based on delay, could be expected from an NDOR AAW system installation. Consequently, these could serve as additional guide regarding installation.



## Chapter 6 Conclusions and Recommendations

### 6.1 Conclusions

#### *6.1.1 Safety Effects*

This research examined the safety changes attained as a result of implementing the NDOR actuated advance warning system at 26 signalized high-speed intersections using a fully Bayesian model. The expected crash reduction rates estimated based on 30,000 Monte Carlo samples are summarized in table 2.3. It was found that there were reductions of 0.5% in crashes involving heavy vehicles, 1.2% in rear-end crashes, 43.6% in right-angle crashes, 20.7% in fatal and non-fatal injury crashes, and 8.2% for all crashes combined.

The results also suggested that there is a greater than 90% probability that the installation of an NDOR AAW system is effective at improving overall safety at high-speed signalized intersections in Nebraska. The NDOR AAW system design is therefore worth considering for dilemma zone protection at other high-speed signalized intersections in the state from the perspective of potential safety improvements.

#### *6.1.2 Operational Effects*

The study also examined the performance of NDOR AAW systems in the field with respect to the efficiency of traffic operations at two intersections. Overall, the results indicated that high-speed approaches equipped with NDOR AAW devices had a significantly lower than expected number of vehicles in their dilemma zones at the onset of amber. In particular, it was found that the number of vehicles in their dilemma zones when the signal indication changed from green to amber was 77.2% smaller than the number that would have been expected if the NDOR AAW system had not been installed. In addition, drivers upstream of the advance warning sign when flashing began had a higher than expected tendency to slow down. This

indicates that the NDOR AAW devices may have performed as intended because drivers seemed to heed the warning to “prepare to stop” when the AAW alerted drivers to the impending end of the green signal.

The average waiting time on minor road approaches was less than 30 s at both intersections. As expected, the frequency at which the through green intervals on the high-speed approaches ended by max-out was low (less than 0.05%) at both study sites. This is further evidence that the NDOR AAW systems seemed to be working as intended at both sites.

#### *6.1.3 Simulation Model*

One objective of this paper was to develop a traffic microsimulation modeling framework that could be used by NDOR engineers to perform consistent, detailed analyses of NDOR AAW systems. The genetic algorithm based model calibration and validation procedure developed in this research appeared to be effective in the calibration and validation (for VISSIM) of high-speed signalized intersections equipped with NDOR AAW systems. The procedure was successfully applied to the calibration of the two test intersections in Lincoln and Omaha. In each case, the calibrated model provided more realistic results than the uncalibrated model (default values) and reaffirmed the importance of the calibration of microscopic traffic simulation models.

#### *6.1.4 Sensitivity Analysis*

The sensitivity analysis explored the effects of different geometric, traffic, and signal timing parameters on the efficiency of signals where NDOR AAW systems were used. The total number of traffic conflicts and the average delay to vehicles on the minor road approaches were used as the measures of effectiveness. The results indicated that, in general, both the number of conflicts and the average delay increased with increasing approach speeds, approach volumes,

and turn percentages. Both sets of signal timing plans used in the study resulted in similar patterns of delay and number of conflicts, with values for the (60, 40) s plan generally lower than those for the (50, 30) s plan.

The main road volumes necessary to maintain a pre-specified level of “acceptable” delay to minor road vehicles were determined. These are summarized in figures 5.6 and 5.7.

## 6.2 Recommendations

Based on the results of the research, the following recommendations are made with regard to the implementation and removal of AAW devices on the state highway system in Nebraska:

1. From the perspective of safety improvement, the NDOR AAW system is worth considering for dilemma zone protection at high-speed signalized intersections in the state. Nonetheless, it should be noted that the appropriate choice of any treatment method can only be made after careful diagnosis of the specific intersection.
2. Installation of NDOR AAW devices on the state highway system should be in accordance with the policies provided by McCoy and Pesti (2002).
3. Delay curves presented in Chapter 5 may guide decisions regarding the removal of NDOR AAW devices arising from anticipated changes in demand.

In addition, further research involving additional test sites for model calibration and validation is recommended. One goal for such a study would be to categorize intersections on the state highway system and to develop a “typical” traffic microsimulation model for each category that could readily be used by NDOR engineers.

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## Appendix A Results of Sensitivity Analysis

**Table A.1** Results of Sensitivity Analysis: G = (50, 30) s, S = (45, 35) mph

Model	Maximum Green (s)		Approach Speed (mph)		Two-Way Volume		Turn	Waiting Time (s)		Number of Conflicts	
	Main St	Cross St	Main St	Cross St	Main St	Cross St	Ratio	Average	Std. dev.	Average	Std. dev.
1	50	30	45	35	500	500	0.05	20.1	0.69	55.5	8.25
2	50	30	45	35	1000	500	0.05	23.2	1.24	82.7	11.08
3	50	30	45	35	1000	1000	0.05	26.8	0.87	171.2	11.11
4	50	30	45	35	1500	500	0.05	26.8	1.79	131.9	10.16
5	50	30	45	35	1500	1000	0.05	30.2	1.23	232.5	12.70
6	50	30	45	35	1500	1500	0.05	44.1	5.67		
7	50	30	45	35	2000	500	0.05	29.8	1.48	192.0	13.79
8	50	30	45	35	2000	1000	0.05	33.7	0.69	296.0	11.60
9	50	30	45	35	2000	1500	0.05	56.1	10.07		
10	50	30	45	35	2000	2000	0.05	179.4	91.72		
11	50	30	45	35	500	500	0.1	21.8	0.94	54.6	7.21
12	50	30	45	35	1000	500	0.1	25.4	1.26	83.3	11.17
13	50	30	45	35	1000	1000	0.1	30.5	1.25	169.1	9.01
14	50	30	45	35	1500	500	0.1	29.4	1.10	131.9	14.26
15	50	30	45	35	1500	1000	0.1	34.3	1.53	234.4	14.12
16	50	30	45	35	1500	1500	0.1	99.3	41.77		
17	50	30	45	35	2000	500	0.1	31.7	1.79	193.1	13.40
18	50	30	45	35	2000	1000	0.1	38.2	1.33	305.6	13.82
19	50	30	45	35	2000	1500	0.1	155.1	61.47		
20	50	30	45	35	2000	2000	0.1	839.1	193.46		
21	50	30	45	35	500	500	0.2	24.8	1.35	56.3	6.70
22	50	30	45	35	1000	500	0.2	28.3	1.36	85.9	8.01
23	50	30	45	35	1000	1000	0.2	35.5	1.56	162.4	13.97
24	50	30	45	35	1500	500	0.2	31.6	1.66	135.2	6.96
25	50	30	45	35	1500	1000	0.2	40.8	2.64	229.9	15.91
26	50	30	45	35	1500	1500	0.2	224.7	63.76		
27	50	30	45	35	2000	500	0.2	35.0	1.32	206.2	13.37
28	50	30	45	35	2000	1000	0.2	49.7	5.26	300.0	21.47
29	50	30	45	35	2000	1500	0.2	302.7	83.69		
30	50	30	45	35	2000	2000	0.2	725.5	176.71		



**Table A.2** Results of Sensitivity Analysis: G = (50, 30) s, S = (45, 45) mph

Model	Maximum Green (s)		Approach Speed (mph)		Two-Way Volume		Turn	Waiting Time (s)		Number of Conflicts	
	Main St	Cross St	Main St	Cross St	Main St	Cross St	Ratio	Average	Std. dev.	Average	Std. dev.
31	50	30	45	45	500	500	0.05	19.7	0.66	45.4	4.25
32	50	30	45	45	1000	500	0.05	23.1	1.21	77.0	10.90
33	50	30	45	45	1000	1000	0.05	26.5	1.09	141.2	11.63
34	50	30	45	45	1500	500	0.05	26.3	0.82	114.8	11.12
35	50	30	45	45	1500	1000	0.05	30.3	1.18	209.8	12.06
36	50	30	45	45	1500	1500	0.05	39.9	6.20		
37	50	30	45	45	2000	500	0.05	29.3	0.96	180.5	12.27
38	50	30	45	45	2000	1000	0.05	34.7	0.98	271.4	18.43
39	50	30	45	45	2000	1500	0.05	48.1	3.84		
40	50	30	45	45	2000	2000	0.05	134.9	35.41		
41	50	30	45	45	500	500	0.1	21.6	1.07	47.7	9.51
42	50	30	45	45	1000	500	0.1	25.4	1.04	77.3	8.87
43	50	30	45	45	1000	1000	0.1	30.3	1.42	151.0	7.27
44	50	30	45	45	1500	500	0.1	28.6	1.32	124.7	8.31
45	50	30	45	45	1500	1000	0.1	33.9	1.78	205.4	16.58
46	50	30	45	45	1500	1500	0.1	89.4	29.73		
47	50	30	45	45	2000	500	0.1	31.9	1.17	190.3	17.00
48	50	30	45	45	2000	1000	0.1	39.0	2.61	283.0	17.80
49	50	30	45	45	2000	1500	0.1	139.6	54.66		
50	50	30	45	45	2000	2000	0.1	698.1	147.70		
51	50	30	45	45	500	500	0.2	24.3	1.02	49.2	3.49
52	50	30	45	45	1000	500	0.2	27.9	0.81	82.0	8.98
53	50	30	45	45	1000	1000	0.2	35.6	1.83	145.5	14.00
54	50	30	45	45	1500	500	0.2	31.6	1.54	129.0	7.80
55	50	30	45	45	1500	1000	0.2	40.2	3.29	212.4	13.16
56	50	30	45	45	1500	1500	0.2	194.5	70.97		
57	50	30	45	45	2000	500	0.2	35.1	1.67	195.8	14.37
58	50	30	45	45	2000	1000	0.2	48.6	3.31	281.5	13.93
59	50	30	45	45	2000	1500	0.2	328.0	125.85		
60	50	30	45	45	2000	2000	0.2	659.9	188.24		

**Table A.3** Results of Sensitivity Analysis: G = (50, 30) s, S = (55, 35) mph

Model	Maximum Green (s)		Approach Speed (mph)		Two-Way Volume		Turn	Waiting Time (s)		Number of Conflicts	
	Main St	Cross St	Main St	Cross St	Main St	Cross St	Ratio	Average	Std. dev.	Average	Std. dev.
61	50	30	55	35	500	500	0.05	20.9	0.66	53.9	10.13
62	50	30	55	35	1000	500	0.05	24.4	1.64	81.5	8.03
63	50	30	55	35	1000	1000	0.05	28.4	1.31	166.7	16.93
64	50	30	55	35	1500	500	0.05	27.5	1.47	122.8	10.67
65	50	30	55	35	1500	1000	0.05	31.5	1.47	226.3	14.34
66	50	30	55	35	1500	1500	0.05	46.5	7.82		
67	50	30	55	35	2000	500	0.05	30.4	1.80	177.2	12.09
68	50	30	55	35	2000	1000	0.05	35.9	1.81	287.6	25.27
69	50	30	55	35	2000	1500	0.05	56.7	13.49		
70	50	30	55	35	2000	2000	0.05	154.4	78.10		
71	50	30	55	35	500	500	0.1	22.9	1.44	54.8	4.85
72	50	30	55	35	1000	500	0.1	26.5	1.44	84.7	9.57
73	50	30	55	35	1000	1000	0.1	32.1	1.21	169.9	14.81
74	50	30	55	35	1500	500	0.1	30.7	1.55	130.8	11.38
75	50	30	55	35	1500	1000	0.1	35.4	1.99	224.8	12.91
76	50	30	55	35	1500	1500	0.1	103.0	40.19		
77	50	30	55	35	2000	500	0.1	33.8	1.97	182.6	15.78
78	50	30	55	35	2000	1000	0.1	40.6	2.94	292.3	25.37
79	50	30	55	35	2000	1500	0.1	175.3	52.52		
80	50	30	55	35	2000	2000	0.1	818.3	188.47		
81	50	30	55	35	500	500	0.2	25.6	1.30	53.8	8.82
82	50	30	55	35	1000	500	0.2	29.6	0.91	88.7	9.75
83	50	30	55	35	1000	1000	0.2	36.6	1.84	167.8	15.51
84	50	30	55	35	1500	500	0.2	34.2	1.65	132.3	8.49
85	50	30	55	35	1500	1000	0.2	43.7	1.88	221.5	14.00
86	50	30	55	35	1500	1500	0.2	254.6	58.57		
87	50	30	55	35	2000	500	0.2	37.9	2.02	190.1	23.02
88	50	30	55	35	2000	1000	0.2	54.9	9.06	291.3	24.77
89	50	30	55	35	2000	1500	0.2	385.3	69.95		
90	50	30	55	35	2000	2000	0.2	713.6	206.97		

**Table A.4** Results of Sensitivity Analysis: G = (50, 30) s, S = (55, 45) mph

Model	Maximum Green (s)		Approach Speed (mph)		Two-Way Volume		Turn	Waiting Time (s)		Number of Conflicts	
	Main St	Cross St	Main St	Cross St	Main St	Cross St	Ratio	Average	Std. dev.	Average	Std. dev.
91	50	30	55	45	500	500	0.05	20.5	0.85	45.7	5.70
92	50	30	55	45	1000	500	0.05	23.4	1.03	74.9	10.03
93	50	30	55	45	1000	1000	0.05	28.1	1.18	137.5	14.47
94	50	30	55	45	1500	500	0.05	27.4	1.38	106.0	8.94
95	50	30	55	45	1500	1000	0.05	31.1	1.49	201.7	14.28
96	50	30	55	45	1500	1500	0.05	42.2	4.92		
97	50	30	55	45	2000	500	0.05	29.5	2.02	164.3	13.82
98	50	30	55	45	2000	1000	0.05	35.4	1.46	261.5	17.17
99	50	30	55	45	2000	1500	0.05	54.1	10.42		
100	50	30	55	45	2000	2000	0.05	177.7	93.39		
101	50	30	55	45	500	500	0.1	22.4	0.66	45.1	6.21
102	50	30	55	45	1000	500	0.1	26.2	1.03	73.6	9.99
103	50	30	55	45	1000	1000	0.1	32.0	1.26	147.6	12.61
104	50	30	55	45	1500	500	0.1	30.1	1.15	114.2	7.71
105	50	30	55	45	1500	1000	0.1	35.6	2.05	201.3	8.03
106	50	30	55	45	1500	1500	0.1	90.4	28.23		
107	50	30	55	45	2000	500	0.1	33.4	1.57	173.4	14.50
108	50	30	55	45	2000	1000	0.1	40.3	1.72	268.2	16.37
109	50	30	55	45	2000	1500	0.1	170.6	71.28		
110	50	30	55	45	2000	2000	0.1	731.9	190.35		
111	50	30	55	45	500	500	0.2	25.1	1.29	49.1	6.30
112	50	30	55	45	1000	500	0.2	29.4	1.09	78.5	9.03
113	50	30	55	45	1000	1000	0.2	36.5	1.92	146.3	7.93
114	50	30	55	45	1500	500	0.2	32.9	1.34	122.1	10.54
115	50	30	55	45	1500	1000	0.2	43.4	2.52	198.6	12.44
116	50	30	55	45	1500	1500	0.2	240.3	86.02		
117	50	30	55	45	2000	500	0.2	37.6	1.87	176.3	15.80
118	50	30	55	45	2000	1000	0.2	53.1	4.11	261.8	9.14
119	50	30	55	45	2000	1500	0.2	315.2	101.23		
120	50	30	55	45	2000	2000	0.2	623.1	237.22		

**Table A.5** Results of Sensitivity Analysis: G = (50, 30) s, S = (55, 55) mph

Model	Maximum Green (s)		Approach Speed (mph)		Two-Way Volume		Turn	Waiting Time (s)		Number of Conflicts	
	Main St	Cross St	Main St	Cross St	Main St	Cross St	Ratio	Average	Std. dev.	Average	Std. dev.
121	50	30	55	55	500	500	0.05	20.4	0.69	38.9	5.92
122	50	30	55	55	1000	500	0.05	23.5	1.15	77.3	7.48
123	50	30	55	55	1000	1000	0.05	27.6	0.80	136.6	9.66
124	50	30	55	55	1500	500	0.05	27.2	1.21	108.5	11.33
125	50	30	55	55	1500	1000	0.05	30.3	1.48	186.8	9.38
126	50	30	55	55	1500	1500	0.05	41.6	5.06		
127	50	30	55	55	2000	500	0.05	29.7	1.67	161.8	12.83
128	50	30	55	55	2000	1000	0.05	34.3	1.32	251.1	19.87
129	50	30	55	55	2000	1500	0.05	50.8	11.69		
130	50	30	55	55	2000	2000	0.05	126.1	44.67		
131	50	30	55	55	500	500	0.1	22.2	0.80	41.8	4.37
132	50	30	55	55	1000	500	0.1	26.3	0.92	66.7	8.98
133	50	30	55	55	1000	1000	0.1	30.5	1.42	139.6	17.39
134	50	30	55	55	1500	500	0.1	29.4	1.35	118.4	11.77
135	50	30	55	55	1500	1000	0.1	34.6	1.66		
136	50	30	55	55	1500	1500	0.1	77.3	23.14	192.4	10.50
137	50	30	55	55	2000	500	0.1	32.9	1.05	161.6	14.05
138	50	30	55	55	2000	1000	0.1	40.3	2.12	258.6	15.38
139	50	30	55	55	2000	1500	0.1	155.0	60.58		
140	50	30	55	55	2000	2000	0.1	768.0	174.14		
141	50	30	55	55	500	500	0.2	24.8	1.51	45.3	5.93
142	50	30	55	55	1000	500	0.2	29.2	1.12	77.4	7.38
143	50	30	55	55	1000	1000	0.2	36.6	2.09	142.4	10.00
144	50	30	55	55	1500	500	0.2	32.6	1.89	118.4	12.09
145	50	30	55	55	1500	1000	0.2	41.7	1.79	192.4	13.16
146	50	30	55	55	1500	1500	0.2	225.0	76.59		
147	50	30	55	55	2000	500	0.2	37.1	2.19	177.9	14.11
148	50	30	55	55	2000	1000	0.2	53.8	9.31	258.7	20.94
149	50	30	55	55	2000	1500	0.2	361.1	81.36		
150	50	30	55	55	2000	2000	0.2	660.6	218.12		

**Table A.6** Results of Sensitivity Analysis: G = (50, 30) s, S = (65, 35) mph

Model	Maximum Green (s)		Approach Speed (mph)		Two-Way Volume		Turn	Waiting Time (s)		Number of Conflicts	
	Main St	Cross St	Main St	Cross St	Main St	Cross St	Ratio	Average	Std. dev.	Average	Std. dev.
151	50	30	65	35	500	500	0.05	21.6	1.13	52.6	7.57
152	50	30	65	35	1000	500	0.05	25.5	1.09	80.0	8.65
153	50	30	65	35	1000	1000	0.05	28.9	0.88	165.6	13.74
154	50	30	65	35	1500	500	0.05	27.8	1.60	111.2	7.83
155	50	30	65	35	1500	1000	0.05	32.7	1.30	215.2	12.98
156	50	30	65	35	1500	1500	0.05	48.0	8.75		
157	50	30	65	35	2000	500	0.05	31.7	1.09	164.9	11.24
158	50	30	65	35	2000	1000	0.05	36.4	1.69	265.9	16.62
159	50	30	65	35	2000	1500	0.05	70.6	19.08		
160	50	30	65	35	2000	2000	0.05	169.9	103.76		
161	50	30	65	35	500	500	0.1	23.3	1.28	52.0	6.91
162	50	30	65	35	1000	500	0.1	28.3	1.04	79.2	8.97
163	50	30	65	35	1000	1000	0.1	33.0	1.32	171.3	11.55
164	50	30	65	35	1500	500	0.1	30.2	1.62	124.9	12.56
165	50	30	65	35	1500	1000	0.1	37.4	2.76	220.5	9.98
166	50	30	65	35	1500	1500	0.1	127.7	46.09		
167	50	30	65	35	2000	500	0.1	34.6	1.47	172.7	18.71
168	50	30	65	35	2000	1000	0.1	41.5	2.35	275.3	17.80
169	50	30	65	35	2000	1500	0.1	242.9	98.18		
170	50	30	65	35	2000	2000	0.1	788.6	200.92		
171	50	30	65	35	500	500	0.2	26.7	1.53	57.5	9.68
172	50	30	65	35	1000	500	0.2	30.8	0.79	83.4	9.86
173	50	30	65	35	1000	1000	0.2	38.8	2.83	157.7	12.54
174	50	30	65	35	1500	500	0.2	34.5	1.60	125.6	10.66
175	50	30	65	35	1500	1000	0.2	45.9	4.25	215.7	17.93
176	50	30	65	35	1500	1500	0.2	281.7	94.04		
177	50	30	65	35	2000	500	0.2	39.3	2.14	175.5	10.90
178	50	30	65	35	2000	1000	0.2	55.0	6.35	276.6	19.86
179	50	30	65	35	2000	1500	0.2	373.5	143.03		
180	50	30	65	35	2000	2000	0.2	700.1	219.74		

**Table A.7** Results of Sensitivity Analysis: G = (50, 30) s, S = (65, 45) mph

Model	Maximum Green (s)		Approach Speed (mph)		Two-Way Volume		Turn	Waiting Time (s)		Number of Conflicts	
	Main St	Cross St	Main St	Cross St	Main St	Cross St	Ratio	Average	Std. dev.	Average	Std. dev.
181	50	30	65	45	500	500	0.05	21.3	0.89	45.2	6.88
182	50	30	65	45	1000	500	0.05	24.8	1.19	71.7	8.64
183	50	30	65	45	1000	1000	0.05	29.0	1.15	141.1	13.68
184	50	30	65	45	1500	500	0.05	27.5	1.00	110.3	9.93
185	50	30	65	45	1500	1000	0.05	32.9	2.12	186.3	12.46
186	50	30	65	45	1500	1500	0.05	45.8	5.07		
187	50	30	65	45	2000	500	0.05	30.8	1.35	156.8	17.55
188	50	30	65	45	2000	1000	0.05	36.7	1.83	249.0	11.17
189	50	30	65	45	2000	1500	0.05	60.6	20.61		
190	50	30	65	45	2000	2000	0.05	158.5	85.31		
191	50	30	65	45	500	500	0.1	23.0	0.78	45.6	4.65
192	50	30	65	45	1000	500	0.1	27.2	1.60	68.1	4.58
193	50	30	65	45	1000	1000	0.1	32.5	1.41	147.4	13.75
194	50	30	65	45	1500	500	0.1	31.1	1.29	113.6	10.27
195	50	30	65	45	1500	1000	0.1	37.5	1.87	187.6	11.44
196	50	30	65	45	1500	1500	0.1	125.3	56.27		
197	50	30	65	45	2000	500	0.1	33.4	2.07	168.4	6.83
198	50	30	65	45	2000	1000	0.1	42.4	2.00	247.9	16.28
199	50	30	65	45	2000	1500	0.1	172.3	74.76		
200	50	30	65	45	2000	2000	0.1	783.3	172.06		
201	50	30	65	45	500	500	0.2	26.1	1.03	50.0	5.77
202	50	30	65	45	1000	500	0.2	31.0	1.43	75.9	8.81
203	50	30	65	45	1000	1000	0.2	37.7	2.31	147.2	8.64
204	50	30	65	45	1500	500	0.2	34.3	1.25	119.2	7.66
205	50	30	65	45	1500	1000	0.2	44.7	1.74	190.0	12.54
206	50	30	65	45	1500	1500	0.2	241.0	58.92		
207	50	30	65	45	2000	500	0.2	39.3	2.22	166.4	5.04
208	50	30	65	45	2000	1000	0.2	54.3	9.56	257.3	21.15
209	50	30	65	45	2000	1500	0.2	324.3	79.43		
210	50	30	65	45	2000	2000	0.2	631.5	227.01		

**Table A.8** Results of Sensitivity Analysis: G = (50, 30) s, S = (65, 55) mph

Model	Maximum Green (s)		Approach Speed (mph)		Two-Way Volume		Turn Ratio	Waiting Time (s)		Number of Conflicts	
	Main St	Cross St	Main St	Cross St	Main St	Cross St		Average	Std. dev.	Average	Std. dev.
211	50	30	65	55	500	500	0.05	21.1	0.69	41.9	6.33
212	50	30	65	55	1000	500	0.05	24.7	1.14	68.0	12.77
213	50	30	65	55	1000	1000	0.05	28.1	0.93	132.7	10.11
214	50	30	65	55	1500	500	0.05	28.1	1.53	98.7	12.23
215	50	30	65	55	1500	1000	0.05	32.0	0.87	177.2	11.97
216	50	30	65	55	1500	1500	0.05	43.6	5.66		
217	50	30	65	55	2000	500	0.05	30.2	1.87	165.4	13.24
218	50	30	65	55	2000	1000	0.05	35.0	1.24	244.7	19.18
219	50	30	65	55	2000	1500	0.05	60.2	21.10		
220	50	30	65	55	2000	2000	0.05	119.1	33.15		
221	50	30	65	55	500	500	0.1	23.3	1.29	46.2	4.21
222	50	30	65	55	1000	500	0.1	26.2	1.00	67.3	6.91
223	50	30	65	55	1000	1000	0.1	31.5	1.47	136.5	12.20
224	50	30	65	55	1500	500	0.1	30.8	1.58	106.2	11.36
225	50	30	65	55	1500	1000	0.1	35.8	1.46	188.6	16.65
226	50	30	65	55	1500	1500	0.1	89.0	33.10		
227	50	30	65	55	2000	500	0.1	33.3	1.24	158.1	12.85
228	50	30	65	55	2000	1000	0.1	41.8	3.56	253.2	13.50
229	50	30	65	55	2000	1500	0.1	160.1	79.33		
230	50	30	65	55	2000	2000	0.1	726.7	170.95		
231	50	30	65	55	500	500	0.2	25.4	0.90	48.6	6.85
232	50	30	65	55	1000	500	0.2	29.4	1.50	74.9	8.74
233	50	30	65	55	1000	1000	0.2	38.1	2.96	139.1	9.18
234	50	30	65	55	1500	500	0.2	33.4	1.13	115.6	8.97
235	50	30	65	55	1500	1000	0.2	43.8	2.59	186.0	14.05
236	50	30	65	55	1500	1500	0.2	254.6	71.84		
237	50	30	65	55	2000	500	0.2	38.2	2.05	167.1	11.49
238	50	30	65	55	2000	1000	0.2	52.4	7.21	250.8	23.38
239	50	30	65	55	2000	1500	0.2	423.7	88.95		
240	50	30	65	55	2000	2000	0.2	690.9	198.13		

**Table A.9** Results of Sensitivity Analysis: G = (60, 40) s, S = (45, 35) mph

Model	Maximum Green (s)		Approach Speed (mph)		Two-Way Volume		Turn	Waiting Time (s)		Number of Conflicts	
	Main St	Cross St	Main St	Cross St	Main St	Cross St	Ratio	Average	Std. dev.	Average	Std. dev.
241	60	40	45	35	500	500	0.05	20.2	0.82	55.7	8.93
242	60	40	45	35	1000	500	0.05	23.2	1.19	82.4	10.80
243	60	40	45	35	1000	1000	0.05	26.6	1.01	163.6	10.65
244	60	40	45	35	1500	500	0.05	26.8	1.82	131.6	10.19
245	60	40	45	35	1500	1000	0.05	30.1	1.09	228.4	16.08
246	60	40	45	35	1500	1500	0.05	36.7	2.48		
247	60	40	45	35	2000	500	0.05	29.8	1.41	192.5	12.09
248	60	40	45	35	2000	1000	0.05	33.8	1.08	309.5	19.46
249	60	40	45	35	2000	1500	0.05	42.8	3.92		
250	60	40	45	35	2000	2000	0.05	126.4	35.68		
251	60	40	45	35	500	500	0.1	21.9	1.08	54.8	6.71
252	60	40	45	35	1000	500	0.1	25.3	1.37	85.0	11.10
253	60	40	45	35	1000	1000	0.1	29.5	1.29	171.2	17.05
254	60	40	45	35	1500	500	0.1	29.3	0.83	131.4	13.51
255	60	40	45	35	1500	1000	0.1	33.6	0.77	222.6	13.48
256	60	40	45	35	1500	1500	0.1	53.2	9.42		
257	60	40	45	35	2000	500	0.1	31.7	1.87	192.0	11.42
258	60	40	45	35	2000	1000	0.1	37.2	1.57	296.4	15.92
259	60	40	45	35	2000	1500	0.1	75.1	39.52		
260	60	40	45	35	2000	2000	0.1	605.8	131.66		
261	60	40	45	35	500	500	0.2	24.8	1.52	55.6	7.97
262	60	40	45	35	1000	500	0.2	28.3	1.32	87.2	7.24
263	60	40	45	35	1000	1000	0.2	32.8	1.39	158.1	15.95
264	60	40	45	35	1500	500	0.2	31.6	1.45	134.9	7.14
265	60	40	45	35	1500	1000	0.2	37.9	1.26	218.8	14.83
266	60	40	45	35	1500	1500	0.2	73.7	17.36		
267	60	40	45	35	2000	500	0.2	35.0	1.30	205.0	13.06
268	60	40	45	35	2000	1000	0.2	43.0	2.09	295.9	13.45
269	60	40	45	35	2000	1500	0.2	111.8	36.04		
270	60	40	45	35	2000	2000	0.2	379.4	122.03		



**Table A.10** Results of Sensitivity Analysis: G = (60, 40) s, S = (45, 45) mph

Model	Maximum Green (s)		Approach Speed (mph)		Two-Way Volume		Turn	Waiting Time (s)		Number of Conflicts	
	Main St	Cross St	Main St	Cross St	Main St	Cross St	Ratio	Average	Std. dev.	Average	Std. dev.
271	60	40	45	45	500	500	0.05	19.7	0.65	45.8	4.47
272	60	40	45	45	1000	500	0.05	23.2	1.25	76.7	11.19
273	60	40	45	45	1000	1000	0.05	26.3	0.99	146.0	11.31
274	60	40	45	45	1500	500	0.05	26.3	0.68	117.4	8.82
275	60	40	45	45	1500	1000	0.05	30.4	0.95	207.3	11.60
276	60	40	45	45	1500	1500	0.05	34.8	1.50		
277	60	40	45	45	2000	500	0.05	29.7	1.32	180.5	12.19
278	60	40	45	45	2000	1000	0.05	33.8	1.15	275.8	26.76
279	60	40	45	45	2000	1500	0.05	41.7	3.19		
280	60	40	45	45	2000	2000	0.05	142.4	36.23		
281	60	40	45	45	500	500	0.1	21.4	0.83	47.7	9.87
282	60	40	45	45	1000	500	0.1	25.7	1.15	78.1	10.34
283	60	40	45	45	1000	1000	0.1	29.4	1.00	146.9	9.50
284	60	40	45	45	1500	500	0.1	28.8	1.26	125.8	7.97
285	60	40	45	45	1500	1000	0.1	33.9	1.53	210.0	10.83
286	60	40	45	45	1500	1500	0.1	47.0	7.06		
287	60	40	45	45	2000	500	0.1	31.7	1.06	193.4	15.95
288	60	40	45	45	2000	1000	0.1	37.4	1.08	281.2	14.29
289	60	40	45	45	2000	1500	0.1	61.1	12.68		
290	60	40	45	45	2000	2000	0.1	585.1	173.48		
291	60	40	45	45	500	500	0.2	24.2	1.36	49.1	3.45
292	60	40	45	45	1000	500	0.2	28.0	0.80	81.9	9.07
293	60	40	45	45	1000	1000	0.2	33.6	1.09	149.3	8.73
294	60	40	45	45	1500	500	0.2	31.5	1.34	127.9	7.55
295	60	40	45	45	1500	1000	0.2	37.9	1.89	201.8	12.74
296	60	40	45	45	1500	1500	0.2	65.6	12.16		
297	60	40	45	45	2000	500	0.2	35.0	1.66	195.9	15.00
298	60	40	45	45	2000	1000	0.2	42.5	1.43	270.1	9.86
299	60	40	45	45	2000	1500	0.2	94.7	20.55		
300	60	40	45	45	2000	2000	0.2	481.6	183.74		

**Table A.11** Results of Sensitivity Analysis: G = (60, 40) s, S = (55, 35) mph

Model	Maximum Green (s)		Approach Speed (mph)		Two-Way Volume		Turn	Waiting Time (s)		Number of Conflicts	
	Main St	Cross St	Main St	Cross St	Main St	Cross St	Ratio	Average	Std. dev.	Average	Std. dev.
301	60	40	55	35	500	500	0.05	20.9	0.68	54.0	9.81
302	60	40	55	35	1000	500	0.05	24.5	1.70	81.8	8.04
303	60	40	55	35	1000	1000	0.05	28.0	0.78	169.7	13.75
304	60	40	55	35	1500	500	0.05	27.9	1.47	123.6	10.72
305	60	40	55	35	1500	1000	0.05	31.3	1.41	223.3	16.32
306	60	40	55	35	1500	1500	0.05	37.9	3.86		
307	60	40	55	35	2000	500	0.05	30.6	1.50	178.8	11.98
308	60	40	55	35	2000	1000	0.05	34.8	1.08	285.5	12.96
309	60	40	55	35	2000	1500	0.05	46.1	7.65		
310	60	40	55	35	2000	2000	0.05	147.6	52.61		
311	60	40	55	35	500	500	0.1	22.8	1.42	56.4	4.77
312	60	40	55	35	1000	500	0.1	26.6	1.55	85.6	9.25
313	60	40	55	35	1000	1000	0.1	31.1	0.88	173.0	13.98
314	60	40	55	35	1500	500	0.1	30.8	1.51	128.4	15.44
315	60	40	55	35	1500	1000	0.1	35.0	1.01	223.6	14.80
316	60	40	55	35	1500	1500	0.1	52.0	8.04		
317	60	40	55	35	2000	500	0.1	33.9	1.85	182.6	16.61
318	60	40	55	35	2000	1000	0.1	39.6	1.63	290.1	21.20
319	60	40	55	35	2000	1500	0.1	69.4	16.15		
320	60	40	55	35	2000	2000	0.1	646.5	202.73		
321	60	40	55	35	500	500	0.2	25.8	1.23	55.3	8.47
322	60	40	55	35	1000	500	0.2	29.6	0.80	89.4	9.00
323	60	40	55	35	1000	1000	0.2	35.6	1.40	165.2	9.39
324	60	40	55	35	1500	500	0.2	34.7	1.68	127.5	13.27
325	60	40	55	35	1500	1000	0.2	40.0	1.75	210.3	16.30
326	60	40	55	35	1500	1500	0.2	91.0	29.06		
327	60	40	55	35	2000	500	0.2	38.2	1.92	189.8	21.78
328	60	40	55	35	2000	1000	0.2	45.7	2.36	277.3	22.12
329	60	40	55	35	2000	1500	0.2	159.0	59.17		
330	60	40	55	35	2000	2000	0.2	605.2	240.60		

**Table A.12** Results of Sensitivity Analysis:  $G = (60, 40)$  s,  $S = (55, 45)$  mph

Model	Maximum Green (s)		Approach Speed (mph)		Two-Way Volume		Turn	Waiting Time (s)		Number of Conflicts	
	Main St	Cross St	Main St	Cross St	Main St	Cross St	Ratio	Average	Std. dev.	Average	Std. dev.
331	60	40	55	45	500	500	0.05	20.5	0.90	45.1	5.93
332	60	40	55	45	1000	500	0.05	23.3	0.89	75.4	9.65
333	60	40	55	45	1000	1000	0.05	28.3	0.64	142.1	10.85
334	60	40	55	45	1500	500	0.05	27.6	1.40	105.4	9.24
335	60	40	55	45	1500	1000	0.05	31.0	1.19	195.8	14.05
336	60	40	55	45	1500	1500	0.05	36.7	2.34		
337	60	40	55	45	2000	500	0.05	29.7	2.14	161.3	9.29
338	60	40	55	45	2000	1000	0.05	35.3	1.36	259.3	17.41
339	60	40	55	45	2000	1500	0.05	43.2	4.45		
340	60	40	55	45	2000	2000	0.05	173.8	79.73		
341	60	40	55	45	500	500	0.1	22.6	0.85	44.7	6.40
342	60	40	55	45	1000	500	0.1	26.2	1.05	73.4	9.65
343	60	40	55	45	1000	1000	0.1	31.2	1.13	145.3	8.93
344	60	40	55	45	1500	500	0.1	30.1	1.15	114.0	7.32
345	60	40	55	45	1500	1000	0.1	34.7	1.52	203.2	9.38
346	60	40	55	45	1500	1500	0.1	46.6	4.77		
347	60	40	55	45	2000	500	0.1	33.6	1.41	169.8	12.89
348	60	40	55	45	2000	1000	0.1	39.0	1.21	262.4	17.57
349	60	40	55	45	2000	1500	0.1	68.5	17.44		
350	60	40	55	45	2000	2000	0.1	579.8	103.24		
351	60	40	55	45	500	500	0.2	25.1	1.26	48.4	5.91
352	60	40	55	45	1000	500	0.2	29.5	1.23	78.1	8.70
353	60	40	55	45	1000	1000	0.2	35.3	1.44	149.5	8.41
354	60	40	55	45	1500	500	0.2	33.0	1.31	118.5	10.24
355	60	40	55	45	1500	1000	0.2	40.3	1.56	193.3	14.86
356	60	40	55	45	1500	1500	0.2	75.2	17.19		
357	60	40	55	45	2000	500	0.2	37.6	1.90	175.7	16.92
358	60	40	55	45	2000	1000	0.2	46.6	1.52	267.2	13.36
359	60	40	55	45	2000	1500	0.2	134.4	56.55		
360	60	40	55	45	2000	2000	0.2	444.3	86.06		

**Table A.13** Results of Sensitivity Analysis: G = (60, 40) s, S = (55, 55) mph

Model	Maximum Green (s)		Approach Speed (mph)		Two-Way Volume		Turn	Waiting Time (s)		Number of Conflicts	
	Main St	Cross St	Main St	Cross St	Main St	Cross St	Ratio	Average	Std. dev.	Average	Std. dev.
361	60	40	55	55	500	500	0.05	20.6	0.75	41.0	5.98
362	60	40	55	55	1000	500	0.05	23.7	1.57	75.5	9.81
363	60	40	55	55	1000	1000	0.05	27.3	1.12	134.0	10.21
364	60	40	55	55	1500	500	0.05	27.4	1.20	109.2	11.02
365	60	40	55	55	1500	1000	0.05	30.6	1.39	189.8	6.39
366	60	40	55	55	1500	1500	0.05	34.6	1.46		
367	60	40	55	55	2000	500	0.05	29.6	1.20	164.2	11.00
368	60	40	55	55	2000	1000	0.05	34.4	1.62	244.8	16.73
369	60	40	55	55	2000	1500	0.05	41.3	4.49		
370	60	40	55	55	2000	2000	0.05	140.6	85.27		
371	60	40	55	55	500	500	0.1	22.4	0.92	41.2	4.49
372	60	40	55	55	1000	500	0.1	26.3	0.97	66.5	8.55
373	60	40	55	55	1000	1000	0.1	29.9	0.91	140.5	11.49
374	60	40	55	55	1500	500	0.1	29.1	1.10	119.0	13.98
375	60	40	55	55	1500	1000	0.1	34.5	1.26	194.2	10.67
376	60	40	55	55	1500	1500	0.1	45.7	5.62		
377	60	40	55	55	2000	500	0.1	33.2	1.21	165.2	14.86
378	60	40	55	55	2000	1000	0.1	38.6	1.71	257.3	12.52
379	60	40	55	55	2000	1500	0.1	59.1	9.98		
380	60	40	55	55	2000	2000	0.1	477.6	112.28		
381	60	40	55	55	500	500	0.2	24.7	1.37	45.6	6.47
382	60	40	55	55	1000	500	0.2	29.2	1.19	76.9	8.18
383	60	40	55	55	1000	1000	0.2	34.4	1.76	139.7	7.63
384	60	40	55	55	1500	500	0.2	32.7	1.69	116.7	10.95
385	60	40	55	55	1500	1000	0.2	39.1	1.64	192.4	16.89
386	60	40	55	55	1500	1500	0.2	72.8	11.55		
387	60	40	55	55	2000	500	0.2	37.4	1.93	176.6	17.66
388	60	40	55	55	2000	1000	0.2	44.8	2.25	256.4	18.96
389	60	40	55	55	2000	1500	0.2	119.6	36.92		
390	60	40	55	55	2000	2000	0.2	463.7	181.84		

**Table A.14** Results of Sensitivity Analysis: G = (60, 40) s, S = (65, 35) mph

Model	Maximum Green (s)		Approach Speed (mph)		Two-Way Volume		Turn	Waiting Time (s)		Number of Conflicts	
	Main St	Cross St	Main St	Cross St	Main St	Cross St	Ratio	Average	Std. dev.	Average	Std. dev.
391	60	40	65	35	500	500	0.05	21.6	1.18	52.8	7.79
392	60	40	65	35	1000	500	0.05	25.6	1.00	80.0	8.74
393	60	40	65	35	1000	1000	0.05	28.5	1.14	163.6	6.33
394	60	40	65	35	1500	500	0.05	28.0	1.58	112.0	6.68
395	60	40	65	35	1500	1000	0.05	32.5	1.29	213.5	18.33
396	60	40	65	35	1500	1500	0.05	39.5	2.49		
397	60	40	65	35	2000	500	0.05	31.6	0.80	164.8	9.96
398	60	40	65	35	2000	1000	0.05	35.7	1.06	271.0	11.91
399	60	40	65	35	2000	1500	0.05	45.3	5.60		
400	60	40	65	35	2000	2000	0.05	145.7	83.38		
401	60	40	65	35	500	500	0.1	23.7	1.55	52.2	7.33
402	60	40	65	35	1000	500	0.1	28.3	0.99	79.6	8.51
403	60	40	65	35	1000	1000	0.1	32.0	1.41	169.1	9.33
404	60	40	65	35	1500	500	0.1	30.7	1.35	125.3	10.32
405	60	40	65	35	1500	1000	0.1	36.3	1.16	210.9	15.78
406	60	40	65	35	1500	1500	0.1	54.9	9.85		
407	60	40	65	35	2000	500	0.1	35.0	1.51	171.2	15.78
408	60	40	65	35	2000	1000	0.1	40.5	1.12	269.7	17.37
409	60	40	65	35	2000	1500	0.1	76.6	13.08		
410	60	40	65	35	2000	2000	0.1	671.3	149.70		
411	60	40	65	35	500	500	0.2	26.9	1.80	56.4	8.30
412	60	40	65	35	1000	500	0.2	31.0	1.01	83.9	9.96
413	60	40	65	35	1000	1000	0.2	36.1	1.75	158.5	12.47
414	60	40	65	35	1500	500	0.2	34.7	1.45	125.1	10.25
415	60	40	65	35	1500	1000	0.2	41.7	2.14	208.9	12.90
416	60	40	65	35	1500	1500	0.2	99.1	30.34		
417	60	40	65	35	2000	500	0.2	39.3	1.36	175.0	9.31
418	60	40	65	35	2000	1000	0.2	46.9	2.44	271.5	12.25
419	60	40	65	35	2000	1500	0.2	162.7	56.53		
420	60	40	65	35	2000	2000	0.2	595.5	210.03		

**Table A.15** Results of Sensitivity Analysis: G = (60, 40) s, S = (65, 45) mph

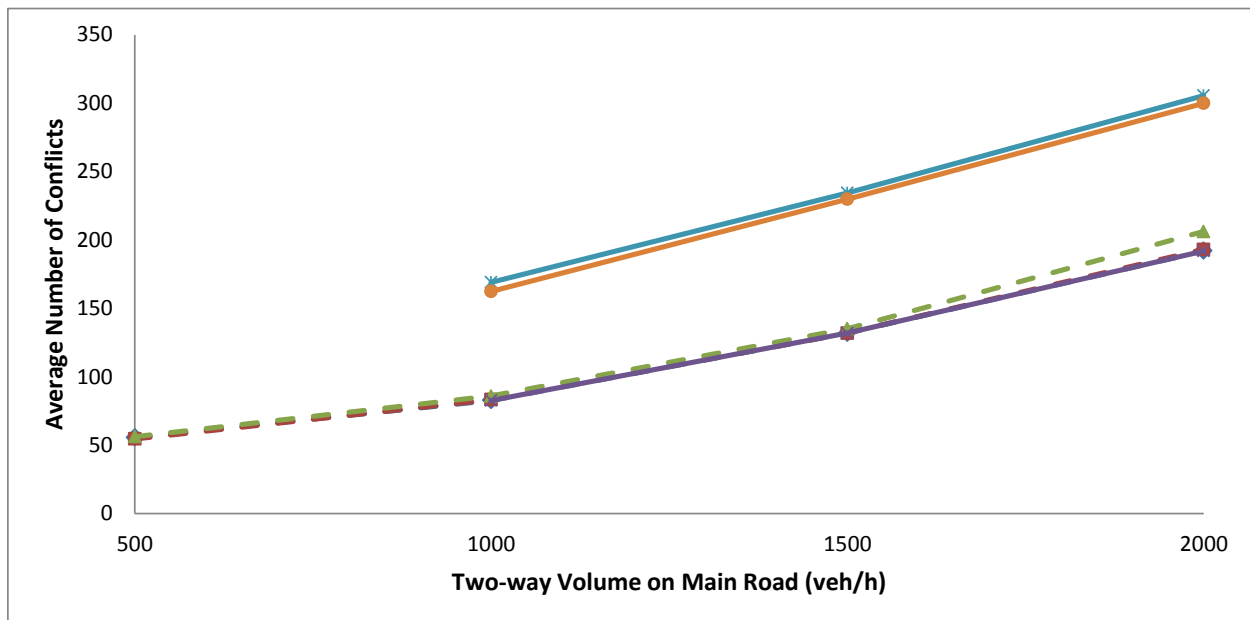
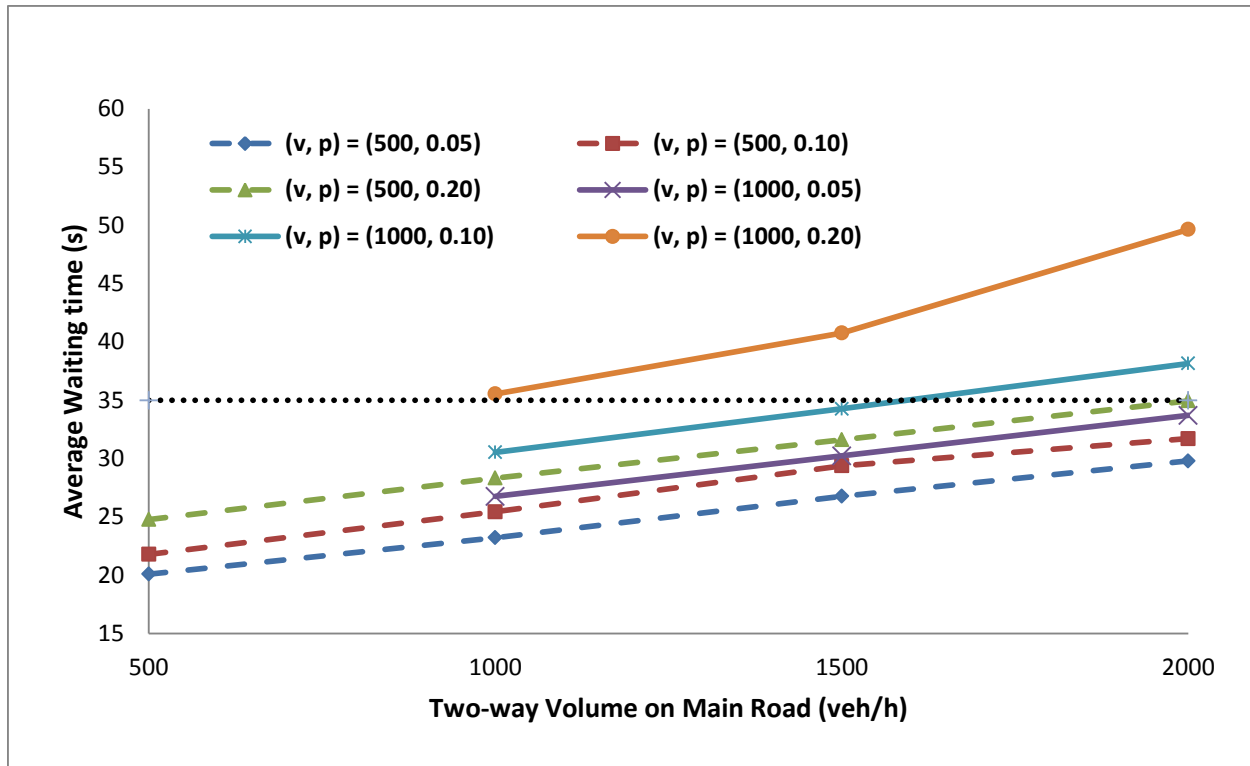
Model	Maximum Green (s)		Approach Speed (mph)		Two-Way Volume		Turn	Waiting Time (s)		Number of Conflicts	
	Main St	Cross St	Main St	Cross St	Main St	Cross St	Ratio	Average	Std. dev.	Average	Std. dev.
421	60	40	65	45	500	500	0.05	21.3	0.94	45.7	7.21
422	60	40	65	45	1000	500	0.05	25.1	1.64	71.9	8.20
423	60	40	65	45	1000	1000	0.05	28.9	0.89	139.8	11.95
424	60	40	65	45	1500	500	0.05	27.7	1.37	111.3	10.89
425	60	40	65	45	1500	1000	0.05	33.0	1.54	183.2	15.27
426	60	40	65	45	1500	1500	0.05	37.7	1.56		
427	60	40	65	45	2000	500	0.05	31.3	1.42	157.3	17.27
428	60	40	65	45	2000	1000	0.05	36.2	1.65	247.3	9.89
429	60	40	65	45	2000	1500	0.05	44.6	10.12		
430	60	40	65	45	2000	2000	0.05	176.7	70.75		
431	60	40	65	45	500	500	0.1	23.0	0.82	45.8	4.94
432	60	40	65	45	1000	500	0.1	27.2	1.49	68.8	5.98
433	60	40	65	45	1000	1000	0.1	32.3	1.35	146.0	15.53
434	60	40	65	45	1500	500	0.1	31.2	1.46	111.6	7.83
435	60	40	65	45	1500	1000	0.1	36.2	1.58	189.0	9.84
436	60	40	65	45	1500	1500	0.1	52.0	5.23		
437	60	40	65	45	2000	500	0.1	33.8	1.87	165.7	8.79
438	60	40	65	45	2000	1000	0.1	39.8	1.07	244.9	21.16
439	60	40	65	45	2000	1500	0.1	62.0	7.76		
440	60	40	65	45	2000	2000	0.1	739.3	152.94		
441	60	40	65	45	500	500	0.2	26.1	1.07	49.9	5.36
442	60	40	65	45	1000	500	0.2	30.8	1.45	77.1	9.92
443	60	40	65	45	1000	1000	0.2	36.7	1.51	143.3	9.31
444	60	40	65	45	1500	500	0.2	34.2	1.22	118.7	8.19
445	60	40	65	45	1500	1000	0.2	41.3	2.19	187.7	13.22
446	60	40	65	45	1500	1500	0.2	81.3	24.26		
447	60	40	65	45	2000	500	0.2	39.6	1.67	165.0	7.54
448	60	40	65	45	2000	1000	0.2	47.4	2.98	258.3	20.71
449	60	40	65	45	2000	1500	0.2	123.4	59.63		
450	60	40	65	45	2000	2000	0.2	511.3	216.49		

**Table A.16** Results of Sensitivity Analysis: G = (60, 40) s, S = (65, 55) mph

Model	Maximum Green (s)		Approach Speed (mph)		Two-Way Volume		Turn	Waiting Time (s)		Number of Conflicts	
	Main St	Cross St	Main St	Cross St	Main St	Cross St	Ratio	Average	Std. dev.	Average	Std. dev.
451	60	40	65	55	500	500	0.05	21.3	0.76	43.7	6.65
452	60	40	65	55	1000	500	0.05	24.8	1.29	68.5	13.63
453	60	40	65	55	1000	1000	0.05	28.0	0.52	134.3	10.07
454	60	40	65	55	1500	500	0.05	28.0	1.27	99.7	12.35
455	60	40	65	55	1500	1000	0.05	31.7	1.64	173.4	12.77
456	60	40	65	55	1500	1500	0.05	37.3	2.18		
457	60	40	65	55	2000	500	0.05	31.1	1.97	161.3	11.26
458	60	40	65	55	2000	1000	0.05	35.1	1.29	244.6	16.95
459	60	40	65	55	2000	1500	0.05	41.4	3.58		
460	60	40	65	55	2000	2000	0.05	118.1	32.65		
461	60	40	65	55	500	500	0.1	23.6	1.49	46.4	5.66
462	60	40	65	55	1000	500	0.1	26.3	1.16	67.6	6.82
463	60	40	65	55	1000	1000	0.1	31.1	1.12	138.1	11.59
464	60	40	65	55	1500	500	0.1	30.6	1.38	106.0	11.67
465	60	40	65	55	1500	1000	0.1	35.7	1.56	178.3	14.40
466	60	40	65	55	1500	1500	0.1	47.4	4.89		
467	60	40	65	55	2000	500	0.1	33.8	1.36	157.8	12.18
468	60	40	65	55	2000	1000	0.1	39.4	1.42	247.4	15.81
469	60	40	65	55	2000	1500	0.1	65.6	18.02		
470	60	40	65	55	2000	2000	0.1	554.1	127.07		
471	60	40	65	55	500	500	0.2	25.4	0.79	47.7	7.18
472	60	40	65	55	1000	500	0.2	29.6	1.58	75.2	9.03
473	60	40	65	55	1000	1000	0.2	35.3	1.59	136.9	14.04
474	60	40	65	55	1500	500	0.2	33.5	1.16	116.6	9.72
475	60	40	65	55	1500	1000	0.2	40.6	2.75	187.3	7.26
476	60	40	65	55	1500	1500	0.2	82.5	19.76		
477	60	40	65	55	2000	500	0.2	38.5	2.06	165.2	12.94
478	60	40	65	55	2000	1000	0.2	46.5	2.90	252.3	6.80
479	60	40	65	55	2000	1500	0.2	112.5	26.94		
480	60	40	65	55	2000	2000	0.2	356.6	76.55		

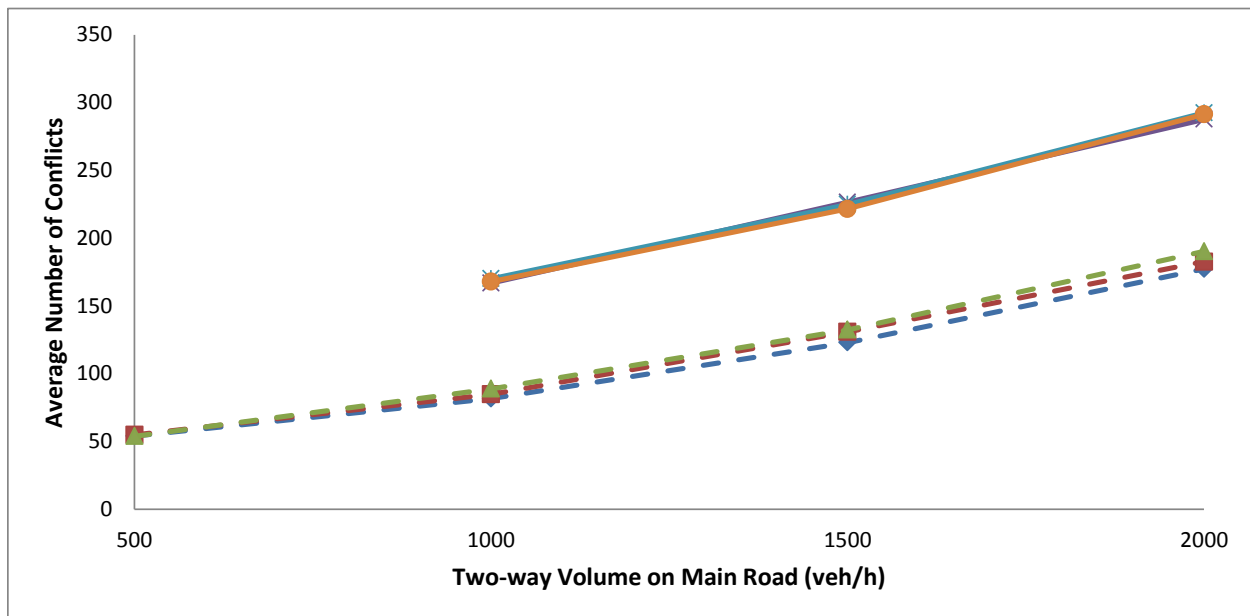
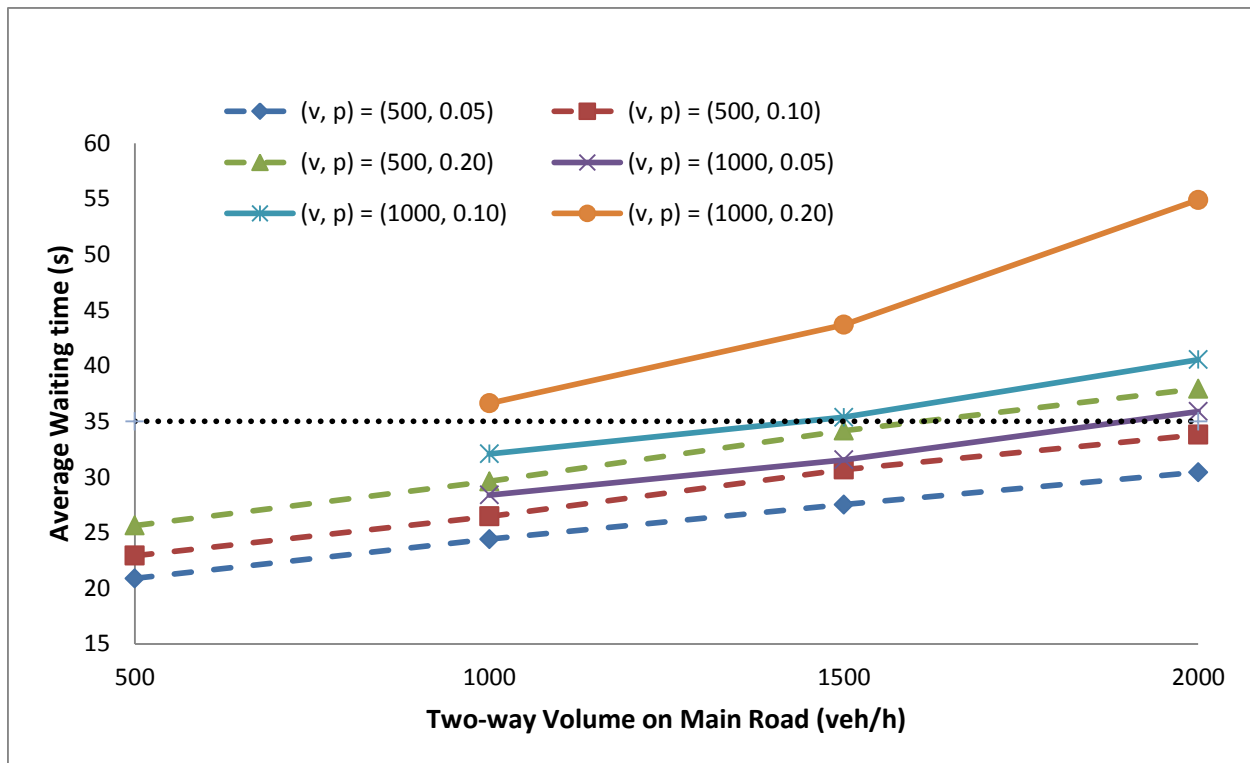
## Appendix B Effects of Traffic Volume and Turn Percentage





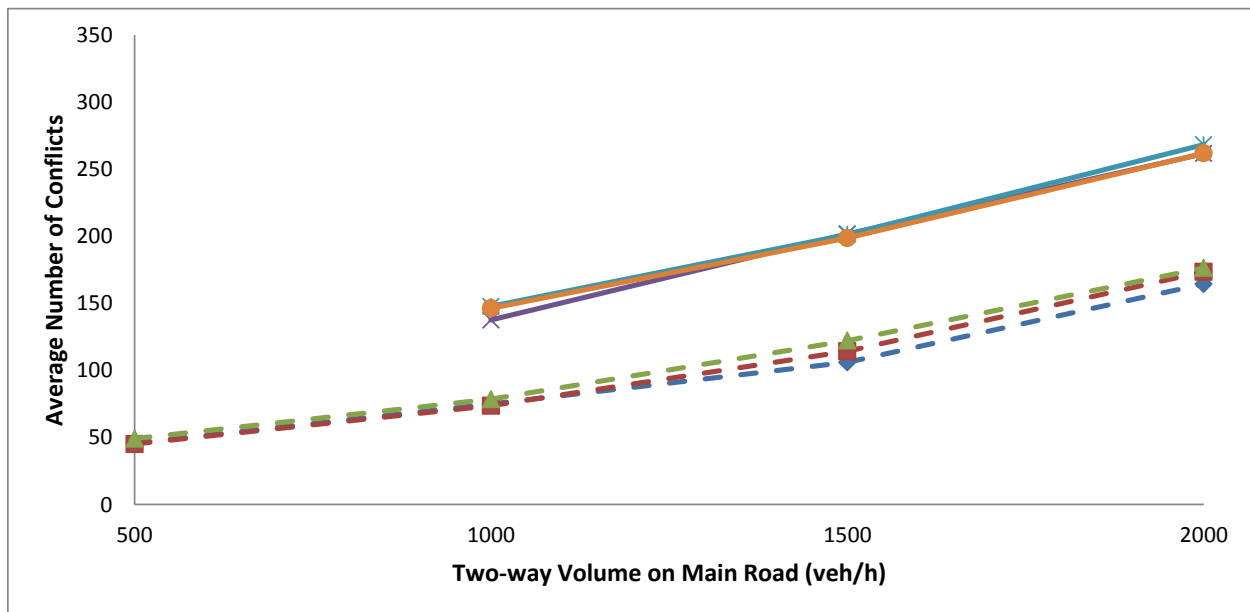
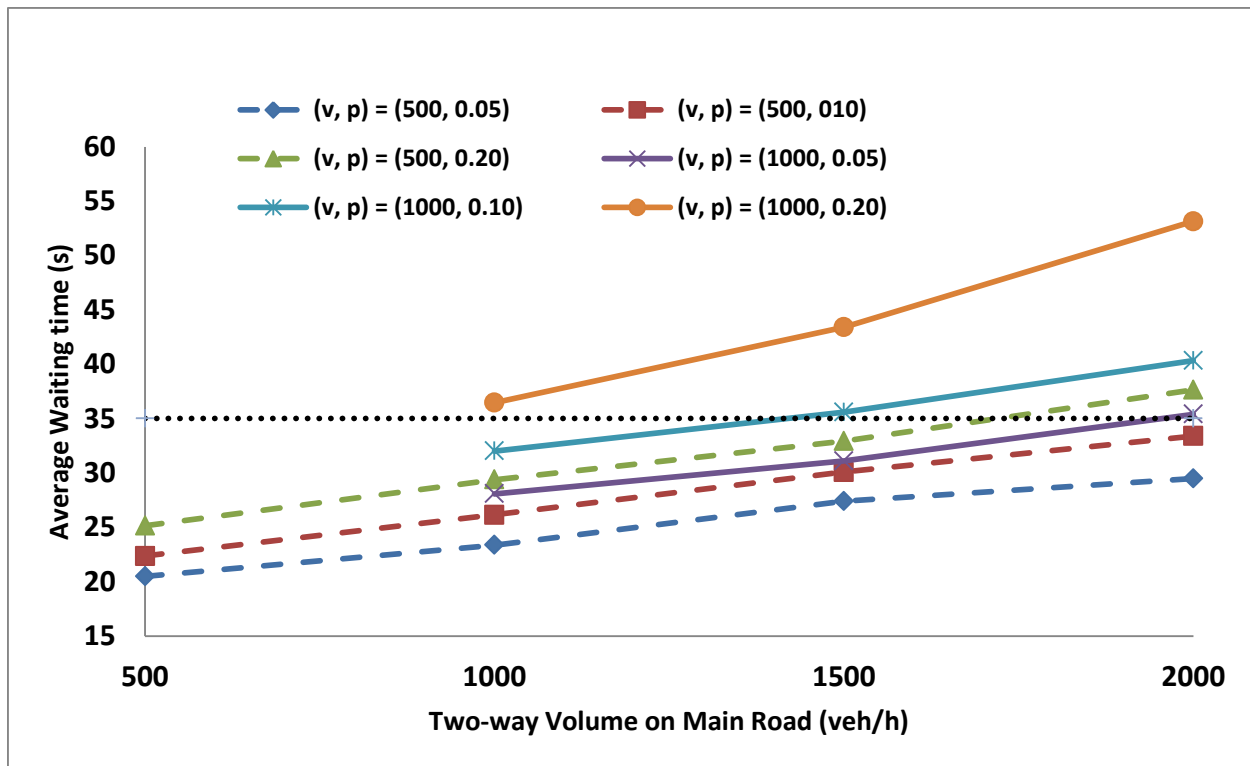
\*(v, p) = (Cross road volume, Turn percentage)

**Figure B.1** Effects of Traffic Volume and Turn Percentage:  $G = (50, 30)$  s,  $S = (45, 35)$  mph



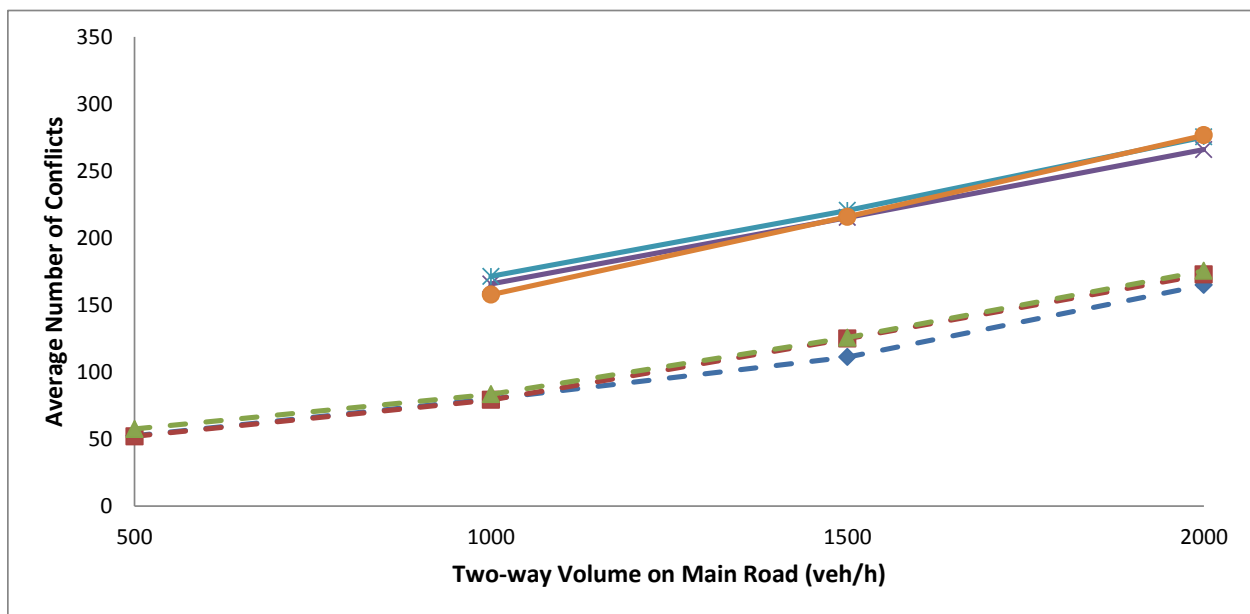
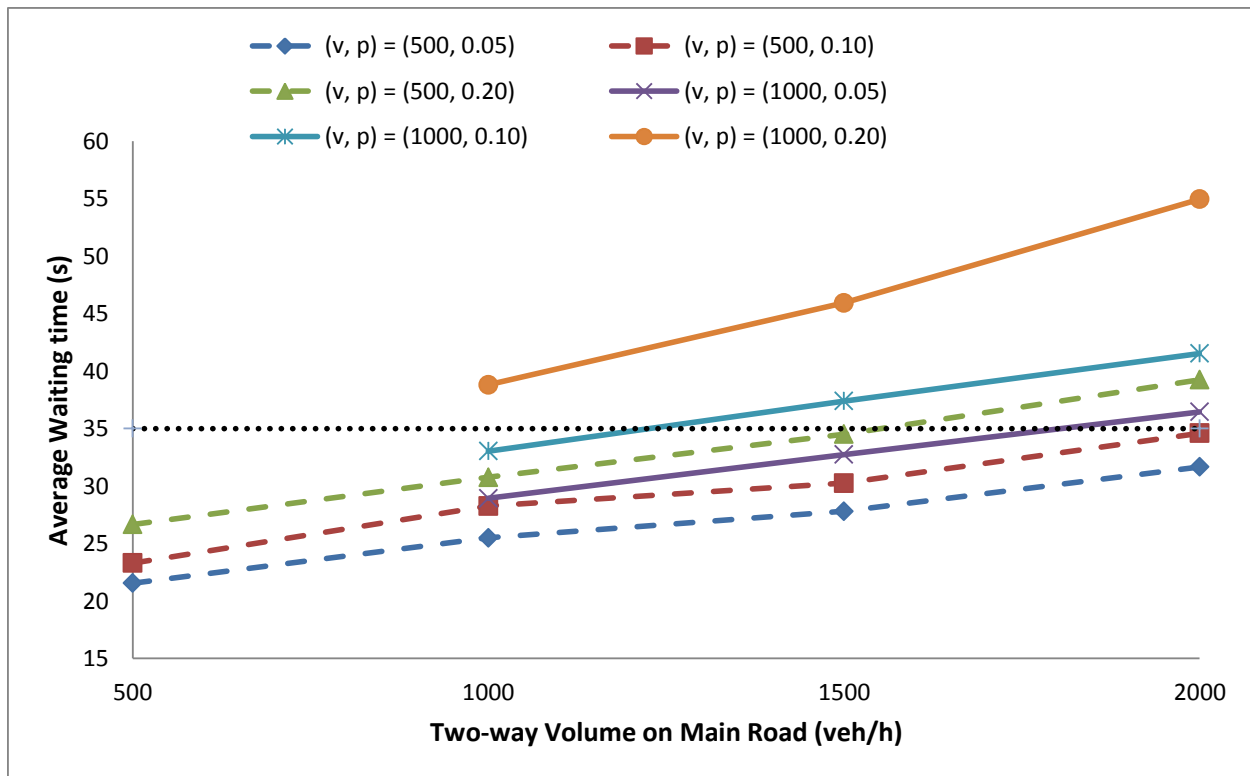
\*(v, p) = (Cross road volume, Turn percentage)

**Figure B.2** Effects of Traffic Volume and Turn Percentage:  $G = (50, 30)$  s,  $S = (55, 35)$  mph



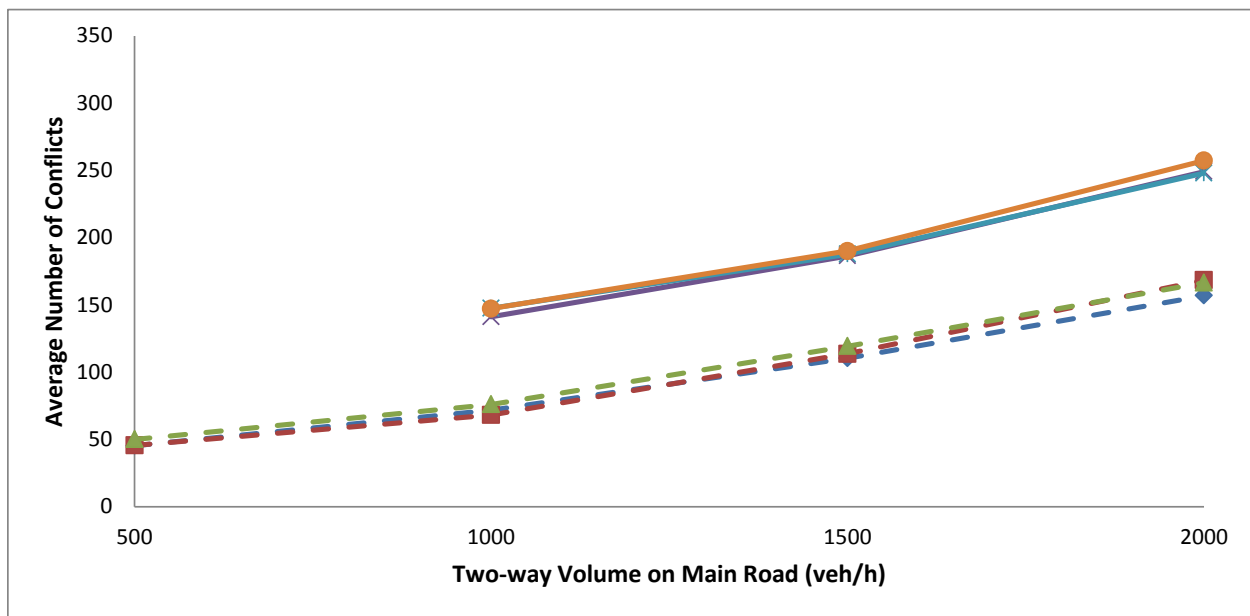
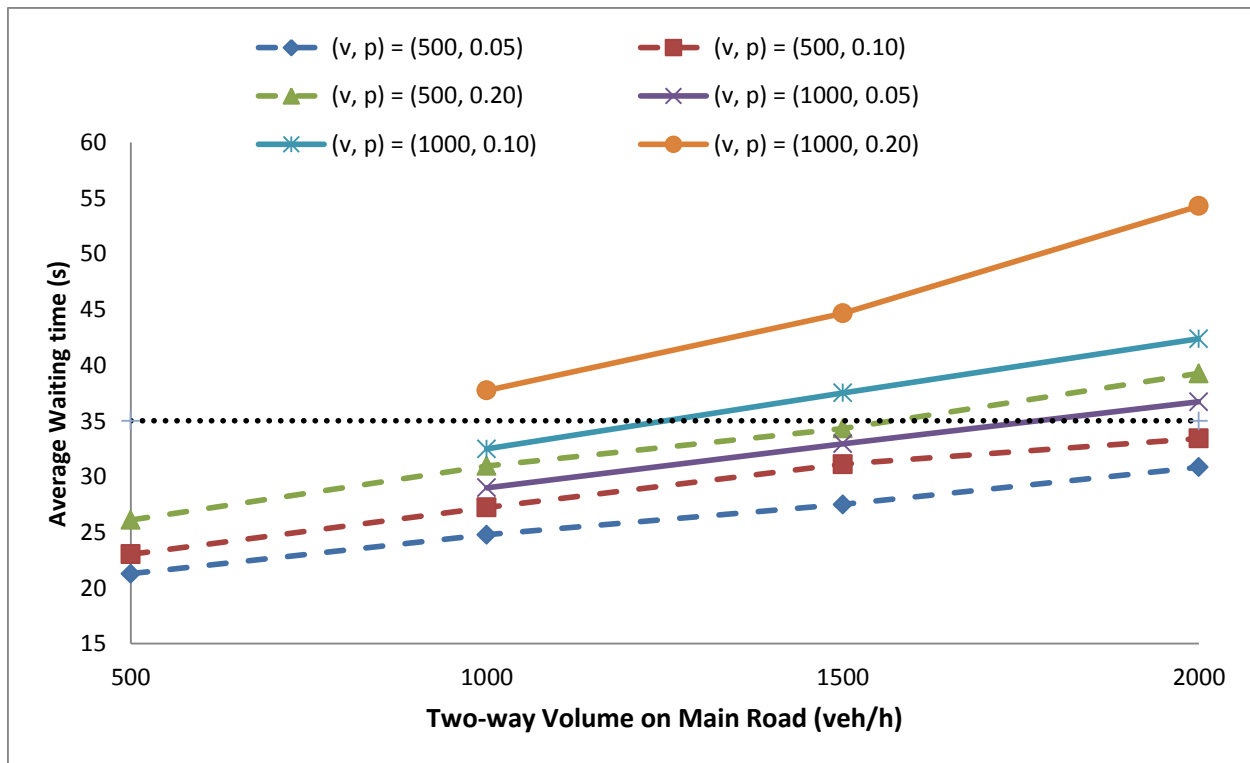
\*(v, p) = (Cross road volume, Turn percentage)

**Figure B.3** Effects of Traffic Volume and Turn Percentage:  $G = (50, 30)$  s,  $S = (55, 45)$  mph



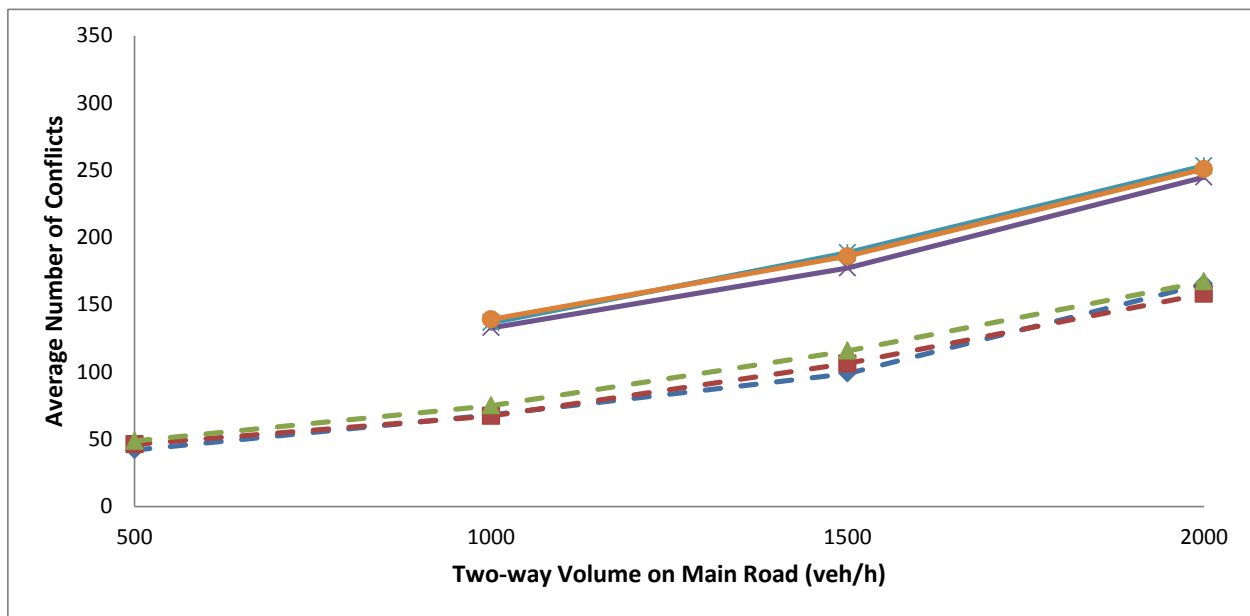
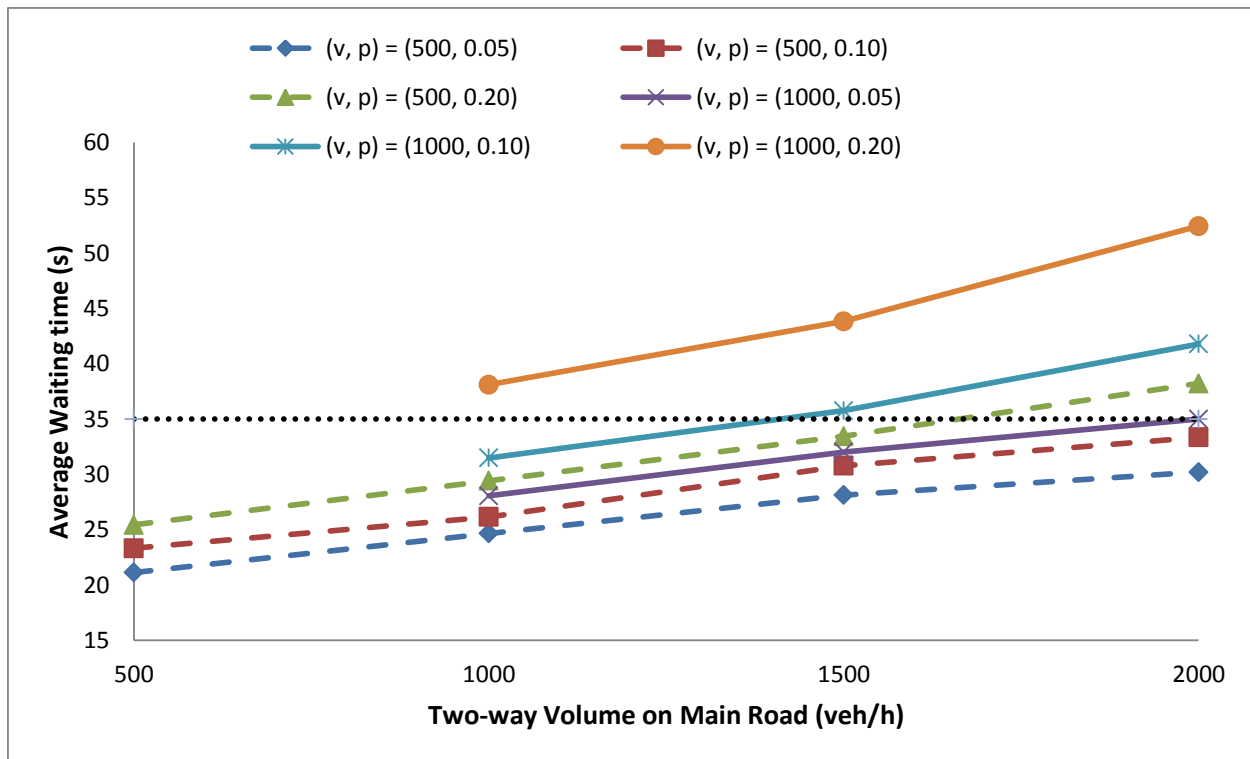
\*(v, p) = (Cross road volume, Turn percentage)

**Figure B.4** Effects of Traffic Volume and Turn Percentage:  $G = (50, 30)$  s,  $S = (65, 35)$  mph



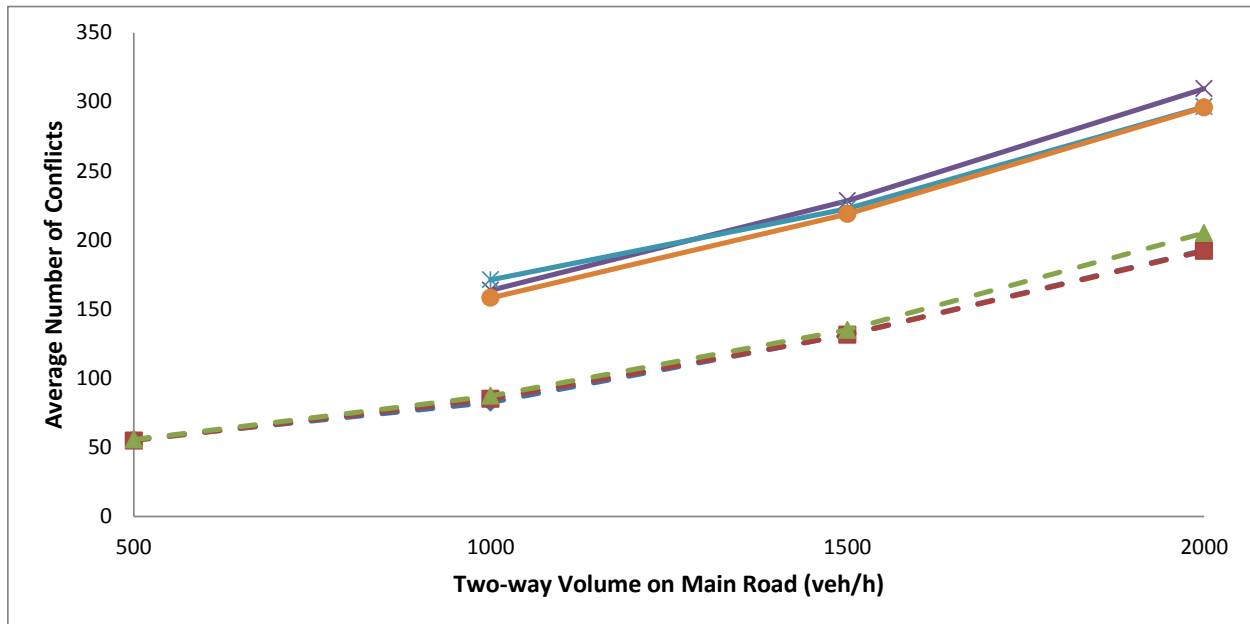
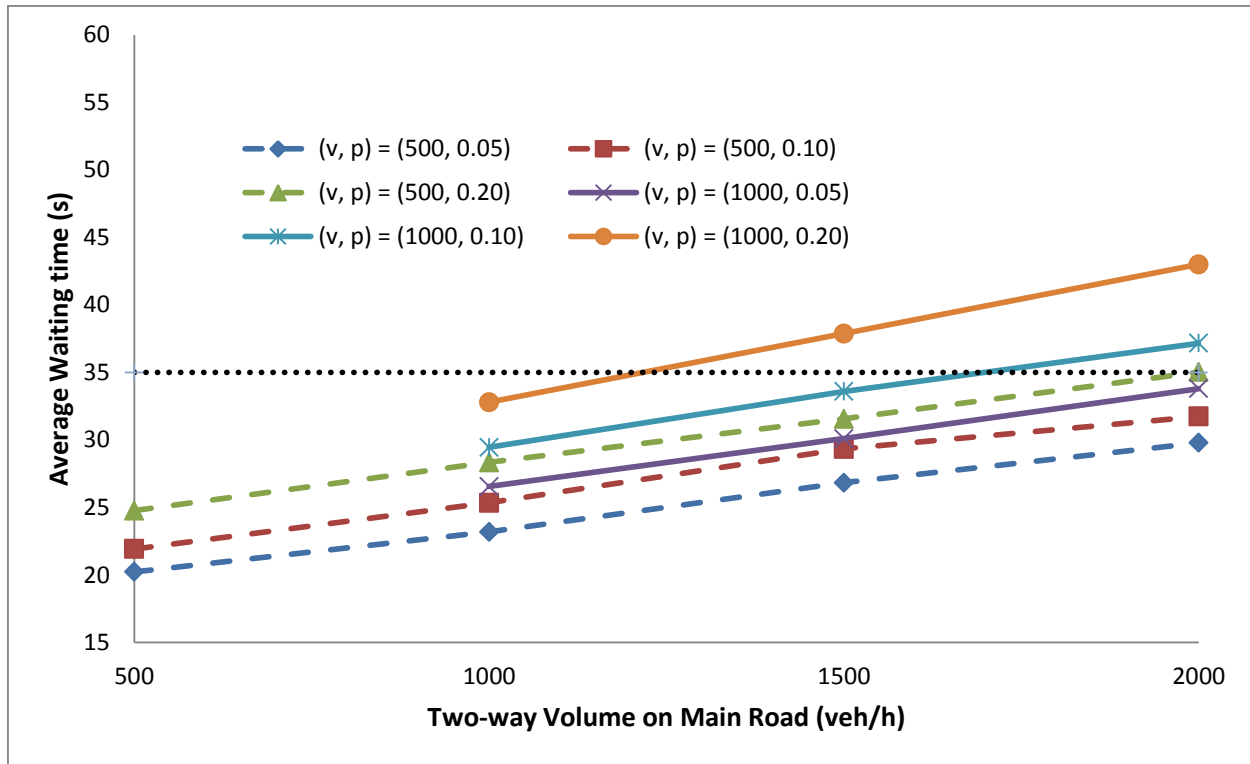
\*(v, p) = (Cross road volume, Turn percentage)

**Figure B.5** Effects of Traffic Volume and Turn Percentage:  $G = (50, 30)$  s,  $S = (65, 45)$  mph



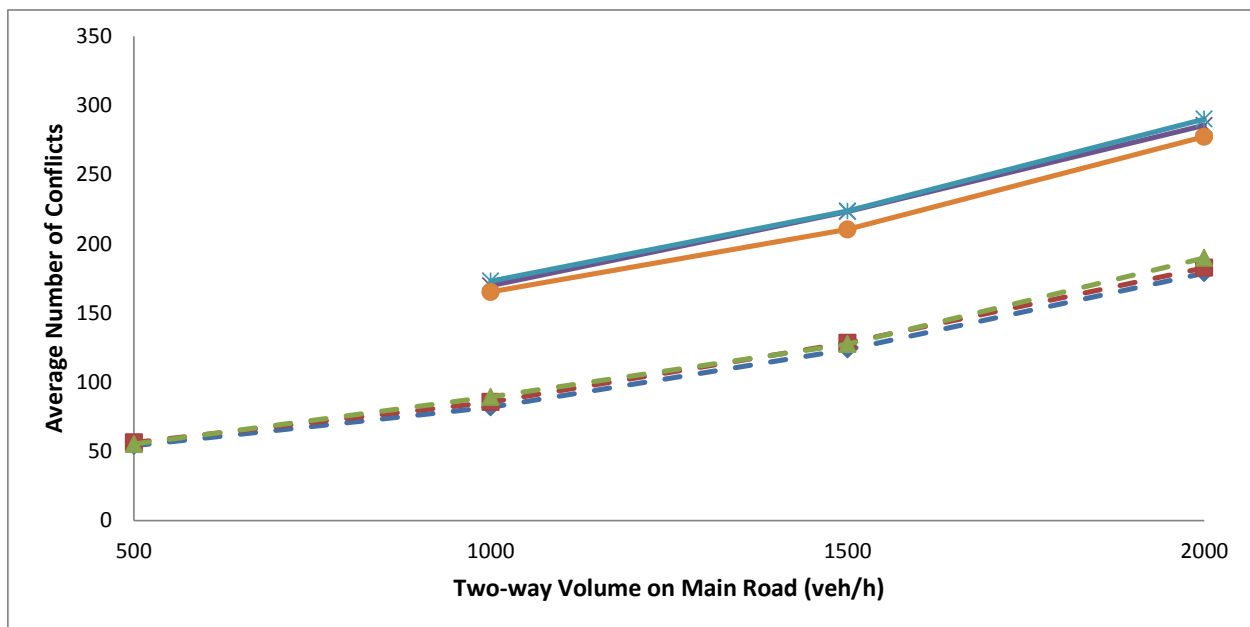
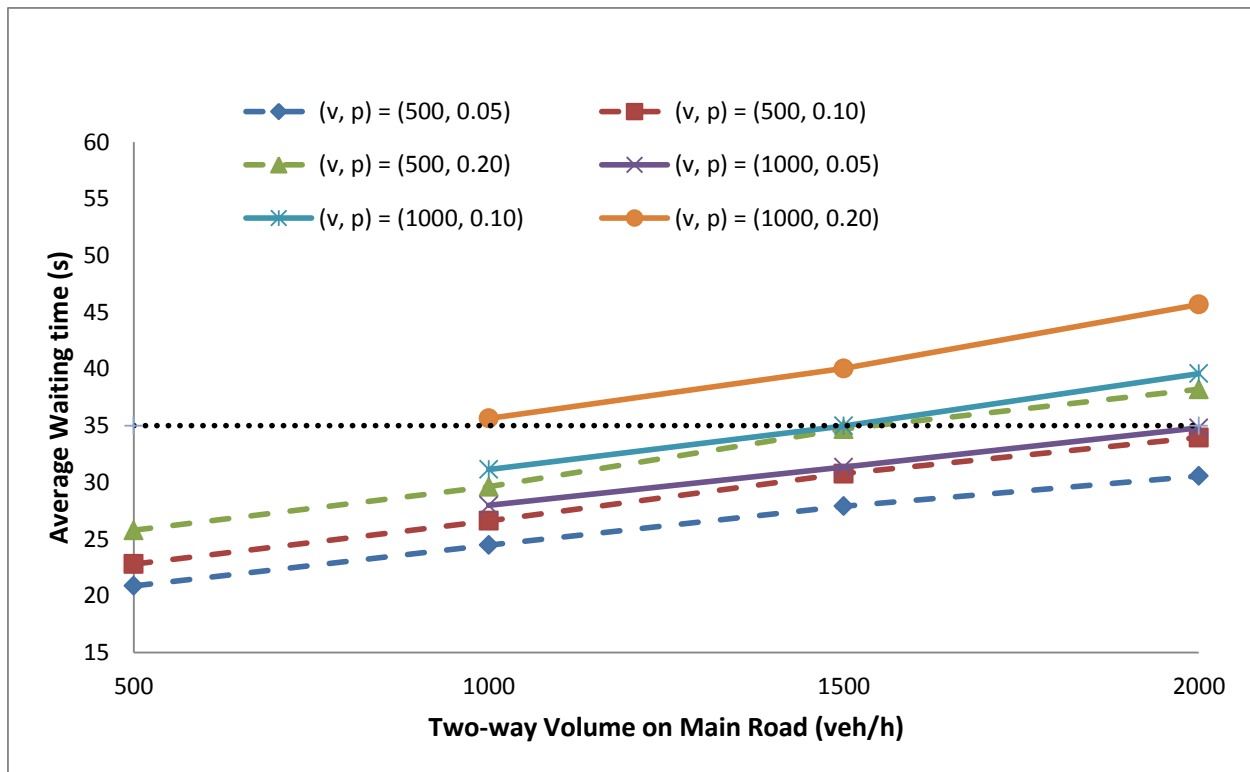
\*(v, p) = (Cross road volume, Turn percentage)

**Figure B.6** Effects of Traffic Volume and Turn Percentage:  $G = (50, 30)$  s,  $S = (65, 55)$  mph



\*(v, p) = (Cross road volume, Turn percentage)

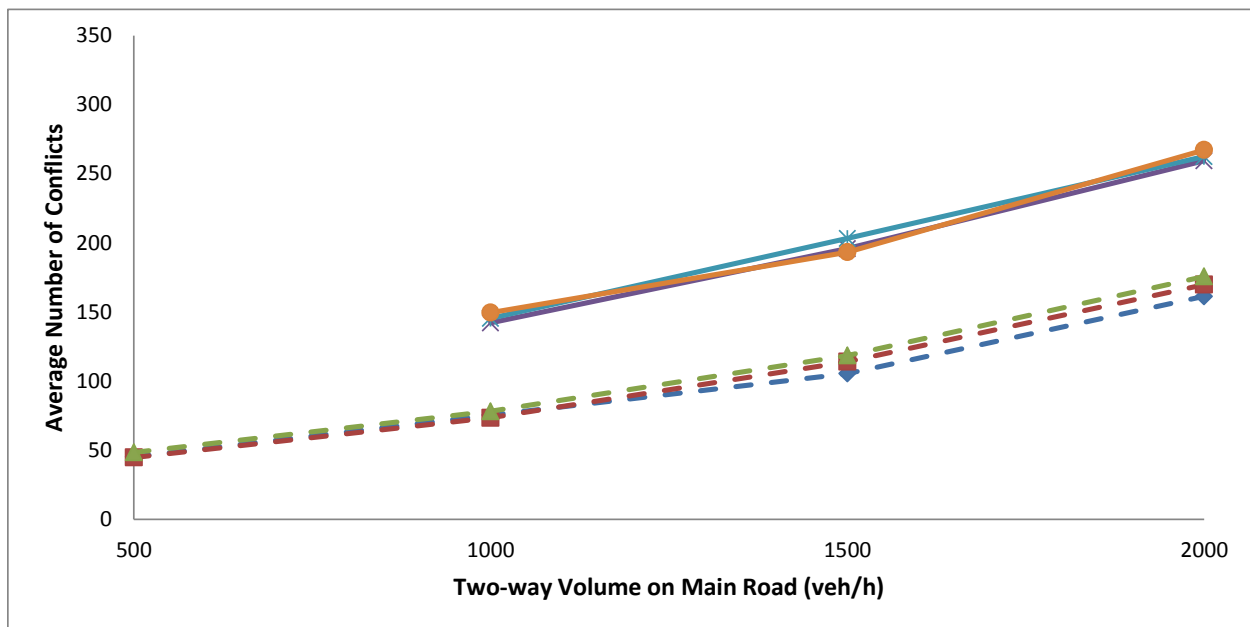
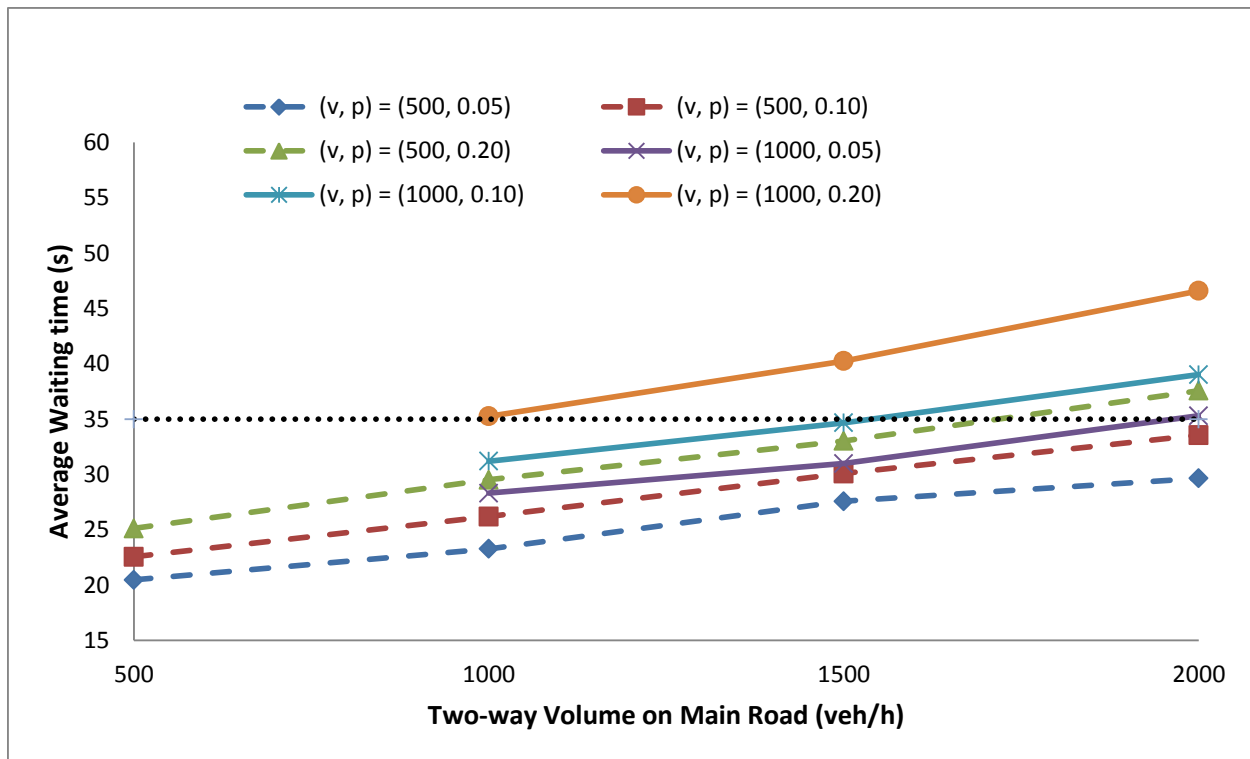
**Figure B.7** Effects of Traffic Volume and Turn Percentage:  $G = (60, 40)$  s,  $S = (45, 35)$  mph



\*(v, p) = (Cross road volume, Turn percentage)

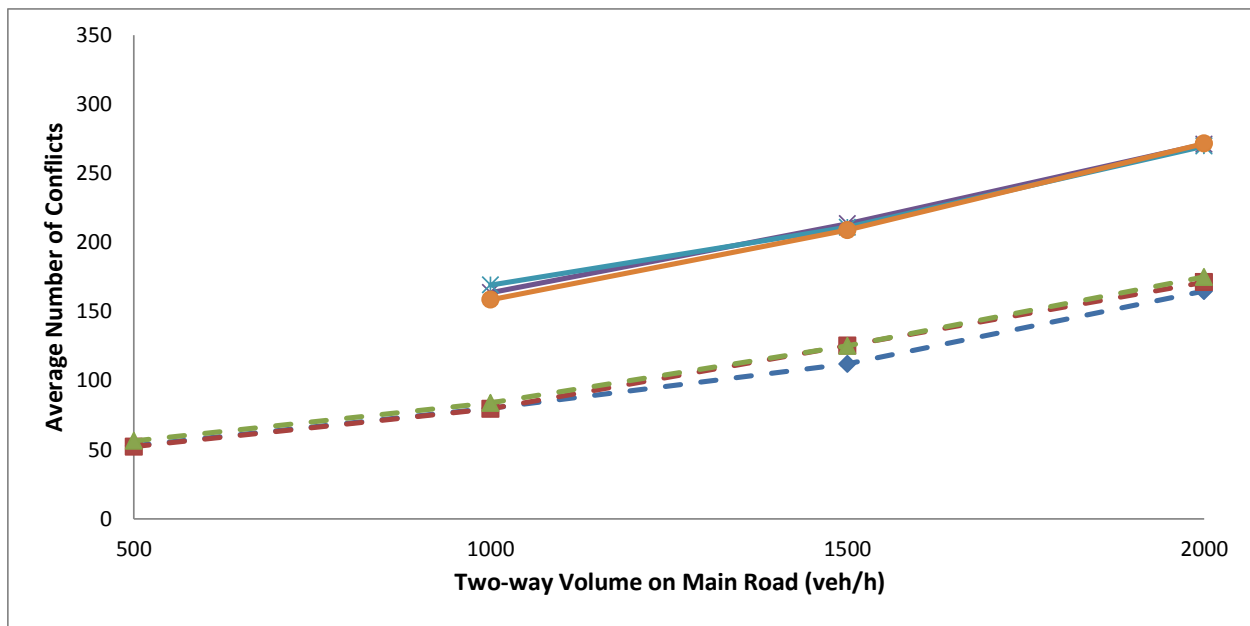
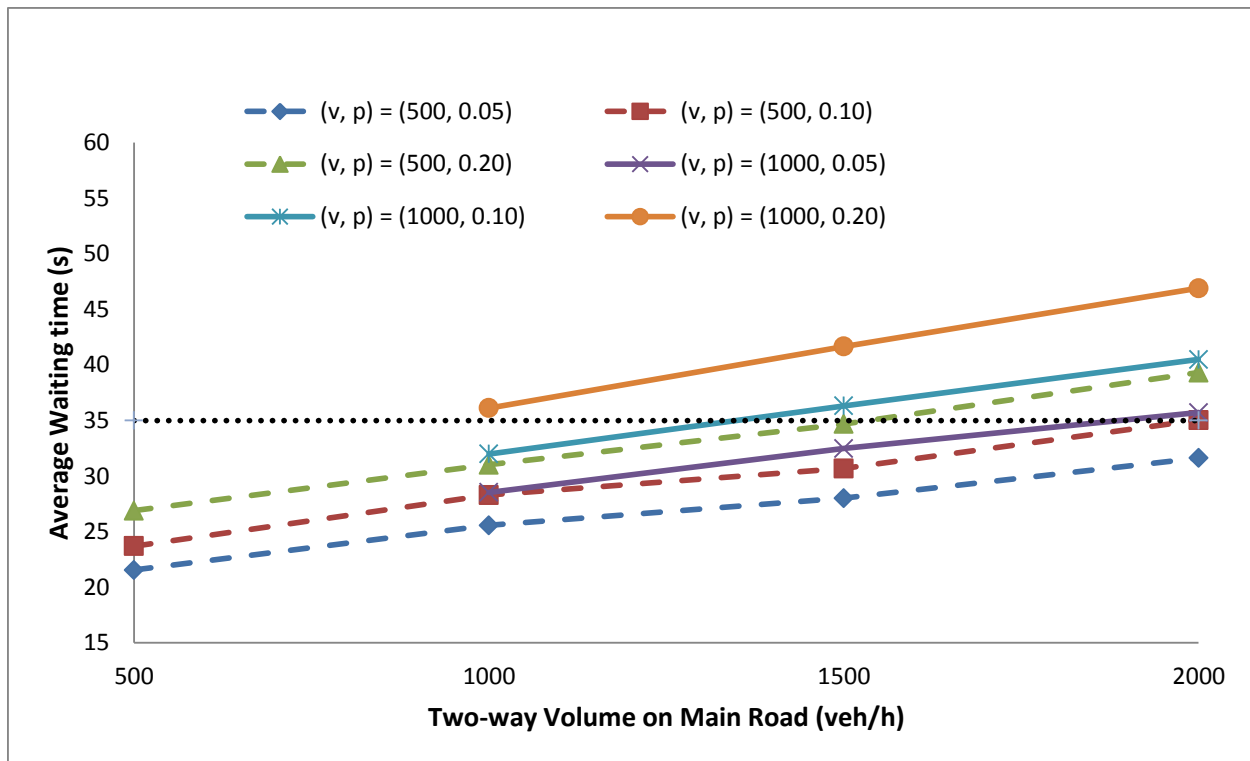
**Figure B.8** Effects of Traffic Volume and Turn Percentage:  $G = (60, 40)$  s,  $S = (55, 35)$  mph





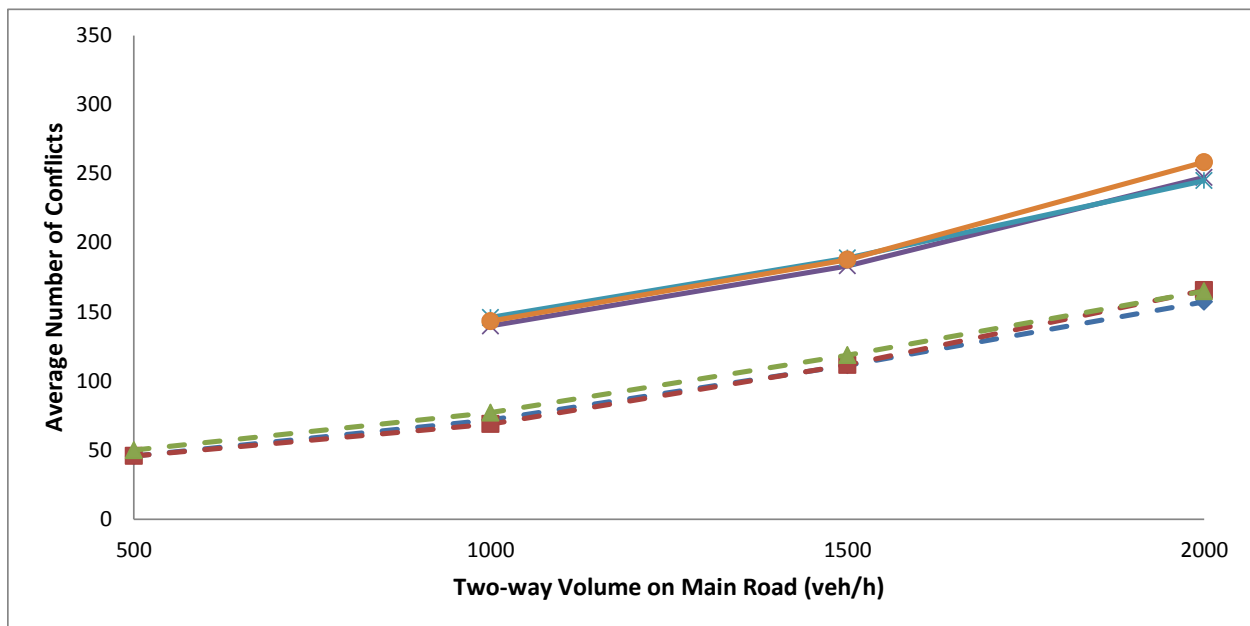
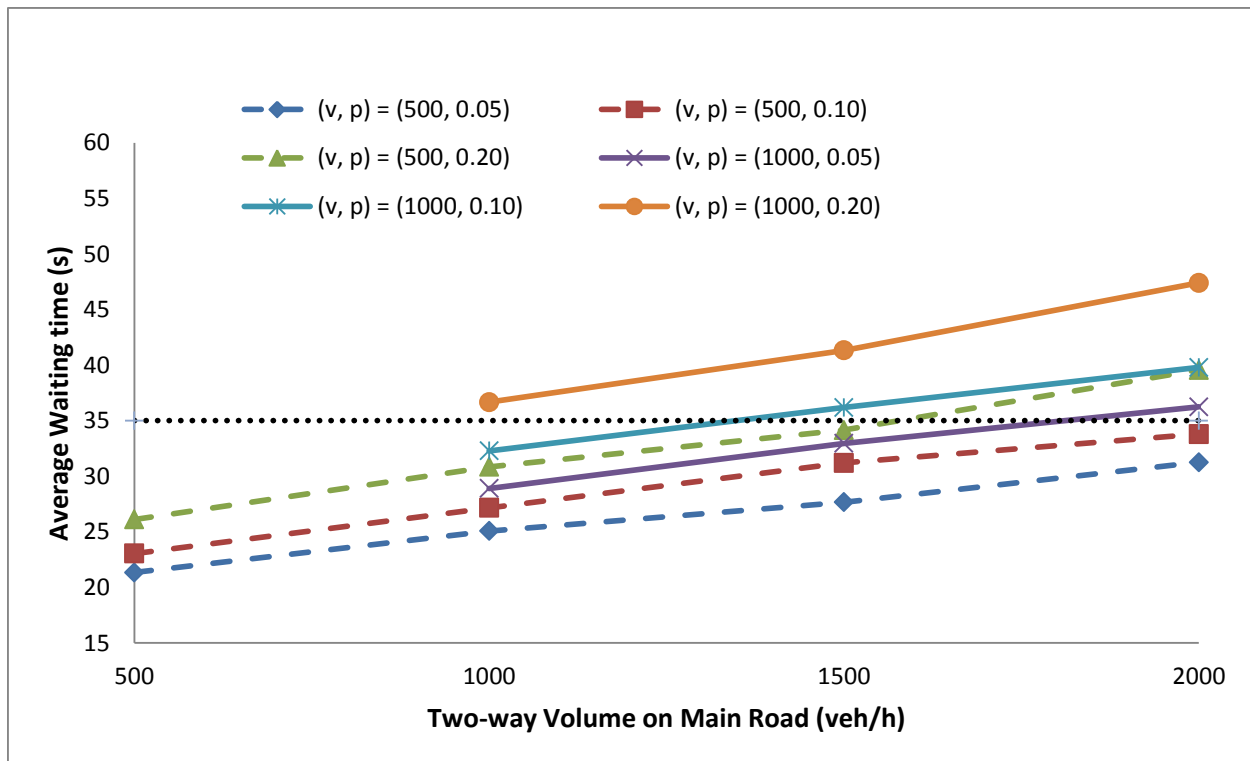
\*(v, p) = (Cross road volume, Turn percentage)

**Figure B.9** Effects of Traffic Volume and Turn Percentage:  $G = (60, 40)$  s,  $S = (55, 45)$  mph



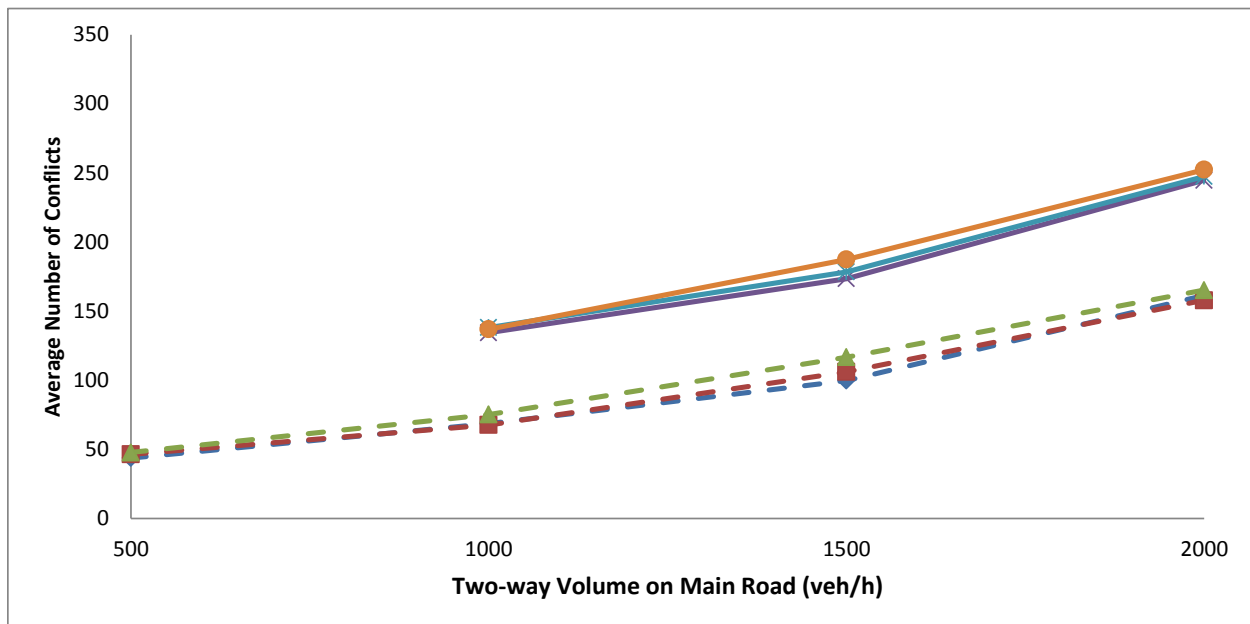
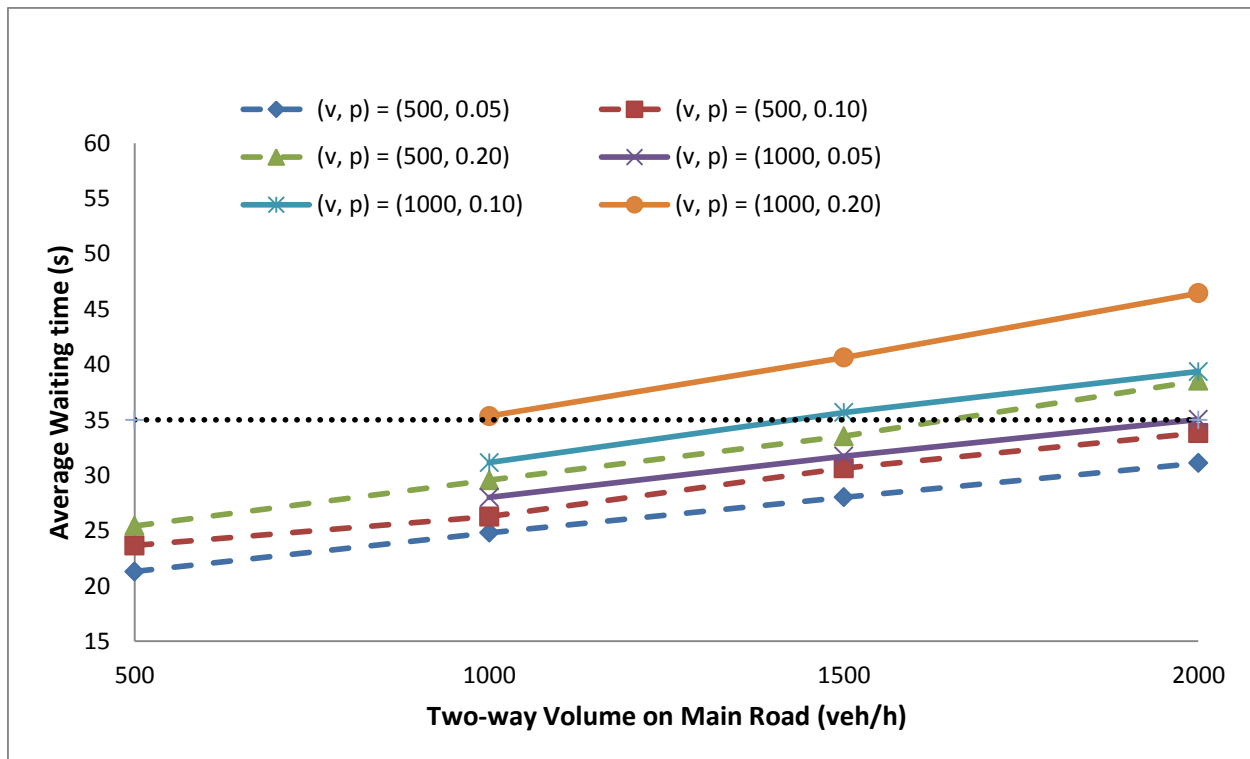
\*(v, p) = (Cross road volume, Turn percentage)

**Figure B.10** Effects of Traffic Volume and Turn Percentage:  $G = (60, 40)$  s,  $S = (65, 35)$  mph



\*(v, p) = (Cross road volume, Turn percentage)

**Figure B.11** Effects of Traffic Volume and Turn Percentage:  $G = (60, 40)$  s,  $S = (65, 45)$  mph



\*(v, p) = (Cross road volume, Turn percentage)

**Figure B.12** Effects of Traffic Volume and Turn Percentage:  $G = (60, 40)$  s,  $S = (65, 55)$  mph