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Effect of Information on Driver's Risk at the Onset of Yellow

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EFFECT OF INFORMATION ON DRIVER’S RISK AT THE ONSET OF YELLOW AT HIGH-SPEED INTERSECTIONS

by

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EFFECT OF INFORMATION ON DRIVER'S RISK AT THE ONSET OF YELLOW AT HIGH-SPEED INTERSECTIONS

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University of Nebraska, 2011

Advisor: Anuj Sharma

Intersection crashes average approximately 8,500 fatal and 900,000 injury accidents a year (*1*). At the onset of yellow at a high speed signalized intersection, a driver may encounter a region of the intersection approach, where they can neither stop safely nor be able to clear the intersection before the red indication. A wrong decision to stop when it would have been safer to proceed can lead to a severe rear-end collision. Conversely, a wrong decision to proceed through the intersection could lead to the driver running the red light and possibly causing a right angle collision. The traditional surrogate measure of safety, dilemma zone, denotes the region of risk but does not quantify the level of risk.

Driver's data was collected at five high speed intersections. A probit modeling technique was used to establish dilemma zone boundaries. Results revealed the effects of providing or lack of providing information. Specifically, the results indicate the effects AWFs have on the probability of stopping and perceived conflict curves. Sites providing information through PTSWF had earlier probability of stopping curves in particular Site 2 and Site 5's probability of stopping curves were drastically different than the other studied sites. The risk associated with being downstream of the severe deceleration distance and upstream of the maximum passing distance was calculated for a variety of speeds at each intersection. An overall weighted average was then computed and compared to the

observed conflicts. An association could be seen in the comparison between the observed conflicts and the computed risks, as sites with larger severe deceleration risk had a larger proportion requiring severe deceleration and vice versa. Thus, caution should be used by engineers before providing drivers with information at a high speed intersection.

DEDICATION – SOLO DE GLORIA

This thesis is dedicated to my family for all the love, support, and encouragement they provided throughout this lengthy process. I am indebted to my parents, Lance and Iva Burnett, who instilled in me at an early age the importance of a quality education. I am thankful to my wife's parents, who have provided their love and support to me more than I could have asked for. To my wife, Rachel, thank you for your patience, inspiration, and belief in me. It was a journey that would have been less joy filled had you not been in my life. To Moses, thank you for keeping me grounded, especially as we travel throughout Lincoln and you help daddy know what he should be doing when approaching an intersection. Lastly, to Grace, when I started this process you weren't even thought of, and now you are five months old and have provided me with many relaxing breaks with you in my arms.

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CHAPTER 1. INTRODUCTION

1.1. Motivation

According to the National Highway Traffic Safety Administration, the total cost of motor vehicle collisions in the United States was estimated at $230.6 billion in 2009 (1). The total cost of motor vehicle collisions in the State of Nebraska was projected at $2.2 billion in 2009 (2). Intersection and intersection-related crashes accounted for nearly 40.1 percent of all reported crashes in 2006 in the U.S (1). Intersection crashes average approximately 8,500 fatal and 900,000 injury accidents a year. Multi-vehicle accidents at intersections in Nebraska comprised of 46 percent of the total reported crashes in 2009 (2). With the exception of 2001, the percentage of multi-vehicle collisions at intersections has stayed relatively constant in Nebraska, as shown in Figure 1.1 (2,3,4,5,6,7,8,9,*10*,*11*,*12*).

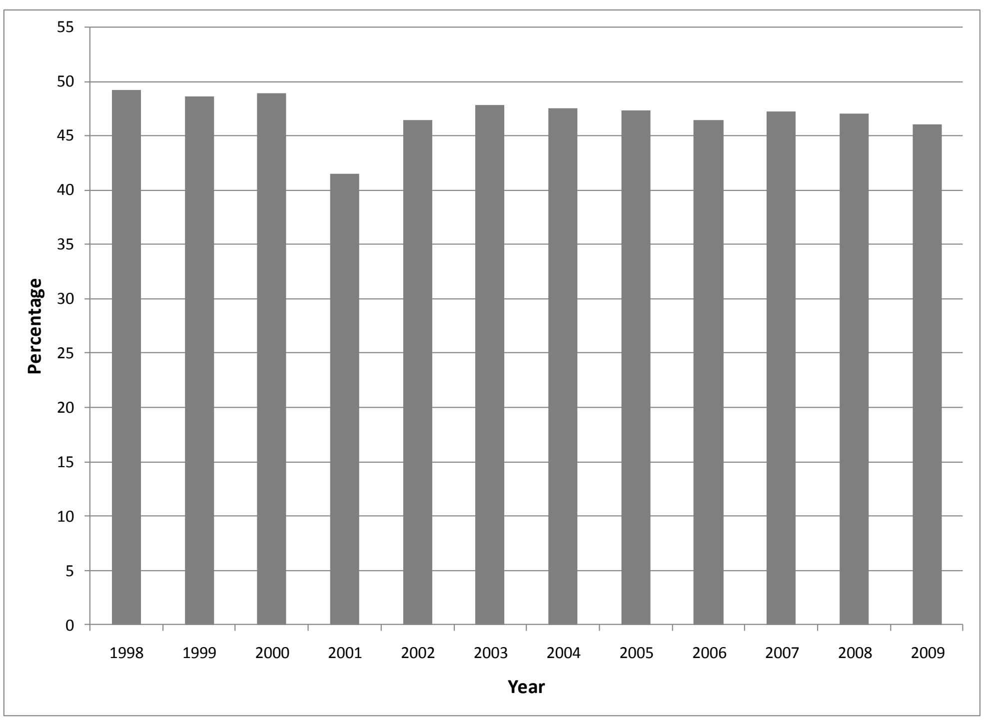


Figure 1.1: Percentage of total crashes multi-vehicle collisions consist of in Nebraska

Every day a typical intersection has approximately 700-800 occurrences of main-street phase terminations transpire where high-speed drivers approaching an intersection have to make a decision on whether to proceed or stop at the onset of yellow (*13*). While approaching the intersection, a driver may encounter being in the decision dilemma zone at the onset of yellow. The dilemma zone has traditionally been defined as the area where a driver can neither stop comfortably nor clear the intersection safely at the onset of yellow. However, the decision dilemma zone has been defined by previous literature as the approach area where the probability of stopping at the onset of yellow is within the range of 10 to 90 percent (*14*,*15*,*16*,*17*). An incorrect decision to stop when it would have been safer to proceed can lead to a severe rear-end collision. Conversely, an incorrect decision to proceed through the intersection could lead to the driver running the red light and possibly causing a right angle collision.

As shown below in Table 1.1, the most frequently occurring collisions at intersections in Nebraska are angle and rear-end collisions, which represent approximately eighty percent of intersection crashes since 1998 (2,3,4,5,6,7,8,9,*10*,*11*,*12*).

Table 1.1: Proportion of crashes by collision type at Nebraska intersections

Crash Type	Percent of Intersection Crashes
Angle	48.63
Rear-end	32.31
Left Turn Leaving	9.74
Sideswipe	6.38
Backing	2.52
Head-on	0.37
Unknown	0.06

Traditional surrogate measures of safety (like number of vehicles in dilemma zone) fail to quantify the risk of crash for a driver approaching an intersection, as a result of not quantifying the positions of vehicles in the dilemma zone. Over the past four decades, the Traffic Conflict Technique (TCT) has evolved and demonstrated its usefulness in indirectly evaluating the safety of intersections. TCT allows traffic engineers the opportunity to provide proactive safety improvements at an intersection instead of waiting for the crash history to evolve. Cooper and Ferguson (*18)* calculated the ratio of the rate of serious conflicts to the rate of crashes to be approximately 2000:1. Therefore, two or three years of reported crash records at an intersection could be observed with 10 hours of conflicts. A recent study by FHWA (*19*) found the ratio of traffic conflicts to actual crashes to be approximately 20,000:1. In addition to quicker data collection, the second advantage of the conflict technique is in identifying safety deficiencies at intersections.

Engineers have been studying the factors associated with rear-end and angle collisions at signalized intersections for decades in an effort to provide maximum safety to drivers approaching intersections. Numerous solutions have been proposed and implemented, specifically at high-speed intersections including: green extension, Self Optimising Signal Control, D-CS, and Advance Warning Flashers (AWFs). Placed upstream of high speed signalized intersections, AWFs provide drivers with information regarding whether they should prepare to stop at the upcoming traffic signal or proceed through the intersection. Specifically, AWFs are designed to minimize the number of vehicles trapped in their respective dilemma zones at the onset of yellow (*20*). Past research

(*20,21,22)* has revealed significant reductions in RLRs at intersections with AWFs; however, rear-end crash potential has increased as a result (*23*,*24*,*25*,*26*,*27*).

1.2. Problem Statement

This thesis will review and examine the effects of one such system, Prepare to Stop While Flashing (PTSWF), on driver behavior, as a result of increased rear-end crash potential. Specifically, this thesis seeks to examine the risk associated with having a conflict while approaching a high-speed signalized intersection. The proposed approach uses current radar-based technology and video to track vehicles approaching an intersection. The traditional surrogate measure of safety, number of vehicles in the dilemma zone, denotes the region of risk but does not quantify the level of risk. The proposed approach applies the dilemma hazard function, an improved surrogate measure of safety, which classifies and quantifies the level of risk.

1.3. Research Objectives

The objective of this thesis is to examine the effects of AWFs on driver's risk at the onset of yellow by comparing six high-speed intersections. The risk will be compared using quantitative analysis. The following characteristics need to be calculated to quantify the level of risk at a high-speed intersection:

1. Probability of stopping curves: Probability of stopping as a function of distance to the stop bar will be graphed. Developing the probability of stopping curves will allow for the calculation of site specific dilemma zone boundaries.

2. Critical acceleration and deceleration thresholds: Thresholds were calculated for distance from the stop bar at which drivers would be required to:
   - Decelerate severely or
   - Heavily accelerate or run the red light
3. Averaged risks: Based on vehicle counts of different speeds, estimated average risk of a person at a specific velocity at a specific intersection having severe deceleration or running the red light was calculated.

1.4. Thesis Outline

The remaining part of this thesis is as follows. Chapter 2 contains a literature review on past research pertaining to the development and advancement of dilemma zone definitions and methods of mitigation. Past and current methods for modeling driver behavior at high-speed intersections at the onset of yellow are presented, as well as current practices of assessing the safety of vehicles approaching an intersection. The limitations of these practices are explained, thus resulting in the following chapters describing the improve method used.

Chapter 3 describes the six data collection sites used: five with AWFs and one without AWFs. The different data collection setups are explained, along with validation of each setup. A combination of radar based detectors and video was used to continuously track vehicles approaching high-speed signalized intersections. In addition, this chapter discusses the steps used in processing the video collected.

Chapter 4 describes the underlying theory of driver behavior as they approach a signalized intersection. The decision process of drivers at the onset of yellow was modeled using the probit modeling technique. Traditional surrogates of safety measure, i.e. dilemma zone, etc., denote the region of risk but does not quantify the level and region of risk. Therefore, along with developing dilemma hazard function, severe traffic conflict thresholds were applied to evaluate the rear-end and RLR risk. Finally, the results of the analysis are presented in this chapter.

Chapter 5 summarizes the research findings and proposes future research steps. The effects of providing information to driver's as they approach a high-speed intersection result in a decision for the traffic engineer on mitigating right-angle and rear-end crash risk.

CHAPTER 2. LITERATURE REVIEW

2.1. Introduction

The following chapter contains a literature review on past research pertaining to the development and advancement of dilemma zone definitions and methods of mitigation. Past and current methods for modeling driver behavior at high-speed intersections at the onset of yellow are presented, as well as current practices of assessing the safety of vehicles approaching an intersection.

2.2. Dilemma Zone Definitions

There are two distinctive types of dilemma zone, Type I and Type II. Type I dilemma zones are caused by improper signal timing of the clearance intervals. Type II dilemma zones, referred to as option or indecision zones, occurs due to variance in driver behavior. The following section will describe the difference in definitions of the two commonly known types of dilemma zone.

2.2.1. Type I Dilemma Zone

Gazis, Herman and Maradudin (GHM) observed problems associated with drivers facing the yellow change interval. In their paper, GHM defined the “Amber Light Dilemma” as a situation in which a driver may neither be able to stop safely after the onset of yellow indication nor be able to clear an intersection before the signal turns red (*28*). Figure 2.1 illustrates the concept of the Type I dilemma zone.

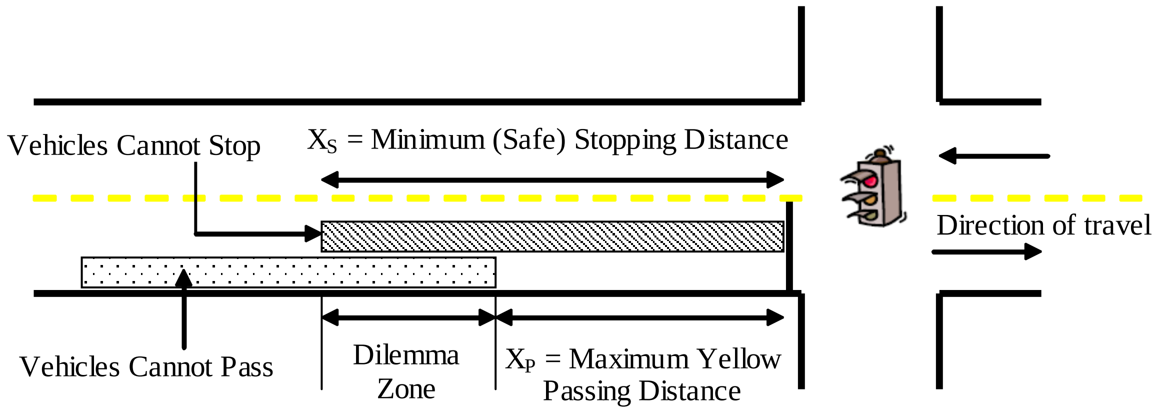


Figure 2.1: Illustration of Type I Dilemma Zone

The two critical distances shown in Figure 2.1 are the maximum yellow passing distance, X_P, and the minimum safe stopping distance, X_S. A vehicle downstream of X_S will not be able to safely stop before the stop bar. Conversely, a vehicle upstream of X_P cannot safely travel and clear the intersection during the yellow phase. As shown above in Figure 2.1, when $X_S > X_P$, a vehicle located within the region between X_S and X_P can neither safely stop nor safely cross the intersection during the yellow phase creating a "dilemma." Thus, the dilemma zone is the physical region between X_S and X_P when $X_S > X_P$. Equations 1 and 2 represent X_S and X_P according to the GHM model.

$$X_S = V_0 \delta_2 + \frac{V_0^2}{2a_2} \qquad \text{Equation 1}$$

$$X_P = V_0 \tau - W + \frac{1}{2} a_1 (\tau - \delta_1)^2 \qquad \text{Equation 2}$$

Where,

X_S = Minimum safe stopping distance (ft)

X_P = Maximum yellow passing distance (ft)

V_0 = vehicle's approach speed (ft/s)

δ_2 = driver's stopping perception-reaction time (s)

a_2 = driver's maximum comfortable deceleration rate (ft/s^2)

δ_1 = driver's crossing the intersection perception-reaction time (s)

a_1 = driver's maximum comfortable acceleration rate (ft/s^2)

τ = duration of yellow interval (s)

W = sum of intersection width and vehicle length (ft)

Equation 2 does not take into account an all-red clearance interval, which will be discussed later. Assuming drivers drive legally and under good weather conditions, the yellow interval or a change in driver behavior, Type I dilemma zone can be eliminated through proper design. In certain instances, drivers may eliminate the dilemma zone by accelerating to or above the speed limit. However as Liu et al. cautioned, advising drivers to use the onset of yellow as an instruction to acceleration would be dangerous (*29*). Thus, assuming a crossing vehicle does not accelerate, Type I dilemma zone may be eliminated by adjusting the yellow interval to set $X_S - X_P$ to zero.

$$\tau = \delta + \frac{v_0}{2a_2} + \frac{L+w}{v_0} \qquad \text{Equation 3}$$

The yellow duration, τ, defined in the GHM model has been divided into two intervals: yellow permissive interval, $y = \delta + \frac{v_0}{2a_2}$, and the all-red clearance interval, $r = \frac{L+w}{v_0}$.

Studies (*14*,*28*,*30*,*31*,*32*,*33*,*34*,*35*,*36*) have shown a wide variability in driver behavior. In an effort to avoid the dilemma zone, May found that some drivers accelerate or decelerate heavily (*14*). Figure 2.2 and Table 2.1 illustrate the variability in perception reaction times and deceleration rates at the onset of yellow. It is the variability in driver behavior that is the main limitation of the GHM model.

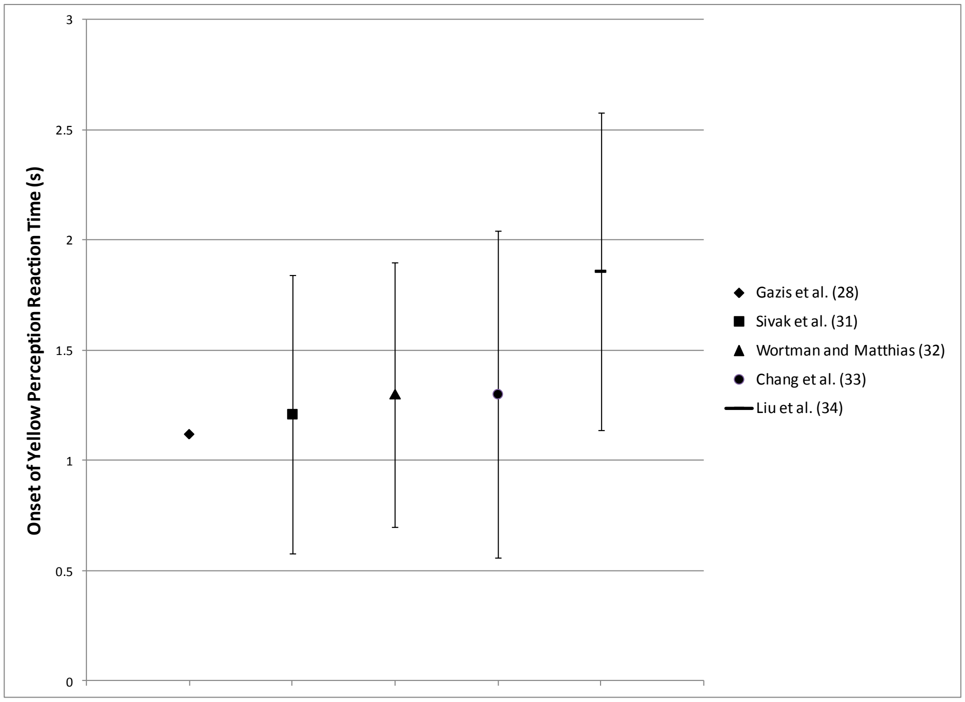


Figure 2.2: Previously reported perception reaction times

Table 2.1: Variability in previously reported deceleration rates

	Speeds studied/calculated	Mean deceleration rate (ft/s^2)
Gazis et al. (28)	45 mph	16
Williams (35)	10-25 mph	9.7
Parsonson and Santiago (36)	-	10
Wortman and Matthias (32)	30-50 mph	11.5
Chang et al. (33)	>20 mph	9.2

2.2.2. Type II Dilemma Zone (Indecision Zone or Option Zone)

To take into account the variability in driver behavior, researchers defined a second type of dilemma zone. Also referred to as indecision or option zone, Type II dilemma zone is based on a probabilistic approach of drivers' decision to the onset of yellow. In this zone, drivers can both stop comfortably or clear the intersection before the end of yellow, thus resulting in the dilemma of whether to stop or proceed through the intersection. Type II dilemma zones were first documented in a technical report by the Southern Section of ITE (*15*). A driver on a high speed roadway encounters a dilemma on whether to stop or proceed through the intersection at the onset of yellow. As a result of the variability previously described, Type II dilemma zone exists at the onset of every yellow indication. A wrong decision to stop when it would have been safer to proceed can lead to a severe rear-end collision. Conversely, a wrong decision to proceed through the intersection could lead to the driver running the red light and possibly causing a right angle collision. Figure 2.3 illustrates the Type II dilemma zone.

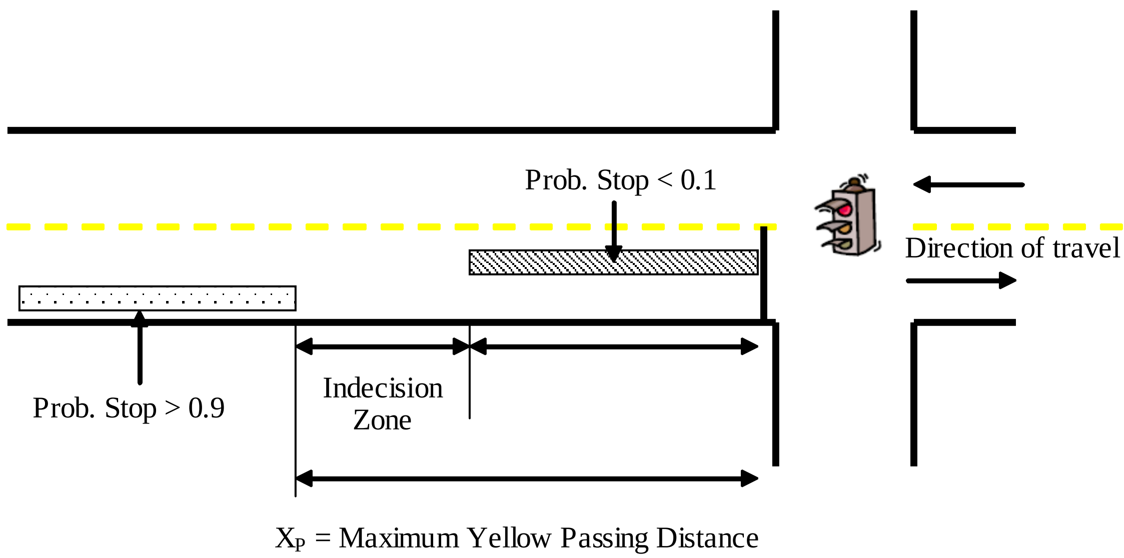


Figure 2.3: Illustration of Type II Dilemma Zone

Zeeger (*16*) defined the zone as "the road segment where more than 10 percent and less than 90 percent of the drivers would choose to stop." Researchers have attempted several approaches for characterizing the indecision zone boundaries. Zeeger (*16*) used a frequency-based approach of drivers stopping decisions at specified distances and speeds to develop a cumulative distribution function. The dilemma zone boundaries have been quantified as the distance and speed or time to the intersection.

At the onset of yellow, a driver can choose from two mutually exclusive courses of action: stop or go. The decision process thus can be modeled by binary discrete choice models. Sheffi and Mahmassani (38) modeled the driver decision process with a probit model to significantly reduce the sample size required for estimating dilemma zone boundaries. A driver's perceived time to reach the stop bar, T, randomly chosen from a population was modeled as a random variable,

$$T = t + \xi \qquad \text{Equation 4}$$

where t is the measured time to the stop bar at a constant speed. The error term, ξ, designating the differences in driver's perception, is a random variable assumed to be normally distributed. Sheffi and Mahmassani hypothesized that a driver would choose to proceed through the intersection if T was less than a critical value, T_{cr}. The critical time, T_{cr}, was also modeled as a normally distributed random variable accounting for a driver's experience, perception of acceleration rates, and aggressiveness.

$$T_{cr} = t_{cr} + \epsilon \qquad \text{Equation 5}$$

where t_{cr}, is the mean critical time. The error term, ϵ, is also normally distrusted across the driver population. The probability of a random driver choosing to stop, P_{STOP} (T), is given by the probit equation:

$$P_{STOP}(T) = \Pr\{T_{cr} < T\} = \Phi\left(\frac{t - t_{cr}}{\sigma}\right) \qquad \text{Equation 6}$$

where $\sigma = \sqrt{\sigma_{\xi}^{2} + \sigma_{\epsilon}^{2} - 2\sigma_{\xi,\epsilon}}$.

In comparison to the required amount of 2000 observations necessary to stabilize dilemma zone curves graphically (*39*), the previously described model was shown to stabilize at approximately 150 observations. Similar results in stability of the probit model were demonstrated by Sharma et al. (*40*).

Advantages of the model described above include:

- Dilemma zone curves directly calculated from the model
- Only a small sample size, 150 observations, required to model dilemma zone curves

In addition to the probit model, dilemma zone boundaries have been estimated with other models primarily the logit model. Similar to the probit model, the logit model is a binary discrete choice model. Recent studies using logit to develop probability of stopping curves include: Bonneson and Son (*40*), Gates et al. (*42*), Papaioannou (43), and Kim et al (*44*). Rakha et al. (*45*) used an empirical model to develop drivers' probability of stopping.

Elmitiny et al. (*46*) used tree-based classification to model the driver's stop/go decision. As a method of splitting the data, classification trees are effective in segmenting the data into smaller and more homogeneous groups. Elmitiny et al. split the data based on distance to intersection and speed at the onset of yellow, position of vehicle (leading or following), and vehicle type. Thus, statistical comparisons can be made based directly on the various nodes defined by the researcher. For example, the study revealed that drivers in the following position are more likely to make go decisions and run the red light than those in the leading position exposing them to the potential of increasing their rear-end crash risk.

Due to the dynamic nature of the decision dilemma zone, studies have examined variables contributing to a driver's decision to stop or proceed through the intersection. Gates et al. (*42*) observed that heavy vehicles have a higher probability to proceed through the intersection than cars, with similar results observed by Wei et al (*47*). Sharma et al. (*40*) proposed probability of stopping to be a function of the required acceleration to cross the stop bar, while Kim et al. (*44*) proposed yellow-onset speed and distance from the stop line, Time to stop bar (TTS), and location of signal head significantly affect a driver's stopping decision. Figure 2.4 shows the variation in dilemma zone boundaries based on previous findings for vehicles approaching an intersection at 50 mph.

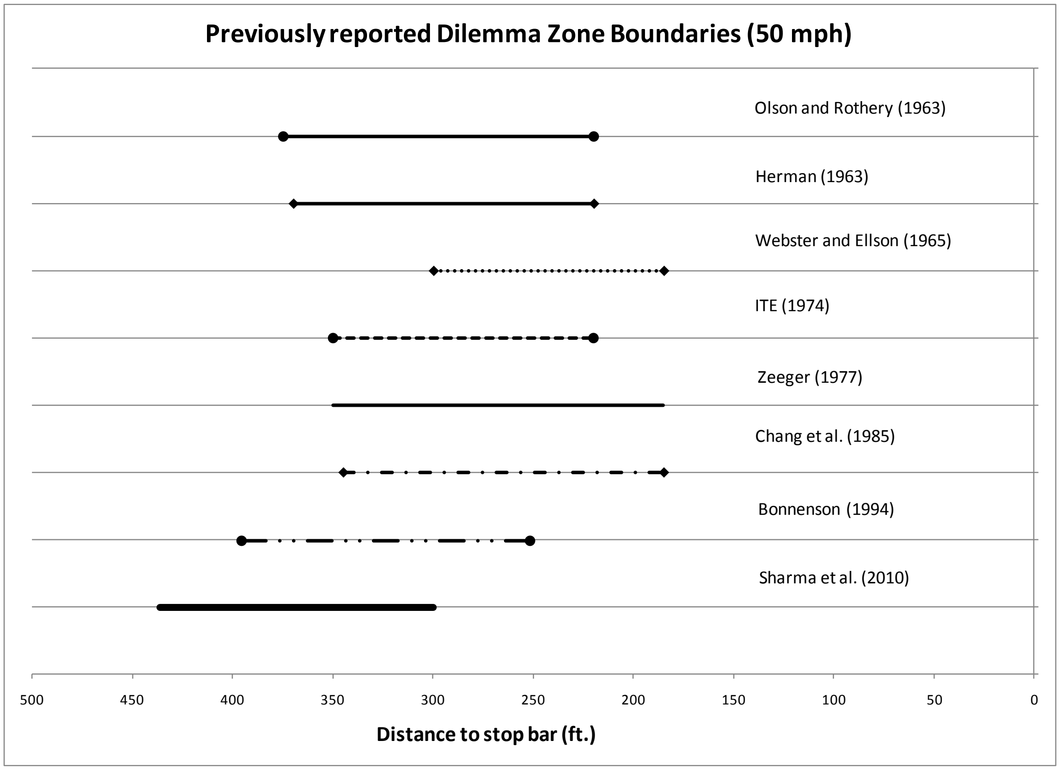


Figure 2.4: Dilemma zone boundaries (50 mph)

The statistical methods of calculating the traditional surrogate safety measure of the number of vehicles in the dilemma zone are sound; however, as is shown in Figure 2.4, the variations that occur in the defined boundaries are a result of the differences in dilemma zone definitions, type of drivers, and environmental and geometric layout of the investigated sites. This method of safety does not quantify the level or risk at different locations in the dilemma zone, as drivers are either at risk (in the dilemma zone) or free from risk (out of the dilemma zone).

2.3. Effects of Yellow Length on Driver Behavior

Studies have also examined the impact on driving behavior as a function of yellow interval length. In his comparison study of intersections equipped with and without flashing green, Knoflacher (*23*) concluded the decrease in right-angle crashes corresponded to increases in the duration of yellow. The effects of yellow interval duration on stopping have also been studied. Lengthy yellow intervals were found by Van der Horst and Wilmink to cause bad driver behavior for last-to-stop drivers at intersections (*48*). Instead of being presented with a red indication as they approached the stop line, the drivers were stopping while the light was still yellow, thus persuading the driver to proceed through the intersection the next time they approached the intersection. Van der Horst and Wilmink found drivers adjusting their stopping behavior as a function of longer change intervals. The probability of stopping for drivers 4 seconds from the intersection decreased from 0.5 for a yellow length of 3 seconds to 0.34 for a yellow length of 5 seconds long. In a study of multiple intersections in Texas, Bonnenson et al. (*49*) noted that drivers do adapt to an increase in yellow duration. Reductions in red light running (RLR) were found to decrease up to 50 percent for increases in yellow ranging from 0.5 to 1.5 s, as long as the yellow duration did not exceed 5.5 seconds. Koll et al. (*27*) concluded that early stops should reduce the probability of right-angle collisions.

Contrary to the previous results, Olson and Rothery (*30*) concluded that driver behavior does not change as a function of different yellow phase durations. Studies have also shown that an overly long amber could lead to greater variability in driver's decision making and potentially increase rear-end conflicts (*14,30,50*). Mahalel and Prashker (*50*) noted a potential increase in the indecision zone for a lengthy “end-of-phase” warning interval. They observed an increase in the indecision zone from the normal zone of (2 to 5 seconds) without a flashing green interval to an indecision zone of 2 to 8 seconds for a 3-s yellow that was preceded by a 3-s flashing green. Mahalel and Prashker presented evidence of increases in the frequency of rear-end crashes due to the increase in the indecision zone.

2.4. Mitigation of Dilemma Zones

2.4.1. Green Extension

Advanced detection systems place several loop detectors upstream of the intersection to detect approaching vehicles and extend the green. These detectors communicate with a computer, which searches the signal controller to determine if an extension is required based on the vehicles’ measured speed. Ideally, the green phase of the high speed approach is extended until there is no vehicle in the dilemma zone; however, a maximum green time, is provided for this operation to avoid excessive delays to the cross street traffic. As long as they are discharging at saturation flow rate, all the phases are allotted green, thus reducing delay. This approach is an all-or-nothing approach. Dilemma zone protection is provided to the high speed vehicles prior to the maximum green time being completely reached, at which time the protection is removed. Developed to reduce the number of trucks being stopped at high speed rural intersections, the Texas Transportation Institute’s (TTI) Truck Priority System is an example of a green extension system (*51*). However, the system does not specifically provide dilemma zone protection. The system extends the phase by as much as 15 seconds past maximum green before reaching max-out, at which time dilemma zone protection is removed. A further description of the limitations in gap out logic is provided by Sharma et al. (52). Another example of a green extension system is Sweden's LHORVA system (*53*).

2.4.2. Green Termination

Green termination algorithms, on the other hand, are relatively new and the systems implementing it exist only at a few intersections. These systems attempt to identify an appropriate time to end the green phase by predicting the value of a performance function for the near future. The objective is to minimize the performance function, which is based on the number of vehicles present in the dilemma zone and the length of the opposing queue. The wide application of these systems has been limited and little quantitative data exists on the trade-off between efficiency, cost, and detector requirements.

2.4.2.1. SOS – Self Optimising Signal Control

Sweden's SOS system is another green termination algorithm designed for isolated intersections. Similar to D-CS, the system utilizes detectors in each lane to project the vehicles as they approach the intersection. The Miller algorithm calculates the cost of ending the green now or in t seconds (*54*). Calculations are performed for different lengths of t, for example 0.5 s up to 20 s. The algorithm evaluates three factors: reduction in delay and stops for vehicles using the green extension, the increase in delay and stops for opposing traffic, and the increase in delay and stops for vehicles that cannot use the green extension and have to wait for the next green period. The percentage of vehicles in the option zone was reduced by 38 percent. Additionally, the number of vehicles exposed to the risk of rear-end collision decreased by 58 percent.

2.4.2.2. D-CS

Texas Transportation Institute's Detection-Control System, or D-CS, is a current state of the art system in the United States that has been implemented at eight intersections in Texas and three in Ontario, Canada (*55*). D-CS uses a green termination algorithm. The D-CS algorithm has two components: vehicle status and phase status.

A speed trap sufficiently far from the intersection (~ 800-1000ft) is used to detect the speed and vehicle length of each vehicle. The projected arrival and departure time of each vehicle in their respective dilemma zone (based on speed and vehicle length) is used to maintain the "dilemma-zone matrix." This matrix is updated every 0.05 seconds. The phase status component uses dilemma-zone matrix, maximum green time, and number of calls registered on opposing phases to control the end time for the main street green phase. The phase status is updated after every 0.5 seconds.

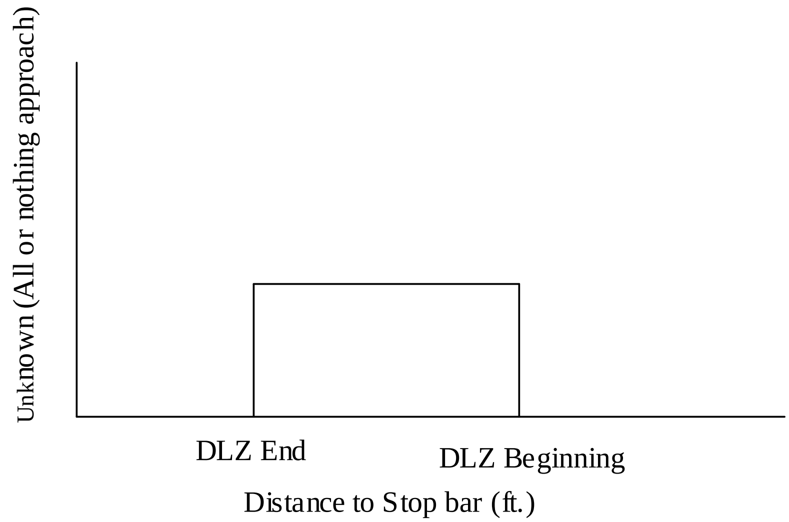
Bonneson et al. (*56*) observed reductions in the frequency of red-light violations at almost every approach. Overall the violations were reduced by 58 percent, with a reduction of about 80 percent for heavy vehicles. D-CS reduced violations 53 percent and 90 percent when replacing systems using multiple advance loop detection and systems with no advance detection, respectively. On the approaches controlled by D-CS, severe crashes were reduced by 39 percent. In addition to severe crashes, crashes influenced by D-CS (i.e. rear-end, left-turn opposed, and sideswipe) appear to provide a 50 percent reduction in severe "influenced" crashes. Intersection operation improved at almost every approach of the five intersections studied. Reductions in control delay and stop frequency were 14 percent and 9 percent, respectively. Most likely the reductions are due to D-CS's more efficient operation than the prior detection and control strategy used.

2.4.2.3. Wavetronix SmartSensor Advance

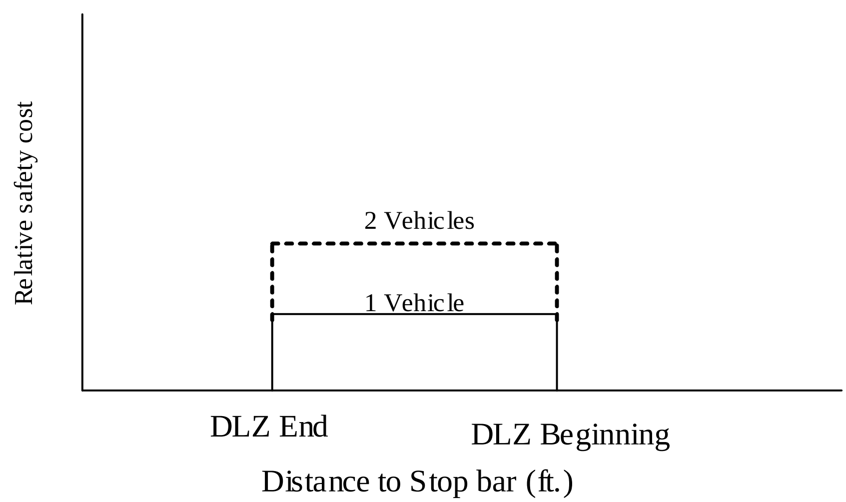
By using digital wave radar, the Wavetronix SmartSensor Advance with SafeArrival technology is one of the newest vehicle detection based systems designed to improve dilemma zone protection (*57*). The system continuously tracks vehicles' speed and range to estimate the time of arrival at the stop bar. SmartSensor Advance formulates the position and size of gaps in flowing traffic to adjust the physical location of the gaps to extend the green time to allow for safe passage if necessary. In a comparison study of dilemma zone protection systems, the Wavetronix system provided a greater reduction in the number of vehicles in the Type II dilemma zone than inductive loops (*58*). In addition, the SmartSensor Advance decreased red light running incidents by more than 3 times the rate of the inductive loop system.

2.5. Traffic Conflicts

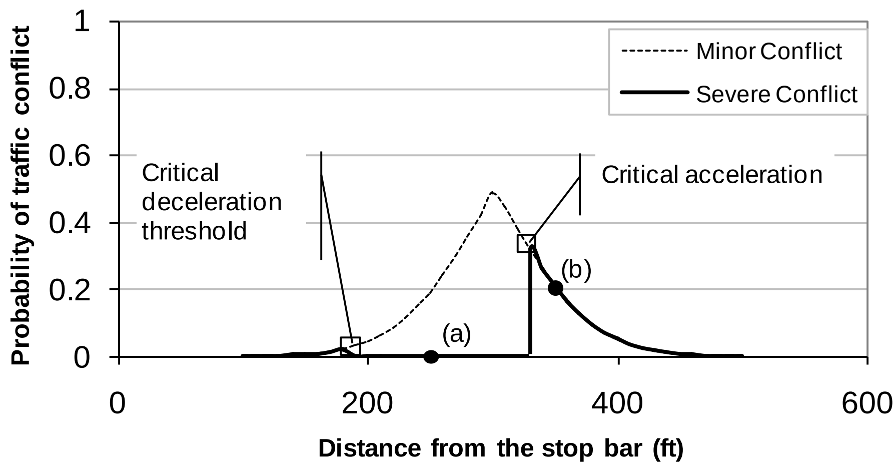
As previously mentioned, traditional surrogate measures of safety (such as the number of vehicles in the dilemma zone) fail to quantify the risk of crash. Meanwhile, traffic conflicts have demonstrated the ability to indirectly evaluate the safety of an intersection. Proposed by Sharma et al. (*40*), Figures 2.5a to 2.5c contrast the present surrogate measures of safety with the dilemma hazard measure of safety. Widely used green extension systems are all-or-nothing approaches. All the vehicles on the high-speed approaches are cleared until the maximum green time is reached. At the end of the maximum green time, none of the vehicles on the high-speed approach are provided protection. As shown in Figure 2.5a, these systems do not have any metric to measure the cost of risk of crash. The green termination systems use the number of vehicles in the dilemma zone as a surrogate measure for quantifying the cost of risk. The number of vehicles is a rank-ordered metric, shown in Figure 2.5b, where the cost of one vehicle in the dilemma zone is less than the cost of two vehicles in the dilemma zone; but the cost is independent of the positions of vehicles in the dilemma zone. In addition, there has been a lack of research to associate a monetary cost of safety for a dilemma zone incursion. Sharma et al. (*59*) modeled the dilemma zone hazard using the observed probability of stop and go at the onset of yellow light. The probability of making an erroneous decision is used as the probability of traffic conflict. The severity of conflict is determined using the observed acceleration and deceleration ranges used by drivers at the intersection. Dilemma hazard function obtained for vehicles traveling at 45 mph as estimated for the study site at Noblesville, Indiana are shown in Figure 2.5c. The probability of conflict curves developed by Sharma was for single vehicle cars only. This thesis will also examine the effect of information provided to drivers on probability of traffic conflicts for single vehicles situations.



a) Safety cost evaluation in current green extension systems



b) Safety cost evaluation in advanced green termination systems



c) Proposed evaluation of safety cost (*40*)

Figure 2.5: Comparison of traditional and recent surrogate measures of safety

2.5.1. <u>Traffic Conflict Technique</u>

Over the past four decades, the Traffic Conflict Technique (TCT) has evolved and demonstrated its usefulness in indirectly evaluating the safety of intersections. The technique originates from research performed at the General Motors laboratory in Detroit, MI for identifying safety problems related to vehicle construction (*60*). Perkins and Harris defined a conflict as "The occurrence of evasive actions, such as braking or weaving, which are forced on the driver by an impending crash situation or a traffic violation." They categorized the conflicts into left-turn conflicts, cross-traffic conflicts, weave conflicts, and rear-end conflicts.

The technique gained popularity as research efforts attempted to establish a direct relationship between conflicts and crashes (*61,62,63,64*). The rationale beyond the gain in popularity of the technique was twofold. First, studies have shown the increased frequency in observing traffic conflicts at an intersection as opposed to waiting for a crash history to develop. This allows for information regarding the safety of an intersection to be collected rather quickly. Cooper and Ferguson (*65*) calculated the ratio of the rate of serious conflicts to the rate of crashes to be approximately 2000:1. Therefore, two or three years of reported crash records at an intersection could be observed with 10 hours of conflicts. A recent study by FHWA (*66*) found the ratio of traffic conflicts to actual crashes to be approximately 20,000:1 though the relationship varied by conflict type. The FHWA study used 83 signalized intersections for their validation study and establishing the following relationship between conflicts and crashes.

$$\frac{Crashes}{Year} = 0.119\ x\ \left(\frac{Conflicts}{Hour}\right)^{1.419} \qquad \text{Equation 7}$$

In addition to quicker data collection, the second advantage of the conflict technique is in identifying safety deficiencies at intersections. TCT allows traffic engineers the opportunity to provide proactive safety improvements at an intersection instead of waiting for the crash history to evolve.

Concerns regarding TCT have been raised by several researchers. Glennon et al. (*67*) expressed concerns on the use of the TCT technique, stating, “The reliability of TCT for estimating accident potential is questionable.” They found for every study in favor of TCT, there is a study that opposes it. Glennon argued the ability to predict the number of accidents at an intersection will be extremely improbable, since both conflicts and accidents are random events. In addition, Hauer and Garder (*68*) criticized the heavily subjective manner of TCT on judging speed and distance of vehicles.

Although concerns have been raised regarding the use of TCT, recent studies have continued to advocate its use as a surrogate measure of safety. Glauz et al. (*69*) investigated two types of expected accident prediction rates, one based on conflict ratios and the other based on accident histories. The study determined the difference to be statistically insignificant, thus an estimate of the expected accident rates using traffic conflicts can be as accurate and precise as predicted by the accident history. Hyden (*70*) concluded that conflicts and accidents did in fact share the same severity distribution based on time-to-accident (TA) and speed values. The use of traffic conflict as a surrogate measure for traffic safety in micro-simulation has been advocated by Fazio et al. (*71*), as well as by Gettman and Head (*66*,*72*) who performed a detailed use-case analysis.

2.5.2. Traffic conflicts at the Onset of Yellow

Zeeger (*16*) identified six conflicts that can occur at the onset of yellow. The following definitions of the six conflicts were used during conflict analysis performed as part of this thesis:

- Red light runner (RLR): A red light violation was defined as occurring when the front of the vehicle was behind the stop line at the onset of red.
- Abrupt stop: An abrupt stop occurs when a vehicle would be able to successfully clear the intersection, yet decides to stop. Abrupt stop conflicts can be viewed both visually and calculated mathematically based on the onset of yellow distance and speed.

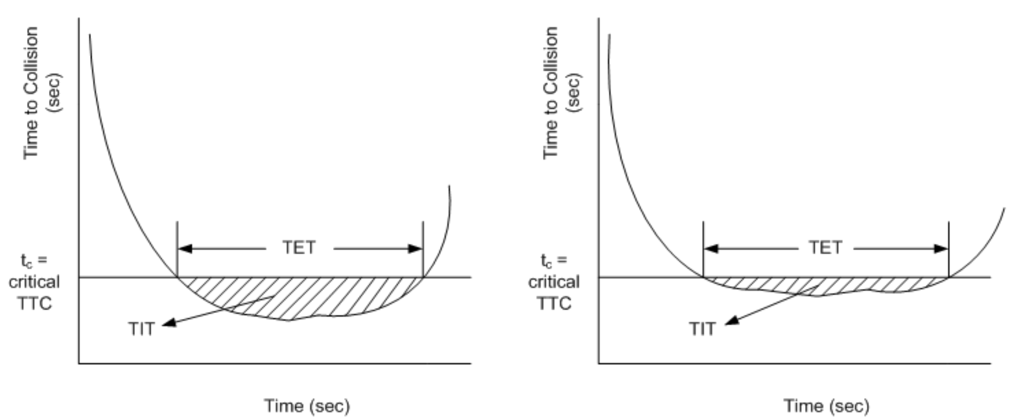
- Swerve-to-avoid collision: Classified as an erratic maneuver of a driver to swerve out of their lane to avoid hitting the preceding vehicle that had stopped for the light in front of them.
- Vehicle skidded: This is a more severe case of abrupt stop, with the vehicle's wheels "locking-up" in order to stop. It can be heard audibly.
- Acceleration through yellow: Acceleration through yellow was identified as either being heard audibly or identified through numerical calculation. Each vehicle's distance was projected at the onset of red based on their onset of yellow distance and speed, assuming constant speed. An acceleration through yellow conflict was assigned if the vehicle successfully crossed the stop bar but would not have if based on the constant speed projection.
- Brakes applied before passing through: This conflict can be viewed visually when at the onset of yellow the driver applied the brakes before passing through the intersection. It indicates the indecisiveness of drivers when approaching the intersection.

2.5.3. Time-to-Collision

Two distinct safety issues at intersections are frequency and severity. Although TCT indicates the frequency at which conflicts occur between road-users, it does not quantify the severity of the conflicts. Time-to-Collision (TTC) is one of the most commonly used measures of effectiveness for rating the severity of traffic conflicts. Hayward (*73*) defined TTC as: "The time required for two vehicles to collide if they continue at their present speed and on the same path." TTC has proven to distinguish between normal behavior and serious conflicts (*74*). In addition, the main advantage of TTC is the considerably less subjective and more objective measures of speed and distance it uses.

Values of TTC are infinite when vehicles are not on a collision course; however, if vehicles are on a collision course, the value of TTC is finite and decreases with time. As the vehicles continue on the collision course, conflict severity is estimated using the minimum TTC value. Moreover, a critical TTC value has been proposed through previous studies to assess the severity of conflicts. Results of the TTC from Hayward found a mean TTC for vehicles on collision paths was 1.5 seconds. However, Hayward proposed a critical TTC value of 1.0 second. Recent studies (*70,75,76,77,78*) have suggested the use of a critical TTC value of 1.5 seconds.

The main drawback of TTC is that it just uses the minimal time to conflict for a vehicle and does not take into account the duration the vehicle was subjected for this conflict. For the case of a following faster vehicle approaching a slower leading vehicle, consider two separate events each having a TTC value of 1.0 second with following vehicle speeds of 55 mph and 35 mph, respectively. Tables 2.2 and 2.3 present a demonstration of the two events.



(a) 55 mph (b) 35 mph

Figure 2.6: Time to collision profiles

Table 2.2: Event with following vehicle traveling at 55 mph

	Lead Vehicle		Following Vehicle		
Time (s)	**Velocity (mph)**	**Distance to Stop Bar (ft.)**	**Velocity (mph)**	**Distance to Stop Bar (ft.)**	**TTC (s)**
0.0	45	300	55	360	5.4
0.5	45	267	55	332.5	5.8
1.0	45	234	55	305	6.2
1.5	45	201	55	277.5	6.6
2.0	41.2	168	55	250	5.0
2.5	37.4	137	55	222.5	4.1
3.0	33.6	110	55	195	3.3
3.5	29.8	85	55	167.5	2.8
4.0	26.0	63	55	140	2.3
4.5	22.1	44	55.0	113	1.8
5.0	18.3	28	55.0	85	1.4
5.5	14.5	15	51.2	59	1.2
6.0	10.7	4	47.4	36	1.0
6.5			43.6	14	1.5
7.0			39.8	-6	1.7

Table 2.3: Event with following vehicle traveling at 35 mph

	Lead Vehicle		Following Vehicle		
Time (s)	**Velocity (mph)**	**Distance to Stop Bar (ft.)**	**Velocity (mph)**	**Distance to Stop Bar (ft.)**	**TTC (s)**
0.0	28	150	35	175	4.4
0.5	27	129	35	157.5	4.1
1.0	26	110	35	140	3.8
1.5	25	90	35	122.5	3.5
2.0	25.0	72	35	105	3.6
2.5	22.5	54	35	87.5	2.9
3.0	18.7	37	35.0	70	2.2
3.5	14.9	23	35.0	52.5	1.7
4.0	11.1	13	35.0	35	1.2
4.5	7.3	4	31.2	19	1.0
5.0			27.4	6	1.6
5.5			23.6	-6	2.0

The definition of TTC quantifies these two events as having the same severity level given that both events have the same minimum TTC. However, it is reasonable to assume the vehicle traveling at 55 mph has a higher severity potential than the vehicle traveling at 35 mph. The following vehicle traveling at 35 mph will be able to decelerate and avoid the collision easier than the following vehicle traveling at 55 mph, as seen by the amount of time the vehicles fall below the TTC threshold value. Therefore, although lower TTC values indicate a higher probability of collision, the values cannot be directly linked to the severity of the collision.

As a result of TTC not implicitly considering speed, Minderhood and Bovy (*79*) proposed two new severity conflict indicators based on traditional TTC measures. Time Exposed Time-to-Collision (TET) calculates the overall length of time all vehicles involved in the conflict are under a predetermined TTC minimum threshold. The second indicator proposed was Time Integrated Time-to-Collision (TIT). TIT integrates the amount of the time the TTC falls below the designated minimum threshold.

Figure 2.6 shows two distinct time-to-collision profiles. As mentioned above, TET only calculates the overall length of time the TTC curve falls below the TTC threshold value. The two events represented below have the same exposed time-to-collision. However, it can be seen that the vehicles in Figure 2.6a are exposed to a more severe threat for the same amount of time than the vehicles in Figure 2.6b resulting in a higher severity of collision.

Previous studies have indicated the usefulness of real-time data using video analysis; however, processing this information has shown to be time consuming (74). Calculating the minimum TTC requires a detailed (time-step or frame) analysis of speed and distance between the two road-users in relation to a common point, which requires significant calculations. Consequently, TTC values have typically been calculated using simulation models. Simulation models automate the speed and distance calculations between vehicles.

2.5.4. Dilemma Zone Hazard Models

Recently, studies have quantified the level of risk associated with being in the dilemma zone by developing dilemma hazard models. The dilemma hazard recently developed is a new traffic conflict potential measure. Li (*80*) validated and calibrated the dilemma hazard model based on an approach developed by the American Society of Civil Engineers. In order to calculate the dilemma hazard, the dilemma hazard model compares driver decisions and their actual driving capability as a function of its Time-to-Intersection (TTI) at the onset of yellow. The approach used driver's decisions at the onset of yellow; their actual capabilities based on vehicle kinematics, and previously reported acceleration and deceleration rates. Data collected was simulated using Monte Carlo simulation to establish dilemma hazard values within the dilemma zone boundaries of two to five seconds. Models were created for single vehicle and multiple (two) vehicle scenarios. Results of the simulation, shown in Figure 2.7, illustrate the effect signal timings have on the dilemma hazard.

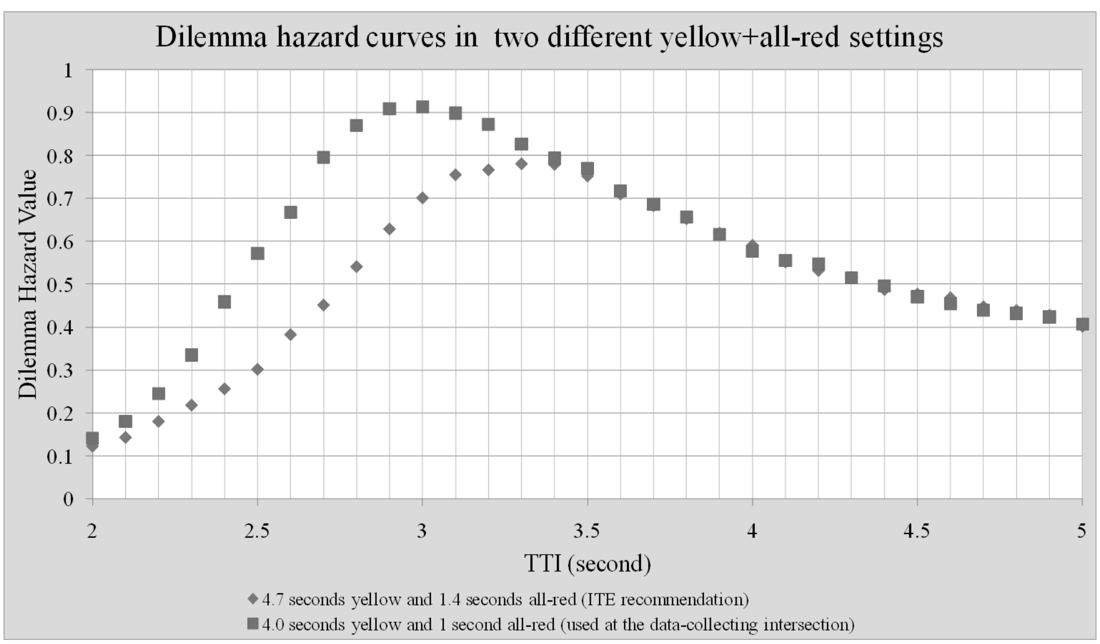


Figure 2.7: Dilemma hazard curves for various yellow and all-red clearance intervals (*80*)

Sharma et al. (*40*) provided a theoretical justification for using probability of stopping to estimate probability of conflict for single vehicles at high-speed intersections. Probability of stopping curves were developed based on the acceleration required by the vehicle to cross the stop bar prior to the onset of red. Sharma theorized vehicles would have a stop conflict if the critical deceleration threshold was greater than the required deceleration; furthermore, vehicles would have a go conflict if the required acceleration was greater than the critical acceleration threshold. Conflicts were classified into minor and severe conflicts based on the magnitude of acceleration or deceleration required to perform the chosen decision. Figure 2.8 illustrates the developed probability of traffic conflict curve.

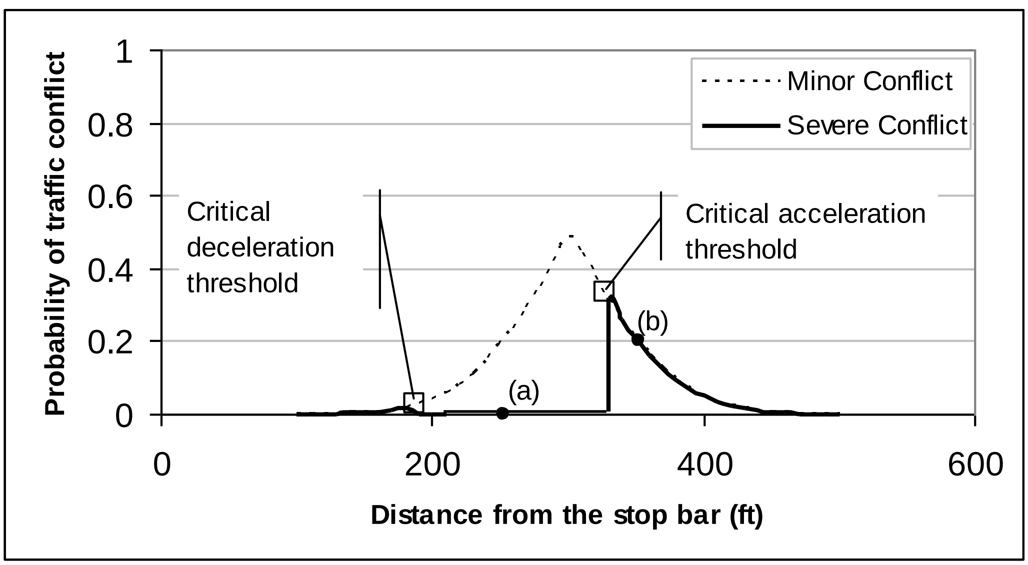


Figure 2.8: Calculated dilemma zone hazard function (*40*)

2.6. <u>Advance Warning</u>

Placed upstream of high speed signalized intersections, AWFs provide drivers with information regarding whether they should prepare to stop at the upcoming traffic signal or proceed through the intersection. Specifically, AWFs are designed to minimize the number of vehicles trapped in their respective dilemma zones at the onset of yellow (*20*). AWFs have been found to improve dilemma zone protection in the state of Nebraska. McCoy and Pesti (*81*) used advanced detection along with AWFs to develop an enhanced dilemma zone protection system. The system was found to reduce the number of max-outs, which would result in a loss of dilemma zone protection. Gibby et al. (*82*) concluded from an analysis of high-speed signalized intersections in California that advance warning flashers significantly reduce accident rates. The approaches with AWFs had lower total, left-turn, right-angle, and rear-end accident rates. Sayed et al. (*83*) calculated the reduction in total and severe accidents at intersections with AWFs to be 10 and 12 percent, respectively.

2.6.1.1. Advanced Warning's Effects on RLR's

Farraher et al. (*21*) observed red light running and vehicles speeds in Bloomington, Minnesota. Installation of advanced warning flashers resulted in reductions of 29 percent in red light running, 63 percent reduction in truck red light running, and an 18.2 percent reduction in the speed of the red light running trucks. In addition, the Texas Transportation Institute (TTI) developed an Advanced Warning for End-of-Green System (AWEGS) that utilized a sign (text or symbolic), two amber flashers, and a pair of advanced inductive loops (*20*). The system capable of identifying different classifications of vehicles (car, truck) has shown to decrease delay due to stoppages at traffic signals, as well as providing extra dilemma zone protection to high-speed vehicles and trucks. Results of the study have shown a reduction in Red Light Running (RLR) by 38 to 42 percent in the first 5 seconds of red.

Although the consensus of AWFs is that the systems provide safety benefits to the users, several concerns have been raised. In their study, Farraher et al. (*21*) detected car drivers running the red light entered speeds above the speed light increasing the risk of crash for opposing traffic. Pant and Huang (*84*) evaluated several high-speed intersections with AWFs and detected increases in speed as the traffic signal approached the red phase. Thus, the authors discouraged the use of Prepare to Stop When Flashing (PTSWF) and Flashing Symbolic Signal Ahead (FSSA) signs along tangent intersection approaches. Further testing performed by Pant and Xie (85) at two intersections verified the previous findings of increased speeds along roadways with a PTSWF or FSSA sign. In a driving simulator study performed by Newton et al. (*22*) on Traffic Light Change Anticipation Systems (TLCAS), RLRs were statistically fewer at intersections with TLCAS.

2.6.1.2. Advanced Warning's Effects on Rear-end

Similar to AWF systems, flashing green systems, have been implemented and tested thoroughly in Europe and Israel. Knoflacher (*23*) studied the decelerations and accidents at intersections equipped with and without flashing green systems. In his study of intersections, Knoflacher found intersections implemented with flashing green systems had larger deceleration rates and increases in the amount of rear-end collisions. Studies (*24,25,26*) of the flashing green system in Israel consistently observed increases in rear-end collisions with negligible changes in right-angle accidents at intersections implemented with the flashing green interval. In a simulated study comparing driver's response at intersections using flashing green, Mahalel et al. (*86*) noted a significant increase in erroneous decisions at the onset of yellow. In particular, the increase in inappropriate stop decisions at intersections with flashing green doubled to 77 percent compared to 38 percent of intersections without the flashing green interval. This increase in inappropriate stops caused a considerable shift in probability of stopping curves. Koll et al. (*27*) compared the effects of flashing green on 10 approaches in Austria, Switzerland, and Germany. Safety impacts considered included the amount of yellow and red stop line crossings observed. A substantial increase in the number of early stops was found in Austria. A larger option zone, area where drivers can both proceed and stop safely, increased as a result.

2.7. Summary

The development and knowledge regarding dilemma zones and traffic conflicts has continuously progressed. The traditional surrogate measure of safety, the dilemma zone, denotes the region of risk but does not quantify the level of risk. Traffic conflicts indirectly evaluate the safety of an intersection, yet have been controversial in their subjective nature, resulting in the development of TTC, TET, and TIT. Although TTC and TET have shown the ability to quantify the risk of conflict, TTC and TET do not implicitly consider speed, since it is reasonable to assume that vehicles traveling with higher speeds will have a larger level of risk when approaching an intersection. While TIT integrates both the overall length in time and magnitude below the critical threshold, the ability to perform this detailed time step analysis with real-time data has shown to be time consuming. Recently, dilemma hazard model and dilemma hazard function have attempted to quantify the level of risk associated with being in the dilemma zone at the onset of yellow.

While the development of dilemma zone and traffic conflicts has continuously progressed, the effect of information on driver behavior at the onset of yellow still remains rather uncertain. Specifically, uncertainly remains on the potential tradeoff in providing safety for either right-angle or rear-end accidents, as past research (*20,21,22*) has revealed significant reductions in RLRs at intersections providing information through AWFs. However, rear-end crash potential has increased as a result (*23*,*24*,*25*,*26*,*27*). This thesis will investigate the effect of information on driver behavior at the onset of yellow using the traffic conflict technique and recently formulated dilemma hazard function (*40*).

CHAPTER 3. DATA COLLECTION

3.1. Introduction

To achieve as thorough analysis as possible, five locations were selected for data collection, with an additional site evaluated for comparison purposes. A combination of radar based detectors and video was used to continuously track vehicles approaching high-speed signalized intersections. This chapter describes the data collection locations, equipment setup and calibration, and video processing tasks.

3.2. Data Collection Sites

This section describes the six intersections studied. Five of the locations were collected as part of this thesis, while the remaining site was collected previously by Sharma et al. (*40*) and is used for evaluation purposes. Four of the five sites were operated by the Nebraska Department of Roads (NDOR). The placement of the AWFs was based upon MUTCD guidelines, as well as feedback from drivers. In order to calculated the length of flashing time before yellow, the distance at which the AWFs were placed was divided by the posted speed limit.

3.2.1. Highway 2 and 84th Street

The high-speed signalized intersection of Highway 2 and 84th St. in Lincoln, Nebraska was selected as the initial data collection site. Highway 2 is a major thoroughfare in Lincoln, particularly for heavy vehicles. The percentage of heavy vehicles at the studied intersection is ten percent**.** The eastbound approach of Highway 2 has two through lanes, two left turn lanes, and a right turn lane. Two PTSWF signs along with flashers are positioned on both sides of Highway 2 563 ft. from the stop bar. Figure 3.1 illustrates what a driver approaching the intersection sees.

Figure 3.1: View of advance warning flashers prior to intersection

3.2.2. <u>US 77 and Saltillo Road</u>

The second intersection studied was the northbound approach of US77 and Saltillo Rd. Located east of Lincoln, Nebraska, US Highway 77 runs north and south. The intersection has two through lanes and both a left and right turn lane. Two PTSWF flashers are positioned on both sides of US77 650 ft. from the stop bar. The speed limit is 65 mph until approximately 1150 ft. before the intersection, when the speed limit changes to 55 mph.

3.2.3. US 77 and Pioneers

US77 and Pioneers Blvd., located 5 miles north of US 77 and Saltillo, was the third intersection studied. The southbound approach of US77 and Pioneers has two through lanes and one left turn lane. Two PTSWF flashers are positioned on both sides of US77 650 ft. from the stop bar. The speed limit is 55 mph along this stretch of US 77.

3.2.4. Highway 34 and N79

The last intersection studied in Lincoln was the westbound approach of Highway 34 and N79. This intersection is northwest of Lincoln, with a speed limit of 60 mph. With no left turn lane and a turnoff for vehicles desiring to travel north prior to the intersection, the westbound approach has only two through lanes. In addition, the intersection is equipped with two PTSWF flashers 650 ft. from the stop bar.

3.2.5. Highway 75 and Platteview Road

Shown in Figure 3.6, US75 and Platteview Road, is located south of Bellevue, Nebraska. The southbound approach of US75 and Platteview has two through lanes and both a right and left turn lane. Two PTWSF flashers are positioned on both sides of US75 438 ft. from the stop bar. Approximately 1550 ft. upstream of the intersection the speed limit changes from 60 mph to 55 mph.

3.2.6. SR32 and SR 37

The last site used for data analysis was the signalized intersection of SR37 and SR32 at Noblesville, Indiana. The southbound approach of SR37 has two through lanes and both a right and left turn lane. The speed limit of SR37 is 55 mph. Unlike the other five sites, this intersection does not have advance warning flashers. In addition, the WAD and camera were mounted on the mast arm contrasting the previous locations where both were located on the side of the road.

3.2.7. Summary

This section has described the six data sites to be used for the analysis. Shown below in Table 3.1 is a summary of site characteristics with aerial photographs of each site presented in Figure 3.2. In addition, Table 3.1 displays the code to be used for the remainder of the thesis for each site.

Table 3.1: Summary of site characteristics

	Saltillo	**Highway 2**	**Pioneers**	**US 34**	**US 75**	**SR 37**
Site Code	Site 1	Site 2	Site 3	Site 4	Site 5	Site 6
Yellow phase	4.4 s	5.6 s	4.9 s	4.4 s	4.5 s	5.0 s
Mean speed (mph)	54.1	48.5	52.8	56.6	51.4	46.6
Posted speed limit (mph)	55	55	55	60	55	55
85th Percentile speed (mph)	64	55	58.3	63	61	55
Use of AWF	Yes	Yes	Yes	Yes	Yes	No
AWF Distance	650 ft.	563 ft.	650 ft.	650 ft.	470 ft.	-
AWF Time before yellow	7.0 s	8.0 s	8.0 s	7.0 s	6.0 s	-
Through Lanes	2	2	2	2	2	2
Right or Left turn lane	Both	Neither	Both	Both	Both	Both

Site 1 – US 77 & Saltillo Rd. Lincoln, NE (NB Approach)	Site 2 – Highway 2 & 84th St. Lincoln, NE (SEB Approach)	Site 3 – US 77 & Pioneers Blvd. Lincoln, NE (SB Approach)
		
Site 4 – US34 & N79 Lincoln, NE (WB Approach)	Site 5 – US 75 & Platteview Rd. Bellevue, NE (SB Approach)	Site 6 – SR 37 & SR 32 Noblesville, IN (SB Approach)
		

Figure 3.2: Data collection sites

3.3. Data Collection

In order to keep consistency during the collection period, data was only collected during good weather days. Good weather days were defined as days with no precipitation and constant wind speeds less than 10 mph. In addition, a variety of traffic conditions were examined by collecting data during both peak and off-peak hours. A summary of the days and times data was collected at each sites is shown in Tables 3.2 and 3.3. Data was collected at five intersections using two different setups. This section will discuss the three different equipment setups and the calibration of each setup.

Table 3.2: Summary of Data Collected at AWF Locations

Site Location	Day Collected	Hours Collected
Site 1 - US-77 & Saltillo Rd., Lincoln (Northbound)	September-29-2010	8:00 AM - 4:00 PM
	September-30-2010	10:45 AM - 6:00 PM
Site 2 - Highway 2 and 84th St., Lincoln (Eastbound)	July-07-2010	10:30 AM - 12:30 PM; 1:00 PM - 3:00 PM
	July-09-2010	10:30 AM - 12:30 PM; 1:00 PM - 3:00 PM
	July-15-2010	10:30 AM - 12:0 PM; 1:00 PM - 3:00 PM
	July-16-2010	10:30 AM - 12:30 PM
	July-19-2010	1:00 PM - 3:00 PM
	July-20-2010	1:00 PM - 1:45 PM; 2:45 PM - 3:00 PM
	November-08-2010	10:45 AM - 5:00 PM
	November-15-2010	3:30 PM - 4:45 PM
	November-16-2010	1:30 PM - 3:00 PM
	November-17-2010	2:30 PM - 4:00 PM
	November-22-2010	1:45 PM - 4:00 PM
	November-23-2010	1:45 PM - 4:00 PM
Site 3 - US-77 & Pioneers Blvd., Lincoln (Southbound)	October-13-2010	8:00 AM - 4:00 PM
	October-14-2010	8:00 AM - 4:00 PM
Site 4 -US-34 & N-79, Lincoln (Westbound)	October-20-2010	8:00 AM - 4:00 PM
	October-21-2010	8:00 AM - 4:00 PM
	November-23-2010	11:00 AM - 4:15 PM
Site 5 -US-75 & Platteview Rd., Bellevue (Southbound)	November-18-2010	8:00 AM - 3:00 PM
	November-19-2010	9:00 AM - 4:00 PM

Table 3.3 Summary of data collection at Noblesville

Site Location	Day Collected	Hours Collected
Site 6 - SR37 and SR32, Noblesville (Southbound) collected by Sharma et al. (*40*)	September-12-2007	6:00 AM - 9:00 AM
	September-20-2007	6:00 AM - 7:00 AM
	September-28-2007	6:00 AM - 12:30 PM & 6:00 PM - 8:00 PM
	October-02-2007	6:00 AM - 10:00 AM
	October-03-2007	1:00 PM - 4:00 PM & 6:00 PM - 8:00 PM
	October-05-2007	9:00 AM - 11:00 AM & 7:00 PM - 8:00 PM
	October-09-2007	3:00 PM - 4:00 PM & 6:00 PM - 8:00 PM
	October-12-2007	3:00 PM - 8:00 PM
	October-29-2007	6:00 PM - 8:00 PM
	October-30-2007	6:00 AM - 10:00 AM & 7:00 PM - 8:00 PM
	November-01-2007	6:00 AM - 12:00 PM & 6:00 PM - 8:00 PM
	November-02-2007	6:00 AM - 11:00 AM &3:00 PM - 5:00 PM
	November-09-2007	8:00 AM - 11:00 AM
	November-27-2007	7:00 PM - 8:00 PM
	February-28-2008	6:00 AM - 6:00 PM
	March-11-2008	9:00 AM - 2:00 PM
	March-12-2008	12:00 PM - 7:00 PM
	March-23-2008	10:00 AM - 8:00 PM
	March-24-2008	8:00 AM - 12:00 PM
	April-02-2008	6:00 AM - 8:00 AM
	April-05-2008	6:00 AM - 9:00 AM
	April-06-2008	6:00 AM - 7:00 PM
	April-07-2008	6:00 AM - 8:00 AM
	April-14-2008	6:00 AM - 3:00 PM & 5:00 PM - 8:00 PM
	April-15-2008	6:00 AM - 2:00 PM
	April-21-2008	7:00 PM - 8:00 PM
	April-22-2008	6:00 AM - 6:00 PM
	April-28-2008	10:00 AM - 1:00 PM
	April-29-2008	6:00 AM - 8:00 PM
	April-30-2008	6:00 AM - 8:00 PM

3.3.1. Site One - Highway 2 and 84^{th} St.

3.3.1.1. Data Collection Setup

Highway 2 and 84^{th} St. is instrumented with three wide area detectors (WAD) which can record individual vehicle information. Two SmartSensor Advance WADs, utilizing digital wave radar technology, installed on the research pole, shown in Figure 3.3, track the vehicles upstream and downstream of the pole and record their distance, speed, lane, and vehicle length up to a distance of 500 ft. A SmartSensor HD acts as the midstream sensor and records the vehicles information equidistant with the research pole. In addition to recording speed, the SmartSensor HD identifies the lane a vehicle travels in and records the vehicle length. The overall data collection schematic is shown below in Figure 3.3. Location A, shown in Figure 3.3, represents the fixed research pole placed 473 ft. from the stop bar. This location will be referred to as the research pole throughout the remainder of this section.

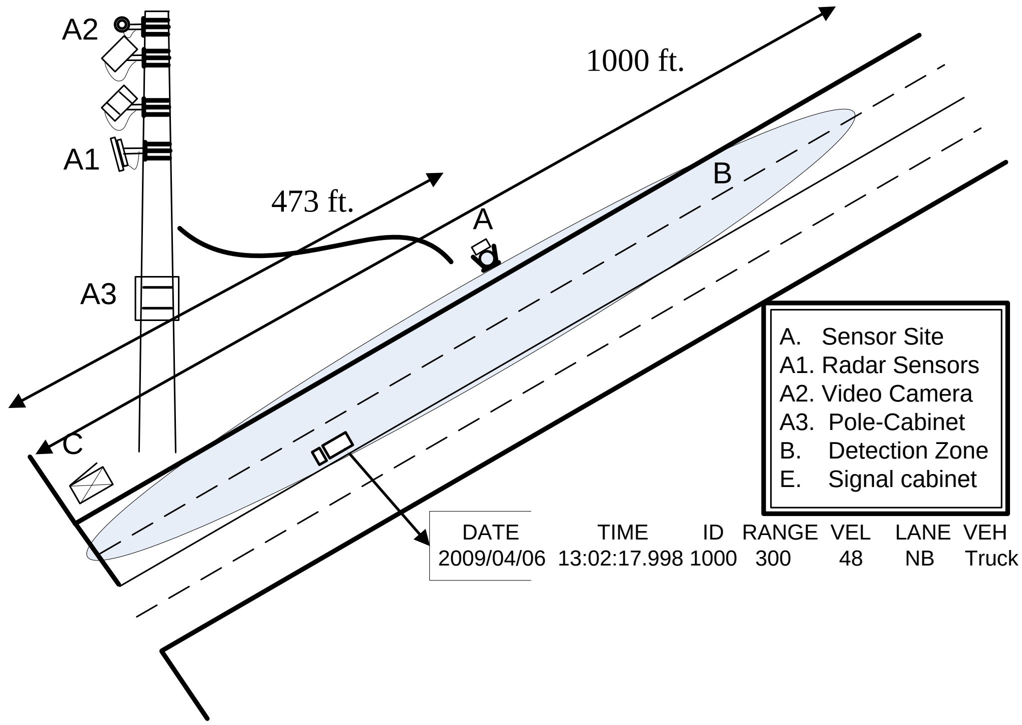


Figure 3.3: Schematic of data collection at Highway 2 and 84th St.

Figure 3.4 and Figure 3.5 display images of the SmartSensor Advance and SmartSensor HD, respectively. Two Click! 500 programmable controllers were used in the field. Signal status was collected by a Click! 500 installed at the traffic cabinet and sent through fiber to a second Click! 500 installed in a cabinet at the research pole. In addition, the Click! 500 located in the cabinet of the research pole extracts the data from three Click! 200's, one for each WAD, and brings together the information.

Figure 3.4: Wavetronix SmartSensor Advance

Figure 3.5: Wavetronix SmartSensor HD

Time synchronization is maintained with reference to the Click! 500 real time clock installed in the cabinet on the research pole. The phase-reading Click! 500 located in the traffic cabinet gets updates from the research pole's Click! 500 through fiber. The time stamping for all three WADs is performed by the research pole's Click! 500. The upstream and downstream latency is 21 milliseconds, while the midstream sensor latency is 6 milliseconds, thus the system is highly accurate.

In addition to the three WADs installed at the data collection site, three cameras were placed to record vehicle movement through the site. Two Axis 232D+ dome cameras, shown in Figure 3.6, were mounted on the research pole approximately 25 feet above the ground. These cameras recorded vehicular movement upstream and downstream of the research pole, while the third Axis camera was mounted on the mast arm. Figure 3.7 illustrates the three vehicular movement views recorded. The top two view on Figure 3.7 represent the upstream and downstream views from the research pole, while the view on the bottom displays the camera mounted on the mast arm. This camera was only used for recording the decision of the driver at the onset of yellow: stop/go.

Figure 3.6: Visualization of Axis 232D+ dome camera

Figure 3.7: Display of recorded vehicular movement through data collection site

Data from the WADs was collected through placing a serial cable connecting the RS-232 on the Click! 500 to a CPU in the research pole. Matlab was used to open the serial connection and save the data. The three cameras were displayed on the computer screen using Active Webcam, which captures images up to 30 fps. Finally, Hypercam 2 was used to record the screen captures from Active Webcam as shown in Figure 3.7. Only instances were a single vehicle was presented were recorded.

3.3.1.2. Validation

The WADs were validated against the Xsens MTi-G, an integrated GPS and Inertial Measurement Unit (IMU). In addition to capturing a vehicle's position from the GPS unit, the MTi-G provides measurement of the vehicle's acceleration in the X, Y, and Z direction at a rate of 100 data points a second. Setup of the MTi-G is shown below in Figure 3.8.

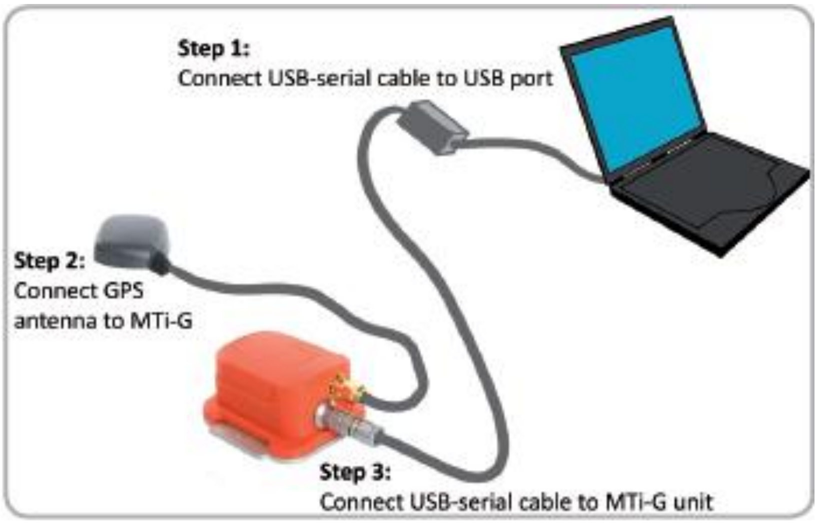


Figure 3.8: MTi-G Setup (*87*)

Validation runs were made using a Honda Civic. Five runs were made using different speeds and lanes to ensure proper performance of the WAD. The times were manually synced between computers using a handheld GPS device. With the GPS as the reference time, both computer times were changed to the time given by the GPS. An example of the tracking performance of the MTi-G and WAD is shown in Figure 3.9. The root mean square error (RMSE) in distance was reported as 9.6 ft.

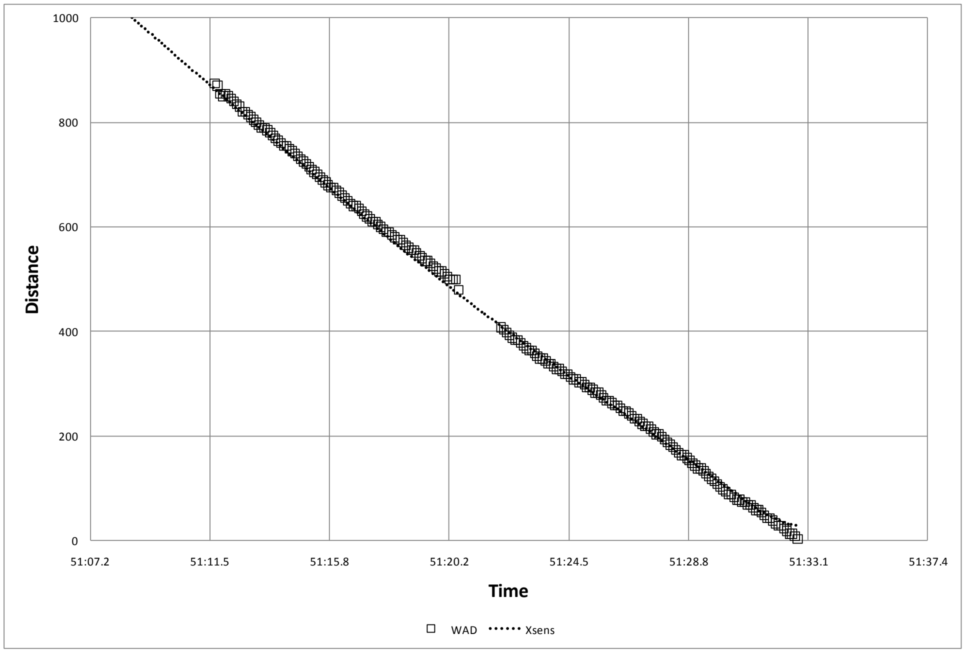


Figure 3.9: Example comparison between WAD and Xsens

3.3.2. Mobile Trailer

3.3.2.1. Data Collection Setup

The remaining sites located were collected using a portable trailer, as shown below in Figure 3.10. The portable trailer could only be used on good weather conditions, as strong continuous or gusting wind would cause the trailer's mast arm to sway and would result in bad data. Based on field experience it was found that average wind speeds of larger than 10 mph caused the mast arm to sway. Therefore, data was collected on days with no precipitation and when the average wind speeds were below 10 mph.

Figure 3.10: Mobile data collection trailer

Similar to the setup in the previous section, the data collection trailer was equipped with three WADs. Two SmartSensor Advance WADs installed on the pole track the vehicles upstream and downstream of the pole, with the SmartSensor HD acting as the midstream sensor. Two Click! 500 programmable controllers were also used. Signal status was received from the Click! 500 installed at the traffic cabinet, through the portable signal phase reader, shown below in Figure 3.11.

Figure 3.11: Safe Track Portable Signal Phase Reader

The signal phase reader communicates the signal phase status by wireless to the portable sensor pole cabinet. This cabinet features three Click! 200's that collect the data from each detector and then send it to the Click! 500; thus, the Click! 500 in the pole cabinet receives the data from the signal and all three detectors. Figure 3.12 displays the portable sensor pole cabinet.

Figure 3.12: Portable sensor pole cabinet

Time synchronization of the portable system is maintained with reference to the trailer's Click! 500 real time clock. The phase-reading Click! 500 syncs from the trailer's Click! 500 through wireless updates. Time stamping for all three WADs is performed by the trailer's Click! 500. The upstream and downstream latency is 21 milliseconds, while the midstream sensor's latency is 6 milliseconds. The calculated drift in synchronization for the entire system is 97 milliseconds by adding the following component drifts:

- 70 ms for the phase information
- 21 ms for the upstream and downstream sensor
- 6 ms for the midstream sensor

Therefore, the entire system has a time resolution accuracy of at least a 10^{th} of a second. The data is pushed or sent from the Click! 500 using the device's serial port and a Serial to USB converter that connects to a laptop. Matlab opens the serial port and saves the data in both .DAT and .txt files. The data was manually truthed through the use of a Mobotix Q24M camera, Figure 3.13. This fisheye camera can record high-resolution views, with a frame rate of up to 30 fps. As shown in Figure 3.14, the camera was setup to view upstream, midstream, and downstream of the trailer.

Figure 3.13: Mobotix Q24M camera

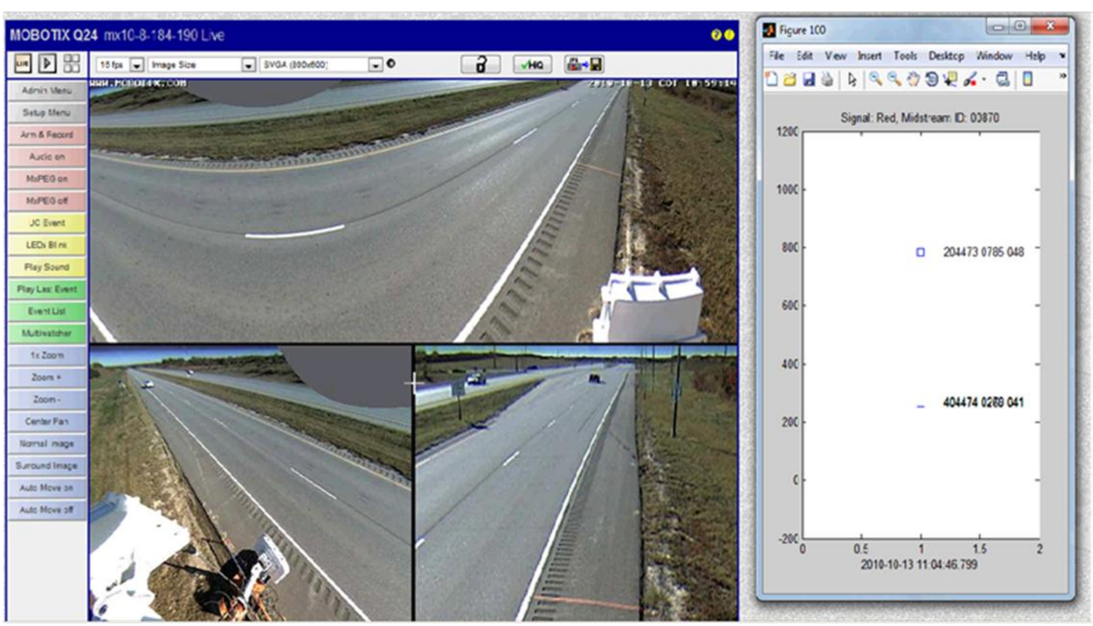


Figure 3.14: Mobile trailer data collection environment

3.3.2.2. Validation

The portable trailer WADs was validated against the MTi-G unit one-time; however, An example of the tracking performance of the MTi-G and WAD is shown in Figure 3.15. The root mean square error (RMSE) in distance was reported as 12.4 ft.

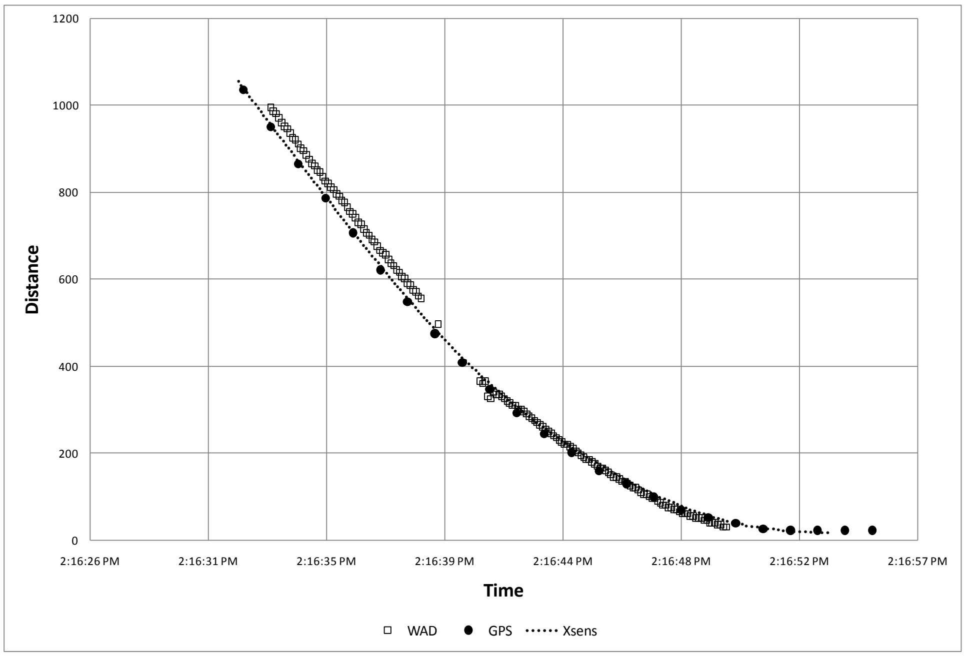


Figure 3.15: Example comparison between WAD, GPS, & Xsens

3.3.3. <u>Noblesville Site</u>

The following section presents the data collected setup performed by Sharma et al. (*40*). A detailed analysis of the performance of the WAD can be found elsewhere (*88*).

3.4. <u>Data Reduction</u>

As a result of the video and WAD data being recorded by Hypercam 2, data reduction was straightforward at most of the sites. The videos were viewed and if any vehicles were present at the onset of yellow their downstream id, range, speed, decision to stop/go, and type of vehicle were recorded. As previously mentioned only single or lead vehicles of a platoon were used for analysis. In addition, if the driver ran the red light it was also noted under the conflict column. Red light runners were defined using the terminology from Section 2.5.2. A sample data reduction form is shown below in Figure 3.16. The date, end of green time, Downstream ID, Range, Speed, decision of driver, and Type of vehicle were noted.

	Wave	Vehicle present control region					
		Lane SA					
Date	Green end	Downstrea	Range	Speed	Stop/Go	Conflict	Type
11/17/2010	49:29.9	546	Data Fusion		Stop		Car S
11/17/2010	51:05.2	557	13	56	Go		Car S
11/17/2010	56:05.1	24	81	44	Go		Suv S
11/17/2010	57:45.1	199	195	54	Go		Suv S

Figure 3.16: Sample data reduction form

Other than the Noblesville site each vehicle has three distinctive vehicle ids (Upstream, Midstream, and Downstream) assigned to them, as a result of using three WADs to track the vehicle. The downstream id was primarily the only id recorded; however, there were drops in the WAD coverage area between the upstream and midstream and midstream and downstream detector or approximately 50 and 100 ft., respectively. At these locations, a vehicle's distance and speed were not picked up by the WADs. If at the onset of yellow a vehicle was present in between the midstream and downstream detectors, the data for all detectors was fit using either a linear or two-degree polynomial, as shown in Figure 3.17 and Figure 3.18 using the information from all three detectors. The best R^2 value was used for determination of using a linear or two-degree polynomial to fit the vehicles trajectory. Table 3.4 reveals the number of vehicles requiring data fusion.

Table 3.4: Amount of vehicles requiring data fusion

	Saltillo	Highway 2	Pioneers	US 34	US 75
Vehicles requiring data fusion	45	59	33	27	78
Total Vehicles	160	437	166	144	247
Linear	9	8	7	16	13
2-degree polynomial	36	51	26	11	65

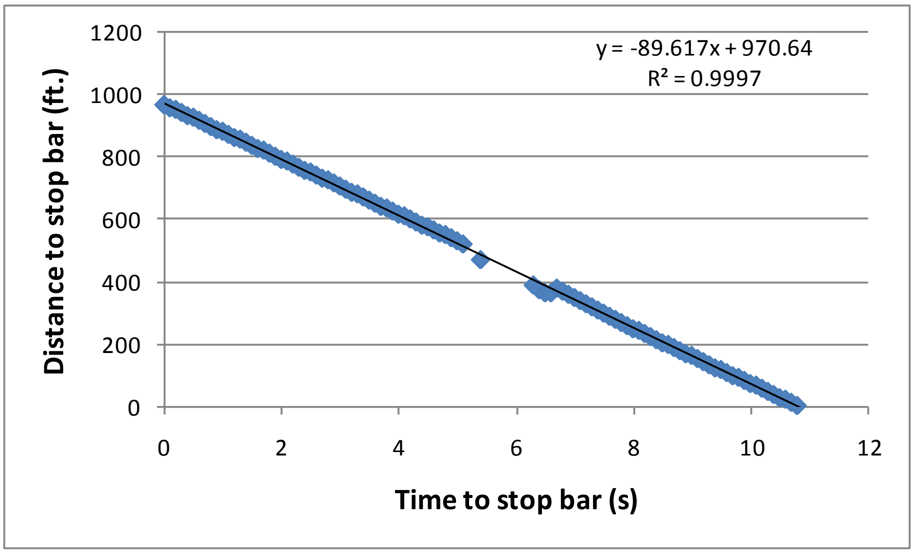


Figure 3.17: Example of linear fit to vehicle

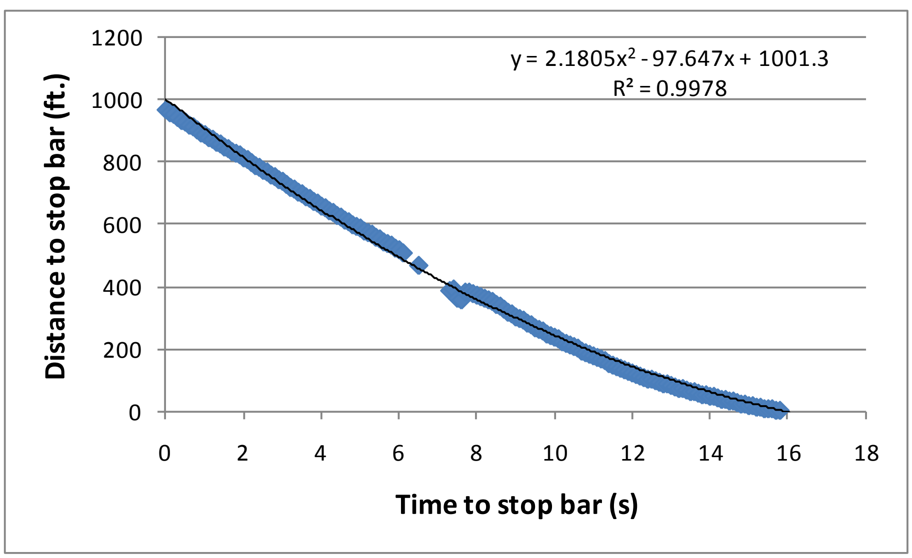


Figure 3.18: Example of two-degree polynomial fit to vehicle

3.5. Summary

In this chapter the data collection and evaluation sites were described, along with the dates and time of collection. The equipment, software and setup used in the field to collect the data was explained. In addition, figures presented the calibration of the three systems with GPS units. Finally, the method of data processing was described.

CHAPTER 4. METHODOLOGY & RESULTS

4.1. Introduction

This chapter develops the dilemma zone hazard function for obtaining probability estimates of a perceived traffic conflict occurring. Data reduced as mentioned in the previous chapter was used for the analysis. The probit model, a binary choice model, was used to model the underlying criteria for a driver's decision at the onset of yellow. Two critical thresholds were calculated for a driver approaching the intersection at the onset of yellow: distance requiring severe deceleration by the driver and the distance at which a driver would be required to heavily accelerate or run the red-light. Any erroneous decision by driver in these zones would result in a severe conflict. Finally, the results of the analysis are presented in this chapter.

4.2. Methodology

4.2.1. Underlying Theory on Driver's Decision - Single Site Example

At the onset of yellow a driver can choose from two mutually exclusive courses of action: stop or go; therefore, driver behavior can be modeled as a binary choice process. Recalling the approach developed by Sheffi and Mahmassani (*39*), let T_p be a driver's perceived time to reach the stop bar randomly chosen from a population. As a result of the variance in driver behavior based on several independent factors such as, perception of the yellow interval based on past experience, perception of the distance from the stop bar, perception reaction time, comfortable deceleration rate, etc., T_p can be modeled as a normally distributed random variable, as shown below in Equation 8.

$$T_p = T_{req} + \xi \qquad \text{Equation 8}$$

where:

T_{req}: is the required yellow time to safely enter the intersection based on the vehicle's distance and speed at the onset of yellow

ξ : is a random variable is assumed to be normally distributed

Figure 4.1 illustrates the resulting probability density function (PDF). If the perceived time, T_p, is greater than the critical time threshold to pass through the intersection, a driver will decide to stop, otherwise they decide to go.

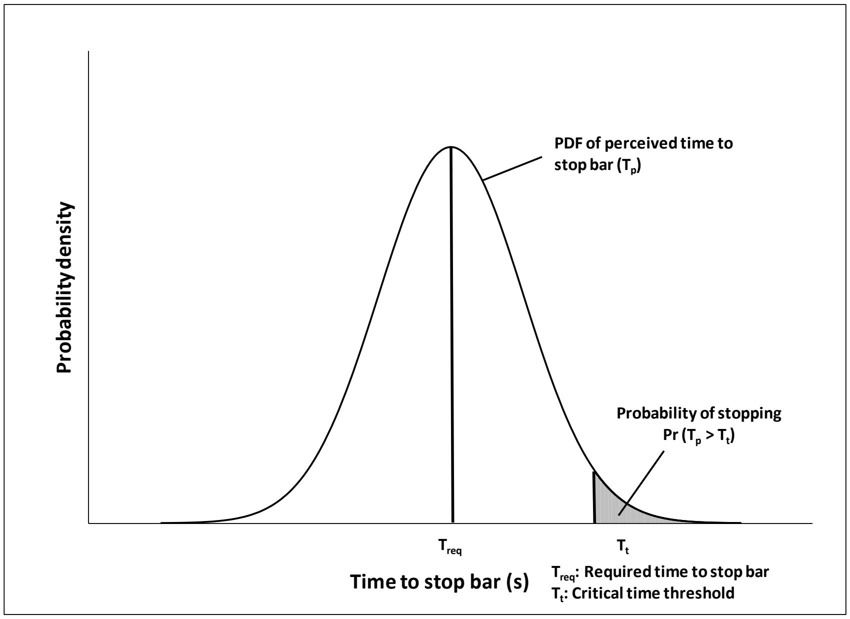


Figure 4.1: Probability density function for perceived time to stop bar

Therefore, the probability of stopping can then be calculated as:

$$P_{Stop} = \Pr\{T_p > T_t\} \qquad \text{Equation 9}$$

$$P_{Stop} = \Pr\{T_{req} + \xi - T_t > 0\} \qquad \text{Equation 10}$$

$$P_{Stop} = \Pr\{\xi > -(T_{req} - T_t)\}$$ Equation 11

$$P_{Stop} = \Pr\left\{\frac{\xi}{\sigma_\xi} < (\frac{T_{req}}{\sigma_\xi} - \frac{T_t}{\sigma_\xi})\right\}$$ Equation 12

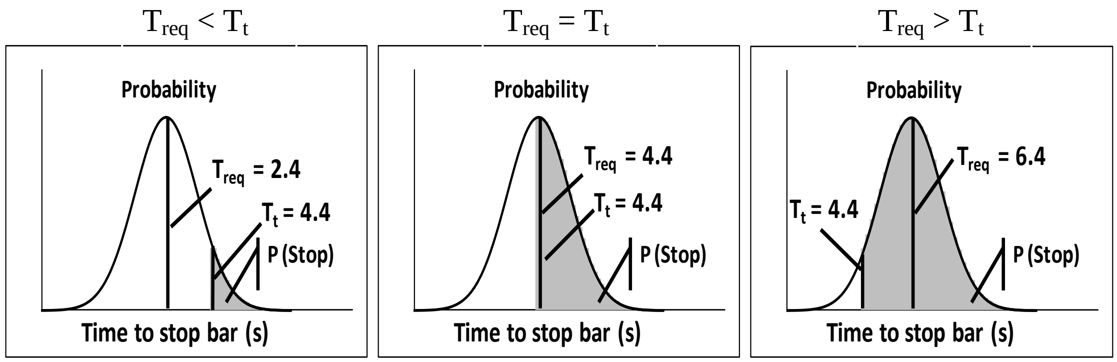
$$P_{Stop} = \Pr\left\{z < \left(\frac{T_{req} - T_t}{\sigma_\xi}\right)\right\} = \Phi\left(\frac{T_{req} - T_t}{\sigma_\xi}\right) = \Phi\left(\mathrm{a} \cdot \mathrm{T_{req}} + \mathrm{b}\right)$$ Equation 13

In addition, the estimates of a and b represent

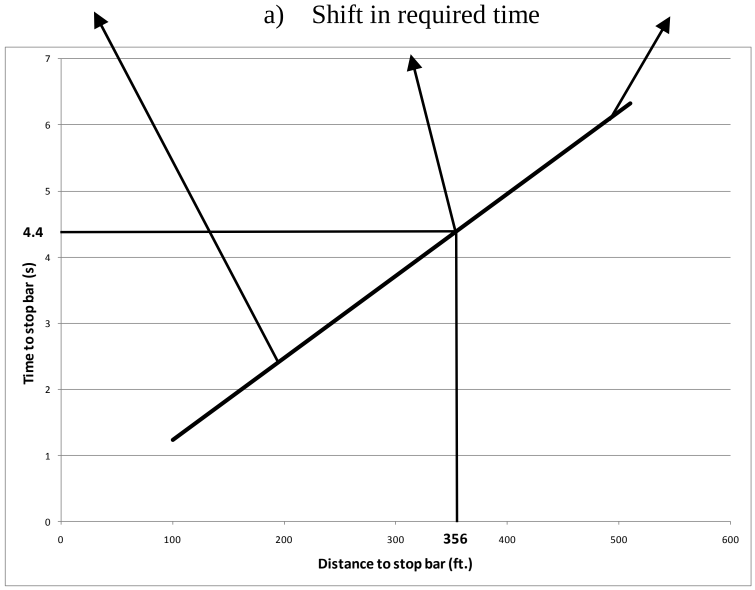
$$a = \frac{1}{\sigma_\xi}\,; b = -\frac{T_t}{\sigma_\xi}$$ Equation 14

Where, Φ(●) represents the standard normal cumulative function and Equation 13 is a probit construct. Estimates *a* and *b* from Equation 13 are imperative to the formation of the probability of stopping curve, as they represent the slope and midpoint. These two estimates, found from a modeling software, allow the standard deviation of the perception error (σ_ξ) and time threshold (T_t) to be calculated. An example of the relationship

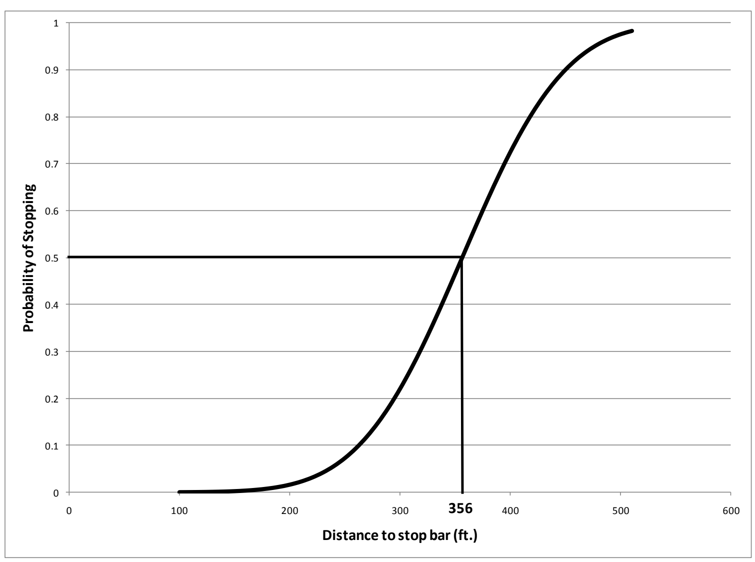
between the time threshold, required time to stop bar, and the probability of stopping is shown below in Figure 4.2. The drivers approaching the stop bar redefine the TTS, thus capturing different areas, as shown in Figure 4.2a. The shifts in the required TTS values result in varying probability of stopping areas. As would be expected, the probability of stopping increases as the required time to stop bar increases, shown in Figure 4.2b. Additionally, Figure 4.2c illustrates that when the time threshold is equal to the time required to stop bar, the probability of stopping is 0.5.



a) Shift in required time



b) Required time to stop bar



c) Probability of stopping curve

Figure 4.2: Relationship between time threshold, required time to stop bar, and probability of stopping

4.2.1.1. Critical Acceleration and Deceleration Thresholds

Two critical thresholds can be calculated for a driver approaching the intersection at the onset of yellow: distance requiring severe deceleration by the driver and the distance at which a driver would accelerate heavily or run the red light. The following calculations were performed as examples of the acceleration and deceleration threshold based off of 85th percentile acceleration and deceleration values from Sharma (*13*). The distance for which a vehicle cannot proceed through the intersection without heavily accelerating or RLR is calculated as shown below:

$$Distance_{Accel} = s \cdot y + \frac{1}{2}a(y-p)^2 \qquad \text{Equation 15}$$

where:

s: speed of the vehicle at the onset of yellow (ft/s)

y: is the length of yellow (s)

a: is the 85th percentile acceleration, 3.19 ft/s^2 (*13*)

p: perception reaction time of 1 s

For a speed of 80.67 ft/s (55 mph) and a yellow length of 4.90s, the critical acceleration distance equals 420 ft. This distance will be referred to as the maximum passing distance throughout the remainder of this thesis and represent the critical acceleration threshold. A vehicle at the onset of yellow upstream of this fixed distance choosing to proceed through the intersection will require heavy acceleration or will run the red light. Similarly, a fixed distance can be calculated where a vehicle will be require to decelerate heavily, as shown in Equation 16.

$$Distance_{Decel} = \frac{s^2}{2\,x\,d} + \ s\ x\ p \qquad \text{Equation 16}$$

where:

d: is the 85th percentile deceleration, 14.41 ft/s^2 (*13*)

Assumptions on formulas:

- Driver behavior can be modeled normally
- Distance calculated using wet coefficients even though data was only on good weather days
- Acceleration and deceleration thresholds used are from Noblesville, IN

Again using 80.67 ft/s (55 mph) and a 4.90s yellow interval, the severe deceleration distance is computed to be 306 ft. A similar recommended severe deceleration rate of 14.76 ft/s^2 can be found in Malkhamah et al. (*89*). A vehicle downstream of this distance choosing to stop will be required to decelerate heavily to stop prior to the stop bar. The two critical threshold distances previously calculated are shown in Figure 4.3.

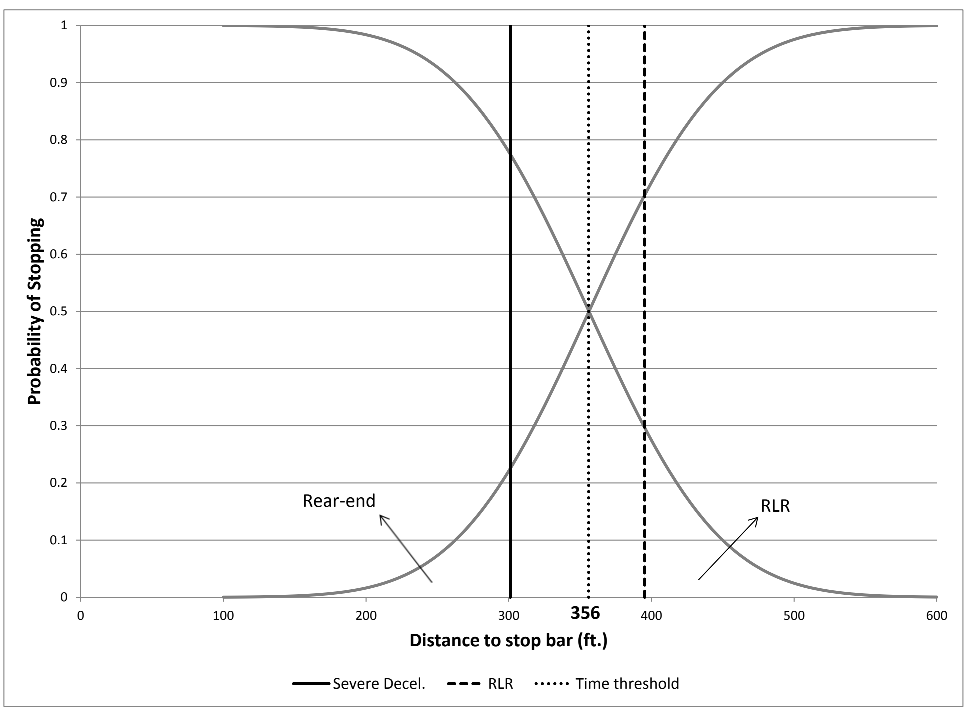


Figure 4.3: Critical distances along probability of stopping curve

Drivers choosing to stop downstream of the severe deceleration distance and choosing to proceed upstream of the maximum passing distance have made an erroneous decision. The consequences of a driver making an erroneous decision at the onset of yellow can lead to a conflict and in the previously mentioned cases a severe conflict. The probability of perceived conflict can be calculated using the critical thresholds and stopping probabilities as shown below in Equation 17 (*40*).

$$P_{CONFLICT} = \begin{cases} P_{STOP} & \forall D_{req} < D_t \\ P_{Go} = 1 - P_{STOP} & \forall D_{req} > D_t \end{cases} \qquad \text{Equation 17}$$

where:

D_{req}: Required distance to perform chosen decision

D_t: critical distance threshold depended on yellow time

Perceived conflicts can be classified into minor and severe based on the magnitude of the acceleration or deceleration required to perform the chosen decision and the typical ranges of acceleration or deceleration used by drivers. The required acceleration or deceleration to complete the chosen action therefore can be used to determine the severity of the evasive action needed. If the required acceleration or deceleration is within the typical operating ranges, a minor traffic conflict would occur; but if the required acceleration or deceleration is greater than the thresholds of the typical ranges, a severe traffic conflict would transpire. Drivers in the zone of a minor conflict are likely to have minor traffic conflicts such as an abrupt stop, applying the brakes before proceeding, or acceleration through yellow. However, the drivers in the zone of severe conflict will have severe traffic conflicts such as running a red light, swerving to avoid a collision, or vehicle skidding. Figure 4.4 displays the severe probability of conflict curves.

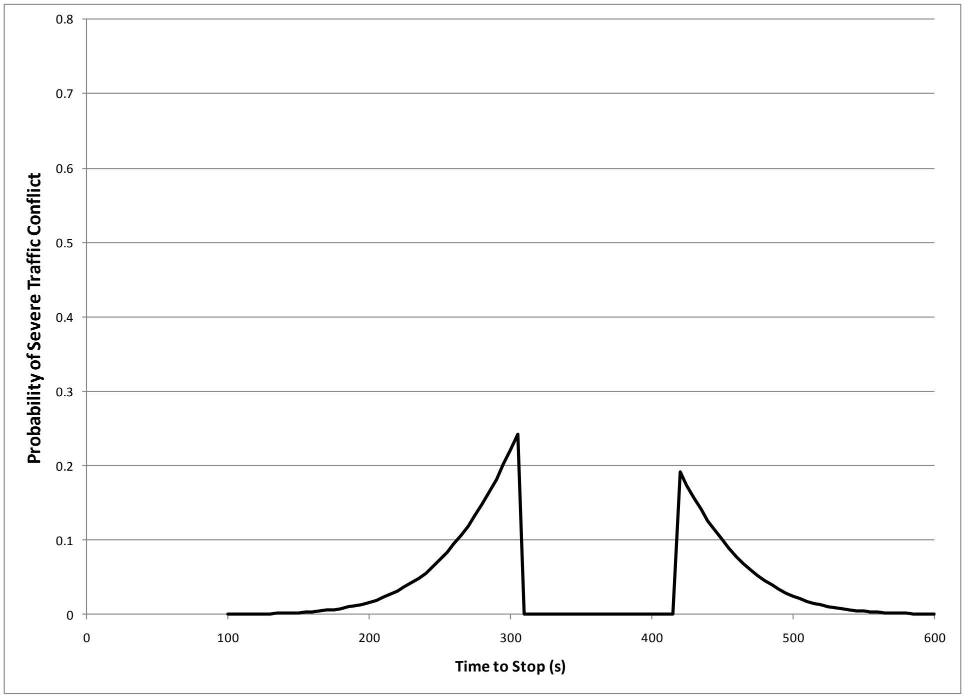


Figure 4.4: Probability of severe conflict

4.2.1.2. Risk Associated By Making an Erroneous Decision

The risk associated with drivers making an erroneous decision can be quantified. Multiple probability of stopping curves must be developed, as a result of the critical acceleration and deceleration thresholds being dependent on time and the speed variability of drivers approaching an intersection. The estimated parameters, *a* and *b*, acquired from the modeling software are plugged into Equation 13 to develop probability of stopping curves for speeds of 35, 40, 45, 50, 55 and 60 mph. Figure 4.5 presents the resulting probability of stopping curves for multiple speed ranges. The first step to quantifying the risk is to integrate the area under both severe conflict thresholds. An average of the integration is computed. Lastly, the proportion of vehicles within each speed category is multiplied by the averaged integration resulting in a weighted average of risk for a driver approaching an intersection.

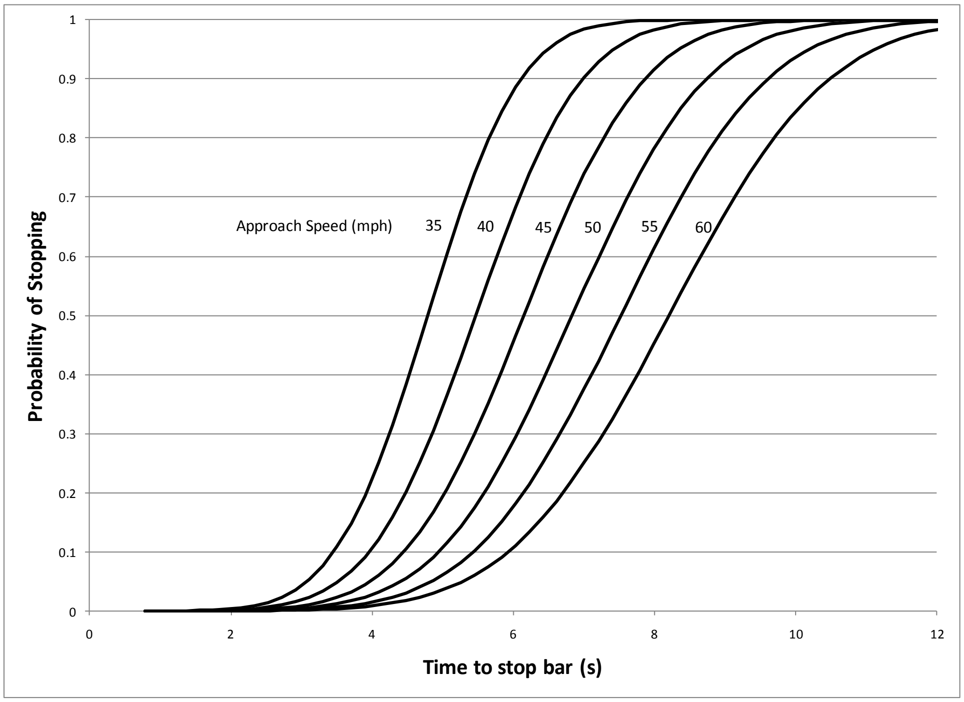
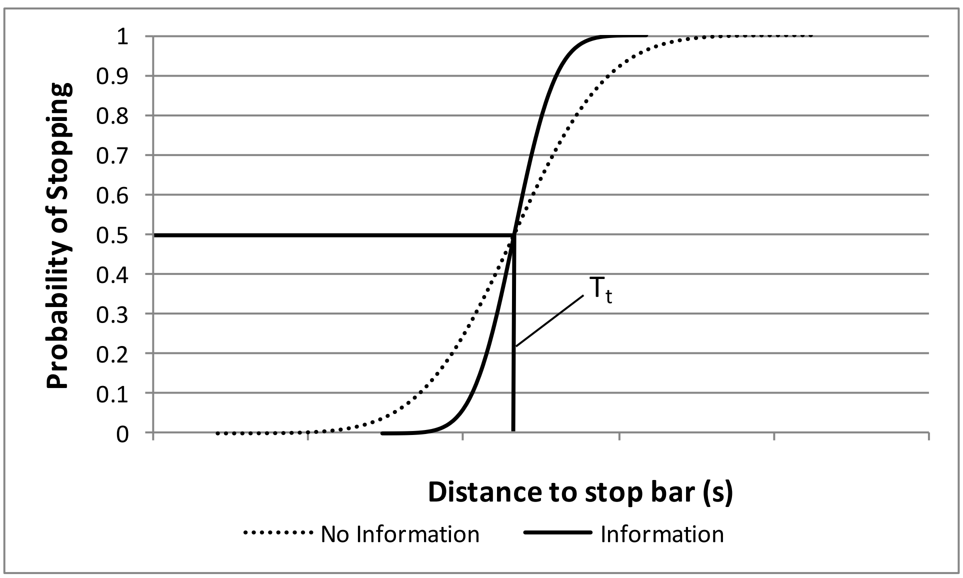


Figure 4.5: Example of varies probability of stopping curves

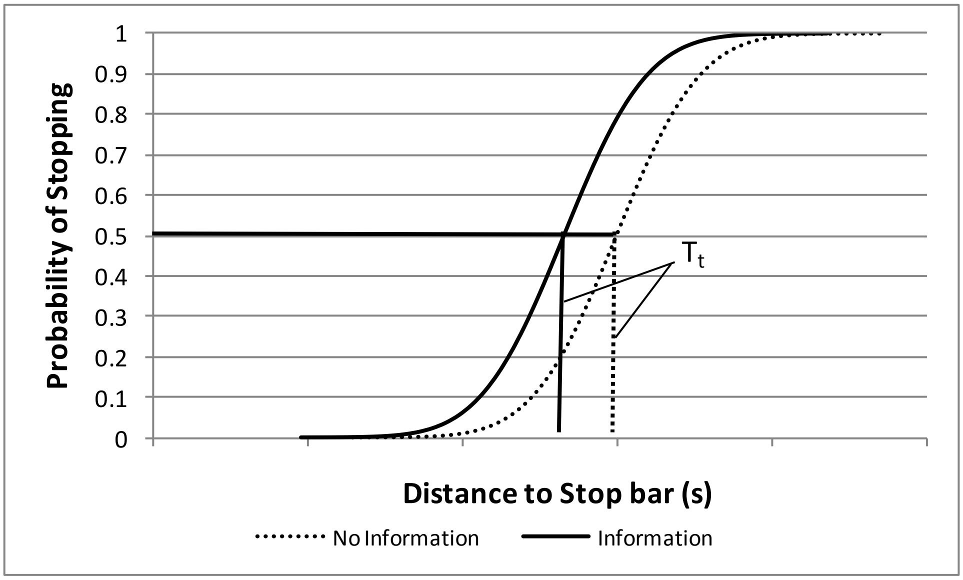
4.2.1.3. Effect of Information

Providing drivers with information through AWFs has shown to alter the probability of stopping curves (*27*). Consider, the potential effect of information at an intersection on the standard error (indecision at the onset of yellow), as shown in Figure 4.6a. It can be seen that by providing information the probability of stopping curves becomes steeper due to a reduction in variability. Ideally, the slope of the probability of stopping curve would be infinity meaning every driver is making the correct decision at the onset of yellow. However, if information shifts the midpoint, the calculated time threshold from Equation 14, the entire probability of stopping curve is shift, as shown in Figure 4.6b.

The probability of stopping curve could be shifted closer or further away from the intersection. Recalling that probability of conflict is dependent upon probability of stopping and the two critical thresholds are fixed results in a shift in the probability of conflict curve. If the probability of stopping curve were shift closer to the intersection the probability of severe deceleration would increase. Conversely, a shift in the probability of stopping curve further away from the intersection would result in an increase in RLRs. This thesis will examine the effects of information on the potential shift in the midpoint as well as on the change in slope on the probability stopping curves.



a) Effect on probability of stopping



b) Effect on probability of stopping

Figure 4.6: Effect of information provided to drivers

4.3. <u>Methodology in Comparing Multiple Sites</u>

Researchers (*90,91*) have found the need to develop site specific dilemma zone boundaries, as a result of both variance in driver behavior and site characteristics. Shown previously in Figure 4.2, variance in driver behavior alters the time threshold values under the PDF potentially shifting or changing the slope and midpoint of the probability of stopping curves. Results similar to the conceptual example of Figure 4.6 are likely in the event of comparing sites providing and not providing information to drivers. Therefore, it is desired to be able to find the statistical significance of the slope and midpoint between sites. The following methodology will be used to determine this statistical significance between multiple sites.

4.3.1. Utilization of Econometric Modeling

The statistical significance is calculated using econometric modeling to find model estimates. By setting up dummy variables dependent on site location and in this analysis a vehicle's time to stop bar, the statistical significance can be tested. Past research (*39,42,44,47*) has advocated the use of time to stop bar as the primary independent variable for modeling driver's decision at the onset of yellow. Dummy variables were setup to test for statistical significance of the variables. This site represents the overall model estimates. The following example will demonstrate this procedure. Two sites are tested to compute the effect of information provided to drivers. One of the sites provides information to drivers through the use of AWFs, while the other does not. Therefore, the site not providing information is selected as the variable to be all zeros. A probit model estimates the following parameters, where:

- Constant = a constant
- Timestop = the instantaneous time to stop bar at the onset of yellow based on the vehicles distance and speed
- Site2 = 1 if the location is Site 1, otherwise if it is Site 2 it equals 0
- Site2_Time = Timestop multiplied by Site1

Results of the analysis are shown in Table 4.1. Model estimates, *a* and *b*, are used to calculate the standard deviation of the perception error (σ_ξ) and time threshold (T_t) from

Equation 13, as shown in Table 4.2. The values in the standard deviation column of Table 4.2 represent the variance in driver's decision at the onset of yellow, while the time threshold values represent when $T_t = T_{req}$. As mentioned previously, this time threshold represents 0.5 on the probability of stopping curve, which is also the midpoint.

Table 4.1: Model results

Variable Name	Value	Standard Error
Constant	b	s_1
Timestop	a	s_2
Site2	b_1	s_3
Site2_Time	a_1	s_4

Table 4.2: Standard deviation and time threshold values

Site	Standard deviation	Time threshold
Site 1 - Overall	$1/a$	-b*a
Site 2	$1/(a+a_1)$	$-(b+b_1)*(a+a_1)$

The effects on parameters a_1 and b_1 are shown below in Table 4.3, assuming a positive b value and negative a value. If a_1 was a negative value, the slope of the probability stopping curve would become steeper, while if a_1 was positive the slope would become gentler. The remainder of this chapter presents the results of the analyses described in the previous sections.

Table 4.3: Effects on model estimates on probability of stopping

	a	b	
a_1 (+)	probability stopping slope becomes gentler	midpoint further from intersection	b_1 (+)
a_1 (-)	probability stopping slope becomes steeper	midpoint closer to intersection	b_1 (-)

4.4. Econometric Modeling for Insight into Effect of Information on Driver Decision

4.4.1. Overall Analysis of Sites

Based on the approach followed by Sharma et al. (*40*), a probit model was used to investigate the influential independent variables on a driver's decision at the onset of yellow. Initially, the data was combined into an overall model for testing the statistical difference between intersections using the dummy variable approach described earlier.

Overall model results are shown below in Table 4.4. Variable descriptions are shown in Table A.1, while the site number references are locate in Table 3.1. As an example, Site1 indicated whether or not a vehicle was present at Site1 and likewise for Site2, Site3, etc. The variable Site1_Time represents the time to stop bar if the vehicle is present at Site1 and so forth for the remaining sites. As expected, time to stop bar was found to be highly significant. In addition, the following variables were found to be statistically significant at the 90 percentile in the model:

- Constant
- Site 1
- Site 2
- Site 5
- Site1_Time

Table 4.4: Results of overall model

Variable Name	Value	Standard Error	T-stats	P-value
Constant	-4.459	0.187	**-23.891**	**0.000**
Timestop	0.926	0.041	**-22.836**	**0.000**
Site1	2.186	0.342	**6.384**	**0.463**
Site2	1.205	0.337	**3.573**	**0.000**
Site3	-0.415	0.920	-0.451	0.652
Site4	-0.429	0.917	-0.468	0.640
Site5	0.957	0.495	**1.935**	**0.053**
Site1_Time	-0.422	0.081	**-5.224**	**0.000**
Site2_Time	0.071	0.092	0.774	0.439
Site3_Time	0.178	0.193	0.922	0.357
Site4_Time	0.104	0.207	0.501	0.617
Site5_Time	0.121	0.136	0.887	0.375

Therefore, Site 1, Site 2 and Site 5 are statistically different from the remaining three sites. The results of the analysis closely adhere to the individual site characteristics shown in Table 3.1. As expected, most of the Nebraska Department of Roads sites (Site 3, Site 4, Site 5) are clustered together. A change in speed limit prior to the intersection could be a possible explanation for Site 1 showing up significant. As mentioned previously, approximately 1150 ft. prior to the intersection at Site 1 the speed limit drops from 65mph to 55mph. In terms of sites providing driver's with information, Site 2 and site 5 provided the longest and shortest combined flasher and yellow time, 13.6s and 10.5s, respectively. In addition, these two sites' AWF distance varies considerably from the other three sites, i.e. 87 ft. and 180 ft. It is also important to note that even though Site 6 data was evaluated from another state, it did not shown up to be statistically significant. Therefore, the results indicated models from Site 3, Site 4, and Site 6 could be used interchangeably.

Table 4.5 presents the overall estimated parameters, as well as the calculated site specific values. The overall model estimates come directly from Table 4.4; however, the site specific values were calculated as previously described.

Table 4.5: Calculated parameter values

	a	**b**
Overall - Site 6	0.926	-4.459
Site 1	0.504	-2.273
Site 2	0.997	-3.254
Site 3	1.104	-4.874
Site 4	1.030	-4.888
Site 5	1.047	-3.502

Table 4.5 displays the values for the standard deviation of the perception error (σ_ξ), time thresholds, and lengths of yellow for each intersection. These values were calculated using Equation 13 and the parameter values in Table 4.5. In terms of making decisions at the onset of yellow, the standard deviation represents the variance in the driver's decision. As previously discussed, an increase in the variance results in an increase in an increase in erroneous decisions made at the onset of yellow. With nearly double the

standard error of the remaining sites, Site 1 had the largest standard error. The change in speed limit prior to the intersection could be an explanation for this. Site 3 was shown to have the smallest variance. Table 4.6 allows for a comparison between the time threshold and length of yellow at each intersection. Site 1 has the strongest correlation between time threshold and the actual length of yellow, while at Site 2 the time threshold is nearly half of the actual length of yellow. The correlation at Site 6 is very close as well between time threshold and yellow time. Four of the intersections' time thresholds varying within 0.5 seconds of the actual yellow length, while the other two intersections differ by more than 1 second. In the case of Site 2, the time threshold fluctuates by 2.3 seconds. The variance in standard deviation and time thresholds is also shown in Figure 4.7 and Figure 4.8.

Table 4.6: Standard error and time threshold values

Site	Standard Error	Time threshold	Yellow times
Overall	1.080	4.8	5
Site 1	1.984	4.5	4.4
Site 2	1.003	3.3	5.6
Site 3	0.906	4.4	4.9
Site 4	0.971	4.7	4.4
Site 5	0.955	3.3	4.5

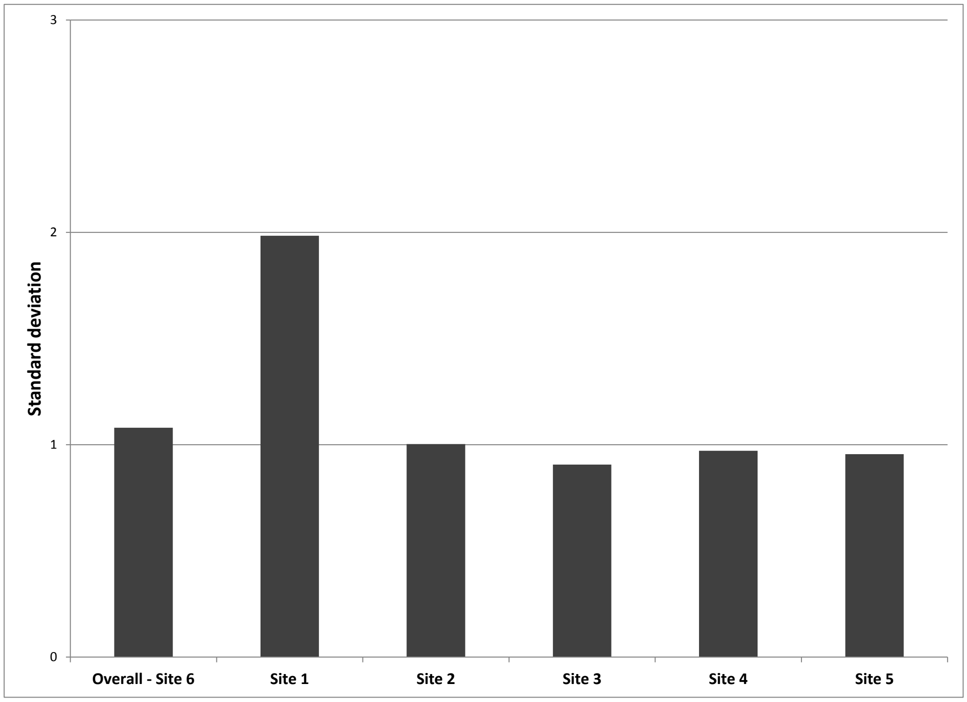


Figure 4.7 Calculated standard deviation

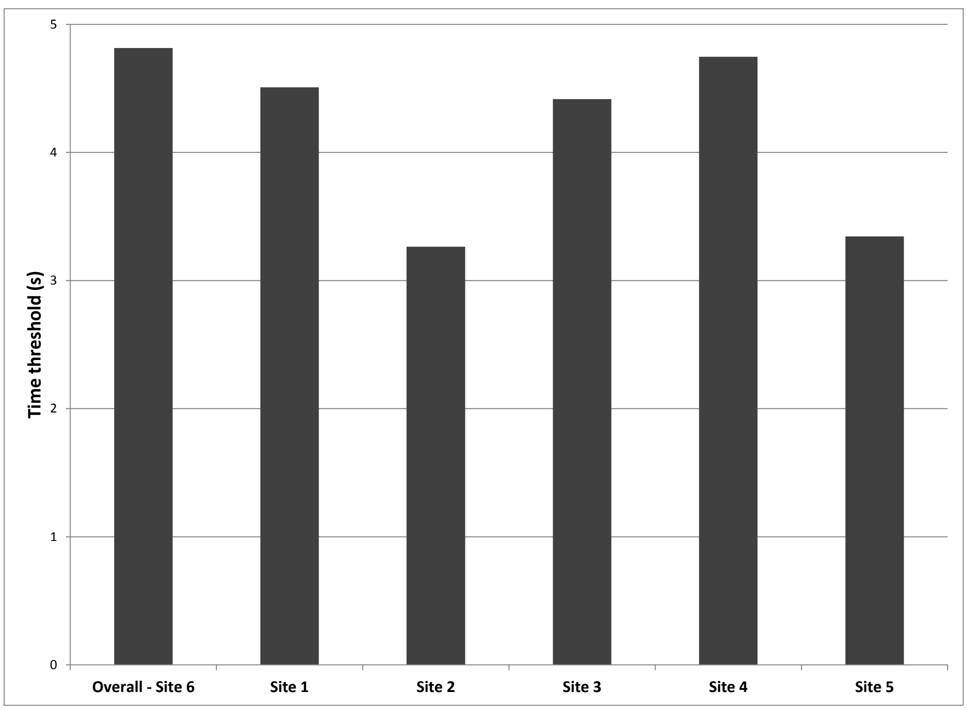


Figure 4.8 Calculated time thresholds

In comparison with previous literature (*27*,*46*,*47*,*92*), the calculated time threshold was plotted against actual length of yellow, as shown in Figure 4.9. Four intersections were graphed from Hurwitz (*92*); however, the time threshold and actual yellow lengths for all four intersections were four seconds. Intersections with AWFs, or, in the case of Koll et al. (*27*) flashing green, were plotted separately from intersections not providing drivers' information. Based on this sample of intersections, drivers approaching intersections without being provided information correctly perceived the time threshold, while drivers inaccurately predicted the time threshold at intersections providing them information. The largest outliers from Figure 4.8 are points A, B, and C, which represent Site 2, Site 5, and Koll's (*27*) studied sites in Austria. In addition, Figure 4.9 displays what type of risk is associated with being above or below the line. The three previously mentioned sites have the potential for increases rear-end risk, as these intersections all fall below the line. Conversely, any intersection above the line would have the potential for increased RLR risk.

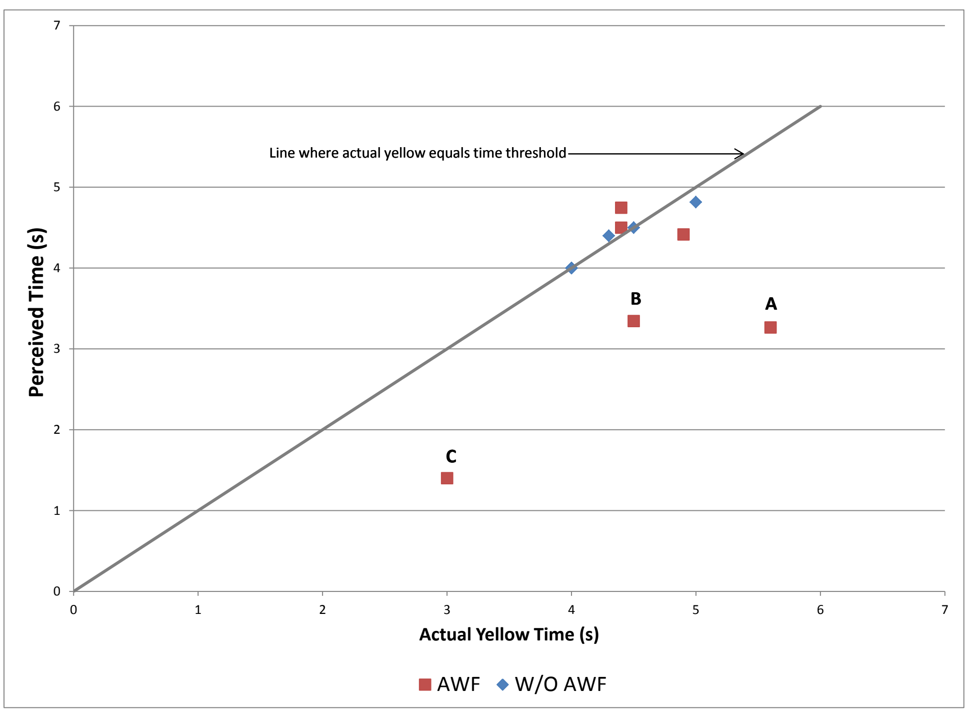


Figure 4.9 Comparison between yellow length and time threshold

Having identified two significantly different groups of intersections, an extensive analysis was performed to understand the underlying criteria between both groups. The first group consisted of Site 2 and Site 5. The remaining sites are classified into group two. This terminology will be used when comparing the two different groups. In addition to time to stop bar, many other variables were tested. A comprehensive list is found in Table A.1. Maximum likelihood estimation technique was used to obtain estimates of the parameters using NLOGIT (*93*). Models were compared using Akaike's Information Criterion (AIC) (*94*). AIC takes into account both the statistical goodness of fit and the number of parameters required to obtain that goodness of fit. As the number of model parameters increase, a penalty is imposed on the model. The best or preferred model is the model that has the lowest AIC value. Results of the analysis are shown below in Table 4.7 and Table 4.8.

Table 4.7: Model results for group 1

Number of observations: 844				
AIC Value: .555				
Unrestricted log likelihood value: -230.5618				
Variable Name	**Value**	**Standard Error**	**T-stats**	**P-value**
Constant	-2.967	0.212	-14.004	0.000
Timestop	0.895	0.054	16.628	0.000
Peak	-0.284	0.134	-2.115	0.034
HV	-0.381	0.218	-1.749	0.080

Table 4.8: Model results for group 2

Number of observations: 3057				
AIC Value: .298				
Unrestricted log likelihood value: -452.1201				
Variable Name	**Value**	**Standard Error**	**T-stats**	**P-value**
Constant	-4.482	0.179	-25.009	0.000
Timestop	0.951	0.039	24.443	0.000
Morning	-0.231	0.114	-2.026	0.043

The estimated parameters were used to develop probability of stopping curves for a speed of 55 mph, as shown in Figure 4.10. The probability of stopping curves reveals the effect of information provided to the drivers from AWFs. Information provided to drivers at the group 1 sites causes a drastic shift in the probability of stopping. The shift in dilemma zone boundaries is shown in Table 4.9. The start and end of the boundaries is shifted closer to the intersection by 1.3 seconds and 1. 5 seconds, respectively for group 1. Results of the models demonstrate the significant shift from information provided to drivers at the onset of yellow from the commonly accepted dilemma zone values of 2.5-5.5 seconds from the stop bar. Figure 4.10 illustrates several critical lines and points represented in Table 4.9.

- Points A and E represent the start and end of the dilemma zone for Group 1
- Points C and F represent the start and end of the dilemma zone for Group 2
- Lines B and D represent the commonly accept dilemma zone boundaries from Bonneson et al. (*95*)

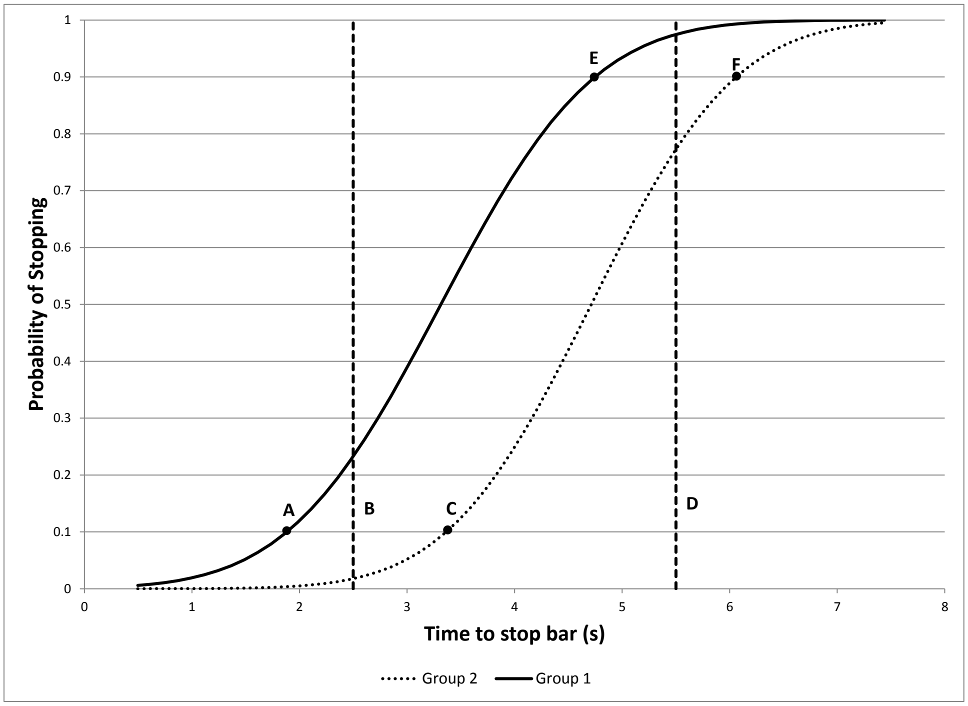


Figure 4.10: Probability of stopping curves for 55 mph

Table 4.9: Dilemma Zone Boundaries

	P = 0.9 (DLZ Start)	P = 0.1 (DLZ End)
Group 1	4.8 s	1.9 s
Group 2	6.1 s	3.4 s
Bonneson et al. (*95*)	5.5 s	2.5 s

4.5. Risk Analysis

The final estimated parameters from Table 4.10 were used to develop probability of stopping curves for speeds of 35, 40, 45, 50, 55 and 60 mph at each site shown in Figure 4.11. The weighted average risk was found for both critical thresholds. Results of the risk analysis are shown in Table 4.11. The effect of information is seen in that the sites seem to mitigate the probability of conflict for one of the two thresholds. As expected, Site 2 and Site 5 have the largest rear-end risk, while Site 4's RLR risk is 4 times higher than any other site.

Table 4.10: Risk for severe conflict

	Site 1	Site 2	Site 3	Site 4	Site 5	Site 6
Rear-end risk	5.55E-05	4.74E-04	1.55E-05	6.86E-06	4.99E-04	1.08E-05
RLR risk	3.27E-05	5.36E-09	2.66E-05	2.04E-04	1.84E-06	4.93E-05

The rear-end and RLR risk were ordered and compared to the actual proportion of vehicles that were required to decelerate heavily. In addition, the RLR risk was ordered and compared with the proportion of RLR's at each site. Results of the comparison are shown in Table 4.12 and Figures 4.14 and 4.15. For the most part, sites with the highest rear-end risk had the highest rear-end average crash history. These results are similar to the previous. Sites 2, 3, and 5 were almost in complete agreement between the calculated risks and observed conflicts. It can also be seen from Figures 4.11 and 4.12 the proportions between the two risks is an inverse relationship. Other than for Site 1 the proportions between calculated risk and observed conflict were in agreement with one another. While some of the sites show a good correlation there appears to be factors not captured by the rear-end risk increasing accidents at Site 1.

Table 4.11: Comparison between risk of conflicts and crash histories

	Site 1	Site 2	Site 3	Site 4	Site 5	Site 6
Rear-end risk	3	2	4	5	1	6
RLR risk	3	6	4	1	5	2
% Severe Deceleration	6	2	4	3	1	5
% RLR	1	6	3	2	4	5

It is evident that providing drivers with information in advance of the intersection using AWFs can potentially cause increased risk in both RLRs and stopping as opposed to decreasing the risk of drivers approaching the intersection. The results are in agreement with previous findings that have revealed significant reductions in RLRs at intersections with AWFs (*20,21,22)*; however, rear-end crash potential has increased as a result

(*23,24,25,26,27*). Thus, caution should be used by engineers before providing drivers with information at a high speed intersection.

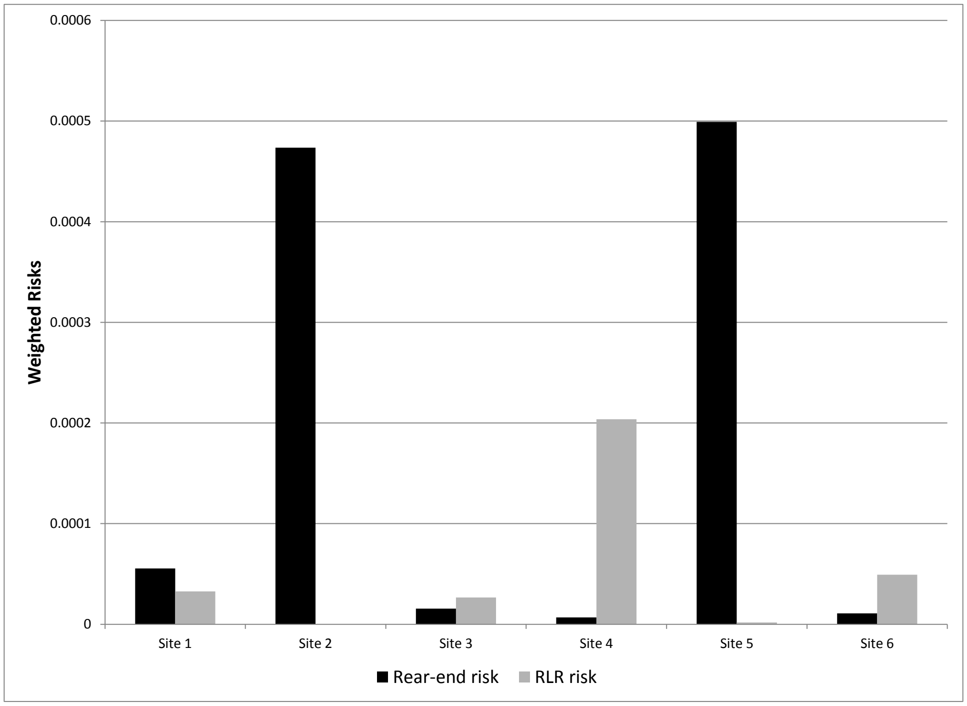


Figure 4.11: Calculated weighted risks

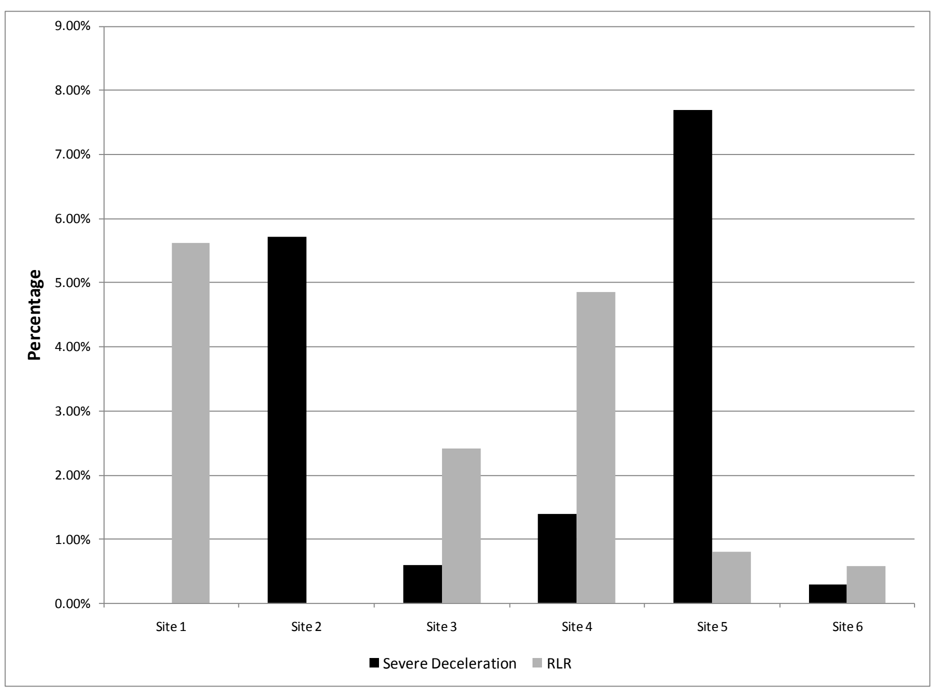


Figure 4.12 Proportion of vehicles performing severe deceleration or RLR

4.6. Further Case Study on Effect of Information

Site 5 revealed specific insights into the effect of information presented to drivers as they approach a high-speed intersection. As a result of the AWFs being located downstream of the mobile trailer, it was possible to view if a vehicle was upstream or downstream of the flashers at their onset. This enabled a probit model to be developed knowing a vehicle's time to the stop bar at both the onset of yellow, as well as the onset of the AWFs.

Using the values from Table 4.15 resulted in a statistical difference in both the slope and midpoint by providing drivers with information through the use of AWFs. This can be seen below in Figure 4.16. The probability of stopping at both the onset of yellow and onset of AWFs is shown. Additionally, the yellow onset stopping curve was shifted by the amount of the time the flashers come on before yellow, i.e. six seconds. By examining and comparing the AWF onset and shift yellow onset curves in Figure 4.17, it can be seen that there is a significant shift in both the slope and midpoint. Remembering the underlying theory in section 4.2, as the slope of the probability of stopping curve increases the indecision at the onset of yellow decreases. Thus, it can be inferred that more erroneous decisions are being made at Site 5, as the AWF curves' slope is less steep than the yellow onset slope.

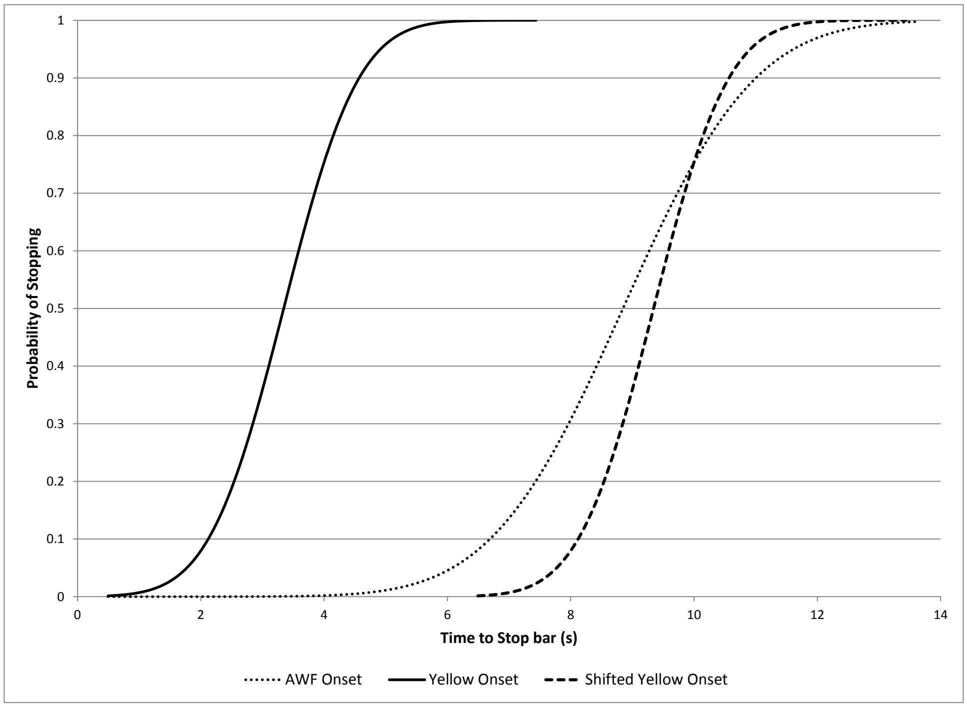


Figure 4.13: AWF Effect at US-75

Specifically, it is desired to decrease the risk of rear-end or RLR to drivers approaching a high-speed intersection. Figure 4.18 shows the resulting probability of severe conflict curves at both the onset of AWFs and yellow. The effect of information on the probability of severe conflict is noteworthy. As a result of have smaller probability of stopping values at both critical thresholds, providing information to drivers has decreased both the rear-end and RLR risk. However, there is still a large potential for rear-end accidents at Site 5 even by providing drivers with information.

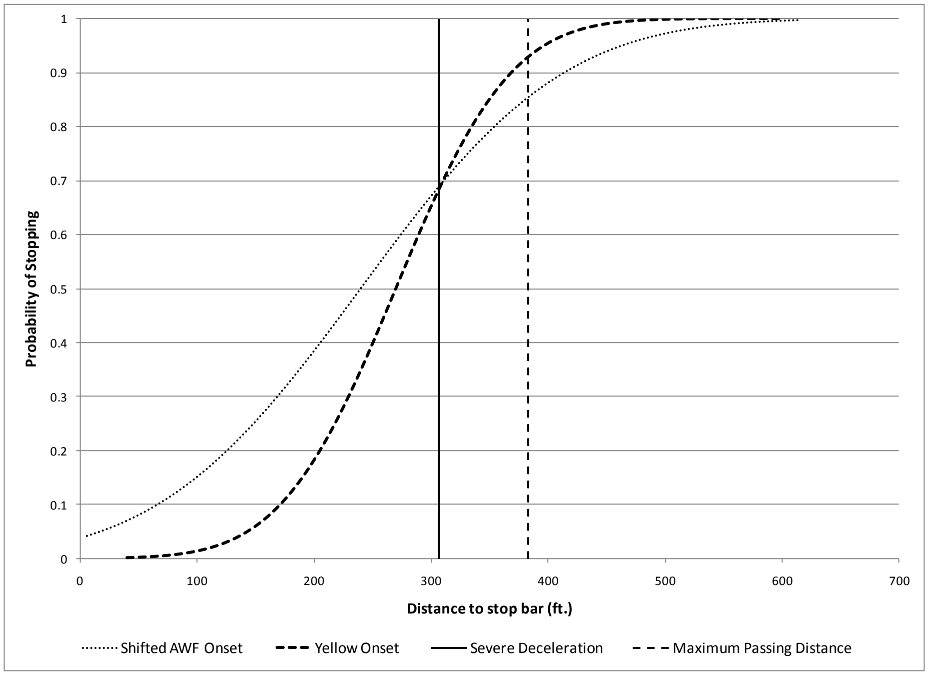


Figure 4.14: Effect of information on critical distances

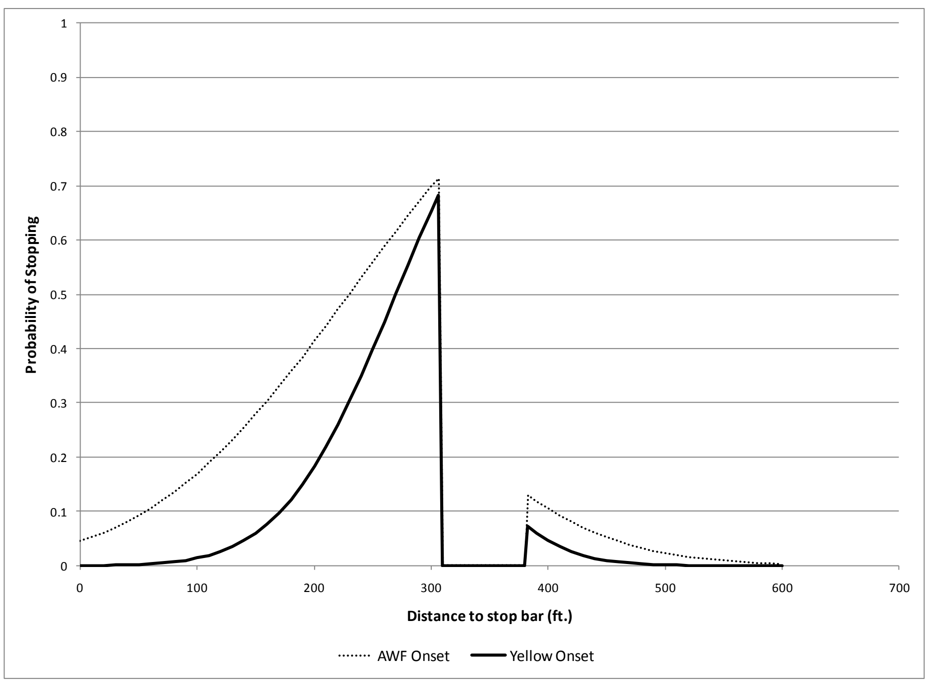


Figure 4.15: Effect of information on probability of conflict

4.7. Summary

This chapter presented probit model estimates for understanding the effect of information on drivers. In addition, an updated methodology of dilemma zone hazard function to assess the risk associated with a driver approaching a high-speed intersection at the onset of yellow was presented. Results of these analyses found providing information to drivers not only has the potential to affect the risk at an intersection but to potentially cause a significant increase in the risk

CHAPTER 5. CONCLUSIONS

5.1. Summary

In this thesis the effect of information on driver's approaching high-speed intersections was analyzed. Data was collected at five intersections providing drivers' information through the use of PTSWF signs, with an additional site used for evaluation. The probit model was used to model a driver's decision at the onset of yellow. Results revealed the effects of providing or lack of providing information. Specifically, the results indicate the effects AWFs have on the probability of stopping and perceived conflict curves. Sites providing information through PTSWF had earlier probability of stopping curves; in particular, Site 2 and Site 5 probability of stopping curves were drastically different than the other studied sites. The shift at Site 5 resulted in virtually all the drivers approaching the intersection having the potential for minor or severe conflict. The risk associated with being downstream of the severe deceleration distance and upstream of the maximum passing distance was calculated for a variety of speeds at each intersection. An overall weighted average was then computed and compared to the crash histories. An association could be seen in the comparison between the crash histories and the computed risks, as sites with larger severe deceleration risk had higher rear-end crash averages and vice versa. Therefore, it is evident that providing drivers with information in advance of the intersection using AWFs can potentially cause increased risk in both rear-end accidents and RLRs as opposed to decreasing the risk of drivers approaching the intersection. Thus, caution should be used by engineers before providing drivers with information at a high speed intersection.

5.2. <u>Conclusion</u>

The main contributions of this thesis are:

1. The effect of information was shown on both probability of stopping curves and the resulting probability of perceived conflicts. Results from Sites 2 and 5 found a shift in the probability of stopping closer to the intersection resulting in an increase in rear-end risk. In particular, the case study of Site 5 offered specific insights into the effect of information provided to drivers. Results of the case study revealed not only a shift in the probability of stopping curves, but a change in the slope of the stopping curve. These results contributed to decreases in both rear-end and RLR risk by providing information to drivers.
2. The effect of information on rear-end and RLR risk was shown to have an inverse relationship. As the rear-end risk increased, the RLR risk decreased as vice versa.
3. A reasonable correlation was found between the rear-end and RLR risk and the observed conflicts at each site similar to previous findings on the correlation between conflicts and crashes.

5.3. <u>Future Research</u>

Development and design of a flasher system to mitigate the risk of severe conflict is recommended for future research. Therefore, to increase the understanding of the effect of information on driver behavior, additional analysis should be performed at sites with and without AWFs. The costs associated with a trade-off between rear-end and right-angle crashes should be determined in the design as well.

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APPENDIX A

Table A.1: Comprehensive List of Variables Investigated

Variable Name	**Description**	**Coding**
Range	Distance to stop bar at onset of yellow	Integer
Speed	Speed of vehicle at the onset of yellow	Integer
TimeStop	Time to stop bar at the onset of yellow	Integer
Accel	Required acceleration to cross the stop bar prior to onset of red	Integer
Decel	Required deceleration to stop prior to the onset of red	Integer
Stop_Go	Decision by driver to stop or proceed through intersection	Go = 1, Stop = 0
MorningP	If observation was during the morning peak	Yes = 1, No = 0
Midday	If observation was during midday peak hours	Yes = 1, No = 0
Afternoon	If observation was during afternoon peak hours	Yes = 1, No = 0
Peak	If observation was during peak hours	Yes = 1, No = 0
Car	If vehicle was a car	Yes = 1, No = 0
HV	If vehicle was a HV	Yes = 1, No = 0
Site1	If vehicle was located at Site 1	Yes = 1, No = 0
Site2	If vehicle was located at Site 2	Yes = 1, No = 0

Site3	If vehicle was located at Site 3	Yes = 1, No = 0
Site4	If vehicle was located at Site 4	Yes = 1, No = 0
Site5	If vehicle was located at Site 5	Yes = 1, No = 0
Site1_Time	Time to the stop bar if at Site 1	Yes = 1, No = 0
Site2_Time	Time to the stop bar if at Site 2	Yes = 1, No = 0
Site3_Time	Time to the stop bar if at Site 3	Yes = 1, No = 0
Site4_Time	Time to the stop bar if at Site 4	Yes = 1, No = 0
Site5_Time	Time to the stop bar if at Site 5	Yes = 1, No = 0
Site1_HV	If vehicle was a heavy vehicle at Site 1	Yes = 1, No = 0
Site2_HV	If vehicle was a heavy vehicle at Site 2	Yes = 1, No = 0
Site3_HV	If vehicle was a heavy vehicle at Site 3	Yes = 1, No = 0
Site4_HV	If vehicle was a heavy vehicle at Site 4	Yes = 1, No = 0
Site5_HV	If vehicle was a heavy vehicle at Site 5	Yes = 1, No = 0