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**METHODS OF SAFETY IMPROVEMENT AT
COORDINATED SIGNALS**

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Methods of Safety Improvements at Coordinated Signals

Introduction

The Indiana Department of Transportation (INDOT) widely employs arterial signal coordination as a means to improve the travel conditions on urban arterial streets and in street grids. Coordinating the signal timings of adjacent traffic signals has proven to be a low-cost and highly efficient means of improving traffic performance. By judiciously selecting the signal coordination parameters, traffic systems engineers are able to reduce the delays to the travelers and minimize the number of unwanted vehicle stops.

Traffic safety received much less attention in the design stage of signal coordination. Traffic at signalized intersections is not as safe as might be expected. In Indiana, 16.4% of year 2000's 220,883 crashes were related to signalized intersections. The most frequent types of crashes are rear-end collisions on approaches to signalized intersections and right-angle collisions after one of the colliding vehicles enters the intersection during a red signal.

In this research, the safety implication of arterial signal coordination was investigated extensively in order to find a method to incorporate safety considerations into arterial signal coordination. Several studies were completed for this main objective. First, crash patterns at the coordinated signals were analyzed and documented. The crashes which were possibly related to signal timings were scrutinized. Second, statistical models were developed to identify traffic characteristics that are affected by signal timing and at the same time have important safety consequences. Third, rear-end and right-angle crash prediction models were developed to facilitate the evaluation of safety performance of a signal timing plan at the design stage. Finally, a prototype safety evaluation software tool was developed based on the models.

Findings

First, it was found through the crash pattern study that most rear-end crashes occur when signals are red or turning green from red. The dilemma-zone related rear-end crashes account for only a small proportion of all rear-end crashes. In addition, most right-angle crashes are caused by red-light-runners from both the arterial street and the crossing street.

Second, arterial signal coordination has significant safety consequences. First, rear-end and right-angle crashes are much less likely when a high density vehicle platoon was scheduled to arrive in the second half of green. Right-angle crash severity level is also lowered for such traffic patterns. Second, the vehicles arriving at the beginning of the green are most susceptible to

crashes. The vehicles arriving at the beginning of the red are also very susceptible to crashes when high density of traffic is concentrated in first half of the red signal. Third, short cycle length is found to be associated with lower rear-end crash risks. Fourth, shorter yellow signal was associated with higher percentage of injury fatal right-angle crashes.

Finally, some other factors that are not controlled by the arterial signal timing were also found to be significant crash factors. Specifically, the existence of an exclusive right-turn lane, a lower speed limit, and a shorter distance from upstream intersections were found to reduce the risks of crashes

Implementation

The presented research developed guidelines for using alternative intersection designs. The guidelines compile the existing knowledge found in existing publications and research reports with the simulation experiments performed with VISSIM. The guidelines are ready

to use and will help planners and designers determine which intersection types are the most promising under considered conditions and should be considered in a detailed way. The simulation results have been summarized in an easy to use format of graphs.

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16. Abstract <p>In this research, the safety impact of arterial signal coordination is investigated. Based on the findings, procedures are proposed to incorporate safety considerations into signal coordination design. In addition, a software tool is developed to facilitate the use of the research findings. Signal coordination has been extensively employed as a tool to optimize an arterial system's mobility performance, commonly measured by the total number of vehicle stops and the time of delays. Although vehicle crashes at signalized intersections have long been recognized as a major safety threat, studies to date have been limited locally to non-coordinated signals or the non-coordinated features of signal timings. Disaggregate statistical models are developed to identify the influential crash likelihood factors of rear-end and right-angle crashes, the two most frequent types of intersection crashes. To capture the severity factors, the outcomes are further classified into property-damage-only (PDO) and injury-fatal (IF) crashes. The crash likelihoods for each 15-minute interval are used as the response variables of the models. For each 15-minute interval, predictive variables are collected or derived from available data, including the volume, signal timings, and traffic patterns. A traffic pattern characterization is designed to represent each 15-minute interval's arrival pattern. Various discrete outcome econometric models are used and compared, including multinomial logit model (MNL), multinomial probit model (MNP), nested logit model (NL), and sequential logit model (SL). A SL framework is proposed as the main model. At the first stage, a MNL models crash likelihoods; at the second stage, a logit model models severity. The key findings are as follows: First, signal coordination significantly affects crash likelihood. Certain traffic arrival patterns are associated with significantly lower crash likelihoods. Second, the dilemma zone contributes in a limited way to the crash likelihoods. Third, adjusted volumes, based on the presented models, were found to be better predictors of crash likelihoods, which indicates that only a portion of all vehicles are highly susceptible to crashes. Fourth, shorter distances from upstream intersection and shorter cycle lengths are associated with lower risk levels of crashes.</p>			
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ABSTRACT

In this research, the safety impact of arterial signal coordination is investigated. Based on the findings, procedures are proposed to incorporate safety considerations into signal coordination design. In addition, a software tool is developed to facilitate the use of the findings.

Signal coordination is a widely used means of improving arterial traffic mobility, commonly measured with the number of vehicle stops and vehicle delays. Although vehicle crashes at signalized intersections have long been recognized as a major safety issue, studies to date have been focused on non-coordinated signals or the non-coordinated features of signal timings.

This research has developed disaggregate statistical models of crash frequency and severity based on observations that represent 15-minute intervals. Predictive variables were collected or derived from available data, including the volume, signal timings, and traffic patterns. Crash likelihood models have been developed to identify the frequency factors of the two most prevalent types of crashes: rear-end and right-angle crashes. To capture the severity factors, the likelihood of injury-fatal (IF) outcome of a crash has been estimated using alternative statistical models. The frequency and severity models are various discrete outcome econometric models, including multinomial logit model (MNL), multinomial probit model (MNP), nested logit model (NL), and sequential logit model (SL). These models have been evaluated and compared and the two-stage SL framework is proposed as the most appropriate one. In the first stage, a MNL models crash likelihoods; in the second stage, a logit model models severity.

The key findings are as follows. First, signal coordination significantly affects crash likelihood. Certain traffic arrival patterns, including concentration of vehicle arrivals in

the second half of a green signal, are associated with significantly lower crash likelihoods. Second, the dilemma zone contributes in a limited way to the crash frequency. Third, certain traffic flow components were found to be better predictors of crash likelihoods than the total traffic volume, which indicates that only a portion of all vehicles are highly susceptible to crashes. Fourth, short distances between intersections and short cycle lengths are associated with a low risk of a crash. Finally, the presence of a right turn bay is associated with a considerable improvement of safety manifested with a lower risk of rear-end and right-angle collisions.

CHAPTER 1. INTRODUCTION

1.1. Motivation

Traffic signal control plays an indispensable role in improving travel conditions on urban arterial streets and in street grids. Traffic signals provide a means of intervening in traffic operations with a level of effectiveness unmatched by other traffic control methods (Gazis 2002; Papacostas and Prevedouros 2000). The implemented signal control strategies in a given area strongly influence urban travelers' mobility and safety.

Coordinating the signal timings of adjacent traffic signals has proven to be a low-cost and highly efficient means of improving traffic performance. By judiciously selecting the signal coordination parameters, including the background cycle, signal offsets, and signal splits, traffic systems engineers are able to reduce both the vehicle delays and the number of vehicle stops. Signal coordination design practices have benefited greatly from dramatically advancing computing technology. Currently, the vast majority of traffic systems engineers routinely use signal optimization software, such as Synchro (Husch and Albeck 2004) and TRANSYT (McTrans Center 1998), for setting arterial signal timings. These software tools provide a means of finding good coordination plans, particularly if the software features are properly and effectively used by the users (Li and Tarko 2006). Usually, coordination plan optimizers aim to minimize the delays, number of stops, and excessive long queues. In other words, such optimization algorithms focus on improving the mobility performance of road networks.

The dilemma zone at signalized intersections is a frequently studied issue induced by poorly designed signal timing (Gazis et al. 1960; Liu et al. 1996). Extensive studies have documented this issue and treatment methods have been proposed through sufficiently long change intervals and advanced controller-based signal timing adjustment

techniques (Zimmerman and Bonneson 2004; Tarko et al. 2006). Unlike the dilemma zone, the safety impact of signal coordination settings is not given considerable attention in the optimization stage of signal coordination design.

Although the conflicting vehicle movements are separated in time by design, traffic at signalized intersections is not as safe as might be expected. For example, in year 2003, among 6.3 million crashes reported in the United States, about 20% occurred at signalized intersections. As a result, nearly 500,000 people were injured and 3,000 fatalities occurred at intersections controlled by traffic signals (National Highway Traffic Safety Administration 2005). In Indiana, 16.4% of year 2000's 220,883 crashes were related to signalized intersections (Indiana Criminal Justice Institute 2005). Table 1-1 shows that around 20% of all crashes of Indiana occurred at traffic control signals. The most frequent types of crashes are rear-end collisions on approaches to signalized intersections and right-angle collisions after one of the colliding vehicles enters the intersection during a red signal.

Table 1-1 Crashes breakdown by traffic control type¹

Traffic Control	2005	2006	2007
01 – Officer/Crossing Guard/Flagman	0.1%	0.1%	0.1%
02 – RR Crossing Gate/Flagman	0.1%	0.1%	0.1%
03 – RR Crossing Flashing Signal	0.1%	0.1%	0.1%
04 – RR Crossing Sign	0.1%	0.1%	0.1%
05 – Traffic Control Signal	20.3%	22.6%	19.2%
06 – Flashing Signal	0.9%	0.8%	0.8%
07 – Stop Sign	11.2%	10.9%	11.3%
08 – Yield Sign	0.9%	0.9%	0.8%
09 – Lane Control	25.9%	23.8%	23.5%
10 – No Passing Zone	3.2%	2.9%	2.8%
11 – Other Regulatory Sign/Marking	0.8%	0.8%	0.8%
12 – None	36.4%	37.0%	40.6%

¹ Calculated from Indiana Vehicle Crash Database.

This naturally leads to an important question: Does signal coordination affect traffic safety at coordinated intersections? The effects of signal coordination on safety have not been investigated extensively. Traffic engineers hold diverse views on this issue. Some engineers think that better coordination also improves traffic safety; others argue that higher speeds resulting from better coordination may deteriorate traffic safety. Anecdotal case studies have documented the safety benefits of coordinated arterial systems after signal optimization (Parsonson 1983; Parsonson and Thomas 1978; Rakha et al. 2000). The focus of these studies, however, is not on safety and the results are not conclusive. Furthermore, these results can hardly be used in the signal optimization stage. The important factors that are associated with higher risk of crashes have not been explored in depth.

We make a hypothesis that the signal coordination settings (cycle, splits, and offsets) impact safety considerably. Signal timings affect the formation of vehicle platoons and schedule the arrivals of such platoons. Certain vehicle arrival patterns may be associated with a higher risk level. For example, it is estimated that approximately 30 percent of fatal crashes and 20 percent of injury crashes at signalized intersections are caused by red-light-running (Retting 2006). Dense platoons of vehicles pose a great risk of rear-end collisions if they arrive at an approach around the end of green. Delaying platoon arrival several seconds gives the drivers an advanced notice about the red signal and may significantly reduce the risk.

1.2. Objectives

The major objectives of this research are to confirm the safety impact of signal coordination settings and to identify a practical means to incorporate safety consideration into design of arterial signal coordination. To accomplish these objectives, several tasks have been performed in the research.

1. First, the patterns of vehicle crashes at the coordinated signalized intersections are analyzed and documented. The crashes which are potentially caused by poorly designed signal timings are scrutinized.

2. Second, the traffic characteristics that are affected by signal timing and at the same time have important safety consequences are identified.
3. Third, models are developed to predict crash occurrence likelihood and crash severity based on available data.
4. Fourth, a prototype software tool is developed to evaluate the crash risks of coordinated arterial systems.

The results of this research will not only provide predictive models for intersection crashes, but also provide insight into the mechanism of crashes. Subsequently, it will provide a better basis for traffic engineers to improve arterial safety performance and for researchers to further understand the causes of crashes.

1.3. Research Scope

Only crashes that are potentially influenced by coordinated signal timings are considered in the research. Therefore, the focus of this research is on arterial approaches to coordinated signalized intersections. Also, only daytime crashes occurred on weekdays are included in the sample because coordination plans typically operate during these periods. Nighttime and weekend periods were excluded.

The two most frequent types of crashes at signalized intersections are rear-end and right-angle crashes. At the same time, these two types of crashes are most susceptible to poor signal timing among all crashes. Therefore, this research will focus on these two types of crashes.

The models were built based on the data provided by the Indiana Department of Transportation (INDOT). These data include the traffic counts at selected intersections, signal timing plans, geometry of the roads, and crash records. Some variables, such as the driver and vehicle characteristics, are important safety factors but they are not easily observable and not documented; thus, these variables were not considered in the model development process.

1.4. Report Organization

The main part of report is organized into eight chapters. Other relevant results and materials are included in the appendices.

Chapter 1 outlines the research. The motivation for the study is introduced as well as the major research objectives. The scope of research is also defined in this chapter.

Chapter 2 summarizes the major literature related to this research and also briefly introduces the available modeling alternatives.

Chapter 3 documents the study of crash patterns at coordinated signalized intersections. The findings serve as a guide for the model development.

Chapter 4 discusses the mechanisms of rear-end and right-angle crashes and develops conceptual basis for the modeling.

Chapter 5 summarizes the collected data used in the model development and describes the methods used to assemble the data from different sources and formats. The methods for deriving important predictive variables are also introduced in this chapter.

Chapter 6 presents and compares describes several developed models of crash occurrence and severity. Statistical details and engineering considerations of these models are presented.

Chapter 7 discusses the safety factors identified for urban arterial streets and their implications for designing coordinated signals.

Chapter 8 presents the computational procedure of predicting crashes for alternative coordination plans and step-by step user instructions for a developed prototype tool SafeArt that implements these calculations.

Chapter 9 summarizes up the research and reiterates the major findings. Potential future research directions are also proposed.

The appendices include reference materials and that can serve as a quick reference for the practitioners who want to consider safety in designing coordinated signals.

CHAPTER 2. LITERATURE REVIEW

2.1. Signal Coordination and Intersection Safety

Very few studies exist concerning the relationship between signal coordination and traffic safety. Recently researchers have begun to investigate the possible impacts of signal coordination on safety on an aggregated level. For example, whether the intersection is coordinated was used as a predictive variable in fitting the usual count data models for rear-end crashes (Wang et al. 2003). The preliminary results revealed the significance, as well as the complexity, of the potential correlations.

Improving signal coordination quality was cited as a proven way of improving traffic safety in a book (Institute of Transportation Engineers 1999). The claim, however, was based on two case studies (Parsonson 1983; Parsonson and Thomas 1978). The two studies simply compared crash frequencies before and after signal modernization projects. There are several drawbacks of this method. First, the criteria of selecting the intersections were not stated explicitly. The regression-to-mean effect may account for a large proportion of the reduction (Hauer 1980). Second, the important safety factors were not controlled. Third, the hypothesis testing procedure was not presented clearly.

Better coordination may also harm traffic safety. A recent study in Florida found that the percentage of road crashes caused by “disregarding traffic signals” was 3-5 times higher for urban one-way streets than for all road types on average (Tindale and Hsu 2005). They suggested that the better coordination on one-way streets may lure drivers to stay in the platoon by driving unsafely. In this case, the red light violations increase with better quality of coordination. The impacts, however, may probably be over-stated since other important factors were not controlled. For example, the one-way streets are more likely to be local roads where the law enforcement is not as strict as the two-way streets.

Another group of researchers reported, on the other hand, a positive correlation between better coordination and crash reductions (Rakha et al. 2000). They projected that after coordinating traffic signals across jurisdiction boundaries, the number of stops of vehicles decreased by 3.6%, and the crash risk decreased by 6.7% in the investigated area. The drawback of this study is the projected crash reduction was obtained through the model instead of observational study. The validity of the crash projection model was not evaluated thoroughly.

All the previous models analyzed aggregated statistics. The detail mechanism of the crashes and signal coordination were still unexplored. A recent study investigated the mechanism of rear-end collisions at signalized intersections. It represented the rear-end likelihood as the product of the probability of the lead vehicle decelerating and the probability of the following vehicle failing to decelerate accordingly (Wang et al. 2003). Although promising, this research didn't model the signal coordination parameters with sufficient accuracy required by this research. It only used an indicator variable to differentiate coordinated and isolated intersections. The important coordination parameters, such as cycles and offsets, were missing. More importantly, the link between traffic patterns and crash likelihoods is not modeled. As a result, it cannot be possibly incorporated into the signal optimization procedures when the key task is to select these parameters optimally.

Other disaggregated transportation safety studies are also available. (Persaud and Nguyen 1998) developed models which are disaggregated by time period, accident severity, and environment class. These models, however, are not designed specifically for capturing the impact of coordination signal timings.

As for right-angle collisions, a promising research is available (Bonneson et al. 2002). In this research, a model linking the cycle length, flow rate, yellow interval, and platoon ratio with the red light running frequencies was developed based on the mechanism of drivers' reaction to signal change. The calibrated model shows great prediction capacity. Another model linking red light running frequencies and right angle crashes was developed. Using these two models, one can link the right-angle crash frequencies with the traffic flow and signal timing parameters. However, the research was mainly used to

fit the red-light running numbers. The rear-end crash likelihood cannot be modeled in the same way. Another study also showed the RLR reduction effect of synchronized signal timings (Shinar et al. 2004).

A very recent research (Grembek et al. 2007) models the red-light running likelihood with real-time traffic monitoring data and establishes a correlation between the red-light running likelihood and different parts of traffic flows. This result shows the potential to improve safety without much loss of mobility through the simulation of TRANSYT (McTrans Center 1998). The model, however, cannot be used directly for our study because the real-time traffic flow information is not available. Moreover, it is unrealistic to assume that traffic systems engineers have these data when optimizing the signal timings of arterial.

As is well known, speed and speed dispersion can be modulated by signal coordination. An informational paper (Aarts and van Schagen 2006) reviews the impact of speed and speed dispersion on crash likelihood. The interactions of speed with other factors are also reviewed.

Many past studies focused on the impact of change intervals on crash likelihood. The dilemma zone issue (Gazis et al. 1960; Liu et al. 1996) has been investigated extensively. It is argued that a short yellow duration poses considerable safety threats. Different measures have been designed to control the dilemma zone occurrence. Among them, the Green Extension System (Zegeer and Deen 1978) is reported to have significant safety benefits. Recently, important improvements have been proposed to use real-time data in determining the optimal extension (Tarko et al. 2006; Zimmerman and Bonneson 2004). Coordination can affect the vehicles arriving around the change interval and thus can reduce the demand for dilemma zone control under such a scenario. This aspect has not been investigated.

2.2. Statistical Models of Crashes

Traffic crash data are a classic example of count data. The number of crashes can only be non-negative integers. The linear regression models are often inappropriate since the

predicted values can be negative and non-integers (Washington et al. 2003). The monograph (Cameron and Trivendi 1998) provides a comprehensive and in-depth treatment of a large variety of count data models.

Numerous studies have been conducted in modeling traffic safety with count data models. In probability textbooks (Casella and Berger 2001; Feller 1968), traffic crashes are often depicted as an example of a Poisson distributed random variable. Several studies showed the superiority of Poisson models relative to linear regression models in modeling traffic crashes (Jones et al. 1991; Miaou and Lum 1993). The Poisson model is often found to be inadequate. In particular, it is almost always discovered that the variances of the crash counts are significantly larger than the mean values of crash counts, which violates the property of Poisson distributed random variables.

The Negative Binomial Model, which allows the variances to be larger than the means (over-dispersion), was consequently used in later studies of traffic safety (Poch and Mannering 1996; Shankar et al. 1995). Another way of accommodating the over-dispersion phenomena is using the mixture models. In these models, the number of crashes is considered a mixture of multiple distributions. For example, some researchers noticed that frequencies of 0 were significantly larger than expected for the Poisson distribution. They thus proposed using a mixture of two processes to represent the crash process. These models were called the “zero-inflated” count data models (Mitra et al. 2002; Shankar et al. 1997). The validity of using this type of models in traffic safety was recently questioned (Lord et al. 2007; Lord et al. 2005).

Another important way of accounting for the over-dispersion is the random effects method (Chin and Quddus 2003; Kim et al. 2007). These models attribute the over-dispersion to the heterogeneity of data and allow complex correlation structures between the observations. For example, the temporally and spatially close intersections may possess similar components which are not captured by the fixed-effect models (Wang and Abdel-Aty 2006).

Most of the models introduced before are concerned with the aggregated number of crashes rather than the crash likelihood in a short period. In modeling such likelihood of

events, the discrete outcome models are the proper choice (Ben-Akiva and Lerman 1985; Greene 2000).

Discrete outcome models are used extensively in modeling human choices including travel demand (Domencich and McFadden 1975). Generally, these models assume that an unobserved utility or index function is associated with each outcome for an individual. The person chooses the outcome with the highest utility level. To capture the properties of human choices, a random term is introduced into the utility function. Depending on the specified distributions of the random part, different models of discrete choices can be derived.

A particularly popular model is the multinomial logit model, in which the random part is specified as independently and identically distributed log-Weibull random variables (Johnson and Kotz 1970; McFadden 1974; Washington et al. 2003). This specification of the random term results in a very simple functional form of the outcome probabilities and can be estimated very efficiently by modern statistical software tools (Greene 1995; StataCorp 2007).

A serious limitation of the multinomial logit model is the independence of irrelevant alternatives (IIA), which follows inevitably from the IID assumption for the random part of the utility function. In practice, the IIA violation is often subtle and difficult to find through conceptual examination. Several statistical tests are available to detect the IIA violation (Hausman and McFadden 1984; Small and Hsiao 1985) and the Small-Hsiao test is recommended by Zhang and Hoffman (1993).

Several remedy methods are available to address this violation. Among them, the nested logit model is a commonly used one (Ben-Akiva and Lerman 1985). It groups the alternatives with significant unobserved common effect into nests and thus accommodates the correlation.

Often unnoticed, the nested logit model has two formulations: the non-normalized multinomial logit (NNML) and the utility maximization consistent multinomial logit (UMNL) (Heiss 2002). Although widely used, the difference between these two formulations is

often overlooked. The difference, however, has very important implications about the estimation, hypothesis testing, and interpretation of the model. As an example, the often cited DZM (Daly and Zachary 1979; McFadden 1978) condition requires that the inclusive value parameters should be within the unit interval. This condition is not necessary for local consistency and is often found to be too strong in practice. Alternative conditions have been found to relax this strong claim for both two-level (Borsch-Supan 1987) and three level-nested logit models (Herriges and Kling 1996). These results are all derived based on the UMNL formulation rather than the NNML formulation. Many statistical software tools offer a choice between NNML and UMNL nested logit models but some of them only offer the NNML estimation procedures. Silberhorn et al. (2006) provides an extensive discussion of the estimation software of nested logit models. When the UMNL estimation software is not available, a method is proposed to trick the NNML software into producing UMNL results by introducing extra layers of dummy nests (Koppelman and Wen 1998).

The sequential logit model (SL) assumes that the event process can be divided into several independent stages. A multinomial logit model is used in each stage to model the process. This multi-stage event process is considered a more reasonable structure in some applications (Ophem and Schram 1997). It was demonstrated in a research (Nagakura and Kobayashi 2007) that the nested logit model is reduced to a sequential logit model when the difference of outcomes with a nest is negligible compared with the difference of outcomes across different nests. In this study, the sequential logit model is considered when the severity level is modeled in addition to the crash types. In the first stage, some risk factors are associated with the likelihood of rear-end and right-angle crashes; in the second stage, a logit model is used to model the severity consequence given that a certain type of crash occurs.

Other alternative discrete outcome models, including a probit model (Lee and Abdel-Aty 2005), an ordered logit model (Wang and Kockelman 2005), and a conditional logit model (McFadden 1974) will be discussed in CHAPTER 6 when these models are actually fitted with the dataset.

CHAPTER 3. INTERSECTION CRASH PATTERN STUDY

An integral part of this research is the study of crash patterns at the coordinated arterial intersections to point out signal time settings that may be a direct or indirect cause of crashes. The results of this crash study may serve as a guide in the later formulation and development of safety models.

3.1. General Procedure

Three Indiana coordinated arterial systems were selected for this study. The signal timings, geometry, traffic counts, and police crash reports were available for these systems.

First, with the help of a special GIS-based tool (Tarko et al. 2007), the publically available TIGER/Lines (US Census Bureau 2007) road maps, and the Indiana State Police (ISP) crash database, the crashes that occurred in the areas of the studied arterial systems have been identified. Then, the GIS tools were used to assign the identified crashes to the studied intersections.

Second, the IDs of crashes selected in the first step were used in querying the ISP crash database to identify the corresponding original crash reports. Then, the electronic versions of the original crash report forms were obtained from Indiana Department of Transportation.

Third, the crash reports were read with a special attention given to the narrative part where the collision circumstances are described. The crashes were classified according to several criteria including crash types, causal factors, signal status at crash moment, and signal violation types.

A total of 1,345 crashes that occurred in the 2003-2006 period were mapped in the vicinity of the studied intersections. In this study, only the crashes that occurred during signal coordinated periods were of interest. Therefore, only 829 crashes that occurred on weekday and inside of the 6:00 AM to 6:00 PM time window were included. Analysis of the electronic reports of these crashes identified 748 of them as having relevant information sufficiently documented for the research purpose.

3.2. Investigated Arterial Systems

The first studied system is located in West Lafayette, Indiana. It includes the intersections on US 52 from Nighthawk Street to Morehouse Street. The system's signal timings were last updated on March 4th, 2004. Table 3-1 lists the specific intersections of this system and Figure 3-1 shows the system layout.

The second system is located in Danville, Indiana, on US 36 between Mackey Road and Tennessee Street. The timings of this system were last updated on November 15, 2004 with a modification in spring of 2006 to improve afternoon westbound progression. Table 3-2 and Figure 3-2 give the basic information of this system.

Table 3-1 Intersections of US-52 system

US 52 Klondike System	
Arterial road	US 52
Crossing Roads	Research Park Entrance
	Win Hentschel Blvd
	Cumberland Avenue
	Yeager #13
	Salisbury St
	Nighthawk

Figure 3-1 US-52 system

Table 3-2 Intersections of Danville system

Danville System	
Arterial road	US 36
Crossing Roads	Mackey Rd
	SR 39
	Jefferson St
	Washington St
	Tennessee St

Figure 3-2 Danville system

The third one is located in Frankfort, Indiana. This system is special because it is coordinated in both west/eastbound and north/southbound. It includes seven signals around the SR 28 crossing SR 39 area. It was last retimed in late February 2004. Table 3-3 and Figure 3-3 show the intersection and layout of the system.

Table 3-3 Intersections of Frankfort system

Frankfort System		
Arterial road	SR 28	SR 39 (Jackson)
Crossing Roads	Columbia	Washington
	Main	Clinton
	SR 39 (Jackson)	SR 28
	Clay	Armstrong St

Figure 3-3 Frankfort system

3.3. Crash Mapping and Record Extracting

Figure 3-4 illustrates the usage of the GIS tool in mapping the crashes onto the geometry files of local roads. The circles and dots are crash locations and the road network information is from the TIGER/Line files.

Table 3-4 summarizes the types of collisions mapped around the intersections in these three systems. It confirms that rear-end and right-angle are the two most frequent types of crashes that occurred at the signalized intersections.

Figure 3-4 Crashes initially mapped on US-52 system

The electronic police crash reports were then extracted from INDOT's database. The crash report filing instructions can be found in Indiana State Police Crash Records Section (2004). Moreover, a sample electronic crash report file is included in 0. Figure 3-5 shows a page from that sample electronic crash report file. On this page, the crash diagram is shown and the police records of narratives from the drivers are recorded. 748 such crash reports were read and classified in this study.

Page 2 of 4																		
Type of Crash	☒ Rear End ☐ Same Direction Sideswipe ☐ Right Angle ☐ Backing Crash ☐ Head On ☐ Opposite Direction Sideswipe ☐ Left Turn ☐ Other ☐ Rear to Rear ☐ Ran off Road ☐ Right Turn ☐ Non-Collision																			
Diagram: (Indicate North by Arrow)

Narrative: VEH#2 WAS STOPPED IN THE LEFT HAND LANE OF E/B US HWY 52 AT MOREHOUSE RD FOR A RED LIGHT. DUE TO THE WET ROAD CONDITIONS AND SPEED UNREASONABLE FOR THE CONDITIONS, VEH#1 TRAVELED IN THE LEFT HAND LANE OF E/B US HWY 52 AND STRUCK THE REAR OF "STOPPED" VEH#2. DR#2 STATED SHE WAS STOPPED BEHIND AVEHICLE FOR THE RED TRAFFIC SIGNAL, AND HEARD "TIRE SQUEALS" BEFORE BEING STRUCK FROM BEHIND. DR#1 STATED SHE WAS UNABLE TO STOP AND "SLID" INTO VEH#2.																				
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		Reviewing Officer																		

Figure 3-5 Page from a police crash report

Table 3-4 Summary of types of crashes

Type (Manner of Collision)	Number	Percent
01 - Rear End	462	34
02 - Head On	58	4
03 - Rear to Rear	21	2
04 - Same Direction Sideswipe	126	9
05 - Opposite Direction Sideswipe	39	3
06 - Ran off Road	39	3
07 - Right Angle	288	21
08 - Left Turn	70	5
09 - Right Turn	21	2
10 - Left/Right Turn	15	1
11 - Backing	155	12
12 - Other (Explain in Narrative)	49	4
13 - Non-Collision	2	0
Total	1,345	100

3.4. Rear-end Collisions

In this section, the information regarding rear-end crashes obtained through reading the crash reports is summarized. After reading the reports, some of the crashes were found to be not relevant to arterial signal timing and were thus excluded from further study. Only 15 of all these 462 rear-end crashes were possibly related to the dilemma zone issue. Some of the rear-end crashes occurred on side street approaches which are not regulated by arterial coordination timings. The breakdown by arterial and approach type are shown in Table 3-6.

Table 3-5 Rear-end crash distribution by arterial and relevancy

	All three Systems	Frankfort	Danville	West Lafayette
Total	462	19	84	359
Not relevant	138	2	38	98
Relevant	324	17	46	261
Dilemma related	15	0	2	13

Table 3-6 Rear-end crash by arterial and approach locations

	All three Systems	Frankfort	Danville	West Lafayette
On arterial approach	274	17	43	214
On side street approach	50	0	3	47
Total	324	17	46	261

Then, the 324 rear-end crashes that occurred on an arterial approach at these three intersections were further classified by the signal status at the moment of the crash as reported by the driver or witness. It was found that 58% of the crashes had signal status documented. In these crashes, only 10% of them occurred when the signals were changing from green to yellow or yellow to red. On the other hand, 40% of the rear-end crashes occurred when the signal was red or was changing from red to green.

Table 3-7 Rear-end crashes by arterial and signal status

Signal Status	Crash Numbers (Percentages)			
	All Systems	Frankfort	Danville	West Lafayette
Green	15 (5%)	0 (0%)	1 (2%)	14 (5%)
Red turning Green	31 (10%)	1 (6%)	4 (9%)	26 (10%)
Yellow turning Red	18 (6%)	1 (6%)	1 (2%)	16 (6%)
Green turning Yellow	11 (3%)	0 (0%)	1 (2%)	10 (4%)
Red	114 (35%)	7 (41%)	21 (46%)	86 (33%)
Unknown	135 (42%)	8 (47%)	18 (39%)	109 (42%)
Total	324 (100%)	17 (100%)	46 (100%)	261 (100%)

3.5. Right-angle Collisions

This section summarizes the information regarding the right-angle crashes obtained through reading the crash reports. It was found that only 70 of the 287 reviewed crashes were actually relevant to arterial signals. Table 3-9 breaks down these right-angle crashes by year and arterial. As is seen from Table 3-10, many of the right-angle crashes are not between vehicles driving in crossing directions.

Finally, Table 3-11 classifies the right-angle crashes by violation type. Conceptually speaking, a right-angle crash cannot occur when no signal control regulation is violated. As can be observed in this table, arterial vehicle red-light running was the most frequent cause of right-angle crashes. The second most frequent violation type was the crossing street through the red signal. The arterial permitted left turn signal violation was the third. The right turn on red (RTOR) related right-angle crash was quite infrequent.

Table 3-8 Breakdown of right-angle crashes by relevancy

Total Reviewed	287
Relevant	70
Not relevant	217

Table 3-9 Breakdown of relevant right-angle crashes by arterial and year

System	2003	2004	2005	2006	Total
Frankfort	5	0	2	7	14
Danville	1	2	2	3	8
West Lafayette	11	19	10	8	48
Sum	17	21	14	18	70

Table 3-10 Breakdown of right-angle crashes by pre-collision movement

Pre-collision vehicle movement	
Parallel	28
Crossing	43

Table 3-11 Right-angle crashes by arterial and violation type

Violation Type	All Three Systems	Frankfort	Danville	West Lafayette
Arterial Through Red	26	5	4	17
Arterial Left Turn Red	4	2	0	2
Arterial Permitted Left Turn	16	1	1	14
Crossing Street Red	17	6	1	10
Arterial RTOR ²	1	0	0	1
Crossing Street RTOR	2	0	0	2
Other	4	0	2	2
Total	70	14	8	48

3.6. Summary of Findings

This study of crash patterns at coordinated arterial systems confirmed that rear-end and right-angle crashes are the two most frequent types of crashes after excluding many crashes that are not relevant to signal timing settings.

Arterial approaches experience about 80% of all rear-end collisions and only 10% of rear-end collisions occur during the end-of-green period. The widely perceived dilemma zone issue is only possibly related to less than 5% of all rear-end crashes. The presence of a queue seems to be the predominant circumstance during which rear-end crashes occur.

² RTOR stands for right turn on red.

For right-angle crashes, red signal violations or priority violations by arterial vehicles lead to about 70% of all right-angle crashes. The permitted left turn violations account for 20% right-angle collisions. Only a very small fraction of all right-angle crashes are related to right turn on red violations.

CHAPTER 4. CRASH MECHANISMS

In this chapter, the mechanisms of rear-end and right-angle crashes at signalized intersections will be analyzed. The purpose of this analysis is to provide a basis and guidance for the statistical model development. Understanding the crash mechanism is crucial in model development because it will help in identifying potential important variables and in selecting the proper functional form of the independent variable in the model. The analysis in this chapter is based on the information obtained through the crash pattern study as documented in CHAPTER 3.

4.1. Rear-end Crash and Signal Timing

A rear-end crash involves two vehicles going in the same direction. It occurs when a following vehicle hits the rear portion of the leading vehicle as a result of contradicting decisions of two drivers.

The first scenario of a rear-end crash that can be affected by the signal timing is the well-known dilemma zone issue. When facing the yellow signal, drivers have two choices: first, proceed driving and try to enter the intersection before red; and second, decelerate the vehicle so as to stop at the intersection. The zone where the expected travel time to the approach is two to six seconds is often considered as a zone where drivers may frequently make different decisions (Gates et al. 2007; Zimmerman and Bonneson 2004). When a leading vehicle decides to stop while the vehicle behind it decides to proceed, a rear-end crash is highly probable. A conceptual model of this type of rear-end crash is presented below. The conceptual model is not meant to be replacement for empirical studies. The purpose is to identify the possible risk factors that can be related to this scenario of rear-end crashes in later model development.

Assume that the probability of choosing to stop is a function of the travel time to the intersection. For example, it is partially estimated to be Gates et al. (2007).

$$\ln\left(\frac{1 - p_{stop}}{p_{stop}}\right) = 6.34 - 1.69t \quad (\text{Eq. 4.1})$$

In addition, assume that the headway distribution follows the lognormal distribution as recommended in Luttinen (1996). Then, it can be shown through simulation that the probability of contradicting initial decisions increases almost linearly with the number of vehicles in the two to six seconds zone. Figure 4-1 and Figure 4-2 illustrate the simulation results of this simplified analysis. Therefore, the instant volume rate at the beginning of yellow has a direct impact on the probability of contradicting initial decisions. This suggests that if the dilemma zone issue is a major cause of rear-end crashes, then the volume rate at the beginning of the yellow has a direct impact on the likelihood of rear-end crashes.

Figure 4-1 Vehicles in 2-6 seconds zone vs. probability of contradicting decision

The other possible scenario of rear-end crashes involves the queuing vehicles. This type of rear-end crash is more frequently reported as observed in the crash patterns study. The dynamic queue forming process involves a great number of vehicle maneuvers.

Figure 4-2 Instant volume rate vs. contradicting decisions in 10000 simulations

This type of rear-end risk is exacerbated during the queue discharging process. When green is shown, leading vehicles accelerate and pass through the intersection. At the same time, vehicles are still coming to join the end of the queue. This process is often referred in the literature as shock wave process (Gazis 2002). Several rounds of braking-accelerating maneuvers may be required even for the same vehicle before it finally passes the intersection. Figure 4-3 illustrates this shock wave process of queue discharging. Therefore, this kind of rear-end crashes is directly affected by the volume rate and the queue length at the beginning of the green phases.

Therefore, in the later modeling process, both the volume rate at the beginning of the red and at the beginning of the green will be tried in the rear-end crash models.

Figure 4-3 Time-space diagram of the queue discharging process

4.2. Right-angle Crashes and Signal Timing

As modeled in a study (Bonneson and Zimmerman 2004), the red-light running probability is directly affected by the duration of the yellow and all-red interval. The conditions for a red-light running (RLR) are as follows:

- 1 A vehicle approaches the intersection near the end of green.
- 2 Driver decides to go when the yellow signal is shown.
- 3 Actual travel time is longer than the change interval (yellow plus all-red).

For a vehicle, define a random binary variable RLR which equals 1 when a vehicle runs the red light and it equals 0 otherwise. Then, the probability for this vehicle to run the red light is a function of its travel time to the intersection t . That is,

$$\begin{aligned} P(RLR) &= P(go)I(T > Y + AR) \\ &= \frac{e^{a-bt}}{1 + e^{a-bt}} I(t > Y + AR) @ h(t) \end{aligned} \quad (\text{Eq. 4.2})$$

where $I(x)$ is an indicator function and a and b are coefficients of the stopping probability function. Assume that the vehicles are uniformly distributed (the expected time headways between every two consecutive vehicles are the same). The travel time of the first vehicle is a uniformly distributed random variable

$$t : \text{uniform}(0, C / Q) \quad \text{P } f_t = Q / C \quad (\text{Eq. 4.3})$$

where Q is the instant traffic flow and C is the width of the time window.

Therefore, the expected probability of RLR is

$$\begin{aligned} E(P(RLR)) &= \int_0^{C/Q} P(RLR) f_t dt \\ &= \int_0^{C/Q} h(t) f_t dt \\ &= \int_0^{C/Q} \frac{Q}{C} \frac{e^{a-bt}}{1 + e^{a-bt}} I(t > Y + AR) dt \\ &= \int_{Y+AR}^{C/Q} \frac{Q}{C} \frac{e^{a-bt}}{1 + e^{a-bt}} dt \\ &= \frac{Q}{bC} \ln(1 + e^{a-b(Y+AR)}) \end{aligned} \quad (\text{Eq. 4.4})$$

The expected aggregated number of right light running is subsequently proportional to the instant volume rate at the beginning of the red interval and is negatively correlated

with the duration of the change interval. The actual predictive power of these variables will be tried in the later model development.

CHAPTER 5. DATA COLLECTION AND ASSEMBLY

Data collection and assembly is a major part of this research. Probably, in part due to the rarity of vehicle crashes, most transportation safety research uses aggregated crash and socio-economic data as the input for model development. In this study, however, such aggregated data are hardly meaningful. As discussed previously, signal timing and traffic conditions constantly change with time and a macro representation of the signal coordination condition will fail to establish defendable and useful correlations that can be used in the signal optimization process. Detailed and disaggregated data must be used to study the impact of adaptive signal timings and crash likelihoods. Past studies (Karlaftis and Tarko 1998; Persaud and Nguyen 1998) tried to use disaggregated crash data in models. None of those studies, however, is suitable for checking the impact of signal timing because signal timings adapt to traffic condition on a minute to minute basis. Due to the unique data requirement, different sources of data must be combined to generate the dataset that can be used in developing the statistical models. Since the smallest format of available data is the 15-minute vehicle counts at each intersection, 15-minute interval at an arterial approach is selected as the building block of the dataset. In other words, a 15-minute interval of the day and the approach uniquely defines an observation of the dataset.

5.1. Raw Data Sources

To construct the dataset needed for this research, several sources of data have to be used.

- Geometry data. This includes the location of the system, the distance between the intersections, the number of through/right turn/left turn lanes, and the posted speed limit at each approach

- Traffic volume data. This should include the traffic counts of all movements at each intersection. The latest data should be used. The counts are aggregated in 15-minute intervals and typically last for at least 12 hours.
- Signal timing data. This should include all the signal timing parameters of all the intersections. The most important parameters are cycles, offsets, and splits.
- Operation periods of the timing plan. Typically, several signal timing plans are used for an arterial system. The periods of all timing plans are required to match a plan to a specific time of day.
- Month of system timing update. Because the arterial system timings are designed and implemented at different times, the starting month of the current signal timing must be provided. The observation will be replicated for every month after the system is re-timed.
- Crash data. The date, time, location, type, and severity level of all relevant crashes that occurred during the period.

The arterial systems included in the model developed are summarized in Table 5-2. Two of the systems reviewed in CHAPTER 3 were excluded because their signal controllers are not interconnected and therefore the offsets vary significantly from the documentation. Table 5-3 summarizes the crash statistics of these arterial systems. The counties of the included arterials are marked on Figure 5-1 with grayed grids.

Table 5-1 summarizes the different sources of raw data and the information included in the final dataset. The data entries marked with “+” are used as record matching columns and those marked with “x” are added into the final dataset from a corresponding data source. Relational database operation language SQL (Colburn 2000) was used to merge data from different sources to obtain the complete interval by interval dataset.

The arterial systems included in the model developed are summarized in Table 5-2. Two of the systems reviewed in CHAPTER 3 were excluded because their signal controllers are not interconnected and therefore the offsets vary significantly from the documentation. Table 5-3 summarizes the crash statistics of these arterial systems. The counties of the included arterials are marked on Figure 5-1 with grayed grids.

Table 5-1 Summary of raw data sources

Variables	Final	Crash Table	Geometry	Signal Timing	Traffic Counts	Plan in Operation	Months After Retime
Arterial	x ¹	+ ¹	+	+	+	+	+
Intersection ID	x	+	+	+	+		
Direction	x	+	+	+	+		
Hour	x	+		+	+	+	
Quarter of hour	x	+		+	+	+	
Sequence of coordination plan	x					x	
In coordination mode	x			x			x
First intersection	x				x		
Cycle	x			x			
Yellow	x			x			
AR	x			x			
Offset from upstream	x			x			
Speed limit (mph)	x		x				
Distance from upstream	x		x				
Number of thru lanes	x		x				
Number of left turn lanes	x		x				
Number of right turn lanes	x		x				
Arterial width (ft)	x		x				
Crossing street width (ft)	x		x				
Upstream primary volume (vphpl)	x				x		
Upstream secondary volume(vphpl)	x				x		
Crossing street volume (vph)	x				x		
Downstream volume (vphpl)	x				x		
Upstream min green	x			x			
Upstream max green	x			x			
Min green	x			x			
Max green	x			x			
Year	x	+					+
Month	x	+					+
Day of month	x	+					
Day of week	x	+					
Crash ID	x	x					
Crash severity	x	x					
Crash type	x	x					

¹The data entries marked with “+” are used as record matching columns and those marked with “x” are added into the final dataset from a corresponding data source.

Table 5-2 Summary of arterial systems in final dataset

Arterial System	SR-28	SR 135	SR 431	US41	US30 A	US30 B
Arterial ID	3	5	6	7	8	8
City (County)	Frankfort (Clinton)	Indianapolis (Marion)	Indianapolis (Hamilton)	(Lake)	(Lake)	(Lake)
Interconnected	Yes	Yes	Yes	Yes	Yes	Yes
No. of Coordinated intersections	7	7	7	5	4	5
No. of timing plans	3	3	6	4	4	4
Month of signal implementation	Feb-2004 34	Jul-2004 29	Dec-2002 48	Aug-2001	Dec-2001	Dec-2001

Table 5-3 Summary of number of crashes by arterial and severity

Arterial	Crash Type				Total
	Rear-end		Right-angle		
	PDO	IF	PDO	IF	
SR 135	9	3	7	5	24
SR 28	1	2	2	0	5
SR 431	160	27	10	4	201
US 30	110	34	22	13	179
US 41	6	4	3	1	14
Total	286	70	44	23	423

Figure 5-1 Locations of Included arterial systems

In addition, the widths of the arterial street and the crossing street are measured based on the satellite photo of the intersection. Figure 5-2 shows the satellite map of the Morehouse Road at US 52 intersection. The stop bars are marked on the map and the widths are measured from stop bar to stop bar.

Figure 5-2 Map of Morehouse Rd. at US 52 intersection³

5.2. Derived Data

In addition to the raw data, certain variables must be derived from the raw dataset. These derived variables, their definitions, and the potential impact on crashes are discussed in this section.

5.2.1. Seasonal Factor

As is shown in Figure 5-3 (Traffic Monitoring Section 2008), Indiana arterials have significantly different levels of traffic volumes in winter and summer. Therefore, indicator variables *Winter* and *Summer* were defined for these two seasons. November,

³ Retrieved on 5/15/2008 from Google Map (<http://maps.google.com/>)

December, January, and February were considered the winter season and June, July, and August were considered the months of summer. Because driving behavior can be prominently different in these two seasons, these seasonal factors may affect both the crash frequency and severity.

Figure 5-3 Monthly traffic level of Indiana arterials

5.2.2. Traffic Volume Data

The raw data included 12 continuous hours of vehicle counts for all movements of each intersection. These traffic counts were converted to traffic volume data. The upstream primary and secondary volumes are calculated from the existing traffic counts. The primary volume is from the upstream through movements and the secondary volume is from the upstream volumes turning onto the arterial. As an example, consider the westbound approach at intersection A in Figure 5-4. The westbound through volume (WBT) from the upstream intersection B is considered the primary volume. The sum of northbound left turn (NBL) volume and the southbound right turn (SBR) volumes are considered the secondary volumes. All these volumes are converted to the unit of vehicles per hour per arterial lane. Figure 5-5 and Figure 5-6 show the histograms of these calculated volumes for each arterial.

Figure 5-4 Illustration of primary and secondary volumes

Figure 5-5 Histograms of primary volume from upstream (vphpl) by arterial

Figure 5-6 Histograms of secondary volume from upstream (vphpl) by arterial

5.2.3. Crossing Street Volume

The number of vehicle accumulated at the crossing street approach when arterial approach is green is also derived. If there are no vehicles accumulated at the crossing street approach, then right-angle crash between the arterial red light running vehicle and crossing street vehicle is impossible. Typically, it is assumed that the crossing street has random arrival. Thus, the expected number of vehicles waiting on the crossing street at the end of arterial green phase can be calculated as

$$CrossV = G \mathcal{V}_{cross} \quad (\text{Eq. 5.1})$$

where $\mathcal{V}_{cross}$ is the crossing street volume rate and G is the length of the arterial green.

This variable is not normalized according to the number of lanes on the crossing street.

5.2.4. Travel Time between Intersections

The travel time between intersections is calculated from the distance and the posted speed limit. The actual travel time may deviate significantly from this value depending on the traffic condition, weather, and drivers' preferences. This variable only serves as an approximate. Figure 5-7 illustrates the histograms of expected travel times from an upstream intersection of each arterial system. Two indicator variables, TrTimeLt15 and TrTimeGt40, are further defined based on the calculated travel. TrTimeLt15 is 1 only when the calculated travel time is shorter than 15 seconds and TrTimeGt40 is 1 only when the calculated travel time is longer than 40 seconds.

Figure 5-7 Travel time from upstream intersection by arterial

5.2.5. Traffic Arrival Pattern

A method of traffic arrival pattern characterization is proposed in this study. Each signal cycle of an arterial approach is divided into four periods: the first half of red, the second

half of red, the first half of green, and the second half of green. The four parts are named R1, R2, G1, and G2 respectively as shown in Figure 5-8.

Figure 5-8 Four periods of a signal cycle

In principle, coordinated arterial signals share a common cycle. This allows the adjacent intersections to maintain fixed relative time differences. The end of the coordinated phase (i.e., the end of the yellow in Figure 5-8), is often selected as the reference point of the coordinated signal. Offset, another important parameter of a signal coordination plan is measured as the time difference between the reference points of adjacent coordinated signals, as shown in Figure 5-9.

Figure 5-9 Offset of adjacent coordinated signals

Generally speaking, the length of the coordinated green phase is not fixed. Typically, the duration is limited to a preset range between minimum and maximum greens. The beginning of the coordinated green fluctuates according to the real-time traffic demand present on the arterial. Therefore, the actually served green in each cycle is a value in the preset range. The medium value of the preset range is calculated and named as *mid green* for usage in the model. These concepts are illustrated in Figure 5-10.

Figure 5-10 Green fluctuation of coordinated green phases

Following the conventional method, it is assumed that the primary volume passes the upstream intersection during the green and secondary volume passes the upstream intersection during the red in calculating the traffic arrival patterns at the downstream approach. Furthermore, it is assumed that all vehicles drive at the posted speed limit. The offset between the two approaches, determines the traffic arrival flow profile at the downstream intersection. Figure 5-11 illustrates the method of calculating downstream arrival flow rate from the upstream discharging flow rate, speed, distance, and signal timing settings. In this example, the downstream flow rate is high in R1 and R2 and the flow rate is low in G1 and G2.

Figure 5-11 Model of arrival flow rate at downstream intersections

In addition, the queue dispersion effect must be considered as well (Papacostas and Prevedouros 2000). When the upstream intersection is far away from the approach, the vehicle platoon tends to disperse and the modulated flow becomes closer to the random arrival flow. Many platoon dispersion models are available (Rakha and Farzaneh 2006; Robertson 1969). In this research, a linear platoon dispersion model was designed and used. It is suggested (Hook and Albers 1999) that for intersections located more than 5,000 feet from each other, coordination is probably not helpful. Therefore, in this study, it is assumed that the vehicle platoon is fully dispersed after 120 seconds of travel.

Assume that the primary volume rate from the upstream approach is V_{prim} and the secondary volume rate from the upstream intersection approach is V_{sec} . Then, the flow rate (vph) at travel time t down the arterial is calculated as

$$V_t = V_0 + \frac{V_T - V_0}{T}t \quad (\text{Eq. 5.2})$$

where t is a time between 0 and T seconds, V_0 is the initial flow rate (vph), and V_T is the fully dispersed flow rate which can be calculated as

$$V_T = \frac{\# \text{ vehicles in a cycle}}{\text{cycle length}} = pV_{prim} + (1 - p)V_{sec} \quad (\text{Eq. 5.3})$$

where p is the proportion of the effective green phase and is calculated as

$$p = \frac{\text{green} + \text{yellow}}{\text{cycle}} \quad (\text{Eq. 5.4})$$

As a result, if the initial volume rate at the upstream intersection is the primary volume V_{prim} , then the downstream flow rate is

$$V_t^{prim} = V_{prim} - (1 - p)\frac{t}{T}(V_{prim} - V_{sec}) \quad (\text{Eq. 5.5})$$

If the initial volume rate is the secondary volume V_{sec} , then the downstream flow rate is

$$V_t^{sec} = V_{sec} + p\frac{t}{T}(V_{prim} - V_{sec}) \quad (\text{Eq. 5.6})$$

Finally, the arrival flow profile of the entire cycle is calculated at the downstream approach. Based on different green signal durations, three different profiles were obtained for max, mid, and min green settings respectively.

Based on the calculated arrival profiles, 12 indicator variables were created: max_r1, max_r2, max_g1, max_g2, mid_r1, mid_r2, mid_g1, mid_g2, min_r1, min_r2, min_g1, and min_g2. The prefix indicates which set of green timings are used. The suffix represents the time period to which the variable corresponds. If a time period concentrates more than 25% of the traffic of the entire cycle, then the corresponding interval is coded as 1. For instance, max_g2 is 1 only when the vehicles arriving in the second half of green account for more than 25% of the volume of the entire cycle, calculated based on the maximum green signal timings.

Based on these 12 indicator variables, we can define a four-digit code to represent the traffic arrival pattern at the downstream interval. For example, the traffic arrival pattern shown in Figure 5-12 will be coded as 0011. If the pattern is obtained based on maximum green, then the variable max_0011 is set as 1.

Figure 5-12 Traffic pattern max_0011

The patterns distribution of the final data set is summarized in Table 5-4. As is shown in the table, the patterns with three 1's were rare. The most frequent types of traffic arrival patterns were 0011, 1100, and 1000. Two variables, G2_max and R2G1_max, were defined to represent two groups of patterns. G2_max is 1 if one of max_0011, max_1001, and max_0001 is 1; R2G1_max is 1 is one of max_0110, max_0100, and max_0010 is true.

Table 5-5 summarizes the percentage of these two groups of traffic arrival patterns for each arterial.

A case study was conducted to verify the validity of the traffic pattern characterization method. The actual observed patterns of the US 52 system, documented in Section 3.2, were compared with the pattern calculated from this method. It was found that the maximum green-based codes were the most similar to the observed patterns. The results are summarized in

Table 5-6. Since the maximum-based pattern codes are found to be the most accurate, in later model development, the max green based variables were used.

Table 5-4 Summary of arrival pattern classification

Pattern	Green Setting		
	Max	Mid	Min
0000	0.00%	0.00%	0.00%
1000	19.1%	27.2%	35.5%
0100	4.11%	12.8%	19.7%
0010	5.39%	0.625%	0.885%
0001	3.50%	1.94%	2.22%
1100	17.9%	14.6%	16.3%
0110	5.35%	7.76%	3.44%
0011	34.2%	18.0%	8.33%
1001	8.20%	11.7%	13.4%
1010	0.00%	0.00%	0.00%
0101	0.485%	0.578%	0.00%
1110	0.143%	0.283%	0.330%
0111	0.378%	0.228%	0.788%
1011	1.24%	2.86%	0.924%
1101	0.00%	1.43%	0.939%

Table 5-5 Percent of G2_max and R2G1_max patterns by arterial

Variable	SR 135	SR 28	SR 431	US 30	US 41	Total
G2_max	48.6%	54.6%	33.9%	50.0%	47.1%	45.9%
R2G1_max	20.8%	4.04%	35.7%	3.64%	1.57%	14.9%

Table 5-6 Verification of traffic pattern classification⁴

Calculated from Max Green				Observed			
R1	R2	G1	G2	R1	R2	G1	G2
22.5%	16.5%	21.5%	39.5%	17.6%	14.7%	33.9%	33.9%
32.4%	17.6%	15.0%	35.0%	24.2%	10.0%	26.2%	39.6%
32.3%	17.7%	15.1%	34.8%	30.2%	11.0%	14.4%	44.4%
23.2%	23.2%	26.8%	26.8%	21.0%	25.7%	35.7%	17.6%
23.2%	23.2%	26.8%	26.8%	21.5%	16.5%	35.8%	26.2%
23.2%	23.2%	26.8%	26.8%	14.8%	13.4%	43.0%	28.9%
26.4%	10.6%	19.4%	43.6%	2.2%	1.9%	57.9%	38.0%
27.0%	8.7%	18.1%	46.2%	3.3%	2.7%	60.0%	34.0%
27.1%	8.4%	17.9%	46.6%	4.7%	2.9%	35.9%	56.5%
9.4%	26.7%	38.8%	25.1%	8.4%	13.3%	58.9%	19.3%
8.9%	26.8%	39.3%	25.0%	4.0%	7.7%	58.8%	29.4%
11.8%	26.0%	36.7%	25.5%	22.3%	13.6%	40.9%	23.1%
8.4%	26.9%	39.7%	25.0%	6.2%	9.5%	56.9%	27.5%
22.5%	10.3%	29.2%	38.0%	12.4%	17.9%	41.3%	28.4%
22.8%	11.6%	28.7%	36.9%	15.7%	25.0%	27.3%	32.0%
22.0%	20.8%	27.7%	29.5%	13.9%	13.3%	42.8%	30.1%
22.2%	21.1%	27.4%	29.3%	11.9%	20.8%	31.5%	35.7%
22.0%	20.8%	27.7%	29.5%	9.7%	23.4%	50.3%	16.6%
14.5%	15.0%	33.0%	37.4%	10.1%	5.6%	49.4%	34.8%
16.3%	16.9%	31.5%	35.3%	6.0%	13.9%	40.8%	39.3%
18.0%	18.6%	30.1%	33.3%	15.0%	6.8%	41.7%	36.4%
11.5%	59.2%	24.9%	4.4%	42.3%	25.7%	16.2%	15.8%
12.6%	57.7%	24.5%	5.2%	30.8%	24.1%	22.4%	22.7%
12.6%	57.7%	24.6%	5.2%	30.4%	30.4%	19.0%	20.2%
12.2%	58.2%	24.7%	4.9%	29.6%	25.6%	17.0%	27.8%

⁴ Traffic percent greater than 25% are shadowed

5.2.6. Platoon Ratio and Arrival Type

Platoon ratio is a traditional measure of arterial signal mobility performance. It is defined as

$$R_p = \frac{P}{\text{green} / \text{cycle}} \quad (\text{Eq. 5.7})$$

where P is the proportion of traffic that passes the effective green phase of the coordinated signal (Institute of Transportation Engineers 2000). This proportion of traffic during the green can be calculated based on the traffic arrival pattern obtained with the method described in Section 5.2.5. Table 5-7 provides summary statistics of the calculated platoon ratios. Figure 5-13 gives the histograms of calculated platoon ratios by arterial based on the maximum green timings.

Table 5-7 Summary statistics of platoon ratios

Variable	Mean	Std. Dev.	Min	Max
pr_max	1.00	0.37	0.23	2.17
pr_mid	1.06	0.62	0.18	2.75
pr_min	1.22	1.22	0.11	5.64

Arrival type is another traditional measure of arterial coordination quality, which can be calculated based on the platoon ratio. The correspondence of the platoon ratio and the arrival type is summarized in Table 5-8. Again, the platoon ratios and arrival types depend on the green timings and thus the values based on max, mid, and min greens were all calculated and considered in the model development. Table 5-9 shows the distributions of arrival type by arterial calculated based on the maximum green. Arrival types 3 and 4, which represent the medium or slightly good progression quality, were most frequently obtained. Arrival types 5 and 6 were rarely found in this dataset.

Figure 5-13 Histograms of platoon ratio based on the max arterial green

Table 5-8 Platoon ratio and arrival type

Platoon Ratio	Arrival Type	Progression Quality
$(-\infty, 0.50]$	1	Worst
$(0.5, 0.85]$	2	Poor
$(0.85, 1.15]$	3	Random
$(1.15, 1.50)$	4	Good
$(1.50, 2.00]$	5	Very Good
$(2.00, \infty)$	6	Best

Table 5-9 Distribution of arrival types by arterial

Arrival Type (max green)	Arterial					
	SR 135	SR 28	SR 431	US 30	US 41	Total
1	1.4%	8.7%	4.4%	16.7%	14.1%	9.2%
2	14.9%	20.8%	24.7%	29.7%	38.8%	25.5%
3	62.3%	18.1%	36.7%	11.2%	29.8%	29.8%
4	21.4%	31.5%	32.2%	28.5%	17.3%	27.5%
5	0.0%	18.3%	1.9%	13.9%	0.0%	7.6%
6	0.0%	2.7%	0.0%	0.0%	0.0%	0.5%
Total	100.0%	100.0%	100.0%	100.0%	100.0%	100.0%

5.2.7. Arterial Change Interval Volume

As analyzed in CHAPTER 4, the change interval vehicles can be the most dangerous vehicles in terms of rear-end and right-angle crash likelihoods. Therefore, two variables were defined to represent the beginning of the green and the beginning of the red volumes respectively. Two two-second time windows were defined at the change interval of the beginning of red and beginning of green. Both of them are from the phase change moment to two seconds after the phase change. Figure 5-14 illustrates the calculation of the change interval volume variables ArtBOR and ArtBOG.

5.2.8. Sufficiency of Change Interval

According to the signal design manual (Federal Highway Administration 2003), given that the posted speed limit is V (ft/s), the appropriate duration of the yellow should at least equal to

$$\bar{Y} = \frac{V}{20 \text{ ft} / \text{sec}^2} + 1 \text{ sec} \quad (\text{Eq. 5.8})$$

to avoid the dilemma zone issue. The all-red interval should be at least

$$\frac{2}{A}R = \frac{S}{V} \text{ sec} \quad (\text{Eq. 5.9})$$

Figure 5-14 Arterial change interval volume

where S is the stop bar to stop bar distance and V is the speed limit. In practice, however, these rules are not always followed. Two indicator variables, *ShortY* and *ShortAR*, were created to represent the sufficiency of the selected yellow and all-red duration. If a yellow is shorter than the recommended minimum value, *ShortY* is set as 1 and when the actual all-red interval is shorter than the recommended value, *ShortAR* is set to 1.

5.3. Summary of Assembled Data

Finally, the next table provides a summary of all the predictive variables that were considered in the later model development. The suffix represents the set of green signal timings in deriving the variable.

Table 5-10 Description of predictive variables

Variable Codes	Variable description
SR28, SR135, SR431, US41, US30 (Yes: 1, No: 0)	Indicator variables of arterial
UpPrimVol /UpSecVol	Primary/secondary volume per hour per lane from upstream
VolTotal	Total volume per hour per lane at the approach
VolCross	Total volume from crossing street per hour
ArtBOR(ArtBOG)_min/_mid/_max	Arterial traffic at the beginning of red/green. (2 second time window)
CrossV_min/_mid/_max	Volume accumulated at the crossing street during the green of arterial.
Winter (Yes: 1, No: 0)	In month Jan, Feb, Nov, or Dec
AM (Yes: 1, No: 0)	Morning indicator. Intervals before 12:00 AM
ArtWidth/CrossWid	Width measured from satellite photo of the arterial/crossing street at the intersection
SpeedLmt	Speed limit
TravelTime	Distance from upstream intersection / speed limit
TrTimeGt40/TrTimeLt15 (Yes: 1, No: 0)	travel time from upstream is greater than 40 seconds /less than 15 seconds
LeftLane/RightLane (Yes: 1, No: 0)	existence of exclusive left/right turn lane
Transition (Yes: 1, No: 0)	Indicator. Is the current interval the coordination changing interval
CyclePerHour	number of cycles per hour
Yellow/AR/ChangeInt	duration of the yellow/all-red/the entire change interval
PR_min/_mid/_max	Platoon ratio
AT12/AT34/AT56 (Yes: 1, No: 0)	Arrival type 1, 2 or 3, 4 or 5, 6. Calculated based on PR_max
Min_R1/Min_R2/Min_G1/Min_G2 (Yes: 1, No: 0)	Is there more than 25% traffic concentrated in the period? Calculated based on min green
Mid_R1/Mid_R2/Mid_G1/Mid_G2 (Yes: 1, No: 0)	Is there more than 25% traffic concentrated in the period? Calculated based on mid green
Max_R1/Max_R2/Max_G1/Max_G2 (Yes: 1, No: 0)	Is there more than 25% traffic concentrated in the period? Calculated based on max green
max(mid/min)_0001/0010/0011 /0100/0101/0110/ 0111/1000/1001 /1010/1011/1100/1101/1110 Yes: 1, No: 0)	4-digit pattern code for max/mid/min arterial green.
G2_max/mid/min (Yes: 1, No: 0)	max_0011 or max_1001 or max_0001 is true
R2G1_max/mid/min (Yes: 1, No: 0)	max_0110 or max_0100 or max_0010 is true
R1xBOR_max	interaction of max_r1 and ArtBOR_max
G1xBOG_max	interaction of max_g1 and ArtBOG_max
VCrxArtBOR/VCrxArtBOG	interaction of VolCross and ArtBOR_max/ArtBOG_max

CHAPTER 6. CRASH LIKELIHOOD AND SEVERITY MODELS

6.1. Overview of Models

6.1.1. General Model Structure

The objective of this research is to incorporate safety considerations into arterial signal coordination timing design. Arterial signal timing directly affects the traffic arrival patterns at each approach of an intersection. The traffic patterns then affect the crash risk levels. Therefore, the central task of this research was to identify the traffic characteristics which significantly impact safety and which can be controlled with coordination settings.

Traffic characteristics that impact vehicle safety can change considerably within the time window of interest. For example, traffic engineers typically use several different coordination plans a day. Traffic arrival patterns affected by these plans will be different in deferent periods of a day. Furthermore, traffic volumes that strongly affect traffic arrival patterns change during a day due to sometimes pronounced peak periods. These essential for safety impact variations of traffic characteristics would be lost if the developed safety models use an aggregate traffic representation through the Annual Average Daily Traffic. Therefore, the traffic should be represented with traffic rates applied to sufficiently short time intervals. Fifteen-minute intervals are proposed for three reasons: (1) It is a sufficiently short interval during which the traffic rate may be assumed constant, (2) Available traffic volume data were presented in this interval, and (3) The interval is sufficiently long to reduce the impact of random fluctuation of traffic when a single traffic count is used as an estimation of the expected value.

Thus, the crash frequency would be predicted in 15-minute intervals. In fact, this frequency can be approximated with the likelihood of a crash occurring during the 15-

minute interval due to the negligible likelihood of two crashes in a single interval and at the same approach to an intersection. The number of crashes expected in a longer period such as a year can be obtained by simply aggregating the predicted likelihoods of crashes of all the intervals during that period. The aggregation can also take place over space by adding up all the expected number of crashes at all approaches of a studied arterial to obtain the expected annual number of crashes on this arterial street. Typically, the aggregation should be made for each coordination plan separately as these plans are designed independently from each other. Suppose that the predicted likelihood of rear-end crash for interval i and intersection approach j is p_{ij}^r , then the predicted number of rear-end crashes of the arterial system will be

$$Y_{RE} = \sum_{i \in I} \sum_{j \in J} p_{ij}^r \quad (\text{Eq. 6.1})$$

Where I is the set of all intervals during a certain coordination period with a single coordination plan under consideration, and J is the set of all arterial approaches to the intersections included in the studied system.

The benefits of such a choice of response variable are apparent. With these disaggregate predictions, an engineer can freely choose the temporal and spatial aggregation ranges to identify potentially problematic locations or time windows. For example, an engineer can obtain the predicted number of crashes of every eastbound approach during the morning peak hour. An approach may have a significantly higher risk of rear-end crash in this time window, which is information that can be lost in the aggregated models.

In optimizing arterial signal coordination, typically 15-minute traffic counts are the finest available data. Therefore, each 15-minute interval at each approach is considered as an observation in the model. Assuming that the likelihood of multiple crashes is negligible, for each observation, the outcome is one of the following scenarios: no rear-end or right-angle crash, rear-end crash (RE), and right-angle crash (RA). Furthermore, if severity is taken into account, the outcomes will be: no RA or RE crash, rear-end property-damage-

only crash (RE-PDO), rear-end injury or fatal crash (RE-IF), right-angle property-damage-only crash (RA-PDO), and right-angle injury and fatal crash (RA-IF). The outcome structure is illustrated in Figure 6-1.

Figure 6-1 Non-nested outcome structure

Naturally, the discrete outcome models are the proper choice in this study. The general model framework was formulated as follows (Ben-Akiva and Lerman 1985; Greene 2000). Assume there is risk index U_i associated with outcome i and the index can be separated into deterministic part V_i and random part e_i . Then, an outcome occurs when its associated risk index value is higher than all other outcomes. In other words,

$$U_i = V_i + e_i \quad (\text{Eq. 6.2})$$

$$\begin{aligned} p_i &= \text{prob}(i) = \text{prob}\{U_i > \max(U_j) \mid j \neq i\} \\ &= \text{prob}\{e_i > \max(V_j - V_i + e_j) \mid j \neq i\} \end{aligned} \quad (\text{Eq. 6.3})$$

Assume the joint probability density function of all random terms is f , and then the probability of outcome 1 can be calculated as:

$$p_1 = \int_{-\infty}^{\infty} \int_{-\infty}^{V_1 - V_2 + e_1} \int_{-\infty}^{\infty} f(e_1, e_2, \dots, e_n) de_n \dots de_2 de_1 \quad (\text{Eq. 6.4})$$

6.1.2. Multinomial Logit Model

If all the random parts are assumed identically and independently distributed (IID) as Gumbel distribution, which is also called type-1 extreme value distribution or log Weibull distribution in the literature (Johnson and Kotz 1970), with location parameter h and scale parameter m , then it has been proven (Domencich and McFadden 1975) that the above model can be simplified as:

$$p_i = \frac{e^{mV_i}}{\sum_j e^{mV_j}} \quad (\text{Eq. 6.5})$$

The scale parameter m appears in each term and thus cannot be identified. The usual practice is to set it to be 1 for convenience. In particular, the rear-end likelihood can be represented as

$$p_{RE} = \text{prob}(\text{rearend}) = \frac{e^{V_{RE}}}{1 + e^{V_{RE}} + e^{V_{RA}}} \quad (\text{Eq. 6.6})$$

It should be noted that this common scale parameter represent the requirement of constant variance for the random parts of the risk index functions. Proper remedy procedures are necessary if this assumption is violated. Moreover, it is conventional to assume that the deterministic part of the risk index has a linear-in-parameter functional form. Let X be a vector of predictive variables and b be a vector of parameters to be estimated, then the deterministic part is $V_i = b'X_i$ and the probability estimates become

$$p_i = \text{prob}(i) = \frac{\exp(b'X_i)}{\sum_j b'X_j} \quad (\text{Eq. 6.7})$$

This model formulation is called the multinomial logit (MNL) model of discrete outcomes. Usually, an outcome is chosen as the base outcome for normalization. Specifically, if no crash scenario is chosen as the base outcome, then the likelihood of each outcome can be represented as:

$$p_{RE} = P(\text{rear-end}) = \frac{e^{b'_{RE} X_{RE}}}{1 + e^{b'_{RE} X_{RE}} + e^{b'_{RA} X_{RA}}} \quad (\text{Eq. 6.8})$$

$$p_{RA} = P(\text{right-angle}) = \frac{e^{b'_{RA} X_{RA}}}{1 + e^{b'_{RE} X_{RE}} + e^{b'_{RA} X_{RA}}} \quad (\text{Eq. 6.9})$$

$$p_{NO} = P(\text{no crash}) = \frac{1}{1 + e^{b'_{RE} X_{RE}} + e^{b'_{RA} X_{RA}}} \quad (\text{Eq. 6.10})$$

An important property of the multinomial model is the log-odd ratio relation:

$$\ln \frac{p_{RE}}{p_{NO}} = b'_{RE} X_{RE}, \quad \ln \frac{p_{RA}}{p_{NO}} = b'_{RA} X_{RA}, \quad \ln \frac{p_{RE}}{p_{RA}} = b'_{RE} X_{RE} - b'_{RA} X_{RA} \quad (\text{Eq. 6.11})$$

And the coefficient b can be interpreted as the marginal effect of the corresponding variable x on the log-odds ratio when other factors are kept as constant. For example, for a continuous factor of the rear-end likelihood $x_{i,RE}$, the coefficient $b_{i,RE}$ can be written as

$$b_{i,RE} = \frac{\partial \ln(p_{RE} / p_{NO})}{\partial x_{i,RE}} \quad (\text{Eq. 6.12})$$

For an indicator variable $x_{j,RE}$, the coefficient $b_{j,RE}$ can be written as

$$b_{j,RE} = \ln \frac{p_{RE}}{p_{NO}} \Big|_{x_{j,RE}} = \frac{\partial}{\partial x_{j,RE}} \ln \frac{p_{RE}}{p_{NO}} \Big|_{x_{j,RE}} = 0 \quad (\text{Eq. 6.13})$$

6.1.3. Conditional Logit and Ordered Outcome Models

A special case of the multinomial logit model is the conditional logit model (CL). In this model, the data supplied to the model consist of outcome-specific attributes rather than observation-specific attributes. For example, a widely cited paper (Hensher 1986)

investigated the route choice travelers. Every traveler is considered as an observation. Some predictive variables in that study, for example travel time, are specific to the outcome of a traveler's choice rather specific to a certain traveler.

In our study, however, all the variables are specific to the observation rather than to the outcome. There is no variable which only exists for a certain type of crash. This limitation can be circumvented by creating interaction variables with each outcome (Greene 2000). In fact, to estimate the nested logit model, many statistical software tools require that data be formatted as outcome-specific. As a result, in this study, each observation was expanded to a group of observations. Each observation in this group represents a possible outcome and a binary variable was created to mark the observation in each group which corresponds to the actual observed event. An illustrative example of the data conversion is shown by Table 6-1 and Table 6-2. The original observation-specific variable SpeedLmt was expanded to four interaction variables of SpeedLmt and the four crash outcomes.

Table 6-1 A sample observation in raw data format

Outcome	SpeedLmt	...
Rear-end PDO	45	...

Table 6-2 A sample observation group in expanded data format

Outcome	Occurred	SpeedLmt xREPDO	SpeedLmt xREIF	SpeedLmt xRAPDO	SpeedLmt xRAIF	...
No Crash	0	0	0	0	0	...
Rear-end PDO	1	45	0	0	0	...
Rear-end IF	0	0	45	0	0	...
Right-angle PDO	0	0	0	45	0	...
Right-angle IF	0	0	0	0	45	...

The estimates of the conditional logit model are often used as the starting values for the nested logit model to accelerate the estimation (StataCorp 2007). The likelihood-ratio

test can be performed to compare the fitness of a conditional logit model and a nested logit model since the predictive variables of these two models are only different by the inclusive value parameters. If the likelihood ratio test fails to reject that the nested logit model is superior in terms of fitness, then it means the nested structure brings little improvement and there is no adequate evidence to claim an IIA violation. The estimation results of the conditional logit model, therefore, will be provided together with the nested logit model when necessary in the following sections.

When modeling the severity of vehicle crash, the ordered outcome models, such as the ordered logit model (Wang and Kockelman 2005) and the ordered probit model (Abdel-Aty 2003; Kockelman and Kweon 2002), are sometimes preferred because these models recognize the ordinal nature of the vehicle crashes. In the setting of this study, when modeling the right-angle crashes, the outcomes have a natural order in terms of severity. For example, the formulation of the ordered probit model assumes that there is an unobserved risk index U which has the form $U = b'X + e$ where e is a normally distributed random term. Then the probabilities of the outcomes are

$$\begin{aligned} p(\text{no crash}) &= 1 - F(b'X) \\ p(\text{rear-end PDO}) &= F(m - b'X) - F(-b'X) \\ p(\text{rear-end IF}) &= 1 - F(m - b'X) \end{aligned} \quad (\text{Eq. 6.14})$$

where b and the threshold m are parameters to be estimated.

The ordered models are potentially more efficient than the unordered models because they enforce an ordered structure of the outcome. The disadvantage, on the other hand, is the difficulty in interpreting the estimation results. The signs of the coefficient show the impact of the factor on the extreme categories but have no clear direction of impact on the intermediate category (Washington et al. 2003). In other words, with the ordered model, it may be found that a factor will reduce the likelihood of a severe crash and increase the likelihood of no crash, but it may remain unclear how this factor affects the likelihood of property damage only crashes. The estimation results of the ordered models will be provided as a candidate model.

6.1.4. Data Sampling

A practical difficulty of this research was the tremendous size of the dataset. The complete dataset includes 202,981 no crash observations, 356 rear-end observations, and 67 right-angle observations. The estimation process became a formidable computational task given the current available computing capacity. A similar situation is described in a paper (King and Zeng 2001) where the authors argued that each positive observation carries much more information than the null observations. A sampling strategy is proposed to keep all the crash observations and randomly sampled no crash observations. This strategy is a special case of the choice-based random sampling strategy (Ben-Akiva and Lerman 1985). It can be prove that the maximum likelihood estimators of such a sample are still consistent except for the choice-specific constant terms. In the multinomial logit model setting, the correction for the constant terms should be (Washington et al. 2003):

$$- \ln\left(\frac{SF_i}{PF_i}\right) \quad (\text{Eq. 6.15})$$

where SF_i is the fraction of observations with outcome i in the sub-sample and PF_i is the fraction of observations with outcome i in the entire population. For example, the total number of observation in this study is 203,404 and the one percent of no crash sample has 2,435 observations. In both samples, 286 rear-end observations and 67 right-angle observations were kept. As a result, when using the one-percent sample in estimating the model, the constant terms should be corrected by

$$- \ln\left(\frac{SF_i}{PF_i}\right) = - \ln \frac{e^{286 / 2435}}{e^{286 / 203404}} - \ln \frac{e^{67 / 2435}}{e^{67 / 203404}} = - 4.43 \quad (\text{Eq. 6.16})$$

6.1.5. Independence of Irrelevant Alternatives (IIA) Violation

A limitation of the multinomial logit model is the IID assumption on the random part of the risk index function. In terms of model interpretation, this leads to the well-known independence of irrelevant alternatives (IIA) property of the model (Washington et al.

2003). The IIA property can be observed from the log-ratio relation introduced above. In short, the IIA property requires that the ratio of the two outcomes' probabilities is independent from the deterministic part of the other outcomes' risk indices. When applying the framework in modeling human choice, the IIA property basically requires that the addition or removal of an alternative does not affect the relative preference level of other alternatives. The issue of IIA is often illustrated by conceptual examples (Ben-Akiva and Lerman 1985), including the Red Bus and Blue Bus paradox. IIA violations are also often observed in actual modeling. The following illustrative example was given in a recent paper based on actual data (Kedar 2005; Kropko 2008). In 2000 presidential election, a liberal voter prefers Nader over Gore and prefers Gore over Bush. If Mr. Bush is not one of the candidates, this voter is going to vote for Nader. The IIA property requires that the addition of the third alternative Mr. Bush has no impact on the voting behavior of this voter. However, it was found that many such voters strategically voted for Mr. Gore because they recognized that Mr. Nader's chances were slim and their votes may results in the election of Mr. Bush.

The violation of the IIA property results in inconsistent estimates of the model parameters and, more importantly, inconsistent prediction of outcome probabilities. Numerous statistical tests are available for this assumption (Hausman and McFadden 1984; Small and Hsiao 1985). A summary and comparison of these tests are available in a paper (Zhang and Hoffman 1993). The authors recommend to the Small-Hsiao test (Small and Hsiao 1985) which is based on the idea that when IIA holds, the parameter estimates should be similar if an irrelevant alternative is removed from the data samples. This test is one of the likelihood ratio tests. Washington et al. (2003) has a succinct description of the test procedure.

6.1.6. Remedies of IIA Violation

There are various ways to deal with IIA violations as documented in Kennedy (2003). Three methods that are potentially helpful in this research are as follows.

First, a better and more comprehensive specification of the deterministic term can often help. A good specification of the risk index may help to limit the IIA effects. This method is the most common practice.

Second, the multinomial probit model (MNP) can be used to circumvent the IIA issue (Alvarez and Nagler 2000). This model assumes that the random parts of the risk index are distributed as a multivariate normal vector. The elements of the random vector can be correlated. The correlation structure of the random terms should be estimated together with the coefficients of the deterministic terms. Due to this flexibility of the multinomial probit model, it is preferred by researchers in modeling events with three or more outcomes (Alvarez et al. 2000).

The major drawback of this method is the computational difficulty. Unlike the multinomial logit model, the estimation of the multinomial probit model involves the evaluation of high-dimensional Gaussian probability density functions. Conventional maximum likelihood estimation will fail in such cases and the simulated maximum likelihood estimation technique is often employed (Train 2003). A widely implemented estimating algorithm is the GHK estimator (Geweke 1991; Hajivassiliou et al. 1996; Keane 1994; StataCorp 2007; Venables and Ripley 2002).

However, recently some researchers argue that the inaccuracy of the estimation method offsets the benefits of relaxing the IIA assumption (Dow and Endersby 2004; Kropko 2008). They directly compared the performance of multinomial logit model versus the multinomial probit model and concluded that the multinomial logit model is superior except for the extreme cases when several alternatives have an extremely high level of correlation.

Therefore, given the current computing technique level, the multinomial probit model still cannot be adopted as the major tool when IIA presents. In addition, it is impractical to assume that traffic engineers will have the computing power for evaluating high-dimensional integrals of multivariate Gaussian density functions. As a result, in this research, multinomial probit models are only fitted for comparison purposes.

The third remedy method is the nested logit model. It is explained in the next section.

6.1.7. Nested Logit Model

Another popular remedy method is the nested logit model. It was derived as a special case of the generalized extreme value (GEV) model (Manski and McFadden 1981). In this model, to accommodate the correlation of the disturbance terms, correlated alternatives are grouped into nests. The alternatives in the same nests may share some unobservable effects. In this study, however, the nested logit model may cause difficulty in interpreting the modeling results, which is discussed in the next section.

In this study, for example, property damage only crashes and the injury fatal crashes share a significant proportion of the unobserved effect. Therefore, nested logit models may be considered in modeling the crash outcomes when severity difference is considered. Conceptually, the outcomes are grouped as in Figure 6-2.

Figure 6-2 Nested structure of crash outcomes

The formulation of the nested logit model can be illustrated with the rear-end severity models. The risk index function is specified as

$$\begin{aligned} U_{\text{RePDO}} &= X_{\text{RePDO}}' b_1 + Z' g_{\text{RePDO}} + e_{\text{RePDO}} \\ U_{\text{ReIF}} &= X_{\text{ReIF}}' b_2 + Z' g_{\text{ReIF}} + e_{\text{ReIF}} \end{aligned} \quad (\text{Eq. 6.17})$$

where X_{RePDO} and X_{ReIF} are variables specific to the alternatives, and Z are the variables specific to the observations (an interval-approach combination). The conditional probability

$$\begin{aligned} P(\text{RePDO}|\text{RE}) &= \frac{\exp\left(\frac{1}{t_{\text{RE}}} V_{\text{RePDO}}\right)}{\exp\left(\frac{1}{t_{\text{RE}}} V_{\text{RePDO}}\right) + \exp\left(\frac{1}{t_{\text{RE}}} V_{\text{ReIF}}\right)} \\ P(\text{ReIF}|\text{RE}) &= \frac{\exp\left(\frac{1}{t_{\text{RE}}} V_{\text{ReIF}}\right)}{\exp\left(\frac{1}{t_{\text{RE}}} V_{\text{RePDO}}\right) + \exp\left(\frac{1}{t_{\text{RE}}} V_{\text{ReIF}}\right)} \end{aligned} \quad (\text{Eq. 6.18})$$

where V_{RePDO} and V_{ReIF} are the deterministic terms of the risk index function and the inclusive value IV_{RE} is defined as

$$IV_{\text{RE}} = \ln \left(\exp\left(\frac{1}{t_{\text{RE}}} V_{\text{RePDO}}\right) + \exp\left(\frac{1}{t_{\text{RE}}} V_{\text{ReIF}}\right) \right)^{\frac{1}{t_{\text{RE}}}} \quad (\text{Eq. 6.19})$$

In addition, the probability of falling in the nest of rear-end crash is

$$P(\text{RE}) = \frac{\exp\left(\frac{1}{t_{\text{RE}}} IV_{\text{RE}}\right)}{\exp\left(\frac{1}{t_{\text{RE}}} IV_{\text{RE}}\right) + \exp\left(\frac{1}{t_{\text{NO}}} IV_{\text{NO}}\right)} \quad (\text{Eq. 6.20})$$

Since the nest of no crash has only one sub-branch and its risk index is normalized to be 0, the above probability can be further simplified as

$$P(\text{RE}) = \frac{\exp\left(\frac{1}{t_{\text{RE}}} IV_{\text{RE}}\right)}{\exp\left(\frac{1}{t_{\text{RE}}} IV_{\text{RE}}\right) + 1} \quad (\text{Eq. 6.21})$$

As a result, the probabilities of rear-end PDO and rear-end IF crashes are

$$\begin{aligned}
 P(\text{RePDO}) &= P(\text{RePDO}|\text{RE})P(\text{RE}) = \frac{\exp\left(\frac{V_{\text{RePDO}}}{t_{\text{RE}}}\right) \exp\left(\frac{IV_{\text{RE}}}{t_{\text{RE}}}\right)}{\exp\left(\frac{V_{\text{RE}}}{t_{\text{RE}}}\right) \exp\left(\frac{IV_{\text{RE}}}{t_{\text{RE}}}\right) + 1} \\
 P(\text{ReIF}|\text{RE}) &= P(\text{ReIF}|\text{RE})P(\text{RE}) = \frac{\exp\left(\frac{V_{\text{ReIF}}}{t_{\text{RE}}}\right) \exp\left(\frac{IV_{\text{RE}}}{t_{\text{RE}}}\right)}{\exp\left(\frac{V_{\text{RE}}}{t_{\text{RE}}}\right) \exp\left(\frac{IV_{\text{RE}}}{t_{\text{RE}}}\right) + 1}
 \end{aligned} \tag{Eq. 6.22}$$

The parameters t_{NO} and t_{RE} are called the inclusive value or dissimilarity parameters.

The Daly-Zachary-McFadden condition requires the inclusive value parameter to be within the unit interval for the model to be consistent with the random utility maximization framework (Daly and Zachary 1979; McFadden 1978). This condition, however, has been prove to be sufficient but not necessary (Borsch-Supan 1987) for local consistency. In many practical models this condition is too strong and can be relaxed. The necessary conditions for local consistency have also been found for two-level nested logit models (Herriges and Kling 1996) and three-level nested logit models (Gil-Molto and Hole 2004).

It should be noted that in the econometrics literature, most notably the classic reference (Greene 2000), the deterministic term of the risk index function V is replaced with a scaled value $\tilde{V}$ (Heiss 2002; Silberhorn et al. 2006)

$$\tilde{V}_{\text{RePDO}} = \frac{V_{\text{RePDO}}}{t_{\text{RE}}} \tag{Eq. 6.23}$$

This alternative nested logit model is often cited as a non-normalized nested logit model (NNNL). As a result of this scaling transformation, this model becomes inconsistent with the random utility model and thus the inclusive value parameters have different meanings. This seemingly minor scaling significantly affects the interpretation of the b estimates as well. Nevertheless, the available econometric software tools, including

Limdep 7 (Greene 1995), SAS 9 (SAS Institute 1996), and STATA 9 (StataCorp 2007) (the version available during this research), only provide the estimation of a non-normalized nested logit model. A method is available to manipulate the NNNL software to estimate the random utility model (Koppelman and Wen 1998) by introducing dummy nests for each outcome. For an extensive discussion of this estimation issue, please refer to the paper (Heiss 2002). It should be noted that the interpretation of the estimated coefficients have a very different meaning in NNML formulation compared with the typical econometric literature.

The nested logit model can be estimated with the limited information method (LIML) or the full information maximum likelihood method (FIML) (Hensher 1986). The former estimates the model sequentially in two steps and is computational much less demanding. Nevertheless, the asymptotic variance has to be corrected. Therefore, the FIML is preferred when computing power is not a limiting factor due to the efficiency of the estimator (Greene 2000). Most software tools offer the option to do LIML or FIML for nested logit models.

6.1.8. Sequential Logit Model

In modeling discrete outcomes, the sequential logit model (Amemiya 1985; Liao 1994) is often used when the event process can be separated into several independent stages. A multinomial logit model is then used to model the outcome at each stage. It has been shown that the sequential logit model (SL) is a limiting case of the nested logit model and statistical tests are available to compare these two models (Nagakura and Kobayashi 2007; Ophem and Schram 1997).

In this study, crash occurrence can be viewed as a multi-stage process, especially when severity levels are taken into account. At the first stage, a multinomial logit model is used to model the likelihood of each type of crash; at the second stage, another multinomial logit model is used to predict the severity level of a reported crash. Given the deterministic term of all risk index functions, the probabilities of two types of crashes are

$$p_{RE} = \frac{e^{V_{RE}}}{1 + e^{V_{RE}} + e^{V_{RA}}}, p_{RA} = \frac{e^{V_{RA}}}{1 + e^{V_{RE}} + e^{V_{RA}}} \quad (\text{Eq. 6.24})$$

and the conditional likelihoods of injury fatal crashes are

$$p_{IF|RE} = \frac{e^{V_{IF|RE}}}{1 + e^{V_{IF|RE}}}, p_{IF|RA} = \frac{e^{V_{IF|RA}}}{1 + e^{V_{IF|RA}}} \quad (\text{Eq. 6.25})$$

As a result, the probabilities of RE-PDO, RE-IF, RA-PDO, and RA-IF crashes are

$$\begin{aligned} p_{REPDO} &= p_{RE} p_{PDO|RE} = \frac{e^{V_{RE}}}{1 + e^{V_{RE}} + e^{V_{RA}}} \frac{1}{1 + e^{V_{IF|RE}}} \\ p_{REIF} &= p_{RE} p_{IF|RE} = \frac{e^{V_{RE}}}{1 + e^{V_{RE}} + e^{V_{RA}}} \frac{e^{V_{IF|RE}}}{1 + e^{V_{IF|RE}}} \\ p_{RAPDO} &= p_{RA} p_{PDO|RA} = \frac{e^{V_{RA}}}{1 + e^{V_{RE}} + e^{V_{RA}}} \frac{1}{1 + e^{V_{IF|RA}}} \\ p_{RAIF} &= p_{RA} p_{IF|RA} = \frac{e^{V_{RA}}}{1 + e^{V_{RE}} + e^{V_{RA}}} \frac{e^{V_{IF|RA}}}{1 + e^{V_{IF|RA}}} \end{aligned} \quad (\text{Eq. 6.26})$$

This characterization of the crash occurrence process has an advantage over the nested logit model mainly in terms of model interpretation. Intuitively, the crash frequency and severity are determined independently. First a crash occurs and then the severity level is determined. The independence of the crash frequency and the severity process is enforced by the formulation of the model.

On the other hand, for the nested logit model, the bottom level outcome factors affect the odd ratio of the upper level nests. The difference between the crash frequency and severity factors is thus obscured and often leads to confusion. For example, a factor may cause the severity of a rear-end crash to increase. As a consequence of the nested logit model structure, this will increase the likelihood of a rear-end crash, other factors being constant.

Therefore, the sequential logit model has more intuitive interpretation and naturally separates the crash frequency and severity factors into two models. It should be noted that in crash data, there may exist a correlation in crash frequency and severity because severe crashes are more likely to be reported (Hauer 2006). For instance, senior drivers are involved in a high proportion of crashes. It may be because the senior drivers are more prone to injury and most injury crashes are reported. Therefore, a driver's age, a severity factor, actually affects the frequency of reported crashes. In interpreting the findings of the sequential logit model, this frequency-severity indeterminacy should be carefully considered.

6.1.9. Model Selection Criteria

In the following sections, for the same dataset, different models' fitted results will be shown. Aside from the researcher's judgment, some statistical criteria are also to be used. These criteria are briefly introduced in this section. Similar to the R^2 goodness-of-fit criterion (Neter et al. 1996) in linear regression, McFadden proposed a pseudo R^2 for the maximum likelihood estimators

$$\text{pseudo } R^2 = 1 - \frac{\text{loglikelihood}(\text{Model})}{\text{loglikelihood}(\text{Intercept})} \quad (\text{Eq. 6.27})$$

This criterion has a similar interpretation as the linear regression. A value close to 1 is preferred. To adjust for the number of variables, the pseudo R^2 can be adjusted as

$$\text{adj. pseudo } R^2 = 1 - \frac{\text{loglikelihood}(\text{Model}) - K}{\text{loglikelihood}(\text{Intercept})} \quad (\text{Eq. 6.28})$$

where K is the number of independent variables used in the model. Another criterion often used in comparing non-nested models is Akaike Information Criterion (AIC) (Akaike 1973; Stone 1977)

$$AIC = 2K - 2\text{loglikelihood}(\text{Model}) \quad (\text{Eq. 6.29})$$

AIC is based on the theory of entropy and is not meant to be used as a hypothesis testing tool but rather a model selection tool. *AIC* is asymptotically equivalent to a cross-validation criterion when the sample size approaches infinity (Stone 1977). Some statistical software offers the function of automatic stepwise model selection based on this criterion (Hastie and Pregibon 1992; Venables and Ripley 2002). The implication of using stepwise model selection based on *AIC* is discussed in Shtatland et al. (2001). The implied significance level can be as high as 15% when sample size is limited (Atkinson 1981).

A similar model selection criterion Bayesian Information Criterion (*BIC*), also called Schwarz Information Criterion (*SIC*), is defined as

$$BIC = K \ln(n) - 2 \log \text{likelihood}(\text{Model}) \quad (\text{Eq. 6.30})$$

where n is the number of observations (Schwarz 1978). As is clearly seen, *BIC* penalizes the extra number of variables more than the *AIC*. Smaller values of *AIC* and *BIC* are preferred.

6.2. Model of Rear-end and Right-angle Crash Likelihoods

6.2.1. Multinomial Logit Model

As illustrated in CHAPTER 3, the most frequent types of vehicle crashes at signalized intersections are rear-end and right-angle crashes. The foremost model is used to identify influential factors that are correlated with the occurrence of these two types of crashes.

When severity is not of concern, then each observation can have three possible outcomes in this study: no rear-end or right-angle crash, rear-end, and right-angle, ignoring the likelihood of multiple crashes in a short 15-minuter interval. In other words, rather than consider five outcomes as illustrated in Figure 6-2, a model is to be developed according to the outcome structure depicted in Figure 6-3 at this stage.

As introduced before, the multinomial logit model is a good starting model to consider in such a scenario. There is no natural order of these three outcomes. The ordered logit/probit model does not have any efficiency advantage and is thus not considered.

A variety of independent variables and their transformations are considered. The description of these variables can be found on page 65.

Table 6-3 lists the final fitted statistics of the multinomial model. Since the evaluation of the likelihood functions of the multinomial logit model requires low computing power, the full sample is used in estimating the model coefficients. If the factors are allowed to be present in only one type of risk index function, then we can drop the insignificant terms for one crash type. The model estimations after dropping those terms are also shown in Table 6-3.

As a comparison, the model is also estimated based on a one-percent sample. The sample consists of all the crash intervals and a randomly sampled one percent of the no crash intervals. As described in Section 6.1.4, the estimates are still consistent when such a choice-based sampling strategy is used. Only the constant terms needed to be corrected. This theoretical result is corroborated by the coefficient estimates listed in Table 6-5. All coefficients have the same signs, and the estimates based on the full sample have higher significance level, as expected. However, the significant levels in the full sample model and the one-percent sample can be very different for some factors. For example, the one-percent sample fails to detect the effect of Winter and AM. The overall goodness of fit statistics are provided in Table 6-4.

As elaborated in Section 6.1.5, the independence of irrelevant alternative assumption is the inevitable result enforced by the IID assumption on the random term of the risk index function. The violation of this assumption has a profound impact on the parameter estimates. Therefore, to guarantee the validity of the fitted multinomial logit model, several statistical tests were conducted on both the full sample and the one percent subset of the data.

The Hausman Test (Hausman and McFadden 1984) is one of the first statistical tests for the IIA assumption. Later, it was found that the violation detection capability of this test is not satisfactory. The Small-Hsiao test, recommended by Zhang and Hoffman (1993), produces more meaningful results. Table 6-6 summarizes the testing results. The tests failed to reject the null hypothesis of IIA for both the full and one-percent subset of dataset. However, for the one percent sample, when rear-end outcomes were omitted, there is marginal evidence that the right-angle estimates are different. This was not verified by the tests based on the full sample and thus was not investigated further.

Figure 6-3 Crash outcomes structure

Table 6-3 Multinomial logit model of rear-end and right-angle crashes

Crash Type	Variable	Full Model			Insignificant terms dropped		
		Coef.	Std. Err.	t-stat	Coef.	Std. Err.	t-stat
Rear-end	SR135	-2.53	0.345	-7.34	-2.53	0.345	-7.34
	SR431	0.864	0.359	2.41	0.864	0.359	2.41
	R1xBOR_max	0.0280	0.00813	3.45	0.0280	0.00813	3.45
	ArtBOG_max	0.0384	0.0110	3.50	0.0384	0.0110	3.50
	Winter	-0.290	0.121	-2.41	-0.290	0.121	-2.41
	AM	-0.291	0.122	-2.39	-0.291	0.122	-2.39
	RightLane	-2.75	0.321	-8.56	-2.75	0.321	-8.56
	SpeedLmt	0.158	0.0305	5.17	0.158	0.0305	5.17
	TrTimeLt15	-0.764	0.202	-3.78	-0.764	0.202	-3.78
	TrTimeGt40	0.861	0.163	5.27	0.861	0.163	5.27
	Max_g2	-0.215	0.122	-1.77	-0.215	0.122	-1.77
	CyclePerHour	-0.0444	0.0157	-2.82	-0.0444	0.0157	-2.82
	Intercept	-11.21	1.61	-6.94	-11.21	1.61	-6.94
Right-angle	SR135	-0.850	0.517	-1.64	-0.831	0.407	-2.04
	SR431	-1.36	0.552	-2.47	-1.26	0.460	-2.74
	R1xBOR_max	0.0346	0.0199	1.73	0.0334	0.0197	1.69
	ArtBOG_max	0.0385	0.0268	1.44	0.0370	0.0258	1.44
	Winter	0.161	0.259	0.62	(dropped)		
	AM	-0.525	0.288	-1.82	-0.559	0.279	-2.00
	RightLane	-1.10	0.341	-3.24	-1.15	0.339	-3.38
	SpeedLmt	0.149	0.0499	2.99	0.159	0.0394	4.05
	TrTimeLt15	-0.638	0.383	-1.67	-0.668	0.381	-1.75
	TrTimeGt40	0.253	0.404	0.63	(dropped)		
	Max_g2	-0.469	0.301	-1.56	-0.471	0.301	-1.57
	CyclePerHour	-0.0131	0.0291	-0.45	(dropped)		
	Intercept	-13.30	2.72	-4.89	-14.00	1.82	-7.70

Table 6-4 Model fitness statistics of the multinomial logit model

Statistics	Full Sample	Full Sample Insignificant terms dropped	One Percent Sample
Number of observations	203404	203404	2449
log likelihood (null)	-3219.68	-3219.68	-1311.83
log likelihood (Model)	-2913.63	-2914.16	-1046.79
Degree of freedom	26	23	23
Pseudo R-square	0.0951	0.0949	0.202
AIC	5879.25	5874.32	2139.59
BIC	6145.05	6109.45	2273.06

Table 6-5 MNL model of rear-end and right-angle crashes (1% sample)

Crash Type	Variable	Coef.	Std. Err.	t-stat	95% Conf. Interval	
Rear-end	SR135	-2.79	0.373	-7.48	-3.52	-2.06
	SR431	0.828	0.376	2.20	0.0906	1.57
	R1xBOR_max	0.0346	0.0107	3.24	0.0136	0.0555
	ArtBOG_max	0.0570	0.0138	4.15	0.0301	0.0840
	Winter	-0.340	0.143	-2.38	-0.620	-0.0597
	AM	-0.334	0.142	-2.35	-0.613	-0.0551
	RightLane	-2.86	0.338	-8.46	-3.52	-2.20
	SpeedLmt	0.158	0.0319	4.94	0.0951	0.220
	TrTimeLt15	-0.811	0.234	-3.47	-1.27	-0.353
	TrTimeGt40	0.796	0.185	4.29	0.433	1.16
	Max_g2	-0.165	0.145	-1.14	-0.450	0.119
	CyclePerHour	-0.0489	0.0172	-2.85	-0.08	-0.02
	Intercept	-6.51	1.71	-3.81	-9.86	-3.16
Right-angle	SR135	-0.930	0.546	-1.70	-2.00	0.140
	SR431	-1.32	0.557	-2.37	-2.41	-0.231
	R1xBOR_max	0.0443	0.0217	2.04	0.00184	0.0868
	ArtBOG_max	0.0498	0.0276	1.80	0.00433	0.104
	Winter	0.158	0.269	0.590	-0.369	0.685
	AM	-0.502	0.292	-1.72	-1.08	0.0708
	RightLane	-1.13	0.355	-3.18	-1.83	-0.433
	SpeedLmt	0.138	0.0508	2.71	0.0383	0.237
	TrTimeLt15	-0.679	0.398	-1.71	-1.46	0.101
	TrTimeGt40	0.172	0.413	0.42	-0.636	0.981
	Max_g2	-0.410	0.312	-1.31	-1.02	0.201
	CyclePerHour	-0.0222	0.0302	-0.740	-0.0815	0.0370
	Intercept	-8.02	2.78	-2.89	-13.47	-2.58

Table 6-6 Small-Hsiao test of IIA assumption

Sample	Small-Hsiao Test (H_0 : IIA)				
	Omitted Outcome	Chi-square	d.f.	p-value	Conclusion
Full Sample (insignificant terms dropped)	Rear-end	9.717	13	0.717	Cannot reject H_0
	Right-angle	6.226	13	0.938	Cannot reject H_0
One Percent Sample	Rear-end	14.455	13	0.343	Cannot reject H_0
	Right-angle	17.56	13	0.175	Cannot reject H_0

Therefore, a satisfactory multinomial logit model was built to model the occurrence likelihood of the outcome set specified in Figure 6-3. There is no evidence to claim that the unobserved risk factors of rear-end and right-angle crashes are correlated when the predictive variables in Table 6-3 are included in the model. In other words, the common risk factors for these two types of crashes are well captured in the deterministic part of the specified form of risk index functions.

Interpretation of the model will be discussed in detail in **Error! Reference source not found..** At this point, some brief results of the model are summarized. The elasticity estimates of the listed variables are tabulated in Table 6-7 and are estimated based on the model fitted with the full sample and without the insignificant crash type-specific terms.

The impact of the traffic pattern on crash likelihood was clearly shown by the elasticity of the variable Max_g2. Traffic arrival patterns fall into this category when vehicles arrivals are concentrated in the second part of the arterial green. When such patterns are present, the likelihoods of rear-end and right-angle crashes are lowered to 81% and 62% levels compared with other patterns, other factors being equal. In other words, the risk reduction factor of Max_g2 is 19% for the rear-end crash and 38% for the right-angle crashes.

Table 6-7 Indicator variables' elasticity of multinomial logit crash model

Indicator Variable 0→1	$\frac{p(\text{rear-end})}{p(\text{no crash})}$			$\frac{p(\text{right-angle})}{p(\text{no crash})}$		
	b	t-stat	Elasticity	b	t-stat	Elasticity
Max_g2	-0.215	-1.77	0.806	-0.471	-1.57	0.624
Winter	-0.290	-2.41	0.748	/	/	/
AM	-0.291	-2.39	0.747	-0.559	-2.00	0.572
RightLane	-2.748	-8.56	0.064	-1.145	-3.38	0.318
TrTimeLt15	-0.764	-3.78	0.466	-0.668	-1.75	0.513
TrTimeGt40	0.861	5.27	2.365	/	/	/

Another finding is that a closely located intersection, as indicated by the variable TrTimeLt15, has significantly lower crash risks. This may be the result of better view for the drivers. When approaching a close downstream intersection, a driver generally has a good view of the queue condition at the approach and thus is better prepared to either stop or go. Furthermore, there is a smaller chance for a change of the signal status at the intersection since the travel time is shorter. The driver has less of a chance for an unexpected and abrupt change of maneuver. When the travel time from the upstream intersection is longer than 40 seconds (TrTimeGt40), on the other hand, the likelihood of the rear-end crashes is significantly higher.

The exclusive right-turn lane is associated with significantly lower likelihood of both rear-end and right-angle crashes. The impact level of this factor is surprisingly high. It reduces both the likelihood of rear-end and right-angle crashes to around 5% as before. In other words, the risk reduction factor of the existence of a right turn lane is greater than 95%. It should be noted that the arterial effects are already controlled. The mechanism of a right-turn lane for reducing rear-end crashes is obvious. If there is no exclusive right turn lane, the right-turning vehicle slows down when approaching the intersection. This causes extra disturbance to the traffic flow and forces the following vehicles to change maneuver in a short time. Particularly, this scenario is highly risky if a high density vehicle platoon is approaching an intersection when the green signal is shown. If an exclusive right turn lane is present, the right-turning vehicles will change to

the lane when approaching the intersection. The following vehicles pass through the intersection with much less chance of being forced to slow down abruptly.

The mechanism of the right-turn lane on right-angle crashes can be partially explained by better assessment of the conflicting traffic flows. Vehicles staying in the through lane will proceed through the intersection. Vehicles coming from a conflicting movement, for example, right-turning vehicles from crossing streets, have a reliable way to judge the behavior of the vehicle. The through vehicles will not be misunderstood as turning vehicles and thus reduce the risk of right-angle crashes. However, these explanations seem to be inadequate. Future studies should investigate this issue further.

6.2.2. Predictive Capability of the Model

A way of verifying the fitness of the model is by comparing the predicted crash likelihoods of the no crash intervals and the crash intervals. As illustrated in Figure 6-4 and Figure 6-5, the intervals in which crashes occurred do have a higher level of predicted crash probabilities. Most crash intervals are predicted to have crash likelihoods higher than majority of non-crash intervals. Figure 6-6 and Figure 6-7 further break down the observations by year, and the trend is still consistent. In each year the actual crash intervals are predicted to be more dangerous. Figure 6-8 and Figure 6-9 break down the observations by arterial systems. Almost all of them possess the same trend which confirms the fitness of the model. It can be seen from the figures that for some arterial systems, the model does not work as well to predict the crash likelihoods.

Figure 6-4 Histograms of predicted rear-end probability

Figure 6-5 Histograms of predicted right-angle probability

Figure 6-6 Histograms of predicted rear-end likelihood by year

Figure 6-7 Histograms of predicted right-angle likelihood by year

Figure 6-8 Histograms of predicted rear-end likelihood by arterial

Figure 6-9 Histograms of predicted right-angle likelihood by arterial

6.2.3. Alternative Multinomial Logit Model

As Table 6-8 shows, the only volume-related predictive variables are R1xBOR_max and ArtBOG_max which are the flow rates (vphpl) of volume arriving right after the phase changes. It suggests that these vehicles are most susceptible to rear-end and right-angle crash risks. The total volume rates, represented by UpPrimVol and UpSecVol, which are the primary and secondary volume rates from upstream intersections, are not statistically significant in the model present in Table 6-8. This does not mean that the vehicles arriving during other time periods are completely immune from rear-end and right-angle crash risks. Statistically speaking, it means that there is not enough evidence

to claim that the total volume rates provide much more information after the R1xBOR_max and ArtBOG_max are included.

Table 6-8 Multinomial logit crash model without volume

Variable	Rear-end			Right-angle		
	Coef.	Std. Err.	t-stat	Coef.	Std. Err.	t-stat
SR135	-2.53	0.345	-7.34	-0.831	0.407	-2.04
SR431	0.864	0.359	2.41	-1.261	0.460	-2.74
R1xBOR_max	0.0280	0.00813	3.45	0.0334	0.0197	1.69
ArtBOG_max	0.0384	0.0110	3.50	0.0370	0.0258	1.44
Winter	-0.290	0.121	-2.41	(dropped)		
AM	-0.291	0.122	-2.39	-0.559	0.279	-0.559
RightLane	-2.75	0.321	-8.56	-1.15	0.339	-1.145
SpeedLmt	0.158	0.0305	5.17	0.159	0.0394	0.159
TrTimeLt15	-0.764	0.202	-3.78	-0.668	0.381	-0.668
TrTimeGt40	0.861	0.163	5.27	(dropped)		
Max_g2	-0.215	0.122	-1.77	-0.471	0.301	-1.57
CyclePerHour	-0.0444	0.0157	-2.82	(dropped)		
Intercept	-11.2	1.61	-6.94	-14.00	1.82	-7.70

Although the total volume rates provide little additional information after the two volume-related variables are added to the model, traffic engineers may want to have included the total volume as a more intuitive measure of risk exposure. Therefore, a model with the upstream primary volume rate is provided in Table 6-9 as an alternative model. Once the total volume is added the other two variables significant in the original model lose their significance. In the matter of fact, the alternative model's performance is only slightly worse than the original one.

Several comments can be made regarding the coefficients of the two models. First, the upstream primary volume rate affects the crash likelihood of rear-end crashes (although marginally from the statistical point of view). It is dropped in the right-angle crash model because the included traffic rate near the change periods is more adequate and consistent with the crash mechanism. Right-angle crashes occur when a vehicle violates red signal and this happens typically in the first seconds of red while red running far from the change interval is very rare. Second, the coefficients of volume-related variables are not much affected by other variables in the model. At the same time, the dependence

among the three volume-related variables (VolTotal, R1xBOR_max and ArtBOG_max) is strong as expected. This behavior of the variable coefficients shows the robustness of the model.

Table 6-9 Multinomial logit crash model with total approach volume

Variable	Rear-end			Right-angle		
	Coef.	Std. Err.	t-stat	Coef.	Std. Err.	t-stat
SR135	-2.56	0.355	-7.22	-0.796	0.398	-2.00
SR431	0.934	0.360	2.59	-1.24	0.448	-2.76
R1xBOR_max	(dropped)			0.0475	0.0165	2.88
ArtBOG_max	(dropped)			0.0437	0.0257	1.70
VolTotal	0.000283	0.000231	1.23	(dropped)		
Winter	-0.290	0.121	-2.41	(dropped)		
AM	-0.400	0.121	-3.30	-0.546	0.279	-1.96
RightLane	-2.791	0.320	-8.71	-1.08	0.337	-3.21
SpeedLmt	0.159	0.0304	5.23	0.160	0.039	4.09
TrTimeLt15	-0.837	0.201	-4.18	-0.611	0.380	-1.61
TrTimeGt40	0.889	0.159	5.60	(dropped)		
Max_g2	-0.406	0.114	-3.55	(dropped)		
CyclePerHour	-0.0355	0.0157	-2.25	(dropped)		
Intercept	-11.08	1.60	-6.92	-14.42	1.80	-8.03

6.3. Multinomial Model of Crash Severity

Frequency of crash occurrence is often not the only concern for traffic engineers. The distribution of crashes in terms of severity is also a very important measure of safety performance. A basic classification of vehicle severity is property-damage-only (PDO) and injury-or-fatal (IF). If two systems have the same number of crashes, the one with a higher proportion of PDO crashes is apparently preferred. In addition, some researchers argue that since minor crashes are often not reported, the observed number of crash already contains severity information (Hauer 2006).

As documented in past research, many crash severity factors are vehicle characteristics or driver characteristics (Bekiroglu 1975; Blincoe et al. 2006; McGwin and Brown 1999; Porter and Berry 2001; Preusser et al. 1998). Ideally, these factors should be controlled when investigating the impact of signal timings on crash severity level. However, most of

these factors are missing in the dataset and the traffic signal timing engineer cannot expect to have these data when applying the models.

Conceptually, when the crash severity level is taken into account, the three-outcome structure has to be expanded to a structure of five outcomes: no crash, rear-end property-damage-only (RE-PDO), rear-end injury-fatal (RE-IF), right-angle PDO (RA-PDO), and right-angle injury-fatal (RA-IF).

The outcome structure of Figure 6-10 is a simple extension of the outcome structure of Figure 6-3. Methodologically, the ordered discrete outcome models and regression tree based methods have been used in classifying the severity levels (Abdel-Aty 2003; Abdel-Aty and Keller 2005). In this study, nonetheless, the ordered structure cannot be directly applied to the five outcome structure. There is no sensible justification to claim that RE-PDO is more severe than RA-PDO, or vice versa.

Figure 6-10 Crash outcomes with severity information

As the first modeling attempt, the multinomial logit model was investigated in this section. If the model is well specified, the common effects of crash outcomes may be explained by the deterministic term of the risk function. The multinomial logit model may be satisfactory, just as in the three-outcome model developed in Section 6.2. Next table lists the best multinomial model specification and the estimation results based on the one-percent sample.

The IIA assumption is then checked to ensure the appropriateness of the multinomial model. The coefficient estimates and significance levels can be seriously biased when

the IIA violation is present. The Small-Hsiao tests found very conclusive evidence against the IIA assumption as outlined in Table 6-11. This result indicates that there are considerable shared unobserved effects for every outcome. The multinomial model is not a proper choice when severity information is added as a classification criterion.

In addition, it was found in Section 6.2 that rear-end and right-angle crashes do not have very a significant level of correlated unobserved effects. Therefore, the correlation mostly lies within each crash type.

Table 6-10 Multinomial logit model of five crash outcomes

Variable	RE-PDO			RE-IF			RA-PDO			RA_IF		
	Coef.	Std. Err.	z	Coef.	Std. Err.	z	Coef.	Std. Err.	z	Coef.	Std. Err.	z
SR135	-2.50	0.40	-6.29	-2.09	0.71	-2.94	-0.54	0.66	-0.82	-1.61	0.98	-1.65
SR431	1.15	0.45	2.59	0.56	0.67	0.83	-0.03	0.77	-0.04	-3.47	1.21	-2.88
ArtBOR_max	0.03	0.02	1.40	0.04	0.03	1.35	-0.07	0.05	-1.35	0.09	0.06	1.37
R1xBOR_max	0.01	0.02	0.69	-0.01	0.03	-0.28	0.09	0.04	2.06	-0.05	0.06	-0.87
ArtBOG_max	0.03	0.01	2.32	0.04	0.03	1.06	0.01	0.06	0.23	0.01	0.07	0.17
G1xBOG_max	0.00	0.01	-0.02	-0.03	0.03	-0.97	0.05	0.06	0.84	-0.03	0.06	-0.46
Winter	-0.27	0.13	-2.01	-0.38	0.28	-1.36	0.40	0.31	1.30	-0.39	0.51	-0.76
AM	-0.32	0.14	-2.36	-0.13	0.27	-0.46	-0.71	0.37	-1.93	-0.18	0.50	-0.35
RightLane	-3.01	0.40	-7.49	-2.26	0.55	-4.14	-1.86	0.55	-3.35	-0.67	0.58	-1.17
SpeedLmt	0.18	0.04	4.63	0.11	0.05	2.10	0.08	0.06	1.24	0.34	0.12	2.80
TrTimeLt15	-0.69	0.24	-2.85	-0.62	0.41	-1.52	-0.70	0.47	-1.47	-0.55	0.73	-0.74
TrTimeGt40	0.93	0.18	5.10	0.52	0.40	1.30	-0.18	0.50	-0.35	1.52	0.81	1.87
Max_g2	-0.38	0.16	-2.38	-0.11	0.31	-0.35	0.01	0.37	0.04	-1.79	0.80	-2.25
CyclePerHour	-0.05	0.02	-2.64	-0.04	0.03	-1.09	-0.05	0.04	-1.33	0.03	0.05	0.58
Intercept	-12.47	1.98	-6.28	-11.15	3.04	-3.66	-9.13	3.51	-2.60	-23.90	5.77	-4.14

Table 6-11 IIA test for the vive-outcome multinomial logit model

	Omitted Outcome	Chi-square	d.f.	p-value	Conclusion
Small-Hsiao (H_0 : IIA)	Rear-end PDO	266.05	15	0.00	Reject H_0
	Rear-end IF	67.80	15	0.00	Reject H_0
	Right-angle PDO	246.23	15	0.00	Reject H_0
	Right-angle IF	265.06	15	0.00	Reject H_0

6.4. Sequential Logit Model of Crash Severity

As we found in previous sections, there is probably a significant level of correlated unobserved effects within each crash type. Therefore, the IIA property of the multinomial logit model is violated and alternative models have to be used when severity is also considered in classifying crash outcomes.

As discussed in Section 6.1.7, the nested logit model is often used to account for the shared unobserved effects within outcome groups. An example of the nested structure of the outcomes is shown in Figure 6-11.

Figure 6-11 Multilevel crash outcome structure

In the context of this study, however, the nested structure of crash outcomes has important and undesirable implications in interpretation. Due to the formulation of the nested logit model, the bottom level probabilities affect the upper level probability of the outcome nest. For instance, in this research, it means that if a factor increases the likelihood of a RE-IF crash without affecting the likelihood of other outcomes, then the likelihood of the RE nest is increased. A particularly undesirable case is that the predicted probability of one crash type can be changed due to a severity factor. An extreme case is that the fitted model may predict that seat belt use makes rear-end crashes less likely. Ideally, this very unrealistic result can be removed with a good model specification. However, in modeling practice, such unreasonable results are very possible due to the complexity of the actual data and event process. There is no sensible way to avoid such results in fitting the nested logit model. Therefore, the nested logit structure is not used to accommodate the shared unobserved effect in each crash type.

A more realistic description of the crash event process is implied by the sequential logit model. This model suggests that the crash process can be divided into two independent stages. First, some factors determine the likelihood of crash. Second, some factors determine the severity of a crash given that a crash occurs. The independence of the two stages avoids the interpretation difficulty that lies in the nested logit model. A severity factor is not going to affect the likelihood of the type of crash. The concept of this sequential logit model is illustrated in Figure 6-12.

Figure 6-12 Two-stage crash outcome structure

As previously described, the logit model used in the severity stage only serves to differentiate between property-damage-only (PDO) and injury fatal (IF) crashes. As a result, the predicted probabilities in the severity stage are conditional probabilities. In other words, the severity stage logit model produces prediction for $P(PDO | RE)$, $P(IF | RE)$, $P(PDO | RA)$, and $P(IF | RA)$. The predicted odds ratios are thus

$$\frac{P(IF | RE)}{P(PDO | RE)} = e^{V_{IF|RE}}, \frac{P(IF | RA)}{P(PDO | RA)} = e^{V_{IF|RA}} \quad (\text{Eq. 6.31})$$

where $V_{IF|RE}$ and $V_{IF|RA}$ are the deterministic part of the crash severity index function.

6.4.1. Logit Model of Rear-end Crash Severity

This model tries to identify the factors associated more severe rear-end crashes. There are two severity categories for a rear-end crash, PDO and IF, as illustrated in Figure 6-13. The PDO severity is selected as the base outcome and its severity index function is normalized to be 0.

Figure 6-13 Logit model structure of rear-end severity

Linear predictor is used (i.e., the severity index function is $V_{IF|RE} = bX_{IF|RE}$). As a result, the probabilities of the two severity levels are

$$P(PDO | RE) = \frac{1}{1 + e^{bX_{IF|RE}}}, P(IF | RE) = \frac{e^{bX_{IF|RE}}}{1 + e^{bX_{IF|RE}}} \quad (\text{Eq. 6.32})$$

where $X_{IF|RE}$ is the vector of the predictive variables and b is the vector of the coefficients to be estimated.

The fitted model is summarized in Table 6-12. Two factors were found to be associated with the severity of rear-end crash. First, the SR-431 system has a significantly lower percentage of severe rear-end crashes. Second, VolTotal, the total traffic volume at this arterial approach, is negatively correlated with the percentage of severe rear-end crashes. It is known that the actual traveling speed is slower when the volume rate is higher, when other factors are kept constant. This explains the severity reduction effect of VolTotal.

Table 6-12 Severity model of rear-end crash

	Coef.	Std. Err.	t-stat	P-value	95% Conf. Interval	
SR431	-0.792	0.281	-2.82	0.005	-1.342	-0.242
VolTotal	-0.00157	0.00059	-2.65	0.0080	-0.00274	-0.00041
Max_g2	0.482	0.276	1.74	0.081	-0.060	1.024
Intercept	-0.302	0.409	-0.74	0.459	-1.104	0.499
Model Statistics						
# Obs	356			d.f.	4	
Log Likelihood(null)	-176.47			AIC	344.06	
Log Likelihood(fit)	-168.03			BIC	359.56	
chi-square(2)	16.88			pseudo R ²	0.0478	

The impact of these two factors on the predicted proportion of IF crashes is shown in Figure 6-14. The reference severity index is calculated without the variable Max_g2, SR431, and VolTotal. As shown on the graph, the predicted likelihood of an IF crash is much lower on SR 431 system when other factors are kept constant. If VolTotal increases by 254 vehicles per hour, the conditional probability of an IF rear-end crash is also noticeably lower. Table 6-13 shows the elasticity of the odds ratio of the two factors. Because SR431 is an indicator variable, the elasticity is

$$\frac{P(IF | RE \text{ } \zeta SR431) / P(PDO | RE \text{ } \zeta SR431)}{P(IF | RE) / P(PDO | RE)} = e^{b_{SR431}} = 0.45 \quad (\text{Eq. 6.33})$$

On the other hand, VolTotal is a continuous variable. If VolTotal increases by 254, the elasticity on the odds ratio is

$$\frac{P(IF | RE \text{ } \zeta V'_{Total}) / P(PDO | RE \text{ } \zeta V'_{Total})}{P(IF | RE \text{ } \zeta V_{Total}) / P(PDO | RE \text{ } \zeta V_{Total})} = e^{b_V (V'_{Total} - V_{Total})} = 0.67 \quad (\text{Eq. 6.34})$$

Figure 6-14 Impact of SR431, VolTotal, Max_g2 on rear-end severity

Table 6-13 Elasticity of odds ratio for rear-end crashes

	b	t-stat	P-value	e^b	$e^{b\text{Std}(X)}$	Std(X)
SR431	-0.792	-2.82	0.005	0.453		0.500
VolTotal	-0.00157	-2.65	0.008		0.671	254.38
Max_g2	0.482	1.744	0.081	1.619		0.484

6.4.2. Logit Model of Right-angle Crash Severity

Right-angle crashes may have a different set of severity factors from rear-end crashes. Therefore, another logit model was developed independently for right-angle crashes. Following the same method as the last section, two severity categories, PDO and IF, are

considered as illustrated in Figure 6-15. A binary logit model with linear predictors is used in this section. The predicted probabilities of the two severity levels are

$$P(PDO | RA) = \frac{1}{1 + e^{bX_{IF|RA}}}, P(IF | RA) = \frac{e^{bX_{IF|RA}}}{1 + e^{bX_{IF|RA}}} \quad (\text{Eq. 6.35})$$

where $X_{IF|RA}$ is a vector of predictive variables and b is the coefficient to be estimated.

Figure 6-15 Right-angle severity outcomes

The fitted logit model for right-angle crashes is summarized in Table 6-14. Three factors were found to be significantly associated with the severity levels of right-angle crashes. *ShortY* is an indicator variable. It is 1 only when the yellow duration is shorter than the recommended value calculated from the MUTCD formula, as introduced in Section 5.2.7. *Max_g2* is an indicator variable describing the traffic concentration in the second half of green. If more than 25% of the vehicles arrive at the approach during this period based on the maximum green timings, then this variable is 1. *Winter* is an indicator variable. It is 1 when the month is November, December, January, or February.

The model suggests that the right-angle crash is likely to be less severe when the yellow signal is sufficiently long, when the crash occurs in winter, and when traffic concentrates more in the second half of green.

- *ShortY* is an indicator variable of the sufficiency of the yellow signal. When the yellow duration is unexpectedly short, high speed arterial right light running is more likely because arterial vehicle drivers may erroneously believe that they can enter the intersection before red and proceed with high speed.

- In *winter*, traffic volumes are generally lower and the actual traveling speed is usually lower. This may explain the lower severity level of right-angle crashes in winter.
- *Max_g2* means more than 25% of the traffic arrives at the approach during the second half of green signal if calculated based on the maximum green. The negative sign of this factor in the severity model indicates that the severity level of a right-angle crash is lower when such a traffic pattern is present. As demonstrated by the crash frequency model, this traffic pattern generally reduces the frequency of right-angle crashes. The severity impact may be because this traffic pattern has more effect on the arterial vehicles than the crossing street vehicles. In addition, the right-angle crashes caused by arterial red light running are generally more severe due to the higher speed of arterial vehicles.

The impact of these factors is illustrated in Figure 6-16. The reference severity level is calculated without Winter, ShortY, and Max_g2 variables. The elasticity of these factors on the odds ratio is shown in Table 6-15. Because all these factors are indicator variables, the elasticity can be calculated as

$$\frac{P(IF | RA \text{ } \zeta X = 1) / P(PDO | RA \text{ } \zeta X = 1)}{P(IF | RA \text{ } \zeta X = 0) / P(PDO | RA \text{ } \zeta X = 0)} = e^{b_x(1-0)} = e^{b_x} \quad (\text{Eq. 6.36})$$

Table 6-14 Logit model of right-angle crash severity

	Coef.	Std. Err.	t-stat	P-value	95% Conf. Interval	
Winter	-1.088	0.667	-1.630	0.103	-2.396	0.220
ShortY	1.926	0.617	3.120	0.002	0.717	3.135
Max_g2	-1.355	0.682	-1.990	0.047	-2.693	-0.018
Intercept	-0.847	0.479	-1.770	0.077	-1.787	0.092
Model Statistics						
pseudo R ²	Obs	logl(null)	logl(model)	d.f.	AIC	BIC
0.187	67	-43.094	-34.7141	4	77.4281	86.2469

Figure 6-16 Winter, ShortY, and Max_g2's impact on right-angle severity

Table 6-15 Elasticity of odds ratio for right-angle crashes

	b	t-stat	P-value	e^b
Winter	-1.088	-1.631	0.103	0.337
ShortY	1.926	3.123	0.002	6.862
Max_g2	-1.355	-1.986	0.047	0.258

6.4.3. Integrated Logit Model of Crash Severity

An integrated model can be used to test the difference of the severity models developed for rear-end and right-angle crashes.

To use this model, interaction terms are created based on the crash type. Such interactions allow that the severity impact of factors be different for rear-end and right-angle crashes. For example, the interaction term of *Winter* and *Right-angle* is only 1 when a right-angle crash occurs during the winter months. This interaction variable is always 0 for the rear-end crashes even if it occurs in winter. Therefore, it has no impact on the severity level of the rear-end crashes.

Table 6-16 shows the initial integrated model with the interaction terms identified for the two crash types, respectively. This model is equivalent to the combination of the two models developed for the two crash types. A ConstxRA interaction term was created to allow the intercept terms be different for rear-end and right-angle crashes. The constant for right-angle crashes can be calculated as $-0.302 - 0.545 = -0.847$, the same as in the right-angle severity model. All coefficients are the same as in the crash-specific severity models.

Table 6-16 Initial integrated logit model of severity

		Coef.	Std. Err.	t-stat	P-value	95% Conf. Interval	
overall	Intercept	-0.302	0.409	-0.74	0.459	-1.104	0.499
rear-end	SR431 x RE	-0.792	0.281	-2.82	0.005	-1.342	-0.2421
	VolTotal x RE	-0.00157	0.00059	-2.65	0.008	-0.00274	-0.0004
	Max_g2 x RE	0.482	0.276	1.74	0.081	-0.060	1.024
right-angle	ShortY x RA	1.926	0.617	3.12	0.002	0.717	3.135
	Winter x RA	-1.088	0.667	-1.63	0.103	-2.396	0.220
	Max_g2 x RA	-1.355	0.682	-1.99	0.047	-2.693	-0.018
	Const x RA	-0.545	0.630	-0.87	0.387	-1.780	0.690

Starting from this model, a series of hypothesis tests were conducted to check whether a severity factor is important for both types of crashes. The likelihood ratio test is well suited in this situation. For example, another interaction term, WinterxRE can be added to the model. We can perform two types of tests.

- 1 Test whether the coefficient of WinterxRE is significantly from 0. This test checks whether *Winter* is also a factor for rear-end severity.
- 2 Test whether the coefficient of WinterxRE is significantly different from the coefficient of WinterxRA. This test checks whether Winter's impact on the two types of crashes are differentiable.

First, it was found that the constant terms of the two types of crashes cannot be differentiated when other factors are controlled. Therefore, the crash-specific constant term can be dropped from the model. Second, all other factors are significant for only one type of crash.

The model after dropping the crash type-specific constant term is presented in Table 6-17. All the signs of the coefficient estimates are the same as in the models developed one crash type. Table 6-18 summarizes the elasticity on the severity odd-ratio of the factors calculated on this integrated model. Figure 6-17 and Figure 6-18 illustrate the impact of the factors on the predicted likelihood of an IF crash when other factors are constant. The reference severity index is calculated without the listed factors.

Table 6-17 Integrated logit model of crash severity

		Coef.	Std. Err.	t-stat	P-value	95% Conf. Interval	
Overall	Intercept	-0.538	0.305	-1.76	0.078	-1.137	0.060
Rear-end	SR431 x Re	-0.726	0.270	-2.69	0.007	-1.256	-0.197
	VolTotal x Re	-0.00129	0.0005	-2.65	0.008	-0.0022	-0.0003
	Max_g2 x Re	0.520	0.273	1.90	0.057	-0.016	1.055
Right-angle	ShortY x Ra	1.720	0.562	3.06	0.002	0.618	2.822
	Winter x Ra	-1.250	0.634	-1.97	0.049	-2.493	-0.007
	Max_g2 x Ra	-1.485	0.660	-2.25	0.024	-2.779	-0.192
Model Statistics							
pseudo R ²	Obs	logl(null)	logl(model)	df	AIC	BIC	
0.0883	423	-222.806	-203.123	7	420.246	448.578	

Table 6-18 Elasticity of factors on the odd-ratio of crash severity

	Variable	b	t-stat	P> z	e ^b	e ^{bStd(X)}
Rear-end	SR431 x Re	-0.726	-2.69	0.01	0.48	
	Voltotal x Re	-0.00129	-2.65	0.01		0.66
	Max_g2 x Re	0.520	1.90	0.06	1.68	
Right-angle	ShortY x Ra	1.720	3.06	0.00	5.58	
	Winter x Ra	-1.250	-1.97	0.05	0.29	
	Max_g2 x Ra	-1.485	-2.25	0.02	0.23

Figure 6-17 Impact of factors on the predicted RA-IF likelihood

Figure 6-18 Impact of factors on the predicted RE-IF likelihood

CHAPTER 7. TRAFFIC SAFETY OF URBAN ARTERIAL STREETS – DISCUSSION OF THE FINDINGS

This chapter discusses the safety impact of signal, traffic, and road characteristics on crash frequencies and severity on urban arterial streets identified through the research described in the preceding chapters of this report. The identified impacts on frequency and severity of rear-end and right-angle crashes are qualitatively summarized in Table 7-1 and Table 7-2.

Table 7-1 Summary of significant crash likelihood factors

Variable y	Description	Rear-end Likelihood	Right-angle Likelihood
RightLane	Exclusive Right-turn lane	Lower	Lower
SpeedLmt	Higher Speed Limit	Higher	Higher
TrTimeGt40	Travel Time from upstream > 40 second	Higher	-
TrTimeLt15	Travel Time from upstream < 15 second	Lower	Lower
Max_g2	High traffic concentration in 2nd half of green	Lower	Lower
Cycle per hour	Shorter Cycle Length	Lower	-
ArtBOG_max	Arterial volume in the change interval at beginning of red phases	Higher	Higher
R1xBOR_max	ArtBOR x max_R1 (interaction)	Higher	Higher
Winter	Month of Nov, Dec, Jan, or Feb	Lower	-
AM	Before noon	Lower	Lower

Table 7-2 Summary of crash severity factors

Crash type	Variable y	Description	Severity Level
Rear-end	SR431	On SR431 system	Lower
	VolTotal	Total volume per hour on the approach	Lower
	Max_g2	More than 25% volume in G2 based on max green setting	Higher
Right-angle	ShortY	Yellow shorter than MUTCD value	Higher
	Winter	Nov, Dec, Jan, or Feb	Lower
	Max_g2	More than 25% volume in G2 based on max green setting	Lower

Second, quantitatively, the level of impact of each factor can be derived from the fitted coefficients reported in Chapter 6 (Table 6-3 and 6-17). Let us begin with the likelihood of a crash. From the formulation of the multinomial logit model, it is clear that the odd ratio of a crash over no crash is

$$\frac{P(\text{crash})}{P(\text{no crash})} = e^{\beta_0 + \beta_1 X_1 + \dots + \beta_k X_k + \dots + \beta_n X_n}$$

Let now assume that variable X_k changes value by Δ_k , thus the new odd ratio is

$$\frac{P_k(\text{crash})}{P_k(\text{no crash})} = e^{\beta_0 + \beta_1 X_1 + \dots + \beta_k (X_k + \Delta_k) + \dots + \beta_n X_n}$$

Recognizing that the likelihood values $P(\text{no crash})$ and $P_{\Delta}(\text{no crash})$ are close to 1, the above equations simplify to:

$$P(\text{crash}) = e^{\beta_0 + \beta_1 X_1 + \dots + \beta_n X_n}$$

$$P_k(\text{crash}) = e^{\beta_0 + \beta_1 X_1 + \dots + \beta_n (X_n + \Delta_k) + \dots + \beta_n X_n}$$

Now, the Risk Modification Factor related to the change Δ_k (RMF_k) defined as $P_k(\text{crash})/P(\text{crash})$ can be calculated as:

$$\text{RMF}_k = \frac{P_k(\text{crash})}{P(\text{crash})} = e^{\beta_k \Delta_k} \quad (\text{Eq. 7.1})$$

The change of variable X_k is equal to 1 if the variable is an indicator (takes only values 0 or 1). The change of variable X_k can take any value if the variable is continuous. We will assume a change of the continuous variable that represents a realistic modification of the traffic or signal control conditions. For instance, a 5 mi/h change in the speed limit is such a reasonable and easy to interpret modification. The risk modification factors corresponding to the indicator and continuous variables for both rear-end and right-angle crashes are given in Table 7-3 Risk modification factors of indicator variables and Table 7-4.

Table 7-3 Risk modification factors of indicator variables

Crash Type	Variable X	Coefficient β	Risk Modification Factor RMF
Rear-end	Winter	-0.290	0.75
	AM	-0.291	0.75
	RightLane	-2.748	0.06
	TrTimeLt15	-0.764	0.47
	TrTimeGt40	0.861	2.36
	Max_g2	-0.215	0.81
Right-angle	AM	-0.559	0.57
	RightLane	-1.145	0.32
	TrTimeLt15	-0.668	0.51
	Max_g2	-0.471	0.62

The change of severity level can be derived similarly. However, since the conditional likelihood of any severity outcome is usually considerably different from 1, the change in the likelihood of the IF outcome cannot be obtained as conveniently as the risk modification factors. The probability of the IF outcome is:

$$P(IF) = \frac{e^{\beta_0 + \beta_1 X_1 + \dots + \beta_n X_n}}{1 + e^{\beta_0 + \beta_1 X_1 + \dots + \beta_n X_n}}$$

Table 7-4 Crash risk reduction factor of continuous variables

Crash Type	Variable X	Variable Change Δ	Coefficient β	Risk Modification Factor RMF
Rear-end	R1xBOR_max	5 veh/h/lane	0.028	1.15
	ArtBOG_max	5 veh/h/lane	0.038	1.21
	SpeedLmt	5 mi/h	0.158	2.20
	CyclePerHour	10	-0.044	0.64
Right-angle	R1xBOR_max	5 veh/h/lane	0.033	1.18
	ArtBOG_max	5 veh/h/lane	0.037	1.20
	SpeedLmt	5 mi/h	0.159	2.22

Let $P_k(IF)$ be the likelihood of the IF outcome after variable X_k changes by Δ_k . It can be shown that the original $P(IF)$ will change by the following value:

$$\Delta P_k(IF) = P_k(IF) - P(IF) = P(IF) \cdot \left(\frac{e^{\beta_k \Delta_k}}{1 + (e^{\beta_k \Delta_k} - 1) \cdot P(IF)} - 1 \right) \quad (\text{Eq. 7.2})$$

Unlike RMF , $\Delta P_k(IF)$ depends on the original likelihood of the IF outcome. To depict the

impact of the crash severity factors identified in CHAPTER 6,

Figure 7-1 Safety impacts on the IF likelihood

Figure 7-1 presents the values of $\Delta P_k(IF)$ as a function of the original likelihood $P(IF)$

and the assumed changes in the model variable X_k . The variable changes are: 1 for indicator variables, and 500 veh/h of total volume arriving on the subject approach.

The *RightLane*, *TrTimeGt40*, and *TrTimeLt15* variables are the geometric characteristics of the arterial. Traffic system designers have a limited opportunity to controlling these variables to improve safety. Nevertheless, highway engineers should be aware of these factors. The presence an exclusive right-turn lane is associated with an exceptionally low frequency of rear-end and right-angle crashes and it should be built on all arterial street approaches. Its strong effect on urban street safety seems to be long overlooked.

Removing vehicles turning right from the dense platoon of vehicles eliminates or at least reduces the braking and lane-changing maneuvers that may be surprising to drivers who are currently focused on traffic signal indications.

The long distance from an upstream intersection represented by the travel time longer than 40 seconds ($TrTimeGt40 = 1$) is associated with a high risk of rear-end crashes. The possible explanations are as follows.

- 1 Long distance between intersections may be associated with higher travel speeds reached near the downstream intersection.
- 2 Long travel time may relax drivers who lose the alertness caused by passing the upstream intersection. This relaxation increases the reaction time and may contribute to unsuccessful collision avoidance at the downstream intersection.
- 3 Longer distances create an opportunity for vehicles queues to grow long posing a source of surprise to drivers who may not expect the back of the queue far away from the downstream intersection.

The risk reduction effect of short distances from upstream approaches, represented by $TrTimeLt15$, can be explained similarly.

1. A shorter distance is associated with a shorter queue at the downstream intersection.
2. Drivers have a better view of the downstream intersection and thus are better prepared.
3. Arterial systems engineers usually try to time the signals such that back-to-back stops are minimized (Li and Tarko 2006). This goal is easier to achieve for closely spaced intersections. The fewer number of stops is associated with a lower risk of rear-end crashes.
4. Drivers are usually more alert right after they leave the upstream intersections.

The speed limit effect is one of the most pronounced safety impacts. Reducing the speed limit by 5 mi/h may reduce the number of rear-end and right-angle collisions by half. The mechanism seems to be quite obvious the ability to stop a car to avoid a rear-end crash or violation of a red signal is highly dependent on the speed.

Short cycle lengths are typically associated with lower traffic volumes and shorter red signals along the arterial streets. As the result, vehicle queues tend to be shorter and together with lower traffic volumes reduce the drivers' exposure to rear-end crashes. It has to be emphasized that the detail inspection of a large number of crash reports described in CHAPTER 3 indicates that the main source of rear-end collisions is not the presence of the dilemma situation (which may be an issue if occurs) but the presence of vehicle queues and slowly moving vehicles on the approach to an intersection. Longer cycle lengths are usually selected when demand for capacity is higher.

High value of *R1xBOR_max* indicates a concentration of the arriving traffic in the first half of the arterial red signal including the first two seconds of red. It is associated with increased likelihood of both rear-end and right-angle crashes. This situation creates conditions for a dilemma zone, rapid braking maneuvers, and violating of the red signal on the arterial road. If the traffic has a low level of concentration during the first half of the red, then the vehicles are not exposed to a significantly higher risk of crashes.

A high value of *ArtBOG_max* means that considerable arterial traffic arrives during the first two seconds of green (when green is maximum on the coordinated intersections). This value is associated with a higher risk of rear-end and right-angle crashes. There are several possible explanations of this effect. (1) Vehicles arriving at the beginning of green are most susceptible to the initial queue shockwave. (2) Another possible cause is related to the operation logic of the actuated signal controller. As introduced in Section 5.2.5, the beginning of the green phase of the arterial movement fluctuates according to the traffic demand. Therefore, vehicles arriving around this time window face a greater variation of signal status. This elevates the degree of uncertainty and surprise to drivers along the arterial street. (3) High value of *ArtBOG_max* is also strongly associated with the increased risk of right-angle collisions. The vehicles arriving at the beginning of the green (*ArtBOG_max*) are susceptible to the red-light runners from the side street.

The value of $Max_g2 = 1$ indicates high density of traffic arriving during the second half of the arterial green signal. This situation is associated with a lower risk of rear-end crashes. These vehicles are the ones least likely to encounter queues of vehicles stopped during the preceding red signal. This result confirms that the presence of queue, is a major cause of rear-end crashes.

The results are consistent with the analysis of the conceptual model developed in CHAPTER 4 and the crash pattern study documented in CHAPTER 3. First, it confirms that arterial signal timing can significantly affect the safety performance. Second, rear-end crashes are caused by the queue-forming and discharging process at the beginning of the green. Third, the results also confirm that the traffic volume rate at the beginning of the red (plus the end of the green) is most susceptible to right-angle crashes. However, the risk reduction effect of longer yellow was not confirmed.

Another crash likelihood factor is the morning effect, represented by the AM variable. It was found that in the morning, both rear-end and right-angle crashes are less likely, which can be attributed to the fatigue level of drivers. In the morning, drivers are generally well rested and make fewer mistakes (Bunn et al. 2005). The effect, however, is not controllable for traffic engineers and thus is not discussed further in this study.

Winter reduces the likelihood of rear-end crashes which may be attributed to the significantly lower level of traffic on arterial roads, as shown in Figure 5-3. It should be noted that the traffic volume in each observation is represented with a single traffic count.

The variables which are tried in the models but are not found to be significant also carry a certain amount of information. Statistically speaking, it does not mean that a variable is not an important factor if it is found to be not significant. The correct conclusion should be: given the current dataset, there is not enough evidence to argue against the claim that the impact of this variable is not present (Neter et al. 1996). A possible reason is that two predictive variables are highly correlated. When one of them is already in the model, the addition of the other variable will not be necessary. Theoretically speaking, it can be tested which one is a better predictor. Practically, however, since the two

predictive variables are highly correlated, it probably will be difficult to differentiate between the impacts of these two factors. In this study, for example, it was found that the primary volume is the best predictor and the addition of the secondary volume variable contributes very little to the overall model fitness. It does not mean that the secondary traffic volume is not exposed to crash risk because the levels of the primary and secondary volumes are highly correlated.

Similarly, the traffic pattern representation proposed in this study outperformed the traditional measure of coordination quality, such as arrival type and platoon ratio. It cannot be interpreted that the traditional measure are not associated with risk levels. It only means the proposed characterization contains better and more accurate information than the traditional measures.

The impact of the arterial traffic pattern on crash severity is represented only by the severity reduction effect of the variable *Max_g2*. *Max_g2* means that more than 25% of vehicles arrive at the approach during the second half of the green signal, calculated based on the maximum green. As demonstrated by the crash frequency model, this traffic pattern generally reduces the frequency of right-angle crashes. This traffic pattern has a stronger effect on the arterial vehicles than on the crossing street vehicles. Also, since the right-angle crashes caused by arterial red light running are generally more severe due to the higher speed of arterial vehicles, the proportion of severe right-angle crashes is thus reduced.

Insufficient duration of the yellow, represented by *ShortY* = 1, is associated with higher severity levels of right-angle crashes, which may cause drivers to misjudge and proceed through the intersection at high speed.

The reducing effect of traffic volume (*VolTotal*) on the severity of rear-end crashes can be explained with the reduction of speed during the congested period and on congested arterial streets. Also, drivers may proceed with an enhanced caution during such conditions. Other variables present in the severity model have less appealing interpretation; thus, no discussion of their meaning is provided. They will be used to

together with other variables and their corresponding coefficients to predict crash severity.

The above findings can serve as a guide for traffic system engineers. A traffic system engineer and a highway designer can lower the risk of both rear-end and right-angle crashes and can lower the crash severity in several ways:

- Offsets that promote traffic arrivals at the stop-line in the second part of the green signal and reduce the possibility of traffic arriving during the second part of red
- Shorter cycle lengths are preferred if they are allowed by low demand for capacity
- Reduce the number of vehicles arriving shortly after the phase changes
- Use sufficiently long yellow signals
- Install right-turn turning bays on arterial approaches
- Consider lower posted speed limits if consistent with the geometric design of the arterial

It should be noted that these recommendations may be difficult to attain simultaneously along the entire arterial street. Also, the traffic mobility must be taken into account. Therefore, using a software tool developed in this project to evaluate and select the safest solution among those alternative solutions that satisfactory meet the traffic mobility criterion seems to be a practical approach. The software tool suitable for this task is presented in the next chapter.

CHAPTER 8. A PROTOTYPE TOOL FOR PREDICTING CRASHES IN COORDINATED ARTERIALS

The prediction models developed and discussed in the preceding chapters allow predicting the frequency and severity of rear-end and right-angle crashes on arterial approaches controlled by traffic signals coordinated with signals at upstream intersections. The crash predictions obtained for 15-minute intervals can be aggregated to produce the number of crashes expected in one year and during the days of week and hours when a specific signal coordination plan is executed. A traffic system engineer may want to develop alternative coordination plans that are comparable from the traffic performance point of view (delays and number of stops) and then predict the number of crashes for each of these plans to identify the safest one. To facilitate this design procedure, a prototype software tool has been developed.

This chapter describes the general procedure of evaluating safety for alternative coordination plans, presents computational details of the procedure, and describes the Excel-based prototype tool, called SafeArt, and its use.

A simple general procedure leading to the selection of the best coordination plan is applicable if the alternative coordination plans are comparable from the traffic performance point of view. Then, the safest plan should be selected as the best solution:

- 1 Develop coordination plans for a certain period of day using Synchro, SimTraffic or other signal optimization/simulation tools.
- 2 For each alternative coordination plan, run SafeArt to obtain the annual prediction of crashes.
- 3 Select the safest coordination plan alternative.

A trade-off between traffic and safety performance is sought in the case when the initial alternatives considerably vary by traffic performance. Accurate prediction of safety is important and the calibration of SafeArt is recommended. The extended general procedure is as follows:

1. Develop coordination plans for a certain period of day using Synchro, SimTraffic or other signal optimization/simulation tools.
2. Calibrate the SafeArt prediction models with data representing the existing coordination plan.
3. For each alternative coordination plan, run the calibrated SafeArt to obtain the annual prediction of crashes.
4. For each alternative coordination plan, calculate the combined traffic and safety performance measure.
5. Select the best coordination plan alternative.

8.1. Computational Procedure

The computational procedure includes:

1. Preparing input data
2. Converting the input data to the variable values
3. Calculating the likelihood of crashes
4. Aggregating the results
5. Determining the calibration parameters (if needed)

8.1.1. Preparing Input Data

Before performing basic safety calculations without models calibration, a traffic system engineer must prepare the required input data including geometry data of the arterial system, signal timing data, and 15-minute traffic counts of all movement of the

intersections. These data are required by Synchro and other similar signal optimization tools to optimize signal timing; therefore, no extra effort is needed.

If the engineer decides to calibrate the safety prediction models for more accurate comparison as introduced in Section 8.1.4, then the annual numbers of rear-end and right-angle crashes during the specified time period are also needed.

Most input data are intuitive and typically in the same format possessed by traffic systems engineers. A few conventions must be followed, however, to obtain correct prediction results. First, the software accepts two scenarios of coordination directions: NB/SB or EB/WB. All the inputs must start from the intersections on the west or south and to the intersections on the east or north. The order should be the same as the geometry locations. This is shown in Figure 8-1 and Figure 8-2. The data must be entered from intersection 3 to intersection 9 in both scenarios.

Figure 8-1 EB/WB arterial layout

Figure 8-2 NB/SB arterial layout

Second, the offsets based on the “Begin of Red” should be used. Synchro provides several representations of offsets. This can be changed in the “Timing Window (F5)” of Synchro as shown in Figure 8-3. This setting should be used by all intersections in the arterial. Otherwise the offset values are not expressed in right convention and the calculations of traffic patterns are wrong.

Figure 8-3 Use begin of red offset

8.1.2. Converting Input to Variable Values

The most involved conversion is determining the traffic arrival pattern and the corresponding variables: *ArtBOG_max*, *ArtBOR_max*, *Max_g2*, *Max_r1*, and *R1xBOR_max*. The derived traffic arrival pattern variables are crucial components of the model. They represent the expected traffic conditions at an intersection approach in 15-minute intervals. The calculation algorithm was described in Section 5.2 with details and repeated here in a modified version for completeness.

The past research shows that any cyclic traffic pattern produced by traffic signals gradually converts along the road to a fully dispersed uniform pattern represented by a constant flow rate over the entire signal cycle. We will represent the initial cycling pattern with a simplified model that includes only two distinct flow rates: primary and secondary rates. The primary volume rate measured in veh/h/lane at the upstream intersection is V_p (arterial through traffic passing the upstream intersection during the arterial effective green phase):

$$V_p = \frac{4 \cdot Q_p \cdot C}{n \cdot g} \quad (\text{Eq. 8.1})$$

where:

V_p = primary volume rate (veh/h/lane),

Q_p = 15-minute traffic count of arterial through movement at the upstream intersection,

n = number of arterial lanes in one direction,

g = arterial green phase including the green and change periods (s),

C = signal cycle (s).

The secondary volume rate measured in veh/h/lane at the upstream intersection is V_s (traffic turning towards the considered downstream approach from the crossing road at the upstream intersection):

$$V_s = \frac{4 \cdot Q_s \cdot C}{n \cdot r} \quad (\text{Eq. 8.2})$$

where:

V_s = secondary volume rate (veh/h/lane),

Q_s = 15-minute traffic count of arterial turning volumes from the crossing roads at the upstream intersection towards the downstream intersection,

n = number of arterial lanes in one direction,

r = arterial red phase which is the rest of the cycle after the arterial green phase (s),

C = signal cycle (s).

As the traffic moves down the street, the difference between the primary and secondary flow rates reduces (primary rates decreases and the secondary flow increases). After

traveling T seconds, the flow has both the rates equal to V_T . The fully dispersed flow rate V_T measured in veh/h/lane is:

$$V_T = \frac{4 \cdot Q_s + 4 \cdot Q_p}{n} \quad (\text{Eq. 8.3})$$

The initial flow rate V_0 (V_p or V_s) after traveling t seconds converts to V_t according to the following linear transformation:

$$V_t = \begin{cases} V_0 + \frac{t}{T}(V_T - V_0) & \text{if } t \leq T, \\ V_T & \text{if } t \geq T. \end{cases} \quad (\text{Eq. 8.4})$$

where:

V_t is the downstream traffic flow rate (veh/h/lane),

t is the travel time to the downstream point where the cyclic profile is observed (s),

T is the travel time beyond which the traffic platoons are fully dispersed (s),

V_0 is the initial flow rate at the upstream intersection (veh/h/lane), and

V_T is the fully dispersed flow rate (veh/h/lane).

In general, the number of vehicles arriving at the downstream intersection during a certain portion of a cycle ΔC (such as the second half of red signal or the first two seconds of a green signal) expressed in veh/h/lane can be calculated as:

$$V_{\Delta C} = \frac{\sum_{i \in \Delta C} V_{ti}}{C} \quad (\text{Eq. 8.5})$$

where:

$V_{\Delta C}$ is the number of vehicles arriving at the downstream intersection during the ΔC time window (veh/h/lane),

$\{\Delta C\}$ are the seconds included in the considered part of cycle,

V_{ti} is the arriving hourly volume rate in second i ,

C is the cycle length (s).

If the hourly traffic rate is constant during the ΔC window, then, (Eq. 8.5) becomes:

$$V_{\Delta C} = \frac{\Delta C \cdot V_t}{C} \quad (\text{Eq. 8.6})$$

For example, the value of $ArtBOG_max$ expressed in veh/h/lane is calculated as:

$$ArtBOG_max = (V_{t1} + V_{t2})/C, \quad (\text{Eq. 8.7})$$

where V_{t1} is the arriving flow rate in the first second of arterial green calculated with , V_{t2} is the arriving flow rate in the second second of arterial green calculated with (Eq. 8.4), and C is the cycle length in seconds. The arterial green traffic signals at the downstream and upstream intersections are assumed at their maximum lengths allowed by the signal controller settings. The value of $ArtBOR_max$ is calculated similarly to the $ArtBOG_max$ but for the first two seconds of arterial red signal.

The value of Max_r1 takes value of 1 if more than 25% of traffic arrives during the second half of the arterial green signal. The arterial green traffic signals at the downstream and upstream intersections are assumed at their maximum lengths allowed by the signal controller settings. The arriving flow rate is summed up during this interval, divided by the average number of vehicles in the cycle, and compared to 0.25:

$$Max_r1 = \begin{cases} 1 & \text{if } \frac{\sum_{t=t1}^{t1+1} V_{ti}}{V_T \cdot C} > 0.25, \\ 0, & \text{otherwise.} \end{cases} \quad (\text{Eq. 8.8})$$

where V_T is the dispersed traffic rate calculated with (Eq. 8.3), C is the signal cycle in seconds, and $\{r1\}$ are the seconds in the first half of the arterial red phase.

Max_r2 , Max_g1 , and Max_g2 are calculated in a similar way as the Max_r1 . The value of $R1xBOR_max$ is the product of Max_r1 and $ArtBOR_max$.

The values of variables $ArtBOG_max$, $ArtBOR_max$, Max_g2 , Max_r1 , and $R1xBOR_max$ are calculated for all the 15-minute intervals and all the arterial approaches with coordinated traffic signals.

Example Calculations

In this section, the calculation of these traffic patterns is demonstrated by an example.

The data need for calculating the percentages of the traffic arriving at an approach include: upstream traffic counts, cycle length, upstream effective green, downstream effective green, speed limit, distance from the upstream intersection, and the offsets of the two intersections. The assumed values of this example are listed in Table 8-1. In addition, the travel time needed to fully dispersed traffic columns is $T = 120$ s.

Table 8-1 Assumed parameter values

Upstream 15-min Vehicle Counts	Geometry	Timing
WBTH $Q_p = 250$ NBLT $Q_{s1} = 35$ SBRT $Q_{s2} = 15$	Speed limit $S = 50$ mph Distance from upstream $L = 2000$ ft Number of arterial through lanes $n = 1$	Cycle $C = 100$ s Upstream offset 3 s Arterial upstream green phase $g = 50$ s Downstream offset 50 s Arterial downstream green phase 70 s

Figure 8-4 Layout of the example intersection

First, the upstream primary traffic flow rate and the secondary traffic flow rate are calculated from the 15-minute traffic counts. Assume that we are interested in the WB approach of the upstream intersection as depicted in Figure 8-4. The traffic engineer recorded at the upstream intersection and in this 15-minute interval: 250 westbound through (WBT) vehicles, 35 northbound left-turn (NBL) vehicles, and 15 southbound right-turn (SBR) vehicles. There is only one westbound traffic lane between the two intersections. Therefore, the primary flow rate and the secondary flow rates discharged from the upstream intersection are calculated as follows:

$$V_p = \frac{4 \cdot Q_p}{n} \cdot \frac{C}{p} = \frac{4 \cdot 250}{1} \cdot \frac{100}{50} = 2,000 \text{ veh/h/lane}$$

$$V_s = \frac{4 \cdot Q_s}{n} \cdot \frac{C}{r} = \frac{4 \cdot (15 + 35)}{1} \cdot \frac{100}{100 - 50} = 400 \text{ veh/h/lane}$$

Figure 8-5 Cyclic pattern of traffic discharging from the upstream intersection

Second, a linear dispersion model is used to calculate the primary and secondary flow rates arriving at the westbound approach of the downstream intersection. We have assumed that the vehicle platoons are fully dispersed after 120 seconds of travel. The fully dispersed flow rate is (Eq. 8.2):

$$V_T = \frac{4 \cdot Q_s + 4 \cdot Q_p}{n} = \frac{4 \cdot (15 + 35) + 4 \cdot 250}{1} = 1,200 \text{ veh/h/lane}$$

Given that the speed limit is $S = 50$ mile/hour and the distance between the intersections is $L = 2000$ feet, the expected travel time is then $t = 2,000 / (50 \cdot 5,280 / 3,600) = 27 \text{ s}$.

Therefore, the primary and secondary arrival flow rates at the downstream intersection are (Eq. 8.4):

$$V_s = \begin{cases} V_p + \frac{t}{T} (V_T - V_p) = 2,000 + \frac{27}{120} \cdot (1,200 - 2,000) = 1,820 \text{ veh/h/lane if } V_0 = V_p, \\ V_s + \frac{t}{T} (V_T - V_s) = 400 + \frac{27}{120} \cdot (1,200 - 400) = 580 \text{ veh/h/lane if } V_0 = V_s. \end{cases}$$

Next, use the end of yellow of the westbound through movement as the reference points for calculating offsets. Since the offset of the downstream intersection is 3 second and the offset of the upstream intersection is 50 second, the local offset between the two signals is 47 seconds (measured between the beginnings of red signals).

Using the timing information, the downstream traffic profile can be readily determined as shown in Figure 8-6. Select the start of red at westbound approach at the upstream intersection as the origin. Then in a 100-second cycle, the arrival rate in the 0-30 second window is secondary, the arrival rate in the 30-80 second window is primary, and the arrival rate in the 80-100 second window is again secondary.

Therefore, the traffic volume in the R1, R2, G1, and G2 windows expressed in veh/h/lane can be calculated according to (Eq. 8.5) or (Eq. 8.6):

$$V_{R1} = V_{R2} = \frac{15 \cdot 580}{100} = 87 \frac{\text{veh}}{\text{h} \cdot \text{lane}}$$

Figure 8-6 Calculate downstream traffic profile

Figure 8-7 Downstream traffic pattern

$$V_{G1} = \frac{35 \cdot 1,820}{100} = 637 \frac{\text{veh}}{\text{h} \cdot \text{lane}}$$

$$V_{G2} = \frac{15 \cdot 1,820 + 20 \cdot 580}{100} = 389 \frac{\text{veh}}{\text{h} \cdot \text{lane}}$$

The volume proportions are thus: 0.073 for r1, 0.073 for r2, 0.531 for g1, and 0.324 for g2. The corresponding arrival profile code is 0011.

The hourly volume during the initial two second windows after the phase changes are

$$ArtBOR = \frac{2 \cdot 580}{100} = 11.6 \text{ veh/h/lane}$$

$$ArtBOR = \frac{2 \cdot 1,820}{100} = 36.4 \text{ veh/h/lane}$$

A software tool is developed to facilitate the use of the findings of this research. This tool is developed with Excel VBA macros. Traffic systems engineers can use this tool as an Excel spreadsheet.

8.1.3. Calculating the Likelihood of Crashes

The equations recommended for predicting the likelihood of a crash and its severity in a 15-minute interval on an arterial approach to an intersection coordinated with the upstream intersection are presented below.

The first step is to calculate risk indices F_{RE} and F_{RA} for rear-end and right-angle crashes, respectively. Also, the severity indices S_{RE} and S_{RA} are calculated. These values obtained in (Eq. 8.9) are then used in the following calculations.

$$\begin{aligned}
 F_{RE} &= 0.028 \cdot R1xBOR_max + 0.038 \cdot ArtBOG_max - 0.290 \cdot Winter - 0.291 \cdot AM \\
 &\quad - 2.7478 \cdot RightLane + 0.158 \cdot SpeedLmt - 0.764 \cdot TrTimeLt15 + \\
 &\quad 0.861 \cdot TrTimeGt40 - 0.215 \cdot Max_g2 - 0.044 \cdot CyclePerHour - 11.21 \\
 F_{RA} &= 0.033 \cdot R1xBOR_max + 0.037 \cdot ArtBOG_max - 0.559 \cdot AM - 1.145 \cdot \\
 &\quad RightLane + 0.159 \cdot SpeedLmt - 0.668 \cdot TrTimeLt15 - 0.471 \cdot Max_g2 - 14.00 \\
 S_{RE} &= -0.0013 \cdot VolTotal - 0.538 \\
 S_{RA} &= 1.720 \cdot ShortY - 1.250 \cdot Winter - 1.485 \cdot Max_g2 - 0.538
 \end{aligned}
 \tag{Eq. 8.9}$$

where:

F_{RE} = frequency risk index for rear-end collisions,

F_{RA} = frequency risk index for right-angle collisions,

S_{RE} = severity risk index for rear-end collisions,

S_{RA} = severity risk index for right-angle collisions,

AM = morning indicator variable, 1 if time before noon, 0 otherwise

BOR_max = number of vehicles arriving during the first two seconds of arterial red signal, (maximum signal splits assumed for the subject and upstream intersection approaches),

ArtBOG_max = number of vehicles arriving during the first two seconds of arterial green signal (maximum signal splits assumed for the subject and upstream intersection approaches),

CyclePerHour = number of signal cycles per hour,

Max_g2 = arrival pattern indicator variable, 1 if more than 25% of traffic arrives in the **second** half of the arterial **green** signal (maximum signal splits assumed for the subject and upstream intersection approaches),

TrTimeGt40 = **long** travel time indicator variable, 1 if the travel time of a vehicle moving at the speed limit between the upstream and the subject stop-lines is longer than 40 seconds, 0 otherwise,

R1_max = arrival pattern indicator variable, 1 if more than 25% of traffic arrives in the **first** half of the arterial **red** signal (maximum signal splits assumed for the subject and upstream intersection approaches),

$R1xBOR_max = R1_max \cdot BOR_max$,

SpeedLmt = posted speed limit (veh/h),

RightLane = exclusive right-turn lane indicator variable, 1 if right-turn lane present on the approach, 0 otherwise,

TrTimeLt15 = **short** travel time indicator variable, 1 if the travel time of a vehicle moving at the speed limit between the upstream and the subject stop-lines is shorter than 15 seconds, 0 otherwise,

ShortY = short yellow signal indicator variable, 1 if the yellow signal is shorter than calculated with the ITE equations, 0 otherwise,

Winter = winter months indicator variable, 1 if November, December, January, or February, 0 otherwise,

VolTotal = total volume arriving at the subject approach per continuous lane (veh/h).

The next step is calculating the likelihood of RE and RA crash and the conditional likelihood of severe crash given that the crash occurs. For that purpose expressions in (Eq. 8.10) are used.

$$\begin{aligned}
\Pr(\text{RE crash}) &= \exp(F_{\text{RE}})/[1+\exp(F_{\text{RE}})+\exp(F_{\text{RA}})] \\
\Pr(\text{RA crash}) &= \exp(F_{\text{RA}})/[1+\exp(F_{\text{RE}})+\exp(F_{\text{RA}})] \\
\Pr(\text{IF given RE crash}) &= 1/[1+\exp(S_{\text{RE}})] \\
\Pr(\text{PDO given RE crash}) &= 1 - \Pr(\text{IF crash given RE crash}) \\
\Pr(\text{IF given RA crash}) &= 1/[1+\exp(S_{\text{RA}})] \\
\Pr(\text{PDO given RA crash}) &= 1 - \Pr(\text{IF crash given RA crash})
\end{aligned}
\tag{Eq. 8.10}$$

where:

$\Pr(\text{RE crash})$ = probability of a **rear-end** crash on a subject arterial approach during a 15-minute interval,

$\Pr(\text{RA crash})$ = probability of a **right-angle** crash on a subject arterial approach during a 15-minute interval,

$\Pr(\text{IF given RE crash})$ = probability of a injury of fatal outcome if a **rear-end** crash happens,

$\Pr(\text{PDO given RE crash})$ = probability of a property-damage-only outcome if a **rear-end** crash happens,

$\Pr(\text{IF given RA crash})$ = probability of a injury of fatal outcome if a **right-angle** crash happens,

$\Pr(\text{PDO given RA crash})$ = probability of a property-damage-only outcome if a **right-angle** crash happens.

Finally, the likelihood of RE and RA crash at two levels of severity is calculated for each 15-minute interval and each arterial approach using the expressions in (Eq. 8.10).

$$\begin{aligned}
\Pr(\text{IF RE crash}) &= \Pr(\text{RE crash}) \cdot \Pr(\text{IF given RE crash}) \\
\Pr(\text{PDO RE crash}) &= \Pr(\text{RE crash}) \cdot \Pr(\text{PDO given RE crash}) \\
\Pr(\text{IF RA crash}) &= \Pr(\text{RA crash}) \cdot \Pr(\text{IF given RA crash}) \\
\Pr(\text{PDO RA crash}) &= \Pr(\text{RA crash}) \cdot \Pr(\text{PDO given RA crash})
\end{aligned}
\tag{Eq. 8.11}$$

where:

$\text{Pr}(\text{IF RE crash})$ = probability of a injury or fatal rear-end crash on a subject arterial approach during a 15-minute interval,

$\text{Pr}(\text{PDO RE crash})$ = probability of a property-damage-only rear-end crash on a subject arterial approach during a 15-minute interval,

$\text{Pr}(\text{IF RA crash})$ = probability of a injury or fatal right-angle crash on a subject arterial approach during a 15-minute interval,

$\text{Pr}(\text{PDO RA crash})$ = probability of a property-damage-only right-angle crash on a subject arterial approach during a 15-minute interval.

8.1.4. Aggregating the Crash Likelihood Values

Other than identifying the intervals with high crash likelihoods and isolating the possible causal factors, an important objective of developing the model was to assist traffic engineers in predicting the aggregated number of crashes of all types and for all coordinated arterial approaches at signalized intersections. Aggregating the likelihood values for the entire year and for all arterial approaches yields the expected number of crashes in that period and along the arterial street. Comparing alternative signal coordination plans becomes possible as explained in the general procedure at the beginning of this chapter.

For example, an engineer may want to predict the numbers of severe rear-end and right-angle crashes in a system. For this purpose, all the predicted likelihoods should be aggregated by crash type. For every combination of interval and approach, there are predicted crash likelihoods for rear-end PDO, rear-end IF, right-angle PDO, and right-angle IF. By totaling all the predicted likelihoods, an engineer then can obtain the estimated number of crashes for each crash type.

8.1.5. Model Calibration

An important issue when applying this aggregating method is the calibration of the model. A unique property of the multinomial logit model is that when the constant term is included, the corresponding aggregated predicted likelihood will equal the actually observed number of 1's. Taking the multinomial logit model to predict the crash likelihoods of rear-end and right-angle crashes developed in Section 6.2 as an example, the aggregated crash likelihoods for SR 28, SR 135, and SR 431 will exactly match the total number of observed rear-end and right-angle crashes. Therefore, we cannot check the aggregated predictive power of the model when the arterial-specific constant terms are also fitted.

The calibration issue also exists for the severity logit model. The constant term, in this case, determines the relative ratio of the PDO vs. IF crashes. The significance of SR431xRe variable implies that the general severity level on the SR-431 system is very different from the other arterials.

This shows the importance of model calibration when an engineer tries to use the model to obtain an aggregated prediction. The engineer should supply the model with the observed numbers of crashes for all types during the past years, Input these values together with the old signal timing and volume counts and then re-run the model only allowing the constant terms to be estimated. Then, the estimated constant terms can be used as calibrated coefficients in the model. Finally, the engineer can input the new signal timings and volume counts and try to predict the safety performance of the new timing plan.

This calibration is not necessary when the engineer is only interested in comparing the relative safety performance of several coordination plans for the same arterial system. The constant terms are only scaling factors and do not change the relative order of different predictions. In other words, the order of the predicted numbers of crashes is preserved no matter what the scaling factors are. For example, a properly calibrated model predicts that a plan will result in four rear-end PDO crashes while another plan will result in six rear-end PDO crashes. Without calibration, the same model may predict 20 and 50 crashes respectively for the two coordination plans. Plan 1 is always preferred over Plan 2 with or without calibration.

Calibration is still strongly recommended, although not necessary, even for the above scenario. The engineer often faces a trade-off between mobility and safety performance. The right magnitudes of crash predictions help the engineer evaluate the worthiness of a safer coordination plan. For instance, consider the same example in the previous paragraph. If Plan 1 has poorer mobility than Plan 2, the engineer has to decide whether the compromised mobility is worthwhile. If not calibrated, the safety benefit is 30 less RE-PDO crashes. This over-estimated value may push the engineer to choose Plan 1 over Plan 2. Nonetheless, a better estimate of the safety benefit is only four less RE-PDO crashes.

8.2. SafeArt - Step-by-step User Instructions

In this section, the developed SafeArt tool is described. SafeArt converts the input data, calculates the crash likelihoods for all time intervals and arterial approaches, and

aggregates the results for one year. SafeArt also calculates the calibration factors at request and use them in new calculations. The tool will be demonstrated using the actual data of several intersections at the studied SR-431 arterial system.

8.2.1. Starting the Tool

Open the Excel file; the first spreadsheet will be displayed as usual. The macro function of Excel must be enabled first. This setting can be changed through menu command of **Tools/Macro/Security**. Excel 2007 gives the security warning of the macro right under the ribbon bar as illustrated in Figure 8-8. Click the “Options” command and then choose “Enable this content”. Detail instructions of enabling macros can be found on the official website of Microsoft Excel (<http://office.microsoft.com/en-us/excel/default.aspx>).

Figure 8-8 Enable macro function of Excel

There are several sheets in this Excel workbook as shown in Figure 8-9. The “System Settings” sheet is the main user interface. Basic inputs are entered and main commands executed in this sheet. In addition, crash prediction results are summarized here. Additional inputs are entered in the “Geometry”, “Signal Timing”, and “Traffic Counts” sheets. These inputs must be entered before executing calculations in the “System Settings” sheet.

Figure 8-9 Sheets of this workbook

	A	B	C	D
1	Example using data of SR431			
2	(1) Basic System Parameters			
3		Model Selection	Default	
4		Number of Coordinated	<div> Default Volume Included </div>	
5			<div> Click the downward triangle button and select "Default" or "Volume Included" </div>	
6		Coordinated Approaches	NB	
7				
8			Hour	Minute
9		Start Time	11	30
10		End Time	12	30
11				
12				
13		Number of days in a week	5	(default 5)
14				
15				
16			Calibration Factor	
17		Rear-end	0.000	(default 0)
18		Right-angle	0.000	(default 0)
19				

Figure 8-10 Step (1) Setting system parameters

8.2.2. System Settings

Go to the A1-D20 cell range of the “System Settings” sheet and enter the basic input values. As illustrated in

Figure 8-10, the traffic engineer first needs to specify which model is used in calculating the crash predictions. There are two alternative models: without total traffic volume included (Default) and with total traffic volume included (Volume included). Although the default model is marginally better, the alternative model with the total volume may be preferred when the safety impact of changes in traffic are to be evaluated. Otherwise, the default model is recommended.

Next, the number of coordinated intersections, the coordinated approaches (arterial orientation), the hours and the number of days a week when the analyzed coordination plan is effective must be entered. Each setting has a valid range of values. Rather than inputting the values directly, the user selects a proper value from the drop down button on the right of the cell (Figure 8-11). The calibration parameters may be left at their default value 0 if they are not used or they are yet to be determined.

Number of days in a week	5	(default 5)
	<div> 1 2 3 4 5 6 7 </div>	
Rear-end		(default 0)
Right-angle	0.000	(default 0)

Figure 8-11 Select number of days from the drop down list

Scroll down to cell A20 and locate the box with crash counts (Figure 8-12). These inputs can be left blank if no calibration of the models is needed. The traffic engineer, however, is encouraged to enter the annual number of rear-end and right-angle crashes occurred at the involved intersections. Only the crashes occurred during the hours and days of week when the coordination plan was executed should be counted. For example, such

coordination periods should be on weekdays between 10:30 am to 12:00 am. In the presented example, these cells are left blank.

	A	B	C	D	E
20					
21			(2) Optional Step		
22				Annual Number of Crashes	
23			Rear-end		
24			Right-angle		
25					
26			These are only used in finding calibration factors.		
27			They can be left blank if calibration is not needed.		
28					

Figure 8-12 Step (2) crash statistics data

Scroll to cell F1 to check the intersection names table (Figure 8-13). The intersections follow a pre-specified order as shown in Figure 8-1 and Figure 8-2. The intersection names table is used to help the traffic engineer identify approaches in the detail results by user-defined names. Click the button (3) and input the crossing street names into this table. In the example, intersection 3 is at the 98th Street, intersection 6 is at the 106th Street, and intersection 9 is at the 116th Street.

Move to cell A35, locate the box with step (4). Button (4.1) *Reset all data input tables* **clears all the current input data** in the “Geometry”, “Signal Timing”, and “Traffic Counts” sheets and resizes these tables according to the current system settings. These tables are then populated with the previously entered names of crossing streets. It may take a few minute to complete this task depending on the speed of the computer and the size of the tables.

	E	F	G	H	I	J	K
1							
2		(3) Click this button to reset the list below. Then input intersection names. Scroll down to see next steps after input intersection names					
3							
4							
5		Intersection ID	Intersection Name				
6		3	98th Street
7		6	106 Street				
8		9	116 Street				
9							
10							
11							
12							
13							
14							
15							
16							
17							
18							
19							
20							
21							
22

Figure 8-13 Step (3) select number of days from the drop down list

	A	B	C	D	E
35					
36	(4) Set input tables and calculate predictions				
37		<div style="border: 1px solid black; padding: 5px; margin-bottom: 5px;">(4.1) Reset all data input tables</div> <div style="border: 1px solid black; padding: 5px; margin-bottom: 5px; background-color: yellow;">(4.2) Input data to Geometry, Signal Timing, and Traffic Counts worksheets.</div> <div style="border: 1px solid black; padding: 5px;">(4.3) Click this button after input all data to calculate crash predictions</div>			
38					
39					
40					
41					
42					
43					
44					
45					

Figure 8-14 Step (4) select number of days from the drop down list

8.2.3. Data Input

Move to the data input sheet “Geometry” (Figure 8-15). Data should be inputted only to the columns with green headers. Values in the columns with yellow headers are generated by the tool. Please notice that the distance is measured in feet and the speed limit in miles/hour. After inputting the geometry data, move to the “Signal Timing” sheet.

Intersection Name	Approach	Distance from Upstream Intersection (ft)	Speed Limit (mph)	Number of Exclusive Right-turn Lanes	Number of Through Lanes (One direction)
98th Street	NB	3456	50		
106 Street	NB	3716	50		
116 Street	NB	5400	50		
98th Street	SB	3716	50		
106 Street	SB	5400			
116 Street	SB	1930			

Figure 8-15 Input geometry data

The “Signal Timing” sheet is shown in Figure 8-16. Input data to the columns with green headers and do not edit the other columns. All the cycle, offset, split, yellow, and all-red intervals are measured in seconds. The offsets are the values obtained from Synchro using the “Begin of Red” option, as illustrated in Figure 8-3. Using other definitions of offsets will lead to incorrect results.

Jmns		Only Input Data To Green Columns				
Intersection Name	Approach	Cycle (sec)	Offset (sec)	Coordinated Movement Split (sec)	Yellow (sec)	AR (sec)
98th Street	NB	140	0	90	4.7	1.7
106 Street	NB	140	44	64	4.3	1.5
116 Street	NB	140	100	56	4.7	2
98th Street	SB	140	0	90	4.7	1.7
106 Street	SB	140	44	62	4.3	1.5
116 Street	SB	140	100	48	4.7	2

Figure 8-16 Input signal timings

Move to the “Traffic Counts” sheet and input the traffic count data for the corresponding intervals and traffic movements. As for other input data sheets, only columns with green headers are used.

Intersection Name	Hour	Minute	NBL	NBT	NBR	SBL	SBT	SBR	EBL	EBT	EBR	WBL	WBT	WBR
98th Street	11	30	2	323	4	6	300	0	9	1	15	19	3	18
98th Street	11	45	1	282	4	8	320	0	12	1	9	23	2	12
98th Street	12	0	4	219	2	6	312	1	5	2	3	28	1	8
98th Street	12	15	2	218	4	13	264	0	0	4	5	12	2	11
106 Street	11	30	7	244	14	13	207	5	14	20	8	12	37	5
106 Street	11	45	21	218	13	10	239	17	20	49	13	20	30	12
106 Street	12	0	19	246	18	12	228	14	18	39	9	14	30	12
106 Street	12	15	14	236	14	17	260	7	10	32	23	19	40	11
116 Street	11	30	120	219	12	70	253	12	71	101	87	37	160	20
116 Street	11	45	126	232	10	52	227	14	33	121	111	17	117	72
116 Street	12	0	153	283	12	119	276	15	19	139	90	13	114	49
116 Street	12	15	99	265	8	64	285	22	22	131	57	5	63	38

Figure 8-17 Input traffic counts

8.2.4. Calculations and Results

Move back to the “System Setting” sheet and click the button (4.3) located at cell B42 (Figure 8-18). This will initiate calculations. It may take several minutes for the tool to finish depending on the size of the system and the computer speed.

	A	B	C	D	E
41					
42					
43					
44					
45					

(4.3) Click this button after input all data to calculate crash predictions

Figure 8-18 Calculate crash predictions after data inputs

Scroll down to cell range A46-E60 and to see the summary of the predicted crashes (Figure 8-19) aggregated by crash type and severity. As shown in Figure 8-19, the

annual number of rear-end crashes between 11:30 am to 12:30 pm and on the three intersections (only coordinated arterial approaches) is 0.349. The annual number of right-angle crashes is 0.131.

For detail results, go to the "Report" sheet. The predictions are grouped by approach, intersection, and hour of coordination (Figure 8-20). This report can be used to identify problematic approaches and time intervals. Notice that the NB approach of 98th Street intersection and the SB approach of 116th Street have 0 predicted crashes. This does not mean that these approaches are safe. These approaches are the first in the coordinated direction and are not coordinated with any upstream traffic signals. Therefore, signal coordination plans do not affect them to the level other approaches are affected. These approaches are excluded from analysis.

	A	B	C	D
47	(5) Results			
48				
49		Predicted Annual Number of Crashes		
50	Crash Type	PDO	Injury & Fatal	Total
51	Rear-end	0.257	0.093	0.350
52	Right-angle	0.079	0.030	0.109
53	Total	0.336	0.123	0.459
54				
55	Recommended Calibration Factors			Calibration objective function
56	Rear-end			
57	Right-angle			
58				
59	For detail predictions, please see sheet "Report"			
60				
61				
62				

Figure 8-19 Brief results of crash prediction

	E	F	G	H	I	J	K	L	M	N
7										
8										
9		Approach	Intersection	Hour	Data					
10		NB	98th Street	11.0	0.000	0.000	0.000	0.000	0.000	0.000
11				12.0	0.000	0.000	0.000	0.000	0.000	0.000
12			98th Street Total		0.000	0.000	0.000	0.000	0.000	0.000
13			106th Street	11.0	0.029	0.011	0.019	0.010	0.007	0.003
14				12.0	0.044	0.017	0.029	0.014	0.011	0.005
15			106th Street Total		0.073	0.027	0.049	0.024	0.018	0.009
16			116th Street	11.0	0.031	0.011	0.022	0.009	0.010	0.001
17				12.0	0.047	0.018	0.035	0.012	0.017	0.002
18			116th Street Total		0.078	0.029	0.057	0.021	0.026	0.003
19		SB	98th Street	11.0	0.009	0.007	0.003	0.000	0.000	0.000
20				12.0	0.014	0.012	0.005	0.000	0.000	0.000
21			98th Street Total		0.023	0.019	0.007	0.000	0.000	0.000
22			106th Street	11.0	0.010	0.005	0.007	0.000	0.000	0.000
23				12.0	0.044	0.015	0.029	0.014	0.010	0.005
24			106th Street Total		0.087	0.027	0.062	0.025	0.015	0.012
25			116th Street	11.0	0.000	0.000	0.000	0.000	0.000	0.000
26				12.0	0.000	0.000	0.000	0.000	0.000	0.000
27		116th Street Total		0.000	0.000	0.000	0.000	0.000	0.000	
28		Grand Total			0.350	0.109	0.257	0.093	0.079	0.030

Figure 8-20 Crash predictions by approach, intersection, and hour

8.2.5. Model Calibration

Model calibration is discussed next. Assume that we have the actual annual number of rear-end and right-angle crashes and want the results from the tool match them. Input the annual crash counts to the table in step (2). There were five rear-end and one right-angle crashes at the analyzed intersections in 2003-2006 and during the 11:30 am - 12:30 pm period. Therefore, the average number of rear-end crashes per year was 1.25 and right-angle crashes was 0.25. These values have been entered to cells C23 and C24, respectively (Figure 8-21).

	A	B	C	D
19				
20				
21	(2) Optional Step			
22			Annual Number of Crashes	
23		Rear-end	1.25	
24		Right-angle	0.25	
25				
26		These are only used in finding calibration factors.		
27		They can be left blank if calibration is not needed.		
28				

Figure 8-21 Crash predictions by approach, intersection, and hour

The calibration parameters are automatically calculated and can be found in cells C56 and C57 as shown in Figure 8-22. These parameters can be copied to cells C17 and C18 (Figure 8-23) to obtain adjusted crash predictions. Click button (4.3) to re-calculate the crash predictions. The obtained adjusted total number of predicted rear-end crashes is 1.242 and the adjusted number of predicted right-angle crashes is 0.248.

	B	C	D
55	Recommended Calibration Factors		Calibration objective function
56	Rear-end	1.465	0.944
57	Right-angle	0.835	
58			

Figure 8-22 Recommended calibration factors

	B	C	D
15			
16		Calibration Factor	
17	Rear-end	1.465	(default 0)
18	Right-angle	0.835	(default 0)

Figure 8-23 Input calibration factors

The obtained crash predictions are close to the actual values 1.25 and 0.25 but do not match exactly which indicates that the calibration factors are not perfect due to certain simplifications made in the calibration procedure. The user has an option to improve the calibration results if the current ones are not satisfactory. To obtain better calibration parameters, the Excel solver function is used. Activate the solver and set the target cell as D56, where the calibration objective function is located. Let cells C17 and C18 change so as to minimize the target value in cell D56. The settings are shown in Figure 8-24. Click “solve” and wait for the results. Then, click “OK” as in Figure 8-25.

Figure 8-24 Use solver to find calibration factors

Figure 8-25 Keep solver solution

This time, the predicted numbers of rear-end and right-angle crashes exactly match the actual numbers. Click button (4.3) to redo calculations with the new calibration parameters to update the results in the “Report” sheet.

	B	C	D
47	(5) Results		
48			
49	Predicted Annual Number of Crashes		
50	PDO	Injury & Fatal	Total
51	0.913	0.337	1.250
52	0.177	0.073	0.250
53	1.090	0.410	1.500

Figure 8-26 Calibrated crash prediction

8.2.6. Evaluating Alternative Coordination Plans

To evaluate the safety performance of an alternative signal timing plan, the geometry and traffic counts data do not have to be input again. Only the “Signal Timing” sheet needs to be updated. Let the alternative coordination plan have cycle length 125 second and all other timings are the same. Go to sheet “Signal Timing” and change all the cycle settings to 125 seconds as shown in Figure 8-27.

	D	E	F	G	H	I	J
9	umns			Only Input Data To Green Columns			
10	Intersection Name	Approach	Cycle (sec)	Offset (sec)	Coordinated Movement Split (sec)	Yellow (sec)	AR (sec)
11	98th Street	NB	125	0	90	4.7	1.7
12	106 Street	NB	125	44	64	4.3	1.5
13	116 Street	NB	125	100	56	4.7	2
14	98th Street	SB	125	0	90	4.7	1.7
15	106 Street	SB	125	44	62	4.3	1.5
16	116 Street	SB	125	100	48	4.7	2

Figure 8-27 Update cycle length

Move back to sheet “System Settings” and click button (4.3) to obtain the new crash predictions (shown in Figure 8-28). Now, the total number of rear-end crashes is 1.033 and the number of right-angle is 0.205. The new numbers are lower than the previous prediction of 1.25 and 0.25. It indicates that the new cycle length has improved safety. The updated detail crash report is provided in the “Report” sheet (Figure 8-29).

	B	C	D
47	(5) Results		
48			
49	Predicted Annual Number of Crashes		
50	PDO	Injury & Fatal	Total
51	0.708	0.318	1.025
52	0.169	0.043	0.212
53	0.876	0.361	1.237
54			

Figure 8-28 Summary of crash prediction with new cycle length

	E	F	G	H	I	J	K	L	M	N
7										
8										
9										
10										
11										
12										
13										
14										
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27										
28										
29										

Approach	Intersection	Hour	Data						
			Rear-end	Right-angle	RE-PDO	RE-IF	RA-PDO	RA-IF	
NB	98th Street	11.0	0.000	0.000	0.000	0.000	0.000	0.000	
		12.0	0.000	0.000	0.000	0.000	0.000	0.000	
	98th Street Total		0.000	0.000	0.000	0.000	0.000	0.000	
	106th Street	11.0	0.120	0.021	0.080	0.040	0.014	0.007	
		12.0	0.147	0.033	0.099	0.048	0.023	0.011	
	106th Street Total		0.267	0.054	0.179	0.088	0.037	0.018	
	116th Street	11.0	0.112	0.020	0.081	0.031	0.018	0.002	
		12.0	0.151	0.036	0.113	0.039	0.033	0.003	
	116th Street Total		0.264	0.056	0.194	0.070	0.051	0.005	
	SB	98th Street	11.0	0.108	0.019	0.075	0.033	0.017	0.002
12.0			0.147	0.034	0.101	0.046	0.031	0.003	
98th Street Total		0.255	0.052	0.176	0.079	0.048	0.005		
106th Street		11.0	0.104	0.018	0.068	0.036	0.012	0.006	
		12.0	0.135	0.031	0.091	0.045	0.021	0.010	
106th Street Total		0.239	0.049	0.159	0.080	0.033	0.016		
116th Street		11.0	0.000	0.000	0.000	0.000	0.000	0.000	
		12.0	0.000	0.000	0.000	0.000	0.000	0.000	
116th Street Total		0.000	0.000	0.000	0.000	0.000	0.000		
Grand Total			1.025	0.212	0.708	0.318	0.169	0.043	

Figure 8-29 Detail crash predictions with new cycle length

CHAPTER 9. CLOSURE

9.1. Summary of Research Results

The primary objective of this research was to investigate the possible impact of signal coordination timing on traffic safety. To accomplish this goal, several tasks were completed as planned.

First, the crash patterns at coordinated signalized intersections were studied. It was found that most rear-end crashes occur when signals are red or turning green from red. The dilemma-zone related rear-end crashes account for only a small portion of all rear-end crashes. In addition, it was found that most right-angle crashes are caused by red-light-runners from both the arterial street and the crossing street. Permitted left turning violations are also a major cause of right-angle crashes. However, this movement is not controlled by arterial signal coordination and thus is excluded from later analysis.

Second, conceptual models were built to identify the possible predictive variables that will be used in later model development. It was hypothesized that the instant volume rates at the beginning of the red and at the beginning of the green have a significant correlation with crash likelihoods. In addition, the right-angle model suggested that the length of the yellow plus the all-red is negatively correlated with the likelihood of red-light running.

Third, data from a variety of sources were assembled to build a disaggregated dataset of 15-minute intervals. Each observation represents a combination of a 15-minute interval and an approach. Methods for deriving important traffic variables were also proposed. In particular, a method to characterize the traffic arrival pattern by a four-digit code was proposed. All the raw data and derived data were used in the model development.

Fourth, crash likelihood models were developed. A sequential multinomial logit model framework was proposed for the crash outcomes. At the first stage, a multinomial logit model was used to model the likelihood of both rear-end and right-angle crashes. At the second stage, a logit model was used to predict the proportion of PDO and IF crashes.

Fifth, with the developed models, a strong traffic pattern effect on the crash likelihood was found as follows:

- Crash likelihoods and right-angle crash severity levels were much lower when a high density vehicle platoon was scheduled to arrive in the second half of green.
- As hypothesized by the conceptual model and observed from the crash pattern analysis, the vehicles arriving at the beginning of the green are most susceptible to rear-end crashes.
- The vehicles that arrive at the beginning of the red, which are associated with the dilemma zone, are much less exposed to rear-end and right-angle risks. The effect is prominent only when high density of traffic is concentrated in first half of the red signal.
- For right-angle crashes, the model confirmed that the vehicles arriving at the beginning of the green were very susceptible to right-angle crashes. There was not enough evidence to support the correlation between traditional measures of coordination quality and the crash likelihood when the above variables were already included in the models.
- Short cycle length is found to be associated with lower crash risks.

Finally, some other factors that are not controlled by the arterial signal timing were also found to be significant factors in crash likelihood. Specifically, the existence of an exclusive right-turn lane, a lower speed limit, and a shorter distance from upstream intersections were found to reduce the risks of crashes. For a succinct summary of the risk factors, please refer to Table 7-1. To evaluate the magnitude of crash reduction, please refer to Table 7-3 and Table 7-4 for the risk reduction factors.

The knowledge obtained through this research will help traffic engineers evaluate the safety consequences of signal coordination. Traffic engineers can eliminate or reduce the signal solutions associated with unacceptably high, but formerly unnoticed, crash risks. When facing the choice of several timing plans of similar mobility performance, the results of this study can be used to single out the one with the lowest associated safety implications. As a result, the final signal plans will have slightly compromised mobility performance and lowered crash risks. When facing such trade-offs, it should be noted that the model calibration is crucial.

9.2. Future Extension

More valid crash data, especially for right-angle crashes, are expected to allow for the development of better model specification. It should be cautioned that the data validity must be checked. For example, it was found during this research that the actual offset settings at the intersections can be vastly different from the original documented timing plans due to the clock drift of signal controllers.

The general research framework can be readily extended to incorporate more variables that are potentially related to crash likelihoods. For example, weather information can be included as a predictive variable. Other possible factors include the sunlight condition, heavy vehicle presence, and ideally real-time traffic counts. Alternatively, the traffic counts can be adjusted by the monthly factors to obtain more reliable estimates.

In terms of methodology, it may be useful to explore a method to model the interdependence of the two stages of crash outcome. Crash occurrence and crash severity may not be independent. Such a possible correlation cannot be captured by the sequential logit framework. Alternative models which can address this issue should be explored. If more crash data are available, time series and panel data methods may be used to control for the unobserved temporal and spatial factors.

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Appendix A. Sample of a Police Crash Report⁵

⁵ Privacy Information Blocked

INDIANA OFFICER'S STANDARD CRASH REPORT										Report ☒ Original ☐ Supplemental Page <u>1</u> of <u>4</u>																																																																																							
State Form 2355R (Revised 5/03) Stock 302 Mail to: Indiana State Police, Crash Records Section 100 North Senate Avenue, Indianapolis, IN 46204										Local ID: <u>2006-1050</u>																																																																																							
Date of Crash Month <u>03</u> Day <u>06</u> Year <u>2006</u>		Day of Week <u>THURS</u>		Actual Local Time <u>5:41</u> ☐ AM ☒ PM		County <u>TIPICANOE</u>		Township <u>WABASH</u>		# Motor Vehicles <u>2</u>	# Injured <u>1</u>	# Dead <u>0</u>	# Commercial Vehicles <u>0</u>	# Deer <u>0</u>																																																																																			
Road Crash Occurred On <u>US HWY 52</u>				Nearest/Intersecting Road/Mile Marker/Interchange <u>MOREHOUSE RD</u>				If not at an intersection, number of feet from <u>APPROX 50'</u>		Direction <u>W</u>	Road Class. ☐ Interstate ☐ County Road ☐ US Road ☐ Local/City Road ☐ State Road ☐ Other																																																																																						
Inside Corporate Limits? ☐ Yes ☒ No		City/Town or Nearest City/Town <u>WEST LAFAYETTE</u>		Property? ☐ D N R ☐ Private ☐ Other		Crash Latitude		Crash Longitude																																																																																									
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Fill in only one Primary Cause for the crash Fill in up to two ovals per vehicle for Driver Contributing Circumstances Fill in one oval per vehicle for Vehicle and Environment Contributing Circumstances																																																																																																	
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<table border="1"> <thead> <tr> <th colspan="2">Witness/Other Participant</th> <th colspan="2">Non-Motorist</th> <th colspan="2">Non-Motorist Action</th> </tr> </thead> <tbody> <tr> <td>☐ Witness</td> <td># (Last Name, First Name, MI)</td> <td>☐ Pedestrian</td> <td>☐ Apparent Physical Condition</td> <td>☐ On designated non-motorist lane</td> <td>☐ On designated non-motorist lane</td> </tr> <tr> <td>☐ Other Participant</td> <td># (Last Name, First Name, MI)</td> <td>☐ Pedalcyclist</td> <td>☐ Normal</td> <td>☐ Not in roadway</td> <td>☐ Not in roadway</td> </tr> <tr> <td>☐ Other Participant</td> <td># (Last Name, First Name, MI)</td> <td>☐ Other</td> <td>☐ Had Been Drinking</td> <td>☐ On shoulder</td> <td>☐ On shoulder</td> </tr> <tr> <td>☐ Other Participant</td> <td># (Last Name, First Name, MI)</td> <td>☐ Other</td> <td>☐ Handicapped</td> <td>☐ On roadway</td> <td>☐ On roadway</td> </tr> <tr> <td>☐ Other Participant</td> <td># (Last Name, First Name, MI)</td> <td>☐ Other</td> <td>☐ Asleep/Fatigued</td> <td>☐ Against traffic</td> <td>☐ Against traffic</td> </tr> <tr> <td>☐ Other Participant</td> <td># (Last Name, First Name, MI)</td> <td>☐ Other</td> <td>☐ Drugs/Medication</td> <td>☐ Crossing at intersection</td> <td>☐ Crossing at intersection</td> </tr> <tr> <td>☐ Other Participant</td> <td># (Last Name, First Name, MI)</td> <td>☐ Other</td> <td>☐ Unknown</td> <td>☐ Crossing not at intersection</td> <td>☐ Crossing not at intersection</td> </tr> <tr> <td>☐ Other Participant</td> <td># (Last Name, First Name, MI)</td> <td>☐ Other</td> <td>☐ Unknown</td> <td>☐ Moving</td> <td>☐ Moving</td> </tr> <tr> <td>☐ Other Participant</td> <td># (Last Name, First Name, MI)</td> <td>☐ Other</td> <td>☐ Unknown</td> <td>☐ Standing</td> <td>☐ Standing</td> </tr> <tr> <td>☐ Other Participant</td> <td># (Last Name, First Name, MI)</td> <td>☐ Other</td> <td>☐ Unknown</td> <td>☐ Working</td> <td>☐ Working</td> </tr> <tr> <td>☐ Other Participant</td> <td># (Last Name, First Name, MI)</td> <td>☐ Other</td> <td>☐ Unknown</td> <td>☐ Getting in or out of a vehicle</td> <td>☐ Getting in or out of a vehicle</td> </tr> <tr> <td>☐ Other Participant</td> <td># (Last Name, First Name, MI)</td> <td>☐ Other</td> <td>☐ Unknown</td> <td>☐ Getting off or on a school bus</td> <td>☐ Getting off or on a school bus</td> </tr> <tr> <td>☐ Other Participant</td> <td># (Last Name, First Name, MI)</td> <td>☐ Other</td> <td>☐ Unknown</td> <td>☐ Other (Explain in Narrative)</td> <td>☐ Other (Explain in Narrative)</td> </tr> </tbody> </table>														Witness/Other Participant		Non-Motorist		Non-Motorist Action		☐ Witness	# (Last Name, First Name, MI)	☐ Pedestrian	☐ Apparent Physical Condition	☐ On designated non-motorist lane	☐ On designated non-motorist lane	☐ Other Participant	# (Last Name, First Name, MI)	☐ Pedalcyclist	☐ Normal	☐ Not in roadway	☐ Not in roadway	☐ Other Participant	# (Last Name, First Name, MI)	☐ Other	☐ Had Been Drinking	☐ On shoulder	☐ On shoulder	☐ Other Participant	# (Last Name, First Name, MI)	☐ Other	☐ Handicapped	☐ On roadway	☐ On roadway	☐ Other Participant	# (Last Name, First Name, MI)	☐ Other	☐ Asleep/Fatigued	☐ Against traffic	☐ Against traffic	☐ Other Participant	# (Last Name, First Name, MI)	☐ Other	☐ Drugs/Medication	☐ Crossing at intersection	☐ Crossing at intersection	☐ Other Participant	# (Last Name, First Name, MI)	☐ Other	☐ Unknown	☐ Crossing not at intersection	☐ Crossing not at intersection	☐ Other Participant	# (Last Name, First Name, MI)	☐ Other	☐ Unknown	☐ Moving	☐ Moving	☐ Other Participant	# (Last Name, First Name, MI)	☐ Other	☐ Unknown	☐ Standing	☐ Standing	☐ Other Participant	# (Last Name, First Name, MI)	☐ Other	☐ Unknown	☐ Working	☐ Working	☐ Other Participant	# (Last Name, First Name, MI)	☐ Other	☐ Unknown	☐ Getting in or out of a vehicle	☐ Getting in or out of a vehicle	☐ Other Participant	# (Last Name, First Name, MI)	☐ Other	☐ Unknown	☐ Getting off or on a school bus	☐ Getting off or on a school bus	☐ Other Participant	# (Last Name, First Name, MI)	☐ Other	☐ Unknown	☐ Other (Explain in Narrative)	☐ Other (Explain in Narrative)
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Page 2 of 4																				
Type of Crash <div style="display: flex; flex-wrap: wrap; padding: 5px;"> <div style="width: 33%;">☒ Rear End</div> <div style="width: 33%;">☐ Same Direction Sideswipe</div> <div style="width: 33%;">☐ Right Angle</div> <div style="width: 33%;">☐ Backing Crash</div> <div style="width: 33%;">☐ Head On</div> <div style="width: 33%;">☐ Opposite Direction Sideswipe</div> <div style="width: 33%;">☐ Left Turn</div> <div style="width: 33%;">☐ Other</div> <div style="width: 33%;">☐ Rear to Rear</div> <div style="width: 33%;">☐ Ran off Road</div> <div style="width: 33%;">☐ Right Turn</div> <div style="width: 33%;">☐ Non-Collision</div> <div style="width: 33%;">☐ Left/Right Turn</div> </div>																						
Diagram: (Indicate North by Arrow) <div style="text-align: right; margin-top: 10px;"> N </div> <div style="text-align: center; margin-top: 20px;"> </div>																						
Narrative: <p> VEH#2 WAS STOPPED IN THE LEFT HAND LANE OF E/B US HWY 52 AT MOREHOUSE RD FOR A RED LIGHT. DUE TO THE WET ROAD CONDITIONS AND SPEED UNREASONABLE FOR THE CONDITIONS, VEH#1 TRAVELED IN THE LEFT HAND LANE OF E/B US HWY 52 AND STRUCK THE REAR OF "STOPPED" VEH#2. DR#2 STATED SHE WAS STOPPED BEHIND A VEHICLE FOR THE RED TRAFFIC SIGNAL, AND HEARD "TIRE SQUEALS" BEFORE BEING STRUCK FROM BEHIND. DR#1 STATED SHE WAS UNABLE TO STOP AND "SLID" INTO VEH#2. </p>																						
<div style="display: flex; justify-content: space-between; font-size: small;"> Time Notified ☐ AM ☒ PM Time Arrived ☐ AM ☒ PM Other Location of Investigation </div> <div style="display: flex; justify-content: space-between;"> 5:45 5:45 ST. ELIZABETH'S HOSPITAL </div> <table border="1" style="width: 100%; border-collapse: collapse; font-size: x-small;"> <tr> <td style="width: 33%;">Assisting Officer</td> <td style="width: 15%;">ID No.</td> <td style="width: 33%;">Agency</td> <td style="width: 19%;">Investigation Complete? ☒ Yes ☐ No</td> <td style="width: 10%;">Photos Taken? ☐ Yes ☒ No</td> </tr> <tr> <td>Assisting Officer</td> <td>ID No.</td> <td>Agency</td> <td colspan="2">Date of Report</td> </tr> <tr> <td>Investigating Officer (printed)</td> <td>ID No.</td> <td>Agency</td> <td colspan="2">03-09-2006</td> </tr> <tr> <td></td> <td></td> <td>TERRELL CO. SHERIFF</td> <td colspan="2">Reviewing Officer</td> </tr> </table>			Assisting Officer	ID No.	Agency	Investigation Complete? ☒ Yes ☐ No	Photos Taken? ☐ Yes ☒ No	Assisting Officer	ID No.	Agency	Date of Report		Investigating Officer (printed)	ID No.	Agency	03-09-2006				TERRELL CO. SHERIFF	Reviewing Officer	
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		TERRELL CO. SHERIFF	Reviewing Officer																			

UNIT INFORMATION				Page 3 of 4	
Unit ID: <u>2006-1050</u>					
Driver Information Driver's Name (Last, First, MI): <u>[REDACTED]</u> Address (Street, City, State, Zip): <u>[REDACTED]</u> Date of Birth: <u>[REDACTED]</u> Age: <u>[REDACTED]</u> Driver's License #: <u>[REDACTED]</u> Lic Type: <u>OP</u> CDL Class: <u>[REDACTED]</u> Lic State: <u>IN</u>				Safety Equipment Used ☐ No restraint ☐ Helmet ☐ Lap Belt Only ☐ Airbag (No Restraints) ☐ Harness (Only) ☐ Airbag + Belt Restraint ☒ Lap + Harness ☐ Child Restraint ☐ Unknown Safety Equipment Effective? ☒ Yes ☐ No ☐ Not Applicable	
Apparent Physical Status ☒ Normal ☐ Glasses/Contact Lenses ☐ Outside Rearview Mirror ☐ Daylight Driving ☐ Handicapped ☐ Ill ☐ Asleep/Fatigued ☐ Drugs/Medication ☐ Unknown ☐ Had Been Drinking ☐ Special Controls ☐ Employment Only ☐ Motorcycle Only ☐ To/From Employment Restrictions ☐ Employer's Vehicle Only ☐ State-Owned Vehicles Only ☐ PP Chauffeurs-Taxi Only ☐ Power Steering ☐ Special Restrictions ☐ Probation (DWI) ☐ Probation HTV ☐ None				Location of Most Severe Injury ☐ Chest ☐ Neck ☐ Eye ☐ Face ☐ Head ☐ Back ☐ Shoulder/Upper Arm ☐ Elbow/Lower Arm ☐ Abdomen/Pelvis ☐ Hip/Upper Leg ☐ Knee/Lower Leg/Foot ☐ Entire Body	
Gender ☐ Male ☐ Female ☐ Unknown Test Given ☐ None ☐ Alcohol ☐ Drug ☐ Alcohol+Drug ☐ Refused Type Given ☐ Blood ☐ Urine ☐ Breath ☐ SFST ☐ PBT Results ☐ PBT - ☐ Certified ☐ Pending ☐ Positive ☐ Negative ☐ Pending				Driver Injury Status ☐ Fatal Injury ☐ Non-Fatal Injury ☐ Incapacitating ☐ Non-Incapacitating ☐ Possible Injury ☐ Unknown ☐ Refused Nature of Most Severe Injury ☐ Severed ☐ Internal ☐ Minor Burn ☐ Severe Burn ☐ Abrasion ☐ Minor Bleeding ☐ Severe Bleeding (Arterial) ☐ Fracture/Dislocation ☐ Contusion/bruise ☐ Complaint of Pain ☐ None Visible ☐ Other (Explain in Narrative)	
Vehicle Information Veh# <u>1</u> Color <u>GRY</u> Vehicle Year <u>2000</u> Make <u>CHEVROLET</u> Model Name <u>IMPALA</u> Style <u>4D</u> # Occupants <u>5</u> Lic Year <u>2005</u> License # <u>[REDACTED]</u> License State <u>INDIANA</u> # Axes <u>2</u> Speed Limit <u>55</u> Insured By <u>Natl. Auto Ins. Inc.</u> Phone Number <u>[REDACTED]</u> Registered Owner's Name (Last, First, MI) <u>[REDACTED]</u> Same as Driver ☒ Address (Street, City, State, Zip) <u>[REDACTED]</u>				Initial Impact Area ☐ Undercarriage ☐ Trailer ☐ None ☐ Unknown Areas Damaged (Multiples) ☐ Undercarriage ☐ Trailer ☐ None ☐ Unknown	
Vehicle Use ☒ Personal (Farm, Company) ☐ Fire* ☐ Ambulance* ☐ Military ☐ Common and Contract Carriers ☐ Highway Department ☐ Rental, not leased ☐ Other Government (Postal, etc) ☐ Public Utilities (Gas, Electric, etc) ☐ School ☐ Police* ☐ Other (Explain in Narrative) Emergency ☐ Run? ☐ Yes ☐ No ☐ Fire? ☐ Yes ☐ No				Vehicle Type ☒ Passenger Car/Station Wagon ☐ Tractor (Cab Only-No Trailer) ☐ Pickup ☐ Motor Home/Recreational Vehicle ☐ Van ☐ Motorcycle ☐ Bus/Seats 9-15 Persons including the driver ☐ Bus/Seats 16+ Persons including the driver ☐ School Bus ☐ Farm Vehicle ☐ Tractor/Trailer (not semi) ☐ Combination Vehicle ☐ Tractor/Double Trailers ☐ Unknown Type (not classified) ☐ Tractor/Triples Trailers ☐ Moped	
Crash Details Towed? ☒ Yes ☐ No Towed To <u>[REDACTED]</u> Towed By <u>[REDACTED]</u> Tri# <u>[REDACTED]</u> Lic State <u>[REDACTED]</u> Lic Year <u>[REDACTED]</u> Registered Owner's Name (Last, First, MI) <u>[REDACTED]</u> Same as Driver ☐ License # <u>[REDACTED]</u> Address (Street, City, State, Zip) <u>[REDACTED]</u> Veh Year <u>[REDACTED]</u> Make <u>[REDACTED]</u> Tri# <u>[REDACTED]</u> Lic State <u>[REDACTED]</u> Lic Year <u>[REDACTED]</u> Registered Owner's Name (Last, First, MI) <u>[REDACTED]</u> Same as Driver ☐ License # <u>[REDACTED]</u> Address (Street, City, State, Zip) <u>[REDACTED]</u> Veh Year <u>[REDACTED]</u> Make <u>[REDACTED]</u> Veh# <u>[REDACTED]</u> Commercial Vehicle: Carrier's Name and Address <u>[REDACTED]</u>				Pre-Crash Vehicle Action ☒ Going Straight ☐ Turning Left ☐ Slowing or Stopped in Traffic ☐ Backing ☐ Making U Turn ☐ Unattended Moving Vehicle ☐ Changing Lanes ☐ Merging ☐ Avoiding Object in Road ☐ Overtaking/Passing ☐ Starting in Traffic ☐ Entering Traffic Lane ☐ Turning Right ☐ Driving Left of Center ☐ Leaving Traffic Lane ☐ Crossing the Median ☐ Parked Direction of Travel ☐ North ☒ East ☐ Northeast ☐ Southwest ☐ South ☐ West ☐ Northwest ☐ Southwest	
HAZMAT Proper Shipping Name: US DOT# <u>[REDACTED]</u> ICC# <u>[REDACTED]</u> State DOT# <u>[REDACTED]</u> Vehicle Identification # <u>[REDACTED]</u> CMV Inspection? ☐ Yes ☐ No If ☐ L1 ☐ L3 Gross Vehicle Weight Rating (GVWR) ☐ Less than 10,000# ☐ 10,001-25,000# ☐ 26,001# or more Cargo Body Type ☐ Grain, Chip ☐ Van/Enclosed Box ☐ Auto Transport ☐ Gravel, Coal ☐ Cargo Tank ☐ Pole ☐ Flatbed ☐ Garbage/Refuse ☐ Other (Explain in Narrative) ☐ Dump ☐ Ruse ☐ Concrete Mixer				Type of Primary/Secondary Roadway One-Way Traffic ☐ One Lane ☐ Two Lanes ☐ Multi-Lanes (3 or more) Two-Way Traffic ☐ Two Lanes ☒ Multi-Lane Divided (3 or more) ☐ Private Drive ☐ Multi-Lanes Undivided 2 way left turn ☐ Alley ☐ Multi-Lane Undivided (3 or more)	
HAZMAT Release of Cargo ☐ Yes ☐ No HAZMAT 4-Digit ID # <u>[REDACTED]</u> Hazard Class # <u>[REDACTED]</u>				If a Collision Crash (Fill in only one oval in this category) ☒ Another Motor Vehicle ☐ Deer ☐ Railway Vehicle ☐ Pedestrian ☐ Animal Other than Deer ☐ Fence ☐ Bicycle ☐ Animal Drawn Vehicle ☐ Mailbox ☐ Impact Attenuator/Crash Cushion ☐ Overhead Sign Post ☐ Tree ☐ Bridge Overhead Structure ☐ Light Support ☐ Curb ☐ Bridge Pier or Abutment ☐ Utility Pole ☐ Ditch ☐ Bridge Parapet End ☐ Culvert ☐ Embankment ☐ Other Post/Pole/In Support ☐ Guardrail Face ☐ Wall/Building/Tunnel, etc ☐ Median Barrier ☐ Work Zone Maintenance Equip. ☐ Highway Traffic Sign Post ☐ Other (Explain in Narrative)	
HAZMAT Placed ☐ Yes ☐ No HAZMAT Release of Cargo ☐ Yes ☐ No HAZMAT 4-Digit ID # <u>[REDACTED]</u> Hazard Class # <u>[REDACTED]</u>				If a Non-Collision Crash (Fill in only one oval in this category) ☐ Overturn/Rollover ☐ Jackknife ☐ Fell from vehicle ☐ Fire/Explosion ☐ Cargo/Equipment Shift or Loss ☐ Immersion ☐ Off Roadway	

UNIT INFORMATION				Page 4 of 4	
Unit ID: 2006-1050					
Driver's Name (Last, First, MI) [Redacted]		Safety Equipment Used ☐ No restraint ☐ Lap Belt Only ☐ Harness (Only) ☒ Lap + Harness ☐ Child Restraint		Safety Equipment Effective? ☒ Yes ☐ No ☐ Not Applicable	
Address (Street, City, State, Zip) WEST LAFAYETTE, IN 47906		Date Month Day Year Age [Redacted]		Ejection/Trapped ☒ Not Ejected or Trapped ☐ Partially Ejected ☐ Ejected ☐ Trapped In ☐ Pinned Under ☐ Unknown	
Driver's License # [Redacted]		Lic Type DP		CDL Class IN	
Apparent Physical Status ☒ Normal ☐ Had Been Drinking ☐ Handicapped ☐ Ill ☐ Asleep/Fatigued ☐ Drugs/Medication ☐ Unknown		Restrictions ☐ Glasses/Contact Lenses ☐ Outside Rearview Mirror ☐ Daylight Driving ☐ Automatic Transmission ☐ Special Controls ☐ Employment Only ☐ Motorcycle Only ☐ To/From Employment		Employer's Vehicle Only ☐ State Owned Vehicles Only ☐ PP Chauffeurs-Taxi Only ☐ Power Steering ☐ Special Restrictions ☐ Probation DWI ☐ Probation MTV ☐ None	
Gender ☐ Male ☐ Female ☐ Unknown		Test Given ☒ None ☐ Alcohol ☐ Drug ☐ Alcohol+Drug ☐ Refused		Alcohol Results ☐ PBT - ☐ Certified ☐ Test - ☐ Pending	
Drug Results ☐ Positive ☐ Negative ☐ Pending		EMS No. 1855		Nature of Most Severe Injury ☐ Severed ☐ Internal ☐ Minor Burn ☐ Severe Burn ☐ Abrasion ☐ Minor Bleeding ☐ Severe Bleeding (Arterial) ☐ Fracture/Dislocation ☐ Contusion/Brise ☒ Complaint of Pain ☐ None Visible ☐ Other (Explain in Narrative)	
Location of Most Severe Injury ☐ Chest ☒ Neck ☐ Eye ☐ Face ☐ Head ☐ Back ☐ Shoulder/Upper Arm ☐ Elbow/Lower Arm ☐ Abdomen/Pelvis ☐ Hip/Upper Leg ☐ Knee/Lower Leg/Foot ☐ Entire Body		Driver Injury Status ☐ Fatal Injury ☐ Non-Fatal Injury ☐ Incapacitating ☒ Non-Incapacitating ☐ Possible Injury ☐ Unknown ☐ Refused		If Cited? ☐ Infraction ☐ Misdemeanor ☐ Felony	
IC Code [Redacted]		IC Code [Redacted]		IC Code [Redacted]	
Veh # 2		Color WHI		Vehicle Year 1998	
Make OLDSMOBILE		Model Name SELTOWING		Style SV	
# Occupants 3		Lic Year 2005		License # IN 02111111	
# Axes 2		Speed Limit 55		Insured By FARMERS	
Registered Owner's Name (Last, First, MI) [Redacted] Same as Driver					
Address (Street, City, State, Zip) [Redacted]					
Towed? ☒ Yes ☐ No Towed To LAFAYETTE Towed By JR'S TOWING					
Tri# 1		Lic State IN		Lic Year 2005	
License # IN 02111111		Registered Owner's Name (Last, First, MI) [Redacted] Same as Driver			
Veh Year 1998		Make OLDSMOBILE			
Tri# 1		Lic State IN		Lic Year 2005	
License # IN 02111111		Registered Owner's Name (Last, First, MI) [Redacted] Same as Driver			
Veh Year 1998		Make OLDSMOBILE			
Commercial Vehicle: Carrier's Name and Address [Redacted]					
HAZMAT Proper Shipping Name: [Redacted]					
US DOT# [Redacted]		ICC# [Redacted]		State DOT# [Redacted]	
Vehicle Identification # [Redacted]		CMV Inspection? If ☐ 1 ☐ 2 ☐ 3 ☐ Yes ☐ No			
Gross Vehicle Weight Rating (GVWR) ☐ Less than 10,000# ☐ 10,001-26,000# ☐ 26,001# or more		Cargo Body Type ☐ Grain, Chlp, Gravel, Coal ☐ Flatbed ☐ Dump ☐ Bus ☐ Van/Enclosed Box ☐ Cargo Tank ☐ Garbage/Refuse ☐ Concrete Mixer ☐ Auto Transport ☐ Pole ☐ Other (Explain in Narrative)			
HAZMAT Placard ☐ Yes ☐ No		HAZMAT Release of Cargo ☐ Yes ☐ No		HAZMAT 4-Digit ID # [Redacted]	
Hazard Class # [Redacted]		HAZMAT 4-Digit ID # [Redacted]			
Vehicle Use ☐ Personal (Farm, Company) ☐ Commercial (Buses, Taxis, Common and Contract Carriers) ☐ Rental, not leased ☐ School ☐ Police* ☐ Fire* ☐ Ambulance* ☐ Military ☐ Highway Department ☐ Other Government (Postal, etc) ☐ Public Utilities (Gas, Electric, etc) ☐ Other (Explain in Narrative)					
Emergency Run? ☐ Yes ☐ No					
Vehicle Type ☐ Passenger Car/Station Wagon ☐ Pickup ☒ Van ☐ Sport Utility Vehicle ☐ Truck (Single Unit 2 axle, 6 tires) ☐ Truck (Single Unit 3 or more axles) ☐ Tractor/Trailer (not semi) ☐ Tractor/Double Trailer ☐ Tractor/Triples Trailers ☐ Tractor (Cab Only No Trailer) ☐ Motor Home/Recreational Vehicle ☐ Motorcycle ☐ Bus/Seats 9-15 Persons including the driver ☐ Bus/Seats 16+ Persons including the driver ☐ School Bus ☐ Farm Vehicle ☐ Combination Vehicle ☐ Unknown type (not classified) ☐ Moped					
Pre-Crash Vehicle Action ☐ Turning Left ☐ Making U Turn ☐ Backing ☐ Changing Lanes ☐ Overtaking/Passing ☐ Turning Right ☐ Slowing or Stopped in Traffic ☐ Unattended Moving Vehicle ☐ Avoiding Object in Road ☐ Entering Traffic Lane ☐ Leaving Traffic Lane ☐ Parked					
Direction of Travel ☐ North ☒ East ☐ Northeast ☐ Southeast ☐ South ☐ West ☐ Northwest ☐ Southwest					
Type of Primary/Secondary Roadway One Way Traffic ☐ One Lane ☐ Two Lanes ☐ Multi-Lanes (3 or more) Two Way Traffic ☐ Two Lanes ☒ Multi-Lane Divided (3 or more) ☐ Multi-Lane Undivided 2 way left turn ☐ Multi-Lane Undivided (3 or more)					
If a Collision Crash Fill in only one oval in this category: ☒ Another Motor Vehicle ☐ Pedestrian ☐ Bicycle ☐ Impact Attenuator/Crash Cushion ☐ Bridge Overhead Structure ☐ Bridge Pier or Abutment ☐ Bridge Parapet End ☐ Bridge Rail ☐ Guardrail Face ☐ Guardrail End ☐ Median Barriers ☐ Highway Traffic Sign Post ☐ Deer ☐ Animal Other than Deer ☐ Animal Drawn Vehicle ☐ Overhead Sign Post ☐ Light Support ☐ Utility Pole ☐ Culvert ☐ Embankment ☐ Other Post/Pole/or Support ☐ Wall/Building/Tunnel, etc ☐ Work Zone Maintenance Equip. ☐ Other (Explain in Narrative)					
Or if a Non Collision Crash Fill in only one oval in this category: ☐ Overturn/Rollover ☐ Fire/Explosion ☐ Immersion ☐ Jackknife ☐ Large Equipment Shift or Loss ☐ Off Roadway ☐ Fell from vehicle					